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Influence of Semi-Rigid joints and Roof Slopes on the Cyclic Behavior of Gabled Frame

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Abstract – This study investigates the impact of semi-rigid joint behavior on the cyclic response of gabled structures with different roof slopes. A numerical analysis is conducted on gable frames featuring both rigid and semi-rigid column-beam joints. The cyclic behavior is examined using the ANSYS V14.5 software, with the Monforton and Wu model employed to simulate the linear response of semi-rigid joints. The primary objective of this research is to evaluate how joint flexibility influences the overall structural performance, particularly in terms of ductility, stiffness degradation, and energy dissipation under cyclic loading. The results indicate that gabled structures with high-pitched roofs and rigid joints exhibit significant flexibility and improved energy dissipation compared to similar structures with lower slopes. However, introducing semi-rigid joints further enhances the ductility of the structure, allowing for better absorption and dissipation of cyclic forces due to their increased rotational capacity. Moreover, the findings reveal that semi-rigid joints contribute substantially to the overall deformation capacity of gable frames, especially in low-slope configurations. In such cases, the structural response demonstrates pronounced ductility, making semi-rigid joints a good option for improving seismic resilience. Conversely, in steep-slope gables, the influence of semi-rigid joints on overall behavior is less significant. These insights contribute to optimizing structural design by adjusting joint stiffness to specific slope conditions.

Keywords – Steel Structures; Semi-Rigid Joints; Numerical Analysis; Cyclic Response; Gable Frame.

I. INTRODUCTION

Steel structures consist of elements connected to each other through joints or assemblies. These connections have been the subject of extensive research in order to determine the impact of their behavior on the overall response of the structures. Understanding the role of joints, particularly semi-rigid ones, is crucial for accurately predicting the performance and resilience of steel frames under various loading conditions. The methods for analyzing semi-rigid connections have made significant advancements, allowing for a better understanding of their true behavior, which can then be integrated into the overall analysis of structures. Since the 1930s, numerous studies have been conducted to predict the behavior of these joints. These studies have shown that semi-rigid joints exhibit a nonlinear moment-rotation [M-ϴ] response [1].

Monforton and Wu [2] were the first to incorporate the effects of semi-rigid connections into the stiffness matrix method in 1963. This was achieved by modifying the stiffness matrix of a beam to account for the effect of the semi-rigidity of the joint in the analysis of frames. Since then, several models have been developed to predict the mechanical behavior of connections. In 1969, Lionberger and Weaver [3] studied the dynamic behavior of frames with semi-rigid joints, and in 1987, Lui and Chen [4] proposed semi-rigid frame analysis methods based on the stiffness matrix approach.

Other empirical models such as Frye and Mouris [5], Kukreti et al. [6], [7], and analytical models like WF Chen et al. [8] have been the subject of extensive research aimed at understanding the real behavior of joints (moment-rotation). Experimental models are generally the most suitable for describing this aspect, although they are costly. On the other hand, mechanical models based on component methods and numerical models can complement the lack of experimental data [9]. The most recognized of these are those by Faella et al. [10], da Silva [11], [12], Nethercot and Zandounini [13], and Jaspart et al. [14]. These studies led to the publication of the first edition of Eurocode 3 in 2005 [15], which addresses the issue of semi-rigid joints in its Annex "J," based on a mechanical model founded on the component method.

Further research has been conducted to better understand the impact and contribution of semi-rigid connections under various loading conditions on the overall behavior of steel structures. Nogueiro et al. [16] implemented the calibration of the modified Richard-Abbott model to determine the cyclic response of a joint with end plate. Bernuzzi et al. [17], Abolmaali et al. [18], and Elghazouli et al. [19] have studied the influence of geometric and mechanical characteristics of joint components on their behavior under cyclic loading. Stoakes et al. [20] investigated the effect of an upper gusset plate in a beam-column connection with an end plate under cyclic loading. Loureiro et al. [21] addressed the interaction between the minor axis and major axis effects on the cyclic response (stiffness and strength) of a three-dimensional joint. These works have been incorporated into structural analysis to observe the effect of joint semi-rigidity on global response.

Static and dynamic analyses have been performed to predict structural behavior. These studies have been conducted on several structures with semi-rigid joints. AI-Bermani et al. [22] addressed the dynamic response of steel frames and determined the hysteretic damping by considering the nonlinear behavior of the joints using the "Bounding line model." Da Silva and Vellasco [23], [24] evaluated the effectiveness of semi-rigid joints and their influence on the economic performance of structures, accounting for both geometric and material nonlinearities. Mathe et al. [25] studied the impact of simplifying the nonlinear behavior of semi-rigid joints (Kishi and Chun law) on the dynamic response of framed structures. Similar cases were treated to determine the advantage of including semi-rigidity in the overall analysis of structures, as discussed by Bhatti [26], Stamatopoulos [27], Koriga et al. [28], Lu, Shengcan and Wang [29] and Karakurt et Meryem [30].

Masoodi and Moghaddam [31], [32] studied the effect of linear behavior semi-rigid joints in a gabled structure under seismic loading to determine the structural response. They concluded that this type of structure exhibits good performance under dynamic and seismic loading.

In light of these studies, it appears that further research is still needed in this field, particularly regarding the behavior of structures with sloped roofs and semi-rigid joints, which serves as the foundation for our work.

This study focuses on the most common type of steel structure, which is the industrial gabled frame, taking into account the effect of semi-rigid connections and the roof slope on the cyclic behavior of the structure.

II. MODEL DESCRIPTION

The structure studied is a gable frame with a fixed base, where the connection between the beam and the column is modeled by a linear rotational spring with negligible length. Its stiffness is calculated according to Monforton and Wu, 1963 [2], as illustrated in Table 3. The dimensions of the structure and the geometric characteristics of the section of the elements used are shown in Figure 1 and Table 2.

The roof slope is provided in the table. 1

The roof Slope %			
S=0,0%	S=20%	S=40%	S=60%

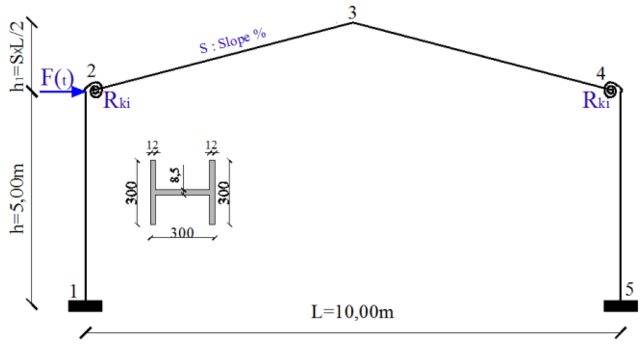


Fig.1 : The studied structure

Table 2: Geometrical characteristics of the element sections.

h (mm)	b (mm)	tf (mm)	tw (mm)	The inertia of section I (cm^4)	Area section A (cm^2)
300	300	12	8,5	16340,1984	94,08

A. Properties of the joints:

The semi rigid joints are incorporated in the model as a rotational spring with a linear moment-rotation (M-Ө) behavior; With variable stiffness given by Monforton and Wu, 1963 [2], expressed by the following equation [1] and [2]. The results are provided in Table 3.

$$\boldsymbol{M_i = R_{ki} \times \theta} \qquad [1]$$

$$\boldsymbol{R_{ki} = \frac{3.E_b.I_b}{L_b} \times \left[\frac{r}{1-r}\right]} \qquad [2]$$

Table 3: Stiffness value of semi-rigid joints

$\left[\frac{3\times r}{1-r}\right]=0{,}5$	$\left[\frac{3\times r}{1-r}\right]=5$	$\left[\frac{3\times r}{1-r}\right]=20$	$\left[\frac{3\times r}{1-r}\right]=\infty$
$R_{ki} = 1396\ kN.m$	$R_{ki} = 13957\ kN.m$	$R_{ki} = 55829\ kN.m$	$R_{ki} = \infty\ kN.m$

R_{ki}: is the linear rotational stiffness of the joints.

According to the Eurocode 3 [15] which defines two limits at which the consideration of their stiffness in the structural analysis becomes necessary and significant. A joint is defined as rigid when its stiffness exceeds 25EI/L, and as a hinged joint when its stiffness is less than 0.5EI/L. Between these two limits, the joints are considered semi-rigid.

B. Material Properties

The material is assumed to have a bilinear kinematic behavior with work hardening (Von Mises criterion), as shown in Figure 3, with an analysis. The elasticity modulus $E_{el} = 2{,}05 \times 10^{5}$MPa with a yield stress fy = 240MPa and the plastic modulus $E = 0.02E_{el}$

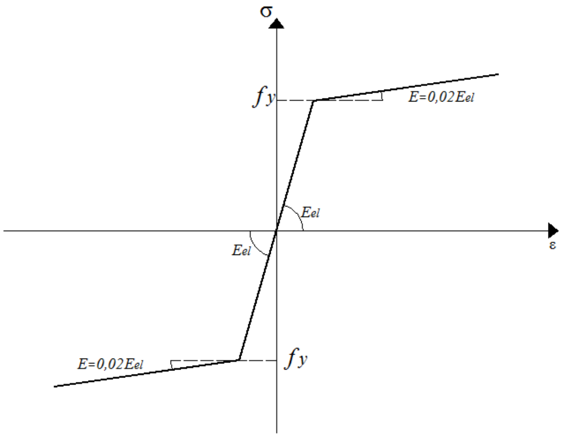


Fig 3. contrainte-déformation curve.

III. CYCLIC ANALYSIS

The load is applied at the top of the column and is shown in Figure 2 with a load increment ΔF = 5 kN

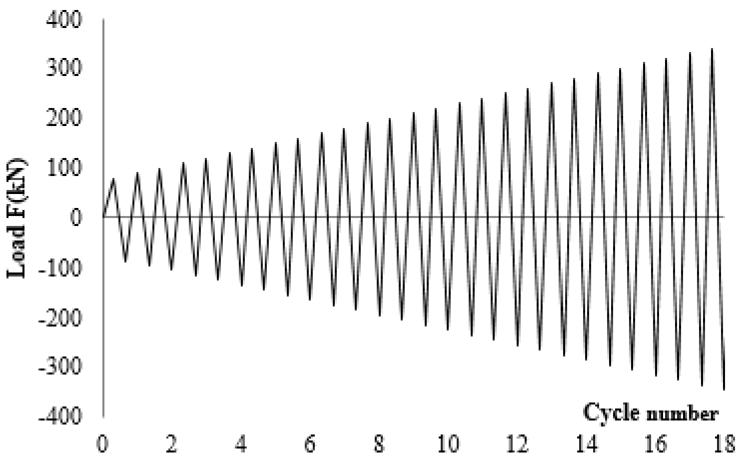


Fig 2. Cyclic loading description

The main objective is to assess the structural displacement at the top of the column (point 2) subjected to a cyclic load "F(t)" as shown in Figure 2, applied at point 2 (Figure 1). This analysis is conducted for each roof slope given in Table 2, considering both rigid and semi-rigid connections. A nonlinear kinematic analysis is performed using the ANSYS V14.5 software.

In the first part, the study focuses on evaluating the effect of integrating semi-rigid connections in the analysis of the gable frame by varying both the roof slope and the connection stiffness (beam-to-column joint, as shown in Figure 1).

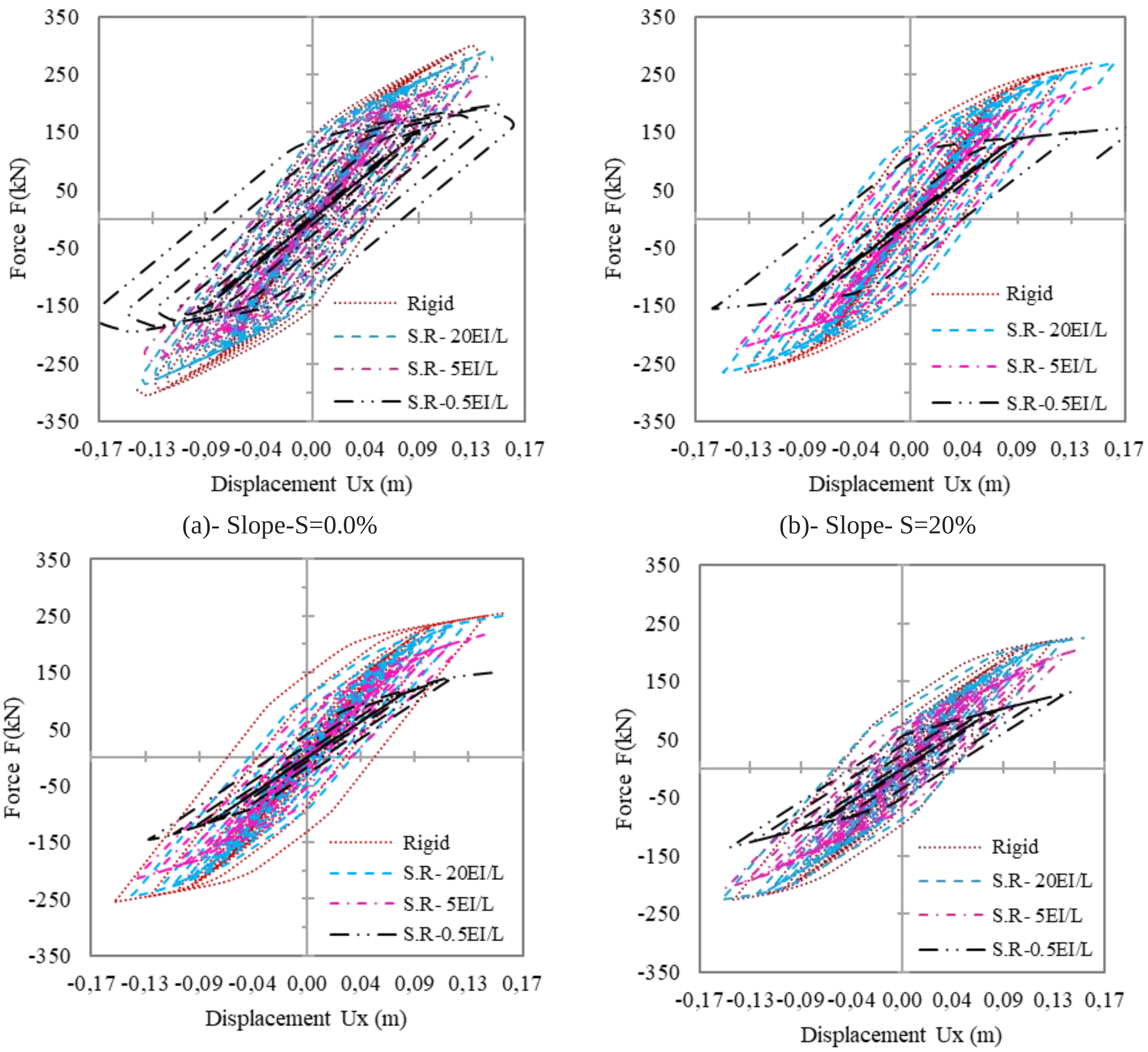
The Algerian steel structure design code CCM97 which is based at the EC03 sets a limit on the maximum allowable displacement.

$U_x \leq \frac{h}{150}$: is the high of the structure

In our case, the maximum displacement is limited to the allowable displacement multiplied by 5 (equation [3]), in order to reach the plastic displacement.

$$U_{x.max} = 5 \times \frac{h}{150} \qquad [3]$$

IV. RESULTS AND DISCUSSION



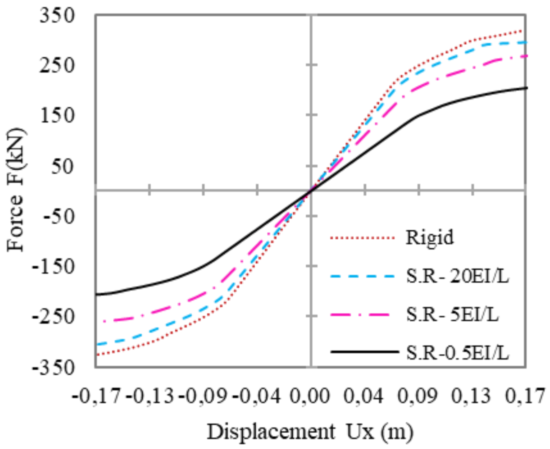
(a)- Slope-S=0.0% (b)- Slope- S=20%

(c)- Slope- S=40% (d)- Slope- S=60%

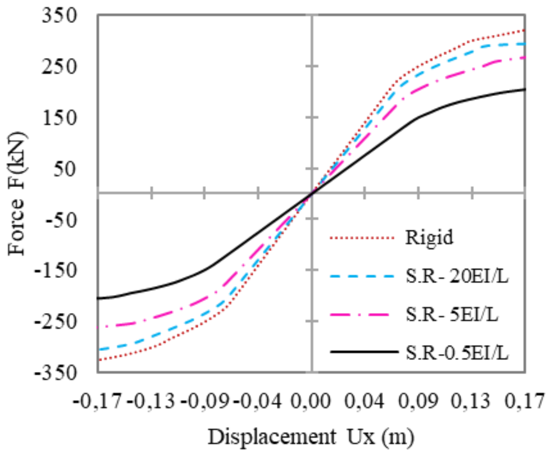
Fig. 4: Hysteretic response at the top of the column as a function of the variation in stiffness connection and roof slope.

From the hysteresis loops illustrated in Figure 4, we observe that the structure with semi-rigid connections becomes increasingly dissipative compared to the same structure with rigid connections.

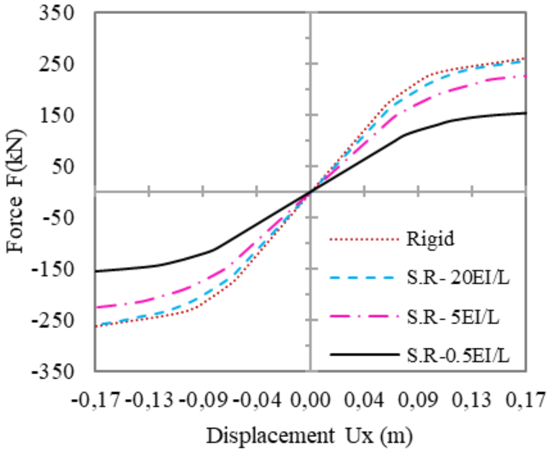
From Figures (a), (b), (c), and (d) in Figure 5, we observe that the structure with a simple frame (S = 0%) exhibits greater stiffness compared to the structure with a double-pitched roof (S = 20%, S = 40%, and S = 60%). Moreover, as the roof slope increases, the structure becomes significantly more ductile.



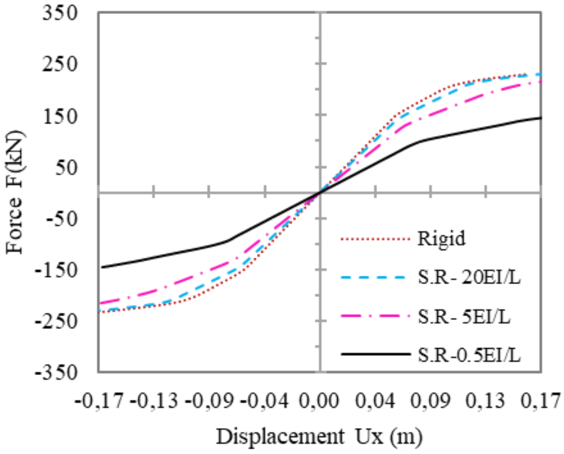
(a)- Slope-S=0.0%



(b)- Slope-S=20%



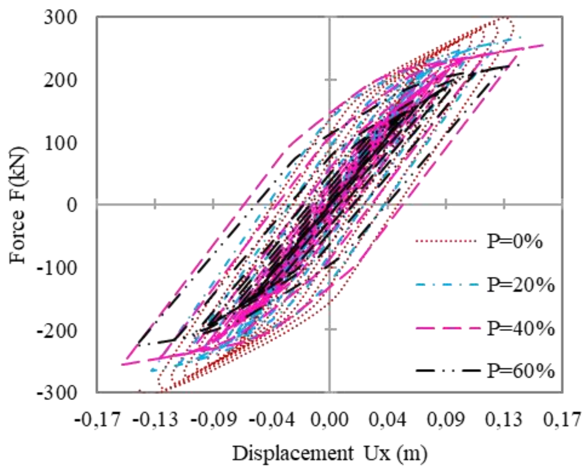
(c)- Slope-S=40%



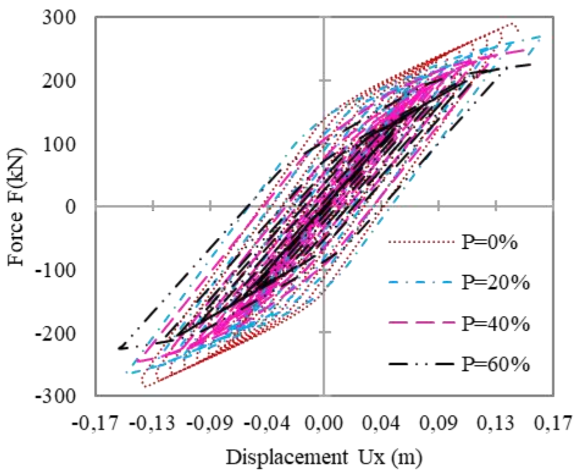
(d)- Slope-S=60%

Fig.5 : Envelope curves of the hysteresis loops from Fig. 4.

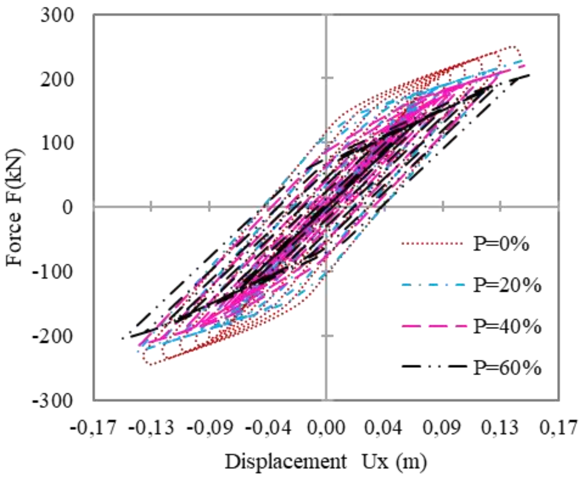
From the envelope curves in Figure 5, we observe that the structure with rigid-joint and slope of "S= 60%" exhibits nearly the same behavior as the same structure with semi-rigid joints with a stiffness of 20EI/L and 5EI/L. This leads us to conclude that the use of semi-rigid connections in high flexible structures (S= 60%) does not significantly affect their overall behavior.



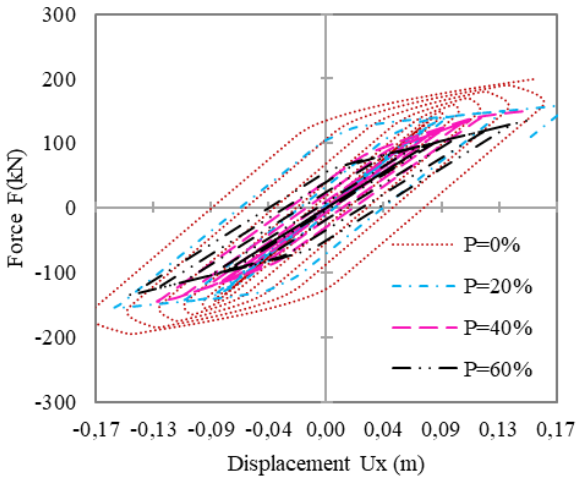
(a)- Rigid joints (R_{ki} =∞)



(b): Semi-Rigid joints (R_{ki} =20EI/L)



(c): Semi-Rigid joints (R_{ki}=5EI/L)

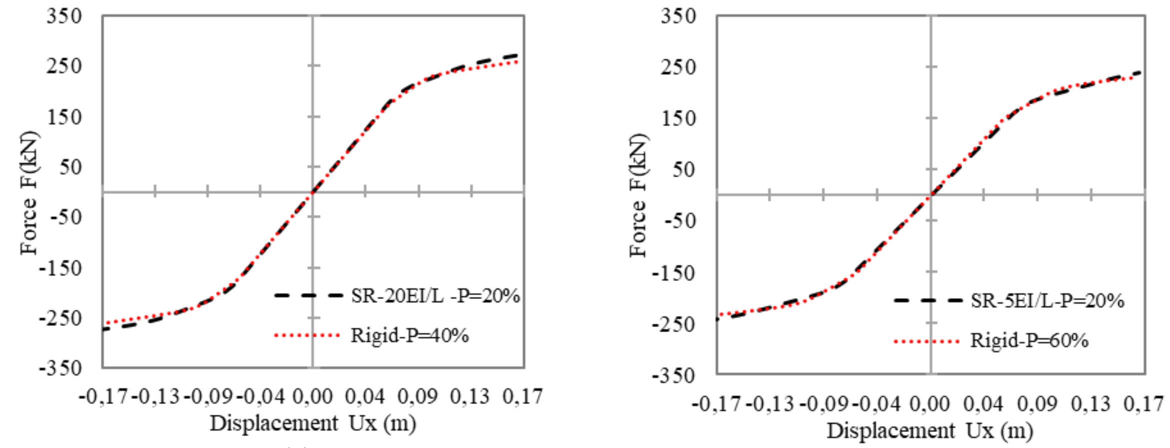


(d): Semi-Rigid joints (R_{ki}=0.5EI/L)

Fig. 6: Hysteretic response at the top of the column as a function of the variation in roof slope and connection stiffness.

From the hysteresis loops illustrated in Figure 4, we observe that as the roof slope increases, the structure becomes increasingly dissipative. Referring to Figure 5(a), we can say that as the slope increases, the structure becomes more and more ductile. Comparing Figure 5(a) to 5(d), we note that as the stiffness of the connections decreases, the structure tends to exhibit the same behavior regardless of the slope.

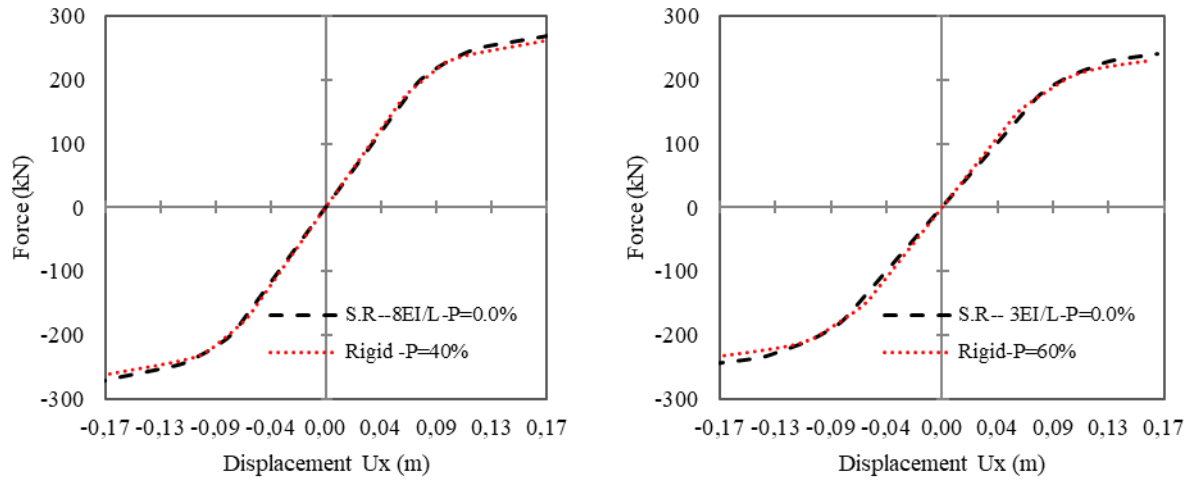
In the second part, the same structure (Fig. 1) was studied using the same data (geometric and mechanical properties) as mentioned earlier, with the aim of determining the equivalence between the traverse slope value and the semi-rigidity of the connections in terms of linear behavior in the overall structural response.



(a) (b)

Fig. 8: Comparison of the cyclic response (force-displacement envelope curve) at the top of the column between two structures with different slopes and connection stiffness.

Figures (8-a) and (8-b) show that for a structure with slopes of 40% and 60% with rigid connections, the behavior is similar to that of a structure with a 20% slope and semi-rigid connections with stiffness values of 20EI/L and 5EI/L, respectively.



(a) (b)

Fig. 9: Comparison of the cyclic response (force-displacement envelope curve) at the top of the column between two structures with different slopes and connection stiffness.

After analyzing the structure and based on the results from the envelope curve of the hysteresis loops shown in Figure 9, it can be inferred that the structure with rigid connections and slopes of 60% and 40% exhibits the same behavior as the structure with a 0.0% slope and semi-rigid connections with stiffness values of 8EI/L and 3EI/L, respectively.

V. CONCLUSION

In this study, a nonlinear cyclic analysis was conducted on a double-pitched structure with different slopes in order to determine the response of the structure, taking into account the connection stiffness using the Monforton and Wu model.

- The higher the slope, the more considerable the energy dissipation;
- The same observation can be made for the case of semi-rigid jointed structures;
- It can also be concluded that frames with semi-rigid joints are more dissipative than those with rigid joints for the same frame study case;

- The envelope curves of the hysteresis loops converge when the roof slope exceeds 60%, especially for joints with stiffness greater than 5EI/L;
- In the case of a 40% slope with rigid joints, the results are almost identical to those of a structure with a 20% slope and 0% slope, with connection stiffnesses of 20EI/L and 8EI/L, respectively;
- The same observation holds for a 60% slope with rigid joints, which corresponds to slopes of 20% and 0% with connection stiffnesses of 5EI/L and 3EI/L.

In summary, considering semi-rigid joints in the analysis of industrial structures provides greater ductility and energy dissipation. Additionally, industrial structures with steeper slopes dissipate energy more effectively than structures with lower slopes.

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