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EFFECT OF COLUMN BASE ROTATIONAL STIFFNESS ON THE SEISMIC RESPONSE OF STEEL BUILDINGS

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Abstract

This paper examines in a numerical fashion the, arguably, most critical connection in a steel building: the column base connection under seismic load. It is well-known that failure of this connection can lead to partial or even complete structural collapse. This connection, whether is assumed fixed or pinned, significantly influences the seismic response of a steel building. The rotational stiffness of this connection is determined by several factors, such as the configuration (number of bolts etc.) and type (exposed or embedded). The study presents the results of non-linear time history analyses where the column base connection is modelled by means of a rotational spring. Emphasis is given to the differences in seismic response when different values of rotational stiffness are applied, including cases where the connection is considered completely rigid or pinned. For comparison purposes, three buildings, different in height and essentially in the number of storeys, are studied. Both local (member-level) and global (building-level) response indices, such as column bending moments, the first significant eigenperiods, and the interstory drift ratio (IDR), are employed in order to assess the effect of different values for rotational stiffness. The results indicate that modelling of column base connection, can significantly alter the seismic response, underscoring the need for further investigation to other types of steel buildings.

Keywords: column base connection, steel building, seismic response, time-history analyses.

1. Introduction

The seismic analysis of a steel building, is particularly sensitive to structural modelling assumptions [1],[2], something that have a direct impact on its final design. Traditional modelling practices, such as the widely adopted bare frame centreline approach, have gradually given way to more sophisticated methods mostly inferred by experimental evidence as well as lessons learned from major seismic events. Among these modelling refinements, that of the column base connection has received significant attention due to its critical role in both local (member-level) and global (building-level) seismic response. Modelling of this connection is important in the context of: i) linear seismic (modal response spectrum) analysis, which constitutes the principal analysis method prescribed by modern seismic codes, e.g., EC8 [3], and ii) non-linear seismic analysis, which provides deeper insights into structural behaviour of any building under any seismic motion.

This paper focuses on the inclusion of column base connection modelling as an option within the seismic analysis of realistic 3D steel MRF (moment resisting frame) buildings. More specifically, both linear and non-linear analyses are performed in order to assess the effect of column base connection modelling on some key member- and building-level response indices, such as column bending moments, fundamental periods, and interstorey drift ratios (IDRs). For comparison purposes, three buildings, different in height and essentially in the number of storeys, are studied. This way, one obtains a clear picture about the influence of column base connection modelling on the overall seismic response of a class of steel buildings. It is stressed that modelling of column base-connection is rarely done in practice and one simply relies on assigning a fixed or a pinned column base connection. While broader considerations such as soil-structure interaction and the effects of non-structural elements remain outside the scope of this study, the present findings may offer valuable guidance regarding the importance of this modelling aspect in seismic analysis.

2. Model description

Three steel buildings of varying heights (number of storeys), i.e., a 3-storey, a 6-storey, and a 9-storey are employed for the study. Each building consists of five and four moment-resisting frames (MRFs) along the x and y horizontal directions, respectively, as shown in Fig. 1. The orientation of the columns and the span of each frame is common for all three buildings and it is indicatively presented in Fig. 2 for the case of the six-storey building. The grade of steel is S355 for both beams and columns. The beam-column and column base connections are assumed to be rigid, and for each floor, a composite steel/concrete slab with a thickness of 0.14 m is considered. The composite slab is modelled using standard shell elements, providing diaphragmatic action in each floor plane. Gravity loads consist by dead load g_{dead} and live load q_{live}=2.5 kN/m^2. Load g_{dead} is 5.0 kN/m^2 on floors and 3.0 kN/m on walls. The selected beam and column sections are IPE400 and HEM340, respectively. To address lateral buckling, secondary IPE330 beams, pinned to the main IPE400 beams, are placed at intervals of 2.0 m along the x direction of the buildings.

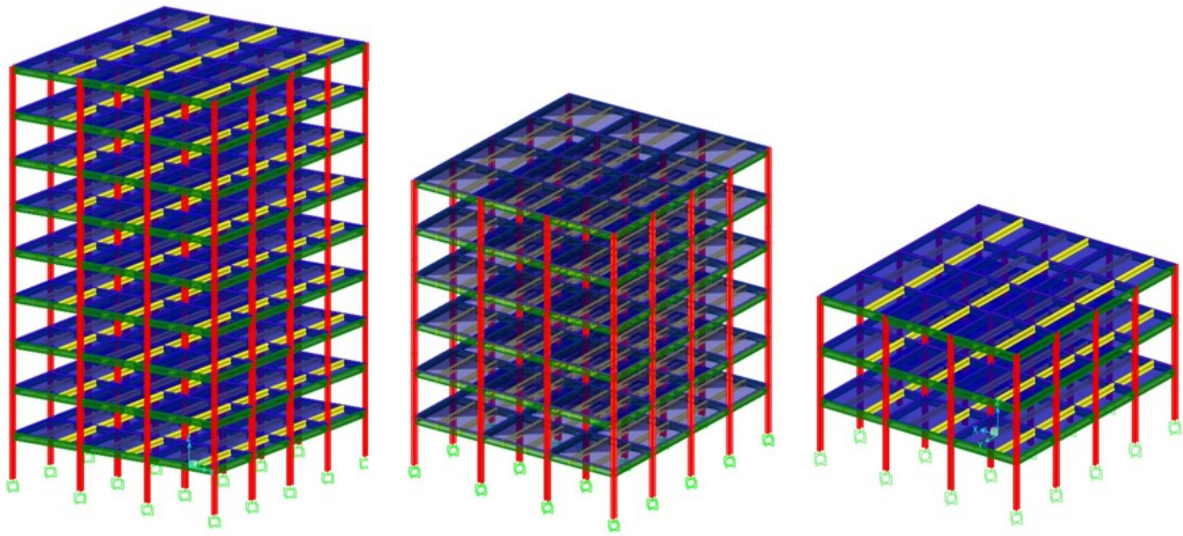

Figure 1. Nine- , six- and three-storey steel buildings (left to right)

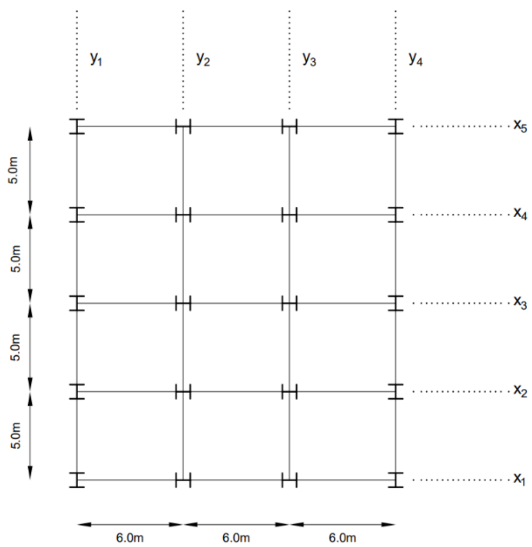

Figure 2. Column orientation along horizontal directions x and y of the steel buildings

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For the linear (modal response spectrum) seismic analysis of the building, the design spectrum of EC8 [3] that corresponds to $a_{gR} = 0.24g$, soil class C, spectrum type 1 is utilized considering an importance factor equal to 1.0 (ordinary building). A value of $q = 4$ for the behavior factor is assumed. Accidental eccentricities of 5% in both horizontal directions are included. For the linear analyses, loading combinations have as follows: $1.35G + 1.5Q$, $G + 0.3Q$, $G + 0.3Q \pm E_x \pm 0.3E_y$ and $G + 0.3Q \pm E_y \pm 0.3E_x$, where E_x, E_y are the seismic loads along directions x and y, respectively. Modal analysis provides the fundamental periods for the 3-storey, 6-storey, and 9-storey buildings, with the first eight modes utilized in modal superposition, satisfying the 90% mass participation requirement of EC8 [3]. The design satisfies EC3 [4] well as the capacity design, inter-storey drift sensitivity coefficient, and damage limitation criteria of EC8 [3], in both directions x and y of the structure.

Then the 3-storey, 6-storey, and 9-storey steel buildings are subjected to non-linear seismic analyses. For the purposes of these analyses, three pairs of seismic motions (accelerograms), appropriately scaled to match the design spectrum of EC8 mentioned above, are utilized.

Several seismic response indices, including maximum interstorey drifts, and bending moments, are derived by both linear and non-linear seismic analyses in order to further discuss the influence of column base connection modelling on the anticipated seismic performance.

A separate comparison of the response indices obtained from the linear and non-linear analyses is presented. This comparison serves in understanding the differences in response results produced by these two analysis methods with respect to column base modelling. While linear analysis provides a simplified approach easily applied in praxis, it cannot capture the real seismic behaviour. Non-linear time-history analysis, on the other hand, having the capability of refined modelling allows for a detailed assessment of the seismic performance of a building. This highlights the importance of conducting non-linear time-history analyses in order to gain deeper insights into the seismic performance of structures, especially when refined modelling assumptions are sought.

3. Column base connection

The column base connection is perhaps the most critical connection of a steel building. Failure of this connection may lead to partial or even complete collapse of the steel building. The rotational stiffness of this connection, i.e., the consideration of a fixed or a pinned connection, affects the seismic response of the steel building. Along the same lines, assigning an erroneous value to the rotational stiffness of this connection, may significantly alter, usually from the non-conservative side, the force and moment demands of the whole building. The rotational stiffness of a column base connection depends essentially on the configuration (base plate, number and placement of anchor rods, vertical stiffeners, footing) and the type (exposed or embedded). A formula that provides the rotational stiffness k_{base} of the column base connection can be found in He et al. [5] and has as follows:

$$k_{base} = \lambda \frac{EI}{L} \tag{1}$$

In Eq.(1), E is the Young's modulus of the material of the steel column, I is the second moment of inertia with respect to the strong axis of the steel column, and L is the height of the steel column, whereas the parameter λ accounts for different k_{base} values. The upper (for a fixed connection) and lower (for a pinned connection) k_{base} value is obtained if $\lambda = 1.67$ and $\lambda = 0.10$ is assumed, respectively. A different procedure to determine k_{base} is summarized in Torres-Rodas et al. [6], however it is not employed herein, in view of the simplicity offered by Eq.(1) in parametric analyses.

On the basis of Eq.(1), the column base connection can be modelled using a rotational spring having a rotational stiffness k_{base}. By varying λ, one may perform seismic analysis for different values of k_{base} in order to investigate their effect on seismic response. The case of not using a rotational spring is indicated as ''Rigid'' in the plots that follow, and essentially corresponds to the analysis of the building described in section 2. The λ values assumed herein are 1.67, 1.50, 1.20, 1.00 and 0.80. The value of $\lambda = 1.67$ provides the maximum rotational stiffness that can be achieved in praxis for a fixed column

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base connection according to He et al. [5], whereas lower λ values represent the case that the actual rotational stiffness of the column base connections is lower than that of the design.

The influence of the rotational stiffness at the column base is examined by considering both the strong and weak axes of the base columns. This consideration acknowledges that the stiffness contribution of the column base can vary significantly between these two axes, depending on the structural design and detailing. The following two cases regarding the assignment of the base rotational stiffness, k_{base}, calculated using Eq. (1), are studied:

- Case 1**:** The rotational stiffness k_{base} is assigned only to the strong axis of the base column, while the weak axis is assumed to be rigid. This case reflects a scenario where the rotational flexibility of the base primarily affects the stronger bending axis of the column, potentially due to construction or design constraints that enhance the rigidity of the weak axis.
- Case 2: The same rotational stiffness value k_{base} is applied to both the strong and weak axes of the base column. This scenario assumes uniform rotational flexibility in both axes, providing a more balanced representation of the behaviour of the base connection.

By exploring these two distinct cases, the study aims to assess how the column base rotational stiffness influences the global and local response of the structure, considering both axes independently. This approach provides insight into whether a simplified assumption (e.g., uniform stiffness for both axes or treating one axis as rigid) is adequate for capturing the structural behavior, or if more detailed modeling is necessary to account for differential stiffness effects between the strong and weak axes. The results of this analysis contribute to understanding the role of base flexibility in the seismic performance of steel building comprised entirely by MRFs.

4. Results

4.1. Modal analysis results

The first three natural periods of the building, corresponding to the various λ values considered, are summarized in Tables 1–6. These results reveal the influence of column base rotational stiffness modelling on the dynamic characteristics of the building.

For the 2 cases mentioned above, it is observed that introducing the rotational stiffness of the column base connection leads to an increase in the fundamental period of the building. This increase can be attributed to the reduced overall stiffness of the building caused by the flexibility at the base connections, which allows greater deformation.

When comparing the periods for the same λ value across the two cases, the differences in periods are relatively small. This suggests that while the additional flexibility of the weak axis in Case 2 slightly influences the dynamic properties, the effect is not pronounced within the range of k_{base} values considered. These findings underscore that while accounting for rotational stiffness is essential for accurately capturing the dynamic behaviour of the building, the distinction between applying stiffness to only the strong axis versus both axes does not significantly alter the global dynamic properties for the studied cases.

Table 1. First three periods (in seconds) - Case 1 - 3-storey building

Mode	*Rigid*	*1.67EI/L*	*1.50EI/L*	*1.20EI/L*	*1.00EI/L*	*0.80EI/L*
1	0.574	0.633	0.636	0.643	0.648	0.655
2	0.569	0.621	0.624	0.630	0.635	0.640
3	0.491	0.553	0.557	0.565	0.571	0.580

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Table 2. First three periods (in seconds) - Case 2 - 3-storey building

Mode	*Rigid*	*1.67EI/L*	*1.50EI/L*	*1.20EI/L*	*1.00EI/L*	*0.80EI/L*
1	0.574	0.652	0.658	0.670	0.681	0.695
2	0.569	0.643	0.648	0.660	0.671	0.685
3	0.491	0.563	0.568	0.579	0.589	0.601

Table 3. First three periods (in seconds) - Case 1 - 6-storey building

Mode	*Rigid*	*1.67EI/L*	*1.50EI/L*	*1.20EI/L*	*1.00EI/L*	*0.80EI/L*
1	1.127	1.188	1.192	1.199	1.205	1.212
2	1.095	1.149	1.152	1.159	1.163	1.170
3	0.965	1.030	1.034	1.043	1.050	1.059

Table 4. First three periods (in seconds) - Case 2 - 6-storey building

Mode	*Rigid*	*1.67EI/L*	*1.50EI/L*	*1.20EI/L*	*1.00EI/L*	*0.80EI/L*
1	1.127	1.209	1.215	1.228	1.240	1.255
2	1.095	1.173	1.178	1.192	1.204	1.219
3	0.965	1.041	1.046	1.059	1.070	1.083

Table 5. First three periods (in seconds) - Case 1 - 9-storey building

Mode	*Rigid*	*1.67EI/L*	*1.50EI/L*	*1.20EI/L*	*1.00EI/L*	*0.80EI/L*
1	1.652	1.712	1.716	1.723	1.729	1.736
2	1.598	1.652	1.655	1.661	1.667	1.673
3	1.410	1.475	1.479	1.488	1.495	1.504

Table 6. First three periods (in seconds) - Case 2 - 9-storey building

Mode	*Rigid*	*1.67EI/L*	*1.50EI/L*	*1.20EI/L*	*1.00EI/L*	*0.80EI/L*
1	1.652	1.733	1.739	1.752	1.764	1.779
2	1.598	1.675	1.681	1.695	1.706	1.722
3	1.410	1.486	1.491	1.504	1.515	1.529

4.2. Interstorey drift (IDR) results

The interstorey drift ratio (IDR) results for the aforementioned two cases of column base rotational stiffness are presented next, comparing outcomes from linear response spectrum analyses and nonlinear time history analyses using three spectrum-compatible accelerograms. These comparisons reveal key insights into the influence of the column base flexibility in conjunction with the analysis method on the IDR values.

For Case 2 (where rotational stiffness is assigned to both the strong and weak axes of the base column), the IDR values are consistently higher than those observed in Case 1 (where rotational stiffness is assigned only to the strong axis, with a rigid assumption for the weak axis). This trend holds true for

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both linear and nonlinear analyses, suggesting that additional flexibility introduced in Case 2 leads to greater deformation across the building height (see Figs. 3-5 for the case of the 9-storey steel building).

A notable observation is that the IDR values obtained from linear response spectrum analyses are higher than those from nonlinear time history analyses for both cases. This indicates that linear analyses may overestimate the IDR, likely due to their reliance on simplified assumptions regarding the seismic demand distribution and seismic displacements induced, whereas nonlinear analyses simulate the realistic dynamic behaviour and the progression of inelastic behaviour under earthquake loading.

Moreover, for both cases and all types of analyses, the IDR value of the 1st story is higher than the one observed in the ''Rigid'' assumption case, while they tend to converge closer to the ''Rigid'' case values for the upper floors. This pattern reflects the significant influence of base flexibility at lower stories, which diminishes progressively with height as the global behaviour of the building becomes less dependent on the base conditions.

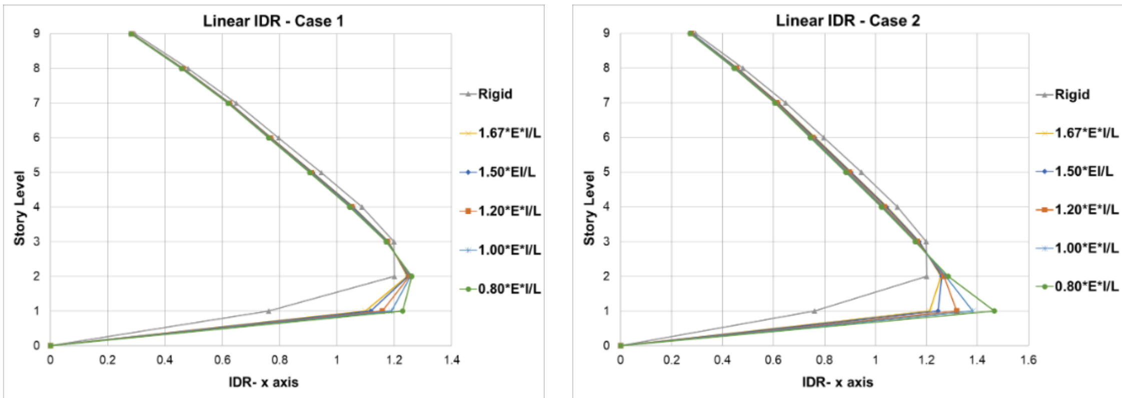

Figure 3. Linear analyses results for the 9-storey building: IDR distribution along direction x for different k_{base} values, Case 1 (left) and Case 2 (right)

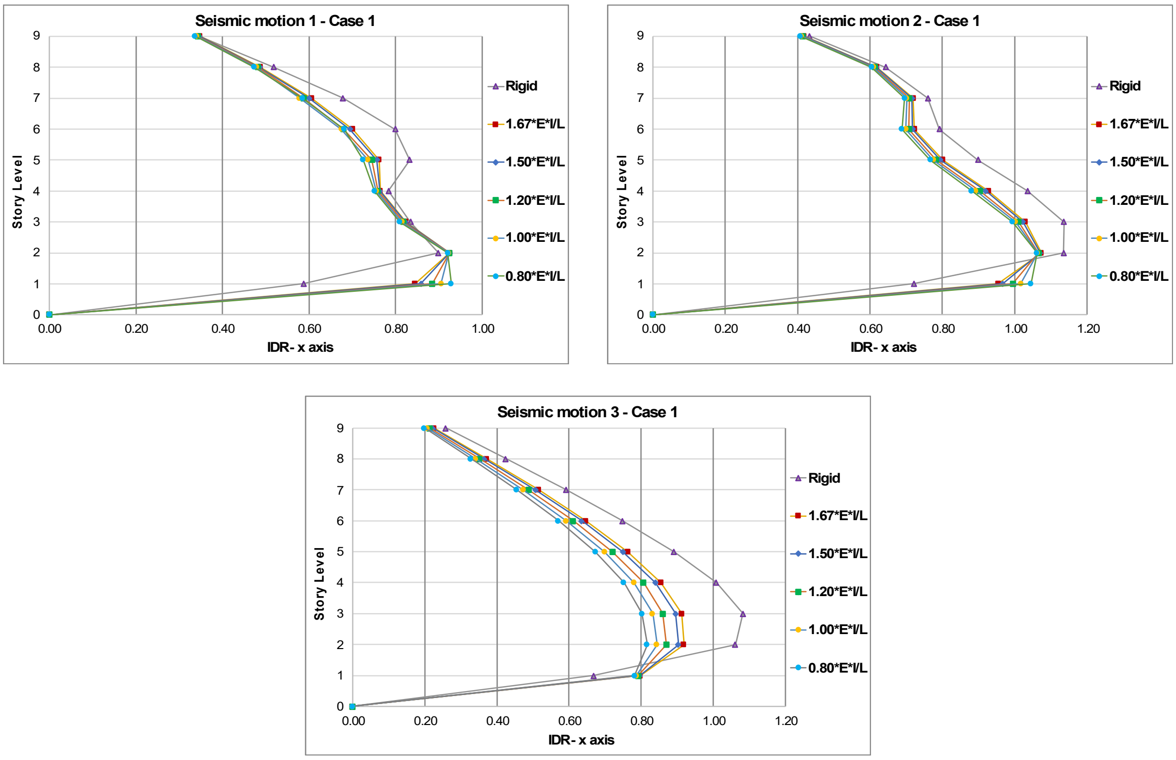

Figure 4. Non-linear analyses results for the 9-storey building: IDR distribution along direction x for different k_{base} values, Case 1

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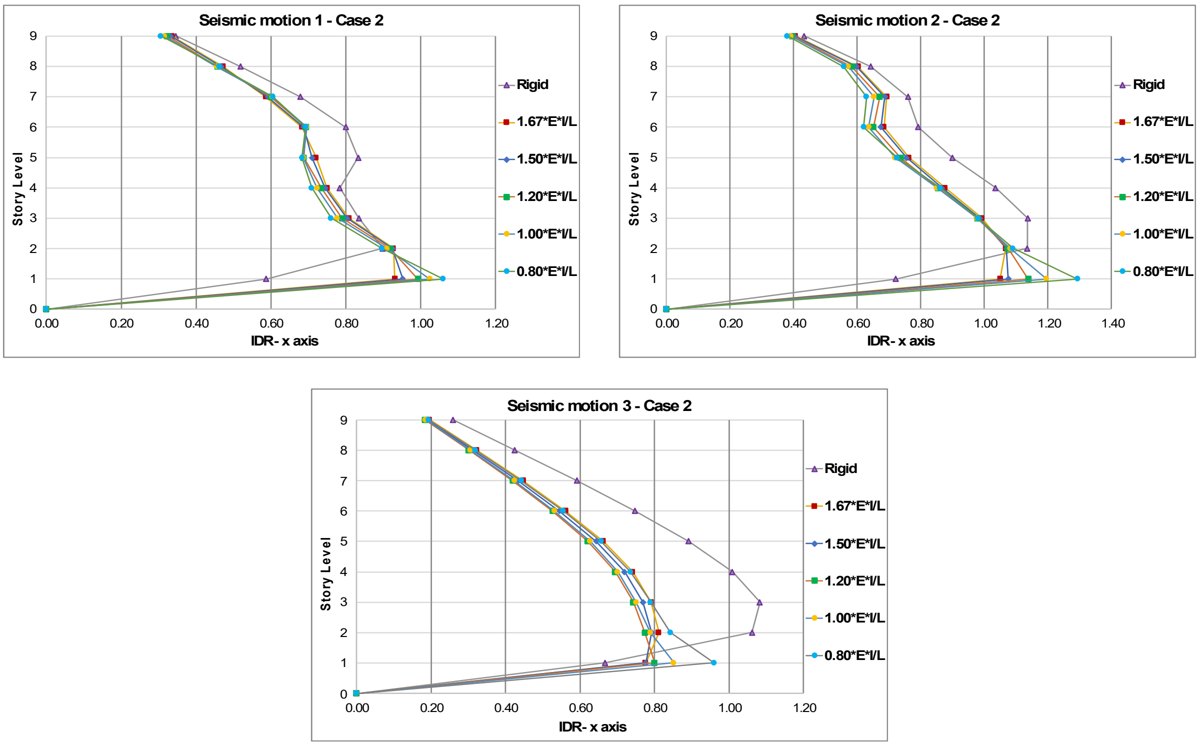

Figure 5. Non-linear analyses results for the 9-storey building: IDR distribution along direction x for different k_{base} values, Case 2

While only the figures for the 9-storey building and the values along the x direction are presented herein, the same observations are identified for the 3-storey and 6-story steel buildings in both horizontal directions, x and y. These results emphasize the importance of considering base flexibility in modelling and demonstrate that nonlinear time history analyses can provide more accurate estimates of interstorey drifts, particularly under flexible base conditions.

4.3. Bending moment results

The bending moments at the base and top of the base column, denoted as M3 (strong axis) and M2 (weak axis), are analysed for the two cases of column base rotational stiffness, presented previously. Comparisons are made between results obtained from linear response spectrum analyses and nonlinear time history analyses using the three spectrum compatible accelerograms, highlighting critical differences in expected behaviour.

The results indicate that for both M3 and M2 bending moments, the bending moments at the base of the most critical base column derived from linear analyses are significantly smaller than those obtained from nonlinear analyses, often underestimating them by, as much as, a factor of four. This stark disparity underscores the limitations of linear analyses in capturing the dynamic inelastic behaviour of the structure under seismic excitation. Nonlinear analyses, which account for material and geometric nonlinearities, provide a more accurate representation of the actual demand on the base columns during earthquakes.

Two notable trends emerge when examining the influence of rotational stiffness on the bending moments:

- Case 1**:** When the rotational stiffness is assigned only to the strong axis (while the weak axis remains rigid), an increase in rotational stiffness leads to a decrease in M3 at the base. This reduction occurs because the greater stiffness along the strong axis redistributes the rotational demands, reducing the bending moment about the strong axis. On the other hand, M2 at the base increases with greater rotational stiffness, indicating an increased demand on this axis.

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This interplay between the axes highlights the sensitivity of bending moment distribution to the modelling assumptions of the base flexibility.

- Case 2: When the rotational stiffness is applied to both the strong and weak axes, increasing the rotational stiffness results in an increase in both M3 and M2 bending moments at the base. This behaviour reflects the overall increase in bending demand as the base flexibility is reduced in both axes, causing the columns to resist greater rotational forces. The simultaneous increase in M3 and M2 bending moments indicates that the flexibility provided by both axes plays a significant role in mitigating bending moment demands at the base.

The remarks above are indicated in the Figs. 6-10, where now the results of the 6-storey steel building are employed for illustration purposes. For the ‘‘Rigid’’ case, the M3 and M2 bending moments at the base, coming from linear analyses, are equal to 320 kNm and 122 kNm, respectively, while for the same indices the values coming from, e.g., the spectrum compatible seismic motion 3, are equal to 1421 kNm and 558 kNm, respectively. A noticeable increase was also identified on the other two seismic motions.

For clarity and ease of interpretation, the remaining results for the 3-storey and the 9-storey steel buildings, are summarized in Tables 7 and 8, rather than being presented as individual figures. Table 7 summarizes the values of the bending moments at the top and the base of the most critical base column for different k_{base} values, along with the percentage difference with the ‘‘Rigid’’ case, allowing for an easy comparison of the influence of base flexibility. Table 8 convey the same information for the 9-storey building. In Tables 7 and 8, a decrease of up to 70% is observed for M3 at the base, when comparing the Rigid base case to the most flexible case (0.80*E*I/L) considered in this paper, for both the 3-storey and 9-storey steel buildings. Moreover, an increase of more than 100% is observed for M3 at the top in Case 2, when comparing the same two limits for rotational stiffness. From the results, Case 2 is presented as the governing one for M3 at the base and the top of the base column and Case 1 as the governing one for M2 at the same locations. Another remark is that for the top of the base column, for both Cases and all seismic motions, the demand is almost always increasing when the rotational stiffness is decreasing.

The results presented in Figures 6-10 and Tables 7 and 8 show the importance of nonlinear time history analyses in providing a more reliable estimation of bending moments under seismic loading, especially in cases where base flexibility and rotational stiffness significantly influence the column response.

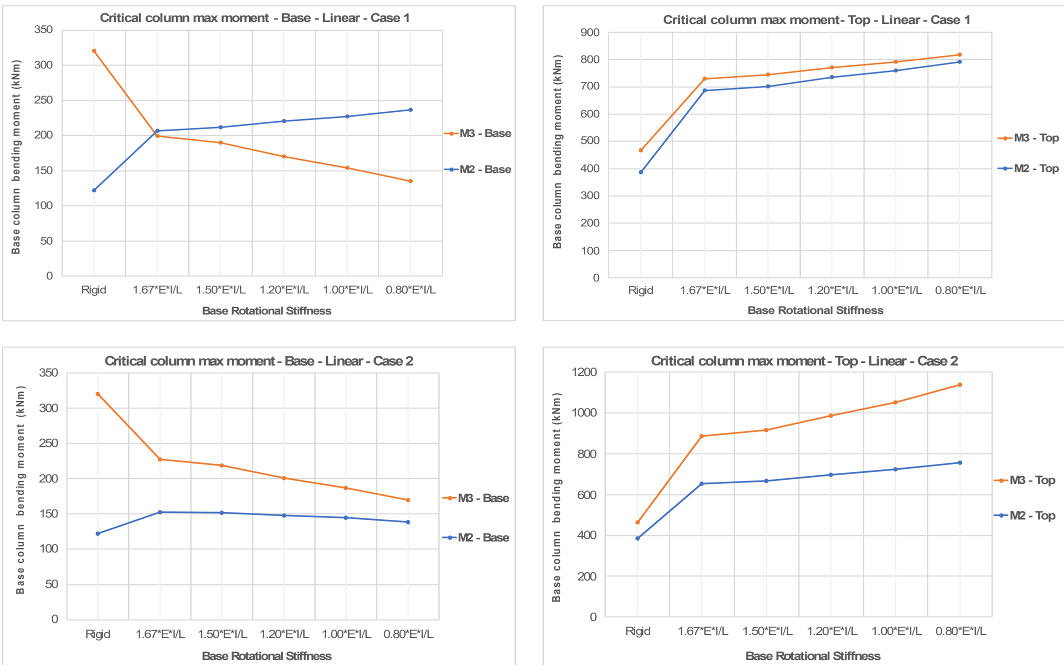

Figure 6. Linear analyses results for the 6-storey building: critical column bending moments for different k_{base} values for Case 1 (top) and Case 2 (bottom)

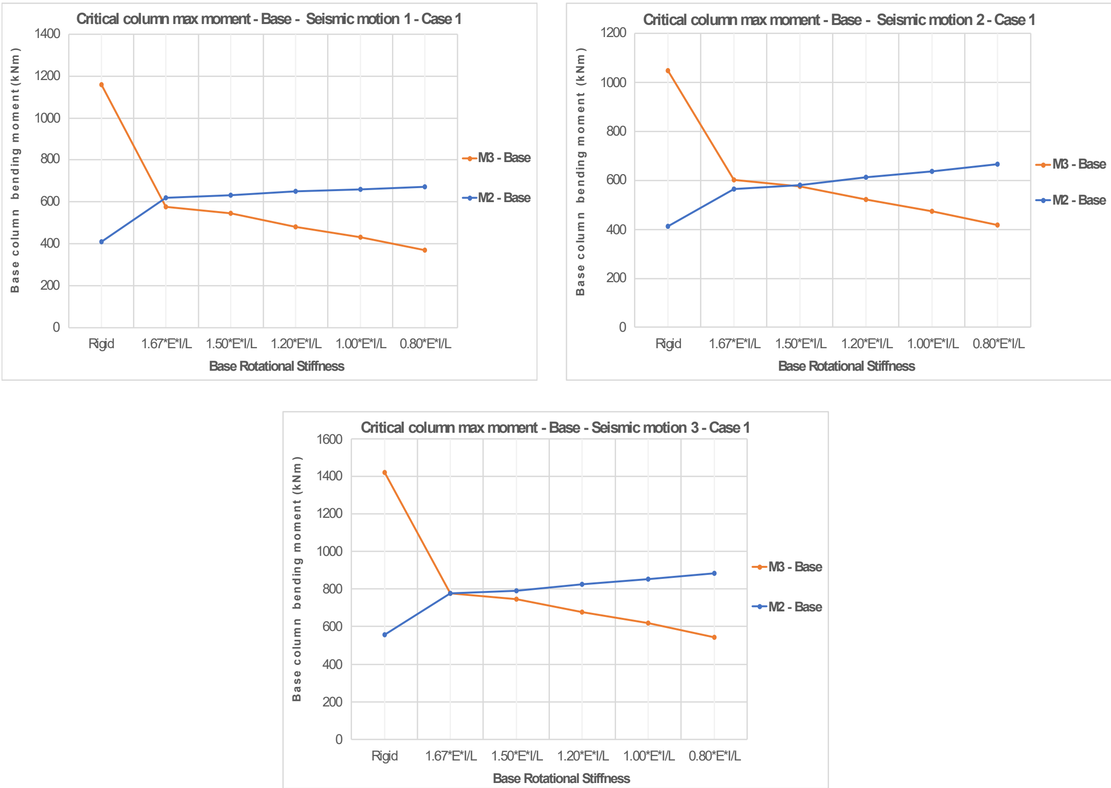

Figure 7. Non-linear analyses results for the 6-storey building: bending moments at the base of the critical column for different k_{base} values, Case 1

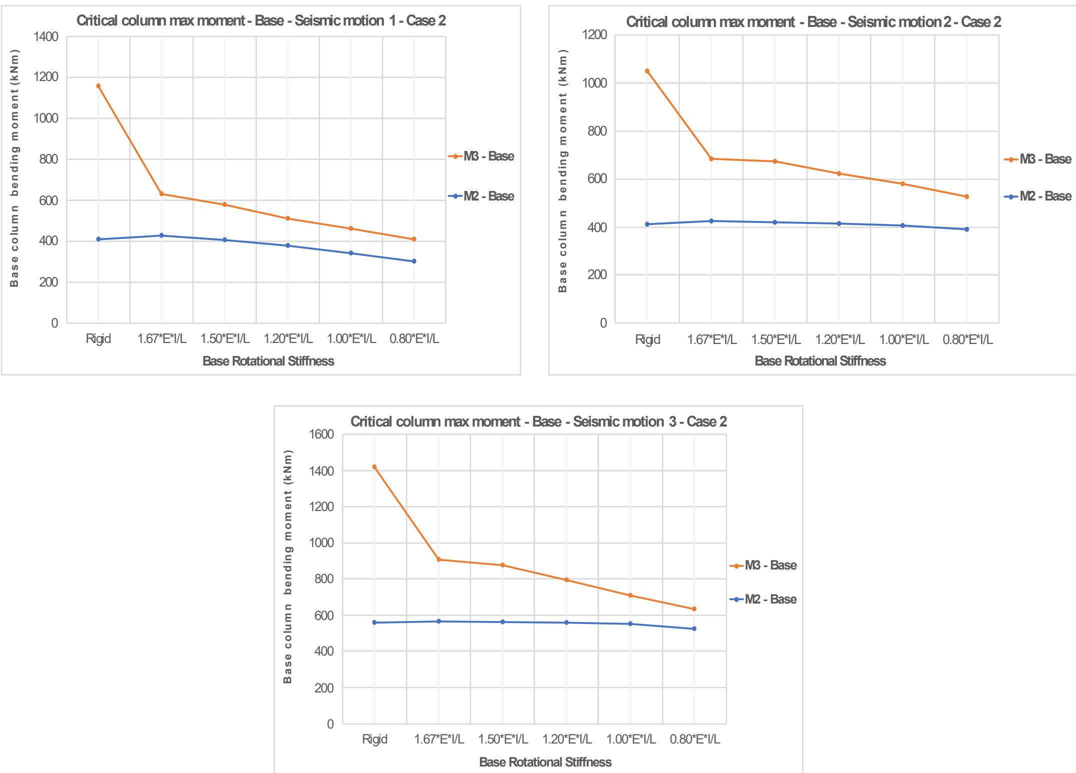

Figure 8. Non-linear analyses results for the 6-storey building: bending moments at the base of the critical column for different k_{base} values, Case 2

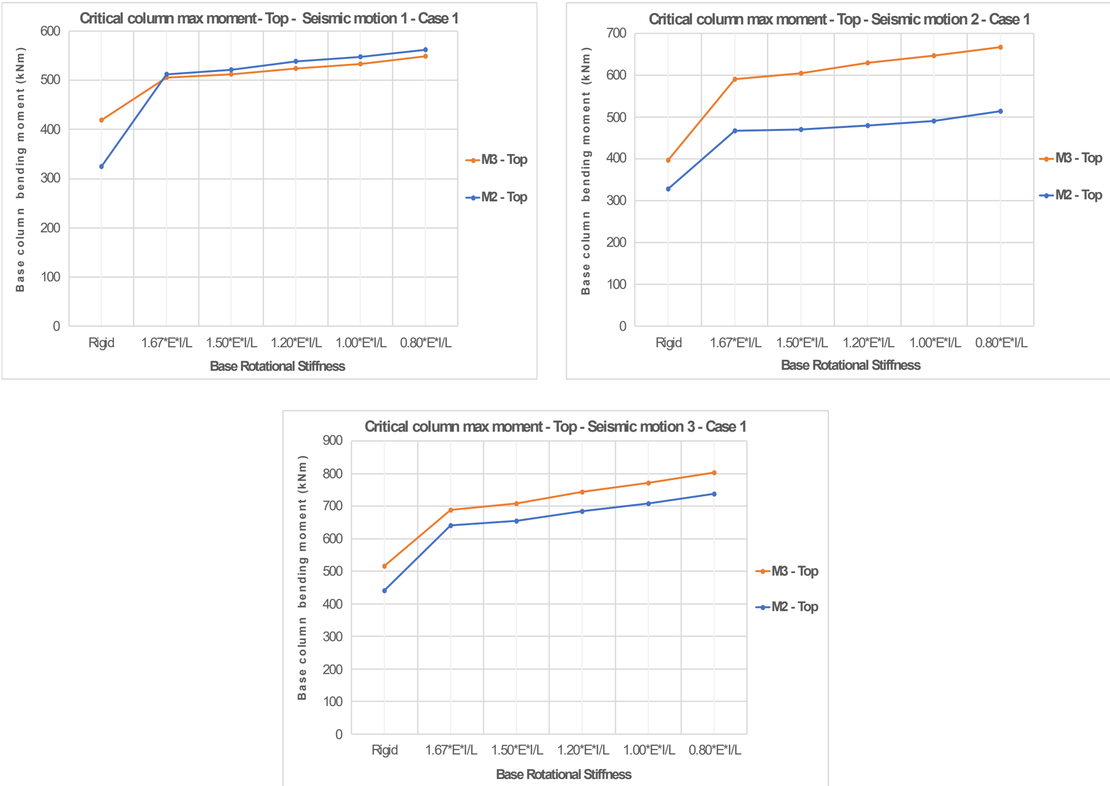

Figure 9. Non-linear analyses results for the 6-storey building - bending moments at the top of the critical column for different k_{base} values, Case 1

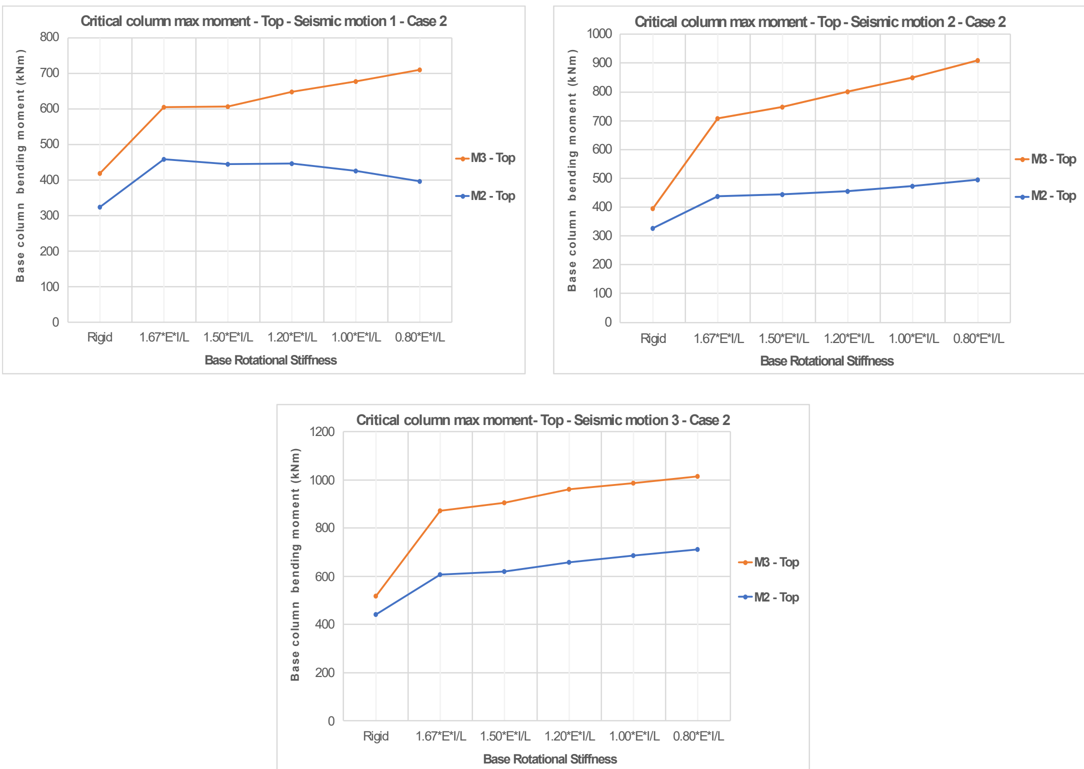

Figure 10. Non-linear analyses results for the 6-storey building - bending moments at the top of the critical column for different k_{base} values, Case 2

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Table 7. Linear and non-linear results for the 3-storey building - Column bending moments (at base and top) for different k_{base} values - % difference from Rigid case

3-storey - Linear	Base Rotational Stiffness										
Base Column Moment (kNm)	Rigid	1.67*E*I/L	% Diff.	1.50*E*I/L	% Diff.	1.20*E*I/L	% Diff.	1.00*E*I/L	% Diff.	0.80*E*I/L	% Diff.
M3 - Base - Case 1	303.1	165.4	**-45.4**	157.0	**-48.2**	139.4	**-54.0**	125.4	**-58.6**	108.9	**-64.1**
M3 - Base - Case2	303.1	183.1	**-39.6**	175.3	**-42.2**	158.9	**-47.6**	145.4	**-52.0**	129.2	**-57.4**
M2 - Base - Case 1	113.4	170.2	**50.1**	173.1	**52.6**	179.0	**57.8**	183.7	**62.0**	189.2	**66.8**
M2 - Base - Case 2	113.4	120.4	**6.2**	118.7	**4.7**	114.3	**0.8**	109.8	**-3.2**	103.1	**-9.1**
M3 - Top - Case 1	436.4	587.4	**34.6**	594.8	**36.3**	610.0	**39.8**	622.0	**42.5**	635.9	**45.7**
M3 - Top - Case2	436.4	691.0	**58.3**	709.2	**62.5**	749.2	**71.7**	783.7	**79.6**	827.5	**89.6**
M2 - Top - Case 1	329.3	514.6	**56.3**	524.1	**59.2**	543.6	**65.1**	559.1	**69.8**	577.1	**75.3**
M2 - Top - Case 2	329.3	475.8	**44.5**	482.5	**46.5**	495.9	**50.6**	506.2	**53.7**	517.7	**57.2**
3-storey - Seismic motion 1	Base Rotational Stiffness										
Base Column Moment (kNm)	Rigid	1.67*E*I/L	% Diff.	1.50*E*I/L	% Diff.	1.20*E*I/L	% Diff.	1.00*E*I/L	% Diff.	0.80*E*I/L	% Diff.
M3 - Base - Case 1	1045.8	440.0	**-57.9**	415.2	**-60.3**	364.8	**-65.1**	329.7	**-68.5**	298.5	**-71.5**
M3 - Base - Case2	1045.8	491.9	**-53.0**	489.8	**-53.2**	482.1	**-53.9**	462.8	**-55.7**	419.8	**-59.9**
M2 - Base - Case 1	374.4	473.3	**26.4**	472.0	**26.1**	478.4	**27.8**	489.5	**30.7**	500.3	**33.6**
M2 - Base - Case 2	374.4	317.4	**-15.2**	309.4	**-17.4**	320.5	**-14.4**	326.4	**-12.8**	323.7	**-13.5**
M3 - Top - Case 1	425.3	467.9	**10.0**	463.4	**9.0**	451.3	**6.1**	448.6	**5.5**	458.7	**7.9**
M3 - Top - Case2	425.3	503.7	**18.4**	525.5	**23.6**	593.5	**39.5**	657.7	**54.6**	723.5	**70.1**
M2 - Top - Case 1	285.4	353.5	**23.9**	356.6	**24.9**	360.1	**26.2**	360.1	**26.2**	359.3	**25.9**
M2 - Top - Case 2	285.4	291.2	**2.0**	295.6	**3.6**	327.7	**14.8**	356.1	**24.8**	387.6	**35.8**
3-storey - Seismic motion 2	Base Rotational Stiffness										
Base Column Moment (kNm)	Rigid	1.67*E*I/L	% Diff.	1.50*E*I/L	% Diff.	1.20*E*I/L	% Diff.	1.00*E*I/L	% Diff.	0.80*E*I/L	% Diff.
M3 - Base - Case 1	1087.2	597.4	**-45.1**	544.8	**-49.9**	500.3	**-54.0**	459.3	**-57.8**	407.2	**-62.5**
M3 - Base - Case2	1087.2	678.3	**-37.6**	659.9	**-39.3**	608.3	**-44.0**	558.1	**-48.7**	490.0	**-54.9**
M2 - Base - Case 1	413.1	642.8	**55.6**	628.6	**52.2**	622.7	**50.7**	626.1	**51.6**	660.5	**59.9**
M2 - Base - Case 2	413.1	426.6	**3.3**	430.8	**4.3**	425.9	**3.1**	415.9	**0.7**	390.5	**-5.5**
M3 - Top - Case 1	446.8	593.9	**32.9**	609.8	**36.5**	642.8	**43.9**	669.9	**49.9**	695.7	**55.7**
M3 - Top - Case2	446.8	747.8	**67.4**	775.6	**73.6**	824.8	**84.6**	857.1	**91.8**	887.6	**98.7**
M2 - Top - Case 1	334.3	536.5	**60.5**	525.6	**57.2**	526.6	**57.5**	530.2	**58.6**	560.5	**67.7**
M2 - Top - Case 2	334.3	454.4	**35.9**	485.1	**45.1**	511.7	**53.1**	529.7	**58.5**	539.4	**61.4**
3-storey - Seismic motion 3	Base Rotational Stiffness										
Base Column Moment (kNm)	Rigid	1.67*E*I/L	% Diff.	1.50*E*I/L	% Diff.	1.20*E*I/L	% Diff.	1.00*E*I/L	% Diff.	0.80*E*I/L	% Diff.
M3 - Base - Case 1	1276.9	648.1	**-49.2**	612.8	**-52.0**	540.9	**-57.6**	483.9	**-62.1**	417.9	**-67.3**
M3 - Base - Case2	1276.9	704.5	**-44.8**	671.3	**-47.4**	601.9	**-52.9**	554.5	**-56.6**	501.7	**-60.7**
M2 - Base - Case 1	472.6	663.8	**40.5**	673.4	**42.5**	693.6	**46.8**	709.6	**50.1**	728.2	**54.1**
M2 - Base - Case 2	472.6	464.3	**-1.8**	456.3	**-3.4**	436.7	**-7.6**	422.4	**-10.6**	403.3	**-14.7**
M3 - Top - Case 1	525.7	637.5	**21.3**	642.5	**22.2**	652.6	**24.1**	660.9	**25.7**	670.7	**27.6**
M3 - Top - Case2	525.7	731.3	**39.1**	746.0	**41.9**	777.7	**47.9**	822.2	**56.4**	879.8	**67.4**
M2 - Top - Case 1	380.1	553.5	**45.6**	562.3	**47.9**	580.8	**52.8**	595.5	**56.7**	612.5	**61.1**
M2 - Top - Case 2	380.1	503.8	**32.5**	509.3	**34.0**	520.1	**36.8**	536.4	**41.1**	556.7	**46.5**

Table 8. Linear and non-linear results 9-storey building - Column bending moments (at base and top) for different k_{base} values - % difference from Rigid case

9-storey - Linear	Base Rotational Stiffness										
Base Column Moment (kNm)	Rigid	1.67*E*I/L	% Diff.	1.50*E*I/L	% Diff.	1.20*E*I/L	% Diff.	1.00*E*I/L	% Diff.	0.80*E*I/L	% Diff.
M3 - Base - Case 1	373.2	217.1	**-41.8**	207.3	**-44.5**	186.4	**-50.1**	169.4	**-54.6**	148.9	**-60.1**
M3 - Base - Case2	373.2	249.1	**-33.3**	241.0	**-35.4**	223.8	**-40.0**	209.4	**-43.9**	191.8	**-48.6**
M2 - Base - Case 1	145.6	229.1	**57.3**	234.2	**60.9**	245.0	**68.3**	253.8	**74.3**	264.3	**81.5**
M2 - Base - Case 2	145.6	167.9	**15.3**	167.2	**14.8**	164.8	**13.2**	161.6	**11.0**	156.2	**7.3**
M3 - Top - Case 1	448.5	660.1	**47.2**	673.2	**50.1**	700.8	**56.3**	723.3	**61.3**	750.2	**67.3**
M3 - Top - Case2	448.5	817.5	**82.3**	850.1	**89.5**	924.6	**106.2**	992.1	**121.2**	1085.1	**141.9**
M2 - Top - Case 1	393.6	648.8	**64.8**	664.5	**68.8**	697.6	**77.2**	724.6	**84.1**	756.8	**92.3**
M2 - Top - Case 2	393.6	620.5	**57.6**	635.9	**61.6**	669.4	**70.1**	698.0	**77.3**	734.5	**86.6**
9-storey - Seismic motion 1	Base Rotational Stiffness										
Base Column Moment (kNm)	Rigid	1.67*E*I/L	% Diff.	1.50*E*I/L	% Diff.	1.20*E*I/L	% Diff.	1.00*E*I/L	% Diff.	0.80*E*I/L	% Diff.
M3 - Base - Case 1	986.5	526.6	**-46.6**	499.0	**-49.4**	441.7	**-55.2**	396.0	**-59.9**	345.3	**-65.0**
M3 - Base - Case2	986.5	588.4	**-40.4**	566.4	**-42.6**	531.1	**-46.2**	496.9	**-49.6**	450.4	**-54.3**
M2 - Base - Case 1	386.7	570.2	**47.5**	579.5	**49.9**	598.6	**54.8**	613.2	**58.6**	629.7	**62.8**
M2 - Base - Case 2	386.7	397.4	**2.8**	390.1	**0.9**	370.8	**-4.1**	351.4	**-9.1**	336.3	**-13.0**
M3 - Top - Case 1	370.0	468.1	**26.5**	474.1	**28.1**	486.1	**31.4**	509.1	**37.6**	535.8	**44.8**
M3 - Top - Case2	370.0	578.3	**56.3**	607.6	**64.2**	668.9	**80.8**	719.6	**94.5**	781.5	**111.2**
M2 - Top - Case 1	306.7	472.4	**54.0**	481.0	**56.8**	498.4	**62.5**	511.7	**66.8**	526.8	**71.8**
M2 - Top - Case 2	306.7	432.1	**40.9**	436.5	**42.3**	443.6	**44.6**	447.4	**45.9**	449.5	**46.6**
9-storey - Seismic motion 2	Base Rotational Stiffness										
Base Column Moment (kNm)	Rigid	1.67*E*I/L	% Diff.	1.50*E*I/L	% Diff.	1.20*E*I/L	% Diff.	1.00*E*I/L	% Diff.	0.80*E*I/L	% Diff.
M3 - Base - Case 1	1211.7	599.7	**-50.5**	570.1	**-53.0**	511.6	**-57.8**	465.3	**-61.6**	408.7	**-66.3**
M3 - Base - Case2	1211.7	689.1	**-43.1**	667.3	**-44.9**	617.6	**-49.0**	572.1	**-52.8**	515.8	**-57.4**
M2 - Base - Case 1	475.2	643.5	**35.4**	652.8	**37.4**	673.1	**41.6**	689.5	**45.1**	709.2	**49.2**
M2 - Base - Case 2	475.2	448.5	**-5.6**	441.6	**-7.1**	425.9	**-10.4**	410.7	**-13.6**	395.2	**-16.8**
M3 - Top - Case 1	418.9	538.3	**28.5**	550.4	**31.4**	574.6	**37.2**	594.3	**41.9**	616.0	**47.1**
M3 - Top - Case2	418.9	670.2	**60.0**	697.0	**66.4**	753.9	**80.0**	802.1	**91.5**	871.4	**108.0**
M2 - Top - Case 1	375.1	531.1	**41.6**	539.7	**43.9**	558.4	**48.9**	573.5	**52.9**	591.4	**57.7**
M2 - Top - Case 2	375.1	485.6	**29.5**	492.2	**31.2**	507.4	**35.3**	522.6	**39.3**	548.7	**46.3**
9-storey - Seismic motion 3	Base Rotational Stiffness										
Base Column Moment (kNm)	Rigid	1.67*E*I/L	% Diff.	1.50*E*I/L	% Diff.	1.20*E*I/L	% Diff.	1.00*E*I/L	% Diff.	0.80*E*I/L	% Diff.
M3 - Base - Case 1	1349.3	753.5	**-44.2**	715.8	**-47.0**	637.2	**-52.8**	573.9	**-57.5**	499.2	**-63.0**
M3 - Base - Case2	1349.3	845.5	**-37.3**	812.3	**-39.8**	741.9	**-45.0**	683.5	**-49.3**	611.7	**-54.7**
M2 - Base - Case 1	520.5	745.8	**43.3**	758.8	**45.8**	785.7	**51.0**	806.6	**55.0**	830.1	**59.5**
M2 - Base - Case 2	520.5	525.0	**0.9**	516.8	**-0.7**	506.9	**-2.6**	494.6	**-5.0**	473.8	**-9.0**
M3 - Top - Case 1	515.2	704.8	**36.8**	715.9	**39.0**	739.3	**43.5**	758.0	**47.1**	780.0	**51.4**
M3 - Top - Case2	515.2	849.3	**64.8**	876.9	**70.2**	938.4	**82.1**	992.2	**92.6**	1061.1	**106.0**
M2 - Top - Case 1	409.8	614.1	**49.9**	625.9	**52.7**	650.3	**58.7**	669.2	**63.3**	690.4	**68.5**
M2 - Top - Case 2	409.8	566.9	**38.3**	580.4	**41.6**	609.1	**48.6**	632.9	**54.4**	661.7	**61.5**

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5. Conclusions

This study highlights the critical role of column base rotational stiffness in the seismic response of steel moment-resisting frame (MRF) buildings. By examining the numerical results derived by linear and nonlinear seismic analyses, it is demonstrated that modelling assumptions at the base connection significantly influence key response indices, including interstorey drift ratios (IDRs) and bending moments. Two distinct cases are examined herein: in the first one the variation of the rotational stiffness is applied only to the strong axis (Case 1) and in the second one by applying the same rotational stiffness to both the strong and weak axes (Case 2). The results reveal that Case 2 generally produces larger IDRs and bending moments compared to Case 1, underlining the impact of weak axis flexibility on the response.

Moreover, the study illustrates the limitations of linear response spectrum analyses in estimating seismic demands, as they overestimate IDRs and underestimate bending moments compared to those found by the arguably accurate nonlinear time-history analyses.

The findings of this study underscore the necessity of refined modelling approaches for column base connections, particularly for cases where rotational stiffness significantly influences seismic performance. By assigning different values to the rotational stiffness and by comparing the results for three different buildings, this study provides some insights with respect to the improvement of seismic analysis and design of steel MRFs.

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