Behavior of Cyclically Loaded Squat Reinforced Concrete Bridge Columns Upgraded with Advanced Composite-Material Jackets

Medhat A. Haroun, F.ASCE,[1] and Hussein M. Elsanadedy, M.ASCE[2]

Abstract: This paper summarizes comprehensive experimental studies on scaled models of squat bridge columns repaired and retrofitted with advanced composite-material jackets. In the experimental program, a total of 14 half-scale squat circular and rectangular reinforced concrete columns were tested under fully reversed cyclic shear in a double bending configuration. In order to provide a basis for comparison, a total of three as-built columns were tested. Another 10 column samples were tested after being retrofitted with different composite jacket systems. One circular as-built column was repaired after failure. The repair process involved both crack injection as well as addition of carbon/epoxy composite jacket. The repaired column was then retested and evaluated. Experimental results showed that all as-built columns developed an unstable behavior and failed in brittle shear mode. The common failure mode for all retrofitted samples was due to flexure with significant improvement in the column ductility. The repaired column demonstrated ductility enhancement over the as-built sample.

DOI: 10.1061/(ASCE)1084-0702(2005)10:6(741)

CE Database subject headings: Bridges, concrete; Composite materials; Concrete columns; Concrete, reinforced; Cyclic loads.

Introduction

One of the major problems associated with the seismic performance of reinforced concrete bridges is the brittle shear failure of squat bridge columns. Such short and, hence, relatively stiff members tend to attract a greater portion of the seismic input to the bridge during an earthquake and require the generation of large seismic shear forces to develop the moment capacity of columns. Estimation of design flexural strength based on elastic methods, along with much less conservative shear strength provisions during the 1950s and 1960s, frequently resulted in actual shear strength of as-built bridge columns being significantly less than the flexural capacity. Generally the transverse reinforcing steel was inadequately anchored in the cover concrete, which can be expected to spall off under cyclic loading, and therefore, the problem was compounded. Hence, shear failure is likely in such columns, accompanied not only by rapid strength, stiffness, and physical degradation, but also by poor energy dissipation characteristics. This has been evidenced by the brittle shear failure of bridge columns in recent California earthquakes.

In order to upgrade and retrofit bridge columns with insufficient shear reinforcement, several retrofitting measures have been developed by researchers and practicing engineers. After being tested at the University of California, San Diego, steel jackets have been proven to be an effective means to retrofit substandard squat bridge columns (Verma et al. 1993). Advanced composite materials have been recently recognized as a reliable alternative to steel jacketing, and have been applied to bridge retrofit. The advantages of composite retrofit systems include lightweight, high strength- or stiffness-to-weight ratios, corrosion resistance, and in particular, the ease of installation. All these advantages make such materials most suitable for retrofitting bridge columns, and moreover, contrary to other retrofitting techniques, composite jacket retrofit will not affect the lateral stiffness of the columns, and hence will not alter the bridge dynamic characteristics. Carbon fiber systems are generally applied to columns either by hand-lay operations or by using filament-winding techniques. E-glass systems are either prefabricated shells manufactured in the factory and bonded to the column on site or applied to columns by hand lay-up. This retrofit technique is essentially a passive system in which the overwrap is not under any significant stress until an earthquake occurs.

Several composite jacket systems have been developed and validated in laboratory or field conditions. Priestley and Seible (1991) undertook experimental testing on 40% scale bridge piers retrofitted by glass fiber jackets. Tests demonstrated significant improvement of seismic performance with increased strength and ductility.

The cyclic performance of composite-jacketed squat bridge columns was studied through an experimental program conducted at the University of California, Irvine, on both circular and rectangular columns (Haroun et al. 1997; Haroun et al. 1998; Haroun et al. 1999; Elsanadedy 2002). Column samples were constructed with continuous longitudinal steel and were tested for shear and confinement enhancement in a fixed–fixed condition.

[1]Dean and AGIP Professor, School of Sciences and Engineering, The American Univ. in Cairo, Egypt; and Professor Emeritus, Univ. of California, Irvine.

[2]Assistant Professor, Dept. of Civil Engineering, Helwan Univ., Cairo, Egypt; formerly, Graduate Student, Dept. of Civil and Environmental Engineering, Univ. of California, Irvine.

Note. Discussion open until April 1, 2006. Separate discussions must be submitted for individual papers. To extend the closing date by one month, a written request must be filed with the ASCE Managing Editor. The manuscript for this paper was submitted for review and possible publication on July 24, 2002; approved on March 28, 2005. This paper is part of the ***Journal of Bridge Engineering***, Vol. 10, No. 6, November 1, 2005. ©ASCE, ISSN 1084-0702/2005/6-741–748/$25.00.

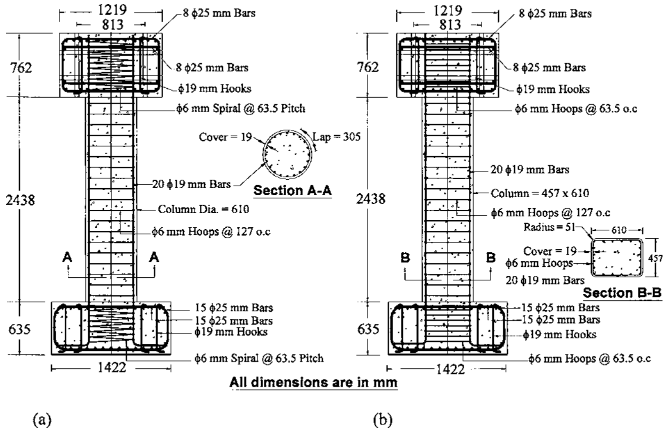

Fig. 1. Reinforcement details for squat column samples: (a) circular columns and (b) rectangular columns

Experimental Program

Fourteen half-scale squat columns were tested for shear and confinement enhancement in a double bending configuration. These samples included seven circular and seven rectangular columns. Circular columns are classified as two as-built columns (CS-A1 and CS-A2), four columns retrofitted by four different jackets (CS-R1 to CS-R4), and one column repaired by carbon/epoxy composite jacket (CS-P1). Rectangular columns are detailed as one as-built column (RS-A1), and six rectangular columns retrofitted by six different types of jackets (RS-R1 to RS-R6). It should be noted that the letters "C" and "R" denote circular and rectangular columns, respectively; "S" symbolizes shear testing; and "A," "R," and "P" stand for as-built, retrofitted, and repaired columns, respectively. Dimensions and reinforcement details of test columns are shown in Fig. 1. All samples were built with a strong RC footing and a strong RC top box as indicated in Fig. 1. All columns had an aspect ratio of 2 with a clear height of 2.44 m, and were reinforced longitudinally by 20 ϕ19 mm bars, uniformly distributed around the column section with a concrete cover of 25 mm. Circular columns had a diameter of 610 mm and were transversely reinforced by ϕ6-mm hoops spaced at 127 mm on centers. Rectangular columns were built with a 457-mm $\times$ 610-mm rectangular cross section, and had transverse steel of ϕ6-mm rectangular hoops spaced at 127 mm on centers. Columns CS-A2, CS-R3, CS-R4, and CS-P1 were reinforced with Grade 60 steel (nominal strength = 414 MPa). All other columns were reinforced with Grade 40 steel (nominal strength = 276 MPa). Ready-mixed concrete with a nominal strength of 34.5 MPa was used throughout the test program. Actual material properties for all squat columns are shown in Table 1.

All composite jackets for the test matrix were designed by the manufacturers to comply with current design requirements of the California Department of Transportation (Caltrans) for composite casings (Chapman et al. 1997). For squat columns, the areas within the top and bottom 610 mm of the column were regarded as the plastic hinge zones, whereas the rest of the column was considered as the nonplastic hinge zone. In general, composite jackets were designed to provide a minimum confinement pressure of 2.1 MPa within the plastic hinge regions and 1.0 MPa in the nonplastic hinge region without exceeding a jacket strain of 0.004. Composite jackets for Columns CS-R1 and CS-R2 were designed to meet these requirements, except that the plastic hinge zones were taken as the top and bottom 914 mm of the column. Composite jacket configuration for Sample CS-R1 is shown in Fig. 2(a). Jackets for Columns CS-R3 and CS-P1 were designed to provide a minimum confinement pressure of 2.1 MPa for the entire height of the column. However, Sample CS-R4 was retrofitted with two layers of carbon/epoxy composite jacket, according to a minimum confinement pressure of 1.0 MPa for the entire height of the column. All rectangular columns were retrofitted using rectangular composite jackets. Jackets for

Table 1. Properties of Squat Column Samples

Test sample	Concrete strength (MPa)	Yield stress of main steel (MPa)	Composite jacket properties: Type	Thickness (mm)	Tensile strength (MPa)	Tensile modulus (GPa)
CS-A1	36.8	299.1	As-built circular column			
CS-R1	40.8	299.1	carbon/epoxy[a]	0.7	4168	231.5
CS-R2	39.2	299.1	carbon/epoxy[a]	0.7	4430	230.1
CS-A2	35.3	480.7	As-built circular column			
CS-R3	34.2	480.7	E-glass/epoxy[b]	10.3	424	18.5
CS-R4	37.6	480.7	carbon/epoxy[b]	1.2	1245	103.8
CS-P1	35.7	480.7	carbon/epoxy[b]	2.3	1245	103.8
RS-A1	37.1	299.1	As-built rectangular column			
RS-R1	38.1	299.1	carbon/epoxy[a]	1.0	4382	226.0
RS-R2	39.3	299.1	carbon/epoxy[a]	1.0	4430	230.1
RS-R3	44.0	299.1	carbon/epoxy[a]	1.0	4168	231.5
RS-R4	44.0	299.1	E-glass/vinyl ester[c]	7.6	744	36.5
RS-R5	44.0	299.1	carbon/epoxy[d]	5.2	937	63.0
RS-R6	42.6	299.1	E-glass/polyester[c]	7.6	641	36.4

[a]Properties are based on net-fiber area.

[b]Properties are based on gross-laminate area.

[c]Pre-cured shells.

[d]Low-modulus carbon/epoxy composite system with properties based on net-fiber area.

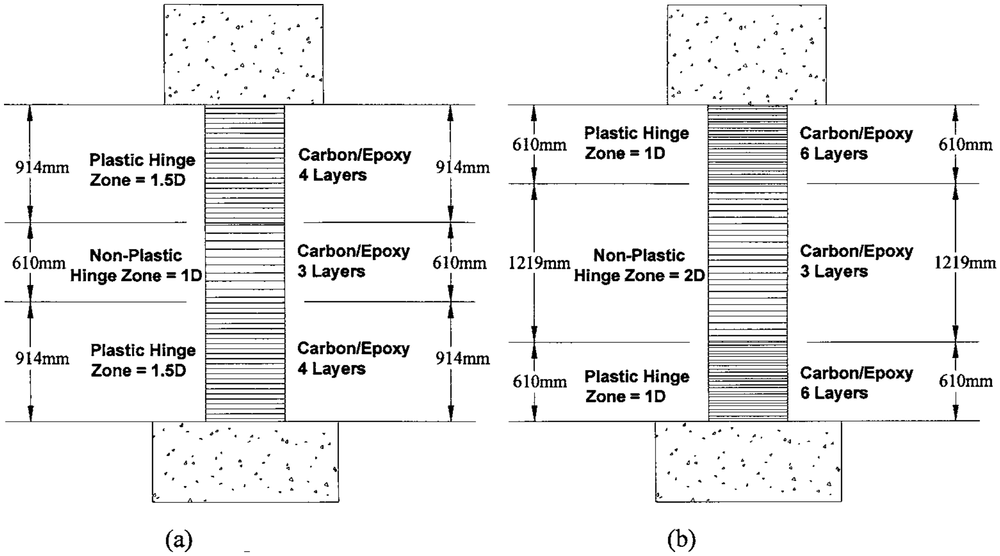

Fig. 2. Examples of composite-jacket configuration: (a) Sample CS-R1 and (b) Sample RS-R1

Columns RS-R1 and RS-R5 were designed similarly to Samples CS-R1 and CS-R2, except that the plastic hinge zones were taken as the top and bottom 610 mm of the column and the required jacket thickness was increased by a factor of 1.5. However, Columns RS-R2, RS-R3, RS-R4, and RS-R6 were designed according to a minimum confinement pressure of 2.1 MPa for the entire height of the column with the increase of the required jacket thickness by a factor of 1.5. Composite jacket configuration for Sample RS-R1 is shown in Fig. 2(b). The effectiveness of composite jackets in enhancing seismic resistance of bridge columns depends upon confinement of the column concrete. Thus, high strength and stiffness are required in the hoop direction throughout the design life of the overwrap. Therefore, all composite systems used in this study had fibers running in the hoop direction (i.e., 90° angle to the column axis). Details of composite jackets for all test samples are listed in Table 1. It should be noted that the composite jacket properties in Table 1 are average values based on the test results of control samples. In order to prevent an increase in the flexural strength and stiffness of jacketed columns, a 25-mm gap was left between the jacket and the column ends (i.e., top of footing and load stub soffit) for all retrofitted and repaired samples.

Test Setup

The test setup was designed to subject the columns to a constant axial compressive load and cyclic horizontal loads. As shown in Fig. 3, each of the shear enhancement columns was subjected to a lateral load applied by a specially designed test rig using a pantagraph arrangement to restrain the rotation at the top of the column, thereby simulating the fixed–fixed condition. The applied axial load satisfies the requirements of Caltrans for 10% of the column's axial load capacity based on the original concrete design strength of 22.4 MPa. The shear enhancement circular columns were loaded with 645 kN of axial load. The rectangular shear columns were loaded with 676 kN of axial load.

Instrumentation

Calibrated load cells were used to monitor and record applied lateral forces. Lateral displacements at different levels up the column were measured by 508-mm travel string potentiometers relative to a free-standing reference frame. Electric resistance strain gages were mounted on the surfaces of longitudinal reinforcing bars, transverse hoops, and composite jackets. Full details of strain gage positions are given elsewhere (Haroun et al. 1999; Elsanadedy 2002).

Test Procedure

An axial load was applied to each column sample by post-tensioning two steel rods with a hydraulic jack at the top of the column as shown in Fig. 3. According to the guidelines of Caltrans (1993), peak forces controlled the initial loading cycles for each test until the column developed the lateral load corresponding to the first yield of longitudinal steel, V_y. Then, the test

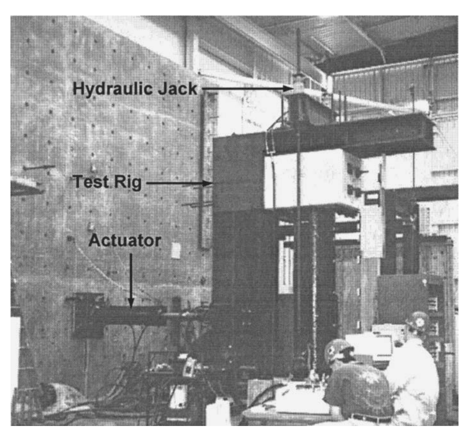

Fig. 3. Test setup for squat columns

Table 2. Test Results for Squat Column Samples

Test sample	V_y(kN)[a]	V_{if}(kN)[b]	Δ_1(mm)[c]	Δ_y(mm)[d]	K^e_{col}(kN/mm)[e]	$V_{u\text{-exp}}$(kN)[f]	$\Delta_{u\text{-exp}}$(mm)[g]	$\mu_{\Delta u\text{-exp}}$[h]	Drift ratio[i] (%)
(a) Circular columns with Grade 40 steel									
CS-A1	328.4	442.5	5.3	7.1	61.57	426.5	9.7	1.4	0.40
CS-R1	322.3	456.4	5.1	7.1	63.44	497.2	81.0	11.4	3.32
CS-R2	320.3	453.7	5.3	7.4	60.04	507.5	89.4	12.1	3.67
(b) Circular columns with Grade 60 steel									
CS-A2	461.3	603.0	16.3	21.1	28.4	445.2	15.5	0.7	0.64
CS-R3	447.5	618.0	16.5	22.9	27.11	740.6	103.4	4.5	4.24
CS-R4	450.6	625.1	12.7	17.5	35.49	684.5	71.1	4.1	2.92
CS-P1	446.1	623.2	15.7	21.8	28.34	741.9	97.3	4.5	3.99
(c) Rectangular columns									
RS-A1	405.7	507.1	6.4	7.9	63.90	405.7	6.4	0.8	0.26
RS-R1	394.4	531.8	7.4	9.9	53.54	560.0	77.7	7.9	3.19
RS-R2	395.4	536.3	7.4	9.9	53.68	558.2	95.8	9.7	3.93
RS-R3	402.8	549.6	7.6	10.4	52.86	559.1	103.6	9.9	4.25
RS-R4	403.2	547.1	8.4	11.2	48.10	569.8	109.7	9.7	4.50
RS-R5	403.0	552.7	8.1	10.9	49.58	581.4	101.3	9.2	4.15
RS-R6	401.9	546.7	8.6	11.7	46.54	581.4	114.8	9.7	4.71

[a]Lateral load at first yield of longitudinal reinforcement.
[b]Lateral load at ideal flexural strength.
[c]Average measured displacement at first-yield state.
[d]Idealized yield displacement.
[e]Effective lateral stiffness.
[f]Average maximum measured lateral load.
[g]Ultimate average displacement at 80% of peak lateral load.
[h]Ultimate displacement ductility.
[i]Drift ratio at ultimate ductility.

was stopped and the yield displacement was determined from

$$\Delta_y = \frac{V_{if}}{V_y}\Delta_1 \tag{1}$$

where Δ_1=average of the measured peak displacements corresponding to the first-yield lateral load, V_y, in the push and pull directions. The ideal flexural lateral load capacity, V_{if}, is computed based on the extreme concrete compressive strain of 0.004 (0.005 for the jacketed columns) and on measured material properties (Xiao et al. 1997). After the column developed the first yield capacity, loading cycles were controlled by the peak displacement. For each load (or displacement level), three fully reversed cycles were completed. It is important to mention that the frequency of applied load (or induced displacement) is constant throughout the test program; it was picked up to be around one cycle per minute, which corresponds to a frequency of 0.0167 Hz. All cycles started with the push direction first, then went into the pull direction.

Experimental Results

Presented in this paper are only representative samples from the test results. Detailed experimental results can be found elsewhere (Haroun et al. 1999; Elsanadedy 2002). A summary of the experimental results for all test columns is presented in Table 2. The key elements of the response of each sample are indicated in the last

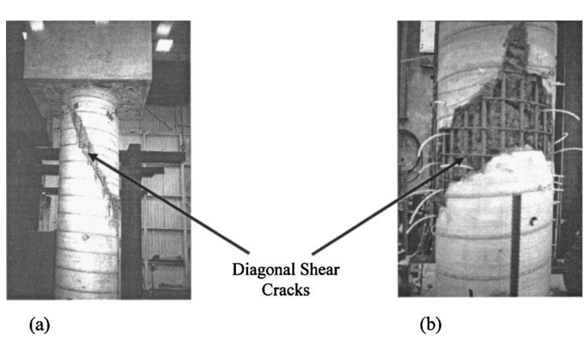

Fig. 4. Shear failure of circular as-built columns: (a) Sample CS-A1 and (b) Sample CS-A2

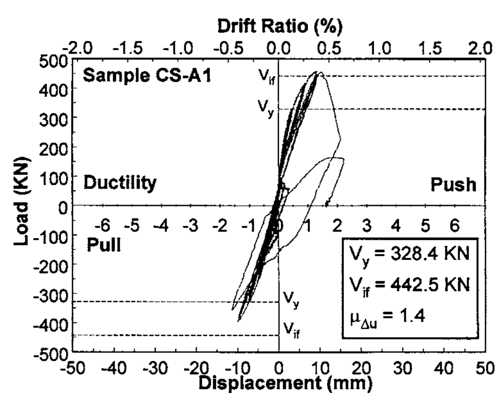

Fig. 5. Hysteresis loops for Sample CS-A1

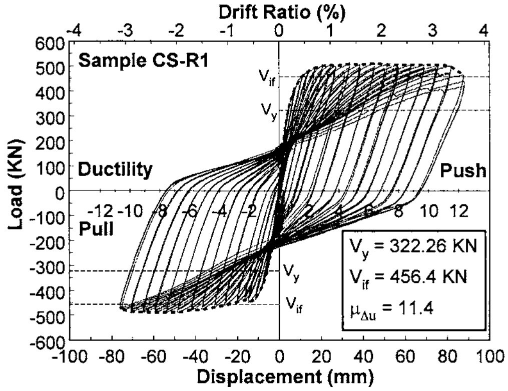

Fig. 6. Hysteresis loops for Sample CS-R1

four columns of the table. The effective lateral stiffness of each sample was calculated by dividing the first-yield lateral load of the column by the corresponding average measured displacement. The lateral strength of each column was represented by the average of the maximum lateral load in the push and pull directions. The ultimate average displacement was estimated per Caltrans guidelines as that corresponding to 80% of the peak lateral load (Caltrans 1993). The ultimate displacement ductility of the column sample was determined by dividing the ultimate average displacement by the idealized yield displacement, which was calculated in Column 4 in Table 2 from Eq. (1). From the table, it is evident that the first-yield lateral load for columns with Grade 60 steel is greater than that for columns with Grade 40 steel by about 1.5 times, which is the same ratio between the nominal yield stresses for both steel grades. In addition, the idealized yield displacement for columns with Grade 60 steel is more than twice that for columns with Grade 40 steel. Yet, it is noted that columns with Grade 40 steel achieved higher ductility levels than columns with Grade 60 steel. In general, it is illustrated that the effective lateral stiffness of both repaired and retrofitted columns was approximately the same as that for corresponding as-built samples. Therefore, composite-material jackets are superior to steel and concrete jackets as they did not alter the lateral stiffness of the column, and consequently, the bridge's dynamic characteristics were not affected. A brief description of the behavior of test samples follows.

(a) (b)

Fig. 8. Details of repair procedure: (a) after injection of all cracks and (b) after composite jacket wrapping

Circular As-Built Columns

The two as-built columns (CS-A1 and CS-A2) developed an unstable response due to brittle shear failure as shown in Fig. 4. Sample CS-A1 failed at ductility of 1.4 as shown in the hysteresis loops in Fig. 5. However, Sample CS-A2 did not even reach ductility 1.0 and failed at ductility 0.7. For both samples, a large diagonal shear cracking, with an angle of about 30° to the column axis, was observed at column failure.

Circular Retrofitted Columns

In contrast to the as-built columns, all circular retrofitted samples performed in a consistently stable fashion throughout the test. Samples CS-R1 and CS-R2 exhibited excellent performance up to a ductility of more than 10. Hysteresis loops for Sample CS-R1 is shown in Fig. 6. Failures of Columns CS-R1 and CS-R2 were due to concrete crushing within the plastic hinge regions. Sample CS-R3 experienced flexural performance up to a ductility of 4.5 when the test was stopped due to limitations of the test rig displacement. No signs of failure were observed for this column. Sample CS-R4 underwent ductile flexural behavior up to a ductility of 4.0 when failure occurred due to concrete crushing associated with composite jacket failure in the hoop direction and

Fig. 7. Flexural failure of retrofitted circular Column CS-R4

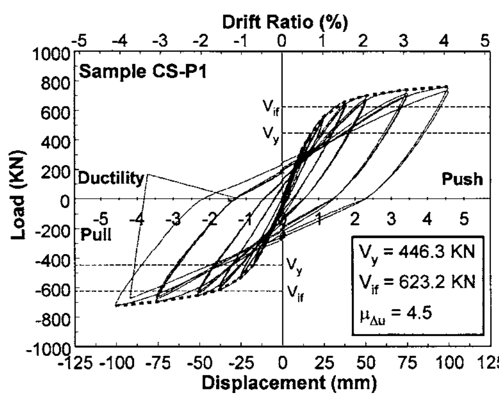

Fig. 9. Hysteresis loops for Sample CS-P1

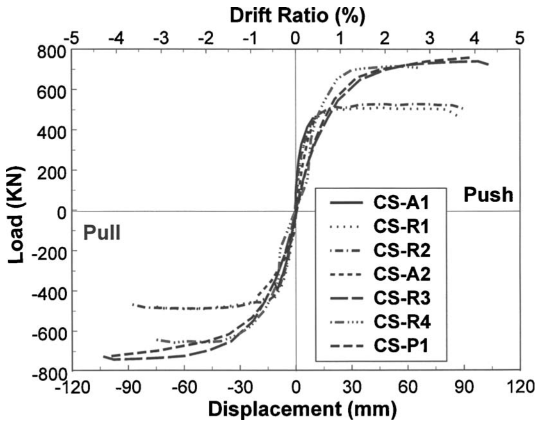

Fig. 10. Load-displacement envelopes for circular columns

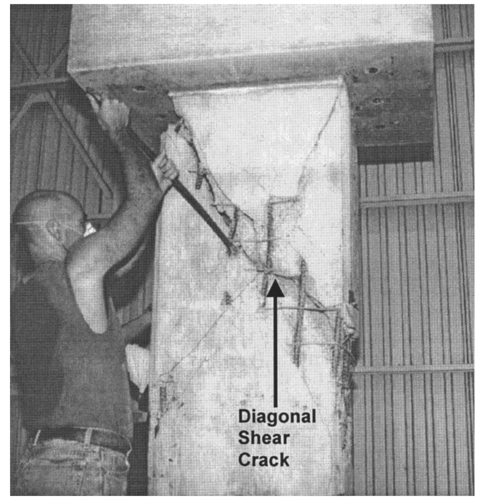

Fig. 12. Shear failure of as-built rectangular Column RS-A1

longitudinal bar buckling within the plastic hinge region at the top of the column (Fig. 7). For all columns, no rupture in the longitudinal steel was observed throughout the test.

Circular Repaired Column

After the as-built Column CS-A2 failed prematurely in shear as shown in Fig. 4(b), it was repaired and then retested. The repair procedure included:

1. Removal of all loose concrete,
2. Cleaning out cracks and column surface with compressed air,
3. Insertion of 6-mm diameter plastic tubes into all major crack openings,
4. Sealing all exposed cracks through all plastic tubes,
5. Injection of grout into the column and restoration of cover concrete using special polymer concrete [Fig. 8(a)], and
6. Hand lay-up of four layers of carbon/epoxy composite jacket [Fig. 8(b)].

The repaired Column CS-P1 was retested with the same loading pattern as Sample CS-A2. The test showed very satisfactory performance and the column developed flexural behavior up to ductility 4.5 as shown in the hysteresis loops in Fig. 9. Failure of this column was due to concrete crushing within the plastic hinge zones.

All Circular Column Data

Summary of load-displacement envelopes for all circular shear columns is shown in Fig. 10.

Rectangular As-built Column

Sample RS-A1 failed in shear without showing ductile behavior. The column failed at ductility 0.8 as shown in the hysteresis loops in Fig. 11. At failure, the column had a large shear cracking with an angle of 30° to its longitudinal axis. This may be seen in Fig. 12.

Rectangular Retrofitted Columns

In general, all rectangular retrofitted columns (RS-R1 to RS-R6) behaved extremely well in a flexural ductile fashion. All samples continued to carry 80% of their maximum lateral load up to more than a ductility of 7.0. An example of the load-displacement hysteresis loops is shown in Fig. 13 for Sample RS-R3. None of the tested samples failed in shear, but rather in extreme concrete crushing within the plastic hinge regions. This failure is seen in Fig. 14 for Sample RS-R2.

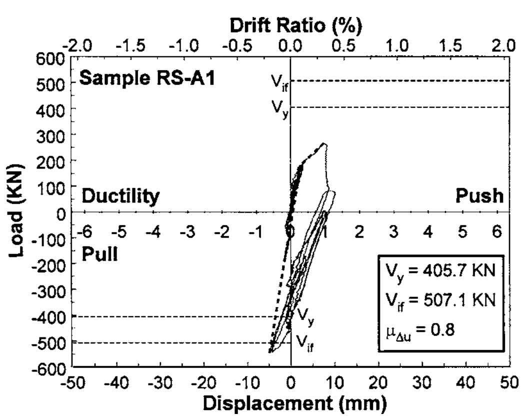

Fig. 11. Hysteresis loops for Sample RS-A1

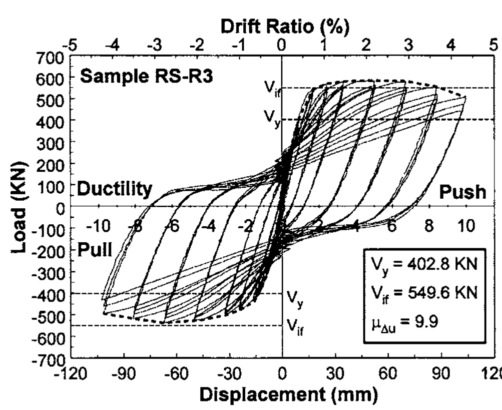

Fig. 13. Hysteresis loops for Sample RS-R3

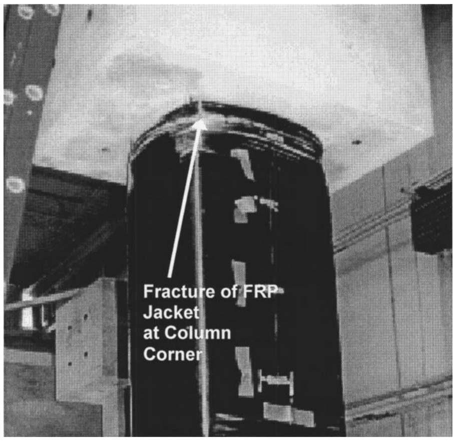

Fig. 14. Failure of retrofitted rectangular Column RS-R2

All-Rectangular Column Data

A summary of load-displacement envelopes for all rectangular columns is shown in Fig. 15.

Conclusions

The following conclusions are derived from the inclusive experimental program conducted on squat columns:

1. The brittle shear failure mode of short and stiff circular and rectangular reinforced concrete bridge columns designed and constructed in the pre-1971 era, as evidenced in the past California earthquakes, was experimentally verified in this study. The short columns, with larger stiffness and hence a short natural period tend to attract a greater portion of the seismic input, resulting in the generation of large seismic shear forces. With inadequate transverse reinforcement for confinement and shear provisions, these columns tend to fail in a sudden brittle manner, involving severe stiffness, strength, and physical degradation at a very limited displacement ductility capacity. All as-built samples underwent sudden shear failure at a very low level of displacement ductility.
2. Contrary to the common assumption of a 45° shear plane inclination, steeply inclined shear failure planes of about 30° to the vertical axis were observed in the as-built column tests. Based on this observation, it can be concluded that most codes still follow a conservative design approach in estimating the shear capacity of RC columns, as the conservatism stems from the continuous use of the traditional 45° truss mechanism approach for predicting the shear carried by transverse steel.
3. In general, all circular and rectangular retrofitted samples tested in this study met their design requirements considering the ductility demand. Composite-material jackets effectively provided the requisite passive confinement within the plastic hinge zone and imparted enhancement to the shear strength of the column, and hence changed the mode of failure from brittle shear failure in the as-built columns to flexural ductile failure for both repaired and retrofitted samples.
4. This study also revealed that as-built columns that failed in shear and then were repaired by crack injection plus composite jacketing can achieve a performance comparable to retrofitted columns.
5. It should be noted that the effective lateral stiffness of all retrofitted and repaired columns was approximately the same as that for corresponding as-built samples. Accordingly, composite jackets have shown advantage over steel jackets as they did not alter the column lateral stiffness, and consequently, the bridge dynamic characteristics were not affected.
6. In addition to the comprehensive experimental program described herein, numerical models were developed in order to predict the seismic performance of column data. The models were calibrated using the experimental data in this study as well as data of columns tested elsewhere. Besides performance prediction, the developed models were used to propose generic retrofit design guidelines for bridge columns with insufficient shear reinforcement.

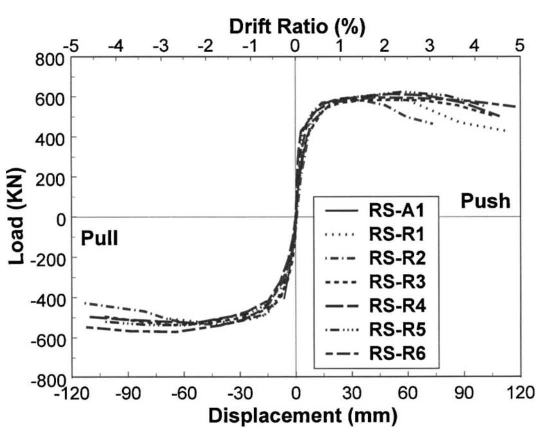

Fig. 15. Load-displacement envelopes for rectangular columns

Notation

The following symbols are used in this paper:

K^e_{col} = effective lateral stiffness;
V_{if} = lateral load at ideal flexural strength;
$V_{u\text{-exp}}$ = maximum experimental lateral load;
V_y = lateral load at first-yield state;
Δ_1 = displacement at first-yield state;
$\Delta_{u\text{-exp}}$ = ultimate experimental displacement;
Δ_y = idealized yield displacement; and
$\mu_{\Delta u\text{-exp}}$ = ultimate experimental displacement ductility.

References

California Department of Transportation (Caltrans). (1993). *Bridge design specifications*, Caltrans, Sacramento, Calif.

Chapman, W., et al. (1997). *Pre-qualification requirements for alternative column casings for seismic retrofit*, California Department of Transportation, Sacramento, Calif.

Elsanadedy, H. M. (2002). "Seismic performance and analysis of ductile composite-jacketed reinforced concrete bridge columns." PhD thesis, Univ. of California at Irvine, Irvine, Calif.

Haroun, M. A., and Feng, M. Q. (1997). "Lap splice and shear enhancements of composite-jacketed bridge columns." *Proc., 13th U.S.–Japan Workshop on Bridge Engineering*, Tsukuba Science City, Japan.

Haroun, M. A., Feng, M. Q., Bhatia, H., Baird, K., and Elsanadedy, H. M. (1999). "Structural qualification testing of composite-jacketed circular and rectangular bridge columns." *Final Report to the California Department of Transportation*, Dept. of Civil and Environmental Engineering, Univ. of California at Irvine, Calif.

Haroun, M. A., Feng, M. Q., Bhatia, H., and Sultan, M. (1998). "Cyclic qualification testing of jacketed bridge columns in flexure and shear." *Proc., 16th Int. Modal Analysis Conf.*, Santa Barbara, Calif.

Priestley, M. J. N., and Seible, F. (1991). "Seismic assessment and retrofit of bridges." *Structural Systems Research Project, Report No. SSRP-91/03*, Dept. of Applied Mechanics and Engineering Sciences, Univ. of California at San Diego, La Jolla, Calif.

Verma, R., Priestley, M. J. N., and Seible, F. (1993). "Assessment of seismic response and steel jacket retrofit of squat circular reinforced concrete bridge columns." *Structural Systems Research Project, Report No. SSRP-92/05,* Dept. of Applied Mechanics and Engineering Sciences, Univ. of California at San Diego, La Jolla, Calif.

Xiao, Y., and Rui, M. (1997). "Seismic retrofit of RC circular columns using prefabricated composite jacketing." *J. Struct. Eng.*, 123 (10), 1357–1364.