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Assessment of the impact of column-to-beam strength ratio on seismic response of RC beam-column connections

Mohamed Mimouni[a*], Djamel El Ddine Kerdal[b] and Lamia Kharroubi[a]

[a]*Laboratoire Matériaux et Structures, Civil Engineering Department, University of Tiaret, Zaaroura, B.P. 78, Tiaret, Algeria*
[b]*LM2SC, Civil Engineering Department, U.S.T.O.M.B., B.P. 1505 El M'Naouer, Oran, Algeria*

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A B S T R A C T

Beam-column joints play an important role in the overall resistance of reinforced concrete frames during seismic events. The Earthquakes of Al Asnam on October 10, 1980, and Boumerdes on May 21, 2003 in Algeria, have highlighted the failure of beam-column joints as a cause of building collapse. 3D nonlinear finite element analysis with ANSYS software has been used in order to examine exterior RC beam-column joints under monotonic loading. The research has investigated the influence of the value of the column-to-beam strength ratio (CBSR) of these joints on shear strength and seismic performance of concrete structures by changing the height and longitudinal reinforcement of the beam while keeping column dimensions constant. It has been shown that the beam-column depth ratio and the beam longitudinal reinforcement ratio have a significant effect on the shear capacity and seismic behaviour of exterior beam-column joints. An appropriate value of the flexural strength ratio between the column and beam is crucial in order to establish a "strong column-weak beam" mechanism in reinforced concrete frames, because inadequate values can lead to premature failure or reduced shear capacity at the junction. The Algerian seismic code – RPA99/version2003 suggests a minimum value of column-to-beam strength ratio equal to 1.25 at joints for seismic design when building codes in other countries call for higher ratios. Nevertheless, this approach might not accurately represent the actual behaviour and could constrain the optimal performance of structures.

1. Introduction

The coastal zone and the Atlas Mountain range of Algeria have always been subject to intense seismic activities mainly due to the geological nature of the Maghreb region and the tectonic features on the border of the African and Eurasian plates that are in a constant motion compression. The earthquake magnitude in this region can be very high and therefore lead to catastrophic damage in the vicinity of their hypocenter. A magnitude of 6.8 was reported in the earthquake of May 21, 2003 that struck the wilayas of Boumerdes and Algiers and covered an area of approximately 150 km x 80 km.

The beam-column connections of reinforced concrete frames could represent the most vulnerable zone of the structure. These connections play an essential role in the behaviour of the structures. Their realization is often delicate because of the high density of reinforcement and the complex geometry of formwork collapses of structures. During severe earthquakes, column-beam joints experience significant forces, and their behaviour has a significant impact on the overall response of the structure. Shear failure, which is brittle, is not an acceptable structural performance, particularly under seismic conditions. It is important to understand the behaviour of the nodes if we want to design them adequately.

* Corresponding author.
E-mail addresses: mohamed.mimouni@univ-tiaret.dz (M. Mimouni)

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There have been many catastrophic collapses of structures reported in past earthquakes that have been attributed to column-beam joints (**Fig. 1**).

(a) (b)

Boumerdes earthquake in Algeria 2003 (Davidovici, 2003)

(c) Izmit earthquake in Turkey 1999
(Sezen et al. 2000)

(d) Abruzzo earthquake in Italy 2009
(Günay & Mosalam, 2010)

Fig. 1. Damage of beam-column joints in past earthquakes

For several decades, the behaviour of column-beam joints under seismic conditions has been studied experimentally and analytically. The effects of the variation of different parameters on the shear strength and the load-displacement curves of reinforced concrete joints such as the joint panel geometry, the beam longitudinal reinforcement, the joint transverse shear reinforcement, the concrete compressive strength and the column axial load have been studied. Various international codes (ACI 318R, 2014; Eurocode 8, 2004; NZS3101.1, 2006) have been revised accordingly in order to incorporate new research findings.

The seismic behaviour of reinforced concrete beam-column joints subjected to a lateral seismic load has been the subject of several experimental studies. A method that could predict the ductile capacity of reinforced concrete beam-column joints, failing in shear after the development of plastic hinges at both ends of the adjacent beams, was proposed by (Lee et al., 2009) who then tested experimentally five joint specimens with varying tensile reinforcement ratios in order to check the validity of the proposed equations. The effect of plastic hinges that developed at both ends of the adjacent beams on the resistance of the joint was evaluated by checking the values of the longitudinal strains of beam bars within the joint. The ductile capacity of BJ-failure, which occurs when joint shear failure happens simultaneously with beam reinforcement yielding joints, was estimated using the proposed method and compared with the experimental results of RC joints. Moreover, an experimental investigation of four full-scale RC corner beam-column joints with orthogonal transverse beams and floor slabs, without transverse reinforcement in the joint region, was conducted by (Park & Mosalam, 2013). They investigated the effect of the joint aspect ratio and beam longitudinal reinforcement ratio on the shear strength and deformability of these joints under cyclic loading. The test results were then compared with the ASCE 41 recommendations. The authors concluded that the damage of the four test specimens was concentrated at the corner joint region due to the absence of transverse reinforcement. Additionally, they found that the shear strength of unreinforced corner joints was reduced by the increase in joint aspect ratio. Furthermore, an experimental study was performed by (Hwang et al., 2014) to evaluate the seismic performance of interior and exterior beam-column connections under cyclic lateral loading using Grade 600 MPa (87.0 ksi) bars for beam flexural reinforcement. The specimens were designed according to the special seismic provisions in ACI 318R (2014). The column depth-to-beam reinforcing bar diameter ratios (hc/db) ranged from 20.5 to 25.0. (Liu et al., 2021) conducted tests on three 2/3-scale exterior beam-column joints without seismic design details, subjected to reversed cyclic loads. They demonstrated that the eccentricity between the centerlines of the beam and the column had significant effects on the damage characteristics, shear strength, and displacement ductility of the specimens. Specimens with a 1/4 column width eccentricity exhibited brittle failure with premature joint shear failure, while those with less or no eccentricity displayed ductile failure after beam flexural yielding. Comparison of test results with predictions from three seismic design codes and two non-seismic design codes indicated that these codes generally underestimated the shear strength of eccentric joints without seismic details.

Analytical models that described the shear behaviour of RC joints in the nonlinear analysis of reinforced concrete frames have been proposed by various researchers. A shear-strength analytical model describing the shear strength of exterior RC beam-column joints without transverse reinforcement in the joint region was proposed by (Park & Mosalam, 2012a, 2012b). To investigate the relevance of the strength recommendations in ASCE 41, previous unreinforced exterior or corner joint test data entries from published literature were collected by the authors, following these selection criteria: (1) the beam longitudinal reinforcement hook is bent into the joint region; (2) the beam width is equal to or smaller than the column width; and (3) the column shear and lap splice failures do not govern the response. The assessment of the joint shear strength for the collected database reveals that the shear strength of unreinforced exterior joints may be underestimated by ASCE 41. In a similar context, an analytical shear strength model for exterior beam-column connections based on the average plane stress concept was developed by (Tran & Hadi, 2017). The contribution of shear reinforcement and column axial load to the shear strength of the joint was considered. The proposed model was validated by comparing its results with those available from a large database of exterior beam-column joints subjected to cyclic loading, that was collected from the literature.

An extensive database of reinforced concrete beam-column connection test specimens exhibiting joint shear failure with a minimum amount of joint confinement was constructed by (Kim & LaFave, 2007). The objective of the study was to assess the most influential parameters affecting the joint shear behaviour. The parameters examined by the authors were grouped by categories such as material property, joint panel geometry, reinforcement confinement, column axial load, and reinforcement bond condition. The authors concluded that the parameters that influenced joint shear behavior were associated with connection type (interior, exterior, or knee joint) and failure mode sequence. However, concrete compressive strength was identified as the most common determinant parameter on joint shear behavior for all groups in the constructed database. The comparison of the respective influence of the five key parameters on the shear resistance of exterior and interior reinforced concrete beam-column connections has also been studied by (Tran, 2016) who analyzed 172 experimental tests existing in the published literature. The ranking by level of importance of these parameters on the shear strength of the joint was found to be: (1) the compressive strength of the concrete, (2) the vertical shear reinforcement of the joint, (3) the longitudinal reinforcement of the beam, (4) the shear reinforcement of the horizontal joint and (5) the axial stress of the column. The most important therefore turned out to be the compressive strength of the concrete. Additionally, Parate and Kumar (2019) assessed shear strength provisions in various national codes for RC beam-column joints, such as ACI 318-2014, NZS 3101-1:2006, EN 1998-1:2004, CSA A23.3:2004, AIJ:2010, and IS 13920:2016. Their analysis of 492 experimental results highlighted the impact of joint aspect ratio and column-to-beam area ratio on code-based predictions. They proposed two approaches to improve code predictions: one based on strut angle variation and the other using empirical modification factors based on the area ratio of the column to the beam cross-section. These approaches helped reduce the disparities between code predictions and experimental outcomes, making them more suitable for design purposes.

Several numerical analyses have been carried out on RC beam-column joints using nonlinear finite element analysis. In a numerical investigation carried out by Niroomandi et al. (2014), the shear failure in nonductile RC exterior joints was studied. The finite element modelling of the RC beam-column joints was performed using the (ANSYS program, 2004). The research involved a parametric study on reinforced concrete exterior beam-column joints without transverse reinforcement in the joint region. The parameters under investigation were the joint aspect ratio and the beam longitudinal reinforcement ratio, and their effects were thoroughly discussed. The results of the research confirmed that these parameters exert a significant influence on the shear failure in RC joints. Similarly, a nonlinear finite element analysis using (ABAQUS, 2014) of two RC beam-column connections, one exterior and one interior, was performed by Najafgholipour et al. (2017) in order to observe the joint shear failure under quasi-static lateral loads and the numerically obtained results were verified by using experimentally test results on exterior and interior beam-column connections. The primary failure mode observed in these experiments was the joint shear failure type. Comparing the experimental and numerical results, it was found that the FE model successfully simulated the performance of the beam-column connections and effectively captured the joint shear failure in RC beam-column connections. Likewise, a nonlinear finite element analysis, using the (ABAQUS software, 2014), of a reinforced concrete exterior beam-column connection subjected to lateral loading was carried out by (Diro & Kabeta, 2020). The investigation focused on the joint shear failure mode, considering joint shear capacity, deformations, and cracking patterns. In their study, the most influential parameters, column axial load, beam longitudinal reinforcement ratio, joint panel geometry, and concrete compressive strength, affecting joint shear failure were evaluated. The authors concluded that the concrete compressive strength was the most important parameter for predicting joint shear failure. In their research, (Maosheng et al., 2021, 2022) explored how varying column-to-beam flexural strength ratios (CBFSR) in RC frames designed to Chinese seismic codes impacted collapse capacity and failure modes. They found that increasing the flexural strength ratio of columns to beams (FSRCB) significantly improved collapse capacities, particularly in low-rise (three-story) frames with FSRCB = 2.0, resulting in a 1.6–2.0 times increase compared to FSRCB = 1.2. They also noted that the required CBFSR to ensure a strong column-weak beam (SCWB) failure mode depended on the beam-column connection type and seismic intensity. (Kim et al., 2022) conducted a numerical study to determine the required strength ratio in structural design, considering factors like earthquake and gravity loads, stiffness ratios, joint locations and frame heights. Their findings showed that the strength ratio varies based on these factors. They developed a design method for the strength ratio and conducted nonlinear static and dynamic analyses on frames designed using this approach, demonstrating its potential to enhance seismic performance.

The work presented in this article is part of a study related to the seismic evaluation of existing reinforced concrete buildings. In this work, we were interested in the behaviour of the external beam-column connection of an existing reinforced concrete frame. The RC beam-column joint plays a fundamental role in the overall stability and strength of structures, and understanding how the value of the column-to-beam strength ratio (CBSR) can affect the seismic performance of the beam-to-column external RC joints is of paramount importance. Our objective was to examine how different values of the column-to-beam resistance ratio can influence the mechanical characteristics of column-to-beam connections. Traditionally, this ratio is often considered constant, with a generally accepted value in design codes (ACI 318R, 2014; Eurocode 8, 2004; NZS3101.1, 2006; RPA99/version 2003). Compared to previously published research, our study places a special focus on the minimum column-to-beam strength ratio required by the Algerian seismic code (RPA 99/version 2003, 2003). In order to establish a "strong column-weak beam" mechanism in reinforced concrete frames, the Algerian seismic code recommends a minimum column-to-beam strength ratio (CBSR) of 1.25 at joints for seismic design when building codes in other countries call for higher ratios. However, this value might not accurately reflect reality, potentially limiting the optimal performance of structures. In this study, the column-to-beam strength ratio (CBSR) value is varied by changing the longitudinal reinforcement and the height of the beams while maintaining the dimensions and reinforcement of the column's constant. These two parameters are chosen due to their significance in the behaviour of RC beam-column joints.

In order to predict the response of exterior RC beam-column joints subjected to monotonic loading, a 3-D nonlinear finite element model was constructed using (ANSYS Commercial Software, 2009). The nonlinear analysis was performed using the modified Newton-Raphson method with a step-by-step load application. The numerically obtained results compared well with the experimental results reported by (Mahini & Ronagh, 2011). Subsequently, a finite element analysis was conducted to investigate the performance of a typical exterior RC beam-column joint in an existing reinforced concrete frame in Algeria. A parametric study was conducted to investigate how the column-to-beam strength ratio (CBSR) affects the shear strength and seismic performance of RC exterior beam-column joints. This study focused on exploring the effects of variation of the ratio of beam depth to column depth and amount of beam longitudinal reinforcement. By examining these key factors, we aimed to gain a better understanding of how exterior RC joints behave under different conditions.

2. Strong-column weak-beam concept

The principle of capacity design, also known as "strong column - weak beam", aims to improve the seismic performance of buildings by promoting the formation of plastic hinges in the beams rather than in the columns during an earthquake. The formation of plastic hinges in the columns results in the transformation of the structure into a mechanism. This explains the concern of seismic codes to give columns a higher resistance than beams. The capacity design philosophy, introduced by (Park & Paulay, 1975), forms the basis of the strong column/weak beam (SCWB) concept, aimed at improving seismic design. The concept is based on the prevention of brittle fractures of the columns during major seismic events. To achieve this aim, the structures are designed so that the plastic hinges form at specific locations in the beams.

One of the basic requirements of the strong column/weak beam concept is that the columns above and below the node must have sufficient bending resistance when adjacent beams develop bending over-resistance at their plastic hinges. The ratio of the bending resistance of columns to that of beams is an important parameter to ensure that plastic hinges form in beams rather than columns. In order to satisfy the philosophy of the strong column and the weak beam, the bending resistance of the column M_c must be higher than that of the beam M_b and it can be written:

$$Mc \geq Mb \tag{1}$$

Previous research has shown that the resistance ratio between columns and beams is an important parameter that conditions the resistance of reinforced concrete frame buildings during earthquakes. The seismic performance of frame buildings with different strength ratios was analysed by Dooley and Bracci (2001), it was found that a resistance ratio of at least 2.0 is required to prevent the formation of a story mechanism under a seismic load. Increasing the strength ratio alone is more effective than simultaneously increasing the strength and stiffness ratios. Similarly, buildings with different column-to-beam resistance ratios were studied by Sunitha et al. (2014), who concluded that a ratio between 2.5 and 3.0 is needed for a sway mechanism in moment resisting RC buildings. A column-to-beam resistance ratio of about 1.2 was found to be insufficient in preventing the story mechanism. Additionally, the seismic fragility of RC frame buildings that were or were not designed according to the strong column-weak beam design criterion, was analysed by Surana et al. (2018). It was found that buildings with a SCWB ratio of 1.4 had an increased overall ductility capability. However, 3-bay, 3-story RC frames were tested by (Su et al., 2020), and observed that the beam-to-column linear stiffness ratio influenced failure patterns. Larger beam-to-column stiffness ratios concentrated plastic hinges in the columns and slowed the development of the beams. Design codes specifying bending resistances from beams to columns may not guarantee SCWB seismic behaviour. In a related context, non-seismically designed exterior RC beam-column joints were tested by Liu et al. (2019), and it was found that the beam-to-column depth ratio significantly affects shear strength and failure mode. Adding cross links to the joint web improved ductility and seismic behaviour. However, Tsonos (2007) who examined the seismic performance of exterior beam-column subassemblies found that the strong beam-weak column design philosophy in the current building codes can sometimes lead to severe damage in critical joint and column regions.

3. Finite element modelling of exterior RC beam-column joints

3.1 Description of the finite element model

A three-dimensional finite element model using (ANSYS, 2009) was developed to predict the monotonic behaviour of RC beam-column joints. The concrete was modelled using the Solid65 element. This element which has eight nodes with three degrees of freedom at each node is capable of modelling both cracking in tension and crushing in compression. The longitudinal and transverse reinforcements were modelled using the LINK8 truss element. Two nodes are required for this element, each node having three degrees of freedom. The element has the ability to undergo plastic deformation. To avoid stress concentration issues and prevent localized crushing of concrete elements near the load application areas, steel plates were implemented at the loading locations using the Solid45 element. This element consists of eight nodes, with three degrees of freedom assigned to each node.

To accurately model concrete, the Solid65 element necessitates both linear isotropic and multilinear isotropic material properties. In order to define the failure of the concrete, the multilinear isotropic material uses the Von Mises failure criterion along with the Willam and Warnke model (Willam & Warnke, 1975). The modulus of elasticity of the concrete (E_c) was based on the equation (ACI 318R, 2014):

$$E_c = 4700\sqrt{f_c'} \tag{2}$$

with f_c' the ultimate uniaxial compressive strength of concrete.

The conditions at the crack face are represented by the shear transfer coefficient for open cracks, βt, and the shear transfer coefficient, βc, for closed cracks. The value of βt ranges from 0.0 to 1.0, with 0.0 representing a smooth crack (complete loss of shear transfer) and 1.0 representing a rough crack (no loss of shear transfer) (ANSYS, 2009). The value of βc is found to be equal to 1.0 (Kachlakev et al., 2001). In this study, shear transfer coefficient was taken 0.25 for open cracks and 0.99 for closed cracks (ANSYS 2009; Kachlakev et al., 2001).

The uniaxial tensile strength σ_t, can be calculated, based on the following equation (ACI 318R, 2014):

$$\sigma_t = 0{,}623\sqrt{f_c'} \tag{3}$$

The uniaxial crushing stress in this model was entered as -1 to turn off the crushing capability of the concrete element as suggested by past researchers (Kachlakev et al., 2001).

In order to achieve an accurate steel model, multi-linear kinematic hardening (MKIN) was used to describe the stress-strain relationship and define curves based on the elastic modulus, yield stress, and tangent modulus (Kachlakev et al., 2001). While it is important to consider the bond strength between the concrete and steel reinforcement, this study assumed a perfect bond between the materials. In order to establish a perfect bond, the link element for the steel reinforcement was connected between the nodes of each adjacent concrete solid elements, allowing both materials to share the same nodes

In nonlinear analysis, the total load was applied step by step and the solution was achieved using the modified Newton Raphson method (ANSYS, 2009).

The finite element model was constructed according to a test specimen selected from the literature (**Fig. 2**). The test specimen was a part of an experimental program conducted on an exterior RC joint by Mahini and Ronagh (2011). The selected specimen was an RC joint without retrofitting. The geometry of the studied specimen (Mahini & Ronagh, 2011) as well as the considered reinforcements for beams and columns are presented in Fig. 2. The joint consisted of 180 mm wide and 230 mm deep beams with 220 mm × 180 mm columns. Beams were reinforced with 12 mm diameter (N12) high-strength longitudinal reinforcing steel bars, two bars in the top and two bars in the bottom of the beam. Columns were reinforced with four N12 reinforcing bars, with one bar positioned at each corner of the columns. The average yield strength of deformed N12 reinforcing steel bars and plain R6.5 mm stirrups and ties, was 507 MPa and 382 MPa respectively and the modulus of elasticity of both reinforcements was 200 GPa. R6.5 bars were used for stirrups with a spacing of 150 mm in both column and beam. Ties were also placed in the joint's region in accordance with the requirements of AS3600 (2001). Additional stirrups and ties and N16 (φ = 16 mm); threaded rods were placed near the ends of the beam and column in all specimens to ensure that local failure does not occur at the load and support points respectively. The concrete had a compressive strength around 40 MPa and a modulus of elasticity around 27.6 GPa.

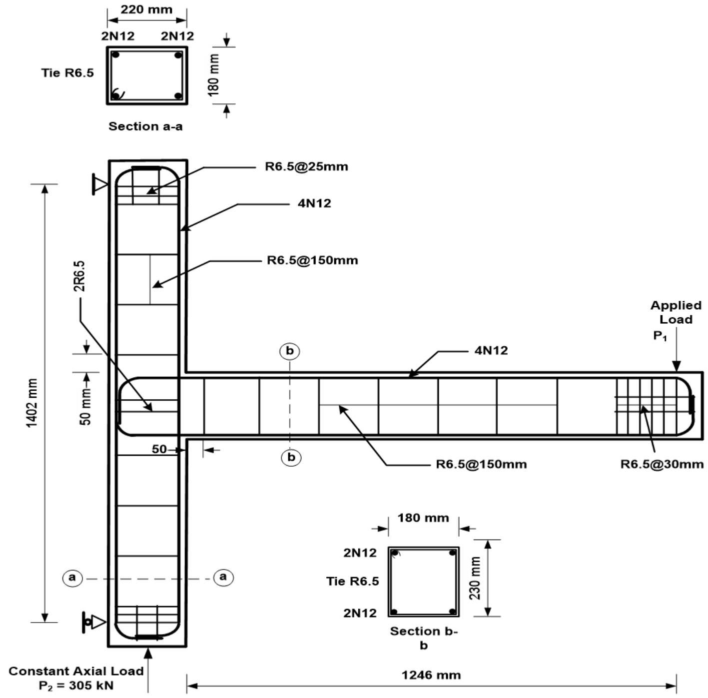


Fig. 2. Specimen's details (Mahini & Ronagh, 2011)

Fig. 3 shows the stress-strain relationship of concrete modelled through the Multilinear Isotropic Hardening option was based on work done by Kachlakev et al. (2001). The multi-linear curve was used to help with convergence of the nonlinear solution algorithm.

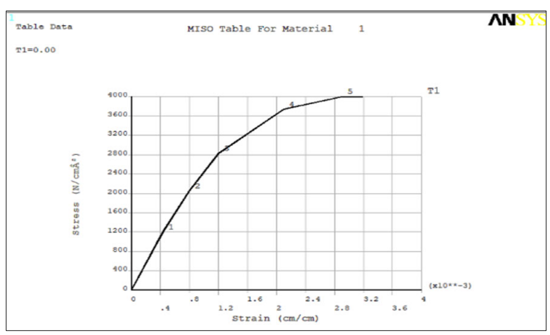
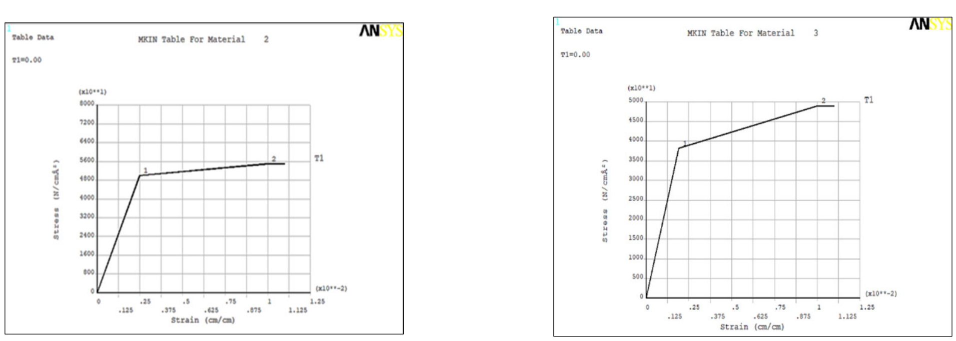


Fig. 3. Uniaxial compressive stress–strain curve for concrete

Fig. 4 shows the stress–strain relationship used in this study for longitudinal and transverse reinforcement. It is assumed to be multi-linear kinematic hardening.

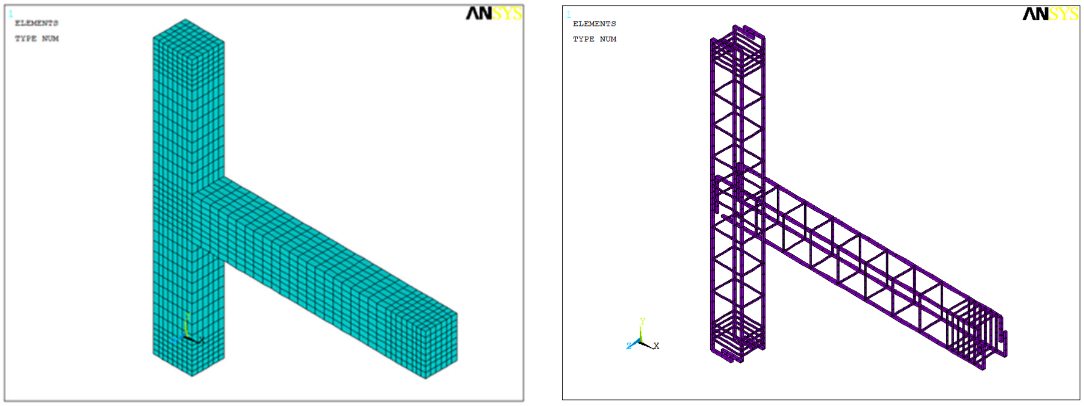


a) for longitudinal reinforcement

b) for transverse reinforcement

Fig. 4. Stress–strain curve for steel reinforcement.

The finite element mesh of the beam-column model and the reinforcement details in the joint are shown in **Fig. 5**.



a) Finite element mesh of the joint model b) Reinforcement details in the joint

Fig. 5. Finite element modelling of the joint

3.2 Comparison of numerical and experimental results

In order to investigate the behaviour of RC beam-column joints under monotonic loading, the result obtained by the numerical finite element model was compared with the experimental test conducted by (Mahini & Ronagh, 2011). Fig. 6 represents the beam tip load-displacement curves obtained from FE results (in solid lines) with the experimental results (in dashed lines). Comparative results show a good correlation between the experimental test and the numerical model regarding initial stiffness and ultimate load capacity. Therefore, the proposed FE model can accurately reproduce the behaviour of the joints.

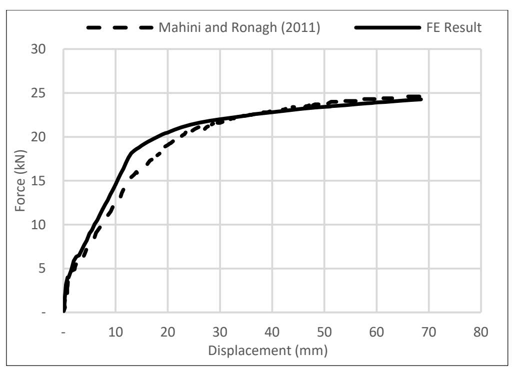


Fig. 6. Comparison of load–displacement curve between the experimental test (Mahini and Ronagh 2011) and the FE numerical results.

4. Parameters affecting the joint behaviour

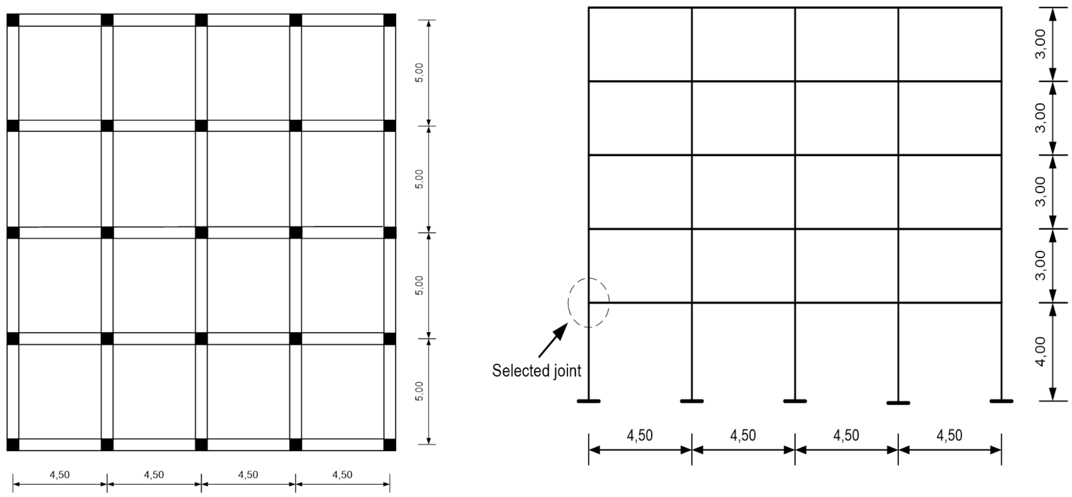
After validating the numerical model by comparing its results with the experimental findings of (Mahini & Ronagh, 2011), nonlinear numerical analyses were employed to evaluate the impact of the column-to-beam strength ratio (CBSR) on the seismic performance of exterior RC beam-column joints. The CBSR value was varied by changing the height and longitudinal reinforcement of the beams while keeping the dimensions and reinforcement of the column's constant. Six different variants for each parameter were selected to assess the influence of CBSR on the strength, ductility, and probability of joint failure. The value of the column-to-beam strength ratio (CBSR) is defined as:

$$\beta = \frac{\sum M_{Rc}}{\sum M_{Rb}} \tag{4}$$

$\sum M_{Rc}$: the sum of the ultimate resistant moments of the ends of the columns,
$\sum M_{Rb}$: the sum of the ultimate resistant moments of the ends of the beams,

The ultimate resistant moments of columns and beams were determined according to (Eurocode 2, 2004). To investigate the influence of the column-to-beam strength ratio (CBSR) on the seismic performance of RC beam-column joints, a numerical study was conducted on an external RC beam-column joint belonging to an existing building in Algeria. The selected building consists of five levels and is located in a high seismicity zone (Zone III) according to the Algerian seismic code

(RPA99/version 2003, 2003). **Fig. 7** illustrates the building's floor plan and the elevation of the transverse frame with the selected joint.



(a) (b)

Fig. 7. Details of the building: (a) plan of the building, (b) elevation of the transverse frame with the joint selected

Respective representations of the material properties of concrete and steel can be found in **Tables (1-2).**

Table 1. Materials properties of concrete

Concrete compressive strength f'_c (MPa)	Concrete tensile Strength f_t (MPa)	Modulus of elasticity (MPa)	Ultimate strain	Poisson's ratio
25	2.1	32164.19	0.2%	0.2

Table 2. Materials properties of steel

	Yield trength (MPa)	Ultimate tensile strength MPa)	Modulus of elasticity (MPa)	Ultimate strain	Poisson's ratio
Longitudinal reinforcement	400	480	200000	0.01	0.3
Transverse reinforcement	235	330	200000	0.01	0.3

Fig. 8 shows the geometrical details and the overall dimensions of the exterior RC beam-column joints examined in this study (the lengths in cm, the diameters of the steel bars in mm).

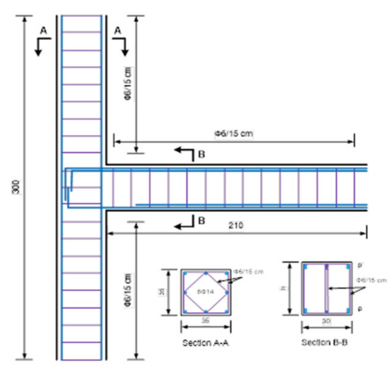


Fig. 8. Geometry and reinforcement detailing of the beam-column joints model

4.1 Effect of beam-to-column depth ratio

The beam-to-column depth ratio, as defined in this study, represents the ratio between the heights of the beam and column cross-sections. Hence, to assess this parameter, six models (A1 to A6) have been selected for examination. The beam dimensions and reinforcement ratios, along with the column-to-beam strength ratio, for the six models, are indicated in Table 3. As the beam-to-column depth ratio increases from 0.86 to 1.57, the values of the column-to-beam strength ratio (CBSR) decrease from 2.98 to 0.82.

Table 3. Beam-to-column depth ratio models

Models	A1	A2	A3	A4	A5	A6
Beam dimensions (cm^2)	30 × 30	30 × 35	30 × 40	30 × 45	30 × 50	30 × 55
Beam upper reinforcement	5ϕ14	3ϕ16+2ϕ14	5ϕ16	2ϕ20+3ϕ14	3ϕ20+2ϕ16	5ϕ20
Beam lower reinforcement	5ϕ14	3ϕ16+2ϕ14	5ϕ16	2ϕ20+3ϕ14	3ϕ20+2ϕ16	5ϕ20
ρ % = ρ' %	0.95	0.96	0.93	0.9	1.0	1.0
Column dimensions (cm^2)	35 × 35					
Column reinforc.	8×14					
h_b/h_c	0.86	1	1.14	1.28	1.43	1.57
M_{Rb} ($kN.m$)	68.97	81.02	120.15	146.85	200.18	256.95
M_{Rc}^{N} ($kN.m$)	99.80	100.40	101.06	101.48	102	102.50
M_{Rc}^{S} ($kN.m$)	105.63	106.17	106.74	107.09	107.51	107.93
β	2.98	2.55	1.73	1.42	1.05	0.82

4.2 Effect of Beam Longitudinal Reinforcement Ratio

The beam longitudinal reinforcement ratio represents the amount of steel reinforcement (rebars) in the beam relative to the concrete cross-sectional area. The reinforcement ratio is expressed as a percentage ($\rho\%$). In this study, six different aspect ratios have been chosen to evaluate this parameter. The models, namely B1 to B6, exhibit varying beam longitudinal reinforcement ratio. As the beam longitudinal reinforcement ratio rises from 0.52 to 1.94, the corresponding column-to-beam strength ratio (CBSR) values decrease from 3.01 to 0.85. Table 4 provides the dimensions of the beams, their reinforcement ratio, and the column-to-beam strength ratio for all six models.

Table 4. Beam longitudinal reinforcement ratio

Models	B1	B2	B3	B4	B5	B6
Beam dimensions (cm^2)	30 × 40					
Beam upper reinforcement	5ϕ12	3ϕ14+2ϕ12	5ϕ16	3ϕ20+2ϕ16	5ϕ20	3ϕ25+2ϕ20
Beam lower reinforcement	5ϕ12	3ϕ14+2ϕ12	5ϕ16	3ϕ20+2ϕ16	5ϕ20	3ϕ25+2ϕ20
ρ % = ρ' %	0.52	0.64	0.93	1.24	1.45	1.94
Column dimensions (cm^2)	35 × 35					
Column reinforcement	8×14					
M_{Rb}($kN.m$)	68.95	83.49	120.15	156.87	183.14	244.93
M_{Rc}^{N}($kN.m$)	101.06	101.06	101.06	101.06	101.06	101.06
M_{Rc}^{S} ($kN.m$)	106.74	106.74	106.74	106.74	106.74	106.74
β	3.01	2.49	1.73	1.32	1.13	0.85

4.3 Results and discussion

The moment-rotation curves for models (A1 to A6) and (B1 to B6) are presented in **Figs. (9-10)** respectively. The results show that reducing the column-to-beam strength ratio (CBSR) leads to a more brittle joint behaviour, as indicated by the capacity curves depicted in both figures for each model.

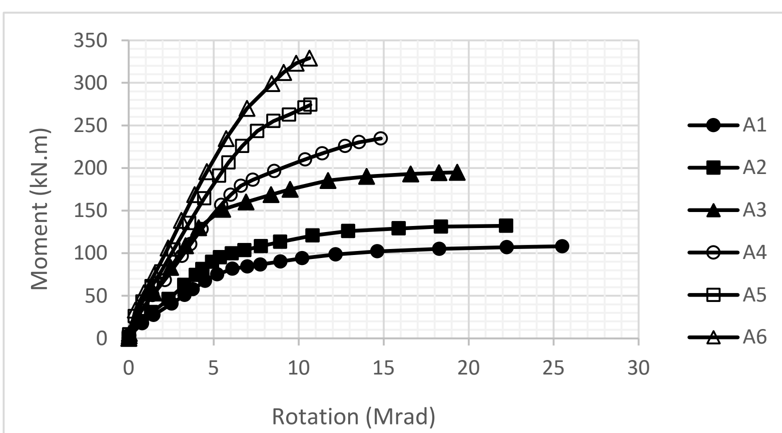


Fig. 9. Moment-Rotation curve of RC Joints for varying Beam-to-Column Depth Ratio

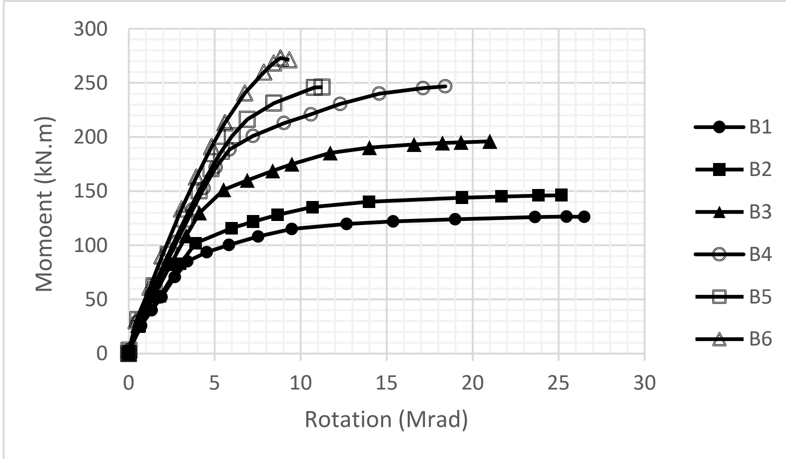


Fig. 10. Moment-rotation curve of RC joint for varying beam longitudinal reinforcement ratio.

Tables (5-6) present the elastic and ultimate moments, corresponding elastic and ultimate rotations and ductility factors for the six variants (A1 to A6) and (B1 to B6) respectively. The idealized moment-rotation curve enables the identification of the yield point and the point of maximum rotation, while also allowing for the determination of the structure's ductility factor, defined as the ratio between the maximum rotation and the yield rotation. Moreover, the moment-rotation curves provide the means to derive both the yield strength and maximum strength of the structure.

Table 5. Elastic and ultimate moments-deformations and ductility factors (Models A1 to A6)

Models	Elastic state		Ultimate state		Ductility factor
	M_y (***kN.m***)	θ_y $(Mrad)$	M_u (***kN.m***)	θ_u $(Mrad)$	μ_θ
A1	90	5.6	108.11	25.51	4.55
A2	107	5	132.08	22.21	4.44
A3	170	5	195.95	21	4.2
A4	180	5.8	234.67	14.83	2.55
A5	255	5.8	274.23	10.69	1.84
A6	270	6	329.02	10.63	1.77

Table 6. Elastic and ultimate moments-deformations and ductility factors (Models B1 to B6)

Models	Elastic state		Ultimate state		Ductility factor
	M_y (***kN.m***)	θ_y $(Mrad)$	M_u (***kN.m***)	θ_u $(Mrad)$	μ_θ
B1	106	3.8	126.24	26.49	6.97
B2	121	3.8	146.16	25.16	6.62
B3	170	5	195.95	21	4.2
B4	195	5	246.74	18.41	3.68
B5	185	4.5	246.03	11.25	2.50
B6	225	5.2	271.26	9.34	1.79

In the case of models A1 to A6 shown in Fig. 11, a reduction in the column-to-beam strength ratio (CBSR) from 2.98 to 0.82 leads to a decrease in the rotational ductility factor from 4.55 to 1.77. Similarly, as shown in Fig. 12, for models B1 to B6, a decrease in the column-to-beam strength ratio (CBSR) from 3.01 to 0.85 results also in a reduction of the rotational ductility factor from 6.97 to 1.79.

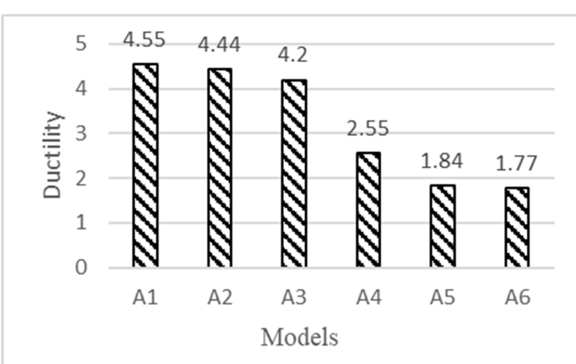


Fig. 11. Rotational ductility factor of beam-to-column depth ratio

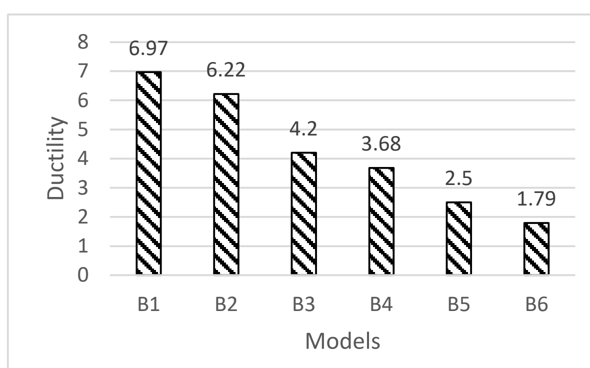


Fig. 12. Rotational ductility factor beam longitudinal reinforcement ratio

The reduction in the column-to-beam strength ratio (CBSR) signifies a transition to a "weak column-strong beam" configuration, making the column relatively weaker compared to the beam. As a result of this design approach, higher forces may be transmitted to the joints during seismic events. If the joints are not adequately designed to handle these increased forces, they may become susceptible to shear failure. According to Eurocode 8 (2004), the horizontal shear force acting on the concrete core of the beam-column joints can be evaluated for exterior beam-column joints as follows:

$$V_{jhd} = \gamma_{RD}.A_{s1}.f_{yd} - V_c \tag{5}$$

A_{s1} : is the area of the beam top reinforcement;
V_c : is the shear force in the column above the joint, from the analysis in the seismic design situation;
f_{yd} : yield strength of steel;
γ_{RD} : is a factor to account for overstrength due to steel strain-hardening and should be not less than 1.2.

Figs. (13-14) represent the relationship between the shear force applied to a reinforced concrete beam-column joint and the strength ratio between the column and the beam for models (A1 to A6) and models (B1 to B6) respectively. These curves show how the strength ratio influences the joints capacity to resist shear forces. As the column-to-beam strength ratio increases, the curve exhibits a gradual decrease in shear force, indicating an enhanced capacity of the beam-column joint to resist shear forces. It is found in **Fig. 13** that the joint shear force is reduced by about 54% when the column-to-beam strength ratio increases from 0.82 to 2.98.

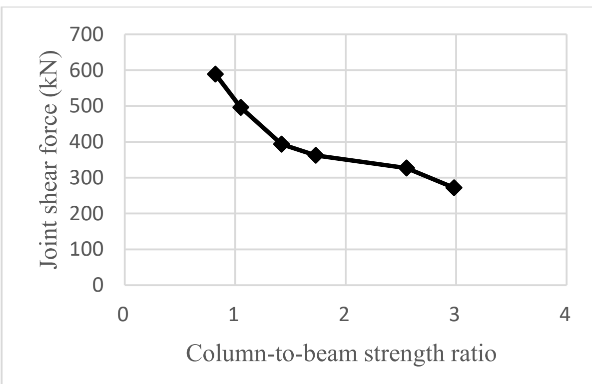


Fig. 13. Relationship between the shear force and column-to-beam strength ratio for models (A1 to A6)

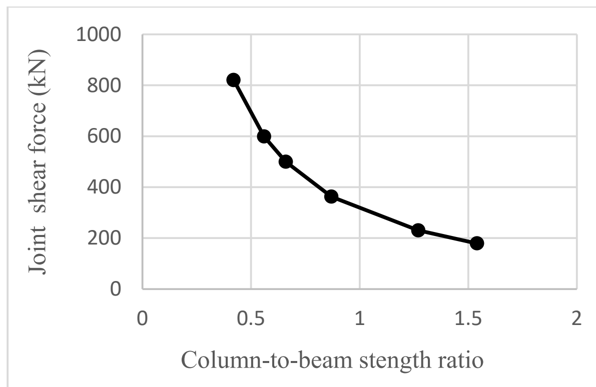


Fig. 14. Relationship between the shear force and column-to-beam strength ratio for models (B1 to B6)

Fig. 14 illustrates a decrease of approximately 78% in the joint shear force as the column-to-beam strength ratio increases from 0.85 to 3.01. **Figs. (15-16)** depict curves illustrating the maximum deformations at the end of the analysis, recorded both in the joint panel and in the beam at the column juncture corresponding to various values of the column-to-beam strength ratio (CBSR) for models (A2, A4, A5, A6) and (B1 to B6) respectively.

Regarding models A2, A4, A5, and A6, when the ratio between the heights of the beam and column increases from 1.0 to 1.57, the value of (CBSR) decreases from 2.55 to 0.82. In this case, we observe an increase in strains within the joint panel and a decrease in the beam near the column juncture. When the value of (CBSR) is equal to 1.05, the strain in the beam at the column juncture is equivalent to the strain in the joint panel. However, when this ratio drops to 0.82, the strain in the joint panel becomes two and a half times greater than that in the beam, indicating an increased concentration of shear strains in this area. This phenomenon can lead to the plastic hinges relocating from the beam's end to the joint panel, potentially reducing the strength and ductility of the connection. As a consequence, premature joint failure may occur, especially under seismic loads.

For models B1 through B6, when the longitudinal reinforcement ratio of the beam varies from 0.52 to 1.94, a gradual reduction in the corresponding (CBSR) value from 3.01 to 0.85. In this scenario, there is a noticeable increase in strains within the joint panel, coupled with a simultaneous decrease at the beam's end. When the column-to-beam strength ratio (CBSR) is approximately 1.2, both the beam's end and the joint panel experience equal strains. However, as the CBSR decreases to 0.85, the strain in the joint panel surpasses that in the beam, exhibiting a three and a half times increase in shear strains at this location. Consequently, similar to the effect of the beam-to-column depth ratio, this shift in strains may cause the plastic hinges to relocate from the beam's end to the joint panel, thereby diminishing the connection's strength and ductility. This, in turn, elevates the risk of premature joint failure, particularly under seismic loads.

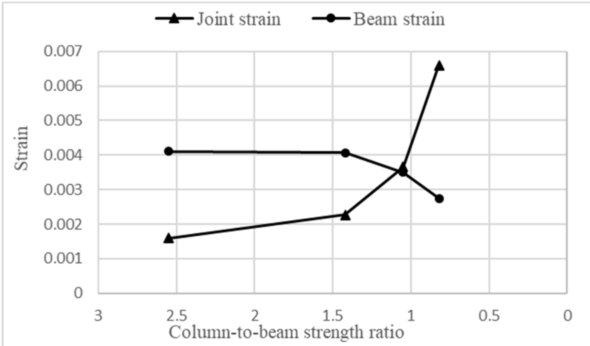


Fig. 15. Visualization of shear strain distribution across the joints for models (A2, A4, A5, A6)

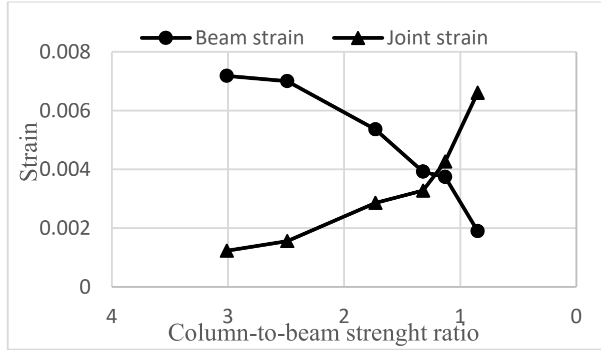
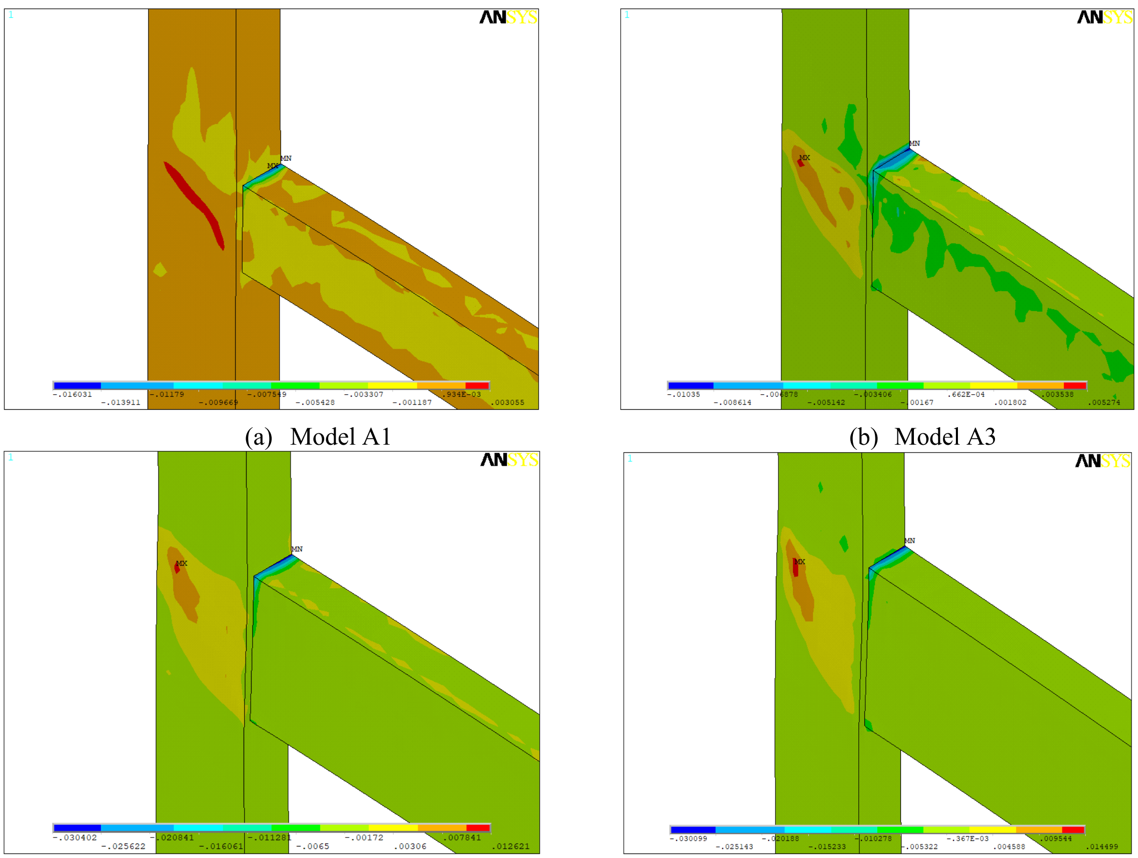


Fig. 16. Visualization of shear strain distribution across the joints for models B1 to B6

The representation of shear strain variations in the joint regions for models (A1, A3, A5, A6) and (B1, B3, B5, B6) are illustrated in **Figs. (17-18),** respectively. According to **Fig. 17**, model A1 exhibit a failure mode characterized by the formation of a plastic hinge in the beam at the column juncture. For models A3, A5 et A6, a relocation of plastic hinges from the end of the beam towards the joint panel. This relocation often results in a change in the failure mode, shifting from beam failure to joint failure. The joint panel diagonal exhibits a higher concentration of shear strain which is generally undesirable, as this can lead to premature joint failure and compromise the stability and safety of the structure.



(a) Model A1

(b) Model A3

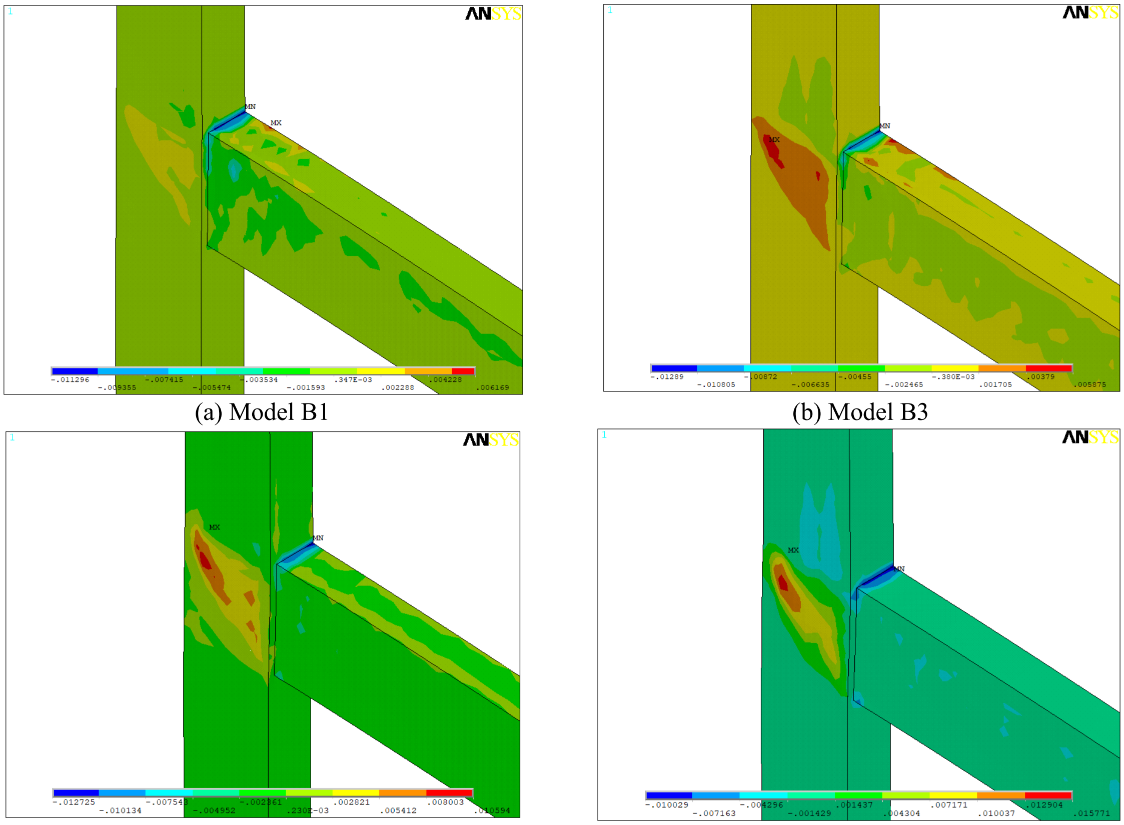
(c) Model A5

(d) Model A6

Fig. 17. Shear strain variations in the joint regions

In model B1, as shown in Fig. 18, the shear strain in the diagonal panel is so small which indicates that the joint's failure is primarily due to the flexural bending of the beam. By increasing the longitudinal reinforcement ratio of the beams for models (B3, B5 and B6), a notable shift of plastic hinges from the beam's end towards the joint panel becomes evident. This shift often alters the failure mode, transitioning from beam failure to joint failure. Diagonal joint shear failure, marked by the initiation of failure along a diagonal path, has been consistently identified by other researchers (Park & Mosalam 2012a;

Niroomandi et al., 2014; Tsonos, 2007). This critical failure location primarily manifests at the left corner of the joint panel, where the maximum joint shear strain is observed, which is generally undesirable as it can lead to premature joint failure.



(a) Model B1 (b) Model B3

(c) Model B5 (d) Model B6

Fig. 18. Shear strain variations in the joint regions

5. Conclusions and recommendations

In earthquake-prone areas, beam-column joints play an essential role in RC moment resisting frames. The sections of beams and columns near these joints are of utmost importance. During strong earthquake events, these sections experience significant bending moments and shear forces. Brittle failure modes, like joint shear failure or anchorage slip of beam longitudinal bars, significantly reduce overall ductility. This study examined the performance of exterior RC beam-column joints under monotonic loading using a three-dimensional nonlinear finite element model. The results were validated against experimental data reported by other researchers (Mahini & Ronagh, 2011), showing similar trends and accurately capturing the non-linear response of specimens up to peak load.

To enhance the understanding of the impact of the column-to-beam strength ratio (CBSR) on the shear strength and seismic performance of RC exterior beam-column joints, a parametric study was conducted. The study involved changing the height and longitudinal reinforcement of the beams. These two parameters were chosen due to their significance in the behavior of RC beam-column joints.

After conducting the study, the following results can be presented:

- The behaviour of beam-column joints is more influenced by the change in geometry of beam and the amount beam reinforcement. Increasing the height or longitudinal reinforcement of the beam, while keeping the dimensions and reinforcement of the column's constant, shifts plastic hinges from the beam's end to the joint panel. This change modifies the failure mode from beam failure to joint failure. This relocation leads to a higher concentration of shear strain along the joint panel diagonal, posing a risk of premature joint failure and compromising the structure's stability and safety.
- The shear resistance capacity of the RC beam-column joint changes according to the resistance ratio between beam and column. An increase in the column-to-beam strength ratio generally results in an improvement in shear strength. A lower column-to-beam strength ratio (CBSR) creates a "weak column-strong beam" condition, making the column relatively weaker than the beam. This can lead to higher stresses on the column and increase the risk of brittle failure. Additionally, reduced CBSR may limit the system's ability to withstand significant deformations without failure, resulting in restricted force and deformation redistribution, ultimately reducing ductility.

- It was found through the present study that the value of the column-to-beam strength ratio specified in the Algerian seismic code - RPA99/version2003 is insufficient to guarantee the principle of "strong column-weak beam and to avoid the formation of a story mechanism under design seismic loading. The results also showed that the optimal failure mechanism with plastic hinges in the beams near their adjacent column can only be achieved if the column-to beam strength ratio is relatively large (on the order of three). Therefore, as this is impractical in most cases and resulting in over-design and unnecessary construction costs, a strength ratio of between 1.3 and 1.4 is recommended. Thus, an intermediate mechanism, where plastic hinges form simultaneously in the beams and columns, can be considered, and the columns should be designed accordingly.

RC beam-column joints vary in configurations and stresses, necessitating specific beam-to-column strength ratio values. Seismic codes should offer more flexibility to adjust the CBSR according to the structure's specific characteristics. To better understand the influence of the column-to-beam strength ratio (CBSR) on seismic performance of RC beam-to-column junctions, extensive experimental and numerical analyses are required taking into account the seismic hazard level in the region, the structural system being used, the number and heights of storeys and the design objectives.

Declaration of Competing 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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