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SEISMIC BEHAVIOR OF RC COUPLING BEAMS WITH SIMPLISTIC REINFORCEMENT LAYOUT

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Abstract

Diagonally reinforced coupling beams (DCBs) are commonly used as seismic-force resisting members for medium- to high-rise buildings in high seismic zones. The diagonal reinforcing bars in DCBs are most effective when the beam has a span-to-depth ratio less than 2. However, modern construction typically requires span-to-depth ratios between 2.4 to 4, which leads to a very shallow angle of inclination for the diagonal reinforcement. The lower angles of inclination, when combined with the detailing requirements specified in ACI 318, result in reinforcement congestion and construction difficulties. These issues can be considerably minimized by utilizing an innovative and simplistic reinforcing scheme consisting of two separate cages similar to those used for typical beams in reinforced concrete special moment frames. The proposed coupling beam has high stiffness and acts like a conventional coupling beam under small displacements. When large displacements occur, cracks begin developing at the beam's mid-span and mid-height area where the narrow unreinforced concrete strip is located, gradually propagating towards the beam's ends. The cracks eventually separate the coupling beam into two slender beams where each has nearly twice the aspect ratio of the original coupling beam. This split essentially transforms the shear-dominated deep beam behavior into a flexure-dominated slender beam behavior. Because damage initiates from the center of the beam and then spreads towards the ends, the beam's ends maintain their integrity even under very large displacements thereby eliminating the sliding shear failure at the beam-to-wall interface. Testing results on half-scale specimens with span-to-depth ratios of 2.4 and 3.3 showed that the proposed coupling beam not only has high ductility and shear strength, but can significantly reduce construction issues in conventional DCBs. In addition, because the cracks always initiate at mid-span and mid-height, the damage location can be easily predicted, which makes repair work easier after moderate earthquakes.

Keywords: RC coupling beam, diagonally reinforced coupling beam, double-beam coupling beam, span-to-depth ratio

1. Introduction

Reinforced concrete structural walls are commonly used as the primary seismic-force-resisting system in buildings. Based on architectural requirements, these walls have numerous openings for entities such as elevators, windows, and doors, which divide a single wall into slenderer walls connected by substantial beams. These beams are known as coupling beams. The use of the coupled wall system leads to a more efficient and economical structure system than single walls because properly designed coupled wall systems possess significantly higher strength, stiffness, and energy dissipation capacity. To attain the desired behavior in the coupled wall system, the coupling beam is required to sustain high shear forces while undergoing large displacement. However, the coupling beams must also yield before the wall piers, behave in a ductile manner, and exhibit significant energy dissipating characteristics.

Prior studies [1, 2] have shown that conventional longitudinally reinforced concrete coupling beams that are flexure-dominant have exhibited satisfactory seismic performance under a shear stress below $0.25\sqrt{f'c}$ (MPa). Beyond this stress level, the sliding shear at the beam-to-wall interface starts to affect the response and eventually leads to failure. Also, recent experiments have shown that slender conventional longitudinally reinforced coupling beams can reach approximately 4% chord rotation at a peak stress of $0.28\sqrt{f'c}$ (MPa) prior to strength degradation [3]. On the other hand, prior nonlinear time-history analyses [4] indicated that coupling beams would need average rotation capacities of 3% and 6% for design basis earthquakes (DBE) (10% probability of exceedance in 50 years) and maximum considered earthquakes (MCE) (2% probability of exceedance in 50 years) level ground motions, respectively, to maintain the integrity of the coupled wall system. Based on the shear resistance and adverse failure mechanisms of conventional coupling beams, Paulay and Binney [5] recommended a detailing consisting of two intersecting diagonal reinforcement groups combined with closely spaced transverse reinforcement (Fig. 1). In this reinforcement detail, the diagonal bars need to be well confined by transverse reinforcement and carefully anchored in the walls. In a design using this type of coupling beam, the whole shear transfer mechanism is resisted by heavily reinforced diagonal cages. Experimental results have shown that diagonal reinforcement detailing can sustain high shear stress and significantly improve deformation and energy dissipation capacity compared to conventional detailing for coupling beams subjected to reversed cyclic loading [5]–[8].

Other alternative reinforcement schemes have been investigated [9], such as the addition of dowels at the ends of the coupling beams or a diagonal reinforcement located only at the beam-wall interface. However, Tassios et al. [9] experimentally demonstrated how coupling beams with these alternative reinforcement details will not exhibit satisfactory seismic behavior, and they can also cause construction difficulties. Furthermore, for coupling beams with a span-to-depth ratio less than or equal to 2.0, diagonal reinforcement over the full beam span has proven to be an efficient solution. On the other hand, modern architectural specifications typically require span-to-depth ratios between 2.4 to 4, which leads to a very shallow angle of inclination for the diagonal reinforcement (can be as low as approximately 10 degrees). The lower angles of inclination, combined with the detailing requirements specified in ACI 318 [10], can cause several major issues for both design and construction [7, 11]–[13].

1. A small angle of inclination significantly decreases the efficiency of diagonal reinforcement in resisting shear forces, which in turn requires an even smaller angle to accommodate the bars; thus, more reinforcing bars are needed, which ultimately increases the difficulty of construction. There is significant difficulty in placing the diagonal reinforcement because they can be easily obstructed by transverse reinforcement used to confine the diagonal bars (Fig. 1). Additional reinforcement detailing is required when the extension of the diagonal bars are bent at the top of the wall or at openings in the wall.

2. The minimum width requirement for diagonal elements causes interlock of the two diagonal elements. This in turn demands an increased clear distance between reinforcing bars in order for one diagonal element to pass through the other. The minimum dimensions and required reinforcement clearances can make the coupling beam very wide, which controls the wall width.

3. It can be very challenging and time-consuming to thread the diagonal reinforcement through the congested

vertical and horizontal bars in the wall's boundary elements (Fig. 1).

4. Although ACI 318-14 [10] Sect. 18.10.7.4(d) allows the second confinement option where the transverse reinforcement is provided for the entire beam cross section rather than around the diagonal bars, there is still obvious difficulty of passing the diagonal bars through these hoops and crossties.

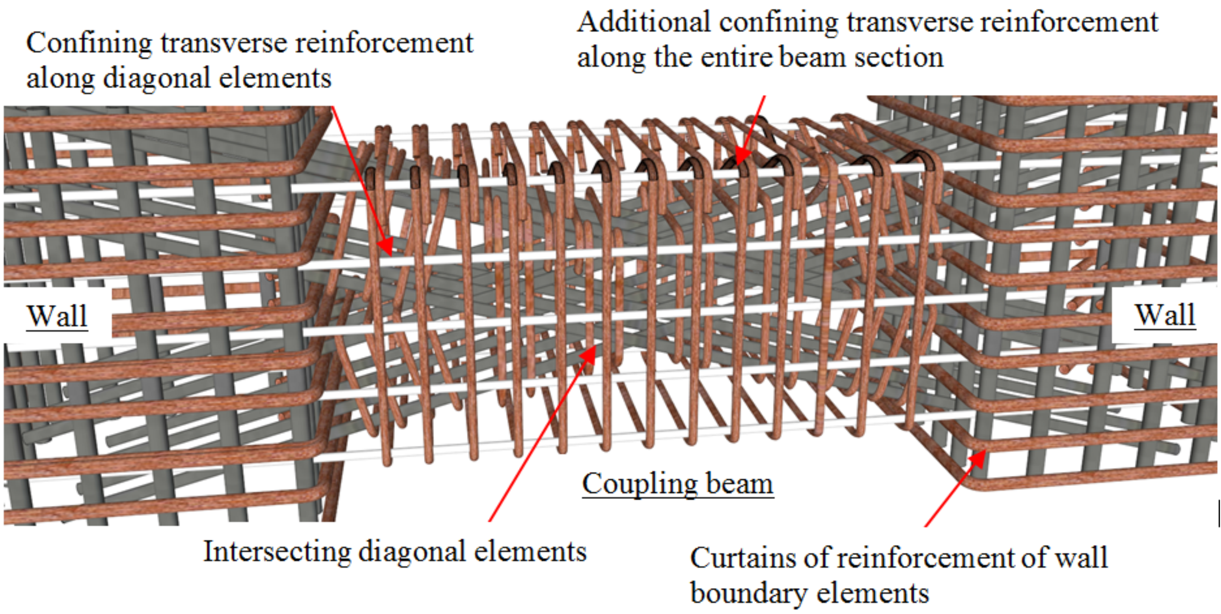

Fig. 1 – Reinforcement detail of DCB with aspect ratio greater than 2.0, according to ACI 318-14 [10]

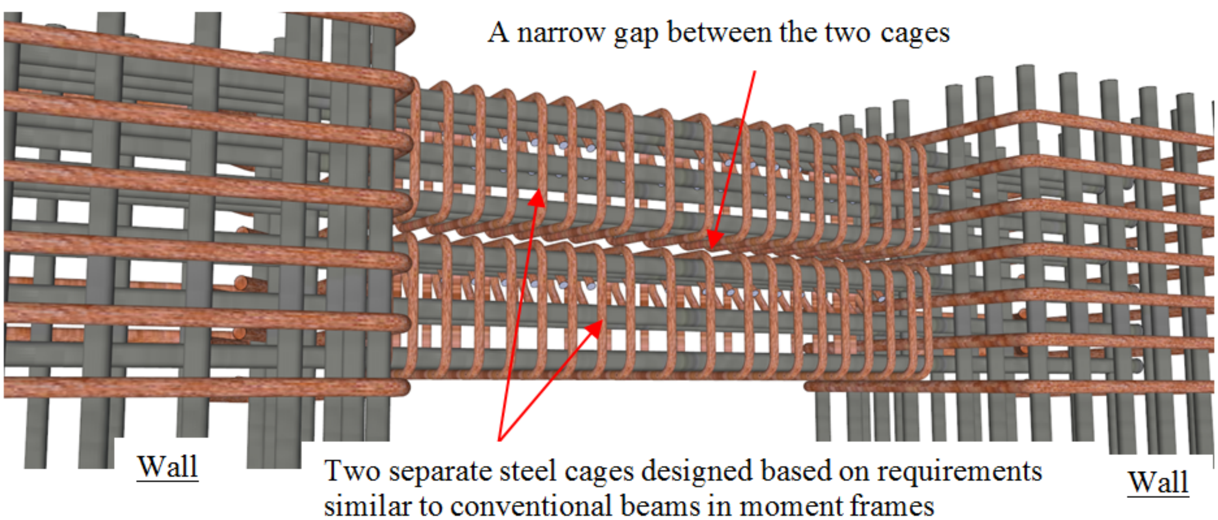

Fig. 2 – Proposed reinforcement detailing (DBCB) for RC coupling beam

2. Proposed Alternative Reinforcement Scheme for RC Coupling Beams

The above-mentioned construction and design issues with DCBs can be considerably minimized by utilizing an innovative and simplistic reinforcing scheme as proposed in this research (Fig. 2). This reinforcement consists of two separate cages similar to those used for typical beams in RC special moment frames. The proposed coupling beam (double-beam coupling beams, DBCBs) has high elastic stiffness and acts like a conventional coupling beam under small displacements. With large displacements, cracks begin developing at the beam's mid-span and mid-height where the narrow unreinforced concrete strip is located, gradually propagating towards the ends. The cracks eventually separate the DBCB into two slender beams where each has nearly twice the aspect ratio of the original coupling beam. This essentially transforms the shear-dominated behavior into the flexure-dominated behavior common to conventional slender beams. Because damage initiates from the mid-span of the beam, then spreads towards the ends, the beam's ends maintain their integrity even under very large displacements, thereby eliminating the sliding shear failure at the beam-to-wall interface, as is commonly seen in conventional coupling beams [1, 2]. Fig. 3 illustrates the difference in the reinforcement between the two ACI 318 [10] compliant DCBs (Fig. 3a – individual diagonal elements are confined; Fig. 3b – only the full beam section is confined) and the proposed DBCB. All three coupling beams have an aspect ratio of 2.4 and the same nominal shear strength. The proposed coupling beam's reinforcement provides simpler detailing and greater constructability. DBCBs can also have a narrower section width due to the elimination of diagonal reinforcing bars. In addition, because the cracks always initiate at the mid-span and mid-height of DBCBs, the damage location can be easily predicted, which makes repair work much easier after moderate earthquakes.

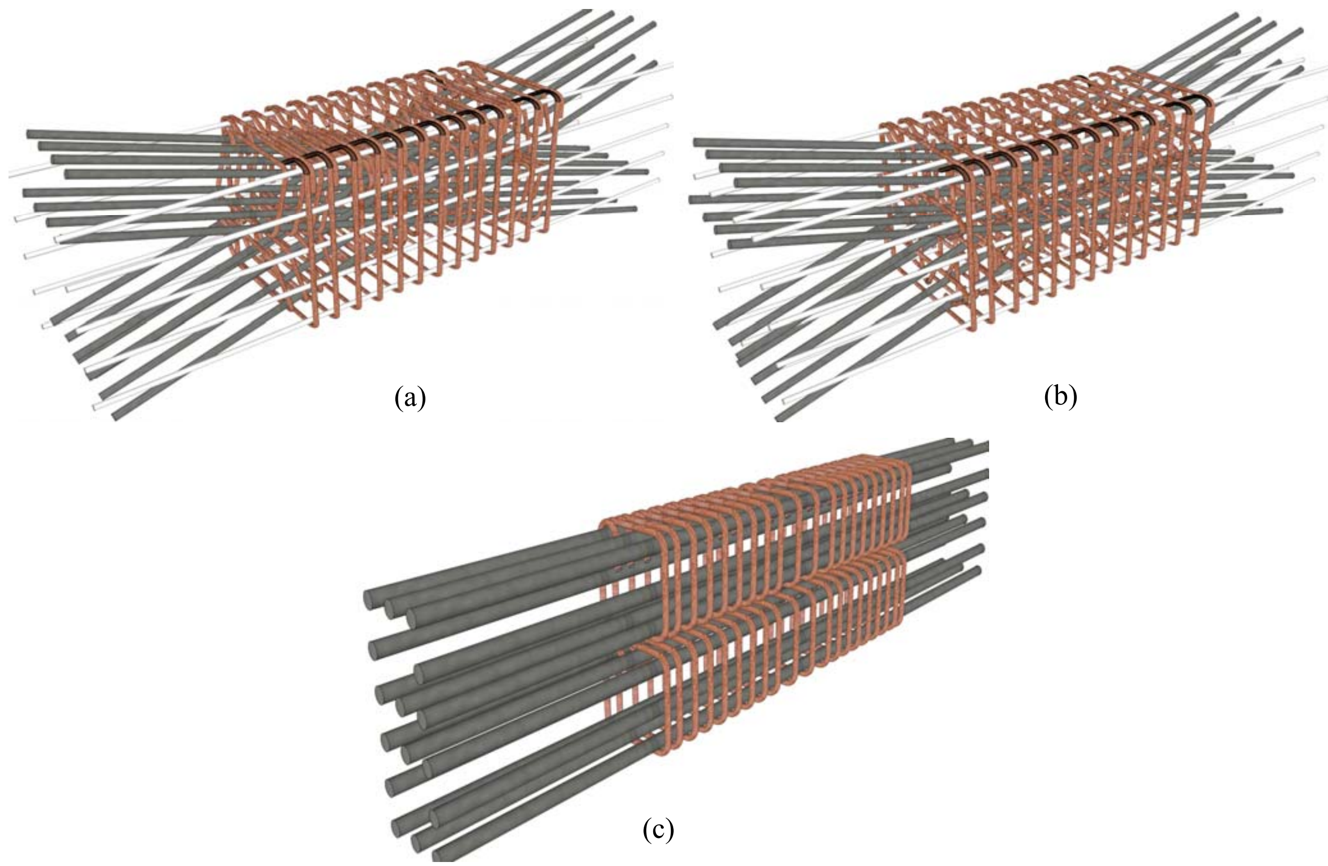

Fig. 3 – Comparison between reinforcement details between RC coupling beams: (a) individual diagonal elements are confined [10], (b) only the full beam section is confined [10], and (c) proposed DBCB reinforcement details

3. Experimental Program

2.1 Test specimens

Experimental results of three coupling beam specimens (Table 1) are presented in this paper. Their performance is compared with recent research results on the diagonally reinforced coupling beam (DCB) arrangement by Naish et al. [7]. The DBCB specimens have the same span to-depth-ratio, 2.4 and 3.3, as the DCB specimens tested by Naish et al. [7]. These specimens are about one-half scale replicas of the coupling beams in typical residential buildings (Fig. 4). However, instead of using a beam width of 305 mm as in the DCB specimens [7], the width of DBCBs was reduced by fifty percent (152 mm) because this width is sufficient to accommodate the straight bars and to achieve the very high factored gross section shear stress level. In addition, the development length is only about 60% of that required by ACI 318-14 Sect. 18.8.5.3(b), due to the fact that the beam-wall boundary did not experience severe damage (shown later). This is opposite to DCBs where the major damage is at the beam-wall boundary as a result of the slip and extension of the diagonal bars [3].

The transverse reinforcement in DBCB specimens was designed according to the confinement and shear requirements specified for flexural members of special moment frames in Sect. 18.6 in ACI 318-14 [10]. The transverse reinforcement area ratios in the plastic hinging zone (defined as $2d$ from the beam-wall boundary, where d is the effect depth of each cage) and beyond the plastic hinging zone is also shown in Table 1. Fig. 4 shows the reinforcement details of the DBCB specimens. The gap between the two cages was 1" wide (clear distance between transverse reinforcement). The one-inch gap was chosen based on nonlinear finite element analyses using VecTor2 [14]. A large gap could lead to a reduced moment arm for each cage; consequently, its moment capacity as well as the overall shear strength of the DBCBs can be reduced. On the other hand, if the gap is too small the beam cannot completely separate into two slender beams before the major shear cracks dominate the behavior thereby causing premature strength degradation. The nominal design concrete strength is 34.5 MPa, and the actual average compressive strength obtained on the testing dates were 39 MPa for both R2.4-SC-1 and R3.3-SC-1 and 43 MPa for R-2.4-NC-1.

Table 1–Specimen information

Specimen	l/h	l/d	**Gap, (mm)**	**Transverse Reinforcement** $\rho_{plastic\ hinging}$ % (bar size, spacing)	$\rho_{non-plastic\ hinging}$ % (bar size, spacing)	ρ_l, %[1] (main bar, sub bar)	f'_c, (MPa)
R2.4-SC-1	2.4	5.71	25	4.4 (#4, 38 mm)	2.67 (#4, 64 mm)	5.87 (#6, #6)	39
R2.4-NC-1	2.4	5.71	25	4.4 (#4, 38 mm)	2.67 (#4, 64 mm)	5.87 (#6, #6)	43
R3.3-SC-1	3.3	7.89	25	4.4 (#4, 38 mm)	1.67 (#4, 64 mm)	8.00 (#7, #7)	39

[1]: ρ_l is total steel area divided by gross cross-sectional area of DBCB.

2.2 Test setup and instrumentation

Each specimen consisted of a coupling beam, and a pair of big and small reinforced concrete blocks representing adjacent structural walls. The specimens were cast horizontally, then rotated and placed in the test setup with the big block fixed to the strong floor (Fig. 5). The cyclic load was applied via a vertical actuator, with the actuator forces' line of action passing through the mid-span of the test specimen to produce an anti-symmetrical moment pattern in the coupling beam and zero moment at the beam's mid-span. The actuator was connected to the small block through a wide flange steel section. The load was transferred to the small block by means of direct bearing and unbonded threaded bars passing through the small block. Two steel links were used to provide some moderate axial restraints for the beams because, in reality, the adjacent structural walls and surrounding slab can only provide non-negligible resistance to beam expansion upon cracking [15, 16]. The specimens were subjected to cyclic loading in a displacement control mode which produced predefined reversed cyclic displacement patterns. Two loading protocols were used, starting from a coupling beam chord rotation of 0.25% and reaching a maximum rotation of 12%. The first loading protocol consisted of symmetric cyclic loading utilizing 2–3 cycles per deformation level (Fig. 6a). However, this type of loading is not representative of near-collapse level response, which would be unsymmetrical and would contain fewer loading cycles. Hence, the loading protocol should contain displacements that are representative of the ratcheting effect, which leads to structural collapse. Such a protocol was developed based on preliminary nonlinear analyses (Fig. 6b).

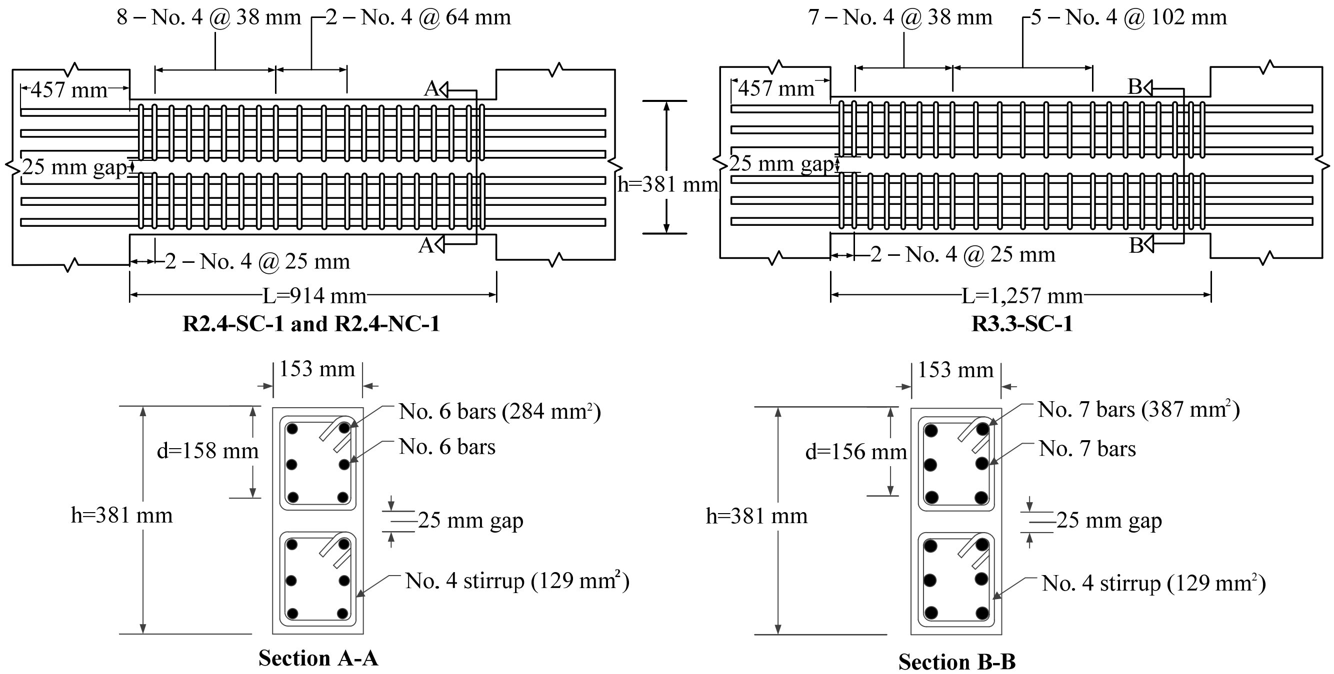

Fig. 4 – Reinforcement details of specimens

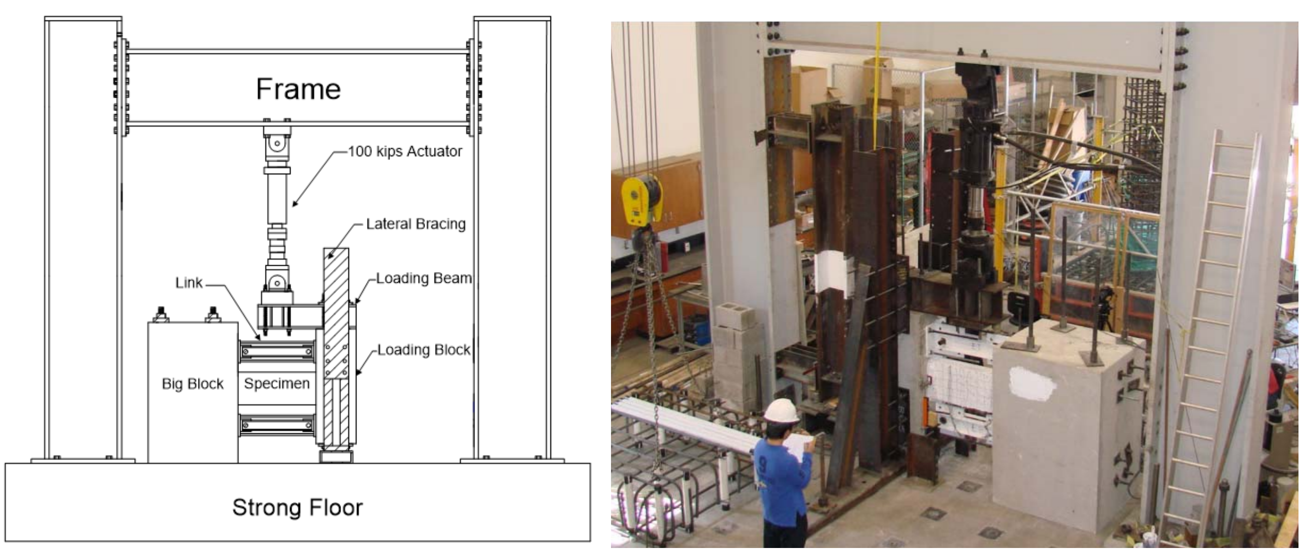

Fig. 5 – Test setup

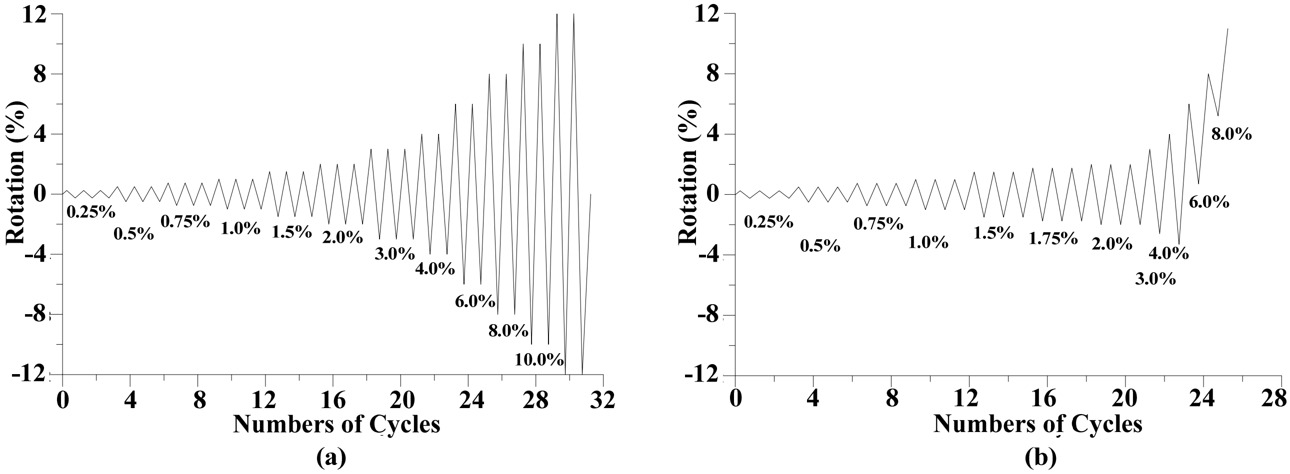

Fig. 6 – (a) Symmetrical loading protocol, (b) Near collapse loading protocol

3. Experimental Results

2.1 Cracking pattern and damage progress

The crack development and damage pattern for the DBCB specimens are very similar to each other. Progressive damage patterns of R2.4-SC-1 are shown in Fig. 7 beginning with 0.25% rotation where the initial cracking developed at the narrow unreinforced concrete strip was located. It was a shear-type cracking near the mid-span and mid-height of the beam. Upon large displacements, the cracks gradually propagated along the intended location and towards the beam's ends. The cracks eventually separated the DBCB into two slender beams where each beam had nearly twice the aspect ratio of the original coupling beam. This essentially transformed the shear-dominated behavior into a flexure-dominated behavior, thereby duplicating the behavior of conventional slender beams. Because the damage initiated from the center of the beam, then spread towards the ends, the beam ends maintained their integrity even under very large displacements, which effectively eliminated the sliding shear failure at the beam-to-wall interface; this is commonly seen in conventional coupling beams. At 1.5% beam rotation, diagonal cracks widened up to 4 mm. Canbolat et al. [17] reported that the first crack for all their diagonally reinforced coupling beam specimens occurred at 0.25% rotations. At 1.5% beam rotation, diagonal cracks widened up to 3 mm (0.12 in.). This indicated that the occurrence of initial cracks in DBCBs and their widths are similar to that of conventional DCBs. The crack patterns of R2.4-NC-1 at 11% beam rotation and of R.3.3-SC-1 at 9% beam rotation are shown in Fig. 7.

2.2 Hysteretic loops

Fig. 8a shows the shear force/stress versus beam chord rotation response for the DBCB specimen under symmetric loading protocol. It is seen in Fig. 8a that R2.4-SC-1 was able to maintain very high shear stress (~0.83√f'c (MPa)) without significant strength degradation up to a beam rotation of 6% (approximate demand for MCE level ground motions). Also, R2.4-SC-1 could still resist 80% of the peak stress at 8% rotation. In addition, R2.4-NC-1 showed no strength degradation up to 11% rotation while shear stress increased (~1.0√f'c (MPa)) when subjected to the near-collapse loading protocol as shown in Fig. 8b. Furthermore, strength in R3.3-SC-1 did not drop until 8% rotation, maintaining a shear stress of about 0.83√f'c (MPa) (Fig. 8c). All DBCB specimens has shear strengths greater than the maximum allowed factored shear shears as specified by ACI 318-14 [10].

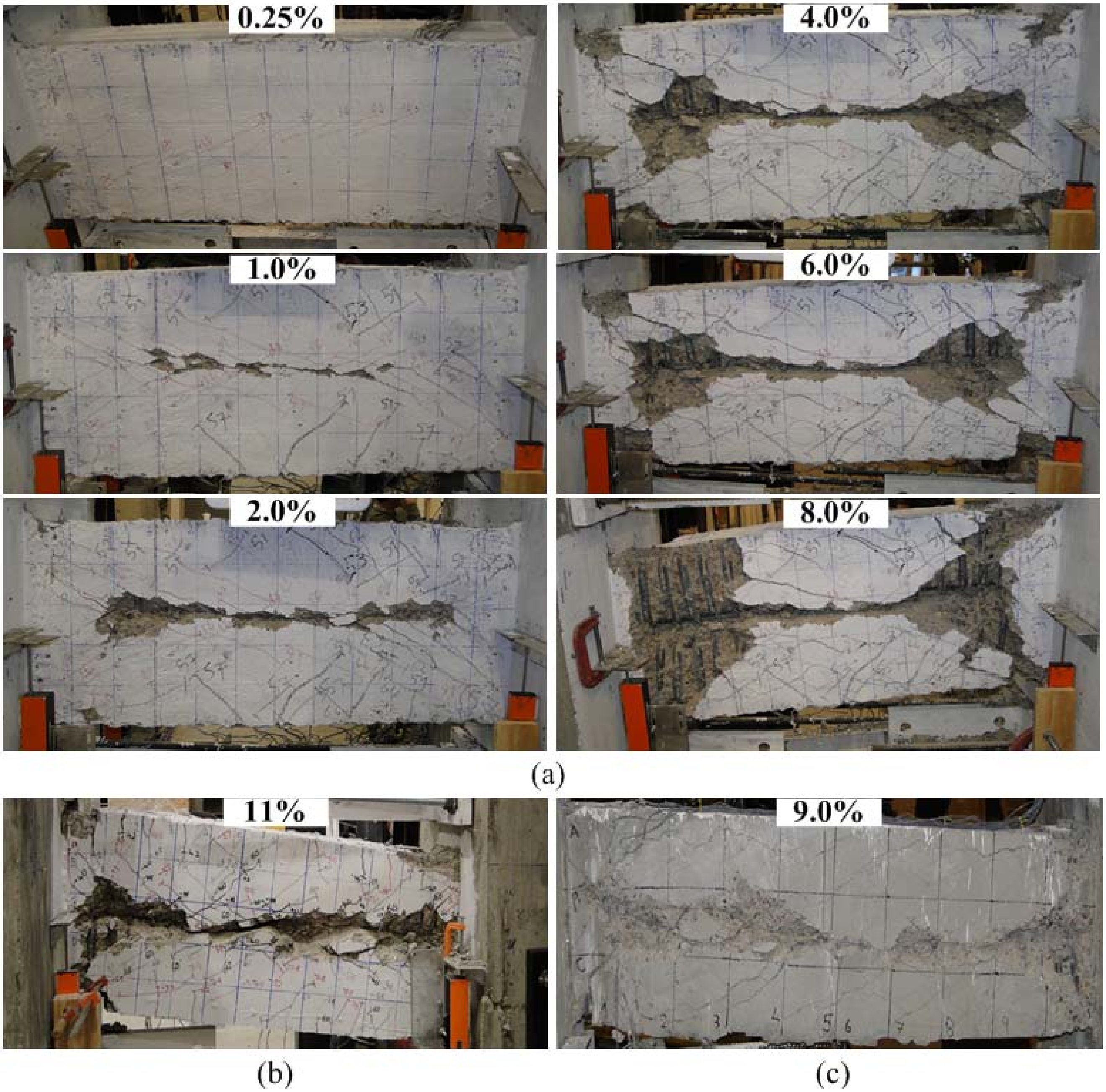

Fig. 7 – Damage pattern in DBCB specimen subjected to the symmetric cyclic loading protocol: (a) R2.4-SC-1, (b) R2.4-NC-1, and (c) R3.3-SC-1

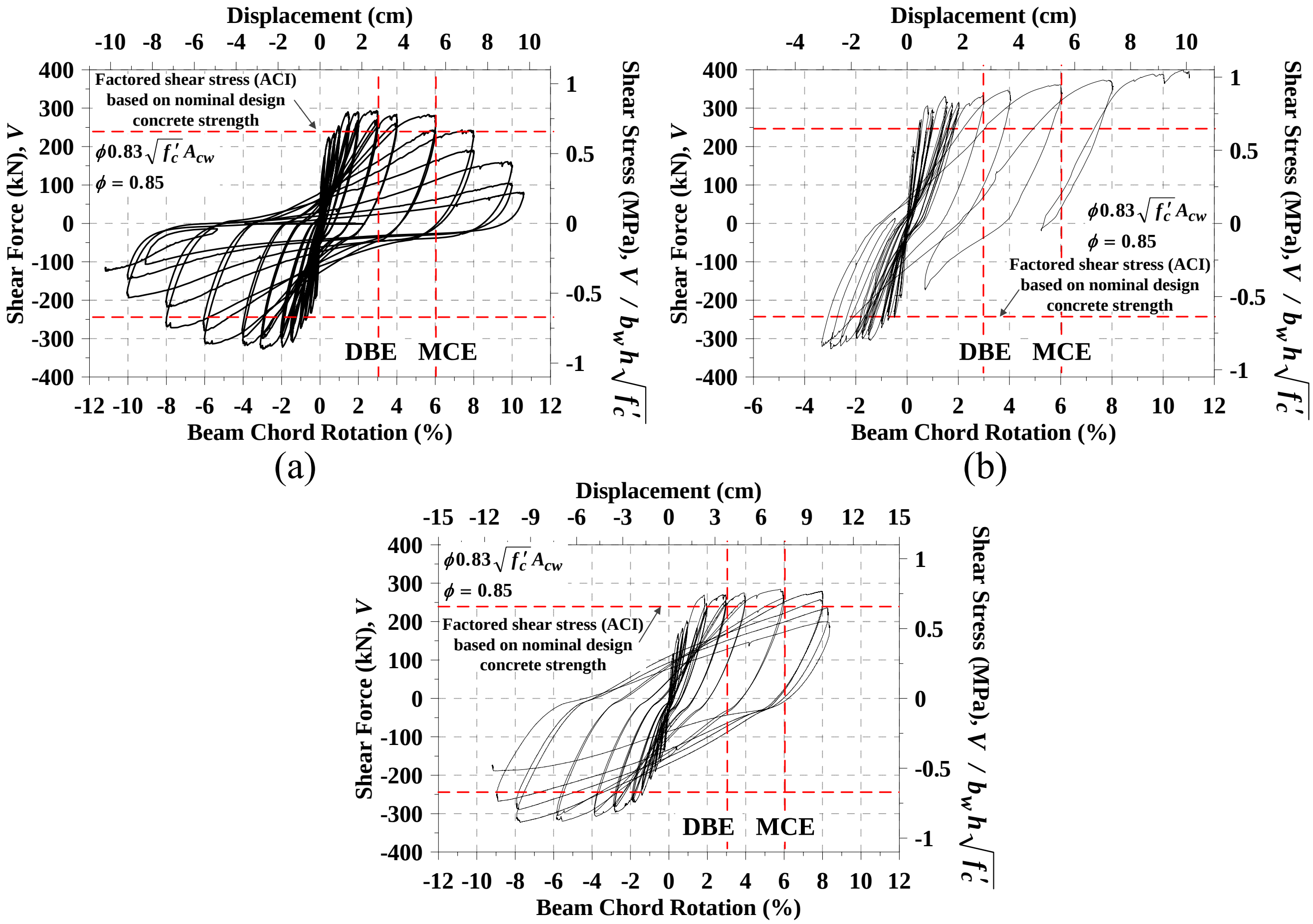

Fig. 8 – Hysteretic responses of DBCBs: (a) R2.4-SC-1, (b) R2.4-NC-1, and (c) R3.3-SC-1

2.3 Comparison between DCB and DBCB

The performance of the double-beam coupling beams (DBCBs) was compared with that of the diagonal coupling beams (DCBs) tested by Naish et al. [7]. The design of the DCBs followed current ACI design code. The DCBs can be divided into two groups according to their cross sectional design; one group had transverse reinforcement around diagonal bar groups (CB24D or CB33D), and the other group had transverse reinforcement around the entire cross section (CB24F or CB33F). Since the performance of the latter group, in terms of strength and ductility, was slightly better, the DBCBs can be compared to CB24F and CB33F. Figs. 9a and 9b show the comparisons. The shear stress, normalized by the square root of actual concrete compressive strength, was used for comparison because the specimens have different concrete compressive strengths and cross-sectional areas. Two beam chord rotations, 3% and 6%, are highlighted in the figure because previous research [4] showed that in order to maintain the integrity of the coupled wall, coupling beams will need average rotation capacities of 3% and 6% for design basis earthquakes (DBE) (10% probability of exceedance in 50 years) and maximum considered earthquakes (MCE) (2% probability of exceedance in 50 years) level ground motions, respectively. The specimens with the span-to-depth ratio of 2.4 for both the DCBs and DBCBs exceeded a shear stress level of 0.71√f'c (MPa) which is the factored ACI shear stress limit. Although the strength in the DBCB slightly decreased after 3% beam rotation during the process of the separation, the overall ductility and strength of the DBCB specimen (R2.4-SC-1) were similar to that of the DCB specimen (CB24F) up to 6% beam rotation. For the specimens with a span-to-depth ratio of 3.3, the shear strength of CB3.3F was 0.55√f'c (MPa). In addition, the strength degradation of this specimen began after 3% beam rotation. Although DBCB specimen, R3.3-SC-1, sustained much higher shear stress (about 0.83√f'c (MPa)), no strength degradation occurred up to 8% beam rotation. As discussed earlier, although DCBs can reach the nominal shear strength of 0.83√f'c (MPa), it is very difficult to construct when it has a span-to-depth ratio higher than 3. On the other hand, DBCBs can reach the high shear strength and ductility with a much simpler design and construction process. The width of adjacent walls does not have to be increased due to the required width of coupling beams because the width of DBCBs can be largely reduced compared to DCBs. In addition, the longitudinal reinforcing bars in DBCBs can be easily adjusted to accommodate the vertical longitudinal reinforcing bars in the wall boundary elements.

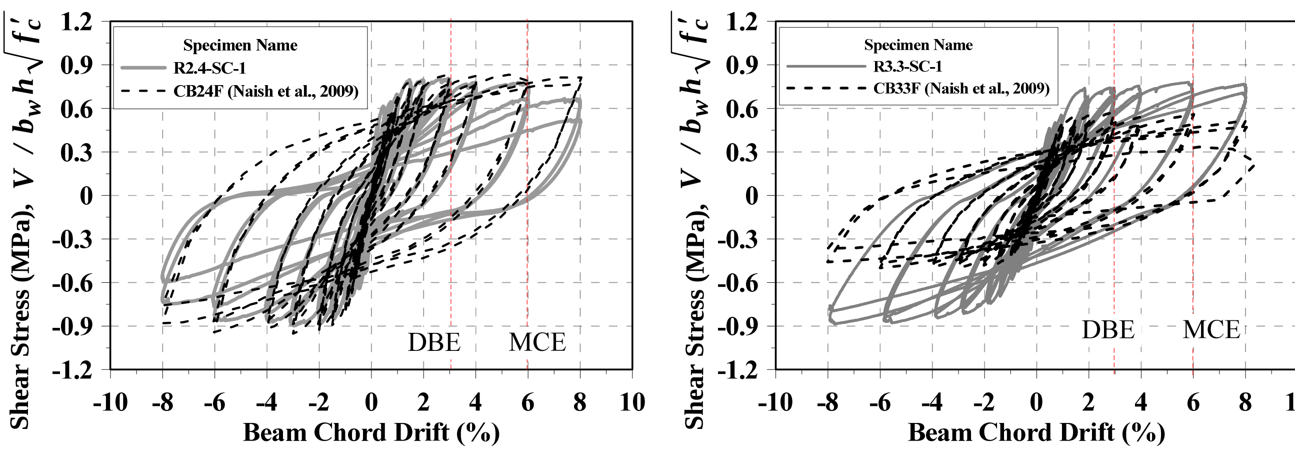

Fig. 9 – Damage patterns in DBCB specimen subjected to the symmetric cyclic loading protocol

3. Summary and Conclusions

This study proposes an innovative and simplistic reinforcing layout for RC coupling beams that significantly reduces design and construction difficulties when using diagonally reinforced coupling beams (DCBs). The proposed double-beam coupling beams (DBCBs) consist of two separate cages similar to those used for typical beams in reinforced concrete special moment frames. Upon large displacements, cracks begin developing at the DBCB's mid-span and mid-height, then gradually propagate toward the beam's ends. The cracks eventually separate the coupling beam into two slender beams where each has nearly twice the aspect ratio of the original coupling beam. This split essentially transforms the shear-dominated single coupling beam behavior into a flexure-dominated slender beam behavior. Because damage initiates from the center of the beam, and then spreads towards the ends, the beam ends are able to maintain their integrity even under very large displacements, thereby eliminating the sliding shear failure at the beam-to-wall interface. Experimental testing results on half-scale coupling beam specimens with span-to-depth ratios of 2.4 and 3.3 showed that coupling beams with the proposed reinforcement scheme were able to sustain high shear stresses ($0.83\sqrt{f'c}$ ~$1.0\sqrt{f'c}$ (MPa)) and large beam chord rotations (8~11%) before significant strength degradation occurred. Furthermore, the results showed that the ductility of a DBCB with a 2.4 span-to-depth ratio was similar to that of an ACI compliant diagonally reinforced coupling beam (DCB). DBCB with a 3.3 span-to-depth sustained greater shear stress and rotation than a DCB with the same span-to-depth ratio. In addition, because the cracks always initiate at mid-span and mid-height of the DBCBs, the damage location can be easily predicted, which makes repair work easier after moderate earthquakes.

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