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論 文

[2175] ELLIPTICAL STEEL JACKETING OF RECTANGULAR REINFORCED CONCRETE BRIDGE COLUMNS FOR ENHANCED SHEAR STRENGTH

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1. INTRODUCTION

Several recent moderate earthquakes in California have confirmed that column shear failure is a one of the major danger to most existing reinforced concrete bridges designed prior to 1971[1)2)3)]. This is due to the adopting of the elastic design to permissible stress levels, which typically results in actual flexural strength of columns exceeding their actual shear strength.

In order to develop adequate earthquake retrofit measures for bridge columns designed before 1971, an extensive research program has been carried out at the University of California, San Diego. This paper describes the experimental results from one phase of this program, namely the seismic retrofit of short rectangular columns to enhance shear strength through elliptical steel jackets.

2. RETROFIT DESIGN

In the light of current capacity design for bridges, the retrofit of inadequately designed existing short columns, should be carried out to satisfy the following principles:

i) enhanced shear strength exceeds new flexural strength
ii) a large ductility is assured
iii) unexpected failure mechanism is prevented.

An elliptical steel jacket is proposed for the retrofit of short rectangular columns, which will be site welded to enclose the column with the gap between the jacket and the column filled by concrete. The retrofit stops just short of the critical section to ensure that the jacket provides a confining effect only to the ductility of plastic hinges and the shear strength, rather than an increase in the section size and hence an augmentation of the moment capacity at the critical regions, which might result in a large shear input and overload on the footing.

For the retrofit design, the following methods for the calculations of shear strength and flexural strength are proposed. Although approximate analytical procedures are available for determining flexural ductility of columns confined by transverse reinforcement[4)], it is necessary to obtain experimental evidence for ductility capacity of columns retrofitted by steel jackets.

2.1 SHEAR STRENGTH OF RETROFITTED COLUMNS

Shear strength of an elliptical jacketing column can be simply

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considered as the summation of the shear carried by concrete shear resistant mechanisms and those carried by the original hoops and the retrofit jacket assuming a 45-degree truss mechanism:

$$V = V_c + V_{hs} + V_{sj} \tag{1}$$

The shear carried by concrete can be taken as:

$$V_c = v_c A_g \tag{2}$$

where, Ag is the gross area of the elliptical section, $Ag=\pi D'B'/4$, D',B' are the longer and shorter diameters respectively, and vc is the nominal shear stress carried by concrete. Here, the following equation from Referrence (5) is adopted for the concrete contribution:

$$v_c = 0.37\alpha\sqrt{f_c'}\,[1+3P/(f_c' A_g)] \quad \text{(Mpa)} \tag{3}$$

where, $\alpha := 2VD/M$, P is the axial load, fc' is the average strength (Mpa) of the concrete of original column and that of the concrete in the gap between original column and jacket.

The shear carried by the original hoops, Vsh and the elliptical jacket, Vsj can be expressed as follows:

$$V_{sh} = A_v f_{yh} d/s \tag{4}$$

$$V_{sj} = 2tf_{yj}D'[1-B'/D'+\pi/4(B'/D')] \tag{5}$$

where, Av is the area of the hoops within the pitch of s, fyh and fyj are the yield strength of hoops and steel jacket respectively, t is the thickness of the jacket, d can be taken as the distance between centroids of tension and compression bars. Equation (5) is derived from a numerical integration of the stress carried by the elliptical jacket in the direction of the applied shear force. Putting $B'/D'=1$ and replacing 2t by Av/s, Eq.(5) gives the same expression of the shear carried by circular hoops as Reference(5).

2.2 FLEXURAL STRENGTH OF RETROFITTED COLUMNS

The flexural strength of a column retrofitted by an elliptical jacket depends on the moment capacities of the critical sections at the column ends, where the jacket is only expected to provide an transverse confinement. Mander et al[4] have proposed a sophisticated stress - strain relation for concrete confined by various types of hoop, spiral and tie. The confinement of column section by an elliptical jacket over a unit length can be estimated by using an average confining stress shown in Fig.1. Assuming the jacket is yielded, the average confining stress σr can be calculated as follows:

$$\sigma_r = 2f_{yj}t\cos\beta/B \tag{6}$$

where, t: thickness of the jacket, β: tangential angle of elliptical jacket at the corner of column section (Fig.1), B:section width of the original column.

Thus, from Mander's model, a stress-strain relation for the concrete confined by an elliptical jacket can be obtained. Consequently, the moment capacity of critical section can be calculated.

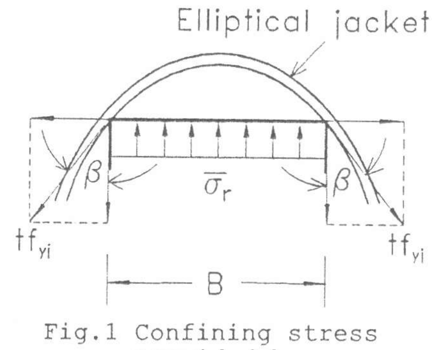
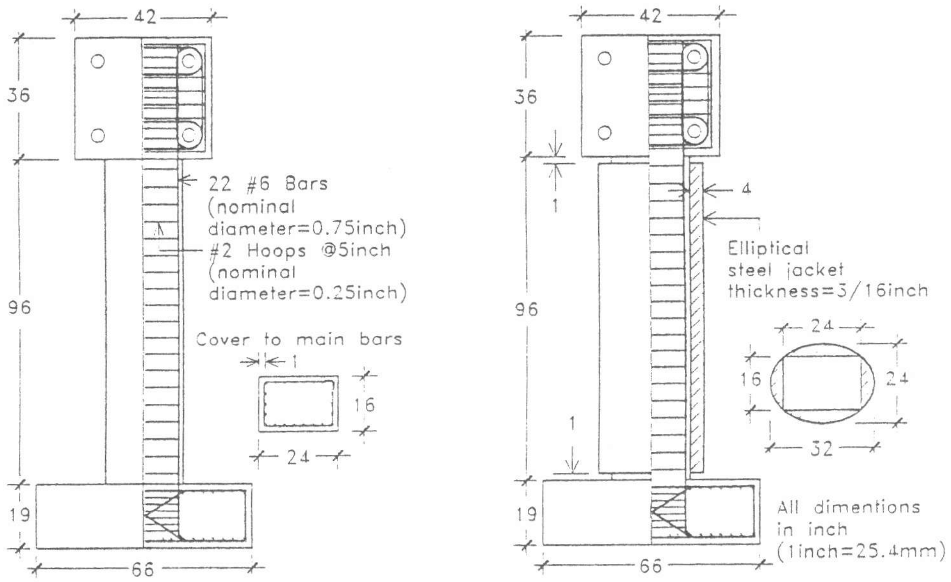


Fig.1 Confining stress provided by elliptical jacket

3. TEST OF "AS-BUILT" AND RETROFITTED COLUMNS

Two columns, as detailed in Fig.2(a),(b), have been tested. Both were designed and constructed according to the design practice in the 1960s. Column R-1 was tested in the condition of "as-built", and R-2 was tested after retrofitting with an elliptical steel jacket with thickness of 3/16inch (4.8mm). The calculated shear strength of column R-2 was 3.8 times of its calculated flexural strength which was derived considering strain hardening of longitudinal reinforcement.

Ready mixed concrete was used throughout. Concrete strength for R-1



(a) "As-built" column R-1 (b) Retrofitted column R-2

Fig.2 Details of the test specimens

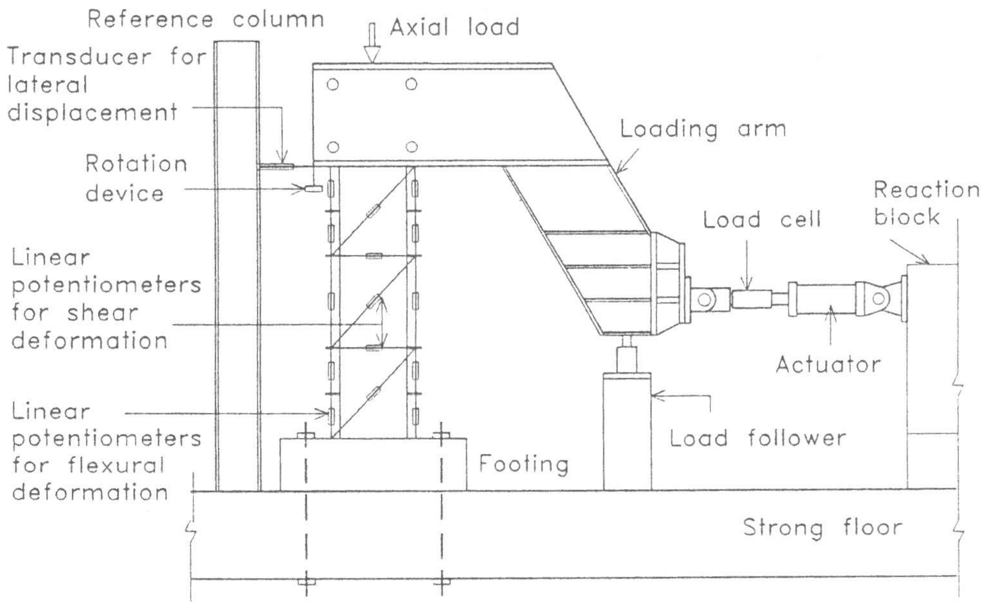


Fig.3 Test set-up

and R-2 was 5.50ksi(37.9MPa) and 5.56ksi(38.3MPa), respectively, and that for the retrofit of R-2 was 4.58ksi(31.6MPa). The yield strength of longitudinal bars (#6) and hoops (#2) was 45ksi(310MPa) and 41ksi(283MPa), respectively. Steel grade of the jacket was A36 with yield strength of 45ksi(310MPa).

As shown in Fig.3, the test set-up is designed to subject the column to axial load and cyclic lateral load acting at its mid height. The measurement gives the information of lateral displacement and rotation of the column top, strain of longitudinal and transverse reinforcements, and the flexural and shear deformation along column height.

Up to the calculated lateral load capacity at first yield point, which is defined as the point when the extreme tensile steel reaches its yield strain, the peak of every loading cycle was controled by load increments. Thereafter, cyclic displacements were increased by multiples of the ductility factor μ, which is the ratio of peak displacement to yield displacement Δy defined as follows:

$$\Delta_y=\Delta_{1y}V_p/V_{1y} \qquad (7)$$

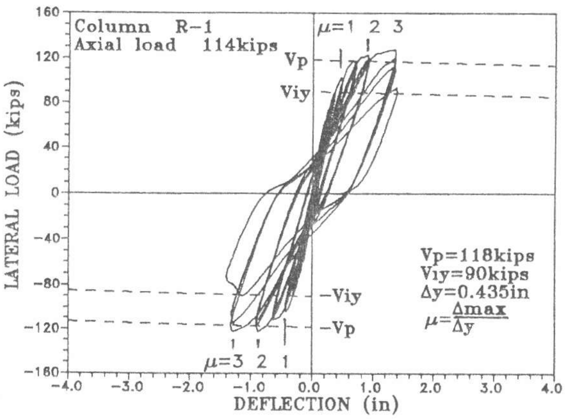
where, V_p: ultimate flexural strength, V_{1y}: lateral load capacity at the first yield point, and Δ_{1y}: measured displacement at the first yield.

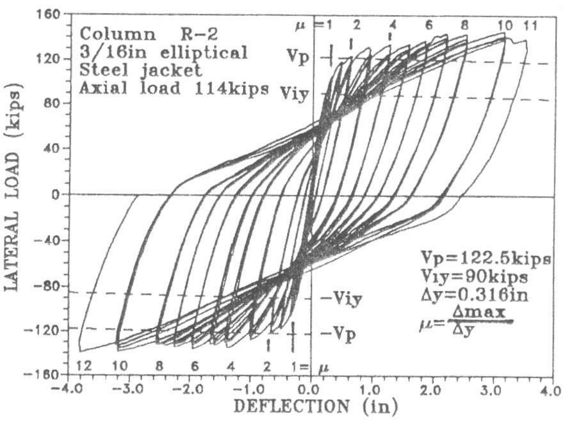
4. LOAD-DEFLECTION RESPONSE

Lateral load - displacement hysteresis responses for column R-1 and R-2 are shown in Fig.4(a), (b). The theoretical ultimate flexural strength V_p and lateral force corresponding to the first yield V_{1p}, are shown by dashed lines in Fig.4.

As shown in Fig.4(a), although the "as-built" column R-1 developed its theoretical ultimate flexural strength at a displacement ductility factor μ=1.5, the degradation of lateral force became significant during the three cycles at μ=3. At this stage, considerable spread of bond cracks and steeply inclined shear cracks were observed. Finally, following the crushing of concrete in the bottom compression zone, a major diagonal crack penetrated through the lower half of the column, and the column lost its load carrying capacity.

The retrofitted column R-2 demonstrated a significantly improved lateral load - displacement hysteresis response, showing impressive

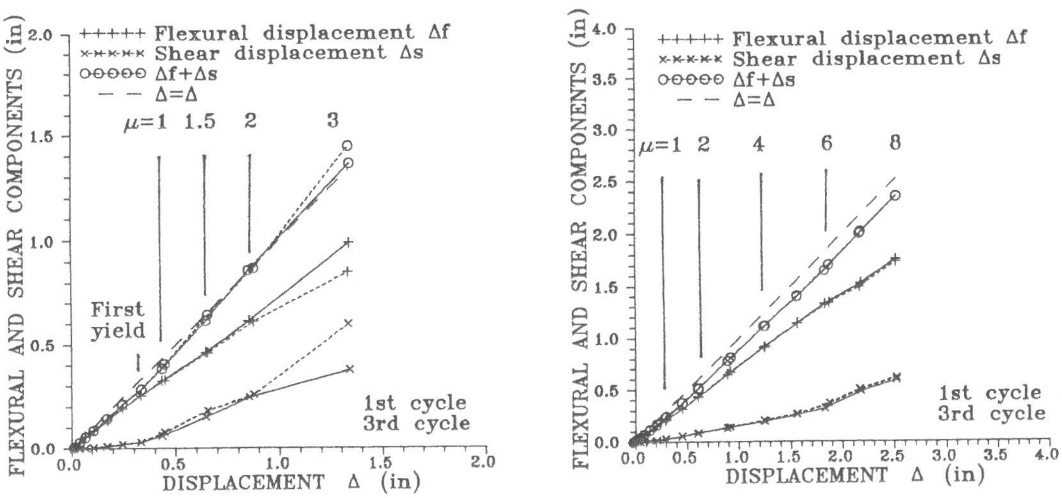




(a) "As-built" column R-1 (b) Retrofitted column R-2

Fig.4 Lateral load - displacement responses

(1kips=4.448kN, 1inch=25.4mm)



(a) "As-built" column R-1 (b) Retrofitted column R-2

Fig.5 Flexural and shear components of drift displacement (1inch=25.4mm)

ductility and energy absorption capacity. The lateral load capacity was very stable up to μ=10 or drift ratio of 3.3% in one direction, and μ=12 or drift ratio of 4.0% in the other. Low cycle fatigue fracture of longitudinal reinforcement occurred during the loading to achieve μ=12 in the first direction and was accompanied by a drop of lateral load capacity. The column exceeded its theoretical ultimate flexural strength Vp at μ=2, the further augmentation in lateral load capacity is due to the strain hardening of longitudinal reinforcement, which was confirmed at about the same stage. Column R-2 developed a maximum lateral load of 145.8kips(649kN), which was very close to the calculated flexural strength of 148kips(658kN) considering strain hardening of longitudinal reinforcement.

5. FLEXURAL AND SHEAR DEFORMATION

The flexural and shear components of drift displacement, Δf and Δs of the columns can be decomposed using the experimental results of the linear potentiometers shown in Fig.3. The components Δf, Δs against total drift displacement, Δ, at peak displacement points for R-1 and R-2 are shown in Fig.5. Solid and dotted lines in Fig.5 describe the results of first cycle and third cycle, respectively.

As shown in Fig.5(a), there is an apparent increase in the shear deformation after the "as-built" column R-1 experienced its first yield point. At the same stage, it was observed that flexural cracks became inclined and new shear cracks started to form in the middle of the column faces in the loading direction. Significant increase of shear deformation and decrease of flexural deformation occurred at μ=3, during the repetitive cycling. This indicates the shifting of the failure mode from flexure to shear. At this stage, the column failed in shear. On the other hand, the retrofitted column R-2 exhibited a stable deformation dominated by flexural deformation. The shear component was maintaining about 1/3 of the flexural component up to μ=8 and a drift angle of 2.6%. However no further data was available after this displacement level, since some of the transducers ran out of stroke.

6. CONCLUDING REMARKS

The poor shear capacity of existing inadequately designed short rectangular reinforced concrete columns can be significantly improved by using an elliptically shaped steel jacket. Based on the philosophy of current capacity design, the retrofit can be done by setting the enhanced shear strength to exceed the new flexural strength. The factor of shear strength over flexural strength for the retrofitted column described in this paper was set to be 3.8, and as a result, a ductile flexural response was obtained. A further reduction of this margin will be considered in future studies.

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