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INSIGHTCORE

Journal of Buildings and Sustainability

2017
Vol. 2
No. 2

Column Slenderness Analysis for Reinforced Concrete Frame Structures using Finite Element Modelling

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A B S T R A C T

This research is done to understand the slenderness effects of Reinforced Concrete (RC) columns in a frame structure through a parametric study considering column design inputs and using a Finite Element Method (FEM) modelling procedure. Three ten-story commercial buildings are modeled using FEM software ETABS. A parametric study is done on three different square floor panels, five different column heights, and three different column locations in the RC beam-column frame structures. It is observed that the corner column is the most sensitive to become slender in comparison to the edge and the inner columns. A 17.5′ (5334 mm) long column becomes slender according to the slenderness criteria recommended by the American Concrete Institute (ACI) code. All RC frames should be considered as a sway frame, since a column can be a non-slender member when considering the RC frame as a non-sway frame but is a slender member while considering it as a sway frame. Also, column slenderness should be checked using both the Slenderness Ratio (*kl/r*) and the Moment Magnification Factor (δns and δs). A column is a non-slender member when considering the Slenderness Ratio, but it behaves as a slender member when considering the Moment Magnification Factor.

Keywords: Column, Finite Element Modelling, Non-Sway, Slenderness, Sway, Moment Magnification Factor

1. Introduction

Slender columns are the structural members whose ultimate load carrying capacities are affected by the slenderness effect and produce additional bending stresses and instability of columns due to excessive deflection. A slender column has less strength than a short column of the same sectional area, and hence it carries a lesser load as compared to a short column (Kumar, 2005). The slenderness of a column dramatically increases due to increase in column height and buckling under gravity or horizontal loads (Halder, 2007). Therefore, evaluation of a slender column involves consideration of the column height, its cross-sectional size in addition to the type of load.

Slender columns exhibit deflections when being subjected to eccentric loads. These deflections produce additional flexural stresses due to the increase in eccentricity by the amount of transverse deflection (Δ). This phenomenon is known as the P-Δ effect. P-Δ is sometimes referred to as the "secondary moment." This secondary moment reduces the axial load carrying capacity of a slender column (Nilson, Darwin, & Dolan, 2003). P-Δ is a non-linear effect that occurs in every structure where columns are subjected to horizontal loads such as wind loads. P-Δ is only one of many second-order effects (Dobson & Arnot, 2003). The P-Δ effect should be included in the analysis for the design of high-rise buildings in which the story drift exceeds 1/85 radians during expected earthquake excitations in seismic regions (Mollick, 1997). The P-Δ effect should be considered as structures become even more slender and less resistant to deformation. Rathbone (2002) noted that P-Δ effects in structures will always occur and always requires consideration. As a result, codes of practice are referring to the engineers - to use the second order analysis progressively so that P-Δ effects are accounted for when appropriate in design. Second order analysis is as essential in concrete and timber design as it is in the design of steelwork (Dobson & Arnott, 2003).

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If the total moment including the secondary moment reaches the ultimate capacity of a section, the column fails to owe to material failure. Parameters such as column-buckling effect, elastic shortening, and secondary moments due to lateral deflection, are not needed when designing short columns but are needed for the design of slender columns (Hassoun, 2005). The concept of diagnosing whether the existence of any slender column effects is imperative and should be considered before the actual slenderness effect calculation procedure is performed (Ferguson, Breen, & Jirsa, 1987).

If a column is very slender, it becomes unstable before reaching material failure, and such instability failure is observed in those columns (Bazant & Know, 1994). Slenderness effects are more pronounced in a column of a unbraced frame than a braced frame. Frames that do not have adequate bracing against lateral loads show excessive sway, which jeopardizes the stability of columns. Adequate bracing in a frame helps to stabilize secondary deformations at column ends and produces more stable columns. Columns are treated differently depending on the bracing conditions in their frames because of different behavior between a braced and an unbraced frame.

The vertical extension of the buildings is now essential for Bangladesh due to deficiency of land, cost of property, and to accommodate a vast number of growing human population in small areas. Most of the buildings are concrete beam-column or flat-plate frame structures. For the high-rise structures, it is frequently seen that there is an increased ground floor height from conventional ground floor heights due to architectural or functional purpose, which subsequently turns a column slender.

However, with the increasing use of high-strength materials and improved methods of dimensioning members, it is now possible to design much smaller cross-sections. Together with the use of more innovative structural concepts, that are rational and reliable, the design procedure for slender columns have become increasingly important. The American Concrete Institute (ACI) code provides different guidelines for designing slender columns with relation to critical parameters in non-sway and sway frames. Commercially available software for structural design of building structures incorporates the ACI code design guidelines. The use of commercially available structural analysis and design software are increasing on a daily basis in design offices. However, until recently, there are very few studies done to understand slender column behavior using finite element modelling (Abdallah, Shazly, Mohamed, Masmoudi, & Mousa, 2017; Mustafa, Elhussieny, Matar, Alaaser, 2016; Rodrigues, Manzoli, Bitencourt Jr, dos Prazeres, & Bittencourt, 2015). Also, application of slender column design procedures is not straightforward, and this also increases the chances of having an under-designed structure.

2. Objective and Methodology

Considering the effect of slenderness in column design is becoming very important especially with the rapid increase in materials properties in high-rise construction. The concept of diagnosing whether any slender column effect exists is crucial and should be considered before the actual slenderness effect calculation procedures are performed. For this reason, it is essential to understand the ACI code design process for slender columns. The parameters that influence the design of slender columns need careful evaluation. The objective of this research is to study the effect of slenderness in Reinforced Concrete (RC) column design by carrying out a parametric approach and identifying the critical parameters for the RC beam-column structures.

In this research, one typical model of a high-rise building is developed using ETABS software (Computers & Structures, 2003). The model is developed considering all possible gravity and horizontal loads. Essential parameters that affect column slenderness such as column height, column dimension, and span length are considered as a parameter and are varied in the normal range.

It should be noted that ACI code (318-1999) is followed. The reason for using an old version of the code is because the Bangladesh National Building Code (BNBC, 2006) was developed by following an old version of the ACI code. In this study, BNBC is used to perform dead load, live load, wind load, and earthquake load calculations and other necessary parameters required for a building design. BNBC is a comprehensive code, which is developed using ACI, ASCE (American Society of Civil Engineers), IBC (International Building Code), and UBC (Uniform Building Code) codes. Though, BNBC revision is currently underway to adapt and address the new ACI code (318-2011); however, use of factors to magnify load magnitudes is still being utilized by the current and updated appendix sections of the ACI code (318-2011). Moreover, the basic slender column design procedure has not been changed in the current edition of the ACI code.

3. Slender Column Design Procedures

As suggested in the ACI Commentary 10.11.4, a compression member can be assumed braced if it is located in a story in which the bracing elements (i.e., shear walls) have a stiffness that is substantial enough to limit lateral deflection to the extent that the column strength is not substantially affected. Visual inspection can often make such a determination. If not, the ACI Code 10.11.4 provides two quantitative criteria for determining if a story is treated as non-sway or a sway frame. In one of the methods, it has been mentioned that if the Stability Index is less than 0.05, then the frame is considered as braced. The Stability Index can be determined by using the equation (1).

$$Q = \frac{\sum P_u \Delta_0}{\sum V_u l_c} \tag{1}$$

where Q is the stability index, is the total factored vertical load in the story, V_u is the total factored shear load in the story, l_c is the height of the column measured center-to-center of the joints in the frame, and Δ_0 is the first-order relative deflection between the top and the bottom of the story due to V_u.

A frame could have both non-sway and sway effects. For a column, it could have a more substantial non-sway effect than a sway effect and vice-versa. Primarily, Slenderness Ratio (kl/r) is determined for each column to determine non-sway or sway effect. For a non-sway frame, a designer can neglect slenderness effect if and for a sway frame slenderness effect can be neglected if $kl_u/r < 22$, where k is the effective height factor; l_u is the unsupported height taken as the clear distance between two floor slabs, beams, or other members providing lateral support; r is the radius of gyration of the cross-section of the column associated with the axis about which bending is occur; M_1 is the value of the smaller end moment calculated from a conventional first-order elastic analysis (positive if the column is bent in a single curvature and negative if it is bent in a double curvature); and M_2 is the value of the larger factored end moment on the column, and is always positive.

If a column is slender, it will fail by buckling into the shape of a sine wave when the load reaches a particular value P_c, which is called the Euler buckling load or critical load that is given by the equation (2),

$$P_c = \frac{\pi^2 E I_{\min}}{(kl)^2} \tag{2}$$

where E is the elastic modulus of the column and I_{min} is the minimum moment of inertia of the column. It is seen that the buckling load decreases rapidly with the increase in the Slenderness Ratio (kl/r).

If a column falls under either a non-sway or a sway frame and crosses the limiting value, then a Moment Magnification Factor (δ_{ns} or δ_s) is required to be calculated. Steps used to compute a Non-Sway Moment Magnification Factor (δ_{ns}) are shown in the equations (3) to (6),

$$EI = \frac{0.4 E_c I_g}{1 + \beta_d} \tag{3}$$

$$\beta_d = \frac{\text{Factored dead load within a story}}{\text{Total factored shear in the story}} \tag{4}$$

$$\delta_{ns} = \frac{C_m}{\left(1 - \dfrac{P_u}{0.75 P_c}\right)} \tag{5}$$

$$C_m = 0.6 + 0.4\frac{M_1}{M_2} \geq 0.40 \tag{6}$$

where E_c is the elastic modulus of the column and I_g is the effective moment of inertia of the girder, which is equal to $0.35I_{gross}$, where I_{gross} is the gross moment of inertia of the girder, P_u is the ultimate load of the column, and C_m is a factor which is a function of the column moments. The column's original moments are then multiplied by the δ_{ns} to get the moment required for the calculation of steel ratio. The δ_{ns} value could be less than 1.0, and for that case, it is assumed as 1.0. If δ_{ns} is greater than 1.0, then the column is considered to have a slenderness effect. However, a column could have a δ_{ns} value of less than 1.0, but the *kl/r* crosses the limiting value or vice-versa. For such cases, considering a column as a slender column would give some safety factor in the design.

The step required to calculate a Sway Moment Magnification Factor (δ_s) is shown in the equation (7),

$$\delta_s = \frac{1}{\left(1 - \dfrac{\sum P_u}{0.75\sum P_c}\right)} \tag{7}$$

P_c is calculated for each column in the story of the column being designed and then $\sum P_c$ is calculated for the given story. Similarly, P_u is calculated for each column in the story of the column being designed and then $\sum P_u$ is calculated for the given story.

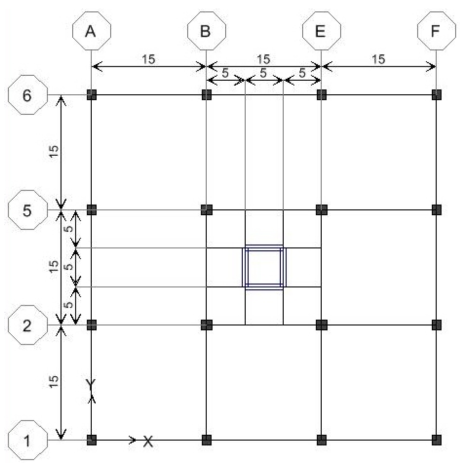
In a RC frame, columns are rigidly attached to girders and adjacent columns. The effective height of a particular column between stories will depend on how the frame is braced and on the bending stiffness of the girders. For frames braced against side sway, *k* varies from 1/2 to 1, whereas for laterally unbraced frames, it varies from 1 to ∞, depending on the degree of rotational restraint at both ends.

It should be noted that ACI code describes two methods for determining *EI* for a column. However, it has been observed that ETABS uses equation (3) to calculate *EI* for the slender column design (Hossain, 2008; Computers & Structures, 2003).

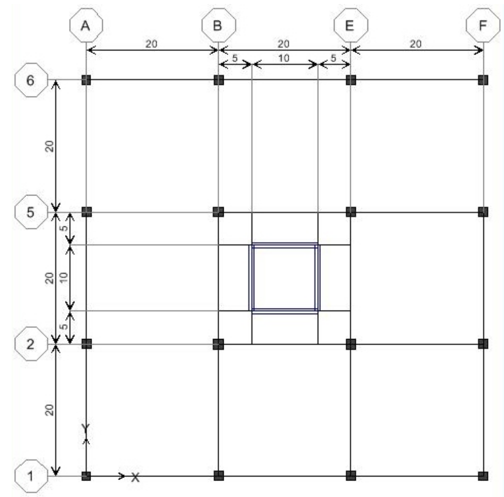
4. Analysis

ETABS Version 8.4.6 is chosen for the study. Figures 1(a), (b), and (c) present the plan view of the ground floor of the models consisting of 15'x15' (4572 mm x 4572 mm), 20'x20' (6096 mm x 6096 mm), and 25'x25' (7620 mm x 7620 mm) slab panels respectively. For simplicity, all the beam-column structure models are square shaped and ten stories in height. Each floor consists of three panels in horizontal and transverse directions and a box-shaped shear wall, which is located in the center of the building. The foundations for the columns and the shear walls are assigned as a fixed support. The height of the basement column is 5' (128 mm). The dimension of the rectangular grade beams is 20"x12" (508 mm x 305 mm). The dimensions of the beams other than grade beams are 18"x12" (257 mm x 305 mm). The clear cover of the grade beam is specified as 2.5" (64 mm) whereas the regular beam's clear cover is taken as 1.5" (38 mm). The ground floor height is increased from 10' to 20' (3048 mm to 6096 mm) with an increment of 2'-6″ (762 mm) for the parametric study. The other story height is 10' (3048 mm), and it is kept unchanged in all structures and analysis. The floor slabs are 6" (216 mm) in thickness which confirms the minimum thickness for the beam-column structure. All the floors and the shear wall were assigned an auto mesh using 4' by 4' (1219 mm x 1219 mm) mesh. The tube shape shear wall is 9" (229 mm) thick. The clear cover of the concrete column is taken as 1.5" (38 mm). The compressive strength of the concrete is assumed as 4.0 ksi (27.6 MPa), the strength of the steel is assumed as 60.0 ksi (413.7 MPa), and modulus of steel is assumed as 29,000.0 ksi (200.0E3 MPa). Concrete properties are the typical values considering Bangladesh perspective.

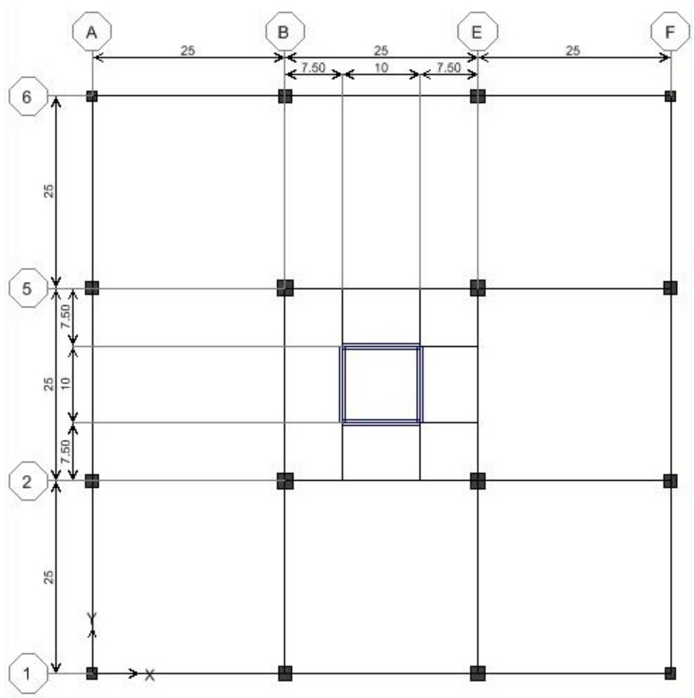
Gravity loads include the live loads on the building and the self-weight of the building. In Bangladesh, only wind and earthquake loads are taken as environmental loads. For this analysis, the imposed live load is considered as 60 psf (2.9E-3 MPa), and the dead load is 60 psf (2.9E-3 MPa). The deal load includes 20 psf (9.6E-4 MPa) of floor finish, 30 psf (1.4E-3 MPa) of boundary wall, and 10 psf (4.8E-4 MPa) of false ceiling. The dead load from the slabs, beams, and columns are calculated automatically by the ETABS using the unit weight of the concrete. The unit weight of the concrete is taken as 150 lb/cubic feet (2403 kg/cubic meter). According to the BNBC, the live load is fixed as 40 psf (1.9E-3 MPa) for a residential building; the live load varies from 75 to 150 psf (3.6E-3 to 7.2E-3 MPa) for other public buildings. For a practical point of view, the live load of the commercial building models is considered to be more than residential buildings but lower than crowded public gathering places.



(a) 15'x15' (4572 mm x 4572 mm) Floor Panel



(b) 20'x20' (6096 mm x 6096 mm) Floor Panel



(c) 25'x25' (7620 mm x 7620 mm) Floor Panel

Figure 1. FEM Models Used for Parametric Study (Dimensions are in Feet)

The wind and earthquake loads are calculated following the BNBC standard for a commercial building in Dhaka. From the BNBC table titled "Basic wind speeds for selected locations in Bangladesh," the Basic Wind Speed (V_b) is chosen as 130.5 mph (210 km/hr). The exposure type is selected as "Exposure A" because the building is sited in the urban area. The windward coefficient is calculated as 1.4 from the BNBC article titled "Overall pressure coefficient (C_p) for rectangular building with flat roofs." As the structure is ranked as the standard occupancy structure, the corresponding value of the Structural Importance Coefficient (C_I) equals to 1.00. The wind load is assumed as acting on the four faces of the building. From the Seismic Zone Map of Bangladesh, the Seismic Zone Coefficient (Z) is taken as 0.15 corresponding to the seismic zone 2. In the basic design information, it has been acknowledged that the building system is a dual system defined as the concrete with concrete IMRF (Intermediate Moment Resisting Frame). So, from the BNBC, the Response Modification Coefficient for the Structural Systems (R) is taken as 9 for both horizontal directions of the building. The Structural Importance Coefficient is taken as 1.00 for the earthquake analysis, and it is same as the wind load analysis. The earthquake force is acting from both horizontal directions of the building. The Structural Period is intended from "Method A" that is described in the BNBC. According to the BNBC, the site soil characteristic is considered as S_3. As it is a commercial building, the BNBC classified it as a standard occupancy structure and ranked the Structural Importance Category as IV. In the model, the following loads are used: dead load (DL), live load (LL), wind load from X direction (WLx), wind load from Y direction (WLy), earthquake load from X direction (EQx), and earthquake load from Y direction (EQy). The Wind and the earthquake loads are applied perpendicular to the building axis. No eccentric load is applied to the model for simplicity. The ETABS version 8.4.6 uses the default cases of load combination of ACI code (318-1999). Total 18 load cases are considered in the models.

4.1 *Parametric Study*

Table.1 shows the floor panel sizes, column positions, column heights, column sizes, and periphery beam sizes, which are used for the parametric study. In this parametric study, total 15 models (3 models for each floor panel size with 5 varying column heights) are generated for the beam-column structure with a tube shape shear wall in the core of the structure. So, among these 15 beam-column structures, a total of 45 ground floor columns are considered for the slender column behavior analysis. The cross-section of columns and beams are selected from a practical point of view.

Table 1. Parameters for parametric study

Floor panel size	*Column position*	*Column height @ ground level*	*Column size*	*Periphery beams size*
15'x15' (4572 mm x 4572 mm)	Corner	10' to 20' @ 2'-6″ increment (3048 mm to 6096 mm @ 762 mm increment)	14"x14" (356 mm x 356 mm)	10"x18" (254 mm x 457 mm)
	Edge	similar	15"x15" (381 mm x 381 mm)	
	Inner	similar	16"x16" (406 mm x 406 mm)	
20'x20' (6096 mm x 6096 mm)	Corner	similar	15"x15" (381 mm x 381 mm)	Similar
	Edge	similar	17"x17" (432 mm x 432 mm)	
	Inner	similar	18"x18" (457 mm x 457 mm)	
25'x25' (7620 mm x 7620 mm)	Corner	similar	16"x16" (406 mm x 406 mm)	Similar
	Edge	similar	20"x20" (508 mm x 508 mm)	
	Inner	similar	24"x24" (610 mm x 610 mm)	

For reinforced concrete column structure, a slab panel size larger than 25' (7620 mm) is not a common scenario. The slab panel greater than 25′ (7620 mm) needs special design requirements. A slab panel smaller than 15' (4572 mm) does not frequently use in commercial buildings because the column spacing is small for ground floor car parking. The beam dimensions were kept the same for all the models because of the consistency in the analysis. After each model is analyzed using ETABS, the moment values of the selected column end and design load for each targeted column are extracted from the software. All the parameters are calculated using a spreadsheet and cross-checked each time with the ETABS design output. Both δ_{ns} and δ_s are determined, and relative graphs are plotted to describe the behavior of the slender column under different conditions.

5. Results and Discussions

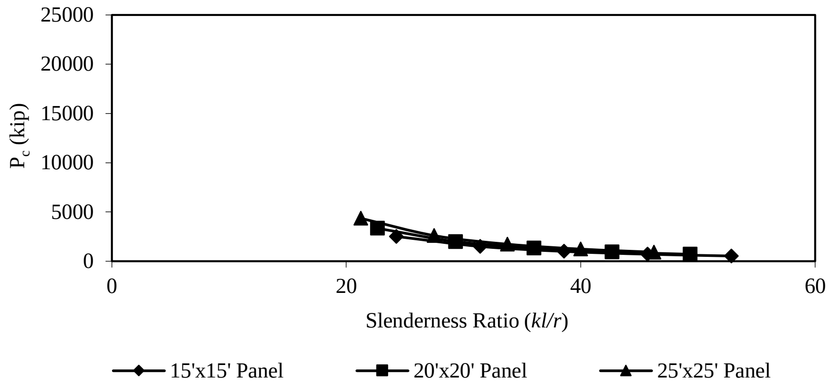
5.1 *Critical Buckling Load*

The limiting value of Slenderness Ratio (*kl/r*) is checked with the recommended ACI code guideline. According to the ACI guideline, a column should be treated as a slender column if the value of the *kl/r* is greater than $34\text{-}12M_1/M_2$. The maximum value of $34\text{-}12M_1/M_2$ that is limited by the ACI code is 40.0. M_1 and M_2 are the column end moments, and M_1 is less than M_2. If the column exhibits double curvature, then the ratio of M_1/M_2 will represent a negative value, and if the column exhibits single curvature then the ratio will have a positive value. The curvature can be recognized by reading the sign of the end moments computed by ETABS. If the end moments represent the same sign (positive or negative), then the column has a single curvature or vice-versa.

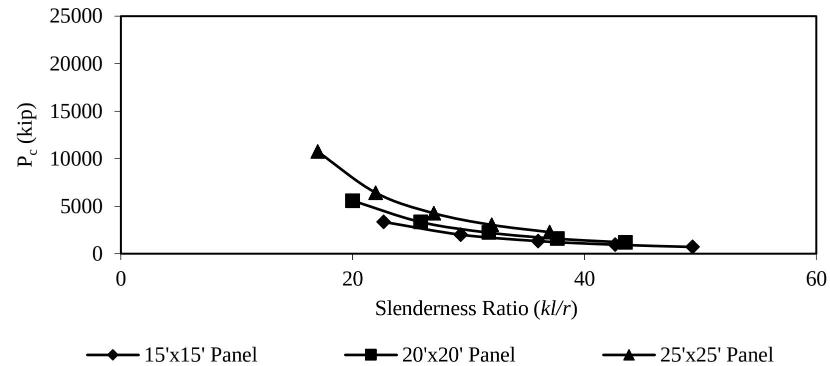
Figure 2 represents Slenderness Ratio (*kl/r*) for three column locations with corresponding Critical Buckling Load (P_c). According to Figure 2(a), the value of *kl/r* is greater or equals to 40 for six corner columns. The P_c value decreased by around 25% when the column height increased by 2.5' (762 mm).

P_c verses *kl/r* for the edge column is illustrated in Figure 2(b). It can be seen that the *kl/r* is greater than 40.0 for three edge columns. The 20' (6096 mm) edge column for the 25'x25' (6096 mm x 6096 mm) panel size showed slenderness effect. The size of the edge column is 17"x17" (432 mm x 432 mm) for the 25'x25' (7620 mm x 7620 mm) size panel. The steel ratio increases by more than 8% in some cases if the column size is reduced from the 17"x17" (432 mm x 432 mm) dimensions. The *kl/r* is greater than 40.0 for one column in the 15'x15' (4572 mm x 4572 mm) size panel.

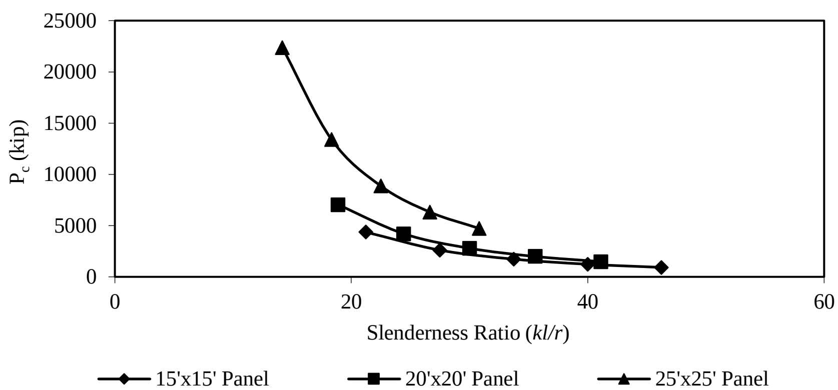
Figure 2(c) shows the *kl/r* for the inner columns. It is seen that three inner columns showed slenderness effect. No column showed slenderness behavior for the 25'x25' (7620 mm x 7620 mm) slab panel. In the FEM model, the steel ratio increased such that the column fails due to over reinforcement if the column dimension is reduced for the 25'x25' (7620 mm x 7620 mm) slab panel. The column section for the 25'x25' (7620 mm x 7620 mm) slab panel is increased compared to the 20'x20' (6096 mm x 6096 mm) slab panel to keep the steel ratio in the tolerable limit. For this reason, the column of this slab panel does not show slenderness compared to the lower sized slab panels. For all three panels, the P_c value decreases by around 25.0% while the column height is increased from 17.5' (5334 mm) to 20.0' (6096 mm). Therefore, it will not be wise to judge slenderness by the only variation in the PC and the *kl/r*. From the above discussions and observations, it can be concluded that corner columns are most sensitive to become slender.



(a) Corner Column



(b) Edge Column

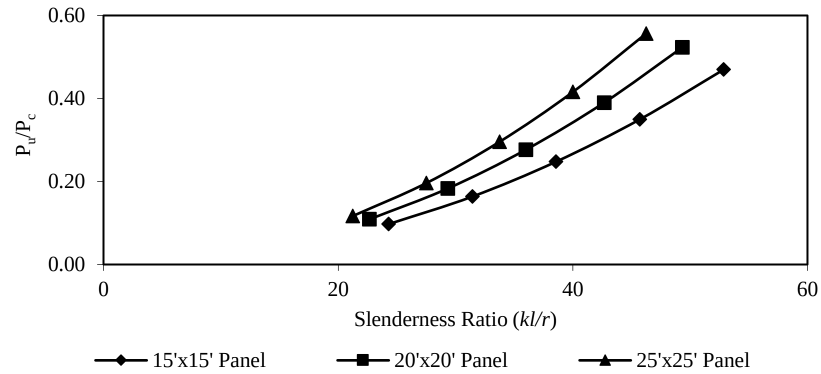


(c) Inner Column

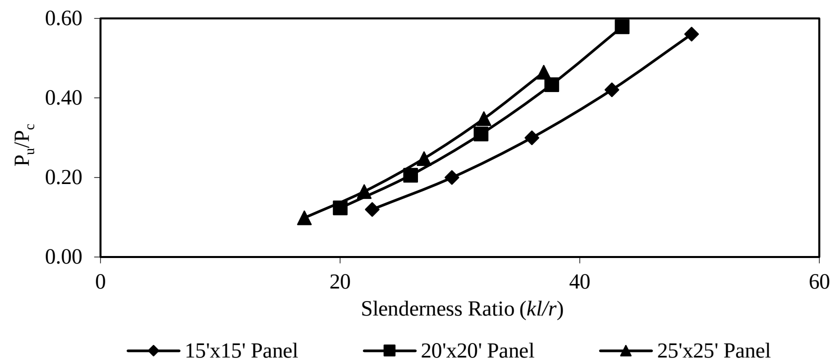
Figure 2. Critical Buckling Load (P_c) for Different Column Positions

5.2 *Ration of Ultimate Load to Critical Buckling Load*

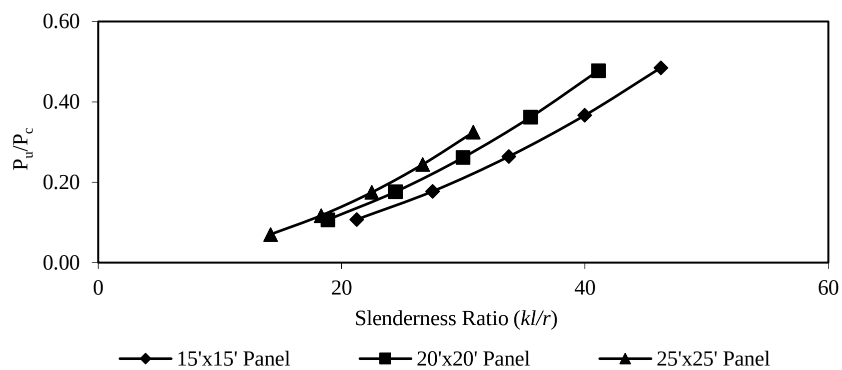
Figure 3 shows three column locations to understand the influence of loads in column buckling. Figure 3(a) represents the ratio of P_u/P_c with *kl/r*. According to Figure 3(a), the slenderness needs to be considered when the design load (P_u) reaches 30% of the critical buckling load (P_c). According to Figures 3(b) and 3(c), the edge and the corner columns showed a slenderness behavior when P_u is increased by more than 40% of P_c. The above observation also proves that the corner columns showed a higher slenderness behavior when compared to the edge and inner columns.



(a) Corner Column



(b) Edge Column

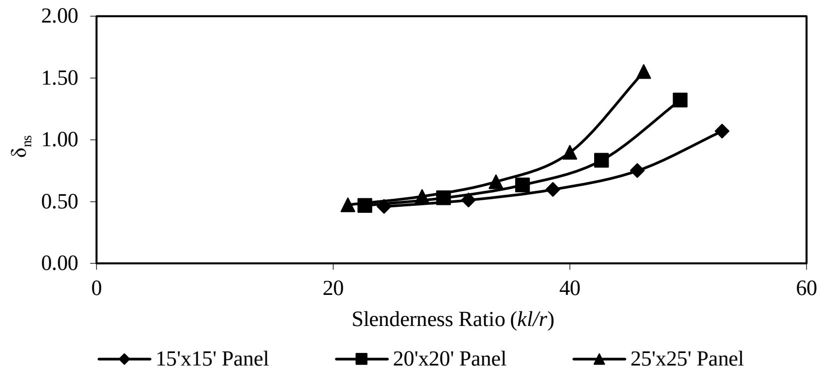


(c) Inner Column

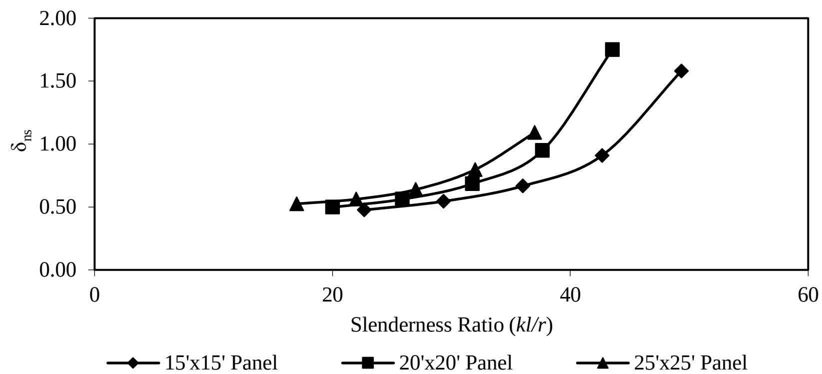
Figure 3. Ratio of Ultimate Column Load (P_u) to Critical Buckling Load (P_c) for Different Column Positions

5.3 *Non-Sway Moment Magnification Factor*

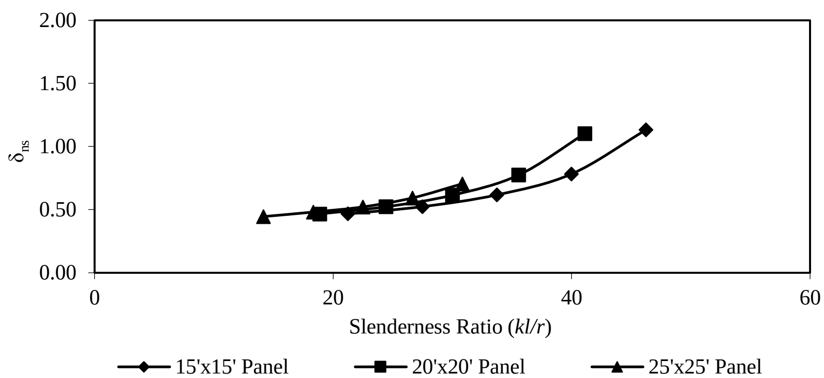
Non-sway Moment Magnification Factor (δ_{ns}) is considered when the slenderness of a column predominantly depends on the gravity load effect. The increase of dead and live load is directly influenced by this type of slenderness behavior. For this study, the columns are checked for the *1.4DL+1.7LL* load combination, since this load combination provides maximum gravity load on the structures. The moment and the load data are collected from the ETABS software calculation. Figure 4 is illustrated to understand the δ_{ns} for the columns at different locations. If δ_{ns} is higher than 1.0, then it is multiplied by the column moments to magnify the value and is then used to calculate the steel ratio required. However, if the value is lower than 1.0, then it is taken as 1.0 considering the safety factor.



(a) Corner Column



(b) Edge Column



(c) Inner Column

Figure 4. Non-Sway Moment Magnification Factors (δ_{ns}) for Different Column Positions

Figure 4(a) shows the δ_{ns} variations for the corner columns. The δ_{ns} value is increased by about 43.0% when column height is increased from 17.5' (5334 mm) to 20.0' (6096 mm) for the 15'x15' (4572 mm x 4572 mm) slab panel. The δ_{ns} value is increased by about 25.0% when column height is increased from 15.0' (4572 mm) to 17.5' (5334 mm) for the 15'x15' (4572 mm x 4572 mm) slab panel. Also the δ_{ns} is increased by about 59.0% for the same increment in column height (17.5' to 20.0') for the 20'x20' (6096 mm x 6096 mm) slab panel and δ_{ns} is increased by about 72.0% for the 25'x25' (7620 mm x 7620 mm) slab panel. So, there is a significant increase in the δ_{ns} value which is observed when the column height increases from 17.5' (5334 mm) to 20.0' (6096 mm).

According to Figure 2(b), the value for *kl/r* is higher than 40.0 for one column in the 15'x15' (4572 mm x 4572 mm) panel but according to Figure 4(b) and the ETABS calculation, the δ_{ns} value is lower than 1.0. If the design engineers only rely on the ETABS analysis and design output, then it is highly likely that it may result in an under reinforced column design. According to the ACI code approach, the 17.5' (5334 mm) column should be considered as a slender column, but ETABS does not consider it as a slender column as δ_{ns} value is lower than 1.0. It is seen that δ_{ns} for a corner column exceed 1.0 only for three columns even though from slenderness point of view six columns should be designed as slender columns. The variation between ETABS and ACI analysis shows that the ACI code design approach is conservative.

Figure 4(b) is illustrated for the δ_{ns} against the *kl/r* for the edge columns of the three panels. The δ_{ns} value is increased by about 74.0% while the column height is increased from 17.5' (5334 mm) to 20.0' (6096 mm) for the 15'x15' (4572 mm x 4572 mm) panel. The δ_{ns} value is increased by about 84.0% while column height is increased from 17.5' (5334 mm) to 20.0' (6096 mm) for the 20'x20' (6096 mm x 6096 mm) slab panel and it is increased by about 36.0% for the 25'x25' (7620 mm x 7620 mm) panel. The δ_{ns} value is not significantly high for the edge column for the 25'x25' (7620 mm x 7620 mm) panel compared to the other two panels. Similar to the corner column, a drastic change is observed when column height is increased from 17.5' (5334 mm) to 20.0' (6096 mm) for the edge column.

Figure 4(c) is illustrated for the δ_{ns} against *kl/r* for the inner columns of the three slab panels. The δ_{ns} value is increased by about 45.0% while the column height is increased from 17.5' (5334 mm) to 20.0' (6096 mm) for the 15'x15' (4572 mm x 4572 mm) slab panel size. The δ_{ns} is increased by about 43.0% for the column while the height is increased from 17.5' (5334 mm) to 20.0' (6096 mm) for the 20'x20' (6096 mm x 6096 mm) slab panel size. The δ_{ns} is increased by about 20.0% for the 25'x25' (7620 mm x 7620 mm) slab panel for the same height increment of the column. Compared to the other two column locations, the value of δ_{ns} is not much higher for the inner column for all the panel size. For this reason, it is concluded that the inner column is not slender susceptible compared to the corner and the edge column.

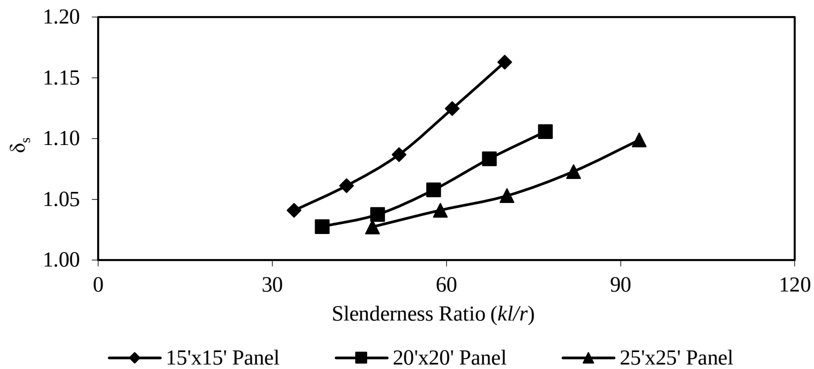
5.4 *Sway Moment Magnification Factor*

There is no straightforward procedure for designing a sway column considering Sway Moment Magnification Factor (δ_s) in ETABS. For sway frame, the magnified moment in ETABS is determined after performing the P-Δ analysis. Even for ETABS, the k value is taken as 1.0 for all cases. Considering k value as 1.0 is conservative for δ_{ns} analysis because k value ranges from 0.5 to 1.0 in a non-sway frame. However, this is not correct for the δ_s analysis because a sway frame k value ranges from 1.0 to infinity. For this study, the critical load combination that is used to calculate δ_s is 0.9DL-1.43EQy since it had the largest sway effect on the frame. Largest sway is observed due to the presence of earthquake load. The Stability Index (Q) is calculated for each frame and is given in Table 2.

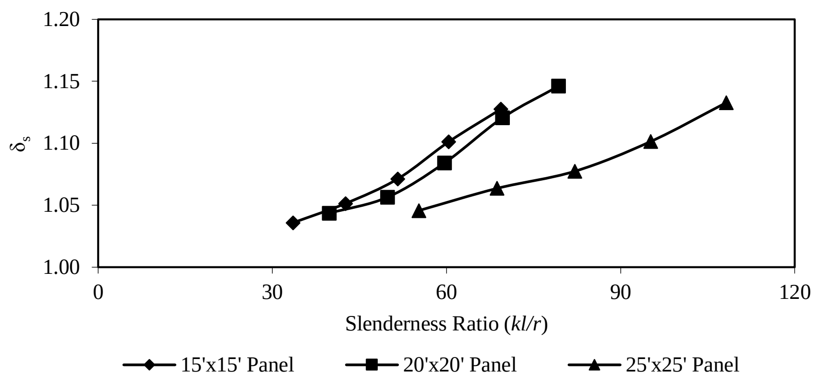
Table 2. Calculated Stability Index (Q) for Different Column Locations

Column Length (ft)	Stability Index (Q)								
	15'x15' Panel			20'x20' Panel			25'x25' Panel		
	Corner Column	Edge Column	Inner Column	Corner Column	Edge Column	Inner Column	Corner Column	Edge Column	Inner Column
10	0.04	0.04	0.03	0.03	0.03	0.04	0.03	0.03	0.04
12.5	0.06	0.06	0.05	0.04	0.04	0.05	0.04	0.04	0.06
15	0.08	0.08	0.07	0.05	0.05	0.08	0.05	0.05	0.07
17.5	0.11	0.11	0.09	0.08	0.08	0.11	0.07	0.07	0.09
20	0.14	0.14	0.11	0.10	0.10	0.13	0.09	0.09	0.12

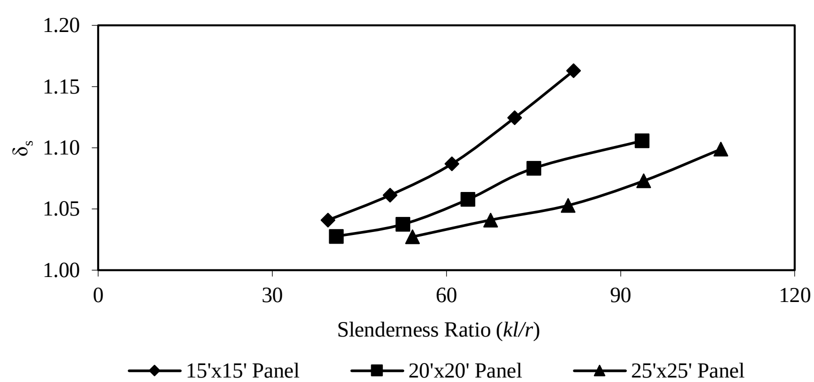
The limiting value of the *kl/r* is checked with the recommended ACI code guidelines. In the case of a sway frame, according to the ACI code, the column should be treated as a slender column if the *kl/r* value is greater than 22.0. It is observed that the value for *kl/r* was higher than 22.0 for all the 15 corner columns. Therefore, the columns which are neglected due to a lower value (i.e., less than 1.0) of δ_{ns} must be considered carefully for the sway moment effect even if the value is low. It has been checked that these 15 columns Stability Index (Q) are lower than 5.0% for the five columns, but according to Figure 5(a), it is seen that even they are in the non-sway frame but have slenderness effect. The *kl/r* value is nearly 100 for the 25'x25' (7620 mm x 7620 mm) slab panel and 20' (6096 mm) column. The 20' (6096 mm) column is very slender and requires a second order computer analysis. Therefore, this concludes that slenderness effect occurs in all cases for the corner column.



(a) Corner Column



(b) Edge Column



(c) Inner Column

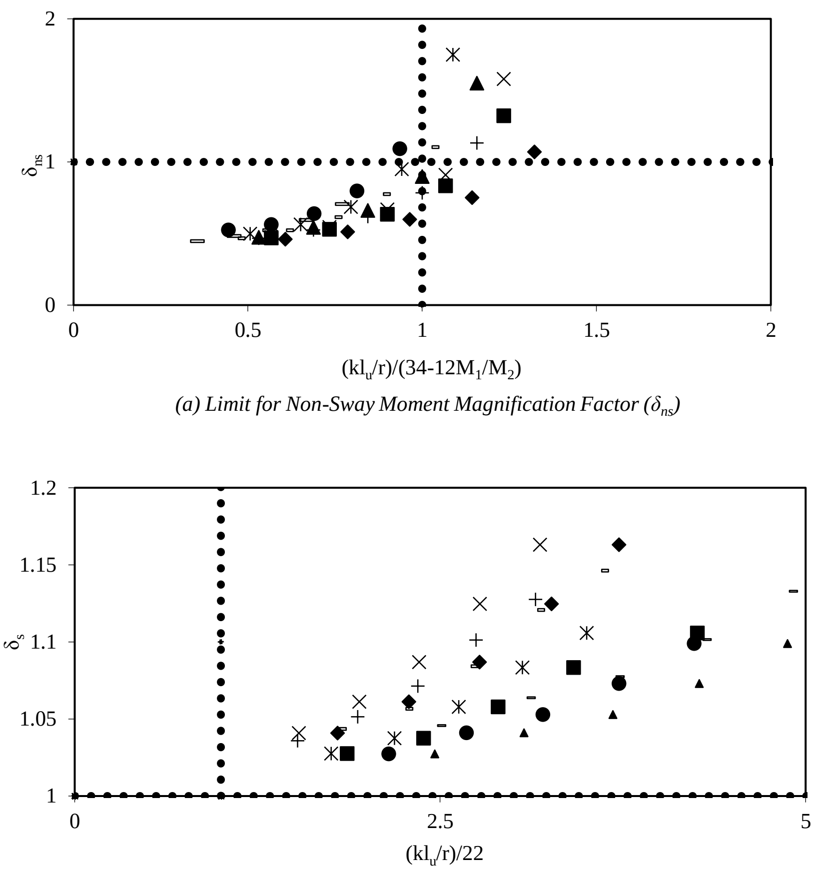
Figure 5. Sway Moment Magnification Factors (δ_s) for Different Column Positions

Figure 5(b) presents δ_s versus *kl/r* for the 15 edge columns for the three panels, and all the columns have a sway effect similar to the corner columns. The *kl/r* is more than 100.0 for the 20' (6096 mm) column for the 25'x25' (7620 mm x 7620 mm) slab panel. So, the column needs second order computer analysis. From comparing Figures 5(a) and (b) it is seen that value of the δ_s varies from 1.0 to 1.18. The 15′x15′ (4572 mm x 4572 mm) size panel showed higher δ_s values compared to the 25′x25′ (7620 mm x 7620 mm) size panel but another way around while compared the *kl/r*. For the 25'x25' (7620 mm x 7620 mm) panel, the *kl/r* values are higher in comparison to the other two panels. Therefore, considering only δ_s or *kl/r* would not be sufficient to compute column slenderness.

In Figure 5(c), the δ_s value is plotted against the *kl/r* for the inner column. Except for the 20′ (6096 mm) column, no *kl/r* value exceeds 100.0, but every column has a slenderness effect. The increment of δ_s in a column for the 15'x15' (4572 mm x 4572 mm) slab panel is steeper than the other two panels. From the preceding description of δ_{ns}, it was concluded that slenderness effect in the inner column is as important as it is for other two locations.

5.5 *ACI Code Limitations*

According to the ACI code, for compression members in the non-sway frames, the effect of slenderness may be neglected when $kl_u/r \leq 34\text{-}12M_1/M_2$, where $34\text{-}12M_1/M_2$ is not taken higher than 40.0. So, if $(kl_u/r)/(34\text{-}12M_1/M_2)$ is less than 1.0, then the column will not show any slenderness effect. Figure 6(a) is plotted for the δ_{ns} against the $(kl_u/r)/(34\text{-}12M_1/M_2)$ for the beam-column frame structures. The dotted lines show the slenderness limits. It is seen that nine columns exceeded the limit bounded by the ACI code. The column with a height greater than 20' (6096 mm) is at risk of slenderness effect. Column heights ranging from 17.5' (5334 mm) to 20' (6096 mm) are substantially considered as slender columns. In Figure 5(a) only one column δ_{ns} value is higher than 1.0, but the $(kl_u/r)/(34\text{-}12M_1/M_2)$ value is less than 1.0 for that column. For three columns the ratio of the $(kl_u/r)/(34\text{-}12M_1/M_2)$ is higher than 1.0, but the δ_{ns} value is less than 1.0. These four cases could be considered as an outlier considering the 45 columns result. So, from Figure 6(a) this can conclude that the guideline for considering δ_{ns} is satisfactory.



(a) Limit for Non-Sway Moment Magnification Factor (δ_{ns})

(b) Limit for Sway Moment Magnification Factor (δ_s)

Figure 6. ACI Code Limitations for the Non-Sway Moment Magnification Factor (δ_{ns}) and Sway Moment Magnification Factors (δ_s)

According to the ACI code, for the compression member in a sway frames, the effect of slenderness may be neglected when the kl_u/r value is less than 22.0. So, if the value of $(kl_u/r)/22$ is lower than 1.0, then the column will not show any slenderness effect. Figure 6(b) is a plot of δ_s versus $(kl_u/r)/22$ for the beam-column frame structures. The dotted lines indicate the limits for slenderness. From Figure 6(b) it is seen that all 45 columns have exceeded the limit bounded by the ACI code. The column with a height higher than the 20' (6096 mm) is at risk of slenderness effect. Column heights ranging from 17.5' (5334 mm) to 20' (6096 mm) are substantially considered as slender columns. So, from Figure 6(b) this concludes that the guideline for considering δ_s is satisfactory, and every column is at risk for the sway effect even it may not be in danger for the non-sway effect. So, the designer should check a column both for the non-sway and the sway effect.

6. Conclusions and Recommendations

The objective of this study is to understand the effects of column design inputs on the slenderness of RC columns for beam-column frame structures. The study is limited considering symmetric beam-column frame structures with only ten stories high. Based on the limited study and from the above discussions, it can be concluded that:

1. Every RC frame in a structure needs to be treated as a sway frame since ground floor columns in the structure have a Sway Moment Magnification Factor (δ_s) of varying magnitudes. So, if a designer uses ETABS, then it is necessary to run the P-Δ analysis option with the specified load combination.
2. Corner columns are more sensitive to becoming slender in comparison to the edge and the inner columns. For a corner column, the bi-axial moment is not balanced, but it is partially balanced for an edge column and mostly balanced for an inner column. A corner column is most likely to become slender due to the presence of the imbalanced moment.
3. The effect of Non-Sway Moment Magnification Factor (δ_{ns}) is higher than Sway Moment Magnification Factor (δ_s). The column that is longer or equal to 17.5' (5334 mm) is vulnerable due to the higher value of δ_{ns}. The variation in δ_{ns} sometimes increases significantly when the column height is increased from 17.5' (5334 mm) to 20.0' (6096 mm). This sudden increment in δ_{ns} value will affect the steel ratio of the column.
4. The structural models are developed considering a commercial building located in a moderate earthquake influence zone and occupancy. In Bangladesh as other two earthquakes influence zones are present, so it is a need to study the effect of slenderness in columns in different earthquake zones and another occupancy, i.e., residential.
5. The study is done considering increment of all columns of a particular floor level. In Bangladesh, it is observed that only columns in one or two frame make double height while other columns keep in nominal height. So, the influence of slenderness in this type of building requires a thorough study.
6. The 15 models developed by ETABS are square shape, and all the columns considered in these models have a square cross-section. Rectangular and irregular shapes of building with different column shapes require further studying. A guideline for the design of slender columns in reference of Bangladesh could be established by summarizing all the ACI Code.

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