# POST-EARTHQUAKE REPAIR AND STRENGTHENING OF REINFORCED CONCRETE BEAM-COLUMN CONNECTIONS (THEORETICAL & EXPERIMENTAL INVESTIGATION)

**Alexander G. Tsonos[1] and Konstantinos V. Papanikolaou[1]**


## ABSTRACT

In this study the effectiveness of all the repair and strengthening techniques proposed by the United Nations Industrial Development Organization (UNIDO) Manual and by Eurocode 8: part 1-4 for reinforced concrete beam-column joints damaged by strong earthquakes is investigated experimentally and analytically. Five one-half-scale exterior beam-column joint specimens were submitted to reverse cyclic pseudo-static displacements. Three of these specimens were then repaired by the epoxy pressure injection technique or by the removal and replacement technique. The other two specimens were strengthened by partial three-sided jacketing. All the repaired and strengthened specimens were then subjected to the same displacement history as that imposed on the original specimens. It can be concluded that all the repair and strengthening techniques proved to be satisfactory.



## KEY WORDS

beams (supports); buildings; columns (supports); damage; earthquake resistant structures; joints (junctions); lateral pressure; load (forces); reinforced concrete; repairs; shear properties; cement grout.


## INTRODUCTION

Investigations into recent earthquake damage in Greece (Thessaloniki, 1978; Corinth, 1981; Kalamata, 1986; Aegion, 1995; Athens, 1999) have shown that, in many cases, damaged areas of reinforced concrete buildings were localised in beam-column connections. Furthermore, considering the commonly accepted idea that failure of joints may quickly lead to general failure (Park and Paulay, 1975), the important issue of the effective repair or strengthening of beam-column connections damaged in cyclic loading has arisen (Popov and Bertero 1975, Rodriguez and Park 1991, Penelis and Kappos 1997, Elnashai 1997).

In trying to address the issue of the effectiveness of any repair and strengthening technique, it is essential to bear in mind that the behaviour of reinforced concrete connections involves the influence of complex interacting phenomena such as shear, bond, confinement, fatigue, which even independently are not yet well understood. Therefore, since unanswered questions exist even concerning the behaviour of undamaged connections designed to withstand seismic excitations, it is justifiable to allow a certain measure of uncertainty for the mechanics of applied repair and strengthening techniques. Additional questions concerning the effectiveness of generally applied reinforced concrete joint repair and strengthening techniques also arise from the existing variety of shear reinforcement design practices.

## REPAIR AND STRENGTHENING TECHNIQUES FOR BEAM-COLUMN JOINTS ACCORDING TO UNIDO 1983 AND ACCORDING TO EUROCODE 8: PART 1-4 (STRENGTHENING AND REPAIR OF BUILDINGS) 1995

In 1983, the United Nations Industrial Development Organization, with the participation of several countries in the Balkan region, produced a manual based on experience gained in this region, which provides mainly qualitative guidelines for the repair and strengthening of buildings. Some case studies are also presented in this manual (UNIDO (1983), Rodriguez and Park (1991)).

Eurocode 8: part 1-4 (strengthening and repair of buildings) (1995) covers the repair and strengthening of buildings and, where applicable, monuments, considering commonly used structural materials (concrete, steel, masonry and timber).

Field reports after damaging earthquakes often indicate that beam-column joints are one of the most vulnerable structural regions. Under earthquake loading, joints often suffer shear and/or bond (anchorage) failures. Three possible repair and strengthening techniques exist, namely:

### (i) Epoxy injections

Epoxy injections can be applied in the repair of damaged joints with slight to moderate cracks without damaged concrete or bent, or failed reinforcement.

[1] *Aristotle University of Thessaloniki, School of Engineering, Dept. of Civil Engineering Division of Structural Engineering, 541 24 Thessaloniki, Greece.*

### (ii) Local replacement of damaged concrete and steel ("equal section method")

Removal and replacement should be carried out for heavily damaged joints with crushed concrete, buckled longitudinal bars or ruptured ties. Depending on the amount of concrete removed some additional ties or reinforcement may be added. Before concreting, the existing joint should be saturated with water as necessary (UNIDO (1983), EC8: part 1-4 (1995)).

### (iii) Reinforced concrete jacketing

In the case of heavily damaged joints of space frames, a reinforced concrete jacket is required, which can be located in the joint area only. The reinforced concrete jacketing of a joint is performed in such a way that all the members connected at the joint collaborate together.

## TESTS CONDUCTED

An investigation was conducted at the University of Thessaloniki to evaluate the effectiveness of the techniques proposed by UNIDO 1983 and the Eurocode 8: part 1-4 (Strengthening and repair of buildings) 1995 for the repair and strengthening of reinforced concrete beam to column connections damaged by severe earthquakes. More specifically, five reinforced concrete exterior beam-column subassemblages were constructed. The subassemblages were subjected to cyclic lateral load histories so as to provide the equivalent of severe earthquake damage. The damaged specimens were then repaired or strengthened according to the UNIDO Manual Techniques (UNIDO 1983) and according to Eurocode 8: part 1-4 (1995). These repaired and strengthened specimens were again subjected to the same cyclic lateral load history. The measured response histories of the original and repaired or strengthened specimens were subsequently compared and evaluated.

## BACKGROUND AND PREVIOUS RESEARCH ON THE PERFORMANCE OF REPAIRED AND/OR STRENGTHENED R/C BEAM - COLUMN SUBASSEMBLAGES

Despite the many unanswered questions related to the behaviour of reinforced concrete structures repaired by epoxy injection, this technique has been used extensively in the aftermath of past earthquakes around the world.

Popov and Bertero (1975), presented a comparison of the performance of a reinforced concrete interior beam-column subassembly tested under cyclic loading with its performance after repairing it with epoxy resin. They observed that the bond around the reinforcing bars in the joint region once destroyed does not seem to be completely restored by epoxy injection. UNIDO 1983, EC8: part 1-4 (1995) and NEHRP 1985 (FEMA-97) state also that epoxy injection is not effective in restoring the bond between reinforcement and concrete. Corazao *et al.* (1988) investigated the effectiveness of different repair and strengthening techniques in restoring or improving the properties of reinforced concrete beam-column subassemblages damaged by earthquake-type loading. Based on their test results, they concluded that large variations in the performance of subassemblages repaired by epoxy injection can occur depending on the quality of the injection work. Data obtained from a testing subassemblage after repairs involving the removal and replacement of the damaged concrete in the beam and beam-column joint region demonstrated that the stiffness and strength of the specimen had been completely restored.

Lee *et al.* (1980) investigated the effectiveness of repair of reinforced concrete exterior beam-column subassemblages. The epoxy injection technique and the removal and replacement technique using high-early-strength materials were used to repair the beams of the subassemblages. They observed that because of the increase in beam strength due to the use of high strength repair materials, there is the possibility of damage moving from the beam to the unrepaired joint and column. The same was also observed by Corazao *et al.* (1988).

The effectiveness of the epoxy injection technique for the repair of reinforced concrete beam-column connections damaged by cyclic loading was also investigated by French *et al.* (1990), Karayannis *et al.* (1998). They concluded that the epoxy injection technique is an effective method to repair earthquake damage of beam-column joints. They also found out that bond between reinforcement and concrete in the joint region was restored by this repair procedure.

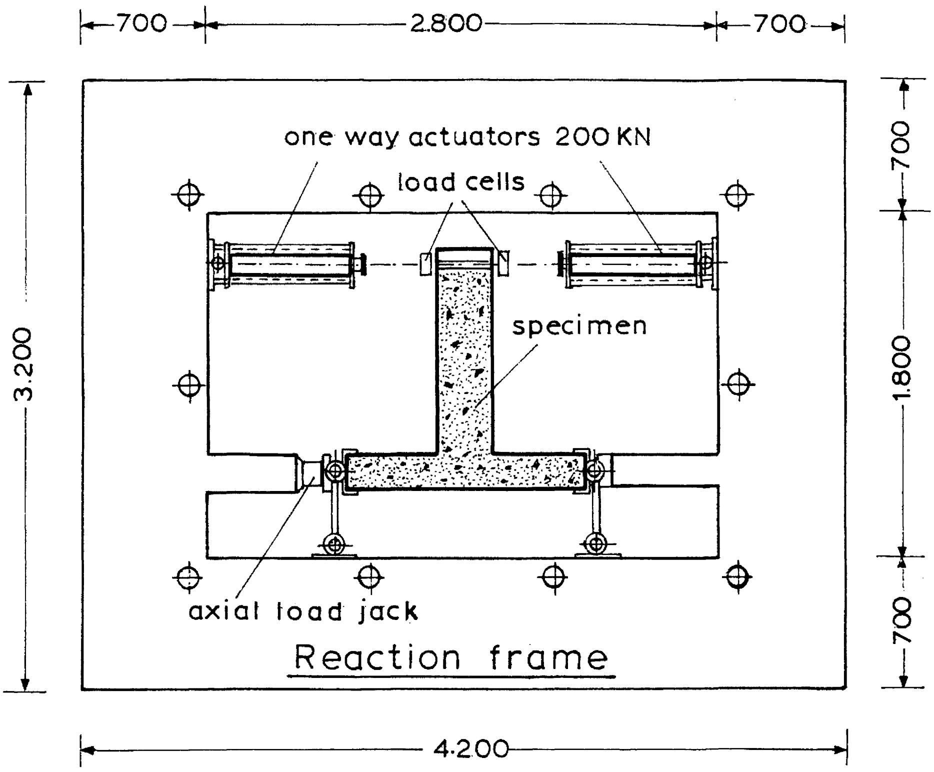
Gulkan (1977), Corazao *et al.* (1988), Alcocer and Jirsa (1990), Paultre and Mitchel (1990), Mitchel (1995), Tsonos (1999) showed that beam-column joint specimens strengthened by jacketing exhibited higher strength, greater stiffness and better energy dissipation capacity than the as-built specimens. However, Gulkan (1977) and Corazao *et al.* (1988) concluded that the beam-column joint is undoubtedly the most difficult structural member to strengthen because of the great number of elements, connected in this region.

Alcocer and Jirsa (1990) and Tsonos (1999) assessed experimentally the behaviour of beam-column subassemblages strengthened by jacketing where the joint was confined with collar stirrups. They found that this joint reinforcement provides adequate confinement and shear capacity to the joint.

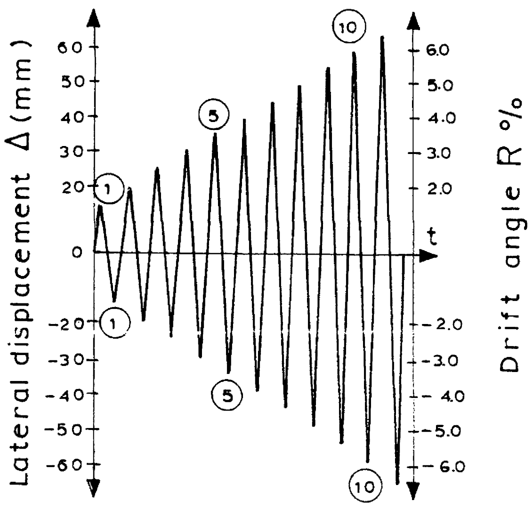
An important work in this field was recently published by Hakuto *et al.* (2000). Test results from poorly detailed reinforced concrete beam-column joints retrofitted by jacketing with new reinforced concrete are reported in this work. They concluded that: "The jacketing of beam-column joints with new reinforced concrete was identified as a useful technique for enhancing the stiffness, strength and ductility of poorly detailed as-built beam-column joint regions. The technique, however, is very labour-intensive and the placement of the new joint core hoops, passing through holes to be drilled in the existing beams is difficult".

## EXPERIMENTAL SET - UP LOADING SEQUENCE

Each specimen was tested before and after repair or strengthening, under reverse cyclic loading in the Laboratory of Reinforced Concrete Structures at the Aristotle University of Thessaloniki. The general arrangement of the experimental set-up is shown in Figure 1(a). All specimens (before and after repair and/or strengthening) were subjected to a large number of cycles applied by slowly displacing the beam's free end, according to the load history shown in Figure 1(b), without reaching the actuator stoke limit. The amplitudes of the peaks in the displacement history were 15 mm, 20 mm, 25 mm, 30 mm, 35 mm, 40 mm, 45 mm, 50 mm, 55 mm, 60 mm and 65 mm, corresponding to drift angle R of up to 6.5%. One loading cycle was performed at each displacement amplitude. An axial load approximately equal to 045$P_b$ ($P_b$: balanced column load) was applied to the columns of the subassemblages and kept constant throughout the test. Approximately 10 electrical-resistance strain gauges were bonded in the reinforcing bars of each specimen of the programme.




(a). Test setup (dimensions in mm)




(b). Loading sequence

***Figure 1. Test setup and loading sequence.***

## DESCRIPTION OF ORIGINAL TEST SPECIMENS – MATERIAL PROPERTIES

Five half-scale exterior beam-column subassemblages were designed and constructed. The first two specimens (specimens $A_1$ and $E_1$) incorporated full seismic details. The purpose of these specimens $A_1$ and $E_1$ was to represent typical construction configurations for beams, columns and beam-column joints used in modern-day buildings. More specifically, specimen $A_1$ was designed according to the requirements of ACI 318-95 (1995) and ACI-ASCE Committee 352 (1985) and specimen $E_1$ was designed according to the requirements of Eurocode 2 (1993) and Eurocode 8 (1994).

The reinforcement details of specimens $A_1$ and $E_1$ are shown in Figure 2. The subassemblages $A_1$ and $E_1$ can be expected to fail in flexure and, more specifically, to develop flexural hinges in the beams without severe damage concentration in the joint region.

The other three specimens were typical of existing structures in Greece built before 1984. ACI-ASCE Committee "Recommendations for Design of Beam-Column Joints in Monolithic Reinforced Concrete Structures (ACI 352R-85)" specifies the maximum allowable joint shear stress factor $\gamma$ is a function of the joint type (i.e. interior, exterior, etc.) and of the severity of the loading, and $f'_c$ which is the concrete compressive strength. Lower limits of the flexural strength ratio, $M_R$, and joint transverse reinforcement are also specified by this Committee. For the beam-column connections examined in this investigation, the lower limits of $M_R$ and $\gamma$ are 1.40 and 1.0 respectively, where $M_R$ is the sum of the flexural capacity of columns to that of the beam.

As seen in Figure 2(c), the joint transverse reinforcement of specimens $F_1$, $F_2$ and $L_1$, did not satisfy the requirements of the ACI-ASCE Committee: $s_h = 70$ mm $> s_{h(required)} = 200/4 = 50$ mm ($A_{sh} \cong A_{sh(required)} = 90$ mm$^2$). The values of the joint shear stress factor were greater than 1.0 for both the specimens $F_1$ and $L_1$, whereas the values of flexural strength ratio were less than 1.40 for both the specimens $F_2$ and $L_1$, see Fig. 2(c). Thus, the beam-column connections of the original specimens can be expected to fail in shear.

The concrete 28-day compressive strengths of specimens $A_1$, $E_1$, $F_1$, $F_2$ and $L_1$ were 35 MPa, 35 MPa, 20 MPa, 31 MPa and 34 MPa, respectively.

### Failure mode of the original subassemblages

***Specimens $A_1$ and $E_1$***: Failure mode of specimens $A_1$ and $E_1$, as expected, involved the formation of a plastic hinge in the beam near the column juncture. The formation of a plastic hinge caused severe cracking of the concrete near the fixed beam end of each subassemblage. The behaviour of the original specimens $A_1$ and $E_1$ was as expected and as documented in the seismic design philosophy of the modern codes (ACI-318 (1995), EC2 (1993), EC8 (1994)). Significant inelastic deformations occurred in the beams' longitudinal reinforcement in both specimens (strains of over 40.000$\mu\varepsilon$ were obtained in the beams' longitudinal bars), while the shear mechanisms of their joints remained in the elastic range. Figure 4 shows the strain gauges data of hoop reinforcement of the joint regions of both subassemblages $A_1$ and $E_1$. As is clearly shown in Figure 4, the maximum strain recorded in the joint hoop reinforcement was below 2500$\mu\varepsilon$.

One difference between the failure modes of specimens $A_1$ and $E_1$ was that hairline cracks appeared in the joint region of $E_1$, and partial loss of the concrete cover in the rear face of the joint of $E_1$ took place during the three last cycles of loading ($9^{th}$, $10^{th}$ and $11^{th}$), while the joint region of subassemblage $A_1$ was intact at the conclusion of the test (see Fig. 3).

***Specimens $F_1$ and $L_1$***: The specimens $F_1$ and $L_1$ failed by yielding of the joint ties during their first cycles of loading. A shear failure occurred, as expected, in the joint region of both the original structures $F_1$ and $L_1$. This shear failure occurred before the formation of plastic hinges in their beams. The maximum strain recorded in the longitudinal bars of the beams was below 2500$\mu\varepsilon$. The longitudinal column reinforcement of specimen $F_1$, consisting of ∅14 bars, was not buckled in the joint region. On the contrary, the longitudinal column reinforcement of specimen $L_1$, consisting of ∅10 bars, was significantly buckled in the joint region (see Fig. 3). The beams in both specimens $F_1$ and $L_1$ remained intact at the conclusion of the tests (Fig. 3).

***Specimen $F_2$***: Specimen $F_2$ developed a flexural hinge in its beam. Damage occurred both in this region and on the beam-column joint (Fig. 3).

Strains of over 40.000$\mu\varepsilon$ were obtained in the beam longitudinal bars of $F_2$. Yielding of the joint ties of $F_2$ was recorded after the $5^{th}$ cycle of loading.

### UNIDO and EC8: Part 1-4 repair and strengthening techniques, specimens $RA_1$, $RE_1$, $RF_1$, $RF_2$ and $RL_1$

The repair procedure applied to specimens $A_1$ and $E_1$ included the following operations:

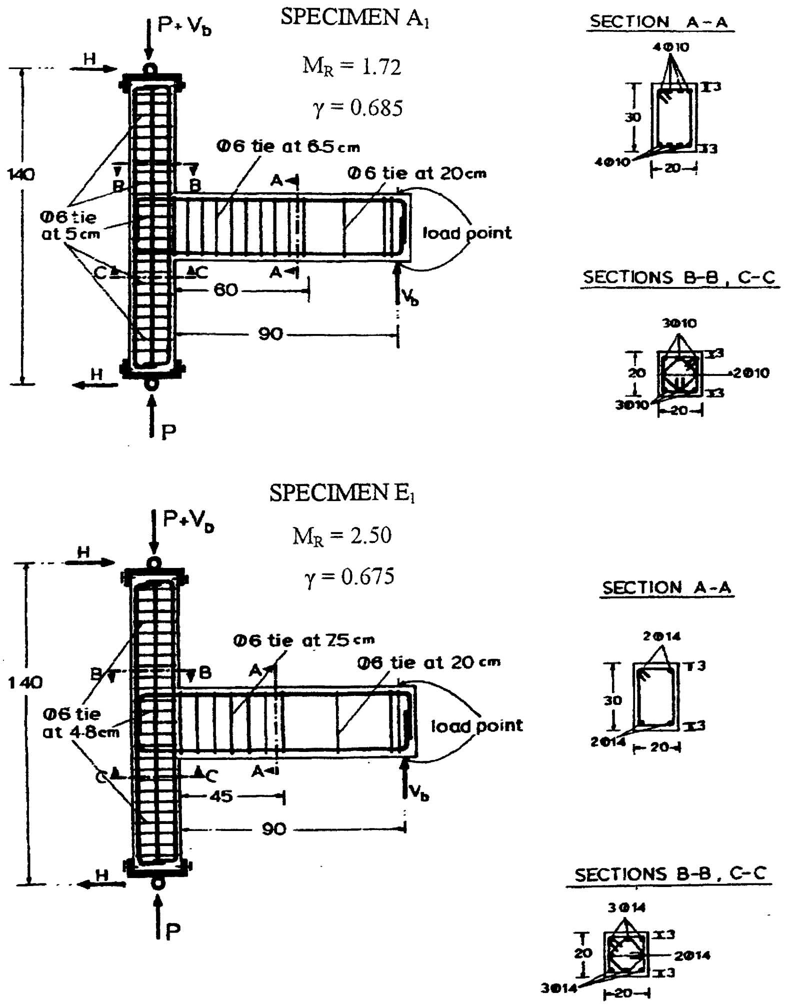
- Removal and replacement of the crushed and loose concrete in the beam near the fixed end of both specimens $A_1$ and $E_1$ by a premixed, non-shrink, rheoplastic, flowable and non-segregating mortar of high strength with 9.5 mm maximum size aggregate.

- Removal and replacement of the spalling and loose concrete cover of the rear face of the joint of specimen $E_1$ by a thick layer of epoxy resin paste.

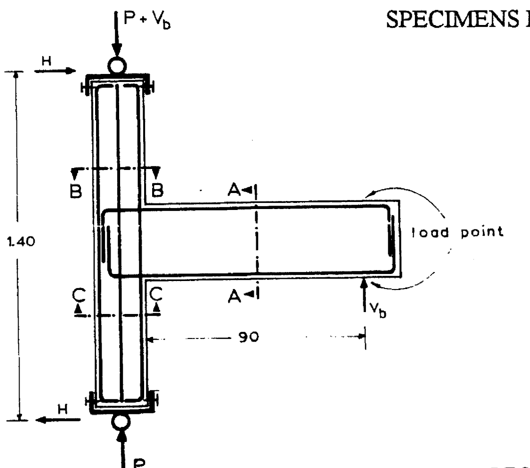
- Superficial sealing of all visible cracks with a thick layer of epoxy resin paste except for plastic inserts located along the cracks which serve as ports allowing inlet access for thin epoxy resin to be injected into the system and outlet access for air to escape from the crack voids.

(a)

(b)




(c)




| Specimen | $F_1$ | $F_2$ | $L_1$ |
|---|---|---|---|
| $A_{sb}$ | 3∅14 | 2∅12+2∅10 | 4∅14 |
| $A_{sc}$ | 3∅14 | 2∅10 | 2∅10 |
| $A_{sv}$ | 2∅14 | 4∅10 | 4∅10 |
| $M_R$ | 1.80 | 1.30 | 0.72 |
| $\gamma$ | 1.36 | 0.9 | 1.40 |

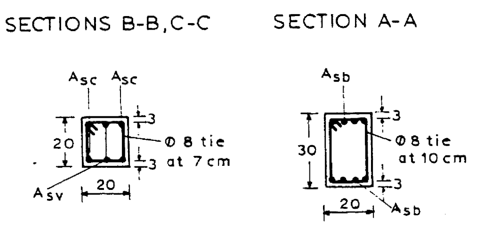



*Figure 2. Dimensions and cross-sectional details of original specimens $A_1$, $E_1$, $F_2$ and $L_1$ (dimensions in cm).*

- Injections under pressure of thin epoxy resin into the crack system of the damaged area of the joints until total fill-up. The whole infusion procedure requires special care in order to avoid local air entrapment.

- After the hardening period of the high-strength mortar used for the replacement of crushed concrete of the beams of both subassemblages $A_1$ and $E_1$, the resin injection procedure was also applied to restore the crack system of the damaged area of both specimens around the added high-strength mortar.

- The repaired specimens remained unloaded during the period of resin hardening (for at least seven days).

The highest percentage of crushed concrete in the joint region of specimen $F_1$ was lost during the tests, as is clearly shown in Figure 3. The longitudinal column reinforcement of this specimen, consisting of ∅14 bars, was not buckled in the joint regions.

Repair was performed as follows: The remaining concrete in the joint regions and the damaged concrete cover of part of the columns' critical regions was removed with a chipping hammer. The concrete surfaces were cleaned of dirt and dust. The same pre-mixed mortar of high strength with 9.5 mm maximum aggregate size, which was used for the repair of the beams of specimens $A_1$ and $E_1$, was also used for the replacement of the crushed concrete in the joint region and of the loose cover of the columns' critical regions of $RF_1$ (Fig. 5(a)). The values of the flexural strength ratio of $RF_1$ were higher than 1.40 and the values of joint shear stress were lower than $1.0\sqrt{f_c'}$ MPa, (Specimens $A_1$ ($RA_1$), $E_1$ ($RE_1$) and $F_1$ ($RF_1$) are first reported in earlier publications (Tsonos (2001, 2002)) and are reported here again in order to present all the repair and strengthening techniques for beam-column joints according to UNIDO (1983) and according to Eurocode 8: part 1-4 (1995)).

Although, it is strongly recommended in the UNIDO Manual that columns and beam-column joints be jacketed on all four sides for the optimum performance in future earthquakes, it also gives examples of three-sided or two-sided jackets of columns and beam-column joints. These types of jackets are inevitable when there are adjacent structures abutting the original building to be strengthened from one or more sides. Thus, it was considered worthwhile to investigate the seismic performance of exterior reinforced concrete subassemblages upgraded by three-sided jackets. It is worth noting that the strengthened beam-column joint subassemblages in the literature were all four-sided jackets.

The removal and replacement of crushed and loose concrete in the beam near the fixed end of specimen $F_2$, by the aforementioned mortar of high strength was carried out. The beam of specimen $L_1$ was intact at the conclusion of the tests (Fig. 5(b)).

A strengthening technique has been applied to the subassemblages, consisting of both raising the reinforcement quantity to minimum levels and covering the whole joint area and parts of the critical regions of the columns of the specimens with a three-sided cement grout jacket reinforced with additional ties (Fig. 5(b)). The additional longitudinal reinforcement was placed at each corner of the jacket, which was then welded to the existing column reinforcement. The existing longitudinal column reinforcement of specimens $F_2$ and $L_1$ consisting of ∅10 bars was significantly buckled in the joint region (Fig. 3). Thus, additional column reinforcement was needed to replace the buckled reinforcement. This additional longitudinal column reinforcement was also needed in order to increase the flexural strength ratios of specimens $F_2$ and $L_1$, and especially that of specimen $L_1$ ($M_R(F_2) = 1.30 < 1.40$, $M_R(L_1) = 0.72 << 1.40$, Fig. 2). The flexural strength ratios of strengthened specimens $RF_2$ and $RL_1$ were significantly higher than those of the original specimens $F_2$ and $L_1$, respectively (Fig. 5(b)).

The same pre-mixed mortar of high strength with 9.5 mm maximum size of aggregate was used for the construction of the cement grout jacket of specimens $RF_2$ and $RL_1$.

As shown in Fig. 5(b) both specimens $RF_2$ and $RL_1$ had the same three-sided cement grout jacket plus ∅10 longitudinal bars at each corner of the column connected by ∅8 supplementary ties at 70 mm centres. All longitudinal bars in the jackets extended into the beam-column region of the subassemblages.

Both the original and repaired and/or strengthened subassemblages were constructed using deformed reinforcement. All (both the original and the repaired or strengthened) specimens' steel yield stress can be summarised as follows: ∅8 = 520 MPa, ∅10 = 535 MPa, ∅12 = 530 MPa and ∅14 = 520 MPa. (Note: ∅8, ∅10, ∅12 and ∅14 = bar with diameter 8 mm, 10 mm, 12 mm and 14 mm).

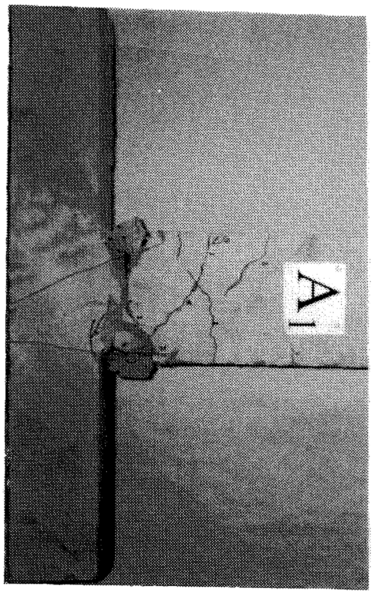
The compressive strengths of the high strength mortar used for the removal and replacement of crushed concrete of specimen $RF_1$ and for the construction of the jackets of specimens $RF_2$ and $RL_1$ were 70 MPa, 68 MPa and 66 MPa, respectively. Electrical-resistance strain gauges were bonded in the reinforcing bars within the joint region of both the original and repaired and/or strengthened subassemblages.

**Additional joint transverse reinforcement**

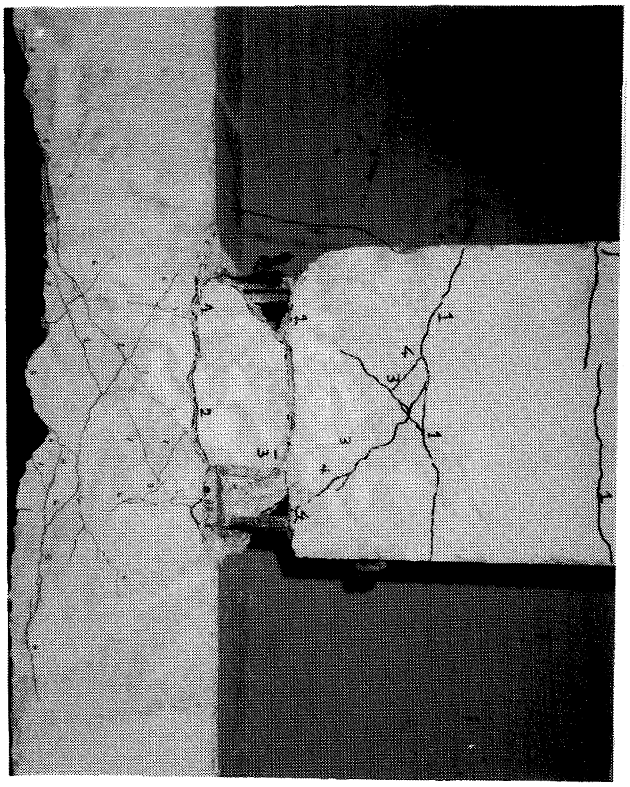
Two additional horizontal ties were placed in the joint region of specimens $RF_1$, $RF_2$ and $RL_1$ in order to increase their shear strength (Fig. 5(c).

The values of the flexural strength ratio were higher than 1.40 and those of the joint shear stress were lower than $1.0\sqrt{f_c'}$ MPa for all the repaired and strengthened specimens $RA_1$, $RE_1$, $RF_1$, $RF_2$ and $RL_1$ (Figures 2 and 5). The joint transverse reinforcement of specimens $RF_1$, $RF_2$ and $RL_1$ with the two additional ties was ∅8 at 35 mm centres. This reinforcement satisfied the requirements of the Committee (ACI 352R-85): $s_h = 50$ mm = 200 mm/4 ($A_{sh} \cong A_{sh(required)} = 90$ mm$^2$).

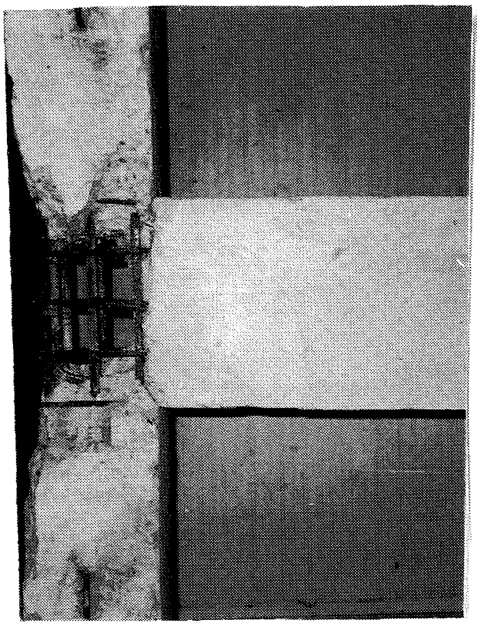
The provision of transverse reinforcement, made of short bars placed and connected under the bends of a group of rebars, was made to ensure the anchorage of the beam bars in the joint region (Eurocode 8 – 1994, see Fig. 5).



Specimen $A_1$

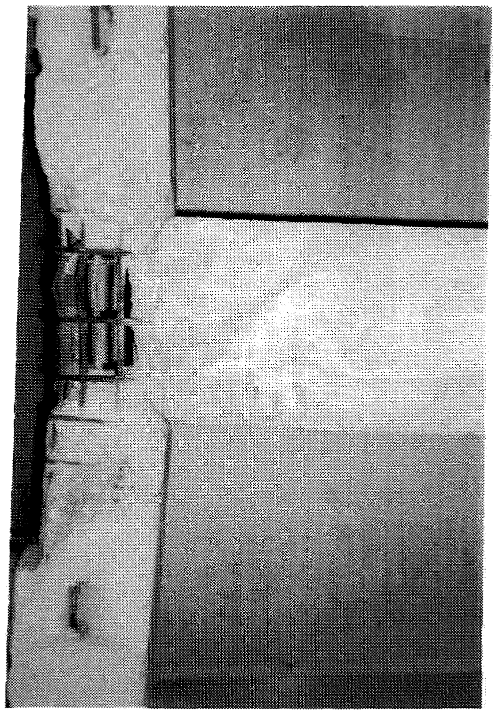


Specimen $E_1$



Specimen $F_1$

Specimen $F_2$



Specimen $L_1$

***Figure 3. Views of the failed subassemblages $A_1$, $E_1$, $F_2$ and $L_1$.***

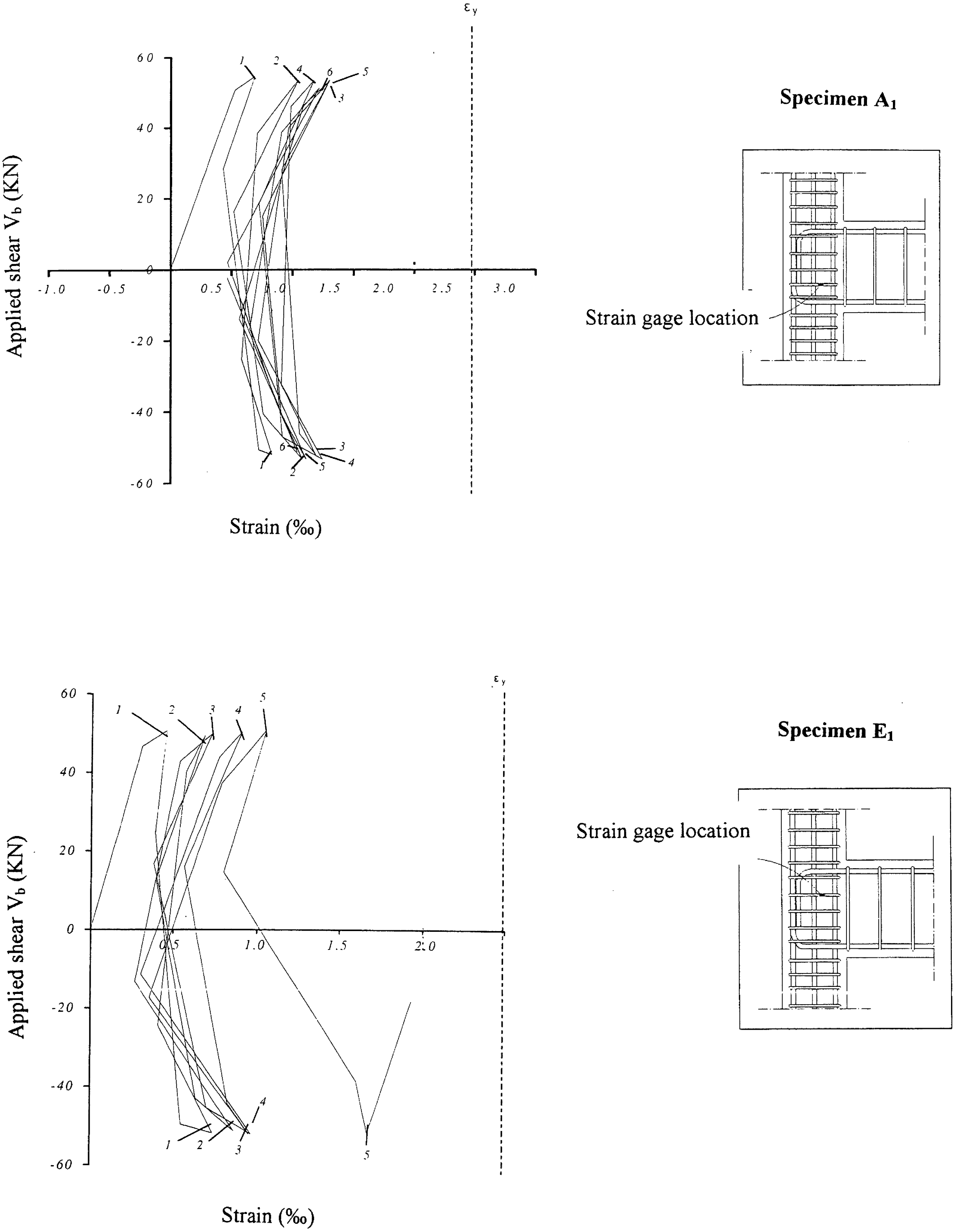
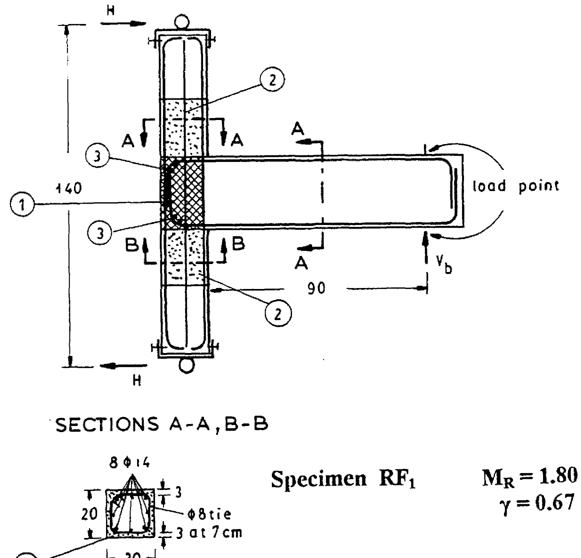



*Figure 4. Applied shear-versus strain in beam-column joint hoop reinforcement of the subassemblages $A_1$ and $E_1$.*

(a)



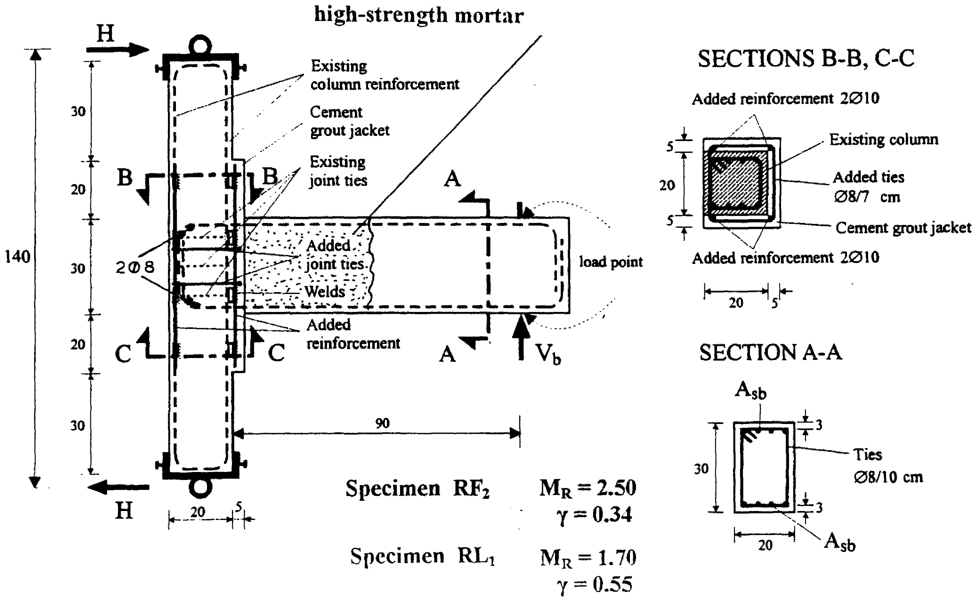

① removal and replacement of the crushed concrete in the joint region with a pre-mixed, non-shrink, rheoplastic, flowable and non-segregating high-strength mortar

② removal and replacement of the damaged concrete cover of part of the columns' critical regions with the same mortar

③ the provisions of transverse reinforcement, made of short bars placed and tightly connected under the bends of a group of rebars, was made to ensure the anchorage of the beam bars in the joint region

removal and replacement of the crushed concrete of part of the beam's critical region of specimen $RF_2$ with a pre-mixed, non-shrink, rheoplastic, flowable and non-segregating high-strength mortar

(b)




(c)

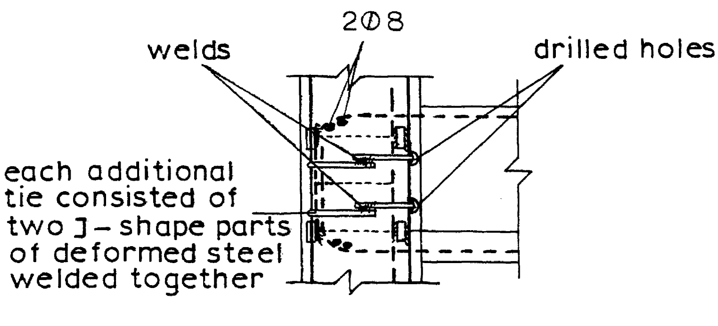



*Figure 5. (a) Details of repaired specimen $RF_1$ (dimensions in cm), (b) Jacketing of beam-column connections of subassemblages $RF_2$ and $RL_1$ (dimensions in cm) and (c) Additional joint ties of specimens $RF_1$, $RF_2$ and $RL_1$.*

The strengthened subassemblages could, therefore, be expected to fail in flexure and, more specifically, to develop flexural hinges in the beams without severe damage concentration in the joint regions.

## BEHAVIOUR OF REPAIRED AND STRENGTHENED SPECIMENS

### Failure modes of the repaired and/or strengthened subassemblages $RA_1$, $RE_1$, $RF_1$, $RF_2$ and $RL_1$

Both specimens $RA_1$ and $RE_1$ repaired by epoxy injections exhibited similar failure modes, nearly identical to that of the original specimen $A_1$. Thus, the failure mode of both the repaired specimens $RA_1$ and $RE_1$ involved the formation of a plastic hinge in the beam near the column juncture and damage concentration in this region only, see Figure 6. In both specimens the rupture of some longitudinal beam reinforcing bars in the plastic hinge region took place during their last two cycles of loading ($10^{th}$ and $11^{th}$ for $RE_1$, $8^{th}$ and $9^{th}$ for $RA_1$). It is obvious that the failure mode of the repaired specimen $RE_1$ was better than that of the original specimen $E_1$.

Failure mode of specimens $RF_1$, $RF_2$ and $RL_1$, as expected, also involved the formation of a plastic hinge in the beam near the column juncture and damage concentration in this region only. During the final cycles of loading of specimens $RF_1$, $RF_2$ and $RL_1$ when large displacements were imposed, the damaged concrete cover could not provide adequate support for the beam longitudinal reinforcement. As a result, buckling of the beam reinforcement in specimens $RF_1$, $RF_2$ and $RL_1$ occurred after the $6^{th}$, $6^{th}$ and $9^{th}$ cycles of loading, respectively.

Strains of over 40.000$\mu\varepsilon$ were obtained in the beam longitudinal bars of all the strengthened specimens $RA_1$, $RE_1$, $RF_1$, $RF_2$ and $RL_1$. The maximum strain recorded in the joint hoop reinforcement of all the repaired and strengthened specimens was below 2500$\mu\varepsilon$. In Figure 7 are demonstrated strain gauge data of the hoop reinforcement of the joint regions of specimens $RA_1$, $RE_1$ and $RF_1$.

The three-sided jacketing of beam-column joints is more critical than the four-sided jacketing, especially in the rear face of the joint along the column, where the hooked ends of the beam longitudinal reinforcement move outward to split the cover. As is clearly demonstrated in Figure 6, the rear faces of both specimens $RF_2$ and $RL_1$ were intact at the conclusion of the tests. It is worth noting that the rear face of specimen $RF_1$ repaired by the removal and replacement technique was also intact at the conclusion of the tests (see Fig. 6).

The beam in the subassemblage $RF_2$ was repaired using the removal and replacement technique and was much stronger than the original one. It is worth noting that the increase in beam strength due to the use of high strength repair mortar did not result in damage moving from the beam to the strengthened joint and column of specimen $RF_2$. As is clearly shown in Figure 6, both the column and the joint of specimen $RF_2$ were intact at the end of the tests.

### Load drift angle curves

The performance of the test specimens is presented herein and discussed in terms of applied shear-versus-drift angle relations. Drift angle R, which is plotted in the figures that follow, is defined as the beam tip displacement $\Delta$ divided by the beam half span L, and is expressed as a percentage (see inset on Fig. 8). Plots of applied shear-versus-drift angle for all the specimens $A_1$, $RA_1$, $E_1$, $RE_1$, $F_1$, $RF_1$, $F_2$, $RF_2$, $L_1$ and $RL_1$ are shown in Figure 8.

The original beam-column specimens $A_1$, $E_1$, $F_1$, $F_2$, and $L_1$ showed stable hysteretic behaviour up to drift angle R ratios of 4.0 percent, 4.0 percent, 2.0 percent, 4.0 percent and 2.0 percent respectively (Fig. 8). They showed a considerable loss of strength, stiffness and unstable degrading hysteresis beyond drift angle R ratios of 4.5 percent, 2.0 percent, 4.5 percent and 2.0 percent respectively (Fig. 8).

The repaired specimens $RA_1$ and $RE_1$ exhibited stable hysteresis up to $6^{th}$ cycle of drift angle R, of 4.5 percent, after which a significant loss of strength began due to noticeable buckling of the beam reinforcement (Fig. 8(a)). The extreme loss of strength, stiffness and energy dissipation capacity observed in specimen $RA_1$ during $8^{th}$ and $9^{th}$ cycle of loading, and in specimen $RE_1$ during the $11^{th}$ cycle of loading, was due to the fracture of almost half of the beam reinforcing bars during these cycles of loading.

The repaired specimen $RF_1$ and the strengthened specimen $RF_2$ exhibited stable hysteresis up to the $6^{th}$ cycle of drift angle R, of 4.0 percent, after which a significant loss of strength began, due to the noticeable buckling of the beam reinforcement (Fig. 8(b)).

The strengthened specimen $RL_1$ showed stable hysteresis up to the $9^{th}$ cycle of drift angle R of 5.5 percent, after which also a serious loss of strength began, due to the remarkable buckling of the beam reinforcement (Fig. 8(b)).

### Comparison of the strength, stiffness and energy dissipation capacity between the original $A_1$, $L_1$ and repaired and/or strengthened $RA_1$, $RL_1$ subassemblages respectively.

For a further evaluation of the effectiveness of the UNIDO 1983 and EC8: part 1-4 (1995) repair and strengthening techniques in restoring and increasing the strength, stiffness and energy dissipation capacity of the damaged subassemblies, it is interesting to compare the peak-to-peak stiffness, the energy dissipated and the peak strength observed for every load cycle of the original specimens $A_1$ and $L_1$ with those of the repaired and strengthened specimens $RA_1$ and $RL_1$.

The peak-to-peak stiffness and energy dissipated for every load cycle of each specimen $A_1$, $L_1$, $RA_1$ and $RL_1$ are illustrated in Fig. 9(b) and Fig. 9(c), respectively.

Figure 9(a) compares the peak strength observed throughout the tests. The comparison is made by observing the ratio of the peak strengths of the repaired and strengthened subassemblages $RA_1$ and $RL_1$, to that of the original subassemblages $A_1$ and $L_1$.

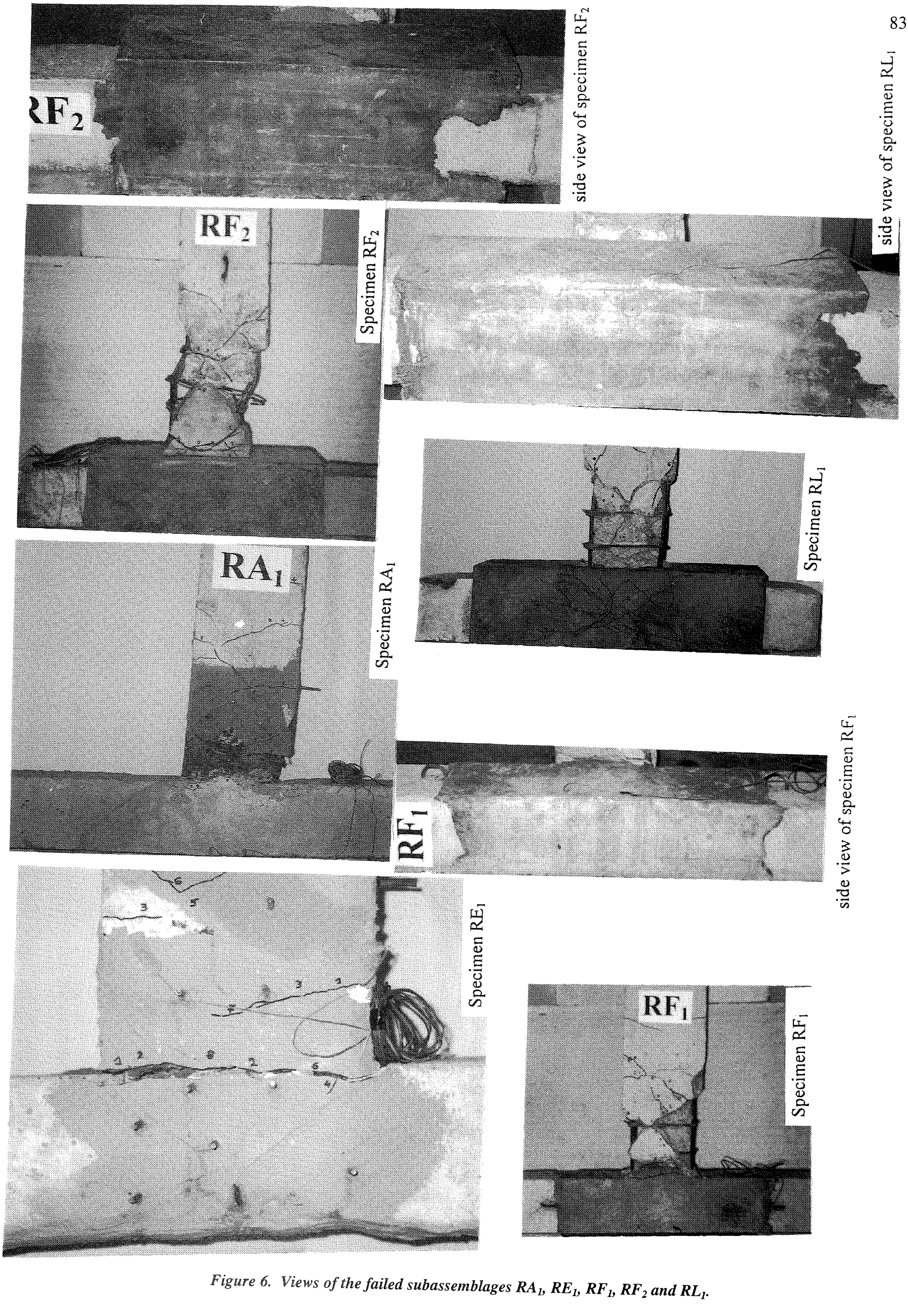



*Figure 6. Views of the failed subassemblages $RA_1$, $RE_1$, $RF_1$, $RF_2$ and $RL_1$.*

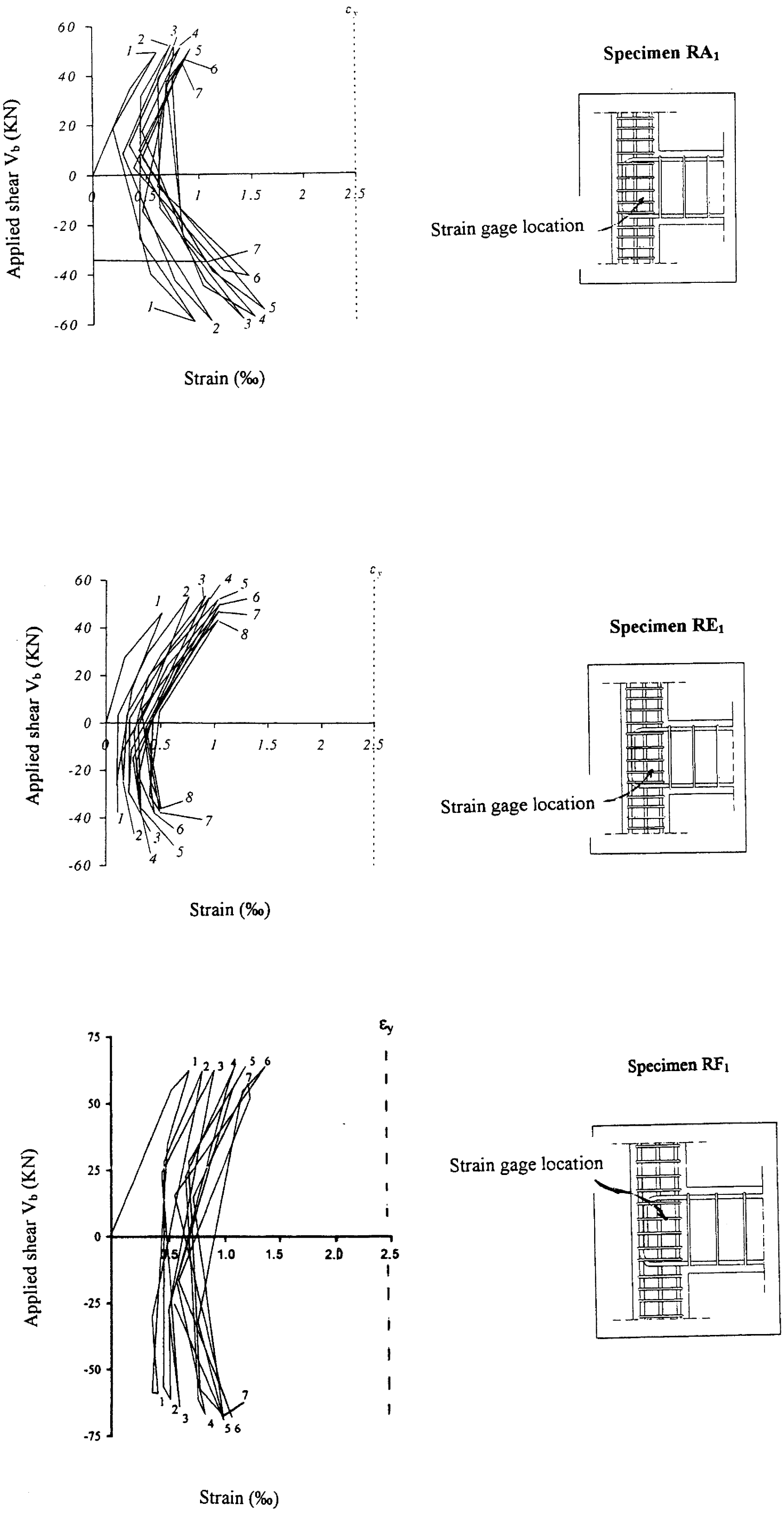



***Figure 7. Applied shear-versus-strain in beam-column joint ties of specimens $RA_1$, $RE_1$, and $RF_1$.***

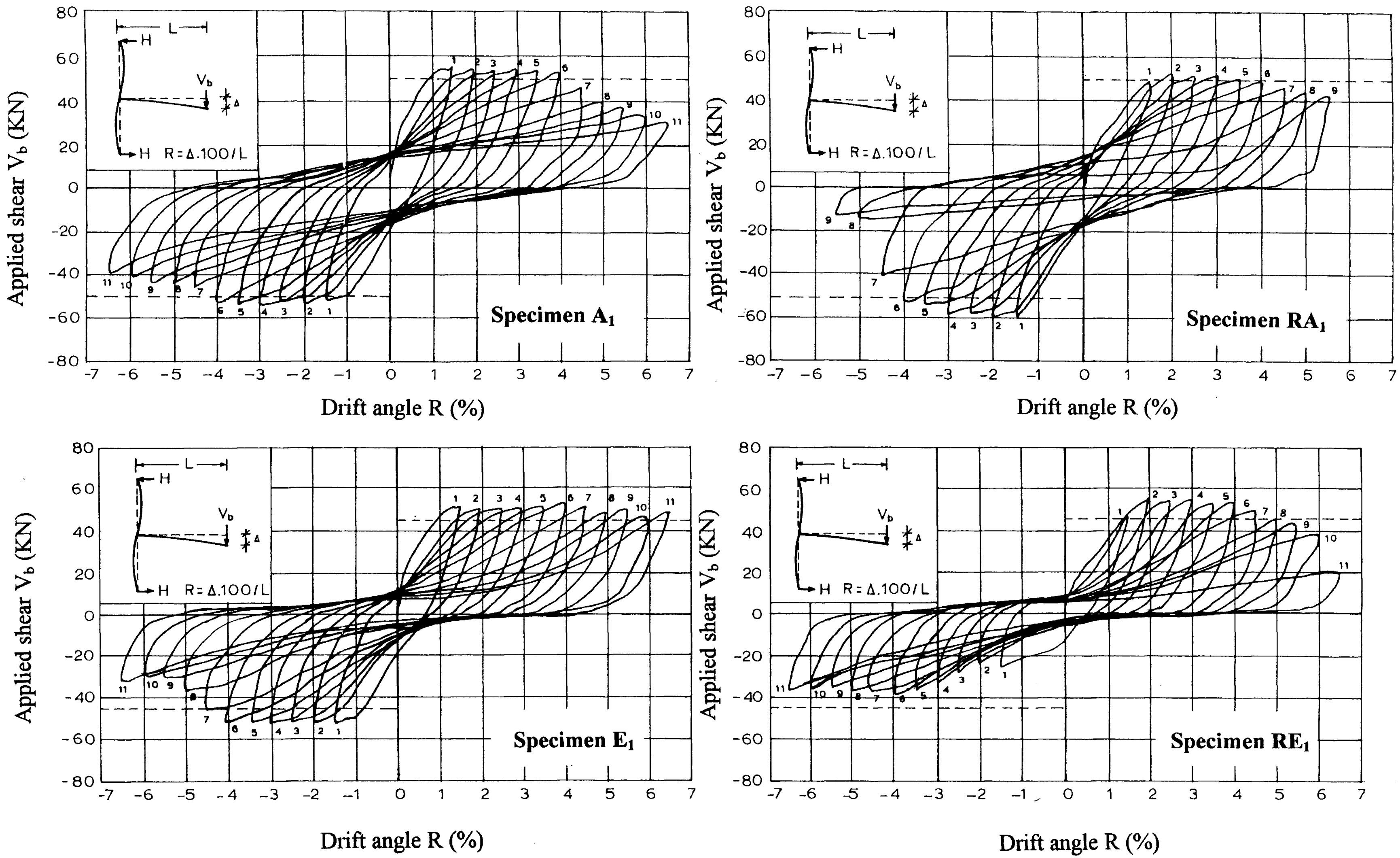



*Figure 8(a). Hysteresis loops of specimens $A_1$, $RA_1$, $E_1$ and $RE_1$.*

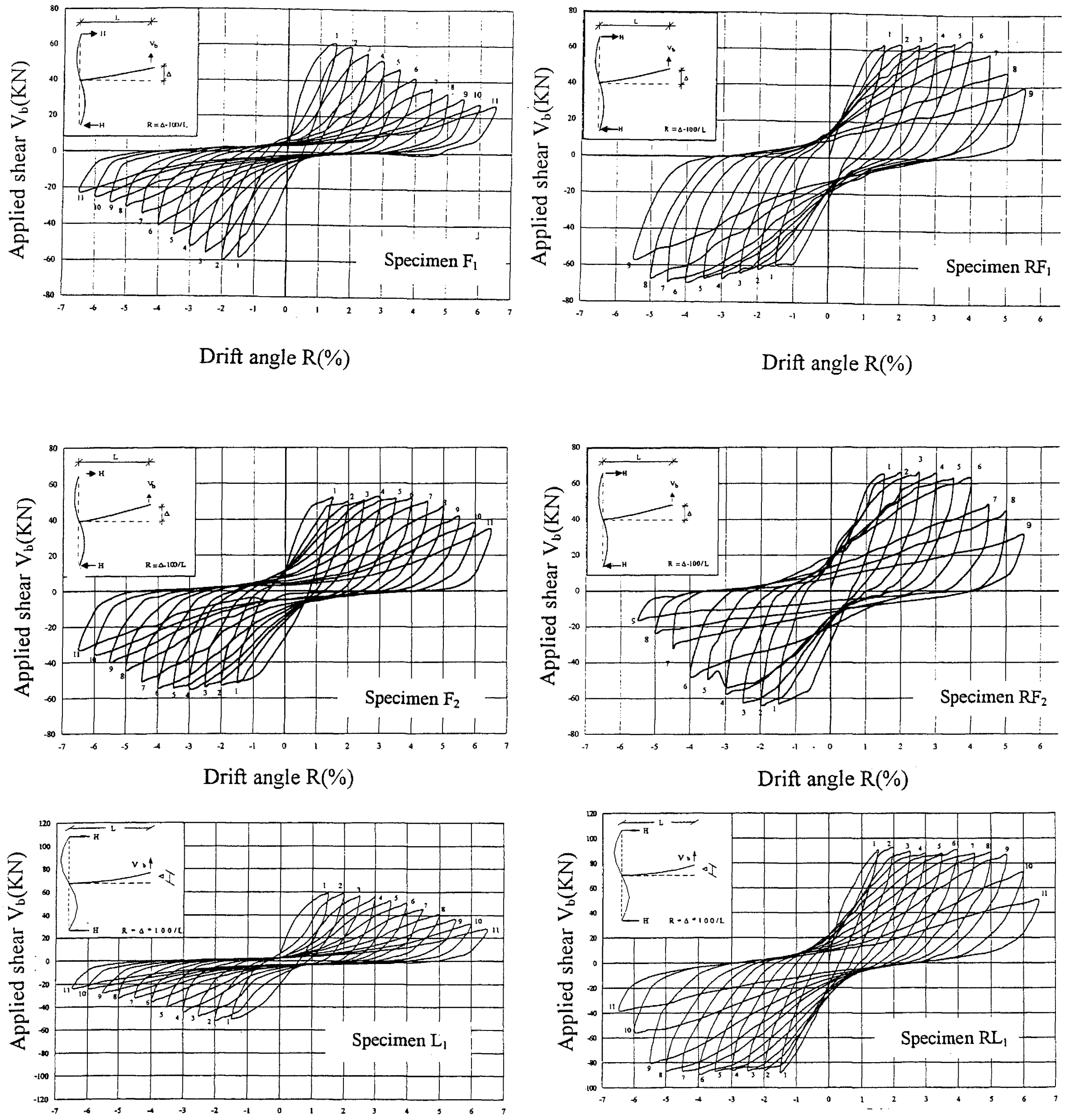



*Figure 8(b). Hysteresis loops of specimens $F_1$, $RF_1$, $F_2$, $RF_2$, $L_1$ and $RL_1$.*

From these diagrams the effectiveness of the examined epoxy resin repair technique in restoring satisfactorily the strength, stiffness and energy dissipation capacity of the repaired specimen $RA_1$ is shown. Thus, the repaired subassemblage achieved almost the same strength, stiffness and energy dissipation capacity as compared with those of the original subassemblage $A_1$ (see Figures 9(a), 9(b) and 9(c)).

From these diagrams, it is also clear that the strengthened specimen $RL_1$ achieved significantly increased strength, stiffness and energy dissipation capacities as compared with those of the original specimen $L_1$, even in the large displacement amplitude cycles of drift angle R between 5.5 percent and 6.5 percent.

## THEORETICAL CONSIDERATIONS

Figure 10(a) shows a reinforced concrete exterior beam-column joint for a moment resisting frame. Park and Paulay (1975) on the basis of experimental findings postulated the existence of two shear mechanisms in R/C beam-column joints. Thus, according to the approach of Park and Paulay, the total shear within the joint core is carried partly by a diagonal concrete strut, formed between the corners of the joint subjected to compression [see Fig. 10(a)], and partly by an idealized truss consisting of horizontal hoops, intermediate column bars and inclined concrete struts between shear cracks (Fig. 10(a)).

Both mechanisms depend on the core concrete strength. Thus, the ultimate concrete strength of the joint core under compression/tension controls the ultimate strength of the connection. After failure of the concrete, strength in the joint is limited by the gradual crushing along the cross - diagonal cracks and especially along the potential failure planes (Fig. 10(a)).

For instance, consider the section I - I in the middle of the joint height (Fig. 10(a)). In this section, the flexural moment is almost zero. The forces acting in the concrete are shown in Fig. 10(b). $T_i$ are the forces acting in the longitudinal column bars between the corner bars in the side faces of the column. These bars compress the joint core by equal and opposite direction forces. Each force acting in the joint core is analysed into two components along the X and Y axes (Fig. 10(b)). Thus, the vertically acting forces are:

$$D_{cy} + (T_1 + \ldots + T_4 + D_{vy}) = D_{cy} + D_{sy} = V_{jv}$$

$$\downarrow \qquad\qquad \downarrow$$

compression strut truss model

where $V_{jv}$ is the vertical joint shear force (EC8-1994, NZS3101-1982).

The sum of the horizontally acting forces also gives the horizontal joint shear force as

$$D_{cx} + (D_{1x} + \ldots + D_{vx}) = V_{jh}$$

The vertical normal compressive stress $\sigma$ and the shear stress $\tau$ uniformly distributed over the section I - I are given by the equations (1) and (2) below. The stress state in the finite sized block, which is located in the middle of the potential corner to corner failure plane, is shown in Figure 10(c). The normal stress in the longitudinal direction is small and can be neglected. Thus,

$$\sigma = \frac{D_{cy} + D_{sy}}{h'_c \text{ x } b'_c} = \frac{V_{jv}}{h'_c \text{ x } b'_c} \tag{1}$$

$$\tau = \frac{V_{jh}}{h'_c \text{ x } b'_c} \tag{2}$$

where $h'_c$ and $b'_c$ are the length and the width of the joint core, respectively.

It is now necessary to establish a relationship between the average normal compressive stress $\sigma$ and the average shear stress $\tau$. From Equations (1) and (2):

$$\sigma = \frac{V_{jv}}{V_{jh}} \cdot \tau \tag{3}$$

It is well known that

$$\frac{V_{jv}}{V_{jh}} = \frac{h_b}{h_c} = \alpha \quad \text{(EC8-1994, NZS3101-1982)}, \tag{4}$$

where $\alpha$ is the joint aspect ratio. Thus,

$$\sigma = \alpha \cdot \tau \tag{5}$$

The maximum principal stresses are given by Mohr's circle (Fig. 10(c)) and the following expressions can be recovered:

$$\sigma_{I,II} = \frac{\sigma}{2} \pm \frac{\sigma}{2}\sqrt{1 + \frac{4\tau^2}{\sigma^2}} \tag{6}$$

From the diagram of the behaviour of concrete under biaxial stresses (Kupfer *et al* 1969), it was found that the branch AB could be represented by a 5th degree parabola (Tegos 1984) (Fig. 10(d)). Thus, for branch AB:

$$\boxed{-10\frac{\sigma_I}{f_c} + \left[\frac{\sigma_{II}}{f_c}\right]^5 = 1} \tag{7}$$

where $f_c$ is the increased joint concrete compressive strength due to confining, which is given by the model of Scott, Park and Priestley (1982) according to the equation

$$f_c = K \cdot f'_c \tag{8}$$

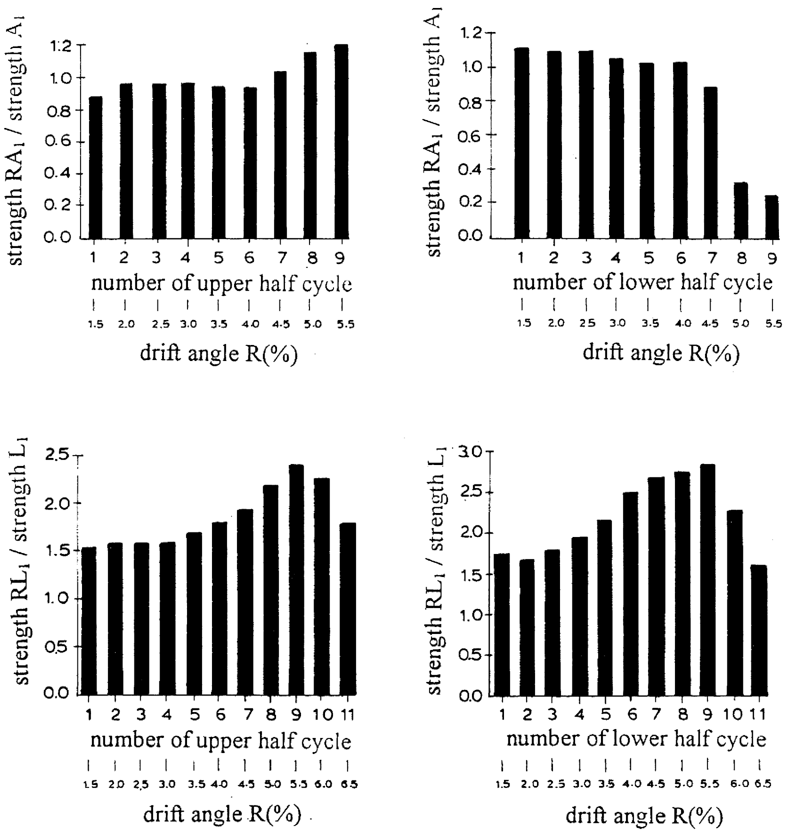



*Figure 9(a). Strength ratio of repaired or strengthened model to original models.*

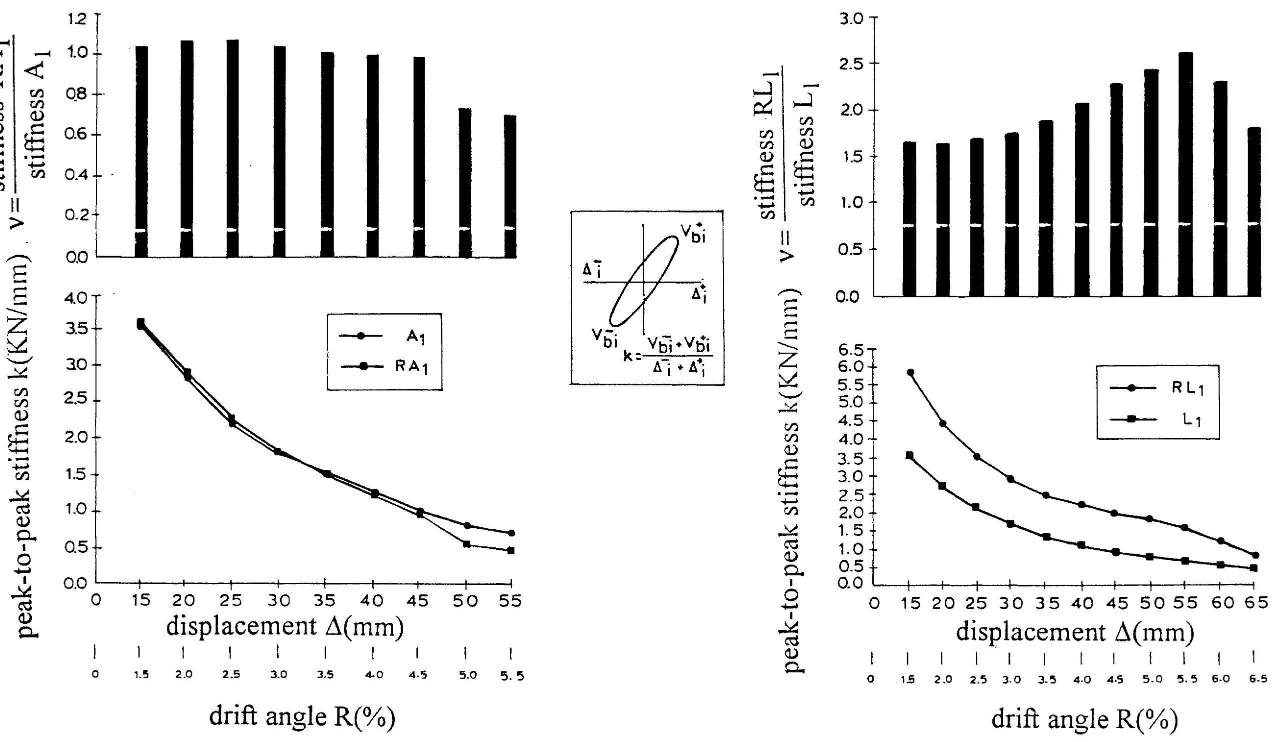
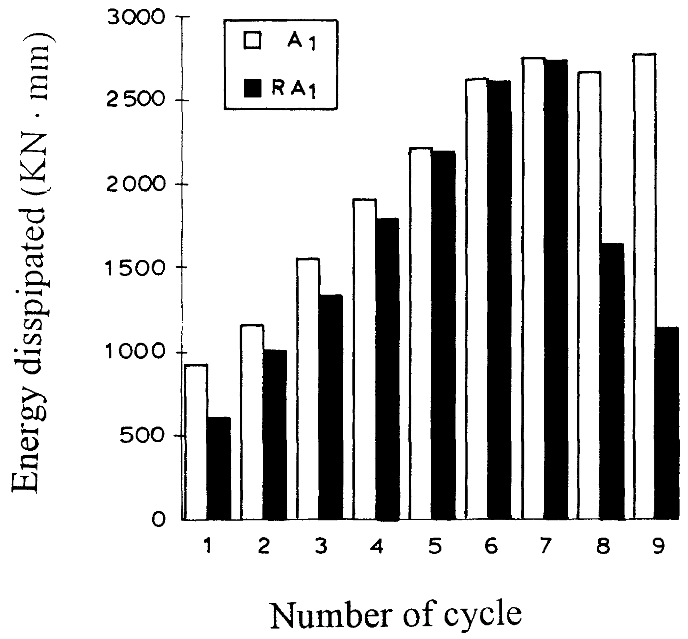



*Figure 9(b). Stiffness comparison between original $A_1$ and $L_1$, and repaired or strengthened subassemblages $RA_1$ and $RL_1$*



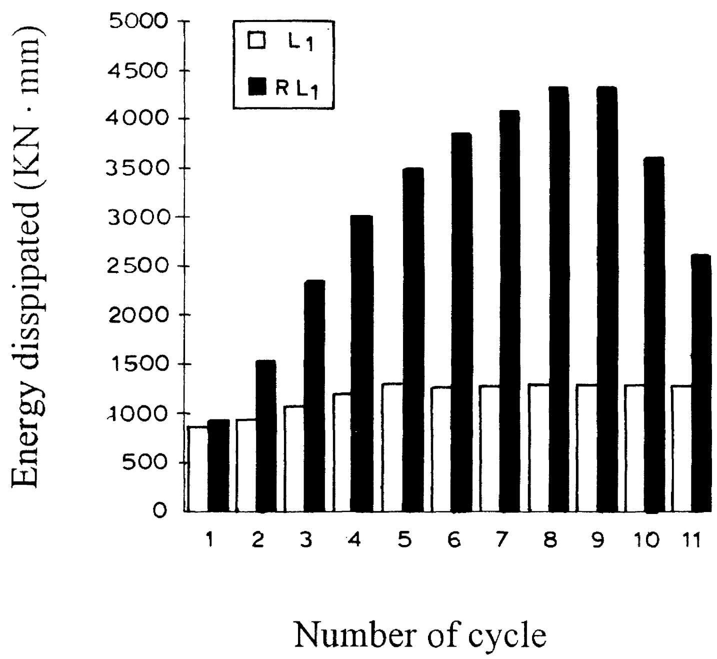




***Figure 9(c).** Energy dissipation comparison between original $A_1$ and $L_1$, and repaired or strengthened subassemblages $RA_1$ and $RL_1$.*

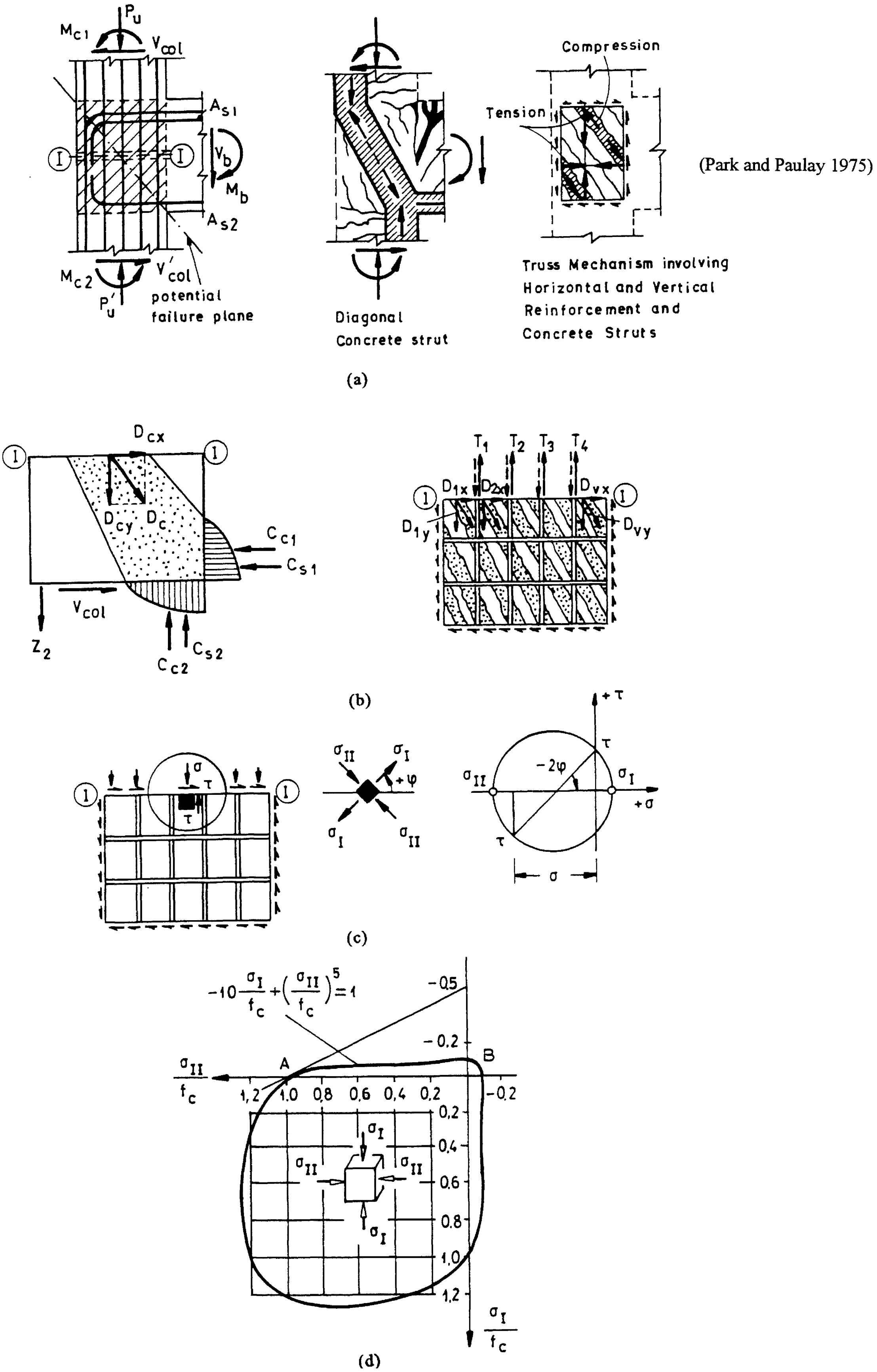



*Figure 10. (a). External beam-column connection and the two mechanisms of shear transfer (diagonal concrete strut and truss mechanism), (b). Forces acting in the joint core concrete through section I-I from the two mechanisms, (c). Stress state of element of this studied region and (d). Representation of concrete biaxial strength curve by a parabola of the $5^{th}$ degree.*

Also, $f_c'$ is the concrete compressive strength, and K is the parameter of the model (Scott *et al* 1982) and is expressed as:

$$K = 1 + \frac{\rho_s \cdot f_{yh}}{f_c'} \quad , \tag{9}$$

where $\rho_s$ is the volume ratio of transverse hoop reinforcement, and $f_{yh}$ is the yield strength of the transverse reinforcement.

Substituting Eqs. (5) and (6) into Eq. (7) and using $\tau = \gamma\sqrt{f_c}$ gives the following expression:

$$\left[\frac{\alpha\gamma}{2\sqrt{f_c}}\left(1+\sqrt{1+\frac{4}{\alpha^2}}\right)\right]^5 + \frac{5\alpha\gamma}{\sqrt{f_c}}\left(\sqrt{1+\frac{4}{\alpha^2}}-1\right)=1 \tag{10}$$

Assume here that

$$x = \frac{\alpha\gamma}{2\sqrt{f_c}} \tag{11}$$

and $$\psi = \frac{\alpha\gamma}{2\sqrt{f_c}}\sqrt{1+\frac{4}{\alpha^2}} \tag{12}$$

Expression (10) is then transformed into:

$$(x+\psi)^5 + 10\psi - 10x = 1 \tag{13}$$

The solution of the system of equations (11)-(13) gives the beam-column joint ultimate strength. This system is solved each time for a given value of the joint aspect ratio using standard mathematical analysis.

For simplicity's sake, the presentation of the above methodology was for exterior beam-column joints. The approach is the same for interior joints. For the development of the above formulation, it has been assumed that bond conditions of the beam and column bars anchoring or passing through the joint region are generally favourable.

The proposed shear strength formulation can be used to predict the failure mode of the subassemblages and thus the actual values of connection shear stress. Therefore, when the computed joint shear stress is greater or equal to the joint ultimate capacity $\gamma_{cal} \geq \gamma_{ult}$, the predicted actual value of connection shear stress will be near $\gamma_{ult}$ because the connection fails earlier than the beam(s). When the calculated joint shear stress is lower than the connection ultimate strength $\gamma_{cal} < \gamma_{ult}$, then the predicted actual value of the connection shear stress will be near $\gamma_{cal}$, because the connection permits its adjacent beam(s) to yield.

More details concerning the above formulation can be found in Tsonos (1996, 1997, 1999), where the validity of the formulation was checked using test data for 40 exterior and interior beam-column subassemblages that were tested in the Structural Engineering Laboratory at the Aristotle University in Thessaloniki, as well as data from similar experiments carried out in the United States.

The improved retention of strength in the beam-column subassemblages, as the values of the ratio $\gamma_{cal} / \gamma_{ult}$ decrease, was also demonstrated. It is worth noting that for $\gamma_{cal} / \gamma_{ult} \leq 0.50$ the beam-column joints of the subassemblages performed excellently during the tests and they remained intact at the conclusion of the tests (Tsonos (1996, 1999)).

The shear capacities of the repaired and/or strengthened beam-column connections of specimens $RA_1$, $RE_1$, $RF_1$, $RF_2$ and $RL_1$ were computed using the above methodology.

Table 1 shows that $\gamma_{cal} / \gamma_{ult}(RA_1) = 0.47 < 0.50$, $\gamma_{cal} / \gamma_{ult}(RE_1) = 0.46 < 0.50$, $\gamma_{cal} / \gamma_{ult}(RF_1) = 0.37 < 0.50$, $\gamma_{cal} / \gamma_{ult}(RF_2) = 0.21 < 0.50$ and $\gamma_{cal} / \gamma_{ult}(RL_1) = 0.35 < 0.50$. Thus, the safe formation of plastic hinge in the beams near the columns is expected without any serious damage in the joint regions and, as a result, there will be satisfactory performance for all the subassemblages $RA_1$, $RE_1$, $RF_1$, $RF_2$ and $RL_1$. As predicted, the repaired and/or strengthened specimens failed in flexure exhibiting remarkable seismic performance (Fig. 6).

In all cases, the observed capacity was predicted to within approximately 15 percent of that computed using the joint shear strength formulation (Table 1).

## CONCLUSIONS

The following conclusions are drawn based on the work presented herein.

1. The original strength, stiffness and energy dissipation capacities were restored in the subassemblages repaired by epoxy resins. The bond between reinforcement and concrete also appeared to be restored by the repair procedure. In general, the epoxy-repaired cracks did not reopen in the tests of the repaired structures, new cracks tended to develop in the concrete adjacent to the repaired cracks.

2. The beam in both subassemblages $RA_1$ and $RE_1$ were repaired using epoxy injection technique, as well as the removal and replacement technique and were stronger than the original ones. The increase in beam strength due to the use of high strength repair materials did not result in damage moving from the beam to the unrepaired joint and column of specimen $RA_1$ (as observed by Lee *et al.* 1980). Both the column and the joint of specimen $RA_1$ were intact at the conclusion of the tests.

3. Specimen $RF_1$ repaired by the Removal and Replacement Technique and specimens $RF_2$ and $RL_1$ strengthened by three-sided jacketing exhibited significantly increased strength, stiffness and energy dissipation capacities as compared with those of original specimens $F_1$, $F_2$ and $L_1$, respectively.

**Table 1. Experimental and Predicted values of the strength of repaired and/or strengthened specimens $RA_1$, $RE_1$, $RF_1$, $RF_2$ and $RL_1$.**

| Specimen | Joint aspect ratio $\alpha=h_b/h_c$ | $\gamma_{cal}$ | $\gamma_{exp}$ | $\gamma_{ult}$ | Predicted shear strength $\tau_{pred}$ (MPa) | Observed shear strength $\tau_{exp}$ (MPa) | $\mu=\frac{\tau_{pred}}{\tau_{exp}}$ | $\frac{\gamma_{cal}}{\gamma_{ult}}$ | Predicted failure mode | Observed failure mode |
|---|---|---|---|---|---|---|---|---|---|---|
| $RA_1$ | 1.50 | 0.685 | 0.584 | 1.46 | 5.05 | 4.31 | 1.17 | 0.47 | flexure | flexure |
| $RE_1$ | 1.50 | 0.675 | 0.554 | 1.46 | 5.00 | 4.10 | 1.20 | 0.46 | flexure | flexure |
| $RF_1$ | 1.50 | 0.670 | 0.600 | 1.80 | 6.03 | 5.45 | 1.10 | 0.37 | flexure | flexure |
| $RF_2$ | 1.20 | 0.340 | 0.395 | 1.58 | 3.03 | 3.52 | 0.86 | 0.21 | flexure | flexure |
| $RL_1$ | 1.20 | 0.550 | 0.520 | 1.55 | 4.83 | 4.57 | 1.05 | 0.35 | flexure | flexure |

For $\gamma_{cal} < \gamma_{ult}$ , $\gamma_{pred} = \gamma_{cal}$ (an overstrength factor a = 1.25 for the beam steel is included in the computations of joint shear stress $\tau_{cal} = \gamma_{cal}\sqrt{f'_c}$ MPa).

4. All the repaired and strengthened specimens $RA_1$, $RE_1$, $RF_1$, $RF_2$ and $RL_1$ developed flexural hinges in their beams near their column juncture. They showed high strength, without any appreciable deterioration after reaching their maximum capacity. The beam-column joints of all the repaired and strengthened specimens were intact at the conclusion of the tests.

5. The proposed Techniques by the UNIDO and by Eurocode 8: part 1-4: Epoxy Injections, Removal and Replacement, and Reinforced Concrete Jacketing for the repair and strengthening of reinforced concrete beam-column joints have proven to be effective methods of repairing severe earthquake damage to this structural element.

## NOTATION

$M_R$ = sum of the flexural capacity of columns to that of beam

$\varnothing$ = bar diameter

$f'_c$ = compressive strength of concrete

$\gamma$ = joint shear stress expressed as a multiple of $\sqrt{f'_c}$

$\gamma_{cal}$ = design values of the parameter $\left[\gamma_{cal} = \frac{\tau_{cal}}{\sqrt{f'_c}}\right]$

$\gamma_{exp}$ = actual values of the parameter $\left[\gamma_{exp} = \frac{\tau_{exp}}{\sqrt{f'_c}}\right]$

$\gamma_{ult}$ = values of the parameter $\gamma$ at ultimate capacity of the connection $\left[\gamma_{ult} = \frac{\tau_{ult}}{\sqrt{f'_c}}\right]$

a = overstrength factor

R(%) = drift angle

## REFERENCES


1. ACI-ASCE Committee 352 (1985), "Recommendations for Design of Beam-Column Joints in Monolithic Reinforced Concrete Structures (ACI 352R-85)", *American Concrete Institute*, Farmington Hills, Mich., 19 pp.

2. ACI Committee 318 (1995), "Building Code Requirements for Structural Concrete (ACI 318-95) and Commentary (318R-95)", *American Concrete Institute*, Farmington Hills, Mich., 369pp.

3. Alcoser, S.M., and Jirsa, J.O. (1990), "Assessment of the Response of Reinforced Concrete Frame Connections redesigned by Jacketing", *Proceedings of the Fourth U. S. National Conference on Earthquake Engineering*, California, U.S.A., pp. 295-304.

4. Corazao, M., Durrani, A.J., and Taylor, H. (1988), "Repair and Strengthening of Concrete Structures damaged by Earthquakes", *Proceedings of Ninth World Conference on Earthquake Engineering*, Vol. III, Tokyo-Kyoto, Japan, pp. 389-395.

5. Elnashai, A.S. (1997), "Repair and Strengthening of Earthquake – Damaged Reinforced Concrete and Steel Buildings", Lecture delivered at Advanced Study Course on Seismic Risk (SERINA), Thessaloniki, Greece, pp. 523-544.

6. Eurocode No 2 (1993), "Concrete Design of Structures", *Commission of the European Communities.*

7. Eurocode No 8 (1994), "Earthquake Resistant Design of Structures", *Commission of European Communities.*

8. Eurocode No 8 (1995), "Design Provisions for Earthquake Resistance of Structure, Part 1-4: Strengthening and repair of buildings", Draft prENV1998-1-4, CEN *(European Committee for Standardization),* Ref. No ENV1998-1-4.

9. French, C.W., Thorp, G.A., and Tsai, W. (1990), "Epoxy Repair Techniques for Moderate Earthquake Damage", *ACI Struct. Journal,* Vol.**87**, No 4, pp.416-424.

10. Gulkan, P. (1997), "The Inelastic Response of Repaired Reinforced Concrete Beam-Column Connections", *Proceedings of Sixth World Conference on Earthquake Engineering,* Vol. III, New Delhi, India, pp. 2473-2479.

11. Hakuto, S., Park. R., and Tanaka, H. (2000), "Seismic Load Tests on Interior and Exterior Beam-Column Joints with Substandard Reinforcing Details", *ACI Structural Journal,* V. **97**, No 1, pp. 11-25.

12. Karayannis, C., Chalioris, C., and Sideris, K. (1998), "Effectiveness of R/C Beam-Column Connection Repair using Epoxy Resin Injections", *Journal of Earthquake Engineering,* Vol. 2, No 2, pp. 217-240.

13. Kupfer, H.; Hilsdorf, H.K. and Rusch, H. (1969), "Behavior of Concrete under Biaxial Stresses", *ACI Journal,* Proceedings, V. **66**, No 8, pp. 656-667.

14. Lee, D.L.N., Wight, J.K., and Hanson, R.D. (1980), "Repair of Damaged Reinforced Concrete Frame Structures", *Seventh World Conference on Earthquake Engineering,* Istanbul, Turkey, Vol. 7, pp. 2486-2491.

15. Mitchell, D. (1995), "Controversial Issues in the Seismic Design of Connections in Reinforced Concrete Frames", Proceedings of the TOM PAULAY SYMPOSIUM "Recent Developments in Lateral Force Transfer in Buildings", *American Concrete Institute*, SP 157-4, pp. 75-95.

16. NEHRP (1985), "Recommended Provisions for the Development of Seismic Regulations for New Buildings – Part 3, Appendix: Existing Buildings", FEMA 97, *Buildings Seismic Safety Council*, Washington, D.C., 142pp.

17. NZS 3101:1982, "Code of Practice for the Design of Concrete Structures", *Standards Association of New Zealand,* Wellington, Part 1, 127pp., Part 2, 156 pp.

18. Park, R. and Paulay, T. (1975), "Reinforced Concrete Structures", *John Wiley Publications*, New York, 769pp.

19. Paulay, T., and Park, R. (1984), "Joints in Reinforced Concrete Frames Designed for Earthquake Resistance", Research Report 84-9, *Department of Civil Engineering, University of Canterbury*, Christchurch, New Zealand, 72pp.

20. Paultre, P., and Mitchell, D. (1990), "Some considerations for achieving Ductility in Reinforced Concrete Frame Structures", *Journal of European Earthquake Engineering*, No 2, pp. 27-37.

21. Penelis, G.G., and Kappos, J.A. (1997), "Earthquake – Resistant Concrete Structures", *E & FN SPON*, An Imprint of Chapman & Hall, 572pp.

22. Popov, E. P.; and Bertero, V.V. (1975), "Repaired R/C Members under Cyclic Loading", *Earthquake Engineering and Structural Dynamics*, (Chichester), Vol. 4, pp. 129-144.

23. Rodriguez, M., and Park, R. (1991), "Repair and Strengthening of Reinforced Concrete Buildings for Seismic Resistance" *Journal of Earthquake Spectra*, Vol. **7**, No 3, pp. 439-459.

24. Scott, B.D.; Park, R.; and Priestley, M.J.N. (1982), "Stress-Strain Behavior of Concrete Confined by Overlapping Hoops at Low and High Strain Rates", *ACI Journal,* Proceedings, Vol. **79**, No 1, pp. 13-27.

25. Tegos, I. A. (1984), "Contribution to the Study and Improvement of Earthquake-Resistant Mechanical Properties of Low Slenderness Structural Elements", PhD thesis, *Aristotle University of Thessaloniki,* Appendix 13, V. 8, 185 pp. (*in Greek*).

26. Tsonos, A. (1996), "Towards a new Approach in the Design of R/C Beam-Column Joints", Technika Chronika, *Scientific Journal of the Technical Chamber of Greece,* Vol. **16**, No 1-2, pp. 69-82 (*in Greek*).

27. Tsonos, A.G. (1997), "Shear Strength of Ductile Reinforced Concrete Beam-to-Column Connections, for Seismic Resistant Structures", *Journal of European Earthquake Engineering*, No 2, pp. 54-64.

28. Tsonos, A.G. (1999), "Lateral Load Response of Strengthened Reinforced Concrete Beam-to-Column Joints", *ACI Structural Journal*, Vol. **96**, No 1, pp. 46-56.

29. Tsonos, A.G. (2001), "Seismic Rehabilitation of Reinforced Concrete Joints by the Removal and Replacement Technique", *Journal of European Association for Earthquake Engineering*, No 3, pp. 29-43.

30. Tsonos, A.G. (2002), "Seismic Repair of Reinforced Concrete Beam-Column Subassemblages of Modern Structures by Epoxy Injection Technique", *Structural Engineering and Mechanics*, Vol. **14**, No 5 (in press).

31. UNDP/UNIDO PROJECT RER/79/015, UNIDO (1983), "Repair and Strengthening of Reinforced Concrete, Stone and Brick Masonry Buildings", *Building Construction Under Seismic Conditions in the Balkan Regions*, Vol. **5**, Vienna, 231 pp.