Shear strength formula for interior beam-column joints with plain bars in existing buildings

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A B S T R A C T

Beam-column joint behavior in Reinforced Concrete (RC) structures is a key issue for earthquake engineering, because, under seismic loads, these elements are subjected to high stresses, which could lead to unexpected collapses, due to the development of brittle failure mechanisms. In Italy, many existing RC buildings constructed before the mid-1970's present structural deficiencies, as they were designed in the absence of the prescriptions of modern seismic codes. In particular, the use of plain reinforcing bars, inappropriate anchorage solutions and the lack of joint horizontal hoops are rather widespread.

This paper presents an in-depth overview on seismic behavior of interior beam-column joints reinforced with plain bars, under cyclic loads, based on experimental results available in the literature. The inadequacies of certain reinforcement arrangements and details of tested joints are highlighted. A critical discussion of the damages and failure mechanisms developed in the joints, in relation to the reinforcement details and to the effects of the main influencing parameters, is provided. These parameters are the mechanical properties of materials, the geometry of the joint and the converging elements, and the column axial load. A formula to predict shear strength of joints with plain bars is proposed.

Due to the lack of similar papers in the literature, the present analysis constitutes the basis for a comprehensive understanding of behavior of interior joints with plain bars under seismic actions. Moreover, due to the absence in the main current Codes of design formulas for the shear strength assessment of existing beam-column joints with plain bars, a design formula is also proposed. This formula is a valuable tool to design the retrofit intervention of existing buildings reinforced with plain bars, constituting a not negligible part of the built heritage.

1. Introduction

It is well known that Reinforced Concrete (RC) buildings under seismic actions are subjected to horizontal forces which impose large deformations on the structure, and lead the members' response into the post-elastic range [1].

In modern seismic codes, the design of RC frames is based on the principles of capacity design, which provide for the development of plastic hinges in specified regions of the structural elements, avoiding the occurrence of brittle failure mechanisms. Conventionally, the strong-column/weak-beam design approach is assumed, because it ensures the best ductile behavior of the structure, by developing plastic hinges at the ends of the beams.

Before the mid-1970's, due to the limited extension of the zones classified as seismic, RC Italian buildings were normally designed to resist to gravity and wind loads only, without considering the seismic action. Hence, many existing RC structures in Italy cannot resist even minor earthquakes, as they present several structural deficiencies, such as the absence of horizontal hoops in the joints, inadequate reinforcement anchorages, and the use of plain reinforcing bars. This condition of existing RC buildings in Italy is representative of buildings present in other seismic regions of the Mediterranean area.

Under seismic load conditions, the framing elements can transmit high stresses to the joint core, whose behavior is highly influenced by the geometric and mechanical properties of the adjacent members.

The lack of transverse shear reinforcement in beam-column

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connections could cause brittle shear failure of the joints and the sudden collapse of the building [2]. Furthermore, the use of plain bars instead of deformed ones heavily influences the steel–concrete bond mechanism, due to the different bond-slip relationships [3], and could lead to slippage of the reinforcement. In particular, beam bars slippage in interior joints can produce additional lateral deformation of the structure with the potential onset of unexpected soft-story failure mechanism. It follows that beam-column joints are critical elements in RC existing structures, because their behavior is governed by shear and bond-slip mechanisms, which may lead to brittle and sudden failures. Hence, in order to design retrofitting of existing RC buildings reinforced with plain bars, it is fundamental to study the seismic performance of beam-column joints.

In recent years, many experimental tests have been conducted on interior beam-column joints reinforced with deformed bars, like in [5–9], and empirical and mathematical models to predict these joints shear strength have been proposed, like in [10–13]. However, only few experimental studies have been carried out on beam-column joints reinforced with plain bars, even though many existing RC buildings have this kind of reinforcement. Moreover, no model for shear strength calculation of beam-column joints with plain bars is present in the literature. A relevant gap in this research field is given also by the absence of a comprehensive collection of different joints' typologies based on geometry, reinforcement, mechanical properties, and load configuration. In addition, also a comprehensive discussion on the failure modes of these joints is lacking, even if their failure may be one of the principal causes of collapse of existing buildings with plain bars.

The motivation to this research study just comes from the recognition of these deficiencies. Hence, the principal objectives that this study aims to achieve are to provide a comprehensive discussion on failure typologies of beam-column joints with plain bars based on their geometry, reinforcement, mechanical properties, and load configuration, and to propose a formula to predict the shear strength of RC joints with plain bars.

This paper reports the largest possible collection of experimental tests, available in the literature, on interior beam-column joints reinforced with plain bars subjected to horizontal cyclic loads. Another similar collection is not present in the literature currently. Besides the presence of plain bars, the investigation considers several factors influencing the joint seismic response, including horizontal hoops amount and column axial load. An in-depth analysis and discussion on the failure modes of the test units is provided. The considered test specimens represent typical joints of existing RC buildings designed before the mid-1970's, which proved to be particularly prone to seismic failure in the past and, recently, also during the Türkiye earthquake (2023).

Regarding the analytical models available in the literature for the prediction of RC beam-column joint shear strength, considering the principal codes, the provisions of Eurocode 8 [14] for the design of interior beam-column connections are based on the verification of the principal compression and tensile stresses in the joint. In ACI Building Code [15], the shear strength only depends on the geometrical characteristic of the joint and the cylindrical compressive strength of concrete. Japanese standards AIJ [16] essentially focus on the diagonal concrete strut failure, assuming that sufficient joint shear reinforcements are provided in order to avoid other premature mechanisms.

Regarding research works in the literature, several authors proposed empirical and mathematical models to evaluate joint shear strength taking into account the main resisting contributions due to the concrete, the passing bars within the joint panel and the geometrical and mechanical characteristics of the elements [10–12,17–21]. Some models are formulated for specific conditions, like that of Kim and LaFave [17], which applies only to joints with horizontal reinforcement. Moreover in some cases, the proposed models provide a closed formulation [10,17–19], while in others the shear strength calculation is based on iterative procedures, like those of Hwang and Lee [20] or Wong and Kuang [21]. However, all the mentioned shear strength models, both those of the building codes and those from research works, have been derived and are valid only for beam-column joints reinforced with ribbed bars, not for joints with plain bars.

Since generally closed formulations are easier to use than iterative ones, the applicability of the closed form shear strength formulation of Pauletta et al. [10] for joints with deformed bars to joints with plain bars is studied herein. This formulation is modified to take account of the very low bond stresses transferred by plain bars. The provided shear strength formula is relevant because no other similar formulas are currently present in the literature, even if many existing buildings have beam-column joints similar to those presented herein.

This research study is significant in relation to the state of the practice, because, for the first time in the literature, provides:

- the largest possible collection to the best of the authors' knowledge of experimental tests on interior beam-column joints reinforced with plain bars subjected to horizontal cyclic loads, which describes geometry, reinforcement, mechanical properties, and load configuration of these joints;
- an in-depth analysis and discussion on the failure modes of the test units on the basis of their features;
- a shear strength formula, which takes account of the low bond of plain bars.

The impact of this investigation develops in a better understanding of the seismic behavior of interior joints RC of existing buildings reinforced with plain bars, and, consequently, in a greater awareness in the design of upgrade solutions for these joints.

By considering the different types of failure described in the following (section 3) for interior beam-column joints, different upgrade solutions can be undertaken to improve the joint behavior. Particularly interesting for the minimal invasiveness in terms of size are the applications of carbon fiber-reinforced polymer sheets [22] (FRP) or ropes [23,24].

2. Bond deterioration mechanism

The modern seismic design of RC structures ensures the development of plastic hinges in the beams, rather than in the columns, thereby avoiding soft-story failure mechanism in the perspective of the strong-column/weak-beam approach. Under seismic load conditions, the beams (and the columns) framing into the joint are subjected to

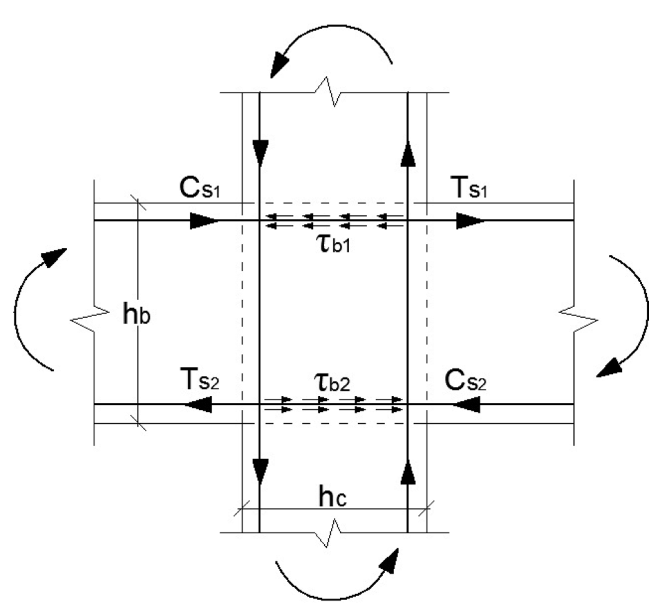


Fig. 1. Forces and stresses acting on the joint under seismic bending moments.

moments in the same direction (Fig. 1), caused by the horizontal seismic forces. As a consequence of this, the longitudinal beam bars passing through the joint core are in tension on one side and in compression on the opposite side. Therefore, under severe cyclic loads, high bond stresses develop along the beam bars in the joint core and bond deterioration may occur, if the upper limit of bond strength is reached.

By assuming the bond stress τ_{b1} acting on the bars to be constant along the joint core, and considering, for simplicity, one single bar, the equilibrium between the tensile force at one side, T_{s1}, the compressive force at the opposite side, C_{s1}, and the bond stress along the beam top bar (Fig. 1) are related by the expression

$$C_{s1} + T_{s1} = \tau_{b1} \bullet d_b \pi \bullet h_c \tag{1}$$

where d_b is the bar diameter and h_c is the column depth.

Since at the development of the plastic hinge the beam bar under tensile force yields at the column interface, by assuming an elastic-ideally plastic constitutive relationship for steel, the tensile and the compressive forces at the opposite sides of the joint can be computed as follows, respectively

$$T_{s1} = \frac{d_b^2}{4} \pi \bullet f_y \tag{2}$$

$$C_{s1} = \frac{d_b^2}{4} \pi \bullet f_s \tag{3}$$

where f_y and f_s are the yield strength and the compressive stress of the bar at the two sides, respectively. The compressive stress is assumed to be minor compared to the yield strength, since on the compression side the concrete also contributes to the resultant of compressive forces. To avoid bond deterioration, the bond strength developed along the bar should be greater than the forces acting on the bar at the two sides of the joint, i.e. the sum of tension T_{s1} and compression C_{s1} (Fig. 1).

By substituting Eq. (2) and (3) in Eq. (1), this conditions results in

$$\tau_{b1} \bullet d_b \pi \bullet h_c \geq \frac{d_b^2}{4} \pi \bullet (f_y + f_s) \tag{4}$$

By simplifying Eq. (4), the limiting value of the ratio h_c/d_b can be obtained

$$\frac{h_c}{d_b} \geq \frac{f_y + f_s}{4\tau_{b1}} \tag{5}$$

In many Codes Eq. (5) represents the condition to be respected to avoid bond deterioration inside RC beam-column joints. Since this issue is of fundamental importance for the structures' strength under seismic actions, many researchers have devoted time to its study.

Among these researchers Hakuto et al. [25] carried out an in-depth theoretical study on interior beam-column joints demonstrating the importance of having adequate ratios d_b/h_c. In particular, they observed that, when bond deterioration occurs, the beam bars at the tension side slide and remain anchored to the side in compression. Therefore, the penetration of the tensile stress in the joint through the bar and the transition of the bar on the compressed concrete side from compression to tension occur. These phenomena are the most probable for large diameter bars and short column depths. This is because large bar diameters and short column depths cause higher bond stresses concentrations. Furthermore, the phenomenon becomes critical when the bars are plain. Hakuto et al. also evaluated the reduction in flexural strength and available ductility of the beams, as a result of bond deterioration along the longitudinal bars passing through the joint. To calculate the flexural strength and the curvature ductility $\frac{\varnothing_u}{\varnothing_y}$, they assumed that sections remain plane under bending moments (Fig. 2), except for the strain in the reinforcement of the compression zone of the beam section (ε_1 greater than 0), whose compressive stress, after bond deterioration and bar slippage, becomes tension (f_1).

In the literature, the effect of bars' slips within the joint on members' flexural strength and ductility has been investigated also for the case of joints with deformed bars, as made in [26–32].

In the case of perfect bond conditions, for positive bending moments (left side of Fig. 1), the compressed reinforcement could develop a curvature ductility factor of 18. Conversely, in presence of bond deterioration, the stress in the compression reinforcement switches from compression to tension, the available curvature ductility factor, when the tensile stress in that reinforcement reaches the yield strength, is 5. This occurs just before the concrete, which remains under compression, crushes.

Furthermore, Hakuto et al. compared the limiting values of longitudinal beam bar diameter to column depth ratio (d_b/h_c) permitted by seismic design standards NZS 3101:1995 [33], ACI 318–95 [34] and AIJ draft design guidelines [35]. They observed that the codes provide different maximum d_b/h_c ratios. These differences depend on how each code weights the advantages and disadvantages of considering a specific d_b/h_c limit. Indeed, it has to be considered that very small d_b/h_c ratios correspond to very small diameters of the reinforcing bars, or to large columns, which lead to design and construction difficulties. On the other hand, very large d_b/h_c values lead to strong bond degradation and to the decrease of the global stiffness of the structure. In any case, Hakuto et al. suggested that a reduction in ductility of the beam plastic hinge should be considered when specifying the maximum permitted d_b/h_c value.

On the basis of all previous observations, it can be said that the ratio of longitudinal beam bars diameter to column depth is fundamental in designing interior beam-column joints. This ratio can seriously

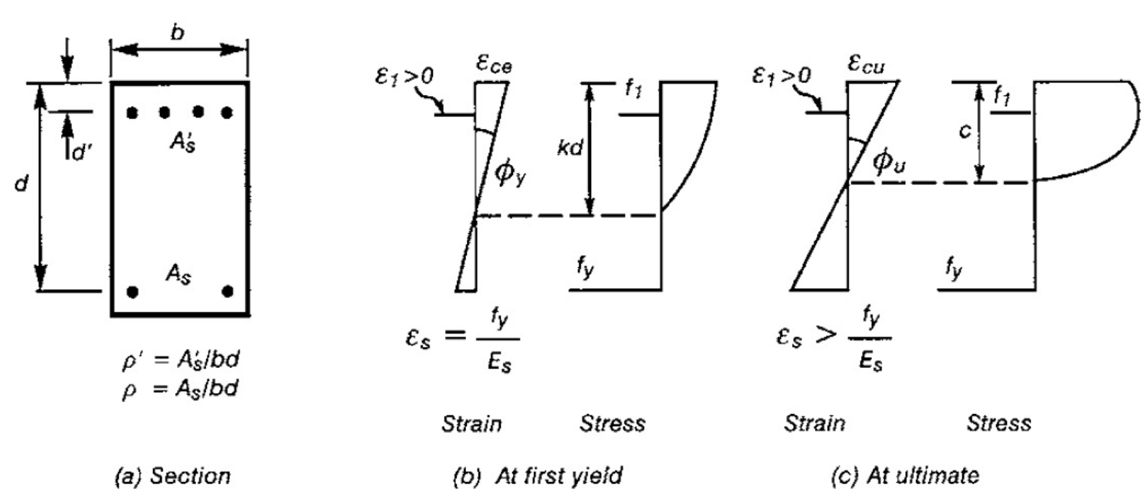


Fig. 2. (a) Doubly reinforced beam section at column interface; (b) at first yield; (c) at ultimate [25].

compromise the global behavior of RC structures under seismic loads, by producing bond deterioration and the development of bar slippage, with the potential onset of brittle failure mechanisms. Many existing buildings do not provide the required value of this ratio, and this should be considered for the correct assessment of their seismic behavior, especially when plain bars are present in the building.

In particular, according to the main building codes, the limiting ratios are the followings.

- NZS 3101:1995 [33]

$$\frac{d_b}{h_c} \leq 3.3\alpha_f \frac{\sqrt{f_c'}}{\alpha_0 f_y} \tag{6}$$

where f_y = lower characteristic (5 percentile) yield strength (MPa);

f_c' = specified concrete compressive cylinder strength (MPa);

$\alpha_f = 0.85$ when beam bars pass through a joint in two directions, as in two-way frames, or 1.0 when beam bars pass only in one direction, as in one-way frames; and

$\alpha_o = 1.25$ when plastic hinges in beams form at the column faces, or 1.0 when plastic hinges in beams form away from the column faces; hence, the sections at the column faces remain in the elastic range.

or, as an alternative to Eq. (6) and to take account of the column axial load

$$\frac{d_b}{h_c} \leq \frac{6\alpha_t \alpha_p}{\alpha_s} \alpha_f \frac{\sqrt{f_c'}}{\alpha_0 f_y} \tag{7}$$

where $\alpha_t = 0.85$ for a top beam bar when more than 300 mm of fresh concrete is

cast below the bar and $\alpha_t = 1.0$ in other cases;

$$\alpha_p = \frac{P}{f_c' A_g} + 0.95 \tag{8}$$

with the limitation of $1.0 \leq \alpha_p \leq 1.25$, where P = minimum axial compression load in the column consistent with the governing ultimate limit state load combination N, and A_g = gross area of the column, mm^2; and

$$\alpha_s = \frac{A_s'}{A_s} \tag{9}$$

with the limitation of $0.75 \leq \alpha_s \leq 1.25$, where A_s' = area of the smaller of the bottom or top beam reinforcement, and A_s = area of the other beam reinforcement.

- ACI 318–95 [34]

$$\frac{d_b}{h_c} \leq \frac{1}{20} \tag{10}$$

- AIJ [35]

$$\frac{d_b}{h_c} \leq \frac{6\left(1 + \frac{P}{A_g f_c'}\right) \bullet \frac{f_c'^{2/3}}{f_{yu}}}{1 + \gamma} \tag{11}$$

where P = axial compressive load on column (kg);

A_g = gross area of column (cm^2);

f_c'= compressive cylinder strength of concrete, (kgf/cm^2);

f_{yu} = upper bound strength of beam bar (kgf/cm^2); and

γ = ratio of area of beam compression reinforcement to area of beam tension, but not to exceed 1.0.

- EC8 [14]

$$\frac{d_b}{h_c} \leq \frac{7.5 \bullet f_{ctm}}{\gamma_{Rd} \bullet f_{yd}} \bullet \frac{1 + 0.8\upsilon_d}{1 + 0.75 k_d \bullet \frac{\rho'}{\rho_{max}}} \tag{12}$$

where f_{ctm} = mean value of the tensile strength of concrete;

f_{yd} = design value of the yield strength of steel;

υ_d = normalized design axial force in the column, $\upsilon_d = \frac{P}{f_{cd} A_g}$, with f_{cd} the concrete compressive design strength;

k_d = factor reflecting the ductility class, equal to 1 for DCB and to 2/3 for DCM;

ρ' = compression steel ratio of the beam bars passing through the joint;

ρ_{max} = is the maximum allowed tension steel ratio

$$\rho_{max} = \rho' + \frac{0.0018}{\mu_\phi \varepsilon_{sy,d}} \bullet \frac{f_{cd}}{f_{yd}} \tag{13}$$

with μ_ϕ the curvature ductility factor depending on the behavior factor of the building, $\varepsilon_{sy,d}$ and f_{yd} the design yield strain and stress of the beam bars, respectively;

γ_{Rd} = model uncertainty factor on the design value of resistances, taken as being equal to 1.2 or 1.0 respectively for DCB or DCM (due to overstrength owing to strain-hardening of the longitudinal steel in the beam).

It has to be stressed that, among previous formulations, ones of NZS, ACI and AIJ may be used to calculate the d_b/h_c ratios also for existing buildings, which could help in the assessment of these buildings' seismic behavior. Instead, formulation of EC8 is not suitable for existing buildings, because it requires the knowledge of the building behavior factor and ductility class, which are generally not defined for existing buildings with plain bars.

3. Experimental investigations available in the literature

The main research findings about interior beam-column joints with plain bars available in the literature are summarized below, in order to attain a comprehensive understanding of their behavior under seismic action.

Liu and Park [36].

Liu and Park investigated the seismic behavior of two RC interior beam-column joints with plain bars having low transverse reinforcement amount in beams and columns and no shear reinforcement in the joint core, representing conditions of existing buildings designed according to pre-1970′s codes. The two specimens, Unit 1 and Unit 2, were identical (Fig. 3) and had the same mechanical and geometric properties of Unit O1, belonging to another study conducted by Hakuto et al. [37], except

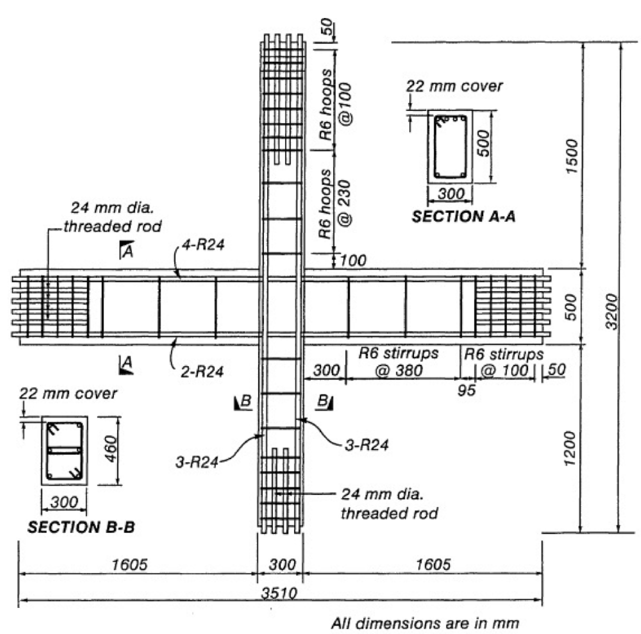


Fig. 3. Reinforcement details of Unit 1 and Unit 2 [36].

for the use of plain round bars for longitudinal reinforcement [25] instead of deformed bars [37]. These allowed a direct comparison between the behavior of the joints made with the two types of reinforcement.

Unit 1 was tested under zero axial load, while Unit 2 was tested with a constant compression equal to 0.12 $f_c^{'}A_g$, where $f_c^{'}$ is the concrete cylinder compressive strength and A_g is the column section gross area [36].

Theoretical considerations on sub-assemblages strength, as well as the inadequate development length of the plain bars within the joint core, led the authors to expect significant bond degradation and longitudinal bars slippage. The theoretical strengths of the beams and columns were obtained with the hypothesis of perfect bond between steel and concrete (plane section theory). The development of the plastic hinges was expected in the columns for Unit 1, and in the beams for Unit 2. The story shear, imposed at the column end, was calculated at the theoretical flexural strengths of the critical members and was equal to 80 kN for Unit 1 and 128 kN for Unit 2.

Liu and Park compared the experimental results of the two RC joints reinforced with plain bars to that of the joints reinforced with deformed bars, in terms of bond deterioration. In particular, test results for Unit 1, with zero axial load, revealed that the damage was concentrated at the column-joint interface, with horizontal flexural cracks, as a result of bond deterioration and slippage of the column longitudinal bars that increased the column fixed-end rotations. The authors observed that, for these bars, according to NZS 3101:1995 [33], the required h_b/d_c ratio was equal to 30.2, while the effective value was lower and equal to 20.8, where h_b is the beam height and d_c is the column bar diameter. Vertical cracks due to slippage of the longitudinal beam bars also occurred, at the beam-joint interface, but less pronounced than the horizontal cracks. According to NZS3101:1995 [33], the required h_c/d_b ratio for the beam longitudinal bars was 33, while the effective value was 12.5. Moreover, the tests revealed vertical cracks running through the columns and the joint core due to column bar buckling, as a result of the inadequate transverse reinforcement of the members. In fact, the stirrup spacing was equal to 230 mm and 380 mm in the columns and in the beams, respectively. There were no inclined tension cracks in the beams or in the columns, indicating that no more transverse reinforcement was needed for preventing shear cracks. The joint core, which was without stirrups, presented some minor diagonal cracks at the end of the test. The strains measured along the beam bars indicated that the bars slip through the joint induced the bars, which were theoretically under compression at one side of the joint, to be effectively in tension. As a result, the strains on the beam and column longitudinal reinforcement adjacent to the joint panel, as well as the flexural curvature, were higher. By comparing the theoretical strengths in terms of the story shear to the actual strengths, it emerged that the formers were overestimated, due to the plane section theory assumption for the column flexural strength at the plastic hinge. The use of plain bars led to a reduction in structural stiffness and flexural strength, with respect to the predicted values.

From the comparison of results of Units 1 and 2 with the results obtained by Hakuto [37] for the specimen O1 reinforced with deformed bars, Liu and Park concluded that the final failure of the sub-assemblages with plain reinforcing bars [36] was governed by bond degradation and column bar buckling, rather than joint shear failure, and attributed the units' low structural stiffness and strength to slippage of the plain bars. Conversely, the use of plain round bars was found to improve the joint shear strength. At the theoretical flexural strengths of the columns, the nominal horizontal shear stresses were 0.5 $\sqrt{f_c^{'}}$ for Unit 1, with plain bars [36], and 0.61 $\sqrt{f_c^{'}}$ Unit O1, with deformed bars [37]. As a consequence, Unit 1 [36] evinced less diagonal cracking and shear distortion in the joint core than Unit O1.

Test results on Unit 2 revealed that column axial compression enhanced the transmission of beam bar forces to the joint core by bond, and led to extensive diagonal shear cracking. As a consequence, the joint core deformation had a bigger contribution to the total story drift, which was greater than that of Unit 1. Furthermore, the damage of Unit 2 spread to the regions near the joint, with wide flexural cracks in the beams. No diagonal tension crack occurred in the members adjacent to the joint for both Unit 1 and Unit 2, since the shear reinforcement in beams and columns was sufficient to provide adequate shear strength. On the other hand, the compressive axial load in the column of Unit 2, when combined with severe bond degradation, led to severe column bar buckling and extensive concrete spalling within the joint core and in the adjacent regions, due to the lack of joint transverse reinforcement. In the end, the presence of column axial compression on Unit 2 developed different cracking patterns and damages and enhanced column bar buckling, which caused the final failure of the sub-assemblage [36].

Pampanin et al. [4].

Pampanin et al. investigated the seismic vulnerability of RC beam-column joints of the typical Italian structures built from the 1950's through the 1970's, having plain bars as longitudinal reinforcement and no joint transverse reinforcement. The sub-assemblages were not detailed to have ductile behavior. The study on interior joints considered two different beam bar configurations, one with continuous bars passing through the joint core (Fig. 4(a)) and the other with lap-slices and end hooks for the beam bars just outside the joint region (Fig. 4 (b)).

In order to simulate the actual forces developed in a frame system during a seismic event, diversely from the other tests in the literature, in [4] the axial load applied on the upper column was varied as a function of the vertical load applied to the beams ends. At the local level, the brittle failure of the structural elements was expected. Particularly, for the considered joints, shear cracking was expected to occur in the joint panel before the column hinging.

Instead, test results revealed that the interior joints developed a relevant resource of plastic deformation, even if they were designed without specific details for developing a ductile behavior.

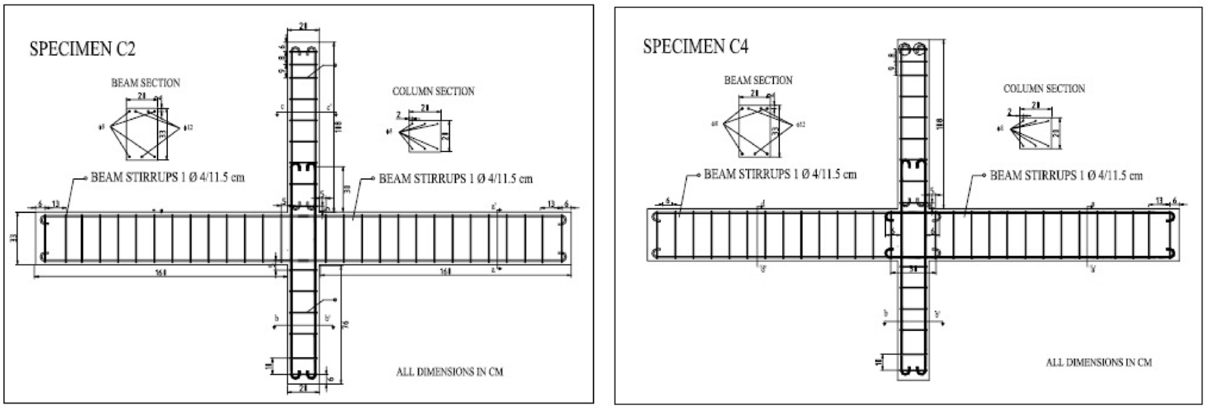
Actually, at early stages, flexural cracks occurred in the column and represented a sort of structural fuse for the joint core, which evinced no damage apart from the slippage of the column bars. From the comparison of the different anchorage solutions, it appeared that, at the local level, the higher deformability due to bar slippage did not result in decreased flexural strength. Anyway, the higher flexibility due to the ductile resource of interior joints, combined with the slippage of the column bars, led to flexural failure at the joint-column interface, which resulted, at the global level, in an undesired soft-story mechanism.

Braga et al. [38].

Braga et al. also investigated the failure mechanisms and their interactions for interior joints designed for gravity loads. They observed that the small section size of the columns and the inadequate longitudinal reinforcement in existing structures were the main causes of failure. In particular, these authors performed three experimental tests on interior joints reinforced with plain bars: two specimens subjected to column axial load, C11-1 and C23-1, built in full scale and 2:3 scale, respectively, and specimen C23-2, built in 2:3 scale subjected to eccentric column load, to study the P-Δ effect. Failures were governed by the bond-slip of the columns longitudinal bars, with lumped yielding of the columns near to the joint-column interface (Fig. 5) and a consequent soft-story failure mechanism.

However, by comparing the behavior of columns with plain bars, subjected to flexure and axial load, with the known behavior of joints reinforced with deformed bars, they observed that the specimens with plain bars show a reduced degradation of the cyclic response.

Afterwards, Braga et al. carried out an investigation of the influence of bond loss of bars passing through the joint panel [39]. They also performed numerical analyses, considering different strength in compression of longitudinal bars, using a simplified model, which provides a stress–strain relationship that accounts for bond slippage [40]. From the comparison, it appeared that the bond-slip of the beam and column longitudinal bars reduces the flexural strength of the sections,



(a) (b)

Fig. 4. Different beam bars configurations for interior joint sub-assemblages [4]: (a) continuous bars passing through the joint; (b) lap-slices with end hook anchorages outside the joint region.

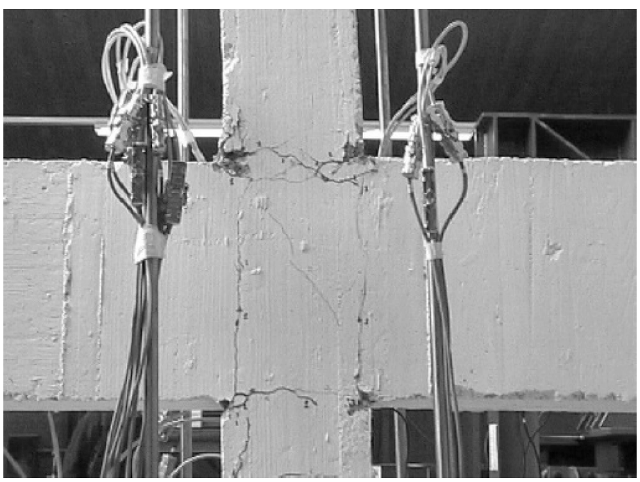


Fig. 5. Final damage pattern of specimen C23-1 [38].

especially when the axial load is high. This phenomenon could promote flexural yielding of columns rather than beams, and modify the local failure mechanism of the structure.

Fernandes et al. [41].

Fernandes et al. carried out a comparative study on six full-scale RC interior joints (Fig. 6), representative of structures built in the mid-1970′s, to assess the influence of bond properties, column axial load and amount of reinforcement on joint behavior.

The investigation on the influence of bond properties focused on two specimens without horizontal hoops in the joint: one reinforced with plain bars, specimen JPA-1, and the other reinforced with deformed bars, specimen JD. Both specimens had normalized column axial load equal to 9.4%. In agreement with the other previously cited research works, the authors observed that the total energy dissipated by the specimen with deformed bars was higher than that of the specimen with plain bars. Moreover, the specimens exhibited different damage modes, and their final cracking patterns well illustrated the influence of bond properties on the cyclic behavior of the joints. In particular, specimen JPA-1 with plain bars showed flexural cracks concentrated at beam-joint and column-joint interfaces, and cracking in the joint core was negligible. Diversely, the joint with deformed JD bars exhibited spread damage, with cracks along the beam and column spans and cracking with concrete cover spalling in the joint core.

By examining the effects of the column axial load, the authors compared test results on specimen JPA-1, with normalized axial load of 9.4%, to that of specimen JPA-3, identical but loaded with column normalized axial load of 21.3%. The comparison revealed that the increase in the compression on the column enhanced the lateral strength of the joint and led to larger strength degradation at maximum drift (4%), and larger energy dissipation. The damage was significant in the joint core, so that specimen JPA-3 exhibited diagonal cracking with concrete cover spalling and displayed larger energy dissipation and reached the conventional failure condition [41].

The influence of the steel reinforcement amount was also studied by Fernandes et al., under the normalized axial load of 21.3%. One unit, specimen JPB, was realized with a large amount of column longitudinal reinforcement; another unit, specimen JPC, with large amounts of column longitudinal bars and transverse reinforcement of both beam and column. The two sub-assemblages showed results very similar to each other, hence it appears that the large amount of transverse reinforcement did not make a significant contribution to specimen JPC's strength. The increase in the column longitudinal reinforcement led to minor damages in the columns, with flexural cracks concentrated at beam-joint interfaces and no damage in the joint region. Comparing the results of specimens JPB and JPC to that of specimen JPA-3, it is evidenced that increasing the amount of steel reinforcement results in marked decrease in energy dissipation.

Melo et al. [42].

Melo et al. studied the cyclic response of interior beam-column joints reinforced with plain bars, by performing tests on six full-scale test units (Fig. 7), representative of RC structures built before the 1970's, without joint shear reinforcement, in the presence of applied column axial load equal to 450 kN. For comparison, an additional unit, specimen ID, with deformed reinforcing bars was built to investigate the bond influence on seismic response of beam-column joints. Moreover, the six specimens reinforced with plain bars presented various geometric and mechanical properties, in order to investigate how the different reinforcement detailing of beams and columns, the presence of floor slabs and the concrete compressive strength might influence the global behavior and the failure mechanism of the sub-assemblages.

The experimental results evidenced that in units reinforced with plain bars, the maximum strength of the joint increased with the concrete grade, while specimen ID, with deformed bars, developed the maximum strength, due to the greater steel grade of the reinforcement. All the tested joints developed shear failure mechanism, with diagonal cracks in the joint core followed by concrete spalling, except for specimen IPD, with lap-slices both in the beams and in the upper column, as shown in Fig. 7.

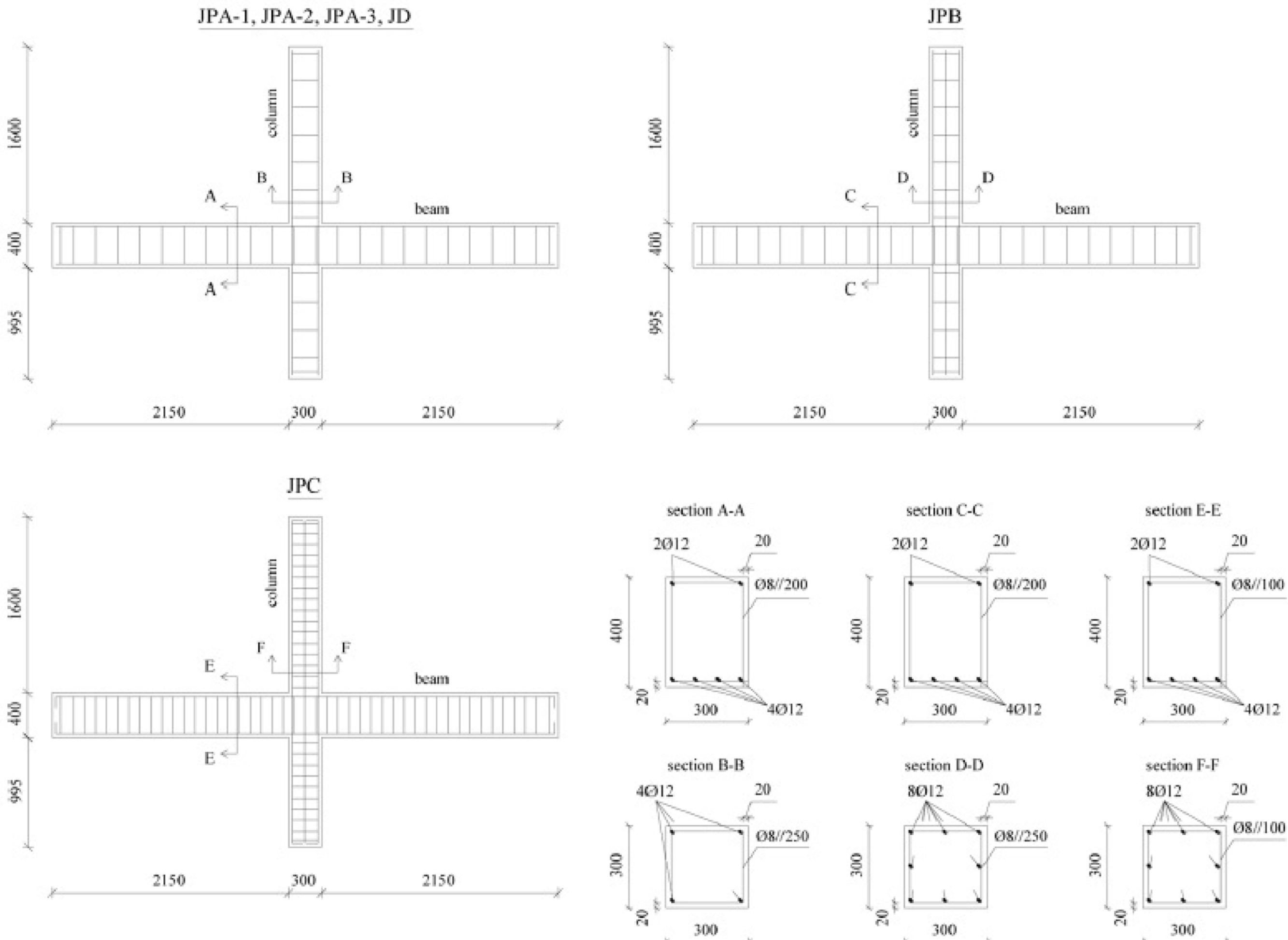


Fig. 6. Geometrical and reinforcing details of the specimens in [41] (dimensions are in mm).

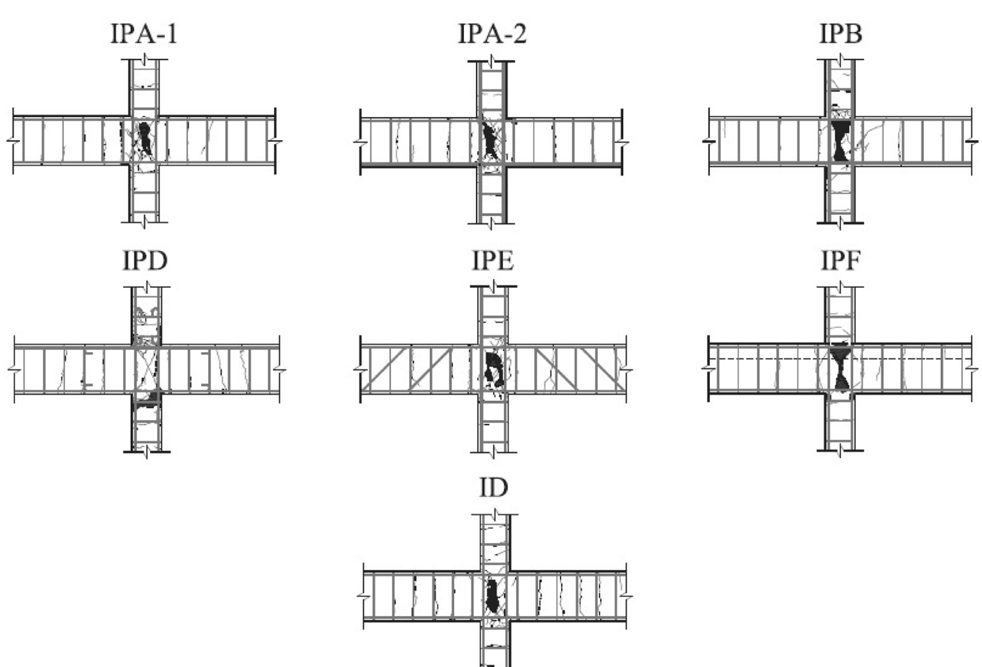


Fig. 7. Final damage patterns of the specimens in [42].

Specimen IPD presented larger damage at the inferior column-joint interface, with flexural cracks, concrete spalling and bars buckling. In fact, the overlapping of longitudinal beam bars in the joint region enhanced the shear strength of the joint and the flexural capacity of the beam-joint interface sections, thus affecting the failure mechanism of the unit, which did not exhibit joint shear failure.

As regards the influence of bond properties, the specimens with plain bars developed concrete damage mainly in the joint core. The authors observed that shear failure occurred due to the lack of joint stirrups and the weak concrete confinement, and that failure was intensified by the slippage of bars through the joint. The cracking pattern for specimen ID, with deformed reinforcing bars, was more distributed along the adjacent beams and columns.

Adibi et al. [43].

Adibi et al. studied the experimental behavior under cyclic load of one interior joint, specimen SC2, reinforced with plain bars, and focused the attention on longitudinal bar slippage as the main failure mode. Moreover, they introduced a systematic procedure to predict the dominant failure mode of the joint, based on the dimensional properties, reinforcement details, and axial and shear load of the joint. From the experimental results, the interior joint sub-assemblage SC2 with 7% constant axial load ratio, developed no diagonal cracks in the joint panel zone. At very small drift ratio (0.2%), vertical cracks occurred both at the left beam-joint interface and at a distance of 11 cm from the right beam-joint interface. Afterwards, at 1.35% drift ratio, several flexural cracks appeared on the beam. At the end of the test, corresponding to 2.7% drift ratio, some minor flexural cracks opened at the upper column-joint interface, but no serious damage occurred, as shown in Fig. 8, though the beams were stronger than the columns. The final damage pattern led the authors to state that the interior joint behavior was controlled by rocking, with beam bar slippage. Hence, the authors observed that specimen SC2 developed 81% of its full nominal flexural capacity, with a gradual strength deterioration and an increasing displacement in the cyclic response due to the presence of plain reinforcing bars.

4. Interior joints shear stress at failure

To interpret the behavior of the collected specimens at varying of the column axial load, the shear stress τ acting in the joint core at failure is derived from the following equation

$$\tau = \frac{V_{jh,test}}{b_j h_c} \tag{14}$$

Fig. 8. Final cracking pattern of specimen SC2 at 2,7% drift ratio [43].

where $V_{jh,test}$ is the experimental shear strength value and b_j is the effective joint width calculated as follows

$$b_j = \begin{cases} \min(b_c, b_b + 0.5h_c) for b_b < b_c \\ min(b_b, b_c + 0.5h_c) for b_b \geq b_c \end{cases} \tag{15}$$

with b_b the beam width and b_c the column width.

Table 1 reports the geometrical and mechanical properties of collected interior joints, while Table 2 reports forces and stresses acting in the joints and their failure modes. In Table 2 the labels used for failure typologies have the following meaning: B = failure with wide cracks at beam-joint or column-joint interfaces, in the presence of bond degradation, CFF = column flexural failure, JSF = joint shear failure.

By considering the failure modes, from Table 2 it is observed that specimens exhibiting bond degradation develop lower strength, as expected. For these test units, the increase in joint shear strength with the increase of axial load ratio is minimum. Whereas, specimens that exhibit joint shear failure and column flexural failure reach generally higher strengths. In these cases, an increase in the axial load ratio results in an apparent increase in the joint shear stress. Hence, it can be said that the axial load is favorable to the joint strength when adequate anchorage of the longitudinal bars is provided.

In Fig. 9 the joint stress ratio $\tau/\sqrt{f_c'}$ (Table 2, column (2)) is plotted versus the axial load ratio $N/f_c' A_g$ (Table 2, column (3)). The diagram also shows the joint failure modes, through the use of different symbols, and reports a linear data interpolation.

On the basis of Fig. 9, it can be observed that, generally, an increase in the axial load ratio involves an increase in the joint shear stress ratio at failure.

In Fig. 10 the ratios d_b/h_c relevant to the collected specimens (Table 1, column (12)) are plotted. For comparison, also the corresponding limit values provided by the main building codes according to NZS, ACI and AIJ (Eqs. (6) or (7), (10) and (11), respectively) are reported. From this figure it can be observed that all the joints have ratios that do not satisfy NZS and AIJ limits, while ACI recommendations are satisfied only by specimens considered in [36] and [4]. Since they represent joints of existing buildings, it is clear that generally these buildings were not designed according to modern building codes prescriptions for seismic actions. As regards the codes limits, it can be observed that the more conservative ones are those of NZS.

5. Shear strength formula

5.1. Critical issues

On the basis of the previous collection of test results on interior beam-column joints reinforced with plain bars, the following observations can be made.

1) First of all, joints with plain bars subjected to cyclic loads generally do not exhibit shear failure, as do joints reinforced with deformed bars, but rather bond degradation and slippage of the longitudinal bars of beams and/or columns [36,38,41,43]. Interior joints reinforced with plain bars exhibit conventional joint shear failure only when quite high axial load, in the range of 16%–27% of the concrete section capacity, acts on the columns [41,42]. This because high column axial load improves bond strength between concrete and beam longitudinal bars in the joint core. Actually, better bond conditions allow the transfer of greater forces from the beam bars to the joint, leading to the complete development of joint shear cracking. In the presence of this cracking, the joint core deformation has a bigger contribution to the total story drift of the structure [36].

For lower axial load, around 12% of the concrete section capacity, and in the absence of stirrups in the joint core, column bar buckling and concrete spalling within the joint core and in the adjacent regions can

Table 1
Geometrical and mechanical properties of the collected interior beam-column joints.

		(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
Author ref.	Specimen labels	b_b	h_b	b_c	h_c	b_j	A_{sb1}	A_{sb2}	A_{sh}	f'_c	f'_{yb}	f'_{yv}	d_b/h_c
		[mm]	[mm]	[mm]	[mm]	[mm]	[mm^2]	[mm^2]	[mm^2]	[MPa]	[MPa]	[MPa]	
[36]	Unit 1	300	500	460	300	450	1809	904	0	43.8	321	321	0.08
	Unit 2	300	500	460	300	450	1809	904	0	48.9	321	321	0.08
[4]	C2	200	330	200	200	200	327	214	0	23.9	346	346	0.06
[41]	JPA-1	300	400	300	300	300	226	452	0	27.8	590	590	0.04
	JPA-2	300	400	300	300	300	226	452	0	27.8	590	590	0.04
	JPA-3	300	400	300	300	300	226	452	0	27.8	590	590	0.04
	JPB	300	400	300	300	300	226	452	0	27.8	590	590	0.04
	JPC	300	400	300	300	300	226	452	0	27.8	590	590	0.04
[42]	IPA-1	300	500	300	300	300	452	452	0	21.5	405	405	0.04
	IPA-2	300	500	300	300	300	452	452	0	30.9	405	405	0.04
	IPB	300	500	300	300	300	452	452	0	24.5	405	405	0.04
	IPD	300	500	300	300	300	452	452	0	18.5	405	405	0.04
	IPE	300	500	300	300	300	452	226	0	21.2	405	405	0.04

occur before the complete development of shear cracking [36].

2) Bond degradation occurs in the joint along beam or column bars due to inadequate development length of the bars within the joint core, which is attested by low values of the ratios h_c/d_b or h_b/d_c, respectively [25] and [36]. This is because large diameters and small depths cause higher bond stress concentrations. Bond deterioration manifests with wide cracks at the joint-beam or joint-column interface, increasing fix-end rotation at these sections and producing lower structural stiffness and strength [36,43] and energy dissipation [41]. For an axial load around 7% of the section capacity, joint strength reduction due to bond-slip can be around 20% of that corresponding to the development of beam flexural capacity [43].

If bond deterioration affects the column bars, it promotes the development of plastic hinges in the columns rather than in the beams, and leads to unexpected soft-story failure in the structure [4,38]. Furthermore, this failure prediction is made inaccurate by the use of plane section assumption for sections reinforced with plain bars, which leads to overestimate the column theoretical flexural strength at the plastic hinge region [36]. Increasing in the column longitudinal reinforcement leads to less damage in the columns, but also to a decrease in energy dissipation [41].

If bond degradation affects the beam bars, flexural cracks concentrated at the beam-column interface appear, while cracking of the joint core is negligible [41,43]. In the presence of low axial loads on the columns, on the order of 7–9% of the section capacity, flexural cracks may contemporarily occur at the column-joint interface [41,43].

3) The presence of overlapping of longitudinal beam or column bars in the joint region enhances the shear strength of the joint [42].

5.2. *Proposed shear strength formula*

On this background, in order to propose a formula for the prediction of shear strength of interior beam-column joints with plain bars, the formula of Pauletta et al. [10] for interior joints with deformed bars (Eq. (16)) is considered and opportunely modified.

$$V_n = 5.28\left(\frac{A_{sb1}}{\Phi_{b1}} + \frac{A_{sb2}}{\Phi_{b2}}\right)l_h + 0.80\chi f'_c a_c b_j \cos\theta_h + 0.14A_{sh}f_{yh} + 0.22\frac{A_{sv}f_{yv}}{\tan\theta_h} \tag{16}$$

In Eq. (16) joint shear strength is obtained by adding the contributions of three inclined concrete struts (first three terms of the equation) to the contributions of the truss mechanism due to joint horizontal reinforcement and column intermediate bars (fourth and fifth terms, respectively). Regarding the three strut contributions, the third term of the equation represents the contribution due to the main concrete strut, which connects the beam and column compression regions. The first and second terms of the equation represent the contributions of the two side struts, which arise thanks to the stresses transferred, through bond, by the beam bars to the joint regions outside the main concrete strut.

By considering the beam-column joints reinforced with plain bars, it is expected that the bond forces transmitted from the bars to the concrete outside the compression region of the column are very low. Consequently, the shear strength contributions due to the two side struts and the truss mechanism can be considered negligible, since these contributions arise due to the bond transferred by the bars. Hence, it is reasonable to consider only the main concrete strut contribution to joint shear strength (Fig. 11). Thus, the shear strength expression (Eq. (16)) reduces to

$$V_n = 0.80\chi f'_c a_c b_j \cos\theta_h \tag{17}$$

where χ is equal to

$$\chi = 0.74 \bullet \left(\frac{f'_c}{105}\right)^3 - 1.28 \bullet \left(\frac{f'_c}{105}\right)^2 + 0.22 \bullet \left(\frac{f'_c}{105}\right) + 0.87, \tag{18}$$

b_j is the width of the diagonal strut, which, in this case, is the minimum value between the beam width, and the column width; a_c is the depth of the diagonal strut, whose value is approximated by.

$$a_c = \left(0.25 + 0.85\frac{N}{A_g f'_c}\right)h_c \tag{19}$$

ϑ_h is the angle of inclination of the diagonal strut, defined as follows

$$\theta_h = \tan^{-1}\left(\frac{h_b}{h'_c}\right) \tag{20}$$

with

$$h'_c = h_c\left(1 - 0.85\frac{N}{A_g f'_c}\right) \tag{21}$$

The ratios between the experimental shear strength values and the nominal shear strength calculated through Eq. (17), $V_{jh,test}/V_n$, for specimens collected in section 2.2 and exhibiting joint shear failure, are reported in Table 2, column (6). The average (AVG) of these ratios is equal to 0.97 and the coefficient of variation (COV) is 0.18. On the basis of these values, it can be said that, for the collected specimens, Eq. (9) is accurate (AVG close to 1) and consistent (COV close to 0) in the prediction of shear strength of interior beam-column joints with plain bars.

To make a comparison between the accuracy and the consistency of the proposed formula and those of other existing semi-empirical models, the formula proposed by Wang et al. [18] and that proposed by Kassem [19] are considered. These formulas are applied to joint specimens of

Table 2
Forces and stresses acting in the collected interior beam-column joints and failure modes.

Author ref.	Specimen labels	(1) $V_{jh,test}$ [kN]	(2) $\frac{\tau}{\sqrt{f'_c}}$	(3) $\frac{N}{f'_c A_g}$	(4) Failure mode	(5) V_n [kN]	(6) $\frac{V_{jh,test}}{V_n}$	(7) $V_{n,K}$ [kN]	(8) $\frac{V_{jh,test}}{V_{n,K}}$	(9) $V_{n,W}$ [kN]	(10) $\frac{V_{jh,test}}{V_{n,W}}$	(11) $V_{n,d}$ [kN]	(12) $\frac{V_{jh,test}}{V_{n,d}}$	(13) $V_{n,ACI}$ [kN]	(14) $\frac{V_{jh,test}}{V_{n,ACI}}$	(15) $V_{n,AIJ}$ [kN]	(16) $\frac{V_{jh,test}}{V_{n,AIJ}}$	(17) $V_{n,EC8}$ [kN]	(18) $\frac{V_{jh,test}}{V_{n,EC8}}$	(19) $V_{n,NZS}$ [kN]	(20) $\frac{V_{jh,test}}{V_{n,NZS}}$
[36]	Unit 1	–	0.564	0.000	B	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–
	Unit 2	–	0.501	0.120	B	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–
[4]	C2	119	0.609	0.126	CFF	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–
[41]	JPA-1	–	0.533	0.080	B	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–
	JPA-2	–	0.547	0.080	B	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–
	JPA-3	286	0.602	0.180	JSF	369	0.78	193	1.48	121	2.36	295	0.97	569	0.50	627	0.46	322	0.89	500	0.57
	JPB	–	0.574	0.180	B	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–
	JPC	–	0.565	0.180	B	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–
[42]	IPA-1	313	0.749	0.233	JSF	261	1.20	138	2.26	176	1.77	209	1.50	501	0.62	524	0.60	287	1.09	387	0.81
	IPA-2	305	0.609	0.162	JSF	334	0.91	198	1.54	221	1.38	267	1.14	600	0.51	676	0.45	337	0.90	556	0.55
	IPB	312	0.700	0.204	JSF	286	1.09	157	1.98	191	1.63	229	1.36	535	0.58	574	0.54	304	1.03	441	0.71
	IPD	312	0.805	0.270	CFF	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–
	IPE	223	0.539	0.236	JSF	258	0.86	136	1.64	175	1.28	206	1.08	497	0.43	519	0.43	285	0.78	382	0.58
						AVG	0.968		1.780		1.683		1.210		0.533		0.495		0.938		0.644
						COV	0.178		0.187		0.253		0.178		0.131		0.144		0.129		0.172

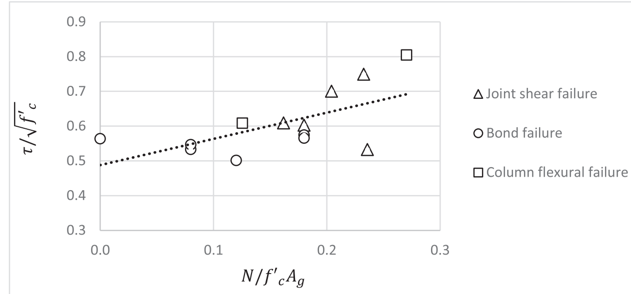


Fig. 9. $\tau/\sqrt{f'_c}$ versus $N/f'_c A_g$ ratios for collected interior joints with plain bars.

Table 2, that exhibited joint shear failure, considering only the concrete strut mechanism. The corresponding shear strength predictions are labeled $V_{n,W}$ and $V_{n,K}$, respectively, and are reported in Table 2. Also the ratios $V_{jh,test}/V_{n,W}$ and $V_{jh,test}/V_{n,K}$ are presented. From Table 2 it can be seen that the proposed formula is more accurate and consistent than those of Kassem and Wang et al., since it provides the lowest AVG and COV. The ratios between $V_{jh,test}$ and the nominal shear strengths obtained from the proposed formula and the other existing models considered mentioned above are plotted in Fig. 12, which clearly shows that the proposed formula is the most accurate and reliable, since it provides shear strength estimations very close to those obtained from the experimental tests.

6. Design formula

The main current building codes do not propose design formulations to predict the shear strength of beam-column joints reinforced with plain bars. Since a not negligible part of the existing RC buildings are built with plain bars, the assessment of beam-column joints shear strength is fundamental in order to evaluate the capacity of these buildings to withstand the seismic actions prescribed by Codes. Eq. (17) cannot be employed for design purposes, since it presents AVG = 1.00. In order to employ Eq. (17) for design purposes, it has to be multiplied by a safety factor. The introduction of a safety factor does not modify the COV value of Eq. (17).

The design shear strength formula proposed herein is derived from that proposed by Pauletta et al. [10] for interior joints with deformed bars by considering only the main concrete strut contribution to joint shear strength. Consequently, the proposed design formula is the following

$$V_n = 0.64\chi f'_c a_c b_j \cos\theta_h \tag{22}$$

The ratios between the experimental shear strength values and the nominal shear strength calculated through Eq. (22), $V_{jh,test}/V_{n,d}$, for specimens used as dataset for assessing Eq. (17) reliability, are reported in Table 2, column (8). The AVG of these ratios is equal to 1.209.

From Table 2 it can be seen that the predicted design shear strengths of specimens are on the safe side for almost all specimens, except one, which presents a $V_{jh,test}/V_{n,d}$ ratio equal to 0.97. Since this ratio is very close to one, it can be concluded that the proposed design formula provides safe estimation of joints shear strength.

To compare the accuracy and the consistency of the proposed design formula with those provided by the main building codes according to ACI 318–14 [15], AIJ 2010 [16], EC8 [14] and NZS 3101–1:2006 [44], these formulas are applied to joint specimens of Table 2 exhibiting joint shear failure. The corresponding shear strength predictions and the ratios between $V_{jh,test}$ and these predictions are reported in Table 2, in columns from (13) to (20). These ratios are also plotted in Fig. 13. This figure clearly shows that the proposed design formula provides the safest predictions, while ACI 318–14 [15], AIJ 2010 [16] and NZS 3101–1:2006 [44] provide shear strength estimations that are much

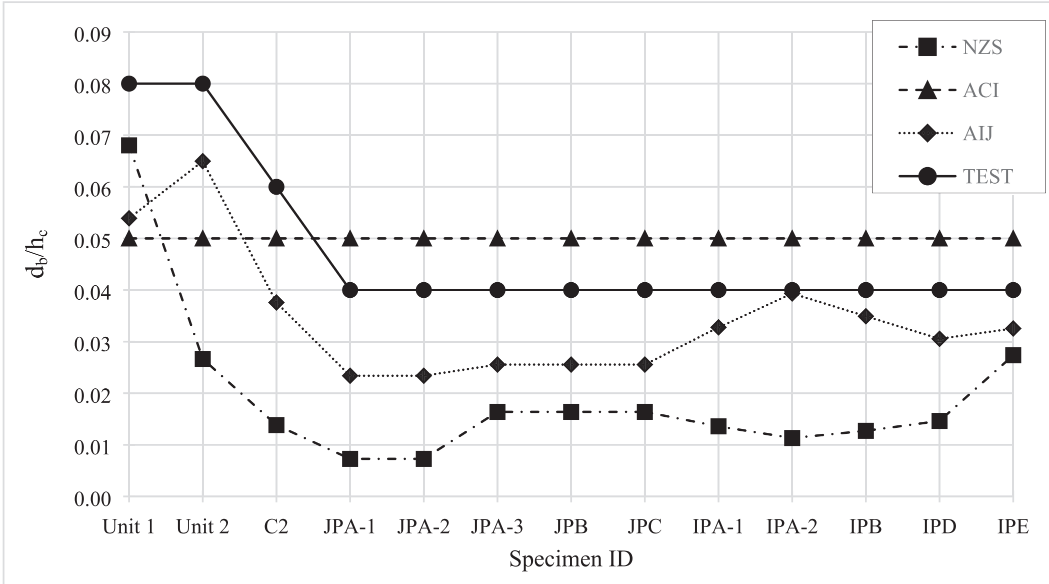


Fig. 10. d_b/h_c ratios calculated for collected interior joints (TEST) and compared to the limit values provided by the main building codes (NZS, ACI and AIJ).

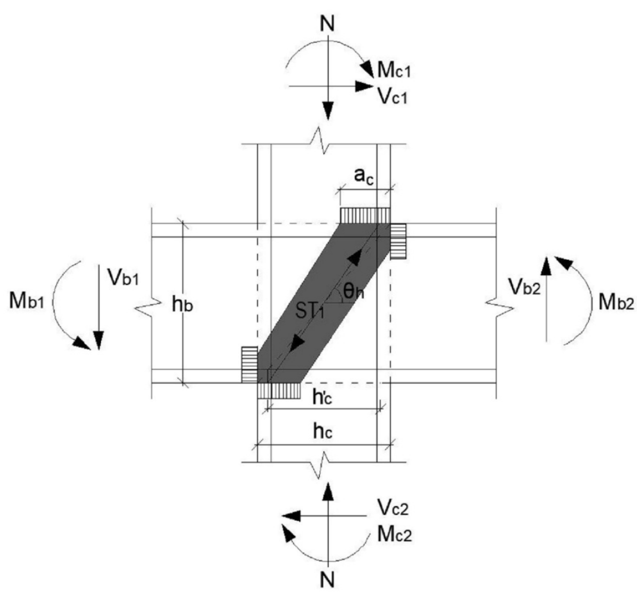


Fig. 11. Concrete strut resisting to shear forces acting in the beam-column joint.

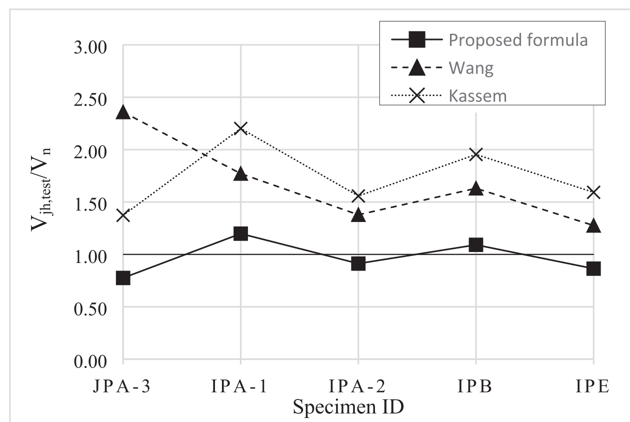


Fig. 12. Comparison between the results obtained from the proposed formula and those of existing models of Wang et al. [18] and Kassem [19].

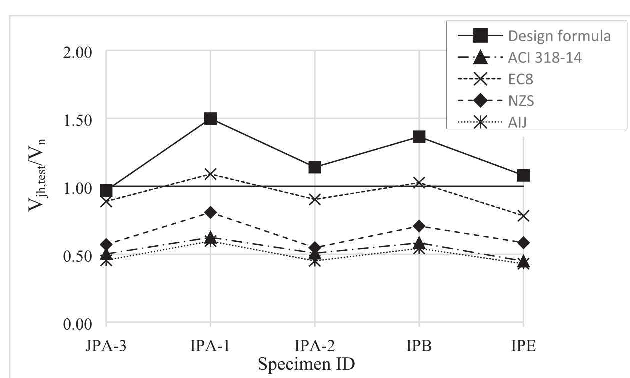


Fig. 13. Comparison between the results obtained from the proposed design formula and those the codes.

greater than the experimental ones. Among the considered building codes, the one that provides the best predictions is Eurocode 8, which are very close to the real ones. However, the AVG of these ratios is 0.938, which indicates that the code formula is not safe enough.

7. Conclusions

On the basis of the informations reported and commented previously, it is remarked that, to predict the possible behavior of beam-column joints reinforced with plain bars under seismic load, the main influencing parameters have to be known as accurately as possible. These are the mechanical properties of materials, the geometry of the joint and the converging elements, and the column axial load.

The principal conclusions on the behavior of interior joints with plain bars subjected to cyclic loads, which can be drawn from the experimental findings considered in this paper, are pointed out in the following.

1. Interior joints with plain bars generally do not exhibit shear failure, but rather bond degradation and slippage of the longitudinal bars of beams and/or columns.
2. Specimens which exhibit bond degradation develop lower shear stress ratios than specimens that exhibit other types of failure.
3. Bond degradation manifests with wide cracks at the joint-beam or joint-column interface increasing fix-end rotation at these sections and producing lower stiffness, strength and energy dissipation of the frame structure.
4. Bond degradation along the beams bars is all the more probable, the larger the diameter of the bars, or the smaller the depth of the column.
5. If bond degradation affects the beam bars, cracking of the joint core is negligible, since shear action transferred to the joint is small.
6. If bond degradation affects the column bars, it promotes the development of plastic hinges in the columns, leading to soft-story failure.
7. In the presence of bond degradation, the use of the plane section theory assumption leads to overestimation of the theoretical flexural strength of beams or columns at the plastic hinge sections.
8. In general, an increase in the axial load ratio involves an increase in the joint shear stress ratio at failure.
9. Joint shear failure occurs only when high axial load (above 16 % of the section capacity) acts on the columns, because it allows the transfer of higher bond stresses from the beam bars to the joint core in the column compression region.
10. For lower axial load and in the absence of stirrups in the joint core, column bar buckling and concrete spalling within the joint core and adjacent regions can occur before complete development of shear cracking.
11. Specimens which exhibit joint shear failure develop higher shear stress ratio than specimens that exhibit other types of failure.

The observations above are helpful for understanding the behavior of joints with plain bars under seismic actions and can form the basis for upgrading these joints when a retrofit intervention on an existing building is planned.

With regard to the formula proposed for shear strenght prediction of interior beam-column joints with plain bars, it can be said:

1. The shear strength contributions linked to bond transfer in the joint regions outside the compression region of the column can be neglected, since the bond strength is very low.
2. It is reasonable to consider only the main concrete strut resisting mechanism for the evaluation of joint shear strength.
3. The proposed formula taking into account only this contribution proves to be both accurate and consistent in the prediction of the collected joints which failed due to shear action.
4. From the predicting shear strength formula a design formula to employ for the assessment of beam-column joints strength is proposed, in order to evaluate the capacity under seismic actions of existing RC building built with plain bars.

CRediT authorship contribution statement

C Di Marco: Conceptualization, Methodology, Software, Investigation, Writing – original draft, Visualization. **G Frappa:** Software, Investigation, Resources, Data curation, Writing – review & editing, Project administration, Funding acquisition. **M. F Sabbà:** Validation, Data curation. **G Campione:** Formal analysis, Supervision. **M Pauletta:** Conceptualization, Methodology, Resources, Writing – original draft, Writing – review & editing, Supervision, Project administration, Funding acquisition.

Declaration of Competing Interest

The authors declare the following financial interests/personal relationships which may be considered as potential competing interests: [Margherita Pauletta reports financial support was provided by Italian Department of Civil Protection and by the University of Udine. Giada Frappa reports financial support was provided by European Union NextGenerationEU.].

Data availability

No data was used for the research described in the article.

Acknowledgements

The research has been partially funded by:

- the Italian Department of Civil Protection, within the framework of Executive Project DPCReLUIS 2022–2024,

- the European Union Next-GenerationEU (PIANO NAZIONALE DI RIPRESA E RESILIENZA (PNRR) – MISSIONE 4 COMPONENTE 2, INVESTIMENTO 1.5 – D.D. 1058 23/06/2022, ECS00000043), within the Interconnected Nord-Est Innovation Ecosystem (iNEST). This manuscript reflects only the authors' views and opinions, neither the European Union nor the European Commission can be considered responsible for them.

- the strategic plan of University of Udine within the framework of the project "ESPeRT",

whose support is greatly appreciated.

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