Rehabilitation of Shear Critical Concrete Columns by Use of Rectangular Steel Jackets
by Riyad S. Aboutaha, Michael D. Engelhardt, James O. Jirsa, and Michael E. Kreger

This paper describes an experimental research program on the use of rectangular steel jackets for seismic retrofit of nonductile reinforced concrete frame columns with inadequate shear strength. Eleven large-scale columns were tested to examine the effectiveness of various types of steel jackets for improving the ductility and strength of columns with inadequate shear resistance. Response of the columns before and after being strengthened with steel jackets was examined. Several types of steel jackets were investigated, including rectangular solid steel jackets and partial steel jackets. The test results indicate that a thin rectangular steel jacket can be a highly effective retrofit measure for reinforced concrete columns with inadequate shear resistance.

Keywords: concrete columns; cross ties; ductility; energy dissipation; seismic retrofit; shear reinforcement; shear strength; steel jacket.

INTRODUCTION
Existing reinforced concrete building columns constructed prior to the 1970s may be vulnerable structural elements in an earthquake due to two major deficiencies: inadequate shear strength, and inadequate flexural strength and ductility due to an inadequate lap splice in the longitudinal reinforcement. Older design and construction practices led to columns with an actual shear strength less than that required to develop the flexural strength of the member. Short columns with inadequate shear strength can experience a brittle shear failure under large cyclic lateral forces, resulting in a rapid degradation of strength and stiffness. Shear often dominates the behavior of columns with small height-to-depth ratios. Such columns may be created during original design or through restraint of a column by structural or nonstructural elements over a portion of its height (a captive column). The restraint shortens the unsupported length of the column and increases its relative stiffness, thereby attracting more shear during an earthquake than was likely considered in design.

Under the cyclic actions of earthquake forces, diagonal cracks form in each direction and cross each other. Shear transfer along diagonal cracks depends on the levels of strains developed in the transverse reinforcement. Yielding of transverse reinforcement may result in significant degradation of shear transfer. For older reinforced concrete buildings, columns generally have small amounts of transverse reinforcement. Consequently, the strains in ties may reach levels well beyond the yield strain, resulting in early strength and stiffness degradation of the column. Past earthquakes have clearly demonstrated the vulnerability of these systems and the need for their retrofit.

BACKGROUND AND PREVIOUS RESEARCH
Several researchers have investigated strengthening of concrete columns with inadequate shear strength using steel jackets, concrete jackets, and fiber reinforced polymer composite jackets. Unjoh and Kawashima investigated the use of 1.0-mm-(0.04-in.)-thick steel jackets for strengthening bridge piers with short development length in the longitudinal bars. A total of three columns were tested. All specimens were 500 x 500 mm (20 x 20 in.) square columns. Every column was reinforced with 46 10 mm (46 No. 3) deformed longitudinal bars. Half of the longitudinal bars were terminated at 900 mm from the top of the base. The columns were transversely reinforced by 6 mm (No. 2) round bars every 250 mm (10 in.). One column (R1) was tested as a basic unretrofitted specimen. Two columns, R2 and R3, were strengthened with 1.0-mm-thick steel jackets of length equal to 1.0 and 1.50 times the depth of the column cross section, respectively. The basic column R1 showed the development of a hinge at the bar cutoff section and a shear failure at the same location. However, the strengthened columns developed the flexural capacity of the column. With the longer steel jacket, shear cracks were completely eliminated near the bar cutoff section.

Yoshimura et al. conducted a study on the use of welded steel plates for seismic retrofit of short columns with inadequate shear strength. A total of nine 7 x 7-in. specimens were tested. Three columns were tested as basic unretrofitted columns with different amounts of longitudinal and transverse reinforcement. The shear span-to-depth ratio was 1.0. Three columns were strengthened with 6-mm-(1/4-in.)-thick steel plates. The steel plates were fabricated in two L-shaped panels in the plan. After installation around the column, the panels were welded at two opposite corners. The 5-mm (0.2-in.) gap between the concrete column and the steel plates was injected with epoxy-based polymer cement. The three basic columns were repaired after testing with similar steel jackets. All of the specimens were tested under a constant axial load equivalent to 0.1A fg c and reversed cyclic lateral load. Test results demonstrated that if a short column is strengthened by welded steel plates, then brittle shear failure can be prevented and the column can develop its ultimate flexural capacity. It was also shown that similar steel jackets can be used for repair of damaged short columns that have failed in shear.

Chai et al. investigated the use of cylindrical steel jackets for seismic shear strengthening of circular columns with inadequate shear strength. Six 24-in.-diameter columns were tested under constant axial load equivalent to 0.1A fg c and reversed cyclic lateral load. Three columns were tested as basic unretrofitted columns with different amounts of steel.
reinforcement. The retrofitted columns were strengthened with 3/16-in.-thick cylindrical steel jackets. The “as built” circular column exhibited a relatively stable response up to a drift ratio of just less than 1.0 percent. Afterwards, the column failed in a brittle shear manner. The hysteretic response of the retrofitted columns showed a large increase in ductility and energy absorption.

Priestley et al.8,9 investigated a total of 14 full-scale columns for enhanced shear strength. Eight circular columns and six rectangular columns were tested. The circular and rectangular columns were strengthened with circular and elliptical steel jackets. Test variables included the moment-to-shear ratio, yield strength of steel, and the shape of the cross section. All columns were tested under reversed cyclic lateral load and constant axial load. While all the basic unretrofitted columns exhibited shear failure, the strengthened columns with steel jackets, both circular and elliptical, developed ductile flexural response at high levels of displacement ductility.

Concrete jackets have been investigated for strengthening columns with inadequate shear capacity. Bett et al.10 conducted an experimental study on the behavior of repaired and strengthened reinforced concrete short columns using concrete jackets. The test specimens had a 12-in. square cross section reinforced with eight No. 6 longitudinal bars, No. 2 ties spaced at eight in., and 1.0-in. cover. One of the specimens was tested, repaired by jacketing, and then retested. The remaining two specimens were strengthened by jacketing prior to testing. All specimens were tested under reversed cyclic lateral load and constant axial load. Strengthening was done by encasing the original column with a shotcrete jacket reinforced with closely spaced transverse ties. Specimens were first roughened by light sandblasting. The jacket reinforcement cages were tied and placed in position, then the columns were shotcreted and float finished. The repaired column was done in a similar way, but after removal of the loose concrete cover. Testing of the basic unstrengthened column showed shear dominated failure with considerable loss of stiffness at displacements in excess of 1.0 percent drift. Both the strengthened and repaired columns performed better than the original column. They were laterally stiffer and stronger than the original unstrengthened column.

Sugano11 reported results of an experimental investigation on seismic shear strengthening of short reinforced concrete columns using welded wire fabric wrapping and mortar. The test specimen had a 250 x 250-mm (10 x 10-in.) cross section. The shear span-to-depth ratio was 1.0. The specimens were tested under reversed cyclic lateral load and constant axial stress of 26.3 kg/cm² (375 psi). Compared to the basic unretrofitted column, the strengthened column showed a considerable increase in strength and stiffness with reasonable ductility.

Jackets made of advanced fiber-reinforced composite materials have also been investigated for seismic strengthening of columns with inadequate seismic shear resistance. Priestly et al.12 investigated the use of fiberglass/epoxy composite wraps for strengthening of circular columns with inadequate shear strength. Eight-ft-high, 24-in.-diameter columns were tested under a comparatively low constant axial load of 133 kips and reversed cyclic lateral load. The shear span-to-depth ratio was 2.0. The column was reinforced with 26 No. 6 Grade 60 longitudinal deformed bars. Transversely, the column was reinforced with no. 2 hoops at every 5 in. The fiber wrap jacket was extended over the full height of the column. Test results indicated that fiber wrap jackets could improve shear strength and ductility of circular reinforced concrete columns with inadequate shear strength.

Saadatmanesh et al.13 experimentally investigated the use of fiber composite straps for seismic strengthening of circular and rectangular columns. One-fifth scale bridge columns were tested under lateral cyclic displacements and constant axial loads. The test columns were designed to model pre-1971 bridge columns. The unidirectional composite straps were impregnated with epoxy resin and wrapped around the potential plastic hinge region of the column. Both active and passive straps were investigated. Test results showed that the composite straps are very effective in seismic strengthening of concrete columns with inadequate seismic resistance.

RESEARCH SIGNIFICANCE

Past research has demonstrated that rectangular steel jackets can be highly effective in improving the shear strength of small rectangular or square columns. However, it appears that little experimental data is available on large rectangular columns and it is unclear if thin rectangular jackets will be as effective in confining and strengthening large columns. Consequently, a major objective of this research program is to evaluate the effectiveness of thin rectangular steel jackets for seismic retrofit of large rectangular reinforced concrete columns with inadequate shear strength. An additional objective is to evaluate steel collars and partial steel jackets, i.e., jackets that do not completely surround the column. Partial steel jackets may be useful where partition walls or curtain walls prevent access to all four sides of a column.

In this research program, large-scale column specimens were tested before and after being strengthened with solid or partial steel jackets or steel collars. The test specimens represent typical building columns designed and constructed in the 1950s and 1960s in the U. S. The specimens were designed with shear strength substantially less than that required to develop the flexural strength of the member. The remainder of this paper summarizes the results of the test program. Further details are available in Aboutaha.14

TEST PROGRAM

Figure 1 shows the test setup. The test specimen was a cantilever, with the fixed end framing into a large footing. The specimen is intended to approximate half of a column in a real building frame. All specimens were subjected to cyclic
lateral force at the tip of the cantilever. Lateral loads were increased in 10 kip increments until significant inelastic displacement was observed. Lateral displacements were then increased in increments corresponding to 0.5 percent drift ratios. The columns were laterally loaded, two complete cycles at every load/drift ratio level. All columns were tested without axial load. A total of 11 large columns were tested. Eight rectangular columns (SC1-SC8) were loaded in the weak direction and three (SC9-SC11) were loaded in the strong direction of the column section.

Details of column reinforcement were based on provisions of the ACI 318-56 and ACI 318-63 building codes. Three columns were tested as basic unretrofitted specimens. Since the shear span-to-depth ratio (a/d) is an important factor for strengthening columns with an inadequate shear strength, the test specimens had two different a/d ratios. Fig. 2 shows the details of the basic reference columns. For all the test specimens, the longitudinal reinforcement bars were No. 8 Grade 60. Transversely, the specimens were reinforced with No. 3 Grade 40 every 16 in. Tables 1 and 2 summarize the details and material properties of the test specimens.

The retrofitted test columns were similar to the basic reference columns but were strengthened with different steel jackets before testing. A total of seven retrofitted columns were tested. Three columns (SC6, SC7, and SC10) were retrofitted with solid steel jackets, two columns (SC8 and SC11) were retrofitted with partial steel jackets, and two columns (SC2 and SC5) were retrofitted with steel collars made of channel sections.

Columns SC1, SC3, SC4, and SC9 were basic reference columns. Columns SC1 and SC4 were transversely reinforced with cross ties at every longitudinal bar and loaded in the weak direction. Columns SC3 and SC9 were transversely reinforced with cross ties at every other longitudinal bar and loaded in the weak and the strong directions, respectively.

Fig. 3 shows typical details of a column retrofitted by the use of a solid rectangular steel jacket. The steel jackets were prefabricated in two L-shaped panels. After being assembled around the column, the ends of the two L-shaped panels were fillet welded together over the full height of the steel jacket. The average size of the fillet weld was 1/4 in. The 1-in. gap between the steel jacket and the concrete column was filled with nonshrink cementitious grout. The steel jacket was terminated 1 in. from the top of the footing to avoid possible bearing of the steel jacket against the footing.

Columns SC2 and SC5 were strengthened with steel collars made of four C4 x 7.25 steel channels. The channels were connected at the column corners using 1/2 in. A325 bolts. The 3/4-in. gap between the steel collars and the concrete column was filled with nonshrink grout. Fig. 4 shows the details of a column retrofitted by the use of steel collars.

Columns SC6 and SC10 were strengthened by the use of solid rectangular steel jackets, as described above. Column SC7 was retrofitted with a bolted rectangular steel jacket.
The BSJ was prefabricated in two L-shaped panels, which were bolted together after being assembled around the column. The bolted steel jacket avoids the need for field welding. This, in turn, avoids the fire hazards and fumes associated with welding in existing buildings.

Column SC8 was strengthened with a 0.06 m (1/4 in.) partial steel jacket as shown in Fig. 5. The use of a partial steel jacket may be useful for strengthening columns with limited access to all four sides of the column. In a real building frame, such limits might result in peripheral columns with curtain walls framing into the sides. Through bolts were provided to assist the steel jacket in confining the concrete section and to resist shear forces. The steel jacket was prefabricated in a U-shaped panel in plan. After the installation of the threaded rods through the column, the gap between the column and the steel jacket was filled with nonshrink cementitious grout. The threaded rods were unbonded to the concrete column.

Column SC11 was strengthened by the use of a C-shaped partial steel jacket (Fig. 6). It represents a situation where a partition wall frames into the middle third of the column sides. The jacket was attached to the column by 16 1-in.-

diameter adhesive anchor bolts. For ease of installation, holes were drilled separately into the concrete column and the steel jacket. After the jacket was erected around the column, adhesive anchor bolts were then placed 8 in. into the concrete column. The 1-in. gap between the column and the
Fig. 6—Details of C-shaped partial steel jacket.

The details of the retrofitted test specimens permitted the following variables to be examined: concrete strength, shear span-to-depth ratio, different types of steel jackets, and field-welded versus field-bolted installations.

TEST RESULTS

Each of the basic unretrofitted columns (SC1, SC3, SC4, and SC9) exhibited shear failure. Figure 7(a) shows the hysteretic response of Column SC3, which was loaded in the weak direction. During the test of Column SC3, the first flexural cracks formed at the column/footing interface at a load of 20 kips. Increased loading to 40 kips caused the develop-
ment of several flexural cracks over the bottom half of the column. During the cycle to 70 kips, most of the flexural cracks extended diagonally, reflecting the influence of shear. Increased loading to 80 kips caused the development of major diagonal shear cracks over 70 percent of the column height. During the 90 kip cycle, the major diagonal shear cracks extended over the full height of the column. The major cracks penetrated the concrete compression zones at the base of the column, reducing the depth of the compression zone to the thickness of the concrete cover. Also, the cross ties yielded as the load was increased to 90 kips. During subsequent cycles, Column SC3 developed limited inelastic deformations, followed by a dramatic loss in strength and stiffness at displacements equivalent to larger than 2 percent drift ratio. The drift ratio is defined as the lateral displacement divided by column height. The major diagonal cracks opened very widely, as shown in Fig. 8(a). This was accompanied by degradation of the concrete compression zones. The column lost its lateral strength due to the major diagonal shear failure mechanism followed by concrete compression shear failure. Column SC3 did not develop its flexural capacity. Based on strain gage readings, the strains in the longitudinal bars did not exceed 75 percent of yield strain. Column SC3 showed only very limited ductility, associated with the yielding of transverse reinforcement crossing the major diagonal crack.

The basic Column SC9, loaded in the strong direction, exhibited a dramatic shear failure. Figure 7(b) shows the hysteretic response of Column SC9. At a load of 30 kips, the first crack was observed at the column/footing interface. The flexural cracks extended diagonally as the load was increased to 70 kips. Several diagonal shear cracks and flexural shear cracks formed over the full height of the column during the cycles to 90 kips. Figure 8(b) shows Column SC9 during the test. Increased loading to 110 kips caused the development of major diagonal cracks over the height of the column. These diagonal cracks did not have any significant effect on the load displacement response of the column at that load. As the load was increased to 130 kips, the transverse reinforcement yielded. Also, several diagonal cracks penetrated the concrete compression zones at the bottom of the column. Increased loading beyond 130 kips caused concrete compression shear failure at the bottom of the column. Column SC9 lost strength and stiffness rapidly during subsequent cycles. Overall, Column SC9 exhibited virtually no ductility.

The remaining two basic columns, SC1 and SC4, showed somewhat better response than Column SC3. This was likely due to the higher concrete strength used in Column SC1 and higher amount of transverse reinforcement used in Column SC4. Both Columns SC1 and SC4 exhibited shear failure but with less severe loss in strength and stiffness as compared to Column SC3.

The retrofitted columns with solid steel jackets (SC6, SC7, and SC10) showed a very ductile flexural response. Column SC6, which was loaded in the weak direction, was a retrofitted version of Column SC3. It was strengthened with a welded solid steel jacket made of 1/4-in.-thick steel plates. The response of Column SC6 is shown in Fig. 7(c). The initial response of the column was essentially elastic, up to a lateral load of 140 kips. The corner longitudinal bars yielded during cycles to 90 kips, whereas the intermediate longitudinal bars yielded during cycles to 130 kips. It appears that the corner bars were well confined by the corners of the steel jacket and, consequently, yielded at an earlier stage. The maximum measured strain on the steel jacket during the test was just below $300 \times 10^{-6}$, approximately 1/6 of yield strain of the steel jacket. The overall response of Column SC6 was excellent, with the specimen maintaining its full flexural strength through drift ratios of 4 to 5 percent. The jacketed Column SC6 developed substantially higher strength and ductility than the retrofitted Column SC3, despite the fact that Column SC6 had only about two-thirds the concrete strength of SC3.

After completion of the test, the steel jacket was removed from Column SC6 and the concrete column was inspected. Investigation of the concrete column revealed the following:
1. Major bond failure occurred between the main longitudinal bars and the surrounding concrete over the bottom two-thirds of the column height.

2. Severe deterioration of the concrete was observed in the compression zones. However, these zones maintained integrity since they were well-confined between the column concrete core on one side and the grout and steel jacket on the other. Figure 9(a) shows a close-up of the concrete compression zone on the west side.

3. Steep diagonal shear cracks were observed. These cracks were narrow and did not open up because they were restrained by the steel jacket. Figure 9(b) shows the crack pattern on the side of Column SC6.

The results of Column SC6 indicate that the rectangular solid steel jacket was a very effective retrofitting system. It considerably improved the strength and ductility of a column with inadequate shear strength. Column SC7 was provided with a 1/4-in.-thick solid steel jacket, similar to Column SC6. In both cases, the jacket was prefabricated as two L-shaped panels (Fig. 3). For Column SC6, the two panels were connected by fillet welds after being placed around the column. For Column SC7, on the other hand, the two panels were connected by a line of bolts extending over the height of the jacket. Column SC7 showed excellent behavior, similar to Column SC6 with the welded jacket. Figure 7(d) shows the response of Column SC7.

Column SC8 was strengthened by the use of partial steel jacket (Fig. 5) and loaded in the weak direction. As indicated by the response plotted in Fig. 7(e), Column SC8 showed a similar response as Column SC6. However, since the steel jacket confined only two-thirds of the column section, the confined and unconfined sides by the steel jacket showed quite different levels of damage. Due to the lack of symmetry of the retrofitted column section, the column developed torsional shear stresses that caused severe damage on the unconfined side of the column. During the 2 percent drift ratio cycles, the cross ties yielded. However, the strains measured on the steel jacket were well below yielding, in the range between 150 and 200 micro strains. Although Column SC8 experienced severe damage on the unconfined side, it maintained its peak lateral load to almost 7 percent drift ratio. Overall, the hysteretic response of Column SC8 was excellent.

Column SC10 was strengthened by the use of a welded solid steel jacket. Column SC10 was a retrofitted version of Column SC9 and was loaded in the strong direction. Fig. 7(f) shows the hysteretic response of column SC10. During the test, the first flexural cracks were observed at the column/footing interface at a load of 50 kips. Increased loading to 110 kips caused the development of major shear cracks behind the steel jacket. Portions of the cracks could be seen on the column just above the top of the steel jacket. At a load of 100 kips, the main longitudinal bars yielded, immediately followed by the development of major diagonal shear cracks. Inelastic deformations were first observed during the cycles to 260 kips.

In the inelastic range, Fig. 7(f) shows the unsymmetric response of Column SC10. That is, large displacements were imposed on the specimen in the push direction of the actuator than in the pull direction. This was a result of stroke limitations of the actuator. A large stroke was available in the push direction rather than in the pull direction. In the pull direction, Column SC10 was loaded to 3 percent drift ratio, the maximum stroke of the actuator in that direction. However, in the push direction, the column was loaded until it developed its flexural capacity at 4 percent drift ratio. During the last cycle, the column was loaded to the maximum stroke of the actuator in the push and pull directions. The flexural capacity was developed by fracturing one of the main longitudinal bars, close to the column/footing interface. At peak load, strains in the transverse reinforcement reached $1500 \times 10^{-6}$, more than 75 percent of yield strain. At the same load, the maximum measured strain on the steel jacket was only $450 \times 10^{-6}$, about 25 percent yield strain of the steel jacket.

After completion of Test SC10, the steel jacket was removed and the column concrete surface was inspected. The damage was very similar to that observed for Column SC6, as described earlier. The overall performance of
Column SC10 was excellent. Compared to the basic unretrofitted Column SC9, the jacketed column showed a large increase in strength and ductility.

Column SC11 was strengthened with a C-shaped partial steel jacket (Fig. 6) and loaded in the strong direction. The response of Column SC11 was not satisfactory. A sudden anchorage failure of anchor bolts was observed as the load was increased to 170 kips. Splitting of the concrete surrounding the anchor bolts caused outward movement of the steel jacket. Consequently, Column SC11 lost most of its lateral capacity and stiffness. The performance of Column SC11 indicates that the use of partial steel jackets with anchor bolts could neither prevent sudden shear failure nor improve displacement ductility. It is believed that connecting the ends of the two C-panels to each other with through bolts might have improved the performance of Column SC11.

Finally, Columns SC2 and SC5 were strengthened by the use of steel collars (Fig. 4). These specimens showed some improvement in response compared to the basic unretrofitted specimens. The steel collars, however, were not nearly as effective as the solid steel jackets. Further details of the response of Columns SC2 and SC5 are available in Reference 14.

**DISCUSSION**

The test results indicate that thin rectangular steel jackets can dramatically improve the seismic response of large reinforced concrete columns with inadequate shear strength. In this section, further analysis of the test results are provided.

**Envelopes of cyclic response**

Figure 10(a) shows the envelopes of the cyclic response of Columns SC3, SC5, SC6, SC7, and SC8. These columns were loaded in their weak direction. The basic unretrofitted Column SC3 did not develop its nominal flexural capacity, which is equivalent to a lateral load of 110 kips. The columns retrofitted with steel jackets, however, exceeded their flexural yield capacity and exhibited large ductility and high energy dissipation. Although the concrete strength of Column SC6 was just two-thirds of that of the basic Column SC3, Column SC6 reached much higher strength and ductility levels. The lateral strength of Column SC6 was slightly higher than that of Columns SC7 and SC8, probably because the longitudinal bars were strain-hardened during a previous test (Column SC4 test). The response of Column SC7 may better represent the response of a retrofitted column with inadequate shear strength, since the longitudinal bars were not previously strain-hardened. The plot also reveals that the steel collar system was not as effective as the solid steel jacket system.

Figure 10(b) shows the envelopes of the cyclic response of Columns SC9, SC10, and SC11. These columns were loaded in the strong direction. The basic unretrofitted Column SC9 exhibited very poor behavior. However, the retrofitted Column SC10 exhibited much higher strength, ductility, and energy dissipation. The flexural capacity of Column SC10 was developed with one longitudinal bar on the tension side fractured during the push cycle to 4 percent drift ratio. However, if the main longitudinal bars of Column SC10 had not strain-hardened during previous tests, the fracture of the longitudinal bar may not have occurred until higher drift ratios were imposed.

The envelopes of cyclic response clearly demonstrate that thin rectangular steel jackets can significantly improve the strength, ductility, and energy dissipation of rectangular columns with inadequate shear strength. Although the steel jackets were terminated 1 in. from the bottom of the column to prevent an increase in flexural capacity, the retrofitted columns showed significant increase in flexural capacity and, consequently, an increase in shear demand. This increase in the flexural strength is likely due to: 1) strain hardening of the longitudinal steel reinforcing bars, and 2) an increase in the concrete compressive strength due to confinement provided by the steel jacket. The increase in the flexural capacity and the consequent increase in shear demand was not critical because the steel jackets provided very high shear resistance.

**Strains in transverse reinforcement**

Strains in the transverse reinforcement of the basic unstrengthened columns reached very high levels while remaining below yield in the strengthened columns, even though the strengthened columns were loaded to higher lateral shear forces and had lower concrete strength. Figures 11(a) and 11(b) show plots of load versus strain in strain Gages SG1 and SG2 for the basic unretrofitted Column SC9 and the strengthened Column SC10, respectively. Both strain gages were installed at mid-length of the peripheral tie located at 24 in. from the bottom of the column. While strain Gage SG1 exhibited very high strains, Column SC9 did not develop its yielding flexural capacity but rather experienced shear failure. On the other hand, the maximum strain measured by the strain gage SG2 was below...
1400 × 10⁻⁶, which is about 30 times smaller that the maximum strain measured at SG1. The maximum strain was low because of the presence of the steel jacket, which did not allow the major diagonal shear cracks to open, even though the column was loaded to very large lateral displacements.

**Strains in steel jacket**

Strain Gages SG3 and SG4 were installed on the steel jackets of Columns SC6 and SC10 on the orthogonal side to the direction of loading. Strain Gages SG3 and SG4 were located 8 in. from the bottom of the column, at the same level as the first layer of transverse reinforcement. Figures 12(a) and 12(b) shows the measured strains in Gage SG3 and SG4 during the test. The plots show that the outside face of the steel jacket experienced tensile strains in the transverse direction. These strains may be caused by the concrete lateral pressure on the compression side, which may develop bending strains in the steel jacket. But since the strains on the outside face of the steel jacket are tensile strains during the push as well as the pull cycles, it is believed that these strains are due to bearing of the concrete column on the sides of the steel jacket. Figure 13 qualitatively illustrates the deformations that developed in the steel jackets under lateral shear load. Although the rectangular steel jacket has poor out-of-plane flexural stiffness, test results showed that a 1/4-in.-thick steel jacket can provide sufficient confinement for columns as wide as 36 in. This observation is supported by the increase in the flexural capacity of the strengthened columns over the basic unstrengthened columns.

**General behavior of jacketed columns in shear**

Before the development of the major diagonal shear cracks, the response of the retrofitted columns with solid steel jackets was almost identical to that of the basic unretrofitted columns, with the exception that the retrofitted columns had a slightly higher stiffness. During the tests, the major diagonal shear cracks developed on both the unretrofitted and retrofitted columns at almost the same lateral loads, which suggests that the steel jackets were passive. No significant shear resistance was provided until the concrete column deformed and developed major diagonal shear cracks.

Prior to major diagonal shear cracking, the strain in the steel jackets was very small, much smaller than the corresponding strain of the concrete. After the development of the major diagonal shear cracks, the steel jacket carried an increasingly higher share of the total shear force. Thus, the steel jacket did not prevent the development of the shear cracks, but it prevented widening of the major diagonal shear cracks. The widening of the major diagonal shear cracks was associated with very high strains in the transverse reinforcement, well beyond yield levels.

The response of the transverse reinforcement to cyclic straining imposed by lateral forces does not involve signifi-
Design Recommendations

Based on the observations and measurements made in this test program, a simple model was developed to predict the shear strength of a jacketed column and to determine the required thickness of the jacket. From the experimental observations, it appears that an important function of the steel jacket is to prevent shear cracks in the concrete from opening widely. The reduction in jacket stiffness that would result from its yielding may permit greater crack opening, and therefore greater deterioration in strength. Consequently, an important premise of this steel jacket design model is that yielding of the jacket should be prevented to obtain satisfactory performance from the retrofitted column.

The shear strength enhancement provided by a rectangular steel jacket is conservatively estimated by considering the jacket to act as a series of independent square ties of thickness and spacing \( t_{sj} \) and \( s_{sj} \), where \( t_{sj} \) is the steel jacket thickness. Conservatively, the shear stresses in the steel jacket between the assumed square ties are ignored. The shear force carried by the steel jacket \( V_{sj} \) can then be computed as

\[
V_{sj} = A_{sj} \cdot F_{sj} \cdot \frac{d_{sj}}{s_{sj}} \quad (1)
\]

where

\[
A_{sj} = \text{area of the assumed square tie (in.}^2\text{)} = (t_{sj})^2
\]

\[
F_{sj} = \text{stress in the steel jacket; a conservative value (based on observed stresses in test specimens) equal to one-half of the actual yielding stress of the steel jacket (ksi)}
\]

\[
d_{sj} = \text{total depth of the steel jacket (in.)}
\]

\[
s_{sj} = \text{spacing between the square ties, equal} t_{sj} \text{ (in.)}
\]

In Eq. (1) it is assumed that shear cracks or diagonal strut inclinations for the retrofitted columns is at an angle \( \theta = 45 \) deg. Based on a maximum stress equal to one-half of the yield stress in the steel jacket, Eq. (1) can be rewritten as

\[
V_{sj} = A_{sj} \cdot \left(\frac{F_{ysj}}{2}\right) \cdot \frac{d_{sj}}{s_{sj}} \quad (2)
\]

As noted above, the steel jacket is designed to remain elastic to limit the width of shear cracks in the concrete. In Eq. (2), the stress in the steel jacket is conservatively limited to one-half the yield stress to accomplish this design objective.

The nominal shear capacity of a retrofitted reinforced concrete column with inadequate shear strength by the use of rectangular steel jackets can then be represented as

\[
V_n = V_c + V_{st} + V_{sj} \quad (3)
\]

where

\[
V_n = \text{nominal shear strength at section (kips)}
\]

\[
V_c = \text{nominal shear strength provided by concrete (kips)} = 2.0 \sqrt{f_c} \cdot b_w d
\]

\[
V_{st} = \text{nominal shear strength provided by transverse ties (kips)} = A_{st} \cdot F_{yt} \cdot d/s
\]

\[
V_{sj} = \text{nominal shear strength provided by the steel jacket (kips)} = A_{sj} \cdot F_{ysj} / 2 \cdot (d_{sj}/s_{sj})
\]

In ordinary reinforced concrete members, \( V_c \) may need to be reduced in plastic hinge regions where reversed inelastic deformations may deteriorate the concrete in the core. However, for steel jacketed columns, a value for \( V_c \) equal to 2.0 \( b_w d \) seems appropriate because the steel jacket prevents deterioration of the concrete. In Eq. (3), the effects of axial load on the shear resistance of the column are neglected. Axial loads were not varied in this study.

For columns with inadequate shear strength, the jacket should extend over the full length of the column. As was done with the test specimens, it is recommended that a gap seems appropriate because the steel jacket prevents deterioration of the concrete. In Eq. (3), the effects of axial load on the shear resistance of the column are neglected. Axial loads were not varied in this study. Axial loads were not varied in this study.

For columns with inadequate shear strength, the jacket should extend over the full length of the column. As was done with the test specimens, it is recommended that a gap of about 1 in. be left between the end of the jacket and the end of the column to prevent bearing of the jacket against an adjoining beam or footing. Bearing at the end of the jacket may damage the jacket or may reduce an unwanted increase in flexural capacity. It is also assumed that welded or bolted connections at the corners of the steel jacket are adequate to develop the force in the ties provided by the jacket.

Conclusions

Results of an experimental research program on the use of rectangular steel jackets for strengthening rectangular reinforced concrete columns with inadequate shear resistance were presented. The tests conducted on large scale columns showed that such columns can be very effectively retrofitted with thin rectangular steel jackets. Up to 36-in.-wide columns having a shear span-to-depth ratio of 1.33 were successfully strengthened with 1/4-in.-thick-steel jackets. Using the test
results and a simple analytical model, design recommendations for rectangular steel jackets were developed.

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CONVERSION FACTORS

1 in. = 25.4 mm
1 ksi = 6.895 MPa
1 kip = 4.448 kN

NOTATION

\[ A_{st} \quad \text{area of the assumed square tie, in.}^2 \]

\[ A_{tr} \quad \text{area of transverse reinforcement, in.}^2 \]

\[ b \quad \text{width of column, in.} \]

\[ d_{tsj} \quad \text{total depth of steel jacket, in.} \]

\[ f_c \quad \text{concrete compressive strength at day of testing, psi} \]

\[ f_{c}^\prime \quad \text{uniaxial compressive strength of 6.0-in.-diameter concrete cylinder at 28 days, psi} \]

\[ f_{sj} \quad \text{effective stress in steel jacket, ksi} \]

\[ F_{sj} \quad \text{nominal shear strength provided by concrete, kips, equal to one-quarter of actual yielding stress of steel jacket, ksi} \]

\[ F_{tr} \quad \text{nominal shear strength provided by transverse ties, kips, equal to one-quarter of actual yielding stress of steel jacket, ksi} \]

\[ s_{tsj} \quad \text{spacing between square ties, equal to the thickness of column, in.} \]

\[ 2.0 \sqrt{f_{cy} / b_d} \quad \text{yield strength of steel jacket, ksi} \]

\[ V_{n} \quad \text{nominal shear strength at section, kips} \]

\[ V_{st} \quad \text{nominal shear strength provided by steel jacket, kips, equal to a conservative recommended value is} \]

\[ V_{tr} \quad \text{nominal shear strength provided by transverse ties, kips, equal to one-quarter of actual yielding stress of steel jacket, ksi} \]

REFERENCES


