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# Numerical model for the non-linear dynamic analysis of multi-storey structures with semi-rigid joints with specific reference to the Algerian code

S. Koriga[1], A.N.T. Ihaddoudene[1], M. Saidani[2]*

[1] *Built Environment Research Laboratory, Faculty of Civil Engineering, U.S.T.H.B., Algiers, Algeria.*

[2] *Faculty of Engineering, Environment and Computing, School of Energy, Construction and Environment, Coventry University, England, UK*

**Abstract:**

The current paper aims at investigating the dynamic response of rigid and semi-rigid connections of steel structures built in high seismic areas. A nonlinear dynamic analysis model, which is an extension to the simplified and direct mechanical model used in the static analysis, is proposed and discussed. The novelty of the model consists in the introduction of a bar element with semi-rigid joint as a single element without the need to discretise it (i.e. without a finite element mesh) in the program where non-linearity is considered in the flexibility factor of the stiffness matrix. The model developed is validated through application to examples of steel frames with different types of connections under dynamic forces. The results obtained were very satisfactory. This work is motivated by the need for the revision of the Algerian seismic code (RPA99v2003) which does not yet consider provisions for the design of structures with semi-rigid joints. Based on the results of the study carried out on a multi-storey structure with different types of joints subjected to seismic loading, it can be seen that the safety justifications recommended by Algerian regulations RPA99 in terms of relative displacements as well as the dimension of the seismic joint prove to be too conservative compared to those by Eurocode 8.

**Keywords:** Semi-rigid; Numerical model; Non-linear; Dynamic loading; RPA99; EC8.

## 1. Introduction

Steel structures are commonly used in high seismic zones due to their ductility and earthquake resistance, as well as their maneuverability in design and execution over other types of structures. They are generally composed of bar elements consisting of laminated or welded sections where the connections of various elements play a very important role in ensuring the transmission and distribution of the different stresses between the connected elements. To simplify their analysis, their behaviour is often considered as fully rigid or ideally pinned. In fact, as is common knowledge, their real behaviour falls between these two extreme cases since the most rigid joint always has a certain flexibility while the pinned joint transmits a certain bending moment [1-4]. Therefore, in structural analysis, where the local deformations of the joint are neglected, this new source of flexibility must be incorporated to quantify the moment transfer ratio as well as the corresponding rotation.

Several studies [7-10] have been carried out on dynamic analyses of structural elements while taking both material and geometrical nonlinearities into account. In this context, tests on flexible connections followed by numerical analyses of Chui and Chan [5] and Nader and Astaneh-Asl [6], have shown the importance of considering the connection flexibility in structural model.

Bahaari and Sherbourne [17] conducted a study on the behaviour of end-plate bolted connections. They proposed characteristics of a model relevant to the semi-rigid joints. The best modeling of the semi-rigid connection behaviour according to Richard-Abbott [18] is realized when the M-θ relation contains power terms. Another approach is developed by Bayo, Coll et al. [19,20], based on the component method, to model the semi-rigid connections. The global joint is considered composed of four joints representing the four solicitations separately. In his research work Aljabri [21] was able to include the effect of the increasing temperatures on connections flexibilities, Hadianfard and Razani [22] used the Monte Carlo simulation technique to illustrate the influence of semi-rigid connections on the reliability of steel structures. Kishi et al. [23] provided an evaluation analysis of the Eurocode 3 classification on the three types of connections in steel construction; they found that the type of connection might change in the post-elastic phase.

Several mechanical models are proposed by several researchers to predict the real behaviour of semi-rigid connections [1-4,23-27]. These models are classified into two main categories, linear and non-linear. The constant stiffness in the linear models [29] has shown an insufficient level of accuracy in the behaviour of semi-rigid joints. This lack of accuracy is reduced by considering their non-linear behaviour in the model [18,24,25]. The actual behaviour of a joint is generally obtained from the experimental tests [19,26].

Based on past research work [7-16,28-34], the effect of the nonlinear behaviour of the joint on the response of the structure was found to be more apparent under cyclic and dynamic loads.

The theory of dynamic analysis of structures with flexible connections considers the moment-rotation relationship as linear [1-4,23,24,29,30]. The non-linear behaviour is idealized as bi-linear. This idealization can affect the response of steel structures due to the reduction in rotational stiffness, especially in the elastoplastic phase [20, 21].

Not only the non-linearity of the moment-rotation curve plays an important role but also the hysteretic effect [7]. Particularly the hysteresis loops of the connection behaviour that directly influence the energy dissipation capacity of the structure, and thereby affect its vibrational characteristics.

A previous study of the vibratory behaviour of semi-rigid joints through comparing the Eurocode 3 approach and the numerical solutions by Apoulos [33] showed that the influence of the connections only appears in the higher modes.

A comparison between rigid and semi-rigid connections in high-rise steel buildings was carried out by Razavi and Abolmaali [34]. They showed that the frame with semi-rigid joints exhibited a better behaviour than the fully rigid frame.

In the current research paper, the effect of the nonlinear behaviour of semi-rigid joints on the response of the structure under dynamic loads is investigated. The mechanical model developed by Ihaddoudene et al. [1] has been used, due to its efficiency in modeling the semi-rigid aspect and the simplicity in its implementation. This simplicity consists in considering the non-linearity by the flexibility factor that exists in the stiffness matrix. This factor results from the M-Φ curves in which the tangent stiffness is present. The advantage of this flexibility factor is that the effect of additional rotation (semi-rigid) to the other components of the stiffness matrix is considered. Validation of the proposed numerical model are performed on examples of steel frame with different types of connections. The results obtained were very satisfactory.

This work was essentially motivated by the need to improve the Algerian seismic code RPA99v2003 (Algerian Parasismic Rules) [39] which does not yet contain provisions for the design of structures with semi-rigid joints. This study is supported by a comparison with Eurocode 8. For this, a study of the effect of the nonlinear behaviour of several types of rigid and semi-rigid connections on the response of a multi-storey structure subjected to the seismic loading was carried out. The justifications provided by RPA99 such as the drift-ratio and the seismic joint are compared with those recommended by EC8.

## 2. Mechanical Model and stiffness Matrix

The adopted model [01] is based on the analogy of three springs (two translations and one rotation) considering the concept of a non-deformable node element. Therefore, the relative displacements and rotations between the nodes and the elements of the structure are taken into consideration in the stiffness matrix.

The objective of the mechanical model is to obtain, in a simple way, the stiffness matrix and the nodal vector of the load. For this, the bar element subjected to transverse loads with semi-rigid joints is taken into consideration (Figure 2).

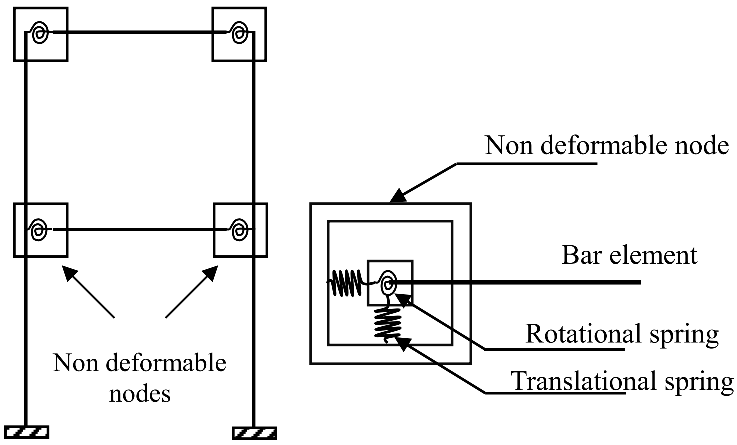



*Figure 1. Adopted mechanical model [1]*

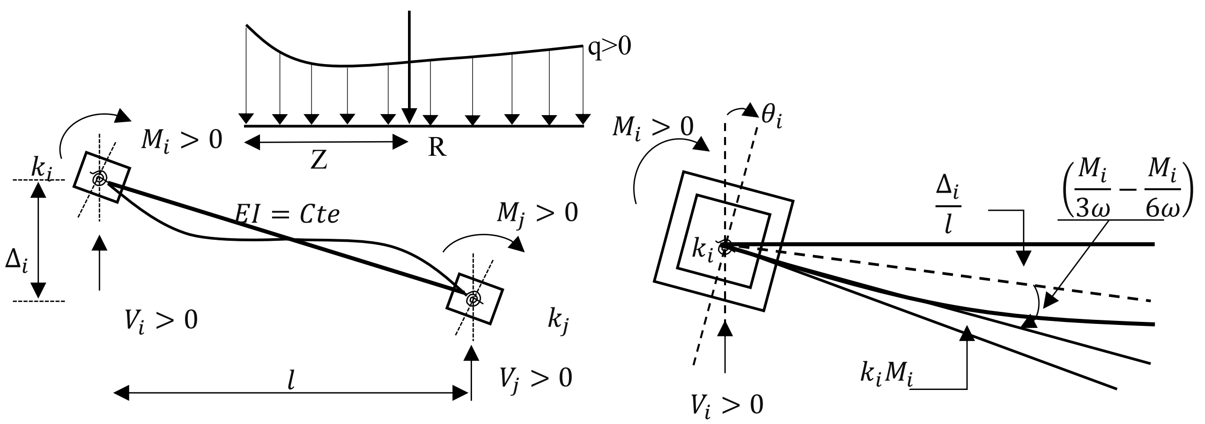



*Figure 2. Different rotations in a non-deformable node [1]*

The establishment of the elementary stiffness matrix may be made by introducing the additional rotation $\theta_i$ [24,25] which reflects the semi-rigid connections (Figure 1). The following equation gives the rotational stiffness $k_i$ of the connection:

$$k_i = \frac{M_i}{\theta_i} \quad (01)$$

$M_i$ being the moment of rotation at node i

### *2.1. Equilibrium equations and rotational deformations*

The equilibrium equations can be written as:

$$V_i + V_j - R = 0 \quad (02)$$

$$M_i + M_j + RZ - V_j l = 0 \quad (03)$$

Where:

$V_i$ , $M_i$, $V_j$ $et$ $M_j$ : are the reactions at nodes i and j, respectively.

$R$: is the applied force

In bending, the spring rotation is the essential component and therefore the rotational deformation equations can be expressed as follows:

$$\theta_i = \frac{\Delta_i}{l} + \frac{m\psi}{\omega l} + \frac{M_i}{3\omega} + k_1 M_i^{\alpha} - \frac{M_j}{6\omega} \quad (04.a)$$

$$\theta_j = \frac{\Delta_i}{l} - \frac{n\psi}{\omega l} + \frac{M_j}{3\omega} + k_2 M_j^{\alpha} - \frac{M_i}{6\omega} \quad (04.b)$$

Where,

$\theta_i$and $\theta_j$ are, the rotations at the nodes i and j respectively.

The modified stiffness matrix $\overline{K}_e$ for this study is expressed as follows:

$$\overline{K}_e = \begin{bmatrix} k_{11} & k_{12} & k_{13} & k_{14} \\ k_{21} & k_{22} & k_{23} & k_{24} \\ k_{31} & k_{32} & k_{33} & k_{34} \\ k_{41} & k_{42} & k_{43} & k_{44} \end{bmatrix} \quad (5)$$

In which, the expression of the elements of the stiffness matrix is given by [1]:

$$k_{11} = \frac{36\omega(1 + (k_1 + k_2)\omega)}{l^2[4(1 + 3k_1\omega)(1 + 3k_2\omega) - 1]} \quad (06)$$

$$k_{12} = -\frac{18\omega(1 + 2k_2\omega)}{l[4(1 + 3k_1\omega)(1 + 3k_1\omega) - 1]} \quad (07)$$

$$k_{13} = -\frac{36\omega(1 + (k_1 + k_2)\omega)}{l^2[4(1 + 3k_1\omega)(1 + 3k_2\omega) - 1]} \quad (08)$$

$$k_{14} = -\frac{18\omega(1 + 2k_1\omega)}{l[4(1 + 3k_1\omega)(1 + 3k_1\omega) - 1]} \quad (09)$$

Where,

ω= EI / L: is the flexural stiffness per unit length,

$k_1$ and $k_2$ : are, the elastic constants of the spring in rotation at nodes i and j, respectively.

It is interesting to note that for frequent steel frames, in general, the joints in a beam member are identical at both ends.

## 3. Semi-rigid connection modeling:

The flexibility of the beam-to-column connection is characterized by a moment-rotation relationship that is practically non-linear on all phases of the static or the dynamic loading. Figure 3 shows the different models proposed to fit a moment-rotation curve [01]. On the other hand, the axial and shear deformations are generally neglected compared to the rotational deformation.

There are some advantages of using these models to describe the nonlinear M-θ relationship of the connections. They can always guarantee a positive first derivative, which is particularly important to prevent the occurrence of negative connection stiffness, which is undesirable for numerical computation.

In addition, they require only a small number of parameters in the expression, so that the procedure for adjusting the curve and calculating the stiffness in the analysis will be simpler and more convenient. Finally, in general, these models give a good fit for the M-θ curves compared to the experimental data [30].

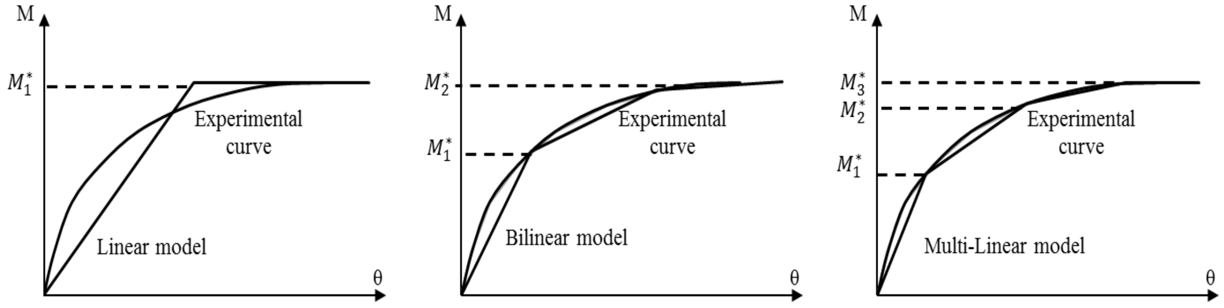



*Figure 3. Idealization of the non-linear behaviour [5]*

## 4. Models

### *4.1. Kishi and Chen [30]*

This is one of the most used models for semi-rigid connections because it only needs three parameters to model the moment-rotation relationship and always gives a positive stiffness. The plastic rotation is defined as a ratio of the ultimate moment capacity and the initial connection stiffness. It is given by the following relation (10):

$$M = \frac{R_{ki}|\theta_r|}{\left\{1 + \left|\frac{|\theta_r|}{|\theta_0|}\right|^n\right\}^{\frac{1}{n}}} \tag{10}$$

Where,

M and $\theta_r$: are the moment and the rotation of the connection, respectively.

n: is a shape parameter,

$\theta_0$: is the plastic rotation,

$R_{ki}$: is the initial stiffness of the connection.

### *4.2. Richard-Abbott [18]*

Richard-Abbott proposed a more accurate four-parameter model [18], which presents the moment-rotation relationship of the connection in the following expression:

$$M = \frac{(R_{ki} + R_{kp})|\theta_r|}{\left\{1 + \left|\frac{(R_{ki} - R_{kp})|\theta_r|}{M_0}\right|^n\right\}^{\frac{1}{n}}} + R_{kp}|\theta_r| \tag{11}$$

Where:

$R_{ki}$ is the initial stiffness of the connection;

$R_{kp}$ is the strain-hardening stiffness;

$M_o$ is a reference moment;

n is a parameter defining the sharpness of the curve [30].

## 5. Program elaboration

The motion formulation of structures with semi-rigid joints is given by the following equation:

$$[M]\ddot{D} + [C]\dot{D} + [K]D = F\{t\} \tag{12}$$

Where [M], [C] and [K] are, the mass, damping and tangent stiffness matrices, respectively.

The Newmark method was used for the numerical integration of the motion equation due to its simplicity [36]. Residual forces in each time step can be eliminated using the Newton-Raphson's iterative method [37]. The incremental motion equation of the structure can be written in the following expression:

$$[M]\{\Delta\ddot{D}\} + [C]\{\Delta\dot{D}\} + [K]\{\Delta D\} = \{\Delta F\} \tag{13}$$

Where $\{\Delta\ddot{D}\}$, $\{\Delta\dot{D}\}$, and $\{\Delta D\}$ are the incremental vectors of acceleration, velocity, and displacement, respectively.

{ΔF} is the external increment load vector. The viscous damping matrix [C] can be defined as the Rayleigh damping matrix [37]. The Newmark algorithm coupled with the Newton-Raphson iterations are presented in Figure 4 [37].

A program in the Matlab language [38] has been developed based on the proposed mechanical model [01] and shown in Figure 5 below. The proposed numerical procedure predicts the elastic and non-linear plastic response of semi-rigid steel structures under dynamic loads. The flowchart below gives the different steps of this procedure.

*NEWMARK Method (nonlinear system)*

Special cases

(01) Average acceleration method ($\gamma = {}^{1}/_{2}, \beta = {}^{1}/_{4}$)

(02) Linear acceleration method ($\gamma = {}^{1}/_{2}, \beta = {}^{1}/_{6}$)

1.0 Initial calculation

1.1 State determination: $(f_s)_0$ and $(k_T)_0$.

1.2 $\ddot{D}_0 = \frac{p_0 - c\dot{D}_0 - (f_s)_0}{m}$

1.3 Selection $\Delta t$

1.4 $a_1 = \frac{1}{\beta(\Delta t)^2} m + \frac{\gamma}{\beta(\Delta t)} c$; $a_2 = \frac{1}{\beta \Delta t} m + \left(\frac{\gamma}{\beta} - 1\right) c$; $a_3 = \left(\frac{1}{2\beta} - 1\right) m + \Delta t \left(\frac{\gamma}{2\beta} - 1\right) c$

2.0 Calculate for each time increment, $i = 0,1,2, \ldots$

2.1 initializes $j = 1, D_{i+1}^{(j)} = D_i, (f_s)_{i+1}^{(j)} = (f_s)_i, and\ (k_T)_{i+1}^{(j)} = (k_T)_i$

2.2 $\hat{p}_{i+1} = p_{i+1} + a_1 D_i + a_2 \dot{D}_i + a_3 \ddot{D}_i$

3.0 For each iteration, $j = 1,2,3$ ...

3.1 $\hat{R}_{i+1}^{(j)} = \hat{p}_{i+1} - (f_s)_{i+1}^{(j)} - a_1 D_{i+1}^{(j)}$.

3.2 Verification of convergence: if the acceptance criterion is not verified, go to steps 3.3 to 3.7;
if not, skip those steps and go to step 4.0

3.3 $(\hat{k}_T)_{i+1}^{(j)} = (k_T)_{i+1}^{(j)} + a_1$.

3.4 $\Delta D^{(j)} = \hat{R}_{i+1}^{(j)} \div (\hat{k}_T)_{i+1}^{(j)}$.

3.5 $D_{i+1}^{(j+1)} = D_{i+1}^{(j)} + \Delta D^{(j)}$.

3.6 State determination: $(f_s)_{i+1}^{(j+1)}$ and $(\hat{k}_T)_{i+1}^{(j+1)}$

Replace $j$ by $j + 1$ and repeat the steps 3.1 to 3.6; denote the final value as $D_{i+1}$.

4.0 Calculates velocity and acceleration

4.1 $\dot{D}_{i+1} = \frac{\gamma}{\beta \Delta t}(D_{i+1} - D_i) + \left(1 - \frac{\gamma}{\beta}\right) \dot{D}_i + \Delta t \left(1 - \frac{\gamma}{2\beta}\right) \ddot{D}_i$.

4.2 $\ddot{D}_{i+1} = \frac{1}{\beta(\Delta t)^2}(D_{i+1} - D_i) - \frac{1}{\beta \Delta t} \dot{D}_i + \left(\frac{1}{2\beta} - 1\right) \ddot{D}_i$.

5.0 Repetition for the next time increment: replace “i” with “i + 1” and implement steps 2.0 through 4.0 for the next time increment.

*Figure 4. Flowchart of the Newmark method*

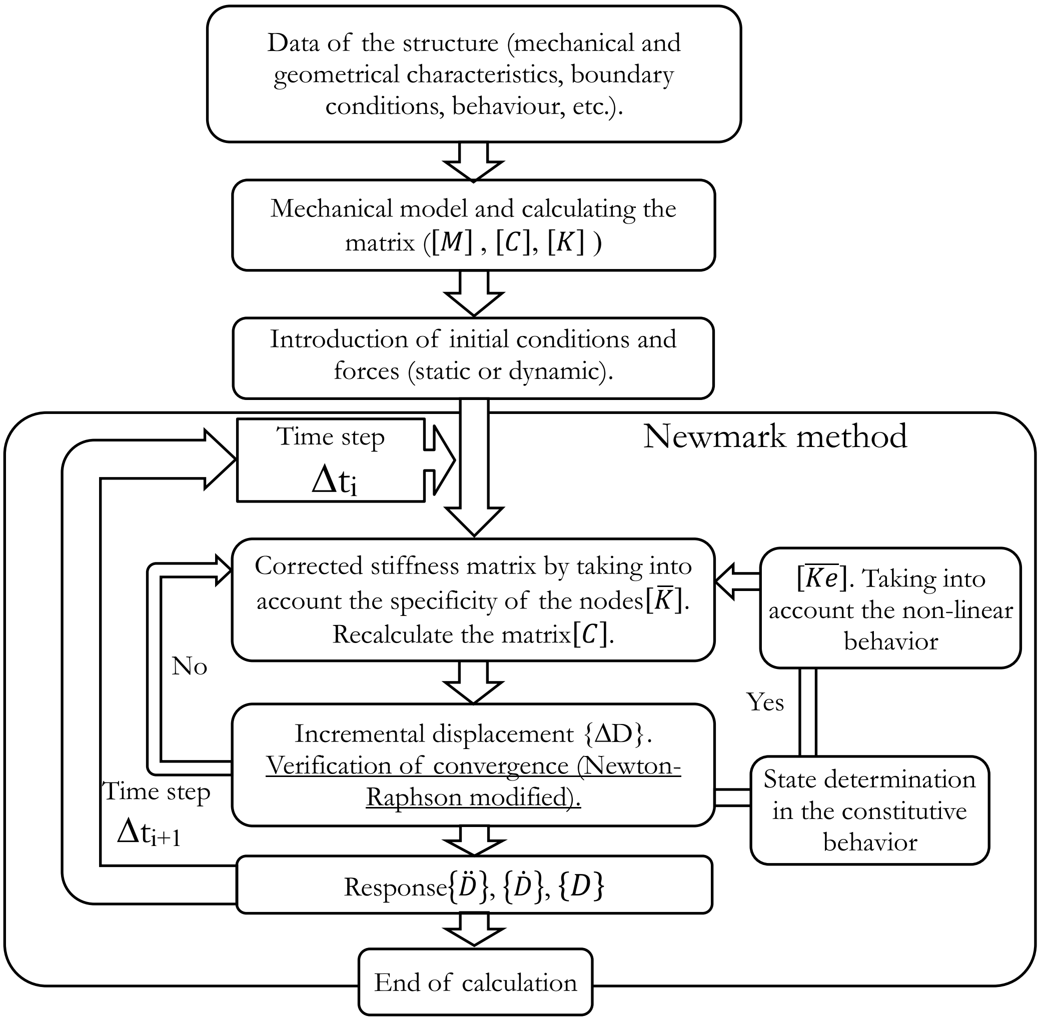



*Figure 5. Flowchart of the numerical model.*

## 6. Validation

In order to verify the accuracy and the computational efficiency of the model, the results of the example below obtained by using the proposed modeling algorithm are compared with those of the experiment and given in reference [30]. The geometry of the structure is shown in Figure (6.a) where all elements of the frame are W5x16 profile; the steel used is A36. The half inch angle connection in used, the details of connection are shown in Figure 7. The structure is assumed to be subjected to a strong pulse during one second as shown in Figure (6.b).

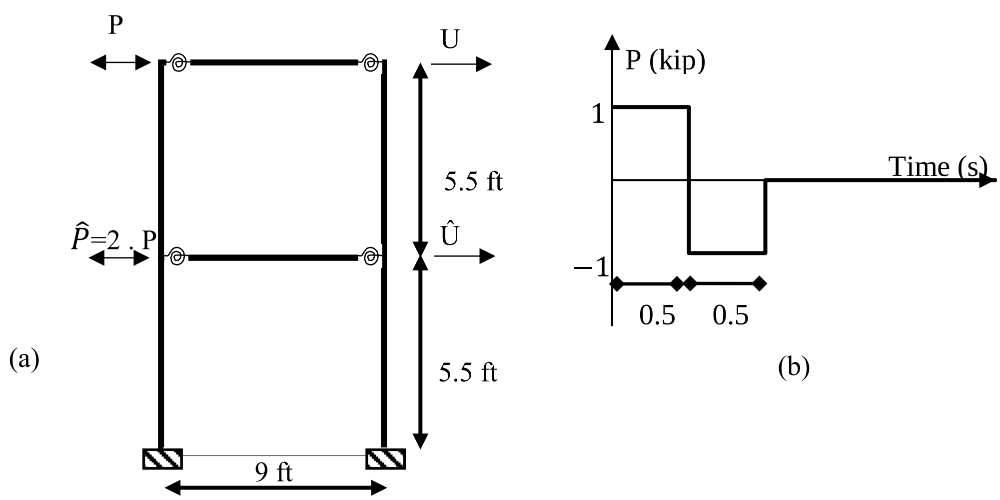



*Figure 6. Two-story frame [30]*

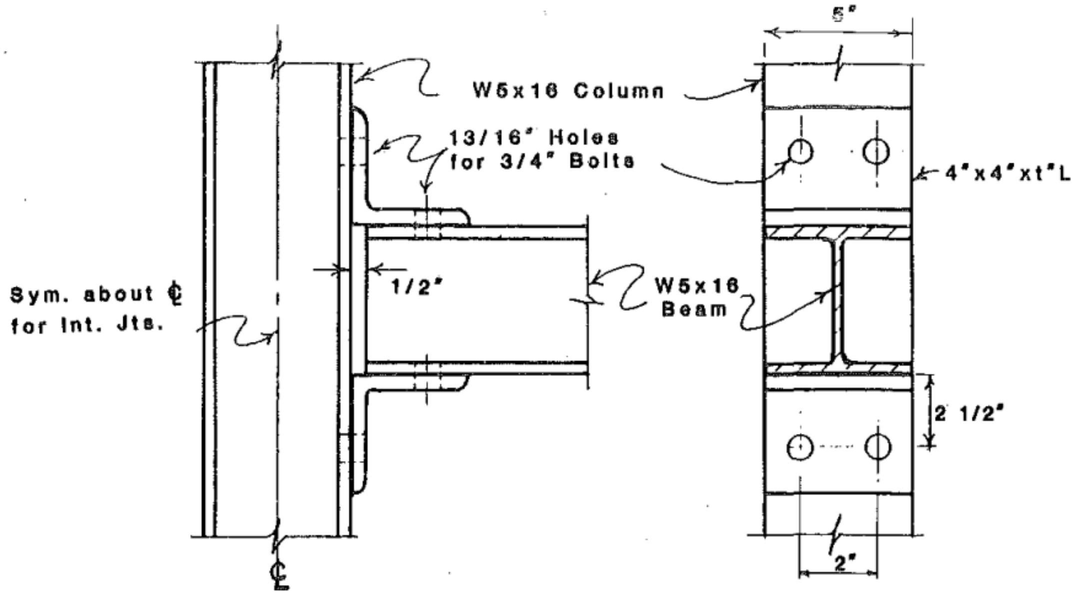
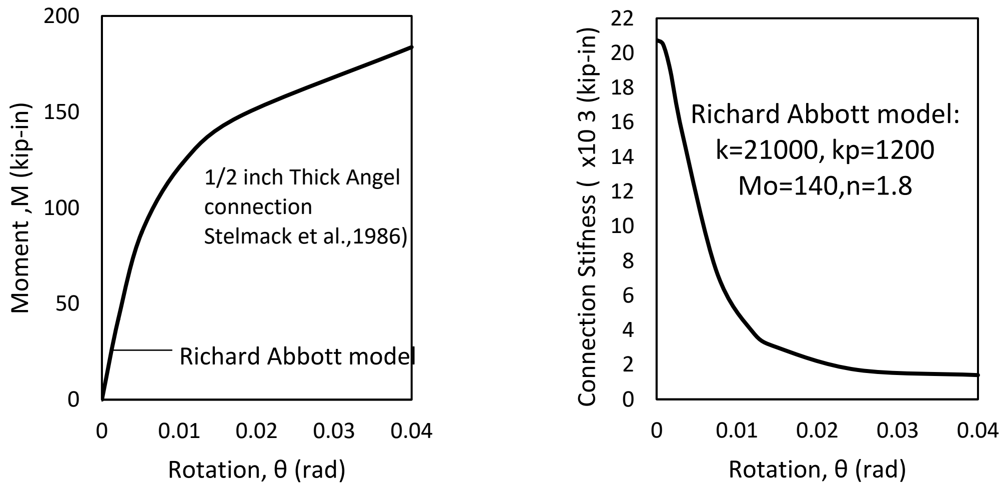



*Figure 7. 1/2 inch thick angle connection [30]*

The non-linear behaviour (M-θ and $R_k$-θ) of Richard-Abbott model (equation (11)), in which the required parameters of the rotational stiffness-rotation curve (Figure 8a) and the Moment-rotation curve (Figure 8b) were used.

The evaluation of the nonlinear behaviour of the semi-rigid connection through their cyclic bounding surface [30] shown in Figure 9, is used. The preceding equation (11) may then have the following expression (14):

$$M = M_a - \frac{(R_{ki} - R_{kp})(\theta_{ca} - \theta_c)}{\left\{1 + \left|\frac{(R_{ki} - R_{kp})(\theta_{ca} - \theta_c)}{2M_0}\right|^n\right\}^{\frac{1}{n}}} - R_{kp}(\theta_{ca} - \theta_c) \quad (14)$$




(a) Moment-Rotation behavior (M-θ)

(b) Stiffness-Rotation behavior ($R_k$-θ)

*Figure8. Constitutive behaviour.*

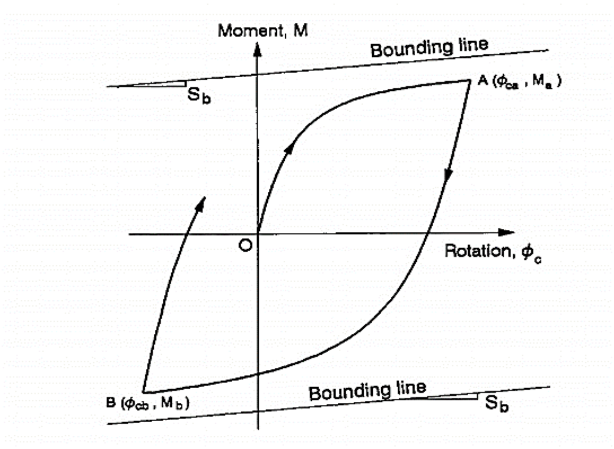



*Figure 9. The Bounding surface model [30]*

Considering an undamped structure, the dynamic analysis is conducted for three cases of connections: rigid, semi-rigid linear (k=21000 kip-in/rad) and semi-rigid nonlinear case

The roof level displacements obtained are compared with those of the reference [30], presented in the Figure 10 below.

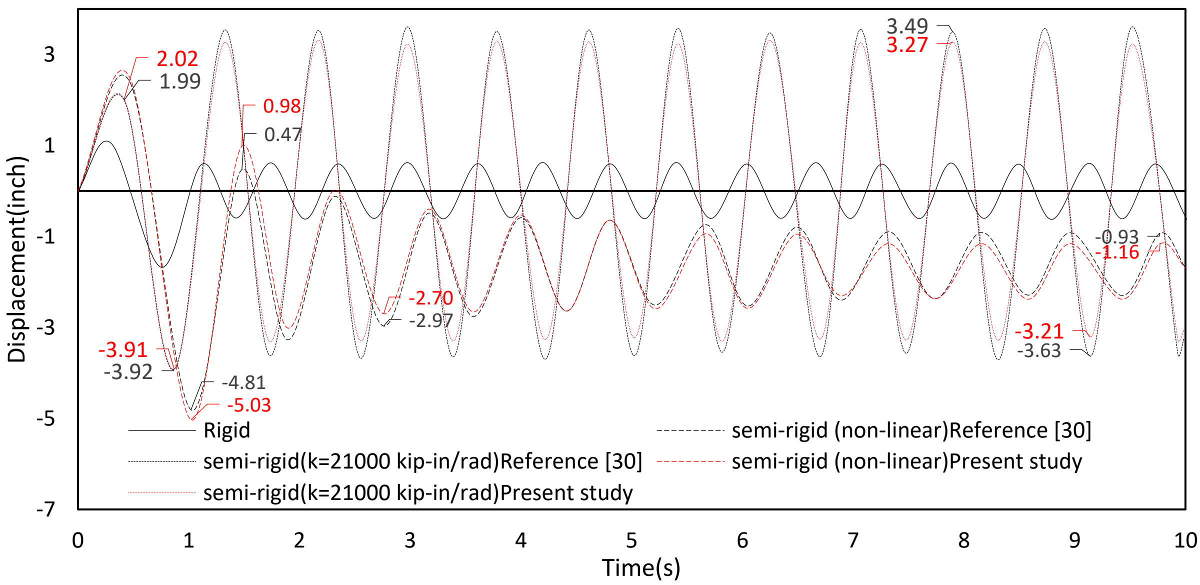



*Figure 10. Displacements in the rigid and semi-rigid case (k=21000 kip-in/rad)*

It may be observed that the response of the frame for the semi-rigid case simulated by these two models of linear and non-linear connections, are close. The error ratio varies between 0 and 8.5% for the linear case (Table 1) and between 0 and 21.28% (Table 2) for the non-linear case. This can be explained by the fact that the results are obtained graphically from reference [30] and therefore subjected to errors

*Table 1. Comparison of displacements for the linear semi-rigid case (k=21000 kip-in/rad)*

| Time (s) | 0.46 | 0.86 | 1.32 | 1.73 | 2.17 | 2.55-10 |
|---|---|---|---|---|---|---|
| Reference [30] displacement (inch) | 1.99 | -3.92 | 3.53 | -3.62 | 3.52 | ≈ 3.52 |
| Present study displacement (inch) | 1.99 | -3.91 | 3.27 | -3.31 | 3.31 | ≈ 3.30 |
| Error (%) | 0.00 | 0.26 | 7.37 | 8.56 | 5.97 | ≈ 6.25 |

*Table 2. Comparison of displacements for the non-linear semi-rigid case*

| Time (s) | 0.46 | 1.02 | 1.89 | 3.17 | 4.80 | 5.21 | 5.67 | 7.74 | 9.43 | 9.77 |
|---|---|---|---|---|---|---|---|---|---|---|
| Reference [30] displacement (inch) | 2.39 | -4.81 | -3.27 | -0.48 | -0.64 | -2.51 | -0.74 | -2.37 | -2.30 | -0.92 |
| Present study displacement (inch) | 2.48 | -5.03 | -3.01 | -0.40 | -0.64 | -2.59 | -0.94 | -2.37 | -2.37 | -1.16 |
| Error (%) | 3.63 | 4.37 | 7.95 | 16.67 | 0.00 | 3.09 | 21.28 | 0.00 | 2.95 | 20.69 |

*Table 3. Parameters for different types of connections*

| Analysis case | Rigid | Linear semi-rigid | Non-linear semi-rigid |
|---|---|---|---|
| Peak number | 31 | 24 | 23 |

In figure 10 concerning the steel frame with semi-rigid connections, the amplitude is larger compared to the rigid connection as shown in Table 3. In addition, the steel frame with the non-linear behaviour dampens and has an irreversible deviation because of the presence of continuous and permanent rotations at the joints.

## 7. Practical case of study

In the Algerian Seismic Regulations (RPA99V2003) [39] semi-rigid connections are not mentioned in a clear way compared to the strict requirements, regulations and codes of safety checks. Some safety objectives of the structure are assumed to be satisfied if the criteria for seismic joints and deformations are simultaneously satisfied (or not). A comparison with Eurocode 8 (EC8) [40] using the same criteria is conducted.

### *7.1. Terms of use*

*7.1.1. Verification of the seismic joint [39]*

The maximum displacements δ1 and δ2 of two blocks calculated at the top of the lowest block include the components due to the torsion and possibly the rotation of the foundations. Seismic joints separating them from minimum width $d_{min}$ must satisfy the following condition:

$$d_{min} = \begin{cases} 15 + (\delta_1 + \delta_2) \geq 40 \text{ mm} & \text{for RPA} \\ \left(\sqrt{{\delta_1}^2 + {\delta_2}^2}\right) * 0.7 & \text{for EC8} \end{cases} \quad (15)$$

*7.1.2. Verification of deformations*

The relative lateral displacements $\Delta_k$ of one level in relation to adjacent floors shall not exceed 1.0% of the floor height for RPA [39] and 2.0% for EC8 [40] could be calculated as:

$$\Delta_k/\mathrm{h}_k \quad \leq \begin{cases} 1\% & \text{for RPA} \\ A & \\ 0.01/\upsilon & \text{for EC8} \end{cases} \tag{16}$$

Where
$h_k$: height of the floor « k »
υ: reduction factor (υ=0.5)

It is proposed through the example of reference [9] to show the influence of the flexibility of the connections on the static and dynamic behaviour of steel structures. The results will be compared with those of the RPA [39] and EC8 [40] codes to show in which situations do the codes agree or not with the calculation results.

### 7.2. Example

The steel frame [9], which has the geometric and material properties shown in Figure 11, is ten levels of 40 m height and 8 m width. Young's modulus is E= 210 GPa, and the structure contains lumped masses in each joint, for the upper level the lumped mass is equal to 6.0 $kNs^2/m$ and for the other, the lumped mass is equal to 8.0 $kNs^2/m$.

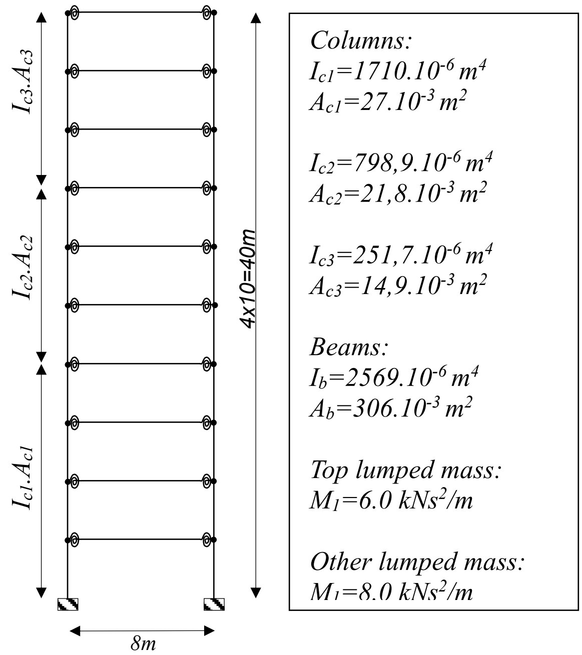



*Figure 11. Multistory frame [9].*

The structure is assumed to be subjected to the Boumerdes seismic excitation [41] (PGA = 0.550 g) as shown in Figure 12. The different parameters defining the types of connections are shown in Figure 13 and given in Table 4 [35].

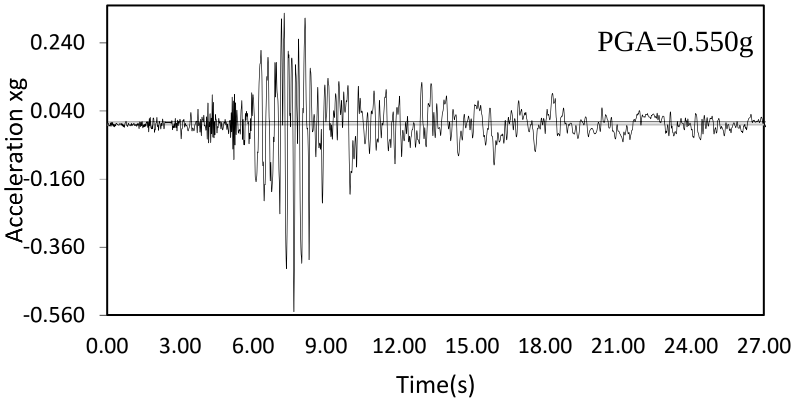



*Figure 12. Boumerdès earthquake (DarBeida station PGA=0.550g) [41]*

*Table 4. Parameters of different types of connections [35]*

| Connections types | $R_{Ki}$(kN.m/rad) | $R_{Kp}$(kN.m/rad) | $M_p$(kN.m) |
|---|---|---|---|
| T-stub (T-S) | 445220 | 9781 | 260.90 |
| End plate (EP) | 62150 | 2509 | 209.05 |
| Top and seat angle (TSA) | 41019 | 3390 | 83.62 |
| Welded top plate (WTP) | 36160 | 2712 | 83.62 |

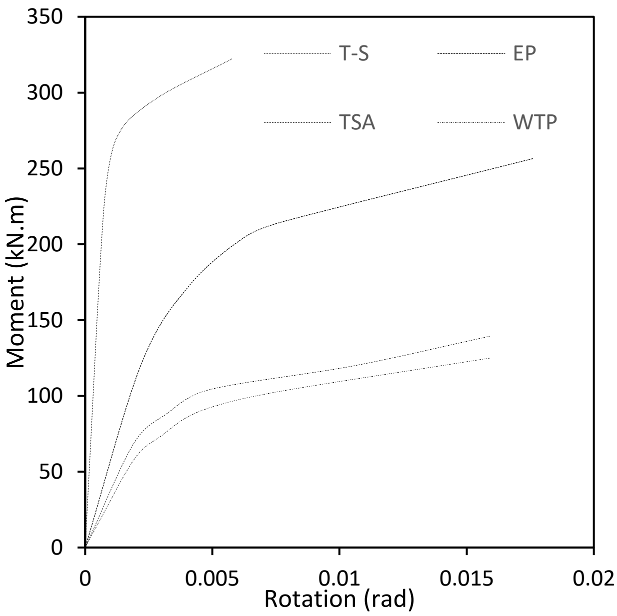



*Figure 13. M-θ curves for different types of connections*

The results of the non-linear temporal analysis of the structure generated by the developed calculation program are presented in the tables (5-7) and the figures (14-16) in which the Periods and the Max displacement are shown (Figure 15) together with the drift ratio in the x direction (Figure 16).

The Periods for the first three calculated vibration modes are presented in Table 5. Noting that the magnitude of the periods has a proportional relationsipt with the connections' flexibilities: the WTP connection gives the highest period (T1 = 1.209s) for the first mode of vibration.

*Table 5. Eigenvalues obtained according to the connection type*

| Connections types | Periods | | | | | |
|---|---|---|---|---|---|---|
| | 1 | | 2 | | 3 | |
| | Sap2000 | Present study | Sap2000 | Present study | Sap2000 | Present study |
| Rigid | 1.006 | 0.993 | 0.364 | 0.359 | 0.205 | 0.202 |
| T-stub (T-S) | 1.025 | 1.013 | 0.370 | 0.365 | 0.208 | 0.205 |
| End plate (EP) | 1.136 | 1.125 | 0.403 | 0.399 | 0.227 | 0.224 |
| Top and seat angle (TSA) | 1.196 | 1.186 | 0.421 | 0.417 | 0.236 | 0.234 |
| Welded top plate (WTP) | 1.219 | 1.209 | 0.428 | 0.424 | 0.240 | 0.238 |

The nonlinear dynamic analysis of the structure in the different cases of connections using the Boumerdes accelerogram [41] giving the results in the form of displacement at the Top of structure are presented respectively in Figures 14 and 15 and Table 6.

The structure with nonlinear connections has larger amplitudes and periods than that in the rigid case. Compared to the parameters in Table 5, these displacements are larger in the case of E-P connection compared to T-S connection. On the other hand, the amplitude of the TSA connection is larger than that of the WTP connection, although the initial stiffness of the WTP connection is lower than that of TSA connection. This can be explained by the fact that the plastic moment of the WTP connection is equal to that in the TSA connections case and could then influence the global response of the structure.

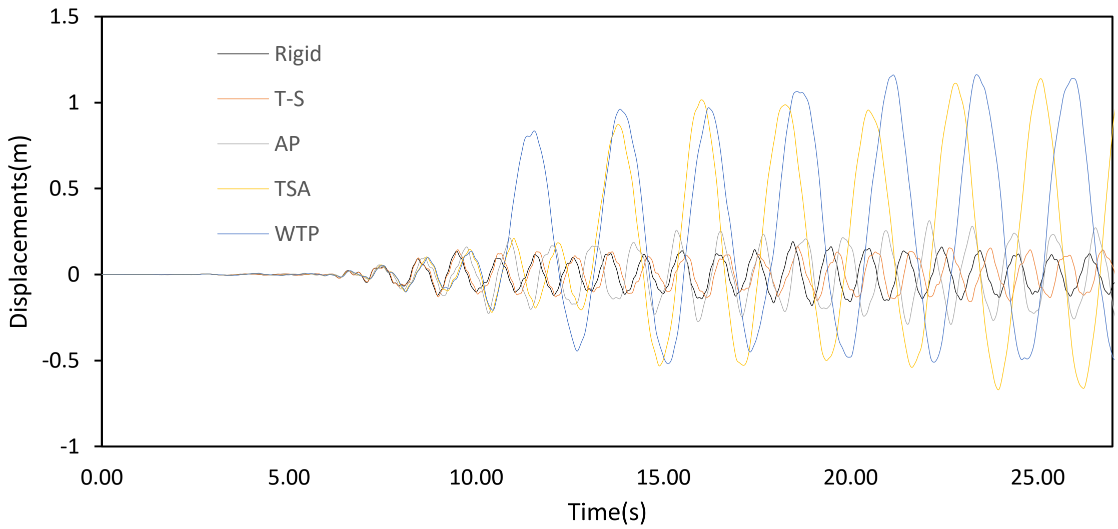



*Figure 14. Displacements at the roof level.*

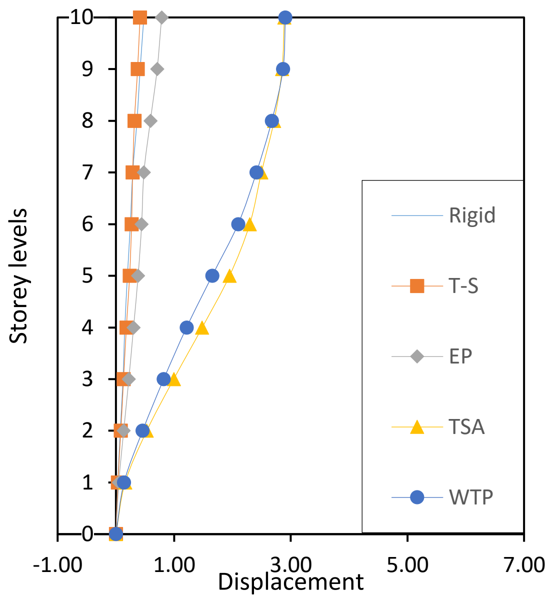



*Figure 15. Maximum displacement by level.*

However, both the RPA and the EC8 specifications are particularly relevant to the seismic joints and drift ratio. For the two types of connections (TS, EP), it is noticed (Figure 15) that the displacements are close and on the safe side compared to the other types of connections where it increases significantly: for the rigid case the seismic joint can reach the value of 398.16 mm calculated using the recommendation given by the RPA [39] and 189.65mm calculated using EC8 [40]. For the other types of connections, the seismic joint increases progressively according to the connection flexibility, as shown in Table 6. In all cases of connections, the seismic joint calculated by RPA [39] is almost as twice as that calculated by EC8 [40].

The evaluation of the drift ratio of the studied structure is presented in Figure 16. The regulation checks of the codes RPA [39] and EC8 [40] are given in table 7. The profiles of the drift ratio of all the cases of connections studied are compared with the required limit (1% of the floor height for the RPA) and (2% of the floor height for the EC8) identified by the vertical lines in Figure 16.

It is noted that for the rigid and TS connections, the deformation conditions are satisfied for the two codes which are considered. On the other hand, for the EP connection, the deformation condition of the RPA regulation is not verified (NotVer) for the levels 8 and 9 where the 1% of the height is exceeded, but it is verified (Ver) for EC8. For the TSA and WTP connections, the conditions of the two codes is not verified (NotVer) because most levels exceed the 1% for RPA except level 10 and the 2% for EC8 except for levels 1.9 and 10 as shown in Table 7.

*Table 6. Comparisons of seismic joints*

| Connection type | RPA | EC8 |
|---|---|---|
| Full Rigid | 398.16 mm | 189.65mm |
| T-S | 342.41 | 163.10 |
| EP | 641.03 | 309.12 |
| TSA | 2321.27 | 1141.69 |
| WTP | 2339.58 | 1149.27 |

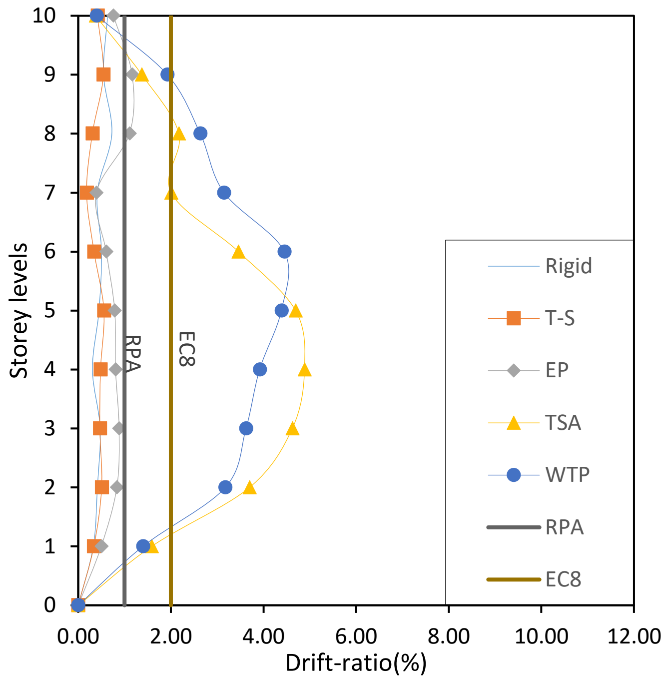



*Figure 16. Investigation of the design inter-story drift-ratio*

*Table 7. Verification of deformations*

| | EP | | TSA | | WTP | |
|---|---|---|---|---|---|---|
| | RPA | EC8 | RPA | EC8 | RPA | EC8 |
| Level 10(mm) | Ver. | | | | | |
| Level 09(mm) | 1.16 ( NotVer ) | Ver. | 1.37 ( NotVer ) | Ver. | 1.93 ( NotVer ) | Ver. |
| Level 08(mm) | 1.11 ( NotVer ) | Ver. | 2.17 ( NotVer ) | | 2.64 ( NotVer ) | |
| Level 01(mm) | Ver. | | 1.59 ( NotVer ) | Ver. | 1.40 ( NotVer ) | Ver. |

## 8. Conclusion

The results presented in the current research work focused on the effect of semi-rigid connections coupled with the effect of inertia on steel structures under dynamic loading. A mechanical model considering the non-linear behaviour and rigidity of the joints is proposed.

The validation of the model is undertaken through the application of the model to an example taken from the literature. Three types of connections are considered, rigid linear, semi-rigid linear and semi-rigid non-linear. The frame is submitted to dynamic loading. The results obtained in terms of displacements showed a convergence.

This model is used for the verification of the safety recommendations of the Algerian seismic rules (RPA 99 v2003) in terms of deformations and seismic joint. The results obtained show that the semi-rigid connections generate relative displacements that exceed the limits allowed by the RPA. Compared with EC8, these connections are satisfactory. In addition, the dimensions of the seismic joint recommended by RPA gives values higher than those recommended by the EC8 relative to the different types of connections.

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