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# SEISMIC RETROFIT OF BRIDGE COLUMN-FOOTING CONNECTIONS

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## ABSTRACT

Reversed cyclic loading tests were carried out on one-half scale reinforced concrete bridge columns, having details typical of those found in older bridges in moderate seismic regions, such as Montréal. The poor performance of the "as-built" column-footing connection using dowels, typical of construction in the 1960's, was demonstrated by low ductility, poor energy absorption and low strength, due to the failure of the lap splices. The proposed retrofit scheme involved adding a reinforced concrete footing block, anchored to the existing footing and existing column using dowels, as well as jacketting the column with a circular steel shell filled with concrete. This retrofit improved the performance by shifting the location of plastic hinging and confining both the column and the lap splice region. This resulted in increased ductility, increased strength and a significant increase in the energy absorption.

## KEYWORDS

Reinforced concrete; bridge; column; column-footing connections; retrofit; lap splice; steel jacket.

## INTRODUCTION

Many of the bridge structures in moderate seismic regions, such as eastern Canada, were designed and built in the 1960's. At that time, typical reinforced concrete column-footing connections were detailed as being fully fixed and had dowel bars which matched the column vertical reinforcement. These connections have limited ductility and energy absorption since the plastic hinge length is restricted in the region of the dowel splices. Furthermore, no additional confinement reinforcement was provided over the splice length (typically 40 $d_b$), therefore reducing its effectiveness under reversed cyclic loading. The scope of this research was to investigate the behavior of "as-built" reinforced concrete bridge columns with lapped column bars and to develop a retrofit technique suitable for use in moderate seismic regions.

## PREVIOUS RESEARCH

Priestley and Park (1987) tested reinforced concrete bridge columns to assess their strength and ductility under seismic loading. The test specimens included square, octagonal and circular columns, with varying amounts and details of confinement reinforcement, loaded under different levels of axial loads. They determined that the amount of transverse reinforcement required increased with increasing axial load. They also investigated the influence of lap splicing the longitudinal reinforcement within the length of the plastic hinge and found that a lap length of 30 $d_b$ was insufficient due to spalling of the concrete cover at high ductilities. In addition, the presence of the lap splices limited the length over which the vertical reinforcement could yield. The yielding of the reinforcement over a small length results in extremely high strains with the possibility of bar buckling on load reversal and bond failure of the lap splice. It was also demonstrated that lapping by cranking bars into the concrete core was more effective than side-by-side lapping. Chai *et al.* (1991) tested circular columns with lap spliced dowels having a splice length of 20 $d_b$ and observed a significant pinching of the hysteresis loops beyond a displacement ductility of 1.5. These specimens exhibited a rapid loss of lateral load carrying capacity and very little energy dissipation. Priestley *et al.* (1994a, b) developed theoretical models and design guidelines for steel jacketting of reinforced concrete bridge columns for enhanced flexural and shear strengths, and to ensure adequate confinement of lap splices. They also presented the results of a series of tests on jacketted columns which showed that this retrofit technique stabilized the hysteretic response up to displacement ductilities of about 8.

## TEST PROGRAM

A half-scale reinforced concrete bridge column, having a "fixed" column-footing connection, was tested under reversed cyclic loading in order to measure the response of an "as-built" column with lapped column bars. A 680 mm lap splice length was provided for the No. 15 bars, which corresponds to a 42 $d_b$ splice length. A companion column was retrofitted to evaluate the effectiveness of the retrofit on improving the ductility and the energy absorption of the existing connection.

Each column was subjected to reversed cyclic lateral loading applied at the approximate inflection point of the prototype structure, assumed to occur at mid-height of the fixed-fixed column. Before applying the reversed cyclic lateral loading, an axial load of 312 kN was applied to simulate the superstructure dead load (i.e., an axial stress of 0.04 $A_g f'_c$).

The applied loads were measured by load cells while linear voltage differential transducers (LVDTs) were used to determine the deflections, as well as the rotations and the curvatures over the lower 700 mm of the column. Strain gages were located on both the dowel bars and the column bars within the lap splice length to monitor the effectiveness of the lap splice in transferring the tensile stresses from the dowels to the column bars. Strain gages were also located on the four ties closest to the base of each column to determine their participation in resisting shear and confining the concrete.

The test procedure used "load control" until general yielding was observed. The deflection at general yielding, $\Delta_y$, was selected as the deflection beyond which a significant reduction in stiffness occurred during loading. After general yielding occurred, "displacement control" was used, by imposing deflections in increments of $\Delta_y$.

## "AS-BUILT" SPECIMEN C6

Figure 1 shows the test setup and the reinforcement details for "as-built" Specimen C6. This 3050 mm tall, 510 mm square column had reinforcing details consistent with those used in typical 1960's bridge columns in the Montréal area. Dowel bars, which matched the column vertical reinforcement, provided the connection between the column and the footing. These dowel bars were extended a distance of $1.7\ell_d$ (680 mm) into the column section resulting in a 42 $d_b$ lap splice length. The dowel bars were anchored into the 600 mm thick footing with 90-degree standard hooks. A clear cover of 25 mm was provided in the column section and a 170x170x36 mm shear key was formed at the base of the column. The longitudinal reinforcement ratio in the column was 1.23%. No supplementary cross ties were provided in the column section and 8 mm diameter ties were spaced at 300 mm. Although these ties had excellent end anchorage, with 135-degree hooks, the lack of supplementary cross ties and the large tie spacing did not provide adequate confinement

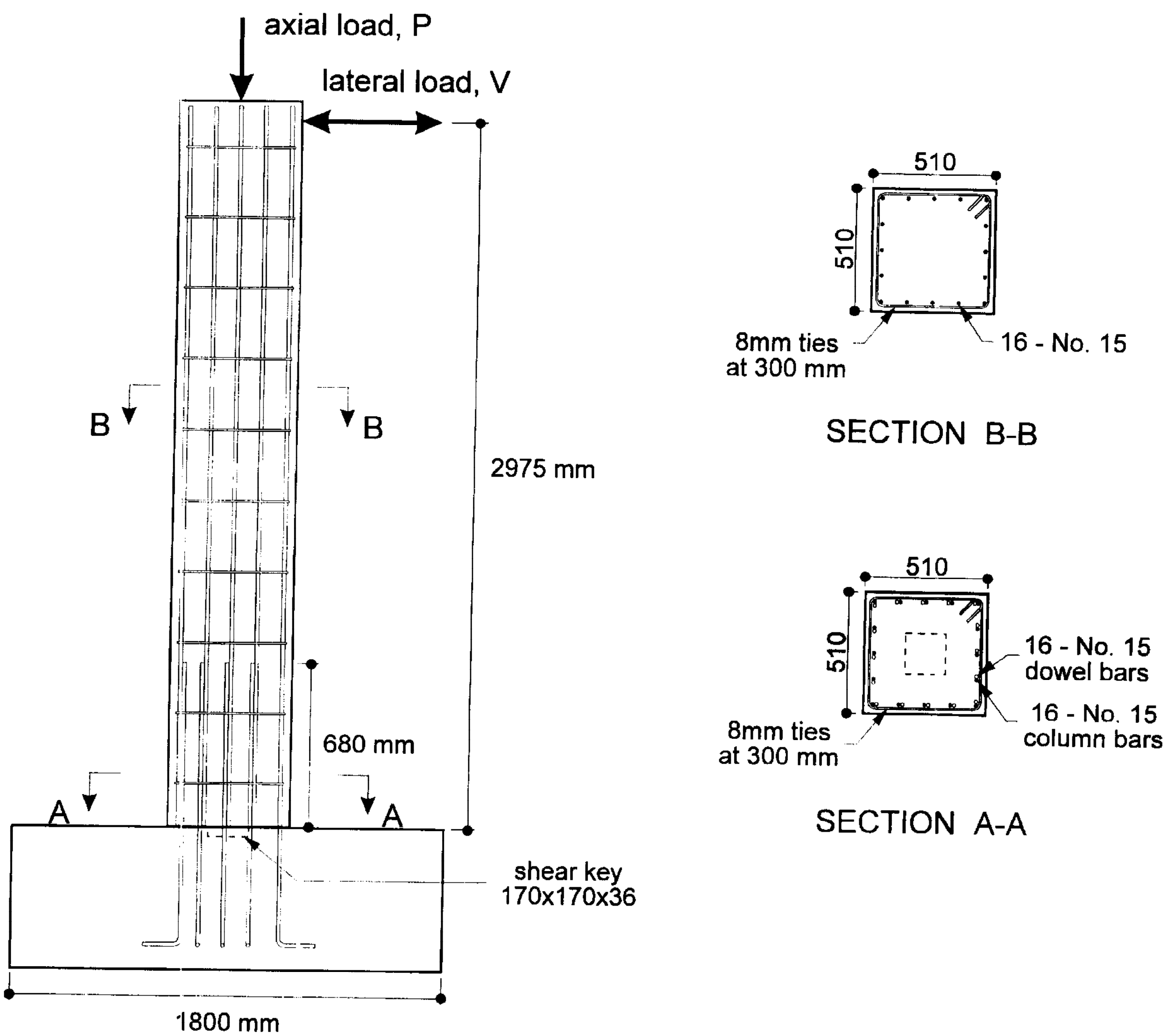


**Fig. 1** Test setup and details of "as-built" specimen C6

of the column and the lap splice, nor did they provide the necessary shear strength to develop plastic hinging in the column.

*Predicted Behavior of "As-built" Specimen C6*

The predicted nominal moment capacity, $M_n$, of Specimen C6 was determined using the computer program RESPONSE (Collins and Mitchell, 1991) using the measured yield stress, $f_y$, of the reinforcing steel. The probable moment capacity, $M_p$, of this column was determined by setting $f_s = 1.25\ f_y$. Figure 2 shows the nominal and probable predicted moment capacities of the column along its height. The moment diagram corresponding to the applied load at first yield is plotted, with first yield occurring at the base of column. As the applied moment is increased, yielding will spread until the limiting moment of $M_p$ is reached (see Fig. 2). The predicted capacities shown in Fig. 2 are based on the assumption that the lap splice is effective under the reversed cyclic loading. The relatively small predicted length of plastic hinging would result in low ductility levels.

*Response of "As-built" Specimen C6*

The shear versus deflection response for "as-built" Specimen C6 is shown in Fig. 3. This test specimen exhibited a stable hysteretic response and minor degradation between successive cycles at each displacement

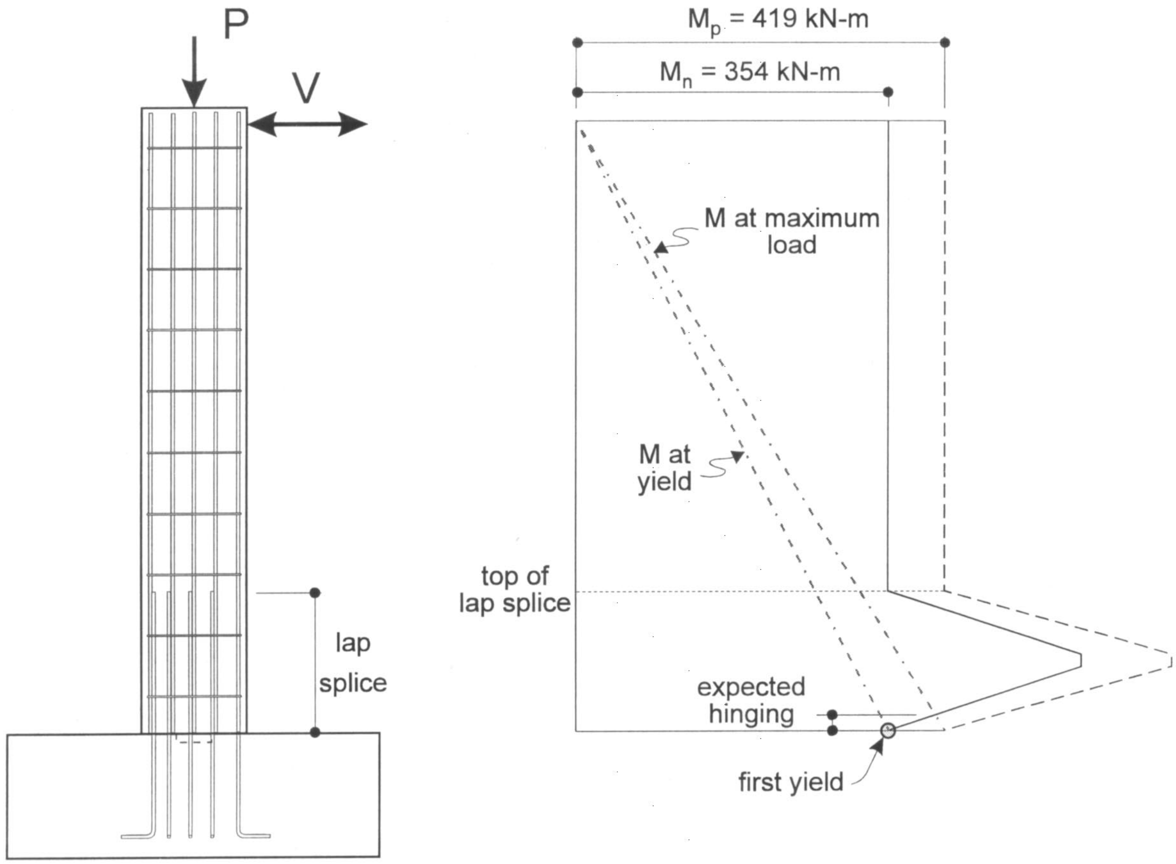


**Fig. 2** Predictions of flexural yielding and hinging for "as-built" specimen C6

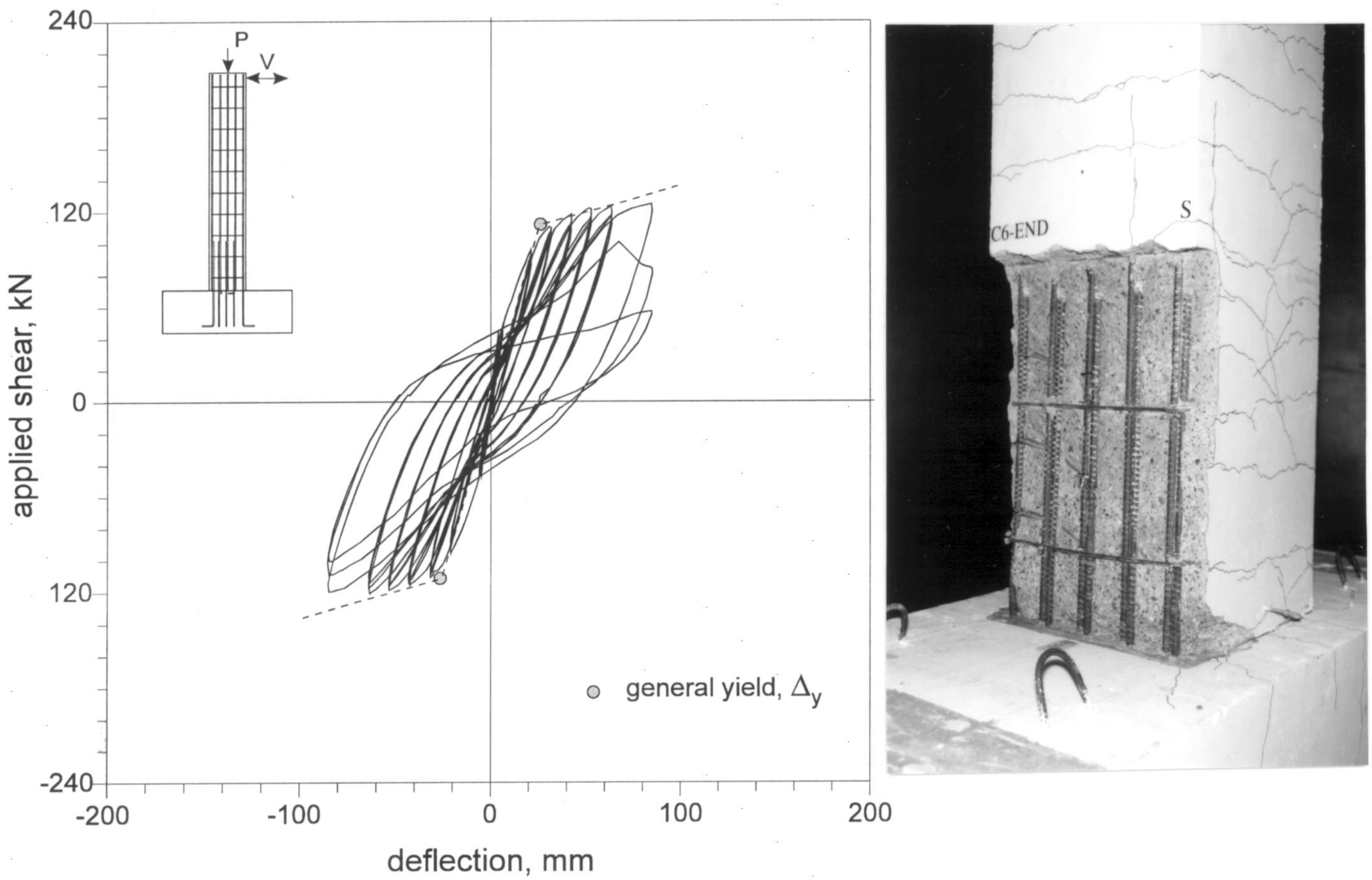


**Fig. 3** Response of "as-built" specimen C6

level up to $2.5\Delta_y$, at which point the lap splice began to fail. Evidence of this lap splice failure could be seen by the propagation of a splitting crack along the outermost dowel bars together with a horizontal crack at the top of the lap splice. This vertical crack extended along the entire height of the lap splice and widened during the second and third cycles at $2.5\Delta_y$. At a ductility of about 3.3, the lap splice failed as exhibited by the complete loss of the concrete cover on one side of the specimen and the sudden loss of strength and stiffness (see Fig. 3).

## RETROFIT SPECIMEN C6R

### *Retrofit Design Philosophy*

The retrofit strategy for Specimen C6R involved adding a reinforced concrete footing block, as shown in Fig. 4, to move the location of yielding from the base of the column to the top of the lap splice. This retrofit would enable significant plastic hinging at the top of the dowels and would add significant confinement to the lap splice. Figure 4 shows the predicted nominal and probable moment capacities over the height of the retrofitted column. The predicted probable moment capacity of the column, $M_p$, above the lap splice is 419 kN-m which corresponds to a moment of 543 kN-m at the top of the existing footing. In order to achieve the desired hierarchy of yielding, vertical dowel bars were drilled and bonded into the existing footing to increase the nominal flexural capacity to 584 kN-m, which exceeds the applied moment at the base of the column when $M_p$ is reached at the top of the splice. In determining the nominal flexural capacities in the added footing block, a conservative approach was taken which neglected the retrofit dowel bars acting in compression and neglected the contribution of the footing block concrete in compression. In addition, the column was jacketted to increase the shear capacity and confinement in the column, and to confine the lap splice. As can be seen from Fig. 4, first flexural yielding is predicted to occur at the top of the lap splice. As the moment is increased, the flexural yielding would spread above and below this point, resulting in a significant plastic hinge length in the retrofit column.

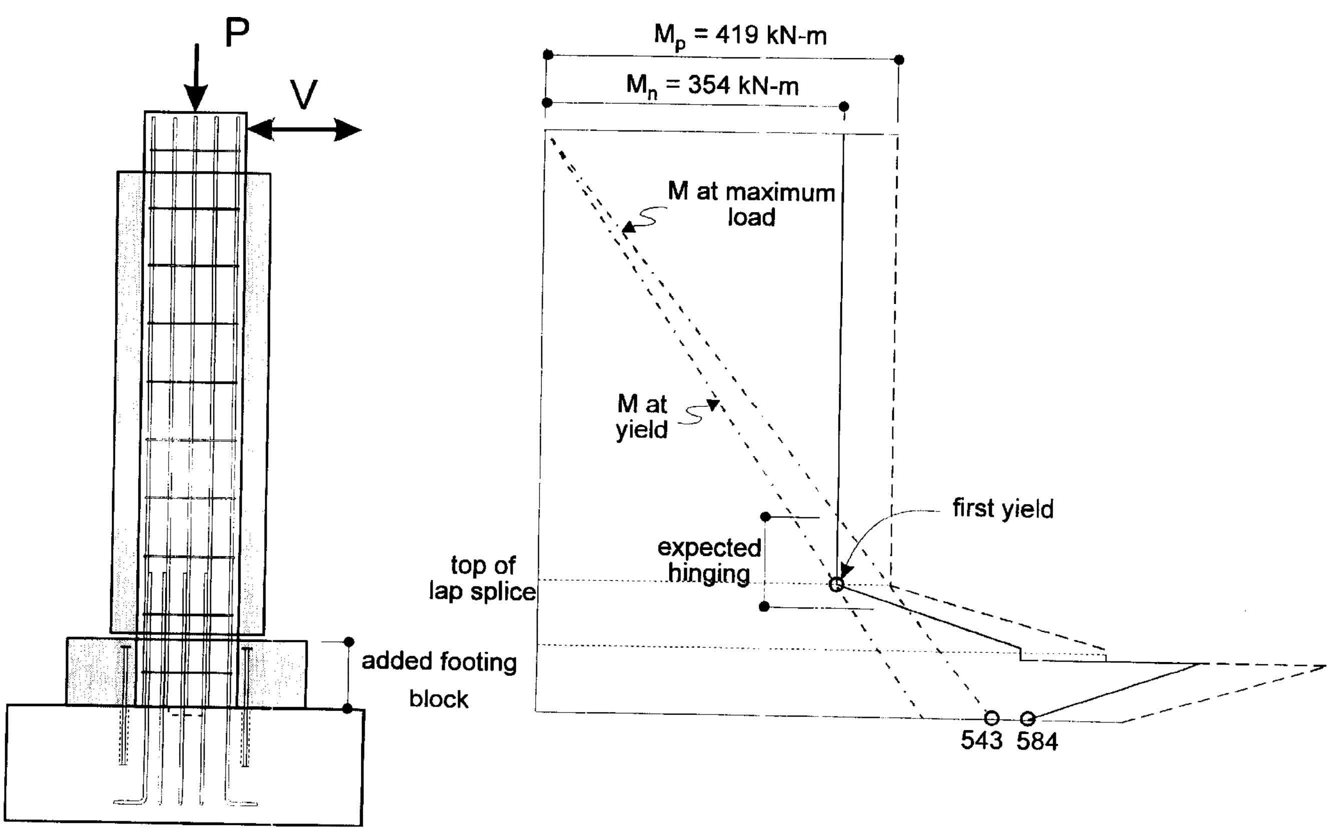


**Fig. 4** Predictions of flexural yielding and hinging for retrofit specimen C6R

*Retrofit Details of Specimen C6R*

The retrofit details for Specimen C6R are shown in Fig. 5. A 350 mm layer of reinforced concrete was added to the existing footing to: (a) increase the flexural capacity at the top of the existing footing; (b) transfer the forces from the retrofit dowel bars to the column; (c) provide confinement over the lower portion of the lap splice; and (d) strengthen the existing footing with added concrete and a top mat of reinforcing steel. The flexural capacity just above the top of the existing footing block was increased by providing sixteen No. 15 vertical dowel bars, anchored with welded plates on one end, with the other end drilled and epoxied into the existing footing. In order to transfer the tensile forces from the retrofit dowel bars into the column, sixteen No. 15 horizontal dowel bars were drilled and bonded into the column core. After the added footing block was cast, a 3.2 mm (1/8 in.) steel shell was placed around the existing column section to provide confinement over the upper portion of the dowel bar lap splice and to increase the shear strength and confinement of the column (Priestley *et al.*, 1994a, b, NCEER, 1993, and Calvi and Priestley, 1991). Concrete was cast to fill the area between the existing column and the circular steel shell. A 25 mm (1 in.) gap was provided between the base of the steel jacket and the top of the new footing block such that the jacketting would not increase the flexural strength of the column at this location.

## RETROFIT PERFORMANCE

The lateral force-displacement response of Specimen C6R is shown in Fig. 6. This specimen exhibited a stable hysteretic response, without significant pinching or strength loss, up to a ductility level of about 6.6. At a ductlility level of about 2.1, some debonding of the steel shell and cracking of the jacket concrete were observed. The significant relative slip observed between the shell and the jacket concrete at ductility levels above 2.1, indicated that the shell did not contribute significantly to the flexural strength. As expected, the new footing block showed some significant cracking, with crack widths as wide as 0.45 mm at the end of testing. It should be pointed out that Figs. 3 and 6 give the applied shear versus deflection plots without the P-delta effects. At the maximum shear load for Specimen C6R, the total moment at the top of the existing footing is $VH + P\Delta = 219.6 \times 2.975 + 312 \times 0.107 = 687$ kN-m. This maximum moment achieved is considerably greater than the predicted moment of 543 kN-m at the top of the existing footing (see Fig. 4). The reasons for this increased capacity include a number of effects that have been ignored in the retrofit design. These include: (a) neglecting the presence of the steel shell and jacket concrete in contributing to the flexural capacity; (b) neglecting the beneficial effects of confinement from the jacketting which increases the concrete strength and ductility; (c) using only 1.25 $f_y$ in the probable moment calculations; and (d) the beneficial effects of confinement in reducing the development length of the spliced bars. At the end of the test, one loading cycle to a ductility of 8.3 resulted in a drop in moment capacity of 17%. At a displacement ductility of 6.6, the weld seam of the steel jacket cracked at its base, reducing its confining effect. At the end of the test, the steel shell and the jacket concrete were removed to expose the original column. Significant cracking of the original column indicated that plastic hinging had occurred over the height of the lap splice. The retrofit increased the flexural strength by approximately 74%, increased the ductility by a factor of 2.5 and significantly increased the energy absorption.

## CONCLUSIONS

These tests demonstrated that bridge columns built in the 1960's, with lap spliced dowel bars, result in columns that have limited ductility and energy absorption since the ability of the plastic hinge to spread is restricted. Furthermore, the provision of lap splice lengths of $42d_b$, together with the small amount of column confinement, resulted in bond failure of the splice region which limited the strength and ductility. The proposed retrofit scheme involved adding a reinforced concrete footing block, anchored to the existing footing and existing column using dowels as well as jacketting the column with a circular steel shell filled with concrete. This retrofit improved the performance by shifting the location of plastic hinging and confining both the column and the lap splice region. This resulted in increased ductility, increased strength and a significant increase in the energy absorption.

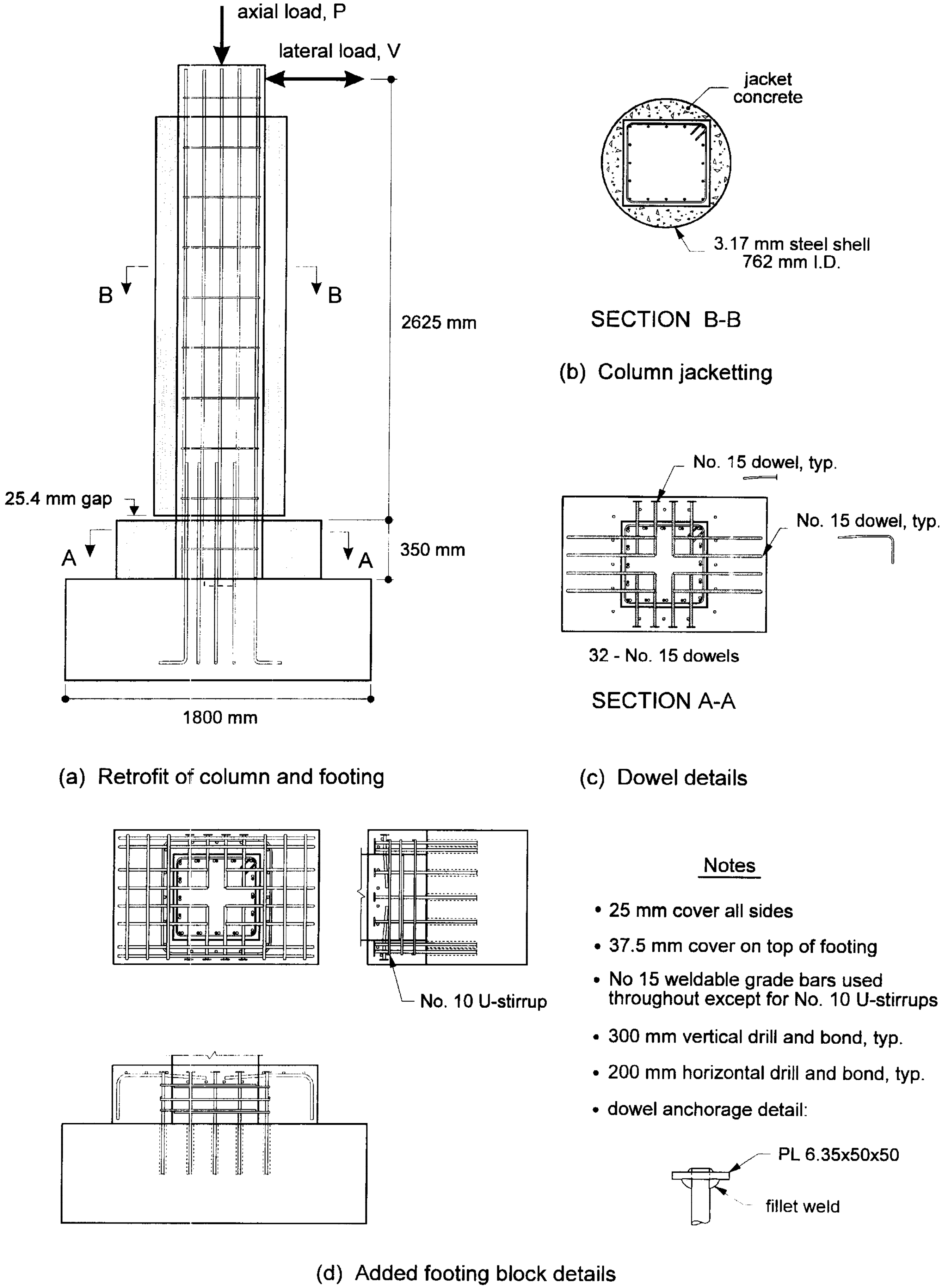


**Fig. 5** Details of retrofit specimen C6R

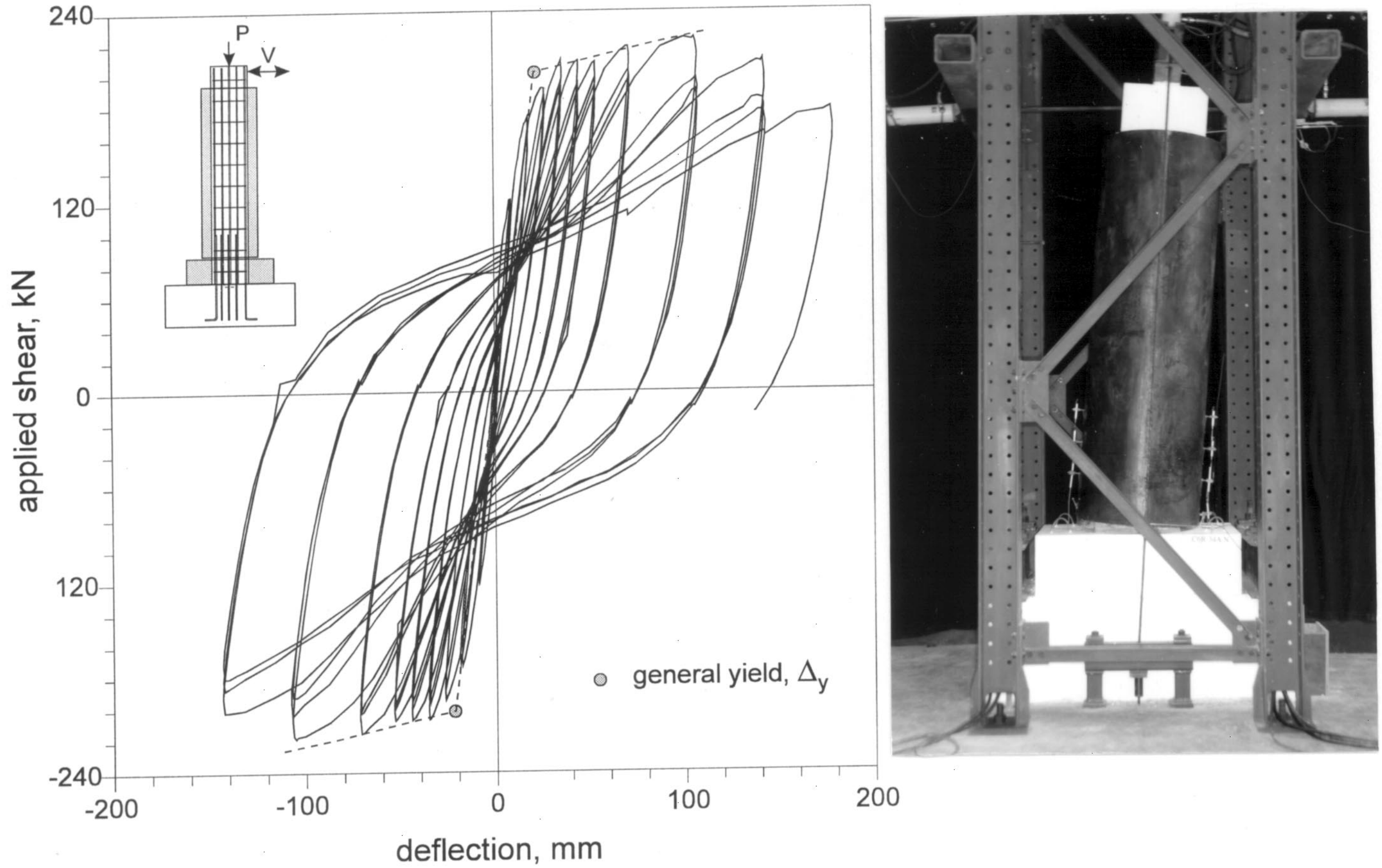


**Fig. 6** Response of retrofit specimen C6R

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