Effect of beam-column joints flexibility on the seismic response of setback RC buildings designed according to the Algerian seismic code

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ABSTRACT. The Algerian seismic code assumes that the beam-column joints in monolithic reinforced concrete (RC) buildings are fully rigid. However, many experiments have proven the existence of relative rotations in these connections, and then the presence of relative transfer of bending moment. The present work aims to investigate the effect of beam-column joints modelling on the global seismic behaviour of reinforced concrete (RC) moment-resisting frame buildings, designed according to the Algerian seismic code recommendations. To consider the nonlinear deformation of the connections, an analytical model developed recently is used. This model includes two important deformation mechanisms; the first one covers the slippage of the continuous reinforcement within the column, whereas the second involves slippage caused by the creation of bending cracks at the extremities of the beams. Three multi-storey RC frames with different setback geometry, including a reference frame, are studied considering the connections as rigid/deformable. The nonlinear static procedure or pushover analysis is used to perform a nonlinear analysis of the studied structures and the results in terms of capacity curve, target displacement, storey drift, storey stiffness and the response reduction factor are presented. The results show

the necessity of considering the beam-column connection flexibility when modelling this type of buildings.

KEYWORDS. Beam-column joints; pushover analysis; RC buildings; setback buildings; seismic behaviour.

INTRODUCTION

Building construction in Algeria has witnessed a remarkable development in recent years, and Reinforced Concrete (RC) [1–4] has become the most widely used material in modern construction. The northern part of the country is considered as a high seismicity area, which prompted engineers to choose stiff systems in order to reduce the impact of earthquakes on buildings. The most commonly used seismic system is the RC shear wall system, which is considered as an expensive solution. Moment resisting frame (MRF) system is not preferred according to the Algerian seismic code RPA [5]. In effect, the use of MRF systems is limited to certain conditions. However, a set of seismic codes [6–8] allows the use of this system due to its ability to dissipate seismic energy without losing its resistance.

In conventional design, beam-column connections are considered rigid in MRF systems. However, there is limited relative rotation between beam and column produced by the effect of beam reinforcement bars' slippage [9–13]. This flexibility is not taken into account when modelling these types of structures and, therefore, their responses are underestimated.

Several studies have been conducted on this topic. In an experimental cyclic test, Geradin and Negro [14] observed that the response of the structure (a 4-storey building designed according to EC8 [6]) was in fact controlled by their beam-column joints deformations rather than the awaited destruction mechanism related to the development of flexural plastic hinges at beams and columns ends. Furthermore, Ferreira [15] and Alva [16] in their studies confirmed the presence of relative rotations between beams and columns.

Several investigations were carried out to numerically model the beam-column connections. Rotational springs were employed by Filippou et al. [17,18] and Mergos et Kappos [19] to model the slippage of reinforcing bars. Other research, on the other hand, employed extra springs to reflect the influence of joint distortion [9,20]. Even though these models are more attractive because of their simplicity and low computation complexity, they still present difficulties in terms of parameters calibration.

Paultre et al. [21] used a tri-linear model to depict the reinforcement slippage moment-rotation relationship inside the joint. A simplified bond stress distribution was employed by the authors in order to determine the fixed end rotations for both elastic and yielding phases. Using the same simplification, Sezen and Setzler [22] devised a new model, which takes into account the slippage of reinforcement bars for both column-foundation and beam-column joints. In their research, the authors also considered the axial strain of the bars in the joint.

Another analytical model was developed by Kwak and Kim [23]. The model reduces the flexural stiffness in the plastic hinges at the beam-ends. Here, besides to the rotations produced by the slippage of reinforcement bars, the flexural cracks at the beam-column interface were also considered.

In 2012, Birely et al. [9] proposed a nonlinear model to reflect the nonlinear behaviour of RC buildings. Two springs were used to consider the beam yielding and the beam-column connection. Later, Alva and El Debs [10] developed a promising model to expect the moment-rotation relationship of RC beam-column joints. The new model considers the relative slip of the flexural reinforcement and the slip introduced by flexural cracking of the beam-ends. Good agreements were obtained when comparing the moment-rotation curves given by the developed model to the experimental curves.

Recently, Santos et al. [12] used the analytical model developed by Alva and El Debs [10] to measure the influence of beam-column joint stiffness on the structural analysis of RC buildings. They found that the joints' flexibility causes a redistribution of internal forces in the structure, which alters the second-order effects. Also, Alva [24] has confirmed these results and proved that the consideration of the bending deformability of the connections leads to significantly better results than the fully rigid consideration.

Most of the above-mentioned studies have examined the behaviour of tall, regular RC buildings. The influence of beam-column connections on the seismic behaviour of RC irregular buildings [25] still gets little attention. One of the most popular irregular buildings is setback buildings, when an abrupt discontinuity in the vertical geometry of the building exists [26]. These discontinuities affect considerably the seismic performance of this type of building.

In the present work, the influence of the beam-column connections in RC buildings is studied. Three RC buildings (regular and irregular) designed according to the Seismic Algerian code, RPA99 v2003 [5] are selected. Alva and El Debs [10] analytical model is used herein to construct the moment-rotation relationship of the beam-column connections. The conventional pushover analysis [27–29] is performed in this work to assess the seismic nonlinear behaviour of these buildings. The results in terms of total drift, storey drift, and shear storey are calculated and compared to the seismic code limits. A secondary aim was to test the validity of the response reduction factor (R) value recommended in RPA99 v2003 [5] and other codes [6,8]for RC MRFs.

BEAM-COLUMN CONNECTION MODELLING

The modelling of RC buildings is a difficult step in the design process. To make this task easier, the engineers set several hypotheses that reduce the number of parameters to be considered. Among the simplifications, the RC beam-column connections in the construction are considered rigid. The experiences of previous earthquakes have shown the flexibility of these joints and then their impact in determining the structural damage. As a result, to address the seismic performance of new or existing RC frames correctly, engineers must use models that can estimate the behaviour of the beam-column connections with acceptable accuracy. The analytical model proposed by Alva and El Debs [10] assumes that the rotations between the beam and the column are produced by two mechanisms (Fig. 1). Mechanism A, represents the relative rotations created by the slippage of the beam reinforcement inside the joint, and Mechanism B, which is the relative rotations produced by the cumulative effect of local slips caused by the cracks opening at the beam ends.
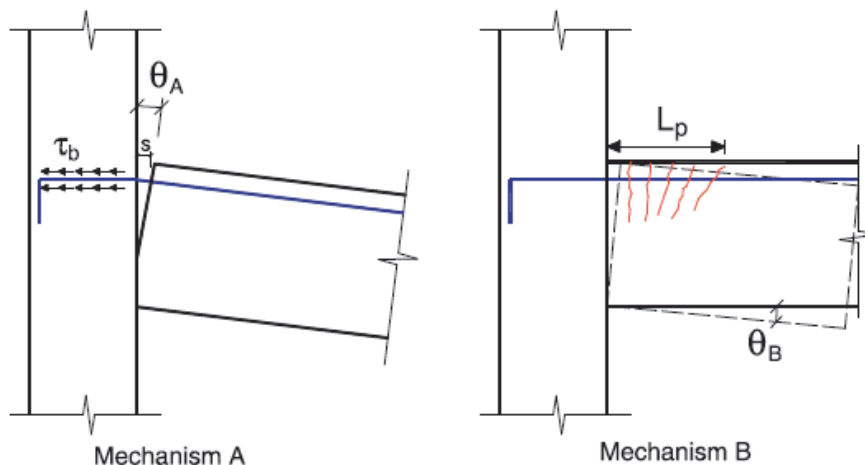

Figure 1: The two mechanisms A and B in beam-column connections [10].

The relative rotation in terms of bending moment and curvature at the beam*column joint can obtained by [10]:

$$
\begin{cases}
\theta = C_1 M^2 + C_2\left(\dfrac{1}{r}\right) & \text{for } M \leq M_y \\[2mm]
\theta = C_1 M_y^2 + C_2\left(\dfrac{1}{r}\right) & \text{for } M_y < M \leq M_u
\end{cases}
\tag{1}
$$

where, M_y is the yielding moment and M_u is the ultimate moment of the beam section (see Fig. 2). Tab. 1 shows the different parameters used in this model.
Comparison of experimental force–displacement curves from the proposed model with experimental data (Fig. 3) demonstrated the analytical model's capacity to predict the effects of flexural reinforcement slippage [10]. According to the authors, the parameters for the proposed model do not require calibration, and the user needs only to define some information like the geometry of cross sections and mechanical properties of used materials. It is crucial to note that the suggested model does not take into account the impact of shear distortion of the joint panel or shear forces at the beam end.

Parameter	Description	Expression
C_1	Constant related with mechanism A	$C_1 = \dfrac{\varnothing}{8 \cdot E_s \cdot \tau_{by} \cdot (d-x) \cdot A_s^2 \cdot z^2}$
C_2	Constant related with mechanism B	$C_2 = 0.5 \cdot (L_p + S_R)$
$(1/r)$	Curvature of the beam section	EC 2 [30]
$\varnothing$	Diameter of longitudinal bending reinforcement of the beam	–
E_s	Elasticity modulus of the steel	–
τ_{by}	Bond stress in the elastic range	$\tau_{by} = \sqrt{f_c}$
f_c	Concrete compressive strength	–
d	Effective depth of the beam	$d = h-c$
h	Depth of the beam	–
c	Concrete cover	EC 2 [30]
x	Neutral axes depth (cracked section)	–
A_s	Area of longitudinal bending reinforcement of the beam	–
z	Lever arm between the tension and compression forces in the beam section	$z = d - \dfrac{x}{3}$
L_p	Connection region length	$L_p \approx d$
S_R	Crack spacing	$S_R = k_3 \cdot c + \dfrac{k_1 \cdot k_2 \cdot k_4 \cdot \varnothing}{\varrho_{eff}}$
k_1	Constant	$k_1 = 0.8$ *(high-bond bars)*
k_2	Constant	*0.5*
k_3	Constant	*3.4*
k_4	Constant	*0.425*
ϱ_{eff}	Reinforcement percentage ratio	$\varrho_{eff} = \dfrac{A_s}{A_{c,eff}}$
$A_{c,eff}$	Effective tension aria of concrete	EC 2 [30]

Table 1: Parameters of the moment-rotation relationship developed by Alva and El Debs [10,12].
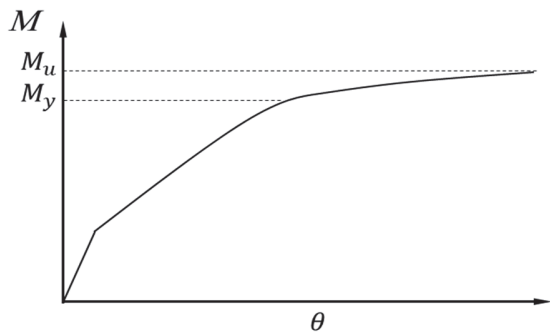
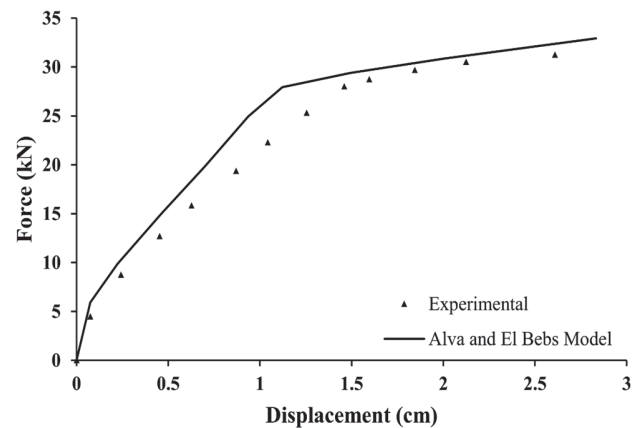Figure 2: The proposed moment-rotation relationship [10].

Figure 3: Experimental and numerical simulation [10].

CASE STUDY

Structures description

To investigate the joint behaviour and its effect on the overall performance of RC frames, three 6-storey structures were selected from the literature [31]. The studied frames are part of three RC structures with the same plan view and are assumed to be the central frames. The frames are equally spaced by 4 m (Figs. 4 and 5). Two frames are irregular in elevation (with setback), designated by F663 and F661, and a third F666 is regular, taken as a reference frame. The storey height is 4 m for the first storey and 3.0 m for the rest of the stories, with 3 bays of 5.0 m for all the frames. They are designed according to the Algerian seismic code RPA99V2003 [5]. The structures are located in a high seismicity zone (Zone III). The soil class is S3 (soft soil) and with viscous damping equal to 7%. Concerning the dead and live loads: They are equal to 5.1 kN/m² and 1.5 kN/m² for the current floors and 5.8 kN/m² and 1 kN/m² for the roof, respectively. The dimensions of the cross-sections, mechanical properties such as the concrete compressive strength f_c, the steel yield stress f_y and the concrete and steel Young's (elasticity) modulus (E_c, E_s) are presented in Tab. 2.
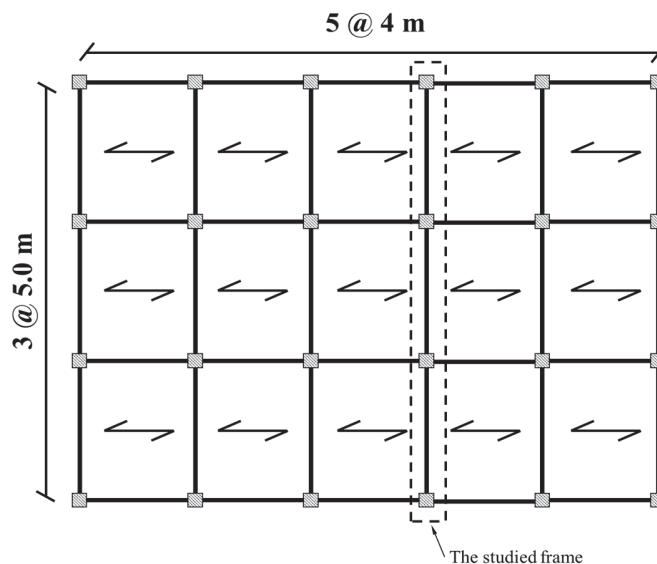

Figure 4: A plan view of the buildings that comprise the three frames under study.
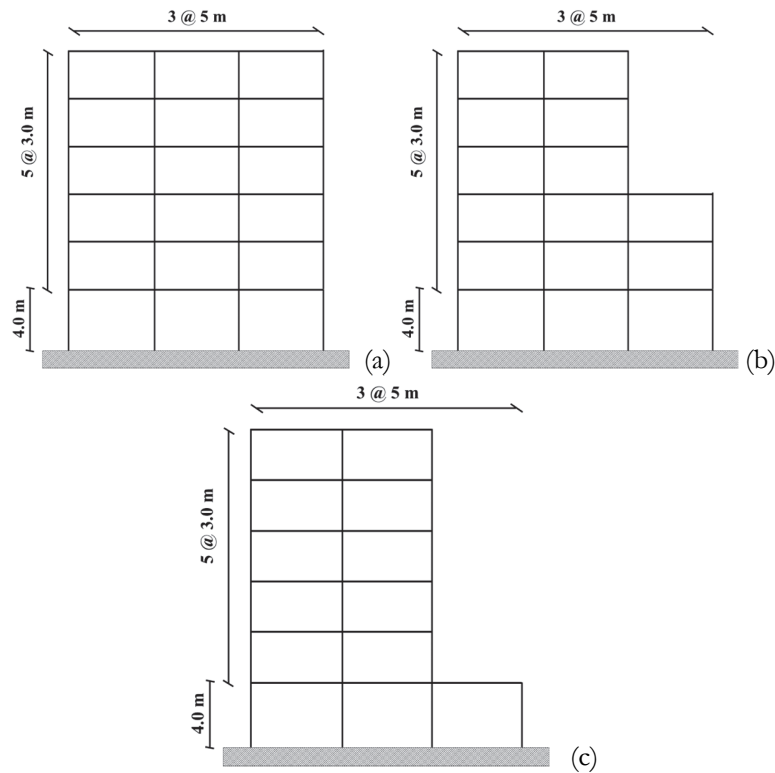
Figure 5: Geometric configurations of the RC moment-resisting frames; a) F666, b) F663, c) F661.

Storey level	Beams			Columns		Mechanical properties
	$b \times h$ cm²	Steel bars		$a \times a$ cm²	Steel bars	
		Top layer	Bottom layer			
F666 frame						
1		$4\Phi20 + 4\Phi16$	$4\Phi16 + 4\Phi14$	50x50	$12\Phi20$	
2		$4\Phi20 + 4\Phi14$	$4\Phi14 + 4\Phi14$	50x50	$12\Phi20$	
3		$4\Phi16 + 4\Phi16$	$4\Phi14 + 4\Phi12$	45x45	$4\Phi20 + 8\Phi16$	
4	40x40	$4\Phi16 + 4\Phi12$	$4\Phi14 + 2\Phi12$	45x45	$4\Phi20 + 8\Phi16$	
5		$4\Phi12 + 4\Phi12$	$4\Phi12$	40x40	$12\Phi16$	
6		$4\Phi12 + 2\Phi12$	$4\Phi12$	40x40	$12\Phi16$	
F663 frame						
1		$6\Phi20 + 2\Phi14$	$4\Phi16 + 4\Phi14$		$12\Phi25$	
2		$4\Phi20 + 4\Phi16$	$4\Phi14 + 4\Phi14$		$12\Phi25$	f_{c28}=25 MPa
3		$4\Phi20 + 4\Phi12$	$4\Phi16 + 2\Phi12$		$12\Phi20$	f_y=500 MPa
4	40x40	$4\Phi20 + 2\Phi16$	$4\Phi14 + 2\Phi14$	45x45	$12\Phi20$	E_c=32.16 GPa
5		$4\Phi16 + 2\Phi16$	$4\Phi14$		$4\Phi20 + 8\Phi16$	E_S=210 GPa
6		$4\Phi16$	$4\Phi12$		$4\Phi20 + 8\Phi16$	
F661 frame						
1		$4\Phi20 + 4\Phi16$	$4\Phi16 + 4\Phi14$		$4\Phi25 + 8\Phi20$	
2		$4\Phi20 + 4\Phi20$	$4\Phi20 + 4\Phi12$		$4\Phi25 + 8\Phi20$	
3		$4\Phi20 + 4\Phi16$	$4\Phi16 + 4\Phi12$		$12\Phi20$	
4	40x40	$4\Phi16 + 4\Phi16$	$4\Phi14 + 4\Phi10$	45x45	$12\Phi20$	
5		$4\Phi16$	$4\Phi14$		$12\Phi16$	
6		$4\Phi12 + 2\Phi12$	$4\Phi12$		$12\Phi16$	

Table 2: Cross-Section and reinforcement details of the beams and columns.

Seismic loads

To perform the nonlinear static pushover analysis and to get the performance point (the target displacement), the seismic demand is represented by the response spectrum in Fig. 6. The three buildings are assumed to be located in a zone of high seismicity, namely zone III, and the site is of type S_3 (soft soil) according to the Algerian seismic code RPA99 v2003 [5].
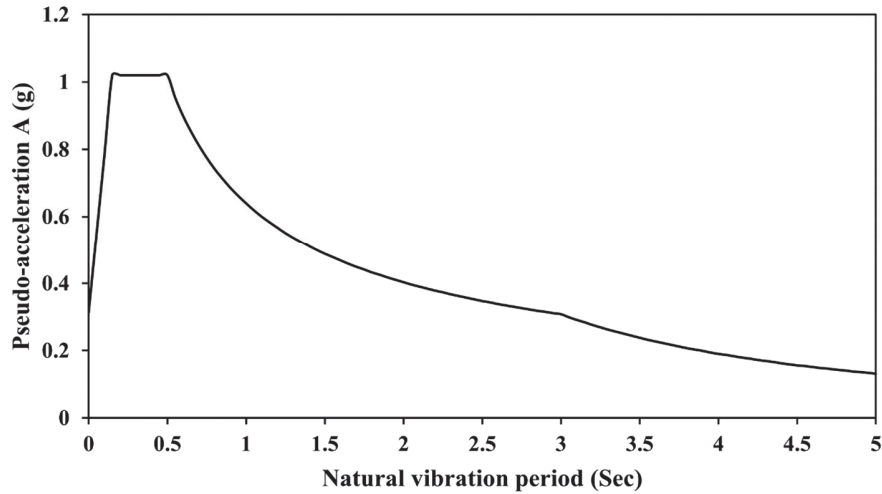

Figure 6: Pseudo-acceleration response spectrum of the designed buildings.

Modelling issues

The modal analysis and the conventional pushover analysis were carried out using the computer program ETABS [32]. Elastic elements coupled with concentrated plastic hinges at the ends of the structural elements were adopted, where two rotational springs are combined in series to represent the inelastic deformations of the beams and the joints [9]. One spring controls the beam response, and the second simulates the joint response. Fig. 7 shows a model of a typical beam with a dual-hinge beam-column element.

The Alva and El Debs' [10] model was used to simulate the joint flexibility (the first rotational spring). The characteristics of the plastic hinges at the ends of beams (second rotational spring) and columns are defined according to FEMA-356 [33]. Fig. 8 shows the force-deformation relationship model used to model the hinges. The P-Δ effect is also included in this study when performing the nonlinear static analysis, FEMA 440 [34] procedure is used to evaluate the performance point of the studied frames (Fig. 9).
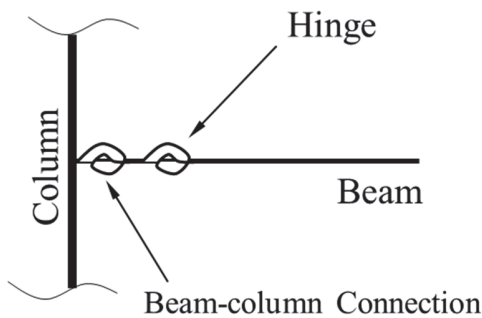

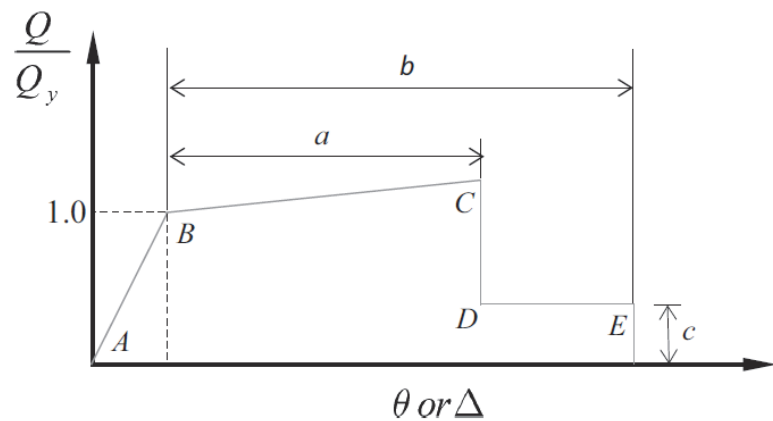Figure 7: The nonlinear model of the RC joints [9].

Figure 8: Generalized force deformation relationship for plastic hinges [33].
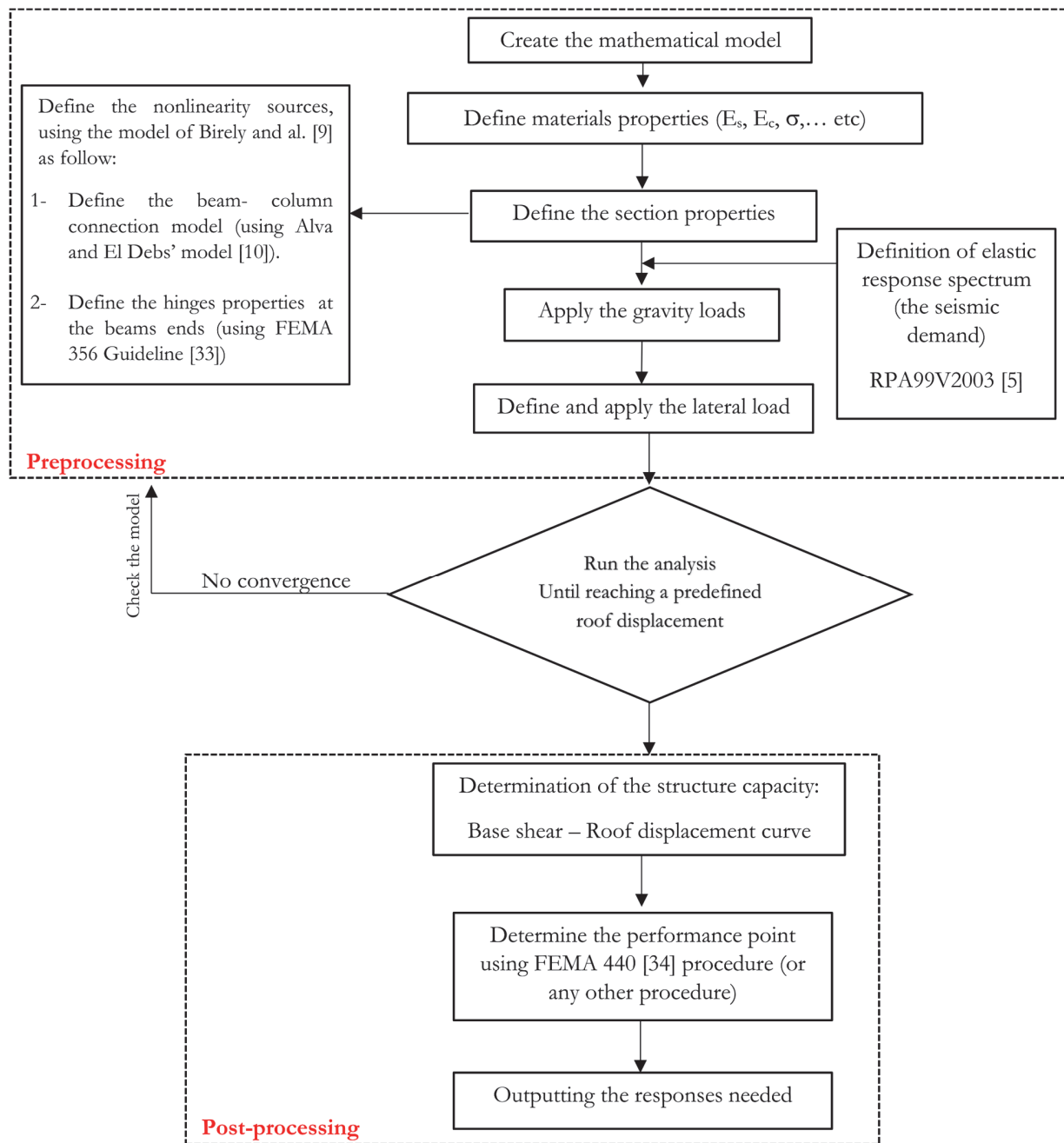
Create the mathematical model

Define materials properties (E_s, E_c, σ,… etc)

Define the nonlinearity sources, using the model of Birely and al. [9] as follow:

1- Define the beam- column connection model (using Alva and El Debs' model [10]).

2- Define the hinges properties at the beams ends (using FEMA 356 Guideline [33])

Define the section properties

Apply the gravity loads

Definition of elastic response spectrum (the seismic demand)

RPA99V2003 [5]

Define and apply the lateral load

Preprocessing

Check the model

No convergence

Run the analysis Until reaching a predefined roof displacement

Determination of the structure capacity:

Base shear – Roof displacement curve

Determine the performance point using FEMA 440 [34] procedure (or any other procedure)

Outputting the responses needed

Post-processing

Figure 9: Flowchart of pushover analysis.

RESULTS AND DISCUSSIONS

Periods and modal mass participating ratios

Tab. 3 presents the periods of the three first modes of vibration of the studied frames. In general, the periods record large values when the joints are flexible compared to the case when the connections are fully rigid, and this is true for all the studied frames. The modal participating mass ratio depicted in Fig. 10 shows that the participation of the first mode decreases if the flexibility of the connection is considered. However, the contribution of the higher modes increases compared to the rigid case. For the F661 setback frame, the modal participating mass ratio reaches 15% and 7% for the second and third modes, respectively.

Frame	Joints	Periods (Sec)		
		Mode 1	Mode 2	Mode 3
F666	Rigid	0.74	0.24	0.13
	Flexible	0.98	0.32	0.16
F663	Rigid	0.68	0.25	0.13
	Flexible	0.86	0.32	0.15
F661	Rigid	0.71	0.23	0.13
	Flexible	0.91	0.30	0.16

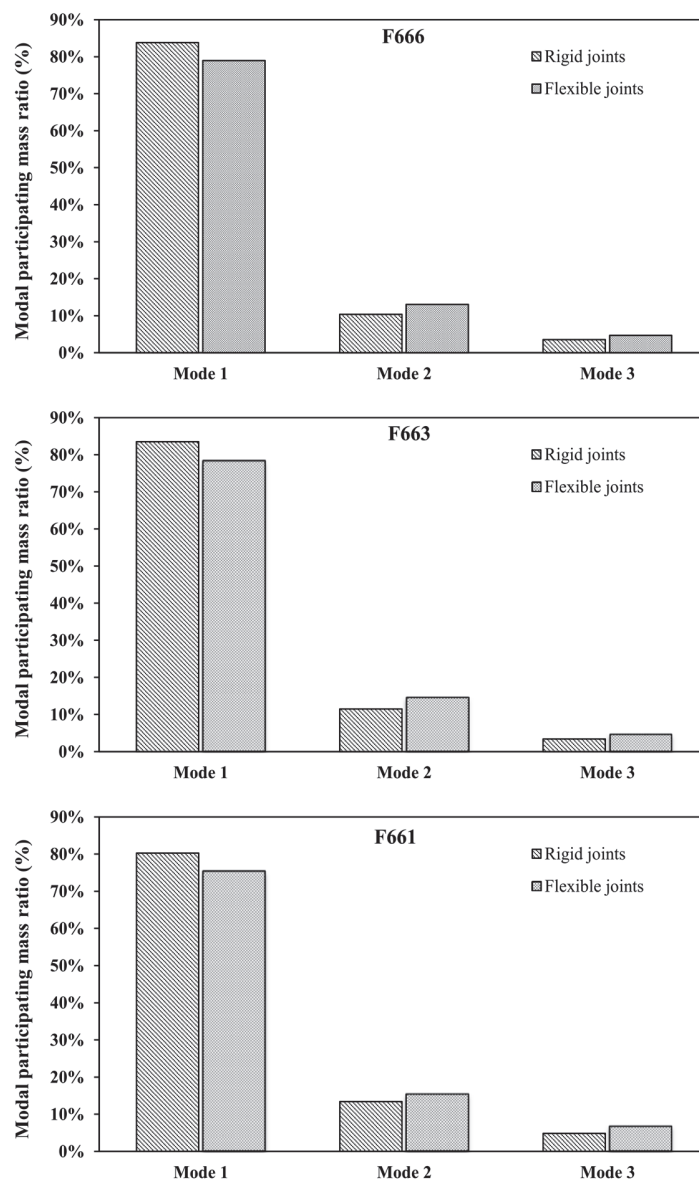Table 3: Periods of the first three modes of the studied structures.

Figure 10: The modal participating mass ratios of the studied buildings.

Capacity curves

The capacity curves of the studied frames are obtained using the pushover analysis to demonstrate the effect of joint flexibility and elevation irregularity on the global structural response (Fig. 11). The capacity curve (or pushover curve) is a relationship between the base shear and the roof displacement. From these curves, one can observe that the flexible joints decrease the strength of the structure for the three studied frames. The first yielding point is marked in each curve for each case. The appearance of the first yield in cases of flexible joints before rigid ones is remarkable. Moreover, after the idealizations of the curves, the global stiffness is decreased by about 50% when the joint flexibility is considered for the three studied frames. Also, it is clear from this figure that the maximum base shear for the setback frames is less than the regular one.
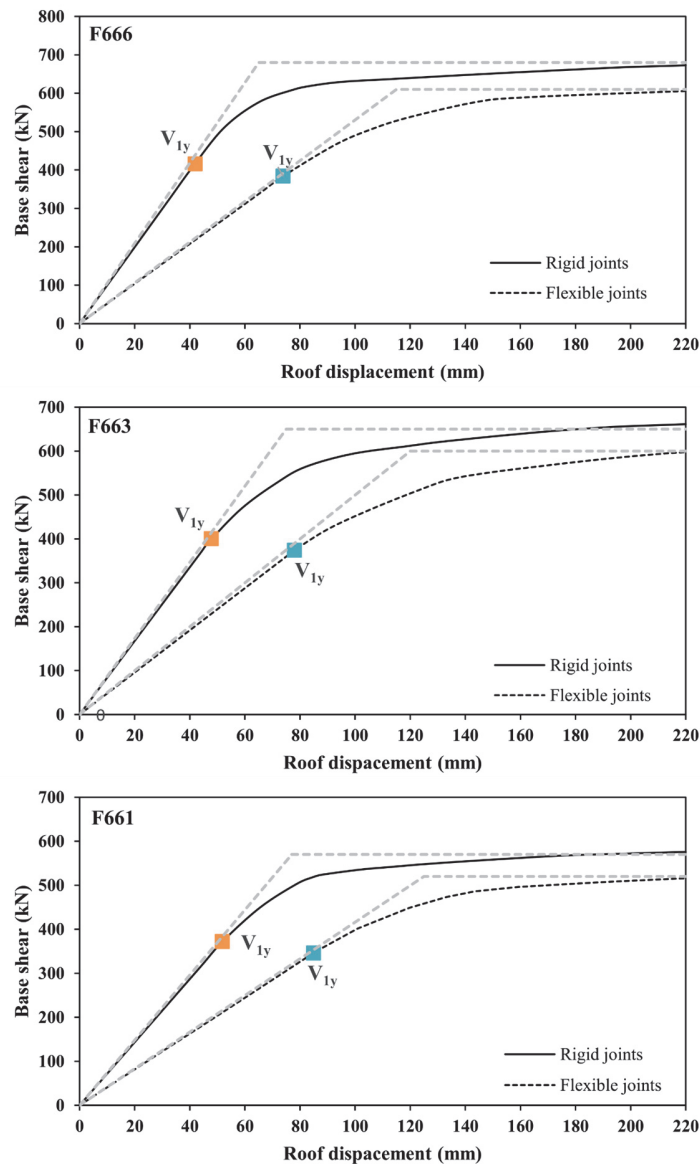

Figure 11: Capacity curves of the studied frames.

Performance point

The performance point is the intersection between the capacity curve (Fig. 10) and the demand curve (Fig. 6). This point is calculated according to FEMA 440 [34]. The two coordinates of this point are the target displacement and the demand base

shear (Figs. 12 and 13). Fig. 12 illustrates the values of the target displacement for the studied frames. The frames with flexible joints give larger values of the target displacement compared to the frames with fully rigid connections. The largest difference is 43%, recorded in the case of the regular frame (F666). For the setback frames, the difference exceeds 33% and 37% for the F663 and F661 models, respectively.
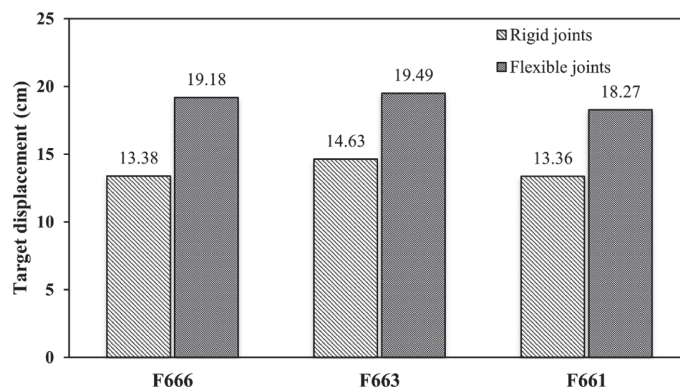

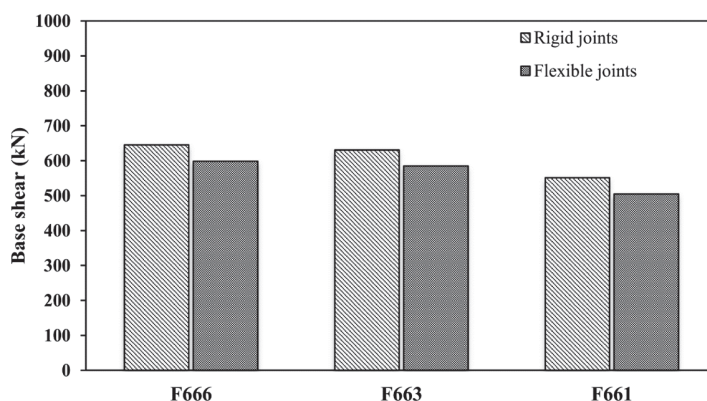Figure 12: The Target displacements of the studied frames.

Figure 13: The demand base shears of the studied frames.

Storey drift ratio

The inter-storey drift ratio (IDR) is the most commonly used parameter for evaluating the structural behaviour under a given seismic load. In this paper, this parameter is calculated to measure the influence of the beam-column connections on the global instability of RC buildings (Fig. 14). The storey drifts in the case when the joints are flexible are greater than those from structures with rigid joints. The values given by considering the connections as rigid remain under the limit value required by the RPA99v2003 code [5] for all the studied frames. However, if the flexibility of the joints is considered, the IDR values exceed the RPA limit (1%) and the design is unsafe. In this case, the beam and column sections must be resized. The Eurocode 8 [6] confirms this conclusion, in which the storey drift limit is the same as the RPA99 v2003. However, for the ASCE 7-16 seismic code [8], the storey drift limit has a large value (2%) and, in this case, both connection types (rigid or flexible) give a safe result.

Storey stiffness

The storey stiffness gives also a clear image of the difference between the two cases of the beam-column connection consideration. Fig. 15 shows the variation of the storey stiffness along the height of the studied frames. It can be observed that the consideration of the flexibility of the connections in the studied RC structures influences the storey stiffness distribution in a remarkable way. The difference can reach 50% for the F666 frame (1st storey), and 40% for the F661 frame.
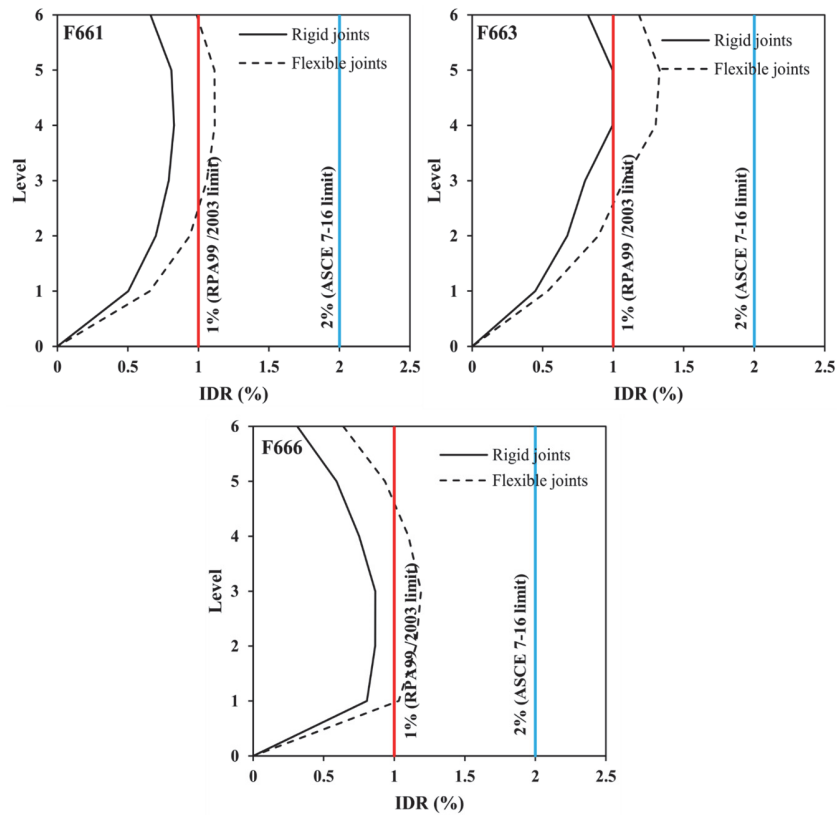
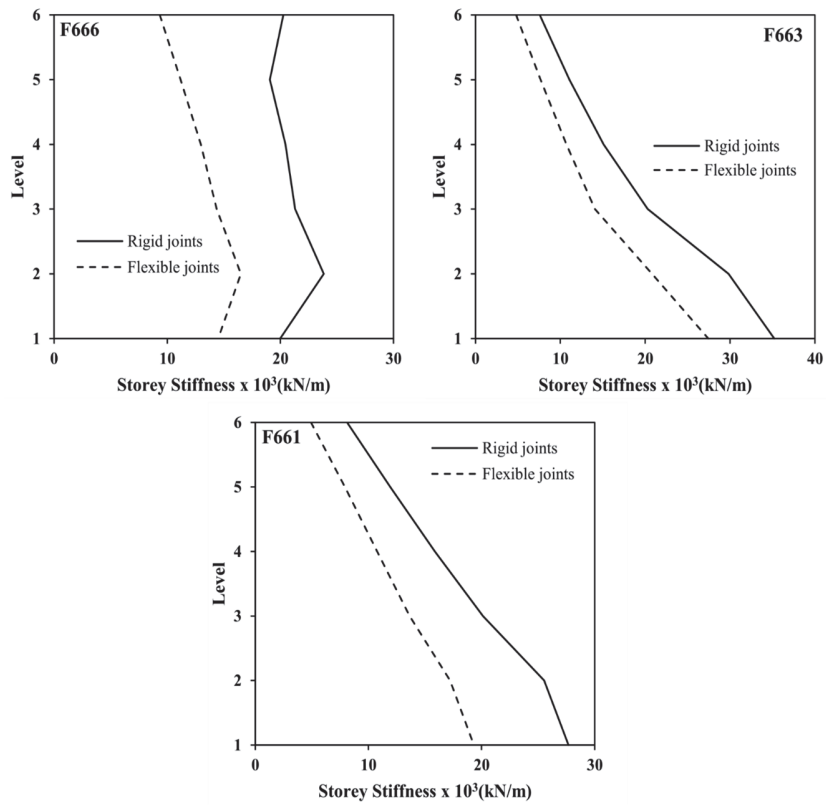Figure 14: The storey drift ratios of the studied frames.

Figure 15: The storey stiffnesses of the studied frames.

Response reduction factor

The evaluation of the behaviour coefficient (or the reduction factor) R is an essential step for structural seismic design because it plays an important role in controlling the energy capacity dissipated in the inelastic behaviour phase. In this context, the ATC-19 [35] proposed a simplified procedure to estimate the response reduction factor given by Equation (2):

$$R = R_\mu \Omega R_R \qquad (2)$$

where, R_μ is the ductility factor. In this study, R_μ is estimated using the relationship proposed by Newmark and Hall [36]:

$$\begin{cases} R_\mu = 1 & \text{for } T < 0.2 \text{ s} \\ R_\mu = \sqrt{2\mu - 1} & \text{for } 0.2 \text{ s} < T < 0.5 \text{ s} \\ R_\mu = \mu & \text{for } T > 0.5 \text{ s} \end{cases} \qquad (3)$$

In which, μ is the global ductility factor (Fig. 16), and T is the fundamental (first) period of the structure.
Ω Is the overstrength factor defined as the greater strength delivered to the building in comparison to the required strength, it is given by:

$$\Omega = \frac{V_y}{V_d} \qquad (4)$$

V_y is the lateral (yielding limit) capacity of the structure, and V_d is the lateral force considered in the design process. Villani et al. [37] and Peres et al. [38] recommended that the response reduction factor value should be defined by supposing that the design base shear, V_d, is equal to the base shear that would result in the creation of the structure's first plastic hinge, V_{1y}. As a result, Ω is defined as:

$$\Omega = \frac{V_y}{V_{1y}} \qquad (5)$$

R_R is the redundancy factor, which is considered to be 1 (Tab. 2, ATC-19 [35]). Tab. 4 presents the values of the three parameters: Ru, Ω and R_R of the studied cases and the final value of R.
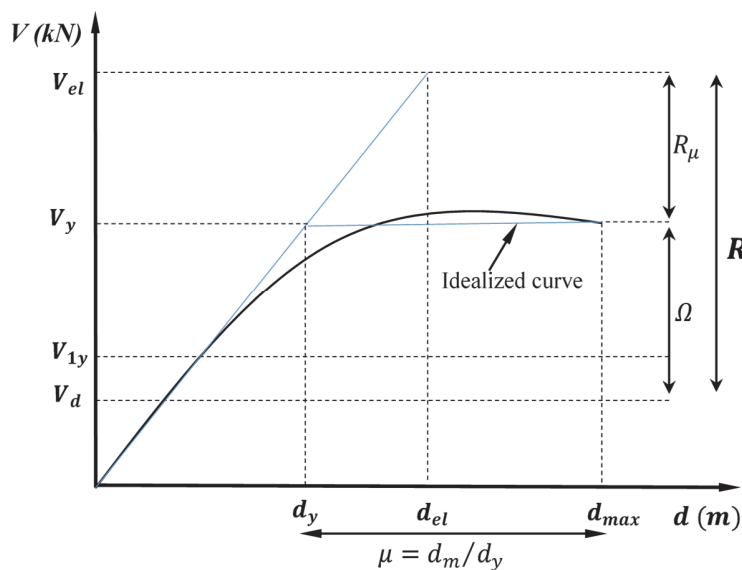

Figure 16: Ductility and overstrength components of the behaviour factor.

Fig. 17 shows the influence of the joint flexibility as well as the irregularity in elevation on the value of the response reduction factor. It is clear that the joint flexibility affects the behaviour and decreases the R-value in all the studied buildings. The setback irregularity affects considerably the response reduction factor when the beam-column joints are flexible, and the F661 gives the lowest value (R = 1.74). In effect, the ductility factor R_u (Tab. 4) for this frame is very small.

For these kinds of structures, the RPA99v2003 [1] recommends a reduction factor of 3.5 (in EC8 and ASCE 7-16, this value is 4 and 5, respectively). When the beam-column joints are assumed fully rigid, the three seismic codes are conservative according to the calculated values (Fig. 17), except for the F666 case, when ASCE 7-16 gives a very close value of R. However, when the joints' flexibility is considered, RPA99v2003, EC8 and ASCE 7-16 overestimate the R factor, resulting in an underestimating of the design base shear.

Building	Joints	R_μ	Ω	R_R	R (calculated)	R RPA99 v2003	R Eurocode 8	R ASCE 7-16
F666	Fully rigid	3.01	1.63	1.00	4.91	3.5	4	5
	Flexible	1.94	1.59	1.00	3.08	3.5	4	5
F663	Fully rigid	3.77	1.62	1.00	6.11	3.5	3.2	5
	Flexible	1.83	1.60	1.00	3.09	3.5	3.2	5
F661	Fully rigid	4.09	1.53	1.00	6.26	3.5	3.2	5
	Flexible	1.16	1.50	1.00	1.74	3.5	3.2	5

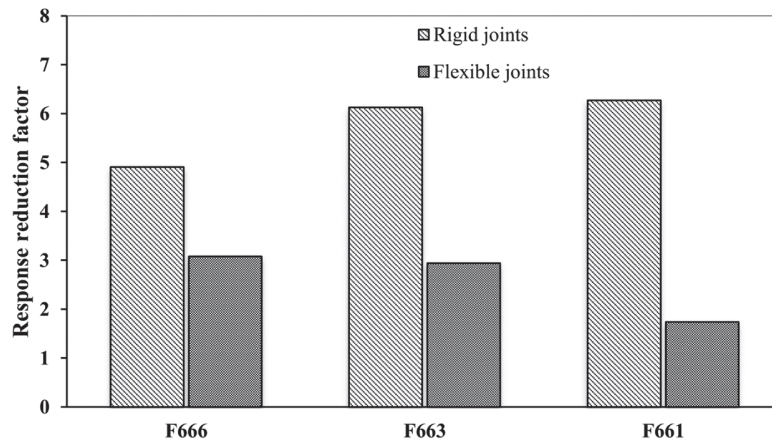Table 4: The R factor values of the buildings under study.

Figure 17: The response reduction factor of the studied buildings.

CONCLUSION

The present paper assesses the seismic performance of RC buildings considering the flexibility of the beam-column connections. A system of two springs is employed to model the nonlinear behaviour of beams. One spring represents the hinges at the extremity of the beam, and the other one represents the nonlinear relationship between the bending moment and the relative rotation at the joints. Three 6-storey building frames designed according to the Seismic Algerian code are studied to measure the influence of the connection flexibility in their seismic responses.

A conventional pushover analysis is performed herein to capture the nonlinear behaviour of the studied frames. After comparing the results of the frames with rigid and flexible connections, the main outcomes are summarised as follows:

- The flexibility of the beam-column connections can affect the modal properties of the buildings under study and gives large values of periods. Moreover, it increases the higher mode effects.
- Based on the results of pushover analysis, the deformable connections always decrease the strength of the structure. Also, they increase the displacement of the storeys.

- The inter-storey drift ratios when the connections are flexible exceed the RPA99 v2003 and EC8 limit (1%) for all the studied structures, and in this case, the design is unsafe. However, for the ASCE 7-16 (storey drift limit equal to 2%), the design has a large margin of safety.
- The setback irregularity in this study does not make any major difference, and the regular structure seems to be the most affected by the flexibility of the joints. This is because the cross-sections and the reinforcement of the structural elements (beams and columns) in setback buildings are overestimated.
- When the beam-column joints are considered fully rigid, the Algerian seismic code is conservative in terms of response reduction factor R. However, when the joints flexibility is taken into account, RPA99v2003 overestimates the R factor, resulting in an underestimating of the design base shear. This conclusion can be generalised for the EC8 and ASCE 7-16 seismic codes.

It is worth noting that the suggested model does not account for shear distortion of the joint panel, shear forces at the beam end and the presence of slabs. Accounting for these factors may increase the effectiveness of the model, making the numerical results relatively close to the experimental responses. Also, the conclusions of the present research were gained for a limited number of frames with different configurations. However, to generalise the outcomes of this work, other analyses should be done for various types of structures.

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