Seismic Behavior of Short Coupling Beams with Different Reinforcement Layouts

by Luciano Galano and Andrea Vignoli

An experimental investigation on the seismic behavior of reinforced concrete coupling beams is presented. The reinforcement layout and the loading history were the main variables of the tests. Fifteen short coupling beams with four different reinforcement arrangements were tested. They were subjected to monotonic and cyclic loading by a suitable experimental setup. All specimens were characterized by a shear span-depth ratio of 0.75. The reinforcement layouts consisted of: a classical scheme (a); diagonal scheme without confining ties (b1); diagonal scheme with confining ties (b2); and inclined bars to form a rhombic configuration (c). Concrete compressive strengths of the specimens varied from 40 to 54 MPa. Test results showed that the beams with diagonal or rhombic reinforcement layouts behaved better than beams with longitudinal arrangement of the steel bars. These results were produced by the different resisting truss mechanisms that were developed in the coupling beams after the first cracking. The differences in energy dissipation were negligible between the diagonal and rhombic layouts. The rhombic arrangement, however, was more advantageous in terms of rotational ductility capacity, and decay in strength and stiffness of the beams. Moreover, cyclic tests demonstrated that the behavior of the rhombic layout was less affected by the different loading histories.

Keywords: cyclic loads; ductility; earthquake-resistant structures; shear walls.

INTRODUCTION

Structural walls have long been used in the planning of multistory buildings. A reliable earthquake-resistant system is represented by coupled shear walls, in which two vertical elements are interconnected by short and deep beams. Since 1977, when the American Concrete Institute (ACI 318-77) stipulated specific rules for the design of ductile reinforced concrete (RC) structures, the concept of ductility capacity has played an increasing role in the seismic design of structures. The beams of a coupled wall are subjected to large cyclic shear deformations during an earthquake. Hence, one of the most critical problems of these structures concerns the brittle failure of the low slenderness coupling beams. If this failure is avoided, a large fraction of the input seismic energy will be dissipated by these elements that are distributed throughout these walls. Consequently, building safety will be improved. This favorable behavior of coupled walls has been previously demonstrated in other research.5-6

In previous experimental research work, shear strength and hysteretic behavior of short reinforced concrete members under cyclic loads have been investigated. In the original experimental studies, the classical layout of the steel bars was used, which consisted of longitudinal reinforcements that were placed parallel to the beam axis. In later experimental studies, different arrangements of main reinforcements were proposed to improve the seismic response of these short members. In 1974, Paulay and Binney first introduced the idea of using diagonal main reinforcements that followed the bending moment distribution along the beam axis. Subsequent research showed that the beams reinforced with diagonal bars performed better than those with parallel reinforcements.7-8 These conclusions were true even when the number of transversal ties increased. Recently, different researchers proposed alternative solutions for the arrangement of the steel bars in these short coupling elements.9 Innovative coupling beams with keyways and through slits along the axis of both faces were tested. Nevertheless, these schemes complicate the construction and consequently the cost of the building.

Given the satisfactory behavior of diagonally reinforced coupling beams, their use in seismic design is recommended by European Standards.3 In a previous paper,10 the main drawbacks of the diagonal arrangement were illustrated and a different layout was proposed to overcome these disadvantages. Tegos and Penelis10 proposed an arrangement of the main steel bars with an inclination that formed a rhombic layout. Their results showed that diagonal and rhombic layouts behaved in a similar manner. By using the rhombic arrangement, however, the occurrence of diagonal explosive cleavage shear fracture was avoided. Despite the satisfactory results, direct comparisons of ultimate strength, stiffness decay, and energy dissipation were not presented.

The results of an experimental program in which 15 short coupling beams with four different reinforcement layouts were tested under monotonic and cyclic shear loading are described in this paper. The research focused mainly on the comparative behaviors of the beams in terms of failure mechanism, ultimate strength, degradation in stiffness, and hysteretic capabilities.

RESEARCH SIGNIFICANCE

When coupled shear walls are used in seismic design of buildings, a satisfactory global response is achieved only if coupling beams respond in a ductile fashion. The results of this research show that the rhombic layout of the main reinforcement improves the ductility capacity of short members under cyclic shear without a significant loss in strength and stiffness. Until now, the practical application of this layout in coupling beams has been limited. Serious problems with construction and difficulties in manufacturing can occur using diagonal layout with confining ties when the thickness of the wall is less than 250 mm. For thin coupled shear walls especially, however, the rhombic scheme simplifies building
construction and reduces the costs when compared with the diagonal arrangement.

**EXPERIMENTAL PROGRAM**

**Specimens**

The literature indicates that the strength and ductility of short coupling beams are mainly affected by: 1) the concrete strength; 2) the shear span-depth ratio; 3) the amount and the arrangement of the main reinforcement; 4) the amount of transversal reinforcement; and 5) the type of the applied loading. In this investigation, the variables that were the object of the study were the main reinforcement layout and the loading histories. There were 15 short coupling beams and four different reinforcement layouts used in this series of tests: longitudinal (a); diagonal without transversal confining ties around the main bars (b1); diagonal with transversal confining ties (b2); and rhombic layout (c). Figure 1 and Table 1 show the overall dimensions and design details of the specimens; two lateral stiff blocks were used to simulate the surrounding walls and apply the required loading histories. To emphasize the adverse shear effects in the inelastic response, the shear span-depth ratio was assumed to be constant and equal to 0.75. The cross section of all coupling beams was equal to 400 (height) x 150 mm (width) and the percentage of main reinforcement \( \rho_l = A_s/\left(hs_l\right) \) was assumed to be equal to 0.524 (four steel bars of 10 mm diameter; refer to Fig. 1).

The b2 scheme differs from b1 (Fig. 1) in the use of transverse ties that are around the four diagonal bars (22 ties of 6 mm diameter). This corresponds to the provisions detailing of Eurocode 8. The c scheme was previously proposed \(^{10}\) and used here for a direct comparison with b1 and b2 by using an equal amount of main reinforcements. Transverse vertical ties followed the capacity design approach. For the specimens of series a the provisions of Eurocode 2 \(^{11}\) were used. For the series c, the suggestions given in Reference 10 for the minimum transverse reinforcement ratio were utilized. Finally, for b1 and b2 schemes, two 6 mm diameter longitudinal bars were placed on the top and the bottom sides of the sections, and a minimum quantity of vertical ties was used (as proposed in Eurocode 8). Table 1 gives designations, reinforcement arrangements, main (\( \rho_l \)) and transversal (\( \rho_v \)) reinforcement ratios for all coupling beams (\( \rho_v \) was defined as the ratio between the total volume of vertical stirrups inside the beams and the concrete volume \( V_c = h s_l H \)).

**Materials**

The steel used for all reinforcements consisted of hot-rolled deformed bars of Fe B 44k grade. The main reinforcement consisted of four 10 mm diameter bars. Bars of 6 and 8 mm were used for vertical ties that were placed with different spacing for the different series of specimens (Fig. 1 and Table 1). Stress-strain tests were undertaken on the reinforcing steel using a universal testing machine. The strains were measured using electric strain gages over a gage length of \( 5 \phi_{nom} \) (\( \phi_{nom} \) is defined as the nominal diameter of the bar). A total number of 20 tests were performed. Average values obtained by the tests were yield strength \( f_{ys} \) equal to 567 MPa (variation in the range 528 to 611 MPa) and ultimate strength \( f_{ut} \) of 660 MPa (variation in the range 641 to 685 MPa). Prior to the fracture of the steel bars, the minimum elongation measured over a length of \( 5 \phi_{nom} \) was 18.3%. No significant differences in modulus of elasticity of the steel bars were found and the average measured value was \( E_s = 206 \times 10^3 \) MPa.

A normal-strength commercial concrete mixture was used and the cement was portland cement Type 425. The concrete was specified to have a 10 mm maximum aggregate size with complete grain size distribution (Fuller bounds) and average water-cement ratios (w/c) = 0.40 were used. Workability was achieved by adding to the mixture a normal-setting water reducer that was approximately 1% of the cement weight. The
specimens were cast horizontally in reusable steel forms. The prescribed nominal value of the concrete cover (equal to 15 mm) was obtained using suitable spacers placed in the bottom (Fig. 2). The specimens were cured at a temperature of 20°C and with relative humidity of 50% for a minimum of 28 days. Subsequently, they were placed in a covered environment that was open to the elements and were subjected to normal atmospheric conditions until the testing. To assess the seismic performances of the coupling beams to an age corresponding to the real conditions of serviceability, experimental tests were conducted after a time that varied from 4 to 5 years from the time of casting. Therefore, the mechanical characteristics of the concrete were determined on cylinder cores extracted from the lateral blocks of each specimen after the test. Maximum care was taken during the extraction of the cores to avoid any disturbance. Three compression tests for each specimen were performed (average dimensions of each cylinder were: 130 mm in height by 65 mm in diameter).

For Specimens P02, P03, P14, P15, and P16, the secant modulus of elasticity at a stress level of approximately 0.3 times the ultimate strength was determined. For these five specimens, one cylinder 150 mm in height by 35 mm in diameter was extracted to evaluate the direct tensile strength. Table 1 gives: 1) the specific weight $\gamma_c$, kN/m$^3$; 2) the average values of compressive strength $f_{cm}$, MPa; 3) direct tensile strength $f_{ct}$, MPa; and 4) secant moduli $E_{cs}$, MPa. The obtained average compressive strengths were similar for $a$, $b_1$, and $c$ series (approximately 48 MPa) but the compressive strength of the $b_2$ series was less (approximately 42.7 MPa). The obtained abnormal compressive strengths of $b_2$ were probably due to the casting and curing conditions that occurred in the concrete, which had a greater complexity of steel bars arrangements in the $b_2$ specimens.

### Experimental procedure and instrumentation

Short coupling beams were tested in a vertical plane under conditions of asymmetric bending and constant shear (Fig. 3). Six steel rollers were used to constraint the specimens in a fabricated stiff testing steel frame. Two rollers, placed laterally, were used to prevent undesirable horizontal movements of the specimens. The other four rollers were utilized to produce the desired loading histories. To assess the seismic performances of the coupling beams to an age corresponding to the real conditions of serviceability, experimental tests were conducted after a time that varied from 4 to 5 years from the time of casting. Therefore, the mechanical characteristics of the concrete were determined on cylinder cores extracted from the lateral blocks of each specimen after the test. Maximum care was taken during the extraction of the cores to avoid any disturbance. Three compression tests for each specimen were performed (average dimensions of each cylinder were: 130 mm in height by 65 mm in diameter).

![Fig. 2—Reinforcement layouts $b_2$ and $c$ of specimens in steel form before casting.](image)

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For Specimens P02, P03, P14, P15, and P16, the secant modulus of elasticity at a stress level of approximately 0.3 times the ultimate strength was determined. For these five specimens, one cylinder 150 mm in height by 35 mm in diameter was extracted to evaluate the direct tensile strength. Table 1 gives: 1) the specific weight $\gamma_c$, kN/m$^3$; 2) the average values of compressive strength $f_{cm}$, MPa; 3) direct tensile strength $f_{ct}$, MPa; and 4) secant moduli $E_{cs}$, MPa. The obtained average compressive strengths were similar for $a$, $b_1$, and $c$ series (approximately 48 MPa) but the compressive strength of the $b_2$ series was less (approximately 42.7 MPa). The obtained abnormal compressive strengths of $b_2$ were probably due to the casting and curing conditions that occurred in the concrete, which had a greater complexity of steel bars arrangements in the $b_2$ specimens.

### Table 1—Details of test specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Reinforcement arrangement</th>
<th>$\rho_l$</th>
<th>$\rho_c$</th>
<th>$\gamma_c$, kN/m$^3$</th>
<th>$f_{cm}$, MPa</th>
<th>$f_{ct}$, MPa</th>
<th>$E_{cs}$, MPa</th>
<th>Loading history</th>
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<tbody>
<tr>
<td>P01</td>
<td>a</td>
<td>0.524</td>
<td>0.84</td>
<td>21.36</td>
<td>48.9</td>
<td>—</td>
<td>—</td>
<td>M</td>
</tr>
<tr>
<td>P02</td>
<td>a</td>
<td>0.524</td>
<td>0.84</td>
<td>21.78</td>
<td>44.5</td>
<td>3.3</td>
<td>24,464</td>
<td>C1</td>
</tr>
<tr>
<td>P03</td>
<td>a</td>
<td>0.524</td>
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<td>4.1</td>
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<td>C2</td>
</tr>
<tr>
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<td>48.7</td>
<td>—</td>
<td>—</td>
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<td>0.39</td>
<td>20.76</td>
<td>39.9</td>
<td>—</td>
<td>—</td>
<td>M</td>
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<td>b1</td>
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<td>46.0</td>
<td>—</td>
<td>—</td>
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<td>—</td>
<td>—</td>
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<tr>
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<td>53.4</td>
<td>—</td>
<td>—</td>
<td>C2</td>
</tr>
<tr>
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<td>b2</td>
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<td>21.45</td>
<td>46.8</td>
<td>—</td>
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<td>P11</td>
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<td>—</td>
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<td>41.6</td>
<td>—</td>
<td>—</td>
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<tr>
<td>P13</td>
<td>c</td>
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<td>0.55</td>
<td>21.37</td>
<td>47.5</td>
<td>—</td>
<td>—</td>
<td>M</td>
</tr>
<tr>
<td>P14</td>
<td>c</td>
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<td>45.0</td>
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<td>23,436</td>
<td>C1</td>
</tr>
<tr>
<td>P15</td>
<td>c</td>
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<td>0.55</td>
<td>21.34</td>
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<td>3.8</td>
<td>24,304</td>
<td>C2</td>
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<td>c</td>
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<td>51.1</td>
<td>3.5</td>
<td>27,342</td>
<td>C3</td>
</tr>
</tbody>
</table>

Note: 1 MPa = 145 psi; 1 kN = 225 lbf; $f_{cm}$ = mean cylinder compressive strength; M = monotonic loading; and C = cyclic loading.
Instrumentation consisted of four linear variable displacement transducers (LVDTs) to measure vertical displacements $\delta_1$, $\delta_2$, $\delta_3$, and $\delta_4$ of the lateral blocks. Additional LVDTs were placed to evaluate accidental movements of the specimens in the horizontal direction ($O_1$ and $O_2$). The two diagonal deformations of the coupling beams on both sides were also measured. Electric signals of the LVDTs and the load values $P_1$ and $P_2$ were continuously measured and recorded during the tests. Due to the dimensional and machine tolerances of the testing setup, the rotations $\theta_B1$ and $\theta_B2$ were slightly different; therefore, the following equations were used to evaluate the rotation $\theta_L$ and the shear $V$ in the coupling beams (Fig. 3)

$$\theta_L = \frac{\theta_{L1} + \theta_{L2}}{2} = \frac{\theta_{B1} + \theta_{B2}}{2} + \theta_G = \frac{0.4}{50}$$

the rotation of the beam axis

$$V = P_1 - R_1 = P_1 \left(1 - \frac{c}{a} \right)$$

Table 1 and Fig. 5 show the different loading histories used for all the tests; $\theta_{ly}$ indicates the yielding rotation of the coupling beams evaluated as specified later in this paper. Three different types of cyclic patterns were used to investigate the effects on strength and stiffness. Tests were conducted applying a velocity of approximately 0.4 mm/s to the actuator’s stroke. The total testing time for each specimen tested using monotonie loading was approximately 50 min. The duration of the cyclic tests varied considerably for different specimens, from approximately 100 to 180 min.

**STATIC AND CYCLIC DUCTILITY IN SHORT COUPLING BEAMS**

The case to be considered concerns a short beam subjected to monotonic increasing shear distortion in an RC coupled wall. Strength and ductility of the member are related to the
shear load \((V)\) rotation \((\theta_L)\) diagram (Fig. 6(a)). A measurement of static ductility \(\mu_s\) is given by

\[
\mu_s = \frac{\theta_{Lu}}{\theta_{Ly}}
\]

where \(\theta_{Ly}\) (yielding rotation) is defined by the 3/4 rule (Fig. 6(a)), and \(\theta_{Lu}\) is the rotation at the point when the shear is reduced to 0.85 times the peak value \(V_u\). The ductility ratio, as suggested by Eq. (3), is largely used in the literature but still has several drawbacks. The main difficulty is that there is no distinction between hinges with a large residual carrying capacity and hinges with no residual capacity after the beam softens to 85% of the peak load. Alternative measurements of ductility are similar to those adopted for flexural toughness and are based on the area under the shear load \(V\) versus \(\theta_L\) curve (Fig. 6(b)). Two ductility indexes are denoted as \(I_5\) and \(I_{10}\); \(I_5\) is the ratio between the area OACD and the area OAB, whereas \(I_{10}\) is the area OACEF divided by the area OAB. Rotation \(\theta_{Ly}\) is defined by the 3/4 rule, as previously given. For a perfectly elastoplastic material, values of \(I_5 = 5\) and \(I_{10} = 10\) are obtained. In cyclic tests, energy dissipation and the concept of cumulative ductility are used to assess the behavior of an RC member under alternating load in the inelastic field. In the present study, the cumulative ductility index \(\mu_{cum}\) as a function of the number of half-cycles was defined by the equations given as follows (the meaning of \(\theta_{L1}, \theta_{Ly}, \text{ and } \Delta\theta_{L2}\) is defined in Fig. 6(c))

\[
\mu_{cum1} = \mu_1 = \frac{\theta_{L1}}{\theta_{Ly}}
\]

\[
\mu_{cum2} = \mu_{cum1} + \Delta\mu_2
\]

\[
\Delta\mu_2 = \frac{\Delta\theta_{L2}}{\theta_{Ly}}
\]

Equation (4) applies only if \(\theta_{L1} > \theta_{Ly}\). Decay of the peak loads \((V_1, V_2, V_3, \text{ etc.) and of the secant stiffnesses } (k_1, k_2, k_3, \text{ etc.) are functions of the cumulative ductility and represent further characteristics of the progressive damage of the members. The seismic behavior of the short coupling beams was investigated based on the following observed characteristics: (1) strength, stiffness, and static ductility as given by monotonic tests, (2) the degradation of strength and stiffness, and (3) energy dissipation based on the cyclic test results.

**Shear-rotation responses**

Results of monotonic tests are summarized in Table 2. The comparison of the load-rotation responses of Tests P01, P05, P10, and P13 is given in Fig. 7. Shear strength \(V_u\) is identified in the diagrams with the maximum values of the shear force \(V\). The configuration of the bars with diagonal reinforcement layouts (P05 and P10) give higher strengths and elastic stiffness \(k_{y}\); small differences between \(b_1\) and \(b_2\) were caused by the different concrete compressive strength. Lower strength values were obtained with the reinforcement layouts \(a\) and \(c\), which corresponded to a decrease of 7.5 and 17%. The percentages of reduction of elastic stiffness between diagonal and the other schemes of reinforcement were more pronounced. Specimens P05 and P10 produced a rotation ductility factor \(\mu_s\) of approximately 8. This value was in accordance with theoretical calculations for these structural...
members. Tests of P01 and P13 specimens produced ductility factor values equal to 6 and 9.3. Finally, the computed $I_5$ and $I_{10}$ ductility factors were similar for all the specimens.

As shown in Fig. 7, the tests were conducted until the coupling beams collapsed due to the fracture of one or more reinforcing bars or when the carried load was reduced more than $2/3$ of the reached maximum load. Specimen P01 showed the typical cracking diagonal pattern resulting from the shear loading, and, after the yield point, the increase of the shear force was negligible. The collapse occurred due to the subsequent failure of the stirrups at the top side of the specimen that resulted in the abrupt decreasing of the sustained load (Fig. 8). Specimens P05 and P10 showed a more ductile response on failure; after the diagonal cracking, two main vertical cracks appeared in the sections where the maximum bending moment was located. This allowed the coupling beams to develop high rotations after yielding without significant loss of strength. The collapse occurred due to the instability of the compressive strut that caused a gradual decreasing of the shear without fracture of steel bars (Fig. 8). Finally, the test on Specimen P13 was characterized by: 1) a large increment of carried shear after yielding; and 2) very high rotation values. A sudden failure, however, occurred due to the fracture of two diagonal reinforcing bars. Before the failure, the specimen developed two distinct separate zones of diagonal cracking. This was followed by the splitting of the concrete on the top and the bottom sides of the sections that were attached to the two lateral blocks.

A comparison between experimental ($V_u$) and theoretical ($V_u^*$) ultimate strengths is given in Table 2. For the $a$ scheme of reinforcement, shear $V_u^*$ was computed based on ultimate flexural capacity using Eurocode 2; for $b_1$ and $b_2$ configurations, $V_u^*$ was calculated by adding the contribution of the flexural minor reinforcements to the shear force carried by the main diagonal bars calculated, as suggested in Eurocode 8. For the $c$ configuration, the truss model proposed in Reference 10 was used. The previously given yielding strength values $f_{sy}$ were used for all the previously mentioned calculations. Theoretical shear values were notably less than experimental values. This was especially true for the $c$ reinforcement layout. These discrepancies between the theoretical and the experimental results were mainly due to the overstrength produced by the hardening of the steel. Therefore, the theoretical truss schemes used for the previsions of ultimate shear are reliable for design purposes.

**Strength and stiffness degradation**

The reduction of strength and stiffness of the reinforced concrete members subjected to cyclic loading is significant for structures in seismic areas. Therefore, members with significant degradation of strength and stiffness due to the imposition of severe cyclic loading must be avoided in seismic design. The reductions of the shear load $V_i$ corresponding to the maximum rotation in each half-cycle versus cumulative ductility

<table>
<thead>
<tr>
<th>Specimen</th>
<th>$V_u$, kN</th>
<th>$V_u^*$, kN</th>
<th>$θ_{ly}$, rad $× 10^3$</th>
<th>$θ_{Lu}$, rad $× 10^3$</th>
<th>$μ_s$</th>
<th>$I_5$</th>
<th>$I_{10}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>P01 ($a$)</td>
<td>223.9</td>
<td>211.2</td>
<td>8.42</td>
<td>50.89</td>
<td>6.04</td>
<td>4.61</td>
<td>9.15</td>
</tr>
<tr>
<td>P05 ($b_1$)</td>
<td>239.3</td>
<td>198.6</td>
<td>8.42</td>
<td>65.41</td>
<td>7.77</td>
<td>4.59</td>
<td>9.26</td>
</tr>
<tr>
<td>P10 ($b_2$)</td>
<td>241.1</td>
<td>198.6</td>
<td>7.75</td>
<td>62.04</td>
<td>8.00</td>
<td>4.57</td>
<td>9.11</td>
</tr>
<tr>
<td>P13 ($c$)</td>
<td>200.5</td>
<td>155.6</td>
<td>8.98</td>
<td>83.63</td>
<td>9.31</td>
<td>4.37</td>
<td>8.97</td>
</tr>
</tbody>
</table>

Note: 1 kN = 225 lbf.

![Fig. 8](image)

Fig. 8—State of specimens after failure stage under monotonic loading: (a) Specimen P01 (layout $a$); and (b) Specimen P10 (layout $b_2$).
to approximately 5. The crushing of the concrete strut produced the instability of the diagonal reinforcements. This was followed by a noticeable loss in strength for cumulative ductility greater than 20. In spite of the absence of the square ties around the diagonal reinforcements, the instability phenomenon did not occur during the tests of Specimens P06 and P07 (b1 layout). Consequently, a less pronounced decay in the strength occurred.

Specimen P02, with main reinforcement placed parallel to the beam axis, demonstrated ratios $V_i/V_u$ comparable with Specimens P06 and P07 but with lower values in strength when the $\mu_{cum}$ value was approximately 40 (Fig. 9(a)). Cumulative ductility values related to the significant ratio $V_i/V_u$ equal to 0.5 were approximately 40 for the b2 layout, 80 for the a and b1 layouts, and 80 for the c layout. Similar conclusions can be drawn from Fig. 9 (b) and (c). With the loading history C2, Specimens P03 and P08 give similar responses within the range in which $\mu_{cum}$ reached values of 60. For values of $\mu_{cum}$ higher than 60, Specimen P15 demonstrated better performances. Finally, tests on Specimen P03 revealed a greater decay in the strength corresponding to intermediate ductility values.

Figure 10(a), (b), and (c) show the decay in the stiffness and plots the ratio $k_i/k_y$ versus cumulative rotation ductility. The secant stiffness values in each half-cycle are $k_i$ (Fig. 6(c)). Again, the c layout of reinforcement demonstrates its superiority for higher values of $\mu_{cum}$. Figure 11 shows the final results of the tests on Specimens P03, P07, P12, and P16. Specimen P12, which crushed due to the instability of the main reinforcements, shows a minor spalling of concrete. The P16 specimen concrete spalled near the four corners of the short beam, but no degradation occurred in the central part. This behavior was due to the favorable effect of concrete confinement produced by the rhombic configuration of the bars.

### Energy dissipation characteristics

The energy dissipation of the specimens under cyclic loading was defined as the area enclosed by the shear-rotation hysteresis loops. Graphs of cumulative dissipated energy $E_h$ versus half-cycles number are plotted in Fig. 12(a), (b), and (c) for the different loading histories. Table 3 presents the values of $E_h$ at 20, 30, and 40 half-cycles for C1, C2, and C3 histories together with the associated cumulative ductility $\mu_{cum}$. Figure 12(a) and (b) shows a slight
but observable superior behavior of Specimens P07 and P08 compared with Specimens P14 and P15. This superiority was due to a more acceptable form of the hysteresis loops. Specimens P02, P03, and P04 demonstrated lower energy dissipation. This was mainly due to the pinched form of the hysteresis loops (Fig. 13). Figure 12(a) shows that Specimens P11 and P12 suffered failures that were caused by the struts’ instability. In the first 15 half-cycles, however, the dissipated quantities of energy were comparable to the energies dissipated by Specimens P06 and P07. From the results of the experimental tests on specimens of b1, b2, and a configuration, no significant differences in energy dissipation were observed when the loading history was varied. The test on Specimen P16 (c layout) that was subjected to a C3 loading history produced higher quantities of $E_h$ compared with P14 and P15 tests.

**CONCLUSIONS**

The previously discussed results lead to the following conclusions:

1. The rhombic layout of the main reinforcements gave the highest rotational ductility values (greater than 9.0 in the monotonic test). Values approximately equal to 8.0 were obtained using the diagonal arrangement. The rhombic layout, however, produced lower values of strength with the same geometrical percentage of steel area. This reduction was approximately equal to 17%.

2. The concrete compressive strength greatly affected the seismic behavior of the coupling beams that were reinforced with diagonal bars (compressive strength of 48 MPa for b1 specimens, and 42.7 MPa for b2 specimens). Despite the fact that the diagonal bars were confined by the square ties, specimens with the lower values of $f_{cm}$ suffered a short time failure for instability of the compressive strut. Specimens of b1 series with the greater values of $f_{cm}$ behaved better and the instability phenomenon was avoided. This different behavior resulted in the cyclic tests in terms of different energy dissipation that reached average value of 59.5 kNrad for coupling beams of b1 series, and 34.8 kNrad for coupling beam of b2 series;

3. A notable superiority of the rhombic layouts in carrying the shear load under repeated loading at high rotation ductility was obtained (Fig. 9);

4. Using the rhombic configuration, the occurrence of the diagonal explosive cleavage shear fracture was avoided;

**Table 3—Dissipated energy and cumulative ductility data in cyclic tests**

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Reinforcement arrangement</th>
<th>No. of half-cycles</th>
<th>$E_h$, kN × rad</th>
<th>$\mu_{cum}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>P02 (C1)</td>
<td>a</td>
<td>20</td>
<td>33.47</td>
<td>101.89</td>
</tr>
<tr>
<td>P03 (C2)</td>
<td>a</td>
<td>30</td>
<td>26.38</td>
<td>76.46</td>
</tr>
<tr>
<td>P04 (C3)</td>
<td>a</td>
<td>40</td>
<td>30.09</td>
<td>134.77</td>
</tr>
<tr>
<td>P06 (C1)</td>
<td>b1</td>
<td>20</td>
<td>52.39</td>
<td>94.84</td>
</tr>
<tr>
<td>P07 (C1)</td>
<td>b1</td>
<td>20</td>
<td>64.97</td>
<td>99.60</td>
</tr>
<tr>
<td>P08 (C2)</td>
<td>b1</td>
<td>30</td>
<td>61.18</td>
<td>71.48</td>
</tr>
<tr>
<td>P11 (C1)</td>
<td>b2</td>
<td>16</td>
<td>34.51</td>
<td>49.46</td>
</tr>
<tr>
<td>P12 (C1)</td>
<td>b2</td>
<td>16</td>
<td>34.99</td>
<td>53.90</td>
</tr>
<tr>
<td>P14 (C1)</td>
<td>c</td>
<td>20</td>
<td>65.14</td>
<td>94.48</td>
</tr>
<tr>
<td>P15 (C2)</td>
<td>c</td>
<td>30</td>
<td>52.72</td>
<td>73.99</td>
</tr>
<tr>
<td>P16 (C3)</td>
<td>c</td>
<td>40</td>
<td>79.52</td>
<td>115.37</td>
</tr>
</tbody>
</table>

Note: 1 kN = 225 lbf.
5. Comparable energy dissipation quantities were achieved with the diagonal and the rhombic layouts. A slight superiority, however, of the rhombic configuration that dissipated an average hysteretic energy of approximately \(65.8 \text{kN rad} \) was achieved (9.6\% more than the energy dissipated by the specimens with the diagonal configuration); and

6. Using this study’s test results, the analytical formulation for the ultimate strength proposed by Tegos and Penelis\(^{10}\) was verified. The test results indicated that the theoretical
truss scheme used for the previsions of ultimate shear is safe for design purposes.

Overall, the results of this study confirm better seismic performances using inclined (rhombic) layout of reinforcing bars for short coupling beams. This layout demonstrated optimal ductility resources and capacity of sustained high-shear forces under repeated loading. This layout of reinforcement is easier to fabricate than the diagonal layout that involves confinement ties. Another disadvantage of the diagonal layout when used in thin RC walls is the difficulties of the placement of the steel bars when the RC walls are being fabricated.

Given the optimal seismic performances and the simplicity of detailing, it is hoped that the rhombic layout of main reinforcements of coupling beams will be used with increasing confidence in aseismic buildings.

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NOTATION

\[ A_s = \text{area of main reinforcements} \]
\[ a, b = \text{distances of lateral rollers from axis of applied loads} \]
\[ c = \text{distance between axes of applied loads} \]
\[ E_{cs} = \text{concrete secant modulus} \]
\[ E_s = \text{dissipated hysteretic energy} \]
\[ E_E = \text{average modulus of elasticity of steel reinforcements} \]
\[ f_{cm} = \text{concrete compressive strength on day of beam testing} \]
\[ f_{ct} = \text{concrete tensile strength on day of beam testing} \]
\[ f_{st} = \text{average ultimate strength of steel reinforcements} \]
\[ f_{ys} = \text{average yield strength of steel reinforcements} \]
\[ h = \text{height of coupling beams} \]
\[ I_{c}, I_{t0} = \text{toughness ductility measures} \]
\[ k_s = \text{secant stiffness at yield point} \]
\[ k_1, k_2, \ldots, k_i = \text{secant stiffnesses at end of each half-cycle} \]
\[ l = \text{distance between extreme supports of specimen} \]
\[ l_P = \text{length of coupling beams} \]
\[ M = \text{bending moment} \]
\[ P_1, P_2 = \text{loads applied to specimens} \]
\[ R_1, R_2 = \text{reactions of boundary supports} \]
\[ s = \text{thickness of coupling beams} \]
\[ V = \text{shear force} \]
\[ V_c = \text{concrete volume of beam} \]
\[ V_u = \text{ultimate shear force} \]
\[ V_1, V_2, \ldots, V_i = \text{shear force values at end of each half-cycle} \]
\[ \gamma_c = \text{concrete specific weight} \]
\[ \Delta \theta_y = \text{accumulated plastic rotation in each half-cycle} \]
\[ \Delta \mu = \text{accumulated rotation ductility in each half-cycle} \]
\[ \delta_1, \delta_2, \delta_3, \delta_4 = \text{displacements of specimen during test} \]
\[ \phi_{nom} = \text{nominal diameter of steel bars} \]
\[ \mu = \text{cumulative ductility} \]
\[ \mu_s = \text{static rotation ductility} \]
\[ \rho_\alpha, \rho_v = \text{main and transversal reinforcement ratios} \]
\[ \theta_{BL}, \theta_{G} = \text{rotations of two lateral blocks} \]
\[ \theta_{BL}-\theta_{2} = \text{right and left rotations of coupling beam axis} \]
\[ \theta_{G} = \text{rotation of coupling beam axis respect to horizontal direction} \]
\[ \theta_L = \text{average rotation of coupling beam axis} \]
\[ \theta_{Lu} = \text{rotation at yield point} \]
\[ \theta_{Lu} = \text{ultimate rotation} \]
\[ \theta_{Lu}, \theta_{Lu1}, \ldots, \theta_{Lu} = \text{rotation values at end of each half-cycle} \]

REFERENCES