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STEEL JACKETS FOR SEISMIC STRENGTHENING OF CONCRETE COLUMNS

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ABSTRACT

Inadequate shear strength and inadequate lap splices in the longitudinal reinforcement are two major deficiencies in old existing reinforced concrete building columns in the United States. In this paper, results of an experimental study on the use of rectangular steel jackets for seismic strengthening of rectangular reinforced concrete columns are presented. Four large-scale columns with inadequate lap splices and four large-scale columns with inadequate shear strength were tested under lateral cyclic loading. The basic unretrofitted columns exhibited non-ductile cyclic response. However, columns retrofitted with rectangular steel jackets exhibited ductile response, higher strength, and improved ductility and energy dissipation.

KEYWORDS

Seismic retrofit, rehabilitation, lap splice, existing structures, steel jackets, anchor bolts, earthquake resistance, shear strength, transverse reinforcement, reinforced concrete.

INTRODUCTION

Existing reinforced concrete buildings constructed in the United States in the 60's and earlier lack adequate seismic resistance. Two major deficiencies limit the seismic resistance of older reinforced concrete columns: inadequate flexural ductility due to short poorly confined lap splices in the longitudinal reinforcement, and inadequate shear strength due to lack of sufficient well detailed transverse reinforcement.

Older building columns were designed primarily for gravity loads. Consequently, they were detailed as compression members which resulted in lap splices in the longitudinal reinforcement bars proportioned as compression splices. However, during an earthquake, column longitudinal bars may experience high tensile forces, which requires a longer, well confined lap splice to develop yield strength of the spliced bars. Column longitudinal bars typically were spliced just above the floor level, a region of potential plastic hinging during an earthquake. Besides being short and located in a critical region, the lap splices were not well confined by transverse reinforcement. As a result the lateral strength and ductility of older reinforced concrete columns generally are not adequate. In the 1971 edition of the ACI 318 code (ACI, 1971), special seismic provisions for reinforced concrete buildings were first introduced. The current seismic provisions of the ACI 318-95 building code (ACI, 1995) allow the use of lap splices in the longitudinal reinforcement but, they must be located near the midheight of the column away from the potential hinge regions.

Older existing short reinforced concrete columns may experience brittle shear failure under large cyclic deformations. Shear failures occur due to a lack of sufficient well detailed transverse reinforcement. Transverse cross ties were usually anchored with 90 degree hooks, which tend to open after the concrete cover spalls off under large cyclic loading. The current provisions of ACI 318-95 ensure adequate shear cover spalls off under large cyclic loading. The current provisions of ACI 318-95 ensure adequate shear strength and ductility by limiting tie spacing in columns to the smaller of one quarter of the minimum column dimension or four inches. In addition, the member is designed for shear forces associated with the development of the flexural capacity at the ends of the member based on tensile stresses in the longitudinal bars equal to 1.25 the specified yield strength of the bars.

Existing rectangular concrete columns with inadequate lap splices or inadequate shear strength can be retrofitted by the use of rectangular steel jackets. In this paper, experimental results of eight large scale columns are presented. Four columns had inadequate lap splices, and are referred to as "flexural columns". Four columns had inadequate shear strength, and are referred to as "shear columns". In one phase, one flexural column was tested as a basic unretrofitted column, and the remaining three were tested after being strengthened with steel jackets and adhesive anchor bolts. In the other phase, two columns were tested as basic unretrofitted shear columns and two were tested after being strengthened with steel jackets. Two shear columns were loaded in the weak direction and two in the strong direction.

EXPERIMENTAL PROGRAM

Figure 1 shows the test setup. The test column is a cantilever type specimen, which represents the portion of a column between the floor and the point of inflection in a building frame. The bottom of the column framed into a large footing prestressed to the laboratory floor. All specimens were laterally loaded at the tip of the column (two complete cycles at every load/drift ratio level,) but without axial load. For the flexural columns, the lateral loads were increased in 22 kN (5 Kip) increments until significant inelastic displacement was observed. However, for the shear columns, the lateral loads were increased in 44 kN (10 kip) increments. When larger displacements were reached, all columns were loaded in 0.5% drift ratio increments.

Flexural columns

The flexural columns were designated FC1 to FC4. All were loaded in the weak direction. Unretrofitted flexural column FC1 represented a reference test. Figure 2 shows the details of column FC1. All longitudinal bars were spliced at the bottom of the column. The splice length was 610mm (24"), corresponding to 24 bar diameters. The column was detailed according to ACI 318-63 provisions (ACI, 1963), which allows the use of a cross tie at every other bar if spacing between the main bars is less than 150mm (6.0").

Retrofitted flexural columns were identical to the basic unretrofitted flexural column FC1, but strengthened with steel jackets before testing. Figure 3 shows the details of column FC2, which was strengthened with a 6mm (1/4") thick steel jacket. The jacket extended over the bottom 725mm (28.50") of the column height, corresponding to 1.2 times the splice length. Since the concrete column cross section was symmetrical about the weak axis, two different patterns of anchor bolts were installed on the opposite large faces of the steel jacket. In this specimen, one side of the steel jacket was stiffened with two vertical lines of four adhesive anchor bolts each and the opposite side was stiffened with two vertical lines of three adhesive anchor bolts each. Figure 4 shows the details of column FC3, which was strengthened with a longer steel jacket. The jacket extended over the bottom 900mm (36") of the column height, which corresponds to 1.5 times the splice length. Only one side of the column jacket was stiffened with anchor bolts. On that side, the steel jacket was stiffened with one vertical line of five anchor bolts. Figure 5 shows the details of column FC4, which was strengthened with a long steel jacket. The jacket height corresponds to 1.5 times the splice length. On one side, the steel jacket was stiffened with two vertical lines of two anchor bolts each. The other side was provided with two vertical lines of three anchor bolts each. In plan, the bolts divided the column width into three 300mm (12") segments.

The steel jackets were prefabricated in two L-shaped panels in plan, as shown in Fig. 6. The free ends of the two L-panels were welded together after being assembled around the column. The 25mm (1.0") gap between the steel jacket and concrete column was filled with commercial non-shrink cementitious grout. The steel jacket was terminated 38mm (1.5") above the top of the footing to avoid any possible bearing of the steel jacket against the footing. All anchor bolts were 25mm (1.0") in diameter and 300mm (12") long. Every bolt was embedded 200mm (8") into the concrete column. Bolts were installed through pre-drilled holes on the steel jacket using an epoxy adhesive. Table 1 shows the properties of the flexural columns.

Table 1 Properties of the flexural columns

Column #	Concrete fc* Mpa (psi)	Grout fc* Mpa (psi)	Height of jacket mm (in)	Bolts east side	Bolts west side	Description
FC - 1	19.7 (2850)	-	-	-	-	basic
FC - 2	19.7 (2850)	41.7 (6045)	685 (27)	2L4B	2L3B	strengthened
FC - 3	20.0 (2905)	36 (5220)	875 (34.5)	1L5B	none	strengthened
FC - 4	22.5 (3265)	43.2 (6260)	875 (34.5)	2L3B	2L2B	strengthened

* Strength at the day of testing. L= Vertical Lines, B= Adhesive Anchor Bolts. 2L3B indicates 2 vertical lines of bolts, with 3 bolts in each line.

Shear columns

The shear columns were designated SC1 to SC4. The basic unretrofitted shear columns SC1 and SC3 (Fig. 7) were reference tests, loaded in the weak and in the strong directions, respectively. The height of the column was only 1220 mm (48"), to develop a shear/moment action. The remaining two columns were strengthened with steel jackets before testing. Table 2 shows the properties of the shear columns.

Columns SC2 and SC4 were strengthened with 6mm (1/4") thick steel jackets. The steel jacket was provided over almost the full height of the column, and was terminated 25mm (1.0") above the top of the footing to avoid any possible bearing of the steel jacket against the footing. Figure 8 shows the details of retrofitted columns SC2 and SC4. The steel jacket was similar to the flexural column steel jackets, but was not stiffened with adhesive anchor bolts. Columns SC2 and SC4 were loaded in the weak and the strong directions respectively, to examine the effectiveness of the steel jacket in strengthening rectangular short columns in the two major directions.

Table 2. Properties of the shear columns

Column #	Concrete fc* MPa (psi)	Grout fc* MPa (psi)	Type of steel jacket	Direction of loading	Description
SC - 1	21.9 (3170)	-	-	weak	basic
SC - 2	15.6 (2255)	40.8 (5910)	full height	weak	strengthened
SC - 3	16.0 (2325)	-	-	strong	basic
SC - 4	16.5 (2390)	45.1 (6540)	full height	strong	strengthened

* Strength at the day of testing.

TEST RESULTS

Flexural columns

The basic unretrofitted flexural column FC1 exhibited non-ductile flexural response. During the test, the first vertical splitting cracks developed over the bottom half of the spliced bars at a load of 154 kN (35 kips). Increased loading to 176 kN (40 kips) caused splice failure, which was associated with vertical splitting cracks over the full length of the splice. The maximum measured strain on the longitudinal bars was just below the actual yielding strain of the bars. Figure 9(a) shows the hysteretic response of column FC1, which had no ductility and very limited energy dissipation.

Figure 9(b) shows the hysteretic response of column FC2, which was strengthened with a short steel jacket. It reached higher strength than the basic unretrofitted column FC1, but with significant loss of strength as deformations increased. Figure 9(c) shows the hysteretic response of column FC3. On the side without anchor bolts, column FC3 developed its yielding capacity at 1.5% drift ratio. Afterwards, the specimen, on that side, lost its strength and stiffness very rapidly. However, on the other side, with anchor bolts, the column developed its yielding strength and maintained it to more than 3.0% drift ratio. Afterwards, the column showed gradual degradation of strength and stiffness. The better performance of the side with anchor bolts is very clear. It appears that the presence of the anchor bolts forces the steel jacket to deform with the concrete column and confine the splice. Figure 9(d) shows the hysteretic response of column FC4, which was strengthened with a longer steel jacket. Both sides of column FC3 developed the member's flexural capacity and maintained it to 4.5% drift ratio. Test results of this column indicated that rectangular steel jackets with adhesive anchor bolts can improve the performance of wide rectangular reinforced concrete columns with inadequate lap splices. Also, it suggests that anchoring the steel jacket by anchor bolts at its upper and lower ends is effective in improving the confinement of the lap splice using the steel jacket. Comparing the behavior of FC2 with FC3 indicates that extending the length of the jacket well beyond the top of the splice significantly improved performance. Also, comparing the behavior of FC3 with FC4 indicates that distribution of anchor bolts has substantial influence on the response of the retrofitted columns. Based on these tests, it appears desirable to use a jacket height on the order of 1.5 times the length of the lap splice. Compared to the basic column FC1, column FC4 exhibited higher strength, and much higher ductility and energy dissipation.

Shear columns

Columns loaded in the weak direction. The response of the basic unretrofitted shear column SC1 was dominated by shear. Flexural response was noted up to a load of 267 kN (60 kips), which corresponds to a nominal concrete shear stress of about 0.17 $\sqrt{f_c'}$ (2 $\sqrt{f_c'}$) . During cycles to 311 kN (70 kips), flexural cracks extended diagonally, forming flexural shear cracks. During cycles to 400 kN (90kips), major diagonal shear cracks extended over the full height of the column as the cross ties yielded. Afterwards, the column showed large inelastic deformations. Figure 9(e) shows the hysteretic response of the shear column SC1. Column SC1 showed dramatic loss in strength and stiffness at displacements that correspond with drift ratios larger than 2.0% .

The retrofitted column SC2 showed excellent performance as indicated in Fig. 9(f). Compared to the basic column SC1, the hysteretic response of column SC2 showed a large increase in strength, even though it had 30 % lower concrete strength. Column SC2 exhibited very stable hysteresis loops, with very large ductility and high energy dissipation. The steel jacket remained elastic throughout the test. The maximum measured strain was below 300 micro-strain, approximately 1/6 the actual yielding strain of the steel jacket. After testing was completed, the steel jacket was removed. The concrete cover in the compression zones showed significant deterioration. Since, it was well confined by the steel jacket and the grout it stayed intact and developed higher useful strains.

Columns loaded in the strong direction. Column SC3 was tested as a basic unretrofitted shear column loaded in the strong direction. The span-to-depth ratio was 1.34. Diagonal cracks formed over the full height of the column during the cycles to 490 kN (110 kips), but the response of the specimen remained essentially elastic until a load of 645 kN (145 kips) was reached. At this load column SC3 lost strength and stiffness very rapidly. The peak load of column SC3 was well below that required to develop its theoretical flexural yielding strength, which corresponds to a lateral load of 935 kN (210 kips). Figure 9(g) shows the hysteretic response of column SC3 which exhibited essentially no ductility and very limited energy dissipation.

Retrofitted column SC4 exhibited excellent hysteretic response as shown in Fig. 9(h). Column SC4 developed its yielding and ultimate flexural strengths. Fracture of one of the longitudinal bars was observed at a 4% drift ratio. Investigation of the concrete column after the test revealed a large number of narrow diagonal shear cracks in the specimen. Deterioration of the concrete compression zones was evident but did not lead to failure because the concrete was confined by the steel jacket. The steel jacket remained elastic. The maximum measured diagonal strain on the steel jacket was about 25% of the actual yielding strain of the steel jacket. Column SC4 exhibited stable hysteretic loops, large ductility and good energy dissipation. Additional details on the performance of the reported columns can be found elsewhere (Aboutaha, 1994).

SUMMARY AND CONCLUSION

Eight large scale columns with either inadequate lap splices in the longitudinal reinforcing bars or inadequate shear strength were investigated before and after strengthening with steel jackets. Test results of the flexural columns with inadequate lap splices show that rectangular steel jackets with adhesive anchor bolts significantly improved the seismic response of columns with inadequate lap splices. The best results were achieved when at least two adhesive anchor bolts at the top and at the bottom of the steel jacket were used and when the height of the jacket was 1.5 times the splice length. Columns with inadequate shear strength developed their flexural capacities and sustained large cyclic deformation levels when strengthened with steel jackets.

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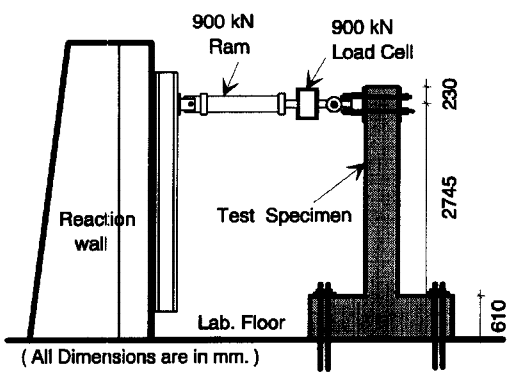


Fig. 1 Test Setup

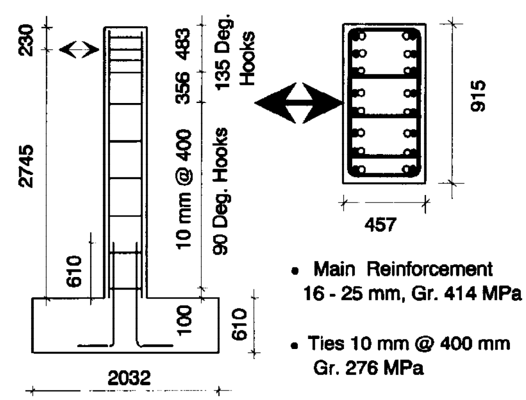


Fig. 2 Basic Column FC1

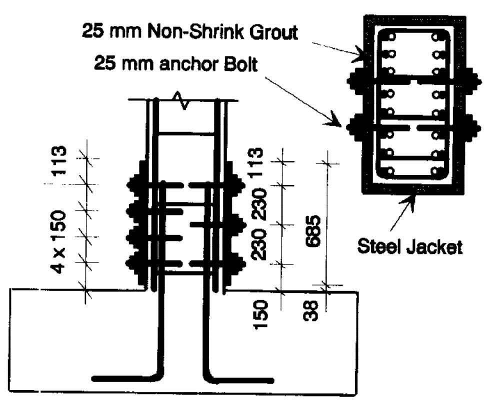


Fig. 3 Strengthened Column FC 2

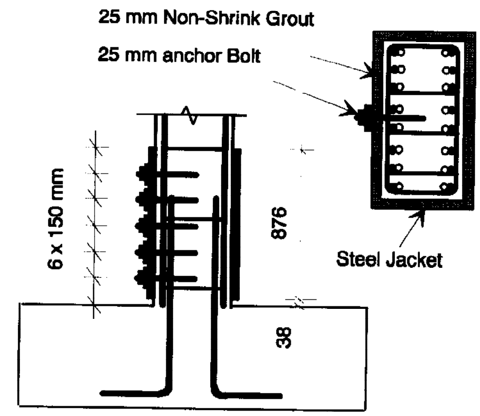


Fig. 4 Strengthened Column FC 3

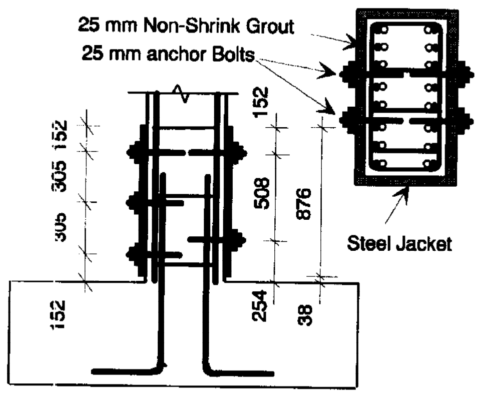


Fig. 5 Strengthened Column FC 4

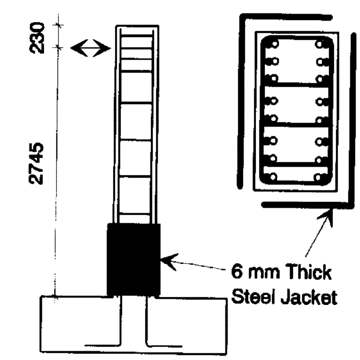


Fig. 6 Typical Strengthened Column

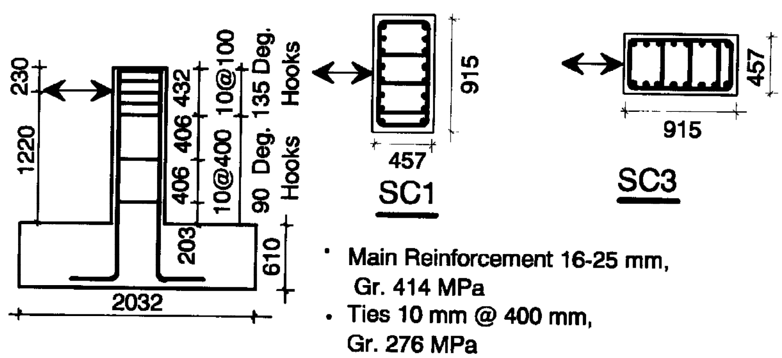


Figure 7 Basic shear columns SC1& SC3

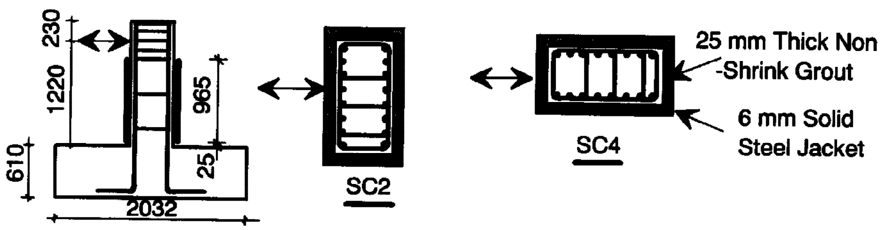
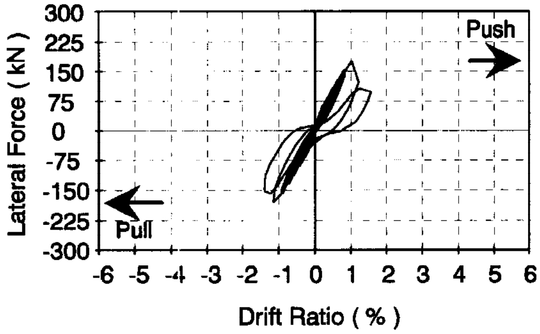
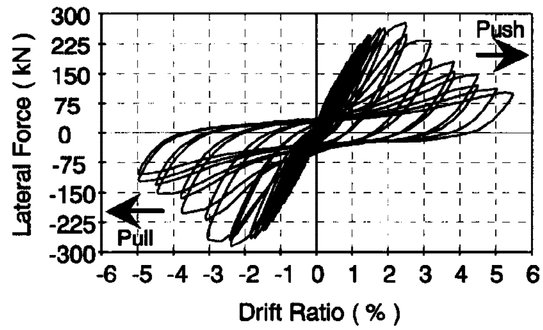


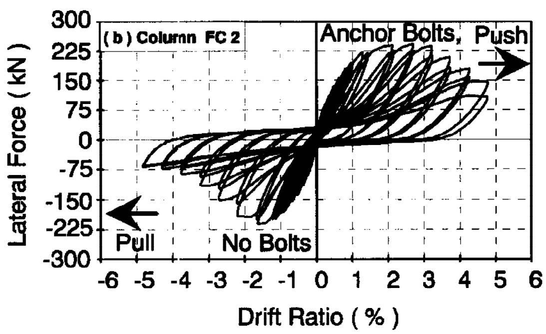
Figure 8 Details of the strengthened shear columns SC2 & SC4



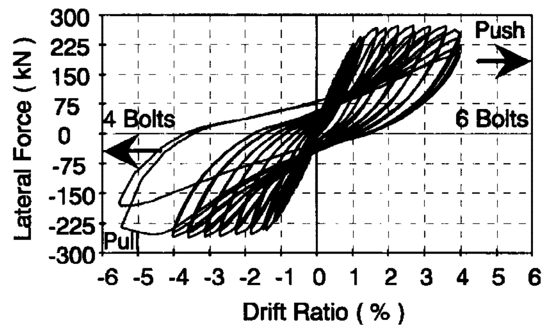
(a) Basic flexural column FC1



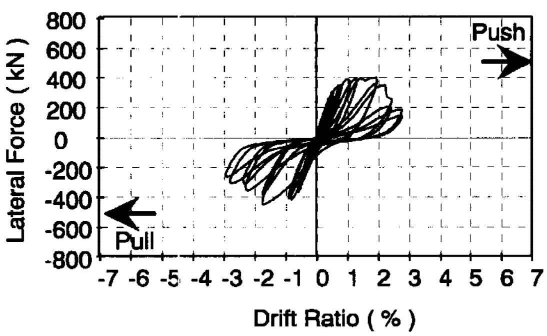
(b) Strengthened flexural column FC2



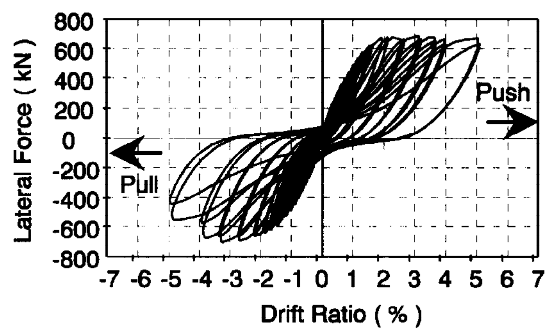
(c) Strengthened flexural column FC3



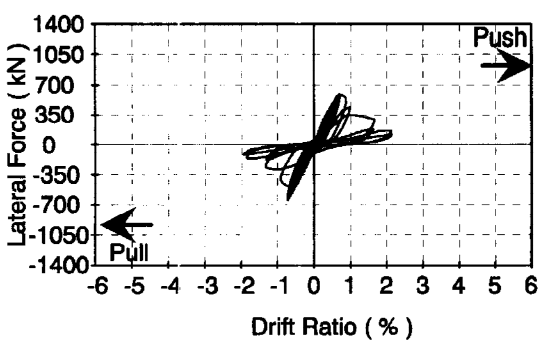
(d) Strengthened flexural column FC4



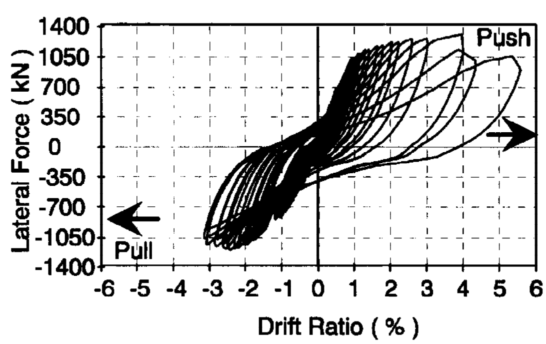
(e) Basic shear column SC1



(f) Strengthened shear column SC2



(g) Basic shear column SC3



(h) Strengthened shear column SC4

Figure 9 (a - h) Hysteretic response of the test columns