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Retrofitting of RC columns with inadequate lap splices by means of external confinement

P.M. Chronopoulos[1], T.P. Tassios[2]

[1] Str. Eng., M.Sc., Ph.D.c, Lab. of RC/NTU of Athens, Greece
[2] Str. Eng., Prof. Emeritus, Lab. of RC/NTU of Athens, Greece

ABSTRACT: Within the framework of an extended theoretical and experimental programme on lap splices of RC members under monotonic and mainly cyclic loads (including critical regions of earthquake resistant structural elements), underway in the Lab. of RC/National Technical University of Athens, a series of tests is completed on columns with inadequate lap splices (scale 1/1 to 1/2, shear span to depth ratio approx. equal to 4, zero axial load). The main parameters of these tests were (i) the detailing of lap splices within the element's section, (ii) the degree of confinement (internal and external), and (iii) the length of lap splices, equal to 30 or 45 bar diameters. The strengthening was applied (prior to testing) by means of additional external confinement, either with CFRP wraps or with light steel cages.

Based on the results of this study and on the first comparisons with the provisions of modern Codes, e.g. the fib/MC 2010, the EC's 2 and 8 (Parts 1 and 3) or the nGCSI (new Greek Code on Structural (Assessment and) Interventions), it could be argued that shorter (than the normative) lap splice lengths could be effective and safe (at least for lengths more than an absolute minimum), depending on the required ductility, if a properly arranged and adequately detailed external confinement is applied.

1 INTRODUCTION

An extensive discussion on the main parameters entering the problems of bond, anchorages and lap splices of RC members, may be found, among others, in Fardis (2008) or in Chronopoulos et al. (2012). Since the lap splicing of longitudinal reinforcement, even in the critical regions (dissipative zones) of RC elements at the base of new buildings, is still the rule (at least for conventional buildings), not to mention all existing ones, a lot of relevant research is ongoing in many earthquake prone countries.

Within this framework, a programme devoted to lap splices of RC members under monotonic and cyclic actions is almost completed in the Lab. of RC/NTU of Athens/GR, aiming at calibrations of the relevant Code provisions (for new or existing buildings). Tests (on almost fifty full scale specimens) and evaluations cover mostly beams and columns, strengthened or not, with adequate or inadequate lap splices.

In this paper, the first results of eight specimens (scale 1/1 to 1/2, shear span to depth ratio approx. equal to 4, zero axial load) under fully cyclic actions (imposed displacement ductility μ_δ up to 7) are presented and discussed. In four specimens the lap splices (with a length of approx. $30d_b$) were inadequate and they were retrofitted before testing by means of additional external confinement (jackets made of CFRP wraps or light cages made of mild steel).

2 PREVIOUS APPLICATIONS AND INVESTIGATIONS

After past experience of significant damage or collapses of buildings due to several major earthquakes, particularly those which hit urban areas since the 70s and 80s, many cases of rehabilitation are reported all over the world. Thus, the number of research works on structural interventions is constantly increasing, indicating the importance of seismic repair and/or strengthening. Local jacketing of critical regions of primary columns (with inadequate lap splices and/or shear reinforcement) is one of the various techniques in use for pre- or post-earthquake upgrading or strengthening of vulnerable RC buildings, see, e.g., CEB/162 (1983), Priestley et al. (1992), Sugano (1996), Moehle (2000), fib/24 (2003), Thermou & Elnashai (2005) and NZSEE (2006 and 2012).

To this end, thin “jacketing” (continuous or discontinuous, in the form of cages) is a versatile technique, offering a limited increase of flexural capacity and a considerable increase of local ductility and of axial and shear capacities, while this intervention does not affect the lateral stiffness of the columns (and consequently does not alter the dynamic characteristics of the building). The present state of practice and research of seismic rehabilitation by means of various types of local jacketing of columns or piers (of buildings, bridges, etc.) is rich enough, not to mention lessons learnt from the observed behaviour of some rehabilitated structures during following strong earthquakes.

The inventory of relevant materials and techniques is long, ranging from typical or conventional to modern or innovative ones, accompanied by fruitful research works and calibrations (both theoretical and experimental) over the last decades, as it is shortly presented in what follows :

- Local intervention, replacement of concrete and insertion of closely spaced welded hoops, with or w/o an additional light and thin mortar jacket (reinforced, e.g., with welded wire fabrics), use of sprayed or, preferably, of casted mortar, with or w/o a pressure; or, finally, application of thin mortar jackets lightly reinforced with innovative metallic fabrics (MF’s, made of twisted/bundled high-carbon high-strength steel cords, coated with micro-fine brass);
- Encasement by means of continuous steel jackets, in the form of thin plates or of channels-clamps, welded or bolted, in contact with the surface of the element or at a distance, single (or even double) layered, use of adhesive agents or of special grouts or mortars, with or w/o a pressure; or, finally, encasement (followed by grouting) by means of special undulated-corrugated sheets, to counterbalance outward bulging of flat steel plates or ties/straps;
- Arrangement of quasi-continuous mild steel jackets-cages, formed by angles (or strips) firmly fixed to the column and closely spaced welded bands-straps (to form hoops), possible use of resin glues or injections and possible final incorporation (forming light and thin jackets);
- Wrapping of spirals (with a small pitch), with adequate end anchorages or connections, by hammering of mild steel wires or by heating of SMA wires (shape memory alloy, NiTi and NiTiNb), or by wrapping and gluing of narrow CFRP bands;
- Arrangement and clamping of various types of discrete closely spaced external collars, secured by various means and devices, made of CFRP’s or of steel (including packaging bands), even slightly pretensioned, by prestressing or preheating; and
- Winding on and wrapping of wide and overlapping CFRP straps (in a certain number of plies), by a wet or dry method, bonded or unbonded to the element (by use of isolation films), or even arrangement, in a single (or a double) layer, of partially prefabricated CFRP sleeves-cases (usually circular or elliptical) and final grouting; or, finally, application of thin mortar jackets lightly reinforced with special textile meshes or fabrics, impregnated with inorganic binders (textile reinforced mortar/TRM jackets).

For most of these techniques there is an accumulation of technical and scientific knowledge (including mechanics and models), covering both main materials, steel and FRP’s or TRM’s.

Reference is also made to recent Guides or Codes [see, e.g., the FEMA (2000) or the ASCE (2007), the fib/24 (2003), the EC8/P.1 (2004) and especially P.3 (2005), and the nGCSI (2012)], as well as to a number of relevant books, reports or papers. The majority of these references are dealing with modeling and redesign (simplified or advanced), concerning both relevant aspects, i.e. the enhanced (axial and) shear capacity (due to the added closed hoops, of various kinds) and the increased ductility (due to the confining action of local jackets or cages and the delayed unzipping of lap splices or buckling of continuous reinforcement). Among these endeavors, the following references were taken into account during the test design and the evaluation of the results of the present study:

a) For local structural interventions by means of steel elements, with or w/o concrete/mortar layers, of various types:
Arakawa (1980), Kahn (1980), Chronopoulos (1982), CEB/162 (1983), Tassios (1983), Priestley et al. (1994a and 1994b), Aboutaha et al. (1996), Ghobarah et al. (1997), Moehle (2000), fib/24 (2003), Hussain & Driver (2005), Thermou & Pantazopoulou (2007), Mokari & Moghadam (2008), Choi et al. (2009).

b) For local jackets, of various types, with G- or C-FRP's, or even TRM's, etc., in contact or near surface mounted:
Katsumata et al. (1988), Saadatmanesh et al. (1994), fib/14 (2001) and fib/24 (2003), ACI (2002), Teng et al. (2002), Galal & Ghobarah (2004), Triantafillou & Papanicolaou (2005), Harries et al. (2006), Tastani & Pantazopoulou (2006), Bousias et al. (2007), Bournas et al. (2007), Elsouri & Harajli (2011), Tassios (2011).

Emphasis is given to the fact that most of the relevant analytical attempts and calibrations are focusing on rational and safe ("adjustments" and) "additions" of the new materials and layers (local jackets of all kinds) and of the existing/original deficient elements.

3 THE PRESENT INVESTIGATION

3.1 Scope

As it was said (§1), this investigation aims at the dimensioning of primary seismic elements of RC buildings with deficient lap splices (due to inadequate length and transverse reinforcement), with or w/o retrofitting. One of the series of tests (on column-like specimens) is presented here below, while the main test results are shortly evaluated in a next paragraph (§4).

3.2 Test arrangement

A series of eight specimens (column-like, almost full scale but under zero axial load) was prepared and tested (in LRC/NTUA/GR), with the main geometrical and mechanical characteristics of the RC elements presented in Figs 1 to 3, here below.

A full instrumentation scheme was used, including a continuous and detailed monitoring of strain of both concrete and steel, as well as of jacketing or caging materials, by means of conventional strain-gauges and of innovative optical fibre sensors ("smart" fibres and monitoring, offered by CRD Ltd/GR).

For all eight specimens, with a concrete cover to stirrups approx. equal to 15 mm (±), the main characteristics are as follows:

- Concrete with $f_c \approx 24$ MPa and $f_t \approx 2$ MPa, at the time of testing (standard cylinders);
- Rebars made of TEMPCORE steel B500C, 4Ø14 mm ($\rho_\ell \approx 1.25\%$),
  with $f_y \approx 540$ MPa, $f_t \approx 660$ MPa and $\varepsilon_u \approx 10\%$; and

- Stirrups made of steel SAE 1008, Ø6 mm/100 mm ($\alpha \approx 0.25$ and $\omega_w \approx 0.10$), with $f_y \approx 380$ MPa, $f_t \approx 460$ MPa and $\varepsilon_u \approx 12\%$.

For the specimens with laps and substandard details (see §3.3), a total of 7 or 5 stirrups was arranged and instrumented within the laps of $45d_b$ (≈ 625 mm) or $30d_b$ (≈ 425 mm), see Fig. 3.

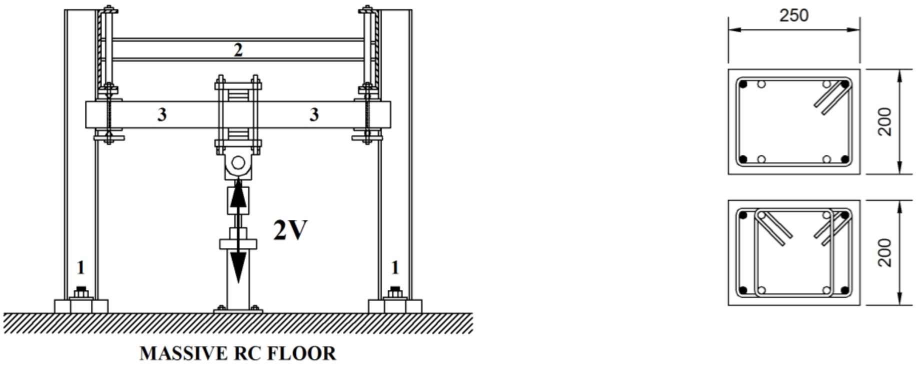


Figure 1. General test arrangement and loading (for N = 0); 1 and 2: Strong and stiff steel reaction frame; 3: RC element (column-like) under fully cyclic loading (imposed displacements). Right: Typical RC cross-sections (in mm). Top: Inside the lap length. Bottom: Inside the load application area.

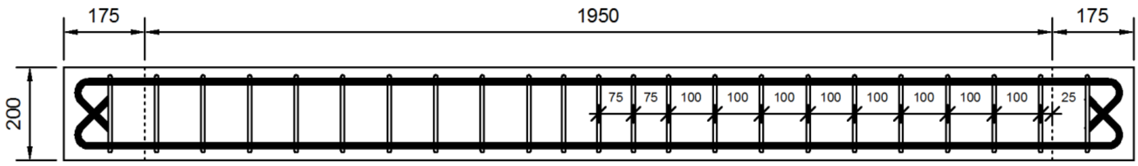


Figure 2. Specimens (CC1 and CC2) with a continuous longitudinal reinforcement.

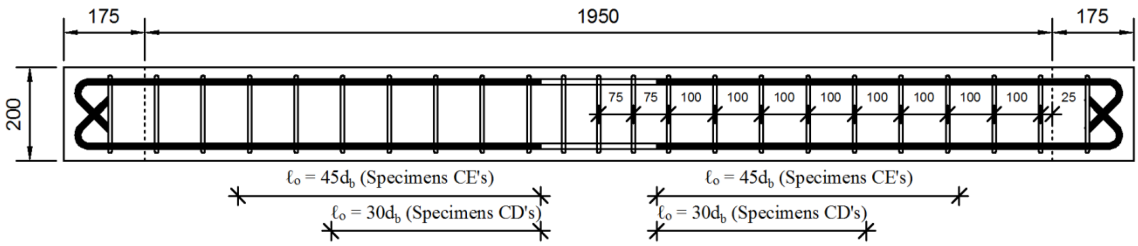


Figure 3. Specimens (CE3 and CE4) with long lap splices (ℓ_o=$45d_b$), considered as "adequate", and specimens (CDS5/CDS6 and CDF7/CDF8) with inadequate lap splices (ℓ_o=$30d_b$), fully rounded and finished, before retrofitting.

3.3 Test design

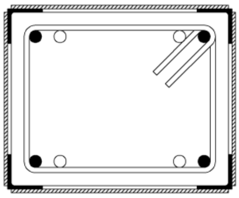
For all spliced specimens, two identical laps were arranged towards both (cantilever-like) ends at each specimen (see Fig. 3); a discussion on this, as well as other general aspects of the test design, may be found in Chronopoulos et al. (2012). All eight specimens had substandard details compared to the following standard provisions of the relevant Euro-Codes:

- Required normative lap splice length ℓ_o=$1.5\ell_b \approx 1.5$x40(up to 45)$d_b \approx 60$(up to 70)d_b, with a transverse reinforcement of min. Ø6 mm/50 to 70 mm;
- Critical regions' length $\ell_{cr,base}$=$1.5\ell_{cr} \approx 1.5$x0.3 m (up to 1.5x0.6 m);
- Anti-buckling reinforcement of min. Ø6 mm/55 to 85 mm (for each and every rebar); and
- Confining reinforcement of min. Ø6 mm, with a min. $\omega_w \approx 0.08$ up to 0.12 (even for N = 0).

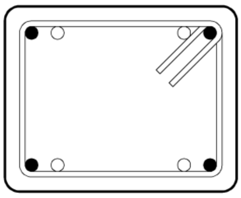
For the design of the test specimens of this investigation, the following simple guide-lines were adopted: Taking into account the "conservatism" of Codes for new structures, specimens with a "nominal" transverse reinforcement and laps of $45d_b$ (approx. equal to 3/4 of the required length) were considered "adequate", while those with laps of $30d_b$ were considered totally inadequate for medium to high ductility demands (with $\mu_\delta = \delta/\delta_y > 3$).

Regarding the retrofitting of inadequate column-like specimens (see Fig. 3), two techniques of local upgrading were applied: Light cages made of mild steel (a "traditional" one) and CFRP wraps/jackets (an "innovative" one). For redimensioning, recent approaches were followed, based on the EC 8 (P/1 and P/3, 2004 and 2005) and the nGCSI (2012), Fardis (2008), Biskinis & Fardis (2009), as well as on an analytical and consistent model proposed by Tassios (2011).

The main details of the strengthened (before testing) deficient specimens are shortly presented in Figs 4 and 5, here below:



Mild steel S235JR (St. 37-2),
corner angles 40·40·4 and closely spaced strips 20·4/75 mm;
full strength SMAW fillet welds with
rutile (TiO_2) electrodes AWS/E6013,
under temporary tightening by means of packaging bands.



Two plies of S & P CFRP – SHEET 240
(230 gr/m^2, t_{nom} = 0.125 mm, Clever Reinforcement AG);
priming and gluing with epoxy based materials / SINMAST S2W.
CFRP's: $f_t \approx 3800$ MPa, $\varepsilon_t \approx 1.6\%$ ($E \approx 237.5$ GPa),
recommended max. $\sigma_d/\varepsilon_d \approx 1425$ MPa/0.6 %.

Figure 4. The main details of the local strengthening, for a length of approx. $1.2\ell_o \approx 36d_b \approx 0.5$ m.

The length of the local strengthening was estimated based on the principle that plastic hinging (although limited) should be restricted within the strengthened area, and not shifted in the region of the column immediately above the cage or the jacket, precluding early failure outside the region with interventions, as expressed by the formula here below, see also Fardis (2008), the nGCSI (2012) etc.:

$$\ell_s = \max[(0.25\ell_c; k \cdot \ell_c); 1.25h_c; 1.2\ell_o] \text{ and} \quad (1)$$

$$k = (1 - aM_o/M_s), \quad (2)$$

where:

ℓ_c and h_c is the shear length of the column (the length between the support and the point of contraflexure) and the depth of its cross-section, respectively;
ℓ_o is the inadequate lap splice length; ℓ_s is the strengthened region's length;
M_o and M_s is the column's flexural strength outside and inside the strengthened region, respectively; and α is a kind of a "safety factor", approx. equal to 0.85.

A narrow gap (of approx. 25 mm) was purposely left at the ends of all strengthened specimens, in order to prevent any increase in flexural resistance.

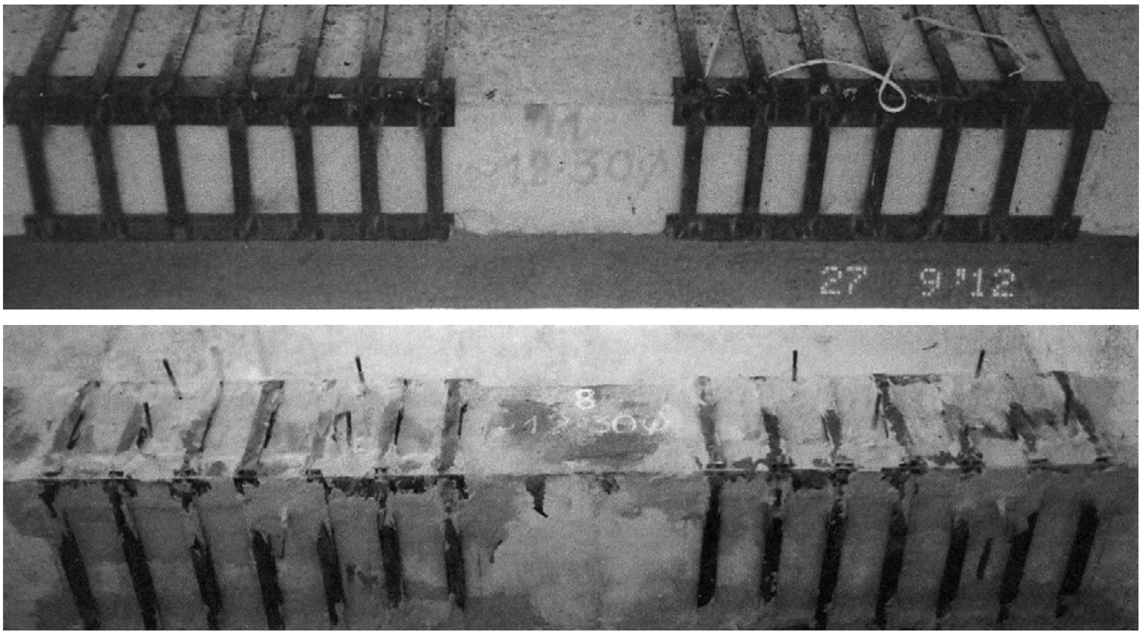


Figure 5.1. Local cages made of welded mild steel elements (angles and strips), before testing. Top: Simply surface mounted. Bottom: Fully surface bonded (by means of resin glue injections).

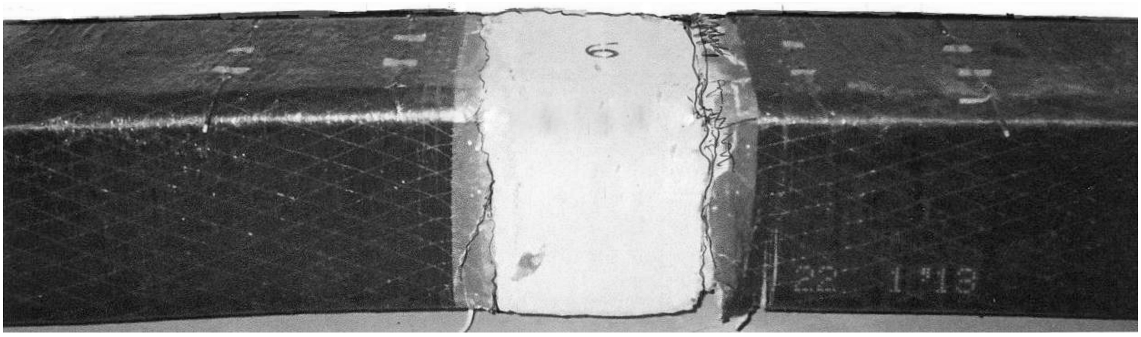


Figure 5.2. Local jacket made by wrapping and resin gluing of two CFRP plies, end overlapping by approx. 150 mm on one of the specimen's sides (transversal to the plane of loading), after testing.

3.4 Test results

The main findings of the experimental investigation are summarized in Figs 6 to 8 here below, in terms of representative P(=2V)-δ diagrammes (in kN and mm).

Regarding the relevant "activation" of the originally incorporated stirrups and of the external caging or wrapping elements, the following were found:

- For stirrups of specimens with a continuous reinforcement or with $45d_b$-spliced rebars, a strain of 0.2 % (or more) was measured close to the end sections (full yielding);
- For internal stirrups of specimens with inadequate $30d_b$ splices and retrofitted before testing, a strain of 0.1 % was measured;
- For steel cages (in particular the fully surface bonded one) and CFRP wraps, a transversal strain of approx. 0.1 % was measured, almost equal to that of the internal stirrups.

In addition, buckling of rebars was observed for all specimens, not to mention breaking of rebars (as well as of stirrups) for the two specimens with $45d_b$ lap splices and the one with $30d_b$ lap splices retrofitted by means of a fully surface bonded steel cage.

Finally, it has to be mentioned that among all four retrofitted specimens, the one which offered an almost “adequate” (or tolerable) high cyclic response is that with the fully surface bonded light cage, made of mild steel, with a higher degree of “monolithic” behaviour, see Fig. 7 (right) in comparison to Fig. 7 (left) and Fig. 8.

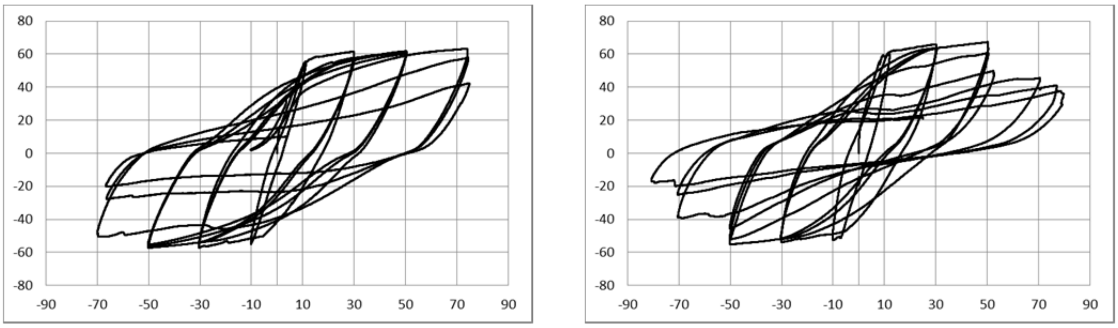


Figure 6. Left: Specimen CC1 (continuous reinforcement). Right: Specimen CE3 (ℓ_o=$45d_b$).
Cycling loading, 3 full cycles at μ_δ=3, 5 and 7.

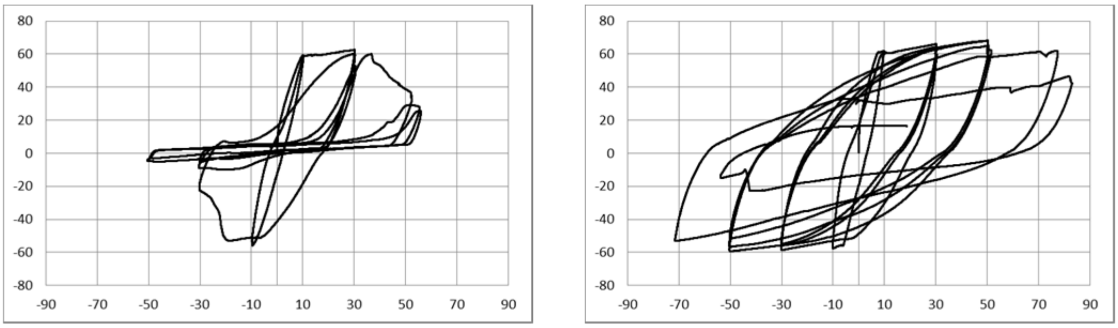


Figure 7. Light cages of mild steel, specimens CDS5 and CDS6 (ℓ_o=$30d_b$).
Left: Simply surface mounted (SSM). Right: Fully surface bonded (FSB).

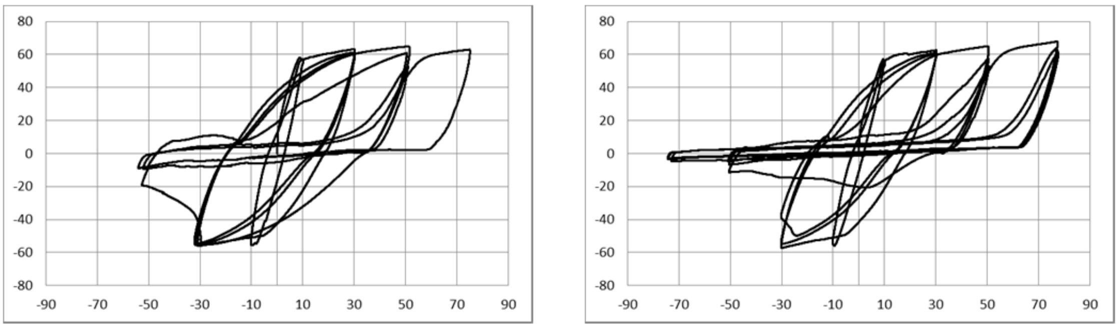


Figure 8. Jackets made of CFRP’s (two plies, by means of a dry method), specimens CDF7 and CDF8 (ℓ_o=$30d_b$).

4 DISCUSSION, CONCLUSIONS

a) Specimens with a continuous reinforcement and specimens with $45d_b$-spliced rebars displayed a similar behaviour (Fig. 6). At force-response degradation of 20% after 3 fully reversed cycles, specimens have exhibited a displacement ductility capacity more than 5. However, in terms of residual response (Fig. 9.1) or of energy dissipation capacity (Fig. 9.2), $45d_b$-spliced specimens were inferior.

b) Specimens with inadequate ($30d_b$) splices, strengthened by means of steel cages (equivalent steel sheet thickness equal to $t_s \approx 1$ mm), displayed a totally different behavior. The one strengthened by means of simple surface mounting (SSM) continued to behave deficiently. On the contrary, the one with full surface bonding (FSB) has shown force-response degradation of 20% under imposed post-yield normalized displacements more than 5 (Fig. 9.1). In addition (see Fig. 9.2), in terms of hysteretic energy dissipation capacity, this specimen was almost identical with the $45d_b$-spliced (not retrofitted) specimens.

c) Specimens with the same inadequate ($30d_b$) splices, strengthened by means of CFRP jacketing (equivalent effective thickness equal to $t_j \approx 0.25$ mm), with a complete rounding of all four corners, were able to undergo a 20% force-response degradation only at $\mu_\delta \approx 4$ (Fig. 9.1). At this ductility level, these specimens exhibited considerably lower energy dissipation capacity than the other specimens (Fig. 9.2), not to mention that they were losing this capacity rather rapidly under larger post-yield imposed displacements. However, these specimens were superior of that with a simply surface mounted steel cage.

d) The positive role of the existing stirrups along the spliced lengths remains to be carefully evaluated before a final judgement on the effectiveness of the strengthening methods used.

e) In all cases however, both retrofitting techniques (under the specific conditions of these tests) proved to be able to reinstate the splicing capacity, up to a level of imposed post-yield displacement ratio equal or higher than $\mu_\delta = 3$. Nevertheless, it has to be stated that this conclusion cannot be generalized; the absence of axial force and the rather considerable amount of existing stirrups have played a positive role which has to be separately investigated.

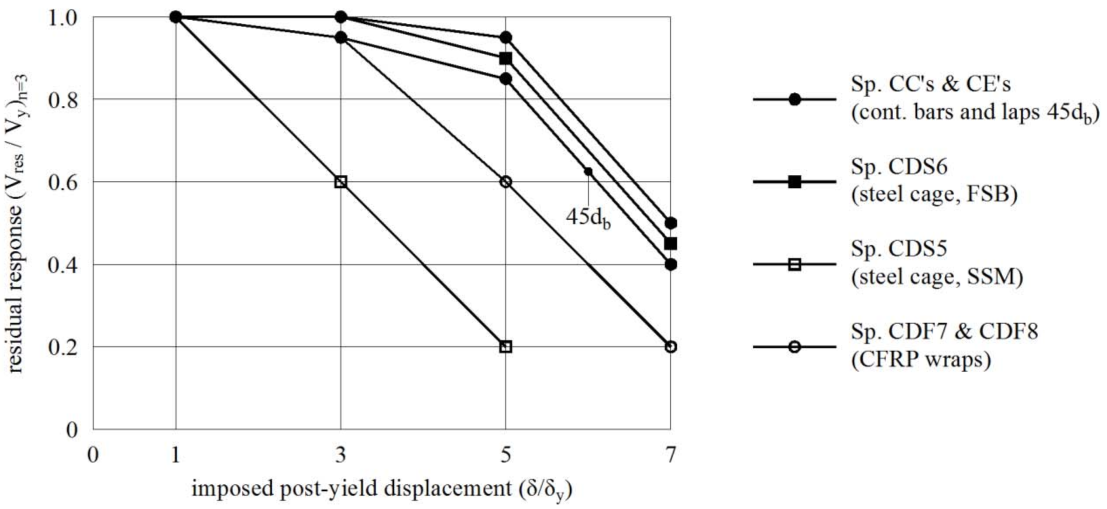


Figure 9.1. Residual response (normalized over the response at full yield) after three fully reversed cycles of imposed displacement; average values of positive and negative direction were taken into account.

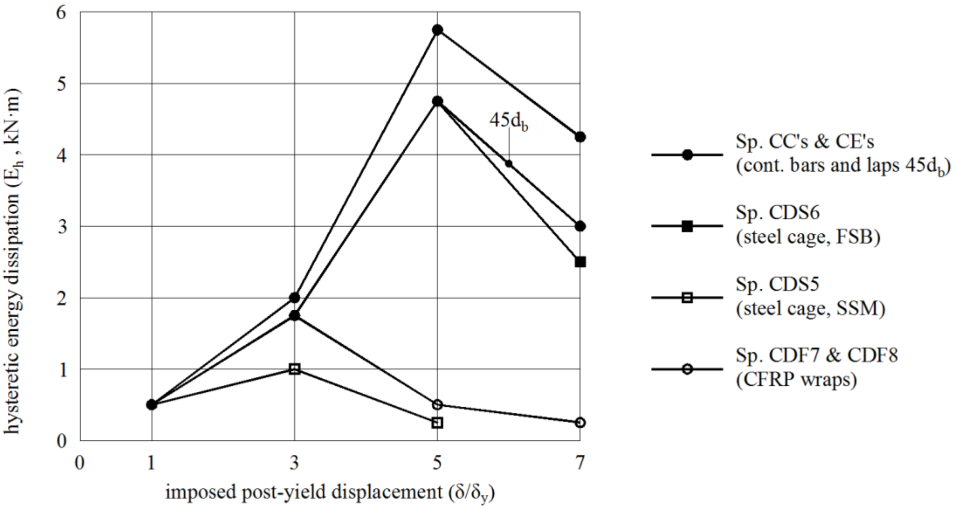


Figure 9.2. Energy dissipation in the 3rd hysteretic loop, at each imposed displacement (imposed ductility μ_δ=3, 5 and 7).

f) Finally, a lap splice length of approx. 30(to 35)d_b seems adequate for developing the actual yielding characteristics of modern steel B500C (in use for new structures of a high ductility class), see also Cho & Pincheira (2006), Harajli & Khalil (2008), Fardis (2008), Chronopoulos et al. (2012) or the nGCSI (2012).

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