

Inelastic cyclic behaviour of RC members incorporating threaded reinforcement couplers

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Abstract

This paper describes an experimental investigation into the inelastic cyclic performance of reinforced concrete members incorporating mechanical reinforcement splices. Based on a survey of available mechanical splicing forms, two types of threaded couplers with different geometric configurations, namely ‘parallel threaded couplers’ and ‘parallel threaded sleeve couplers’ are selected for detailed experimental assessment. The fundamental uniaxial monotonic and cyclic response of reinforcement bars connected with threaded mechanical splices is firstly examined through bare (in-air) and embedded (in-concrete) tests, and their response is compared with that of their non-spliced counterparts. This is followed by an experimental study on four large scale reinforced concrete specimens subjected to lateral inelastic cyclic displacements, which provides a direct comparison between the performance of members with or without the two types of reinforcement couplers as well with or without a co-existing axial load. The test results enable a direct comparative assessment of the key response characteristics of the specimens including stiffness, strength, ductility and energy dissipation. Overall, the results show that members incorporating reinforcement bars connected through threaded mechanical couplers can provide considerable ductility and energy dissipation. However, the coupler-concrete interaction behaviour, which depends primarily on the geometry and location of the threaded coupler, has a significant influence on the inelastic cyclic response and ductility levels exhibited by reinforced concrete members. The experimental results indicate that the presence of the slender coupler alters the plastic hinge behaviour by localising the curvatures between the coupler and interface, reducing the rotational capacity. In contrast, the response of members with compact couplers, and without axial load, is virtually identical to that of the specimen with continuous reinforcement. The axial load enhances the stiffness and lateral capacity yet reduces the ductility performance due to a response strongly characterised by crushing. It is shown that whilst uniaxial in-air reinforcement performance may be adopted as a basis for selecting suitable mechanical couplers for use in dissipative members, the expected levels of ductility implied in current codified seismic procedures need to be carefully and explicitly evaluated when relatively non-compact couplers are employed.

Keywords: Cyclic response; inelastic behaviour; reinforced concrete members; reinforcement splices; threaded mechanical couplers.

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1. Introduction

The continuity of reinforcement in inelastic regions is critical for structural integrity as it controls the performance of reinforced concrete (RC) structures under extreme loading conditions [1]. Besides use in on-site repairs or alterations, mechanical couplers are often employed in newly built structures due to notable practical and economic benefits compared to conventional lap splices, normally more than offsetting the cost of the couplers, and considerably reducing the construction time for RC and precast concrete (PC) systems [2]. The reinforcement congestion problem becomes particularly more critical for cases in which ductile seismic detailing is required. Conventional options such as lap splices can lead to over-reinforcement resulting in potential non-ductile behaviour at the spliced region due to a concentration of stresses at both ends of the lap, inadvertently altering the deformation capacity [3,4]. In comparison to welded laps, mechanical splices are also more practical for site installation.

In dissipative regions of RC members, reinforcement needs rigorous detailing to accommodate large local rotations that allow significant hysteretic energy dissipation [5]. Despite the above-mentioned merits of using mechanical splices, there is relatively limited information on their local deformation response and their influence on the overall inelastic cyclic behaviour of RC members. Previous tests on upset headed couplers (UHC) indicated that RC columns employing reinforcement couplers can mimic the response of corresponding members with continuous reinforcement [6]. In contrast, precast columns provided with grouted sleeve couplers (GSC) [7,8] resulted in alteration of the distribution of plasticity within the potential plastic hinge region and reduced ductility in comparison to monolithic members. Other tests on members with bolt-lock couplers (BLC) and rib-threaded couplers indicated increase in capacity at the expense of reduction in ductility [9,10]. Observations indicate that members incorporating GSC and BLC generally exhibited up to 60% reduction in ductility levels compared to continuous reinforcement, whilst other forms may provide a more favourable inelastic performance, although this is based on limited test results [11]. On the other hand, offset couplers (OBLC and OSWG) may develop unreliable behaviour in RC members at large deformations, characterised by rebar strip-out and significant slip combined with wedge rotation, producing additional detrimental concentration of stresses at rebar-wedge, as shown in previous studies [12].

Current testing performance criteria of mechanical splices are typically governed by the axial strength and/or slip requirements of the bare rebar/coupler assembly [13], without explicitly considering ductility. A recent comparative assessment of mechanical reinforcement splices illustrated in Figure 1a was carried out by the authors [14] using results extracted from a survey of over 350 ‘in-

air' uniaxial reinforcement tests considering the strength, ductility and coupler geometry. The comparison indicated that a full strength criteria based on splice-to-bar ultimate strength (i.e. $f_{u,sp} > f_{u,b}$) is not normally met in most systems, but, as shown in Figure 1b, it is typically satisfied when based instead on splice-to-bar yield strength (i.e. $f_{u,sp} > f_{y,b}$). A ductility criterion, expressed in the form of splice-to-bar ultimate strain ($\epsilon_{u,sp}/\epsilon_{u,b}$) showed a clear difference between the performance of the different coupling systems considered (Figure 1c) [14]. It was also noted that the performance of RC members incorporating couplers is expected to be influenced by the geometry of the coupling system due to coupler/concrete interaction effects. A 'size bulkiness' parameter ($\lambda_s = d_c L_c / d_b L_b$) was used to characterise the geometry of the coupling system as shown in Figure 1d [14], where d_c and L_c represent the diameter and length of the coupler, respectively, whilst d_b and L_b are the diameter and effective length of the splice ($L_c + 4d_b$), respectively. In principle, a favourable balance of 'in-air' strength and ductility, and low 'size-bulkiness', such as that offered by threaded couplers (TC) would have the potential for offering desirable inelastic 'in-concrete' performance.

Following a comparative assessment of the performance of different types of mechanical coupling systems using a collated database, as mentioned above [14], in this paper, two types of tension-compression couplers, including parallel threaded couplers (PTC) and cold forged sleeves with parallel thread couplers (PTSC) were selected for detailed experimental assessment, particularly since tests on RC members incorporating such couplers seem to be non-existent. Although some splices appear to perform well under cyclic in-air loading, the lack of readily available codified guidance limits a more common use of threaded splices in members under inelastic conditions. Eurocode 8 stipulates that the use of mechanical couplers for reinforcement bar splicing in dissipative zones needs to be verified by testing under conditions compatible with the selected ductility class (i.e. medium ductility: DCM, or high ductility: DCH) [15]. However, intrinsic characteristics depending on the surface type, size, coupler-to-rebar connection and coupler-to-rebar interface properties, which influence the local RC behaviour, are not addressed.

Monotonic and cyclic uniaxial 'in-air' and 'in-concrete' tests on PTC and PTSC, as well as non-spliced reference specimens, were first performed. After obtaining an insight into the uniaxial response, a detailed experimental investigation on four large scale RC members subjected to inelastic lateral cyclic deformations, with and without a co-existing axial load, was carried out. Based on the test results and observations, key response characteristics including stiffness and strength as well as ductility and energy dissipation are examined, and the main practical and design considerations are highlighted. Within the scope of the results considered, this paper offers some guidance for the selection and application of mechanical reinforcement couplers in inelastic regions.

2. Experimental programme

To obtain a detailed insight into the in-air and in-concrete response of mechanical splices, uniaxial monotonic and cyclic tests, as well as RC beam-column member tests subjected to lateral deformation with or without a co-existing axial load, were carried out using parallel threaded (PTC) and parallel threaded sleeve (PTSC) couplers. The PTC has standard female threads at the coupler and matching male threads at the rebar, while the PTSC has matching male-female threads forged from a steel sleeve that is mechanically connected to the unaltered reinforcing bars through swaging, similar to SWC and OSWC splices. Both coupling systems, illustrated in Figure 2, are designed to provide ‘full performance mechanical splices’ under tension, compression, cyclic or fatigue loading [16]. In the tests described in this paper, the two coupler types are used to connect 16 mm Grade 500 longitudinal bars. The PTC is a relatively more compact coupler with diameter $\times$ length = $\varnothing 25 \times 45$ mm, in comparison to the PTSC which have a diameter $\times$ length = $\varnothing 25 \times 206$. Hence these are referred to in relative terms throughout the paper as *compact* and *slender* couplers, respectively.

2.1 Testing arrangements and instrumentation

The test rig shown schematically in Figure 3a was designed to enable realistic experimental assessment of the ultimate behaviour of the large-scale members subjected to lateral cyclic displacements and co-existing axial loading [17]. The footing of the test specimen was directly supported and connected by means of two transfer beams pre-stressed through six $\varnothing 33$ mm ties to the strong floor of the laboratory to avoid potential sliding and overturning under lateral loads. The pre-stressing loads were recorded by means of 600 kN load cells positioned at each tie. The actuators were connected to the rig and to the member through two swivel hinges. A vertical actuator of 1000 kN capacity was connected directly to the loading frame as illustrated in Figure 3a, whilst another 250 kN actuator placed horizontally was connected to the reaction frame. The stroke of the horizontal actuator was 250 mm in total. Transfer plates of 30 mm thickness were used between the hinges and member cap, and steel rail guides were provided to ensure restraint against any out of plane movement.

The specimens were tested in an upright position as shown in Figure 3b, with the vertical load applied directly to the member. The inclination of the vertical actuator load was corrected for in the determination of the lateral loading as a function of the level of lateral deformation. After application of the constant axial load that was maintained throughout the test, the lateral cyclic deformations using a displacement rate of about 0.3mm/sec. The lateral displacements were based on one cycle at 0.25, 0.5, $0.75 \times \delta_y$ and $1.0 \delta_y$, followed by three cycles at $2.0 \delta_y$ and $(2+2n) \times \delta_y$ up to failure, where δ_y is the estimated lateral yield deformation and $n=1, 2, 3$, etc (Figure 3c). The tests continued until the fracture of at least one flexural reinforcement bar (rebar) occurred. In addition to the

load cells and displacement transducers incorporated within the horizontal and vertical actuators, the instrumentation included 4 additional displacement transducers, 3 inclinometers, up to 16 strain gauges on the longitudinal, transverse reinforcement and couplers (if present).

All the uniaxial monotonic and cyclic ‘in-air’ and ‘in-concrete’ rebar/splice tests were carried out in a stiff Instron 8802 testing machine provided with load cells with a dynamic capacity of 250 kN, as illustrated in Figure 4a,b. In the monotonic tests, before reaching the inelastic regime, a set of three cycles between $0.02-0.60 \times f_y$ were applied to determine the elastic deformation at the couplers. Afterwards, uniaxial displacement was applied until rebar fracture. After obtaining the complete stress-strain σ - ϵ curve from the monotonic test, for in-air or in-concrete specimens, the yield deformation δ_y was extracted from the test data and used to determine the parameters for the cyclic loading procedure. The cyclic in-air tests were subjected to the S2 low cycle reverse elastic-plastic ISO procedure [14], illustrated in Figure 4c in which the member is deformed monotonically in tension up to $2 \times \delta_y$, followed by four cycles applied at $2 \times \delta_y$ and $5 \times \delta_y$ between yield force in tension and a compressive force equivalent to 50% of the yield strength in compression, then further deformed in tension up to fracture. The in-concrete tests were subjected to a combined S1 (Figure 4d) and S2 low cycle loading test [14] in which, the sample is initially subjected to 20 cycles from $90\% \times f_y$ in tension to $50\% \times f_y$ in compression, followed by S2 depicted in Figure 4c.

A digital image correlation (DIC) system was additionally employed to record detailed local deformations and strains for the uniaxial tests and within the expected plastic hinge zone of the beam-column members. The DIC system, which has been used in several recent studies on RC members [18], offers significant versatility and accuracy. It consists of two cameras and a controller which are used to capture crack propagation and member displacements. High contrast black/white patterns were provided to the front face of the specimens by applying two layers of water-based white paint and, subsequently, a fine black speckle pattern was applied to ensure smooth detection of surface deformations during testing. This was also required to allow the post-processing software to compute surface strain vector fields. A recording rate of 1Hz was used throughout to obtain a balance between the extent of collected data and the required post-processing time. Before the start of recording a detailed calibration was carried out to ensure measurement accuracy [19].

2.2 Specimen and materials details

Four RC beam-column specimens of square cross-section with 300×300 mm dimensions and a cantilever length of $L_s = 1350$ mm were tested as shown in Figure 5a,b. The specimen reference uses the format of C300-Cx-Ny which indicate the member size (Cxxx) in mm, the coupler type (Cx) (0 for non-spliced member, C for compact coupler with diameter $\times$ length = $\varnothing 25 \times 45$, S for slender cou-

pler with diameter $\times$ length = $\varnothing 25 \times 206$), and the level of axial load (N: N0 for members without axial load, N1 for axial load of about 15% the nominal concrete cross-section axial capacity). As illustrated in Figure 5a, the four specimens were provided with $8 \times \varnothing 16$ mm bars as longitudinal reinforcement and $\varnothing 10$ mm stirrups with a spacing $s_w = 70$ having a clear concrete cover of 20 mm. Specimen C300-C0-N0 had continuous longitudinal reinforcement, whilst the remaining three specimens were provided with couplers within the expected plastic hinge zone with their mid-length at 150 mm above the member-to-footing interface (Figure 5c). Specimens C300-CC-N0 and C300-CC-N1 were provided with compact couplers, whilst Specimen C300-CS-N0 had slender couplers.

The region of longitudinal reinforcement embedded in the footing was provided with L-shaped bents at the bottom to ensure sufficient anchorage. The first stirrup was 65 mm from the bottom of the footing, whereas the first stirrup in the free length of the member was located 35 mm from the footing-to-member interface. The specimens were designed to represent a scaled configuration of square building members, with due consideration of experimental constraints. Each specimen had a rectangular footing ($1200 \times 600 \times 400$ mm) at its base, and a rectangular cap ($450 \times 450 \times 300$ mm) at the top, as shown in Figure 5d. The footing and column cap were designed to remain relatively rigid in comparison with the beam-column region. The preparation of the specimens was carried out considering the following sequence: preparation of reinforcement cages directly in the footing formwork, fixing of strain gauges on the reinforcement, casting of the footing, addition of remaining member and cap formwork, and finally casting of the member and cap.

Hot rolled deformed bars with $d_b = 16$ mm diameter of steel Grade B500C, with bond factors above 4.0, which are above the minimum requirements of existing guidelines [20,21], as indicated in a previous experimental study carried out by authors [22], were used for the longitudinal reinforcement in the beam-column members and uniaxial tests. Separate monotonic tests, using the testing arrangement illustrated in Figure 4a, were carried out for the reinforcement used in the non-spliced member (C300-C0-N0), the member with slender couplers (C300-CS-N1) and the members provided with compact couplers (C300-CC-N0 and C300-CC-N1). The properties of the steel materials, as average strengths from steel coupon direct tension tests determined on a minimum of three samples of 350mm length, are provided in Table 1.

Besides the steel material tests carried out on reinforcement samples from the beam-column members, 12 uniaxial tests, selected from a wider experimental programme covering rebars and couplers of different diameters and sizes [23,24], are reported herein. These selected uniaxial tests include 6 in-air and 6 in-concrete tests on $\varnothing 16$ mm diameter rebars, equally divided between monotonic and cyclic tests on non-spliced rebars, spliced rebars with compact couplers, and spliced rebars with

slender couplers, as illustrated in Figures 2 and 4. The in-concrete specimens had an 800 mm long splice of which 600 mm was embedded in a concrete cylinder of 121 mm diameter.

Ready-mix concrete was used in the footing of the RC beam-column specimens and in the uniaxial in-concrete specimens, whilst laboratory-mixed concrete was used for the member and cap regions of the beam-column members. The constituents used in the materials prepared in the laboratory were 361 kg/m³ cement CEM I 52.5, 64 kg/m³ Category S fly ash, 820 kg/m³ sand, 1001 kg/m³ gravel, and 149 l/m³ water. The workability for the low water-to-cement ratio of 0.35 content was maintained by adjusting the initial admixtures quantity given (2.5 kg/m³ plasticiser and 5.1 kg/m³ superplasticiser). The assessment of mechanical properties in terms of compressive strength were carried out on Ø100×200 mm cylinders and 100 mm cubes, and splitting strength on Ø102×254 mm cylinders. Material tests on all concrete materials were carried out at 28 days and on the day of member testing. Average values of tests from a minimum of three samples are provided in Table 2.

3. Test results and observations

3.1 Uniaxial monotonic and cyclic tests

As illustrated in Figure 6a, the cyclic tests on 16 mm samples resulted in levels of deformations at fracture ϵ_f , assessed from the applied actuator displacement divided by the clear length of the specimens, between $\epsilon_f=10.1-15.0\%$ ($\epsilon_u=8.2-12.9\%$). For the non-spliced bar ϵ_f was above 15.0% ($\epsilon_u=12.9\%$), decreasing to $\epsilon_f=13.2\%$ ($\epsilon_u=8.2\%$) for the specimen with compact couplers, and to $\epsilon_f=10.1\%$ ($\epsilon_u=8.2\%$) for the specimen with slender couplers. In the corresponding monotonic tests, ϵ_u were very similar for all configurations with ϵ_u around 12%. In all the in-air samples, fracture occurred in the rebar outside the coupler region. The results show that the minimum criterion for Class C reinforcement (i.e. $\epsilon_u>7.5\%$) in Eurocode 2 was met for all samples; and implicitly the strain requirements for reinforcement in seismic design according to Eurocode 8 for Ductility Class High (DCH) [15,20,25]. Figure 6b shows a comparison between the deformations recorded in the in-air elastic cyclic tests for the non-spliced and spliced samples, which may be used to determine the additional elastic deformation at the couplers. Both types of couplers had marginally higher deformation than the non-spliced reference bar with values below 0.2 mm, yet, after the three elastic cycles, the residual displacement was between 0.01-0.02 mm for all samples showing that no notable slip developed at the couplers.

Figure 6c presents the load-deformation response of the in-concrete cyclic tests, whilst Figure 6d illustrates the crack pattern after rebar yielding. A similar trend is observed for the in-concrete members as in Figure 6a for in-air specimens, in the sense that the member with slender couplers

showed the lowest ε_f (8.0%). The equivalent in-concrete monotonic specimens developed the same relative difference in ε_f between non-spliced and coupled members, yet these were slightly more ductile. These results show lower ε_u for in-concrete members in comparison to the in-air samples, contrary to suggestions in previous studies [26] that in-air performance offers a conservative lower bound for the splice performance. Although the load-strain curves plotted in Figure 6 indicate decreasing trends in respect to ductility, these member tests show that the coupler size does not seem to have a strong influence on the crack propagation, as illustrated in Figure 6d. However, for the members with slender couplers, the potential slip due to lower bond strength at the coupler in comparison to the bonded rebar region, shifted the cracks outside of the coupler and interface. This was shifted initially towards the concrete/rebar region, and ultimately to the weakest cross-section which corresponds to the free rebar region where failure occurred. The ratios between these lengths, combined with bond characteristics, also may influence the performance of in-concrete members under uniaxial loading and are therefore likely to affect the response of members in bending.

3.2 Lateral cyclic response of RC members

As noted before in Section 2.2, the experimental assessment included four large RC beam-column specimens subjected to inelastic lateral cyclic loading. The influence of the horizontal component of the axial load at large lateral deformations δ was accounted for in determining the lateral loads V . A summary of the main results is shown in Table 3, the V - δ relationships are depicted in Figure 7, and the observed damage in the four specimens is illustrated in Figure 8.

The specimen with continuous reinforcement (C300-C0-N0) had a typical RC response characterised by nearly equally spaced flexural cracks, followed by reinforcement yielding, compressive crushing and spalling, and eventually rebar fracture. The yield deformation δ_y corresponding to the first yield of flexural rebars as recorded by strain gauges located above the member-to-footing interface occurred at $\delta_y=14.0$ mm in the positive (pull) direction at $V_y=73.2$ kN. As observed from the V - δ curve in Figure 7a, after each first cycle applied in the post-yield regime ($\delta>15$ mm), the lateral load attained for each level of δ decreased with each increasing cycle. After reaching the maximum lateral capacity $V_{\max}=92.3$ kN, as shown in Figure 7a, the first signs of crushing occurred at $\delta=60$ mm, with significant spalling developing towards the end of the test. The fracture of longitudinal rebars shown in Figure 9a, occurred in the third negative (push) cycle at $\delta=90$ mm. Further cycling at $\delta=105$ mm, produced the first fracture on the opposite side in the second positive (pull) cycle.

Specimen C300-CS-N0 incorporated slender couplers (Figure 5c). As the coupler is considerably long (208 mm) with respect to the column depth (300 mm), the remaining length between the inter-

face and coupler base was around 47 mm, producing crack concentrations below the coupler (Figure 8b). Consequently, first yield was recorded in the pull cycle at about $\delta_y=9.0$ mm, which corresponded to $V_y=49.9$ kN. There were no signs of crushing or spalling; however, as illustrated in Figure 7b, there were notable reductions in capacity accompanied by audible noises without reinforcement fracture, both in the pull and pull cycles, which are attributed to debonding between the coupler and the concrete. These were observed at the second positive (pull) cycle of $\delta=60$ mm, and at $\delta=38$ mm in the first positive cycle of applied $\delta=90$ mm. As there was no visible concrete crushing, combined with coupler-to-concrete debonding that resulted in additional rotation, the cross-section rotation axis moved towards the member edge, producing a rocking-type response (Figure 7b). Reinforcement fracture, shown in Figure 9b, was observed in the second pull cycle at $\delta=90$ mm. The member secant stiffness became nearly flat with the lateral capacity diminishing.

In Specimen C300-CC-N0, which had compact couplers and no axial load, first yield of the longitudinal reinforcement both in the push and pull cycles was $\delta_y=12$ mm at a corresponding average $V_y = 63.5$ kN, whilst the first signs of crushing were observed around $\delta=20$ mm. From the $V-\delta$ relationship shown in Figure 7c, it may be observed that in the post-yield regime for $\delta>15$ mm and up to $\delta=60$ mm, considerable inter-cycle degradation, in the range of 15%, is observed between the first and second cycle, but this stabilises at the third cycle with only 3.6% reduction, which is similar to the response of the member with continuous reinforcement (C300-C0-N0). In contrast, for $\delta>60$ mm, similar degradation is observed between first and second cycle, and second and third cycle, with an average degradation between cycles of about 17%. A notable decrease in V is observed after $\delta>90$ mm due to compressive crushing, as illustrated in Figure 8c, led to considerable spalling.

The specimen C300-CC-N1 with compact couplers, subjected to an axial load of 800 kN, was dominated by significant compressive crushing, as expected. After first yield of flexural reinforcement was recorded in the negative (push) direction at $\delta_y=14$ mm, at a corresponding $V_y=121.5$ kN, immediate crushing of the concrete in compression occurred. This is captured in the $V-\delta$ shown in Figure 7d, in the first cycle at $\delta=30$ mm, with a notable reduction in capacity after the $V_{max}=138$ kN in the positive (push) direction was reached. Compression crushing, depicted in Figure 8d, was followed by more significant spalling of the cover, in comparison to the cantilever member, over a length larger than that corresponding to the member depth. The presence of axial load increased the level of degradation with the increase in δ , which led to an ultimate condition governed by the buckling of the compression reinforcement. The reinforcement buckling produced some sliding in concrete above and below, increasing the demand and strain concentration in the tension reinforcement. This was directly followed by fracture at the coupler region at the top connection between coupler and rebar at $\delta_f=78$ mm (Figure 9c). After fracture, the lateral capacity dropped to zero.

It is worth noting that the couplers influenced the flexural crack kinematics for all coupler types, with minimal influence on the elastic transverse reinforcement. For both specimens with compact couplers (C300-CC-N0 and C300-CC-N1), two cracks formed at the coupler-to-rebar interface capturing the transition from two cross-sections with different rigidities. As these couplers are relatively compact, the overall member behaviour was not notably affected, as the $V-\delta$ responses for C300-C0-N0, without couplers, and C300-CC-N0 were virtually identical. In contrast, the slender coupler influenced the crack kinematics by translating the weak regions below and above the couplers, similar to the uniaxial in-concrete response described in Section 3.1. This, added to the influence from the moment gradient, concentrated the rebar strains at the member-to-footing interface, resulted in early failure in comparison with the other cantilever members. This was captured by the strain gauges located at the interface which showed values about 50% higher at $\delta=30$ mm for C300-CS-N0 in comparison to the C300-C0-N0. Some of the differences in the strain levels obtained at the experimental δ_y , may be attributed to inadvertent slight variations in the strain gauge locations.

Based on the experimental results, it is indicated that the transverse reinforcement detailing plays a significant role in the response of the member provided with couplers in the expected plastic hinge zone. In the test specimens described in this paper, the transverse rebar density was determined from Eurocode 8 seismic detailing requirements [15]. The chosen stirrup spacing ($s_w=70$ mm) was about 10% smaller and the transverse bar diameter ($d_{bw}=10$ mm) was about 50% higher than those required, hence satisfying the code limits. However, a comparison with the more stringent seismic requirements for bridge piers [27], the chosen spacing was 45% above the minimum required. The implicit concentration of curvatures at the coupler-to-rebar interfaces resulting from rebar bending could be limited by a dense reinforcement scheme at the coupler region. Ties located at the coupler and at the rebar-to-coupler interfaces would restrict buckling and reduce the demand on the tension reinforcement. To restrict the rotation of the coupler around the stirrup, a minimum of two stirrups are needed within the coupler length with a minimum spacing complying with codified procedures.

3.3 Curvature profiles

The length of the experimental inelastic member region is assessed here by considering the flexural deformations, as extracted from surface virtual gauges resulting from post-processing of DIC data and validated against strain gauge recordings. Sixteen surface gauges located symmetrically at 100 mm from the member axis were used to determine member curvatures ϕ throughout the instrumented specimen length. Figure 10 depicts these ϕ distributions for the four specimens, by considering the strains at the first cycle of each of the applied lateral deformation levels δ . These show concentrations in the proximity of the member-to-footing interface and minimum values around the

couplers as the strains obtained within that region were largely elastic. The dashed vertical line shows the yield curvature ϕ_y obtained from sectional analysis considering the material strength from Section 2.2. The intersection between the ϕ profiles and the ϕ_y line shows the limit between inelastic-to-elastic deformations, and implicitly, the experimental plastic hinge region L_{pr} .

The distributions indicate a shorter plastic hinge region for the non-spliced member (C300-C0-N0) of around 250 mm, and longer inelastic lengths for C300-CS-N0 with slender couplers (330 mm). For specimens with compact couplers (C300-CC-N0 and C300-CC-N1), the distance between the interface and the inelastic-to-elastic limit was between 330 mm and 265 mm, respectively. These results show that the presence of the coupler alters the plastic hinge region, extending its length. For the member with slender couplers (C300-CS-N0) a concentration of ϕ is observed below the coupler, with maximum values about 75% higher in comparison to the non-spliced case (C300-C0-N0) at $\delta=60$ mm, and further increased at higher δ levels. Also, higher ϕ are observed above the coupler region for the case with slender couplers in comparison to the other cases. This shows that the maximum ϕ are translated above and below the couplers with higher values at the interface due to the influence from the moment gradient. As shown in Figure 10, for the members with compact couplers (C300-CC-N0 and C300-CC-N1), the obtained ϕ are within similar ranges to the non-spliced member (C300-C0-N0).

3.4 Energy dissipation

The energy dissipation in the test members is estimated from the hysteretic behaviour shown in Figures 7, based on the enclosed areas from the first and third cyclic loops at each deformation level as well as the average of the three cycles. These are represented by continuous grey and black lines in Figure 11, respectively. The difference between the two lines at the same deformation level may be regarded as the inter-cycle degradation. As observed from Figures 11a,c the non-spliced and compact members (C300-C0-N0 and C300-CC-N0) had the same level of energy dissipated at each applied δ as the response was virtually identical. At $\delta=90$ mm. A slight difference occurs at the third cycle at $\delta=90$ mm at which C300-CC-N0 dissipated about 3% less energy, an effect further enhanced at the final applied deformation $\delta=105$ mm (1% difference on average and 75% at the third cycle). However, at that level δ , there was significant crushing and spalling for both members. Also, up to $\delta=60$ mm, the inter-cycle degradation seems to be minimal and rather constant.

In contrast, the member with slender couplers (C300-CS-N0) had a distinctly different behaviour in comparison to other members without axial load. The E_d - δ average curve of C300-CS-N0 in Figure 11b changes in slope after $\delta=60$ mm as at this δ , the capacity dropped significantly by about 55%.

Also, starting from $\delta=60$ mm, the first-to-third inter-cycle degradation is larger for this specimen (about 30%) in comparison to C300-C0-N0 and C300-CC-N1 for which it was around 13%. All these members had a rather proportional E_d - δ increase with δ while the hysteretic loops were stable and no significant crushing (C300-C0-N0 and C300-CC-N0) or debonding (C300-CS-N0) occurred. A relatively steeper E_d - δ slope is observed in Figure 11d for C300-CC-N1 up to $\delta=30$ mm as the presence axial load produced wider loops in comparison to members with no axial load. After $\delta=30$ mm, a gradual decrease in the average dissipated energy is observed with crushing governing the behaviour after $\delta=60$ mm. As reinforcement fracture occurred, at the first cycle of $\delta=90$ mm, the lateral capacity dropped significantly, and hence the energy dissipated diminished rapidly in comparison to other inelastic deformation levels.

4. Comparative assessments

4.1 Moment-rotation response

The moment-rotation M - θ envelope curves shown in Figure 12, in which the ultimate rotation θ_u is assumed to correspond to a reduction of 20% from the ultimate moment M_u , indicate similar stiffness for all members without axial load. The average member stiffness considering a proportionality limit at 30% of ultimate lateral capacity for both positive and negative envelopes was $k_{cr}=8.3$ kN/mm for the non-spliced member (C300-C0-N0), $k_{cr}=9.6$ kN/mm for the member with slender couplers (C300-CS-N0) and $k_{cr}=7.8$ N/mm for Specimen C300-CC-N0, with compact couplers. In contrast, the presence of axial load enhanced the stiffness to $k_{cr}=20.8$ kN/mm for C300-CC-N1. Although the member stiffness is slightly influenced by the coupler size, for the investigated members, as listed in Table 3, the V_{max} is largely the same. This is also confirmed through an assessment of the moment-axial (M - N) interaction curve, which is typically used to characterise the cross-sectional strength of members under combined loading. The top left panel of Figure 10 illustrates the interaction diagram for the investigated cross-section. The test results, represented by a pair of (M_u , N) values, as given in Table 3, consistently coincide with the predicted M - N curve, with the members without axial load grouped together.

From the M - θ curves in Figure 12, θ_u is shown to be a function of the coupler type and level of axial load. As expected, the presence of axial load increases the post-peak degradation with an increase in the applied θ and reduces the θ_u capacity. A comparison between C300-CC-N1 and C300-CC-N0 shows that θ_u is halved for the member with axial load ($\theta_u=33.9$ mrad) in comparison to the non-axially loaded member ($\theta_u=70.2$ mrad). Additionally, the slender coupler (Specimen C300-CS-N0) influenced the rotation capacity with a reduction in θ_u of about 23% in comparison to the member with compact couplers (C300-CC-N0). In contrast, as shown in the chart in the bottom right panel

of Figure 12, the non-spliced member (C300-C0-N0) and the member with compact couplers and without axial load (C300-CC-N0) had essentially identical responses and θ_u of about 70 mrad. Additionally, the member rotational ductility ratio $\mu_\theta = \theta_y / \theta_u$ was examined by considering the chord rotation at yield θ_y and the ultimate chord rotation θ_u . The θ_y was obtained from a simplified bilinear M- θ curve by equating energy between the average test M- θ envelope and the bilinear M- θ . The analysis shows μ_θ of around 9.0 for non-spliced C300-C0-N0 member and C300-CC-N0 with compact couplers, with a difference of only 1%. In contrast, the member with slender couplers (C300-CS-N0) showed a 10.6% reduction in μ_θ in comparison to the non-spliced counterpart (C300-C0-N0). The axial load reduced μ_θ by about 12.5% for the members with compact couplers C300-CC-N1 in comparison with C300-CC-N0.

4.2 Plastic hinge characteristics

In plastic hinge models, the inelastic deformation is considered to be lumped within a finite length L_{pl} in which a constant curvature is often accounted for. For RC members with seismic detailing, an estimate of L_{pl} may be obtained from Equation (1a) or Equation (1b) [27,28]. For the investigated configurations, the estimated plastic hinge lengths L_{pl} assessed with Equation (1a) vary between 211-216 mm with an average $L_{pl}=214$ mm, whilst L_{pl} assessed with Equation 2b are around 254 mm, as in the second equation the influence from the concrete strength f_c is not included. A comparison between the experimental L_{pl} for the member C300-C0-N0 ($L_{pr}=250$ mm) and the estimated L_{pl} shows that Equation (1b) offers better predictions in comparison to Equation (1a) with a deviation of only 1.5% from the test results. Hence, Equation (1b) is considered thereafter in this section.

$$L_{pl} = L_s / 30 + 0.20D + 0.11d_b f_y / \sqrt{f_c} \quad (1a)$$

$$L_{pl} = 0.1L_s + 0.015f_{yk}d_b \text{ where } f_{yk} = f_y / 1.15 \quad (1b)$$

As illustrated in Figure 10, the experimental curvature ϕ profiles for the members with couplers differ from those of the non-spliced member. A sudden change in ϕ is observed as the strains in the coupler are typically below yield strain due to the increase in cross-section. With the increase in δ , and implicitly in moment gradient, the location at which yielding occurs with respect to the moment capacity M_R is modified by the couplers (Figure 13). Consequently, the inelastic region is longer in the presence of couplers, whilst a higher concentration of ϕ is developed when the coupler is longer and the distance between the coupler and member-to-footing interface is shorter. This is captured by a comparison between the considered test ϕ which show higher values for the member with the slender coupler (Figure 10b) in comparison to that with the compact coupler (Figure 10c).

The total chord rotation θ_u may be assessed as a function of the ultimate curvature ϕ_u and the estimated L_{pl} (Equation 2). The θ_u has two components namely the yield rotation θ_y and the plastic rotation θ_p , represented by the first and second term on the right-hand side of Equation (2). As observed, θ_p , implicitly accounts for L_{pl} and is a function of the difference between ultimate and yield curvatures ($\phi_u - \phi_y$), i.e. plastic curvature ϕ_p . For assessments, ϕ_y and ϕ_u are typically obtained from a $M-\phi$ analysis of the section with due account for the co-existing axial load. In addition to θ_y and θ_p , under cyclic loading, the yielding of the tension bars penetrates into the anchorage zone, increasing the fixed-end-rotation of the end due to slippage between the rebars and concrete. Equation (3) may be used to estimate the contribution of strain penetration to the total member rotation [21]. Hence, in this paper, the total chord rotation θ_u is estimated with Equation (4).

$$\theta_u = \theta_y + \theta_p = \phi_y L_s / 3 + (\phi_u - \phi_y) L_{pl} (1 - 0.5 L_{pl} / L_s) \quad (2)$$

$$\Delta\theta_{slip,p} = 5.5 d_b \phi_u \quad (3)$$

$$\theta_u = \theta_y + \theta_p + \Delta\theta_{slip,p} \quad (4)$$

A sectional analysis was carried out [29] for RC sections in which the complete $\sigma-\epsilon$ characteristics, material parameters and axial load (if present) described in Section 2 were considered. Firstly, to obtain the $M-\phi$, the complete $\sigma-\epsilon$ curve of the unconfined concrete properties was modelled considering the member cross-section from Figure 5. An idealised bilinear representation of the non-linear $M-\phi$ obtained, as illustrated in Figure 14, was considered for the evaluation of ϕ_y and ϕ_u . The obtained ϕ_u , listed in Table 4, is strongly influenced by the axial load, being about quarter the value in C300-CC-N1 compared to C300-CC-N0.

Figure 15 illustrates the $M-\theta$ test envelopes obtained as an average between the positive and negative envelopes from Figure 12. A bilinear idealisation of the test $M-\theta$ is also indicated, by equating the area under the $M-\theta$ curves and using an elastic stiffness obtained from $M-\theta$ at 30% of the non-linear stiffness. These curves are compared with the estimated $M-\theta$ using the approach described above, with the main results also listed in Table 4. As illustrated in Figures 10 and 12, in comparison to the non-spliced member (C300-C0-N0), the slender coupler (C300-CS-N0) led to curvature localisation between the coupler and member-to-footing interface, whilst the compact coupler (C300-CC-N0) had some influence on the curvature profiles, yet no influence on the ductility.

An assessment of the ultimate rotation θ_u using the full length of the plastic hinge estimated using Equation (1b) offered unconservative estimates in comparison to the test θ_u for Specimen C300-CS-N0, whilst for C300-CC-N0 these were in a good agreement. To account for the influence of the

slender coupler on the rotational response of the member, a shorter plastic hinge length L_{pl} reduced by 50% was considered, as illustrated in Figure 13b. As observed from Figure 15, in this manner the estimated ultimate total rotation θ_u for C300-CS-N0 is within the same conservative ranges as for C300-C0-N0 and C300-CC-N0, for which L_{pl} determined with Equation (1b) was considered. For these members, θ_u is 14-23% lower than the test values for θ_u . For the member subjected to combined axial load and bending moment (C300-CC-N1), the estimated θ_u is over-conservative, with θ_u being about half of the test θ_u at 20% reduction in M_u as this member had a more crushing governed response compared to others. However, it is worth noting that, the predictions of θ_u are in close agreement with θ at crushing, as shown by the change in slope after M_u was reached in Figure 7d.

4.3 Inelastic member performance

Seismic codes, such as Eurocode 8 [15], typically define required levels of curvature or rotational ductility ratios depending on the required inelastic performance levels and the associated ductility class. As shown in Section 3.3, the couplers influence the curvature distribution along the member length, and as discussed before in Section 4.1, the levels of ductility obtained are not substantially reduced for relatively compact couplers, and are likely to satisfy high ductility requirements. Nonetheless, it is evident from this study that explicit assessment of the level of reduction in ductility is necessary when non-compact couplers are employed.

As noted in the introduction, some coupler configurations may impair the member ductility in comparison to their non-spliced counterpart. Such reduction in performance typically arise from the poor performance of the coupler-to-rebar connection, which may instigate rebar fracture at the coupler region. The relationship between the size/bulkiness factor $\lambda_s = d_c L_c / (d_b L_b)$ depicted in Figure 1d and the ultimate rotation ratio $\theta_{u,co} / \theta_{u,cs}$ for RC members incorporating rebar couplers and subjected to cyclic action, is extracted from previous studies [6,9-11,30-32] and presented in Figure 16. The parameter $\theta_{u,co}$ represents the ultimate rotation of the member with couplers, whilst $\theta_{u,cs}$ represents the corresponding ratio in non-spliced counterparts, considering that these members have similar material properties and geometries; they were also subjected to the same loading procedure. Values of $\theta_{u,co} / \theta_{u,cs}$ below unity indicate a reduction in ductility due to the presence of couplers.

Besides the information extracted from the literature, $\theta_{u,co} / \theta_{u,cs}$ for members without axial load investigated in this paper are also plotted in Figure 16. As observed, the member with compact (PTC) couplers (C300-CC-N0) had the most favourable performance with $\theta_{u,co} / \theta_{u,cs} \approx 1.00$, whilst the member with slender (PTSC) couplers (C300-CS-N0) had $\theta_{u,co} / \theta_{u,cs} = 0.77$. In most cases, as illustrated in Figure 16, members with couplers showed reduced performance. For the member with

RTC, the poor interlock at the coarse coupler-rebar thread led to rebar slip inside the coupler and reduction in ductility of about 14%, whilst for GSC, the slip between the rebar and the grout reduced the ductility by up to 50% or more. A similar situation was observed in Specimen C300-CS-N0 employing slender couplers, in which the coupler-concrete slip altered the plastic hinge behaviour and reduced the ductility. Although controlled debonding between the rebar and grout in previous GSC member tests minimised the loss of ductility in comparison to the monolithic counterpart [30], such dissipation mechanisms can become unreliable when significant non-linear behaviour develops. Other compact systems, such as OSWC or OBLC are not recommended for use in plastic hinge regions, as the eccentric loading on the coupler from offset bars produces unrestrained rotation, concentrating the damage around the coupler [12]. Additionally, intrinsic concentration of stresses in rebars at the bolts of BLC and OBLC splices, renders these systems less reliable than threaded systems, as previous results on embedded splices subjected to cyclic loads showed a decrease in displacement ductility reaching up to 60% in comparison to their non-spliced counterparts [9,33].

It is clear that members with compact threaded couplers such as PTC and UHC, and subjected to low and moderate levels of axial load, exhibit no reduction in ultimate rotation in comparison to the non-spliced counterparts. This can be directly related to the uniaxial performance of splices incorporating these types of couplers as they typically develop uniaxial ductile response with failure outside the coupler region. Although in the uniaxial tests described in this paper, the spliced specimens had reduced fracture strain in comparison to their non-spliced counterpart, exhibiting a similar decreasing trend in terms of ductility as the beam-column members, the responsible mechanisms differ. Due to the relatively low bond and high rigidity at the coupler region, the curvatures within the plastic hinge region in flexural RC members create concentration of strains at the rebar-coupler interface. This may lead to non-uniform rebar-thread to coupler-thread bearing, hence magnifying the influence of the coupler on the member performance. Clearly, this effect would be less significant in relatively compact couplers.

The above discussions suggest that couplers with size factors λ_s around 1.00, grouped towards the left-hand side of Figure 1d (e.g. PTC, TTC, UHC), are generally compact. In contrast, couplers with λ_s approaching 2.00 (e.g. BLC, GSC) are relatively slender and rigid. Using λ_s as a comparative measure, coupling systems with $\lambda_s \leq 1.0$, and that exhibit favourable ‘in-air ductility’, may be preferred in dissipative zones, as these performed better in terms of ductility in the tests depicted in Figure 16. For example, the member with PTC had $\lambda_s=0.65$ and showed no reduction in θ_u , the member with PTSC had $\lambda_s=1.19$ and exhibited 23% reduction in θ_u , whilst specimens with GSC and with $\lambda_s=1.83-1.98$ developed θ_u with up to 50% less than the corresponding non-spliced specimens.

Based on the observations above, a conservative representation of the λ_s versus $(\theta_{u,co}/\theta_{u,cs})$ relationship is plotted in Figure 16. This may be used to estimate the reduction $\theta_{u,co}$ in comparison to $\theta_{u,cs}$ as a function of the size/bulkiness factor of the couplers. Additionally, the couplers must satisfy the elastic slip (related to serviceability) and strength criteria required for the specific design case.

The test results described in this paper show that slender couplers shift the maximum curvatures outside the coupler region, hence altering the curvature distributions within the dissipative zone. For slender configurations, a minimum distance, equivalent to the member depth, between the base of the coupler and the interface should be considered. Improved performance may be obtained when slender couplers are located at the interface, as these may increase capacity by translating the weak section above the coupler region [34]. Consequently, the design should consider that the plastic hinge is translated or altered as a function of the coupler location. In contrast, for relatively compact couplers, a response similar to a non-spliced counterpart can be achieved, and the design could therefore follow typical codified guidance without modification.

5. Concluding remarks

In this paper, two types of threaded reinforcement couplers were selected for detailed experimental assessment including monotonic and cyclic uniaxial in-air and in-concrete behaviour, as well as tests on RC specimens subjected to inelastic lateral cyclic deformations. Based on the test results and observations, a number of response characteristics were investigated, including stiffness, strength, ductility, spread of plasticity, rotation capacity and energy dissipation. These were used to highlight key design considerations which may be used for practical application of mechanical couplers in dissipative zones. The main conclusions from this study are summarised below.

The uniaxial in-air tests showed that the threaded couplers perform well in the elastic regime without notable slip at the threads and seem to provide marginally less ductility, as a function of coupler length, than non-spliced specimens, yet developing ultimate deformations above 7.5% that comply with European seismic design requirements. The relatively long length of the slender coupler in comparison to the member section size from lateral cyclic tests, combined with the short coupler to footing distance, localised the reinforcement strains below the coupler, at the interface, altering the plastic hinge behaviour and reduced the rotational capacity by about 23% in comparison to the non-spliced member, increasing curvatures and leading to earlier failure. Also, due to the smooth coupler surface, some debonding occurred resulting in a shift in the neutral axis towards the member edge and producing a rocking-type response.

The response of the member incorporating compact couplers, and without axial load, was virtually identical to that of the specimen with continuous reinforcement. The stiffness, strength, inter-cycle degradation, ductility and energy dissipation were not notably affected by the presence of the compact coupler and the lateral capacity was largely the same. The axial load enhanced the stiffness and lateral capacity in comparison to the non-axially loaded specimen with compact couplers, yet halved the ultimate rotation, increased the level of inelastic degradation with the increase in lateral deformation, and the response was strongly governed by crushing in which buckling of the compression reinforcement increased strain concentration and instigated rebar fracture at the coupler, emphasising the need for dense and specific stirrup arrangement at the coupler region.

An estimation of the spread of plasticity indicated a shorter continuous plastic hinge region for the non-spliced member, and longer interrupted inelastic lengths for the members with couplers characterised by governing maximum curvatures localised below the couplers and with curvatures around yield, above the couplers. Estimates of the plastic hinge length using existing methods showed that codified procedures may be used without modification for the assessment of ductility characteristics in non-spliced members and members with compact couplers, whilst for assessment rotational response of members in which curvature localisation occurs between the coupler and member-to-footing a shorter plastic hinge length should be considered. The observations from this paper show that current codified provisions may be used for the assessment and design of members incorporating compact couplers whilst, for other configurations, the design should account for the implications on ductility.

Based on the test results from this paper and others available in the literature, a simple relationship for predicting the reduction in ultimate rotation of a spliced member in comparison to a non-spliced counterpart as a function of the size/bulkiness factor of the couplers was proposed in this paper. Key observations suggest that compact couplers, particularly threaded types, exhibiting a favourable balance of ‘in-air ductility’ and ‘size-bulkiness’, seem to have the best performance in terms of strength and ductility. Also, relatively slender threaded configurations may be an alternative for connecting reinforcement bars in plastic hinge zones, provided they are optimally located and the influence on ductility is assessed. In comparison to grouted or bolt lock couplers, threaded systems have superior connection characteristics, working effectively both in tension and compression. However, depending on the level of ductility required, other systems may perform well in inelastic RC regions.

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Figure 16 Relationship between coupler size factor and ductility ratio

Nomenclature

Greek letters

δ – displacement/deformation
 δ_y – yield deformation
 ε – strain
 ε_f – fracture strain/deformation
 ε_u – ultimate strain/deformation
 $\varepsilon_{u,sp}$ – ultimate strain of a splice
 $\varepsilon_{u,b}$ – ultimate strain of a bar
 ϕ – curvature
 ϕ_{test} – test curvature
 ϕ_u – ultimate curvature
 ϕ_y – yield curvature
 λ_s – size/bulkiness factor
 μ_δ – displacement ductility factor
 μ_ϕ – curvature ductility factor
 μ_θ – rotational ductility factor
 σ – stress
 θ – rotation
 θ_p – plastic rotation
 θ_u – ultimate rotation
 θ_y – yield rotation
 $\theta_{y,EP,pred}$ – estimated bilinear yield rotation
 $\theta_{u,EP,pred}$ – estimated bilinear ultimate rotation
 $\theta_{u,EP,test}$ – test bilinear ultimate rotation
 $\theta_{u,co}/\theta_{u,cs}$ – ultimate rotation ratio
 $\theta_{u,co}$ – ultimate rotation of the member with couplers
 $\theta_{u,cs}$ – rotational ductility factor of the of the reference cast in situ member
 $v = N/A_c f_c$ – axial load ratio
 $\Delta\theta_{slip}$ – slip rotation

Lowercase latin letters

d_b – bar diameter
 d_c – coupler diameter
 f_c – cylinder concrete strength
 $f_{c,cube}$ – cube concrete strength
 $f_{ct,sp}$ – concrete splitting strength
 $f_y, f_{y,b}$ – yield strength of a rebar
 f_{yk} – characteristic yield strength
 $f_{u,sp}$ – ultimate strength of a splice

$f_{u,b}$ – ultimate strength of a rebar
 k_{cr} – member stiffness
 s_w – stirrup spacing

Uppercase latin letters

A_c – concrete section
 D – member section size
 E_d – dissipated energy
 L_b – effective length of the splice
 L_c – coupler length
 L_{pl} – predicted plastic hinge length
 L_{pr} – experimental plastic hinge length
 L_s – span
 M – moment
 M_y – yield moment
 M_u – ultimate moment
 M_R – moment capacity
 $M-\theta$ – moment-rotation
 N – axial load
 $\emptyset$ – diameter
 V – lateral force
 V_y – yield lateral force
 V_{max} – maximum lateral capacity
 $V-\delta$ – load-deformation

Tables

Table 1 Steel material properties in beam-column members

Batch	Connecting couplers	Longitudinal				Transverse		
		f_y (MPa)	f_u (MPa)	ϵ_u (%)	ϵ_t (%)	f_y (MPa)	f_u (MPa)	ϵ_u (%)
C300-C0-N0	None	568	653	8.32	20.3	522	652	10.6
C300-CS-N0	Slender	578	662	8.35	18.8			
C300-CC-N0	Compact	571	674	9.59	18.3			
C300-CC-N1								

Table 2 Concrete strength

Specimen(s)	Coupler	Footing (28 days)			Strength at 28 days (MPa)			Strength at member testing day (MPa)		
		f_c (MPa)	$f_{c, \text{cube}}$ (MPa)	$f_{ct,sp}$ (MPa)	f_c (MPa)	$f_{c, \text{cube}}$ (MPa)	$f_{ct,sp}$ (MPa)	f_c (MPa)	$f_{c, \text{cube}}$ (MPa)	$f_{ct,sp}$ (MPa)
Uniaxial tests	All	-	-	-	44.8	51.4	3.23	53.3	59.8	4.71
C300-C0-N0	None	54.3	61.3	3.30	58.3	81.2	4.54	74.6	87.5	4.40
C300-CC-N0	Compact				64.5	78.7	4.20	73.9	89.5	4.13
C300-CC-N1	Compact							70.0	84.2	4.07
C300-CS-N0	Slender							70.8	84.5	4.15

Table 3 Beam-column members test results

Specimen	N (kN)	V_{y+} (kN)	V_{y-} (kN)	V_y (kN)	V_{max} (kN)	δ_y^* (mm)	δ_u (mm)	δ_f (mm)	M_y (kNm)	M_u (kNm)	θ_y (mrad)	θ_u (mrad)
C300-C0-N0	0	73.2	70.7	71.2	92.3	14.0	94.4	105	96.1	125	10.5	69.9
C300-CS-N0	0	47.4	52.3	49.9	92.9	9.0	70.4	90	67.4	125	5.82	53.9
C300-CC-N0	0	62.4	64.6	63.5	88.4	12.0	92.0	105	85.7	119	9.10	70.2
C300-CC-N1	800	125	121	123	129	14.0	39.6	78	170	187	11.3	33.9

*corresponds to first yield recorded by strain gauges

Table 4a Ductility assessments (I)

0	Coupler	L_c	$\theta_{y,EP}$	$\theta_{u,test}$	μ_0	L_{pl}	L_{pl}^*	ϕ_y	ϕ_u
		(mm)	(mrad)	(mrad)	(-)	(Eq 2b) (mm)		(rad/km)	(rad/km)
C300-C0-N0	none	0	7.71	69.9	9.06	254	254	17.1	182
C300-CS-N0	slender	206	6.65	53.9	8.11	256	128	17.1	182
C300-CC-N0	compact	54	7.83	70.3	8.97	254	254	17.1	182
C300-CC-N1	compact	54	4.32	33.9	7.85	254	254	13.2	45.9

L_{pl}^* - plastic hinge length considered in assessments.

Table 4b Ductility assessments (II)

	Coupler	$\theta_{y,calc}$	$\theta_{p,calc}$	$\Delta\theta_{slip,calc}$	$\theta_{u,calc}$	$\theta_{y,calc} /$	$\theta_{u,calc} /$
		(mrad)	(mrad)	(mrad)		$\theta_{y,test}$ (-)	$\theta_{u,test}$ (-)
C300-C0-N0	none	7.71	37.8	16.0	61.5	1.00	1.14
C300-CS-N0	slender	7.71	20.0	16.0	43.8	0.86	1.23
C300-CC-N0	compact	7.71	37.9	16.0	61.6	1.02	1.14
C300-CC-N1	compact	5.95	7.5	4.0	17.5	0.73	1.93

Figures

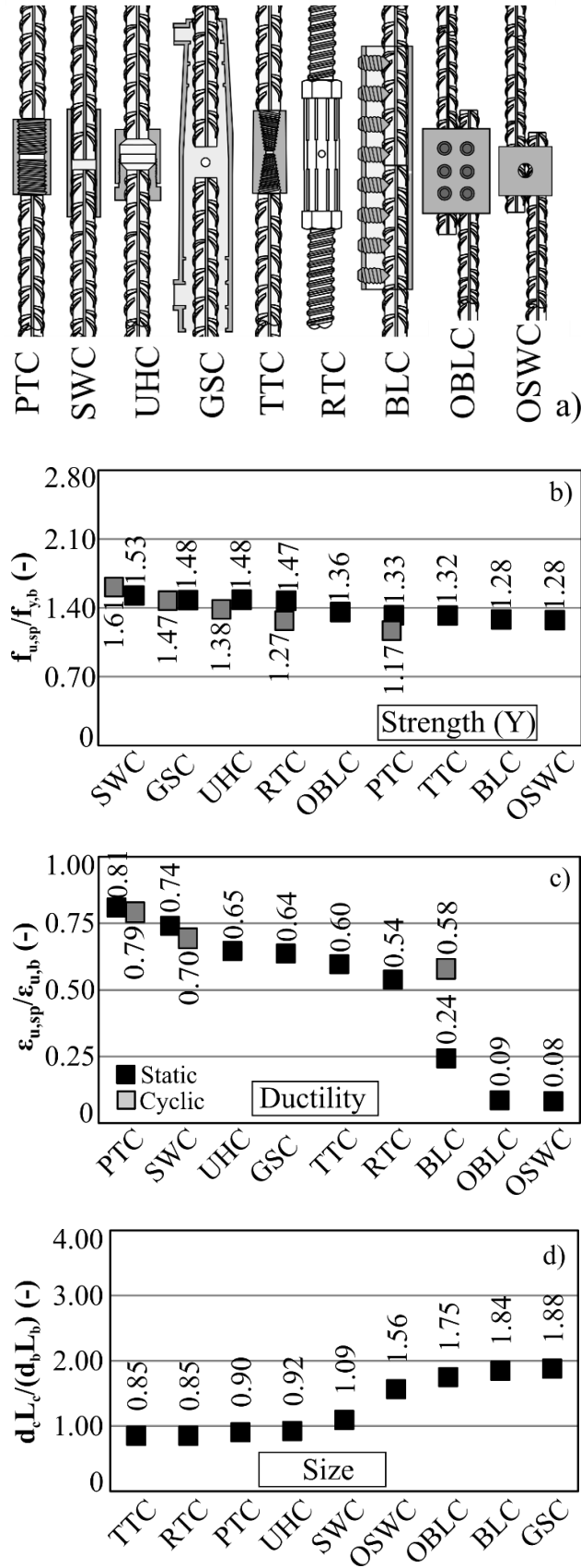


Figure 1 a) Coupling systems considered in survey [14]; Comparative performance of mechanical splices: b) strength at yield - $f_{u,sp}/f_{y,b}$; c) ductility - $\epsilon_{u,sp}/\epsilon_{u,b}$; d) size/bulkiness - $\lambda_s = d_c L_c / d_b L_b$; (PTC Parallel threaded, SWC Swaged, UHC Upset headed, GSC Grouted sleeve, TTC Taper threaded, RTC Rib thread, BLC Bolt lock, OBLC Offset bolt lock, OSWC Offset wedge swaged)

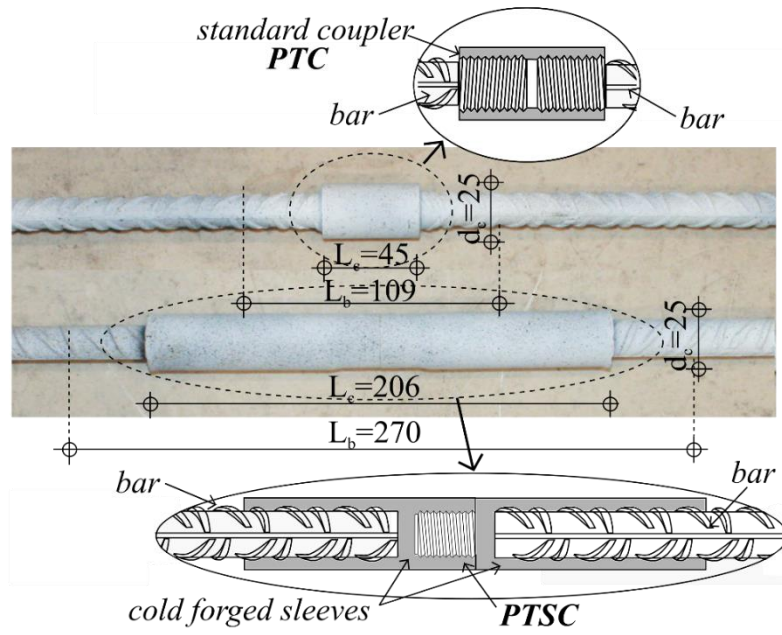


Figure 2 Coupler configuration

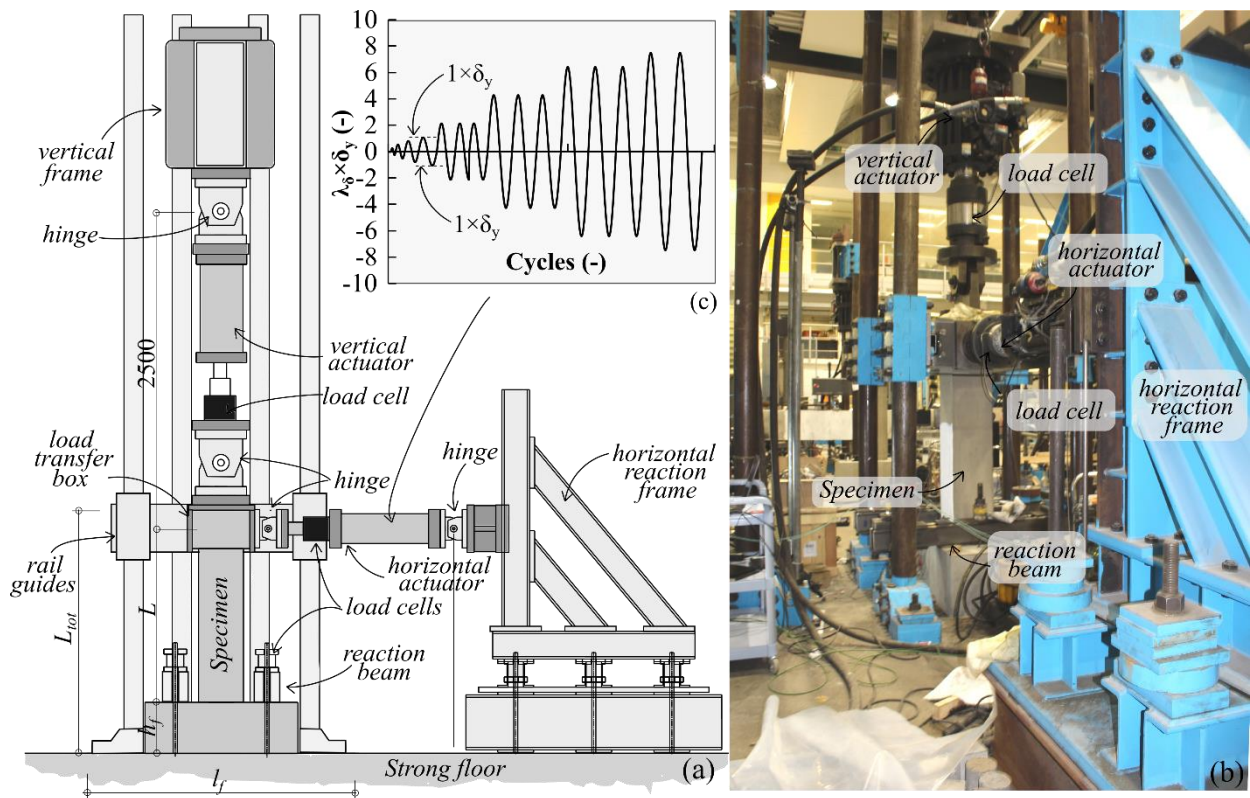


Figure 3 Testing arrangement: a) test-rig layout, b) general view, c) lateral displacement protocol

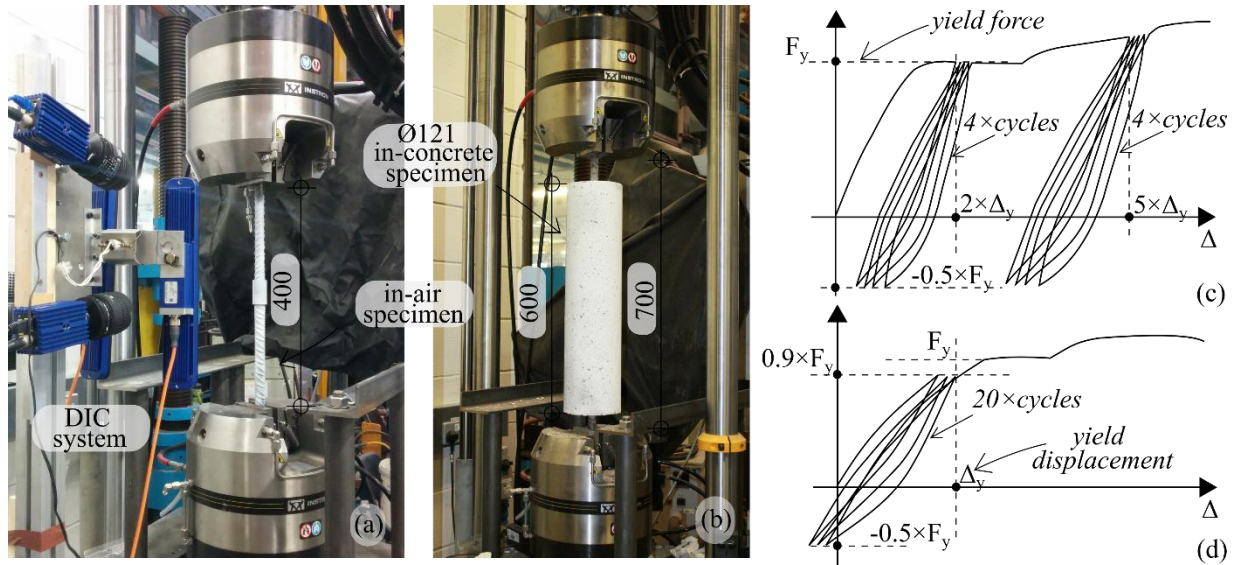


Figure 4 Testing configuration for uniaxial tests: a) in-air tests, b) in-concrete tests, c) S2 loading protocol, d) S1 loading protocol

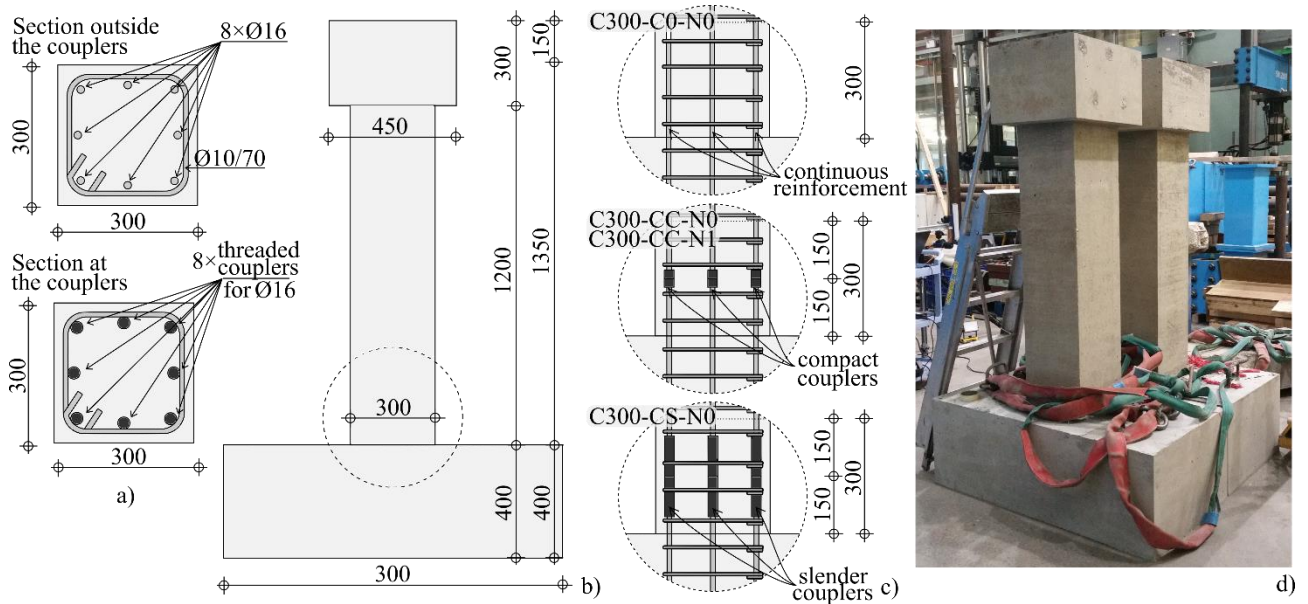


Figure 5 RC specimen details: a) cross-sections, b) side view, c) coupler positions, d) view of two of the members before testing

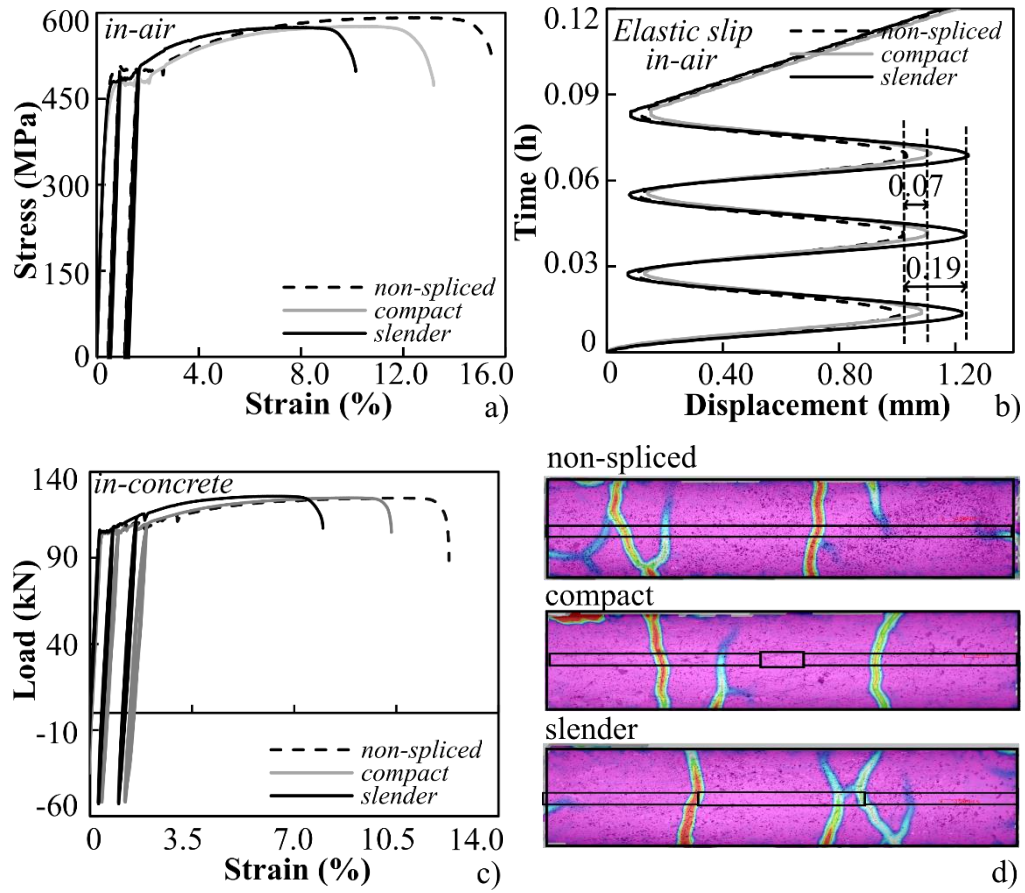


Figure 6 a) Stress-strain results for in-air cyclic tests; b) Elastic slip measurements for in-air specimens, c) Load-deformation of in-concrete specimens under cyclic loading; d) Crack patterns for in-concrete tests.

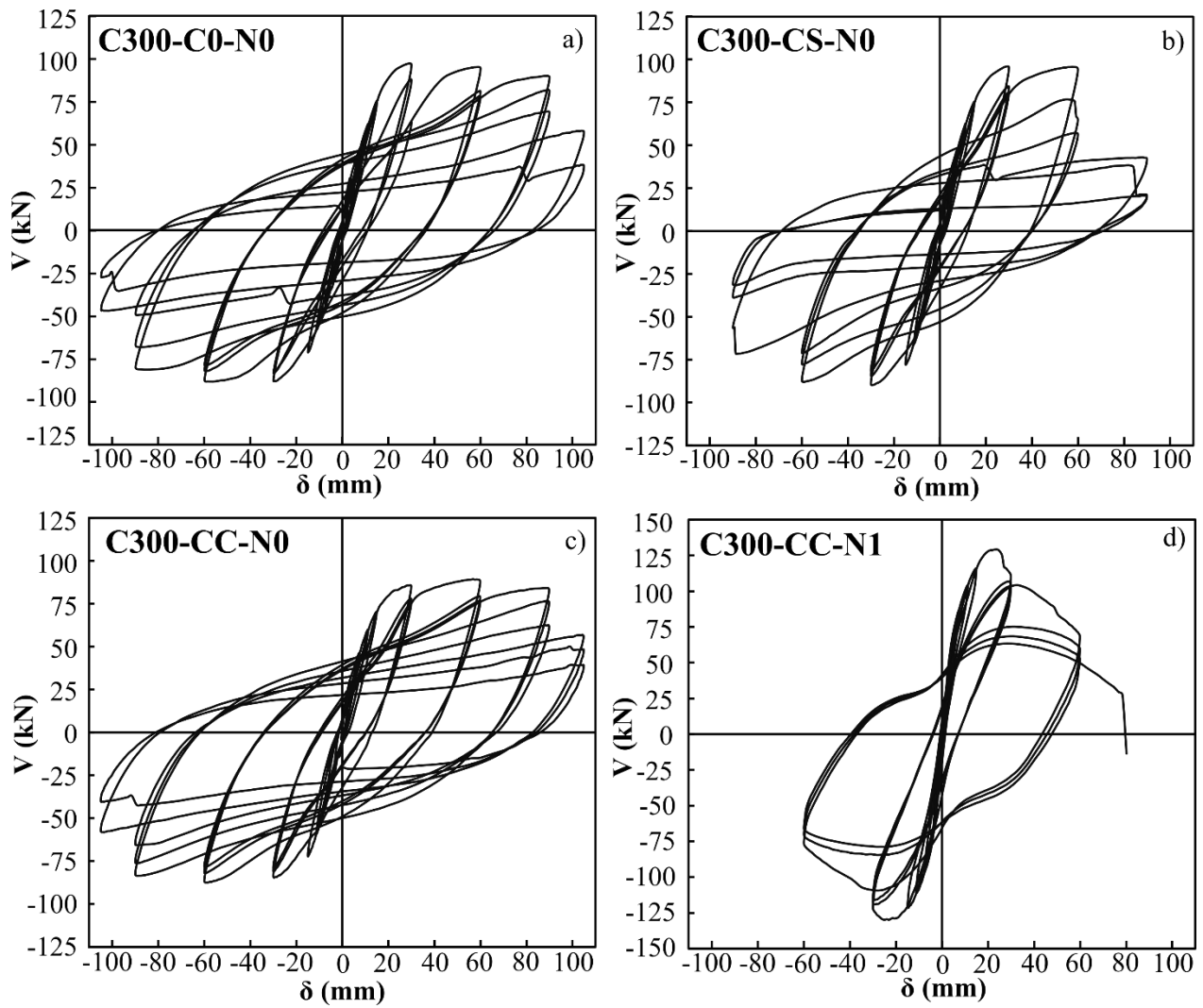


Figure 7 Load – lateral deformation (V - δ) response for Specimens: a) C300-C0-N0, b) C300-CS-N0, c) C300-CC-N0, d) C300-CC-N1

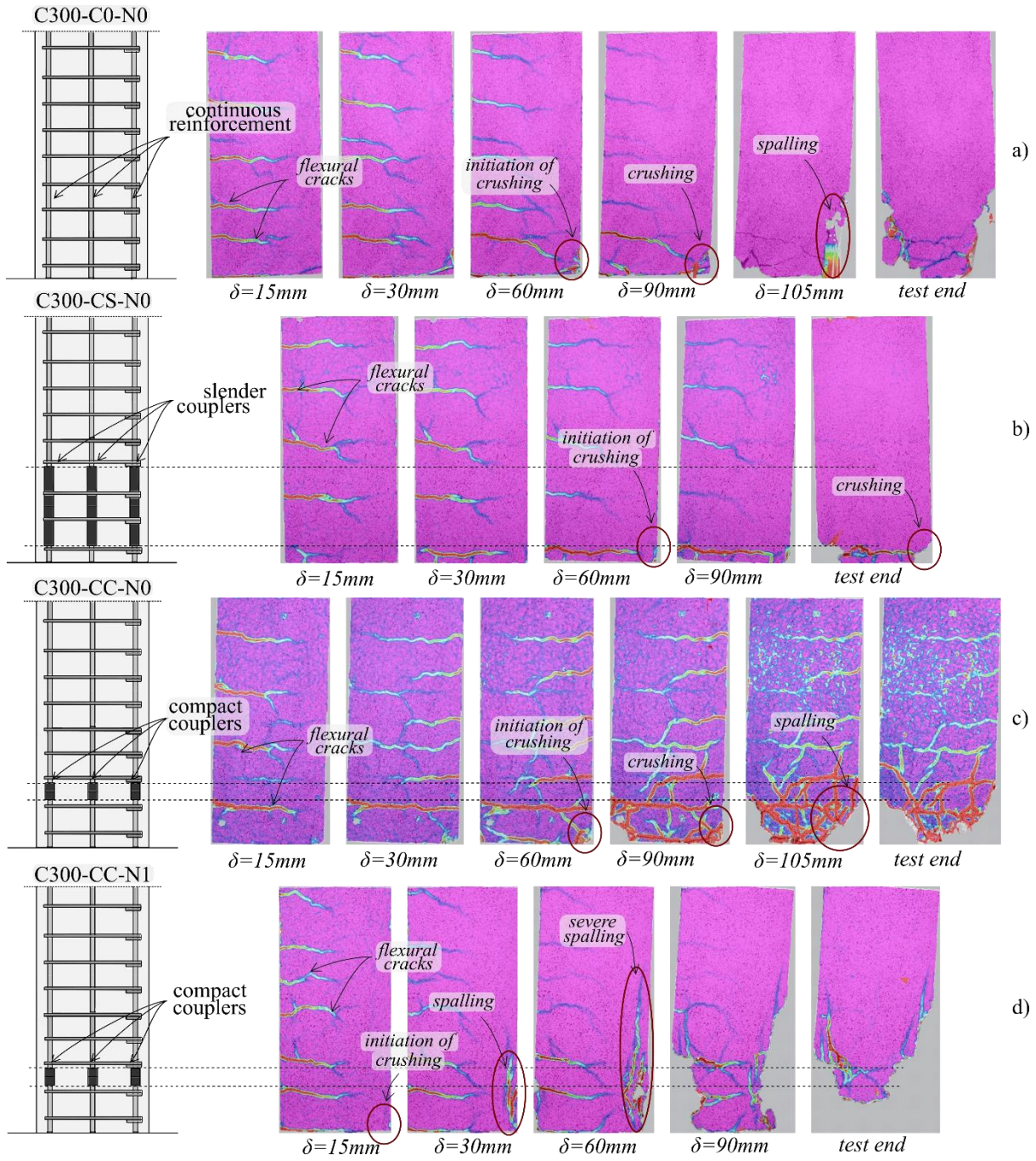


Figure 8 Damage propagation and crack patterns at various applied lateral deformation levels for Specimens: a) C300-C0-N0, b) C300-CS-N0, c) C300-CC-N0, d) C300-CC-N1



Figure 9 a) Reinforcement fracture mode for C300-C0-N0 and C300-CC-N0, b) reinforcement fracture in C300-CS-N0, c) fracture at the couplers-to-rebar interface in C300-CC-N1 following buckling of reinforcement

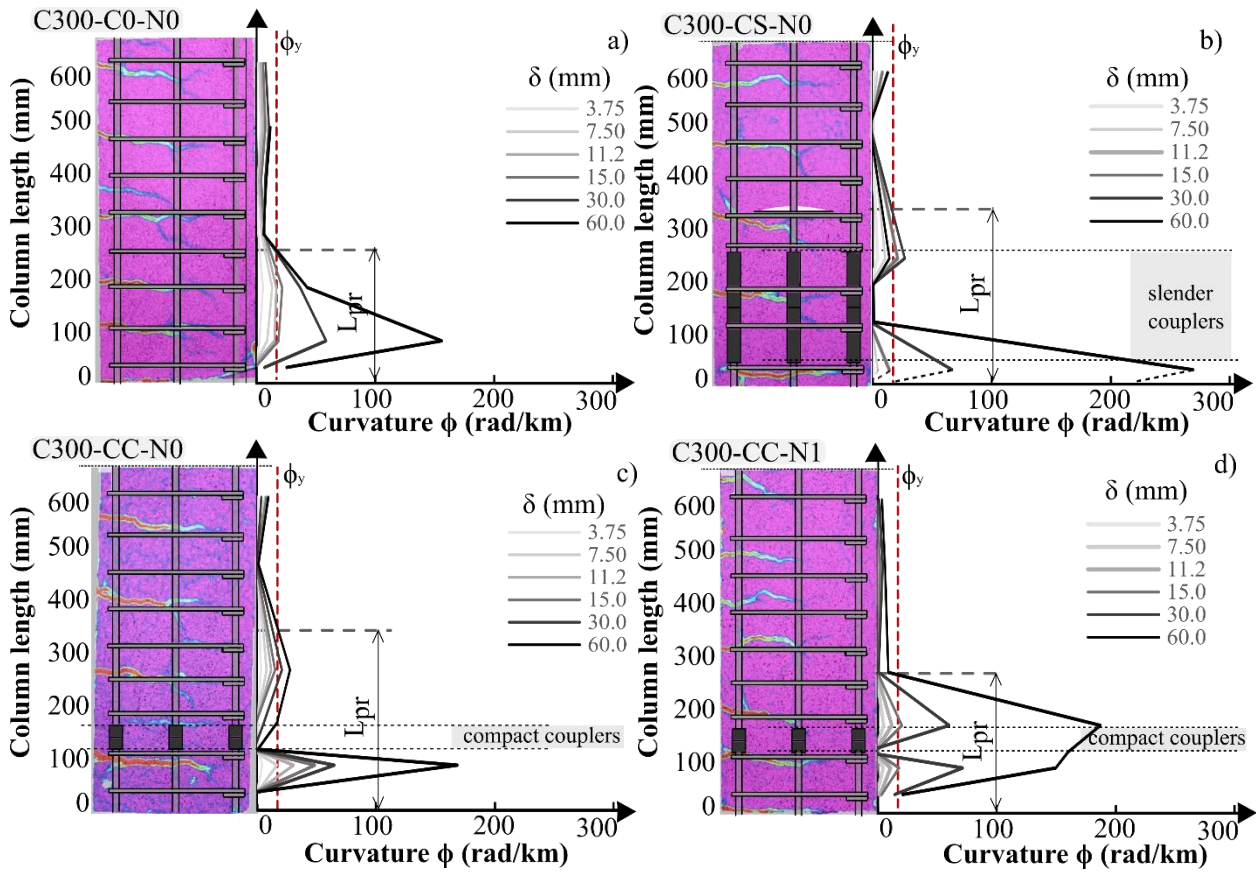


Figure 10 Experimental curvature profiles: a) C300-C0-N0, b) C300-CS-N0, c) C300-CC-N0, d) C300-CC-N1

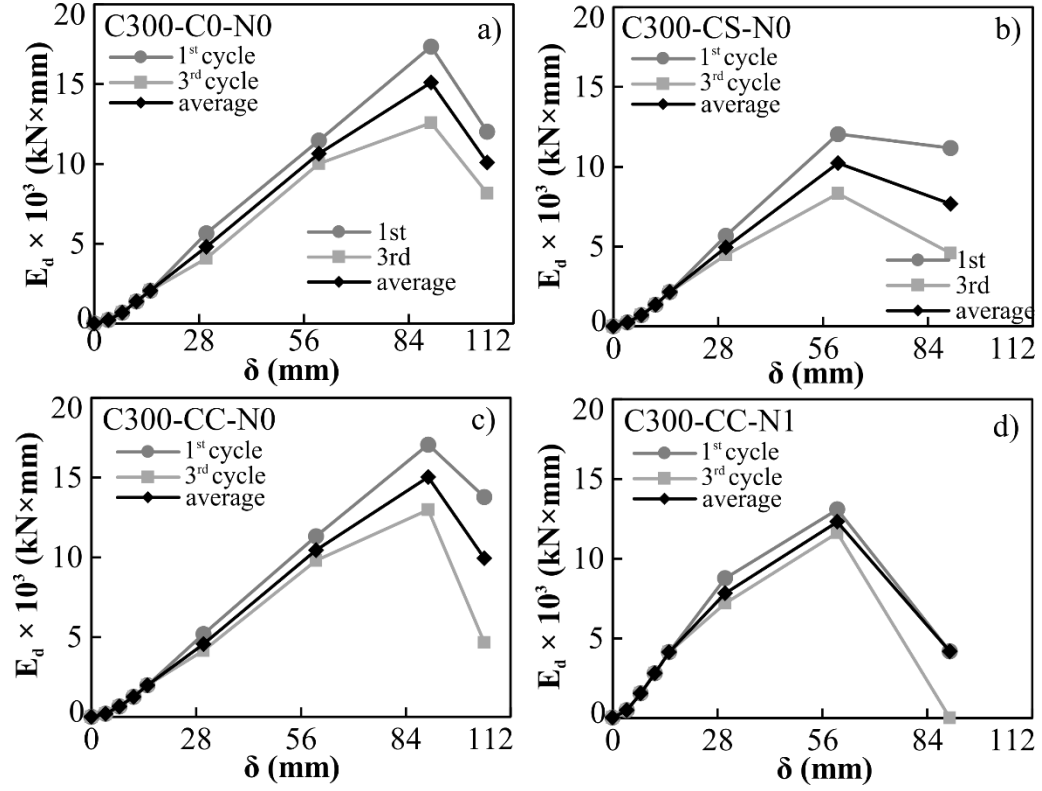


Figure 11 Energy dissipation for Specimen: a) C300-C0-N0, b) C300-CS-N0, c) C300-CC-N0, d) C300-CC-N1

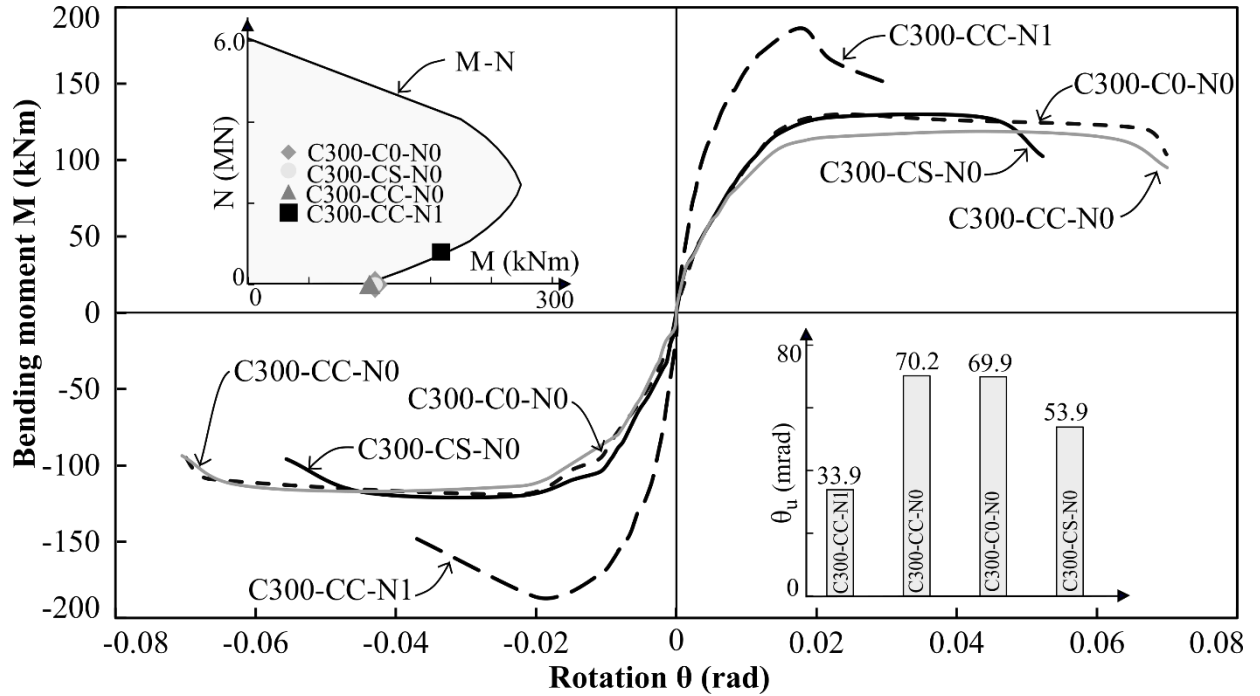


Figure 12 Moment-rotation $M-\theta$ envelope curves

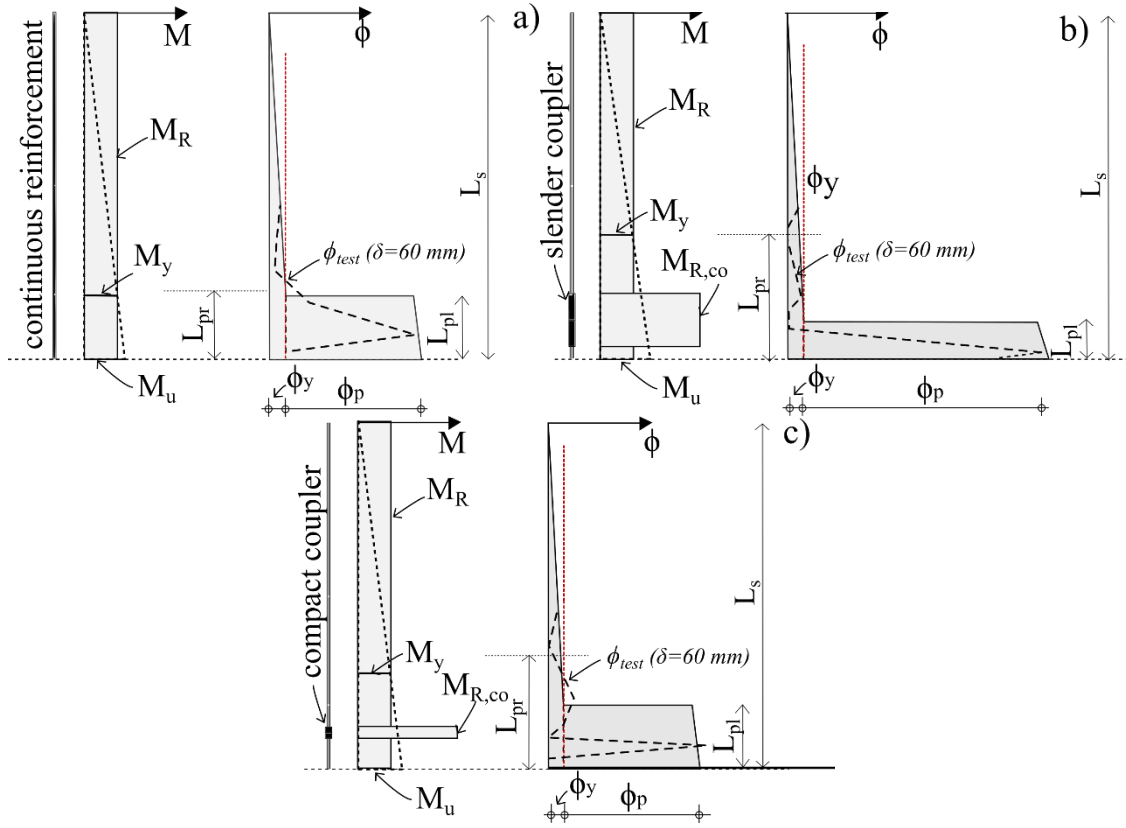


Figure 13 Moments (M) and curvatures (ϕ) for cantilever members: a) C300-C0-N0, b) C300-CS-N0, c) C300-CC-N0

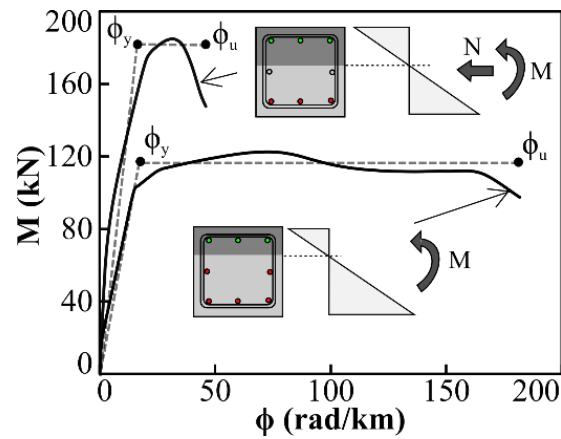


Figure 14 Moment-curvature M - ϕ analysis of the cross-section with reinforcement

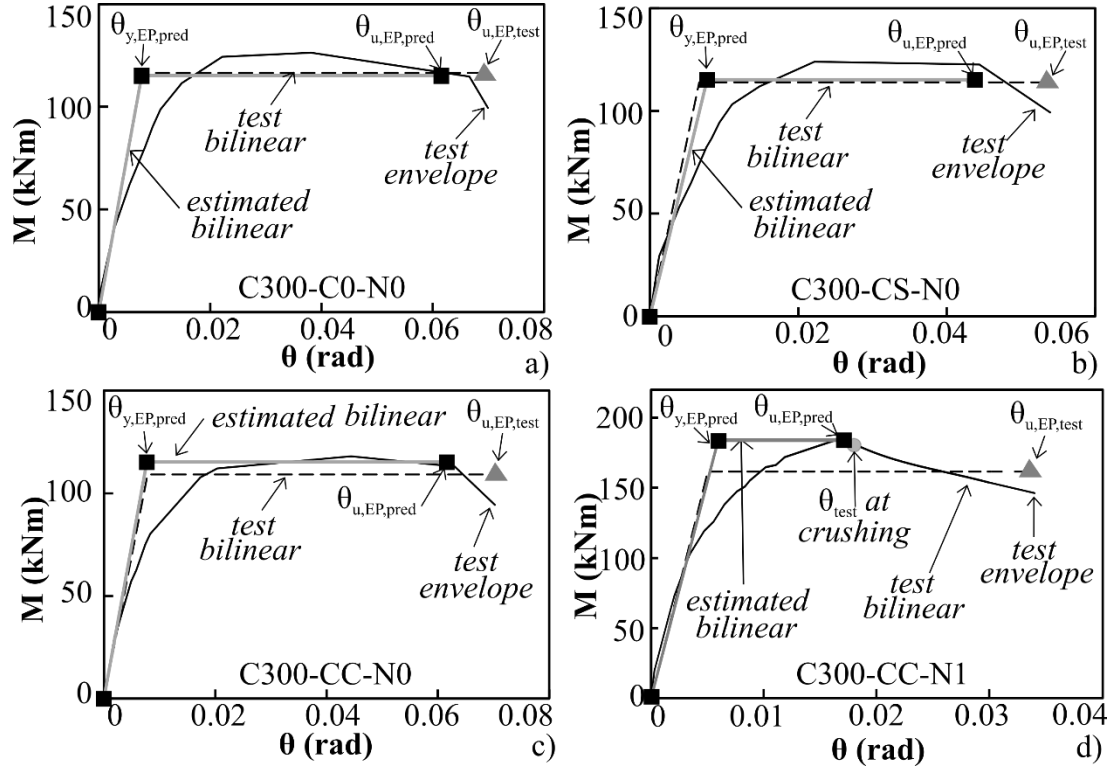


Figure 15 Test and predicted $M-\theta$ envelopes for Specimen: a) C300-C0-N0, b) C300-CS-N0, c) C300-CC-N0, d) C300-CC-N1

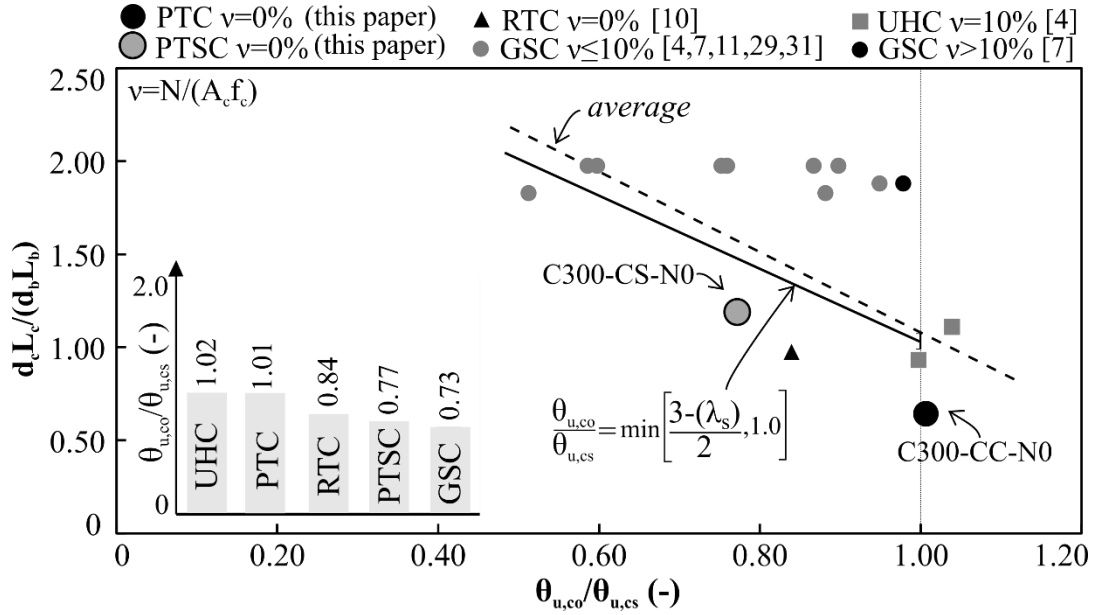


Figure 16 Relationship between coupler size factor and ductility ratio