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Location of semi-rigid connections effect on the seismic performance of steel frame structures

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ABSTRACT: In designing steel frames, combining semi-rigid and rigid connections can result in better structural performance, particularly in seismic locations. In this study, the effects of semi-rigid beam-to-column connections located on the seismic performance of steel frame structures are investigated. The analysis uses six and twelve-story moment resisting steel frames (MRSF) with rigid, semi-rigid, and dual beam-column connections. These frames are designed according to the Egyptian design codes. The Drain-2Dx computer program and seven earthquake ground motions are used in the non-linear dynamic analysis. The rotational stiffness of beam-to-column connections is indicated through the end fixity factors with a value equal to 0.6. The performances of these frames are evaluated through the roof drift ratio (RDR), the maximum story drift ratios (SDR), and the maximum column axial compression force (MACF). The results indicated that the quantities of fundamental periods, the roof drift ratio, the story drift ratio, and the column axial compression force are related to stiffness, rigidity, and the number of semi-rigid connections in steel frames.

Keywords: semi-rigid connections; end fixity factor; steel frames; seismic performance

1. INTRODUCTION

Computational mechanical and nonlinear analysis developments are provided structural engineers with general and systematic approaches for modeling and analyzing complex structures. Under both static and dynamic loads, a certain structure may be studied using a numerical simulation model that involves the effect of geometric and material nonlinearities. Computational process and nonlinear analysis developments have provided structural engineers with broad and systematic approaches for modeling and analyzing complex structures. In dynamic loadings, almost any complex model may be investigated using a numerical simulation framework that includes into consideration material and geometric nonlinearities. Despite advances in the analysis, the design procedure has remained unchanged. Most of today's designs are based on conventional trial-and-error methods, from which a structure is designed, examined, and evaluated for agreement with design requirements. If the structure's performance fails to satisfy the specified design requirements, the structure is redesigned. The design, analysis, and verification process continue until the design has been completed. In general, the final design isn't optimal in any way. The trial-and-error process is especially inefficient for sophisticated designs that go beyond the designer's intuition and experience.

The rotational stiffness of the beam-to-column connection plays a significant effect in the optimal design and response of the structure. The design elements and connections of structures take into account some improvements in the steel frame system. Most design responsibilities in structural engineering are based on the connections in the steel frame being fully rigid. Therefore, the level of flexibility of the connection in MRSF is ignored. Consequently, the predictions of structural response are inaccurate. Several kinds of research show that the real beam-to-column connections have some stiffness, in between the cases of fully rigid and ideal pinned cases. The semi-rigid effect on many parameters of the structure such as the frame drift, the moment distribution alongside the beams and columns, and the cost of the design frame structure [1-2]. Modern design codes such as Eurocode 3 [3] and the AISC-LRFD [4] Specification license semi-rigid connection should be considered in the analysis to provide a correct stiffness of the structure and give more accurate results.

The behaviors of semi-rigid connections are investigated by several researchers in recent years. Akbas and Shen [5] investigated the seismic behavior of steel buildings with combined rigid and semi-rigid frames. 5- and 10-stories SMRFs are designed according to the LRFD (1995) code. The DRAIN-2DX program is used for nonlinear dynamic time history analysis of the two-dimensional models of the frames. The results indicated that in high seismicity regions might be used bolted semi-rigid steel frames with rigid steel MRFs. To simulate the semi-rigid response of the connections, the mathematical representation through the end-fixity factor and the modified stiffness matrix were used to merge such behavior into structural analysis packages. To confirm the written program, a computer-based analysis was conducted using PROKON software and comparing analysis results with that obtained from the excel spreadsheet. It demonstrates that Excel's results were perfectly exact. Consequently, the procedure of establishing spreadsheets as finite element analysis software for a certain form of frames demonstrates its validity.

Kartal et al. [6] investigated the effects of the semi-rigid connection of the steel braced RC frame, steel truss, and prefabricated structural system responses. SEMIFEM program was used in numerical analysis. The semi-rigid connections were defined by the rotational spring stiffness-connection ratio of structural members connected. The results indicated that semi-rigid connection degrees are an important factor in structural systems and their effect differs from one structure to another. Ghassemieh et al. [7] investigated how the flexibility of the extended end-plate connections influences the 4-, 8-, and 16-story steel moment frames. ABAQUS program was used for the frame models' nonlinear static pushover and incremental dynamic analyses. The results indicated that by increasing the connection flexibility, the strength and stiffness of the frame are reduced. So, the natural period is increased. Feizi et al. [8] investigated steel frames with three, eight, and fifteen stories with rigid, semi-rigid, and dual beam-column connections under seismic force. The Drain-2Dx computer program and five earthquake ground motions are used in the non-linear dynamic analysis. The results indicated that in general, the seismic performances of dual-frame models are better than that of the rigid frame.

Sagiroglu and Aydin [9] studied the nonlinear behavior of beam-to-column connections in different designs of steel frames by the MRVSSF program. The top and bottom angles with double web angle connection types and the Frye-Morris polynomial model is used to describe the non-linear behavior of semi-rigid connections. The results indicated that in the semi-rigid connection cases, the beam's end-moments decrease, and the column's end moments increase compared with the rigid connection cases. Ana [10] investigated the behavior of semi-rigid connections in 4- and 6-stories steel structures under seismic loads. These frames are designed according to Romanian design codes. Different types of connections with different degrees of semi–rigidity are used in the analysis. The results indicated that by increasing the flexibility of connections, the lateral displacements are increased.

Bayat and Zahrai [11] investigated the seismic performance of steel frames with rigid and semi-rigid connections under five earthquake records. 10, 15, and 20-story steel frames are modeled, designed, and nonlinear analyzed by ETABS software. The analysis results showed that by using semi-rigid connections, the base shear decreases, and smaller sections of beams and columns can use and leading to reduced cost. Nandeesha and Kashinath [12] investigated the effect of end fixity factors of joints on multi-story steel space frames under static loads. The results indicated that the structural behaviors of the frame depend on the type of connections. Also, the end fixity factors from 0.60 to 0.70 are the best range for beam design.

Guha et al. [13] investigated the semi-rigid beam-column connection effect on the response of the frame structure. SAP2000 program has been used in the analysis.
In contrast to rigid connections, semirigid connec tions were expected to enhance displacements, M idspan moments, and end moments while decreas ing end moments and improving the seismic resp onse of the structure.

Van et al. [14] investigated the effects of connection flexibility with different end-fixity factors on plane steel frame structures. MATHCAD software programming was used in the analysis of numerical examples. The analyses show that the behaviors of actual structures are the best compared to both pinned and fixed connections. Farhadi and Anvarsamarin [15] evaluated the nonlinear dynamic response of six, twelve, and eighteen-story steel moment frames with different rigidity of connections under Far-Field earthquake records. Seismo-Struct software was used in the analysis. The obtained results indicated that the dispersion of the collapse fragility curve and the fundamental period of frames are increased by decreasing the rigidity of the beam-column connections. Rigia et al. [16] studied the seismic performance of five, ten, and fifteen-story of rigid and semi-rigid steel moment-resisting frames under the twenty-two pairs of far-field earthquake ground motions existing in FEMA P695. The OpenSees computer program uses non-linear static and incremental dynamic analysis. The results indicated that the lateral stiffness and strength will be calculated to be lower with the more accurate rigidity modeled of the structural frame.

Hou et al. [17] investigated experimental and numerical detection, of semi-rigid connections damage in space frame structures in a two-step method. The sparsity of the damage can be used to identify the presence and position of the damaged component. Identifying the location and degrees of damage to the damaged elements or joints is the goal of the next condition monitoring step. According to the numerical analysis, joint damage has a much smaller impact on the modal characteristics than damage to bar members.

Most studies are based on semi-rigid connections in the design and analysis of frame structures. Although the semi-rigid connections are the source of the structure ductility level increased the story drifts. Some research started to use the combined rigid and semi-rigid connection (dual frame) to take advantage of the two types of connections and to reduce the cost of structure design [18]. This paper focuses on a study of the effects of semi-rigid beam-to-column connections location for the performance of steel frame structures under nonlinear dynamic analysis. The analysis uses six and twelve-story moment resisting steel frames with rigid, semi-rigid, and combined configurations. These frames are designed according to the ECP-201 [19] and ECP-205 [20]. The rotational stiffness of beam-to-column connections is indicated through the end fixity factors with a factor equal to 0.6. The performances of the MRSFs with strong columns and weak beams are evaluated with different locations of semi-rigid connections. Drain -2Dx software is used in the nonlinear dynamic analysis of all frame cases [21]. The performance of these frames is incident through the roof drift ratio (RDR), the maximum story drift ratios (SDR), and maximum axial compression forces (MACF).

2. CONNECTION CLASSIFICATION

Fully and partly restrained steel construction types are described by the American Institute of steel construction and load and resistance factor design specifications [22]. This specification requires that the connections of the partly restrained type constructions be considered flexible (semi-rigid) and, this flexibility be evaluated by a reasonable analysis or experimental works. On the other hand, three types of connection: rigid; semi-rigid, and normally pinned are proposed in Eurocode 3 [3]. Hence, there is not any information about semi-rigid connections in Egyptian steel design specifications [20]. Nader and Astaneh [23] indicated that rigid connections are capable of developing a moment at the beam end equal to or greater than 90% of the fixed end moment, while pinned connections can only develop a moment at the beam end less than 20% of the fixed end beam. Chen et al. [24] indicated that the end-fixity factor is the conventional characteristic to calculate the end restraints beam. This factor defines the rotation of the beam end divided by the joint rotation of the beam and the connection due to a unified end-moment. The equation to calculate the end-fixity factor, “r” is defined as:

$$r = \frac{1}{1+\frac{3EI}{RL}} \tag{1}$$

Where “R” is a spring stiffness connection, and “EI/L” is the flexure stiffness of the fixed elements. This factor, r, is equal to 0 and 1 for pinned and fixed connections, respectively. Therefore, the end-fixity factor lies between 0 and 1 for a semi-rigid connection. The end-fixity factor value of 0.6 is used in this study.

3. STRUCTURE MODELING

Six and twelve-story moment resisting steel frames are designed according to the ECP-201 [19] and ECP-205 [20]. These frames can be considered demonstrative of the low and medium-rise moment-resisting steel frames. The two frames have the same symmetrical square floor plan of 3 by 3 bays shown in Fig. 1. Each bay is 8.00 m wide. Also, Fig. 1 shows the lateral resistance of the buildings is provided by the middle steel moment-resisting frames. The story heights of the two buildings are 4.0 m for the ground floor and 3.6 m for the other floors. The total heights of the building are 22.0 and 43.60 meters in six and twelve-story frames, respectively.

6- and 12-story-frame contained a rigid, semi-rigid, and the dual frames with different combinations of the rigid and semi-rigid connection locations are shown in Figs. 2-3. The building frames were assumed to be in the city of Alexandria, Egypt. The building floors are assumed to consist of a metal deck with normal-weight concrete topping. The dead load value is 5 kPa and includes deck weights, beams, girders, ceiling, partitions, mechanical, and electrical systems. The weight of the exterior walls is considered equal to 1.25 kPa of the surface. The applied live load considered is taken to 2.5 kPa for frame buildings.

The design internal forces are calculated by considering the critical combination of gravity and seismic or wind loading. The special moment resisting frame is designed with a reduction factor of 7. These frames are designed to withstand significant inelastic deformations. The modulus of elasticity of steel is considered 200 GPa and the strain hardening ratio is 0.01. The frames were designed to make sure that the columns are stronger than the beams. The frames required for design purposes are analyzed using the SAP-2000 computer program [25]. Wide flange sections were used in the design of columns and beams elements. Detailed descriptions of the column and beam cross-sections are summarized in table 1 for the six and twelve-story frames.

A mathematical model of the structure is introduced as a two-dimensional (2D) assemblage of non-linear elements. The model structures with semi-rigid connections are applied in the Drain-2dx computer program with considering the P-Δ effect [21]. The Drain-2dx computer program is a general-purpose computer program for static and dynamic analysis of inelastic plane structures. The mass of the structure model is taken at the end nodes of element structures. The fiber beam-column element type (15) is used to model the beam-column elements. The fiber element model is based on dividing the element into segments and fibers to capture the inelasticity alongside the member. The connection behavior is represented by a rotational spring element type (4) that is introduced at the beam-column interface. The inelastic stiffness of the connections is depending on the connection end-fixity factor.

The partial end-fixity factor is the relationship between the moment and the rotation at the connection, or the equivalent rotational spring constant. The effects of the rigid, semi-rigid and combined configurations under dynamic analysis have been studied on the overall behavior of the steel structures. In table 2, seven earthquake ground motions from the PEER network with different frequency contents and motion measurements are used in the analysis. 3.0 % viscous damping ratio for the first and second natural modes of the frame structures was used in the analysis [26].

Table 1. Cross-section details of the six- and twelve-story frame

Story	6-story			12-story		
	Exterior columns	Interior columns	Beams	Exterior columns	Interior columns	Beams
1	W 14 x 109	W 14 x 176	W 24 x 68	W 14 x 283	W 14 x 342	W 30 x 116
2	W 14 x 109	W 14 x 176	W 24 x 68	W 14 x 283	W 14 x 342	W 30 x 116
3	W 14 x 82	W 14 x 132	W 24 x 68	W 14 x 193	W 14 x 283	W 30 x 116
4	W 14 x 82	W 14 x 132	W 24 x 68	W 14 x 193	W 14 x 283	W 30 x 99
5	W 14 x 53	W 14 x 82	W 21 x 62	W 14 x 176	W 14 x 257	W 30 x 99
6	W 14 x 53	W 14 x 82	W 21 x 62	W 14 x176	W 14 x 257	W 30 x 99
7	-	-	-	W 14 x 145	W 14 x 233	W 30 x 90
8	-	-	-	W 14 x 145	W 14 x 233	W 30 x 90
9	-	-	-	W 14 x 109	W 14 x 159	W 24 x 76
10	-	-	-	W 14 x 109	W 14 x 159	W 24 x 76
11	-	-	-	W 14 x 53	W 14 x 109	W 21 x 44
12	-	-	-	W 14 x 53	W 14 x 109	W 18 x 35

Table 2. the characteristics of the selected earthquakes

Rec. No.	Earthquake	Date	Station	Magnitude	Distance (Km)	PGA (g)
1	San Fernando	1971	Pacoima Dam	6.6	3.5	1.17
2	Imperial Valley	1979	EL Centro Array	6.6	27	0.459
3	Coalinga	1983	Transmitter hill	6.0	9.5	1.17
4	Westmorland	1983	CA-Fire Station	6.0	7.2	0.47
5	Palm Springs	1986	Desert Hot	5.9	12.0	0.30
6	Northridge	1994	Sylmar	6.7	18.0	0.38
7	Park field	2004	Fault Zone 14	6.0	8.0	1.31

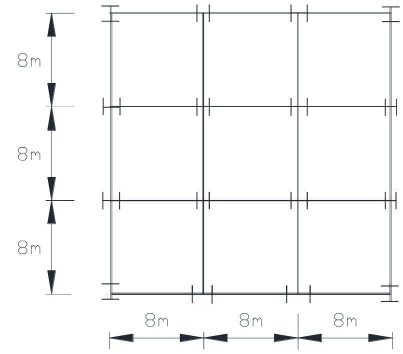


Figure1 Plan view of the steel frames.

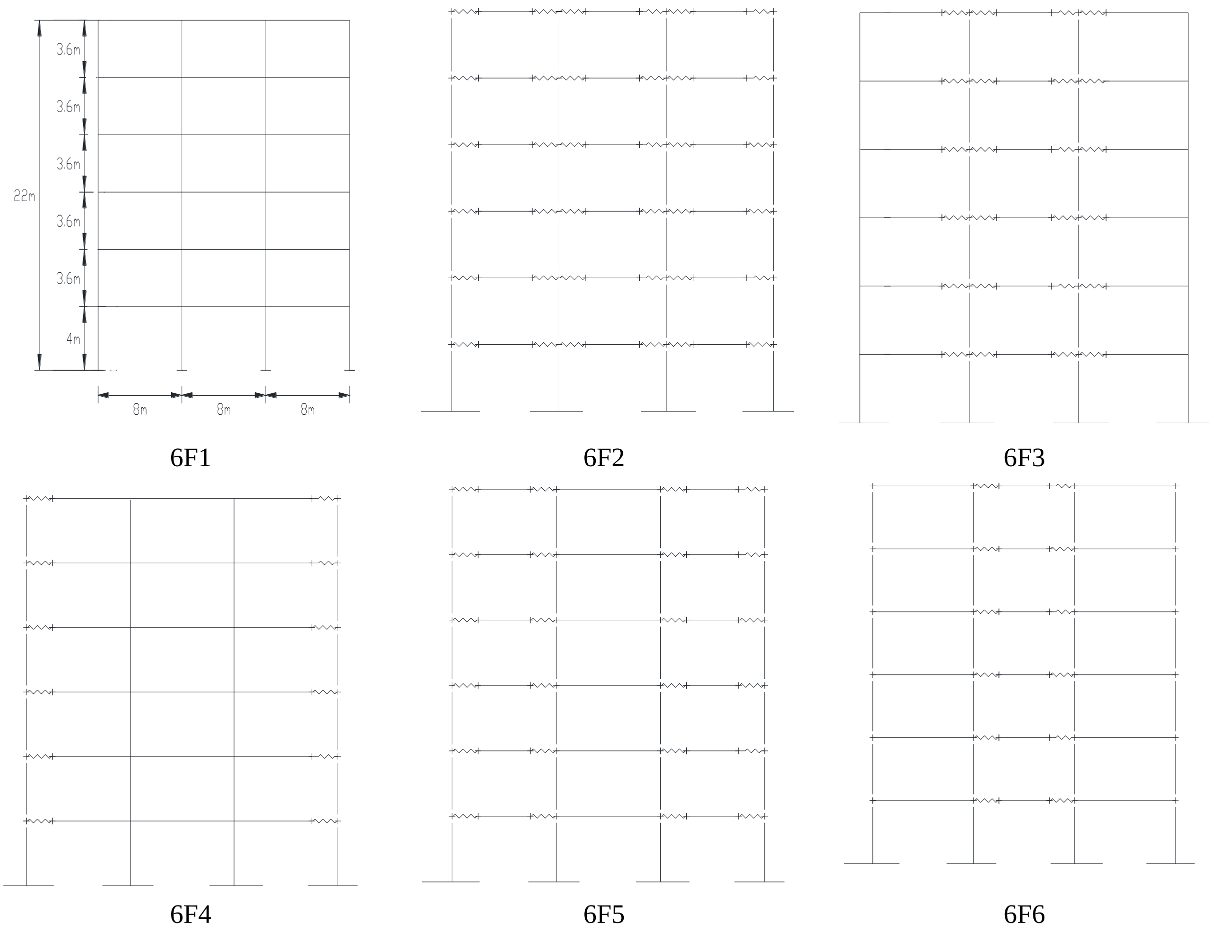
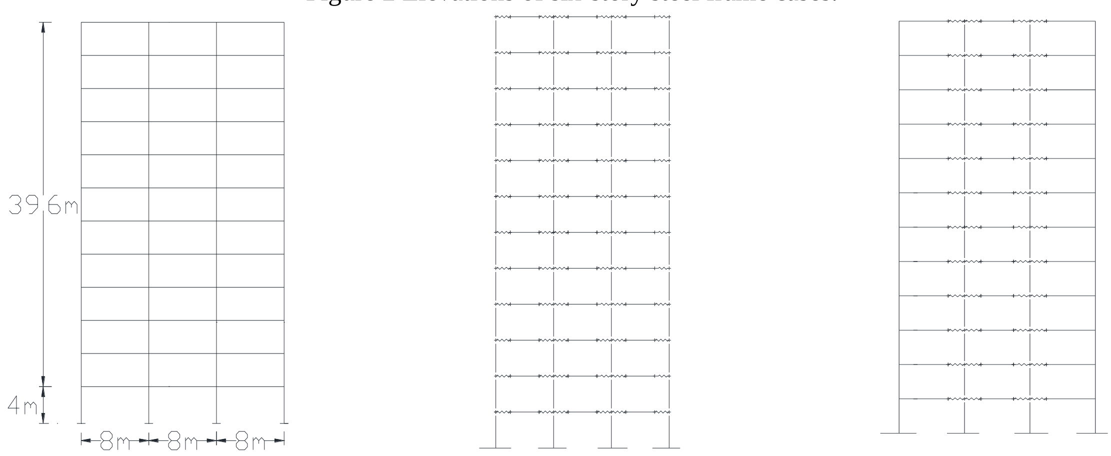
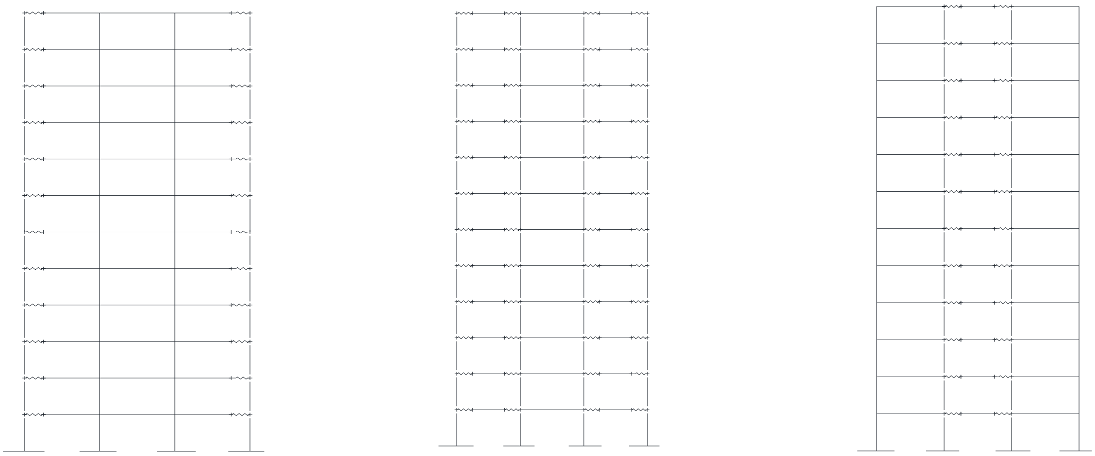


Figure 2 Elevations of six-story steel frame cases.



12F1 12F2 12F3



12F4 12F5 12F6

Figure 3 Elevations of twelve-story steel frame cases.

4. DYNAMIC TIME HISTORY ANALYSIS

4.1 The fundamental period

The fundamental periods calculated by the Drain-2dx computer software for all MRSFs cases are shown in table 3. It is seen that for all of the frame cases, as the stiffness of the beam-column connections decreases, the fundamental period increases, which can be inferred as a decrease in the overall stiffness of the structures. The frame with a fully rigid connection (F1) has the lowest fundamental period and the frame with all semi-rigid connections (F2) have the greatest period in both 6- and 12- story MRSFs. By increasing the number of semi-rigid connections, the fundamental periods of frames are increased. In both 6- and 12- story MRSFs, the periods of dual frames (F3 and F5) cases are similar. Also, the periods of dual frames (F4 and F6) cases are similar. From table 3, by increasing the heights of the frame, the fundamental periods increased. So, the frame height, position, and a number of the semi-rigid connections are affected by the fundamental periods. These results are per those obtained by Feizi, et al. [8].

Table 3. The fundamental period of the frame structures

Frame	T (sec)	Frame	T (sec)
6F1	0.541	12F1	2.7193
6F2	0.642	12F2	2.9847
6F3	0.604	12F3	2.8908
6F4	0.573	13F4	2.8042
6F5	0.600	14F5	2.8844
6F6	0.567	15F6	2.8012

4.2 Maximum roof drift ratios

The roof drift ratio (RDR) is the roof displacement divided by the frame height. The Values of RDR to the 6-story MRSFs of all frame cases are summarized in table 4. It is observed that as the number of semi-rigid connections increases, the RDR of the frame increase. Therefore, on average, the 6F2 frame case is the greatest RDR value in all frame cases. Moreover, the results in table 4 indicate that, on average, the RDR of the fully rigid frame (6F1) is close to the value in the hybrid frames (6F4, and 6F6) cases. Also, the RDR of 6F1, 6F4, and 6F6 cases are less than the other cases. Additionally, there is a little difference among the predictions of the RDR in the other two hybrid frames (6F3, and 6F5) cases.

Table 4. Values of RDR for the 6-story MRSFs of all cases

Record NO.	6F1	6F2	6F3	6F4	6F5	6F6
1	0.61	0.50	0.44	0.43	0.44	0.43
2	0.08	0.10	0.10	0.09	0.10	0.09
3	0.07	0.07	0.08	0.08	0.09	0.08
4	0.59	0.99	0.89	0.77	0.87	0.74
5	0.25	0.26	0.27	0.27	0.27	0.26
6	0.68	0.95	0.93	0.72	0.90	0.72
7	1.17	1.43	1.30	1.17	1.29	1.13
Average	0.49	0.61	0.58	0.50	0.56	0.49

The Values of RDR for the 12-story MRSFs of all frame cases are summarized in table 5. It is observed that as the number of semi-rigid connections increases, the RDR of the frame increases. Therefore, on average, the 12F2 frame case is the greatest RDR value in all frame cases. Moreover, the results in table 5 indicate that, on average, the RDR of the fully rigid frame (12F1) is close to the value in the hybrid frames (12F4, 12F5, and 12F6) cases.

The results presented in tables 4-5 show that the RDR in the rigid frame is close to the hybrid frames (F4 and F6) cases. Moreover, the RDR of the frame increased as the number of semi-rigid connections and the frame height increased.

Table 5 Values of RDR for the 12-story MRSFs of all cases

Record NO.	12F1	12F2	12F3	12F4	12F5	12F6
1	1.67	1.70	1.69	1.68	1.69	1.68
2	0.44	0.53	0.45	0.44	0.45	0.44
3	0.43	0.50	0.47	0.45	0.47	0.45
4	0.79	0.72	0.77	0.79	0.77	0.79
5	0.60	0.57	0.58	0.59	0.59	0.59
6	2.09	2.30	2.24	2.18	2.23	2.17
7	1.07	1.03	1.04	1.07	1.04	1.04
Average	1.01	1.05	1.04	1.03	1.03	1.02

4.3 Maximum story drift ratio

The maximum story drift ratio (SDR) is a significant seismic demand measure that may be extracted from a load of information obtained from incremental dynamic analysis results to estimate the potential damage to structural elements [27]. The use of semi-rigid connections in steel frames increases the story drift ratio, especially in the higher stories [18]. Fig. 4 shows the variance of the mean of maximum SDRs along with the height of the 6-story frame for all frame cases. Furthermore, the maximum SDRs of the earthquake loading cases occur in the third story of all frame cases. The maximum SDRs that occur in the frame with all connections are semi-rigid (6F2) compared to all frame cases.

Moreover, the results in Fig. 4 indicate that the hybrid frames (6F4 and 6F6) cases are showing lower in the SDR compared to the fully rigid frame (6F1) case. This trend is attributed to the results in table 4 indicate that, on average, the RDR of the 6F1 is close to the value in the 6F4, and 6F6 cases. The plastic hinges may move from the elements to the semi-rigid connectors as a result of these connections. As a result, the elements' plastic rotation would slow down. It is possible to reduce the likelihood of local bulking and the production of soft story systems by reducing the plastic rotation of the parts. The story drifts would lessen as the connection strength decreased.

Additionally, there is a little difference among the predictions of the SDR in the other two hybrid frames (6F3, and 6F5) cases.

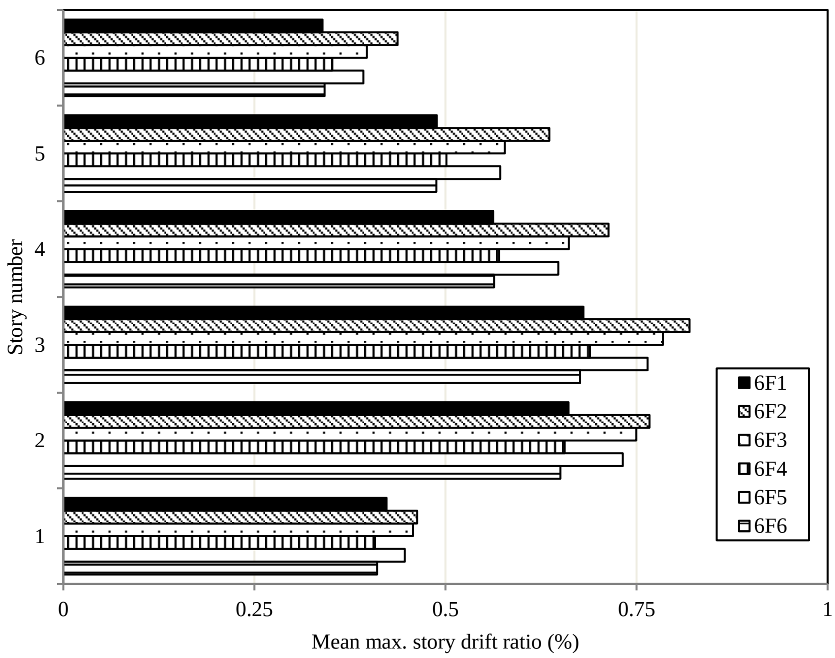


Figure 4 Height-wise distribution of mean SDRs for 6-story frames.

Fig. 5 shows the variations of mean maximum SDRs along with the height of the 12-story frame for all frame cases. The maximum SDRs of the earthquake loading cases occur in the twelfth story in all frame cases. In these cases, the story drift's requirement was not exceeded. An increase in the size of some beams and columns is necessary in these cases to reach the story drift limitation. The maximum SDRs that occur in the frame with all connections are semi-rigid connections (12F2) compared to all frame cases. Moreover, the results in Fig. 5 indicate a little difference between the predictions of the SDR in a fully rigid frame (12F1) with hybrid frames (12F4, and 6F6) cases. Additionally, there is a little difference among the predictions of the SDR in the other two hybrid frames (12F3, and 12F5) cases.

The results presented in Figs. 4-5 show that by increasing the number of semi-rigid connections in steel frames, the story drift ratio is increased. These results are per those obtained by Feizi, et al. [8].

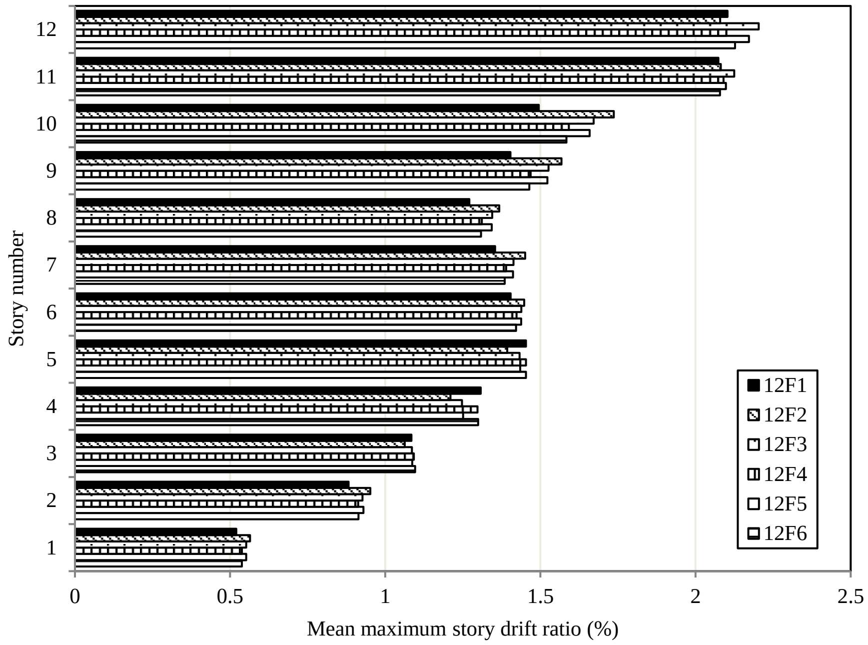


Figure 5 Height-wise distribution of mean SDRs for 12-story frames.

4.4 Column Maximum Axial-Compression-Forces

Fig. 6 shows the variations in mean MACFs in columns along with the height of the 6-story frame for all frame cases. The results shown in the figure indicate that the mean column MACFs occur in the first story in all frame cases under the earthquake loading cases. Moreover, the mean column MACFs of hybrid frame cases are greater than that in a fully rigid frame (6F1). Additionally, there is a little difference between the predictions of the MACFs in the two-hybrid frames (6F3 and 6F4) cases.

Fig. 7 shows the variations in mean MACFs in columns along with the height of the 12-story frame for all frame cases. The results shown in the figure indicate that the mean column MACFs occur in the first story in all frame cases under the earthquake loading cases. Moreover, the mean column MACFs of hybrid frame cases are greater than that in a fully rigid frame (12F1).

The results presented in Figs. 6-7 show that by increasing the number of semi-rigid connections in steel frames, the maximum column MACFs is increased. These results are per those obtained by Feizi, et al. [8].

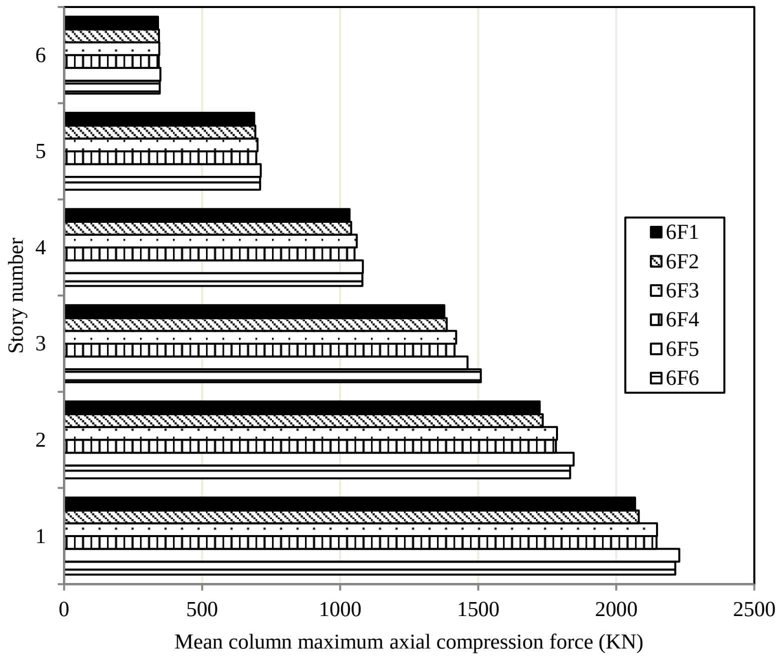


Figure 6 Height-wise distribution of mean column MACFs for 6-story frames.

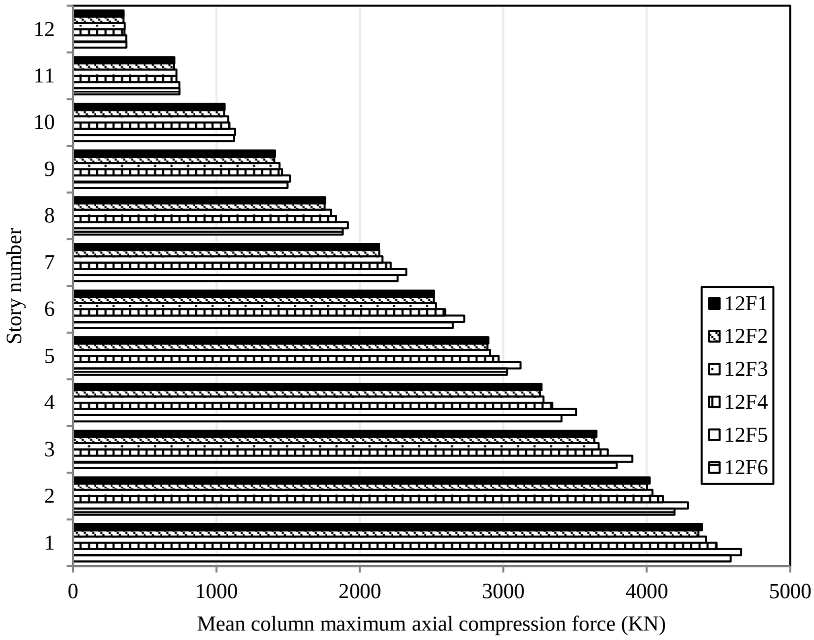


Figure 7 Height-wise distribution of mean column MACFs for 12-story frames.

5. CONCLUSIONS

In this study, six and twelve-story moment resisting steel frames with rigid, semi-rigid, and dual beam-column connections were designed according to the Egyptian design codes. The Drain-2Dx computer program and seven earthquake ground motions are used in the non-linear dynamic analysis. The rotational stiffness of beam-to-column connections is indicated through the end fixity factors with a value equal to 0.6. The following conclusions based on the results obtained are drawn.

- The fundamental periods of the frame structures are increased by increasing the frame height and increasing the number of semi-rigid connections.
- The roof drift ratio in the rigid frame is close to the frames with combined rigid and semi-rigid connections frame (F4 and F6) cases.
- Moreover, the roof drift ratio of the frame increased as the number of semi-rigid connections and the frame height increased.
- The behavior of the dual frames under the earthquake records will change as the number increases and the connection locat ion changes.
- In designing steel frames, combining semi-rigid and rigid connections can result in better structural performance, particularly in seismic locations.
- In both six and twelve frames, there is a little difference between the predictions of the SDR in a fully rigid frame (F1) with hybrid frames (F4, and F6) cases. Additionally, there is a little difference between the predictions of the SDR in the other two hybrid frames (F3, and F5) cases.

6. ACKNOWLEDGMENT

The corresponding author states that there is no conflict of interest.

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