LATERAL LOAD RESPONSE OF STRENGTHENED REINFORCED CONCRETE BEAM-TO-COLUMN JOINTS

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SUMMARY

Two test series were conducted to determine the effectiveness of UNIDO Manual guidelines for repair and strengthening of beam-column joints damaged by severe earthquakes. Seven exterior reinforced concrete subassemblages were subjected to a series of cyclic lateral loads to simulate severe earthquake damage. The specimens were then repaired and strengthened by jacketing according to UNIDO Manual guidelines. The strengthened specimens were then subjected to the same load history as that imposed on the original test specimens. The repaired and strengthened specimens exhibited higher strength, higher stiffness and better energy dissipation capacity than the original specimens.

INTRODUCTION

In the past, a large number of reinforced concrete structures have been damaged by severe earthquakes, 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), Japan (Rodriguez & Park 1991), Mexico (Aguilar et al. 1989, Jara et al. 1989) and Peru (Kuroiwa & Kogan 1980).

Systematic studies to determine the behaviour of the repaired and/or strengthened members under 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 be extremely important (Rodriguez & Park 1991, Popov & Bertero 1975).

After the Thessaloniki earthquake (1978), the Halcyonides earthquake (1981) and the Kalamata earthquake (1986) in Greece, many of the buildings were repaired and/or strengthened. The repair and/or strengthening of structures after these earthquakes were undertaken in accordance with the techniques proposed by the United Nations (1977), which were later incorporated in the United Nations Industrial Development Organization (UNIDO 1983). 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 techniques proposed by UNIDO (1983) for the repair and strengthening of reinforced concrete beam-to-column connections damaged by severe earthquakes. More specifically, seven reinforced concrete exterior beam-column subassemblages were constructed with non-optimal design parameters: flexural strength ratio, joint shear stress, without joint transverse reinforcement or having joint transverse reinforcement less than that required by the modern Codes, representing the common construction practice of joints before 1984 and encompassing the vast majority of beam-column connections which were subjected to the above earthquakes in Greece. It is worth mentioning that in 1984 there was a major revision of the Greek Earthquake Resistant Code of 1959. The subassemblages were subjected to cyclic lateral load histories so as to provide the equivalent of severe

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earthquake damage. The damaged specimens were then repaired and strengthened 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.

REPAIR AND STRENGTHENING TECHNIQUES FOR BEAM-COLUMN JOINTS ACCORDING TO UNIDO (1983)

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:

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 the bond between the reinforcement and the concrete by injections is inadequate and unreliable. Removal and replacement should be applied in cases of crushed concrete, deteriorated bond or rupture reinforcement.

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 existing reinforcement, the 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.

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

DESCRIPTION OF THE SPECIMENS

Original Test Specimens O_1, O_2, P_1, P_2, P_3, M_1 and M_2

Seven test specimens O_1, O_2, P_1, P_2, P_3, M_1 and M_2 were constructed using normal weight concrete and deformed reinforcement. All 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-1985)" specifies the maximum allowable joint shear stresses in the form of $\gamma\sqrt{f_c'}$ psi, 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, the lower limits of M_R and γ are 1.40 and 12 respectively.

The specimen O_1 had no 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). As seen in Fig. 1, the joint transverse reinforcement of specimens P_1, P_2, P_3, M_1 and M_2 did not satisfy the requirements of the Committee s_h = 7cm > 20cm / 4 = 5cm ($A_{sh} \cong A_{sh(required)}$ = 0.90cm^2), whereas the values of flexural strength ratio were less than 1.40 and/or those of the joint shear stress were greater than $12\sqrt{f'_c}$ psi for all the specimens O_1, O_2, P_1, P_2, P_3, M_1 and M_2, see Fig. 1. 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 one-half scale. The concrete compressive strengths of specimens O_1, O_2, P_1, P_2, P_3, M_1 and M_2 were 2320 psi, 3190 psi, 4780 psi, 2760 psi, 3050 psi, 4500 psi and 4900 psi respectively.

UNIDO Strengthening Techniques, Specimens RO_1, RO_2, RP_1, RP_2, RP_3, RM_1 and RM_2

Strengthening involved encasing the original beam-column joint and the critical regions of the columns of the specimens with a three-sided cement grout jacket reinforced with additional ties in the joint region and the columns (Fig. 2). 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. 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, O_2, P_1, P_2, P_3, M_1 and M_2 after the tests) was chipped away and the surface was roughened by light sandblasting.

EMACO was used for the construction of the cement grout jacket. EMACO is a trademark name of a premixed, non-shrink, rheoplastic, flowable and non-segregating mortar of high strength with 0.95cm maximum size of aggregate. 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. 2, all specimens RO_1, RO_2, RP_1, RP_2, RP_3, RM_1 and RM_2 had the same three-sided cement grout jacket, plus ∅14 longitudinal bars for specimen RO_1, and plus ∅10 longitudinal bars for specimens RP_1, RP_2, RP_3, RM_1 and RM_2, at each corner of the column connected by ∅8 supplementary ties at 7cm. All longitudinal bars in the jackets extended into 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 in this region (Gulkan 1977, Corazao et al. 1988).

The concrete compressive strengths of the jackets of specimens RO_1, RO_2, RP_1, RP_2, RP_3, RM_1 and RM_2 were 9130 psi, 9200 psi, 7970 psi, 9490 psi, 8990 psi, 8700 psi and 8700 psi respectively. Both the original and strengthened subassemblages were constructed using deformed reinforcement. As summary of all (original and strengthened) specimens' steel yield stress, are in ksi, bar size: ∅8 = 71.74, ∅10 = 67.45, ∅12 = 76.67, ∅14 = 70.30 (NOTE: ∅8, ∅10, ∅12, ∅14 = bar with diameter 8mm, 10mm, 12mm, 14mm). Electrical-resistance strain gages were bonded to the reinforcing bars within the joint region of characteristic original and strengthened subassemblages of the program.

Additional joint transverse reinforcement

For these joints with additional ties (joints of strengthened specimens) the technique proposed by the UNIDO Manual was used (1983). The same technique was also applied to the repaired and strengthened buildings in Mexico City following the 1985 earthquake (Jara et al. 1989).

Four horizontal ties were placed in the joint of specimen RO_1 in order to provide enough confinement and shear capacity to the joint. Two additional horizontal ties were placed in the joint region of specimens RO_2, RP_1, RP_2, RP_3, RM_1 and RM_2 in order to increase their shear strength (Fig. 2).

The values of the flexural strength ratio were higher than 1.40 and those of the joint shear stress were lower than $12\sqrt{f'_c}$ psi for all the specimens RO_1, RO_2, RP_1, RP_2, RP_3, RM_1 and RM_2, (Figures 2(a), 2(b)). The additional joint transverse reinforcement of specimen RO_1 was ∅8 at 5cm. This reinforcement satisfied the requirements of the Committee. The joint transverse reinforcement of specimens RO_2, RP_1, RP_2, RP_3, RM_1 and RM_2 with the two additional ties was ∅8 at 3.50cm. It is obvious that the joint reinforcement of RO_2, RP_1, RP_2, RP_3, RM_1 and RM_2 also satisfied the requirements of the Committee.

The provision of transverse reinforcement, made of short bars placed and tightly connected under the bends of a group or rebars, was made to ensure the anchorage of the beam bars in the joint region (Eurocode 8 1993). 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.

TEST SETUP - 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 of the specimens. All specimens were loaded transversely according to the load history shown in Fig. 3.

COMPARISON OF TEST RESULTS

Failure Modes

Specimens O_1, O_2, P_1, P_2, P_3, M_1 and M_2 : the connections of all 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. Of course, more rapid deterioration was observed in specimen O_1 (without joint shear reinforcement); the extreme joint shear deformations are obvious in this specimen, see Fig. 4. It is worth mentioning that the column longitudinal reinforcement of specimens O_1 and P_1, consisting of ∅14 bars, was bent into permanent waves in the joint region (Fig. 4), while the column longitudinal reinforcement of specimens O_2, P_2, P_3, M_1 and M_2 consisting of ∅10 bars, was buckled into permanent waves in this region. The beams in all specimens O_1, O_2, P_1, P_2, P_3, M_1 and M_2 remained intact at the conclusion of the tests.

Specimens RO_1, RO_2, RP_1, RP_2, RP_3, RM_1 and RM_2 : the failure mode of specimens RO_1, RO_2, RP_1, RP_2, RP_3, RM_1 and RM_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 retrofit area, see Fig. 4. 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, RO_2, RP_1, RP_2, RP_3, RM_1 and RM_2 occurred after the seventh, eighth, ninth, eighth, seventh, eighth and seventh cycles of loading, respectively.

The three-sided jacketing of beam-column joints is more critical than the four-sided jacketing, especially on the rear face of the joint along the column, where the hooked ends of the beam longitudinal reinforcement move outward to split the cover. The rear faces of all specimens strengthened by local three-sided jacketing RO_1, RO_2, RP_1, RP_2, RP_3, RM_1 and RM_2 were intact at the conclusion of the tests.

In summary, the strengthened subassemblages RO_1, RO_2, RP_1, RP_2, RP_3, RM_1 and RM_2 exhibited cracking patterns dominated by flexure. In contrast, the original subassemblages O_1, O_2, P_1, P_2, P_3, M_1 and M_2, exhibited cracking patterns dominated by shear (Fig. 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. 4). Plots of applied shear-versus drift angle for representative specimens O_1, RO_1, P_1, and RP_1 are shown in Fig. 4.

The original beam-column specimens O_1, O_2, P_1, P_2, P_3, M_1 and M_2 showed stable hysteretic behaviour up to drift angle R ratios of 1.0 percent, 3.0 percent, 2.5 percent, 3.0 percent, 3.0 percent, 3.5 percent and 2.0 percent. They showed a considerable loss of strength, stiffness and unstable degrading hysteresis beyond drift angle R ratios of 2.0 percent, 3.0 percent, 3.0 percent, 3.5 percent, 3.5 percent, 3.5 percent and 2.0 percent, respectively (see Fig. 4 for the representative specimens O_1 and P_1).

Strengthened specimens RO_1, RO_2, RP_1, RP_2 and RM_1 exhibited stable hysteresis up to the eighth 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 (see Fig. 4 for the representative strengthened specimens RO_1 and RP_1). The strengthened specimens RP_3 and RM_2 demonstrated stable hysteresis up to the seventh cycle of drift angle R of 4.5 percent.

CONCLUSIONS

An effective retrofit 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, O_2, P_1, P_2, P_3, M_1 and M_2 representing an existing beam-column subassemblage, performed poorly under reversed cyclic lateral deformations. The connections of these subassemblages exhibited explosive shear failure during early stage of seismic loading, and damage to all subassemblages was concentrated in the joint region.

2. The UNIDO Technique for the local strengthening of reinforced concrete beam-column joints by three-sided jacketing has proven to be an effective method of repairing severe earthquake damage of this structural element. Strengthened specimens RO_1, RO_2, RP_1, RP_2, RP_3, RM_1 and RM_2 exhibited significantly increased strength, stiffness and energy dissipation capacities as compared with those of original specimens O_1, O_2, P_1, P_2, P_3, M_1 and M_2 respectively.

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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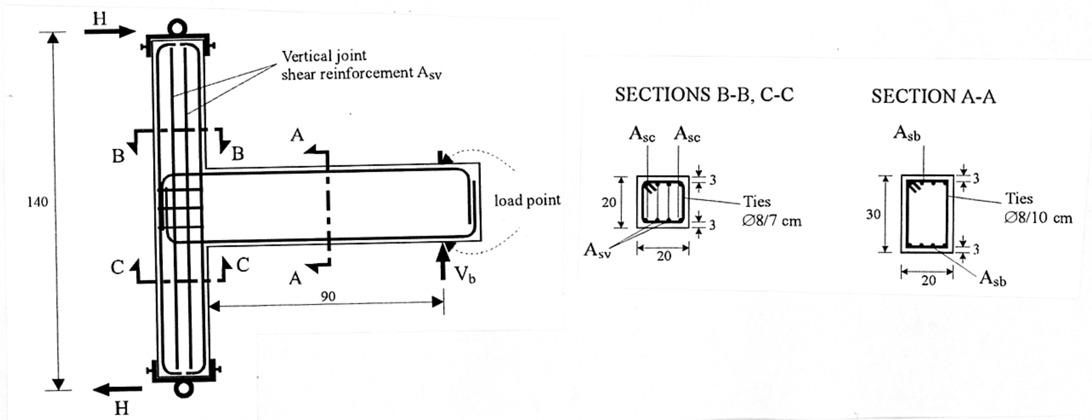


Specimen	O_1	O_2	P_1	P_2	P_3	M_1	M_2
A_{sb} (top or bottom beam reinforcement)	4Ø14	2Ø10+2Ø12	4Ø14	2Ø10+2Ø12	2Ø12+2Ø14	2Ø10+2Ø12	4Ø14
A_{sc} (column reinforcement (each face))	2Ø14	2Ø10	2Ø14	2Ø10	3Ø10	2Ø10	2Ø10
A_{sv} (vertical joint shear reinforcement)	–	–	–	4Ø14	–	–	–
M_R (flexural strength ratio)	1.03	1.10	1.04	1.04	0.97	1.29	0.72
γ (joint shear stress expressed as a multiple of $\sqrt{f'_c}$)	24.23	12.24	16.43	13.16	17.83	10.50	9.00

Fig. 1 Dimensions and cross-sectional details of original specimens O_1, O_2, P_1, P_2, P_3, M_1 and M_2

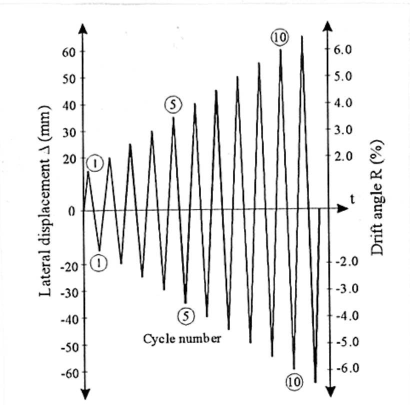
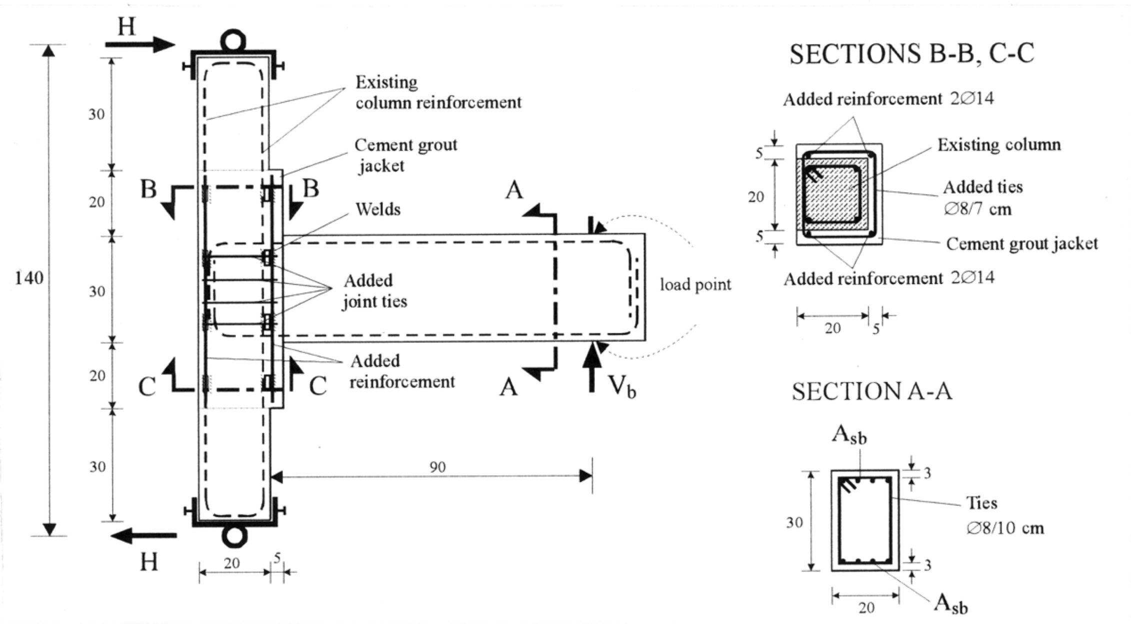
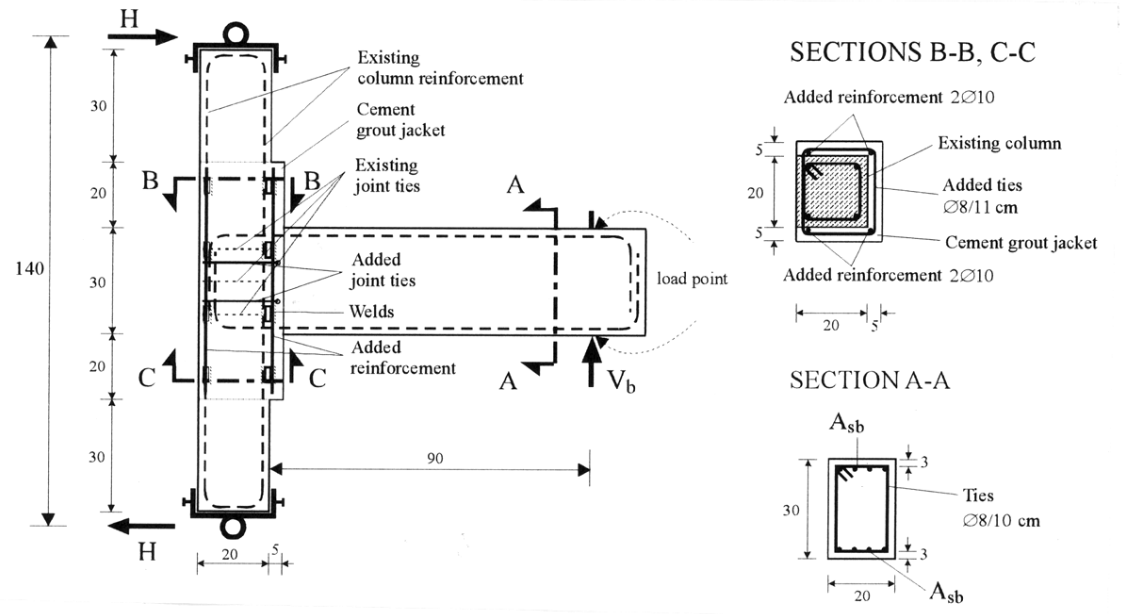


Fig. 3 Lateral displacement history



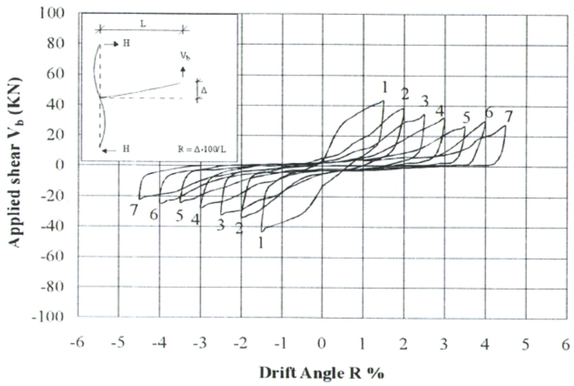
Specimen RO_1 $M_R = 2.50$

$\gamma = 6.15$



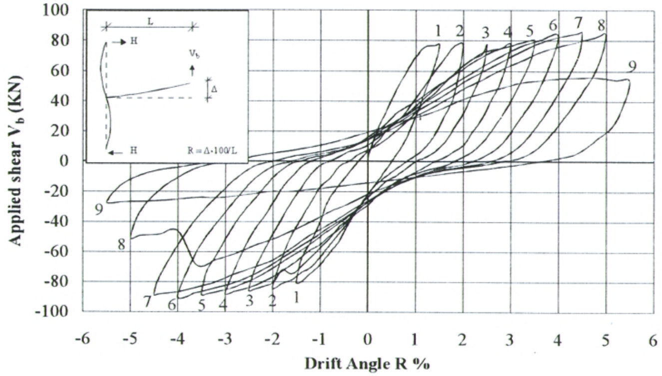
Specimen RO_2	$M_R = 2.50$, $\gamma = 3.80$	Specimen RP_2	$M_R = 2.40$, $\gamma = 3.80$	Specimen RM_1	$M_R = 2.50$, $\gamma = 4.10$
Specimen RP_1	$M_R = 1.90$, $\gamma = 6.60$	Specimen RP_3	$M_R = 2.00$, $\gamma = 5.40$	Specimen RM_2	$M_R = 1.70$, $\gamma = 6.70$

Fig. 2 (a) Jacketing of Beam-Column Connection of Subassemblage RO_1 (dimensions in cm, 1cm=0.39in), (b) Jacketing of Beam-Column Connection of Subassemblage RO_2, RP_1, RP_2, RP_3, RM_1 and RM_2 (dimensions in cm, 1cm=0.39in)



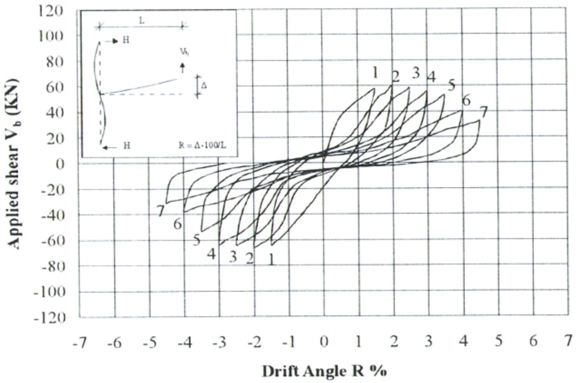
Specimen O_1

Specimen O_1



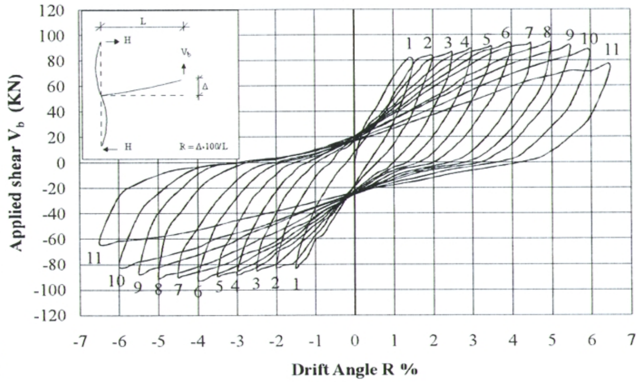
Specimen RO_1

Specimen RO_1



Specimen P_1

Specimen P_1



Specimen RP_1

Specimen RP_1

Fig. 4 Load deflection curves and cracking configuration of specimens O_1, P_1, RO_1 and RP_1