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Seismic retrofit of bridge columns using steel jackets

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ABSTRACT: Bridge failures in the 1971 San Fernando earthquake, the 1987 Whittier earthquake, and the 1989 Loma Prieta earthquake have drawn much attention to the consequences of substandard detailing in bridges constructed before 1970. Some of the problem areas associated with concrete bridge columns are (i) inadequate and undependable flexural strength as a result of low seismic design coefficients and the common practice of lapping flexural reinforcement within the potential plastic hinge region, (ii) inadequate flexural ductility, as a result of insufficient and poorly detailed transverse reinforcement, and (iii) inadequate shear strength, again as a result of insufficient and poorly detailed transverse reinforcement, and the use of elastic design methods which provided lower shear strength than flexural strength in many designs. The research described in this paper was directed towards developing an inexpensive and effective retrofit methods to rectify these deficiencies in many of the older bridges in California.

1 INTRODUCTION

Recent earthquakes in California (see Fung et al (1971), Priestley (1988), and Lew (1990)) have provided graphic evidence of the potential hazard that exist in many of the older bridge structures in the United States. Considerable progress has made by the California Department of Transportation (Caltrans) in implementing retrofit measures to upgrade the seismic resistance of these bridges in California (see Zelinski (1990)). Deficiencies inherent in the columns of the older bridges are inadequate flexural strength and ductility, undependable flexural capacity, inadequate shear strength, insufficient joint and footing strength. These problems have been discussed elsewhere (see Priestley and Seible (1991)).

Current approaches for seismic design of bridge relies on column ductility provided by proper confinement of the potential plastic hinge regions by closely-spaced transverse hoops or spirals. Such provision allows the ultimate compressive strain to increase from a value of about 0.005 in unconfined concrete to a value of 0.03 or higher in confined concrete. The increase in ultimate compressive strain significantly enhances the ductility capacity of the concrete section. Research results by Priestley and Park (1987) have shown that well confined columns can develop stable hysteresis loops during inelastic cycling to displacement ductility factors exceeding six.

Effective confinement can be provided to existing substandard circular columns by encasing the potential plastic hinge regions with a site-welded cylindrical steel sleeve or jacket. The jacket is introduced slightly oversize for ease of construction, and the gap between the jacket and column is subsequently filled with a cement-based grout. The increase in ultimate compressive strain as a result of confinement from the steel jacket may be estimated from the equation suggested by Priestley and Seible (1991):

$$\epsilon_{cu} = 0.004 + 1.4\frac{f_{yj}}{f'_{cc}}\epsilon_{su}\rho_{sj} \qquad (1)$$

where ϵ_{cu} = ultimate compressive strain of concrete, f_{yj} = yield strength of the steel jacket, f'_{cc} = compressive strength of confined concrete, ϵ_{su} = ultimate tensile strain of the steel jacket. The term ρ_{sj} = confining ratio of the steel jacket, and is given by $4t_j/(D_j - 2t_j)$ where D_j and t_j are the outside diameter and thickness of the steel jacket, respectively. It should be noted that Eqn. 1 was derived from the energy balance method suggested by Mander et al (1988).

The use of a close-fitted steel jacket to a rectangular column, however, would not be effective in enhancing the flexural behavior, since the out-of-plane flexibility of the rectangular jacket cannot provide effective confinement to the column except at the jacket corners. However, by encasing the rectangular column with an elliptical jacket and placing concrete between the jacket and the existing concrete, continuous confinement can be achieved in both principal directions of the column.

The failure to provide adequate transverse steel, as with the pre-1971 design, may lead to brittle shear failure in short columns, with very limited displacement ductilities. Inelastic cyclic response of these columns is characterized by poor energy dissipation, rapid strength, stiffness and physical degradation. The use of steel jacket would, in addition to provide confinement, enhance the shear strength of the column. A satisfactory shear retrofit would be to increase the shear strength of the column to a level above the flexural strength to avoid brittle

shear failure so that the column after retrofit can exhibit ductile flexural response. For circular columns, a cylindrical jacket is appropriate, while for rectangular columns, an elliptical jacket is desirable. The contribution to column shear strength by steel jackets, based on 45^0 truss analogy, has been proposed by Priestley and Seible (1991) as:

For circular jacket:

$$V_{sj} = \frac{\pi}{2} t_j f_{yj} D_j \tag{2}$$

For elliptical jacket (strong direction):

$$V_{sj} \approx 2 t_j D' f_{yj} (1 - \frac{B'}{D'} + \frac{\pi}{4} \frac{B'}{D'}) \tag{3}$$

where D' and B' = long and short diameters of the elliptical jacket, respectively. The shear strength of the elliptical jacket in the weak direction can be obtained by replacing B' by D', and D' by B' in Eqn. 3.

In all cases, the jacket is terminated slightly short of the connecting member or footing to ensure that the jacket provides only confinement at the critical section, rather than contributing to the flexural strength by bearing against the adjacent member on the compression side. Unexpected moment enhancement from this cause could overload the adjacent member.

In order to verify the effectiveness of the retrofit techniques for enhancing the seismic performance of bridge columns, a series of large-scale circular and rectangular columns were recently tested at the University of California, San Diego. Test columns were constructed at 0.4 geometric scale of typical prototype dimensions, using materials and design details appropriate for columns designed in the 1950's and 1960's.

2 COLUMN FLEXURAL TESTS

2.1 Column Details

The flexural program included testing of circular columns of 24 inches (610 mm) in diameter and rectangular columns of 28.75 inches (730 mm) by 19.25 inches (489 mm) in cross-section. The column height was 144 inches (3658 mm) from the top of footing to the center of horizontal force. An axial force of 400 kips (1779 kN) was applied to the test column using two 2 inch (51 mm) diameter high-strength bars before imposing lateral displacements. Each bar was stressed with a center-hole jack which reacted against the test floor and the bar forces were transferred to the column by a cross-beam mounted on top of the column loadstub. Fig. 1 shows the test configuration for a circular flexural column.

A target concrete compressive strength of f'_c = 5000 psi (34.5 MPa) at 28 days was used in all test columns, and Grade 40 reinforcement (f_y = 40 ksi or 276 MPa) was used in all flexural columns. Longitudinal reinforcement for circular columns consisted of 26 #6 (19 mm diameter) deformed bars, uniformly distributed around the column. A concrete cover of 0.8 inch (20 mm) was provided for the longitudinal reinforcement. Yield strength for the #6 (19 mm diameter) bar av-

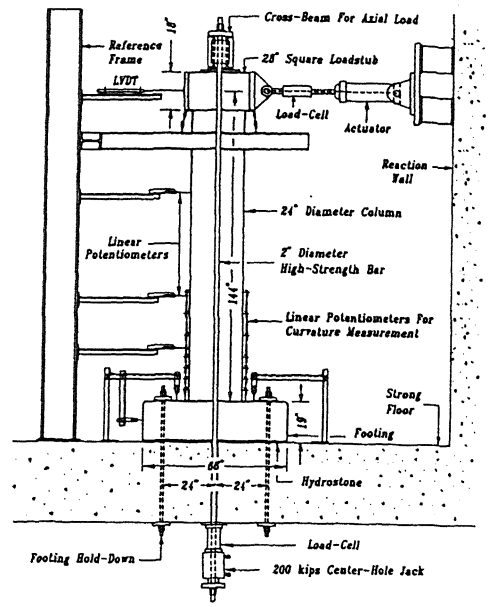


Fig. 1 Flexural Test Setup (1 in = 25.4 mm)

aged 45.7 ksi (315 MPa). Transverse reinforcement was provided by #2 (6.4 mm diameter) circular hoops at 5 inches (127 mm) uniform spacing. Fig. 2 shows the reinforcement details for a circular flexural column.

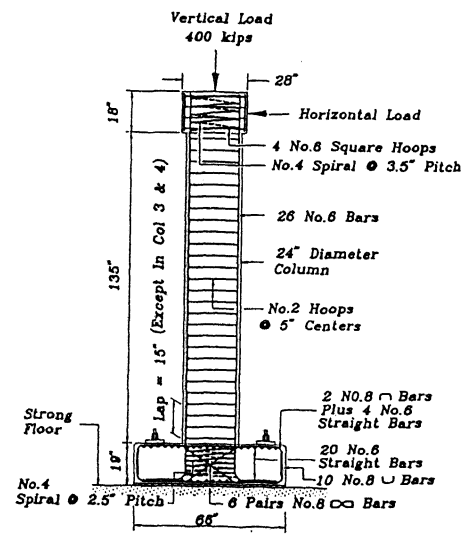


Fig. 2 Details for Circular Flexural Column (1 in = 25.4 mm)

Circular columns were retrofitted with cylindrical jackets fabricated from 3/16 inch (4.8 mm) thick A36 hot-rolled steel (f_y = 36 ksi or 248 MPa). A 1/4 inch (6.4 mm) gap was provided between the jacket and column, and subsequently pressure filled with a cement-based grout which contained a small dose of water-reducing,

expansive additive. To ensure that the jacket did not bear against the footing when in compression, a nominal gap of 1 inch (25.4 mm) was provided between the toe of the jacket and the footing. The length of cylindrical jacket was chosen to be 48 inches (1219 mm) to ensure that the moment demand immediately above the jacket did not exceed 75% of the original flexural capacity. Fig. 3 shows the cross-section of a steel jacketed circular column.

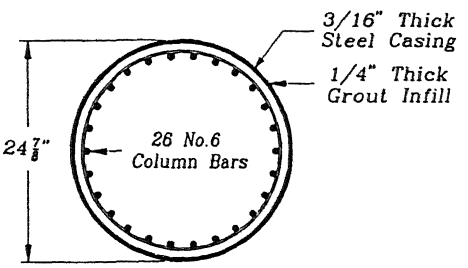
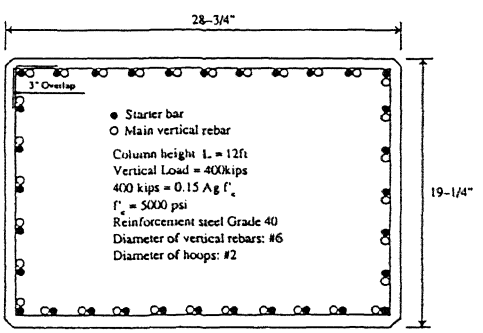
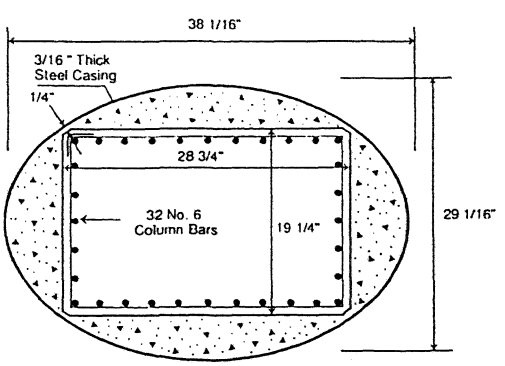


Fig. 3 Steel Jacketed Circular Column (1 in = 25.4 mm)

Two reinforcement contents were investigated for the rectangular columns; namely, 2.53% and 5%. For the rectangular columns with 2.53% steel area ratio, the longitudinal reinforcement consisted of 32 #6 (19 mm diameter) bars distributed into single layer, as shown in Fig. 4(a).



(a) 'As-Built'



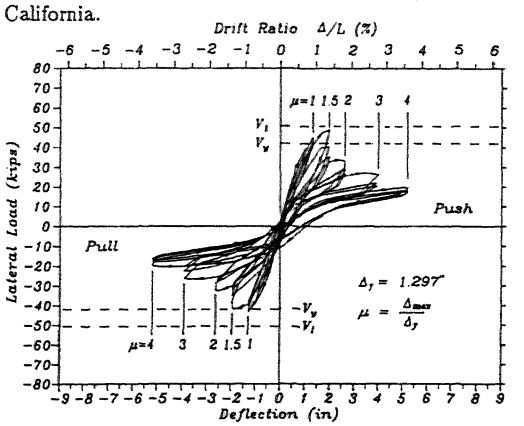
(b) Elliptical Jacket

Fig. 4 Details of Rectangular Columns 1 in = 25.4 mm)

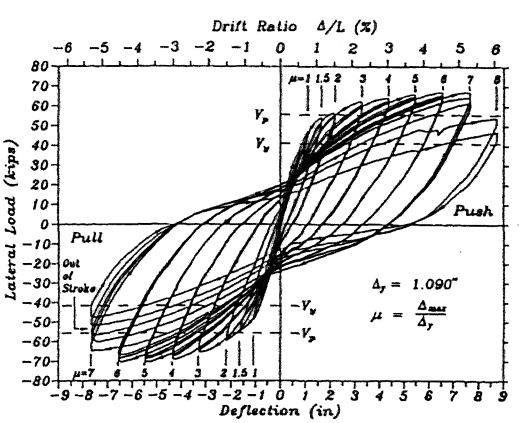
The transverse steel for rectangular columns was similar to that of circular column i.e. #2 (6.4 mm diameter) perimeter hoops at uniform spacing of 5 inches (127 mm). Fig. 4(b) shows the retrofit of a rectangular column with an elliptical jacket which had centerline dimensions of 38.06 inches (967 mm) and 29.06 inches (738 mm) in the two principal directions. The elliptical steel jacket was fabricated from 3/16 inch (6.4 mm) thick A36 hot-rolled steel and 48 inches (1219 mm) in height, as was the case for the circular column.

2.2 Response of Circular Flexural Columns

The hysteresis loops for an 'as-built' circular column detailed with a lap-splice of 20 times the longitudinal bar diameter in the potential plastic hinge region is shown in Fig. 5(a) and (b). Column base lap-splices were common for pre-1971 design of moment-resisting column in California.



(a) 'As-Built'



(b) Cylindrical Jacket Retrofit

Fig. 5 Hysteretic Loops of Circular Flexural Columns (1 in = 25.4 mm, 1 kip = 4.448 kN)

In Fig. 5, V_y corresponds to the lateral force at first yield of the extreme tension reinforcement, V_i corresponds to the theoretical ideal flexural capacity of the unconfined column section, and V_p is equivalent to V_i

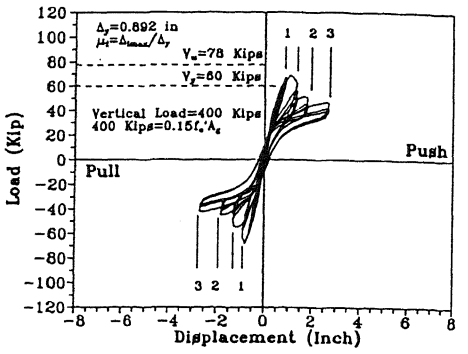
but includes the effect of confinement provided by the steel jacket for the retrofitted column and was assessed at an extreme compressive strain of 0.005 in the concrete core. As can be seen from Fig. 5(a), the response of the 'as-built' column with lapped starter bars was characterized by severely pinched hysteresis loops with a rapid degradation of strength after the first cycle to $\mu = 1.5$. A maximum lateral force of 49 kips (218 kN) was noted during the push cycle to $\mu = 1.5$ and was 97% of the theoretical ideal capacity V_i. The strength envelope was seen to degrade asymptotically after $\mu = 1.5$ to the moment resisted purely by the axial load which corresponded to a shear force of 19 kips (84.5 kN).

In contrast, the circular column retrofitted with a steel jacket exhibited a much improved performance, as shown in Fig. 5(b). The hysteresis loops were remarkably stable and demonstrated high energy absorption up to $\mu = 7$, or corresponding to a drift ratio (displacement divided by height) of 5.3%. Very little degradation of peak lateral force was noted upon displacement cycles to a given ductility level. Peak lateral forces at $\mu \geq 3$ exceed V_p as a result of strain-hardening in the longitudinal reinforcement. Low-cycle fatigue fracture of longitudinal reinforcement occurred during the first cycle to $\mu = 8$, and was accompanied by comparatively rapid strength degradation, although good energy absorption capacity was maintained.

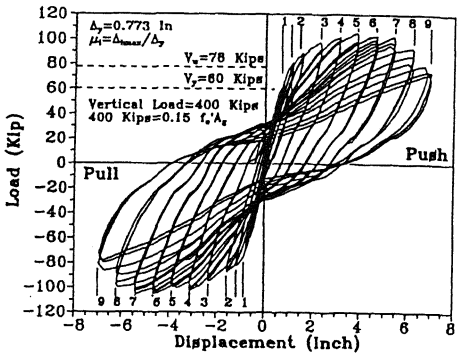
2.3 Response of Rectangular Flexural Columns

The hysteretic loops for rectangular columns loaded in the strong direction are shown in Fig. 6(a) and (b). These columns were provided with lap-splices of 20 times the longitudinal bar diameter. In Fig. 6, V_y is the lateral force at first yield of the extreme tension steel, and V_u is the ideal capacity calculated using the ACI equivalent stress block. The response of 'as-built' rectangular column was very similar to that of circular column, with a bond failure at the lap-splices of the main reinforcement. The ideal capacity of the column in the strong direction was estimated to be 78 kips (347 kN), but could not be reached by the column (see Fig. 6(a)). Bond failure occurred prior to reaching $\mu = 1.5$, and subsequent response was characterized by rapid degradation of strength with very narrow energy loops.

With an elliptical jacket, the rectangular column showed a substantial improvement in the load-deflection response as can be seen in Fig. 6(b). The ideal capacity of the original column was exceeded at $\mu = 1.5$. A significant increase in lateral load was subsequently noted. The maximum load recorded was 106 kips (471 kN). Unlike the circular columns with cylindrical jackets which failed by low-cycle fatigue fracture of the longitudinal reinforcement, the final failure of rectangular column was caused by bond failure at the lap-splice of the main reinforcement. Strength degradation due to relative sliding between the main reinforcement and starter bars was minor and was not significant until after $\mu = 7$. The presence of the elliptical jacket restrained the spalling of cover concrete, and therefore allowed a more grad-



(a) 'As-Built'



(b) Elliptical Jacket

Fig. 6 Hysteretic Loops of Rectangular Flexural Columns (1 in = 25.4 mm, 1 kip = 4.448 kN)

ual degradation of strength. Hysteretic loops showed a rather impressive dissipation of energy by the column.

3 COLUMN SHEAR TESTS

3.1 Shear Column Details

The shear retrofit program involved testing of circular columns 24 inches (610 mm) diameter and rectangular columns of 24 inches × 16 inches (610 mm × 406 mm) section.

The test setup used for the shear columns is shown in Fig. 7. The lateral load mechanism consisted of displacing the column in double-curvature with the point of inflection occurring at mid-height of the column. A stiff loading arm connected the top of the column to a horizontal double-acting actuator located at column mid-height. Loadstub rotation was minimized by a load-balancing system which was designed to compensate for the weight of the loading arm. The axial load was applied to the column using high-strength flexible rods, as for flexural columns.

Retrofit of circular and rectangular columns involved encasing to almost the full height of the column with a 3/16 inch (6.4 mm) or 1/8 inch (3.2 mm) thick cylindrical jackets. Elliptical jackets of 32 inches (813 mm) by 24 inches (610 mm) outside dimensions in the two prin-

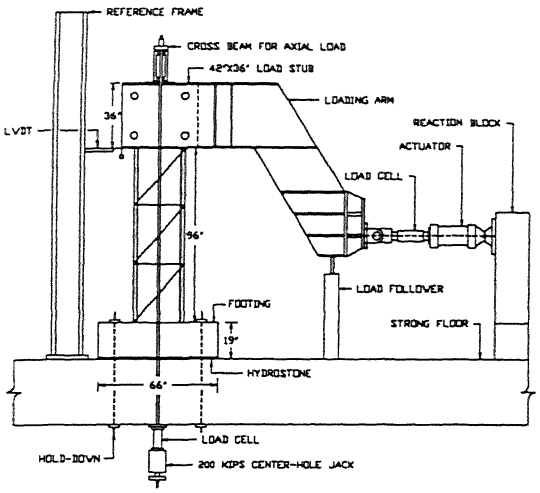


Fig. 7 Shear Test Setup (1 in = 25.4 mm)

cipal axes were used for retrofit of rectangular columns. Continuous longitudinal reinforcement were used in all shear columns i.e. without lap-splice in the potential plastic hinge regions. Fig. 8 shows the reinforcement details for a circular shear column.

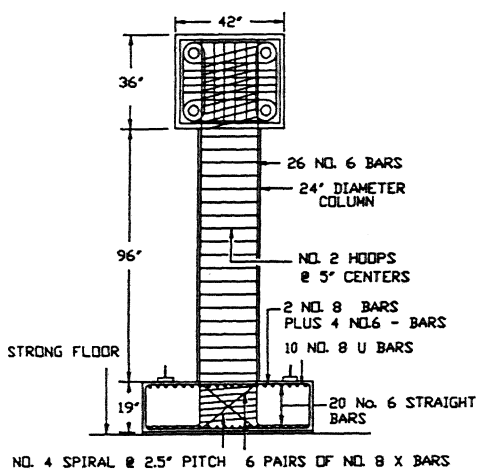
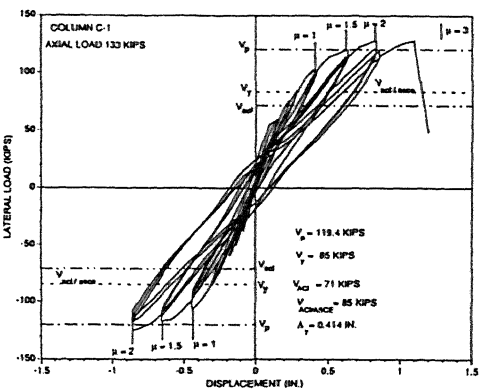


Fig. 8 Details for Circular Shear Column (1 in = 25.4 mm)

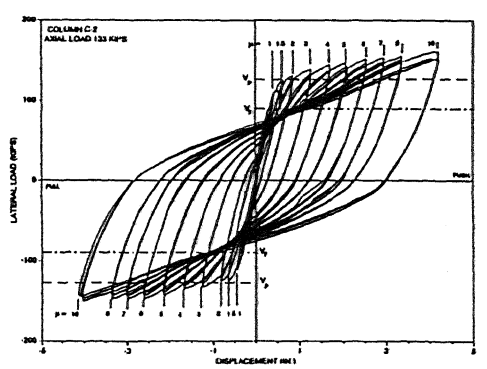
3.2 Response of Circular Shear Columns

Typical lateral force-deflection hysteresis loops for the circular shear columns are shown in Fig. 9(a) and (b). The lateral forces corresponding to the theoretical ultimate flexural strength of the column, V_p, and at first yield of extreme tension reinforcement, V_y, calculated using the Mander model for confined concrete (see Mander et al (1988)), are shown by dashed lines in these figures. In addition, the theoretical shear capacity predicted by the ACI (see ACI Code 318 (1989)) is included as a linked line.

Flexural crack patterns developed in the 'as-built' column exhibited strong shear influence, with the angle between inclined shear cracking and the column axis being less than 25^0. The 'as-built' circular shear column exhibited relatively stable response up to displacement ductility factor of $\mu = 2$ (see Fig. 9(a)). Although the theoretical flexural capacity, V_p= 119.4 kips (531 kN) was reached at $\mu = 1.5$, the column failed in brittle shear during the first cycle to $\mu = 3$. The maximum load attained was 129 kips (574 kN) which was considerably higher than the ACI (see ACI Code 318 (1989)) shear strength prediction of 71 kips (316 kN). Final failure of the column involved a major diagonal crack initiated from crushing of concrete in the compression zone in the upper region of the column.



(a) 'As-Built'



(b) Cylindrical Jacket

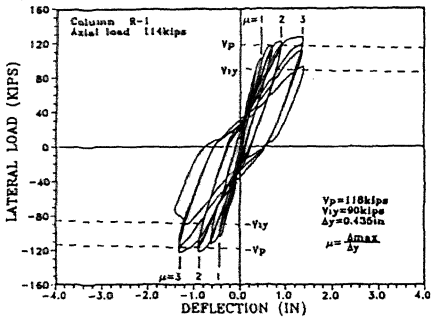
Fig. 9 Hysteretic Loops of Circular Shear Columns (1 in = 25.4 mm, 1 kip = 4.448 kN)

The hysteretic response of circular shear columns after retrofit with a cylindrical jacket showed an impressive increase in displacement ductility and energy absorption (see Fig. 9(b)). Displacement to ductility factors of $\mu = 10$ or drift ratio of 4.37% was possible without serious strength or stiffness degradation. The shear force corresponding to theoretical flexural strength, V_p, calculated using the yield strength of the main steel,

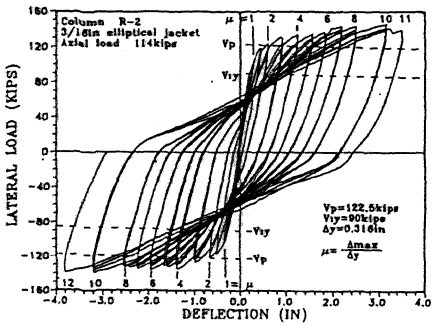
was 127 kips (565 kN). This load was first exceeded at $\mu = 2$, about the same ductility level when strain-hardening was noted to occur in the circular flexural columns. The maximum lateral force recorded was 162 kips (720 kN), occurring at peak displacement in the first push cycle to $\mu = 10$. Testing was discontinued after three cycles to $\mu = 10$ due to displacement limitations.

3.3 Response of Rectangular Shear Columns

The lateral force-deflection hysteretic response for rectangular shear columns loaded in the strong axis direction are shown in Fig. 10(a) and (b). The axial load applied on the columns was 114 kips (507 kN). The link lines shown in these plots correspond to the lateral forces at first yield of the extreme tension reinforcement, V_y, and the theoretical ultimate flexural capacity, V_p, computed using Mander's model for confined concrete (see Mander et al (1988)).



(a) 'As-Built'



(b) Elliptical Jacket

Fig. 10 Hysteretic Loops of Rectangular Shear Columns (1 in = 25.4 mm, 1 kip = 4.448 kN)

Although the theoretical ultimate flexural capacity of $V_p = 118$ kips (525 kN) was achieved at displacement ductility factor $\mu = 1.5$, the degradation of lateral strength became excessive during the three cycles to $\mu = 3$ (see Fig. 10(a)). Considerable spread of bond cracks and inclined shear cracks had developed by this stage resulting in a combined shear/bond failure. Final failure of the column occurred at $\mu = 3$ when concrete crushed in the bottom compression zone and a major diagonal crack propagated through the lower region of the column, destroying the vertical load carrying capacity of the column.

In contrast, the rectangular column retrofitted with an elliptical jacket showed significantly improved hysteretic response, as shown in Fig. 10(b). Ductile plastic hinging developed at both ends of the column, and displacements to a ductility factor exceeding $\mu = 10$ was possible without serious degradation of strength. As with retrofitted circular columns, the rectangular column with an elliptical jacket exhibited high energy absorption capacity.

4 CONCLUSIONS

Cylindrical steel jacketing of the potential plastic hinge region has been shown to enhance the flexural strength and ductility of flexurally dominated circular columns. A dependable drift ratio exceeding 5% was available. For circular columns with small aspect ratio, brittle shear failure was avoided by encasing the full height of the column with a cylindrical steel jacket. Stable and ductile plastic hinging could be developed, providing drift ratio in excess of 4%. Elliptical shaped steel jackets were also shown to be effective in enhancing the flexural and shear performance of rectangular columns.

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