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Research Article

Spectral Assessment of the Effects of Base Flexibility on Seismic Demands of a Structure

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Base flexibility of structures changes and can increase the demands on structural elements during earthquake excitation. Such flexibility may come from the base connection, foundation, and soil under the foundation. This research evaluates the effects of column base rotational stiffness on the seismic demand of single storey frames with a range of periods using linear and nonlinear time history analysis. The base rotational stiffness ranges considered are based on previous studies considering foundation and baseplate flexibility. Linear and nonlinear spectral analyses show that increasing base flexibility generally increases frame lateral displacement and top moment of the column. Furthermore, moments at the top of the columns and the nonlinear base rotation may also increase with increasing base flexibility, especially for shorter period structures. Since many commonly used baseplate connections may be categorized as being semirigid, it is essential to design and model structures using realistic base rotational stiffness rather than simply use a fixed base assumption. The overall results also illustrate the range of increased seismic demand as a function of normalized rotational stiffness and structural period for consideration in design.

1. Introduction

The rotational stiffness at the base of a column affects the force and displacement demands on frame elements during an earthquake. In most analyses conducted for design, the column base is considered to be fully fixed. However, in real structures, there is foundation flexibility due to the soil, foundation, and base connection, all of which can change the rotational stiffness and violate this assumption leading to increased demands.

A number of studies have been conducted to evaluate the effect of base flexibility. Maan and Osman [1] modelled five- and ten-storey buildings with different column base flexibility values. They showed that pinned to fully fixed column base cases bounded the responses for all frames. Also, while frame displacements increased with increasing base flexibility, the frame displacement capacity also increased. The location of inelasticity could change with base flexibility and could increase at some levels. Aviram et al. [2] showed that increasing base flexibility increased displacement demands and concentrated deformations in the first storey of a three-storey building. Ruiz-Garcia and Kanvinde [3] showed that ideal pinned based connection leads to larger interstorey drift demands but smaller residual drift demands compared to the fixed base condition. Zareian and Kanvinde [4] found that increasing base flexibility results in the collapse mechanism with large deformations concentrated in a fewer storeys in 2-, 4-, 8-, and 12-storey steel moment resisting frames.

From this discussion, there appears to be a need to evaluate the effect of the base flexibility on structural demands for a wide range of structural periods.

This paper addresses this need for a single storey moment frame structure by seeking answers to the following questions:

(i) What rotational stiffness is likely at the base of columns in realistic steel frames?

(ii) What is the effect of column base flexibility on the frame top displacement, column top moment, and nonlinear base rotation?

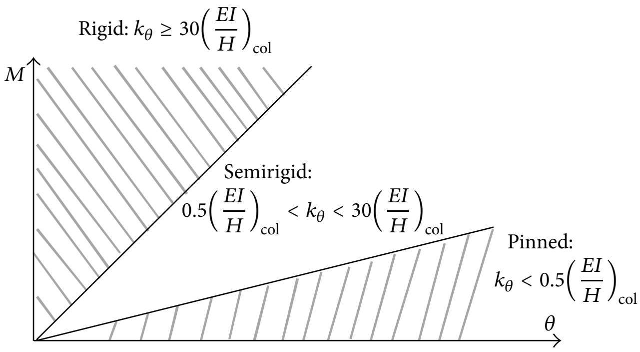


Figure 1: Eurocode 3 [5] column base rotational stiffness boundaries.

(1) Code Background to Connection Flexibility. Few standards state much about column base rotational stiffness, k_θ, which may be defined as the base moment, M, divided by the base rotation, θ. However, Eurocode 3 [5] defines base connections into categories depending on k_θ as shown in Figure 1. Here, k_θ is normalized by EI/H, where EI is the flexural stiffness of the column section and H is the height of the column to the point of inflection. Column bases with rotational stiffness, k_θ, of $30EI/H$ or more do not change the column ultimate strength by more than 5% or column lateral displacement under service load by more than 10% from the fixed base case according to Jaspart et al. [6]. In this case, the connection may be considered to be fully rigid. Base connection stiffness values lower than $0.5(EI/H)$ capture the fully pinned condition and connections may be modelled as being fully pinned. It is suggested that if the real rotational stiffness is between these two boundaries, then rotational stiffness should be explicitly considered in the analyses.

(2) Base Connections and Flexibility. The most common column base connections used in practice are either (1) exposed baseplate connections (EPBC) or (2) embedded column bases (ECB), as illustrated in Figures 2(a) and 2(b), respectively. The EPBC, with the column directly welded to the baseplate and the baseplate bolted to the foundation, has been most widely used on low to medium rise structures (Cui et al. [7]). Here, the rotation of the column, θ_{column}, under the column moment is equal to the footing rotation, $\theta_{footing}$, plus the baseplate rotation, θ_{plate}, and the base and footing stiffness can be considered as two rotational springs in series. For the ECB, with the base embedded in the reinforced concrete foundation, the beam and the footing rotations, θ_{beam} and $\theta_{footing}$, are the same as the column rotation, $\theta_{footing}$, and the rotational stiffness of the base may be considered to result from the two rotational springs of the footing and beam in parallel.

(a) Exposed Baseplate Connection (EBPC). Up to now, lots of tests were conducted on the exposed baseplate connections and their strength and stiffness were reported. In this section, some of the most referenced experimental tests and also the tests' results on different types of base connections that were conducted by authors are mentioned.

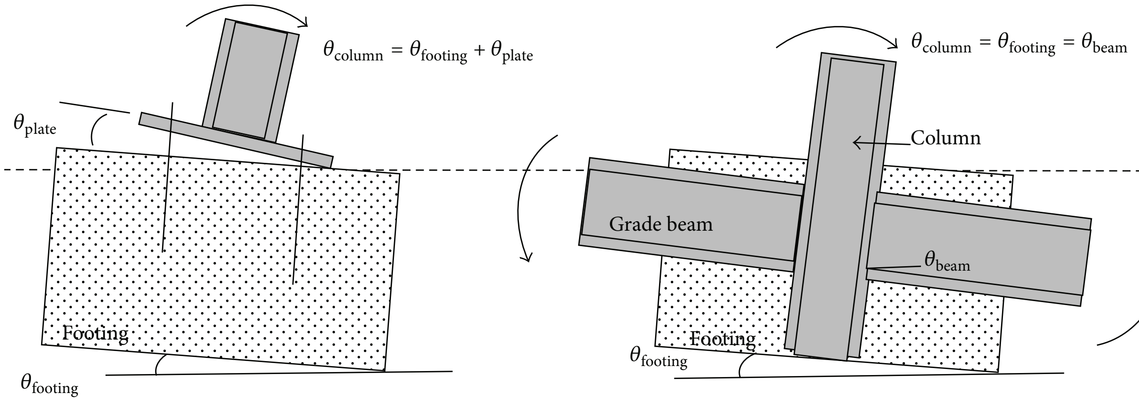
Picard and Beaulieu [8] showed that axial loading increased baseplate connection rotational stiffness of an EBPC with snug tightened nuts through the baseplate. Rotational stiffness, k_θ, defined as the 70% of the maximum applied, divided by the base rotation at that moment, θ, ranged between 662 kN·m/rad and 14492 kN·m/rad for the range of sizes and axial forces considered. Rotational stiffness may be expressed as a degree of fixity as per Eurocode 3 [5] by writing it in terms of EI/H. Here, EI is the material elastic modulus multiplied by the second moment of area for the column section tested, but the actual column base rotational stiffness, k_θ, is dependent only on moment and rotation, and it is independent of the position of loading H. The appropriate value of EI/H depends not only on the column section, but also on the likely height to the point of contraflexure from the base for that particular column. Here, the column stiffness ranges above may be written as lying between $1.65EI/H_{st}$ and $7.6EI/H_{st}$, where H_{st} is simply taken as 2.2 m (EI is different since three different column sections were tested). This is approximately 63% of the base storey column height of 3.5 m to the point of contraflexure for a typical full-size column.

Robertson [9] tested a frame where the column base rotational stiffness could be set. He used rotational stiffness values of 150 kN·m/rad ($0.1EI/H_{st}$), 1500 kN·m/rad ($1.3EI/H_{st}$), and 3300 kN·m/rad ($2.8EI/H_{st}$) representing a pinned, and 2 different degrees of semirigid, base, respectively. Robertson also stated that the rotational stiffness for typical 20 mm thick baseplate connections was likely to range from 500 kN·m/rad ($0.4EI/H_{st}$) to 3500 kN·m/rad ($3EI/H_{st}$).

Gomez et al. [10] assessed the effect of axial load, baseplate thickness, and anchor-rod strength on total performance where the nuts were snug tightened. The rotational stiffness of these base connections was in the range of 7,760 kN·m/rad ($1.2EI/H_{st}$) to 41,310 kN·m/rad ($5.8EI/H_{st}$). Deformation of the plate and of the concrete under the compression side and elongation of the anchor bolts were the main sources of observed flexibility.

Borzouie [11] conducted some experiment tests on the exposed baseplate connections and the base connections with yielding angles and friction connections to evaluate them from low damage aspect. The rotational stiffness for the exposed baseplate connections was in the range of 14,736 kN·m/rad ($1.47EI/H_{st}$) to 38,181 kN·m/rad ($3.8EI/H_{st}$). According to these tests, the main source of the flexibility was the anchor rods; therefore, the anchor rods were posttensioned in the second series of the tests to evaluate the performance of this type of base connection without elongation of the anchor rod. The rotational stiffness was increased up to 55,443 kN·m/rad ($5.54EI/H_{st}$) due to the preloading of the anchor bolts. The rotational stiffness was lower for the tested based connections with friction devices and yielding angles that were ranged from 1,100 kN·m/rad ($0.11EI/H_{st}$) to 8,994 kN·m/rad ($0.9EI/H_{st}$).

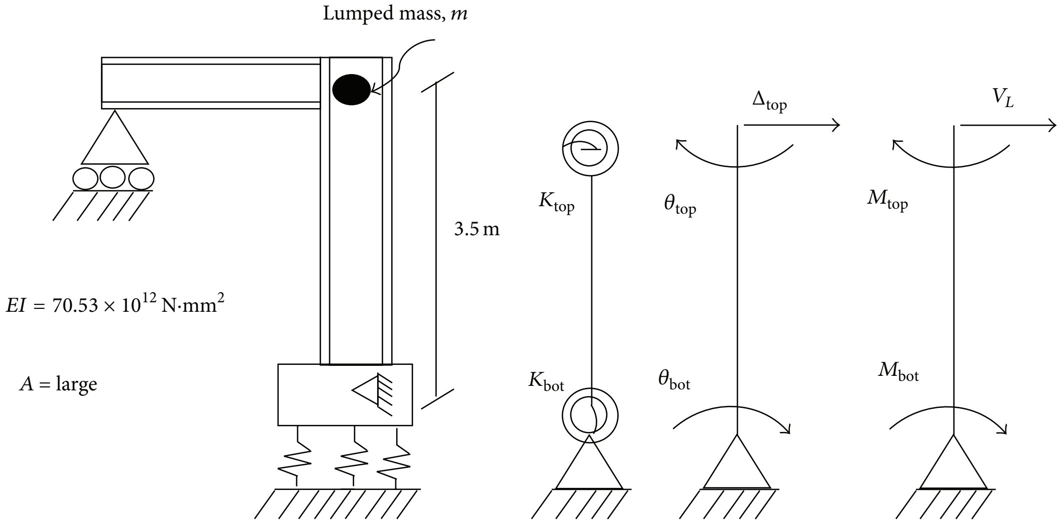
(b) Embedded Steel Column Base (ECB). Nakashima and Igarashi [12], Morino et al. [13], and Grauvilardell et al. [14] categorised the performance of ECB based on embedment length. For deeply embedded types, when the embedment length was no less than two times the lateral dimension of the



(a) Exposed baseplate connection (EBPC)

(b) Embedded column base (ECB)

Figure 2: Common base connections (Zareian and Kanvinde [4]).



(a) Column (b) Springs (c) DOFs (d) Forces

Figure 3: Analysis model.

column cross section in the plane of bending, the column may be assumed to be fixed. For shallower embedments, rotational stiffness decreased and the failure mechanism changed from column yielding to cracking of the concrete and yielding of the anchor bolts.

(3) Foundation and Soil Flexibility. The soil below the foundation also contributes to overall column base stiffness as investigated for different soils by Winterkorn and Fang [15] and modified by Melchers [16] to consider uplift effects.

(4) Preface. It may be seen from the above discussion that columns bases may not be rigid as a result of connection, foundation, and soil flexibility. Previous studies have quantified these effects individually. Also, the base flexibility effect on a number of individual frames has been quantified and none of these studies was general enough to provide design guidance for buildings with different periods. In addition, all these studies have evidenced failure or nonlinear deformation in base connections, although they are designed to remain elastic and studying the impact of nonlinear base behaviour with different levels of strength is also required. In this study, the likely range of rotational stiffness considering the combination of connection, foundation, and soil flexibility is first obtained. Then, linear and nonlinear spectral analyses are conducted to determine the likely change in response for a wide range of structural periods and strengths to understand the likely range of responses of single storey structures due to this base flexibility.

2. Methodology

A single storey steel moment frame is modelled. Figure 3(a) represents half of a single storey frame with the beam roller pin support at zero moment connection at the beam midlength. The base of the column is fixed laterally but can rotate. The rotational stiffness of the beam can be represented by a rotational spring with stiffness, K_{top}, as

Table 1: Test data of baseplate from Gomez [17] tests and evaluation of K_θ for different types of soil based on Melchers [16].

BP thickness (mm)	N^*/N_s [d]	$K_{\theta,\mathrm{BP}}$ [a,b] (EI/H_{st})	Type of soil	$K_{\theta\,\mathrm{footing}}$ [a,c] (EI/H_{st})	$K_{\theta,\mathrm{total}}$ [a] (EI/H_{st})
25.4	0	1.24	Soft clay	0	0
			Loose sand	0	0
			Dense sand and gravel	0	0
			Basalt	0	0
	0.16	3.49	Soft clay	0.14	0.14
			Loose sand	1.82	1.20
			Dense sand and gravel	8.55	2.48
			Basalt	3590	3.49
	0.27	5.81	Soft clay	0.15	0.15
			Loose sand	1.95	1.46
			Dense sand and gravel	9.19	3.56
			Basalt	3863	5.80
38.1	0.16	3.07	Soft clay	0.12	0.12
			Loose sand	1.57	1.04
			Dense sand and gravel	7.33	2.17
			Basalt	3097	3.07
50.8	0.16	3.02	Soft clay	0.12	0.12
			Loose sand	1.54	1.02
			Dense sand and gravel	7.25	2.13
			Basalt	3046	3.01

[a]The test's column section was W200 × 7, Grade 300.
[b]For tests that were conducted several times, the median results are presented in the table.
[c]The footing is assumed to be rigid relative to the soil that it rests on, with dimension of 1.2 m × 1.2 m.
[d]N_s is the nominal column section capacity and N^* is the applied axial force to the column.

shown in Figure 3(b). K_{bot} represents the total base rotational flexibility from the connection, foundation, and soil below. The resulting column degrees of freedom consist of one mass degree of freedom, Δ_{top}, and two massless degrees of freedom, θ_{top} and θ_{bot}, as shown in Figure 3(c). Associated forces are in Figure 3(d), where M_{top}, M_{bot}, and V_L are the top and base moment and lateral shear force applied to the frame. The period of the structure is changed from 0.3 s to 5 s in steps of 0.1 s by changing the mass, m. Tangent stiffness proportional damping with a ratio of 5% is assumed for the first mode.

To estimate realistic column base rotational stiffness for analysis, the base connection stiffness of Gomez et al. [10], together with the soil effect from Melchers [16], is used as shown in Table 1. It may be seen that baseplate rotational stiffness, $K_{\theta\mathrm{BP}}$, is less than $6EI/H_{\mathrm{st}}$. When soil flexibility is also considered, assuming a 1.2 m × 1.2 m foundation block on different soil types (soft clay, loose sand, dense sand and gravel, and basalt), the total foundation flexibility, $K_{\theta,\mathrm{total}}$, is obtained considering the springs in series. According to Table 1, the column baseplate connection stiffness is significantly less than the rigid level of $30EI/H_{\mathrm{st}}$ of Eurocode 3 [5], implying that no baseplate connections should be considered to be fully rigid.

In this study, rotational stiffness values of $0EI/H$ (fully pinned), $5EI/H$ (intermediate), and $2000EI/H$ (fully fixed) are considered. Here, H is the height of the frame of 3.5 m, as the point of contraflexure depends on the end fixities. Baseplate connections will generally result in stiffness between about $0EI/H$ and $5EI/H$ while those with greater stiffness represent embedded connections with stiff beams on hard soil.

The double curvature period, T_{dc}, is used in this analysis because the period of the structure is only affected by the stiffness and mass of the frame, and it is independent of rotational stiffness at top and base. T_{dc} is the period of the column when it is rotationally fixed at the base and the top with lateral stiffness of $12EI/H^3$. For example, the true period of a fully fixed base column that is pinned at the top (with lateral stiffness of $K_L = 3EI/H^3$) is twice that of the double curvature period, T_{dc}. Therefore, for a fixed base structure with top stiffness of $0EI/H$, $5EI/H$, and $2000EI/H$, the actual period, T, is $2T_{\mathrm{dc}}$, $1.22T_{\mathrm{dc}}$, and $1.0T_{\mathrm{dc}}$, respectively. This approach allows consistent comparison across all cases analysed and is easily converted to a normal period.

Twenty medium suite earthquake records (La 10 in 50) from the SAC steel project for Los Angeles with a probability of 10% in 50 years were used for time history analysis. The elastic spectral displacement for these records and the median value are shown in Figure 4. All 20 records were used for each configuration of structure, and the median result was presented.

The elastic analysis study involves the following parameters:

(i) K_{top} of $0EI/H$, $5EI/H$, and $2000EI/H$.

(ii) K_{bot} of $0EI/H$, $5EI/H$, and $2000EI/H$.

(iii) Double curvature period, T_{dc}, between 0.3 s and 5.0 s, $\Delta T_{\mathrm{dc}} = 0.1$ s.

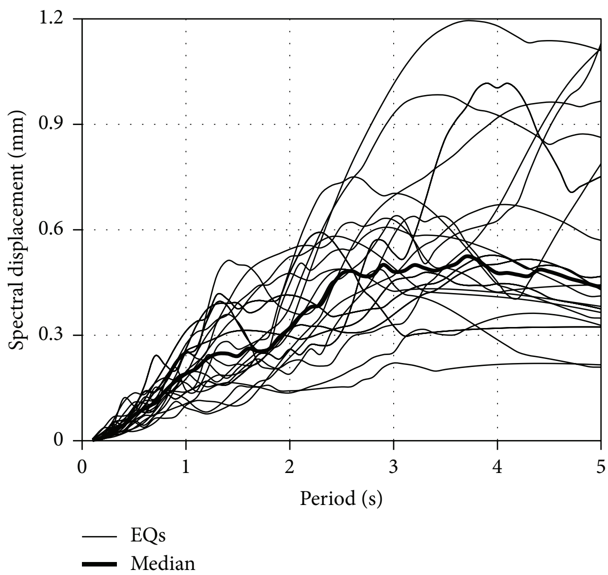


Figure 4: Median and record displacement response spectrum for SAC La 10 in 50 suite.

The results are shown as a spectral plot ratio of $x^{K_{\text{top}}=\beta EI/H}_{K_{\text{bot}}=\alpha EI/H}$ to $x^{K_{\text{top}}=\beta EI/H}_{K_{\text{bot}}=\text{fixed}}$ over T_{dc}, where x is the demand parameter, such as top displacement and top moment, and K_{top} and K_{bot} are top and base rotational stiffness, respectively. When the response causes greater demands than for the rigid base case, the ratio is greater than one.

The entire study is repeated for nonlinear analysis where the column is rotationally fixed at the top and the same column base flexural strength. The nonlinear base rotation of a frame with column base rotational flexibility is compared with that for a rigid base over a range of T_{dc}. Note that P-Δ effects were not considered in linear and nonlinear analysis. The base yielding moment is defined according to the maximum elastic moment of the fixed top and base column, $M_{\text{Elastic,FB}}$.

The Menegotto and Pinto [18] M-Δ hysteretic curve was used for modelling the column moment-rotation behaviour as shown in Figure 5(a) for the nonlinear analysis. Curve properties are given in Table 2. The curve is not fully piecewise linear due to rounding of the corners represented by γ. This nonlinearity is used to consider yielding. Figure 5(b) presents a representative hysteresis loop used in the analysis.

For nonlinear analysis, the results are presented as a plot of the ratio of the nonlinear base rotation of a frame that is fixed at the top with rotational stiffness, $\theta_{\text{NL},K_{\text{bot}}}$, to nonlinear base rotation of the fixed top and base frame, $\theta_{\text{NL,Fixed}}$, over base yielding moment, M_y, and base rotational stiffness, K_{bot}. This ratio is presented for three ranges of T_{dc} to simulate short ($0.3\,\text{s} \le T_{\text{dc}} < 1\,\text{s}$), medium ($1\,\text{s} \le T_{\text{dc}} < 3\,\text{s}$), and long ($3\,\text{s} \le T_{\text{dc}} < 5\,\text{s}$) periods. For each level of M_y ($0.3M_{\text{Elastic,FB}} \le M_y \le 0.9M_{\text{Elastic,FB}}$) and K_{bot} ($1EI/H \le K_{\text{bot}} \le 9EI/H$), the maximum value of this ratio over the range of T_{dc} is plotted. Ratios larger than one represent a nonconservative design, and ratios less than one show that the design is conservative.

Table 2: Properties of the Menegotto-Pinto hysteretic loop and its governing equation.

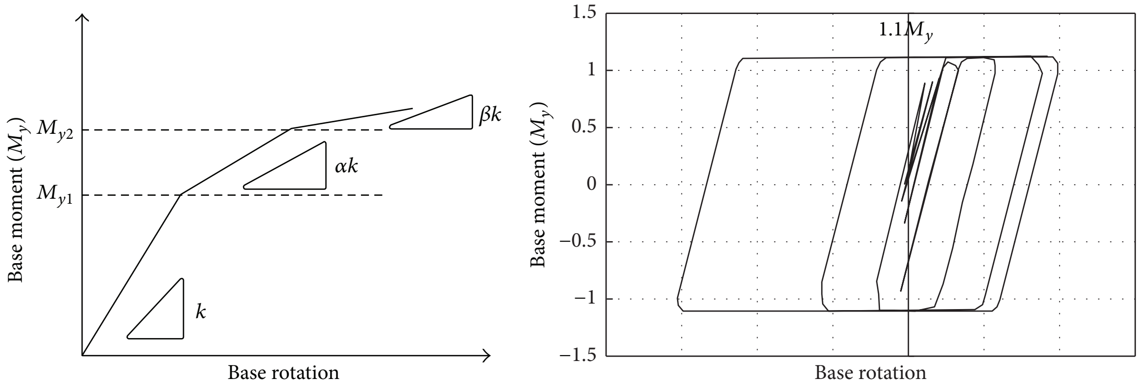
Properties	Value
α	0.1
β	0
M_{y2}	$1.1M_{y1}$
γ (curvature of loop's corners)	10
Equation: $F = \dfrac{k}{(1+(k/M_y)^{\gamma})^{1/\gamma}}$	

Based on simple theoretical considerations, it would be expected that, for a frame with a rigid beam, by changing from a fixed base to a pinned base, the frame stiffness decreases by a factor of 4 ($=3EI/H^3/(12EI/H^3)$) so the period doubles. For structures in the range where the response spectra increase linearly with period and when the equal displacement method holds, the doubling of period is associated with doubling of total displacement. In the elastic range, this results in the same moment at the top of the column as for the rigid base case because the displacement is double and the moment diagram is in single rather than double curvature. It also reduces the moment at the top of the column and the base plastic hinge rotational demand in the inelastic case. For this reason, it is possible that, apart from displacement demands, other moment and plastic hinge demands may be reduced. However, there may be significant variation from this due to the actual beam stiffness, the shape of the response spectra, P-delta effects, variation from the equal displacement assumption, and so forth.

3. Results and Discussion

3.1. Elastic Response Variation. Figure 6 shows roof displacement, Δ_{top}, increased with reduction of base rotational stiffness relative to a fixed base structure with T_{dc} less than 1.5 s, 2.4 s, and 3 s for $K_{\text{top}} = 0$, $5EI/H$, and $2000EI/H$, respectively. For T_{dc} greater than these values (1.5 s, 2.4 s, and 3 s), the displacements do not change very much from the fixed base assumption. The demands are consistent with the response spectrum of Figure 4, where longer periods see an increase in spectral displacement up to a true period of about 3.0 s, which is consistent for T_{dc} given. The response is greater than 2 times the fixed base elastic response for the column with top stiffness and short period ($T_{\text{dc}} < 0.8$ s). This is because the shape of the elastic spectra is not linear with period. In Figure 6(a), $K_{\text{bot}} = 0$ is not shown because it is statically unstable.

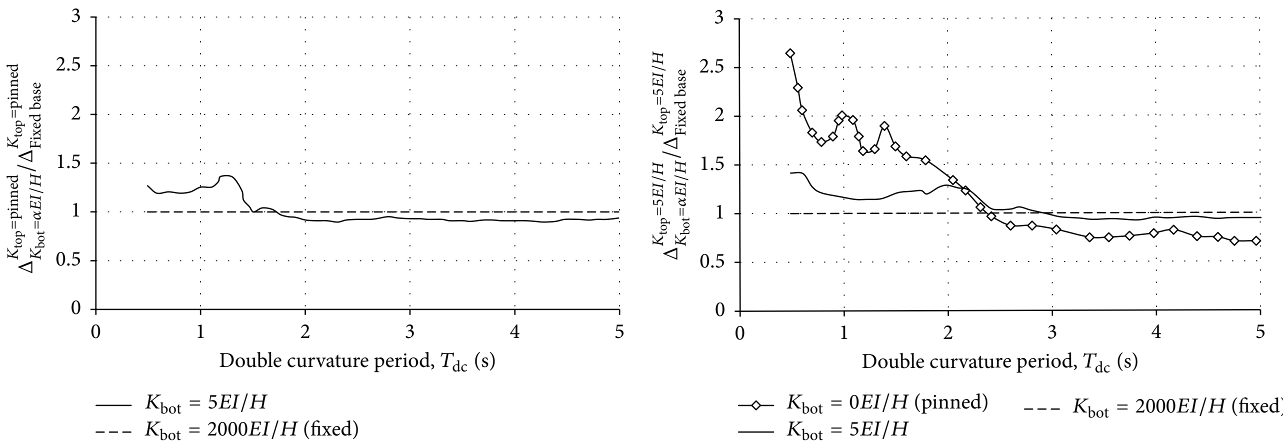
Figure 7 shows that the moment demand at the top of the column, M_{top}, increases with the base flexibility when $T_{\text{dc}} < 0.8$ s. Such an increase in moment increases the possibility of yielding at the top of the column, as well as the likelihood of a soft storey mechanism. This increase in moment response is consistent with the range of period causing amplification of displacement by more than 2 according to the theoretical considerations above.



(a) Idealized loading response for first half cycle

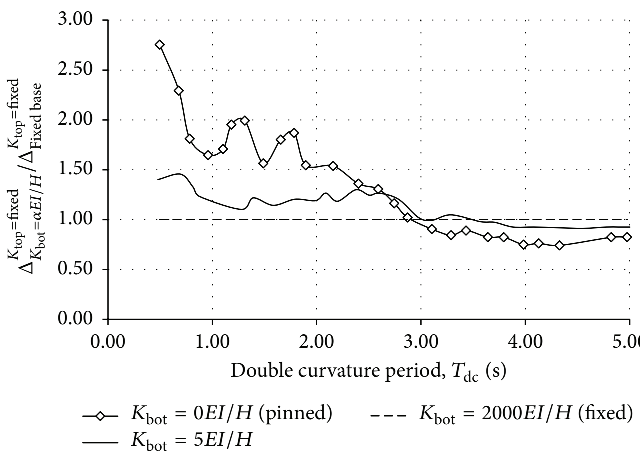
(b) Cyclic response

Figure 5: Menegotto-Pinto hysteretic loop.



(a) Pinned top, $K_{top} = 0EI/H$

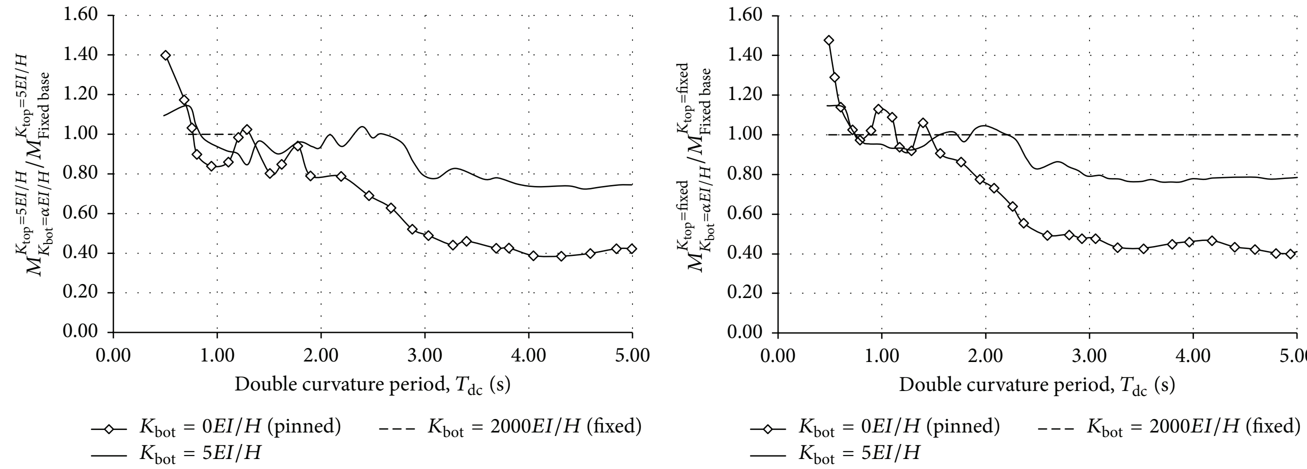
(b) $K_{top} = 5EI/H$



(c) Fixed top, $K_{top} = 2000EI/H$

Figure 6: Base flexibility effect on top displacement for various period (elastic analysis: $K_{top}H/(EI) = 0$, 5, and 2000).

3.2. Nonlinear Base Rotation Variation. Figure 8 shows the ratio of the median maximum nonlinear base rotation considering base flexibility, $\theta_{NL,K_{bot}}$, to that with a fixed base, $\theta_{NL,Fixed}$. The column is rotationally fixed at the top, the rotational stiffness at the base, $K_{bot}/(EI/H)$, ranges from $0EI/H$ *to* $9EI/H$, and the yield strength, M_y, ranges from 0.3 to $0.9M_{Elastic}$. Here, M_y is computed based on $M_{Elastic}$ for each record. For $M_y > 1.0M_{Elastic}$ the base performs elastically. For $M_y = 0M_{Elastic}$, the base is considered pinned. In both cases, there is no inelastic base rotation, so these cases were not



(a) $K_{top} = 5EI/H$

(b) Fixed top, $K_{top} = 2000EI/H$

Figure 7: Base flexibility effect on elastic top moment for linear case.

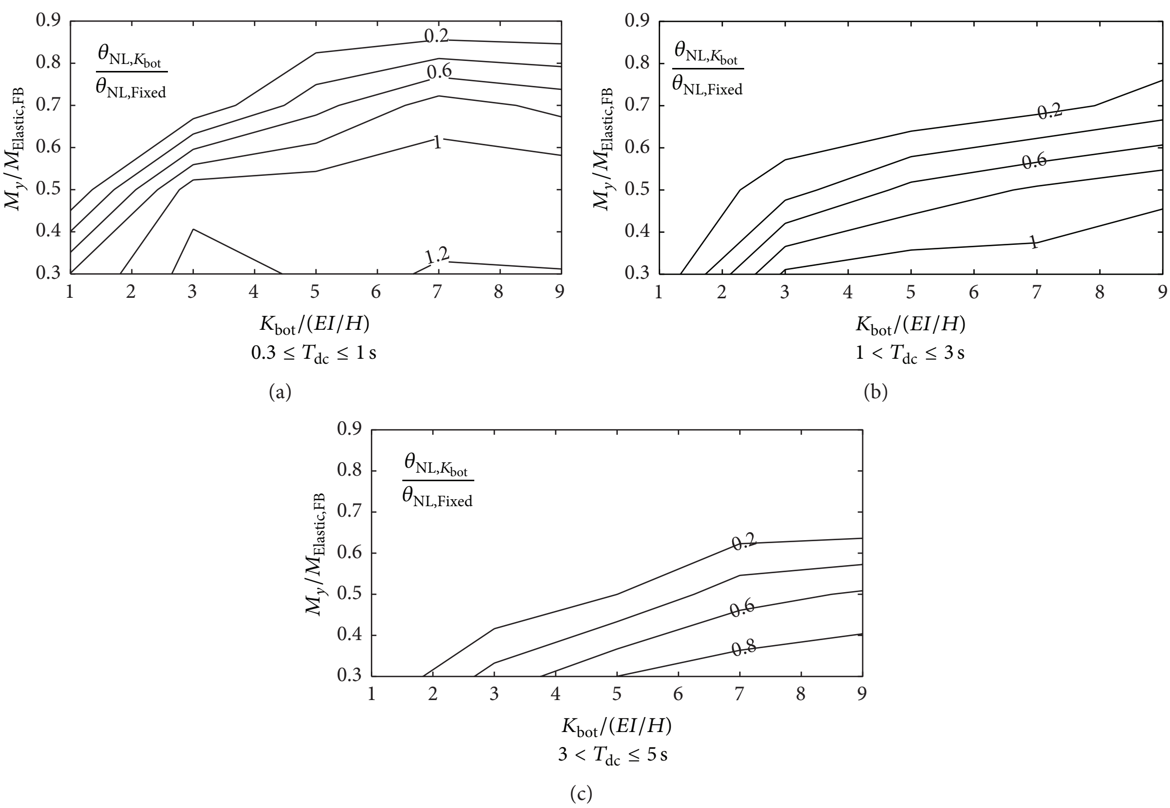


Figure 8: Median nonlinear base rotation ratio, $\theta_{NL,K_{bot}}/\theta_{NL,\text{Fixed}}$, for different base rotational stiffness ($K_{bot}/(EI/H)$) and yielding moment ratio (M_y/M_{Elastic}) for the suite of twenty records.

considered. The ratio of $\theta_{NL,K_{bot}}/\theta_{NL,\text{Fixed}}$ is plotted for period ranges of $0.3\,\text{s} \leq T_{dc} \leq 1.0\,\text{s}$, $1\,\text{s} < T_{dc} \leq 3.0\,\text{s}$, and $3\,\text{s} < T_{dc} \leq 5.0\,\text{s}$ in Figure 8. It may be seen that $\theta_{NL,K_{bot}}/\theta_{NL,\text{Fixed}}$ tends to increase as T_{dc} decreases, K_{bot} increases, and M_y decreases. The shaded area in Figure 8 is the range associated with greater nonlinear rotation of the base compared to a fixed base case where $\theta_{NL,K_{bot}}/\theta_{NL,\text{Fixed}} > 1.0$. It increases with lower T_{dc} and lower M_y. In the figures shown, it also generally increases with greater K_{bot} (except in Figure 8(a) where it peaks and is starting to reduce), but $\theta_{NL,K_{bot}}/\theta_{NL,\text{Fixed}}$ will return to unity as K_{bot} tends to infinity. It is up to about 20% more than the fixed base rotation.

4. Conclusion

This paper presents linear and nonlinear spectral analysis of the impact of base flexibility on seismic demands over a suite of design level ground motions. The following were found:

(1) Column base flexibility is affected by the flexibility of the column base connection, the foundation beneath the connection, and the soil supporting the foundation and can vary from being pinned to rigid. Rigid ones are likely to be embedded columns. For typical baseplate connections, it is difficult to get rotational stiffness greater than $6EI/L$. Since base flexibility affects the response, it should be considered in design.

(2) The effect of base flexibility on structural demands depends on the period of the frame, fixities at the column ends, and strength as follows:

(i) For top displacement of elastically responding frames, the rigid base assumption was conservative for structures with periods above 3 s period for all cases. Results are nonconservative below periods of 3 s for fixed top connections, reducing to 1.5 s for pinned top connections. Displacements were more than twice the fixed base displacement for periods less than 0.8 s.

(ii) The moment demand in an elastically responding frame at the top of the column is increased for periods below 0.8 s. This is consistent with the displacement change.

(iii) The nonlinear base rotation increased by up to about 20% on average for structures with low period, moderate K_{bot}, and low M_y.

For these reasons, neglecting the base flexibility effects arising from soil, foundation, and base connection flexibility can lead to nonconservative designs. In addition, nonlinear base rotation has a direct relation to the base connection damage and is not generally reduced by the base flexibility. Therefore, base flexibility effects may not reduce damage as is often assumed.

Disclosure

All opinions expressed are those of the authors.

Competing Interests

The authors declare that they have no competing interests.

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