Earthquake Engineering, Tenth World Conference © 1992 Balkema, Rotterdam. ISBN 90 5410 060 5

Rehabilitation of RC frame connections using jacketing

S. M. Alcocer
Institute of Engineering, National Autonomous University of Mexico & National Center for Disaster Prevention, Mexico City, Mexico

ABSTRACT: The behavior of frame connections repaired and/or strengthened by jacketing was assessed experimentally. Four slab-beam-column joints were constructed and rehabilitated. The specimens were tested under bidirectional cyclic loading that simulated earthquake-type motions. Variables were the jacketed element, level of damage of the existing structure prior to rehabilitation, and layout of jacketed column reinforcement. Test results indicated that jacketing was effective to rehabilitate the existing structure, thus improving the strength, stiffness and energy dissipation characteristics of the existing structures. Recommendations are developed for design of frame connections rehabilitated by jacketing.

1 INTRODUCTION

Numerous reinforced concrete non-ductile moment resisting frames designed in the 1950's and 1960's are in the inventory of vulnerable structures worldwide. Characteristics of these buildings include flexible columns, non-ductile reinforcement detailing, and "strong beam - weak column" systems (Rosenblueth & Meli 1986). One of the rehabilitation techniques used for this type of structures is jacketing of frame elements. R/C jacketing is the addition of a concrete shell reinforced to improve the strength, stiffness and ductility of the element (UNIDO 1983). In Mexico City, jacketing was the most widely used rehabilitation scheme for R/C frames (Jirsa 1987). However, little experimental work has been conducted to verify the performance of concrete jacketing of frame elements and its suitability as a rehabilitation technique. To aid in developing analysis and design guidelines, an experimental program, aimed at studying the behavior of slab-beam-column joints rehabilitated by jacketing, was implemented.

2 EXPERIMENTAL PROGRAM

2.1 Specimen description

Four large-scale identical reinforced concrete slab-beam-column joints were constructed and rehabilitated by jacketing the columns only or both the columns and beams. The specimens represented an interior joint of a lower story of a multistory building. Specimen dimensions are shown in fig. 1. In each test, the existing structure was the same and was designed

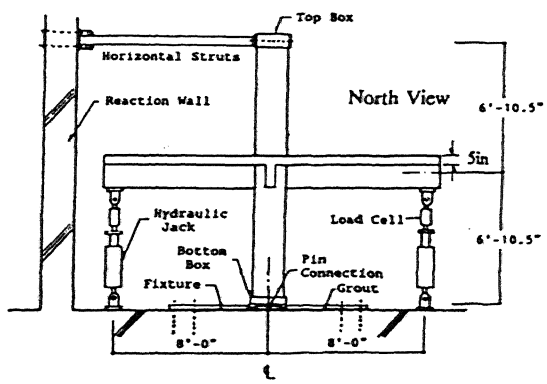
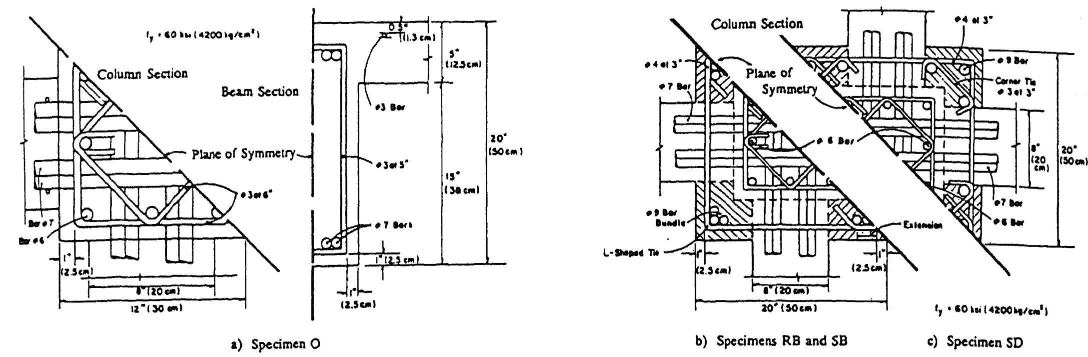


Figure 1. Specimen dimensions and setup (1 ft = 304.8 mm).

according to the American practice of the 1950's and 1960's (ACI 1963). Three experimental variables were studied: the level of damage in the specimen before the test, the jacketed frame element, and the layout of the jacketed column longitudinal reinforcement. The experimental program is summarized in table 1.

Table 1. Summary of experimental program.

Test	Damage before test	Jacketing Columns	Jacketing Beams	Bundled column bars
O	no	-	-	-
RB	yes	yes	no	yes
SB	no	yes	no	yes
SD	no	yes	no	no
SD-B	no	yes	yes	no



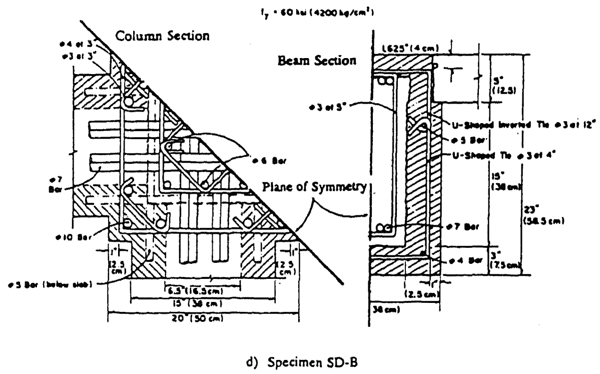


Figure 2. Reinforcement details (1 in.= 25.4 mm; 1 ksi = 6.89 MPa).

2.2 Reinforcement details of existing structures

Reinforcement details of column and beam sections outside the joint region are shown in fig. 2. The beam flexural reinforcement of the existing structures was continuous through the joint and was designed to provide large joint shear stresses after jacketing the column. No ties were provided in the joint region of the existing structures since this was more representative of 1950's construction. The concrete compressive strength of the existing structures at time of test was 17.2 MPa on the average, except for SB, which was 27.6 MPa. Slab reinforcement in each direction consisted of a top and a bottom layer of continuous #3 bars. Top reinforcement was spaced at 305 mm and the bottom steel at 610 mm.

2.3 Reinforcement details of column jackets

Rehabilitated specimens were designed so that flexural strength of columns was greater than that of the beams. For specimens RB and SB, the column jacket was reinforced with three continuous bundled bars placed at the corners through perforations in the slab (fig. 2b). Transverse steel above and below the joint, was provided and detailed following ACI 318 Building Code Requirements (1989), and consisted of two L-shaped ties that overlapped in diagonally opposite corners. Specimens SD and SD-B had distributed column longitudinal steel around the perimeter (figs. 2c and 2d). Main transverse steel was similar to that of RB and SB, but additional ties were placed between bars away from the corners following ACI requirements. Corner ties with 180-deg hooks extended into the joint region.

2.4 Reinforcement details of beam jackets

Beams of specimen SD-B were also jacketed and the reinforcement was added to increase beam flexural capacity moderately and to produce high joint shear stresses (fig. 2d). Top bars crossed the orthogonal beams through holes, and the bottom bars were placed under the soffit of the existing beams, at each side of the existing column. Beam transverse steel consisted of sets of U-shaped ties fixed to the top jacket bars, and of inverted U-shaped ties placed through perforations in the slab. Closely spaced ties were placed near the joint region, where beam hinging was expected to occur.

2.5 Joint confinement steel of rehabilitated specimens

To confine the joint concrete not confined by transverse beams, and to confine the column bars, a structural steel cage was welded

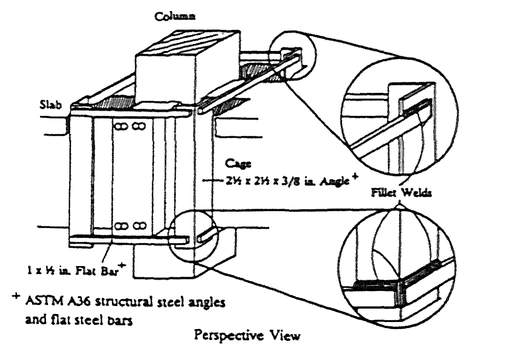


Figure 3. Joint confinement cage (1 in.= 25.4 mm).

around the joint for all rehabilitated specimens, as illustrated in fig. 3. The cage eliminated the need for ties and the need to drill holes through the beams for placing the ties. Steel angles were dimensioned to provide confinement to the core equivalent to spiral reinforcement. Construction details of the rehabilitation scheme can be found elsewhere (Alcocer & Jirsa 1991).

The concrete compressive strength of the jackets was 37.9 MPa on the average.

3. TEST SETUP AND LOADING PROGRAM

Column ends were pinned and the lateral movement restricted. Beams were connected to the reaction floor through double-action hydraulic actuators (fig. 1).

All specimens were tested using the same bidirectional cyclic load history, shown in fig. 4. The load history was displacement controlled based on the interstory drift angle. Four levels of deformation were applied.

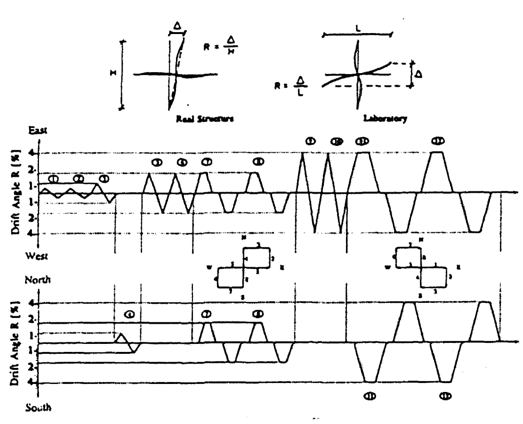


Figure 4. Loading program.

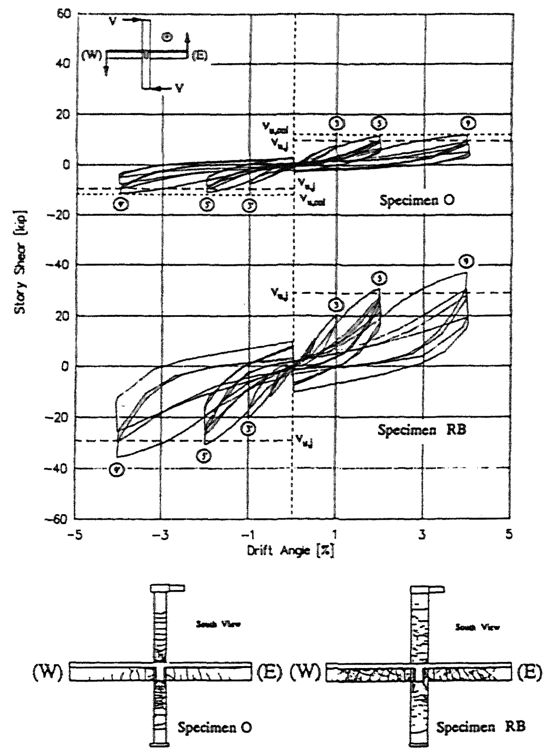


Figure 5. Response of specimens O and RB (1 kip = 4.45 kN).

4. TEST RESULTS

4.1 Hysteresis loops of specimen O and RB

The E-W story shear versus drift angle curves and final crack patterns for specimens O and RB are presented in fig. 5. The ultimate story shears corresponding to joint failure $V_{u,j}$, and column hinging (for O), $V_{u,col}$, are also shown in the figure. The story shear corresponding to joint failure was based on the joint shear strength recommended by ACI-ASCE Committee 352 Report (1976). The story shear to produce column hinging was calculated from measured material properties and dimensions, considering an equivalent rectangular stress block for the concrete and assuming that plane sections remain plane. The ratio of calculated column to beam flexural capacities in the E-W direction for specimens O and RB were 0.32 and 2.53, respectively. The hysteresis loops are nearly symmetrical and show considerable pinching, especially at drifts to 4%, and severe stiffness degradation. The hysteresis curves for O are dominated by the response of the most damaged elements, namely the columns and joint. Severe concrete crushing and spalling were observed in the columns. Loops are characteristic of column hinges without ductile detailing. Consistent with a "strong beam - weak column" system, the beams and slab of O experienced minor flexural cracking and remained elastic.

Following the completion of test O, loose concrete was removed and the subassemblage was repaired by jacketing the columns. The beams and slab were not modified or repaired. In general, cracking in specimen RB was more uniformly distributed than in specimen O. While beams yielded, columns showed only minor flexural cracking compatible with the "strong column - weak beam" design philosophy. Slab cracking was more extensive than in test O. Most of the damage was concentrated around the joint, which failed in shear after beam hinging. The joint confinement cage was sufficient to prevent spalling into the joint core and confined effectively the concrete and column bars (Alcocer & Jirsa 1991). Although the loops showed higher stiffness and strength than those for specimen O, stiffness and strength degradation, as well as pinching, were noted (fig. 5). Bond deterioration along beam bars contributed to the S-shaped response. The measured E-W story shear exceeded the calculated strength in cycles 5, 9 and 10.

The behavior described for specimen RB was typical for all rehabilitated specimens (Alcocer & Jirsa 1991). The ratios of calculated column to beam flexural capacities for the E-W direction were 2.68, 2.65 and 2.29 for SB, SD and SD-B, respectively. Failure of the specimens were due to joint failure after beam hinging. Column damage was minor. The hysteresis diagrams showed better stiffness, strength and total energy dissipation characteristics than specimen RB. The maximum measured story shears in the E-W direction exceeded the calculated joint capacities. Analysis of strain gauge data showed good bond in jacketed column bars of RB, SB, SD and SD-B.

4.2 Response envelopes

The envelopes for positive cycles in the E-W direction are presented in fig. 6. All specimens maintained their strength even at deformations to 4% drift.

The strength of specimen RB was 2.8 times greater at 2% drift than the strength of specimen O. At 0.5% drift, specimen RB was 2.3 times stiffer than specimen O. Comparison of RB and SB shows that by jacketing the most damaged elements, the columns and joint, the strength at 2% drift and stiffness at 0.5% drift were 63% and 52%, respectively, of the values obtained in the undamaged specimen. Among the rehabilitated specimens with only columns jacketed, specimen SB was the stiffest and strongest because of the location of the jacket bars and the strength of the concrete. The performance of specimen SD was nearly equal to that of SB. At 0.5% drift, stiffness for SD was 88% of the stiffness for SB.

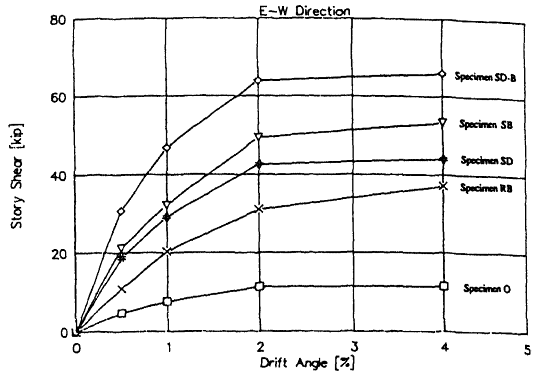


Figure 6. Response envelopes (1 kip = 4.45 kN).

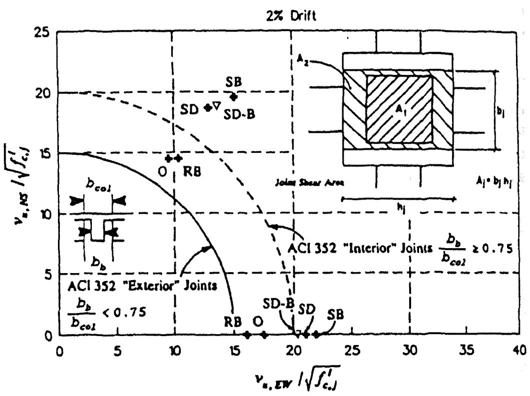


Figure 7. Normalized joint shear stress.

Strength at 2% drift was 86% of the value obtained in test SB. As expected, specimen SD-B was the stiffest and strongest specimen since the beams were also jacketed. Initial stiffness was 6.4 times the stiffness for test O. The strength was 5.7 times that of specimen O at 2% drift.

4.3 Joint shear strength

Since no design recommendations exist for rehabilitated frame connections, one of the objectives of this study was to investigate the mechanism of shear resistance in jacketed joints. Current ACI 352 Recommendations for new construction assume that the transverse reinforcement in the joint is provided to confine the concrete, and that the joint shear forces are mostly carried by a concrete diagonal strut.

To assess the joint shear strength, the joint geometry was determined according to ACI 352 (fig. 7). The maximum measured joint shear stresses v_u were calculated from measured story shears for both loading directions during unidirectional and bidirectional cycles to 2% drift. The joint shear stresses were divided by $\sqrt{f'_{c,j}}$ (fig. 7). Curves are shown to reflect joint confinement conditions according to the number and geometry of the lateral members based on the joint geometry and joint classification suggested by ACI Committee 352. A circular interaction curve was assumed for bidirectional loading, in which the unidirectional strength is the radius. Specimens O, RB, SB and SD are classified as "exterior" joints because the beam width was less than three quarters of the column width. The joint region of the rehabilitated specimens consisted of two different concretes thus having distinct strengths. A weighted average of the concrete strength was computed to calculate the joint shear stress. The average strength was given by

$$A_j\sqrt{f'_{c,j}} = A_1\sqrt{f'_{c,1}} + A_2\sqrt{f'_{c,2}} \tag{1}$$

where
$A_j = b_j h_j$ = joint area
$f'_{c,j}$ = weighted average concrete strength
A_1 = gross area of existing column
$f'_{c,1}$ = strength of the existing column joint
$A_2 = A_j - A_1$ = area of column jacket included in joint area (see fig. 7)
$f'_{c,2}$ = strength of the jacketed column joint.

From test results (fig. 7), it is clear that ACI 352 recommendations provided a safe estimate of the strength of jacketed joints with good confinement. Checking the joint shear strength in each direction will provide adequate bidirectional joint shear strength. For unidirectional loading, joint stress of $1.25\sqrt{f'_{c,j}}$, *MPa* and $1.66\sqrt{f'_{c,j}}$, *MPa* should not be exceeded for "exterior" and "interior" type joints, respectively. The uniaxial joint shear should be calculated from a beam hinge mechanism where the beam moment capacity accounts for increased stresses (due to higher yield strength than the nominal value and due to strain hardening) in longitudinal bars and for the participation of slab reinforcement. For all rehabilitated specimens, the beam critical section should be assumed at the jacketed column face (Alcocer & Jirsa 1991).

4.4 Slab participation

Floor system strength, under positive and negative bending, is of particular interest because it must be considered for shear strength of the beams, the flexural strength of the columns and the shear strength of the joints. Design guidelines for T-beam action under positive moment are based on an effective width (ACI 1989). For negative flexure (i.e. slab in tension), there are no design guidelines, even for new construction, to estimate the slab participation.

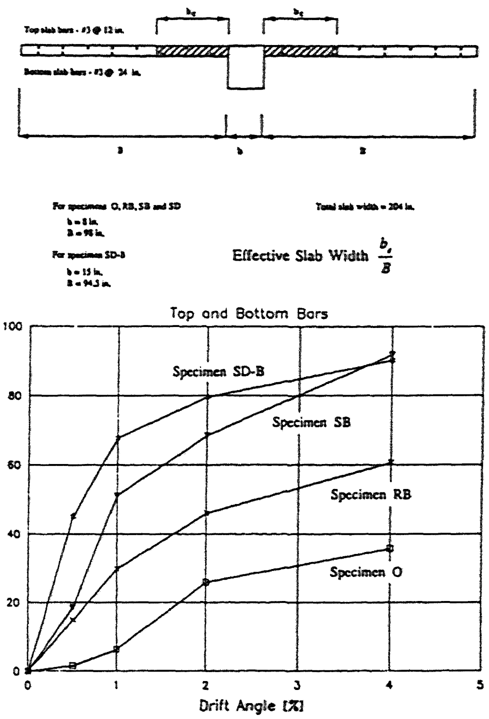


Figure 8. Slab participation (1 in.= 25.4 mm).

The slab participation was evaluated using two methods. First, the effective slab width was calculated from the stress distribution of the bars across the slab (Alcocer & Jirsa 1991). The effective slab width ratio (effective width b_e over total slab width B, see fig. 8) was computed as the ratio of tensile force in the slab reinforcement over the maximum tensile force when all bars reach yielding. The contribution of the top and bottom layers of the reinforcement is presented in fig. 8. At 2% drift, the slab participation of specimen O was half that of the repaired specimen RB. Specimen SD and SD-B showed the largest participation at about 75%. Comparison of RB and SB shows that the slab participation of the repaired specimen (damaged before rehabilitation) was smaller than that of the undamaged specimen. Due to malfunctioning slab gauges, the effective slab width ratio could not be determined for specimen SD.

The slab participation was also assessed through comparison of maximum measured moments and calculated beam capacities at 2% drift. Top and bottom slab bars were considered effective in the flange portion. Slab reinforcement outside the effective width was ignored. The beam capacity for O and RB was best estimated by the rectangular beam section strength. For the other specimens, $b_e = 0.30B$ was used.

Averaging the effective slab widths obtained from the two methods described, an effective slab width equal to 20% of the transverse span can be considered for a damaged structure to estimate the beam negative moment capacity. For undamaged structures to be rehabilitated, an effective slab width equal to half the transverse span can be used. Both top and bottom bars can be used in the calculations as long as bottom bars have enough anchorage in the transverse beams.

5. CONCLUSIONS

Jacketing of frame elements improved the strength, stiffness and energy dissipation characteristics, and changed the mode of failure of the existing structures. Jacketing was effective as a rehabilitation technique. In general, modern building codes (such as ACI 1989) can be used for designing a jacketing scheme. However, constructibility must be always borne in mind.

Mode of Failure. In the design of structures rehabilitated by jacketing, the change in mode of failure and redistribution of forces must be considered. Beam shear capacity must be sufficient to develop beam hinging. The location of the beam critical section, and the participation of the existing reinforcement and slab reinforcement should be taken into account (Alcocer & Jirsa 1991).

Effect of Damage. Tests showed that for a damaged specimen, the strength at 2% drift and stiffness at 0.5% drift were 63% and 52%, respectively, of the values obtained in the undamaged specimen.

Strength. Shear strength of rehabilitated frame connections can be estimated using current ACI 352 recommendations for design of beam-column joints in new construction. All specimens maintained their strength even at deformations to 4% drift. Ductile behavior is associated to beam hinging, compatible with the "strong column - weak beam" design philosophy.

Slab participation. To estimate beam negative capacities, an effective slab width equal to 20% of the transverse span (eight times the slab thickness) can be considered for a damaged structure. For undamaged structures to be rehabilitated, an effective slab width equal to half the transverse span (twenty times the slab thickness) can be used.

Column Layout. The layout of the column longitudinal bars is an important design consideration. The use of column distributed reinforcement (versus column bundles) is appropriate to reduce the possibility of bond damage.

ACKNOWLEDGEMENTS

The author gratefully acknowledges the guidance and advice of Dr. James O. Jirsa, supervisor of this research. The study was sponsored by the National Science Foundation (NSF-USA). The financial support of the Consejo Nacional de Ciencia y Tecnologia (CONACYT-Mexico), and help of the staff at the Ferguson Structural Engineering Laboratory of the University of Texas at Austin are thankfully acknowledged.

REFERENCES

ACI committee 318 1963. Building code requirements for reinforced concrete. ACI 318-63, American Concrete Institute, Detroit, Mich.

ACI committee 318 1989. Building code requirements for reinforced concrete. ACI 318-89, American Concrete Institute, Detroit, Mich.

ACI-ASCE committee 352 1976. Recommendations for design of beam-column joints in monolithic reinforced concrete structures. American Concrete Institute, Detroit, Mich.

Alcocer, S.M., & J.O. Jirsa 1991. Reinforced concrete frame connections rehabilitated by jacketing. PMFSEL Report No. 91-1, Phil M. Ferguson Structural Engineering Laboratory, The University of Texas at Austin.

Jirsa, J.O. 1987. Repair of damaged buildings - Mexico City. Proc. Pacific Conference on Earthquake Engineering: 1, 25-34, New Zealand.

Rosenblueth, E., & R. Meli 1986. The 1985 earthquake: causes and effects in Mexico City. Concrete International: 8 (5), 23-34.

United Nations Industrial Development Programme 1983. Building construction under seismic conditions in the balkan region: repair and strengthening of reinforced concrete, stone and brick-masonry buildings. Vol. 5, first edition, Vienna.