Location of Semi-Rigid Connection Effect on the Seismic Performance of Steel Frame Structures

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Research Article

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Abstract

In design 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. 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, 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.

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 (Kim and Chen 1996; Barsan and Chiorean 1999). Modern design
codes such as Eurocode 3 (EC3) and the AISC-LRFD 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 (2003) investigated the seismic behavior of steel buildings with combined rigid and semi-rigid frames. 5- and 10-stories SMRF designed according to the LRFD (1995) code. 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 was 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. (2010) investigated the effects of semi-rigid connection on the steel braced RC frame, steel truss, and prefabricated structural system responses. SEMIFEM program was used in the 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 differ from structure to others. Ghassemieh et al. (2015) 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. (2015) investigated steel frames with three, eight, and fifteen stories with rigid, semi-rigid, and dual beam-column connections under seismic force. 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 (2015) studied the non-linear 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 are 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 columns end moments increase compared with the rigid connection cases. Ana (2016) 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 (2017) 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 analyzing by ETABS software. The analysis results showed that by using semi-rigid connections, the base shear decreases, and smaller sections for beams and columns can use and leading to reduced cost. Nandeesha and Kashinath (2017) 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 are depending on the type of connections. Also, the end fixity factors from 0.60 to 0.70 are the best range for beam design.

Van et al. (2020) 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 the numerical examples. The analyses show that the behaviors of actual structures are the best compared to both pinned and fixed connections. Farhadi and Anvarsamarin (2020) 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. (2021) 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 is used 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.

Most studies are based on semi-rigid connections in the design and analysis of the frame structures. Although the semi-rigid connections are the source of the structure ductility level but 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 (Dubina et al. 2000). This paper focuses on a study the effects of semi-rigid beam-to-column connections location on 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 and ECP-205. 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 MRSF’s 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 (Parkash et al. 1993). The performance for these frames is incident through the roof drift ratio (RDR), the maximum story drift ratios (SDR), and maximum axial compression forces (MACF).

2. Connections Classification

Fully and partly restrained steel construction types are described by the American Institute of steel construction and load and resistance factor design specification ((AISC 2002, LRFD). 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 (EN 1993-1-1). Hence, there is not any information about semi-rigid connections in Egyptian steel design specifications (ECP-205). Nader and Astaneh (1992) 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. (1996) 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 unit end-moment. The equation to calculate the end-fixity factor, “r” is defined as:

\[
r = \frac{1}{1 + \frac{3EI}{RL}}
\]

Where “R” is spring stiffness connection and “EI/L” is 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 and ECP-205. 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 shows that 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 for the two buildings are 4.0 m for the ground floor and 3.6 m for other floors. The total heights of the building are 22.0 and 43.60 meters for 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 connections 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 be consisting 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 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 frame is designed with a reduction factor of 7. 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. 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 (Prakash and Powell 1992). 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 of 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.

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 x 176 | 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  |
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 measurement 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.

| 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 #7 | 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 Springs | 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    |

4. Dynamic Time History Analysis

4.1 The fundamental period

The fundamental periods calculated by the Drain-2dx computer software to 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) has 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 the 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 on the fundamental periods. These results are per those obtained by Feizi, et al. (2015).
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 a 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. Additionally, 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 six-story MRSFs of the six-frame 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 increase. Therefore, on average, the 12F2 frame case is the greatest RDR value in all frame cases. Moreover, the results in a
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.

| 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 (Gupta and Krawinkler 1999). The use of semi-rigid connections in steel frames increases the story drift ratio, especially in the higher stories (Dubina et al. 2000). Fig. 4 shows the variations 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 a little difference among the predictions of the SDR in a fully rigid frame (6F1) with hybrid frames (6F4, and 6F6) cases. Additionally, a little difference among the predictions of the SDR in the other two hybrid frames (6F3, and 6F5) cases.

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 of all frame cases. 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 that a little difference among the predictions of the SDR in a fully rigid frame (12F1) with hybrid frames (12F4, and 6F6) cases. Additionally, 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 drifts ratio is increased. These results are per those obtained by Feizi, et al. (2015).

**Column Maximum Axial-Compression-Forces**

Fig. 6 shows the variations 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 of all frame cases under the earthquake loading cases. Moreover, the mean column MACFs of hybrid frames cases are greater than that in a fully rigid frame (6F1). Additionally, a little difference among the predictions of the MACFs in the two-hybrid frames (6F3 and 6F4) cases.

Fig. 7 shows the variations 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 of all frame cases under the earthquake loading cases. Moreover, the mean column MACFs of hybrid frames 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. (2015).

**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. 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 by increasing the number of semi-rigid connections in steel frames, the story drifts ratio and the maximum column MACFs are increased.
- In design steel frames, combining semi-rigid and rigid connections can result in better structural performance, particularly in seismic locations.

**Declarations**
Acknowledgment

On behalf of all authors, the corresponding author states that there is no conflict of interest.

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Figures

Figure 1

Plan view of the steel frames.
Figure 2

Elevations of six-story steel frame cases.
**Figure 3**

Elevations of twelve-story steel frame cases.
Figure 4

Height-wise distribution of mean SDRs for 6-story frames.
Figure 5

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