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BEHAVIOR OF DIAGONALLY-REINFORCED CONCRETE COUPLING BEAMS WITH HIGH-STRENGTH STEEL BARS

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Abstract

Coupling beams reinforced with Grade 60 (420 MPa) steel and with a clear span-to-depth ratio less than 2.0 often require heavily congested diagonal reinforcement to ensure adequate strength, stiffness, and deformation capacity. The constructability of these members would be markedly improved if the diagonal reinforcement were replaced with a smaller quantity of high-strength steel reinforcement. The aim of this study is to determine whether coupling beam performance is compromised when high-strength (Grade 120, or 830 MPa) steel is used in place of conventional reinforcement.

Tests have been designed to investigate the behavior of diagonally reinforced coupling beams constructed with Grade 120 (830 MPa) steel reinforcement under reversed cyclic loading. Three beams have been designed and constructed; one with Grade 60 (420 MPa) reinforcement and two with Grade 120 (830 MPa) reinforcement. One of the specimens with Grade 120 reinforcement was designed based on a shear stress of $15\sqrt{f'_c}$, psi ($1.25\sqrt{f'_c}$, MPa), where f'_c is the specified concrete compressive strength. The other specimens were designed for a shear stress of $10\sqrt{f'_c}$, psi ($0.83\sqrt{f'_c}$, MPa), the upper limit permitted by the ACI Building Code. High target shear stresses are feasible, from a constructability standpoint, because of the significant reduction in reinforcement volume associated with the use of high-strength reinforcement. The specimens have dimensions of 10 by 18 in. (25.4 by 45.7 cm) and a length of 34 in. (86.4 cm), which results in an aspect ratio of 1.9. The beams were cast monolithically with larger concrete sections at each end to simulate interfaces with wall boundary elements. A summary of the experimental program is presented with details on test setup, loading protocol, and specimen design. Preliminary test results are reported for the specimen with Grade 120 (830 MPa) reinforcement with a target shear stress of $10\sqrt{f'_c}$, psi ($0.83\sqrt{f'_c}$, MPa).

Keywords: coupled walls, deformation capacity, reversed cyclic loading, seismic performance, shear walls.

1. Introduction

Reinforced concrete buildings designed for seismic resistance often have structural walls (shear walls) due to their lateral strength and stiffness. Architectural considerations commonly result in openings in these structural walls, which divide a single wall into multiple walls connected by short deep beams denoted as coupling beams. The use of coupling beams leads to a more cost-effective structural system than individual walls, because properly designed coupled wall systems have considerably higher strength, stiffness, and energy dissipation capacity. Studies on the seismic behavior of coupling beams have shown that beams reinforced with diagonally oriented reinforcing bars exhibit acceptable strength and deformation capacity [1]. In such beams, it is assumed that all imposed shear and moment demands are resisted by the diagonal bars. Closely spaced transverse reinforcement is necessary to delay buckling of the diagonal reinforcement. Such reinforcement detailing, however, creates construction problems due to the reinforcement congestion associated with the placement of the diagonal and transverse reinforcement.

A series of tests is underway that have been designed to study the behavior of specimens constructed with high-strength steel bars, the use of which has been limited by US building codes [2] for many years due to lack of experimental data. The aim of the test program is to minimize construction difficulties while maintaining or improving overall performance of coupling beams. The test program consists of coupling beam specimens with the following variables: nominal yield strength of diagonal reinforcement, target coupling beam shear strength, axial restraint, and development length of secondary (non-diagonal) longitudinal reinforcement. To address the first two variables, three specimens have been constructed, two of which use high-strength (Grade 120, or 830 MPa) steel as diagonal reinforcement. Two different shear stress levels are targeted in this group of specimens.

2. Research Background

For satisfactory performance of coupled-walls during a seismic event, coupling beams must retain a significant shear force capacity through large displacement reversals without severe reductions of strength and stiffness. The ACI Building Code (ACI 318-14) [2] requires the use of diagonal reinforcement to resist all of the shear demand in short and highly stressed coupling beams. This is because short coupling beams (with an aspect ratio less than 2.0) reinforced with moment frame-type detailing have been shown [3] to be susceptible to either diagonal tension or sliding shear failures that limit the deformation capacity. Diagonally reinforced coupling beams have been shown [1] to provide a stable behavior under earthquake-type displacement reversals; however, the large amount of diagonal steel required to resist all of the imposed shear demand can be difficult and time consuming to place through the adjacent wall reinforcement. The use of high-strength steel is being proposed to both simplify the construction process by reducing the amount of steel and also as a means of increasing the design shear stress capacity without sacrificing performance under large displacement reversals.

For many years, the use of high-strength steel bars with a nominal yield strength greater than 80 ksi (550 MPa) has been limited by the ACI Building Code. Coupling beams are, however, excellent candidates for use of higher strength reinforcement. This is because concerns related to the use of higher grades of reinforcement, including compatibility with concrete under compression (assuming a limiting concrete strain of 0.003) and control of crack widths at service load, are likely not problematic for the case of coupling beams. Furthermore, there are insufficient test data for coupling beams designed for shear stresses exceeding the code-specified upper limit of $10\sqrt{f'_c}$, psi ($0.83\sqrt{f'_c}$, MPa) because reinforcement congestion limits the constructability of such beams. Use of higher grade diagonal reinforcement may allow for higher design shear stresses in coupling beams.

Previous research [4] has shown that the ACI Building Code recommendation to terminate secondary (non-diagonal) longitudinal reinforcement near the intersection with the wall may lead to undesirable localization of damage along the wall-beam interface. Providing an embedment length for these bars that satisfies development length requirements can lead to improved strength and deformation capacity [5].

3. Research Significance

The research program includes the first tests of coupling beams reinforced with Grade 120 (830 MPa) steel and the first with an aspect ratio near two designed for a nominal shear stress of 15$\sqrt{f'_c}$, psi (1.25$\sqrt{f'_c}$, MPa), which is 50% greater than that permitted by the ACI Building Code. The use of high-strength steel bars in conventional reinforced concrete has been limited due to restrictions in existing building codes along with insufficient experimental data and design guidelines. If successful, research outcomes will facilitate construction of coupling beams with reduced amounts of reinforcement and, thereby, reduced construction time and cost.

4. Experimental Program

4.1 Specimens

Testing is underway of three coupling beam specimens under reversed cyclic loads. Details of the specimens are listed in Table 1 and shown in Fig. 1. The specimens had a length of 34 in. (86.4 cm), depth of 18 in. (45.7 cm), and width of 10 in. (25.4 cm), resulting in an aspect ratio (ratio of clear span-to-overall depth) of 1.9. The specimens had either Grade 60 or 120 (420 or 830 MPa) steel as diagonal reinforcement and Grade 60 (420 MPa) steel for all non-diagonally oriented reinforcement. Specimens CB2 and CB3 were designed to have nominal shear strengths, calculated assuming the diagonal reinforcement resists all imposed shear force, of 10 and 15$\sqrt{f'_c}$, psi (0.83 and 1.25$\sqrt{f'_c}$, MPa), respectively. This resulted in Specimens CB2 and CB3 having 8 and 12 No. 6 (19 mm) diagonal bars (Fig. 2). Transverse reinforcement was nominally identical in all specimens, with No. 3 (10 mm) hoops and crossties spaced at 3 in. (7.62 cm) on center, or 4 times the diameter of the diagonal bars. The specimens also had No. 3 bars oriented longitudinally and distributed around the perimeter of the beam. To be consistent with the detailing recommended in the ACI Building Code commentary, the horizontal reinforcement was terminated 2 in. (5.1 cm) into the top and bottom blocks.

The test setup was designed to test the specimens vertically, with a top block and bottom block designed to simulate wall boundary elements (Fig. 1). The top and bottom blocks were reinforced with Grade 60 (420 MPa) steel. The specimens were cast monolithically with the top and bottom block (laying horizontally).

Table 1 – Summary of coupling beam specimens

Specimen ID	Diagonal Bar Yield Strength f_y, ksi	Transverse Bar Yield Strength f_{yt}, ksi	Longitudinal Bar Yield Strength f_{yl}, ksi	Nominal Shear Strength [a], psi	Diagonal	Transverse	Longitudinal
CB1	60	60	60	$10\sqrt{f'_c}\,A_{cw}$	12 #7	#3@3in	8 #3
CB2	120	60	60	$10\sqrt{f'_c}\,A_{cw}$	8 #6	#3@3in	8 #3
CB3	120	60	60	$15\sqrt{f'_c}\,A_{cw}$	12 #6	#3@3in	8 #3

[a] Based on ACI 318-14 Eq. 18.10.7.4 using specified material properties; A_{cw} is the cross-sectional area of the coupling beam.

4.2 Materials

Measured material properties for both the concrete and reinforcement are listed in Table 2. Ready-mixed concrete provided by a local supplier was used to cast the specimens. The target compressive strength was 6,000 psi (41 MPa). Mill certifications for reinforcing bars used as conventional Grade 60 (420) steel showed compliance with ASTM A706/A706M-15 (2015) Grade 60 (420) [6]. Mill certifications of reinforcing bars used as Grade 120 (830) showed compliance with ASTM A1035-16a Grade 120 (830 MPa) [7].

Table 2 – Measured material properties

Specimen ID	Concrete: Compressive Strength at Test Day f_{cm} ksi (MPa)	Reinforcing Steel: Nominal Bar Diameter in. (mm)	Reinforcing Steel: Yield Strength (f_y) ksi (MPa)	Reinforcing Steel: Tensile Strength (f_t) ksi (MPa)
CB1	–	0.875 (22)	63 (434) [2,3]	91 (627) [2]
CB2	7.2 (50) [1]	0.75 (19)	127 (876) [2,3]	168 (1158) [2]
CB3	7.0 (49) [1]	0.75 (19)	127 (876) [2,3]	168 (1158) [2]

[1] Cylinder size of 6 by 12 in. (150 by 300 mm)
[2] Reported on manufacturer mill certification (will be verified with laboratory testing)
[3] Based on the 0.2%-offset method

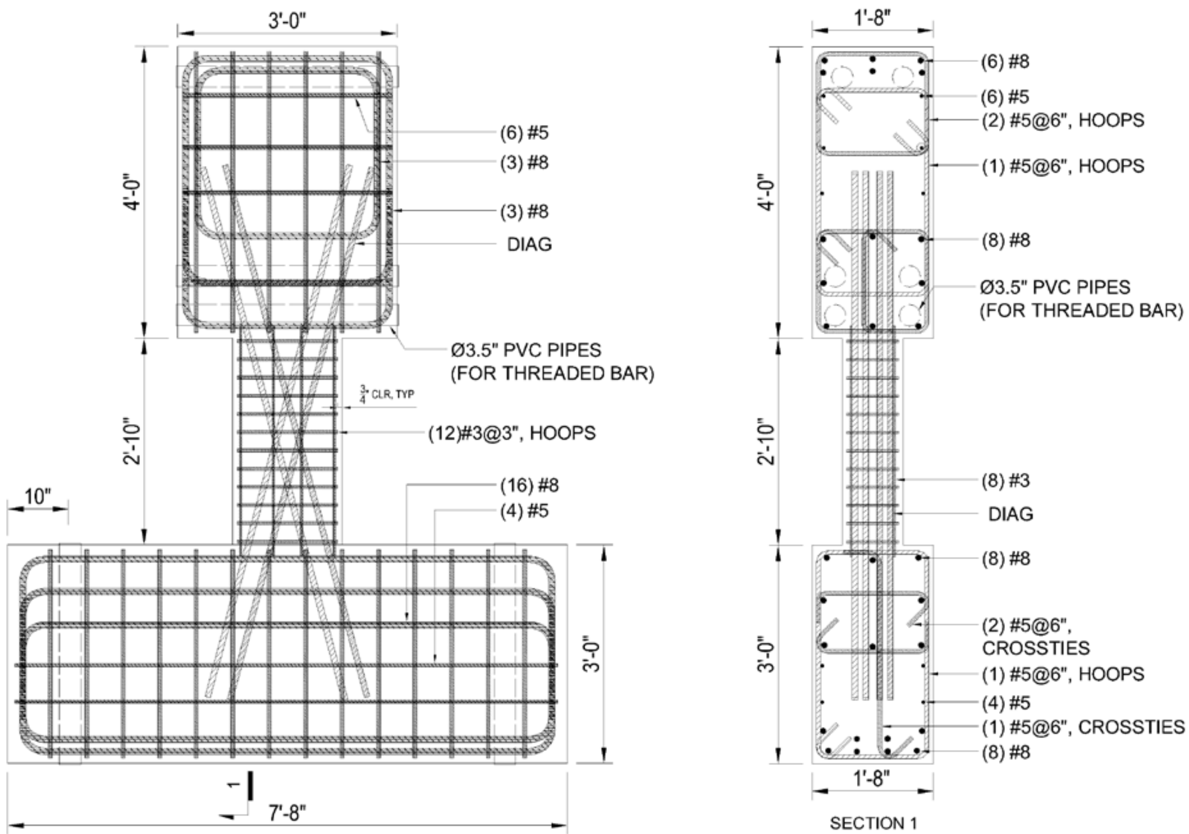
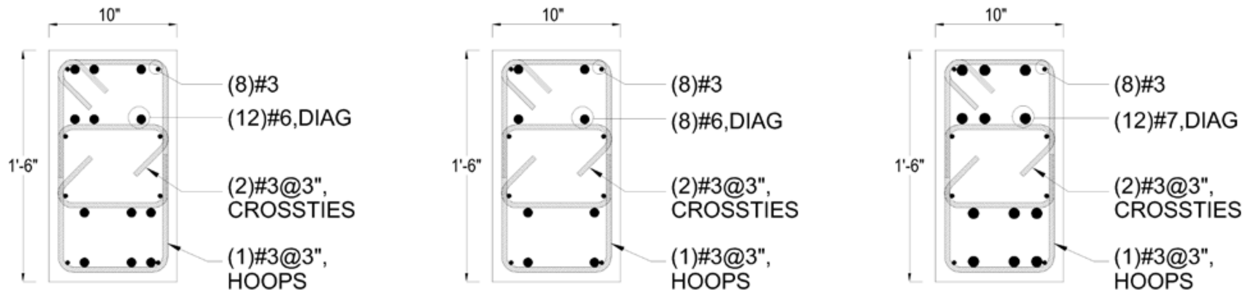


Fig. 1 – Specimen details



(a) CB1 (b) CB2 (c) CB3

Fig. 2 – Coupling beam cross-sections near wall intersection

4.3 Test setup

A typical specimen and testing setup is shown in Fig. 3. For testing, the bottom block of the specimens was bolted to the laboratory strong floor with two 2.5-in. (6.35-cm) diameter high-strength threaded rods passing through the base block (Fig. 3). Two MTS 201.70 Hydraulic Actuators were used to load the specimens, with force-based actuator control for cycles prior to yielding and displacement-based control for later cycles. The actuators each have a stroke length of 40 in. (102 cm) and a capacity of 220 kip (979 kN). The ratio between forces or displacements applied by the two actuators was selected such that a zero-moment inflection point remained near mid-span of the beam throughout the tests (specimens were under double-curvature).

The two actuators were connected to the strong wall and the specimen by means of vertically oriented HP steel sections. The HP section connected to the top block of the specimen used a series of hollow structural steel (HSS) sections for transmitting compression and six 2.26-in. (5.75-cm) diameter high-strength threaded bars for transmitting tension (Fig. 3). Additional steel fixtures were present to provide the HP section with out-of-plane bracing.

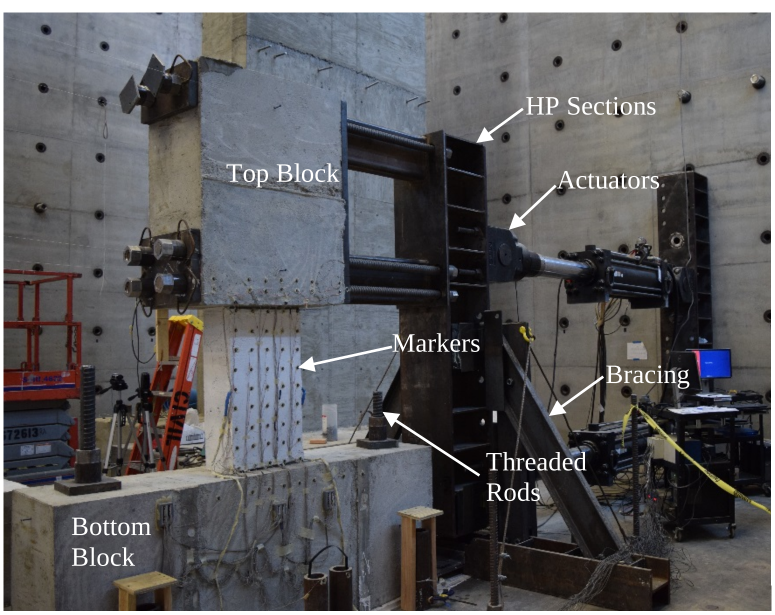


Fig. 3 – Test setup

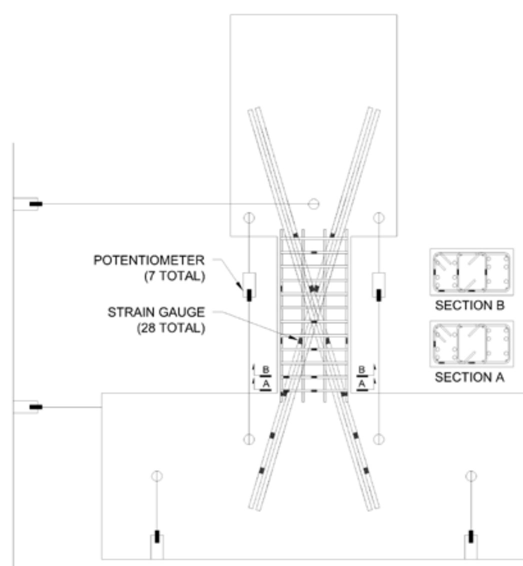


Fig. 4 – Instrumentation

4.4 Instrumentation

The location of the instrumentation is shown in Figs. 3 and 4. The lateral deflection of the top block was measured with two potentiometers installed horizontally on opposite sides of the top block. To measure the rotation of the top block with respect to the bottom block, two potentiometers were positioned vertically connecting the top and bottom blocks. Three potentiometers (two vertical and one horizontal) were used to monitor rotation and sliding of the base block relative to the strong floor.

In addition to potentiometers, an infrared-based non-contact position measurement system was used to record the position of 72 markers (Fig. 3), attached directly to the surface of the specimens (the markers emit infrared light pulses that are detected by cameras, allowing their spatial coordinates to be triangulated). The markers were arranged in a 4 in. (100 mm) square grid over one face of the specimen and part of the top and bottom blocks. Data from this system will be analyzed to determine the distribution of deformations.

Longitudinal, transverse, and horizontal reinforcing bars were instrumented with 28 electrical resistance strain gauges placed at the locations shown in Fig. 4. In each specimen, two diagonal bars were instrumented with six strain gauges each, eleven strain gauges were attached to the hoops and ties and the No. 3 (10 mm) longitudinal bars were instrumented with five strain gauges. The strain gauges were rated for 15% strain to allow measurements throughout the test.

4.5 Loading protocol

Specimens were subjected to a series of reversed cyclic displacements following the protocol shown in Fig. 5, which is patterned after the protocol recommended in FEMA 461 (2007) [8]. Prior to yielding of the diagonal reinforcement, force-based control was used to impose pairs of cycles to approximately 0.3, 0.6, and 0.9 times the load associated with anticipated yielding of the diagonal reinforcement. The remainder of the cycles were imposed using displacement control.

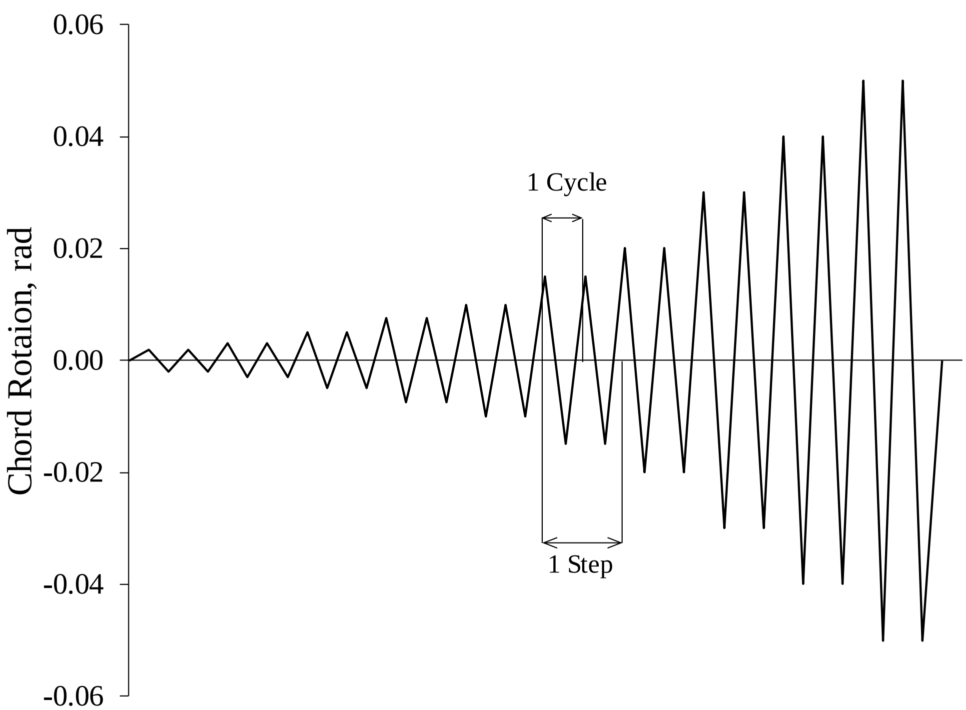


Fig. 5 – Target loading protocol

5. Preliminary Test Results and Observations

The measured force versus chord rotation response for CB2 is shown in Fig. 6 (results from tests of CB1 and CB3 are not yet available). Beam chord rotation (or beam drift ratio) was calculated using data from the infrared-based non-contact position measurement system as the relative displacement between top and bottom blocks, corrected for rotation of both the top and bottom blocks, divided by the clear span of the beam. The maximum force resisted by the specimen was 207 kips (921 kN), which corresponds to a shear stress of $13.6\sqrt{f'_c}$, psi ($1.13\sqrt{f'_c}$, MPa), higher than the target shear stress of $10\sqrt{f'_c}$, psi ($0.83\sqrt{f'_c}$, MPa). This peak load occurred at a chord rotation of 0.03 rad.

Despite the high shear demand, the beam response shown in Fig. 6 is stable until the final push towards +6% chord rotation. Failure of the specimen was sudden and dominated by fracture of several diagonal bars. The shape of the fractured and adjacent bars, observed after testing, indicated that bar fracture was preceded by bar buckling. Prior to bar fracture, the specimen retained more than 80% of its peak strength in both loading directions.

The chord rotation capacity of the specimen is defined as the average of the maximum chord rotations imposed in both loading directions prior to failure. CB2 had a chord rotation capacity of 5.1% (5.6% in one direction and 4.5% in the other) without a significant reduction in load carrying capacity. Fig. 7 shows the deformed CB2 specimen during at a chord rotation of +5.2%. It is clear in Fig. 7 that deformations concentrated near the beam-to-wall interface where the diagonal bars ultimately fractured. This may be attributable to the termination of secondary (non-diagonal) longitudinal reinforcing bars near that intersection.

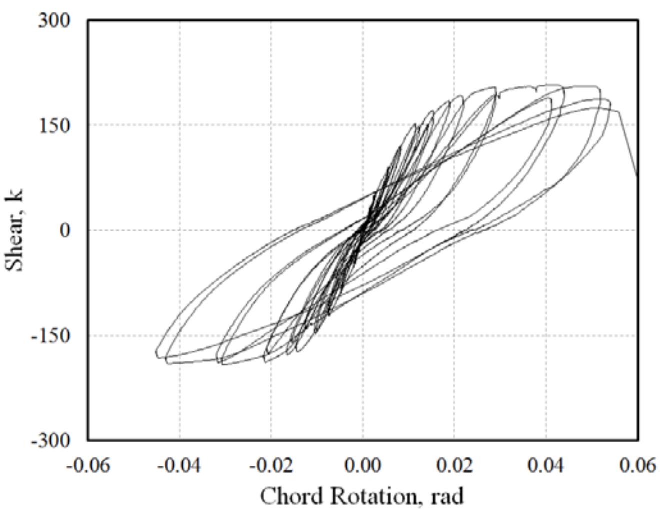


Fig. 6 – Shear versus chord rotation for CB2

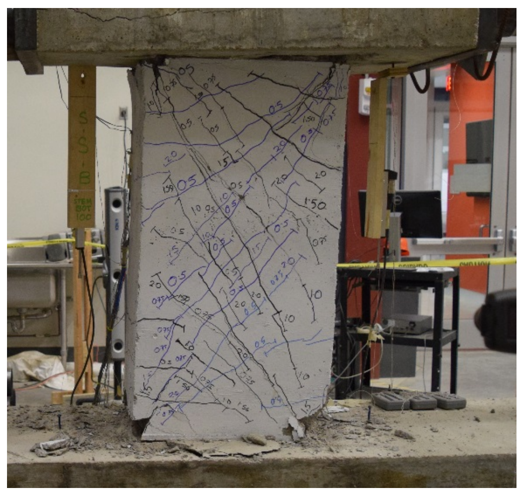


Fig. 7 – Deformation of CB2 at a chord rotation of +5.2%

Because the control specimen has yet to be tested, this performance is evaluated in the context of previous tests with clear span-to-overall depth ratios (aspect ratios) close to 1.9. The most similar test was reported by Tassios, Moretti and Bezas [9]. One of their specimens was diagonally reinforced and had an aspect ratio of 1.7. Under a peak shear stress of $10.2\sqrt{f'_c}$, psi ($0.85\sqrt{f'_c}$, MPa), the specimen exhibited a maximum chord rotation of 5%; similar to that exhibited by CB2. The specimen had diagonal reinforcement with a yield stress of 73 ksi (504 MPa) and fully developed secondary longitudinal reinforcement. Naish et al. [10] tested a somewhat more slender coupling beam with aspect ratio of 2.4. The specimen exhibited an ultimate chord rotation of 8% under a peak shear stress of $11.5\sqrt{f'_c}$, psi ($0.95\sqrt{f'_c}$, MPa). The specimen had diagonal reinforcement with a yield stress of 70 ksi (482 MPa) and, similar to CB2, the secondary longitudinal reinforcement was terminated near the coupling beam-to-wall interface. This relatively large deformation capacity may be attributable to the larger aspect ratio of the specimen. The tests reported by Naish et al. [10] exhibited localized deformations at the beam ends similar to CB2.

6. Summary and Conclusions

An experimental program is underway to investigate the deformation capacity of coupling beams reinforced with high-strength steel under reversed cyclic displacements. The main variables for the tests described herein are the yield stress of the diagonal reinforcement and the target beam shear stress.

Results from one test are available at this time. The specimen, which had Grade 120 (830 MPa) diagonal reinforcement and was subjected to shear stresses greater than $10\sqrt{f'_c}$, psi ($0.83\sqrt{f'_c}$, MPa), exhibited a chord rotation capacity of 5.1%, which is similar to, but somewhat smaller than, the deformation capacities reported for similar specimens reinforced with Grade 60 (420 MPa) diagonal bars. Additional testing is planned that will include a more similar control specimen (CB1) and beams with alternative detailing to limit the concentration of deformations at the face of the wall.

7. Acknowledgements

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