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Effect of Beam-Column Connection Types on the Response Modification Factors of Steel Frames

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Abstract

A key component of steel structural design is the careful selection of structural modeling joints in steel structures, as the behavior of the joints affects the structure's strength and displacement characteristics. According to the capacity for transferring moment, connections in the analysis of frames are categorized as rigid, semi-rigid, or pinned. Also, the response modification factor (R-factor) is an effective parameter used in the seismic design of structures. However, the influence of the beam-column connection's stiffness factor on the response modification factor did not seem to have been considered in seismic design codes. Consequently, the R-factor under static pushover and dynamic loading is being studied for moment-resisting steel frames (MRSFs) with 3-, 6-, and 12-story using three different forms of beam-column connections depending on the connections' stiffness (m). The rigidities of the connections are taken 20, 10, and 5 for rigid, stiff semi-rigid, and flexible semi-rigid connections, respectively. The MRSFs are subjected to ten records with varying frequency contents and ground motion durations. The ductility reduction factor (R_μ), the over-strength reduction factor (R_o), and the R-factor were determined. The results indicated that the beam-column connection rigidity factors affected the R_0, R_μ, and R-factors. Also, the R-factors were more affected by the rigidity factors for the beam-column connections and the number of story frames.

Keywords Response modification factor · Steel frame · Semi-rigid connection · Pushover analysis · Dynamic analysis

1 Introduction

The equivalent static analysis is the basis for the usual seismic design technique used in modern design codes due to its simplicity (IBC, 2021; ASCE/SEI 7–16 2016; FEMA356, 2000; UBC, 1997). These codes decrease earthquake loads by a response modification factor based on the structure's nonlinear seismic response to an earthquake event. The response modification factor R, in most cases, controls this inelastic seismic behavior (FEMA356, 2000). Both the ground motion factors, and structural characteristics affect a structure's nonlinear performance. The response modulation factor (R) is one of the most important parameters used in seismic design to govern the seismic responses of structures, with moment-resisting steel frame structures (MRSFs) being particularly useful for resisting lateral loads. The primary characteristics of these kinds of structures are their plasticity and high consumption of energy capacity. The major component of these frames that withstand lateral forces are the connections between the beam and columns.

The traditional design of these structures is based on the beam-column connection's stiffness as the pinned, semi-rigid or rigid connection. The actual behavior of beam to- column connection of steel frame According to AISC (1999) is generally classified as rigid-joint or pinned-joint behavior. All internal forces are transferred between rigid joints, and the transmission of bending moments is prevented by pinned joints. While beam-column steel connections are categorized into three types based on the connection stiffness (FEMA 355D, 2000). First, there are rigid connections, including extended end plates and bolted flange plates, which provide greater bending forces at smaller rotations. Partially Restrained (PR) Connections, such as bolted T-Stub, are the second type. In the analysis of structures, these connections are often required considering the connection stiffness. The third type is the flexible partially restrained connections such as the Top-Bottom Clip Angle and Bolted Web-Angle. In addition, just a small percentage of the member's plastic

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moment capacity will be developed by the more flexible PR connections.

Most research has shown that the behavior of connections is important for the strength and displacement characteristics of a structure. Materials, geometrical structures, loading situations, and boundaries are some of the factors that might result in nonlinear structural behaviors. However, an element's stiffness, strength, reactivity, and connection to other components can all be greatly affected by geometrical configurations, which include an element's shape, size, and connection types between elements. Also, nonlinearity arises from the connections in steel frames, which can lead to a decrease in load capacity. And therefore, careful selection of structural shapes may help address this issue (Chandrasekaran, 2019; Chandrasekaran et al., 2021).

Based on the amount of a rigidity factor "m," Nader and Astaneh (1992) divided the connections' rigidity into four types. A connection's rigidity factor is calculated by dividing its elastic rotational stiffness by its elastic bending stiffness. If m is less than 0.5, the connection is taken to be either simple or pinned, and if m is greater or equal to 18, it is taken to be rigid. In contrast to the condition of $8 < m < 18$, where it is thought to be equivalent to a stiff semi-rigid connection, the case of $0.5 \leq m < 8$ is thought to be flexible semi-rigid.

Elnashai and Elghazouli (1994) investigated experimentally and analytically two types of beam-to-column connections of two-story steel frames under static and dynamic loading. It was discovered that the semi-rigid frame showed good behavior and can be employed to great benefit in earthquake-resistant construction. Aksoylar et al. (2011) evaluated the seismic performance of a three-span three-bay frame with semi-rigid connections under static pushover and dynamic analyses. The Zeus-NL software program was used in the analysis. Without utilizing the perimeter frame technique, the issue of the overdesign characteristics in low-rise, long span buildings is rather mitigated. Also, the top displacements in semi-rigid frames under some specific ground motion records have lower top displacements than their rigid ones.

Shooshtari et al. (2015) investigated the behavior of the semi-rigid connections and supports of gabled frames under static pushover analysis. The Open Sees software was used in the analysis. It can be concluded that the target point's final displacement can be inferred to be more affected by the support flexibility coefficient than by the connection flexibility coefficient. Although both states have the same overall base shear range. Movaghatia and Abdelnaby (2019) experimentally investigated the nonlinear behavior of bearing-type semi-rigid steel connections under monotonic and cyclic loadings. According to the test results for monotonic loading, the maximum connection rotations were up to 19%, and the slip between the beam flanges and the angular legs heavily influenced the bearing-type nonlinear behavior. Also, in the bearing-type deformations in cyclic loading tests improved the pinching behavior. Rigi et al. (2021) investigated the effect of connection rigidities on the performance of 5, 10, and 15 stories MRSFs under static pushover analysis and dynamic analysis. During the construction and repair of MRSFs, consideration for the rigidity in the beam-column connection cannot be neglected as it may affect the performance of the steel moment frames.

Most research has shown that beam-column connections are designed to be semi-rigid connections to model actual structural behavior (Aksoylar et al., 2011; FEMA 355D, 2000; AISC, 1999; Elnashai & Elghazouli, 1994; Nader and Astaneh, 1992). However, under seismic excitations, the dynamic behavior of frames with semi-rigid connections will differ greatly from that of frames with rigid connections according to connection flexibility. Response modification coefficient, behavior factor, response reduction factor, and response modification factor are the names given to it by ASCE/SEI 7–16 (2016), EC-8 (2004), the Indian seismic code (IS 1893, 2002), and EC-201 (2012), respectively. The target of earthquake engineering is to be able to manage the kind, position, and amount of the damage as well as the detailed technique. Figure 1 illustrates this, showing both elastic and inelastic responses, as well as how the design force is reduced from elastic to design force levels using the equal energy principle (Tasnimi & Masoumi, 2006). The R-factors of structures built to withstand earthquakes have been investigated in numerous studies. Mahmoudi and Zaree (2010) evaluated the R-factors of steel frames with different brace configurations under nonlinear static analysis. Thirty conventional concentric braced frames (CBFs) and twenty buckling restrained braced frames (BRBFs) with different stories were implemented in the SNAP-2DX program. The findings demonstrated that BRBFs had larger response modification factors than CBFs. It was discovered that the R-factors were more influenced by the quantity of bracing bays and building height.

Izadinia et al. (2012) investigated the reduction factor, over-strength factor, and R-factors from capacity curves obtained from Adaptive pushover analysis and conventional pushover analysis. 3, 9, and 20-story MRSFs implemented by the SAC steel project are used in the analysis. The results indicated that the ductility ratios and R-factors obtained by the two pushover analysis methods tend to be dissimilar. Steel frames with gate bracing were examined by Fanaie and Ezzatshoar (2014) for over-strength, ductility, and R-factors under three types of analysis: linear, nonlinear incremental dynamic, and a static pushover. The analysis was conducted using the Open Sees software on 3, 5, and 7-story buildings with braking systems and was done by Iranian Standard No. 2800. The investigation showed that, for the ultimate limit state and acceptable stress approaches, respectively, values of 3.5 and 5 are suggested for the response modification factor. The gate bracing's fragility curves have been plotted for the first time. Such curves can be utilized as the basis for evaluating seismic requirements.

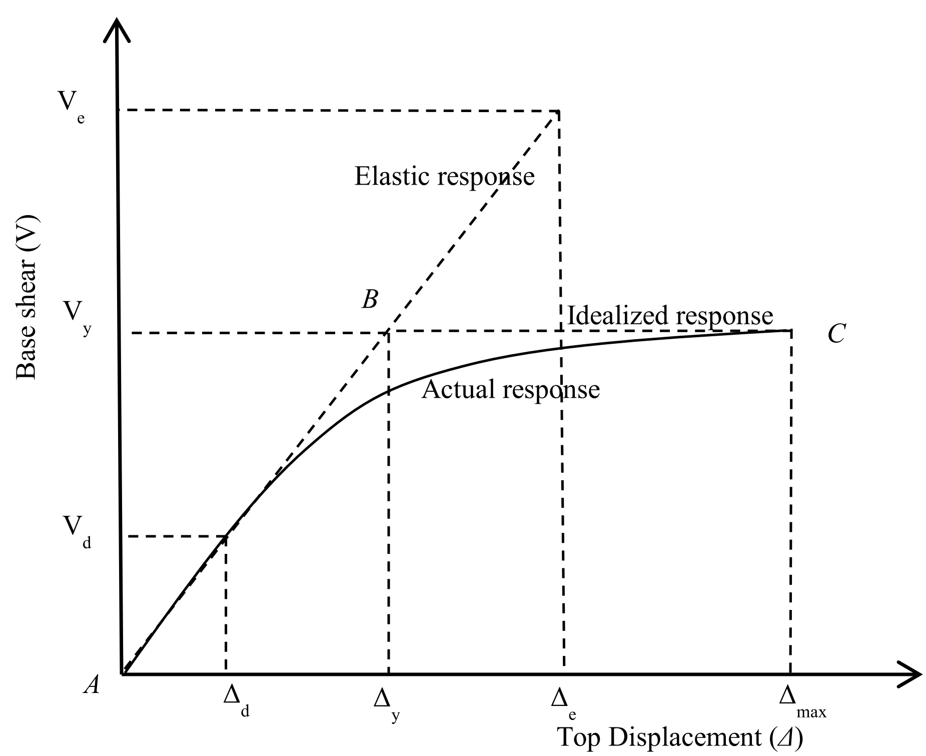
Ferraioli et al. (2014) investigated the R-factors of 12 MRSFs designed according to the Italian Code (NTC08, 2008) under static pushover and nonlinear dynamic analyses. The obtained results demonstrate that the Italian Seismic Code and Euro Code 8 both propose a conservative redundancy reduction factor for these frames. However, these Codes' recommended behavior factors may not be conservative. Using incremental dynamic studies, Soltangharaei et al. (2016) examined the behavior factor for 3, 6, and 10 stories of special MRSFs for near and far fault records. The behavior factor for recordings with close faults is demonstrated to be, on average, 23% lower than that for records with far faults. The response modification, over-strength, and ductility parameters of the Linked Columns Frame (LCF) lateral load-resisting system were evaluated by Golestani et al. (2023). In the Open Sees program, 31 building frames with LCF systems and 3, 5, 7, 9, and 11 stories were developed. These frames were then subjected to incremental dynamic, nonlinear static, as well as linear and nonlinear dynamic analysis. To get more specific results, factors other than the number of stories, such as the length and link beam behavior, were also studied. The findings suggested that lengthening the link beams will result in a decrease in the mean values of the response modification factors. The suggested range for the R-factors of the LCF lateral load-resisting system was 4.0 to 6.5 based on the results.

The influence of the beam-column connection's stiffness factor on the response modification factor is not considered, according to a thorough review of the previous research and seismic design codes. Therefore, the main objective of this study is to investigate the R-factors of three MRSFs designed in accordance with the Egyptian design codes using various beam-column connection types [EC-201, 2012; ECP-205, 2008]. Three types of beam-column connections based on the rigidity of the connections are used in the analysis. The rigidities (m) of the connections are taken 20, 10, and 5 for rigid, stiff semi-rigid, and flexible semi-rigid connections, respectively (Nader & Astenah, 1992). Under static pushover and seismic loading, three steel frames of 3, 6, and 12 stories are examined to estimate the R-factors of the structures. Ten records covering a wide range of frequency contents and ground motion durations are applied to the MRSFs to analyze earthquakes which are chosen from COSMOS Virtual Data Center (2017).

2 Calculation Methodology of the R-Factor

Static and dynamic methods are used to calculate the R factors. Applying a pushover loading with the lateral load distribution indicated in the seismic design code is the basis of the static approach, which looks at the structure's displacement behavior. The variation of structural base shear versus deformation in a typical pushover analysis is illustrated in Fig. 1 (Tasnimi & Masoumi, 2006). The

Fig. 1 Behavior of both elastic and inelastic structures of forces and displacements



dynamic method, in contrast, relies on calculating the R-factor from a series of nonlinear time-history analyses while being subjected to seismic loading that increases in intensity. The R-factor was defined by three factors in ATC-19 (1995) with the ductility reduction factor (R_μ), the over-strength reduction factor (R_o), and the redundancy factor (R_r) as:

$$R = R_\mu R_0 R_r \tag{1}$$

R_r, it is called a redundancy factor. This parameter considers how many structural elements exist in the structure with enough strength. In fact, this consideration is directly related to 'Structural Reliability'. In the scope of that study, the R_R was assumed to be one by evaluating the designed frames' having enough elements with enough strength. Also, the over-strength factor (R_0) is calculated as:

$$R_0 = \frac{V_y}{V_d} \tag{2}$$

The ductility factor R_μ is calculated as (Uang, 1991):

$$R_\mu = \frac{V_e}{V_y} \tag{3}$$

where V_e, is the max base shear coefficient if the structure remains elastic, V_y is the base shear coefficient corresponding to the actual yielding of the structure; and V_d is the design base shear. In addition to being related to the structure's natural period (T), Newmark and Hall (1982) provided a formula to compute R_μ as:

$$\begin{aligned} R_\mu &= 1 && for\ T < 0.2s \\ R_\mu &= \sqrt{2\mu - 1} && for\ 0.2s < T < 0.5s \\ R_\mu &= \mu && for\ T > 0.5s \end{aligned} \tag{4}$$

$$\mu = \frac{\Delta_{max}}{\Delta_y} \tag{5}$$

where Δ_{max} and Δ_y are the maximum and yield roof displacement, respectively.

3 Structure Prototyping and Modeling

In this study, the building's short-direction perimeter SMRFs designed according to ECP-201 (2012) and ECP-205 (2008) are considered as shown in Figs. 2, 3, 4 and 5. It is estimated that perimeter MRSFs will support the buildings' lateral resistance in both directions. The analysis used three-, six-, and twelve-story MRSFs with three-bay in each direction that located in Alexandria city, Egypt. A 2.5 kPa applied live load is considered, and the dead load is computed to be 5 kPa. The standard American-wide, wide flange W-sections with the ASTM A992 (2011) specification are used to produce the steel members. The yield of steel and the modulus of elasticity of steel are taken at 345 MPa and 200 GPa, respectively. Strong-column weak-beam design as well as

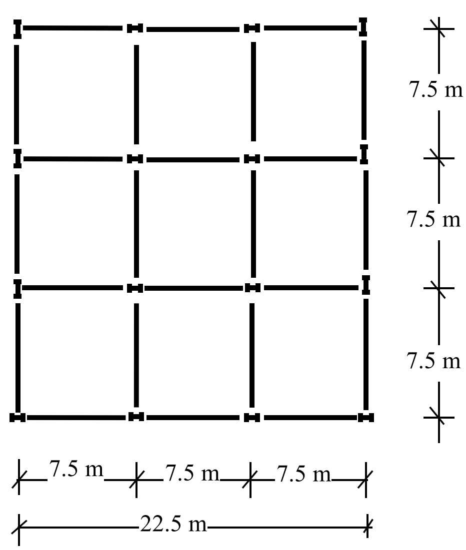


Fig. 2 The floor plan for all frames

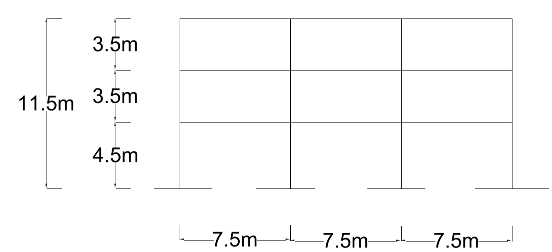


Fig. 3 Elevation of the three-story SMRF

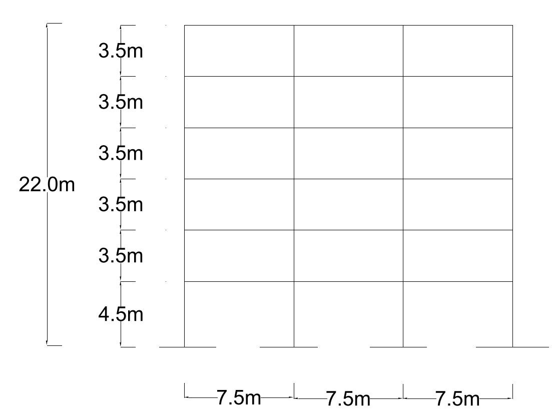


Fig. 4 Elevation of the six-story frame

Fig. 5 Elevation of the twelve-story frame

panel zone yielding, and member stability requirements have been implemented. The MRSFs are designed with a story drift ratio equal to 1.5% according to the Egyptian code ECP-201 (2012) drift provisions. Table 1 shows the cross-section details of beams and columns for the three SMRFs.

DRAIN-2DX software program was used in the analysis of the structure models (Prakash & Powell, 1992). Where it is one of the most accurate and efficient computer programs to do nonlinear dynamic and static analysis of the plane structure. The fiber element (type 15) was used for modeling beams and columns and the element (a zero-length element type 4) was used for beam-column connections. The distribution of inelasticity over the structural members is represented by fiber modeling, which divides the element length into segments and the cross sections into steel fibers. By using an event-to-event method, static nonlinear analysis is carried out. Each event represents a significant modification in stiffness. The P-Δ effect has been considered in addition to the assumption that the structure mass is concentrated at the nodes. Additionally, a rotational spring approach is used to represent the connection model, which is based on the moment-rotation relationship's bilinear piece-line curve. The initial stiffness, post-elastic stiffness, and plastic moment of the connection are the characteristics of the bilinear curve. Different types of beam-column connections based on the rigidity factors (m = 20, 10, 5) are considered in the current study (Nader & Astenah, 1992).

4 Non-Linear Static (Push-Over) Analyses

The elastic analysis does not account for the consequent force redistribution in successive production, nor does it predict the failure of processes in the building's structure. The static non-linear analysis that considers the structure's inelastic behavior can better predict them. The concept of pushover analysis developed from the necessity to use non-linear modeling for predicting the performance of the building over its elastic limit. Pushover analysis is a kind of non-linear static analysis that has developed in recent years to determine the force–displacement relationship of structural parts. This method is used to explain how progressive collapse develops in buildings and how to identify the mode

Table 1 Cross section details of the 3-, 6- and 12-story MRSFs

Story	3-story			6-story			12-story		
	Beams	Exterior column	Interior column	Beam	Exterior column	Interior column	Beams	Exterior column	Interior column
1	W21×50	W14×61	W14×109	W24×68	W14×109	W14×176	W30×108	W14×257	W14×311
2	W21×50	W14×53	W14×109	W24×68	W14×109	W14×176	W30×108	W14×257	W14×311
3	W21×50	W14×53	W14×109	W21×62	W14×82	W14×132	W30×99	W14×193	W14×257
4	–	–	–	W21×62	W14×82	W14×132	W30×99	W14×193	W14×257
5	–	–	–	W21×44	W14×53	W14×82	W30×90	W14×145	W14×233
6	–	–	–	W21×44	W14×53	W14×82	W30×90	W14×145	W14×233
7	–	–	–	–	–	–	W27×84	W14×120	W14×211
8	–	–	–	–	–	–	W27×84	W14×120	W14×211
9	–	–	–	–	–	–	W24×76	W14×109	W14×159
10	–	–	–	–	–	–	W24×76	W14×109	W14×159
11	–	–	–	–	–	–	W21×44	W14×53	W14×109
12	–	–	–	–	–	–	W21×44	W14×53	W14×109

of final failure. Pushover Analysis may also detect possible weak points in a structure by examining the ordered pattern of damage in each member. This procedure enables the assessment of overall structural behaviors and performance aspects. It also allows for the analysis of the sequential creation of plastic hinges in the separate structural elements that comprise the overall structure (Chandrasekaran & Pachaiappan, 2023; Chandrasekaran & Roy, 2006).

The analysis includes sequentially increasing horizontal forces to the structure in a predetermined sequence till the structure exhibits a collapsed state or a limit that is established (FEMA356, 2000). Pushover analysis can be thought of as a technique for estimating seismic force and deformed requirements that essentially illustrates the redistribution of internal forces that happen when the structures are subjected to inertia forces that cannot be resisted within the elastic range of structural behavior. According to UBC 97, the maximum permissible inter-story drift, the models were pushed until the roof displacement equals 2% of the system's overall height. With each step, the lateral forces are increased until an element yields. The next paragraphs provide the static analysis for the different design cases.

Figure 6 shows the roof displacement of the 3-story frame versus base shear coefficients to three values of the connection's rigidity factors. The rigidities (m) of the connections are 20, 10, and 5, for rigid, stiff semi-rigid, and flexible semi-rigid connections, respectively. According to the data shown in the figure, as the connection's rigidity factors increase, so do the ultimate base shear coefficient and the initial stiffness. This result is explained by the fact that when the rigidity factors of the connection increase, the frame fundamental period reduces, and the design base shear coefficient increases. Figure 7 shows the distribution of the 3-story frame's Max. Story drift ratio (MSDR) by height at 2% roof displacement. The MSDRs for m = 5, 10, and 20 at the end of the pushover study were 2.45%, 2.66%, and 2.77%, respectively. In each of the three frame cases, the greatest MSDRs are found in the first story.

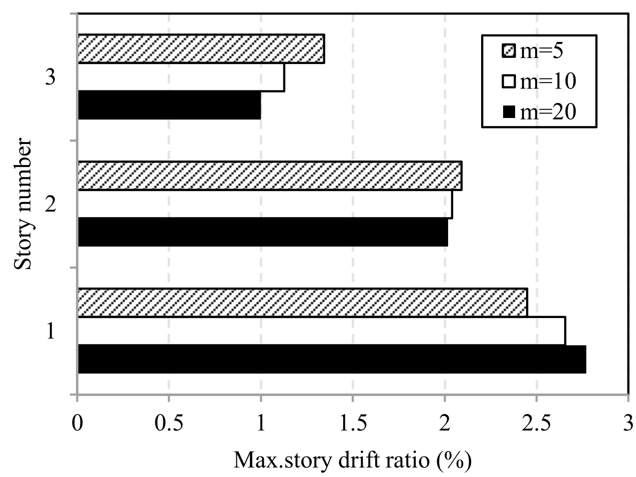


Fig. 7 Distribution of the 3-story frame's Max. story drift ratio by height

Figure 8 shows the roof displacement of the 6-story frame versus base shear coefficients to three values of the connection's rigidity factors. According to the data shown in the figure, as the connection's rigidity factors increase, so do the ultimate base shear coefficient and the initial stiffness. Figure 9 shows the distribution of the 6-story frame's MSDR by height at 2% roof displacement. The MSDRs for m = 5, 10, and 20 at the end of the pushover study were 2.3%, 2.6%,

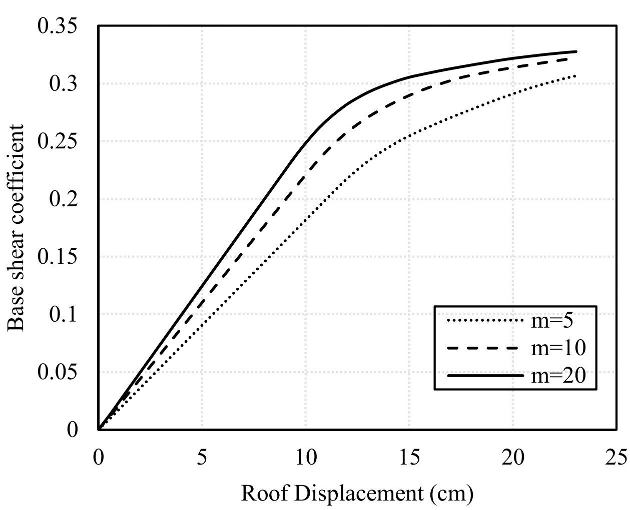


Fig. 6 Roof displacement (cm) of the 3-story frame versus base shear coefficients

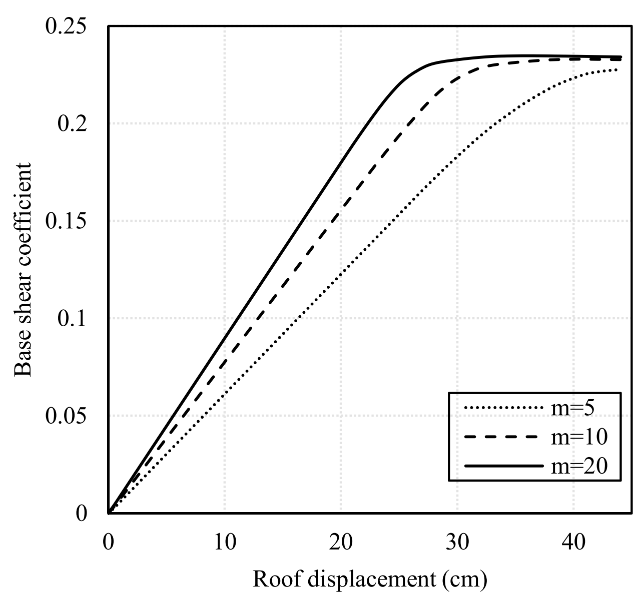


Fig. 8 Roof displacement (cm) of the 6-story frame versus base shear coefficients

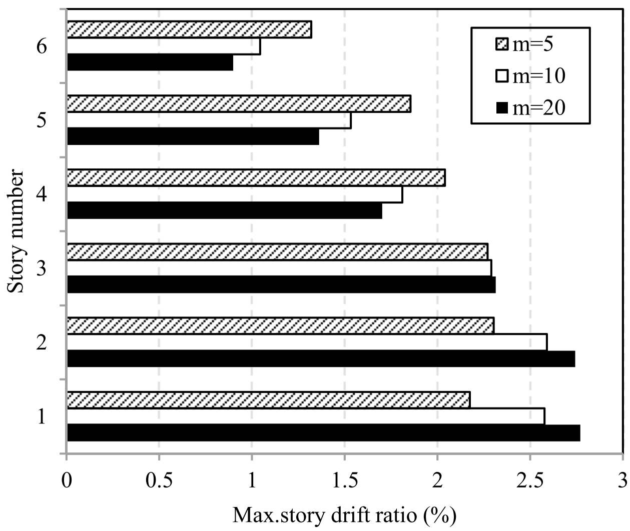


Fig. 9 Distribution of the 6-story frame's MSDR by height

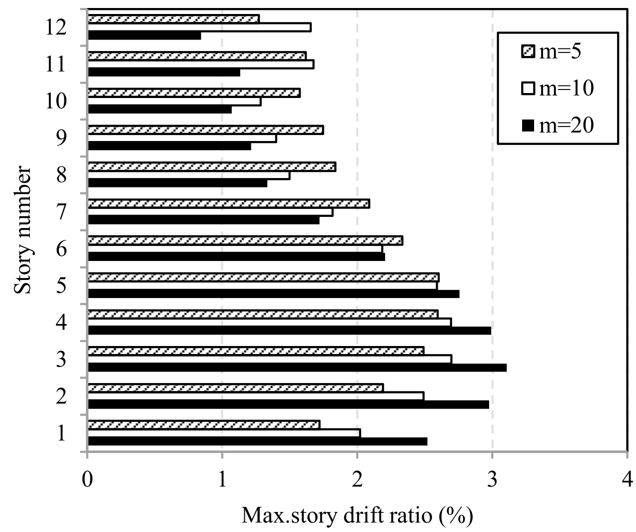


Fig. 11 Distribution of the 12-story frame's MSDR by height

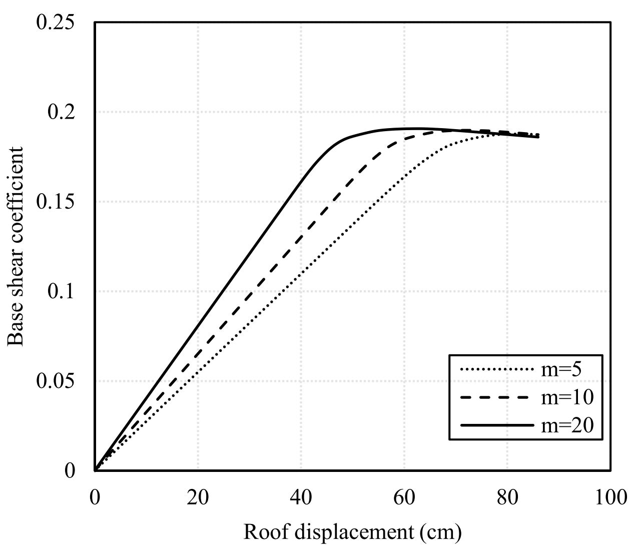


Fig. 10 Roof displacement (cm) of the 12-story frame versus base shear coefficients

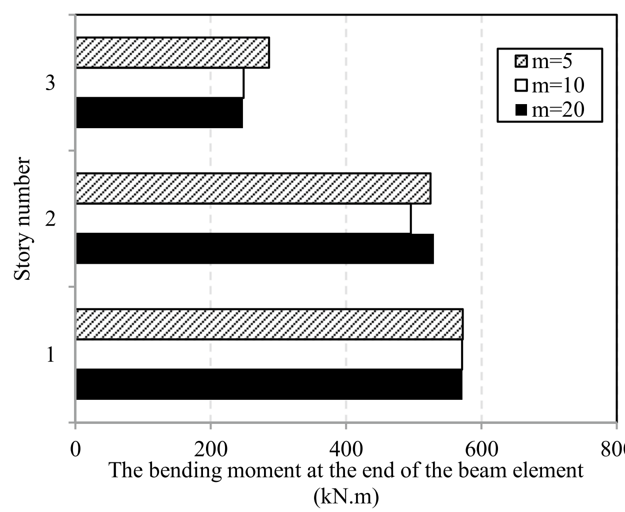


Fig. 12 Height-wise distribution of the bending moment at the end of the beam element for 3-story frames

and 2.7%, respectively. The greatest MSDRs are found in the first story to frame with m = 20 and in the second story for frames with m = 10 and 5.

Figure 10 shows the roof displacement of the 12-story frame versus base shear coefficients to three values of the connection's rigidity factors. According to the data shown in the figure, as the connection's rigidity factors increase, so do the ultimate base shear coefficient and the initial stiffness. Figure 11 shows the distribution of the 12-story frame's MSDR by height at 2% roof displacement. The MSDRs for m = 5, 10, and 20 at the end of the pushover study were 2.6%, 2.7%, and 3.1%, respectively. The greatest MSDRs are found in the five-story frame with m = 5 and in the third story in the frames with m = 10 and 20.

Figure 12 Height-wise distribution of the bending moment at the end of the middle beam element for 3-story frames at 2% roof displacement. The maximum the bending moment at the end of the middle beam element for m = 5, 10, and 20 at the end of the pushover study were 572, 571, and 572, respectively. The maximum bending moment at the end of the middle beam element is found in the first story in all frame cases

Figure 13 Height-wise distribution of the bending moment at the end of the middle beam element for 6-story frames at 2% roof displacement. The maximum the bending

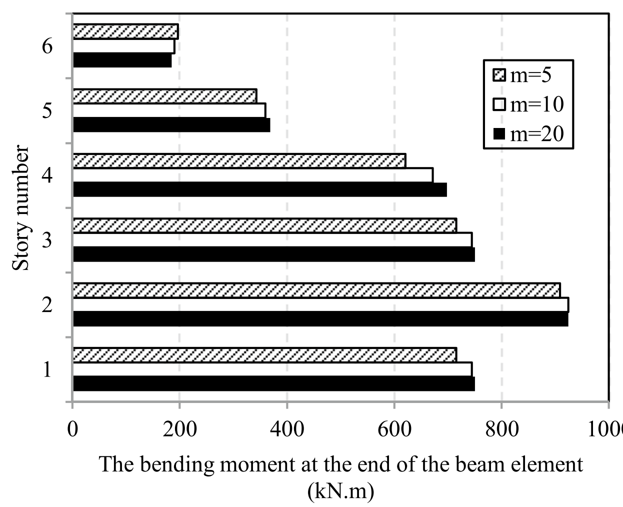


Fig. 13 Height-wise distribution of the bending moment at the end of the beam element for 6-story frames

moment at the end of the middle beam element for m = 5, 10, and 20 at the end of the pushover study were 909, 924.2, and 924.6, respectively. The maximum bending moment at the end of the middle beam element is found in the second story in all frame cases

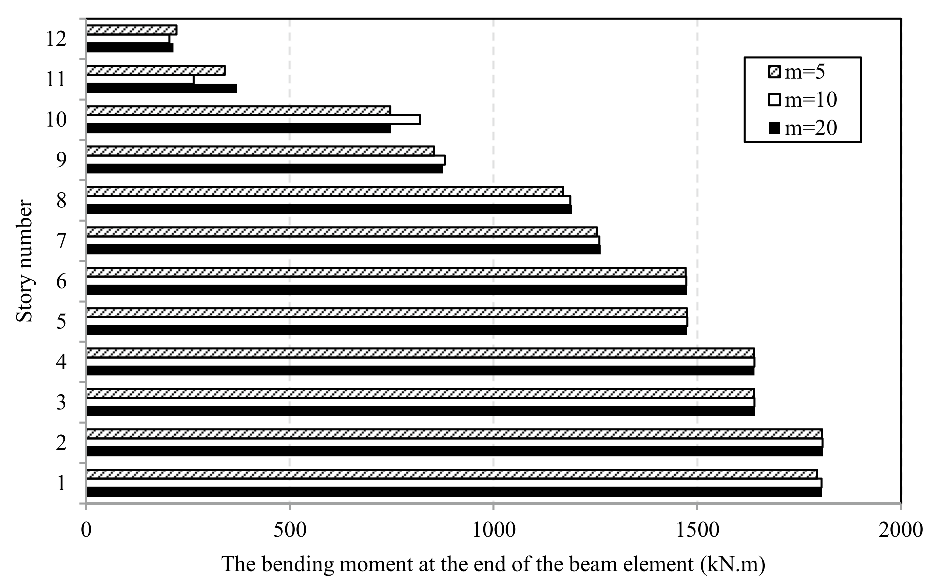
Figure 14 Height-wise distribution of the bending moment at the end of the middle beam element for 12-story frames at 2% roof displacement. The maximum the bending moment at the end of the middle beam element for m = 5, 10, and 20 at the end of the pushover study were 1807, 1808, and 1809, respectively. The maximum bending moment at the end of the middle beam element is found in the second story in all frame cases. This indicated that the bending moment at the end of the middle beam element for the rigid case is larger than in other cases. Additionally, demonstrate how joint stiffness affects the bending moment distribution of the beam elements. Results from research like those of Haapio and Heinisuo (2010) are consistent with this explanation.

For m = 5, 10, and 20, the behavior factors (R_μ, R_0, and R) are determined for the three-, six-, and twelve-story MRSFs. Figure 15a displays that the R_μ increases with the increase in the connection rigidity factors for all MRSF cases. Also, the R_μ decreases with the increased number of stories at the same connection rigidity factors. Its value varies from 1.31 to 1.59 for the 3-story case, 1.19 to 1.38 for the 6-story case, and 1.18 to 1.31 for the 12-story case.

Figure 15b displays that the overstrength factor values (R_0) increase with the increase in the connection rigidity factors for all MRSF cases. However, the R0 increases with the increase of story numbers at the same connection rigidity factors. Its value varies from 5.58 to 5.85 for the 3-story case, 5.95 to 6.13 for the 6-story case, and 6.24 to 6.34 for the 12-story case. These values of (R_0) are high compared to as defined in Table 12.2-1 of ASCE/SEI 7-16 (2016). But given the restriction, it was suggested that this factor be equal to 3. In general, the sizing of MRF members is based on drift limitations. Based on this observation, the frame member sizes should be different for frames with different connection flexibilities (Prakash & Powell, 1992). So, the frame member sizes have been designed up to a story drift ratio equal to 1.5% according to the Egyptian code ECP-201 (2012) drift provisions.

Figure 15c displays that the R-factor increases with the increase in the connection rigidity factors for all MRSF cases. Also, the R-factor decreases with the increased number of stories at m = 10, 20. Its value varies from 7.44 to 8.32 for the 3-story case, 7.04 to 8.49 for the 6-story case, and 7.29 to 9.33 for the 12-story case. However, these factors are

Fig. 14 Height-wise distribution of the bending moment at the end of the beam element for 12-story frames



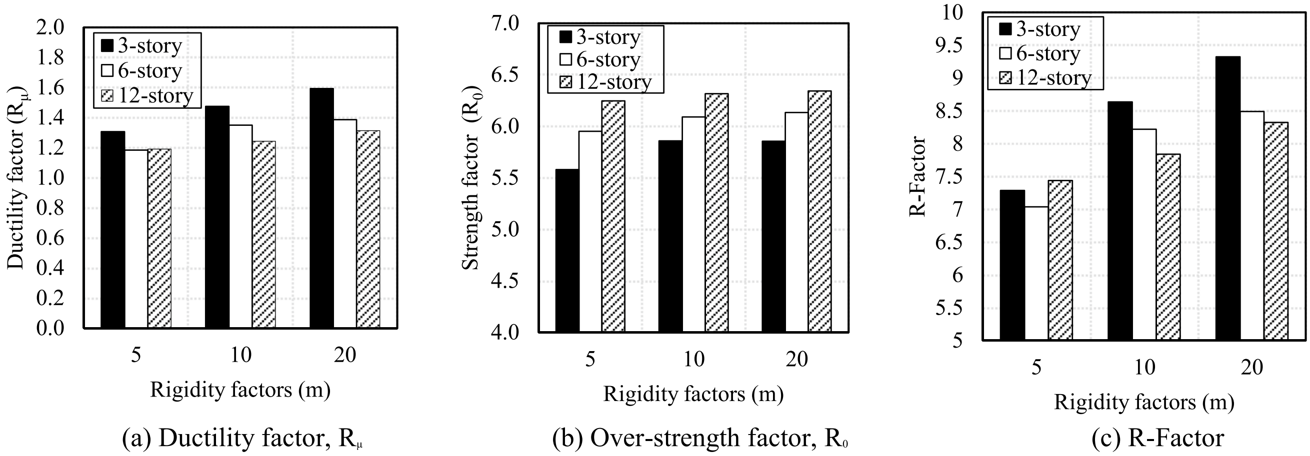


Fig. 15 The 3-, 6-, and 12-story MRSFs' behavior factors

greater than the value recommended by the Egyptian code ECP-201 (2012), which equals 7. The increase in the yielding displacement value of the frame with an increase in the number of stories is what results in the above-mentioned. When the yield displacement levels increase, both R_μ and the R-factor of the frame reduce. These results show that the response modification factor depends on both the connection rigidity factors and the height of the structures.

5 Dynamic Analysis

The incremental dynamic analysis method was taken ten records of earthquake ground motions. These records covering a wide range of frequency contents and ground motion durations are applied to the MRSFs to analyze earthquakes which are chosen from COSMOS Virtual Data Center. Each record of these earthquakes is scaled to several intensity levels in the computer processing to get different levels of dynamic analysis. The PGA values which correlate to the yield and the maximum limit levels are calculated by scaling the chosen earthquakes to provide several intensity levels. Table 2 displays the characteristics of the earthquake records. It is assumed that the structural mass is assembled at the nodes. The P-Δ effect and 3.0% viscous damping in the first two natural modes of the building have been considered.

Mwafy et al. (2002) proposed an equation based on the structure earthquake response to calculate the ductility reduction factor (R):

$$R_\mu = \frac{PGA_{max}}{PGA_y} \tag{6}$$

where PGA_{max} and PGA_y are the peak ground accelerations of the earthquake that related the maximum and the yield displacement responses, respectively. Thus, the ductility reduction factor (R) and the over-strength factor (R_0) which calculated from Eq. (2) can be mixed to get the R-factor, as follows:

Table 2 The selected earthquakes details

Rec No	Earthquake	Date	Station	Magnitude (M)	distance (Km)	PGA (g)
1	San Fernando	1971	Pacoima Dam	6.6	3.50	1.17
2	Imperial valley	1979	EL Centro Array #7	6.6	27.0	0.459
3	Coalinga	1983	Transmitter hill	6.0	9.50	1.17
4	Westmorland	1983	CA-fire station	6.0	7.20	0.47
5	Palm springs	1986	Desert hot springs	5.9	12.0	0.30
6	Limon, costa Rica	1991	San Jose—Guatuso	5.5	22.0	0.10
7	cape Mendocino	1992	CA—Petrolia	7.0	15.0	1.04
8	Northridge	1994	Sylmar	6.7	18.0	0.38
9	Park field	2004	Fault Zone 14	6.0	8.0	1.31
10	Whittier	2014	Scott & Whittier	5.1	9.3	0.20

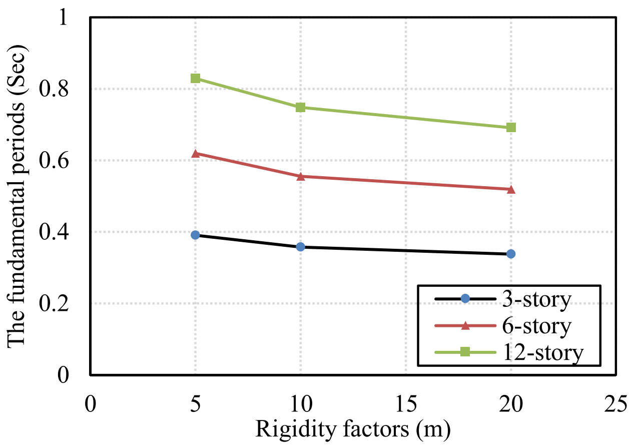


Fig. 16 The fundamental periods of all MRSF cases

$$R = \frac{PGA_{max}}{PGA_y} \times \frac{V_y}{V_d} \quad (7)$$

Figure 16 shows the fundamental periods of MRSF cases determined by the Drain-2dx computer software. According to the results shown in Fig. 16, the fundamental periods of the MRSFs decrease as the connection rigidity factors increase. Additionally, as the story number of frames increases, the fundamental periods of the frames also increase.

Table 3 provides a summary of the R-factors of 3-, 6-, and 12-story MRSFs determined by the dynamic time-history analysis. Figure 17 shows the R-factors of 3-, 6-, and 12-story MRSFs calculated by static and dynamic analysis with three rigidities of beam-column connection. The results shown in the figure demonstrate that, the R-factors obtained by dynamic analysis in all MRSF cases are higher than the equivalent obtained values by static pushover analysis. Additionally, for all MRSF cases, the R-factor increases with the increase in the connection

Table 3 R-factors of all MRSFs under earthquake loadings

Rec No	3-story			6-story			12-story		
	m = 20	m = 10	m = 5	m = 20	m = 10	m = 5	m = 20	m = 10	m = 5
1	10.07	10.76	6.66	10.85	8.12	6.63	10.39	7.62	7.28
2	14.91	13.43	8.40	9.08	8.52	7.25	7.76	7.77	7.06
3	11.87	10.85	6.96	8.16	8.00	7.58	8.98	8.83	7.44
4	12.14	11.95	7.43	9.62	10.08	7.26	13.46	10.58	7.45
5	14.78	11.28	10.95	11.97	7.89	7.01	8.36	7.86	7.45
6	14.75	12.06	11.72	9.14	9.75	7.46	13.57	11.41	7.45
7	10.43	9.22	8.14	11.60	8.62	7.02	8.29	7.83	7.43
8	10.39	9.31	8.20	8.46	11.84	8.48	13.64	9.23	7.46
9	10.71	10.03	9.31	11.94	10.56	7.02	10.02	7.82	7.44
10	14.23	9.74	9.38	10.33	10.85	6.87	8.00	7.33	6.78
Average	12.43	10.86	8.72	10.12	9.42	7.26	10.25	8.63	7.32

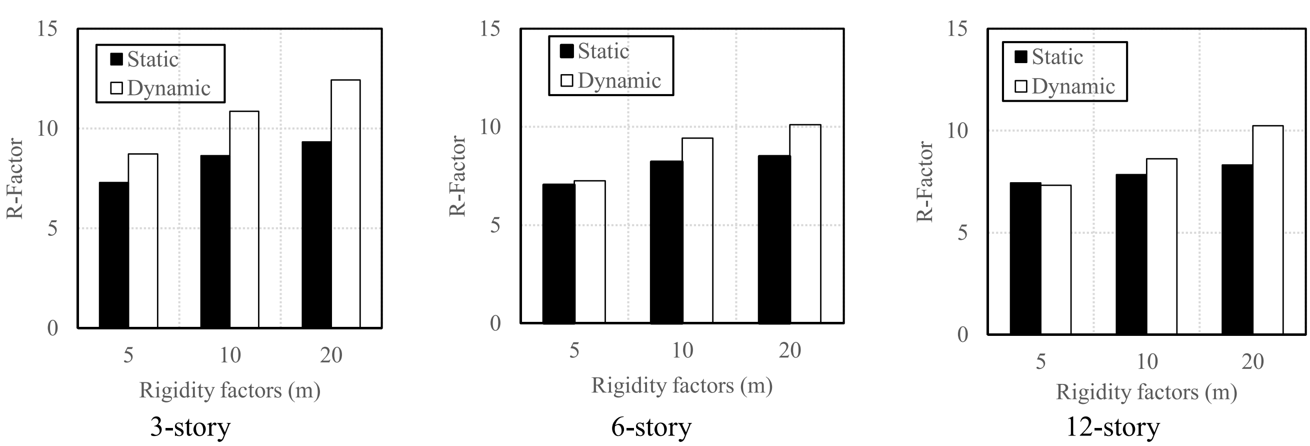


Fig. 17 R- factors determined using static and dynamic analysis of 3-, 6-, and 12-story MRSFs

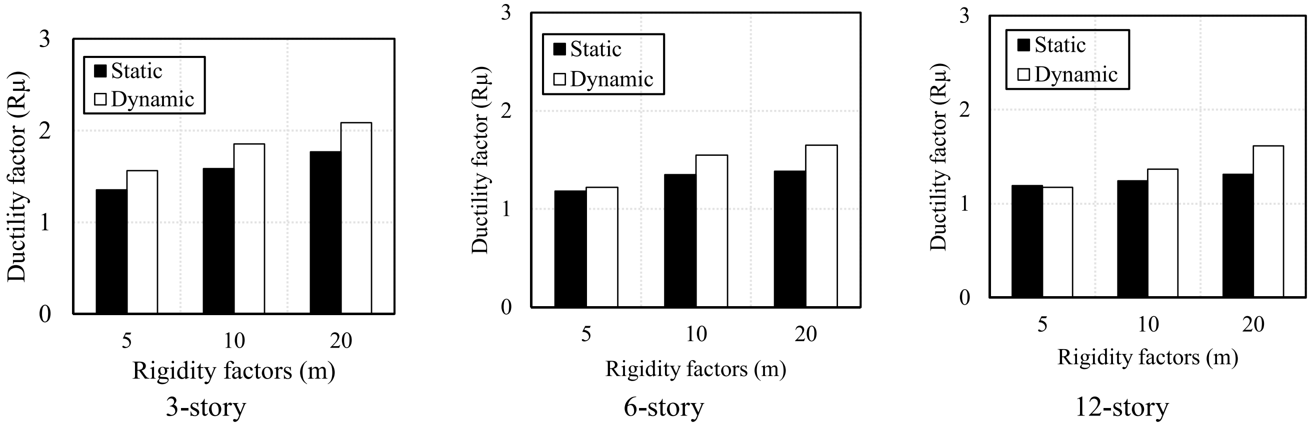


Fig. 18 Ductility factors determined using static and dynamic analysis of 3-, 6-, and 12-story MRSFs

rigidity factors in both static and dynamic analysis. Also, the R-factor decreases with the increased number of stories at the connection rigidity factors in both static and dynamic analysis. Its value varies from 7.04 to 9.33 in static analysis and its value varies from 7.32 to 12.43 in dynamic analysis. The Egyptian code's R-factor values are the lowest of MRSF cases, which suggests the code uses a minimum value to be R-factor-conservative. This value will be an important concept for such a frame. This value will represent the frame's critical value.

Figure 18 displays that R_μ increases with the increase in the connection rigidity factors for all MRSF cases in both static and dynamic analysis. R_μ estimated from the static pushover analysis is usually less than the equivalent values estimated by the earthquake analysis. For dynamic analysis, the R_μ decreases with the increased number of stories at the same connection rigidity factors. Its value varies from 1.56 to 2.08 for the 3-story case, 1.22 to 1.65 for the 6-story case, and 1.17–1.62 for the 12-story case. For static analysis, the R_μ decreases with the increased number of stories for m = 10 and 20. Its value varies from 1.31 to 1.59 for the 3-story case, 1.18 to 1.38 for the 6-story case, and 1.19 to 1.31 for the 12-story case.

6 Conclusions

In this study with different rigidity beam-column connection types, the R-factors of 3, 6, and 12 stories MRSFs designed according to the Egyptian code are investigated under static-pushover and seismic loadings. The rigidities (m) of the connections are taken 20, 10, and 5 for rigid, stiff semi-rigid, and flexible semi-rigid connections, respectively. The following conclusions are made considering the obtained results.

1. As the beam-column connection rigidity factors increased, the values of R_0, R_μ, and R-factors increased in all MRSFs.
2. The mean values of the R_0 and R-factors both increased as the number of stories increased, whereas R_μ decreased.
3. Both the R_μ and the R-factors estimated by the static pushover analysis are usually less than those values estimated by the dynamic analysis.
4. The R-factor is sensitive to both the rigidity factors for the beam-column connections and the number of story frames.
5. The R-factor value for MRSFs varies from 7.04 to 9.33 by static analysis and its value varies from 7.32 to 12.43 by dynamic analysis. Most design codes' R-factor values are the lowest of all cases for MRSFs, which means the code uses the lowest value possible to be more R-factor-conservative, and this value will represent the frame's critical value.
6. The behavior of the R-factor value studied is based only on a 1.5% story drift ratio. It is necessary to evaluate the R-factor value of frame buildings with other drift ratios in the future.

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