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STUDY ON THE INFLUENCE OF STIFFNESS OF BEAM–COLUMN CONNECTIONS ON THE SEISMIC BEHAVIOR OF COMPOSITE MOMENT RESISTING FRAMES

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Abstract
In the design of steel or composite structures, the connections are usually considered infinitely rigid or perfectly pinned. However, the real behavior of these connections is semi-rigid. Consequently, this semi-rigidity can influence the overall behavior of the composite structures, especially the moment-resisting frames. Seismically, the most critical parameter that characterizes the frames behavior is the response factor R. In this context, the research work consists of studying the semi-rigidity effect of the connections on the behavior of the composite frames by evaluating the response modification factor R by using the Pushover method. To accomplish this task, three types of portal frames of 3, 4 and 5 storeys were analyzed for different degrees of connection (beam-column). An easy and practical solution has been proposed to determine the approximate value of the coefficient behavior R for the composite frames with semi-rigid connections.

Keywords: **Composite frames; Rigid joint; Semi-rigid joint; Pushover analyses; Behavior factor R.**

1. INTRODUCTION

The use of moment-resisting frames with steel-concrete composite floor systems offers several structural and constructive advantages when compared to bare structural steel and other alternatives, such as high strength, high rigidity and ductility, good fire resistance, reduced costs and time of realization. Previous research has indicated that composite frames show favorable behavior from the elastic to the plastic stage due to their improved response characteristics, especially their ductility properties, in terms of seismic performance [1].

In the traditional approach to designing these structures, they are considered either pinned (i.e. absence of any resistance and flexural rigidity), or rigid offering total resistance and infinite rigidity. The models with such ideal connections simplify the analysis procedure, but often they cannot represent the real structural behavior. Therefore, this idealization is not adequate because it is proved by numerous experimental investigations that most steel or composite connections are more or less flexible or semi-rigid [2-11]. Also, the use of semi-rigid connections offers several structural and economic advantages [12].

On the other hand, the dynamic behavior of frames with semi-rigid connections can be significantly different from frames with rigid connections, especially those subjected to strong seismic excitations [13]. This can be explained by the fact that in semi-rigid frames, the internal distribution of forces, the amplitude of lateral displacements and the modes of collapse are a function of the flexibility of the connections, especially in semi-rigid connections [14]. Therefore, it is necessary to study the influence of the semi-rigidity of the connections (composite beam to steel column) on the seismic behavior of composite frames. Generally, this inelastic seismic behavior is controlled by the response factor R [15].

In this context, this paper deals with the effect of the semi-rigidity of the composite beam to steel column connections on the seismic behavior of composite frames through the evaluation of their behavior factor R. To highlight the work carried out, composite moment resisting frames of three, four, and five storeys designed according to the Eurocode 4 [16] and the Algerian seismic code RPA 99/version 2003 [17] have been analyzed as a base case study. The degree of connection J was subsequently modified for each frame to introduce the principle of semi-rigidity according to the Eurocode 4 classification [16]. A total of 36 rigid and semi-rigid frames were subjected to a non-linear static analysis to identify the relationship between the shear force at the base and the displacement at the top. From the latter, the behavior factor R of all frames is evaluated using the method proposed by ATC-19 [18]. The results showed that the behavior factor is strongly influenced by the semi-rigidity of the connections, which is not considered in seismic design code RPA 99/version 2003. Therefore, a simplified approach for the calculation of the behavior factor R of composite moment resisting frames has been proposed in this study.

2. CLASSIFICATION OF CONNECTIONS

The composite connections are classified in Eurocode 4 [16] according to three characteristics [19] their stiffness, strength and ductility. This classification depends on the stiffness and strength properties of the adjacent cross-section beams (Eurocode 3) [20].

2.1. Classification by stiffness

The degree of connection J shown in equation (1) is defined as the ratio between the connection stiffness to the flexural stiffness of the beam. It is used to describe the semi-rigidity of the connection [21].

$$J = \frac{\left(S_{j,ini}\right)_{connection}}{\left(EI_{eq}/L\right)_{beam}} \tag{1}$$

where:

$S_{j,ini}$ – the initial stiffness of the connection (see equation 2),

E – the modulus of elasticity or longitudinal deformation of the beam material,

I_{eq} – the equivalent moment of inertia of the composite beam (see Figure 8),

L – the length of the composite beam.

According to the classification of Eurocode 3 [20] and Eurocode 4 [16], the connection is considered semi-rigid when J varies from 0.5 to 25 for non-braced frames and from 0.5 to 8 for braced frames (Figure 1) .

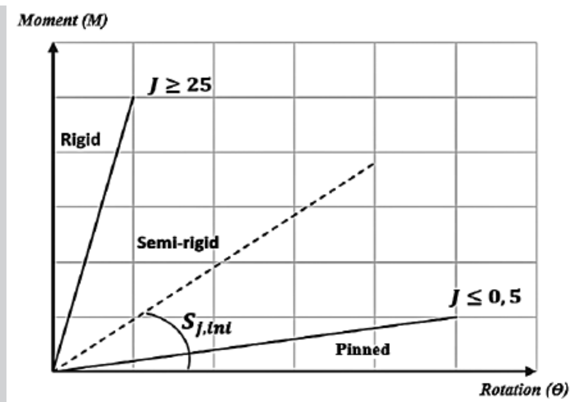

Figure 1.
Boundaries for stiffness classification of beam-column joints for non-braced frames

2.2. Classification by résistance

A beam-column connection can be classified as a full-strength, hinged or partial-strength connection by comparing its moment resistance to the moments of resistance of the elements it connects [20].

Full-strength beam-column connections shall meet the criteria given in Figure 2.

A pinned connection has low strength, not more than 25% of the minimum strength required for a full-strength connection. $M_{j.Rd} < 0.25\ M_{b.pl.Rd}$.

A partial-strength connection is the intermediate solution $0.25\ M_{b.pl.Rd} < M_{j.Rd} < M_{b.pl.Rd}$

where:

$M_{c.pl.Rd}$ – the plastic moment resistance of the column,

$M_{b.pl.Rd}$ – the plastic moment resistance of the beam.

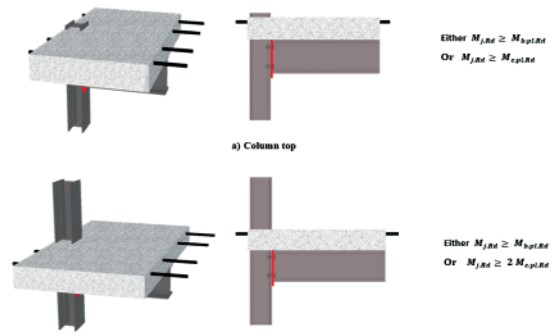

Figure 2
Full strength assembly

3. BEHAVIOR LAW OF COMPOSITE JOINTS

The moment-rotation relationship proposed in this paper for full-strength semi-rigid bolted end-plate composite joints is that of the Eurocode 4 [16]. The key characteristics of the connection such as the moment resistance, initial stiffness and rotational capacity depend on several parameters such as the cross-section area of the longitudinal reinforcement of the concrete slab, the thickness of the end-plate and the thickness of the column flange. The idealized M-θ curve for the monotonic loading that puts the concrete slab in tension which was proposed by Eurocode 4, according to Figure 3, can be described by the following expressions.

The initial stiffness of the connection [16]:

$$S_{j,ini} = \frac{E_a z^2}{\sum_i \frac{1}{k_i}} \tag{2}$$

The moment resistance [16]:

$$M_{Rd} = \sum F_{L,t,i,Rd}\, h_i \tag{3}$$

The rotation capacity [22]:

$$\theta_u = \frac{\Delta_{us}}{D + D_r} + \frac{s^B}{D} \tag{4}$$

where:

E_a – steel elastic modulus,

z – relevant lever arm,

n – number of relevant components,

k_i – stiffness coefficient for basic joint component i,

$\sum F_{L,t,i,Rd}$ – the total tensile force,

h_i – the lever arm of the internal forces,

Δ_{us} – the inelastic elongation of the slab reinforcement,

D_r – the distance from the center of the reinforcement area to the centerline of the lower flange of the beam,

D – the distance from the steel-concrete interface to the centerline of the lower flange of the beam,

s^B – slip at the end of the steel-concrete interface.

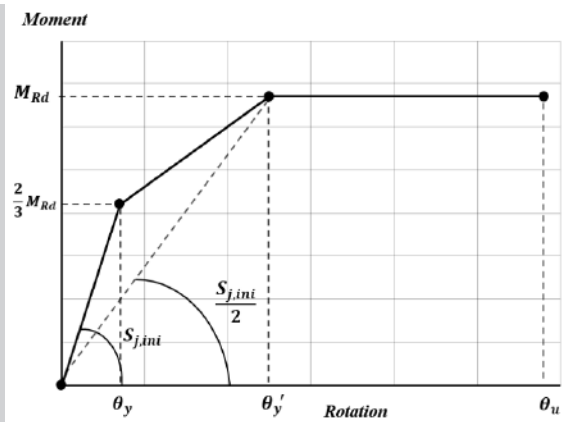

Figure 3
Moment-Rotation Relationship of bolted end plate connection proposed by Eurocode 4

Although Figure 3 and equations 2, 3, and 4 provide an adequate model for beam-column connections under negative moments, very little information has been reported on the behavior of composite connections under positive moments. In the absence of appropriate data, it was decided to assume a point reflection image of Figure 3 [23]. The full M–θ curve used in this study was therefore that of Figure 4.

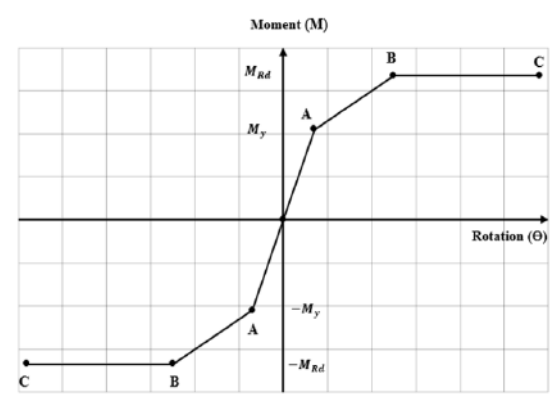

Figure 4.
Complete behavior of a column-beam composite joint

Table 1.
The coordinates of the points in Figure 4

	Moment (KN·m)	Rotation (rad)
Point A	$\pm\frac{2}{3}M_{Rd}$	$\pm\frac{M_{Rd}}{S_{j,ini}}$
Point B	$\pm M_{Rd}$	$\pm\frac{2M_{Rd}}{S_{j,ini}}$
Point C	$\pm M_{Rd}$	$\pm\theta_u$

4. CALCULATION OF BEHAVIOR FACTORS

The behavior factor is noted q in the Eurocode 8 [24] and R in the Algerian seismic code RPA99/version2003 [17]. The review of the existing literature shows that the response of the reduction factor depends on four parameters, namely: one factor that takes into account ductility, another that expresses the resistance reserve, and a third and fourth to take into account redundancy and damping [18]:

$$R = R_\mu \Omega R_R R_\zeta \tag{5}$$

4.1. The ductility factor Rμ

The ductility factor is a measure of the global non-linear response of a structure. It is a function of several structural characteristics like ductility, damping and fundamental period of vibration (T). The global ductility (μ) is represented as following:

$$\mu = \frac{\Delta_u}{\Delta_y} \tag{6}$$

Where Δ_u and Δ_y are respectively the ultimate and yield displacement.

In this study, in order to define the yield displacement point, an elastic perfectly plastic idealization with reduced stiffness (EPI) of the real system is employed. The initial stiffness is assumed as the secant stiffness at 75% of the ultimate strength, and the point at which the secant stiffness reaches the ultimate strength is considered as the global yield point (Figure 5) [12].

During the last four decades, the ductility factor has been the subject of much research. In this study, the relationship R-μ-T developed by Newmark and Hall (1982) [25] is used to calculate the ductility factor R_μ:

$$\begin{aligned} R_\mu &= 1 && for \quad T < 0,2\,s \\ R_\mu &= \sqrt{2\mu - 1} && for \quad 0,2\,s < T < 0,5\,s \\ R_\mu &= \mu && for \quad T > 0,5\,s \end{aligned} \tag{7}$$

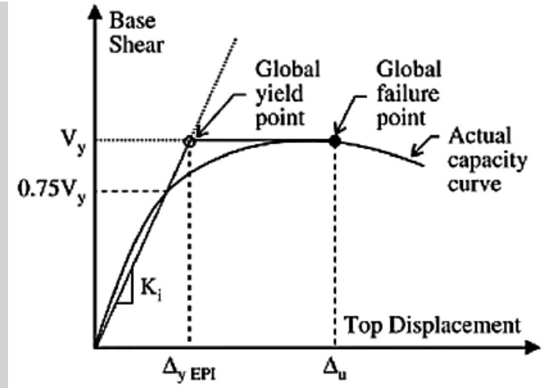

Figure 5.
EPI idealizations of actual capacity curve [12]

where:

T – the fundamental period of the structure.

4.2. Overstrength factor

The overstrength factor is a measure of the additional strength of the structure beyond its design strength due to various reasons. Among those according to Louzai [15] are the following: the actual resistances of the used materials are more significant than the resistances used in the design, the redistribution of internal forces in the inelastic domain, the dimensions of the selected elements of the structure are generally greater than the dimensions strictly necessary because of the availability or rounding of their sizes, the phenomenon of strain-hardening in the steel material as well as the multiple load combinations, and finally the contribution of non-structural elements.

$$\Omega = \frac{V_u}{V_d} \tag{8}$$

where:

V_u – the ultimate shear force calculated using inelastic static and dynamic analyses,

V_d – the design shear force calculated using linear elastic methods.

4.3. Redundancy factor R_R

A redundant structure must be composed from several vertical lines. Four lines of frames or walls in each direction is the minimum number recommended by the Algerian seismic code RPA 99/version 2003 [17] to have adequate redundancy.

4.4. Damping factor R_ξ

The damping factor (R_ξ) is used for structures provided with additional energy dissipation devices (viscous damping). The damping factor is assumed 1 for buildings without such devices.

In this study, the redundancy factor and the damping factor are taken equal to 1 (Table 4.3 of ATC-19) [18]. Thus, the seismic behavior factor is determined as the product of the ductility factor (R_μ) and the overstrength factor (Ω), as shown in Figure 6.

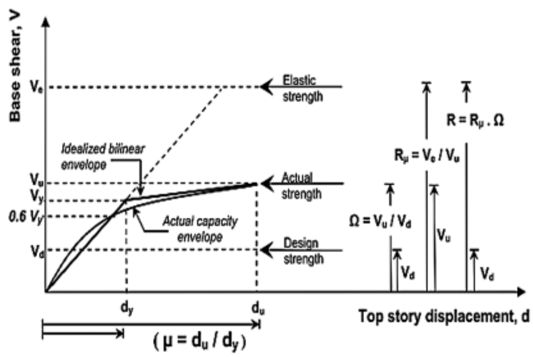

Figure 6.
Relationship between seismic behavior factor (R), overstrength factor (Ω), ductility factor (R_μ) and global ductility μ [26]

5. PUSHOVER ANALYSIS

Through the dynamic time history analysis is believed to set more exact results for seismic evaluation and design of structure. It is believed to be more time and efforts consumptive. So that, to overcome such difficulties and act nonlinear seismic analysis in a practical but still exact procedure, a method called the nonlinear static procedure has been used.

Pushover analysis is a tool used to verify the structural performance of existing or reinforced buildings and new structures. The pushover analysis is a nonlinear static analysis. The method consists of applying a progressive increment of lateral loads distributed over the floors. These loads increase monotonically from zero to the ultimate state corresponding to the initiation of structural damage. This provides the relationship between the shear force at the base and the displacement of the top floor at the top. This curve is generally called the Pushover curve or capacity curve.

There can be many alternatives for the distribution pattern of the lateral loads, and it may be expected that different patterns of lateral loads result in pushover curves with different characteristics and different sequence of plastic hinge formation. In a study done by Mwafy and Elnashai (2001) [26], it is shown that the inverted triangular distribution pattern of the lateral loads produces better estimates of the maximum drift and R factor compared with uniform and multi-modal distributions [15].

This study deals with structures that have a simple structural configuration which is chosen to avoid the influence of different modes of vibration, namely the upper modes of translation or torsion modes. Therefore, the structures analyzed are exclusively influenced by the first vibration mode (translation mode). Also, the horizontal force distribution that is obtained during the lateral force analysis corresponding to the first elastic translation mode of the structure, and which corresponds to a triangular load distribution.

5.1. Failure criteria

To evaluate the R factor, a number of response criteria are needed to define the collapse limit states of a structure. Three failure criteria are used here. These are classified into two groups, local and global criteria.

A local criterion is defined based on the limitation of plastic hinge rotation of different elements (beams, columns) to the ultimate rotation.

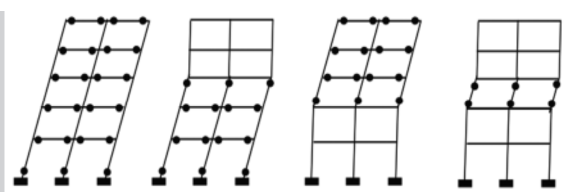

Figure 7.
Four overall instability cases [27]

The adopted global failure criteria are:

– structural instability due to formation of a column hinging mechanism (Figure 7) [27],
– an upper limit of the inter-storey drift, equal to 2% of the storey height. This limit is also specified by Balendra [28], and closed to those adopted by seismic design codes Eurocode 8 [24], which vary between 2 and 3%.

5.2. Moment-rotation (M-θ) relationship for composite beam section

The main parameters needed to develop moment-rotation curve are yield rotation, yield moment, ultimate rotation and ultimate moment under both hogging and sagging. The model adopted in this study to calculate these parameters is that of Senthil kumar [21]. These parameters are calculated according to Table 2 and the M-θ model shown in Figure 8.

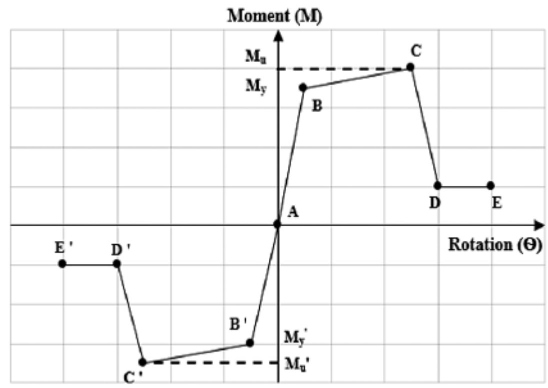

Figure 8.
Plastic hinge behavior of composite beam [21]

Table 2.
Moment-Rotation (M-θ) Relation for the composite beam [21]

Hogging (Liu et all. 2011)	sagging
Length of plastic hinge . $L_p = 1{,}75 \cdot h_t$	
Yield curvature $\phi_y = \dfrac{\varepsilon_{xy}}{y_b}$	Yield curvature $\phi_y = \dfrac{Z \cdot f_y \cdot L}{6 \cdot E \cdot I_b}$
Yield moment $M_y' = E.I' \cdot \phi_y$	Yield moment $M_y = \dfrac{f_{xy}}{y_{mo}} \cdot Z_e$
Ultimate moment $M_u' = M_P'$	Ultimate moment $M_u = \dfrac{f_y}{y_{mo}} \cdot Z_p$
Ultimate curvature $\phi_u = \dfrac{10 \cdot \varepsilon_{xy}}{y_{pb'}}$	Ultimate rotation θ_u from table 5-6 of FEMA 356:2000 [29]

where:

ε_{xy} – yielding of the steel beam,

y_b – distance between the elastic neutral axis and the lower flange of the steel beam,

E – Young's modulus,

I' – Hogging moment of inertia of the transformed section of composite beam,

M_p' – the plastic moment capacity of the composite beam under hogging,

$y_{pb'}$ – distance between the plastic neutral axis and the lower flange of the steel beam,

Z_p and Z_p – elastic and plastic section moduli of the pure steel beam,

f_y – yield strength of the steel beam,

y_{mo} – partial safety factor.

5.3. Simplified model of composite semi-rigid frames

The analyses have been performed using SAP2000 program [30], which is a general-purpose structural analysis program for static and dynamic analyses of structures. In this study, a description of the modeling details is provided in the following.

The program used (SAP2000) does not include composite sections in its library. The type of section used to simulate the behavior of the composite beam is "general" with plastic hinges at both ends defining the behavior of the composite section (Figure 8).

In the composite beams, two different flexural stiffnesses have to be taken into account: EI_1 for the part of the spans subjected to positive bending (uncracked section) and EI_2 for the nodal part subjected to negative bending (cracked section) Figure 9. The properties are calculated using a steel equivalent section with an equivalent inertial stiffness moment of I_{eq} equation 9 [24]:

$$I_{eq} = 0.6 I_1 + 0.4 I_2 \tag{9}$$

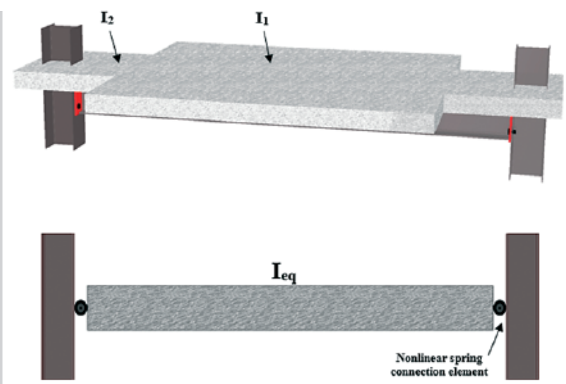

Figure 9.
The equivalent section of the beam

Columns were modeled as steel columns with auto-generated P-M hinge properties based on FEMa 356:2000 at their ends. The non-linear behavior of the connections is modeled by non-linear link elements. Modeling of elements with hinges is shown in Figure 10.

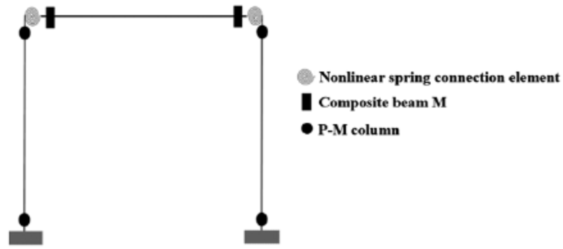

Figure 10.
Modelling of elements in SAP 2000 [21]

6. NUMERICAL APPLICATION

The three portal frames studied are part of three composite buildings (steel column, composite beam) having the same plan views and are assumed to be the central frames (Figure 11). The total dead and live loads on the floor slabs are taken equal to 4.05 and 2.5 kN/m^2 respectively, and for the top floor slab, they are assumed equal to 5.53 and 1.0 kN/m^2.

The frames (Figure 12) were designed according to the Eurocode 4 for composite sections and the Algerian seismic code RPA 99/version 2003 with the following parameters: zone of moderate seismicity, zone IIa, importance class group 2, soil type S3 (soft soil) and quality factor Q = 1. The analysis will be performed for an acceleration factor A = 0.2. The seismic behavior factor R is taken equal to 6. The

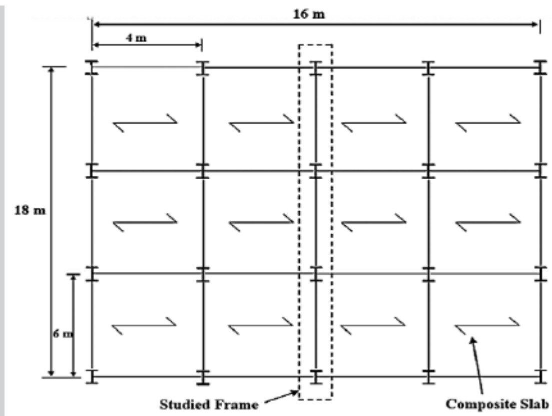

Figure 11.
Plan view of buildings at 3, 4 and 5 storey levels containing the studied frames

member cross-section sizes of the columns-beams elements are given in Table 3.

The Algerian seismic code RPA99/2003 requires the use in seismic zone rigid joints capable of developing the total plastic capacity in beams "article 8.2.4" [17], in fact, this condition can be met by using semi-rigid joints with full resistance. To assure this full strength, the condition of Eurocode 4 shown in Figure 2 must be respected ($M_{j.Rd} \geq M_{b.pl.Rd}$). The initial stiffness of the connection takes 11 values by varying the connection degree J (equation 1). The rotational capacity of

Table 3.
Geometrical and mechanical characteristics of composite frames

Storey level		Columns	Beams
three-storey frame	1, 2 and 3	HEB 300	**Concrete slab:** Thickness, h_c=80 mm, Effective widths: b_{eff+} = 1050 mm, b_{eff-} = 750 mm, Compressive strength, f_{ck} = 25 N/mm^2, Tensile strength, f_t = 3 N/mm^2, E_c = 29000 N/mm^2.
four-storey frame	1, 2, 3and 4	HEB 400	
five-storey frame	1,2 and 3	HEB 550	**Reinforcing bars:** 6Ø12, Yield stress, f_{yr} = 500 N/mm^2. E_r = 200000 N/mm^2. **Steel beam**: IPE300 (S235), E_a=210000 N/mm^2.
	4 and 5	HEB 400	**Stud shear connectors:** Diameter×length=19 mm×60 mm, Degree of shear connection, η=100%.

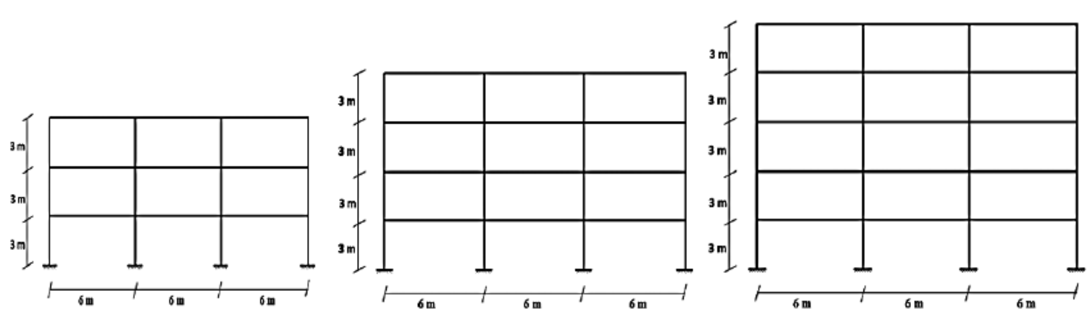

Fig. 12.
Studied frames

the connections must be higher than that of the beams (equation 10) to allow the development of ultimate rotation of the plastic hinges in the beams as required in the Eurocode 4 [16]. This work ends to fix the ratio of 1.2 of the rotational capacity of the composite joints to that of the beams:

$$\left(\theta_u\right)_{connection} > \left(\theta_u\right)_{beam} \tag{10}$$

After determining the key parameters that characterize the connection (see Table 4), its complete moment-rotation behavior is also determined using Table 1.

Table 4.
The joint stiffnesses correspond to the degrees of connection

J	M_{Rd} [kN·m]	$S_{j,ini}$ [kN·m/rad]	θ_u [rad]
0.5		2540.13	
2.5		12700.63	
5		25401.25	
7.5		38101.88	
10		50802.50	
12.5	197.81	63503.13	0.053
15		76203.75	
17.5		88904.38	
20		101605.00	
22.5		114305.63	
25		127006.25	

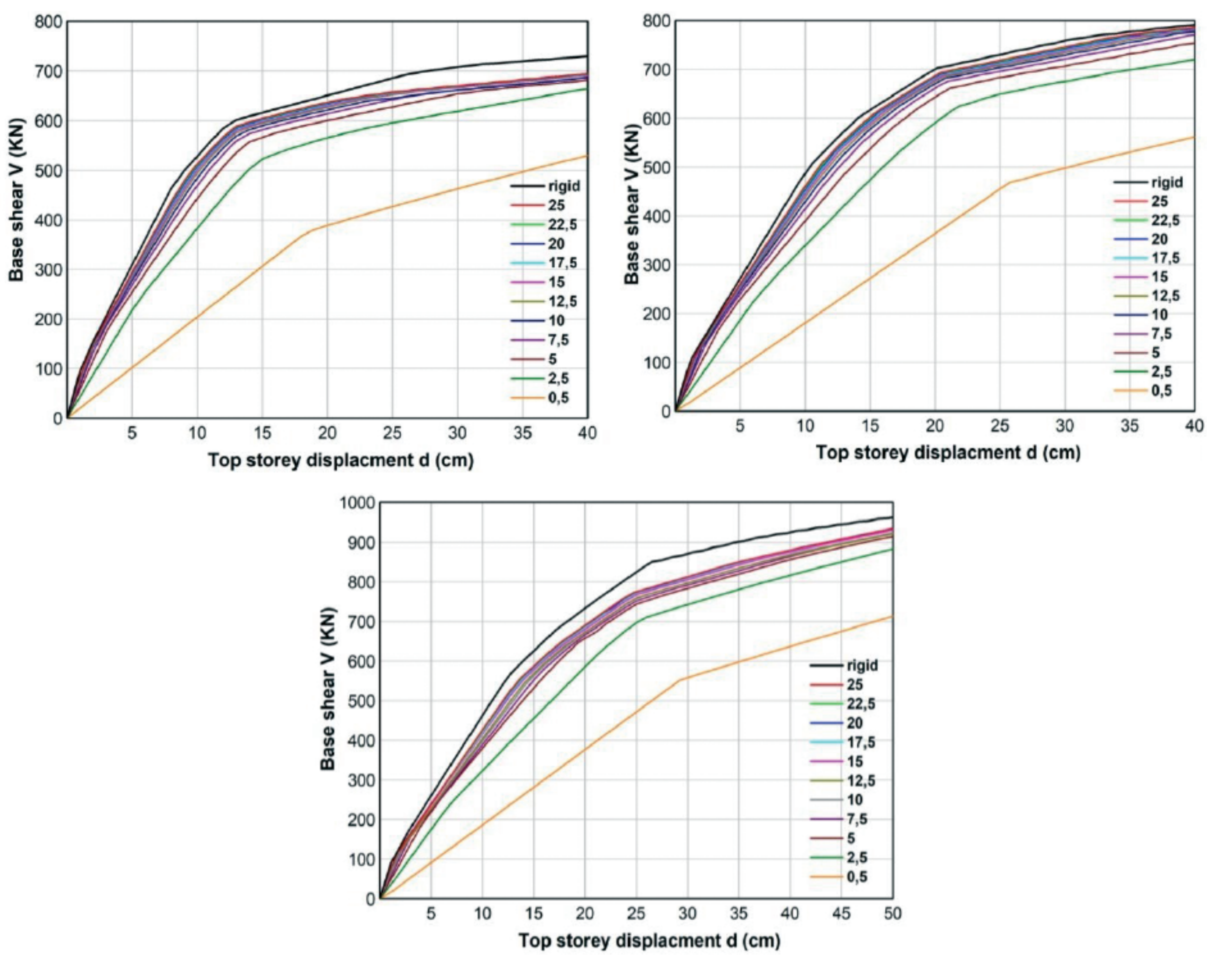

Fig 13.
Pushover curves for rigid and semi-rigid composite frames (a) 3 storeys (b) 4 storeys (c) 5 storeys

7. RESULTS AND DISCUSSION

Figure 13 shows the base shear force curves as a function of the top displacement of 3, 4 and 5 storeys rigid and semi-rigid composite moment-resisting frame. From this figure, it can be seen that the frames with connection degree $J > 5$ have a behavior very close to that of rigid frames. Indeed, the limit of their linear behavior is characterized by a shear force between 468.82 and 498.50 kN for frames of 3 storeys, between 494.72 and 507.05 kN for frames of 4 storeys and between 452.54 and 536.78 kN for frames of 5 storeys. For frames with a degree of connection $J = 5$, the linear behavior is limited by a base shear force equal to 406.55, 506.47 and 542.98 kN respectively for 3, 4 and 5 storey frames. For frames with a degree of connection $J = 2.5$, the limiting values of base shear force for the linear behavior of frames of the 3, 4 and 5 storeys are 475.83, 575.63 and 613.47 kN respectively. This figure also shows that the frames with a degree of connection of their joints $J = 0.5$, the limit of the linear behavior in terms of base shear reached 367.57 and 458.23 kN for the 3 and 4 storey frames, and 551.86 kN for the five storeys frames.

The values of the yield displacement Δ_y, limit displacement Δ_u, Fundamental Period T, ductility factor R_μ, design shear force V_d, ultimate shear force Vu, overstrength factor Ω and the behavior factor R of frames studied are presented in the tables below (Tables 5, 6 and 7). For all the frames analyzed and regardless of the degree of connection J, the determinant limit state of failure is the inter-storey drift at 2%. The latter is observed between 1.44% and 1.66% top displacement. In this context, it can be seen that the reduction of connection stiffness has no effect on the limit instant of inter-storey drift.

Table 5.
Yield displacement Δ_y, limit displacement Δ_u, Fundamental Period T, ductility factor R_μ, design shear force V_d, ultimate shear force V_u, overstrength factor Ω and the behavior factor R of frames with 3 storeys

Degree of connection	Δ_y [cm]	Δ_u [cm]	T [s]	R_μ	V_d [kN]	V_u [kN]	Ω	R
rigid	10.61	14.98	0.571	1.41	91.26	615.31	6.74	9.52
25.0	11.19	14.98	0.606	1.34		604.59	6.62	8.87
22.5	11.24	14.98	0.610	1.33		603.47	6.61	8.81
20.0	11.31	14.98	0.614	1.32		602.13	6.60	8.74
17.5	11.40	14.98	0.620	1.31		600.42	6.58	8.65
15.0	11.51	14.98	0.628	1.30		598.16	6.55	8.53
12.5	11.65	14.98	0.638	1.29		594.94	6.52	8.38
10.0	11.83	14.98	0.653	1.27		589.76	6.46	8.18
7.5	12.13	14.98	0.676	1.24		581.79	6.38	7.87
5.0	12.65	14.99	0.718	1.18		566.50	6.21	7.35
2.5	13.71	15.00	0.817	1.09		523.21	5.73	6.27
0.5	18.66	15.01	1.164	1.00		306.29	3.36	3.36

Table 6.
Yield displacement Δ_y, limit displacement Δ_u, Fundamental Period T, ductility factor R_μ, design shear force V_d, ultimate shear force V_u, overstrength factor Ω and the behavior factor R of frames with 4 storeys

Degree of connection	Δ_y [cm]	Δ_u [cm]	T [s]	R_μ	V_d [kN]	V_u [kN]	Ω	R
Rigid	14,48	19,23	0.666	1.33	112.45	688.37	6.12	8.13
25.0	14,01	19,23	0.714	1.37		675.21	6.00	8.24
22.5	15,06	19,16	0.719	1.27		672.84	5.98	7.61
20.0	15,33	19,52	0.725	1.27		676.89	6.02	7.66
17.5	15,64	19,94	0.733	1.27		681.44	6.06	7.73
15.0	15,67	19,54	0.743	1.25		672.08	5.98	7.46
12.5	15,72	19,46	0.756	1.24		667.07	5.93	7.34
10.0	16,08	19,65	0.775	1.22		665.16	5.92	7.23
7.5	16,25	19,32	0.805	1.19		649.68	5.78	6.87
5.0	16,94	19,51	0.858	1.15		635.24	5.65	6.51
2.5	17,86	19,26	0.981	1.08		575.63	5.12	5.52
0.5	19,19	19,14	1.370	1.00		348.27	3.10	3.10

Table 7.
Yield displacement Δ_y, limit displacement Δ_u, Fundamental Period T, ductility factor R_μ, design shear force V_d, ultimate shear force V_u, overstrength factor Ω and the behavior factor R of frames with 5 storeys

Degree of connection	Δ_y [cm]	Δ_u [cm]	T [s]	R_μ	V_d [kN]	V_u [kN]	Ω	R
Rigid	18.39	23.29	0.753	1.27	124.37	794.00	6.38	8.09
25.0	18.20	22.70	0.807	1.25		740.04	5.95	7.42
22.5	18.28	22.76	0.812	1.25		739.93	5.95	7.41
20.0	18.22	22.94	0.819	1.25		768,80	5.93	7.42
17.5	17.89	21.90	0.828	1.22		720.53	5.79	7.09
15.0	17.96	21.99	0.837	1.22		716.43	5.76	7.06
12.5	18.22	22.00	0.853	1.21		708.97	5.70	6.88
10.0	18.51	21.99	0.874	1.19		707.28	5.69	6.76
7.5	18.92	22.01	0.906	1.16		701.66	5.64	6.56
5.0	19.45	22.67	0.962	1.17		693.73	5.58	6.50
2.5	21.21	22.08	1.087	1.04		636.27	5.12	5.33
0.5	22.30	22.22	1.441	1.00		418.66	3.37	3.37

7.1. Ductility factor

The ductility factor values calculated by equation 7 for the three frames studied are presented in the Tables 5, 6, 7. These ductility factors for all rigid and semi-rigid frames vary between 1.04 and 1.41. Figure 14 shows that the number of storeys and the degree of connection have a negligible effect on the ductility factor. An exception is the degree of connection $J = 0.5$, where the ductility factor is taken to be equal to 1 because the failure occurs in the elastic domain. The main reason is the absence of plastic hinges in the frame elements as shown in Figure 15.

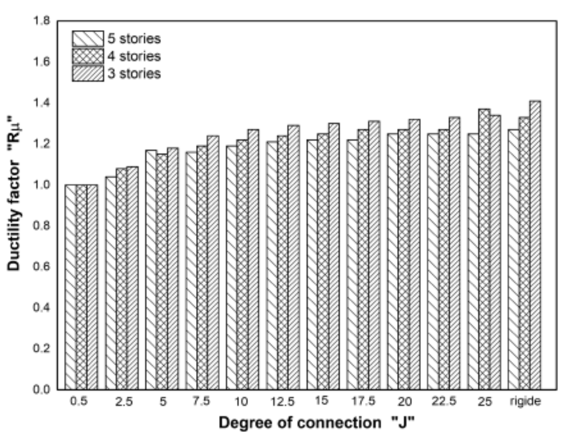

Figure 14.
The effect of degree of connection J and the number of storeys levels on the ductility factor

7.2. The overstrength factor

The overstrength factor values for the three frames studied are presented in the Tables 5, 6, and 7. These overstrength factors are significant and vary between 5.58 and 6.64 for rigid and semi-rigid frames with a degree of connection $J \geq 5$. The number of storeys has a negligible effect, as shown in the histogram in Figure 16. However, the use of semi-rigid connections influences the values of the overstrength factor, knowing that it decreases with the decrease of the degree of connection J. This is clearer, especially for frames designed with semi-rigid connections having a connection degree $J < 5$.

These overstrength factors high values are caused by:

– the harsh limitation to choose the columns cross

Figure 15.
The distribution of plastic hinges in frames with a J=0.5 at the limit instant of the appearance of inter-storey drift criteria

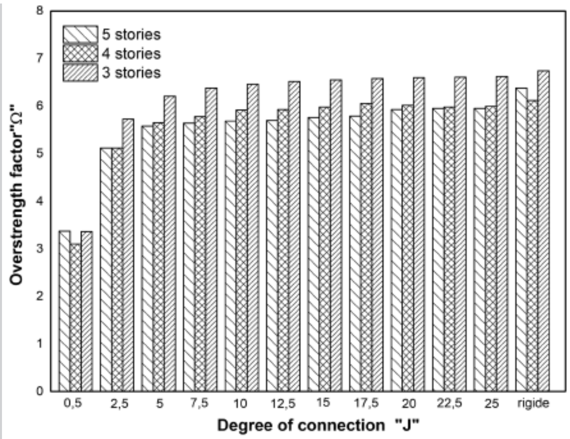

Figure 16.
The effect of the degree of connection on the overstrength factor of the different frames (3, 4 and 5 storeys)

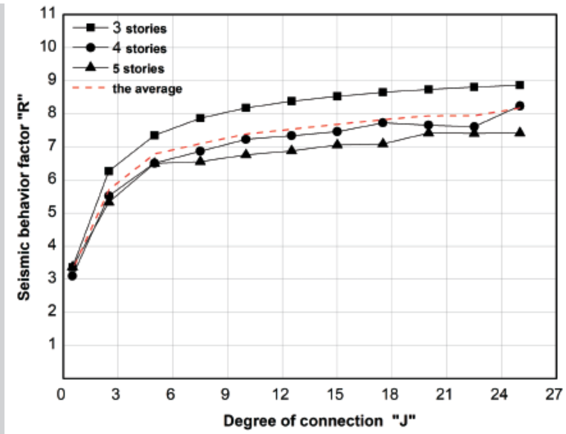

Fig 17.
The effect of the degree of connection on the behavior factor of the different frames (3, 4 and 5 storeys)

sections. The lateral resistance of a moment resisting frame rely basically on the stiffness of the columns. Thus, so as to watch the inter storey drift, the size of the columns has been taken up till the limits are reached to (inter-storey drift ≤ 1% of storey height) [31],

– the assumptions made in the modeling with Sap 2000 Nonlinear may have altered the behavior of the frame. The fact that the composite beam has modeled as a steel equivalent section with an equivalent inertial stiffness moment of I_{eq},

– the strain hardening and the difference between the characteristic values of material strengths, used in design and those used in analysis are other reasons contributing to the huge noticed overstrength [31].

7.3. The behavior factor R

Figure 17 shows the effect of the degree of connection J on the behavior factor R of the different frames (3, 4 and 5 floors). According to this figure, it can be seen that the behavior factor of portals with 3, 4 and 5 storeys increases as the degree of connection increases. For frames with maximum semi-rigidity of the connections ($J = 25$), the calculated behavior factor value is very close to that of the rigid frames (Table 5, 6 and 7). These results show that the seismic behavior factor does not depend on the height of the structures. However, it is influenced by the semi-rigidity of the composite connections (column-beam).

Until now, the semi-rigidity of composite connections is not considered in the seismic codes to design the structures in our case (composite moment-resisting frame). Therefore it is necessary to make some comparisons between the R values of the composite frames (steel-concrete) estimated in this study and the only value given by the Algerian seismic code RPA99/version 2003.

Figure 18 shows a comparison of the variation in the behavior factor R as a function of the degree of connection with the R value recommended by the RPA99/Version 2003 code, which equals 6. In the case of rigid frames, the calculated R-factor value is equal to 8.5 (average value), which is greater than 41.6% recommended by RPA99/2003. For semi-rigid frames, the R-factor decreases as the degree of connection J decreases (Figure 16).

This is due to the reduction of the ultimate base shear force V_u with the decrease in the degree of connection J. This decrease is proportional to overstrength Ω, which is a factor determining the behavior coefficient R.

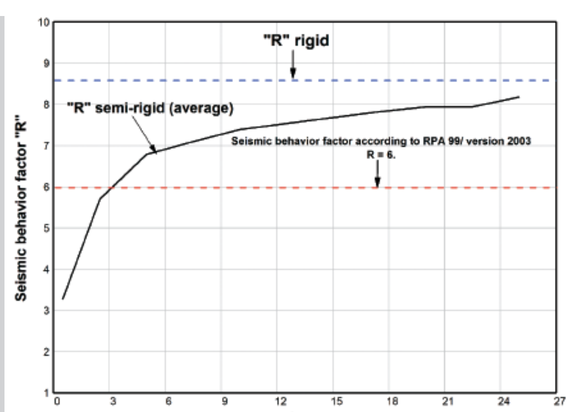

Fig 18.
Comparison between the variation of the behavior factor R as a function of the degree of connection

From Figure 18 we can see that the behavior factor R varies considerably from J = 0.5 to J ≥ 5. However, it varies slightly if J > 5. At this point and in order to simplify the use of behavior factor R in the seismic design of semi-rigid composite structures, Figure 19 breaks down this variation into two parts, the first part is represented by a logarithmic function and the second one is represented by a single constant value equal to 7.5:

$$\begin{aligned} &R = 8.5 && J > 25 \\ &R = 7.5 && 5 < J \le 25 \\ &R = 1.52\,Ln(J) + 4.3271 && 0.5 \le J \le 5 \end{aligned} \qquad (11)$$

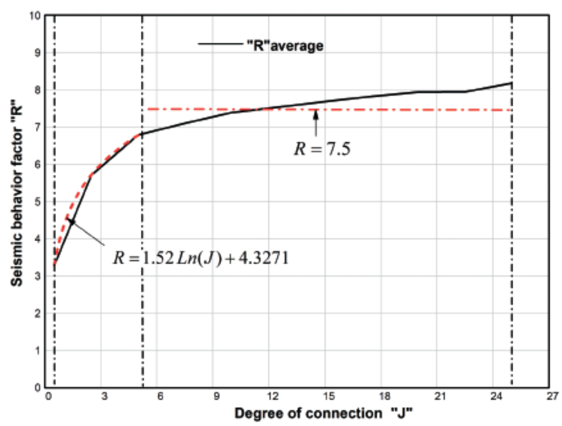

Fig 19.
The Simplification usage of the behavior factor values in the design of semi-rigid composite structures

8. CONCLUSION

In this paper, the effect of semi-rigidity of connections on the behavior of composite moment resistant frames structures has been highlighted. For this purpose, the seismic behavior of these structures has been analyzed through the evaluation of their behavior factor R. In this regard, three types of frames 3, 4, and 5 storeys were analyzed for different degrees of connection (column-beam). The findings of the study lead to the following conclusions:

– in the case of Pushover analyses performed to calculate the behavior factor R and among the associated failure criteria, the inter-story drift is the parameter controlling the global failure of all frames analyzed. Moreover, it was concluded that the semi-rigidity of the connections does not influence the time of occurrence of this criteria in terms of limit top displacement,
– the degree of connection J does not affect the ductility factor values, which are relatively low because of the conservative limitation of the inter-story drift criterion (2%),
– the semi-rigidity of composite connections has an influence on the overstrength factor, and this influence is proportional,
– the capacity curves of semi-rigid frames with degrees of connection J > 5 show very similar behavior. Therefore, the effect of the variation of the semi-rigidity in this range is moderate. On the other hand, one must be very careful in designing frames with degrees of connection J ≤ 5 where the effect of the variation of the semi-rigidity is significant on their seismic behavior,
– the behavior factor R does not keep the same value in the case of introducing semi-rigidity of the connections. This value varies with the variation of the degree of connection J but always remains lower than that of rigid connection frames.

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