Structural Engineering and Mechanics, Vol. 13, No. 1 (2002) 17-34
DOI: http://dx.doi.org/10.12989/sem.2002.13.1.017

Seismic repair of exterior R/C beam-to-column joints using two-sided and three-sided jackets

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Abstract. The use of local two-sided and three-sided jacketing for the repair and strengthening of reinforced concrete beam-column joints damaged by severe earthquakes is investigated experimentally and analytically. Two exterior beam-column joint specimens (O_1 and O_2) were submitted to a series of cyclic lateral loads to simulate severe earthquake damage. The specimens were typical of existing older structures built in the 1960s and 1970s. The specimens were then repaired and strengthened by local two-sided or three-sided jacketing according to UNIDO Manual guidelines. The strengthened specimens (RO_1 and RO_2) were then subjected to the same displacement history as that imposed on the original specimens. The repaired and strengthened specimens exhibited significantly higher strength, stiffness and better energy dissipation capacity than the original specimens.

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

1. Introduction

Damage caused by earthquakes through the years indicated that some reinforced concrete buildings designed and constructed in the 1960s and 1970s are found to be inadequate for resisting seismic forces. As a result, lateral strength and ductility of these structures were minimal (Hakuto *et al.* 2000). The challenge to structural engineers after an earthquake is to recommend if a damaged structure should be repaired and/or strengthened or torn down. The final answer to this question does not depend entirely on technical issues, but also on economic, social and political factors. Reconstruction and rehabilitation are nowadays often preferred to demolition and redevelopment because of cost advantages.

In the past, severe earthquakes damaged a large number of reinforced concrete structures, and some of these structures have been repaired and strengthened. Several examples of the repair and strengthening of reinforced-concrete buildings damaged by earthquakes have been reported in earthquake-prone countries such as in the Balkan region (UNIDO 1983, Penelis and Kappos 1997, Penelis 1999), Japan (Rodriguez and Park 1991), Mexico (Aguilar *et al.* 1989, Jara *et al.* 1989, Teran and Ruiz 1992) and Peru (Kuroiwa and Kogan 1980) and Greece (Penelis and Kappos 1997, Penelis 1999).

Systematic studies to determine the behavior of the repaired and/or strengthened members under

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cyclic loading are still very limited. The importance of this information can hardly be underrated. Because of a possible future major earthquake affecting highly populated, industrialized centres, basic information on the performance of repaired and/or strengthened members will become extremely important (Popov and Bertero 1975, Rodriguez and Park 1991).

Reinforced concrete beam-column joints are considered vulnerable structural elements during earthquakes. The failure of a joint or a group of joints can result in at least partial collapse of the structure. An investigation was conducted at the University of Thessaloniki to evaluate the effectiveness of the technique proposed by UNIDO (1983) for the repair and strengthening of reinforced concrete beam to column connections damaged by severe earthquakes. More specifically, two identical reinforced concrete exterior beam-column subassemblages were constructed with non-optimal design parameters flexural strength ratio, joint shear stress with and without joint transverse reinforcement representing the common construction practice of joints of older structures built in the 1960s and 1970s. 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 and strengthened by two-sided and three-sided jackets according to UNIDO Manual Techniques (1983). These upgraded specimens were again subjected to the same cyclic lateral load history. The measured response histories of the original and strengthened specimens were subsequently compared and evaluated.

2. Repair and strengthening techniques for beam-column joints according to UNIDO (1983)

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

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

2.1 Local repairs

Epoxy injections can be applied for the repair of damaged joints with slight to moderate cracks without damaged concrete or bent, or failed reinforcement. However, the restoration of bond between the reinforcement and the concrete by injections is inadequate and unreliable (UNIDO 1983). Removal and replacement should be applied in cases of crushed concrete, deteriorated bond or rupture reinforcement (UNIDO 1983, Popov and Bertero 1975, Karayannis *et al.* 1995, Karayannis and Chalioris 1998, Karayannis *et al.* 1998) .

2.2 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. For an adequate bond between original and new concrete and possibly for the welding of new reinforcement to the

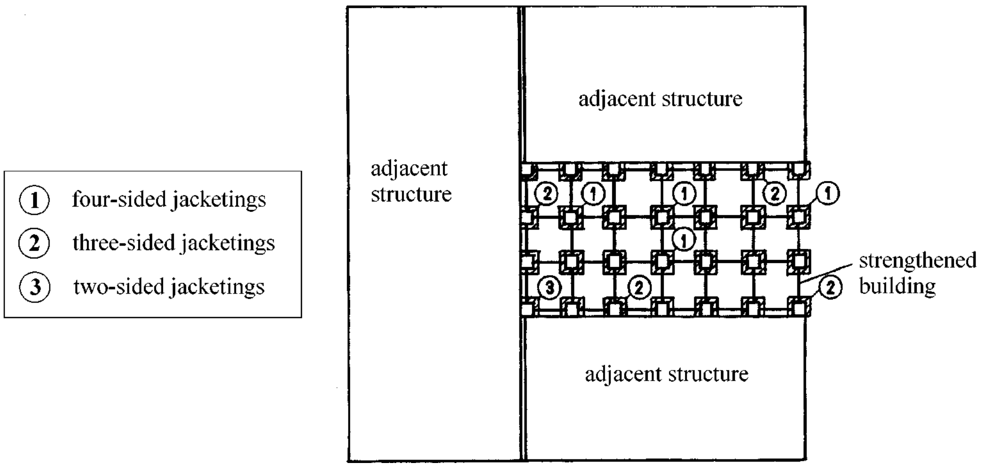


Fig. 1 Type of jacketing of columns and beam-column joints of strengthened building with adjacent structures

existing reinforcement, the old concrete cover must be chipped away. Additional horizontal ties and vertical reinforcement must be placed in the joint region in order to provide adequate joint shear strength. This is achieved by passing the new horizontal ties through holes drilled in the beam webs, and by passing the new vertical reinforcement through holes drilled in the floor slabs since the jacket must project above the top of the structural slabs. It is necessary that sufficient thickness of the jacket be provided in order that the large number of reinforcement bars required can be installed (UNIDO 1983, Karayannis *et al.* 1998, Tsonos 1999).

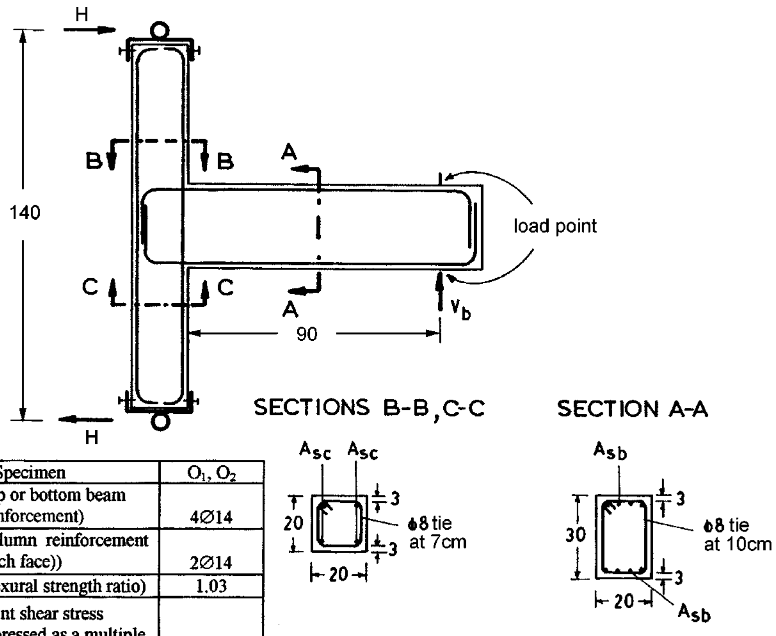
Although it is strongly recommended by the UNIDO Manual that columns and beam-column joints be jacketed on all four sides for best performance in future earthquakes, it also gives examples of three-sided or two-sided jacketings of columns and beam-column joints. These types of jacketings are inevitable when there are adjacent structures abutting the original building to be strengthened, from one or more sides, as shown in Fig. 1.

Thus, it was considered worthwhile to investigate the seismic performance of exterior reinforced concrete subassemblages upgraded by two-sided and three-sided jacketings. It is noticeable that the strengthened beam-column joint subassemblages in the literature were all four-sided jacketings.

3. Description of the specimens

3.1 Original test specimens O_1 and O_2

Two identical test specimens O_1 and O_2 were constructed using normal weight concrete and deformed reinforcement. Both specimens were typical of existing older structures built in the 1960s and 1970s. ACI-ASCE Committee "Recommendations for Design of Beam-Column Joints in Monolithic Reinforced Concrete Structures (ACI 352R-1985)" specifies the maximum allowable joint shear stresses in the form of $\gamma\sqrt{f_c'}$ MPa, where joint shear stress factor γ is a function of the joint type (i.e., interior, exterior, etc.) and of the severity of the loading, and f_c' is the concrete compressive strength. Lower limits of the flexural strength ratio M_R and joint transverse reinforcement are also confirmed by this Committee. Thus, for the beam-column connections examined in this investigation,



Specimen		O_1, O_2
A_{sb}	(top or bottom beam reinforcement)	4Ø14
A_{sc}	(column reinforcement (each face))	2Ø14
M_R	(flexural strength ratio)	1.03
γ	(joint shear stress expressed as a multiple of $\sqrt{f_c'}$)	2.02

Fig. 2 Dimensions and cross-sectional details of original specimens O_1 and O_2 (dimensions in cm)

the lower limits of M_R and γ are 1.40 and 1.00 respectively.

As seen in Fig. 2, both specimens O_1 and O_2 did not have joint transverse reinforcement (often ties in the joint region were simply omitted in the construction process in the past because of the extreme difficulty they created in the placing of reinforcement), whereas the values of flexural strength ratio were less than 1.40, and those of the joint shear stress were greater than $1.0\sqrt{f_c'}$ MPa for both specimens O_1 and O_2, see Fig. 2. Thus, the beam-column connections of the original specimens can be expected to fail in shear. The dimensions of the test specimens were primarily dictated by the availability of formwork and laboratory testing capacities, resulting in a beam-to-column joint model of approximately 1 : 2 scale. The concrete compressive strengths of specimens O_1 and O_2 were 16.00 MPa and 16.10 MPa respectively.

3.2 UNIDO strengthening technique, specimens RO_1 and RO_2

Strengthening involved encasing the original beam-column joint and the critical regions of the columns of the specimens O_1 and O_2 with a three-sided or two-sided cement grout jacket reinforced with additional ties in the joint region and in the columns (Figs. 3 and 4). To support the transverse steel, additional longitudinal reinforcement was placed at each corner of the jacket, which was then welded to the existing column reinforcement (Fig. 5). To improve the bond between the old and new concrete and for the welding of the new reinforcement to the existing reinforcing bars, the concrete cover of the original specimens (O_1 and O_2 after the tests) was chipped away and their surface was roughened by light sandblasting.

A premixed, non-shrink, rheoplastic, flowable and non-segregating mortar of high strength with

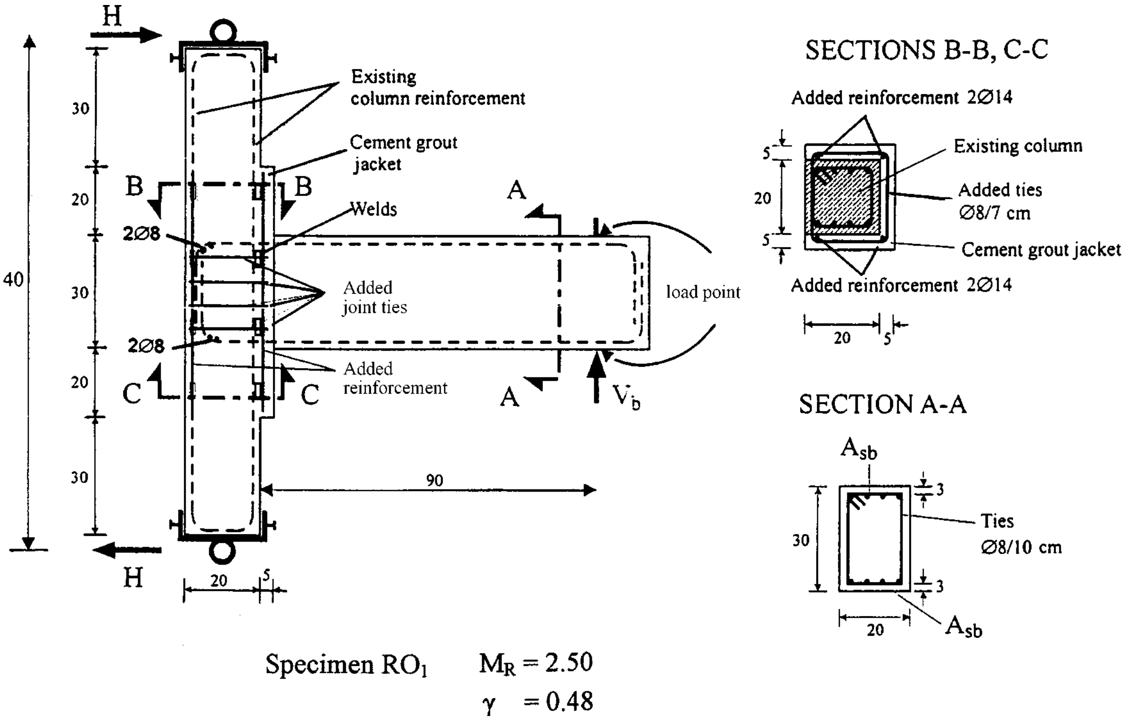


Fig. 3 Jacketing of beam-column connection of subassemblage RO_1 (dimensions in cm)

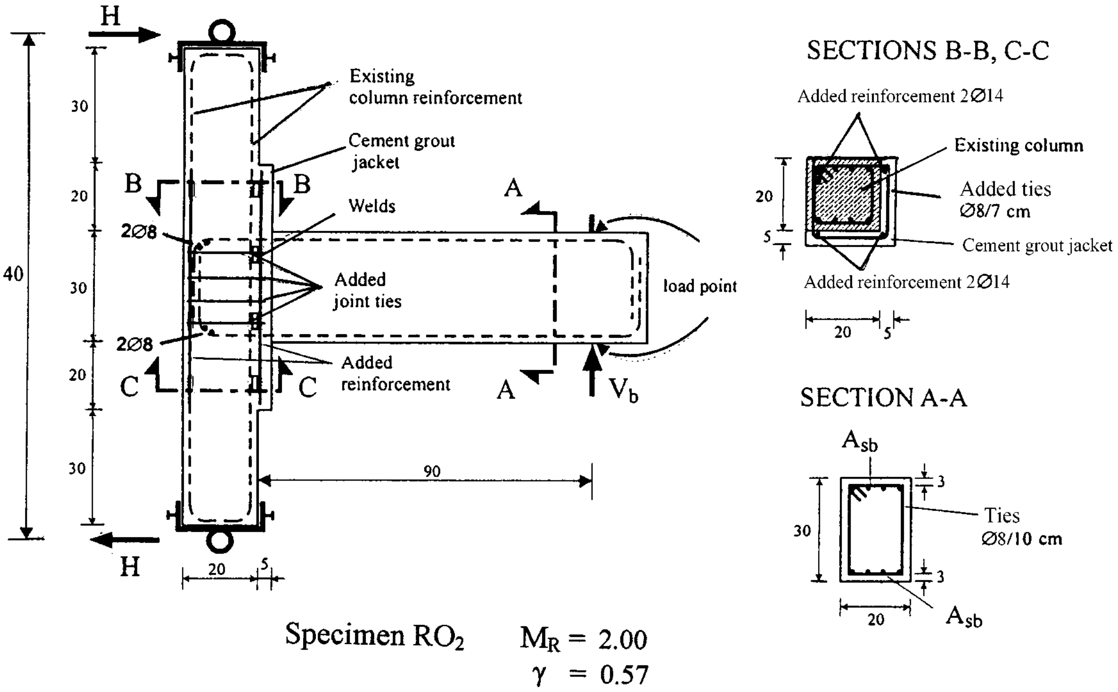


Fig. 4 Jacketing of beam-column connection of subassemblage RO_2 (dimensions in cm)

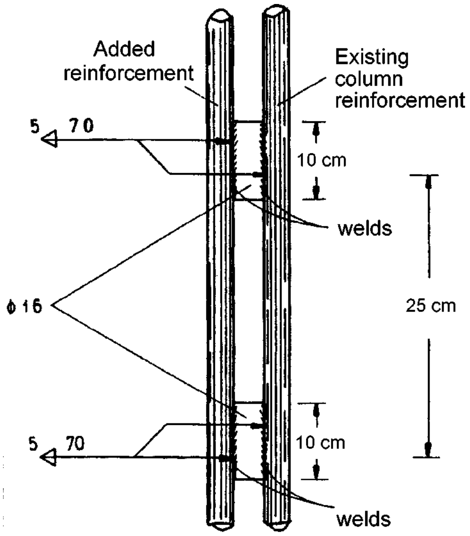


Fig. 5 Welding of new reinforcement to existing column reinforcement

0.95 cm maximum size of aggregate was used for the construction of the cement grout jacket. Using wooden formwork, the specimens were jacketed by an experienced contractor. The forms used were rigid, sufficiently tight fitting and sealed to prevent leakage.

As shown in Fig. 3, specimen RO_1, had a three-sided cement grout jacket, plus ∅14 longitudinal bars at each corner of the column connected by ∅8 supplementary ties at 7 cm, and as shown in Fig. 4 specimen RO_2 had a two-sided cement grout jacket, plus ∅14 longitudinal bars at each corner of the column connected by ∅8 supplementary ties at 7 cm. All longitudinal bars in the jackets extended through the beam-column region of the subassemblages. The beam to column joint is undoubtedly the most difficult to strengthen because of the great number of elements assembled at this point (Gulkan 1977, Corazao *et al.* 1988).

3.3 Additional joint transverse reinforcement

Four horizontal ties were placed in the joint region of specimens RO_1 and RO_2 in order to provide enough confinement and shear capacity to the joint (Fig. 6). The technique proposed by the UNIDO Manual was used. The same technique was also applied in the repaired and strengthened buildings in Mexico City following the 1985 earthquake (Jara *et al.* 1985). This was achieved by threading the new horizontal ties through holes drilled in the beam webs as follows (UNIDO 1983): each additional horizontal tie consisted of two "⊔" shape parts connected by welds (Fig. 6). Holes were drilled through the beams, and the first part of the tie was inserted and cemented with epoxy adhesive gel (Figs. 7 and 8). The two ends of this part were bent and then, along with the ends of the second "⊔" shape part, were brought together and finally welded to each other at their ends (Fig. 6).

As is clearly shown in Table 1 both specimens RO_1 and RO_2 satisfied all the requirements of the ACI-ASCE Committee. Thus 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 both specimens RO_1 and RO_2 (see Figs. 3 and 4). The additional joint transverse reinforcement of both specimens RO_1 and RO_2 was

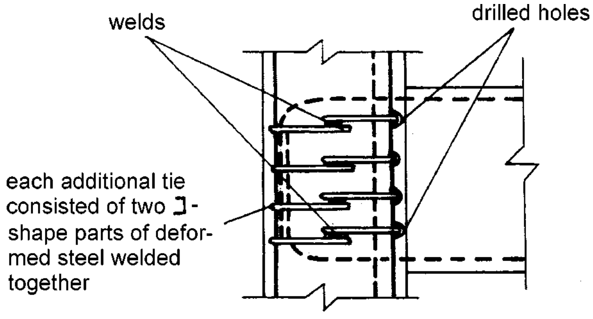


Fig. 6 Detail of additional joint transverse reinforcement

Fig. 7 Drilling of holes in the beam

Fig. 8 Additional joint ties were inserted through the holes and cemented with epoxy adhesive gel

∅8 at 5 cm. This reinforcement satisfied the requirements of the Committee $s_h = 5 = 20$ cm/4, $A_{sh} = A_{sh(required)} \simeq 0.90$ cm^2. The requirements for development of reinforcement in the joint region were also satisfied (Table 1).

In order to secure the anchorages of the beam bars terminating in a standard hook in the exterior joint region, two 8 mm diameter short bars were placed and were tightly connected on the top bends of the beam reinforcing bars and two on the bottom, running in the transverse direction of the joint, as shown in Fig. 9. This is the setup recommended by the Eurocode 8 (1993).

Both strengthened subassemblages RO_1 and RO_2 could therefore be expected to fail in flexure and, more specifically, to develop flexural hinges in their beams without severe damage concentration in their joint regions.

The concrete compressive strengths of the jackets of specimens RO_1 and RO_2 were 70.0 MPa and 70.20 MPa respectively.

Both the original and strengthened subassemblages were constructed using deformed reinforce-

Table 1 Comparison of strengthened specimens' design parameters to the ACI-ASCE recommendations

Specimen number	M_R	γ	s_h (cm)	l_{dh} (cm)	h_b/column bar diameter
RO_1	2.50(1.40)	0.48(1.00)	5 (6.25)	35.00(34.22)	21.43(20)
RO_2	2.00(1.40)	0.57(1.00)	5 (6.25)	35.00(34.11)	21.43(20)

Numbers outside the parentheses are the provided values.
Numbers inside the parentheses are the required by the recommendations
$l_{dh} = 1.25\, f_y$ (MPa) $d_b/6.2\sqrt{f_c'}$ MPa
d_b = beam bar diameter
h_b = total depth of beam
s_h = center-to-center spacing of hoops in the joint region, cm

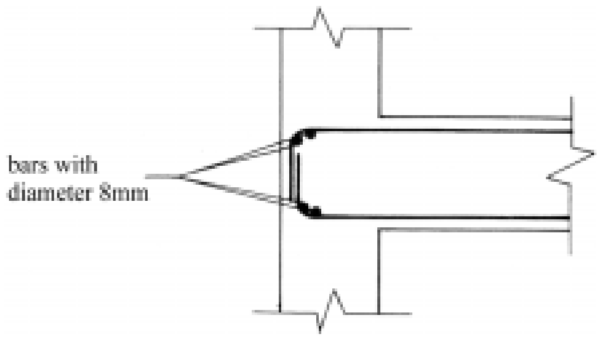


Fig. 9 The provision of transverse reinforcement was made to ensure the anchorage of the beam bars in the joint region

ment (NOTE: ∅8, ∅14 = bar with diameter 8 mm, 14 mm). The subassemblages' steel yield stresses are shown in Table 2. Electrical-resistance strain gauges were bonded in the reinforcing bars within the joint region of both the original and strengthened subassemblages in Fig. 10 are shown the locations of the strain gauges in the joint region of the specimens.

4. Test set-up – loading sequence

A testing frame in the Laboratory of Reinforced Concrete Structures at the Aristotle University of Thessaloniki was used to apply cyclic displacements to the beam while maintaining a constant axial load in the column (Fig. 11(a)). Both specimens were loaded transversely according to the load history shown in Fig. 11(b). The axial load was imposed by a hydraulic jack on one of the two specimen columns, as shown in Fig. 11(a). In this column, the axial load was kept constant at approximately 0.40 P_b (P_b: balanced column load) during the test.

Table 2 Original and strengthened specimens' steel yield stress

Bar diameter	Steel yield stress (MPa)
∅8 (8 mm)	495
∅14 (14 mm)	485

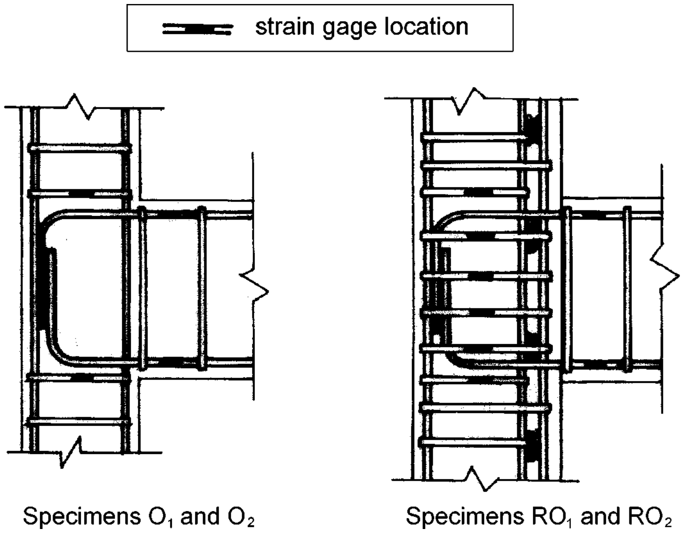
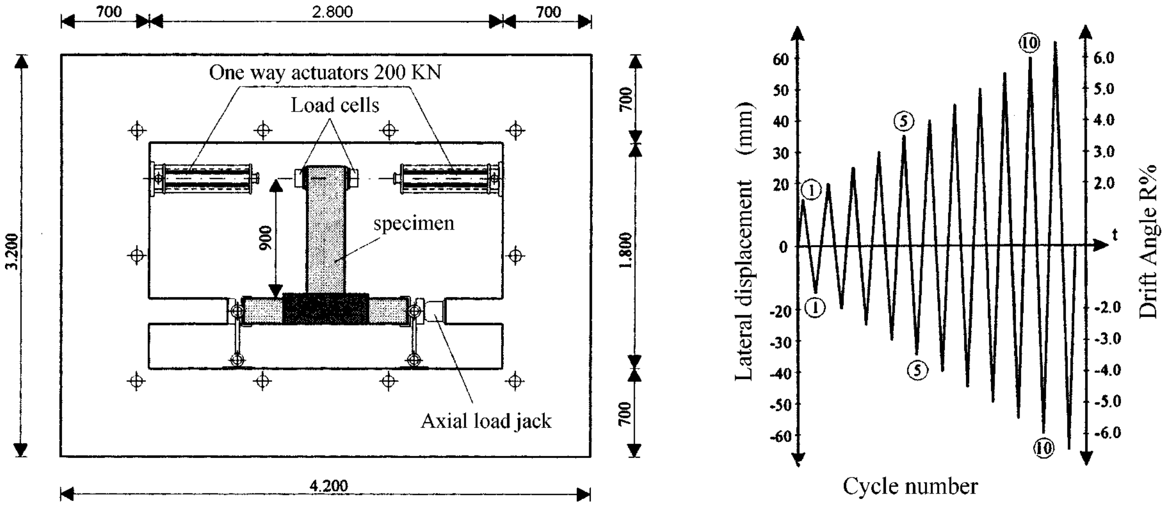


Fig. 10 Locations of the electrical-resistance strain gages in the joint regions of the specimens O_1, O_2, RO_1 and RO_2



(a) (b)

Fig. 11 (a) Test setup (dimensions in mm), (b) Lateral displacement history

5. Comparison of test results

5.1 Failure modes

<u>Specimens O_1 and O_2</u>: the connections of both these subassemblages, as expected, exhibited explosive shear failure during the early stages of seismic loading. Damage occurred both in the joint area and in the columns' critical regions. The extreme joint shear deformations are obvious in these

specimens, see Fig. 12.

The beams in both specimens O_1 and O_2 remained intact at the conclusion of the tests (Fig. 12).

Specimens RO_1 and RO_2: failure mode of both specimens RO_1 and RO_2, as expected, involved the formation of a plastic hinge in the beam near the column juncture and damage concentration in this region only. It is worth noting that the flexural hinges occurred just outside the retrofitted areas, see Fig. 13. The formation of plastic hinges caused severe cracking of the concrete near the fixed end of the beam.

In particular, during the final cycles of loading 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 RO_1 and RO_2 occurred after the seventh and eighth cycles of loading, respectively.

The three-sided and two-sided jacketing of beam-column joints are 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 Fig. 13, the rear faces of both specimens RO_1 and RO_2 were intact at the conclusion of the tests.

In summary, the strengthened subassemblages RO_1 and RO_2 exhibited cracking patterns dominated by flexure. In contrast, the original subassemblages O_1 and O_2 exhibited cracking patterns dominated by shear (Fig. 13).

5.2 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 expressed as a percentage (see the inset on Fig. 14). Plots of applied shear-versus-drift angle for all the specimens (O_1, RO_1, O_2 and RO_2) are shown in Fig. 14.

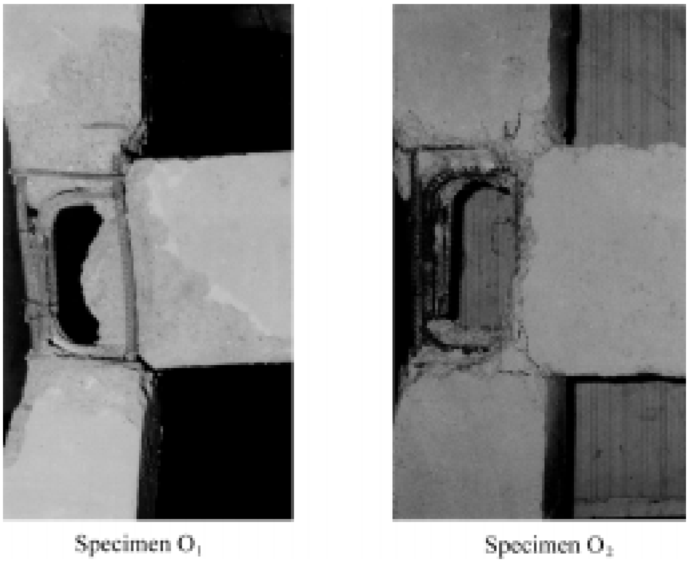
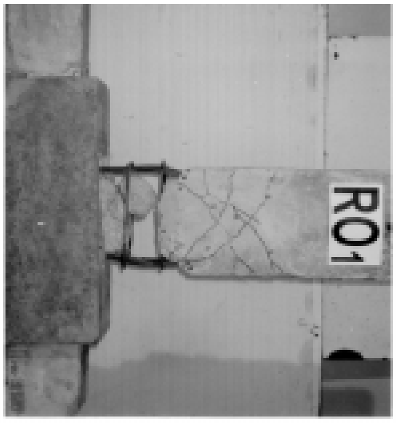
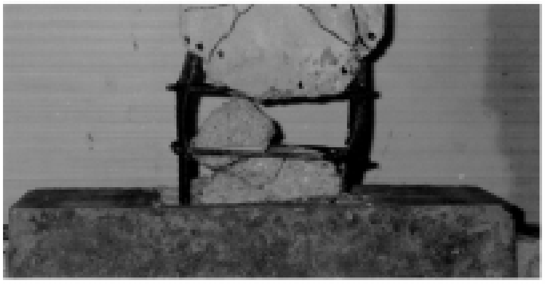


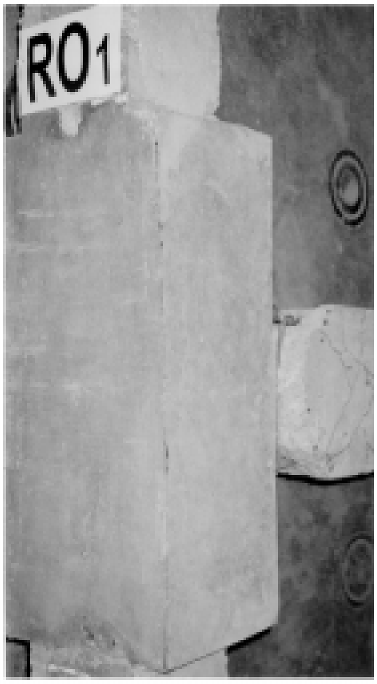
Fig. 12 Cracking configuration of specimens O_1 and O_2



front view of specimen RO_1



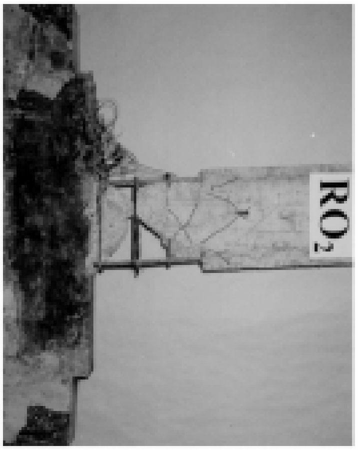
Specimen RO_1 (detail)



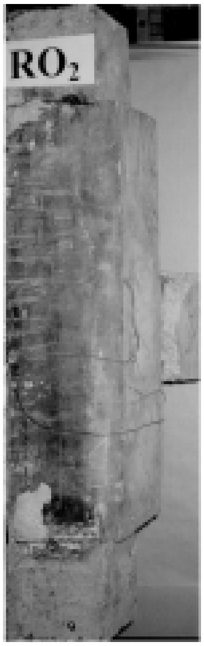
side view of specimen RO_1

Fig. 13 (a) Cracking configuration of specimen RO_1

front view of specimen RO_2



rear view of specimen RO_2



side view of specimen RO_2

Fig. 13 (b) Cracking configuration of specimen RO_2

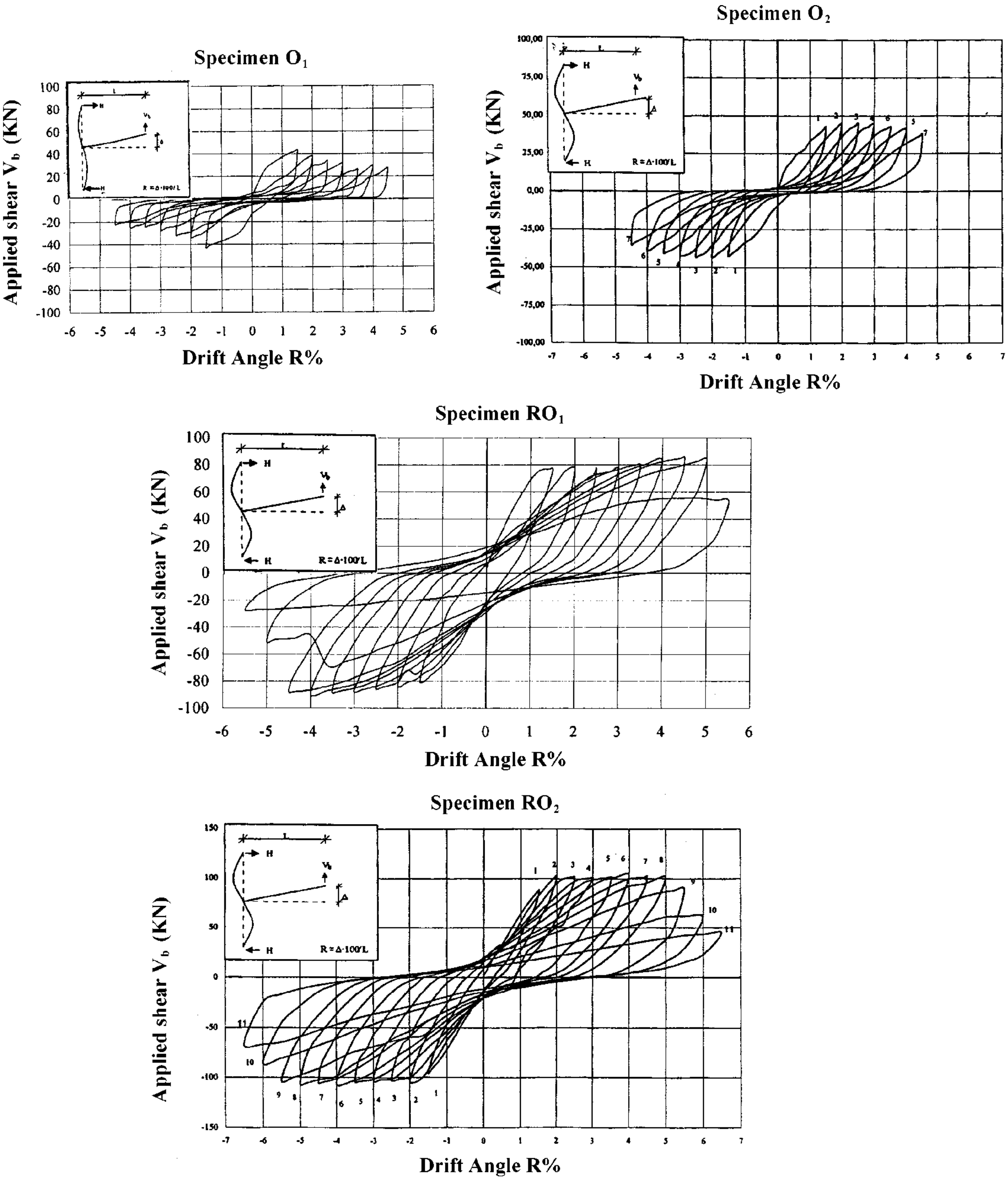


Fig. 14 Load deflection curves of specimens O_1, RO_1, O_2 and RO_2

The original beam-column specimens O_1 and O_2 showed stable hysteretic behavior up to drift angle R ratios of 2.0 percent and 2.5 percent respectively. They showed a considerable loss of strength, stiffness and unstable degrading hysteresis beyond drift angle R ratios of 2.0 percent and 3.0 percent respectively (Fig. 14).

Strengthened specimens RO_1 and RO_2 exhibited stable hysteresis up to the 8th cycle of drift angle

R, of 5.0 percent, after which a significant loss of strength began due to the noticeable buckling of the beam reinforcement (Fig. 14).

5.3 Comparison of strength, stiffness and energy dissipation capacity between the original (O_1 and O_2) and strengthened (RO_1 and RO_2) subassemblages

For a further evaluation of the effectiveness of the UNIDO strengthening technique in restoring and increasing 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 O_1 and O_2 with those of the strengthened specimens RO_1 and RO_2 respectively. The beam-column connection of specimens O_1 and O_2 represent the oldest building beam-column connections in Greece which have low joint shear strength $\gamma = 2.02 >> 1.0$ and which do not have joint transverse reinforcement.

The peak-to-peak stiffness and energy dissipated for every load cycle of each specimen are illustrated in Fig. 15 and Fig. 16 respectively. Fig. 17 compares the peak strength observed throughout the tests. The comparison is made by observing the ratio of the peak strengths of the strengthened subassemblages to that of the original subassemblages. From these diagrams, it is clearly seen that

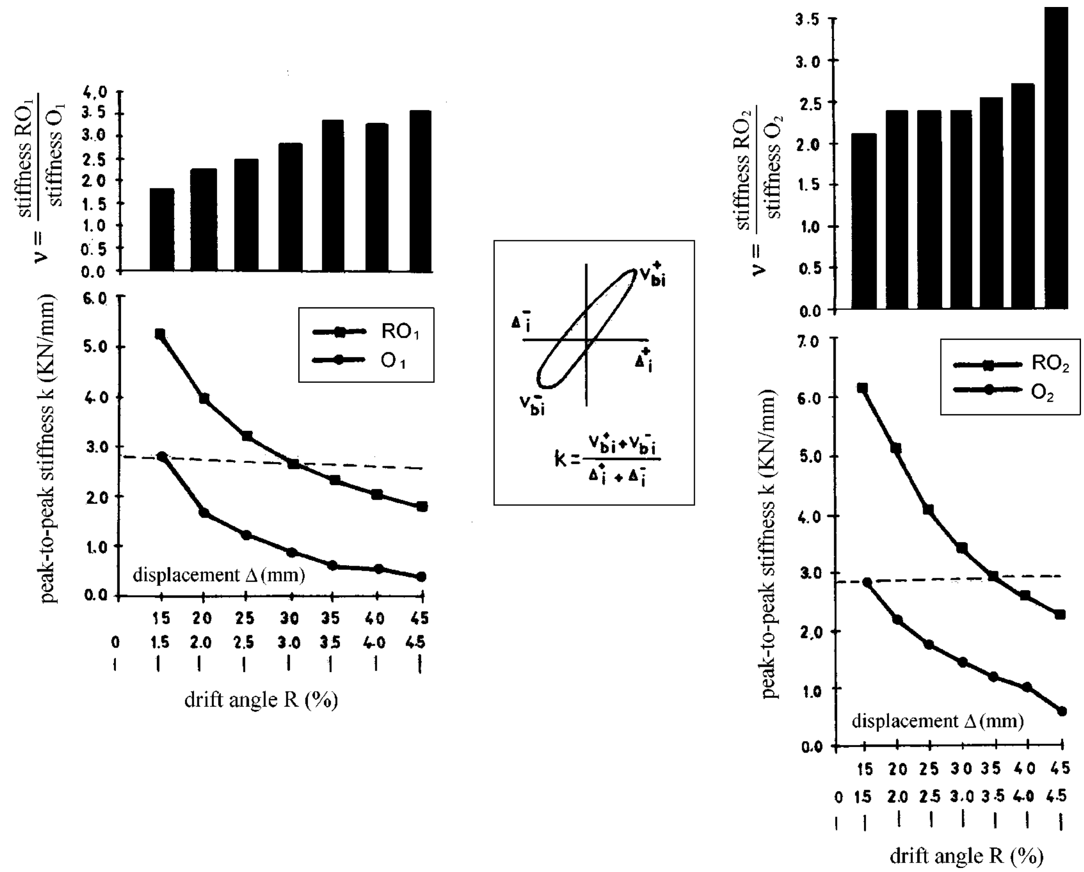


Fig. 15 Stiffness comparison between original and strengthened specimens

the strengthened specimens achieved significantly increased strength, stiffness and energy dissipating capacities compared to the original specimens, even in the large displacement amplitude cycles of drift angle R ratios between 3 percent and 4.5 percent.

From the diagrams of Figure 15, it is easy to compare the peak-to-peak stiffness developed by the strengthened subassemblages RO_1 and RO_2 during each load cycle to the original peak-to-peak stiffness of the O_1 and O_2 respectively. It is worth noting that the stiffness of RO_1 was greater or equal to the original stiffness of O_1 until the 4th cycle of drift angle R ratio of 3.0 percent and the stiffness of RO_2 was greater or equal to the original stiffness of O_2 until the 5th cycle of drift angle R ratio of 3.5 percent (Fig. 15).

The above indicates the effectiveness of UNIDO techniques in restoring and increasing the strength, stiffness and energy dissipation capacity of heavily damaged beam-column connections.

5.4 Strain gauges data

The original specimens O_1 and O_2 failed by the yielding of the ties in the joint region during their first cycles of loading. A brittle shear failure occurred in the joint region of both the original structures O_1 and O_2. This shear failure occurred before the formation of a plastic hinge in the beams. The maximum strain recorded in the longitudinal bars of the beams was below 2000 $\mu\varepsilon$.

The strengthened specimens RO_1 and RO_2 displayed significantly improved strength and ductility. They developed plastic hinges in their beams while their joint regions remained intact at the conclusion of the tests (Fig. 13). Strains of over 40.000 $\mu\varepsilon$ were observed in the beam longitudinal bars.

The maximum strain recorded in the ties of both strengthened specimens RO_1 and RO_2 was below 2000 $\mu\varepsilon$ (Fig. 18).

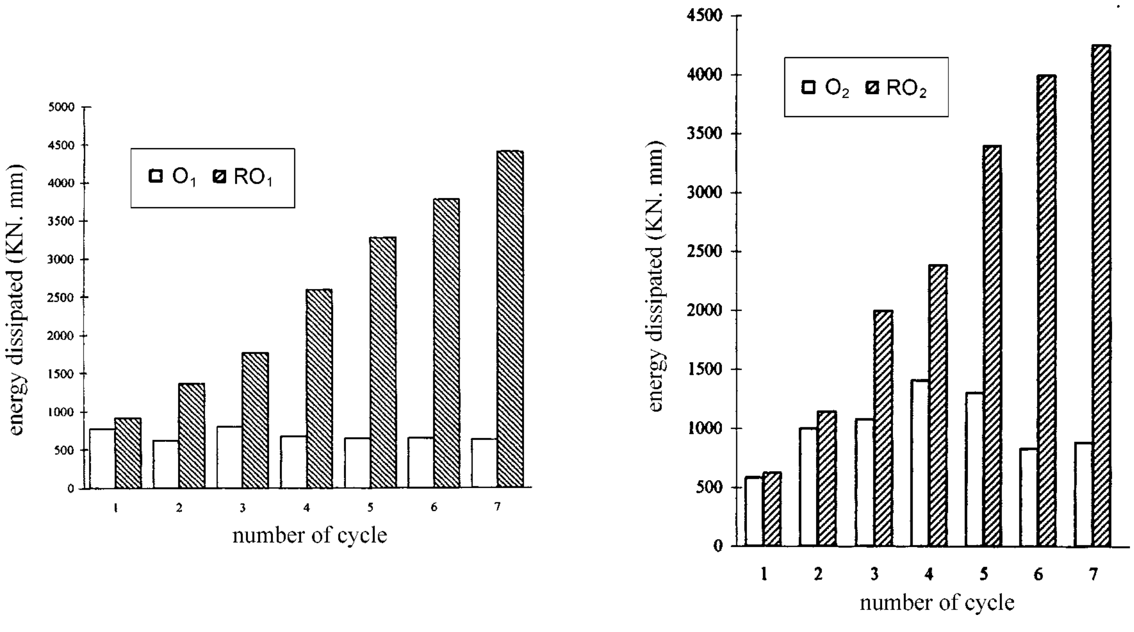


Fig. 16 Energy dissipation comparison between original and strengthened subassemblages

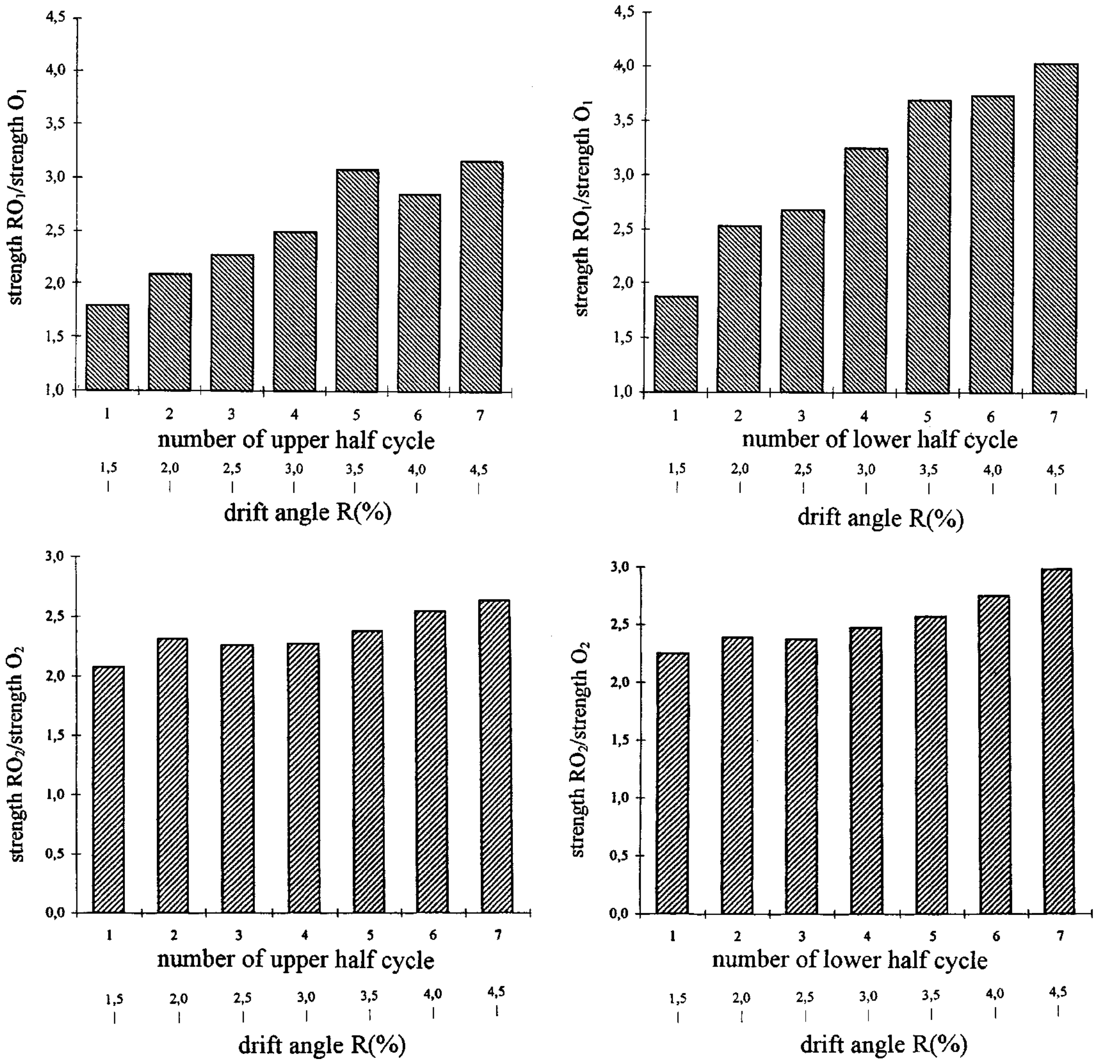


Fig. 17 Strength ratio of strengthened model to original models. (a) RO_1/O_1: number of upper half cycle; (b) RO_1/O_1: number of lower half cycle; (c) RO_2/O_2: number of upper half cycle; (d) RO_2/O_2: number of lower half cycle

6. Theoretical considerations

A formulation was developed at the Aristotle University in Thessaloniki, which gives the beam-column joint ultimate shear strength.

This 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's 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

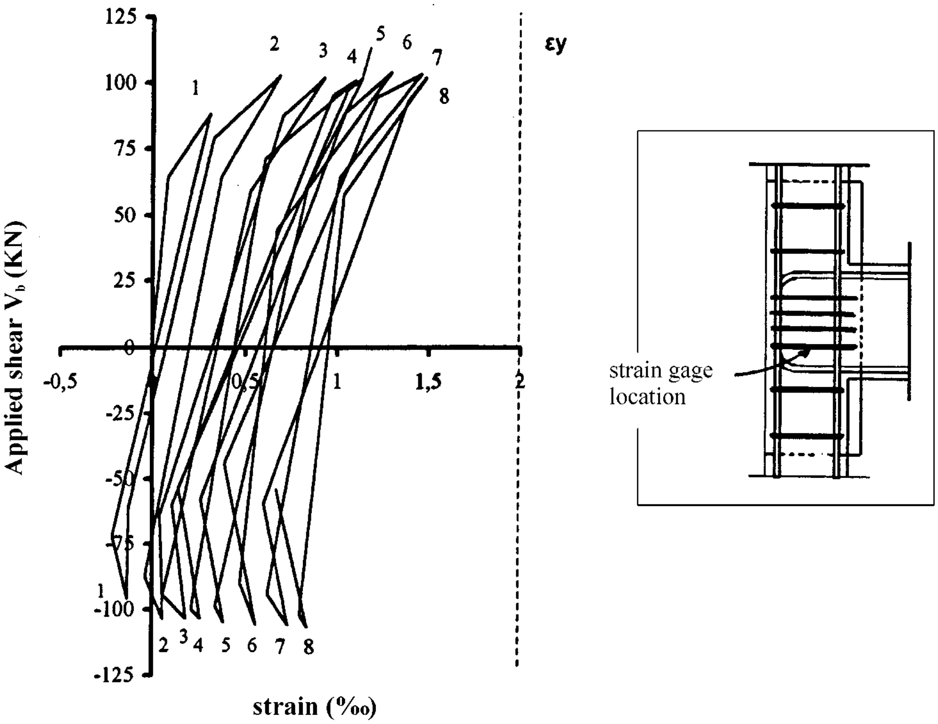


Fig. 18 Applied shear-versus-strain in beam column joint additional ties of strengthened specimen RO_2

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).

Table 3 Experimental and Predicted values of the strength of strengthened specimens RO_1 and RO_2

Specimen	γ_{cal}	γ_{exp}	γ_{ult}	Predicted shear strength $\tau_{pred}^{(1)}$	Observed shear strength $\tau_{exp}^{(2)}$	$\mu = \frac{\tau_{pred}}{\tau_{exp}}$	$\frac{\gamma_{cal}}{\gamma_{ult}}$	Predicted failure mode	Observed failure mode
RO_1	0.48	0.49	1.51	$0.48\sqrt{f_c'}$	$0.43\sqrt{f_c'}$	1.10	0.31	flexure	flexure
RO_2	0.57	0.57	1.51	$0.57\sqrt{f_c'}$	$0.57\sqrt{f_c'}$	1.00	0.38	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 $\tau_{pred} = \gamma_{cal}\sqrt{f_c'}$ MPa).

$^{(1)}\tau_{pred} = \gamma_{cal}\sqrt{f_c'}$ MPa

$^{(2)}\tau_{exp} = \gamma_{exp}\sqrt{f_c'}$ MPa

The shear capacities of the strengthened beam-column connections of specimens RO_1 and RO_2 were computed using the above methodology.

Table 3 shows that $\gamma_{cal}/\gamma_{ult}$ (RO_1) = 0.31 < 0.50 and $\gamma_{cal}/\gamma_{ult}$ (RO_2) = 0.38 < 0.50, which means that the three-sided jacket of specimen RO_1, and the two-sided jacket of specimen RO_2 in their joint regions are adequate. 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 RO_1 and RO_2. As predicted, the strengthened specimens failed in flexure exhibiting remarkable seismic performance (Fig. 13).

In both cases, the observed capacity was predicted to within approximately 10 percent of that computed using the joint shear strength formulation (Table 3).

7. Conclusions

An effective repair method has been studied for damaged beam-column joints in reinforced concrete frames.

Based on the test results described in this paper, the following conclusions can be drawn.

1. Specimens O_1 and O_2, representing existing beam-column subassemblages of old structures, performed poorly under reversed cyclic lateral deformations. The connections of these subassemblages exhibited explosive shear failure at early stages of seismic loading, and damage to both subassemblages was concentrated in the joint region.
2. The strengthening of beam-column joints with new partial two-sided and three-sided reinforced concrete jackets was identified as a useful technique for enhancing the stiffness, strength and energy dissipation capacities of poorly detailed as-built beam-column joint regions.
3. The strengthened specimens failed in flexure and showed high strength, without any appreciable deterioration after reaching their maximum capacity. Also, spindle-shaped hysteresis loops were observed with large energy dissipation capacity.
4. In general, the ACI-ASCE Recommendations can be used for designing a jacketing scheme in the joint regions.

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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
P = applied column axial load during the test
τ = joint shear stress
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 $\left[\gamma_{\text{ult}} = \dfrac{\tau_{\text{ult}}}{\sqrt{f_c'}}\right]$
l_{dh} = development length of hooked bars measured from the face of the column core to back side of the hook
a = overstrength factor
h_b = total depth of beam