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Ruiz Pinilla, JG.; Pallarés Rubio, FJ.; Giménez Carbó, E.; Calderón García, PA. (2014). Experimental tests on retrofitted RC beam-column joints underdesigned to seismic loads. General approach. Engineering Structures. 59:702-714. https://doi.org/10.1016/j.engstruct.2013.11.008

The final publication is available at

http://dx.doi.org/10.1016/j.engstruct.2013.11.008

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Additional Information

TITLE PAGE

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Experimental Tests on Retrofitted RC Beam-Column Joints Underdesigned to Seismic Loads. Global results

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Abstract This paper presents an experimental program designed to determine the behaviour of steel jacketing used as a seismic strengthening system for reinforced concrete frame structures. Tests were carried out on a total of 20 full scale interior beam-column joints. Geometry and reinforcements were selected according to existing buildings, designed solely to gravity loads under strong beam-weak column concept. Column strengthening was performed in all specimens, and four different types of column-joint connection strengthening have been tested. Two types of beam reinforcement have been included in the experimental program. Tests were carried out by subjecting specimens to gravity and cyclic loads.

Keywords Cyclic test, Experimental test, RC beam-column joint, Seismic strengthening, Steel cage

1. Introduction

In order to ensure that reinforced concrete structures show good behaviour under seismic loads, the structural components must have a certain degree of ductility. As this concept was only introduced into building standards in the 70's, buildings erected before that time are not usually equipped to deal with earthquakes. This has been made manifest on numerous occasions in earthquakes such as those occurring in Northrige in 1994, Kobe 1995, Izmit 1999, Taiwan 1999, Bingol 2003, Sumatra 2004, Sichuan 2008 and L´Aquila in 2009.

A number of research groups [1-7] have studied the most commonly found causes of the failure of reinforced concrete structures under seismic loads. These include: soft stories, short columns, strong beam-weak column and deficient building practices, among others. However, the principal cause of building collapse is critical damage to columns and beam-column joints.

Retrofitting structures against seismic loads has now become a relatively common operation. A considerable amount is already known about how to strengthen isolated elements like beams and columns by means of different techniques; however the treatment of the beam-column joint is a more complicated task, due to the high concentration of loads on a relatively small area difficult to access in existing buildings.

At the present time several research groups in different parts of the world are studying the behaviour of the beam-column joint elements in reinforced concrete frame structures in order to improve its response to seismic loads. It is also advisable to submit existing underdesigned buildings to a study on seismic resistance to avoid structural damage during earth movements. The following section deals with a review of the state of the art of the research at present being carried out on this structural element.

1.1. Background: Tests and strengthening pattern

The earliest studies on the behaviour of beam-column joints in RC structures under seismic loads were carried out in the 60's and focused on the joint in isolation [8-11]. This meant the joint could be deformed without the restraints it would have had as part of a complete structure. During the 90's, experiments began to include other structural elements, such as columns, beams and the beam-column joint [12].

All the research carried out to date can be divided into three main groups. The first deals with failures in joints subjected to cyclic loads designed to withstand gravity loads only [13, 14]. These studies did not consider strengthening the joints but they did throw light on how they work, the deficiencies in present-day buildings and the key points that must be taken into account when designing this structural element.

The second group focuses on solutions that improve the seismic behaviour of joints in new buildings and carry out tests on new arrangements for strengthening the joint core [15, 16] and also on the use of special concretes [17].

The third group is involved with joints in existing buildings that are strengthened with the idea of improving their behaviour under seismic loads and can be roughly divided into two sub-groups. In some cases the strengthening is added before the tests [18-20] and in others the original element is first tested before being strengthened and re-tested [21,-23] to evaluate the results of the strengthening.

There are three main techniques for strengthening RC columns: concrete covers [24, 25], FRP composite materials [26-29] and steel jacketing [30, 31]. According to [32], the strengthening technique that has received most attention in the literature is the use of composite materials and most of the recently published papers deal with this type [33, 34]. However, in practice the most commonly used strengthening technique is by steel strips and angles (a variety of steel jacketing), which, as [35] point out, is fully effective in increasing the strength and ductility of RC columns.

Steel jacketing has been widely used in many European countries. For example, in the Czech Republic the technique had been used more than 5,000 times up to the year 2000 [36]. It was also generally applied in Greece after the earthquake disaster in Kalamata in 1986 [35]. It is also known to have been used in Japan to retrofit RC columns that had been affected by earth movements [24].It was used in Mexico to repair buildings after an earthquake in 1985 [37]. The CEB-FIB Bulletin Nº 24 [38] analyzed the different strengthening techniques for RC columns in high seismic risk areas and classified the steel angles and strips method as being among the most suitable.

According to the American Concrete Institute [39], when structural columns are being strengthened, one of the most important aspects is the treatment of the beam-column joint. Different studies have been carried out on this method of strengthening columns [31, 32, 36, 40-42] but very few on the behaviour of the beam-column joint in structures with steel strengthened columns subjected to cyclic loads [43].

1.2. Research objectives and significance

The main objective of this research is to determine the seismic behaviour of strengthened interior beam-column joints in RC frame structures originally designed solely to withstand gravity loads, therefore lacking in ductile details. The paper considers strengthening by steel jacketing on columns, and all the beam-column joint retrofitting have been designed in order to achieve that they are easy to fit to existing structures, as in general access to these zones tends to be difficult.

One of the main novelties of the paper is the way how cyclic loads are applied and their combination with gravity loads in such a way that the forces applied to the specimens are as near as possible to the real forces they would experience as part of a complete structure.

2. Experimental Program

Many variables with a strong influence on the final response of the specimen can be included in a test like the one described here: consideration/non-consideration of gravity loads both in beams and columns, level of gravity load, loading protocol, number of cycles, increments of loading between cycles, loading speed, force-control or displacement control, type and number of specimens and reinforcement, symmetry of the reinforcement, strengthening technique, data acquisition, data points, etc.

The variables included were intended to cover a wide range of cases. A high number of specimens were tested as compared to other studies, as we pursued specific goals with different strengthening techniques in order to acquire as much practical information as possible.

In this section, the insights, drawbacks, advantages, philosophy and criteria for the design of the experimental program are given from a critical point of view, in order to justify the decisions adopted and allow well-reasoned critical comparisons with other experimental tests and future designs in similar tests.

The authors of the article have a background of useful experience in similar tests performed on retrofitted elements of framed structures [31, 32, 40-43].

References and comments to other studies are given with the aim of a positive review of the experimental procedure here described, since we learned much valuable information from reading previous experiences. The key points of these references are highlighted and were taken into account in the design of the experiments described in the following subsections.

2.1. Details of specimens

2.1.1. Geometry

It is well known that when properly retrofitting a column in an underdesigned beam-column structure under horizontal loads, the failure usually shifts to the joint as the next weakest part. With the aim of studying the joint, the specimens represent a common interior beam-column joint in framed buildings with strong beams and weak columns. The columns are strengthened according to [31], so that the joint core and its interfaces with beams and columns become the next weakest parts. One of the first decisions we took was on the number and scale of the specimens.

Previous investigations [40-43] have shown the influence of the capital in the joint under monotonic load. Here, two tests (plain specimens without joint strengthening, A.W. specimens) were planned as reference tests to compare against the joint strengthening technique using only capitals (A.C. specimens). The latter were used as reference tests to compare against the strengthened specimens (all including capitals) and to study the behaviour of the joint under cyclic loads. We wanted to test three other strengthening techniques (apart from the use of capitals only), either because we had had experience with them or because we wanted to find the most efficient under earthquake loads. As two axial load levels, two types of beam reinforcement and at least two specimens for each technique were involved to allow for errors or accidents, an initial total of 19 specimens were planned.

The success (each of the two matching specimens in the tests had similar behaviour) and experience acquired from the tests led to a minor change in this sequence. One of the A.VB.L1 test was not carried out and was tested as A.C.L1, so that two capital-only specimens were tested (with and without axial gravity load in the column). The failed specimen was then repaired and retrofitted with vertical bars and re-tested, so a second A.VB.L1 test was performed and re-labelled A.VB.L1-R, which gave a total of 20 tests, as seen in Table 1.

The specimens geometry is based on the dimensions of seismically underdesigned real buildings from past decades, and is similar to other studies as shown in Table 2, except [17, 18] that conducted scaled tests. This is done using the points of contra flexure, which are approximately at mid span of the beams and columns in all the referenced studies.

The points of contra flexure are clear in the interior columns and are located at mid height when horizontal loads are acting on the structure. However, these points are not clear in the beams if the gravity load is considered, since gravity loads lead to contra flexure points located between one fourth and one fifth of the span, while they are at mid spans for horizontal loads. The variation in the position depends on the ratio between gravity and horizontal loads. Authors like [13, 17, 29], among others, assume the points of contra-flexure to be located at the mid-height of columns in two successive stories and the centre-points of beams in two adjacent bays, since gravity loads are not considered.

The present study considers the effect of gravity loads on the response of the frame to seismic loads, so that the experiments were designed in this way. Since the location of zero points is load-dependent, a compromise was reached by adopting a length between one half and one fifth of the span, avoiding the D-region close to the joint and adjusting the ends of the specimen so that the jacks could be attached to ground. Some authors [19, 22, 47] also consider gravity loads in their experiments, but no reference is made to this fact in the geometry of the specimens. Although this is not a major point, the authors are unaware of any information on this matter in the literature.

Fig. 1 shows the final dimensions of the specimens. When the accessories required for the test are in place the specimens measure 4000x2800 mm. These accessories are comprised of UPN steel pieces and plates used to link the specimens to the test frame and to strengthen the areas where the point loads are applied, avoiding failures in these parts due to load concentrations. Fig. 1 gives details of these accessories, which were clamped to the beam and columns by ϕ16 mm steel bars. The cross-section dimensions for beams and columns are typical of framed RC buildings constructed in Spain during the 80’s and 90’s (see sections A-A and B-B in Fig. 1), having 5 m span length and story height around 2.8 meters.

Table 2 shows some of the references considered in this paper and summarizes the main aspects of dimensions, type of reinforcement and strengthening technique, axial load and other information about the experiments conducted.

A final comment on the slab effect: different studies have shown that the slab effect significantly withstands horizontal loads. There are many methods of simulating lateral pressure and lateral confinement due to the presence of the slab: [22, 29] and [14, 47] include slabs on top of beams.

However, Beres *et al.* [22] showed that there was no significant effect on strength, stiffness degradation or total energy dissipation but that some of the damage shifted to the column. The authors alluded to in this and the above paragraph were the only ones to include the slab effect in their work. No slab effect was considered in the present work.

2.1.2. Material properties and construction

The specimens were made of reinforced concrete strengthened with structural steel.

When designing the materials used in the preparation of the specimens prior to strengthening, the idea was to obtain materials similar to those used in the 80's and 90's. The steel used in these RC buildings had an elastic limit of 400 MPa and a certain amount of ductile behaviour. B500SD steel bars (elastic limit 500 MPa) were used in the specimens since B400S were not available.

The usual design compressive strength of the concrete in these constructions was usually 25 MPa, but as the aim was not only to update underdesigned buildings for seismic loads but also to strengthen deteriorated beam-column joints, the specimens had different compressive strengths, ranging from 10 MPa to 25 MPa. This range covered both poor and normal concrete when studying the response of the specimen and the increase in ultimate load after strengthening. These different strengths could be either due to defective construction, or as a result of accidental loads such as earthquakes or because loads higher than those foreseen in the initial design have to be imposed on the structure (possibly due to a change in building use).

The values used by other research groups (see Table 2) were usually higher than 20 MPa, since the studies were performed on good performance concretes.

S275 structural steel with an elastic limit of 275 MPa was used for strengthening.

The specimens were cast in horizontal position using wooden formwork stiffened with steel bars (see Fig. 2), after which the specimen columns were strengthened by the different techniques described in the following subsection.

2.1.3. Reinforcement

Two types of beam reinforcement were used in the specimens: symmetric (S-type, Fig. 3) and non-symmetric (A-type, Fig. 3) since we had to cover two sets of conditions. Non-symmetric reinforcement was usually employed in buildings in the 80's and 90's for gravity loads, according to the current Standard, in which the bottom bars and some of the top bars overlap inside the joint core, while the rest are continuous. These steel reinforcing bars are known to have inadequate bond capacity against cyclic loads, due to lack of embedding in the joint. Symmetric reinforcement has no overlaps and aims to cover cases in which the reinforcement has been correctly designed to resist horizontal loads or the beam has been strengthened, so the failure will be forced to occur in the joint or the column, avoiding bar pullout.

In both cases the joints have deficient steel reinforcement to provide enough joint shear capacity for seismic loads, since no transverse steel hoops are specified in the joint core.

The beams in the S-type specimens had five 20 mm diameter bars at both top and bottom and two 8 mm diameter hoops spaced at 100 mm.

The A-type specimen beams had two 12 mm diameter bars at the top corners and two 16 mm diameter bars at the bottom corners, overlapping the joint core by 250 mm. Three additional continuous 16 mm diameter bars were placed at the top, crossing the joint continuously at the point of maximum moment during lateral loading in both beams and columns (see Fig. 3).

Other authors such as [13] designed similar schemes, but overlapping outside the joint. Li *et al.* [14] try to reproduce as-built joints from existing buildings, but the overlapping is outside the joint. There is no overlapping in [28] or in [15]. Pantelides *et al*. [29] point out that few studies address the pullout of beam bottom steel bars in the beam-column joint region, usually due to inadequate embedding in the column. The A-type described here is typical of the 80's and 90's and includes failure by pullout of beam bottom steels bars due to inadequate anchorage length in the joint core under cyclic load. As already mentioned, transverse steel hoops are not included in the joint in any of the studies cited in this paper.

The columns were reinforced in the same way for all the specimens according to the Standard, but no lap splices were designed above the beam level (Fig. 3). The reinforcement consisted of one 12 mm diameter bar in each corner of the square column and transverse reinforcement based on 6 mm stirrups with 150 mm spacing.

Some authors consider these splices in their studies [29, 22]. However, the latter states that the existence of splices in the beam leads to failure due to lack of bonding, with good performance from the column splices.

Also, since this area is under additional compression due to the confinement effect of the steel cage, which improves the bond capacity of the longitudinal bars, and also due to the large number of tests planned, it was decided to avoid lap splices above the beam in order to reduce the number of test variables.
Table 2 shows the use of symmetric and non-symmetric reinforcement in the studies. Since the reinforcement was varied for each study and also for the sake of brevity, the reinforcement type is not included in this table. They do not differ to a great extent, except for the tests performed by [28] and [13], which present a much higher steel ratio than the other studies.

2.2. Strengthening techniques

The starting point was a specimen in which the column had been strengthened for gravity loads according to the design rules proposed in [31]. As commented previously, these columns were strengthened in such a way that the joint core and its beam and column interfaces became the next weakest parts. The initial steel cage had longitudinal angle sections L70.7 fixed to the corners of the column, to which rectangular transverse steel strips were welded. The space between cage and column was filled with cement to improve axial, shear and bending capacity. The columns were strengthened by angles and rectangular strips measuring 280x160x8 mm and 260x120x8 mm. The cement mortar between cage and column had a cement/sand ratio of 1:2. This can be observed in Fig. 4a, where the steel strips are closer to the ends of the columns due to the maximum bending moment in this part of the columns.
Besides the initial reference specimen, four different strengthening techniques were tested for cyclic load, the aim being to design easy to install practical strengthening without damaging the existing elements.
- C-series: This series used L100.14 steel angles with stiffeners (capitals) at the corners between beams and columns, as can be observed in Fig. 4b. The dimensions and layout were based on the study by [43].
- CA-series: This series had eight 16 mm diameter bars joining the capitals to the beam through chemical anchors (Fig. 4c).
- VB-series: This series had four 16 mm diameter exterior vertical bars with threaded ends, joining the opposing capitals through the outer part of the beam (Fig. 4d).
- DB-series: This series had four 16 mm diameter exterior diagonal bars joining the opposing capitals through the outer part of the beam (Fig. 4e).
The second and third series (CA and VB) were a cyclic version of the monotonic tests performed in [43], which achieved good results for monotonic loads. The aim of this series was to maintain beam and capitals in

contact, increasing confinement in the region and improving bonding. The vertical bars were designed to force both capitals to work together (a step forward in the evolution of chemical anchors), giving similar behaviour to a steel-concrete composite structure.

The third strengthening technique was novel. Au *et al.* [15] identified the forces acting on the beam–column joint under seismic actions and the effectiveness of using diagonal reinforcing bars in the joints (in agreement with [44, 45]. With this in mind, we devised a strengthening technique with exterior diagonal bars on the assumption that they could contribute to joint shear resistance when the compression struts were alternatively developed, and would also be helpful in transmitting tensile stresses between opposite sides of the column reinforcing bars.

It is worth mentioning here that the strengthening bars were placed in the outer position in the VB and DB series to avoid damage to the interior reinforcement when drilling. In this way the joint core was not weakened and the longitudinal reinforcing bars were untouched.

The authors are unaware of any steel strengthening techniques in beam-column elements focused on the joint tested under cyclic loads. Corinaldesi and Moriconi [46] can be cited here, but they tested an external beam-column joint under cyclic load focusing on sustainable concrete with an overdesigned steel cage only to avoid spalling in the joint area and to induce damage to the beam. Most of the scientific literature relating to cyclic or seismic strengthening in the joint involves FRP, so comparisons with other strengthening techniques using steel cannot be made here.

2.3. Instrumentation and data acquisition

Instruments were placed at different points on the specimens to record strains, displacements and loads at all times. Ad-hoc software was developed to follow the experiments in real time, monitoring parameters of interest and giving the operator direct control. Fig. 5 shows the two computers employed in the experiment. One of them was used to control inputs (hydraulic equipment and jacks) and the other controlled important test outputs through numerical and graphical data.

Strain gauges, linear variable displacement transducers (LVDT's), load cells and digital information were used to follow the response of the specimens to load. They were located on the reinforcement, strengthening and concrete, as described below. The instrumentation was varied according to specimen type (A or S) and strengthening technique.

- Reinforcement: strain gauges were located at several points and on different longitudinal bars. Up to 14 points were monitored in the columns, 27 in A-type, and 22 in the S-type beams. One of the main goals was to study bonding evolution between bar and concrete throughout the reverse cycles, since bonding degradation has been observed by many researchers [22, 15, 14].

- Strengthening: 14 strain gauges were placed at several points of steel cage, angles and plates, as well as on strengthening bars.
- Concrete: the joint was equipped with 6 LVDT's forming a square with two diagonals.

They monitored joint distortion from its original shape. This instrumentation was removed at the first pause, at 1% drift, to avoid being damaged, since heavy degradation was experienced in the joint in some specimens from this point onwards. Additionally, digital image correlation was used to obtain displacements at any desired point in the joint. A camera was mounted in front of the joint and programmed to record a sequence every 1.5 seconds. A few points were selected and marked in the specimens and displacements were registered by comparing two images. Lee *et al*. [28] used digital image correlation, measuring the full joint strain field.

The sample rate was seven measurements per second for the input control equipment and two measurements per second for the output equipment. The input equipment needed a higher rate to control the jacks and the reversing cycles. The rates were kept constant throughout the tests and gave continuous control of the equipment. The large amount of data registered was stored in a computer for the post-processing phase.

2.4. Loading protocol

Another important issue in the experimental program is the loading procedure. The aim was to reproduce as closely as possible the real situation when building frames are subjected to earthquake loads. This was a key point in the development of the tests, since trying to reproduce real conditions makes the loading protocol extremely complicated, as explained below.

Two types of loads were considered in the tests: dead and live gravity loads and cyclic seismic loads.

2.4.1. Gravity loads

No gravity loads were put on the S-type beams, since they had no 'real' reinforcement. Their reinforcement was overdesigned to prevent beams failure.

The A-type beams were preloaded with point loads at the ends simulating gravity loads. A load value of 30 kN was applied, taking into account all the loads acting on beams in a normal floor. The point load generates a bending moment equal to the one produced in the joint by gravity during seismic loading, so that the shear force is accurately reproduced in the joint. Few authors consider the gravity load in their experiments. Beres *et al.* [22] consider preloading with 89 kN (20 kips) simulating constant dead and service loads at the ends of each beam, while Benavent-Climent *et al*. [47] placed 40 kN as four point loads (sand bags). Prota *et al.* [19,20] applied gravity loads as point forces (40.1 kN) at the ends of the beams, since this value generates a maximum flexural moment equal to half the design moment, obtained according to ACI provisions. Hakuto *et al.* [16] preferred to impose a prescribed vertical displacement at the base of the column.

Gravity loads can be exerted on columns in both beam types through constant axial compression by means of a jack at the top of the frame; other possibilities are the use of tensioned rods or cables (e.g.[47]), although the use of prestressing tendons sometimes makes it difficult to keep this value constant [13]. As can be seen in Table 2, some specimens (labelled x.xx.L1) were given an axial compression load 0.3 times the strength capacity of the column, while the rest of the specimens had no axial compression on the columns. This represents the gravity loads acting on columns located close to the base of the building from the floors above the columns or close to the upper floors (no axial load). It was important to determine the influence of axial compression on the response of these columns in a building and the effect of this compression on the retrofitting techniques tested.

In the authors' opinion, the gravity load, especially on the beams, must be considered if the real situation is to be reproduced as it has a great impact on the results obtained. However, it does add to the experimental complications as force-controlled and displacement-controlled tests are usually required, as explained below.

2.4.2. Cyclic loads

Cyclic loads can be introduced into the experiments in two ways: on the columns or on the beams (see Fig. 6 and Table 2). The latter is a more complicated process, requiring two coordinated actuators, but the gravity loads can be kept constant and complete control of the load and displacements applied to the beam ends is maintained, as explained below.

The loading protocol can be configured in different ways when considering gravity loads. One of the main problems is to keep the gravity load constant and to reproduce the real phenomena of an earthquake.

Of the four works previously mentioned, [22] decided on a two-phase test; in the first the gravity load was kept constant through a force-controlled test in four sets of increasing load with three cycles per set, and the reversal movement was governed by the prescribed load values in each jack. In the second phase a displacement-controlled test was performed while keeping the algebraic sum of the beam forces (initial preload) constant and increasing the values of positive beam rotation, although no specific details are provided about these increments. It can be presumed that the displacement-controlled test is so called because the reversal movements are applied when the prescribed drifts are attained, but the test is still force-controlled. This is a key point, since it allows the initial gravity load to be kept constant but has the disadvantage that if a plastic hinge is developed in one of the beams, this will concentrate most of the rotation and will reach the prescribed drift, limiting the capacity of the other beam to provide more strength. This deduction is based on the fact that the plot representing the beam shear forces are symmetric with respect to the 89 kN (20 kips) value, while the beams do not have symmetrical reinforcement.

On the other hand, Prota *et al.* [19, 20] made two force-controlled phases, switching the load configuration after reaching zero shear force at one of the beam ends. This way of controlling the tests makes it impossible to record the softening branch of the envelope, as can be seen from the figures in their papers, where a divergent response is obtained after the peak load is reached.

Benavent-Climent *et al.* [47] and Hakuto *et al.* [16] both perform displacement-control tests, capturing the stiffness degradation beyond the peak load.

In the present paper, the authors decided to do a mixed control test on the A-type specimens, with force and displacement strictly controlled, mainly influenced by the incorporation of the gravity load, to reproduce seismic conditions as closely as possible. As the S-type specimens did not include gravity loads, only displacement-control tests were conducted. Three cycles per set were chosen to facilitate identification of a steady state (no stiffness degradation with increasing cycles) in each set. Some authors, such as Zerbe and Durrani [48] or Beres *et al.* [22] prefer to interpose small cycles during the test to check variations in stiffness.

Fig. 7a, Fig. 7b and Fig. 7c show the loading protocol in A-type specimens, where the two phases are clearly displayed, separated by a break during the test, which was necessary to update the system while shifting from force-control to displacement-control.

Fig. 7a presents drift versus time. Drift is defined as the algebraic difference of the two beam actuator displacements divided by the distance between them. Fig. 6 displays the equivalent column drift between the two usual ways of testing specimens using cyclic loads.

It was decided to conduct a slow first phase, when the movements are still small and the specimen is undamaged, and a faster second phase to avoid a long time-duration test, since the displacements of the damaged specimen are considerable at higher drifts. In the first phase it was decided to keep a constant slow movement of 240 seconds per cycle in displacement-controlled tests, while the load rate was kept constant at

0.33 kN/s in the force-controlled test. The drift was increased every three cycles in steps 0.25%, 0.5%, 0.75% and 1% to facilitate the observation of specimen degradation. The second phase was always displacement controlled with a faster movement of 120 seconds per cycle, increasing drift in increments of 0.5% until failure.

In the A-type specimens the first phase was conducted under a force-controlled test to guarantee constant application of the gravity load, stopping at 1% drift while the specimen was still relatively undamaged. It was known from other studies that drifts between 1% and 2% considerably damage the specimen, and are seldom higher than 3% in real buildings. However, the tests were conducted until total specimen degradation or jack limitations (6% drift or higher). The second phase was from 1% drift to failure, and was conducted under displacement-control to register stiffness degradation and softening. In this phase the gravity load depends on the degradation of the specimens, as can be seen in Fig. 7c. Fig. 7b shows the initial displacement required to obtain constant gravity load during the first phase and how it increases over time with specimen degradation, since it is a force-controlled test. In the second phase the displacement symmetry is clear under a displacement-controlled test, while the response in terms of forces is non-symmetric. By mixing the tests in this way, the gravity load is kept constant during the first phase and plastic redistribution of forces is allowed during the second phase, which is somewhat more realistic.

In the S-type specimens the two phases were displacement-controlled, as previously commented. Fig. 7d shows the symmetry of the prescribed displacements in both phases. Fig. 7e shows the drift applied over time, with no break between phases. Fig. 7f shows the force applied by one of the actuators, with stiffness degradation within each set of prescribed drift and symmetric behaviour. The rest of the test conditions are similar to those of the A-type specimens, described above.

2.5. Test set-up

Summarizing, the main goal in the experimental phase was the setup of a cyclic test for real specimens in existing buildings in conditions as close as possible to actual ones.

A self-reacting steel frame was constructed and attached to the floor (Fig. 8a), in which the specimens were placed in for the tests (Fig. 8b).

The end plate of the upper column of the specimen was attached to a sliding device that allowed rotation in the loading plane and vertical movement, constraining horizontal movement and was used to apply the vertical load through an upper 1000 kN compression jack when required. The end plate of the lower column of the specimen was attached to a fixed point, while the end plates of the beams were pinned joints simulating the inflection point, with 250 kN, tension-compression jacks operating independently with 500 mm of maximum displacement to impose the reversed cyclic load (see Fig. 9). The use of pinned joints is common in this type of test [22], although some authors such as [13] clamped the lower part of the column to the frame. The moment applied by these jacks is resisted by an opposite moment created by the horizontal reactions of the top and bottom hinges, according to the scheme in Fig. 6b.

2.6. Critical review

This subsection presents a critical review, brief summary and comments on important points in the design of an experimental program similar to the one described here.

One of the elements that could be included in a program (not included here) is putting lap splices in the column reinforcement, but this would involve a large number of variables, as commented in subsection 2.1.3.

Another important point is the fact that testing as shown in Fig. 6b, fixing the column ends, prevents considering P-Δ effects in the element. In the authors' opinion this would lead to moving the column end, as shown in Fig. 6a, complicating the test, since a horizontal-moving and always vertical jack at the top of the column would be required in the testing frame. The use of tensioned cables could be argued, since the direction of the load is not kept vertical. If gravity loads are not simulated in the test (as in many tests in the scientific literature), this must be considered when interpreting the results and conclusions.

The slab effect was not considered here (see subsection 2.1.1).

As will be shown in the following section, the external strengthening bars were 16 mm, but the failure was always at the threaded end, so care must be taken at this point since the real bar diameter is governed by the thread (approximately 12 mm in the present study).

The axial force induced on the beams in a real earthquake was not taken into account. None of the references cited in this paper considers this effect, which is more important in exterior than interior columns [48].

3. Global Results

This section contains the global results of the experimental program. It indicates the differences between the tests with and without gravity loads, gives the results of the comparisons between the specimen responses and also shows the influence of the different variables studied together with comments on the main specimen failure modes.

3.1. Significant features of tests with and without gravity loads

In order to highlight the special features of the S-type (which were tested with cyclic loads only) and A-Type specimens (with both cyclic and gravity loads), we give here the hysteresis cycles of one specimen of each type. The graphs in Fig. 10 show S-Type and A-Type specimens. The figure contains graphs showing the stories drift *vs* shear force on columns (V_c) and the story drift *vs* the shear force applied by the hydraulic jacks on the beams (V_1 and V_2). The 'V_c'definition can be seen in Fig. 6b.

In Fig. 10a y Fig. 10b (S-Type beams) a high degree of symmetry can be seen between the columns response (Fig. 10a) in both directions of the cyclic movement and also in the response of both beams in both directions (Fig. 10b).

For the test on the specimen with Type-A beams and gravity load (Fig. 10c y Fig. 10d) a symmetrical response of the column can also be seen (Fig. 10c), but not in the beam response (Fig. 10d). This is due to two reasons: (i) the beam reinforcement is not symmetrical on its upper and lower faces and (ii) neither are the cycles applied, since the values of V_1 and V_2 start with the value of the gravity load applied on the beams (30 kN).

As the test progresses, the forces on the beams increase both positively and negatively and the bending moments become inverted when the cyclic loads rise above the value of the gravity loads on the beams. In this case the load inversion generates tensile forces in the lower beam reinforcement and brings its overlap capacity into play. The degradation of the specimens under cyclic loads ends by reducing the reinforcement bonding inside the joint. It should also be noted that the loss of strength is faster in the direction of positive than negative moments.

The effect of the axial gravity load on the column, in the form of hysteretic cycles, can be seen in the comparison between both tests. When this load is not applied there is a marked pinching effect (Fig. 10a y (b)) which is not evident in the test shown in Fig. 10c y Fig. 10d in which an axial load is applied. The axial load also makes the specimen behaviour more brittle and causes faster strength degradation and loss of ductility.

3.2. Load-displacement hysteretic results

Fig. 11 shows graphs obtained from different specimens in which the hysteretic responses are compared in pairs, column shear force versus story drift.

In the reference specimen (A.W.L0-2), in which the column is strengthened without a column-joint connection, the column reinforcement yields while the concrete remains almost intact, keeping the strength with high ductility. The use of capitals connected by vertical bars increases the load capacity of the specimen to 2.5 times that of the reference (Fig. 11a), but there is not such a great difference between the types of external strengthening when high axial loads are exerted on the columns (Fig. 11b).

The mere fact of using capitals increases maximum load by 63% and gives similar ductility. When an axial load is applied to the column, the specimen is seen to withstand a considerably higher load but loses ductility (Fig. 11c).

The failure of the specimens with Type-S beams and column-joint connections by capitals and diagonal bars is due to the breaking of the bars, which causes a sudden loss of the column's bending capacity. After this point its behaviour is similar to the specimen equipped with capitals only (Fig. 11d).

Fig. 11e shows the different load capacities between specimens with A and S-Type beams, which were strengthened in the same way (column-joint connections with capitals and vertical bars) in which axial loads were not applied to the columns.

The repair carried out on specimen A.C.L1-S after its test not only increased its ultimate load but also made it more ductile (Fig. 11f), thus improving the seismic resistance of the original specimen and transferring the failure to the beams. The joint of the original specimen was seriously damaged during the test and the repair involved replacing the damaged concrete by high-strength mortar and welding the discontinuous beam reinforcement. External vertical bars were also added to connect capitals and columns.

3.3. Failure modes

The specimens were tested under a strong-beam weak-column design. In this type of structure the failure usually takes place in the columns, which fail due to lack of either confinement, bending resistance or shear resistance, putting the structure's stability at serious risk. Using steel jacketing on the columns is an efficient way of avoiding damage to them and transfers the failure to the next-weakest zones.

During the experimental program we found damage to columns where they meet the joint, also in the beams and in the joint core. Damage was also detected in the strengthening elements used for connections, such as in external bars and chemical anchors. A selection of photographs of the failures can be seen in Fig. 14.

When the column is strengthened by angles and strips with no external connection to the joint and no axial load is applied, failure is due to yielding in the column-joint connections (Fig. 12a) as the weakest part of the strengthened column. Most of the deformation is concentrated in these sections.

When using capital-only or chemical anchors, it is worth noting the marked separation of the capital from the beam on which it was resting as a result of the concentration of deformation in the interface section (Fig. 12f). After various cycles this causes the failure of the chemical anchors.

The large cracks in the column-joint connection may also have caused the failure of the diagonal bars used to connect with the columns (Fig. 12e), which were not able to prevent or resist the deformation.

In other situations in which the column and its connections were strong and ductile enough to withstand bending (Fig. 12c) considerable damage caused by shear force was observed.

When an axial load is applied and the column connections make the column and joint stronger than the beams, the latter tend to suffer bending failures on both their upper and lower faces, as seen in Fig. 12b.

Fig. 12d shows the damage to specimen A.VB.L1-R, which was the repaired and strengthened A.C.L1-S that had sustained a severely damaged joint. On retesting, the damage was seen to move away from the joint with the failure finally occurring in the beams.

4. Conclusions

This research was conceived from the need to make good the lack of information on the behaviour of steel jacketing used to strengthen RC columns against cyclic loads, with special attention to their effects on the beam-column joint.

This paper gives a detailed description of the experimental program designed for this purpose and the global results obtained from the tests carried out.

Twenty tests were carried out on full-scale internal beam-column joints to represent the seismic behaviour of an RC frame structure. The specimen geometry and reinforcement were designed to represent typical structures planned to resist gravity loads only, with no allowance for seismic loads under a strong-beam weak-column concept. All the columns were strengthened in the same way, the only variations being in the system of connecting columns and joints.

Before planning the tests we reviewed the existing bibliography on research studies on the behaviour of joints, the various ways of strengthening them to improve their seismic response and on the different loads applied to the specimens. A summary is included in this paper, together with all the references. When designing the experimental program, special attention was given to obtaining conditions as close to reality as possible. The aspects considered included:

- Beam and column cross-sections and lengths were chosen according to the location of the inflection points for the application of point loads during the tests.
- Maximum and minimum axial load values were applied to columns so as to include columns from both upper and lower stories.
- Application or absence of gravity loads on beams according to beam reinforcement.
- Gravity loads considered together with cyclic loads. This is an extremely important aspect, since the relationship between these two load values affects the beam stress state and the transmission of shear forces through the joint.
- Joint strengthening was designed considering access to joints in existing buildings with a view to ensuring practical applications.

Considerable differences were observed between the tests according to the type of strengthening used to connect columns and joints, type of beam reinforcement and the combinations of loads applied to the specimens. Using steel jacketing on columns prevents column failure, which is transferred to the next-weakest zones.

The experimental program proposed in this paper therefore makes it possible to study the behaviour patterns under cyclic loads of steel jacketing and the systems used to connect columns with joints, as well as to obtain improvements in the behaviour of joints under shear forces and in the reinforcing bars bonding of beams and columns in the joint.

Acknowledgements The authors wish to express their gratitude for the financial support received from the Spanish Ministry of Science and Innovation under Research Project BIA 2008-06268.

Thanks are also due to Jose M. Adam for his invaluable assistance from the beginning of the project.

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Figuras

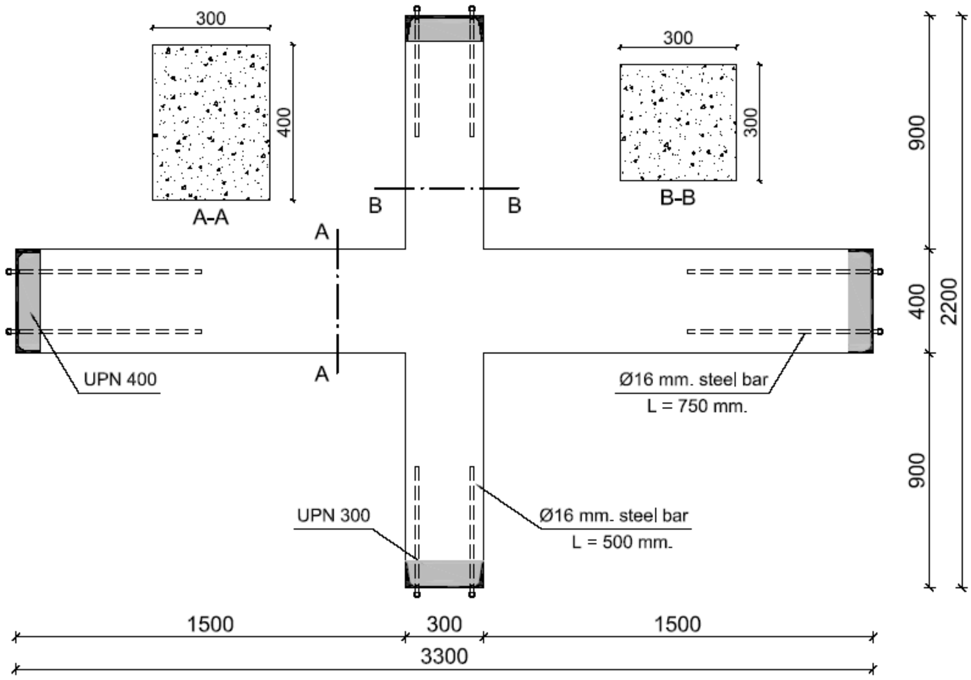

Figure 1. Specimen used in the tests

Figure 2. a) Specimen ready for concrete pouring, b) specimens strengthened with steel cage

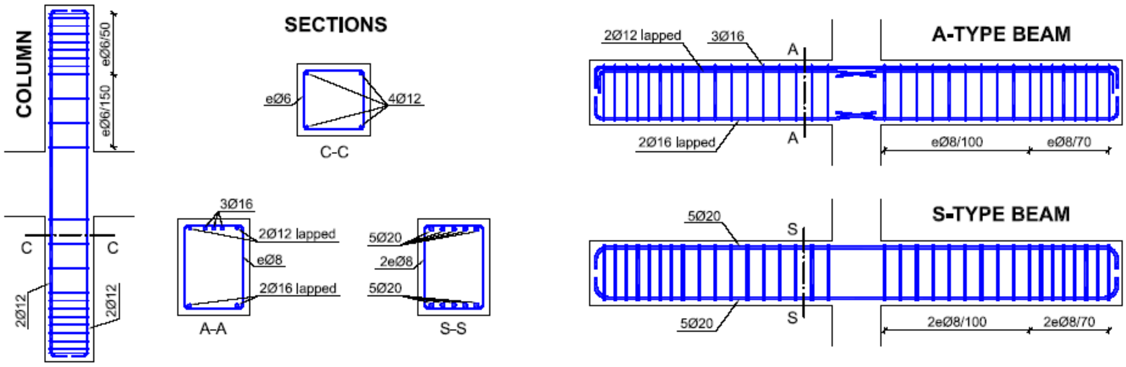

Figure 3. S-type beam reinforcement, A-type beam reinforcement and column reinforcement

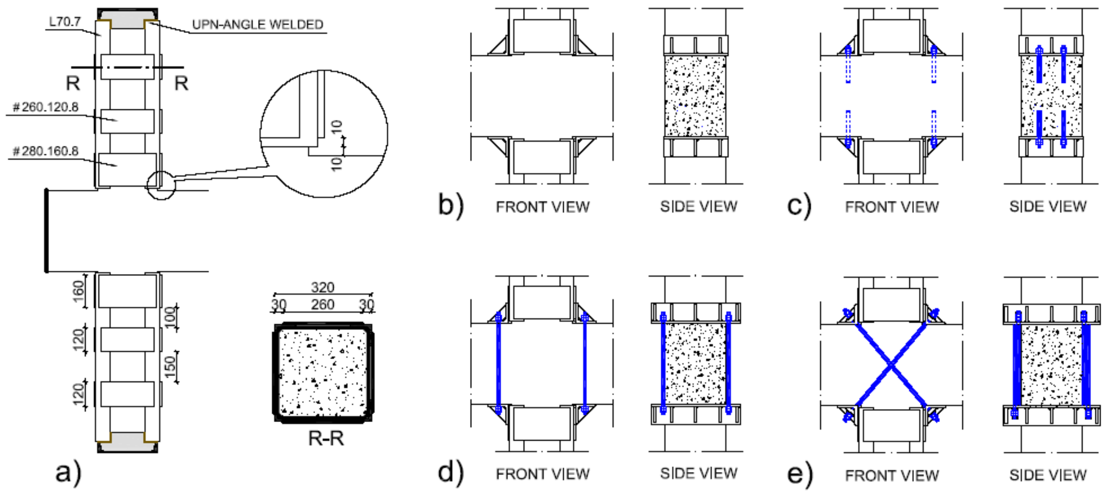

Figure 4. Strengthening columns and joints. a) columns, b) C-series, c) CA-series, d) VB-series, e) DB-series

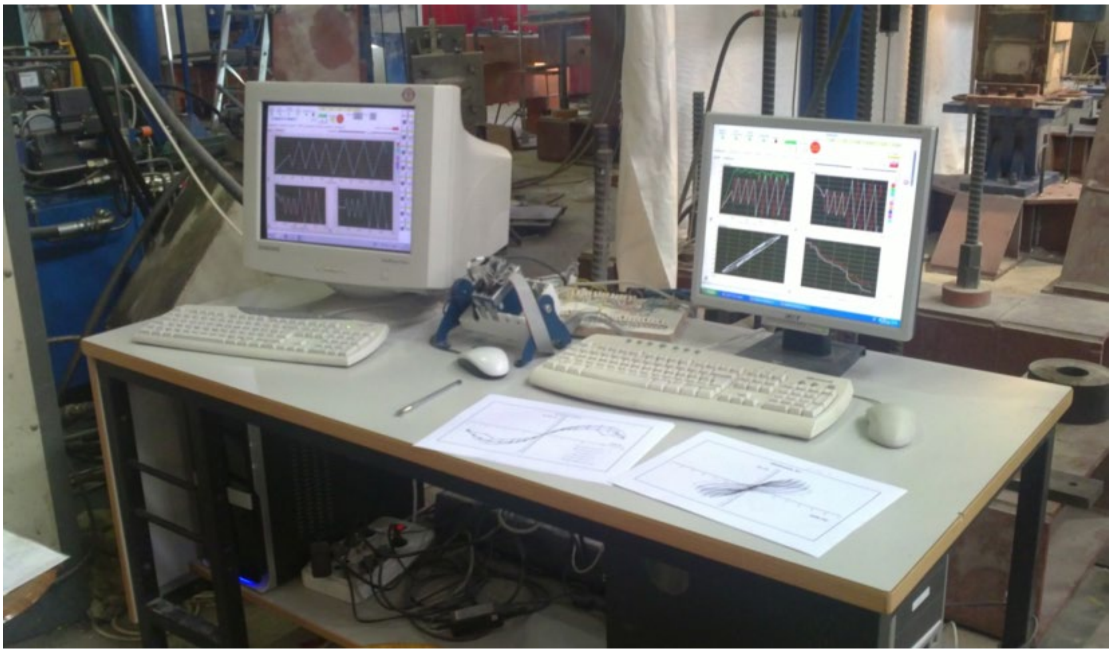

Figure 5. Computers giving full control of input and output parameters

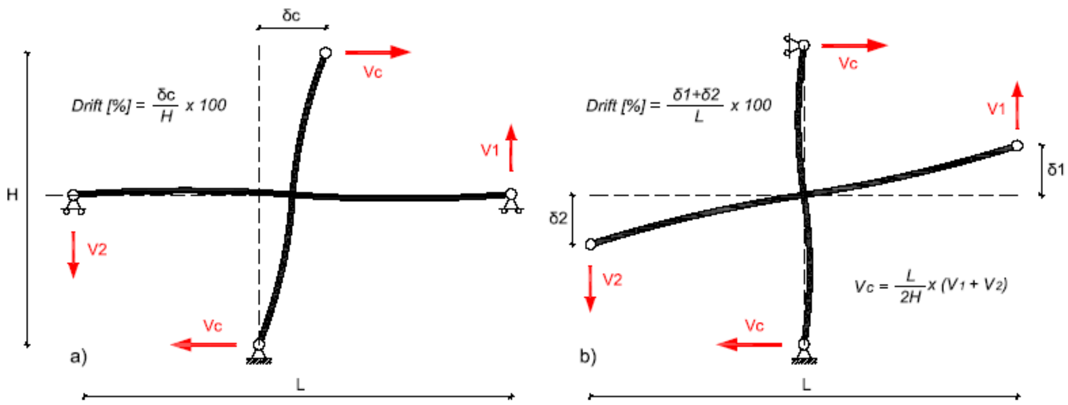

Figure 6. Drift expression and cyclic forces applied. a) on columns, b) on beams

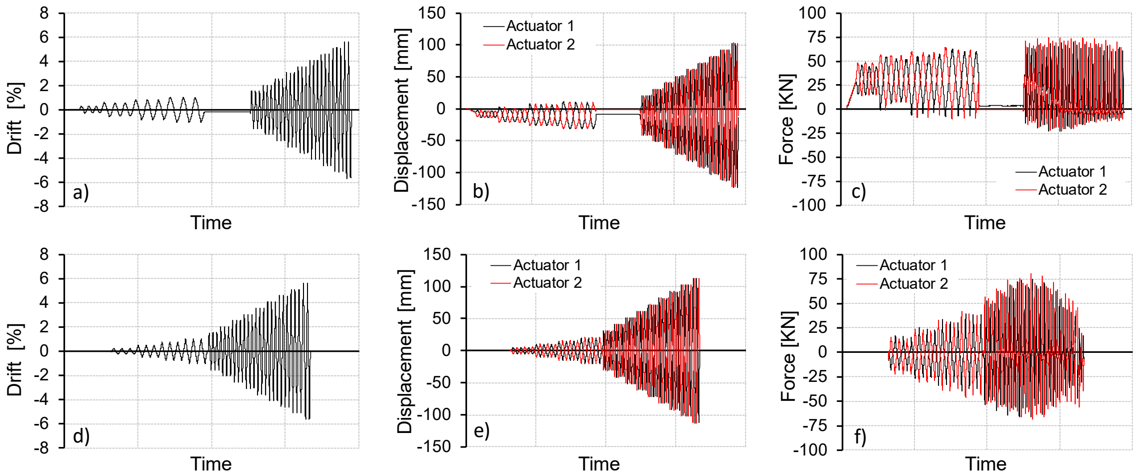

Figure 7. Loading protocol. a) A-type Drift vs time, b) A-type Displacement vs time, c) A-type Force vs time, d) S-type Drift vs time, e) S-type Displacement vs time, f) S-type Force vs time

Figure 8. a) Frame attached to floor, b) Specimen set-up ready for test

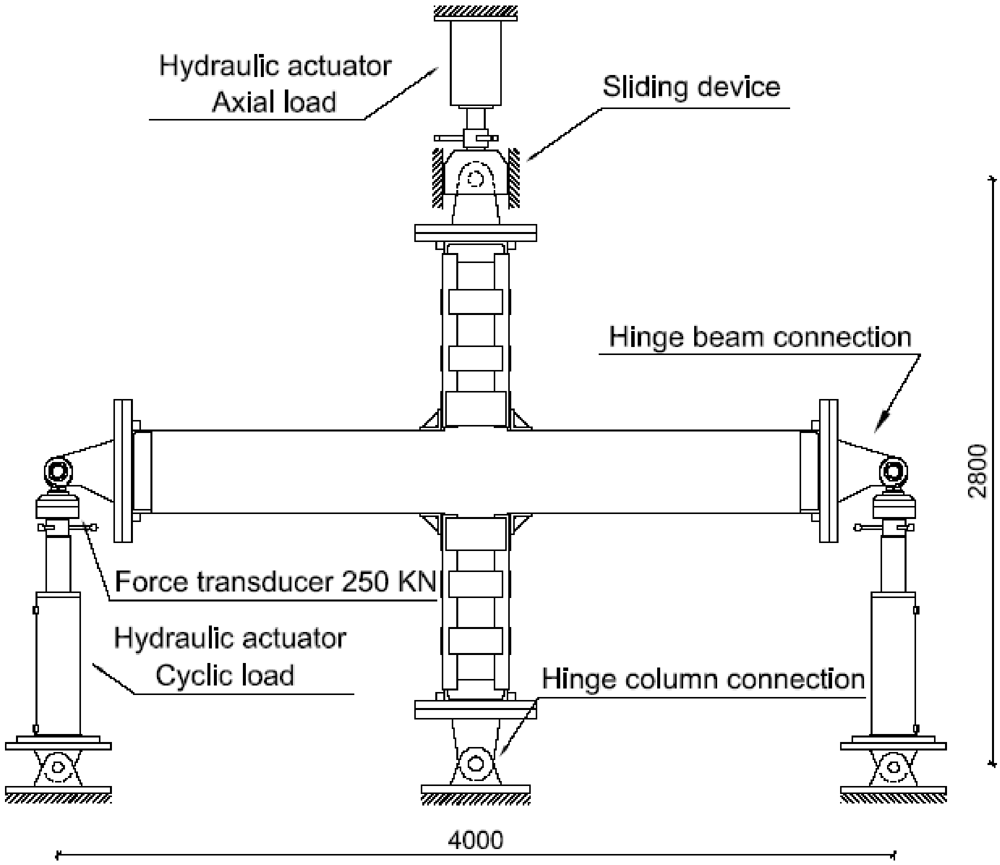

Figure 9. Schematic details of set-up

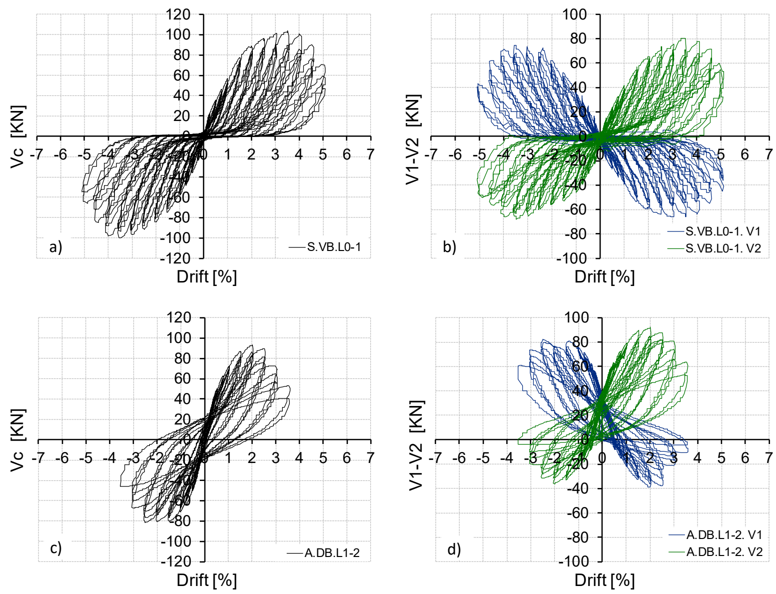

Figure 10. Hysteretic loops of test without (specimen S.VB.L0-1) and with (specimen A.DB.L1-2) gravity load on beams. a,c) Column shear force versus storey drift. b,d) Beam shear forces versus storey drift

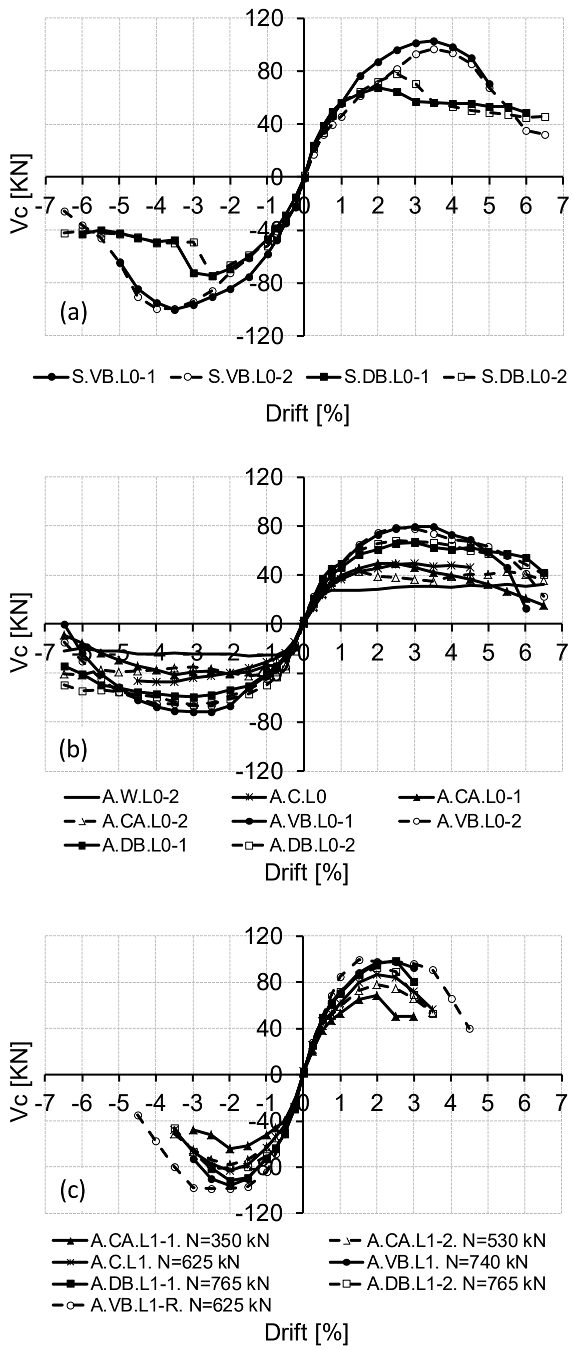

Figure 11. Lateral load–displacement envelopes for all specimens. a) Specimens Type S. b) Specimens Type A, N=0. c) Specimens Type A, N≠0

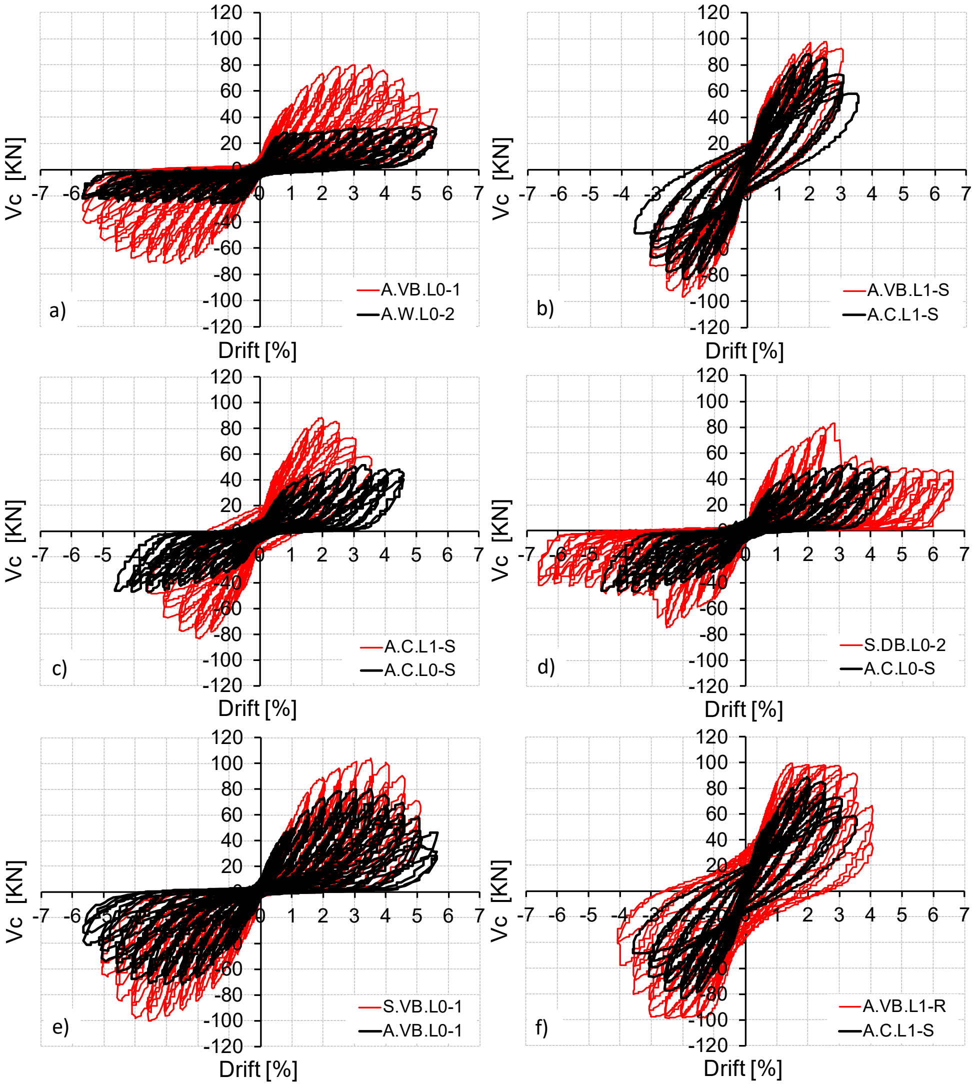

Figure 12. Column shear force versus storey drift. a) Joint connection effect, b) Specimens with axial load, c) Axial force effect, d) Diagonal bars effect, e) Beam reinforcement effect, f) Improvement after repair

Figure 13. Photographs of principal failure modes. a) Bending failure in column-joint connections (specimen A.W.L0-2), b) Positive and negative bending failure in beams (specimen A.DB.L1-1), c) Shear failure in joint (specimen VB.L0-1), d) Failure transferred to beams after repair (specimen A.VB.L1-R), e) Tensile failure of external bars (specimen S.DB.L0-1), f) Pull-out failure of chemical anchor (specimen A.CA.L0-2)

Number	Specimen Code	Beam Type	Strengthening	Axial Load Level (P/P_{ult})*
1	A.W.L0-1	Asymmetrical	Reference	0.0
2	A.W.L0-2	Asymmetrical	Reference	0.0
3	A.C.L0-S (single)	Asymmetrical	Capital	0.0
4 (4.1)	A.C.L1-S (single)	Asymmetrical	Capital	0.3
5	A.CA.L0-1	Asymmetrical	Capital + Chemical Anchor	0.0
6	A.CA.L0-2	Asymmetrical	Capital + Chemical Anchor	0.0
7	A.CA.L1-1	Asymmetrical	Capital + Chemical Anchor	0.3
8	A.CA.L1-2	Asymmetrical	Capital + Chemical Anchor	0.3
9	A.VB.L0-1	Asymmetrical	Capital + Vertical Bar	0.0
10	A.VB.L0-2	Asymmetrical	Capital + Vertical Bar	0.0
11	A.VB.L1-S (single)	Asymmetrical	Capital + Vertical Bar	0.3
12 (4.2)	A.VB.L1-R (repaired)	Asymmetrical	Capital + Vertical Bar	0.3
13	A.DB.L0-1	Asymmetrical	Capital + Diagonal Bar	0.0
14	A.DB.L0-2	Asymmetrical	Capital + Diagonal Bar	0.0
15	A.DB.L1-1	Asymmetrical	Capital + Diagonal Bar	0.3
16	A.DB.L1-2	Asymmetrical	Capital + Diagonal Bar	0.3
17	S.VB.L0-1	Symmetrical	Capital + Vertical Bar	0.0
18	S.VB.L0-2	Symmetrical	Capital + Vertical Bar	0.0
19	S.DB.L0-1	Symmetrical	Capital + Diagonal Bar	0.0
20	S.DB.L0-2	Symmetrical	Capital + Diagonal Bar	0.0

* P_{ult} is the ultimate load of the column under pure compression

Table 1. Number and properties of the tests performed.

	Total number of specimen (interior joints)	Cross section dimensions (mm)		Specimen dimension (mm) Wide/Height (Beam/Column) (*) include accessories	Reinforcement		Axial load level (P/P_{ult}) (**) P_{ult} no provided by authors	Type of strengthening	Introduction of load	Concrete (MPa)
		Beam Wide/Height	Column Wide/Height		Continuous	Symmetric				
Au et al 2005 [15]	6	250x300	300x300	3300x2060	Yes	Yes	0, 0.30	Interior joint reinforcement	On beams	40
Li and Chua 2009 [27]	6	300x230 600x230	820x280 1600x300	3600x3275*	No	No	0.35	GFRP, CFRP	On Column	19
Lee et al 2010 [28]	3	300x600	400x400	4500x3850	Yes	Yes	0.19	CFRP	On column	27
Dhakal et al 2005 [13]	6	300x550	350x500	5400x3700	No	No	0.12	None	On beams	31.6 32.7
Pantelides et al 2008 [29]	8	406x610 406x406	406x406	3251x4064*	No	No	0.10	CFRP	On beams	43
Shannag et al 2005 [17]	6	125x200	125x150	1600x745	Yes/No	No	0	HPFRC	On column	27 75
Mukherjee and Joshi 2005 [18]	13	100x100	100x100	900x900	Yes/No	Yes	0.50	GFRP, CFRP	On beam	30
Li et al 2009 [14]	5	200x400	200x400 400x200	4500x2460	Yes	No	0	None	On column	31
Parra-Montesinos et al 2005 [49]	2	150x350	350x350	4880x2600*	Yes	Yes	0.04	HPFRCC	On column	35
Beres et al 1992 [22]	20	356x610	406x406	2794x3391	Yes/No	No	1557 kN**	Steel plates	On beams	24.1
Benavent-Climent et al 2008 [47]	1	1740x18	270x270	4200x1800*	No	No	355 kN**	None	On column	19.4
French et al 1990 [21]	4	305x510	380x380	4800x2400*	Yes	Yes	0	Epoxy	On column	63
Prota et al 2000 [20]	12	-	-	3048 x2642	Yes	No	125 kN**, 249 kN**	FRP	On beams	31.7
Prota et al 2004 [19]	11	200x355	200x200	3050x2640	Yes	No	0.10, 0.20, 0.30	CFRP	On beams	30

Hakuto et al 2000 [16]	6	300x500	300x460	3510x2900	Yes	No	0	New RC	On column	31-53
Lu et al. 2012 [50]	10	250x400	400x400	3500x3500	Yes	Yes	200 kN**	Interior joint reinforcement	On column	21.4-40
Present paper Ruiz-Pinilla et al.	20	300x400	300x300	3300x2200 (4000x2800 *)	Yes/No	Yes/No	0, 0.30	Steel cage Capital Anchorage Steel bars	On beams	10-25

Table 2. Some characteristics on similar test.