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Seismic repair of reinforced concrete beam-column subassemblages of modern structures by epoxy injection technique

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Abstract. The use of the epoxy pressure injection technique to rehabilitate reinforced concrete beam-column joints damaged by strong earthquakes is investigated experimentally and analytically. Two one-half-scale exterior beam-column joint specimens were exposed to reverse cyclic loading similar to that generated from strong earthquake ground motion, resulting in damage. Both specimens were typical of new structures and incorporated full seismic details in current building codes. Thus the first specimen was designed according to Eurocode 2 and Eurocode 8 and the second specimen was designed according to ACI-318 (1995) and ACI-ASCE Committee 352 (1985). The specimens were then repaired with an epoxy pressure injection technique. The repaired specimens were subjected to the same displacement history as that imposed on the original specimens. The results indicate that the epoxy pressure injection technique was effective in restoring the strength, stiffness and energy dissipation capacity of specimens representing a modern design.

Key words: beams (supports); columns (supports); earthquakes; epoxy resins; joints (junctions); reinforced concrete; repairs.

1. Introduction

Seismic provisions in most modern building codes are designed to provide adequate stiffness, strength and ductility to the main structure. The seismic design philosophy can be summarized in the following requirements:

a. Serviceability limit state: Structures must resist low-intensity earthquakes without any structural damage. Thus, during small and frequent earthquakes all structural components forming the structure should remain in the elastic range.
b. Ultimate limit state: Structures should withstand an earthquake of moderate intensity (a “design earthquake” having a peak acceleration with 90% probability of not being exceeded in 50 years) with very light and repairable damage in the structural elements, as well as in the infill elements.
c. Collapse limit state: Structures should withstand high-intensity major earthquakes with a return period much longer than their design life without collapsing (Penelis and Kappos 1997).

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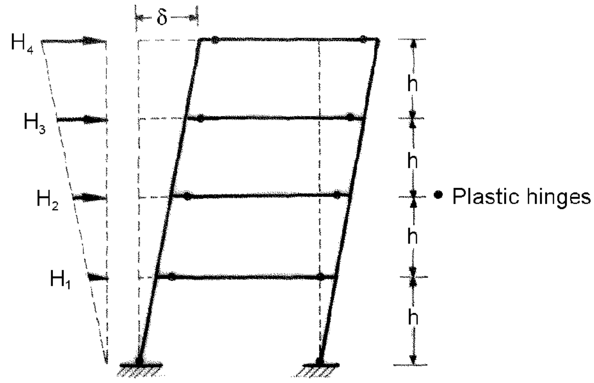

Fig. 1 Optimal failure mechanism of a frame (beam mechanism)

According to the above, the seismic design philosophy implies that a structure should exhibit some flexural cracking under moderate shaking and should deform well into the inelastic range when subjected to a major earthquake by developing flexural hinges in the beams near their adjacent column faces according to the requisite "strong column/weak beam" (Fig. 1). Primary structural elements (columns, shear walls and beam-column joints) must remain intact after a major earthquake (except the base of the structure where flexural hinges are allowed for both the columns and the shear walls).

The need for the post-earthquake strengthening of structures arises in case where existing structures must comply with more recent code requirements (Rodriguez and Park 1991). Thus after major earthquakes, modern structures should theoretically only require repair and not strengthening in order to resist new major earthquakes without collapsing for the following three reasons:

- The flexural strength ratios in the joint regions and the strength of the joints ensure the safe formation of flexural hinges in the beams of the structure. Thus the strengthening of the columns in order to increase the values of flexural strength ratios is not needed.
- Strengthening of columns, beams, beam-column joints and shear walls in order to increase their shear capacity is not needed.
- Addition of new shear walls is not needed because the modern structures have the properly located and designed shear walls.

The above are theoretically valid and, of course, there are some issues which have to be resolved.

For example, the question arises whether or not well-designed and constructed beam-column joints in a modern structure which performed excellently during a first major earthquake (allowing the safe formation of plastic hinges in the beams framing into them), are able to perform in the same successful way during a second major earthquake after repair works. After a major earthquake the repair works in the beams near the column face sometimes include removal and replacement of the crushed concrete with high strength premixed mortars. Lee *et al.* (1980) state that because of the increase in beams' strength due to the use of high strength repair materials the beam-to-column joints are usually stressed to a higher level during another major earthquake, thus creating the possibility of damage moving from the beams to the unrepaired joints and columns. However, the commonly accepted idea is that a failure of joints may quickly lead to general failure (Park and Paulay 1975).

Thus, in order to examine and evaluate the exact requirements of the repair of modern structures in practice, an experimental and analytical research is being carried out in the Structural Engineering Laboratory at the School of Engineering of Aristotle University of Thessaloniki, part of which is assessed and documented in the present study.

Systematic studies to determine the behavior of the repaired members under cyclic loading are still limited. Because of a possible future major earthquake affecting highly populated, industrialized centres, basic information on the performance of repaired members will become extremely important (Popov and Bertero 1975, Rodriguez and Park 1991).

2. Background and previous research on the performance of R/C beam-column subassemblages (original and repaired)

Field reports after damaging earthquakes often indicate that beam-column joints are one of the most vulnerable structural elements. Under earthquake loading, joints suffer often shear and/or bond (anchorage) failures (UNIDO 1983).

Since in a R/C beam-column subassemblage the weakest link is usually the beam-column joint, the review of the previous studies on R/C beam-column testing was mainly concentrated on the seismic performance of beam-column joints.

The first studies of beam-column connections were conducted at the Portland Cement Association Laboratories by Hanson and Conner (1967). Seven exterior beam-column subassemblages were constructed and tested. Hanson and Conner did not differentiate between the modes of shear resistance in flexural members and in joints and suggested that the contribution of joint core concrete in resisting the input shear could be determined by the corresponding equations for members subjected to axial load and flexure. Many other experimental investigations of the seismic behavior of beam-column connections were conducted, using R/C beam-column subassemblages. While each research program had slightly different objectives, the thrust of the majority of the programs has been twofold: (1) to develop construction details to provide and maintain ductility under repeated loads on the beam-column connection and (2) to develop guidelines which ensure proper anchorage of beam bars in the joint (Bertero 1979, Meinheit and Jirsa 1981, Paulay and Park 1984, ACI-ASCE Committee 352 (1985), Jirsa 1991).

A detailed review of previous studies on R/C beam-column joints is beyond the scope of the present paper. However, it is worth referring briefly to some studies that had a scope similar to that of the present work. Paulay and Park (1984) on the basis of experimental findings postulated the existence of two shear resisting mechanisms in R/C beam-column joints. Thus, according to the approach of Paulay and Park 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, and partly by an idealized truss consisting of horizontal hoops, intermediate column bars and inclined concrete bars between shear cracks.

The approach adopted by US investigators which gave rise to the corresponding recommendations by ACI-ASCE Committee 352 (1985) is purely empirical and it consists in establishing appropriate lower bounds to the largest available number of test results, defining a maximum allowable nominal shear strength ($\tau_{\max} = \gamma\sqrt{f_c'}$) for each type of joint and providing adequate column confining reinforcement through the connection (it is assumed that the confining steel makes no direct contribution to resisting the shear forces in the joint).

Thus, the main schools of thought are the American one (ACI-ASCE Committee 352) and the New Zealand one (Paulay and Park 1984); the approach of the latter has been adopted by CEB (1985) and to a lesser extent by Eurocode 8 (1994).

Current design practice usually leads to the formation of plastic hinges in the beam regions adjacent to the column face, which usually causes stiffness and strength deterioration to take place in the joint. Joint deterioration can be eliminated by moving beam yielding away from the connection. Bertero and Popov (1977), Abdel-Fattah and Wight (1987), Joh *et al*. (1991) have proposed arrangements of beam reinforcement in such a way that plastic hinges can successfully be moved away from the column face.

Meinheit and Jirsa (1981), Ehsani and Wight (1985), Durrani and Wight (1987), Kitayama *et al*. (1991) investigated the effect of transverse beams on the behavior of R/C joints. They concluded that the presence of transverse beams considerably improves the joint behavior. On the contrary, there was no evidence during the tests of Cheung *et al.* (1991) to indicate that the presence of beams in two directions provided significant confinement to the joint cores during bidirectional seismic loading. The effect of torsion produced by the presence of a floor slab in transverse beams on the behavior of the joint is not clearly understood (Durrani and Wight 1987).

It was shown that reduced compression, or even tension may be experienced by intermediate stories of medium-rise to high-rise reinforced concrete structures. Consequently, the shear resistance mechanisms of beam-column connections, usually considered as concrete and steel contributions, should be re-examined. This was the motivation behind the work of Agbabian *et al.* (1994) and Higazy *et al*. (1996). Through the results of their study it was demonstrated that the seismic behavior of a connection is sensitive to reductions in axial compression and even more sensitive to tension in the column. In areas of potentially high vertical ground acceleration where the development of axial tension is more likely to take place, modified lateral joint reinforcement techniques are warranted. In these areas the concrete contribution to the shear resistance of joints cannot be relied on and modified design procedures are required.

Higazy and Elnashai (1997) introduced the design basis and implemented a reliable technique for the assessment of R/C beam-column joints. The technique, namely the Shear Deformation Energy Index, defines a criterion for satisfactory behavior under earthquake excitations. They also proposed a performance chart, which is a simple and practical design aid for R/C beam-column connections of multistory structures.

Despite the many unanswered questions related to the behavior 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 and NEHRP 1985 (FEMA-97) state also that epoxy injection is not effective in restoring bond between reinforcement and concrete.

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 effectiveness of the epoxy injection technique for the repair of reinforced concrete beam-column connections damaged due to cyclic loading is 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.

3. Significance and objectives of the research

The repair and upgrading of reinforced concrete structures damaged by seismic actions are relatively new and challenging fields of study in earthquake engineering. Since the philosophy of modern seismic design codes (Eurocode 8, ACI 318-95) is based on a specific acceptable degree of structural damage, even in the event of the design earthquake, redesign-repairing structures designed to these codes and subsequently damaged, constitute a requisite part of the conceptual aim of the entire design process for seismic safety. Research in this area is essential, since engineers in seismic regions often face the dilemma of either analysing and designing repair or strengthening works of damaged buildings without quantitative guidance. The applied nature and financial importance of this research field is therefore apparent and its application should be immediate (Karayannis *et al*. 1998).

To help in this direction, the experimental investigation reported herein is aimed at evaluating the effectiveness of damaged exterior reinforced concrete beam-column subassemblages of modern structures repaired using resin injections.

4. Description of test specimens - Material properties

Two one-half scale exterior beam-column subassemblages were designed and constructed for this experimental and analytical investigation. The different reinforcement details of the specimens are shown in Fig. 2. Both specimens 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 all the requirements of ACI-318 (1995) and ACI-ASCE Committee 352 (1985) were used for the design of specimen A_1 and all the requirements of Eurocode 2 (1993) and Eurocode 8 (1994) were used for the design of specimen E_1. These requirements refer to both the estimation of member strength and detailing of reinforcement.

The nominal material properties for both specimens were 35 MPa for the concrete 28-days compressive strength and 500 MPa for the deformed reinforcement yield stress.

5. Experimental set - up loading sequence

Each specimen was tested before and after repair, 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 Fig. 3(a). All specimens (before and after repair) were subjected to a large number of cycles applied by slowly displacing the beam's free end, according to the load history shown in Fig. 3(b) without reaching the actuator stroke limit. The

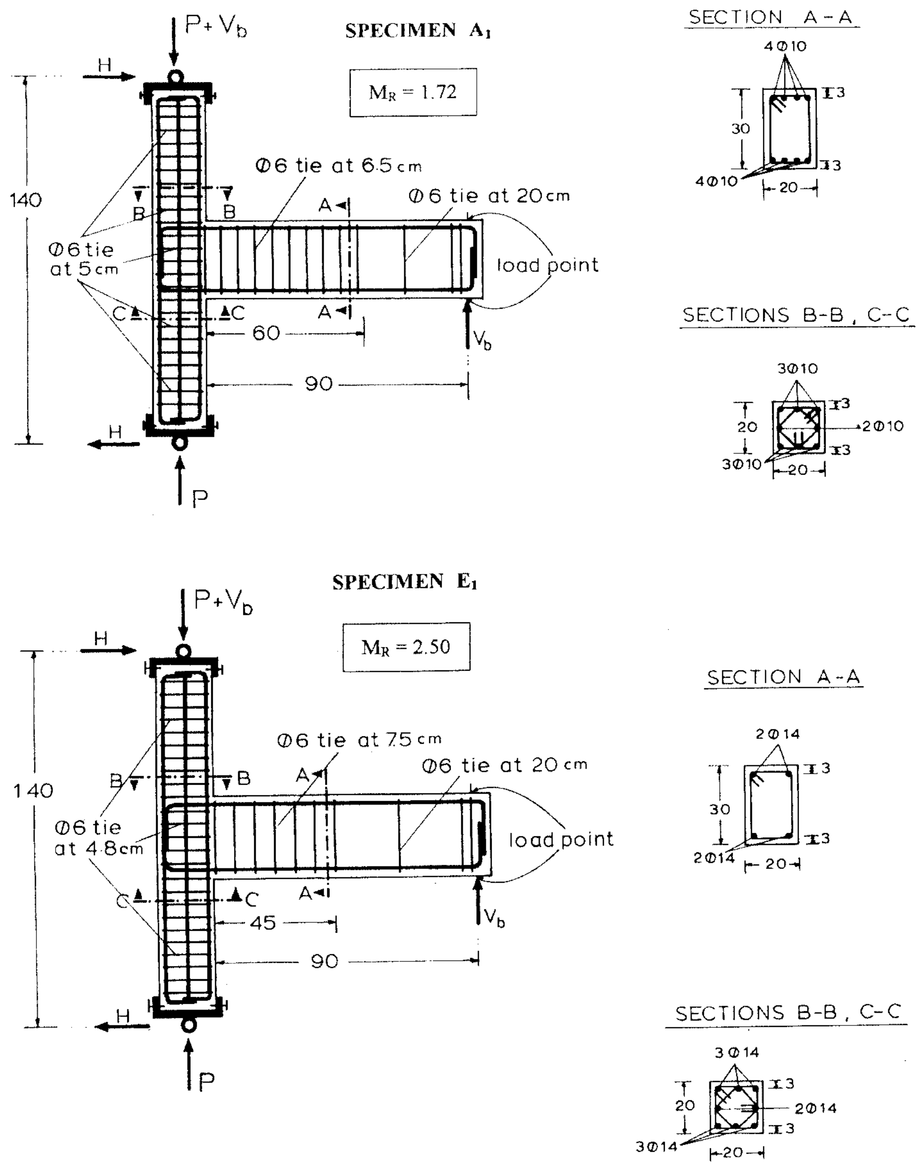

Fig. 2 Dimensions and cross-sectional details of original specimens A_1 and E_1 (dimensions in cm)

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. One loading cycle was performed at each displacement amplitude. A constant axial load 200 KN was applied to the columns of the subassemblages. Approximately 10 electrical-resistance strain gages were bonded in the reinforcing bars of each specimen of the program. In Fig. 3(c) are shown the locations of the strain gages of the specimens. The strain gages which were bonded in the joint hoops of specimens A_1 and E_1 were also used in measuring strains of the repaired specimens RA_1 and RE_1. The strain gages which were bonded in the beams' longitudinal reinforcement of specimens A_1 and E_1 were destroyed during the

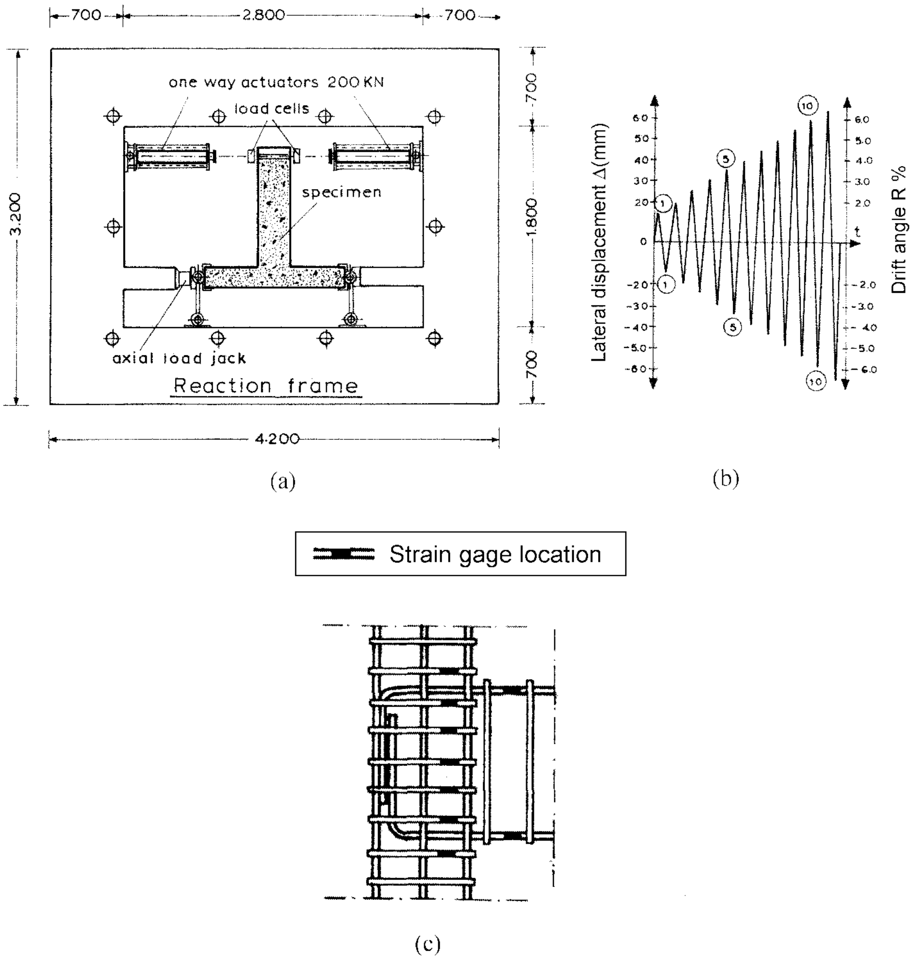

Fig. 3 (a) Test setup (dimensions in mm), (b) lateral displacement history, (c) locations of the electrical-resistance strain gages in the joint regions of the specimens A_1, RA_1, E_1 and RE_1

last cycles of loading. Thus, new strain gages were used in measuring strains of the beams of the repaired specimens RA_1 and RE_1.

6. Experimental results

6.1 Failure mode of the original subassemblages 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 plastic hinges caused severe cracking of the concrete near the fixed beam end of each subassemblage. The behavior 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 elastic. Fig. 4 shows the strain gages data of hoop reinforcement of the joint regions of both subassemblages A_1 and E_1. As is clearly shown in Fig. 4, the maximum strain recorded in the joint hoop reinforcement was below 2500 $\mu\varepsilon$.

A 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

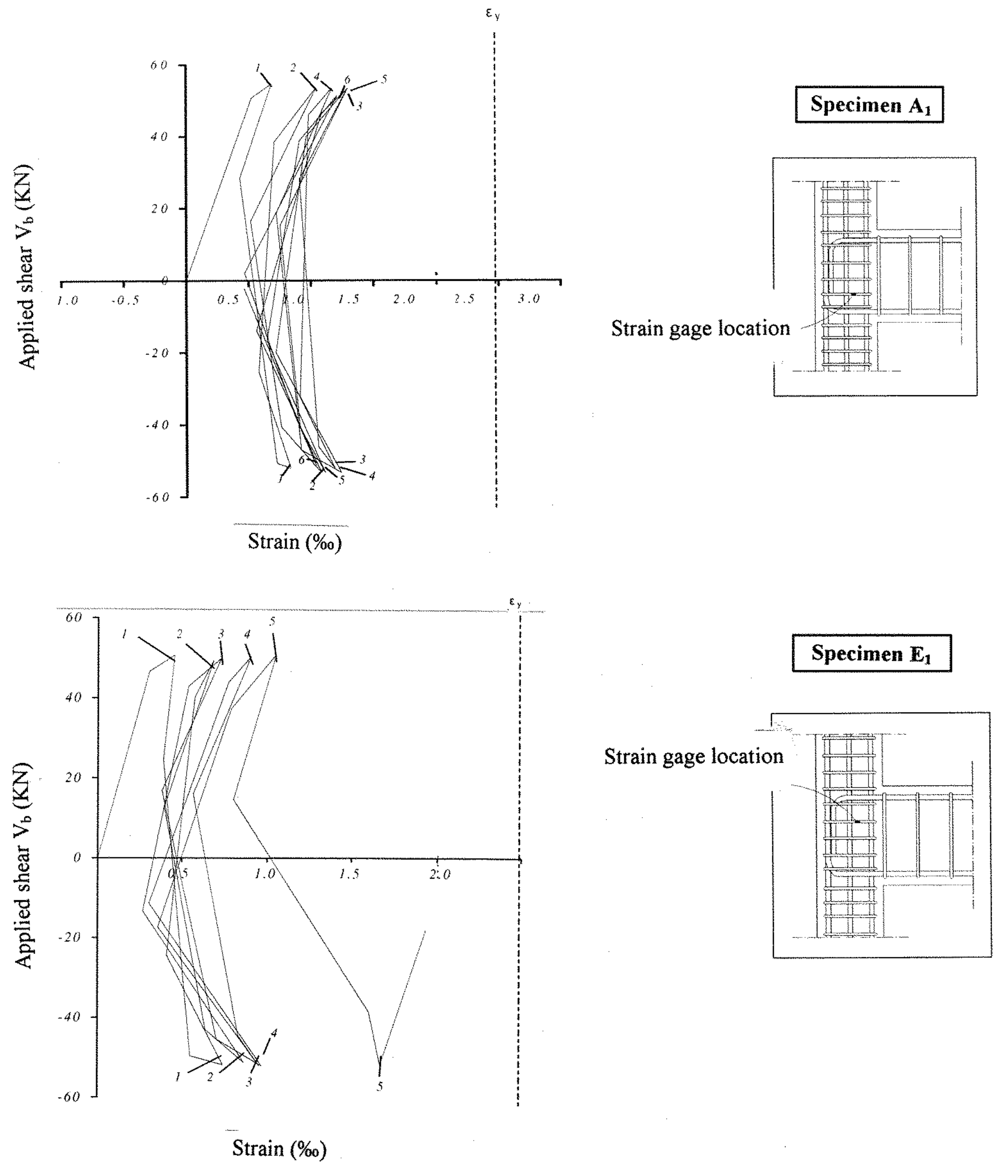

Fig. 4 Applied shear-versus strain in beam-column joint hoop reinforcement of the subassemblages A_1 and E_1

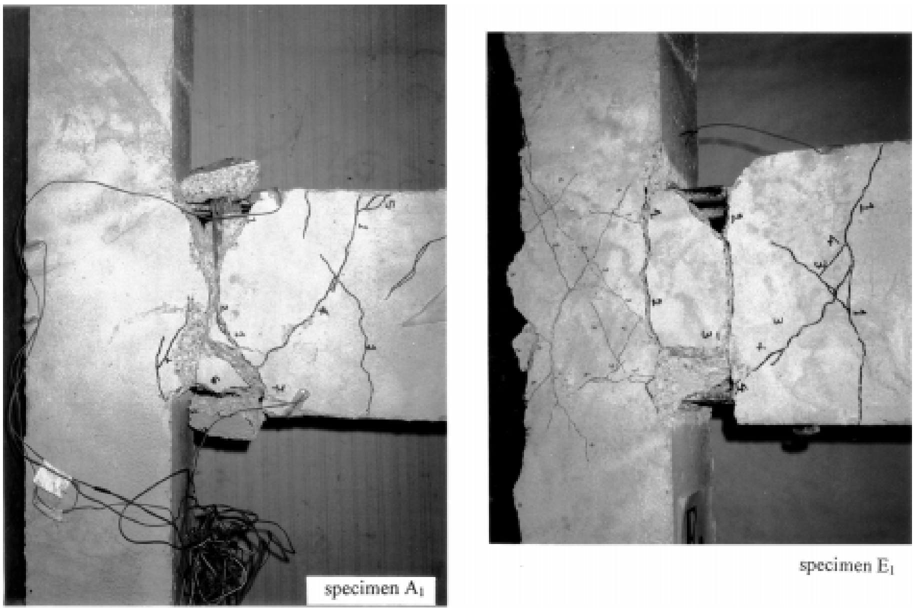

Fig. 5 Views of the collapsed subassemblages A_1 and E_1

place during the three last cycles of loading (9th, 10th and 11th), while the joint region of subassemblage A_1 was intact at the conclusion of the test (see Fig. 5). There were four small bars in the A_1 beam, and two large bars in E_1 (Fig. 2). An anchorage failure of the beam reinforcing bars of specimen E_1 took place during the last three loading cycles. The anchorage failure led to cracking in the joint region of E_1 and to spalling of the exterior face of the joint.

6.2 Repair technique

The repair procedure applied to both damaged specimens includes the following operations:

- Removal and replacement of the crushed and loose concrete in the beams 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 0.95 cm 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 (Fig. 6).
- 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 trapping (Fig. 6).
- 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).

Photographs of the various steps required by the repair procedure are presented in Figs. 6(a) to 6(f).

6.3 General behavior of the repaired subassemblages RA_1 and RE_1 and failure mechanisms

Both the repaired specimens RA_1 and RE_1 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 Fig. 7. In both specimens the rupture of some longitudinal beam reinforcing bars in the plastic hinge region took place during their two last cycles of loading (8^{th} and 9^{th} for RA_1, 10^{th} and 11^{th} for RE_1). It is obvious that the failure mode of the repaired specimen RE_1 was better than that of the original specimen E_1.

Strains of over 40.000 $\mu\varepsilon$ were obtained in the beam longitudinal bars of both the repaired specimens RA_1 and RE_1. The shear mechanisms of the joints of RA_1 and RE_1 remained elastic. In Fig. 8 are demonstrated strain gages data of the hoop reinforcement of the joint regions of the repaired subassemblages. As is clearly shown in Fig. 8, the maximum strain recorded in the joint hoop reinforcement was below 2500 $\mu\varepsilon$.

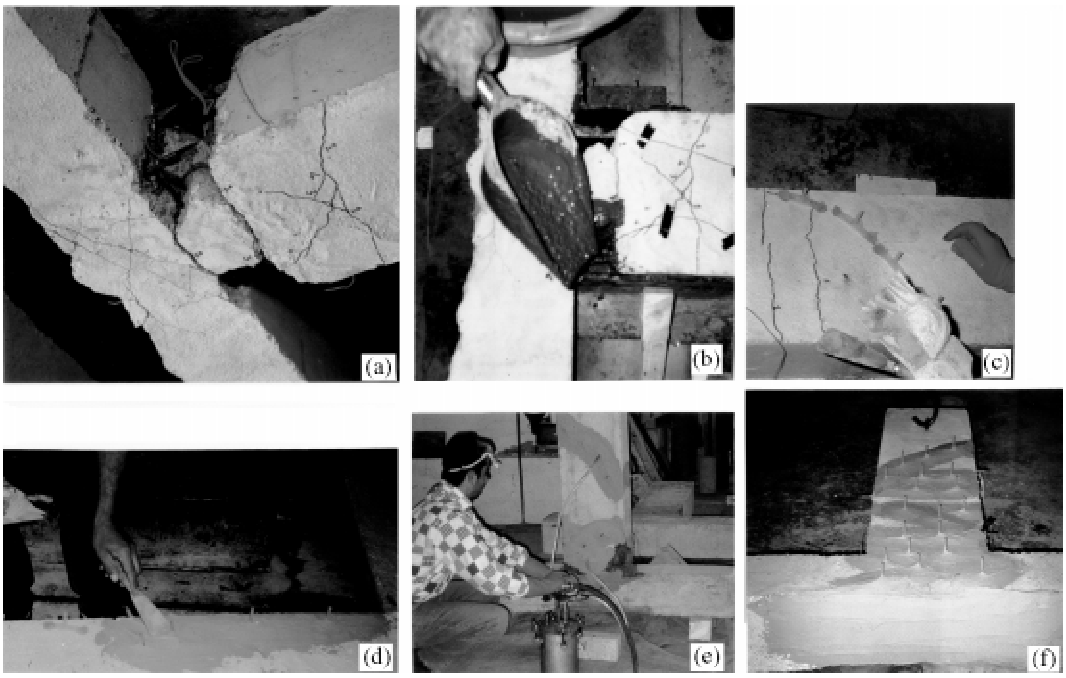

Fig. 6 Repair procedure (a) removal of the crushed concrete; (b) replacement of the crushed concrete by a pre-mixed, non-shrink, rheoplastic, flowable and non-segregating mortar of high strength; (c) placing of plastic inserts; (d) application of quick-setting epoxy; (e) epoxy injection; (f) repaired subassemblage

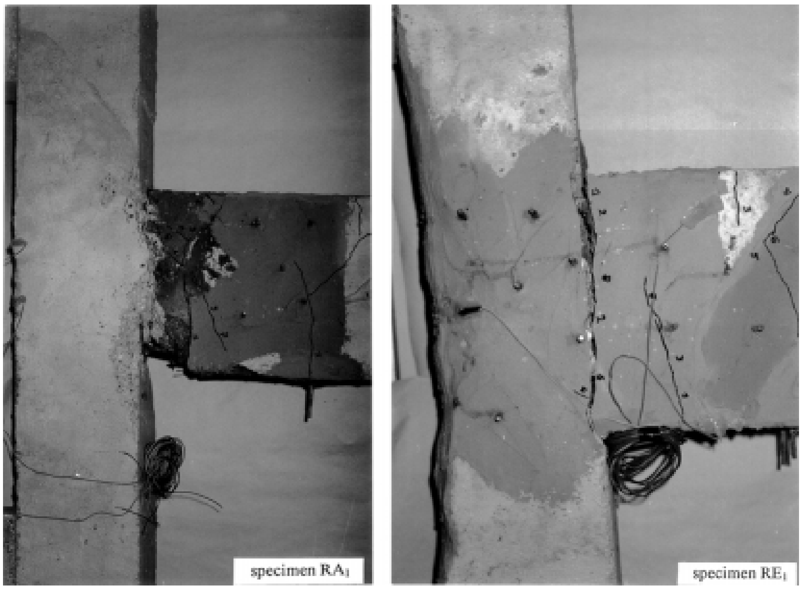

Fig. 7 Views of the collapsed subassemblages RA_1 and RE_1

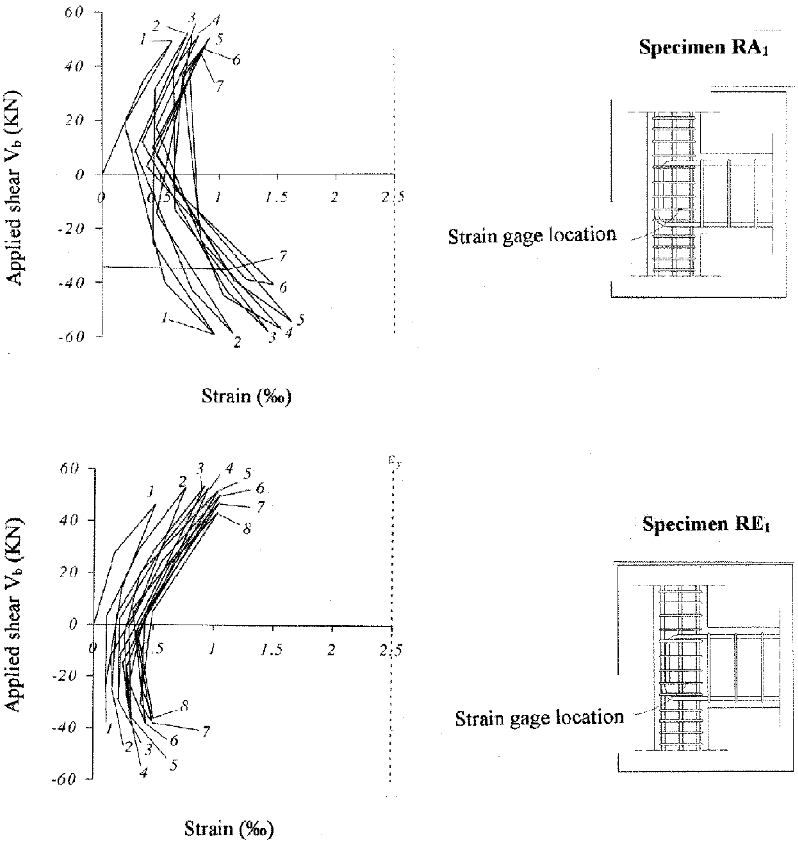

Fig. 8 Applied shear-versus strain in beam-column joint hoop reinforcement of the subassemblages RA_1 and RE_1

6.4 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 which follow, is defined as the beam tip displacement Δ divided by the beam half span *L*, and is expressed as a percentage (see the inset on Fig. 9). Plots of applied shear-versus drift angle for all the specimens A_1, E_1, RA_1 and RE_1 are shown in Fig. 9.

The original beam-column specimens A_1 and E_1 showed stable hysteretic behavior up to drift angle *R* ratios of 4.0. They showed a considerable loss of strength, stiffness and unstable degrading hysteretic behavior at and beyond drift angle *R* ratios of 4.50 (Fig. 9).

Repaired specimens RA_1 and RE_1 exhibited stable hysteresis up to the 6th 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. 9). The extreme loss of strength, stiffness and energy dissipation capacity observed in specimen RA_1 during the 8th and 9th cycle of loading and in specimen RE_1 during the 11th cycle of loading was due to the fracture of almost half of the beam reinforcing bars during these cycles of loading. In fact, it is not surprising that the smaller bars in RA_1 fractured. It is likely that they buckled in the test of A_1 and after repair they further buckled in compression resulting in the fracture of bars in the hinge region. Furthermore, the lower strength of RE_1 (compared to that of E_1) during the first seven half cycles in the negative direction, see Fig. 9, was due to the severe degradation of the anchorage of the top longitudinal beam bars and the inability of the epoxy injection to restore bond between a part of these bars and the surrounding concrete in the joint region of RE_1. This, together with the higher buckling capacity of larger bars, resulted in delaying bar fracture in RE_1.

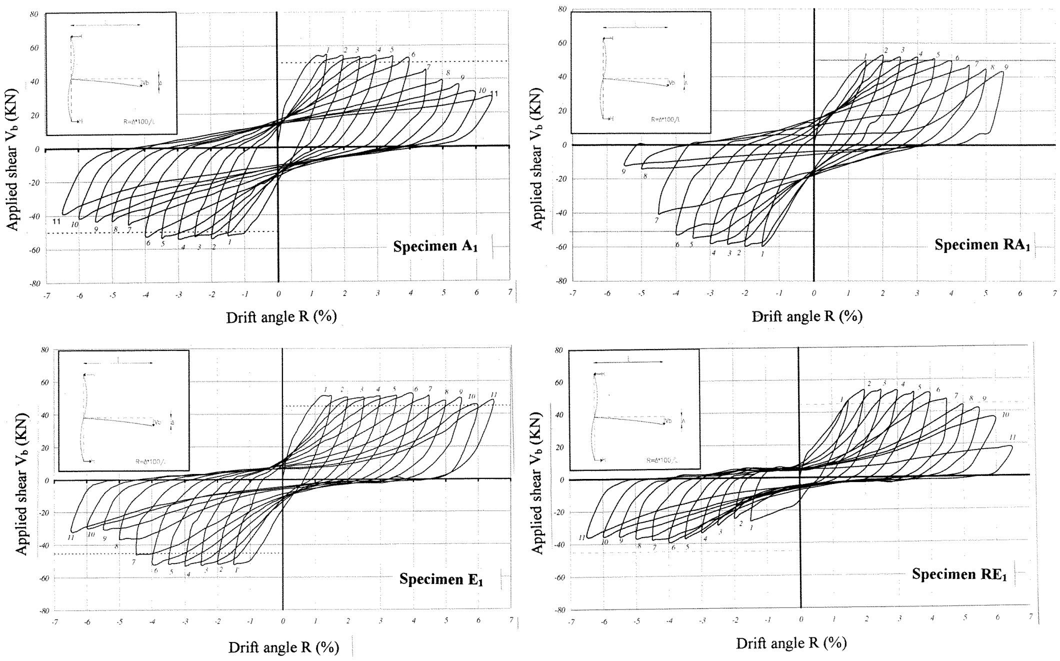

Fig. 9 Hysteresis loops of specimens A_1, RA_1, E_1 and RE_1

6.5 Comparison of the strength, stiffness and energy dissipation capacity between the original A_1, E_1 and repaired RA_1, RE_1 subassemblages respectively

For a further evaluation of the effectiveness of the repair technique in restoring 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 E_1 with those of the repaired specimens RA_1 and RE_1 respectively.

The peak-to-peak stiffness and energy dissipated for every load cycle of each specimen A_1, E_1, RA_1 and RE_1 are illustrated in Fig. 10 and Fig. 11, respectively. Fig. 12 compares the peak strength observed throughout the tests. The comparison is made by observing the ratio of the peak strengths of the repaired subassemblages RA_1 and RE_1, to that of the original subassemblages A_1 and E_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 specimens RA_1 and RE_1 is shown. Thus the repaired subassemblages achieved almost the same strength, stiffness and energy dissipation capacity as compared with those of the original subassemblages A_1 and E_1 respectively (see Figs. 10, 11 and 12). The only exception was observed in the first seven lower half cycles of specimen RE_1. In these half cycles the strength, stiffness and energy dissipation capacity were lower than those exhibited by the corresponding lower half cycles of the original specimen E_1. But all these earthquake – resistant mechanical properties were gradually increased from the first until the seventh lower half cycle, see Figs. 10, 11 and 12.

The bond between the steel reinforcement and the deteriorated surrounding concrete was not restored in part of the development length of the longitudinal beam reinforcement, while this bond

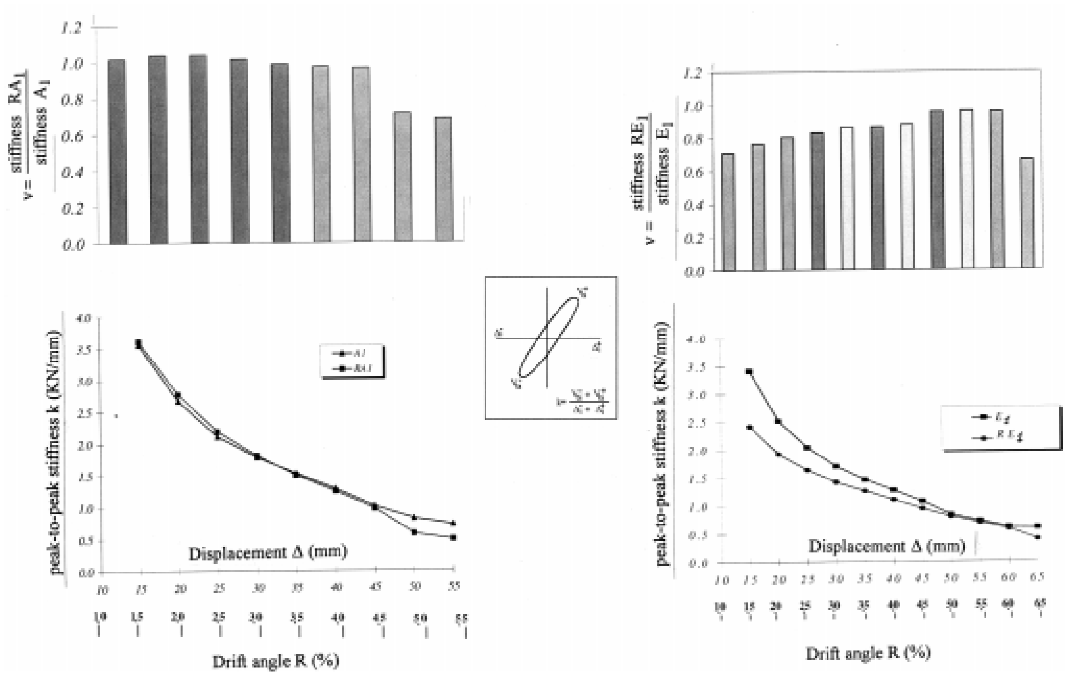

Fig. 10 Stiffness comparison between original specimens A_1, E_1 and repaired specimens RA_1, RE_1 respectively

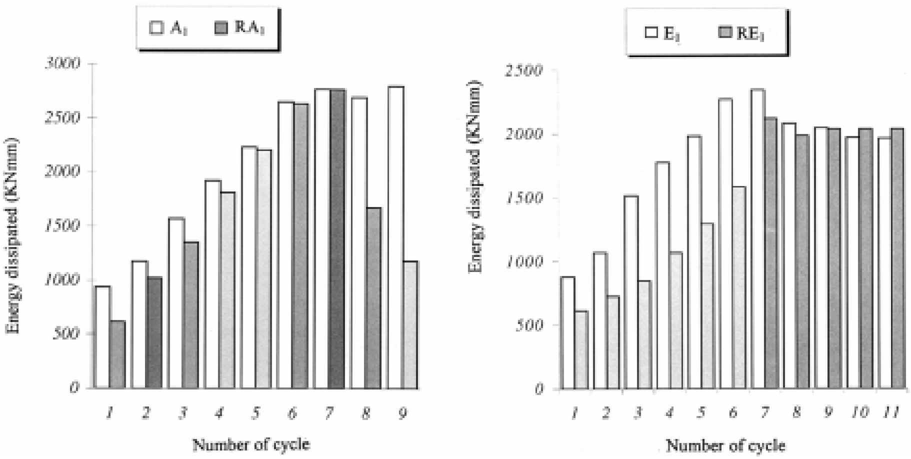

Fig. 11 Energy dissipation comparison between original subassemblages A_1, E_1 and repaired subassemblages RA_1, RE_1 respectively

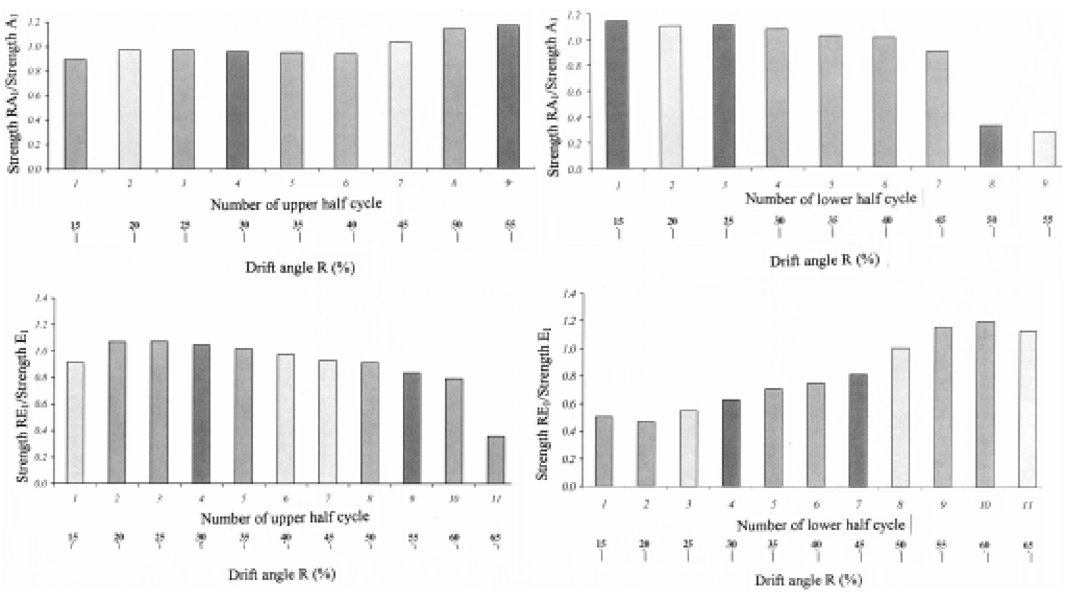

Fig. 12 Strength ratio of repaired models RA_1, RE_1 to original models A_1, E_1 respectively

was fully restored in the remaining part of the development length of the longitudinal beam reinforcing bars. The full activity of this last part of development length resulted in a gradual increase in the strength, stiffness and energy dissipation capacity during the lower half cycles of loading. It is worth noting that in the last four half cycles of loading the earthquake - resistant mechanical properties of RE_1 were similar to those of the original specimen E_1, see Figs. 10, 11 and 12. Thus it was shown that the epoxy pressure injection technique can restore the bond between the steel reinforcement and the deteriorated surrounding concrete up to a certain point.

Although the effectiveness of the pressure injection in restoring bond is not always recognized in design standards (UNIDO 1983, NEHRP 1985, Dritsos 2001), experimental studies have shown that the bond can be, at least, partially recovered by this repair technique (Tasai 1992, Karayannis *et al*. 1998).

7. Theoretical considerations

Fig. 13(a) shows a reinforced concrete exterior beam-column joint for a moment resisting frame. The shear forces acting in the joint core are resisted: (i) partly by a diagonal compression strut that acts between diagonally opposite corners of the joint core [see Fig. 13(a)], and (ii) partly by a truss mechanism formed by horizontal and vertical reinforcement and concrete compression struts. The horizontal and vertical reinforcement is normally provided by horizontal hoops in the joint core around the longitudinal column bars and by longitudinal column bars between the corner bars in the side faces of the column (vertical joint reinforcement), as shown in Fig. 13(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 gradual crushing along the cross - diagonal cracks and especially along the potential failure planes (Fig. 13(a)).

For instance, consider the section I - I in the middle of the joint height (Fig. 13(a)). In this section, the flexural moment is almost zero. The forces acting in the concrete are shown in Fig. 13(b). T_i are the forces acting in the longitudinal column bars between the corner bars in 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. 13(b)). Thus, the vertically acting forces are:

$$\underbrace{D_{cy}}_{\text{compression strut}} + \underbrace{(T_1 + \ldots + T_4 + D_{vy})}_{\text{truss model}} = D_{cy} + D_{sy} = V_{jv}$$

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 σ and the shear stress τ uniformly distributed over the section I - I are given by the Eqs. (1) and (2) below. The stress state in the finite size block, which is located in the middle of the potential corner to corner failure plane, is shown in Fig. 13(c). The normal stress in the longitudinal direction is small and can be neglected. Thus,

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

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

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

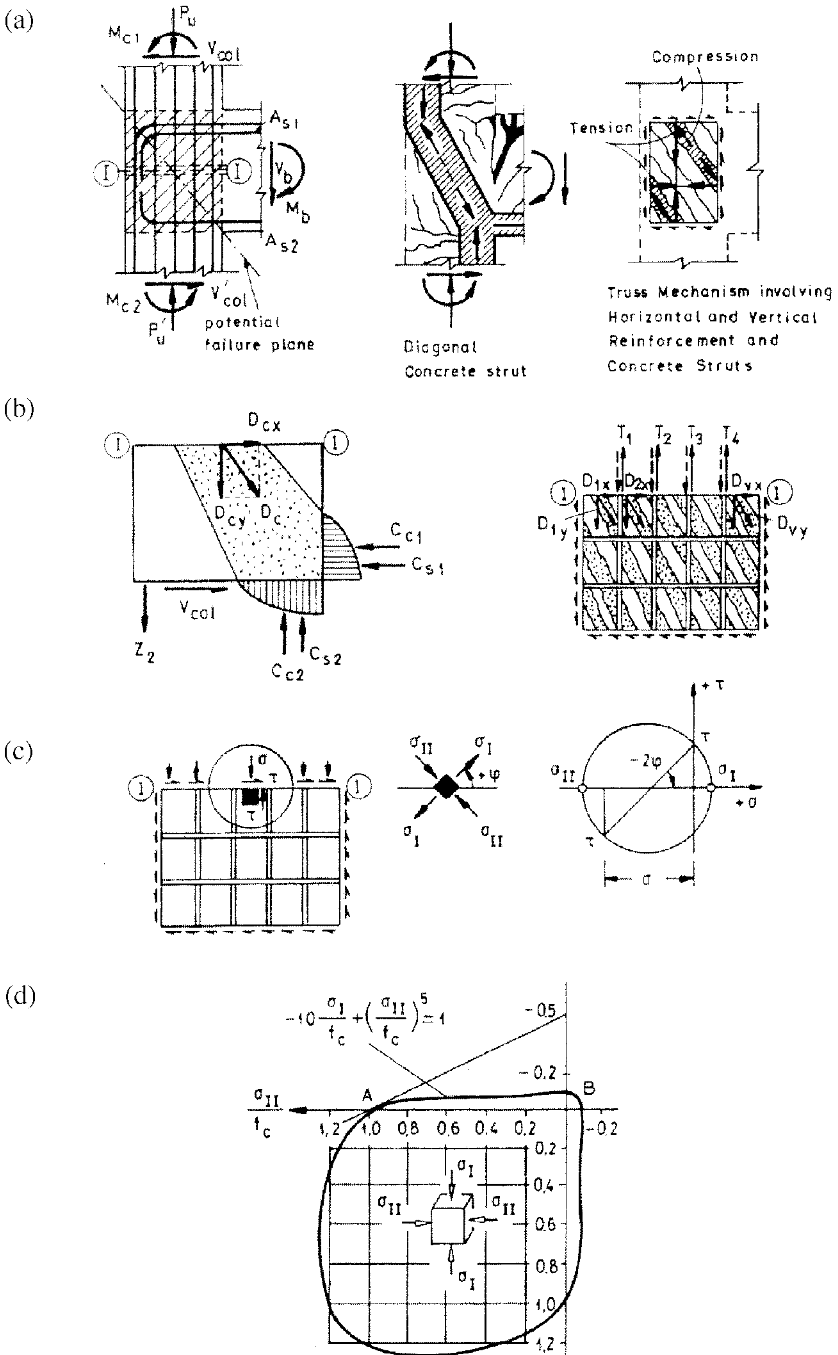

Fig. 13 (a) External beam-column connection and the two mechanisms of shear transfer (diagonal concrete strut and truss mechanism); (b) Forces acting in the joint core concrete through section I - I from the two mechanisms; (c) Stress state of element of this studied region; (d) Representation of concrete biaxial strength curve by a parabola of 5th degree

It is now necessary to establish a relationship between the average normal compressive stress σ and the average shear stress τ. From Eqs. (1) and (2) :

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

It is well known that $\frac{V_{jv}}{V_{jh}} = \frac{h_b}{h_c} = \alpha$ (EC8-1994, NZS3101-1982) (4)

where α is the joint aspect ratio. Thus,

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

The maximum principal stresses are given by Mohr's circle (Fig. 13(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 behavior 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. 13(d)). Thus, for branch AB:

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

where f_c is the increased joint concrete compressive strength due to confining, which is given by the model of Scott *et al.* (1982) according to the equation

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

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

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

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

Substituting Eq. (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:

$$(x + \psi)^5 + 10\psi - 10x = 1 \qquad (13)$$

The solution of the system of Eqs. (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 γ_{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 γ_{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 strengthened beam-column connections of specimens RA_1 and RE_1 were computed using the above methodology.

Table 1 shows that $\gamma_{cal}/\gamma_{ult}$ $(RA_1) = 0.47 < 0.50$ and $\gamma_{cal}/\gamma_{ult}$ $(RE_1) = 0.46 < 0.50$. 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 both the subassemblages RA_1 and RE_1. As predicted, the repaired specimens failed in flexure exhibiting remarkable seismic performance (Fig. 7).

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

Table 1 Experimental and predicted values of the strength of repaired specimens RA_1 and RE_1

Specimen	Joint aspect ratio $\alpha = h_b/h_c$	γ_{cal}	γ_{exp}	γ_{ult}	Predicted shear strength τ_{pred} (MPa)	Observed shear strength τ_{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

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).

8. Conclusions

Two half scale exterior beam-column subassemblages designed either according to Eurocode 2 and Eurocode 8, or according to ACI-318 (1995) and ACI-ASCE Committee 352 (1985), were tested under cyclic loading to failure. The damaged specimens were repaired and were retested under the same loading pattern. The following conclusions are drawn based on the work presented herein.

1. The original strength, stiffness and energy dissipation capacities were restored in the repaired structures. 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 beams in both subassemblages were repaired using the removal and replacement technique, as well as epoxy injection technique and were stronger than the original ones (Fig. 12). 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. The beam-column joints of both the original subassemblages performed satisfactorily during the first cyclic loading sequence to failure, allowing the safe formation of plastic hinges in their adjacent beams. The two repaired specimens performed successfully during an identical cyclic loading sequence to failure. The repaired subassemblages developed also flexural hinges in their beams at the column face. The beam-column joints of both repaired specimens were intact at the conclusion of the tests.

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Notation

M_R : sum of the flexural capacity of columns to that of beam
$\varnothing$: bar diameter
f_c' : compressive strength of concrete
γ : joint shear stress expressed as a multiple of $\sqrt{f_c'}$
γ_{cal} : design values of the parameter $\left[\gamma_{\text{cal}} = \dfrac{\tau_{\text{cal}}}{\sqrt{f_c'}}\right]$
γ_{exp} : actual values of the parameter $\left[\gamma_{\text{exp}} = \dfrac{\tau_{\text{exp}}}{\sqrt{f_c'}}\right]$
γ_{ult} : values of the parameter γ at ultimate capacity of the connection $\left[\gamma_{\text{ult}} = \dfrac{\tau_{\text{ult}}}{\sqrt{f_c'}}\right]$
a : overstrength factor
$R(\%)$: drift angle