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Seismic Strengthening of RC Exterior Beam-Column Joints Using Local Confinement and 45° Steel Haunches: A Comparative FE Study

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ABSTRACT

This study investigates the influence of local confinement and the introduction of metallic haunches on the mechanical and seismic behaviour of reinforced concrete beam–column joints. A numerical comparative approach is adopted to assess the performance of several strengthening configurations relative to a reference unstrengthen joint. Three confinement configurations are examined notably the confinement of the column in the joint region, confinement of the beam, and simultaneous confinement of both the beam and the column. A fourth configuration incorporates two metallic haunches inclined at 45°, acting as an external strengthening system applied to the unconfined reference configuration. The results indicate that confinement generally enhances the strength, ductility, and energy dissipation capacity of beam-column joints under seismic loading. Among the confined configurations, simultaneous confinement of the beam and the column provides the best overall performance. This is followed by confinement of the column alone, while confinement of the beam alone exhibits comparable load-bearing capacity but slightly inferior performance due to less favourable failure mechanisms. In contrast, the introduction of metallic haunches inclined at 45° leads to a very significant increase in mechanical and seismic performance, clearly exceeding the improvements achieved through conventional confinement. These findings highlight the critical role of external strengthening devices in enhancing the seismic behaviour of beam–column joints and emphasize the strong potential of metallic haunches for the seismic design and retrofitting of reinforced concrete structures.

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1. Introduction

Many existing reinforced concrete (RC) buildings were erected before the implementation of current seismic design standards. Consequently, the beam–column joints in these older frames often lack sufficient transverse reinforcement, confinement, or anchorage. When subjected to intense earthquake ground motions, these defects can trigger premature joint shear failure [1], resulting in rapid loss of strength and potential brittle collapse. This failure can occur even while the adjacent beams and columns remain primarily elastic [2, 3]. Due to these vulnerabilities, beam-column joints are considered a critical link governing the seismic performance of RC frames.

Various techniques have been proposed to repair or strengthen deficient joints, including epoxy injection [4], removal and replacement of damaged concrete, RC or masonry jacketing [5, 6], steel jackets and steel cages, and fibre-reinforced polymer (FRP) systems [7-12]. These methods can significantly improve performance, but each has limitations in terms of constructability, weight, intrusiveness [13, 14], durability or anchorage reliability. In particular, RC and steel jacketing may be heavy and disruptive [15, 16], while FRP systems [17-19] are often governed by debonding and sensitive to workmanship and environmental conditions [20, 22].

Metallic retrofit strategies using external steel elements, such as plates, angles and haunches, offer an attractive alternative [23-35]. They can be prefabricated, bolted or post-installed, and applied with limited disturbance to occupants [36, 37]. Previous experimental and numerical studies have shown that steel haunches can improve hysteretic behaviour [38, 39], modify internal force paths and relocate plastic hinges away from critical regions [40, 41]. For RC beam–column joints, fully fastened haunch systems using post-installed anchors have demonstrated significant gains in load-carrying capacity and energy dissipation, with plastic hinges shifted from the joint panel to the beam [42-45].

A key contribution to the field comes from the work of Genesio *et al*. [46-48], who pioneered a retrofit solution for RC exterior joints. This technique employs a fully fastened steel haunch system installed using post-installed anchors. Because it is minimally invasive, it is well-suited for application to existing buildings. Furthermore, the method is highly effective: it forces plastic hinges to form within the beam and leads to a notable enhancement in both the capacity to carry loads and the capacity for energy dissipation. The findings regarding the Fully Fastened Haunch Retrofit were later confirmed by experiments on 3D joints [49] and studies exploring the use of post-tensioned metallic straps [50].

In existing reinforced concrete buildings, the level of internal confinement provided within beam–column joint panels exhibits significant variability. Many older structures were constructed without modern seismic detailing requirements, resulting in joints with minimal or no transverse reinforcement in the core region. In other cases, only partial confinement is present, typically limited to ties around the column longitudinal bars, leaving the joint core inadequately confined. Properly detailed joint confinement is therefore not consistently achieved in such building stocks.

Modern seismic design codes explicitly prescribe minimum transverse reinforcement requirements for columns and beam-column joints to ensure adequate ductility, prevent buckling of longitudinal bars, and enhance shear resistance under cyclic loading. For example, ACI 318-19 specifies detailed transverse reinforcement provisions for joint regions and columns in seismic-design categories, including maximum spacing limits and confinement reinforcement configurations intended to improve post-cracking behaviour and energy dissipation capacity. Similarly, provisions in Eurocode 8 (EN 1998-1) require closely spaced hoops or ties in column plastic hinge regions and enhanced detailing in joint zones for structures designed for seismic resistance. The additional transverse reinforcement introduced in the column models of this study can be interpreted as a numerical representation of these code-based objectives. These reinforcement layouts emulate improved internal confinement consistent with practices of seismic detailing and retrofitting. Such reinforcement also aligns with common strengthening techniques, such as column jacketing or supplemental transverse reinforcement, that are applied in practice to enhance confinement and deformation capacity.

It is therefore important to assess how varying levels of internal confinement interact with external strengthening solutions, such as steel haunch configurations, particularly because many existing buildings lack adequate code-compliant joint detailing. Understanding the extent to which a practical external haunch configuration can compensate for deficient internal confinement enhances the relevance of the study for real-world assessment and retrofit of existing structures.

In this context, the present work focuses on a clear and practically relevant comparison between:

- an unconfined non-ductile exterior joint;
- confined counterparts with improved transverse reinforcement in the joint panel; and
- the same deficient joint externally strengthened with 45° steel haunches.

The goal is to assess a direct comparison between confinement strategies and haunch retrofit under the same protocol and show a mechanism shift evidence, ultimately identifying the configuration that offers the best seismic performance for this joint typology.

2. Numerical Modelling

2.1. Prototype Joint and Retrofit Configurations

The numerical study is based on an exterior RC beam-column joint tested under quasi-static cyclic loading by Genesio, presented in Fig. (**1**), representing a gravity-load-designed frame without proper joint confinement. Lateral displacement is imposed at the beam tip, while the column is subjected to constant axial load. The geometry, reinforcement layout and loading protocol are taken from the experimental programme.

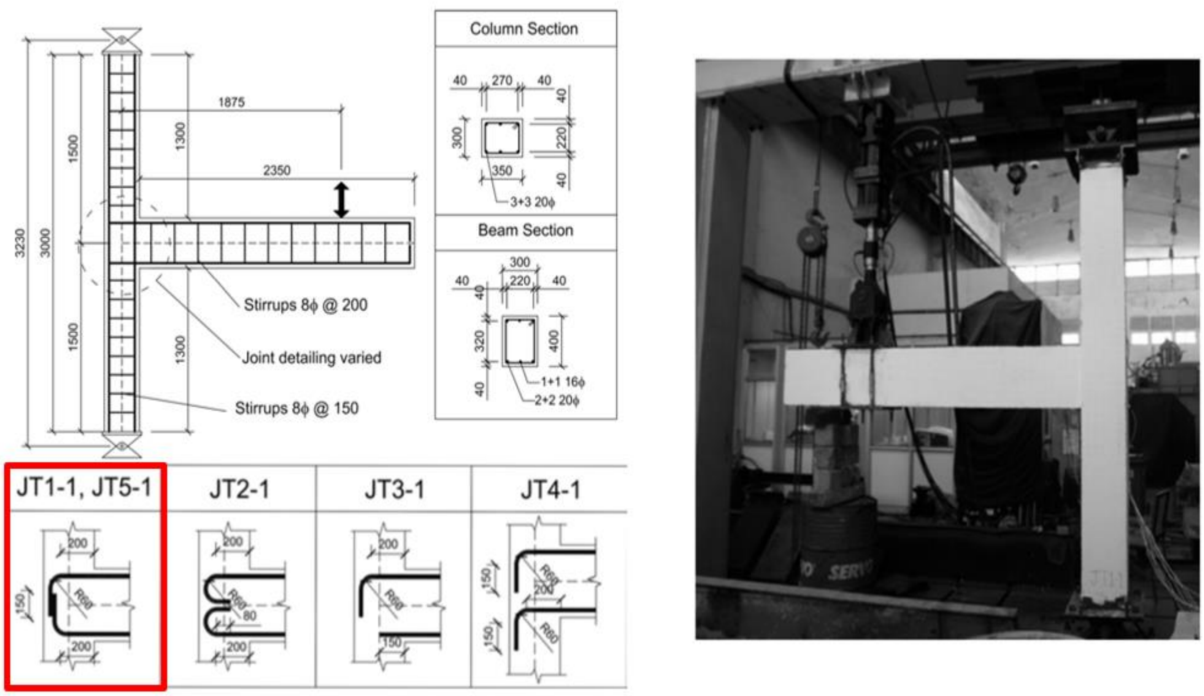


Figure 1: Experimental model [31].

Four main FE models are considered:

- Model A (validation model): numerical replica of the tested non-ductile joint, with the original, poorly confined joint detailing.
- B1 (unconfined joint): joint panel without transverse reinforcement (most deficient configuration), presented in Fig. (**2**).

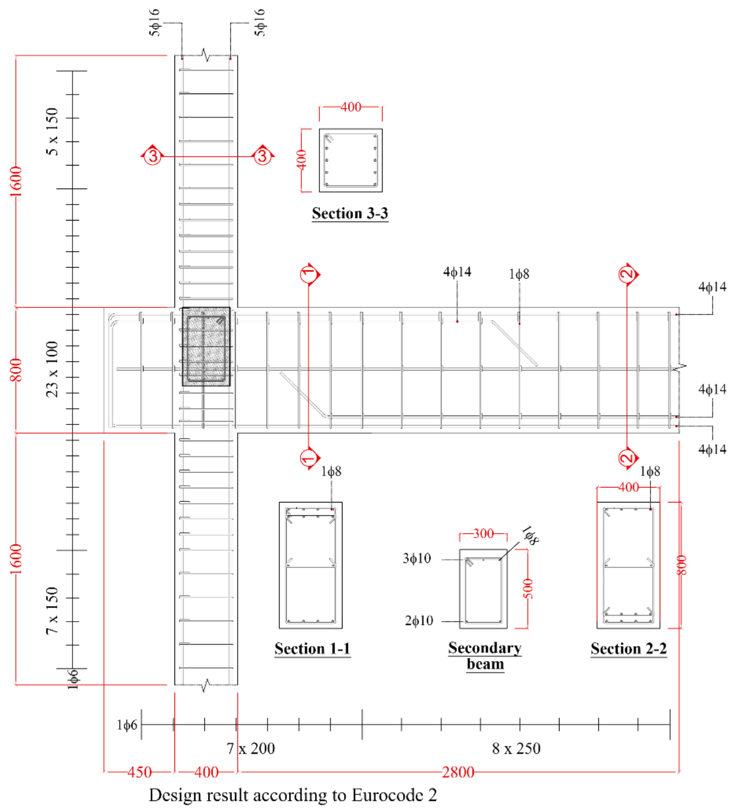


Figure 2: Reinforcement plan of the joint derived from the design at ultimate limit states.

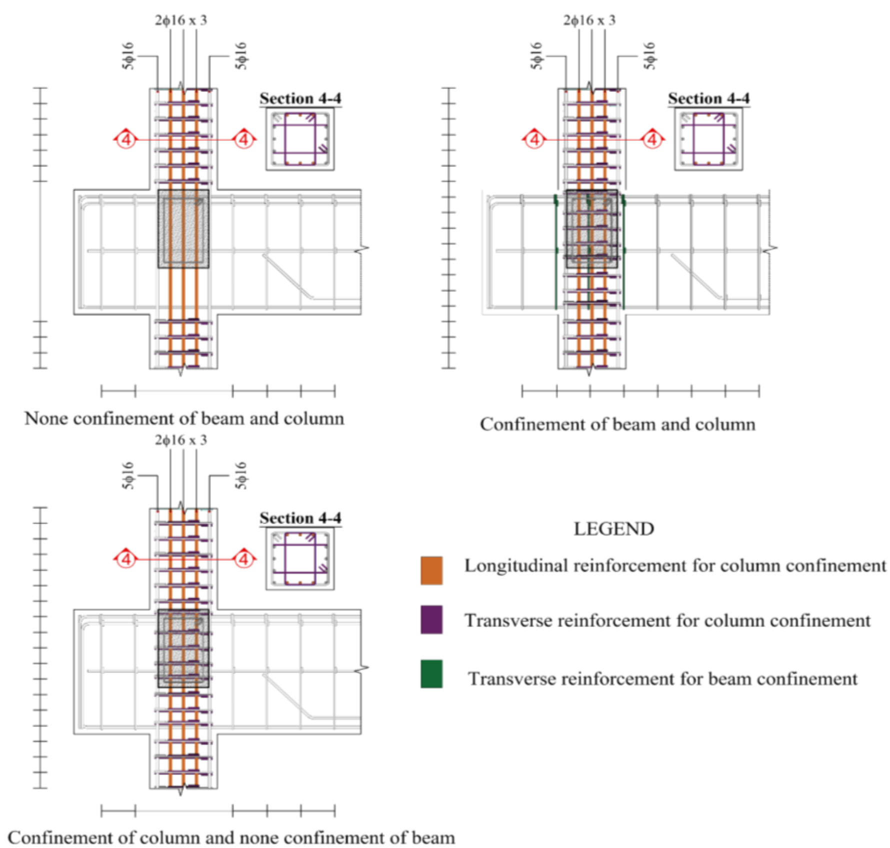


Figure 3: Reinforcement plan of B1 (None confinement), B2 (confinement of beam and column) and B3 (confinement of column and none confinement of beam).

- B2 and B3 (confined joints), presented in Fig. (**3**):
  - B2: transverse reinforcement enclosing both column and beam bars in the joint region (improved confinement);
  - B3: transverse reinforcement provided only around the column bars in the joint panel zone (partial confinement).
- BJS (strengthened joint): the unconfined joint B1 externally strengthened with steel haunches inclined at 45°, symmetric with respect to the beam axis as presented in Fig. (**4**). The haunches are connected to the beam and column through steel plates and post-installed fasteners, following concepts developed for fully fastened haunch retrofits.

This selection allows a direct comparison between internal joint improvement (confined vs unconfined) and external strengthening of the most deficient joint using a single, practically attractive haunch configuration.

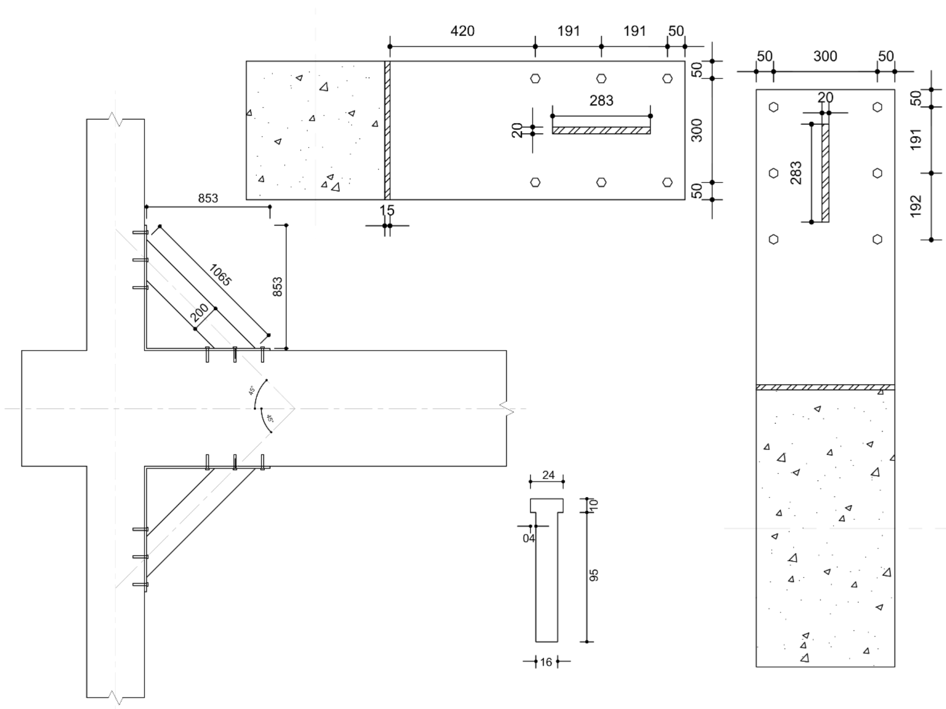


Figure 4: Construction details and the arrangement of the steel haunches.

2.2. Material Models and FE Idealisation

Concrete is modelled with the Concrete Damage Plasticity (CDP) model implemented in ABAQUS [51]. Class C30/37 concrete is assumed, with elastic properties and compressive/tensile strengths taken from the experimental source. CDP parameters (dilation angle, eccentricity, biaxial-to-uniaxial strength ratio, shape factor and viscosity) are calibrated according to previous work [52] on RC joints so as to capture stiffness degradation due to cracking and crushing. The uniaxial stress–strain relationships in compression and tension (Fig. **5**) are defined in terms of inelastic strain and damage variables, given in Table **1**, to capture stiffness degradation due to cracking and crushing as highlighted by Fig. (**6**).

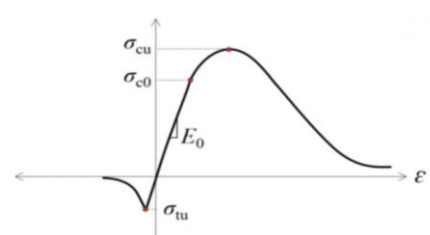
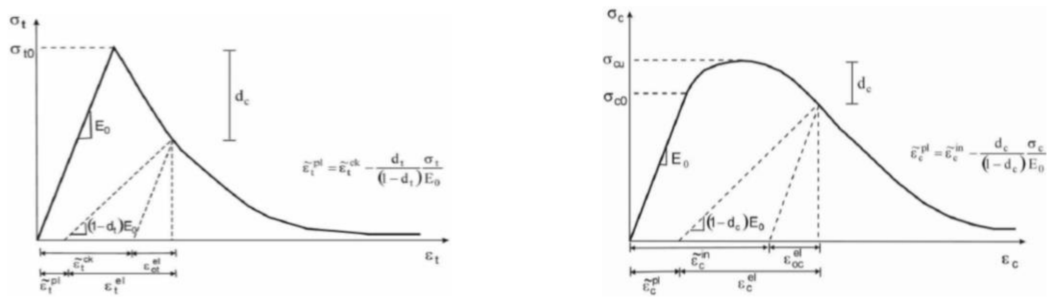


Figure 5: Stress-strain curve of concrete.



a) b)

Figure 6: Decrease in concrete stiffness during unloading: (**a**) in tension; (**b**) in compression [52].

Table 1: Parameters defining the elastic behavior and the plasticity of the concrete [52].

Elastic Phase			Plastic Phase				
ρ	E_c	**Poisson Ratio** ν_c	**Dilation Angle**	**Eccentricity**	**Stress Ratio**	**Shape Factor**	**Viscosity Parameter**
kg/mm^3	MPa	-	°	-	-	-	-
2.4E-06	31000	0.2	40	0.1	1.6	0.667	0

Reinforcing bars and structural steel are modelled as bilinear elastoplastic materials with kinematic hardening. The yield strength, ultimate strength and elastic modulus are taken from the experimental campaign and summarised in Table **2**. The steel haunches, plates and bolts are assigned appropriate elastic–plastic properties to capture yielding and strain hardening.

Table 2: Steel properties (engineering).

Material	**Density**	**Young Modulus**	**Yield Strength**	**Poisson's Ratio**	**Plastic Strain**
	kg/m^3	**GPa**	**MPa**		
Reinforced steel	7.85E-06	200	450	0.3	0.01775
Steel haunches (t=20mm)	7.85E-06	210	275	0.3	0.01869
Plate (t=15mm)	7.85E-06	210	275	0.3	0.01869
Bolt M16	7.85E-06	210	400	0.3	0.01810

Concrete is discretised using eight-node reduced-integration solid elements (C3D8R). The potential influence of bond degradation and slip represents a limitation of the present modelling strategy and suggests that future work could incorporate bond-slip interface elements or nonlinear bond laws to provide a more detailed assessment of joint cyclic behavior. Surface-to-surface contact with hard normal behaviour and frictional tangential response is used between steel plates/haunches and concrete to model bearing and shear transfer. The mesh is refined in the joint region, where high stress gradients and cracking are expected, and coarser elsewhere. A refined 20 mm mesh size was employed in the regions of interest to ensure accurate results. Further mesh refinement led to a significant increase in computational cost and similar results, while previous studies have demonstrated that the adopted mesh provides an effective balance between accuracy and computational efficiency. The meshed geometries of Model A and the B/BJS are presented in Fig. (**7-8**), respectively.

Boundary conditions reproduce the test setup: the column ends are hinged and the beam tip is subjected to a prescribed vertical displacement under displacement control according to the quasi-static cyclic protocol in Fig. (**9**). The same loading history is applied to all B and BJS models.

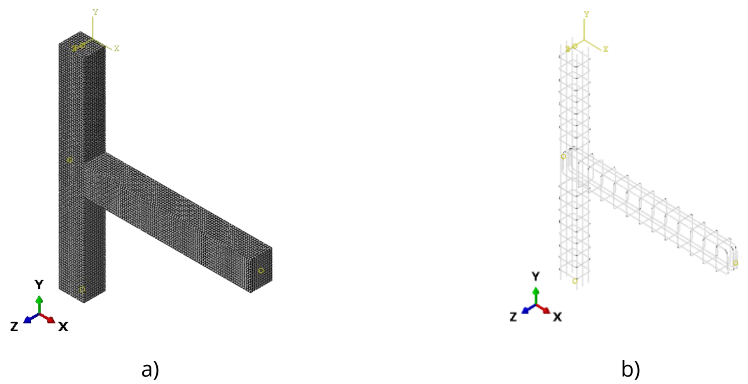


Figure 7: Meshed geometry of FE Model-A: (**a**) Mesh of the concrete matrix; (**b**) Mesh of the reinforcing steel.

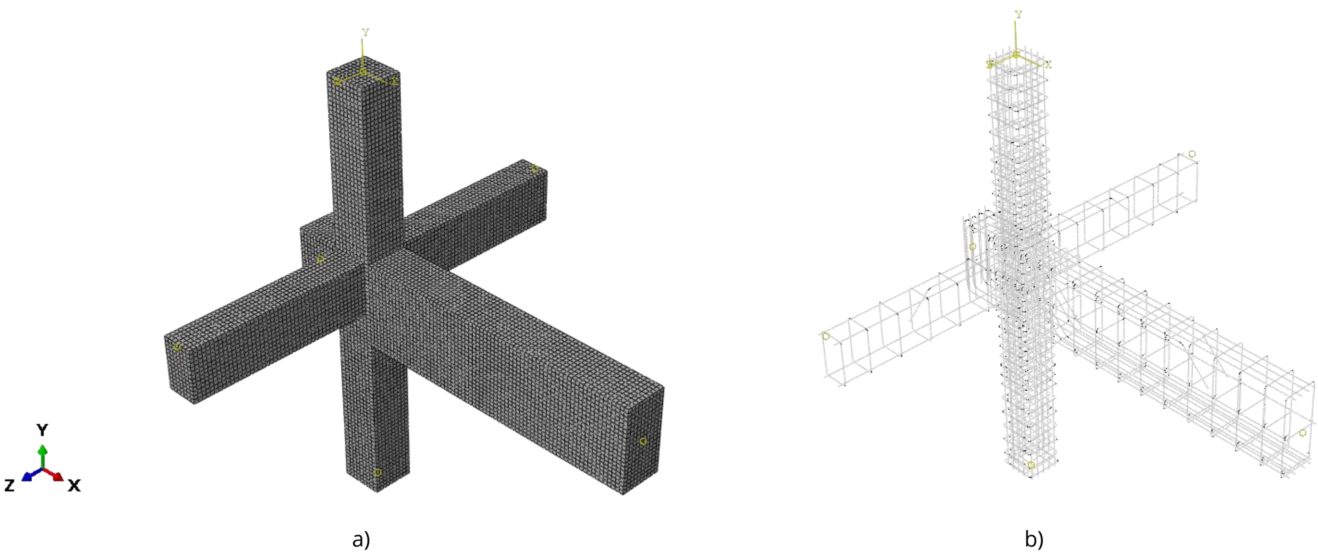


Figure 8: Meshed geometry of FE Model-B: (**a**) Mesh of the concrete matrix; (**b**) Mesh of the reinforcing steel.

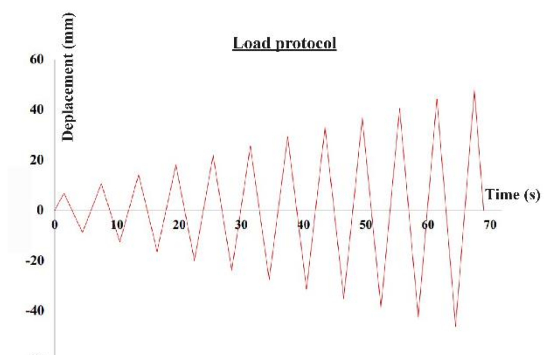


Figure 9: Cyclic loading protocol.

3. Numerical Results

3.1. Validation of the FE Model (Model A)

The validation model (Model A) reproduces the reference non-ductile joint as seen in Fig. (**10**). The FE load–displacement hysteresis curves match the experimental data and an independent numerical study in terms of initial stiffness, peak strength and degradation pattern, with a predicted peak lateral load of about 44 kN compared to 34 kN in the experiment and 47 kN in the reference simulation. These discrepancies are discussed in terms of concrete constitutive modelling, boundary conditions and bond assumptions, but the overall response and failure mode are satisfactorily captured.

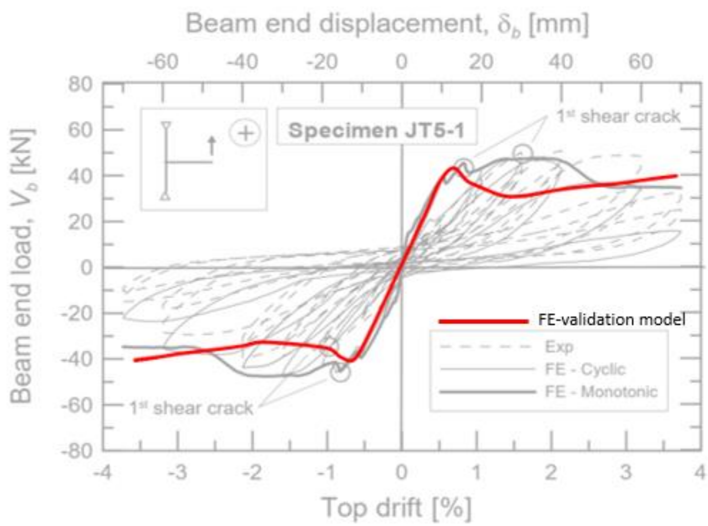


Figure 10: Comparison of experimental and numerical results of force-displacement curves.

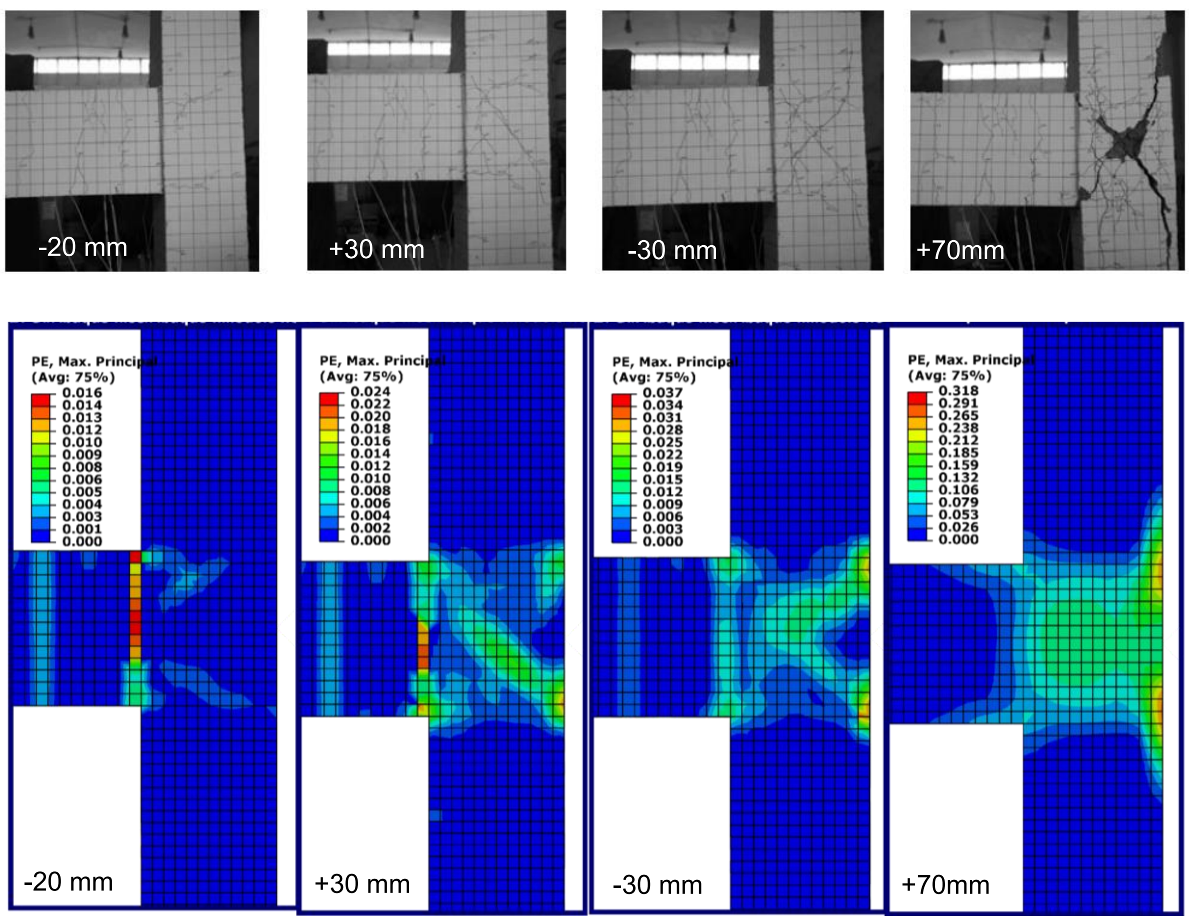


Figure 11: Cracking patterns at increasing beam and end displacement levels: (**a**) Experimental Model [47]; (**b**) FE validation model.

As seen in Fig. (**11**), the numerical damage pattern shows strong diagonal cracking and crushing within the joint panel, while beam and column remain largely elastic, confirming a brittle joint shear failure consistent with the tests. This validates the modelling approach and provides confidence in using the same framework to study the influence of joint confinement and strengthening with haunches.

3.2. Influence of Joint Confinement (B Series)

The B series is used to investigate how internal joint confinement affects the behaviour of unstrengthened joints. The load-displacement curves in Fig. (**12**) show that transverse reinforcement in the joint core has a clear influence on both peak strength and post-peak behaviour.

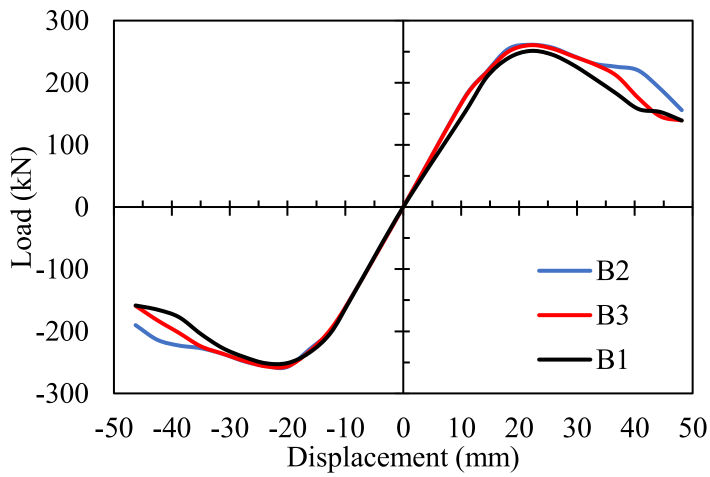


Figure 12: Force-displacement curves of B models.

- Unconfined joint (B1) - The configuration without joint ties exhibits the lowest lateral strength and the most rapid post-peak degradation. Shear cracking develops early and concentrates within the joint panel, leading to severe damage and a brittle loss of stiffness.
- Fully confined joint (B2) - Adding transverse reinforcement encircling both beam and column bars yields the best performance among the unstrengthened configurations. The peak strength increases by several percent relative to B1, and the hysteretic loops become fuller and more stable, indicating improved energy dissipation. Damage spreads more uniformly in the joint core and crushing is delayed.
- Partially confined joint (B3) - When confinement is provided only around the column bars, the joint shows intermediate behaviour: strength and ductility are improved compared to the unconfined case, but remain inferior to B2. Damage tends to localise near the column core and beam-joint interface, confirming that partial confinement cannot fully prevent stress concentration in the panel zone.

Concrete damage maps (Fig. **13** and **14**) in compression and tension confirm these trends: B1 develops large crushed and cracked regions across the joint panel; B2 exhibits more distributed cracking and reduced crushing intensity; B3 lies between these two extremes, with some alleviation of joint shear damage but still significant distress in the panel.

Overall, internal confinement improves the behaviour of non-ductile joints and delays joint shear failure. However, for the investigated configuration, the gain in peak strength and stiffness remains modest, and damage still concentrates in the joint region even in the fully confined case.

3.3. Strengthening the Unconfined Joint with 45° Steel Haunches (BJS)

To evaluate a practical retrofit solution for severely deficient joints, the unconfined configuration B1 is strengthened with steel haunches inclined at 45° (BJS). The haunches connect the beam and column via steel plates and post-installed anchors, forming a triangular subassembly that diverts a portion of the joint shear and bending demand into the beam.

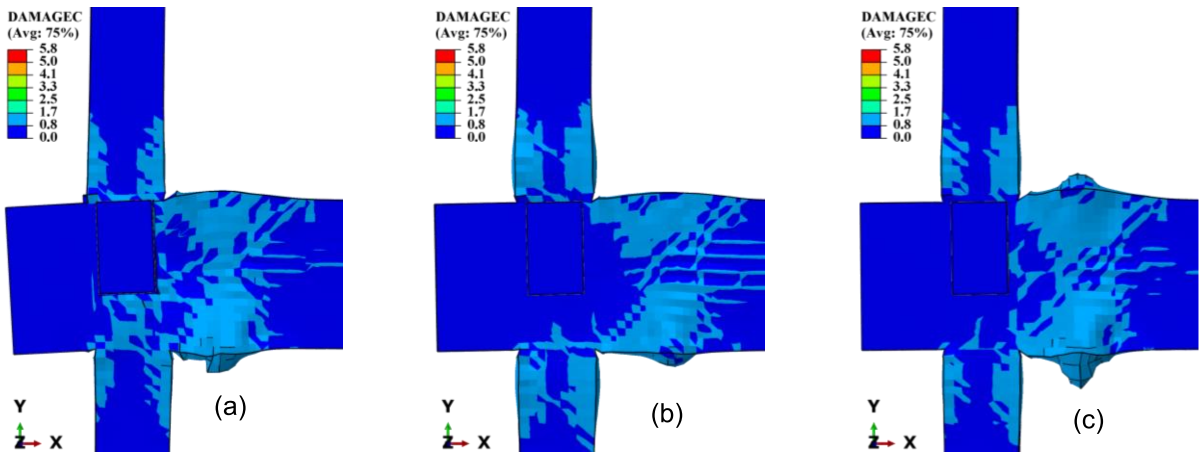


Figure 13: Mapping of compressive failure zones in concrete (DAMAGEC) at the end of cyclic loading: (**a**) B1; (**b**) B2; (**c**) B3.

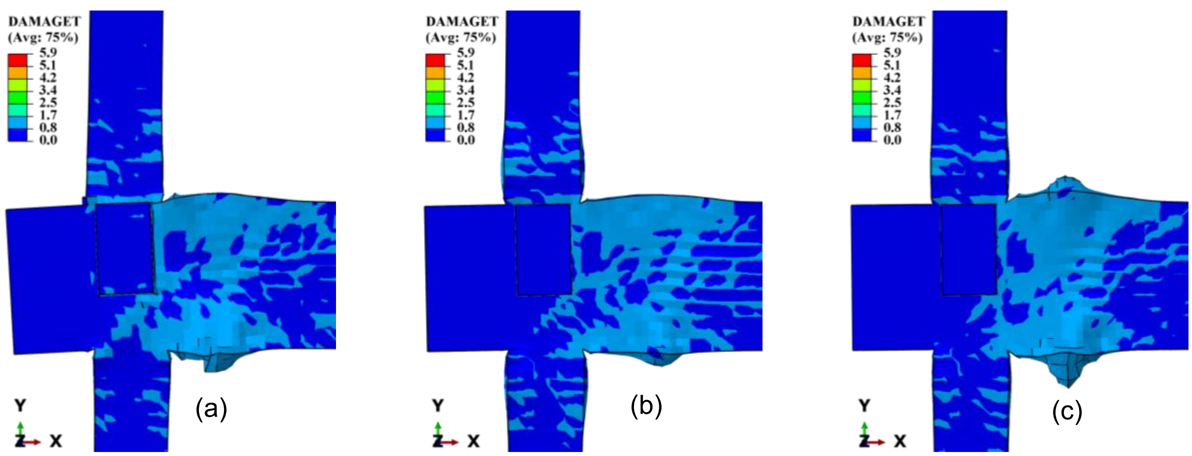


Figure 14: Mapping of tensile failure zones in concrete (DAMAGET) at the end of cyclic loading: (**a**) B1; (**b**) B2; (**c**) B3.

The curves in Fig. (**15-16**) show that the 45° haunch strengthening leads to a substantial increase in peak lateral strength and secant stiffness compared with the reference unconfined joint.

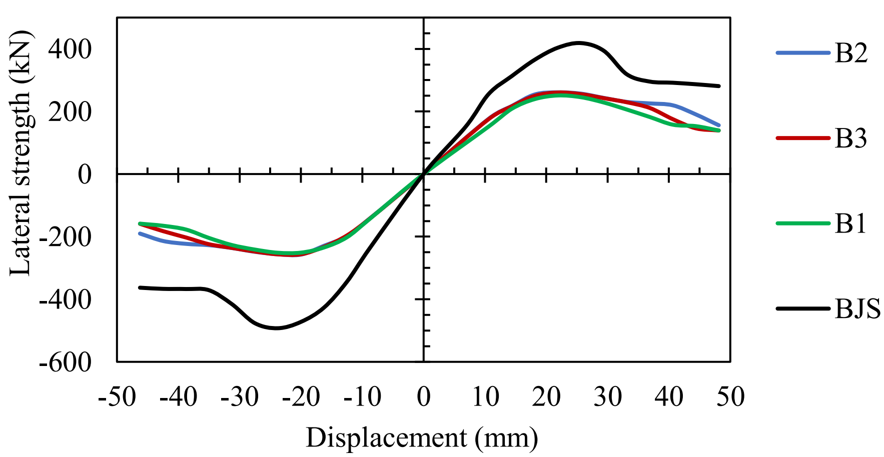


Figure 15: Force-displacement curves of B models.

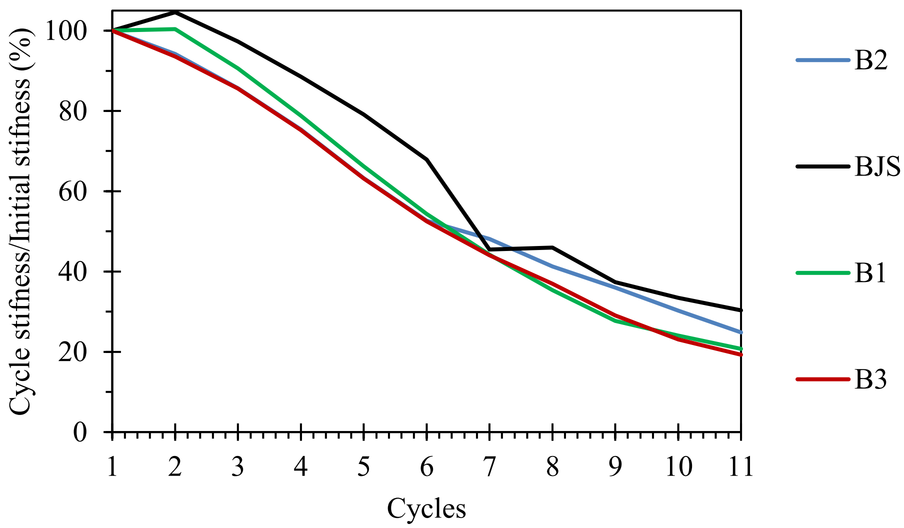


Figure 16: Stiffness degradation curves of B models.

The evolution of normalized cyclic stiffness in Fig. (**16**) further differentiates the specimens. All configurations exhibit progressive stiffness degradation with increasing loading cycles, as expected under cyclic inelastic demand. However, BJS maintains a higher stiffness retention up to approximately the sixth cycle (on the order of 65–70% of the initial stiffness), while B1, B2, and B3 decline more rapidly to approximately 50–55% over the same interval. Beyond the seventh cycle, the stiffness trajectories tend to converge; nevertheless, BJS consistently preserves a modest but discernible advantage. At the final recorded cycle, the residual stiffness of BJS remains around 30%, compared with approximately 20–25% for the other specimens.

The damage distribution at the end of the loading protocol, shown in Fig. (**19**), highlights a major change in failure mechanism. Whereas B1 fails by severe joint shear cracking and crushing, BJS shows only limited damage in the joint panel and column. Instead, damage concentrates in the beam near the haunch anchorage, where a clear plastic hinge forms Fig. (**17**). The compression and tension damage variables in the joint remain significantly lower than in the unstrengthened case, indicating that the haunch effectively offloads the joint core. This behaviour is attributed to the haunch's ability to intercept and redirect the force path, transferring a portion of the axial and bending demands away from the joint panel Fig. (**18**). As a result, the column experiences reduced stress concentrations, and the structure enforces a strong-column/weak-beam mechanism.

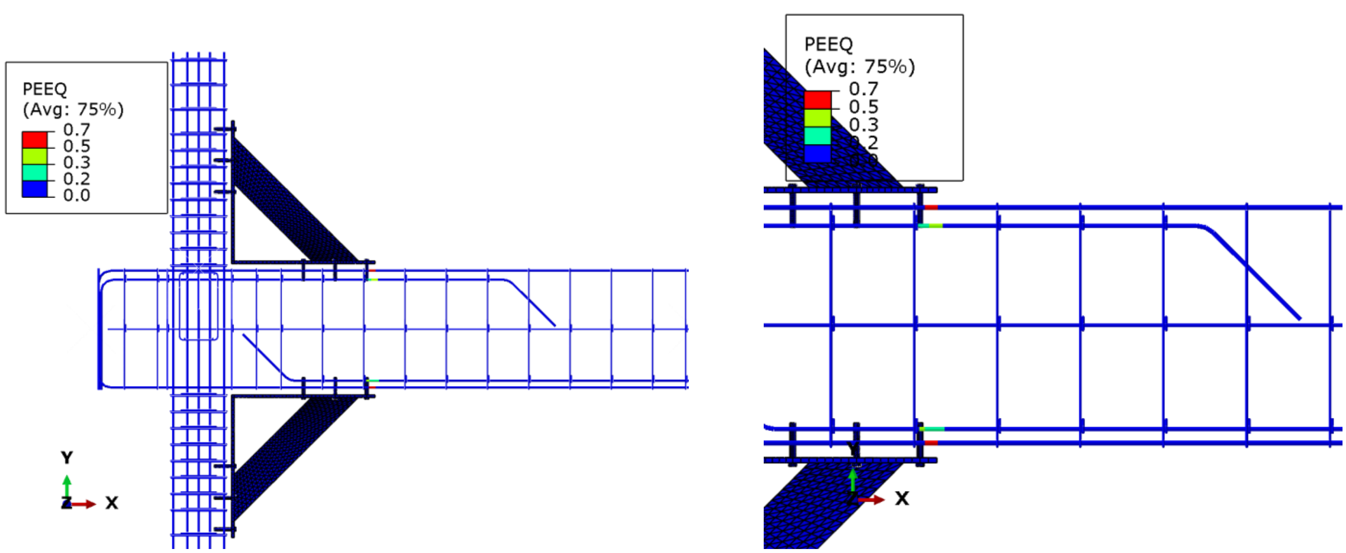


Figure 17: Plastic equivalent strain in beam reinforcement.

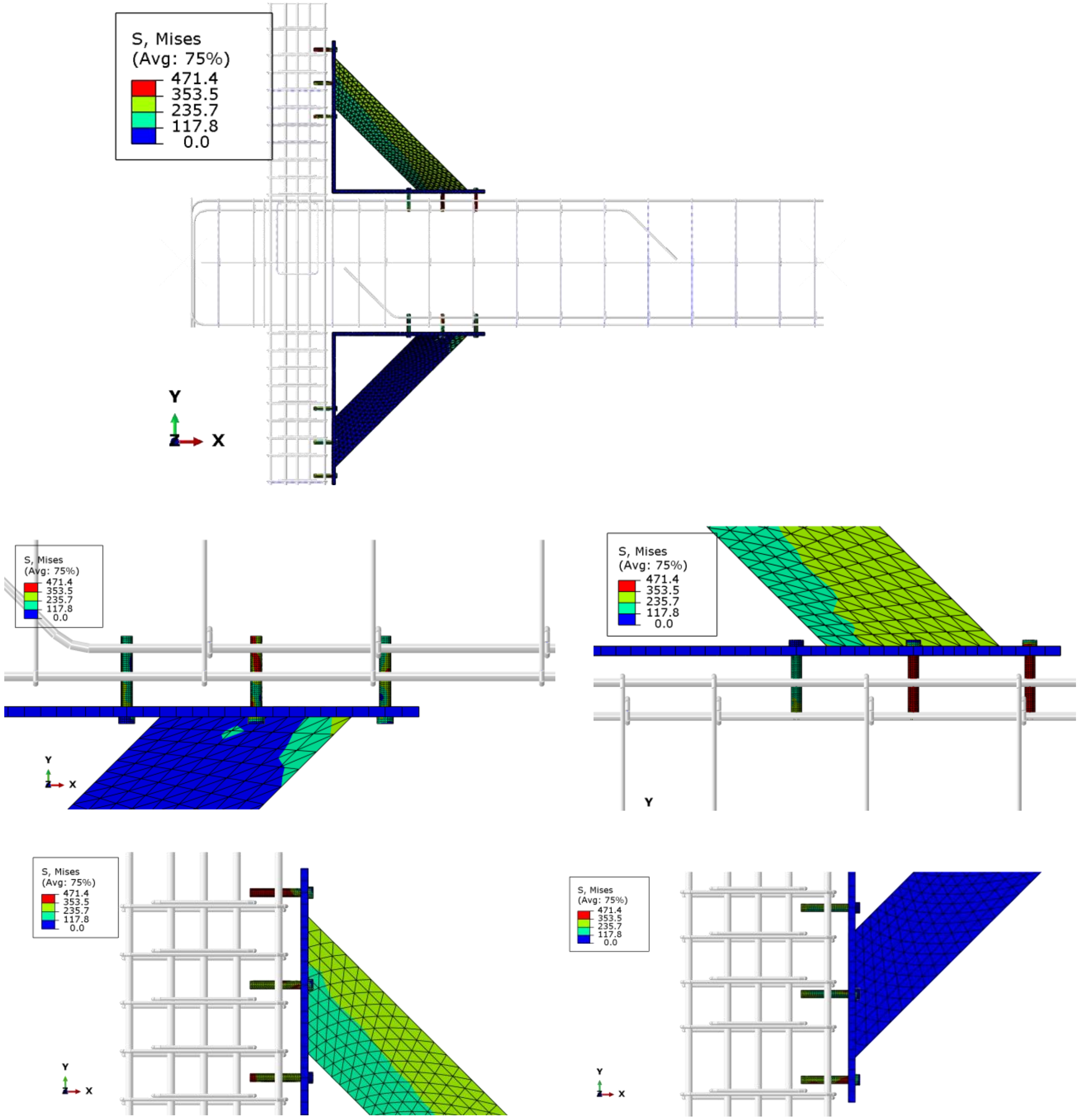


Figure 18: Stress distribution (von Mises) in haunch and bolts (in MPa).

From a comparative standpoint, the three conventional models (B1-B3) demonstrate superior deformation capacity relative to BJS, as reflected by their higher ductility ratios (Table **3**). The differences, although moderate in absolute magnitude, are systematic and consistent across loading directions. BJS exhibits a moderate reduction in ductility relative to the most ductile unstrengthened configuration, but retains an acceptable drift capacity while providing much larger strength and stiffness. For the joint typology investigated, this trade-off is favourable: the retrofit eliminates brittle joint and column failures and replaces them with a more desirable and controllable beam-hinge mechanism.

Accordingly, the structural response of BJS may be characterized by higher strength and stiffness but lower relative ductility, whereas B1, B2, and B3 display a more ductile behavior with greater deformation capacity prior to ultimate conditions. This trade-off between strength–stiffness enhancement and ductility demand are critical in evaluating overall seismic performance and should be carefully considered in design-oriented interpretations.

Table 3: Ductility ratio (Δu/Δy).

	Model	Ductility Ratio (Δu/Δy)
Positive Cycles	B1	0.087
	B2	0.084
	B3	0.084
	BJS	0.061
Negative Cycles	B1	0.080
	B2	0.078
	B3	0.078
	BJS	0.058

Table 4: Comparison between models.

	Model	Peak-load	Percentage Difference	Stiffness	Percentage Difference
		kN	%	kN/mm	%
Positive Cycles	B1	251.13		11	
	B2	260.40	4%	12	9%
	B3	261.08	4%	12	9%
	BJS	418	75%	16	78%
Negative Cycles	B1	-251.50		11	
	B2	-256.52	2%	11	2%
	B3	-257.23	2%	13	18%
	BJS	-492	58%	21	61%

It is worthy to mention that the peak load values presented in Table **4** should not be directly compared with the peak load reported in the validation section, as they correspond to different structural models developed for distinct purposes within the study. The validation model was specifically designed to replicate the geometry, boundary conditions, and reinforcement detailing of the reference experimental specimen. Owing to its smaller scale and reduced cross-sectional properties, this model exhibits a lower global load capacity, with a peak load of approximately 44 kN. Conversely, specimens B1, B2, B3, and BJS belong to the parametric investigation and are based on a different structural configuration. These models incorporate larger geometric dimensions, increased concrete cross-sectional areas, and different reinforcement ratios. Such differences in section properties and overall geometry naturally result in significantly higher load-carrying capacities, leading to peak loads on the order of 250 kN.

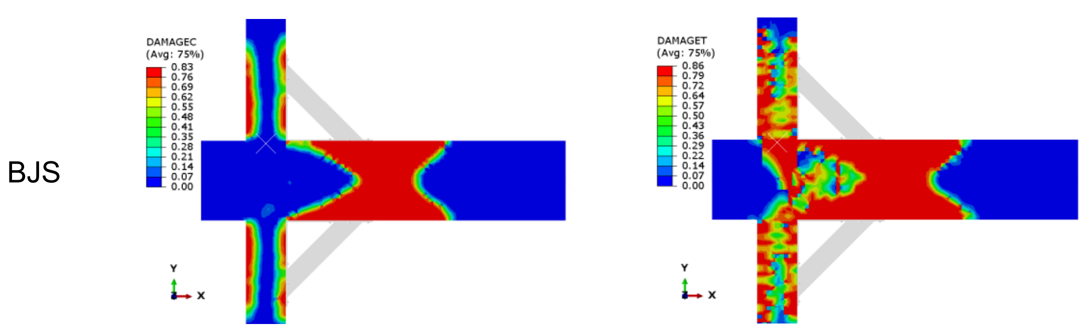


Figure 19: Damage distributions and failure modes of BJS series at the end of analysis: **a**) damage by compression; **b**) damage by tension.

It is therefore emphasized that all reported forces correspond to the total applied load (global structural response) associated with each specific model configuration. The observed discrepancy in peak load values is thus attributable to differences in structural scale and section characteristics.

4. Discussion

The numerical results allow a direct comparison of three practically relevant situations for non-ductile exterior joints:

- Unconfined joint: susceptible to early joint shear failure and rapid strength loss.
- Confined joints: show moderate improvements in strength, stiffness and energy dissipation, but damage still concentrates in the joint panel, and the global mechanism remains joint-controlled.
- Unconfined joint strengthened with 45° steel haunches: exhibits large gains in strength and stiffness, reduced joint and column damage, and a plastic hinge relocated into the beam.

Internal confinement is an important design parameter and should not be neglected where it can be provided. However, for existing structures where joint reinforcement cannot easily be enhanced, the results show that confinement alone may be insufficient to radically change the failure mechanism: the joint remains the most vulnerable region, and brittle shear damage can still govern the response.

By contrast, adding 45° steel haunches to the most deficient (unconfined) joint not only increases capacity but also fundamentally alters the failure hierarchy. The haunches create an alternative load path that bypasses the joint panel and transfers a significant share of the shear and bending to the beam, where a ductile plastic hinge develops. This is fully consistent with capacity design principles and is more reliable than simply increasing joint confinement in isolation.

From an application perspective, the 45° haunch retrofit is relatively simple and minimally invasive: it can be installed externally using steel plates and post-installed anchors, without major demolition or thick jacketing. The present results suggest that, for joints of the type studied, such a retrofit is an effective way to mitigate existing deficiencies, even when the original joint is completely unconfined.

5. Conclusions

This study has evaluated, through a finite element (FE) comparative approach, the influence of local confinement and the introduction of metallic haunches on the mechanical and seismic behaviour of reinforced concrete beam-column joints. The results confirm that confinement is an effective means of enhancing seismic performance, leading to increases in strength, ductility and energy dissipation capacity when compared with an unstrengthened configuration. A clear hierarchy of performance emerges from the comparative analysis of the confined models. Simultaneous confinement of both the beam and the column provides the highest level of performance among the confinement-based solutions, ensuring a more stable global response and more favourable failure mechanisms. Confinement of the column alone also proves to be effective and delivers satisfactory performance, highlighting the dominant role of the column in the seismic response of the joint. Confinement of the beam alone exhibits a comparable load-bearing capacity; however, it remains slightly less effective due to less favourable failure mechanisms. By contrast, the introduction of two metallic haunches inclined at 45°, applied to the unconfined reference configuration, results in a very pronounced improvement in both mechanical and seismic behaviour. This solution clearly outperforms conventional confinement techniques by favourably modifying the force transfer mechanism, reducing shear demand within the joint region, and delaying the onset of critical damage. These findings demonstrate the strong potential of metallic haunches as an efficient solution for the seismic strengthening and retrofitting of reinforced concrete beam-column joints. They also open promising perspectives for future developments involving hybrid design strategies that combine confinement and metallic strengthening, as well as further investigations based on numerical modelling and experimental validation.

Conflict of Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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