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# Strengthening of column splices in infilled shear walls

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ABSTRACT: Strengthening of existing reinforced concrete moment resisting frames often involves addition of infill walls. The benefit of such strengthening is often limited by failure of splices in existing columns which act as boundary elements for new walls. Several different approaches for strengthening column splices were examined. Twelve two-third scale column specimens were constructed with column splice details typical of ordinary moment-resisting frames and were strengthened using a variety of techniques. Providing continuity or confinement in the splice region significantly improved strength and ductility.

## 1 INTRODUCTION

Reinforced concrete (R/C) moment-resisting frames designed and constructed in the 1950's are often found to be inadequate for resisting seismic forces. Gravity loading was of primary concern during design of these frames, and as a result, lateral strength and ductility were minimal.

Retrofitting of such frames often involves construction of cast-in-place or shotcrete infill walls which provide an efficient means of augmenting the lateral capacity of the structural system. Existing columns act as boundary elements for the new walls. As a result, some columns in the frame are subjected to large axial tensile forces which may exceed the capacity of column splices that were originally designed for little or no flexure and only axial compression (see Fig. 1). The benefit of strengthening the R/C moment-resisting frames using infill walls is thus limited by the failure of column splices under the action of large axial tensile forces.

In a previous study by Gaynor (1988), a series of non-ductile R/C frames were strengthened using shotcrete infill walls. Specific features of these frames were compression splices at the base of the columns and minimal column ties in the splice region. The strengthened frames were tested by subjecting them to in-plane reversed cyclic lateral loads. Test results indicated that the behavior of the frame-wall system was controlled by the failure of column splices. Shah (1988) and Jimenez (1989) investigated improvements in performance of the strengthened structure by modifying the details in the boundary elements.

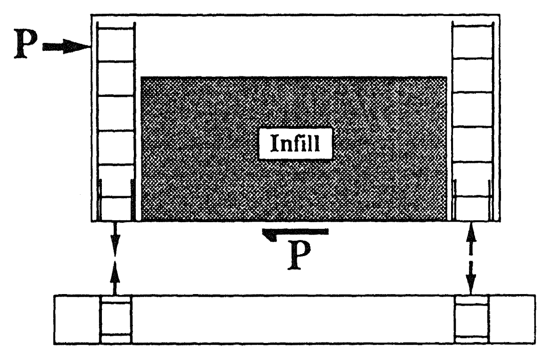



**Fig. 1 Axial forces in columns resulting from overturning of infill walls**

The major objective of this investigation is to develop different techniques for strengthening column splices to permit development of tensile yield during reversed cyclic loading.

## 2 STRENGTHENING APPROACH

Two approaches for strengthening column splices were examined. The first consisted of making spliced bars continuous so that forces could be transferred directly without relying on the bond strength between spliced bars and surrounding concrete. The other involved improving confinement in the column splice region to improve bond along the spliced bars.

## 3 EXPERIMENTAL PROGRAM

### 3.1 *Test specimens*

*Existing columns*

The prototype member selected for study was a 450 mm x 450 mm reinforced concrete column with 30 mm diameter longitudinal bars and 10 mm diameter ties. The test specimen was a two-third scale model of the prototype column. It was designed and detailed in accordance with ACI 318-56 (1956), which specifies a column compression splice length of 24 longitudinal bar diameters and tie spacing of 300 mm. Ties were fabricated with 90 degree hooks, typical of 1950's construction. Details of the test specimen with the existing (unstrengthened) lap splices are shown in Fig. 2. Splices were located at mid-height of the test specimen. The strength of the existing splice was estimated using an equation developed by Orangun, et al (1977). The equation provides a means of calculating the strength of splices, considering concrete cover, splice length, bar diameter, tie size and spacing, and concrete compressive strength.

*Strengthening procedures*

Twelve test specimens were constructed. One specimen was not strengthened to provide a bench mark. Two specimens were strengthened by providing continuity in the splice region. Splices in these two specimens were constructed as non-contact splices with a 6 mm gap between reinforcing bars. This was done in order to simulate adverse field conditions where spliced bars are not in contact after construction is completed. Continuity was provided by welding the bars together with a 10 mm diameter bar serving as a filler. Weld was designed in accordance with ACI 318-89 (1989). One of the two specimens with welded splices was provided with a 10 mm diameter tie near the end of the outer spliced bar to restrain the outward thrust produced by eccentricity between the spliced bars (see Fig. 3).

Nine specimens were strengthened by improving confinement in the splice region. Three different schemes were selected. The first scheme involved provision of steel angles along the column corners over the splice region with steel straps connecting the angles. In the second scheme, the splice region was confined with external steel rebar ties. In the third scheme, additional ties were placed in the splice region by removing concrete cover at each tie location.

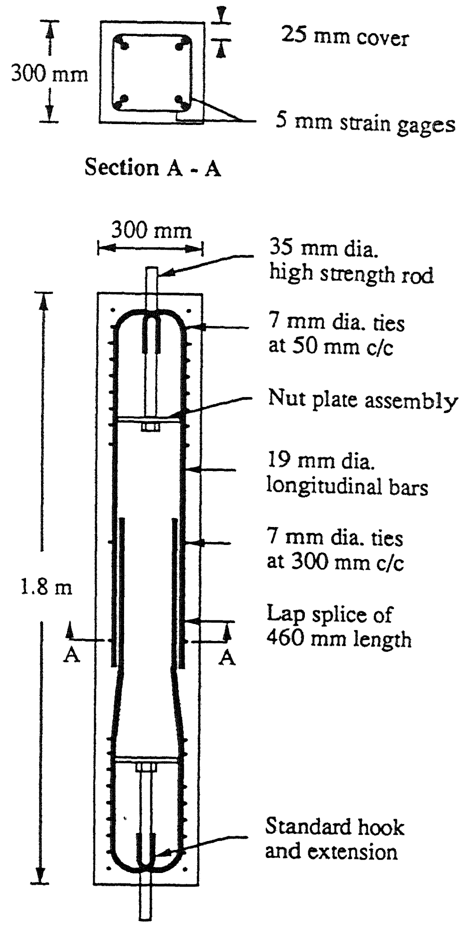



Fig. 2 Test specimen with existing (unstrengthened) splice details

*Strengthening details*

Three specimens were strengthened using steel angles (50 x 50 x 6 mm) and straps (300 x 25 x 6 mm). Column corners were chipped off to provide a better fit for the steel angles, and steel straps (spaced at 150 mm c/c) were welded to these angles. Extreme care was taken to ensure that the angles fit tightly against existing concrete. Two more specimens were strengthened with similar details but with the steel angles and straps grouted to the existing concrete. The angles were fixed along the column corners with a 6 mm spacer between the concrete and each steel angle, and straps were welded to the angles. A dry pack grout (non-shrink and non-corrosive grout mixed with minimal water) was placed and compacted in the gap between the existing column and steel elements (see Fig. 4).

Fig.3 Welded splice with an additional tie

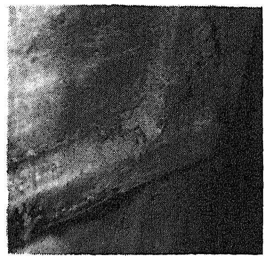


Fig. 4 Grouted steel angles and straps

Three specimens were confined with external ties made of U-shaped 13 mm diameter rebars. They were spaced at 75 mm c/c and their overlapping legs were welded together. One of the three specimens was left ungrouted (Fig. 5a). Ties in the second specimen were grouted with a thin mix forming a 25 mm cover over the existing concrete (Fig. 5b). The third specimen was partially grouted. A dry-pack grout was used to fill the gap between the ties and the existing column (Fig. 5c). The partially grouted scheme was intended to ensure that the external ties were in contact with the existing column.

The last test specimen was provided with additional internal ties. Three 10 mm diameter ties were provided at a 300 mm spacing. The ties consisted of U-shaped rebars with their overlapping legs welded together (see Fig. 6). The existing concrete cover was removed and replaced with a non-shrink grout after the ties were placed and welded.

*Materials*

The actual 28-day concrete compressive strength was 24 N/mm² for all specimens. The maximum size of coarse aggregate was 10 mm (2/3 scale of 15 mm). Grade 60 (414 N/mm²) reinforcing bars were used. Actual yield strengths of reinforcing bars of all sizes ranged between 476 N/mm² and 490 N/mm². Conventional A36 (248 N/mm²) steel was used for steel angles and straps.

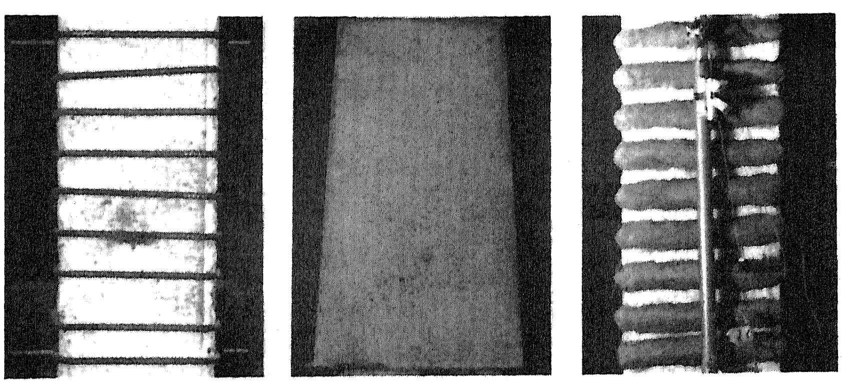
3.2 *Test frame*

The frame was designed to subject the column specimens to alternating axial tensile and compressive forces (see Fig. 7). Tension was applied to the specimen through a 35 mm diameter rod embedded in the test specimen.

3.3 *Testing procedures*

All specimens were subjected to repeated cycles of load reversals. The load cycles consisted of a tension cycle followed by a compression cycle of equal magnitude. The planned loading pattern incorporated one cycle to a tensile load of 267 kN (load at which concrete cracks) followed by two cycles at 401, 534 (load at which rebars yield in tension) and 668 kN. The load patterns were modified slightly during the tests based on the actual behavior of specimens.

Load cells were placed under the hydraulic rams to measure the applied load. Two displacement transducers were installed on opposite faces of the test specimen to measure axial elongation along the splice region. Strain gages were placed at critical locations in each specimen.



(a) Ungrouted (b) Grouted (c) Partially grouted

Fig. 5 External ties

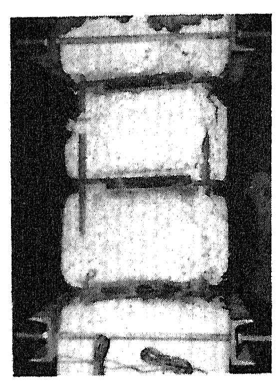


Fig. 6 Additional internal ties

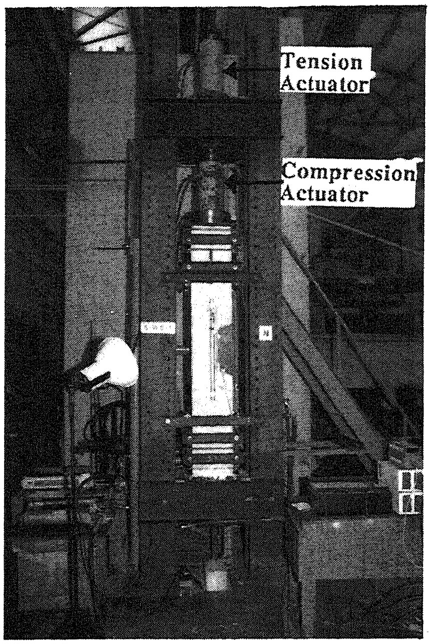



Fig. 7 Test setup

## 4 TEST RESULTS

*Existing column*

In the specimen with existing splice details, cracks initiated near the location of column ties in the splice region and progressed along the splices until a splitting tensile failure occurred during the third cycle at a load level of 379 kN (see Fig. 8). Calculated column strength, based on bars yielding in tension, was 539 kN. Predicted column strength, based on Orangun's equation, was 428 kN.

*Welded splices*

One of the two specimens with welded splices (no additional tie) reached load levels beyond the tensile yield capacity. However, cracks developed near the end of outer spliced bars at early load stages and progressed along the splice region. The concrete cover along the column corners spalled due to the outward thrust produced by the eccentricity between the spliced bars (see Fig. 9). The existing tie near the end of the outer spliced bars yielded in tension and the 90 degree hook opened at the onset of spalling of concrete. Behavior of the column during its final stages was characterized by significant loss of member stiffness and unstable hysteresis loops. Performance of the second specimen strengthened with welded splices and an additional tie was very satisfactory. Not only did the column sustain yield capacity but it also maintained its stiffness with relatively stable hysteresis loops. Cracks initiated near the end of the outer spliced bars as soon as the additional tie reached its yield strain. Behavior of the specimen beyond yield of the additional tie was similar to that of the companion specimen, which was subjected to seven load cycles instead of eight. No visible distress could be observed in the welds of either specimen.

*Confinement with steel elements*

The three specimens strengthened with ungrouted steel angles and straps showed varied performance. While two yielded in tension (testing was discontinued at this stage due to hook failure in an end connection), the third one failed at a load level of

Fig. 8 Failed specimen with unstrengthened splices

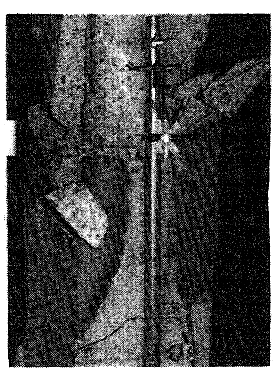


Fig. 9 Damaged specimen with welded splices (no additional tie)

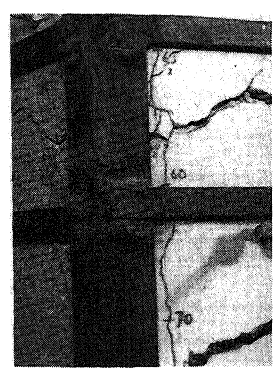
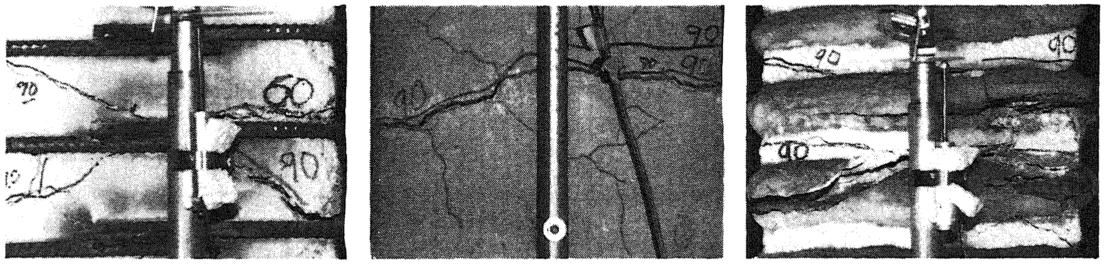


Fig. 10 Failed specimen with ungrouted steel angles and straps



(a) Ungrouted (b) Grouted (c) Partially grouted

Fig. 11 Failed specimens with external ties

401 kN with splitting in tension (see Fig. 10). This very different behavior indicated the difficulties in matching new steel elements with an existing concrete surface. Behavior of the two specimens strengthened with grouted steel angles and straps was similar and extremely satisfactory. Both specimens were subjected to six load cycles with load levels as high as 668 kN in tension and compression. No cracking was observed in the splice region until the final load stages. Column bars started to yield and strain harden outside the splice region with very large crack widths.

*Confinement with ties*

The three specimens with additional external ties and different grouting conditions also exhibited varied performance. The specimen with ungrouted external ties performed no better than the existing (unstrengthened) splice. Splices failed in tension during the fourth cycle at a load level of 423 kN. The specimen with fully grouted external ties continued to deform well past the point at which yield was reached. Cracks developed at early load stages followed by yielding of column bars in the splice region. Column splices failed in tension during the seventh load cycle at a load level of 668 kN. The third specimen with partially grouted external ties failed in tension during the sixth cycle at a load level of 624 kN. The appearance of these specimens is shown in Fig. 11.

Behavior of the column specimen strengthened with additional internal ties was not satisfactory. Cracks developed and progressed in the splice region until splices failed in tension during the fourth cycle at a load level of 512 kN (see Fig. 12). The removal and replacement of concrete cover resulted in microcracking of the concrete core and reduction in effectiveness of concrete cover. Grout placed on the grooves appeared to perform monolithically with the existing cover.

Results of all specimens are summarized in Table 1.

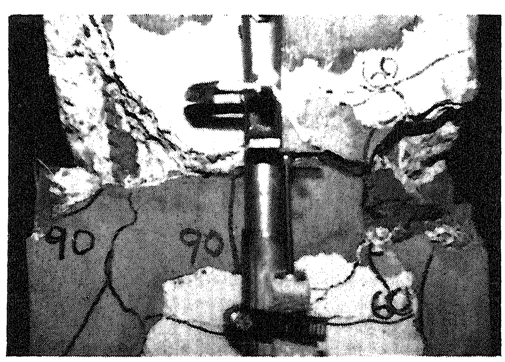


Fig. 12 Column specimen with additional internal ties

## 5 CONCLUSIONS

Columns constructed with splices designed for little or no flexure and only axial compression, and comprising part of a frame-wall system cannot develop tensile yield when subjected to reversed cyclic loads. As a result, the benefit of strengthening R/C frames with infill walls is limited by the failure of column splices.

Providing continuity in the splice region by welding the bars enabled the columns to yield in tension under reversed cyclic loads. However, it was necessary to add ties to restrain the outward thrust produced by the eccentricity between spliced bars.

External reinforcement around the splice region significantly improved confinement and splice strength. The external reinforcement must be grouted in order to permit it to effectively confine the concrete.

Addition of internal ties to the splice region was not an effective method for strengthening column

**Table 1 Summary of test results**

| Specimen number | Details of splice strengthening | Maximum load (kN) | Fraction of tensile capacity ($f_y$ = 539kN) | Failure mode |
|---|---|---|---|---|
| 1 | unstrengthened | 379 | 0.70 | splice failure |
| 2 | welded splice | 584 | 1.08 | large deformations |
| 3 | welded splice with an additional tie | 615 | 1.14 | yielding of additional tie |
| 4 | steel angles & straps (ungrouted) | 535 | 0.99 | end connection failure |
| 5 | " | 535 | 0.99 | " |
| 6 | " | 401 | 0.74 | splice failure |
| 7 | steel angles & straps (grouted) | 668 | 1.24 | no failure |
| 8 | " | 668 | 1.24 | " |
| 9 | external ties (ungrouted) | 423 | 0.78 | splice failure |
| 10 | external ties (grouted) | 668 | 1.24 | " |
| 11 | external ties (partially grouted) | 624 | 1.16 | " |
| 12 | additional internal ties | 512 | 0.95 | " |

splices, because removal of concrete reduced the effectiveness of concrete cover and reduced the splice strength more than additional ties improved it.

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