### Seismic Upgrade with Carbon Fiber-Reinforced Polymer of **Columns Containing Lap-Spliced Reinforcing Bars**

by Kumar K. Ghosh and Shamim A. Sheikh

Existing reinforced concrete (RC) columns detailed with poor lap splices and inadequate transverse confinement reinforcement in the potential plastic hinge regions near beam-column joints, characteristic of pre-1970 design provisions, are found to be deficient for the strength and ductility demands imposed by earthquake loading. The work reported herein was directed toward the evaluation of the effectiveness of carbon fiber-reinforced polymer (CFRP) jackets in strengthening and repair of such columns under simulated earthquake loading. A total of 12 columns, six 356 mm (14 in.) diameter circular and six 305 mm (12 in.) square, were constructed and tested. The columns were 1.47 m (58 in.) long and had a 510 x 760 x 810 mm (20 x 30 x 32 in.) stub at one end with a construction joint at the interface and spliced longitudinal bars in the columns. The variables studied in this program included effect of the presence of lap splices, the effectiveness of CFRP in preearthquake strengthening and post-earthquake retrofitting of deficient columns, as well as effects of level of axial load, shape of column cross section, and transverse steel reinforcement details. The CFRP retrofitting technique was found to be effective in enhancing the seismic resistance of the columns and resulted in more stable hysteresis curves with lower stiffness and strength degradations as compared with the unretrofitted columns.

Keywords: columns; confinement; ductility; lap splice; rehabilitation.

### INTRODUCTION

Most of the existing reinforced concrete structures designed and constructed prior to 1970 in accordance with the prevalent design standards may have inadequate seismic resistance. As a result, during the recent earthquakes, many of these structures collapsed or suffered severe damage. The Loma Prieta earthquake alone caused damage to over 80 bridges, resulting in 40 deaths, \$1.8 billion in damage, and severe economic disruptions.<sup>1</sup> These structures were designed and detailed to resist primarily gravity loads and did not possess adequate lateral strength and ductility and thus the new codes have rendered them substandard. Lap splices are used in the construction of most reinforced concrete structures for preserving the continuity of reinforcement within the structural members. These splices were usually provided in the potential plastic hinge regions at the base of the columns, just above a construction joint and all the bars were usually spliced at the same section. This results in a considerable reduction in the strength and ductility of the structure. Because most of these pre-1971 structures were detailed only for gravity loading, the splice lengths in the columns were designed as per the code requirements for bars under axial compression with little or no flexure. Earthquake forces, however, result in transverse loading causing the spliced bars to be subjected to large tensile forces and inelastic deformations, which lead to high strength and ductility demands. The situation is further worsened by inadequate transverse reinforcement to confine the concrete and thus there is a lack of adequate clamping pressure across the fracture

surfaces in the splice zone. The performance of such columns will thus be limited by the premature failure of the splices. The changes implemented in the ACI design practice as per the splice configuration (length, bond strength, and detailing of added stirrups/ties) over the years until the current design practice has been outlined in a literature review.<sup>2</sup>

There are considerable research efforts being directed at developing and applying retrofit strategies to upgrade the seismic performance of deficient structures. Traditional strengthening systems include steel and reinforced concrete jacketing. Several researchers have validated experimentally as well as through field applications the effectiveness of steel jackets in providing desired confinement to the core concrete and thereby improving the seismic performance of deficient columns.<sup>1,3-6</sup>

The rapid deterioration of the infrastructure coupled with restrained public spending, however, demands constant optimization of the repair and strengthening techniques to improve and economize the retrofitting process of the large number of existing, seismically deficient structures. This has led to research towards the use of new polymers and composite materials to develop alternative retrofitting techniques that are easy to implement, economical from a perspective of life-cycle cost, and more durable. One strengthening scheme that is finding rapid acceptance involves the use of externally bonded fiber-reinforced polymer (FRP) (carbon, glass, and aramid) jackets. Research work is being carried out at various research centers around the world in determining the effectiveness of FRP reinforcement in strengthening columns rendered deficient by the modern seismic codes due to poor confinement and splice details.<sup>2,/-15</sup>

### **RESEARCH SIGNIFICANCE**

The current study was directed toward an evaluation of the effectiveness of carbon FRP (CFRP) jackets in enhancing the seismic resistance of non-ductile reinforced concrete columns with poor details of longitudinal bar splices in plastic hinge regions and inadequate transverse confinement. Results from this work are compared with the database already available from similar FRP-retrofitted columns that did not contain lap splices in the hinge regions.<sup>7,8</sup> The objectives of this research program were to: 1) evaluate the degradation of column strength and ductility due to the presence of lap spliced longitudinal bars; 2) determine the effectiveness of CFRP in pre-earthquake strengthening and post-earthquake repair of such deficient columns; 3) study the effects of shape

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Fig. 1—Specimen details: (a) dimensional details; (b) reinforcement details of square and Type A circular columns; (c) reinforcement details of Type B circular columns; and (d) sectional details.

(circular or square) of column cross section on the effectiveness of CFRP retrofitting; 4) determine the effects of confinement on lap splices; and 5) investigate the effect of axial load level.

#### EXPERIMENTAL PROGRAM Specimen details

Twelve reinforced concrete columns (six circular and six square) were constructed and tested under combined axial load and reversed cyclic lateral displacement excursions simulating earthquake. The circular columns had a diameter of 356 mm (14 in.) while the square ones had a section size of 305 mm (12 in.). The columns were 1.47 m (58 in.) long and had a 510 x 760 x 810 mm (20 x 30 x 32 in.) stub at one end with a construction joint at the interface and spliced longitudinal bars in the columns. The dimensional details of the columns are presented in Fig. 1(a). The test column represented the portion between the point of contra-flexure and the section of maximum moment at the base of the column. The stub represented a supporting member of the column like a beam column joint or a footing. In all the specimens, the ratio of the column core area, measured center-to-center of external hoops, to the gross area of the column section was kept at approximately 75%. The specimens are typical representations from pre-1971 multi-story building and bridge piers.

Table 1 lists four groups of columns out of which the 12 columns of the first two groups represent specimens tested under the present project. Group III lists the details of some of the circular columns tested by Sheikh and Yau,<sup>7</sup> while under Group IV, the details of two square columns tested by Iacobucci and Sheikh<sup>8</sup> are shown. All of the Group III and IV columns have the same dimensional and reinforcement



Fig. 2—Stress-strain behavior of steel.

details as the present columns, except that they were reinforced with continuous longitudinal bars. For columns in Groups I and II, the first letter in the specimen designation indicates the shape of the column, C being circular and S being square. A or B refers to the transverse steel configuration of the specimens. F1, if present, indicates that the column was wrapped with one layer of CFRP and is followed by the testing sequence number of the specimen. The last letter N refers to normal-strength concrete.

*Reinforcing steel*—Three types of deformed steel bars were used to construct the reinforcing cages of the specimens. Grade 400, 20M (area =  $0.465 \text{ in.}^2$  [300 mm<sup>2</sup>]) bars and Grade 60 U.S. No. 3 bars were used for the longitudinal and the transverse reinforcement in the columns, respectively. Grade 400, 10M (area =  $0.155 \text{ in.}^2$  [100 mm<sup>2</sup>]) bars were used to construct the stub reinforcing cages. The stressstrain curves of steel bars along with the yield and ultimate stress and strain values are presented in Fig. 2. The columns were detailed as per the provisions of the ACI 318-56<sup>16</sup> and ACI 318-63<sup>17</sup> codes.

The circular columns were reinforced with six 20M and the square columns with eight 20M longitudinal bars that were lap spliced in the column for a length of 470 mm (18.5 in.) from the stub-column interface (Fig. 1). Two configurations of transverse reinforcement, Types A and B, were used. For the circular specimens, Type A columns were detailed with U.S. No. 3 hoops at 300 mm (11.8 in.) spacing while Type B columns consisted of U.S. No. 3 spirals at 80 mm (3.1 in.). For the square specimens, Type A columns were reinforced with only one set of U.S. No. 3 ties at 300 mm (11.8 in.), such that only the corner bars were held by corners of the ties. The Type B square specimens were reinforced with two sets of U.S. No. 3 ties at 300 mm (11.8 in.) so that every longitudinal bar was laterally supported by tie corners. To ensure that failure of the specimens occurs at the potential plastic hinge region, beyond a distance of approximately 600 mm (23.6 in.) from the stub-column interface, the spacing of the transverse steel was reduced to approximately half the specified spacing in the test zone as shown in Fig. 1.

The reinforcement for the stub consisted of 10M horizontal and vertical stirrups at 64 mm (2.5 in.) spacing. Additional 10M bars with 135-degree hooks were added at two sides to increase stub stiffness. The reinforcement details are illustrated in Fig. 1.

*Concrete*—The stubs and the columns were cast with two separate batches of concrete on two consecutive days to simulate field conditions and to form a construction joint at the interface. Concrete with a target strength of 25 MPa (3630 psi) was used. Concrete strength was monitored by

		Lateral steel				Capacity strengths		Ductility parameters		Work index	
Specimen	f′ <sub>c</sub> , MPa	Туре	ρ <sub>s</sub> , %	Axial load, $P/P_o$ , %	Layers of CFRP	Shear, kN (kips)	Moment, kN-m (k-ft)	$\mu_{\Delta 80}$	$N_{\Delta 80}$	W <sub>80</sub>	$W_T$
Group I: 356 mm (14 in.) diameter circular columns with 470 mm (18.5 in.) lap splice <sup>2</sup>											
CA-1N	24.8	Hoops	0.32	0.05	0	48.2 (10.3)	89.9 (66.8)	2.05	7.2	8.5	63.2
CAF1-2N	24.9				1	76.5 (17.2)	144.6 (106.7)	6.10	39.4	154.5	410.2
CA-3N	25	Hoops	0.32	0.27	0	93.7 (21.1)	181.2 (40.7)	1.76	6.0	3.9	17.3
CAF1-5N	25.1				1	97.1 (21.8)	193.5 (43.5)	5.64	33.9	76.7	287.3
CB-4N	25.1	Spiral	1.2	0.05	0	62.9 (14.1)	118.5 (26.6)	3.48	11.9	16.7	86.5
CBF1-6N	26.5				1	77.9 (17.5)	149.3 (33.6)	7.69	44.3	264.1	426
Group II: 305 x 305 mm (12 x 12 in.) square columns with 470 mm (18.5 in.) lap splice <sup>2</sup>											
SA-7N	26.6	Type A	0.37	0.05	0	62.2 (14.0)	116.5 (26.2)	2.73	10.7	9.1	83.2
SAF1-8N	26.7				1	81.8 (18.4)	152.9 (34.4)			_	_
SA-9N	26.7	Type A	0.37	0.33	0	101.7 (22.9)	198.9 (44.7)	1.93	5.8	5.1	13.3
SAF1-10N	26.8				1	114.0 (25.6)	226.9 (51.0)	4.0	17.2	27.6	119.3
SBF1-11N	27.0	Type B	0.61	0.05	1	81.4 (18.3)	152.2 (34.2)	4.79	22.0	60.7	136.2
SB-12N SBRF1-12N <sup>*</sup>	27.2				0 1 (R)	80.3 (18.1) 64.7 (14.5)	150.4 (33.8) 123.4 (27.7)	2.28 3.05	6.9 12.9	6.8 18.2	NA 47.8
Group III: 356 mm (14 in.) diameter circular columns without lap splice <sup>7</sup>											
S-4NT	39.2	Spiral	0.32	0.27	0	120 (27)	215 (48.3)	1.8	t	t	9
ST-4NT	44.8				1	145 (32.6)	259 (58.2)	5.5	t	Ť	183
Group IV: 305 x 305 mm (12 x 12 in.) square columns without lap splice <sup>8</sup>											
AS-1NS	31.4	Туре В	0.61	0.33	0	108.2 (24.3)	180.4 (40.6)	3.7	9.5	10.2	25.3
ASC-2NS	36.5				1	127.5 (28.7)	228.8 (51.4)	6.1	33.3	110.5	254.6

Table 1—Specimen details and ductility parameters

\*This column is repaired version of SB-12N.

<sup>†</sup>Capacity of these columns did not drop to 80% in both directions before final failure.

Note: Lateral steel in all columns consisted of U.S. No. 3 hoops or spirals at 300 mm (11.8 in.) spacing except in Specimens CB-4N and CBF1-6N, in which spacing was 80 mm (3.1 in.).

regular testing of  $150 \times 300 \text{ mm} (6 \times 12 \text{ in. nominal})$  concrete cylinders, which were cast with the specimens. The strengthage relationship thus developed was used to obtain the strength of each specimen at the time of testing.<sup>2</sup>

*Fiber-reinforced polymers*—A commercially available FRP system was used for retrofitting. The epoxy consisted of two components, A and B, mixed in the ratio of 100 parts of A with 42 parts of B using a mixer for 5 minutes at a speed of 400 to 600 rpm. The fabric was saturated with epoxy and a layer of epoxy was also applied on the column surface using rollers. The fabric was thereafter wrapped around the columns with fiber orientation in the circumferential direction. The corners of square columns were rounded to facilitate FRP wrapping using concave wood sections, with a 16 mm (0.63 in.) radius, placed inside the forms.

Three circular and four square columns were wrapped with one layer of 1 mm (0.04 in.) thick and 610 mm (24 in.) wide CFRP in the potential plastic hinge regions adjacent to the stub-column interface. The column outside the test region was wrapped with three layers of 1.25 mm (0. 049 in.) thick glass FRP (GFRP) to ensure that failure occurred in the test region. The average tensile strength/unit width/layer and the rupture strain of the CFRP measured from coupon tests were 1019 N/mm (5819 lb/in.) and 0.0129, respectively. The corresponding values for GFRP were 568 N/mm (3243 lb/in.) and 0.0228. Stress-strain curves were essentially linear up to failure for both FRPs.

*Instrumentation*—The longitudinal and transverse reinforcing bars of the specimens were instrumented extensively to determine the steel strains at various locations (Fig. 3). In the B-type circular columns, the first three rings of the spirals were instrumented, while in the other specimens the first two





Fig. 3—Locations of strain gauges on longitudinal and transverse reinforcement.

sets of hoops closest to the stub were instrumented with strain gauges of 5 mm (0.20 in.) gauge length. In addition, six external strain gauges of 60 mm (2.4 in.) gauge length



Fig. 4—LVDT arrangements.



Fig. 5—Testing frame.

were used in each of the retrofitted specimens to measure the strain in the CFRP in the direction of fibers. One gauge was placed at each side face of the retrofitted columns at 75 mm (3 in.) from the stub face. Two gauges each were installed at the top and bottom faces of the columns. These gauges were 75 and 150 mm (3 and 6 in.) away from the stub face.

A total of 18 linear variable differential transducers (LVDTs), 10 on one side and eight on the opposite side, were mounted to the rods embedded in the columns to measure the deformations of the concrete core in the test region of the specimens (Fig. 4). The gauge lengths for these LVDTs varied from 55 to 110 mm (2.2 to 4.3 in.) and covered a length of 550 mm (21.6 in.) from the face of the stub. Transverse displacements were measured by LVDTs placed at six locations along the lengths of the specimens.

### Test setup and testing procedure

All the columns were tested under constant axial load and reversed cyclic displacement excursions in a test frame (Fig. 5). Axial load was applied first using a 4450 kN (1000 kips) capacity hydraulic jack through hinges at the ends of the specimens allowing in-plane end rotations. The reversed cyclic lateral load was then applied through a servocontrolled actuator having 1000 kN (220 kips) load capacity and ±152 mm (6 in.) stroke capacity. Displacement control mode of the actuator was used to apply the predetermined displacement history (Fig. 6) at the interface in which the specimen was subjected to a displacement of 0.75  $\Delta_1$  for the first cycle followed by two cycles each of  $\Delta_1$ ,  $2\Delta_1$ ,



Fig. 6—Loading displacement history.



Fig. 7—Variation of fiber-reinforced concrete with load in Specimen CAF1-5N.

 $3\Delta_1$ ...until the specimen was unable to maintain the applied axial load. Deflection  $\Delta_1$  (also defined in Fig. 6) represents the yield displacement at interface and was calculated from the theoretical sectional response of the unconfined specimen. All the data were collected by a high-speed data acquisition system.

### TEST OBSERVATIONS

### **Control specimens**

Six control specimens (CA-1N, CA-3N, CB-4N, SA-7N, SA-9N, and SB-12N) were tested to failure without any CFRP wraps to establish the behavior against which the performance of the retrofitted columns could be evaluated. At advanced stages of testing, all the control columns exhibited considerable damage in the zone of maximum moment near the column-stub interface, displaying cracking, concrete spalling, and slippage/bucking of the reinforcing bars.

The damage in all the control columns started with the appearance of longitudinal cracks at the top and bottom sections of the columns. This was followed by the appearance of vertical flexural cracks at regular intervals within 600 mm (23.6 in.) from the stub face, during the second and third cycles ( $\delta_{max} = \Delta_1$ ). The flexural cracks usually started to deteriorate rapidly from the fourth cycle ( $\delta_{max} = 2\Delta_1$ ) onward and diagonal shear cracks also started appearing at the sides. Moreover, separation cracks started appearing at the column-stub interface indicating the initiation of bar slippage. From the sixth cycle ( $\delta_{max} = 3\Delta_1$ ) onward, all the cracks started deteriorating rapidly and led to the initiation of spalling of the concrete at the top and bottom. During the last cycles, considerable spalling and dilation of the concrete cross section was observed.



*Fig.* 8—*Specimens after testing.* 

For the columns under low axial load of  $0.05P_o$ , (CA-1N, CB-4N, SA-7N, and SB-12N), cracking and spalling were more concentrated near the interface because failure was governed by slippage of the reinforcing bars. Closer spacing and larger amounts of lateral reinforcement in CB-4N and SB-12N provided better confinement of concrete and resulted in a reduction of the region of cracking and spalling of concrete. In columns under high axial load (CA-3N and SA-9N), although failure was initiated by bar slippage, buckling of the longitudinal bars at higher deflection excursions became the governing mode of failure and thus the cracking and spalling of concrete became more concentrated at approximately 200 mm (8 in.) from the interface, where buckling of the reinforcing bars took place. Closer spacing of the lateral reinforcement in CB-4N and SB-12N provided better confinement of concrete and resulted in a reduction of the region of cracking and spalling of concrete.

Specimen SB-12N was repaired after damaging it to a level when the longitudinal bars had yielded, representing a post-earthquake damage scenario. Such a state was reached in the sixth cycle of testing (that is, at a displacement demand of  $\delta_{max} = 3\Delta_1$ , following the loading protocol shown in Fig. 6. At this stage, considerable concrete spalling was observed and the critical longitudinal bars at the top and the bottom of the column section were found to have yielded significantly. This damage was found to be more extensive than planned. The column was returned to zero lateral displacement position and for the purpose of safety during the repair process as well as to maintain its alignment, the axial load was maintained at approximately 50 kN (11.2 kips) that corresponded to approximately 2% of axial load capacity of the column. All the loose concrete was removed and the column was repaired with a high-strength grout. After 3 days of curing, the column was wrapped with one layer of CFRP. The CFRP was allowed to cure for a week before the specimen, redesignated as SBRF1-12N, was tested to failure.

### **Retrofitted specimens**

All the retrofitted specimens had a more stable behavior compared with the control specimens. The columns under

tion of the column from the stub at the column-stub interface as a result of slippage of the spliced longitudinal bars. Thus, for Specimens CAF1-2N, CBF1-6N, SAF1-8N, SBF1-11N, and SBRF1-12N, failure was initiated by the appearance of cracks along the periphery of the columns at the interface usually from the fourth cycle ( $\delta_{max} = 2\Delta_1$ ) onward and there was no rupture of the CFRP wrap. The maximum FRP strains measured in columns tested under lower axial loads ranged between 0.0027 and 0.0033. For the wrapped columns tested under high axial loading, separation cracks appeared first at the interface due to bar slippage. At larger lateral displacements, however, buckling of the reinforcing bars resulted in rupture of the CFRP near the location of bucking. The rupture of the CFRP jacket followed separation of the fabric in the circumferential direction and resulted in considerable degradation of flexural strength of the columns. For CAF1-5N ( $P = 0.27P_o$ ), the CFRP ruptured in the 17th cycle ( $\delta_{max} = 8\Delta_1$ ) at 135 mm (5.3 in.) from the interface, whereas for SAF1-10N ( $P = 0.33P_o$ ), rupture of the fabric took place due to buckling of the reinforcing bars at 150 mm (5.9 in.) from the interface in the 15th cycle ( $\delta_{max} = 7\Delta_1$ ). Typical variation of FRP strain in Specimen CAF1-5N is shown in Fig. 7. The highest tensile strains measured in Specimens CAF1-5N and SAF1-10N were approximately 0.0063 and 0.0058, respectively. The maximum measured FRP strain values reported herein were taken before the longitudinal bars buckled and before columns reached the failure state. It should be noted that FRP ruptured at much higher strain.

low axial load had a gradual mode of failure due to separa-

Specimen SAFI-8N ( $P = 0.05P_o$ ) suffered premature failure at the end of the fourth cycle, which resulted in termination of the test in the fifth cycle. Considerable crushing and diagonal cracks were observed at the end of the stub, where steel plates were used to attach the specimen with the test apparatus. Poorly compacted concrete, causing pullout of the anchor bolts, seemed to be the reason for this unexpected failure. All the specimens except SAFI-8N, at the end of the tests, are shown in Fig. 8.



Fig. 9—Idealization of test specimen.



Fig. 10-Moment-rotation response of Specimen CA-1N.



Fig. 11—Shear (V)—tip deflection ( $\Delta_t$ ) responses of circular columns (Group I).

### **RESULTS AND DISCUSSIONS**

Figure 9 illustrates the idealized test specimen. Responses of the specimens can be presented graphically in the form of shear force V versus tip deflection  $\Delta_t$  and moment at interface M versus total differential rotation  $\theta_t$  plots. The shear force V was determined from the applied lateral load  $Q_L$ . The



Fig. 12—Shear (V)—tip deflection ( $\Delta_t$ ) responses of circular columns (Group II).

total moment at interface was the sum of the primary moment produced by the lateral load and the secondary moment caused by the axial load. The deflected shape of the column obtained from the LVDT readings was used to calculate the secondary moment. The differential rotation angle  $\theta_t$  was measured as the angle made by the tangent at the column-stub interface with the tangent drawn at the column tip. The deflection values obtained from the LVDT readings were used to compute the rotation angles. The *M* versus  $\theta_t$  and *V*- $\Delta_t$ responses of Specimen CA-1N are presented in Fig. 10 and 11, respectively. Because the rotation components are derived from the deflection values and the moments are derived from the shear and axial forces, both the V- $\Delta_t$  and M- $\theta_t$  plots follow similar pattern of cyclic excursions as is obvious from a comparison of the two responses for Specimen CA-1N. Thus for the rest of the specimens only the V- $\Delta_t$  plots are presented in Fig. 11 and 12 due to limited space. The *M*-versus- $\theta_t$ responses of all the specimens are available elsewhere.<sup>2</sup> Key events during testing such as appearance of longitudinal flexural cracks along the splice length, diagonal shear cracks, interface cracks between the stub and the column, spalling of the concrete cover, and rupture of the CFRP in the retrofitted columns are marked on the plots. Similar plots from earlier tests<sup>7,8</sup> are shown in Fig. 13.

### **Ductility parameters**

Ductility factors and work indexes are two of the most common parameters used for the seismic evaluation of structural components. It is easy to define the ductility parameters for elasto-plastic behavior. For reinforced concrete members that do not display perfectly elasto-plastic behavior, however, there is no universal definition of ductility. Thus, in evaluating the performance of the specimens tested under the current research, ductility parameters

proposed by Sheikh and Khoury<sup>18</sup> have been used herein so that comparison can be made with earlier tests.<sup>7,8</sup> These parameters, namely the displacement ductility factor  $\mu_A$ , the cumulative displacement ductility ratio  $N_{\Lambda}$ , and the work damage indicator W have been found to provide a consistent basis for the member behavior and have been defined in Fig. 14. Subscripts T and 80 added to the parameters  $\mu_{\Lambda}$ ,  $N_{\Lambda}$ , and W indicate, respectively, the value of each parameter until the end of the test and until the end of the cycle in which there is 20% reduction in the maximum lateral load. The 20% reduction represents a concrete section with a substantial remaining capacity.<sup>18</sup> Rotation ductility factors can also be similarly defined.<sup>6</sup> Because the rotation angles are derived from the deflection components, a simple relation exists between the displacement and rotation ductility parameters. The performance of the specimens is, therefore, reported herein only in terms of the displacement ductility parameters. The shear and moment capacities, as well as the ductility parameters of the specimens, are presented in Table 1. The test program was designed such that the effects of a variable could be studied by comparing two otherwise similar specimens.

## Effect of confinement by transverse steel reinforcement

The effect of confinement by internal transverse reinforcement on the performance of columns can be evaluated by studying three sets of specimens, CA-1N/CB-4N, CAF1-2N/CBF1-6N, and SA-7N/SB-12N. Specimens in each set were similar in all respects except that the CB and SB series of specimens had higher transverse steel contents. The CA series specimens had a transverse reinforcement ratio of 0.32% (No. 3 hoops at 300 mm [11.8 in.]) as compared with 1.2% (No. 3 spirals at 80 mm [3.1 in.]) for the CB series. The SA series had a transverse reinforcement ratio of 0.37% (perimeter ties only) as compared with 0.61% (perimeter and diagonal ties) for the SB series.

It is evident from Table 1 and Fig. 11 and 12 that there were considerable improvements in the ductility factors and work indexes of the circular columns containing larger amount of transverse steel in more closely spaced spirals. The better confinement of concrete produced by the uniform clamping pressure applied by the transverse reinforcement around the spliced longitudinal reinforcement helped to delay the formation of internal splitting cracks along the splice length, thus increasing the splice/bond strength of the longitudinal bars and resulting in improved ductility. The  $\mu_{\Delta 80}$  and  $W_{80}$  values for CB-4N were found to be 1.6 and 2.0 times the corresponding values for CA-1N. For the retrofitted specimens, the  $\mu_{\Delta 80}$  and  $W_{80}$  values of CBF1-6N increased 1.2 and 1.7 times those for CAF1-2N. The FRP wraps, however, were found to provide more effective confinement as compared with the presence of closely spaced spirals. This can be concluded by comparing the performance of Specimens CAF1-2N (specimen strengthened with FRP wraps) and CB-4N (specimen with closely spaced spirals), respectively, with that of control Specimen CA-1N in Table 1. The  $\mu_{\Lambda 80}$ and  $W_{80}$  values for CAF1-2N were found to be 2.97 and 18.2 times the corresponding values for CA-1N while the  $\mu_{\Delta 80}$  and  $W_{80}$  values for CB-4N were found to be 1.6 and 2.0 times the corresponding values for CA-1N. The best performance out of all the specimens was obviously observed in Specimen CBF1-6N with both closely spaced spirals and FRP strengthening.

In contrast to the circular specimens, the effect of larger amounts of transverse steel was not evident from a comparison



*Fig.* 13—Shear (V)-tip deflection ( $\Delta$ ) responses of columns (Group III and IV) without lap splices.



Fig. 14—Member ductility parameters.<sup>18</sup>

of the square Specimens SA-7N and SB-12N. This was because the ties were placed at 300 mm (11.8 in.) spacing in both types of columns. Thus, it can be concluded that inadequate confinement due to widely spaced ties was the governing factor rather than steel ratio in determining the seismic response of the columns. Higher strength of SB-12N appears to be due to larger amount of shear reinforcement. The ductility and work index values of SB-12N were slightly lower than those for SA-7N, which indicates that the section is unable to sustain the higher forces due to lack of confinement.

### Effect of strengthening with CFRP jackets

The effectiveness of CFRP jackets in the strengthening of deficient columns can be evaluated by studying six sets of specimens, CA-1N/CAF1-2N, CA-3N/CAF1-5N, CB-4N/CBF1-6N, SA-7N/SBF1-11N, SA-9N/SAF1-10N, and SB-12N/SBF1-11N. Specimens in each set were similar in all respects except that the second specimen was strengthened with one layer of CFRP in the plastic hinge region. Specimen SAF1-8N could not be included in this comparison because, as mentioned previously, its premature failure prevented the completion of the test. Thus, the effectiveness of CFRP in strengthening SA-7N was evaluated by comparing it with SBF1-11N. As discussed previously, because of the widely spaced transverse steel, different tie configurations in square

sections did not lead to any significant changes in the performance of specimens.

As is evident from Fig. 11 and 12, all the control specimens exhibited poor hysteretic response due to premature splice failure resulting in low ductility and energy dissipation. Retrofitting of the columns with FRP resulted in more stable response with less pinching and lower strength and stiffness degradation rates. The improvements were found to be much larger in circular columns than in square ones due to higher efficiency of CFRP wraps in circular configuration. Whereas the work index increased by up to approximately 20 times due to retrofitting, increase in the moment capacity ranged from 7% (CAF1-5N) to 61% (CAF1-2N) in the columns tested.

The influence of the CFRP wraps in improving the performance of both circular and square deficient columns was primarily due to the confinement of concrete in the plastic hinge regions. The confining pressure helped to delay the initiation of internal cracking, thereby preventing the splitting of the concrete around the spliced longitudinal bars and also shifted the critical buckling deformation of the spliced bars to higher levels. As evident from Table 1, retrofitting considerably increased the flexural strength and ductility of the columns. Specimens CAF1-2N and CBF1-6N were found to perform the best out of 12 columns of Groups I and II with the highest values of ductility parameters and work indexes. The  $\mu_{\Lambda 80}$  and  $W_{80}$  factors increased by approximately 3.0 and 18.2 times, respectively, for CAF1-2N and 2.2 and 15.8 times for CBF1-6N, when compared with the control Columns CA-1N and CB-4N. The behavior of retrofitted specimens suggests that the adverse effect of inadequate steel confinement could be compensated by external CFRP reinforcement. It is to be noted herein that the efficiency of the CFRP strengthening will also depend on the load combination during a seismic event and the effect of high axial load on column performance discussed in the following sections.

The enhancements in the total work index were highest for Specimens CAF1-5N and SAF1-10N, which were tested under high axial loads, with  $W_T$  values increasing 16.6 and 9.0 times the corresponding values of control Specimens CA-3N and SA-9N, respectively (Table 1). Corresponding enhancement in  $W_T$  for CAF1-2N, tested at low axial load, was 6.5 times that of CA-1N. This was because under low axial load, both the retrofitted and the control specimens were able to sustain a large number of cyclic excursions; the retrofitted columns did so with gradual degradation of strength while the strength degradation in control columns was more severe. For the columns tested at high axial load, the control specimens underwent rapid degradation of strength and failed much earlier than the retrofitted specimens.

# Effect of retrofitting of damaged columns with CFRP jackets

To determine the effectiveness of CFRP wrap in the postearthquake retrofit of damaged columns, SB-12N was damaged and then patched with high-strength grout and retrofitted with one layer of CFRP (renamed SBRF1-12N). The non-shrink natural aggregate grout was prepared with a water cement ratio of 0.22. As per the specifications provided by the supplier, the grout was expected to reach a compressive strength of 27 MPa (3920 psi) in 3 days and 37 MPa (5360 psi) in 7 days.

The extent of damage prior to the repair is shown in Fig. 8. The ductility factor  $\mu_{\Delta 80}$  and work index  $W_{80}$  of the retrofitted damaged column, SBRF1-12N were approximately 34 and 167% higher compared with the original column, SB-12N. The repaired column SBRF1-12N, however, achieved only 63% of the displacement ductility and 30% of the work damage indicator value of a similar Specimen SBF1-11N that was strengthened before any prior damage. It should be noted that Specimen SB-12N was damaged more than anticipated, and it was unable to recover its original load and moment carrying capacity. In spite of the extensive damage, retrofitting with a single layer of CFRP was successful in obtaining a significantly more ductile column behavior. The effectiveness of CFRP in retrofitting damaged columns obviously depends on the extent of the sustained damage.

### Effect of axial load level

The effect of axial load on the cyclic behavior of the test columns can be evaluated by comparing the responses of four sets of specimens, CA-1N/CA-3N, CAF1-2N/CAF1-5N, SA-7N/SA-9N, and SBF1-11N/SAF1-10N. Specimens in each set were similar in all respects except that  $P/P_o$  for the first column was 5% of its axial load carrying capacity and for the second column it was 27 and 33% for the circular and square columns, respectively. The columns under low axial load were found to have a more ductile response than their counterparts tested under high axial load levels.

It can be observed from Table 1 that all specimens tested under high axial load experienced considerable degradation in the ductility and work indexes, and this was more pronounced for the control specimens. Due to higher axial load, the retrofitted Specimens CAF1-5N and SAF1-10N experienced approximately 8% reductions in  $\mu_{\Lambda}$  over Specimens CAF1-2N and SBF1-11N, while the control Specimens CA-3N and SA-9N experienced approximately 15 and 29% reductions in  $\mu_\Delta$  over Specimens CA-1N and SA-7N, respectively. The most significantly affected ductility parameter was the work index, which decreased on an average by almost 50% for all the specimens due to increased axial load levels. In terms of total work index, the retrofitted Specimen CAF1-5N experienced 30% reduction in  $W_T$  over Specimen CAF1-2N, while the control Specimens CA-3N and SA-9N experienced 73 and 84% reductions in  $W_T$ over Specimens CA-1N and SA-7N, respectively.

It is also evident from Fig. 11 and 12 that the control columns tested under high axial load levels underwent rapid degradation of strength and stiffness and failed at much lower levels of displacements excursions than the columns tested at low axial loads. It is to be noted that high axial load improves the performance of the splice by reducing the tensile force produced by flexure during cyclic lateral load excursions. As a result, the splice-deficient columns tested at high axial load levels may be expected to have a better performance. The poor confinement of the core concrete and buckling of the longitudinal bars, however, play a major role in causing the poor performance of columns under high axial load. The presence of FRP improves this behavior substantially resulting in a less severe effect of higher axial load on the retrofitted columns. It is also to be noted that for the columns strengthened with the CFRP jackets, CAF1-5N and SAF1-10N, rupture of FRP was observed to result in ultimate failure as the mode of failure of these columns was shifted from slippage to buckling of longitudinal reinforcement.

### Effect of shape of column cross section

In the current study, six circular columns and six square columns were constructed and tested. In both sets of

columns, the ratio of the core area to the gross area of the column section was kept constant at approximately 75% to allow direct comparison between the two sets. The effect of shape of column cross section on its performance can be evaluated from four sets of specimens, CA-1N/SA-7N, CA-3N/ SA-9N, CAF1-2N/SBF1-11N, and CAF1-5N/SAF1-10N. It is evident that the responses of the control square specimens, SA-7N and SA-9N, including their ductility parameters and work indexes, were similar to those of the control circular columns, CA-1N and CA-3N. This is due to the fact that confinement of concrete in all these columns was minimal. It should be noted that the higher axial loads in both types of columns were approximately equal to their respective balanced loads. A load of  $0.05P_{o}$  represents approximately 19% of the balanced load in circular columns while in square columns, it is only 15% of the balanced load.

The retrofitted circular columns were found to have better seismic performance than the comparable square columns. Circular Specimens CAF1-2N and CAF1-5N had 40% higher ductility factors and approximately 160% higher work indexes  $(W_{80})$  compared with the square columns, SBF1-11N and SAF1-10N. The total work index  $(W_T)$  values for Columns CAF1-2N and CAF1-5N were 67 and 59% higher than those of SBF1-11N and SAF1-10N, respectively (Table 1). The higher ductility and work index values in retrofitted circular sections indicates more efficient confinement provided by external CFRP wraps than in the square sections. With concrete dilation, the CFRP wraps in circular columns are placed in hoop tension and, thus, they exert a continuous circumferential confining pressure. In square sections, however, the confining pressure is applied only at the corners and the external CFRP jacket is more susceptible to rupture at the corners under high stress concentration.

### Effect of lap-spliced longitudinal bars at columnstub interface

The effect of the presence of poorly detailed spliced longitudinal bars at the column/stub interface on the columns performance can be evaluated by comparing the columns of the present study with specimens tested by Sheikh and Yau<sup>7</sup> and Iacobucci and Sheikh.<sup>8</sup> The specimens in each of the sets, CA-3N/S-4NT and CAF1-5N/ST-4NT as well as SA-9N/AS-1NS and SAF1-10N/ASC-2NS were similar except that the first columns were detailed with lap splices at the interface while the second columns contained continuous bars. Shear versus deflection responses of Specimens S-4NT, ST-4NT, AS-1NS, and ASC-2NS are shown in Fig. 13. A direct comparison between the ductility parameters of the columns with and without the splice details can only be made if the columns in both sets were constructed with concrete of similar strengths.

The presence of lap splices in the plastic hinge regions of the square columns reduced the displacement ductility factors  $\mu_{\Delta 80}$  by approximately 35 to 50% and the work indexes ( $W_{80}$ ) by up to 75%. The most noticeable differences between the columns with and without spliced reinforcing bars were observed in their hysteretic loops. It can be observed from Fig. 11 and 12 that the responses of the columns with spliced reinforcing bars exhibited considerable pinching. These columns suffered bond failure due to slippage of the spliced longitudinal bars and, consequently, most of the damage was concentrated near the interface. The failure of the columns with continuous reinforcing bars was governed by concrete crushing in the compression zone followed by buckling of the longitudinal bars and, thus, the failure occurred away from the interface. Moreover, these columns had a more stable hysteretic behavior with lower degradation of strength (Fig. 13). The presence of lap splices in the potential plastic hinge region of the column leads to reduced ductility and unstable hysteretic behavior of the column.

### SUMMARY AND CONCLUSIONS

A total of 12 columns (six with circular and six with square cross sections) were tested under inelastic reversed cyclic lateral loading simulating an earthquake event. The circular specimens had a diameter of 356 mm (14 in.) whereas the square ones had a cross section of 305 mm (12 in.). The columns were 1.47 m (58 in.) long and had a 510 x 760 x 810 mm (20 x 30 x 32 in.) stub at one end. The columns and the stub were cast separately to create a construction joint at the column-stub interface, as would be encountered in most existing reinforced concrete columns in the field. The column details were chosen in accordance with the pre-1971 building code provisions. The variables studied in this experimental program included presence of lap spliced reinforcing bars in the columns, CFRP retrofitting, shape of cross section, transverse steel details, and level of axial load. The seismic performances of the test columns have been evaluated in terms of their ductility parameters to study the effect of the different variables. The following conclusions were reached from this study.

1. Presence of poorly detailed lap splices in the potential plastic hinge region of a column leads to significantly reduced ductility and unstable hysteretic behavior with rapid degradation of strength due to premature splice failure. Strengthening of the plastic hinge region of the columns with CFRP jackets considerably increased their flexural strength, ductility, and energy dissipation capacity, resulting in a more stable hysteretic behavior. The external confinement provided by the CFRP jackets can thus compensate to a large extent for the adverse effects of inadequate confinement and poor splice details of deficient columns;

2. Retrofitting of previously damaged columns with CFRP jackets resulted in improvements in strength and ductility. The level of improvement, however, would be dependent on the damage experienced by the column prior to retrofitting;

3. Higher transverse steel contents with close spacing or efficient steel distribution provided better confinement of the core concrete and created a clamping pressure around the lap spliced longitudinal reinforcing bars thereby delaying the initiation of concrete splitting, onset of bar buckling, and slippage. This resulted in a more ductile performance of the columns;

4. High axial load resulted in considerable reduction in the ductility and energy dissipation capacity of columns, with the work index indicative of energy dissipation capacity, being the worst affected parameter; and

5. Ductility improvements in square columns with lap splices as a result of CFRP retrofitting were significantly lower than that for comparable circular columns due to more efficient confinement mechanism in circular shapes.

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### NOTATION

- $E_s$  = modulus of elasticity of steel
- $F_u$  = ultimate strength of longitudinal steel
- $F_{y}$  = yield strength of longitudinal steel
- $\vec{K}$  = slope obtained from the V- $\Delta_t$  plot as ratio of maximum displacement,  $\Delta_t$  for cycle to corresponding load at deflection
- L =length from column tip to column-stub interface, mm
- M = moment at column-stub interface, kN·m
- $N_{\Delta}$  = cumulative displacement ductility ratio
- $P^-$  = applied axial load, kN
- $P_o$  = unconfined theoretical axial load carrying capacity of column, kN
- $Q_L$  = lateral load applied to columns, kN
- s = spacing of lateral steel along axis of member, mm
- V = shear force sustained by column, kN
- W = work damage indicator
- $\Delta_1$  = yield displacement at column-stub interface computed from theoretical sectional response, mm
- $\Delta_2$  = displacement at section on descending portion of response curve corresponding to certain drop in lateral load, mm
- $\Delta_t$  = deflection of column tip computed with respect to initial tangent from column-stub interface at load, mm
- $\delta$  = deflection at column-stub interface, mm
- $\delta_1 =$  deflection of column tip with respect to test hinge location at point of application of load, mm
- $\delta_2$  = total deflection of column test hinge location at point of application of load with respect to tangent from column-stub interface, mm
- $\varepsilon_u$  = ultimate strain in steel
- $\varepsilon_y$  = yield strain in steel
- $\mu_{\Delta}$  = displacement ductility factor
- $\theta_t$  = angle between tangents at column-stub interface and column test hinge location at point of application of load, radians
- $\rho$  = longitudinal steel ratio
- $\rho_s$  = volumetric ratio of ties to concrete core measured center-to-center of perimeter ties

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