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Local thin jacketing for the retrofitting of reinforced concrete columns

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Abstract. Two series of tests were conducted to investigate the behavior of local thin jacketing for the retrofitting of reinforced concrete (RC) columns. In the first series, four full-scale RC columns with a height of 400 cm and a 30 cm square cross-section were tested under constant axial load and reversed cyclic lateral displacements. The heavily damaged columns were retrofitted with local thin jacketing. Self-compacting concrete (SCC) was used in the production of 7.5 cm thick, four-sided jacketing. The height of the jacketing was 100 cm for one specimen and 200 cm for all others. In the second series, the retrofitted columns were retested with the same axial load and displacement history. The effectiveness of local thin jacketing in the retrofitting of RC columns was examined with respect to lateral strength, stiffness, inelastic load-deformation behavior and energy dissipation.

Keywords: reinforced concrete column; precast column; retrofitting; jacketing; self-compacting concrete.

1. Introduction

Precast frame buildings are widely used in the construction of industrial facilities and commercial malls. Single story warehouses represent the most common structural configuration, which consists of cantilever columns connected by simply supported precast and prestressed beams. Connection of the non-moment resisting beams to the columns was achieved on site. In general, the structural configuration depends entirely on the cantilevered columns for lateral strength and stiffness. Several industrial buildings collapsed in Turkey during the last devastating earthquakes of Ceyhan (1998) and Marmara-Kocaeli (1999), which led to major disruptions in the manufacture of various goods. Posada and Wood (2001) stated that one type of structural damage frequently observed in single story industrial buildings involved the flexural hinges located at the base of columns.

RC jacketing is one of the earliest and most frequently used techniques to strengthen RC members. It involves enlarging the existing cross section with a new layer of concrete that is

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strengthened with both longitudinal and transverse reinforcement. For a column type RC member, the placement of jackets improves the stiffness, strength, ductility and hysteretic response. The use of conventionally poured normal-weight concrete and relatively large jacket thicknesses is a common practice.

The thickness and height of the RC jacket are important parameters that require consideration, especially in regard to the retrofitting of members for industrial buildings. The process of RC jacketing needs to be accomplished in a limited space given the dense cabling and piping that is usually present in industrial buildings. It was therefore decided that the lower one-quarter or one-half of the columns be retrofitted by thin RC jacketing. Minimizing the dimensions of the RC jacketing will limit the amount of additional mass imparted to the structure following retrofitting. SCC was utilized in the jacketing process given the small thickness of the jacketing especially with longitudinal and transverse reinforcement in the jacket. The main advantages associated with the use of SCC include good workability, resistance to segregation, and marked filling and passing abilities.

The efficiency associated with the use of SCC in thin jacketing to retrofit RC columns was evaluated in this experimental study. For this purpose, the performance and behavior of four original and jacketed columns were tested and the results were analyzed.

Ilki *et al.* (1998) tested three full-scale original and jacketed columns. The cross-sectional dimension of the square columns was 30 cm and the specimen height was 400 cm. RC jacketing with a thickness of 10 cm was applied to the damaged columns throughout their height. Concrete compressive cylinder strength at stage of testing used in the jackets for the three specimens was 15, 7 and 13 MPa. The selected water/cement ratio was relatively high in an effort to increase the workability of the concrete. A shear connector was not employed at the interface of the old and new concrete parts, and the jacketed columns were subjected to displacement reversals. Perfect integration between layers of the section was maintained throughout the duration of the testing. A comparison of load-displacement curves revealed that the compressive strength of concrete used in jacketing was highly effectual on the general response of the jacketed columns.

Bett *et al.* (1988) studied the effectiveness of three different repair and/or strengthening techniques in enhancing the lateral load response of identical RC short columns. Three identical columns were constructed at two-thirds scale. The square columns were 12 (30.5 cm) cross section. Strengthening involved encasing the original column with a shotcrete jacket reinforced with closely spaced transverse ties. To support the transverse steel, additional longitudinal reinforcement was placed at each corner of the jacket but it was not anchored to the end blocks. The thickness of the shotcrete shell was 2-1/2 (6.4 cm) and the surface of the column was sandblasted prior to strengthening. The columns repaired by jacketing were much stiffer and stronger laterally than the original column.

Rodrieguez and Park (1993) tested four reinforced concrete columns subjected to simulated seismic loading to investigate repair and strengthening techniques. The as-built columns were 350 mm square and contained low quantities of transverse reinforcement. Two column units were tested, repaired and strengthened by jacketing and retested. The other two column units were strengthened by jacketing and tested. The thickness of jacketing was 100 mm. The as-built columns displayed low available ductility and significant degradation of strength during testing, whereas the jacketed columns behaved in a ductile manner with higher strength and much reduced strength degradation.

Julio *et al.* (2005) performed a series of tests to analyze the effect of interface treatment on the structural behavior of columns strengthened by RC jacketing. They concluded that the monolithic behavior of the composite element could be achieved, even without increasing the surface roughness, using bonding agents or applying steel connectors prior to strengthening by RC jacketing. They reported that a grout with the characteristics of self-compacting concrete (SCC) and high strength concrete (HSC) can be employed to achieve reduced thickness of the jacket.

Gomes and Appleton (1994) reported that the monolithic behavior of the jacketed RC columns could be achieved if a higher percentage of transverse reinforcement is used. They recommend that half the spacing of the original column transverse reinforcement be adopted for the jacket transverse reinforcement.

Ersoy *et al.* (1993) utilized two series of tests to investigate the behavior of columns. In the first series, four columns having identical dimensions and reinforcement were tested under monotonic axial loading. Following this test, the columns were jacketed and retested. In the second series, three jacketed and two monolithic reference specimens having identical dimensions and reinforcement as the jacketed specimens were tested under combined axial load and bending. The cross-sectional dimensions of the original columns were 13 × 13 cm and the thickness of the jacketing was 2.5 cm. It was reported that repaired and strengthened jackets behaved well both under monotonic and reversed cyclic loadings.

Vandoros and Dritsos (2006) investigated the effects of axial preload on RC columns that were strengthened by the placement of RC jackets. They observed a positive effect of preloading with respect to strength and deformation capacities. Preloading reduced the initial stiffness but helped to retain stiffness during testing. Preloading allowed the specimen to dissipate more energy than otherwise would have occurred when constructing the jacket without preloading.

Abdullah and Takiguchi (2003) presented an experimental research program on the use of circular and square ferrocement jackets for strengthening square RC columns with inadequate shear resistance. A total of six identical RC columns based on 1/3 to 1/8 scales were constructed. The ratio of axial load and the effects of jacketing schemes were investigated. Marked stable and ductile responses were observed regardless of the axial load ratio and strengthening scheme employed.

Xiao and Wu (2003) introduced an improved steel jacketing method to retrofit RC columns using a square or rectangular section for enhanced strength and improved ductility. In the retrofitting of existing columns, relatively thin steel plates were welded to form a rectilinear jacket for shear strength enhancement, and then additional confinement elements with various types of desired configuration were welded to the potential plastic hinge regions to ensure ductile behavior. Five 1/3 scale model columns were tested. Test results demonstrated the efficiency of the partially stiffened rectilinear steel jacketing, which prevented brittle shear failure and markedly improved column ductility, with a drift ratio greater than 8%.

Ong *et al*. (2004) investigated the suitability of two existing confined concrete models, to predict the behavior of jacketed reinforced concrete columns when subjected to axial loads. Full length and partial jacketing were studied. They concluded that the two analytical models take into consideration the effects of confinement and may be used to predict the entire stress-strain relation of jacketed RC columns subjected to axial loads.

The behavior of heavily damaged columns retrofitted by local RC jacketing has not been reported in the literature. Bingol (2004) completed a testing program in which several full-scale jacketed columns were tested using SCC in the construction of the local thin jacketing. The current study assessed the performance of jacketed columns by making use of the experimental data obtained in Bingol's study.

2. Objective and scope

The aim of this investigation was to evaluate the effectiveness of local and thin jacketing of retrofitted columns belonging to existing industrial buildings. Use of partial and thin RC jacketing would have advantages over alternative methodologies given the dense cabling and piping that is usually present in these types of buildings.

Self-compacting concrete (SCC) was used in the production of local thin jacketing. One of the major aims of the study was to determine the effectiveness of SCC in the retrofitting of existing structural elements.

The scope of this study essentially details two experimental stages. In the first stage, four full-scale RC columns were tested using reversal displacement cycles. In the second stage, the heavily damaged columns were retrofitted with local thin jacketing and retested using a similar loading pattern. The retrofitted columns were evaluated with respect to cracking pattern, moment curvature relations, load-deflection relations, energy dissipation capacity and structural integrity.

Nonlinear static analysis was performed to estimate the lateral load carrying capacity of the original and jacketed columns. The analytical results obtained were compared with the experimental findings and an improved correlation was observed.

3. Series 1 – Original columns

Four full-scale square columns having identical dimensions and different reinforcing ratios were tested under constant axial load and lateral displacement cycles with gradually increasing amplitude. They are referred to as the original columns. All of the columns were severely damaged with significant reduction in lateral stiffness.

3.1 Test specimens and material properties

A prototype of a precast RC single story industrial building was designed according to the Turkish Earthquake Code of 1998. A classical precast foundation socket was utilized, and the socket

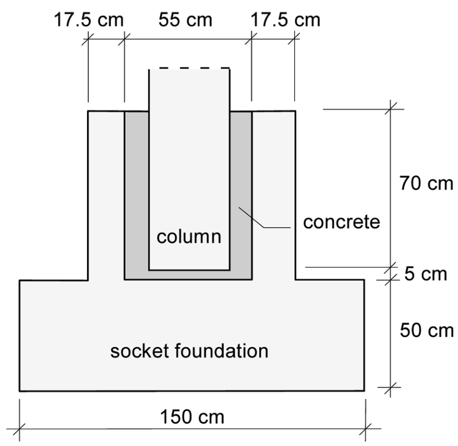


Fig. 1 Geometry of the socket foundation

foundation was proportioned according to the axial load transmitted by the column and to the magnified design moment of the bottom edge of the column.

The socket foundations and columns were fabricated separately and the columns were positioned into the socket foundations in the laboratory. Gaps between the column and walls of the socket foundation were filled with conventionally placed ordinary concrete having compressive cylinder strength of 30 MPa, (Fig. 1).

The original columns were 30 × 30 cm in cross section. The height of the column measured from the top of the socket foundation was 400 cm and the columns had eight longitudinal reinforcement bars. Three different diameters of reinforcement were used in the column cross sections. Specimen

Table 1 Longitudinal reinforcement of the original columns

Specimen	Number of reinforcements	Reinforcement diameter [mm]	Reinforcement ratio %
S30-14	8	14	1.4
S30-14M	8	14	1.4
S30-16	8	16	1.8
S30-18	8	18	2.3

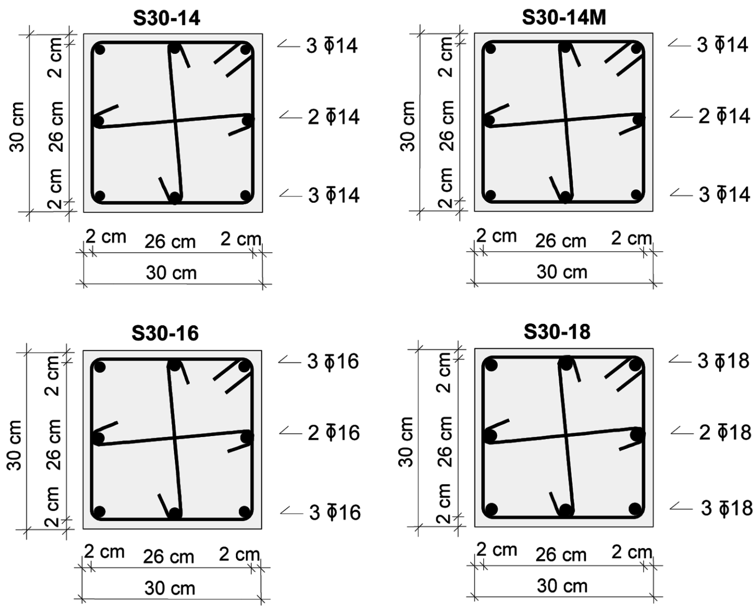


Fig. 2 Geometry and reinforcement details of original columns

Table 2 Average compressive strength of concrete for the original columns

Specimen	Average compressive strength [MPa]
S30-14	45.9
S30-14M	40.9
S30-16	44.2
S30-18	45.8

Table 3 Average yield strength of reinforcement bars

Bar diameter [mm]	Average yield strength [MPa]
8	476.3
14	478.9
16	502.4
18	452.4

designations and longitudinal reinforcement characteristics are given in Table 1. The first number in the designation represents the cross-sectional dimension and the subsequent number indicates the longitudinal reinforcement diameter. Details concerning the geometry and reinforcement arrangement of the columns are shown in Fig. 2. The diameter of the transverse reinforcement was 8 mm for all columns. The spacing of the transverse reinforcement was 10 cm for the critical 230 cm long lower portion of the column height and 15 cm for the upper portion.

The average concrete compressive cylinder strengths obtained from the standard compression tests are shown in Table 2. The average yield strengths obtained from the tension tests performed on the transverse and longitudinal reinforcement bars are shown in Table 3.

3.2 Testing setup, procedure and instrumentation

The socket foundations were fixed to the testing floor by means of high-strength steel bars. A tension force of 100 kN was imposed on each of these bars in order to resist slipping and rotation.

The lateral tip load was applied using a MTS hydraulic actuator with a loading capacity of ±250 kN and a displacement capacity of ±30 cm (Fig. 3). Both ends of the actuator were hinged to avoid secondary effects. In order to generate and measure the axial load, a hydraulic jack and a load cell was placed at the top of the column. A cross-shaped rigid steel beam and four steel cables

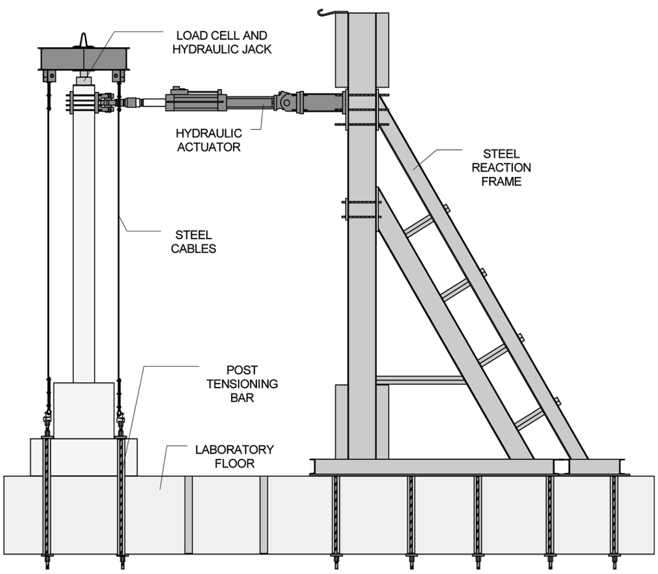


Fig. 3 Loading setup

Table 4 Axial load applied to specimens

Specimen	Axial load [kN]
S30-14	130
S30-14M	130
S30-16	180
S30-18	210

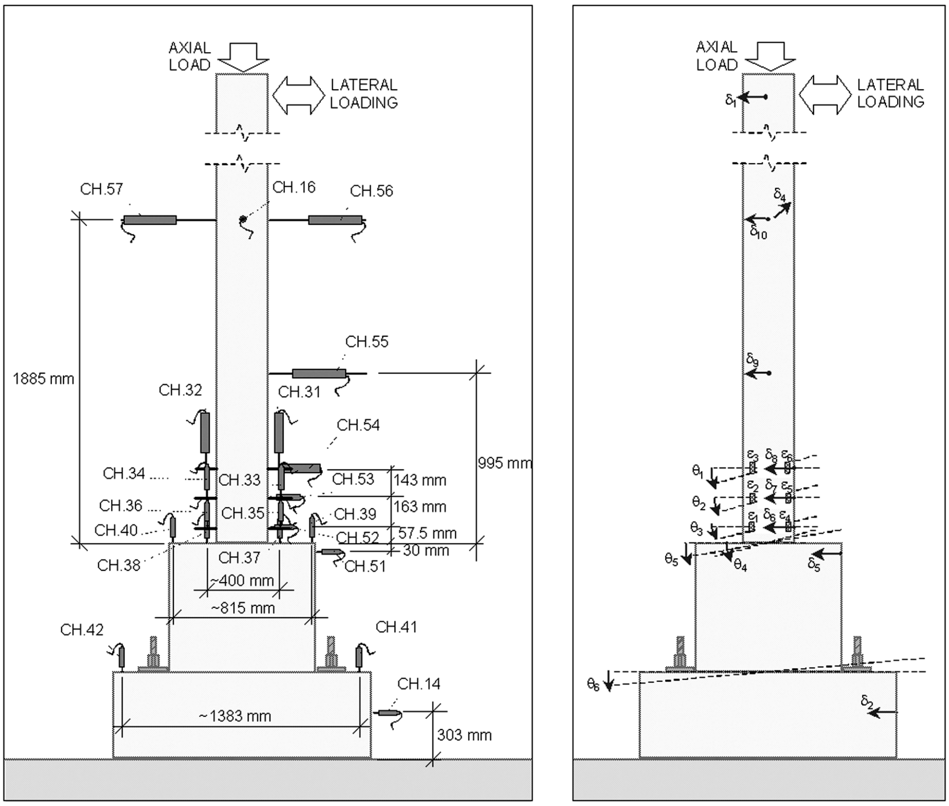


Fig. 4 Instrumentation for displacements and curvatures

transferred the reaction of the applied axial load to the strong floor of the laboratory (Fig. 3).

Lateral cyclic loading was performed by repeating three cycles at each displacement step and the amplitude was increased gradually to simulate a seismic effect. Only two reversal displacement cycles were applied to specimen S30-14M.

A constant axial force corresponding to the vertical support reaction of the roof beam in the prototype structure was applied to the specimens. The applied axial force was in the range and $0.05A_gf_c$, where A_g and f_c are the gross cross-sectional area of the columns and the average concrete compressive strength, respectively (Table 4).

Specimens were monitored using appropriate instrumentation in order to determine the translation and rotation at certain critical sections, including the strain in the longitudinal reinforcements (Fig. 4). Except for minor differences regarding the position of some transducers, the instrumentation used was almost the same for all specimens.

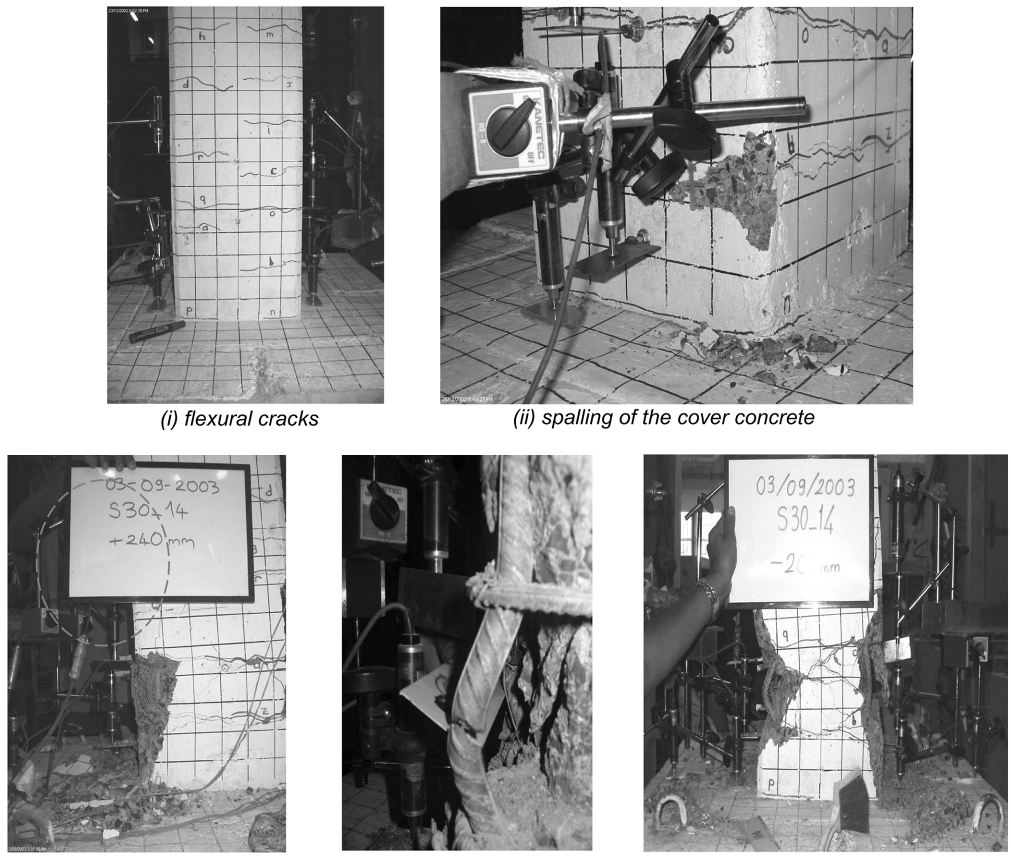
The designation and position of displacement transducers used in the measuring system are shown on the left of Fig. 4, while the monitored displacement and strain are depicted on the right of the same figure. CH 14 was included in the instrumentation to record any potential rigid translation of the socket foundation, while CH 16 was used to monitor the out-of-plane movement of the tested columns. CH 31 to CH 42 were used for measuring the rotations θ_1 to θ_6, which were used for calculating the curvature at the plastic hinge region. CH 51 to CH 57 were employed for measuring the lateral translations δ_5 to δ_{10}, which were used to define the deformed shape of columns.

Six strain gauges were included at the bottom edge of each original column with a view to monitoring the axial deformation of the longitudinal reinforcements. The axial deformation components ε_1 to ε_6 recorded during the tests are shown on the right of Fig. 4.

3.3 Test results

Each original column was tested until considerable crushing of concrete and buckling of longitudinal bars were observed at the maximum moment region of the cantilever. No significant damage was observed in the socket foundation and column-socket interface.

The original columns developed flexural cracks at the bottom edge after cycling to 0.2 percent



(i) flexural cracks *(ii) spalling of the cover concrete*

(iii) buckling of the reinforcement *(iv) failure of the reinforcement* *(v) plastic hinge*

Fig. 5 Failed specimen S30-14

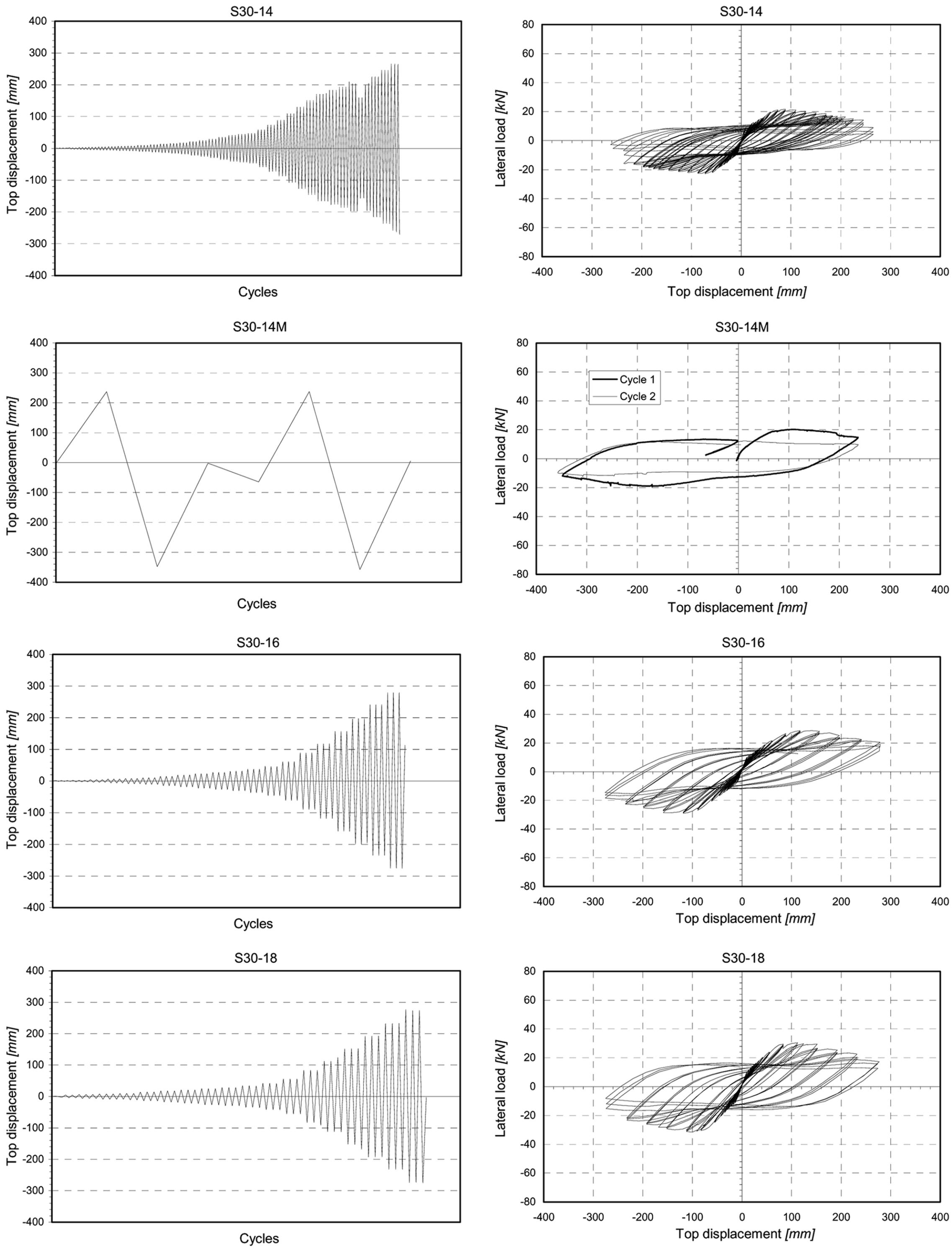


Fig. 6 The applied displacement cycles and response of the original columns

drift. Yielding of the longitudinal reinforcement was observed at 1.4 percent drift and the corresponding width of the flexural cracks was 0.5 mm. Cycling to 2 percent drift caused widening of these flexural cracks to more than 0.7 mm. Crushing of the concrete of all the original columns was observed at 4 percent drift. Buckling of the longitudinal bars was observed at 6.25 percent drift (Fig. 5). All columns yielded a similar damage pattern.

The performance of each specimen is presented in terms of the load-displacement response. The applied displacement patterns and the response curves for the original columns are shown in Fig. 6. The original columns displayed stable hysteretic behavior for the duration of the tests. The most distinct difference in behavior that was observed concerned the strength capacity of the columns, which arose from variation in the longitudinal reinforcement ratio.

Specimens S30-14 and S30-14M are identical except for the loading pattern applied, and no marked difference was observed in their behavior.

4. Jacketing of columns

Before jacketing was introduced, the following processes were performed: Mechanical removal of the damaged concrete cover, cleaning of the substrate to receive the jacket, concrete core repair with resin injections, and restoration of yielded or broken longitudinal bars by means of welding new reinforcing steel to the damaged region.

Local thin RC jacketing was placed onto all columns. The thickness of the jacket was 7.5 cm all around the square column cross section. An additional longitudinal reinforcement ratio of 1% was

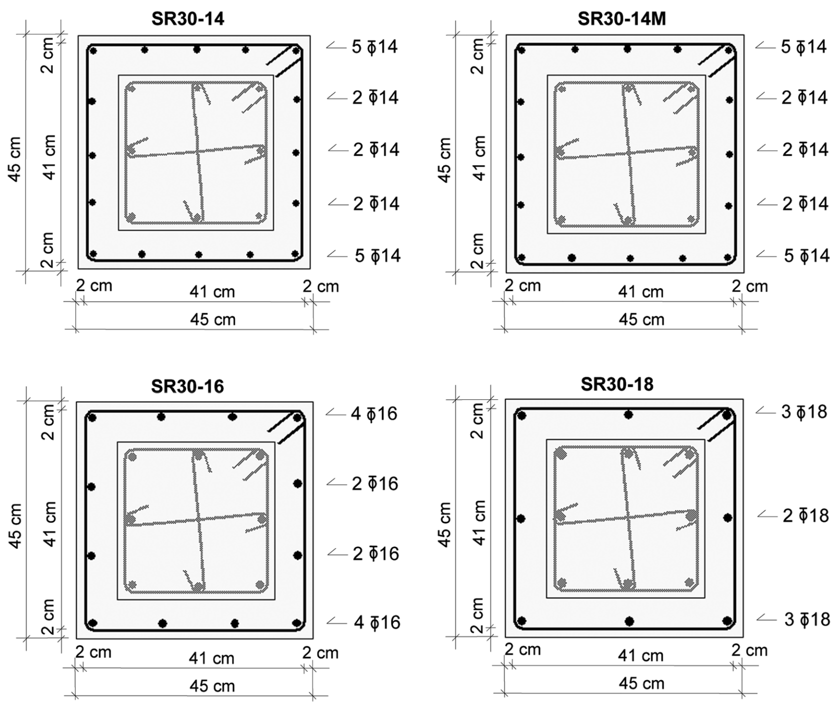


Fig. 7 Geometry and reinforcement details of jacketed columns

configuration of the reinforcements *specimen provided with forms*

Fig. 8 Photographs taken during the jacketing works

utilized in the jacket cross sections. The height of the thin jacketing was 100 cm for specimen S30-14 and 200 cm for all others. The columns were jacketed after relieving the axial load.

The steel longitudinal bars of the jacket were anchored to the socket foundation using a commercially available two-component epoxy resin. The depth of the holes was 30 cm and holes were cleaned using a vacuum cleaner. Longitudinal bars of varying diameter (Φ14, Φ16 or Φ18) were employed in the jacketing in an effort to determine the effect of bar diameter on the anchorage. Details concerning the geometry and longitudinal reinforcement arrangement are shown in Fig. 7. The diameter of the transverse reinforcement was 8 mm. Based on the work of Gomes and Appleton (1994), the spacing of the transverse reinforcement was 5 cm along the whole length of the jackets.

No surface treatment was applied prior to jacketing. To achieve monolithic behavior of the composite section, L-shaped shear connectors made from a Φ16 reinforcement bar were anchored onto all sides of the columns. The distance between the shear connectors was 30 cm (Fig. 8).

SCC was cast into the jacket from the top of the framework (Fig. 8). For curing process, the jackets were covered with wet blankets after removing of the frameworks. The average concrete compressive cylinder strength of the SCC was 34.6 MPa.

Notwithstanding the dense reinforcement cage, the use of SCC provided an outstanding result. Voids in the jacketing were not observed in for any of the specimens after failure.

5. Series 2 – Retrofitted columns

The axial loading and lateral cyclic loading history applied to the jacketed columns were identical to those applied to the original columns. The 'R' shown in the specimen designations refers to a specimen that has been retrofitted. A detailed comparison of the response of the original and jacketed column follows.

5.1 Cracking patterns

For the 100 cm jacketing height (SR30-14), the initial flexural cracks were observed at 0.25

deformed shape of the column *plastic hinge failure above the jacket*

Fig. 9 Failed specimen SR30-14

crushing of the socket concrete *crushing and spalling of the socket concrete*

Fig. 10 Significant on specimens SR30-16 and SR30-18

percent drift at the column section above the jacketing, and concrete crushing was observed at 3 percent drift (Fig. 9). In general, the response of specimens SR30-14 and SR30-14M, which had the same reinforcement ratio but different jacketing height, was similar. For both the aforementioned specimens, flexural cracking was concentrated at the column section just above the jacketing. For specimens with a jacketing height of 200 cm (SR30-16 and SR30-18), initial flexural cracks were observed with 0.15 percent drift at the column section above the jacketing. Cycling to 2 percent drift caused widening of these flexural cracks to more than 1.5 mm. Damage accumulated at the top of the socket foundation during application of 4 percent drift. The concrete of the socket foundation was crushed and some of the reinforcements protruded out (Fig. 10).

No major damage was detected on the jacketed parts of any retrofitted column, although a limited number of thin flexural cracks formed on the jackets.

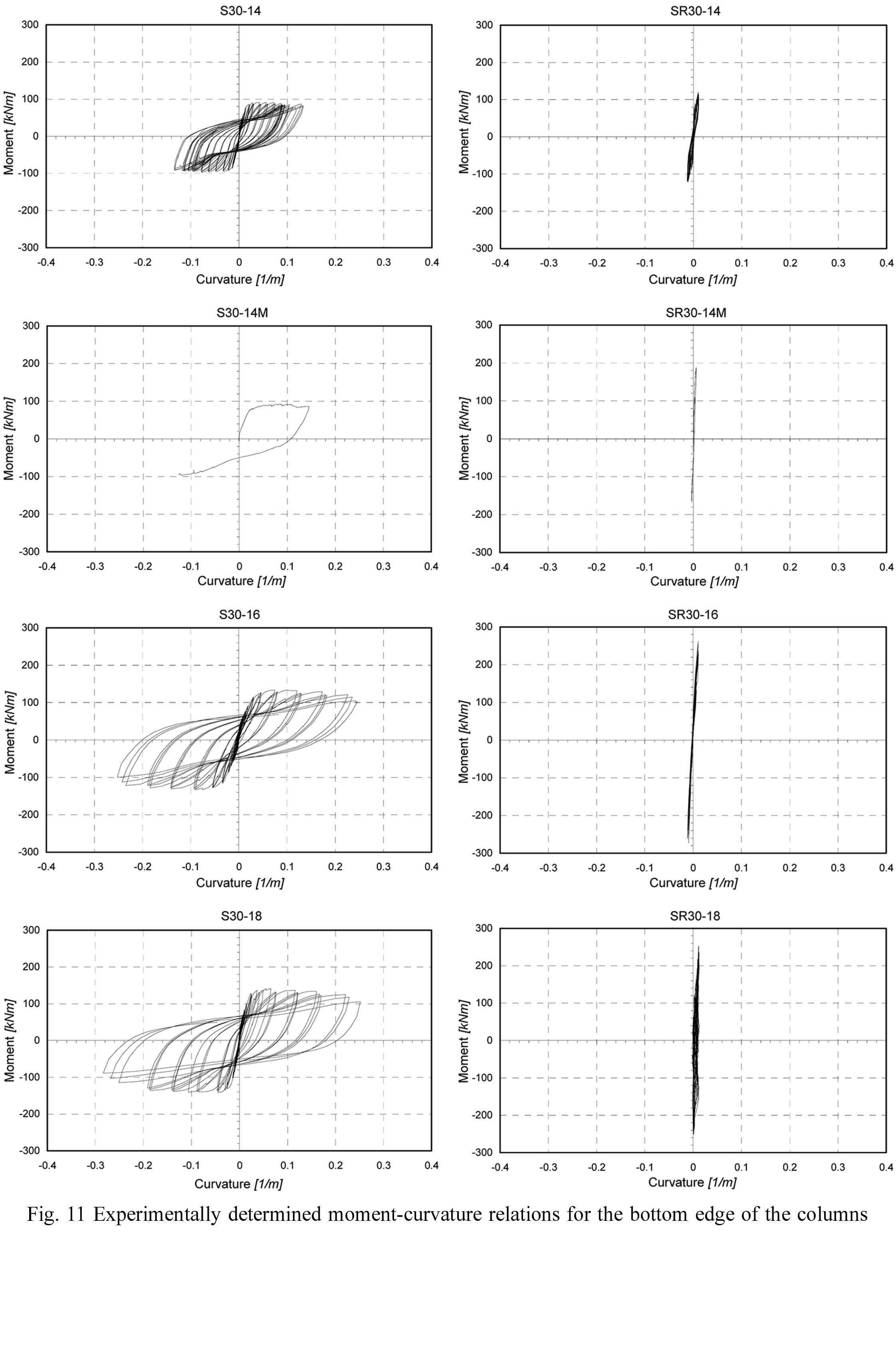


Fig. 11 Experimentally determined moment-curvature relations for the bottom edge of the columns

Table 5 Column bottom edge moment capacities

Specimen	Height of jacketing [cm]	Original case [kNm]	With jacketing [kNm]	Relative increment
SR30-14	100	91.75	120.32	31.1%
SR30-14M	200	91.98	188.98	105.5%
SR30-16	200	134.19	262.73	95.8%
SR30-18	200	141.92	253.36	78.5%

5.2 Moment-curvature relations

The experimentally determined moment-curvature relations determined at the bottom edge of the columns are illustrated in Fig. 11 for the original and jacketed specimens. It can be concluded that flexural stiffness of the section increased significantly for all jacketed specimens. The maximum moments reached at the bottom edge of the columns are shown in Table 5. The increment ratio was 31% for S30-14 and greater than 78% for the other specimens, a result that can be attributed to differences in the height of the jacketing.

5.3 The load-deflection relations

The load-deflection curves for the jacketed columns are shown on the left of Fig. 12, while envelope curves of the original and jacketed columns are compared on the right of the same figure. A substantial increase in the load carrying capacity of the columns with the application of local thin jacketing was observed. A comparison of the ultimate strength capacities is shown in Table 6. As seen from Table 6, the lateral strengths of all the jacketed columns were significantly higher than those of the original columns. For SR30-14, the relative increment in ultimate strength was 35% and for other specimens was greater than 114%.

5.4 Energy dissipation capacities

The energy dissipation was determined by calculating the area inside the hysteretic load-displacement loops for each cycle. The cumulative energy dissipated was calculated as the sum of the area enclosed by all previous hysteretic loops. The cumulative energy dissipation versus drift ratio relation for each specimen is shown in Fig. 13.

The energy dissipation capacity improved due to the increased strength of the jacketed columns with respect to the original columns. For SR30-14, the increment in the dissipated energy was relatively low and the original column exhibited higher energy dissipation following a drift ratio of 4.6%. A steady increment in the dissipated energy was observed throughout the duration of the tests for specimens where the jacketing was applied to only half of the column height. With a 2% drift ratio, the energy dissipation capacity of S30-16 and S30-18 increased 4 times following retrofitting.

5.5 Structural Integrity

The high performance achieved and other observations made during the tests clearly demonstrate the structural integrity and efficiency of local thin jacketing. There was no indication of any

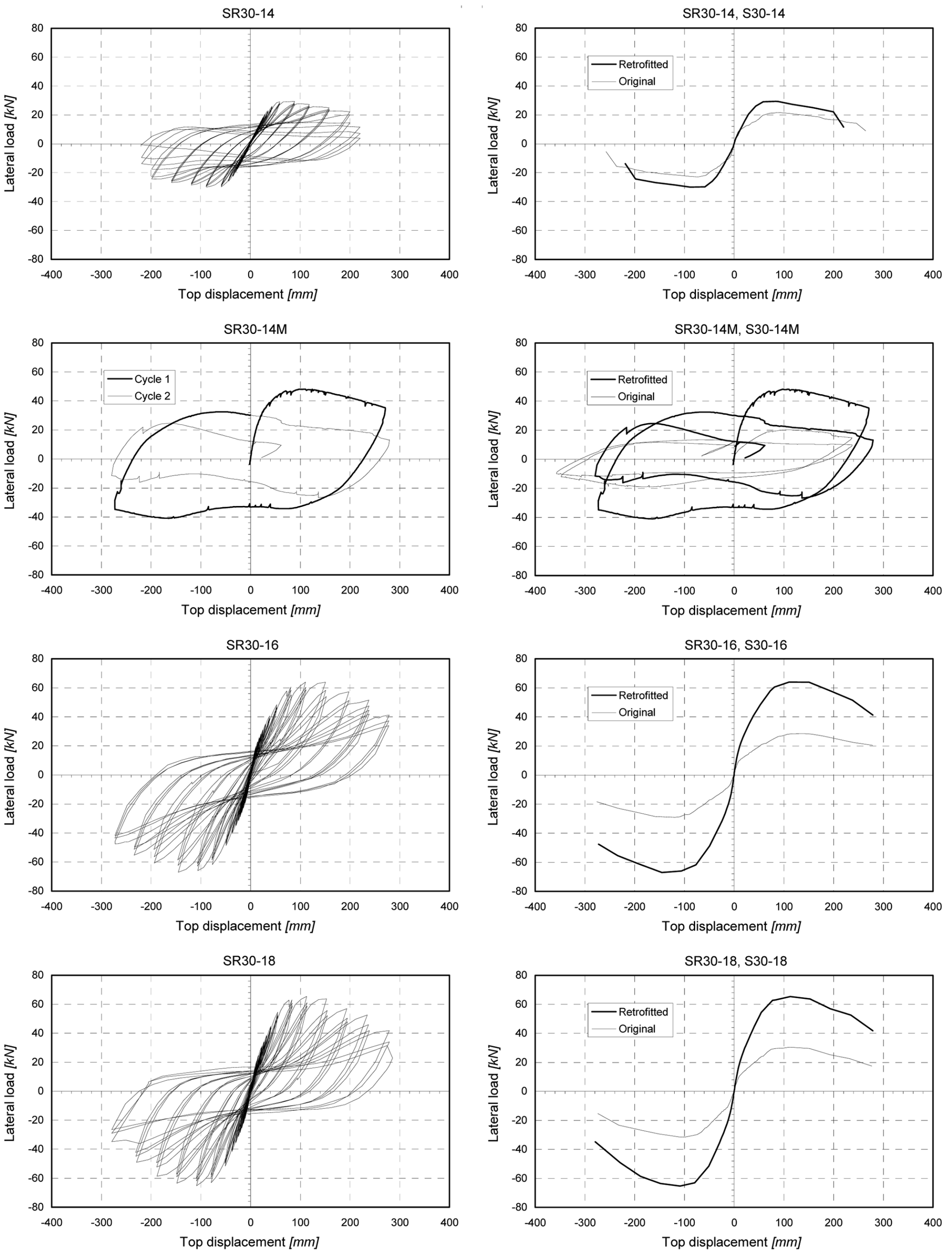


Fig. 12 Lateral load-top displacement curves of jacketed and original columns

Table 6 Comparison of lateral strengths

Specimen	Height of jacketing [cm]	Original case [kN]	With jacketing [kN]	Relative increment
SR30-14	100	21.6	29.3	35.6%
SR30-14M	200	20.3	48.1	136.9%
SR30-16	200	28.6	64.0	123.8%
SR30-18	200	30.5	65.3	114.1%

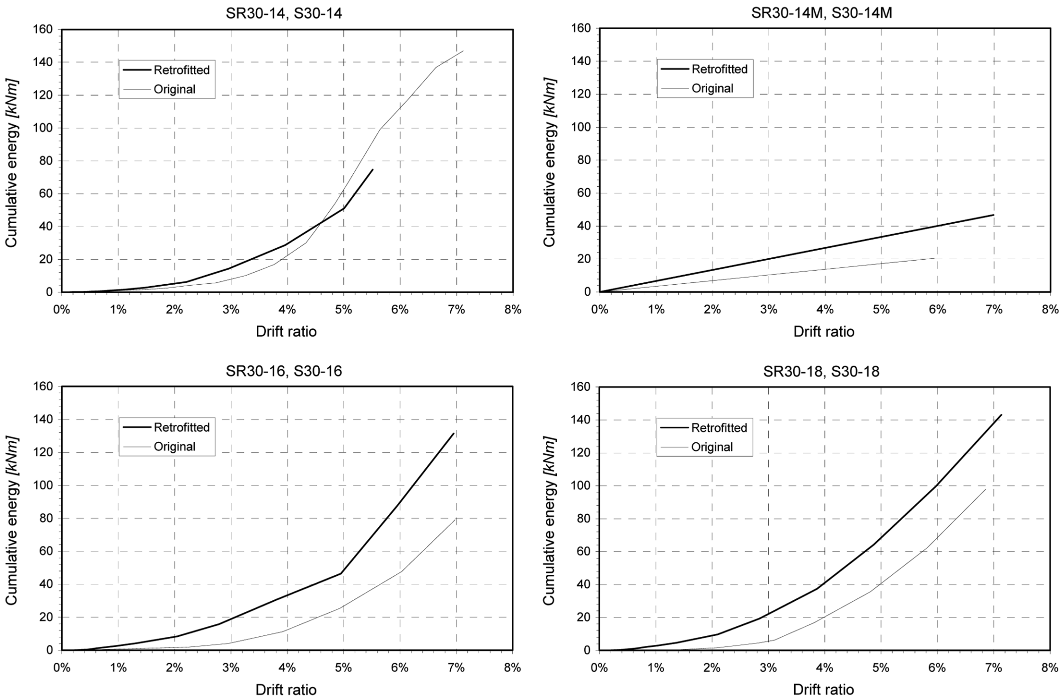


Fig. 13 Cumulative energy dissipation of the specimens

slippage of the longitudinal bars of the local thin jacket from the holes in the socket foundation. As for the top of the jacket, the only cross section where the interface boundary was visible, no cracking was evident and there was no relative movement between the jacket and the original column. It is concluded that a higher percentage of transverse reinforcement and SCC are the essential contributors to the structural integrity of the jacketed column.

6. Analytical studies

The moment curvature relations for the original and jacketed column sections were generated using XTRACT, a fully interactive computer software program that can be employed for the analysis of structural cross sections. To represent the nonlinear behavior of the concrete and steel in the RC cross section, Mander's confined and unconfined concrete models, and a bilinear, parabolic strain hardening steel model were used. The strain-stress relationships obtained from the material

Table 7 Ultimate load carrying capacities

Specimen	Experimental ultimate load [kN]	Analytical ultimate load [kN]	Relative difference %
S30-14	21.63	21.80	−0.8
S30-14M	20.70	20.95	−1.2
S30-16	28.60	28.86	−0.9
S30-18	30.50	30.92	−1.4
SR30-14	29.30	28.40	3.1
SR30-14M	48.10	47.40	1.5
SR30-16	64.00	57.70	9.8
SR30-18	65.30	60.67	7.1

tests were used to assign the parameters of the models. The maximum concrete strain for confined and unconfined concrete was 2% and 0.3%, respectively. For the jacketed cross sections, the longitudinal reinforcement of the original column that was heavily damaged was ignored when calculating the moment-curvature relation.

Nonlinear static analyses of the original and jacketed columns were performed using IDARC2D. Constant axial and incremental lateral tip loads were utilized in the analysis. Bilinear moment curvature relationships idealized from the XTRACT output were employed in the analytical work performed with IDARC2D. The spread plasticity option in IDARC2D was selected for the nonlinear analysis. The ultimate loads for the original and jacketed columns obtained from the experimental and analytical findings and their relative differences are shown in Table 7. The analytical results are in good agreement with the experimental findings. For the jacketed columns, the ultimate loads obtained analytically are smaller than those obtained experimentally. This can be attributed to the longitudinal reinforcements which were heavily damaged in the original column, and ignored when calculating the moment-curvature relations of the jacketed cross sections.

7. Comparison with previous work

The dimensions adopted for the original column and the RC jacket thickness investigated by Julio *et al.* (2005) were 20 × 20 cm and 3.5 cm, respectively. The column height was 135 cm and the corresponding jacket height was 90 cm. The ratio of the dimension of the jacket to the original column was 0.175. The compressive strength of the original column was 35 MPa while that of the RC jacket was 80 MPa. An increase in the yielding load to 123% was observed and the ultimate load increased to 152% of the original. The secant stiffness, obtained by dividing the experimental value of the horizontal yielding force by the corresponding displacement, increased to 151%.

The cross-sectional dimensions of the original column investigated by Bett *et al.* (1988) were 30.5 × 30.5 cm and the column height was 96 cm. A 6.4 cm shotcrete jacket in which No 6 longitudinal bars were used produced a 43.2 cm square column. The shotcreted jacket spanned the full height of the column. The ratio of the dimension of the jacket to the original column was 0.208. The specimens were subjected to a unidirectional, cyclic lateral load. The compressive strength of the original columns was 28 MPa, while that of the shotcrete used for jacketing the columns was 31 MPa. Narrow flexural cracks developed at 1% drift. At 1.5% drift, inclined cracks developed from these flexural cracks and wide flexural cracks opened between the shotcrete jacket and the end blocks. At 2.5% drift, the jacket was crushed and spalled near both end blocks. The lateral load capacity of the jacketed column was almost 83% higher than the original column.

In the current study, full-scale RC 30 cm square columns with a height of 400 cm were tested under constant axial load and reversed cyclic lateral displacements. The ratio of the dimension of the jacket to the original column was 0.25. Two RC jacketing heights were investigated in this study. SCC was utilized for the production of the 7.5 cm thick jacketing. The compressive strength of the original column was 44 MPa, while that of the jacketing was 34.6 MPa. SCC jacketing was applied to the full-scale heavily damaged original columns, which differs from other reported studies. The ratio of the dimensions of the jacketing to the original column was greater than that reported in the two aforementioned investigations. For the jacketing with a height of 200 cm, damage accumulated at the top of the socket foundation during application of 4 percent drift, and no major damage was detected on the jacketed parts except for a limited number of thin flexural cracks that formed on the jackets. The lateral strength increment in the jacketed columns was 35% and 114% for columns with a jacketing height of 100 cm and 200 cm, respectively. With a 2% drift level, the energy dissipation capacity of the columns after retrofitting increased 4 times.

A comparison of the results presented in this study with those reported by Ilki *et al.* (1998) shows that thin jacketing constructed to half the height of heavily damaged columns is more effective than use of a relatively thick jacket spanning the whole height of the column using conventionally placed ordinary concrete.

Application of local thin jacketing with SCC is as effective as the two aforementioned methods with respect to lateral stiffness, strength, damage and energy dissipation capacity.

8. Conclusions

Four full-scale original and jacketed precast RC columns were tested under constant axial and reversed cyclic lateral loads. The original columns were tested until considerable crushing of the concrete and buckling of the longitudinal bars were observed at the bottom edge. SCC was used in the production of 7.5 cm thick, four-sided local thin jacketing. The jacketed columns performed extremely well and showed marked increases in lateral strength in comparison to the original columns.

Based on the results of this investigation using four full-scale columns, the following conclusions can be drawn:

1. The applied local thin jacketing made from SCC markedly increased the lateral stiffness and strength properties of the heavily damaged columns. The lateral strength increased by 35% for the specimen with a jacketing height of 100 cm, while the ratio was above 114% for the other specimens.
2. For the column retrofitted by local thin jacketing, determining the position of the plastic hinge may be dependent on the original column flexural capacity and the height of the jacketing. Although the height of the jacketing differed for specimens SR30-14 and SR30-14M, the position of the plastic hinges was similar. For both of these specimens, the longitudinal reinforcement ratio was 1.4% and the concrete compressive strength was similar. For specimens SR30-16 and SR30-18 for which the longitudinal reinforcement ratio was 1.8% and 2.3%, respectively, the plastic hinges were formed at the top of the socket foundation.

3. Use of local thin jacketing as a retrofitting technique can increase the energy dissipation capacity of existing RC columns. It seems that more energy is dissipated with increasing jacket height. The energy dissipation capacity of SR30-16 and SR30-18 with a 2% drift ratio was four times greater than that associated with the respective original columns.
4. The capacity of the existing socket foundation should also be taken into consideration when determining the height of the jacketing.

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