Effects of Slenderness in Reinforced Concrete Column Design

by

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It is hereby declared that this project or any of it has not been submitted elsewhere for the award of any degree or diploma.

(Mohammad Imran Hossain)

DEDICATION

This Project is Dedicated to My Late Parents

ACKNOWLEDGEMENTS

First of all, the author wishes to express his gratitude to Almighty Allah for giving him this opportunity and for enabling him to complete the project successfully.

The author wishes to express his heartiest gratitude and profound indebtedness to his supervisor Dr. Tahsin Reza Hossain, Associate Professor, Department of Civil Engineering, Bangladesh University of Engineering & Technology (BUET), Dhaka for his generous help, invaluable suggestions, continuous encouragement and unfailing enthusiasm at every stage of this study. His active interest in this topic and valuable advice was the source of author's inspiration.

The author pays his deepest homage to his late parents, whom he believed to be the cardinal source of inspiration for all his achievements. Last but not the least, the author remains grateful to his family specially Samira Binte Kashem, Ayesha Anwar, and Dr. Rojibul Haque for their contribution to complete this project work.

ABSTRACT

To accommodate over growing population, scarcity of land and amplified price of property in Bangladesh, vertical extension of buildings has now become essential and many high-rise buildings are now being constructed in major cities. In addition, the use of high strength materials with innovative design approach with the help of design software, it is now possible to design smaller cross section of structural members than those designed in earlier time. This reduction in cross-section is very important for column design as slenderness might influence the adequacy of the column. The design load of a slender column might approach buckling load and sway of the frame might be large enough to increase the moments obtained from first order analysis. Consideration of these effects is essential as column is the most important member in a structure. An inadequately designed column without any attention to slenderness might result in catastrophic failure of the whole structure.

To build awareness about slender column, this study has been carried out for beam-column and flat-plate concrete structures; where corner, edge and inner columns have been considered. The methods described in ACI Code-318 (1999) for designing slender column have been reviewed. Slender column design procedures of ETABS and PCACOL software have been validated against manual calculation as per the provisions of ACI Code. A limited parametric study has also been performed to identify the influencing factors of slenderness that should be considered before any design process started.

The findings of the study are noticeable. Considering slenderness in column design is extremely important. Designer should check the slenderness effect in column before starting column design. The ACI Code limits for neglecting sway effect has been checked and found acceptable. P-Delta analysis should be run for every high-rise concrete structure to incorporated possible sway effect. Flat-plate frame structure has more sway effect than beam-column frame structure. Corner and edge columns need more attention due to susceptibility to slenderness. Column with height greater than 15' is substantially vulnerable to sway effect. The steel ratio might increase abruptly when slenderness exceeds a certain limit.

Hopefully this study will increase some awareness to structural designers regarding slender column design. The designers should be able to decide when slenderness effect need to be taken care of and he could properly take the advantage of automatic design feature of software and the design will be more effective and correct.

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LIST OF SYMBOLS

The following symbols are used in this project paper.

ACI	American Concrete Institute
A_c	Area of Concrete in Column
A_g	Gross Area of Column
A_{st}	Area of Steel in Column
BNBC	Bangladesh National Building Code
C	Numerical Coefficient
C_I	Structural Importance Coefficient
C_m	Nonsway Modification Factor
C_p	Overall Pressure Coefficient for Rectangular Buildings with Flat Roofs
DL	Dead Load
E_c	Modulus of Elasticity of Concrete
E_s	Modulus of Elasticity of Steel
EQ_X	Earthquake Load from X-Direction
EQ_Y	Earthquake Load from Y-Direction
f_c	Allowable Strength of Concrete
f'_c	Ultimate Strength of Concrete
f_y	Flexural Strength of Steel
I	Structural Importance Coefficient
l_c	C/C Distance of Column
l_u	Unbraced Length of Column
l_e	Effective Length of Column
IMRF	Intermediate Moment Resisting Frame
k	Effective Length Factor
LL	Live Load
mph	Mile per Hour
M_1	Moment at Top End of Column
M_2	Moment at Bottom End of Column
M2	Minor Axis Moment
M3	Major Axis Moment
M_s	Sway Moment
M_{ns}	Nonsway Moment

n	Ratio of Modulus of Steel to Modulus of Concrete
P_c	Critical Buckling Load
psf	Pound per Square Feet
P_u	Ultimate Axial Load
Q	Stability Index
r	Radius of Gyration
R	Response Modification Coefficient
S_3	One Type of Site coefficient for Seismic Lateral Forces
T	Time Period of Structure
UBC	Universal Building Code
V	Shear Force
V_b	Basic Wind Speed in km/h
W	The Seismic Dead Load of Building
WL_X	Wind Load from X-Direction
WL_Y	Wind Load from Y-Direction
Z	Seismic Zone Coefficient
β_d	Ratio of Maximum Factored Axial Load to Maximum Factored Total Load
δ_{ns}	Nonsway Moment Magnification Factor
δ_s	Sway Moment Magnification Factor
Δ	1st Order Relative Deflection between Top and Bottom of the Story
Δ_1	1st Order Relative Lateral Deflection After 1st Order Computer Analysis
Δ_2	2nd Order Relative Lateral Deflection After 2nd Order Computer Analysis
ϕ	Load Reduction Factor
ρ_g	Steel Ratio

CHAPTER 1

INTRODUCTION

1.1 Background

Columns are defined as members that predominantly carry compressive load. The column for which strength is governed entirely by the strength of the materials and cross-section is called "Short Column". On the other hand, for a "Slender Column", the cross-sectional dimension of the column are small enough comparing to its length and strength is controlled by material, cross-section as well as length of column. A slender column has less strength than a short column of the same sectional area and hence can carry lesser load as compared to short column. Two types of frames are commonly seen in structures, one is "Sway Frame" and the other is "Nonsway Frame". The frame which is braced against sideway is termed as nonsway frame and which is not braced against sideway is known as sway frame. Effects of slenderness need to be addressed differently for sway and nonsway frames. For a nonsway frame, the slenderness effect becomes significant as the design load approaches buckling load and for a sway frame the effect is further amplified due to the horizontal movement of the frame caused by lateral loads. In actual structures, a frame is seldom completely braced or completely unbraced.

In Bangladesh, construction of high-rise structure is now a common trend for commercial and residential buildings. The vertical extension of building is essential for Bangladesh due to deficiency of land, cost of property and to accommodate huge number of growing people in a small area. Most of the buildings are concrete beam-column or flat-plate frame structure. For high-rise structures, it is often seen that the ground floor height increases from conventional height due to architectural or functional purposes which may turn the column slender. With the increasing use of high strength materials and improved methods of dimensioning members, it is now possible to design much smaller cross-section than the past. Together with the use of more innovative structural concepts, a rational and reliable design procedure for slender columns have become increasingly important. ACI Code 318 (1999) provides such design procedures for design of slender column for both in nonsway and sway frames.

Many commercially available structural design software like ETABS (CSI, 2003) are capable of designing column considering the slenderness and P-delta effect as per ACI Code provisions. In design offices, the use of structural analysis and design software is increasing day by day. However, the design procedure of ETABS for column has yet been validated with hand calculation. In the design office, the many designers tend to design columns without considering the slenderness at all even though some prefer to analyze the structure with ETABS or other available software. Also, many of those who prefer to use automatic design features of ETABS or other software not always appreciate the concept behind the design for slenderness. In either case, there is a possibility of mistake in the design of column. So, there is clearly a need to study the ACI design procedure as well as the effects of slenderness in designing reinforced concrete columns thoroughly.

1.2 Objective with Specific Aims and Possible Outcomes

It is getting more and more important to consider the effect of slenderness in column design with the rapidly increasing material properties in high-rise construction. The concept of diagnosing whether any slender column effect exists is extremely important and should be considered before the actual slenderness effect calculation procedures are performed. At the same time, it is essential to understand the ACI Code design philosophy of a slender column. It is also important to study ACI Code design procedure manually and at the same time incorporation of these methods in available design software needs to be studied. The parameters which influence the design of slender column need a careful evaluation.

The proposed study will be carried out with a view to attain the following objectives:

- To verify the column design procedure of ETABS and other available software as per provision of ACI Code 318 (1999) for slender columns.
- To study the effect of slenderness in column design by carrying out a parametric study and to identify the important parameters.

With successful completion of the project, the designer can be alerted as to when and how slenderness of column needs to be considered in the design. The designer can perform a correct column design either manually or using available software.

1.3 Outline of the Methodology

The design procedure of slender column has been described in section 10.0 of ACI Code 318 (1999). The guideline and methods for slender column design procedure provided in ACI Code will be studied. The calculation steps will be carried out manually and also with the help of commercially available design software. ETABS is an available software that is being extensively used by engineers in design offices. The design output of this software will be checked both by manually and by another available software PCACOL (PCA, 1999) which is capable of designing columns.

Considering all possible loads (Wind, Earthquake etc.) one typical model of a high-rise building will be developed in ETABS where the different parameters will be changed. The parameters which are important for designing building consistent with slender column like height of column, dimension of column, span arrangement, loads, material strengths etc. will be varied in normal range.

Results of the parametric study should indicate the important parameters which need to be considered during the design phase and should indicate their significant influence on the column design. Finally with the factors that create slenderness in a column being identified, the designer can become confident when to consider slenderness and can perform an efficient and accurate design of slender column.

1.4 Outline of the Project Paper

The project paper consists of five chapters, which are outlined in this section:

Chapter 1- General background of the research program and summary of possible outcomes, objectives and methodology are stated.

Chapter 2- Review of literature is presented in detail with special emphasis on behavior of column in different condition and how the slender column should be design.

Chapter 3- Software validation presents the manual calculation procedure for slender column design as per ACI Code and compare the results obtained from available software e.g. ETABS and PCACOL.

Chapter 4- Parametric study identifies when slenderness needs to be considered and which parameters are significant in designing slender columns.

Chapter 5- Conclusions are drawn with findings, limitations and recommendations.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

Slender columns are those members whose ultimate load capacities are affected by the slenderness effect that produces additional bending stresses and/or instability of columns. A slender column has less strength than a short column of the same cross-sectional area and hence can carry lesser load as compared to short column (Kumar, 2005). The slenderness increases greatly with increasing length which buckling under gravity load (Halder, 2007). Therefore, evaluation of a slender column involves consideration of the column length in addition to its cross-section. Slender columns, when subjected to eccentric loading, show deflections. These deflections produce additional flexural stresses due to the increase in eccentricity by the amount of transverse deflection (Δ). This is known as the P-Δ effect. The additional moment, P-Δ, is sometimes referred to as the "secondary moment". This secondary moment reduces the axial load capacity of slender column (Nilson *et al.*, 2003). If the total moment including the secondary moment reaches the ultimate capacity of a section, the column fails owing to material failure. The parameters like column buckling effect, elastic shortening and secondary moment due to lateral deflection, which are not so important in designing a short column, must be considered for designing slender column (Hassoun, 2005). The concept of diagnosing whether any slenderness effect exits is extremely important and should be considered before the actual slenderness effect calculation procedure are performed (Ferguson *et al.*, 1987). Second order effects in structure will always occur and always need to be considered (Rathbone, 2002). P-Δ is a non-linear effect that occurs in every structure where elements are subjected to axial load. P-Δ is actually only one of many second-order effects (Dobson, 2003). P-Δ effect should be included in the analysis for the design of a high-rise building if the story drift exceed 1/85 radian during an expected earthquake excitation in seismic region (Mollick, 1997). As structures become even more slender and less resistant to deformation, the need to consider second order and to be more specific P-Δ effects arises. As a result, codes of practices are even more referring engineers to use second order analysis in order that P-Δ effects are accounted for. This is as true in concrete and timber design as it is in the design of steelwork (Dobson and Arnott, 2003).

If the column is very slender, it becomes unstable prior to reaching material failure. In this case instability failure occurs. Slenderness effects are more pronounced in columns of unbraced frames. Frames that do not have adequate bracing against lateral loads show excessive sway which jeopardizes stability of columns. Adequate bracing in frames helps to stabilize secondary deformations at column ends and produces more stable columns. Because of the difference in behavior between a braced and an unbraced frame, columns are treated differently depending on the bracing conditions of their frames.

2.2 Some Basic Concept Related to Column

Before going through ACI design criteria for slender column, some basic definitions related to column needs to be clarified. In the following sections some fundamental concepts and classifications are stated.

2.2.1 Types of column

Column may be divided into two broad categories:

2.2.1.1 Short column

It is the column for which the strength is governed by the strength of the materials and the geometry of the cross-section.

2.2.1.2 Slender column

It is the column for which the strength may be significantly reduced by lateral deflections.

2.2.2 Braced and unbraced frame

Structural frame can be divided into two broad categories:

2.2.2.1 Braced frame

Structural frames whose joints are restrained against lateral displacement by attachment to rigid element or by bracing are called braced or nonsway frames. Floors of building are usually braced by attachment to rigid elements such as structural walls (shear walls), elevator shafts, or reinforced masonry walls.

2.2.2.2 Unbraced frame

If a structural frame is not attached to an effective bracing element but depends on the bending stiffness of its columns and girders to lateral resistance, it is termed an unbraced or sway frame.

2.2.3 Effective length

Columns supported by frictionless pins and rollers do not exist in real structures. The ends of real columns are restrained against rotation by their supports, and moments always develop; moreover, the ends of columns are sometimes free to displace laterally. A column hinged at both ends may buckles into a sine curve shown in Fig. 2.1.

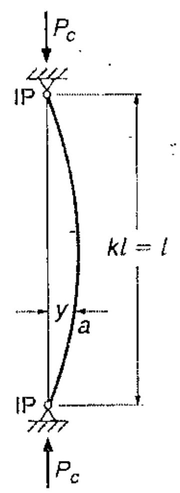


Figure 2.1: Pin ended column (Adopted from Nilson *et al.,* 2003)

If a member is fixed against rotation at both ends, it buckles in the shape of Fig. 2.2 (a), with inflection points (IP) as shown. The portion between the inflection points is in precisely the same situation as the hinge-ended column of Fig. 2.1, and thus the Effective Length (*kl*) of the fixed-fixed column i.e., the distance between inflection points. The point of inflection is equivalent to a pin support for two reasons; (1) the moment is zero, (2) like a frictionless pin, the buckled column below the point of inflection has no bending stiffness to resist rotation. The Effective Length (l_e) is typically expressed as the product of the actual length l times a factor k, called the Effective Length Factor. Effective Lengths of different end conditioned column are shown in Fig. 2.2.

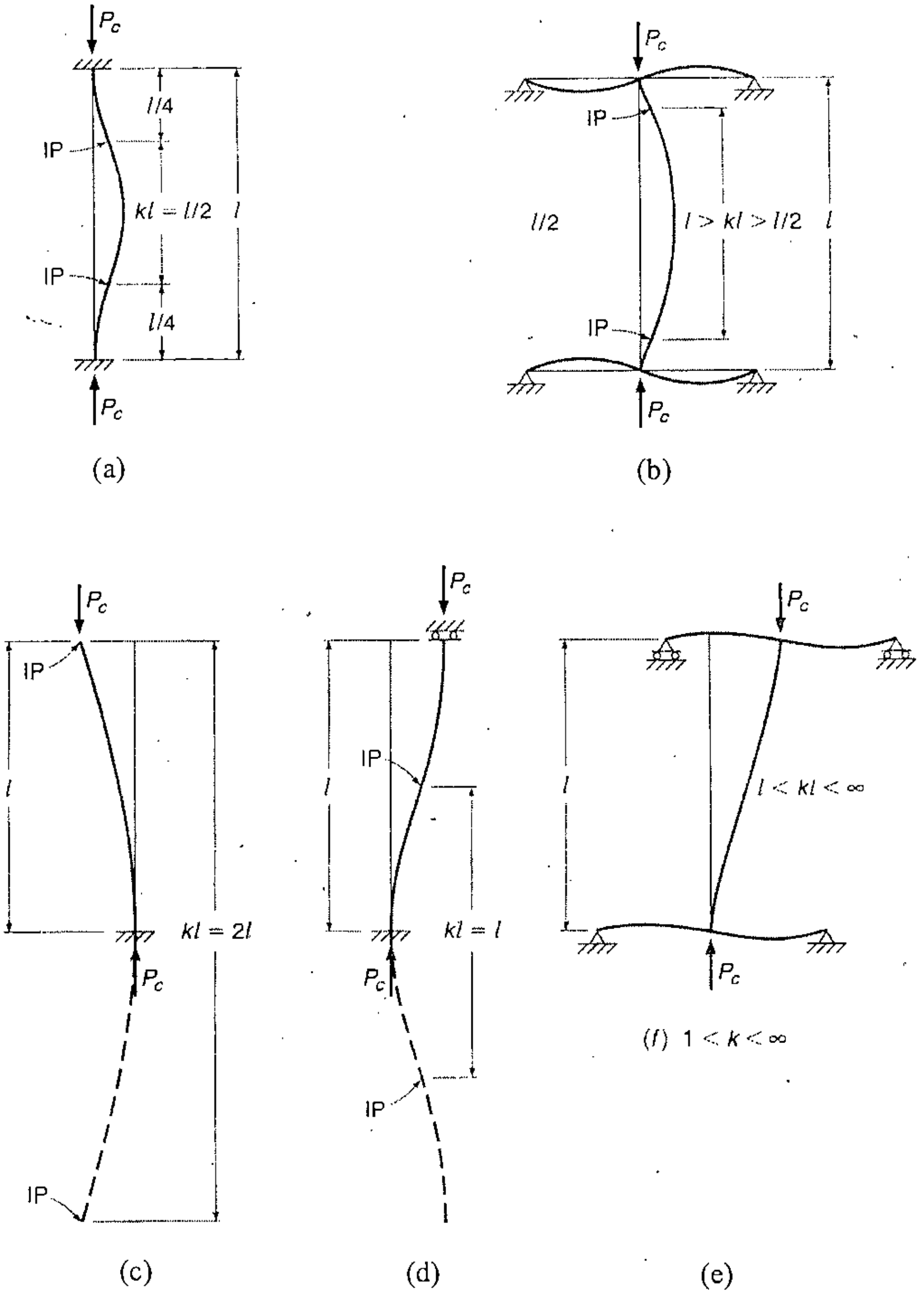


Figure 2.2: Effective length of axially loaded column (Adopted from Nilson *et al.*, 2003)

2.3 Short Column Subjected to Axial Compression

For lower loads for which both materials remain in their elastic range in the short axially loaded compression members, the steel carries relatively small portion of the total load. The Steel Stress (f_s) is equal to n times the concrete stress,

$$f_s = nf_c \tag{2.1}$$

Where, $n = \dfrac{E_s}{E_c}$ is the modular ratio. In this range the axial load P is given by,

$$P = fc\left[A_g + (n-1)A_{st}\right] \tag{2.2}$$

Where, the term in square brackets is the area of the transformed section. Equations (2.1) and (2.2) can be used to find concrete and steel stresses respectively, for given loads, provided both materials remain elastic.

The nominal ultimate strength of an axially loaded column can be found, recognizing the non linear response of both materials, by

$$P_n = 0.85f'_cA_c + A_{st}f_y \tag{2.3}$$

$$P_n = 0.85f'_c(A_g - A_u) + A_{st}f_y \tag{2.4}$$

2.3.1 ACI Code provision for strength reduction

According to ACI Code 10.3.5, the useful design strength of an axially loaded column is to be found based on Eq. (2.4) with the introduction to certain strength reduction factors. The ACI Code factors are lower for columns than for beams, reflecting their greater importance in a structure. A beam failure would normally affect only a local region, whereas a column failure could result in collapse of the entire structure

A further limitation on column strength is imposed by ACI Code 10.3.5 in order to allow for accidental eccentricities of loading not considered in the analysis. This could be included by specifying a certain minimum eccentricity to be used by imposing an upper limit of capacity less than the calculated design strength. This upper limit is taken as 0.85 times the design strength for spirally reinforced columns, 0.80 times the calculated strength for tied columns. Thus, according to ACI Code 10.3.5,

For spirally reinforced columns,

$$\phi P_n = 0.85\phi\left[0.85f'_c(A_g - A_{st}) + f_yA_{st}\right] \tag{2.5}$$

Where, ϕ=0.75

For tied column,

$$\phi P_n = 0.85\phi\left[0.85f'_c\left(A_g - A_{st}\right) + f_y A_{st}\right] \quad \textbf{(2.6)}$$

Where, ϕ=0.70

2.4 Slender Column Subjected to Axial Compression

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

$$P_c = \frac{\pi^2 EI_{\min}}{(kl)^2} \quad \textbf{(2.7)}$$

It is seen that the buckling load decreases rapidly with increasing Slenderness Ratio (kl/r). There is a limiting Slenderness Ratio $(kl/r)_{\lim}$. For values smaller than this, failure occurs by simple crushing, regardless of $(kl/r)_{\lim}$ for values larger than $(kl/r)_{\lim}$ failure occurs by buckling, the buckling load or stress decreasing for greater slenderness.

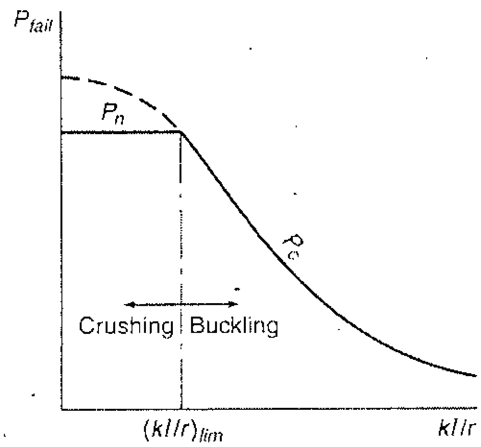


Figure 2.3: Effect of slenderness on strength of axially loaded columns (Adopted from Nilson *et al.*, 2003)

Columns in real structures are rarely either hinged or fixed but have ends partially restrained against rotation by abutting members. It has been seen that compression members free to buckle in a side sway mode are always considerably weaker than when braced against side sway. In columns that are braced against side sway or that are parts of frames braced against side sway, the Effective Length kl, i.e., the distance between inflection points, falls between $l/2$ and l, depending on the degree of end restraint. The

Effective Lengths of columns that are not braced against side sway or that are parts of frames not so braced are always larger than l.

2.5 Slender Column Subjected to Compression with Bending

Most reinforced concrete compression members are also subjected to simultaneous flexure, caused by transverse loads or by end moments owing to continuity. The behavior of members subject to such combined loading also depends greatly on their slenderness. Fig. 2.4(a) shows such a member, axially loaded by P and bent by equal end moments M_e. If no axial load were present, the moment M_a in the member would be constant throughout and equal to the end moments M_e. This is shown in Fig. 2.4(b). In this situation, i.e., in simple bending without axial compression, the member deflects as shown by the dashed curve of Fig. 2.4(a), where y_0 represents the deflection at any point caused by bending only. When P is applied, the moment at any point increases by an amount equal to P times its lever arm. The increased moments cause additional deflections, so that the deflection curve under the simultaneous action of P and M_o is the solid curve of Fig. 2.4(a). At any point, then, the total moment is now

$$M = M_0 + P_y \tag{2.8}$$

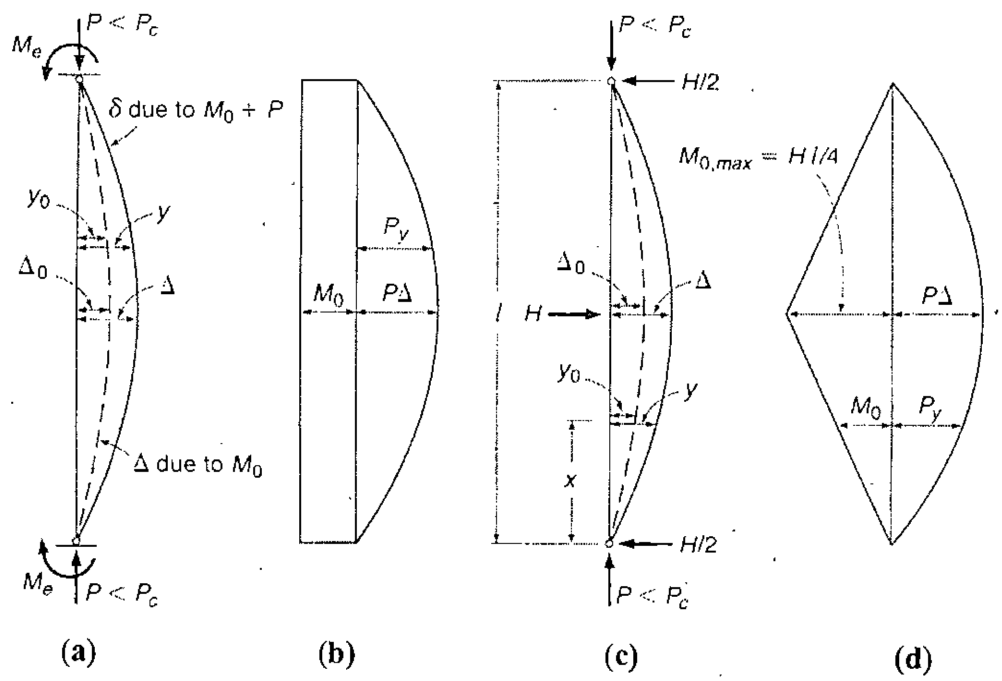


Figure 2.4: Moments with slender members with compression plus bending bent in single curvature (Adopted from Nilson *et al.*, 2003)

The total moment consists of the moment M_o that acts in the presence of P and the additional moment caused by P, equal to P times the deflection. This is one illustration of the so-called P-Δ effect. The deflections y of elastic columns of the type shown in Fig. 2.4 can be calculated from the deflections y_o that is, from the deflections of the corresponding beam without axial load, using the following expression

$$y = y_0 \frac{1}{1 - \frac{P}{P_c}} \tag{2.9}$$

If Δ is the deflection at the point of maximum moment M_{max}, then

$$M_{\max} = M_0 + P\Delta = M_0 + P\Delta \frac{1}{1 - \frac{P}{P_c}} \tag{2.10}$$

The term, $\dfrac{1}{\left(1 - \frac{P}{P_c}\right)}$ is known as the moment magnification factor. The case of equal and opposite end moments shown in Fig. 2.5:

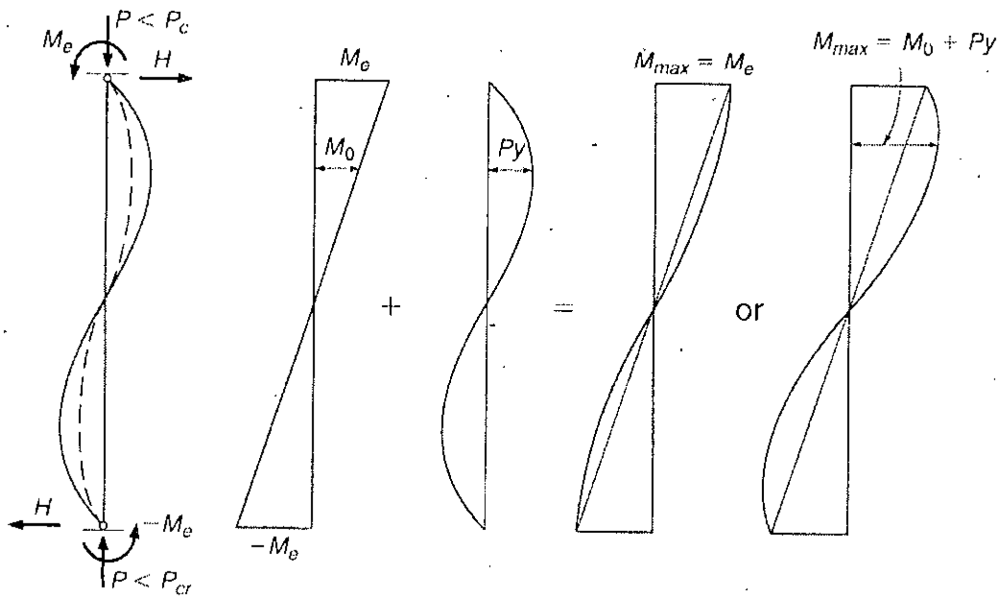


Figure 2.5: Moments in slender members with compression plus bending, bent in double curvature (Adopted from Nilson *et al.*, 2003)

The above equation can be modified into an equation for equal and opposite end moment given next:

$$M_{\max} = M_0 + \frac{C_m}{\left(1 - \frac{P}{P_c}\right)} \tag{2.11}$$

Where,

$$C_m = 0.6 + 0.4\frac{M_1}{M_2} \geq 0.4 \tag{2.12}$$

Here M_1 is the numerically smaller and M_2 the numerically larger of the two end moments. The function M_1/M_2 is defined positive if the end moments produce single curvature and negative if they produce double curvature. It is seen that when $M_1=M_2$ as in Fig. 4(a), C_m=1, so that Eq. (2.11) becomes Eq. (2.10) as it should. It is to be noted that Eq. (2.12) applies only to members braced against side sway. As will become apparent from the discussion that follows, for members not braced against side sway, maximum moment magnification usually occurs, that is, C_m= 1.

2.6 ACI Code Criteria for Nonsway and Sway Frame

As suggested in ACI Commentary 10.11.4, a compression member can be assumed braced if it is located in a story in which the bracing elements (shear walls, etc.) have a stiffness substantial enough to limit lateral deflection to the extent that the column strength is not substantially affected. Such a determination can often be made by inspection. If not, ACI Code 10.11.4 provides two quantitative criteria for determining if columns and stories are treated as non-sway or sway.

2.6.1 Method-1

Columns in a given story may be considered braced or non-sway elements if the column ends moments produced by a second-order structural analysis are not more that 5 percent larger than the moments predicted by a first-order analysis.

2.6.2 Method-2

A story may be considered braced if Stability Index is less than 0.05. Stability Index,

$$Q = \frac{\sum P_u \Delta_0}{\sum V_u l_c} \tag{2.13}$$

Where,

$\sum P_u$ = Total factored vertical load in the story. ($\sum P_u$ should correspond to the lateral loading case for which $\sum P_u$ is greatest)

V_u = Total factored story shear in the story

l_c = The length of the column measured center-to-center of the joints in the frame

Δ_0 = The first-order relative deflection between the top and the bottom of the story due to V_u

2.7 ACI Code Criteria for Neglecting Slenderness Effect

ACI Code 10.12.2 and 10.13.2 provide the limits for neglecting the effects of slenderness. Separate limits are applied to braced and unbraced frames. The code provisions are described below.

2.7.1 For nonsway frame

For compression members in non-sway frames, the effects of slenderness may be neglected when,

$kl_u/r \leq 34 - 12M_1/M_2$. Where, $34 - 12M_1/M_2 \leq 40$.

2.7.2 For sway frame

For compression members not braced against side sway, the effects of slenderness may be neglected when,

$kl_u/r \leq 22$.

Where,

k = Effective length factor;

l_u = The unsupported length taken as the clear distance between floor slabs, beams, or other members providing lateral support.

r = Radius of gyration of cross-section of column associated with axis about which bending is occur.

M_1 = Value of smaller end moment on the column calculated from a conventional first-order elastic analysis. Positive if the member is bent in single curvature and negative if bent in double curvature.

M_2 = Value of the larger factored end moment on the compression number, always positive.

The radius of gyration r for rectangular columns may be taken as $0.30h$, where h is the overall cross-sectional dimension in the direction in which stability is being considered. For circular members, it may be taken as 0.25 times the diameter. For other shapes, r may be computed for the gross concrete section.

2.8 Determination of Effective Length Factor for Columns of Rigid Frame

An accurate determination of the effective length factor k is essential. Fro frames, it is seen that this degree of rotational restraint depends on whether the stiffness of the beams framing into the column at top and bottom as shown in Fig. 2.6. The methods for determining k are described below.

2.8.1 Method-1

In a reinforced concrete frame, columns are rigidly attached to girders and adjacent columns. The Effective Length of a particular column between stories will depend on how the frame is braced and on the bending stiffness of the girders. We know that 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. This is illustrated in Fig. 2.6.

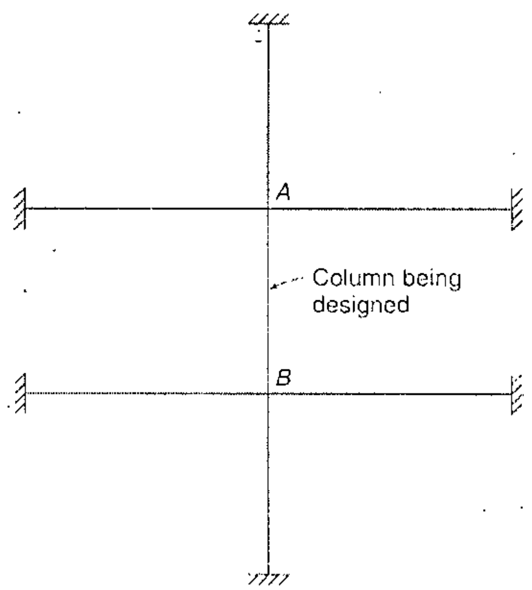


Figure 2.6: Selection of rigid frame including column to be designed

The Jackson and Moreland Alignment Chart (Fig. 2.7) can be used to evaluate the effective length factor. The charts are entered with values of ψ for the joints at each end of a column. Where ψ is defined as,

$$\psi = \frac{\sum\left(\frac{E_c I_c}{l_c}\right)}{\sum\left(\frac{E_g I_g}{l_g}\right)} \tag{2.14}$$

Where,

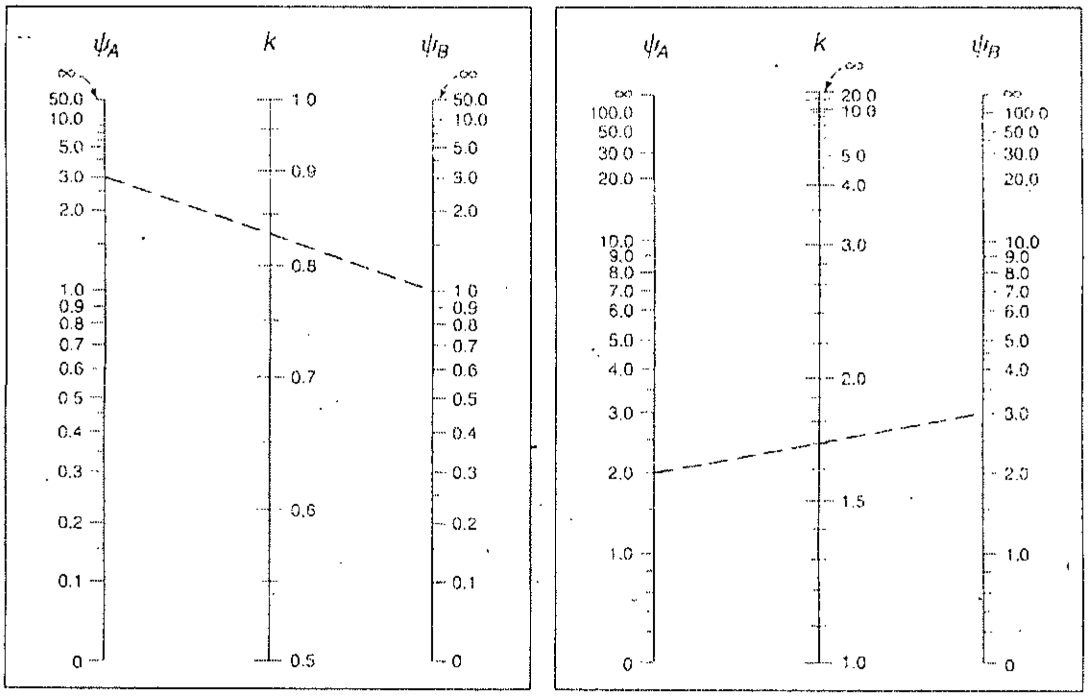
I_c = Effective moment of inertia of column = $0.7I_{gross}$

I_g = Effective moment of inertia of girder = $0.35I_{gross}$

l_c = Length of column, center to center joints

l_g = Length of girder, center to center joints

E_c, E_s = Modulus of elasticity of columns and girders, respectively



(*a*) Nonsway frames (*b*) Sway frames

Figure 2.7: Jackson and Moreland Alignment Chart for Effective Length Factor (k) (Adopted from Nilson *et al.*, 2003)

2.8.2 Method-2

An alternate to using the alignment chart to establish k, ACI Commentary R10.12.1 provides the simplified equation given below.

2.8.2.1 For nonsway frame

For compression members in a non-sway frame, an upper bound to the Effective Length Factor may be taken as the smaller of the following two expressions:

$$k = 0.7 + 0.05(\psi_A + \psi_B) \qquad \textbf{(2.15)}$$

$$k = 0.85 + 0.05\psi_{\min} \qquad \textbf{(2.16)}$$

Where ψ_A and ψ_B are the value of ψ at two ends of the column and ψ_{min} is the smaller of the values. Ψ is determined by Eq. (2.14).

2.8.2.2 For sway frame

For columns in unbraced frames restrained at both ends, first determine ψ_m, is the average of ψ_A and ψ_B.

For ψ_m<2

$$k = 0.05 \times (20 - \psi_m) \times \sqrt{(1 + \psi_m)} \qquad \textbf{(2.17)}$$

For ψ_m>2

$$k = 0.9 \times \sqrt{(1 + \psi_m)} \qquad \textbf{(2.18)}$$

2.8.2.3 Special end condition

- If a column is supported by a frictionless hinge, it can be treated like a joint into which girders of zero stiffness. For this case, $\Psi=\infty$ (infinity).
- A fixed end support may be considered equivalent to a joint into which a girder of infinite stiffness. In this case $\Psi=0$.

2.9 Determination of β_d Factor

For load combinations which include lateral loads, the factor β_d, shall be calculated as follows:

$$\beta_d = \frac{\textit{Maximum factored sustained shear within a story}}{\textit{Total factored shear in the story}} \qquad \textbf{(2.19)}$$

For load combinations which include gravity loads only, the factor β_d, shall be calculated as follows:

$$\beta_d = \frac{\textit{Factored dead load within a story}}{\textit{Total factored load in the story}} \qquad \textbf{(2.20)}$$

2.10 Calculation of *EI*

The formula(s) for calculating *EI* are specified by ACI Code. The appropriate formula for calculating *EI* depends on the context in which the value of *EI* is to be used. Two cases apply.

2.10.1 Case-1: Calculation of *EI* to determine P_c

Under this case *EI* may be calculated as,

$$EI = \frac{0.2E_cI_g + E_sI_{se}}{1+\beta_d} \tag{2.21}$$

or,

$$EI = \frac{0.4E_cI_g}{1+\beta_d} \tag{2.22}$$

Here, β_d, is as the Eq. (2.20).

2.10.2 Case-2: Calculation of EI to determine Ψ value

For this case the value of *EI* is calculated per ACI Code 10.11.1.

$$EI_{beams} = \frac{0.35E_cI_g}{1+\beta_d} \tag{2.23}$$

$$EI_{columns} = \frac{0.70E_cI_g}{1+\beta_d} \tag{2.24}$$

Here β_d is as the Eq. (2.19). Note that for calculating Ψ the value of β_d may be zero.

2.11 Methods of Slender Column Design

Slender column design for sway frames may be accomplished by one of three methods (Yaw, 2005). The three methods are:

- Second order computer analysis
- Direct *P*-Δ analysis
- ACI sway moment magnifier method.

Each of these methods is describe herein.

2.11.1 Second order computer analysis-required when $\frac{kl_u}{r} > 100$

The steps for slender column design by 2nd order computer analysis are described in following section.

2.11.1.1 Lateral load analysis

In this method the analysis of the frame being designed is performed using a second order computer analysis. A second order computer analysis takes into account P-Δ effects automatically. To do this analysis, each necessary load combination which includes lateral loads must be used. From the computer analysis the load combination which causes the worst end moments will be used for design. Since the computer is doing a second order analysis the moments calculated will already be magnified. Hence, the value of P_u, M_{u1} and M_{u2} may be read directly from the computer output. This method is the most accurate method of analysis.

2.11.1.2 Special consideration

None necessary for this method

2.11.1.3 Gravity load stability check

To ensure that gravity loads do not cause an unstable situation ACI Code requires a special check for gravity load stability for the frame being designed. Note that under this stability check, since we are using gravity loads, β_d will be calculated as per Eq. (2.20). This special check is carried out as outlined below.

1) Use gravity load combination $1.4DL+1.7LL$.
2) With the gravity load combination use any reasonable lateral load. A lateral load of 0.005 times the total story gravity load may be used.
3) Using load 1 and 2 together run a first order analysis and determine the first order relative deflection for the story being designed. For example, using any column in the story, the story deflection will be the top column joint lateral deflection minus the bottom column joint lateral deflection. This is the deflection of just the story under consideration. The deflection is called Δ_I.

4) Using load 1 and 2 together, run a second order computer analysis and determine the second order relative lateral deflection for story being designed. The deflection called Δ_2.
5) Calculate the ratio Δ_2/Δ_1. If this ratio exceeds 2.5 the design must be changed. For example, the column cross-section size used in the story will need to be increased or the story needs to be braced.

2.11.2 Direct P-Δ analysis

The procedure for slender column design following direct P-Delta analysis is described in subsequent section.

2.11.2.1 Lateral load analysis

To determine the magnified moments the following procedure may be used. During this process β_d is generally taken as zero.

1) For the worst load combination that includes lateral loads, a first order frame analysis must be done in such a way to obtain nonsway and sway moments. Such moments need to be taken the computer analysis at the top and bottom joints of the column being designed. The moments will be factored and called them M_{1ns} and M_{1s} for the bottom joint end moments and M_{2ns} and M_{2s} for the top joint end moments of the column being designed.
2) The Stability Index, Q, must be calculated per Eq. (2.13) and is based on the worst load combination determined during the process of step 1.
3) Calculate δ_s by using the following formula:

$$\delta_s = \frac{1}{1-Q} \leq 1.5 \qquad \textbf{(2.25)}$$

   If δ_s is greater then 1.5 analysis methods 1 and 3 must be used. Alternatively, the column being designed would need to be redesigned with a bigger cross-section. Note that this limitation on δ_S is exceeded when Q becomes greater than 1/3. δ_s is never to be used less than 1.0.
4) The magnified moment calculated as,

$$M_1 = M_{1ns} + \delta_s M_{1s} \qquad \textbf{(2.26)}$$

$$M_2 = M_{2ns} + \delta_s M_{2s} \qquad \textbf{(2.27)}$$

The larger absolute moment M_1 and M_2, shall be used for design of the column under consideration. This larger moment is usually called M_2 and it is a factored moment.

2.11.2.2 Special consideration

The maximum moment may occur between the ends of the column being designed. This is ordinarily not the case for columns in sway frames. For sway frames the maximum column moment usually occurs at one end of the column. However, under certain conditions this may not be the case. However ACI Code requires that we check for such a condition. Such a condition occurs when,

$$\frac{l_u}{r} > \frac{35}{\sqrt{\frac{P_u}{f'_c A_g}}} \tag{2.28}$$

If Eq. (2.28) is true then the column must be designed as a nonsway column based on,

$$M_c = \delta_{ns} M_2 \tag{2.29}$$

$$\delta_{ns} = \frac{C_m}{\left(1 - \frac{P_u}{0.75 P_c}\right)} \tag{2.30}$$

$$C_m = 0.6 + 0.4 \frac{M_1}{M_2} \geq 0.40 \tag{2.31}$$

With M_1 and M_2 calculated by Eqs. (2.26) and (2.27) and δ_s per Eq. (2.25). The factor β_d (probably zero) is defined per the load combination under consideration and k is defined for a column in a nonsway frame and will likely need to be determined by using calculated ψ values. Once M_c is calculated the column is designed for P_u and the factor moment M_c.

2.11.2.3 Gravity load stability check

This check is unnecessary when $\delta_s M_s$ has been computed by using the direct P-Δ method analysis. Hence, since our present method of analysis is the direct P-Δ method, this check do not required.

2.11.3 ACI sway moment magnifier method

The process of determining moment magnification factor by ACI sway moment magnification method is described in next section.

2.11.3.1 Lateral load analysis

To determine the magnified moments the following procedure may be used. During the process β_d is generally taken as zero.

1) For the worst load combination that includes lateral loads a first order frame analysis must be done in such a way to obtain nonsway and sway moments. Such moments need to be taken from the computer analysis at the top and bottom joints of the column being designed. The moments will be factored and we call them M_{1ns} and M_{1s} for the bottom joint end moments and M_{2ns} and M_{2s} for the top joint end moments of the column being designed. Be careful to maintain the appropriate signs on these moments when you extract them from the computer analysis.
2) Calculate $\sum P_u$ for the story of the column being designed.
3) Calculate P_c for each column in the story of the column being designed. Then calculate $\sum P_c$ for the given story.
4) Calculate δ_s,

$$\delta_s = \frac{1}{\left(1 - \dfrac{\sum P_u}{0.75\sum P_c}\right)} \tag{2.32}$$

5) Calculate the magnified moments per Eqs. (2.26) and (2.27), but in this case use δ_s as calculated by Eq. (2.32). The absolute moment, M_1 or M_2 shall be used for design of the column under consideration. This larger moment is usually called M_2 and it is a factored moment.

2.11.3.2 Special consideration

In this step the same procedure has to be followed that is stated in preceding P-Δ analysis step's special consideration except for the following,

- Use δ_s as determined in this section
- Use M_1 and M_2 as determined in this section

2.11.3.3 Gravity load stability check

To ensure that gravity loads do not cause an unstable situation, ACI Code requires special load stability for the frame being designed. Note that under this stability check, since we are using gravity loads, β_d will be calculated. The value of β_d will be calculated as per Eq. (2.20) given above. This special check is carried out as outlined below.

1) Use gravity load combination $1.4DL+1.7LL$ to calculate $\sum P_u$ for the story of the column being designed. Also, calculate $\sum P_c$ for the columns in the story of the column being designed. The value of β_d used shall correspond to the gravity load combination being used.
2) From $\sum P_u$ and $\sum P_c$ obtained 1 above, calculate δ_s as determined in Eq. (2.32). The value of δ_s so determined shall not exceed 2.5. If is does, the story of the frame under consideration is unstable and need to be redesigned. This likely will require that bigger column cross-sections be used.

2.12 Conclusion

This chapter described the conception of slender column analysis and design procedure. The design principle that is set by ACI Code has been reviewed. The different methods to conduct a detail analysis and design of slender column are described step by step. With the base on this chapter, manual calculation of slender column design has been conducted. The calculation procedure of ETABS and PCACOL has been scrutinized on these guidelines of ACI Code.

CHAPTER 3

SOFTWARE VALIDATION

3.1 Introduction

Column is the most important part of a structure and it is essential to design the column correctly considering all Code provisions. Nowadays, many user-friendly software are available which can take into account these design provisions. In Bangladesh, use of these design software is increasing rapidly even though some designers still prefer to go for manual calculation. In Chapter 2, ACI Code 318 (1999) guidelines for slender column design have been discussed. As for column design, the ACI methods of including slenderness effect for sway and nonsway frames are not straight forward and as a result many designers ignore this effect in manual calculation. On the other hand, those who opt to design using software often fail to appreciate the design philosophy of slender column. In either case, it might result in an inadequate column design.

ETABS (CSI, 2003) is a sophisticated, yet easy to use, special purpose analysis and design program developed specifically for building systems. ETABS Version 8.4.6 features an intuitive and powerful graphical interface coupled with unmatched modeling, analytical, and design procedures, all integrated using a common database. Although quick and easy for simple structures, ETABS can also handle the largest and most complex building models, including a wide range of nonlinear behaviors, making it popular to structural engineers in the building industry. PCACOL (PCA, 1999) is a column design software developed by "Portland Cement Association". This software can analyze and design both sway and nonsway column. The design procedure is almost same like ETABS with some exceptions.

Both ETABS and PCACOL are popular among designers in Bangladesh. However, there is no evidence of verification of these software results. Particularly the design verification of slender column has not been done yet. It is an objective of the current work is to check whether the design procedure of these software are correct as per the provisions of ACI Code. In this Chapter, manual calculations have been carried out to validate the slender column design procedure of ETABS and PCACOL. Current work also aims at understanding the procedure steps of the software so that the designer can easily find out which steps are important and need more attention at the time of design. Although both ETABS and PCACOL use ACI design method, there are some exceptions in their design procedures in using equation for calculating some parameters. Those exceptions have been identified and the results due to these variations are pointed out in this Chapter.

3.2 Design Information

The design information for developing a structural model has been divided in three categories like basic information, loading information and load combination. All the information is necessary for developing the structural model by ETABS. All the categories are briefly described in the following articles.

3.2.1 Basic information

For software validation study, the building model that has been prepared by ETABS software, named as "Basic Model". The basic model is ten stories high. This is a square shape building. There are three panels in each direction; each is 20'-0" apart. The structure has been modeled as beam-column frame type with shear walls in the middle position of building. So, according to BNBC (BNBC, 1994) the structure can be classified as dual system. More specifically concrete shear wall with concrete IMRF. The shear walls are acting as a bracing of the building frame and it reduce the sway of building frame. The thickness of shear wall is 9". The position of stair is around outside of shear walls and connects the shear walls with slabs. The foundation is specified as fixed type and the foundation is shallow foundation. The height of basement column is 5'-0". The ground floor height is 17'-6" and other floor height is 12'-0". The beams other than grade beams are 18"x12" depth. The rectangular grade beams are 20"x12" deep. The grade beam connecting column to column and grade beam to shear wall to transfer the ground shear force effectively. The clear cover of grade beam is specified as 2.5" where as the regular beam clear cover is 1.5". The slab is monolithic with beams heaving 6" in depth. All the columns are square in shape. The corner columns are 14"x14", edge columns are 16"x16" and inner columns are 18"x18" in dimension. The clear cover of concrete column is 1.5". The compressive strength of concrete is 4.0 ksi, strength of steel is 60.0 ksi and modulus of steel is 29,000.0 ksi.

According to BNBC the site soil characteristics is considered as S_3. The building is situated in Dhaka city and it is a commercial building. On this basis the design parameters have been chosen such as those are suitable for commercial building standard in Dhaka. As it is a commercial building, BNBC classify it as a standard occupancy structure and ranked the structural importance category as IV.

3.2.2 Loading information

Normally in Dhaka city a building experiences two types of loads. One is gravity load, other is environmental load. Gravity loads come from self weight of building and impose vertical loads on building. Because of self weight, gravity load is always present in building even it does not have any imposed load. In Bangladesh only wind load and earthquake load are taken as environmental load. The combine presence of wind load and earthquake load is rarely possible. A minimum amount of wind load is always present on structure which turns peak value at the time of storm. The presence of earthquake load is occasional but most devastating. Now people are more concerned about earthquake vulnerability and take precaution in design. Either wind or earthquake load could be critical for building. It sometimes depends on structural shape and size and the zone where the structure is located.

The dead load calculated from slab is 75 psf. Dead load imposed through floor finish is 20 psf and dead load coming from partition and boundary walls has been assumed as much as 30 psf. As the building has been considered as commercial building the loads from false ceiling and other service facilities are consider as 10 psf. So, total dead load imposed on the structure is 60 psf. Some designer may consider the partition wall load as live load for commercial building because the partition wall can rearranged depends on occupancy. In this model the partition wall load has been considered as dead load. The live load coming to building is 60 psf. According to BNBC, for residential building the live load is fixed as 40 psf. For other public building the live load varies from 75 to 150 psf. For practical point of view the live load is consider more than residential but lower that intense public gathering places.

It was stated before that the building is located in Dhaka city. From BNBC table titled "Basic wind speeds for selected locations in Bangladesh" the Basic Wind Speed (V_b)

has been selected as 210 km/hr which is equivalents to 130.5 mph. The exposure type is selected as "Exposure A" because the building is sited in urban area. The windward coefficient has been calculated as 1.4 from BNBC article titled "Overall pressure coefficient (C_p) for rectangular building with flat roofs". As the structure has been ranked as standard occupancy structure the corresponding value of Structural Importance Coefficient (C_I) equals to 1.0. The wind force applied following the projected area method of BNBC.

From Seismic Zone Map of Bangladesh, the Seismic Zone Coefficient (Z) is taken as 0.15 corresponding seismic zone 2. In the basic design information it has been acknowledged that the building system is dual system defined as concrete shear wall with concrete IMRF. So, from BNBC the Response Modification Coefficient for Structural Systems (R) has been taken as 9.0 for both direction of building. Structural Importance Coefficient has been taken as 1.0 for earthquake analysis which is same as wind load analysis. The earthquake force is acting from either direction of building which has been considered for more practical consideration. The Structural Period is calculated from "Method A" that is described in BNBC.

3.2.3 Load Cases

In the model static load cases like dead load (*DL*), live load (*LL*), wind load from X direction (*WLx*), wind load from Y direction (*WLy*), earthquake load from X direction (*EQx*), earthquake load from Y direction (*EQy*) have used. For applying wind load and earthquake load the automation feature of software has been used. Wind and earthquake loads are applied perpendicular to the building axis. No eccentric loads have been applied to the model for simplicity. ETABS version 8.4.6 uses the default cases of load combination of ACI Code. Here are the 18 load cases that have been used in the model.

Table 3.1: Load combination table

Load combination no.	Combination type
1	1.4 DL
2	1.4DL+1.7LL
3	0.75(1.4DL+1.7LL+1.7WLx)
4	0.75(1.4DL+1.7LL-1.7WLx)
5	0.75(1.4DL+1.7LL+1.7WLy)
6	0.75(1.4DL+1.7LL-1.7WLy)
7	0.9DL+1.3WLx
8	0.9DL-1.3WLx
9	0.9DL+1.3WLy
10	0.9DL-1.3WLy
11	0.75(1.4DL+1.7LL+1.87EQx)
12	0.75(1.4DL+1.7LL-1.87EQx)
13	0.75(1.4DL+1.7LL+1.87EQy)
14	0.75(1.4DL+1.7LL-1.87EQy)
15	0.9DL+1.43EQx
16	0.9DL-1.43EQx
17	0.9DL+1.43EQy
18	0.9DL-1.43EQy

3.3 Model Generation

The model generation starts with clicking new model following file menu of the tool bar of ETABS. ETABS has some default building plan grid system. This predefined structural object is very useful for quick generation of model. The custom grid spacing tab has been chosen under grid dimension plan to customize the grid data. There are six grids in each direction, four of them are primary grids and two of them are secondary grid. The primary grids are 20'-0" a part which produce 60'-0" frame size. The grid spacing is same in two directions. The data those are input in "define grid data" table format has been shown in Fig. 1 of Appendix-A.1. After defining the grid data custom storey data are defined by pressing "edit storey data". Only two stories have been defined at beginning of model generation. The first storey (storey-1) is ground floor with grade beams only and the second storey (storey 1-1) is the floor with beams. The ground floor storey is 17'-6" high and the other storey is 12'-0" high. Only ground floor height is extended to observe the slenderness behavior. The data those have been input in "storey data" table format are shown in Fig. 2 of Appendix-A.1. After generation of the model the layout plan has been developed.

The next step is to define the beam and column dimension as well as define member parameters. The columns, beams and shear walls are drawn along the layout plan. After drawn the beams, columns and shear walls the plan view of storey 1-1 is looked like Fig. 3.1. The plan view of grade beam layout has been shown in Fig. 3 of Appendix-A.1. In grade beam plan this is shown that the shear walls are connected with main frame of structure by grade beam.

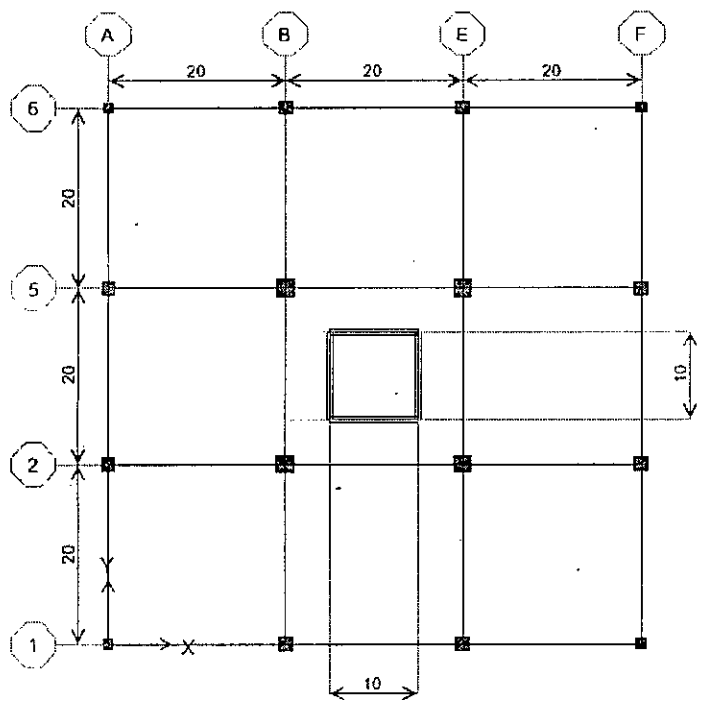


Figure 3.1: Plan view of model (Dimension in feet)

The floor slabs have been drawn by connecting column to column. The floor slabs and shear walls have been generated by using auto mesh feature of software. 4'x4' meshing is used up to the beam line. Slab has been modeled as shell member. The stair model has been simplified and modeled as slab connecting to share wall and surround beams. The supports are assigned as fixed type support. All the slabs are assigned for diaphragm action to minimize relative displacement. After completing the second floor slab the model has been extended up to ten stories by insert new storey. The other nine stories copy the property of storey-1-1. For this reason other storey contains name like storey 1-2, storey 1-3 and so on. After completing the structure the elevation view of model is looked like Fig. 3.2. One three dimensional figure of the model is shown in Fig. 4 of Appendix-A.1.

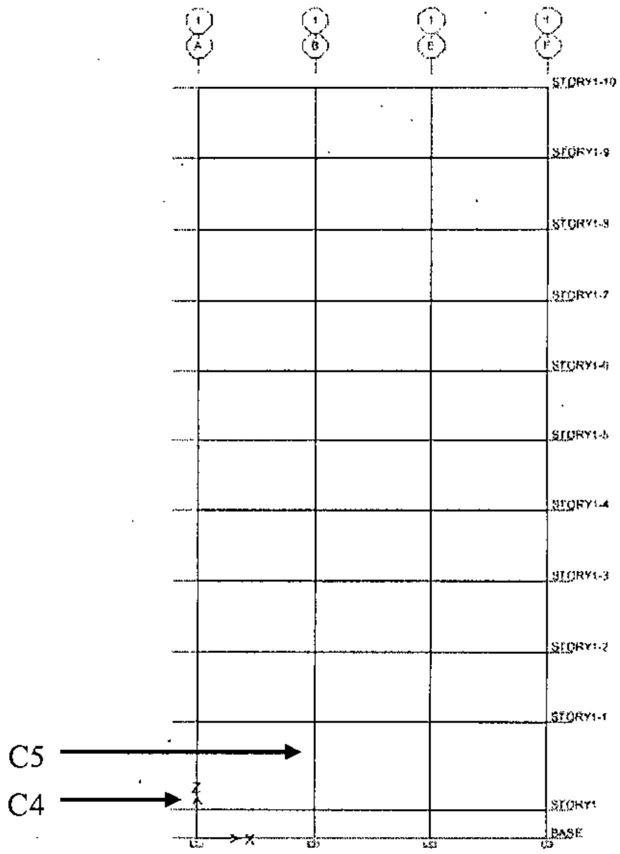


Figure 3.2: Elevation of model

The building designed as sway intermediate moment resisting frame. This is the reason for neglecting seismic data and mass source only from load has been selected. The self weight of building has been automatically calculated and incorporated by the software. The environmental load data inputs in the model by using code UBC-1994. There are some difference in code UBC-1994 and BNBC. Such as exposure type of building in ETABS stats from B but according to BNBC it is A. In ETABS the wind load value input in mph whereas in BNBC it is in km/hr. for X-direction wind load the wind direction angle is 0 (zero) degree. Windward coefficient has been calculated from BNBC and obtains 1.4. Leeward coefficient is negligible and just input for software needful. The exposure height has been considered from storey-1 to storey-1-10. Influence of wind load does not effect below soil. The wind speed value given in mph unit. The structural importance factor has been taken from BNBC specification and for this building it is 1.00. The UBC-1994 wind load data table is shown in Fig. 5 of Appendix-A.1. Same data table has been prepared for Y-direction wind load except the wind direction angle is assign as 90 (ninety) degree. UBC-1994 seismic loading parameters are assigned for earthquake load data input. The seismic load parameters can be defined as BNBC. There is an option for assigning the direction of eccentricity but this was not used. Two data tables have been created, one for X-direction earthquake load and other for Y-direction earthquake load. For time period calculation, the value calculated from Method-1 from BNBC. Here the noticeable thing is that the C_t value input is as ft. where in BNBC the value expresses in terms of meter. For assigning earthquake load, the storey ranges given from base to storey-1-10. The numerical coefficient (R_w) has been taken as 9.0 because the building has been considered as dual system with concrete IMRF. The seismic zone factor is user defined and soil coefficient is matched with BNBC value. The 1994 UBC seismic loading table is shown in Fig. 6 of Appendix-A.1. Only floor load has been provided for all slab members. All the floors including roof has same gravity load distribution.

All the members (i.e. beams and column etc.) have been satisfied for their maximum allowable loads for most critical load condition. The columns that have been checked by calculation are marked on Fig. 3.2 by arrow. C4 is a corner column and C5 is an edge column. C4 column has tested for Sway Moment Magnification factor and C5 column has tested for Nonsway Moment Magnification Factor.

3.4 Calculation Check for Adequacy of Structure

Two simple calculations have been done to check the adequacy of the model. One is to check the storey drift of structure and other is to check the earthquake shear force. If the lateral load imposed on model is high then, the storey drift will be excessive and abnormal deflection will occur. Also when the members are not properly connected with each other at the time of generating the model then some storey shows abnormal deflection shape and value. This is the most primary check of structure for competence. The sway value of storey presents in Fig. 7 of Appendix-A.1.

3.4.1 Storey drifts check of model

According to BNBC, storey drift is the displacement of one level relative to the level above or below due to design lateral force. Storey drift Δ, shall be limited as follow.

$$\Delta \leq 0.04\ h/R \leq 0.005h, \text{ For } T \leq\ 0.7 \text{ sec.} \tag{3.1}$$

$$\Delta \leq 0.03\ h/R \leq 0.004h, \text{ For } T \geq\ 0.7 \text{ sec..} \tag{3.2}$$

Where, h = height of storey. The period T used in this calculation shall be the same that used for determining the base share of earthquake force. The limits involving R in above equation shall be applicable only when earthquake forces are present.

The maximum deflection of building is greater for wind force than earthquake force. So the equation related to height of storey is used for check the model. Equation (3.2) is appropriate for use in this case because time period of the model is greater than 0.7. The maximum deflection has checked for the top two stories. So storey drift for storey-1-10 and storey-1.9 is $(0.004 \times 12' \times 12") = 0.576"$, which is larger than the difference of ETABS calculation. ETABS calculation is presented in Fig. 7 of Appendix-A.1, [i.e. $(2.226"-2.141") = 0.186"$]. So, the storey drift is in tolerable limit for maximum lateral force.

3.4.2 Base shear check of model

According to BNBC, under equivalent static force method the Design Base Shear (V) force of structure can be found by Eq. (3.3).

$$V = \frac{ZIC}{R} W \tag{3.3}$$

Here,

Z = Seismic zone coefficient = 1.0

I = Structural importance coefficient = 1.0

W = The total seismic dead load of building = 11096.17 kip

C = Numerical coefficient = $\dfrac{1.25S}{T^{(2/3)}}$

S = Site coefficient = 1.5

T = Fundamental period of vibration of building = $C_t(h_n)^{(3/4)}$

C_t = 0.049 because the model is intermediate moment resisting frame.

h = Height in meter above the base = $130.5 \div 3.28 = 39.787$ m

$$\therefore T = 0.049 \times 39.787^{(3/4)} = 0.776 \text{ sec.}$$

$$\therefore C = \frac{1.25 \times 1.5}{0.776^{(2/3)}} = 2.220$$

$$\therefore V = \frac{0.15 \times 1 \times 2.220}{9} \times W = 0.0370 \times W = 0.0373 \times 7086.08 = 262.18 \text{ kip}$$

The base shear force calculated by software is 259.42 kip, which is almost matched with the manual calculation. The output of seismic dead load and base shear force is shown in Fig. 8 of Appendix-A.1.

3.5 Calculation Check Manually for Nonsway Moment Magnification Factor (δ_{ns})

The design moment is magnified by Nonsway Moment Magnification Factor (δ_{ns}), when the slenderness of column predominantly depends on gravity loads. As the gravity load increases the design load gets closer to Critical Buckling Load of Column. This might cause failure of column in buckling. Increase of dead and live load directly influence in this type of slenderness behavior. This step involves checking of one column, i.e. edge column (C5) of periphery frame. In edge column the value of magnification factor is different for two axes of column. The basic moment and load data have been obtained from ETABS software calculation. The primary objective is to check δ_{ns} of column. C4 column is checked for design load combination-2 (DCON2). δ_{ns} mostly depends on load combination of gravity loads. For edge column, the major and minor moments are not same for imposed loads because of the position of the column and non-symmetric biaxial moment distribution. The nonsway values are checked in top of column location. The column height is 17.5' and the clear height of column is measured from bottom face of beam in storey-1-1 to top face of grade beam in storty-1. In calculation of slenderness, ETABS consider Effective Length Factor (k) as 1.0 for nonsway frame. By considering k equals to 1.0, ETABS basically design concrete building in a very conservative way. The actual value is usually less than 1.0. For analysis of slenderness the k factor has been considered as 1.0 to take the advantage of automatic design phenomena even the value could be overwritten at any time after the analysis and design for a particular column.

3.5.1 Design data

To determine the δ_{ns} value for edge column the geometric and load data information have been directly recorded from ETABS. The values are:

Column dimension = 16" x 16"

Beam dimension = 18" x 12"

Design load, $P_u = 684.785$ kip

At station location 16'-0" (top of column) and for load combination-2 (DCON2), the major and minor axis moments are presented in Table 3.2.

Table 3.2: Service loads and moments for column C5

Load type	Major axis moment, *M3* (kip-ft)	Minor axis moment, *M2* (kip-ft)	Member load (kip)
Dead	-1.171	17.330	-350.63
Live	-0.428	6.914	-114.06

$M3_{ns} = 1.4 \times (-1.171) + 1.7 \times (-0.428) = -2.367 \text{ kip - ft.}$

$M2_{ns} = 1.4 \times (17.330) + 1.7 \times (6.914) = 36.016 \text{ kip - ft}$

Minimum moment $= P_u \times (0.6 + 0.03 \times h) = (684.785 \times (0.6 + 0.03 \times 16)) \div 12$

$= 61.631$ kip -ft.

Here h is the minimum dimension of column. As the column is square in shape the minimum and maximum dimension is same. This is the reason for obtaining same moment values in major and minor axis. Minimum moment is larger than the actual acting moment. For this reason minimum moment governs for both axes. For column design major axis moment is considered. When the minimum moment governs, ETABS shows the major axis moment (*M3*) equals to minimum moment but minor axis moment (*M2*) remain unchanged. The primary moment at top and bottom location of member has been shown in Table 3.3. The subsequent calculation is dependent on these moments.

Table 3.3: Location wise (top and bottom) moment in C5 column

Type of moment	Location of moment	
	At height 0.0 ft.	At height 16.0 ft
Major axis moment, *M3* (kip-ft)	-2.370	0.830
Minor axis moment, *M2* (kip-ft)	-18.210	36.020

The procedure of finding δ_{ns} is as follows:

The constant C_m could be determined by using Eq. (2.12),

$$C_{m\,(3)} = 0.6 + 0.4 \times \left(\frac{0.830}{-2.370} \right) = 0.460 > 0.400 = 0.460$$

$$C_{m\,(2)} = 0.6 + 0.4 \times \left(\frac{-18.210}{36.020} \right) = 0.398 < 0.400 = .0400$$

The β_d factor could be determined by using Eq. (2.20),

$$\beta_d = \frac{(1.4 \times dead\ load)}{(1.4 \times dead\ load\ +\ 1.7 \times live\ load)}$$

$$\beta_d = \frac{(1.4 \times -350.63)}{(1.4 \times \text{-}350.63\ +\ 1.7\ \times\ \text{-}114.06)} = 0.717$$

The constant *EI* could be determined by using Eq. (2.22),

$$EI = \frac{\left(0.4 \times 3.6 \times 10^6 \times \left(\frac{16 \times 16^3}{12} \right) \right)}{(1 + 0.717)} = 4.580 \times 10^9 \text{ in}^2\text{-lb}$$

The Critical Buckling Load of column could be determined by using Eq. (2.7),

$$P_c = \left(\frac{(\pi^2 \times 4.580 \times 10^9)}{(1 \times 16 \times 12)^2} \right) \div 1000 = 1226.20 \text{ kip}$$

The Nonsway Moment Magnification Factor could be determined by using Eq. (2.30),

$$\delta_{ns\,(3)} = \frac{0.460}{\left(1 - \frac{684.785}{0.75 \times 1226.20} \right)} = 1.801$$

$$\delta_{ns\,(2)} = \frac{0.400}{\left(1 - \frac{684.785}{0.75 \times 1226.20} \right)} = 1.566$$

Design moment, $M3 = 1.801 \times 61.631 = 110.997$ kip - ft

Design moment, $M2 = 1.566 \times 36.016 = 56.401$ kip - ft

The calculated value almost matched with result output of ETABS model. The analysis and design output for C5 column is presented in Fig. 9 of Appendix-A.1.

3.6 Calculation Check by PCACOL for Nonsway Moment Magnification Factor (δ_{ns})

The steps to calculate column forces and other parameters by PCACOL are provided below. The calculation steps are visually presents in Appendix-A.2 in step by step.

3.6.1 Steps of PCACOL

After opening a new file the design data need to be provided by using "Input" tab of menu bar. In the input menu there are seven items which are required for design. The steps for calculating δ_{ns} using PCACOL is described in following sections.

3.6.1.1 Step-1: Provide general information

The data that has been required by this step is primary unit, design code, option for slenderness and design moment application (i.e. biaxial or uniaxial).

3.6.1.2 Step-2: Material properties

Concrete strength, steel strength and basic properties of concrete have to be provided in this step. Only concrete strength and steel strength has been required. The other values i.e. elasticity, beta, ultimate strength has been calculated automatically by the software.

3.6.1.3 Step-3: Sectional properties

Width and length of column are the input parameters. As the column is uniform in length no increment value has been needed.

3.6.1.4 Step-4: Reinforcement information

The limit of minimum and maximum size of reinforcement and number of bars which should be checked by the software has been provided in this step. Additional information like clear cover, layout of column reinforcement has been provided. The minimum and maximum bar No. and size can not be equal. If any invalid data given then warning is show.

3.6.1.5 Step-5: Slenderness option input

In step 1, consideration of slenderness has been selected. So, in this step the slenderness parameters have been provided. The structure has been modeled as braced against sideway and the k factor is provide 1(one) because ETABS consider k factor as one for calculating δ_{ns}.

3.6.1.6 Step-6: Basis loads and moments input

The column has been checked for nonsway condition so, the dead load and live load parameters are essential. These basic loads are imported from ETABS calculation. For inputting the moment data one thing should be noticed that, all the data are positive value because the deflection shape is double curvature and positive moment at the end of the column. This data input has been recommended by PCACOL design guideline. The moment data presentation is different for ETABS and PCACOL software.

3.6.1.7 Step-7: Load combination

The column has been designed for load combination-2. So, only dead load and live load combination (DCON2) has been used for calculation. No lateral load has been considered. The default option of other lateral load combination has been deleted because of simplicity of analysis.

3.6.2 Design output

The design output for PCACOL is presented in Appendix-A.3. The parameters calculated by PCACOL to determine δ_{ns} for two axes are presented in Table 3.4.

Table 3.4: Parameters found by PCACOL for determining δ_{ns} values

Parameters	Value
C_m for X-axis	0.400
C_m for Y-axis	0.460
β_d for X and Y axis	0.717
P_c for X and Y axis	1569 kip
δ_{ns} for X axis	1.000
δ_{ns} for Y axis	1.101

This is noticeable that the value of P_c for X and Y axis, δ_{ns} for X axis and δ_{ns} for Y axis are different from the calculation that has been achieved from ETABS calculation output. This is due to using of one formula for determining EI that is different from ETABS. The equation of EI that is used by PCACOL is Eq. (2.21),

$$EI = \left(\frac{0.2 \times 3605 \times \left(\frac{16 \times 16^3}{12} \right) + 29000 \times \left(8 \times 0.79 \times 5.625^2\right) + 29000 \times \left(4 \times 0.79 \times 1.875^2\right)}{1 + 0.717} \right)$$

$$\text{So, } EI = \frac{\left(1.0059 \times 10^7\right)}{1.717} = 5.858 \times 10^6 \text{ in}^2\text{-kip}$$

The Critical Buckling Load of column could be determined by using Eq. (2.7),

$$P_c = \left(\frac{\left(\pi^2 \times 5.858 \times 10^6\right)}{\left(1 \times 16 \times 12\right)^2} \right) = 1568.36 \text{ kip}$$

The Nonsway Moment Magnification Factor could be determined by using Eq. (2.30),

$$\delta_{ns\,(Y)} = \frac{0.460}{\left(1 - \frac{684.80}{0.75 \times 1568.36}\right)} = 1.101$$

$$\delta_{ns\,(X)} = \frac{0.400}{\left(1 - \frac{684.80}{0.75 \times 1568.36}\right)} = 0.957 < 1.000 = 1.000$$

The value matched with PCACOL result output.

3.7 Result Variation of ETABS and PCACOL for Calculating δ_{ns}

As ETABS and PCACOL use different formula for calculating δ_{ns}, there are some differences in output values. These variations are shown in Table 3.5.

Table 3.5: Comparison between results of ETABS and PCACOL for δ_{ns} calculation

Parameters	ETABS value	PCACOL value	% difference
EI	4.580×10^6 in^2-kip	5.858×10^6 in^2-kip	(+) 27.9%
P_c	1226.20 kip	1568.36 kip	(+) 27.9%
$\delta_{ns(Y\ or\ 3)}$	1.801	1.101	(-) 38.87%
$\delta_{ns(X\ or\ 2)}$	1.566	1.000	(-) 36.14%
% of steel	4.079	3.703	(-) 9.22%

From Table 3.5, this is clear that ETABS design approach is conservative. According to Nilson *et al.* (2003), for lightly reinforced member the use of Eq. (2.22) is more conservative, but for highly reinforced member it greatly underestimates the value of *EI*. Equation (2.21) is more reliable for the entire range of ρ and definitely for medium and higher ρ value. There is no option for choosing or changing equation in these two software. Even the value of *EI* and P_c cannot be overwritten in ETABS. So, this fact is very important while designing by ETABS and PCACOL.

3.8 Calculation Check Manually for Sway Moment Magnification Factor (δ_s)

This section involves checking of one column, i.e. corner column (C4) of periphery frame. Corner column has been selected because lateral load influences design of this column. The basic moment and load data are achieved from ETABS software calculation. The primary objective is to check the value of Sway Moment Magnification Factor (δ_s) of column. The noticeable thing is that there is no prescribed equation for determining δ_s in ETABS design manual. The designer has to run P-Δ analysis to know the sway moment effect on column. The design method that is described in Chapter-2 for direct P-Δ analysis has been followed to determine the δ_s value. C4 column has been checked for design load combination-18 (DCON18). DCON18 is a combination of dead load and earthquake load. The major and minor moments of edge column are same for dead and live load because of the position of the column and symmetric biaxial moment distribution. The sway values checked in top of column location. The column height is 17.5' and the clear height of column measure from bottom face of beam on storey-1-1 to top face of grade beam on storey-1.

3.8.1 Design data

To determine the δ_s value for corner column the geometric and load data information has been directly recorded from ETABS. The values are:

Column dimension = 14" x 14"

Beam dimension = 18" x 12"

Design load, P_u = 684.785 kip

At top of column location (16'-0") and for load combination-18 (DCON18) the ultimate load, maximum shear force and relative storey drift are presented in Table 3.6.

Table 3.6: Parameters for determining Stability Index

Parameters	P_u (kip)	$V3$ (kip)
Corner column	221.25	2.41
Edge column	385.63	3.74
Drift at bottom of column	0.045"	
Drift at top of column	0.243"	

To calculate the Stability Index for the periphery frame, the total ultimate load for column and maximum shear force acting on column is required. In periphery frame, there are two corner columns and two edge columns. So,

$$\sum P_u = 2 \times 221.25 + 2 \times 385.63 = 1213.76 \text{ kip}$$

$$\sum V = 2 \times 2.41 + 2 \times 3.74 = 12.3\ 5 \text{ kip}$$

The Stability Index could be determined by using Eq. (2.13),

$$Q = \frac{\sum P_u \times \Delta_0}{\sum V \times l_c} = \frac{(1213.76 \times (0.243 - 0.045))}{(12.3 \times 17.5 \times 12)} = 0.093 > 0.05$$

So, it is a sway frame.

The Sway Moment Magnification Factor could be determined by using Eq. (2.32),

$$\delta_s = \frac{1}{1-Q} = \frac{1}{1-0.093} = 1.103$$

Minimum moment $= P_u \times (0.6 + 0.03 \times h) = (221.25 \times (0.6 + 0.03 \times 14)) \div 12$

$= 18.806$ kip -ft.

The minimum moment is same for major and minor axis because of square shape column. Minimum moment is larger than the actual acting moment. So, minimum moment governs for both axes. For column design major axis moment is considered. When the minimum moment govern ETABS show the major axis moment (*M3*) equals to minimum moment but minor axis moment (*M2*) remains unchanged.

One special consideration has to be checked for determining δ_s. The maximum moment may occur between the ends of the column being designed. This is not ordinarily the case for columns in sway frame. For sway frames, the maximum moment usually occurs at one end of the column. However, under certain conditions this may not be the case. Hence ACI requires that one check for such a condition. Such a condition occurs when by the Eq. (2.28),

$$\frac{L_u}{r} > \frac{35}{\sqrt{\frac{P_u}{f'_c A_g}}}$$

If above equation is true then the column must be designed as a nonsway column.

$$\therefore \frac{L_u}{r} = \frac{16\times 12}{0.3\times 14} = 45.71 \text{ and } \frac{35}{\sqrt{\frac{P_u}{f'_c A_g}}} = \frac{35}{\sqrt{\frac{221.25}{4\times 14\times 14}}} = 65.88$$

So, $\frac{L_u}{r} < \frac{35}{\sqrt{\frac{P_u}{f'_c A_g}}}$ and no further magnification required.

After overwrite the value of δ_s the design moment has changed. The step of overwrite and after overwrite result of δ_s value in ETABS design is shown respectively in Fig. 1 and Fig. 2 in Appendix-A.4. The model is run with P-Δ analysis with the moment value before overwrites with δ_s. The load combination that is used for P-Δ analysis is 1.4*DL*+1.7*LL* (White and Hajjar, 1999), which has also recommended in ETABS design manual for column. The steps of P-Δ analysis has been shown in Fig. 3 and Fig. 4 in Appendix-A.4. In the Table 3.7 a list presents the value of moments at different condition of column. The moment value after overwrite almost matched with P-Δ analysis. As the P-Δ analysis moments are almost same as overwrite moment it can be concluded that the using of Eq. (2.25) in this calculation is consistent for checking ETABS design calculation.

Table 3.7: Result comparison of different analysis mode

	Types of moment	**Moment value (kip-ft)**
Before overwrite	*M3*	18.807
	M2	17.474
After overwrite	*M3*	18.807
	M2	18.464
After P-Δ analysis	*M3*	18.963
	M2	18.144

3.9 Calculation Check by PCACOL for Sway Moment Magnification Factor (δ_s)

The steps that has been followed in PCACOL to calculate the Sway Moment Magnification Factor has been graphically presented in Appendix-1(E). Major inputs have to be provided in slenderness data. It requires the ratio of design load of all columns of concern frame at design storey to design load of selected column and ratio of critical buckling load of all columns of concern frame at design storey to buckling load of selected column. PCACOL cannot perform P-Δ analysis. This is the reason that, the P-Δ analysis result obtains from ETABS cannot be compared with the PCACOL output. The k is taken 1.0 for both corner and edge columns. This is done to get the result for least conservative condition. The calculation steps are shown below:

$$\sum P_u = 2\times 221.25 + 2\times 385.63 = 1213.76 \text{ kip}$$

$$\frac{\sum P_u}{P_u} = \frac{1213.76}{221.25} = 5.486$$

The EI value for corner column could be determined by using Eq. (2.21),

$$EI = \left(0.2\times 3.6\times 10^6 \times \frac{14\times 14^3}{12}\right) + \left(29\times 10^6 \times 6\times 0.44\times 4.75^2\right) = 4.03\times 10^9 \text{ in}^2\text{-lb}$$

The EI value for edge column could be determined by using Eq. (2.21),

$$EI = \left(0.2\times 3.6\times 10^6 \times \frac{16\times 16^3}{12}\right) + \left(29\times 10^6 \times 8\times 0.79\times 5.625^2\right) + \left(29\times 10^5 \times 4\times 0.79\times 1.875^2\right)$$

$$\therefore EI = 1.01\times 10^{10} \text{ in}^2\text{-lb}$$

The Critical Buckling Load for corner column could be determined by using Eq. (2.7),

$$P_c = \left(\frac{\pi^2 \times 4.03\times 10^9}{(1\times 16\times 12)^2}\right) \div 1000 = 1078.95 \text{ kip}$$

The Critical Buckling Load for edge column could be determined by using Eq. (2.7),

$$P_c = \left(\frac{\pi^2 \times 1.01\times 10^{10}}{(1\times 16\times 12)^2}\right) \div 1000 = 2704.10 \text{ kip}$$

$$\sum Pc = 2\times 1078.95 + 2\times 2704.10 = 7566.10 \text{ kip}$$

$$\therefore \frac{\sum P_c}{P_c} = \frac{7566.10}{1078.95} = 7.01$$

From Eq. (2.32) the δ_s for corner column,

$$\delta_s = \frac{1}{1 - \dfrac{1213.76}{0.75 \times 7566.10}} = 1.27$$

For corner column, the δ_s is same for major and minor axis. There are some difference between the calculation of δ_s by ETABS and PCACOL. The variations in different parameters are presented in Table 3.8.

Table 3.8: Comparison between result of ETABS and PCACOL for δ_s calculation

Parameters	ETABS value	PCACOL value	% difference
$\delta_{s\ (Y\ or\ 3)}$ or, $\delta_{s\ (X\ or\ 2)}$	1.103	1.270	(+) 15.14%
% of steel	1.00%	1.27%	(+) 27.00%

3.10 Conclusion

The objective of this chapter was to review the slender column design guideline according to ACI Code by utilizing software named ETABS and PCACOL. Both δ_{ns} and δ_s have been calculated manually and compared with the results of ETABS and PCACOL output value. The structural model has been developed using ETABS. For manual calculation, the design loads and column moments have been obtained from ETABS analysis output. Manual calculation of slender column design following the ACI Code method has validated the calculation process of ETABS and PCACOL software.

It has been observed that these two software uses two different equations for determining δ_{ns} value. ETABS follows Eq. (2.22) and PCACOL uses Eq. (2.21) in calculating *EI* value. Equation (2.22) is more conservative than Eq. (2.21) because of neglecting reinforcement involvement in determining δ_{ns}. As a result, ETABS yields higher value of δ_{ns} and consequently an increased steel ratio in column. The manual calculation has been done by following both Eqs. (2.21) and (2.22). The values of δ_{ns} obtained manually matched with those of ETABS and PCACOL.

When the moment and load values of column copied from ETABS to PCACOL, it has been noted that PCACOL loading sign convention is different. PCACOL considers top and bottom end moments as positive value when the column shows double curvature. Where as in ETABS the two end moments shows opposite sign when the column shows double curvature.

To incorporate sway effect in a structure, ETABS uses automated feature of *P*-Δ analysis option. By employing this automated feature of ETABS, the designer can include the effect of sway on column moments. It has been observed that for a particular column the magnified moment found in manual calculation by using Eq. (2.25) is almost same as that found after *P*-Δ analysis using ETABS. So, this concludes that the designer could achieve secondary moment effect on a particular column by Eq. (2.25) without running *P*-Δ analysis option. But the limitation of using the equation needs extra attention which is described in Chapter 2. So, this could be recommended that whatever the value of δ_s, *P*-Δ analysis should always be carried out.

There is no option for *P*-Δ analysis in PCACOL to obtain secondary moment effect on single column. PCACOL directly use Eq. (2.32) to calculate δ_s. For a particular column, magnified moment found by using Eq. (2.32) is larger than magnified moment found after *P*-Δ analysis in ETABS. Above this, as PCACOL uses Eq. (2.21) to calculate *EI* value, it gives more conservative result for steel ratio than ETABS. This is due to the fact that when *P*-Δ analysis is run by ETABS it considers the sway effect on the whole building structure whereas at time of using PCACOL only sway of one frame has been considered in the current work. As a result, sway effect of one frame has been found larger than sway effect of the full structure and a higher value of δ_s has been obtained in PCACOL.

CHAPTER 4

PARAMETRIC STUDY

4.1 Introduction

The design guidelines of slender column according to ACI Code 318 (1999) and validation of software which can perform slender column design following ACI Code have been scrutinized in Chapter 3. Conservativeness and accuracy of ETABS (CSI, 2003) and PCACOL (PCA, 1999) design software have been notified in the preceding Chapter. Sometime the designer may ignore or have little idea about the slenderness consideration in column because of cumbersome design procedure of slender column. The designers who use automated feature of ETABS software get magnified moment from result output but designer who do conventional manual calculation need some extra awareness in slender column design. Even ETABS does not calculate Sway Moment Magnification factor automatically and cannot consider it in design unless P-Δ analysis is run. This could results in under designed structures. So, it is important to recognize that which factors need careful judgment in slender column design. For this circumstance a parametric study has been carried out.

Column design depends on various features. The parameters like type of frame, type of column, position of column, end condition, loads, length, cross section, steel ratio etc are the influencing features of column design. In this study, in a building system one particular parameter of a single column has been changed when other parameters has been kept unchanged to observed the effect of that change in column.

4.2 Characteristics of Basic Parameters and Functions

The relationship between one parameter and function with other need to be clarified before the parametric study initiate. This has been made easy to understand the behavior of certain feature or function of column. For one particular column Effective Length Factor *(k)*, Critical Buckling Load *(P_c)*, Nonsway Moment Magnification Factor (δ_{ns}), Unbraced Length of Column *(l_u)* have been taken as basic parameters and functions. The column cross section is 18"x18" and the unbraced length is 13'. The input loads and moments are assumed to take reasonable value. One datasheet has been developed using Microsoft Excel software with the equations for determining δ_{ns}. Two cases have been studied. In case 1, only the k value has been changed from 0.8 to 1.0 with 0.05 increments of 13' unbraced long column. There has been no other variation incorporated in this slenderness calculation. So, case 1 is one column of different end conditions. Using the database δ_{ns} has been calculated for 5 columns and plotted accordingly.

In case 2, this column has been studied for the variation of length with other parameters kept unchanged. The column clear distance has been changed from 12' to 16' with an increment of 1'. The k value has taken 1.0. There has been no other variation incorporated in this slenderness calculation. The loads and moments that have been assumed in case 1 are same for case 2. So, case 2 is one column of different unbraced length with one end condition. By using excel datasheet δ_{ns} has been calculated for 5 columns and plotted accordingly.

The column end condition is one of the major factors for slender column behavior. k is measured from column end condition. From Eq. (2.7) it is seen that the capacity of column is inversely proportional with square of k value. This means when the value of k increases then P_c decreases. The relation between k and P_c of column is present in Fig. 4.1. From Fig. 4.1 this is seen that capacity of column changes about 35% when the column end condition has been changed from hinge toward fixed.

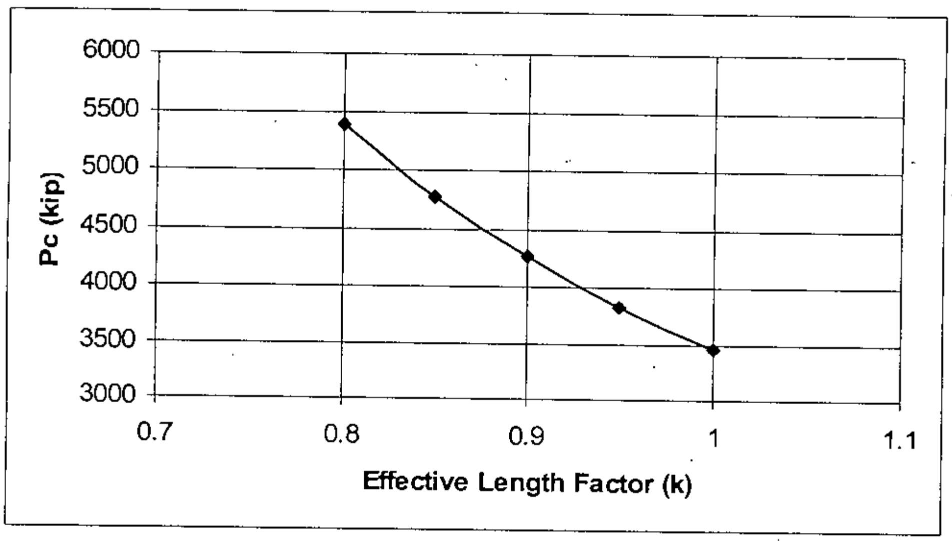


Figure 4.1: Variation of Critical Buckling Load with Effective Length Factor

From Eq. (2.30) it is observed that P_c of column in inversely related with δ_{ns} of column. So, δ_{ns} is proportional with k. This means when the value of k increases δ_{ns} increases. The relation between k and δ_{ns} of column is presented in Fig. 4.2. Increasing the value of k means the column shows more buckling behavior. So, δ_{ns} is higher for a column with hinged end condition than a fixed condition. The variation of δ_{ns} is about 10% when the column end condition approached to fixity.

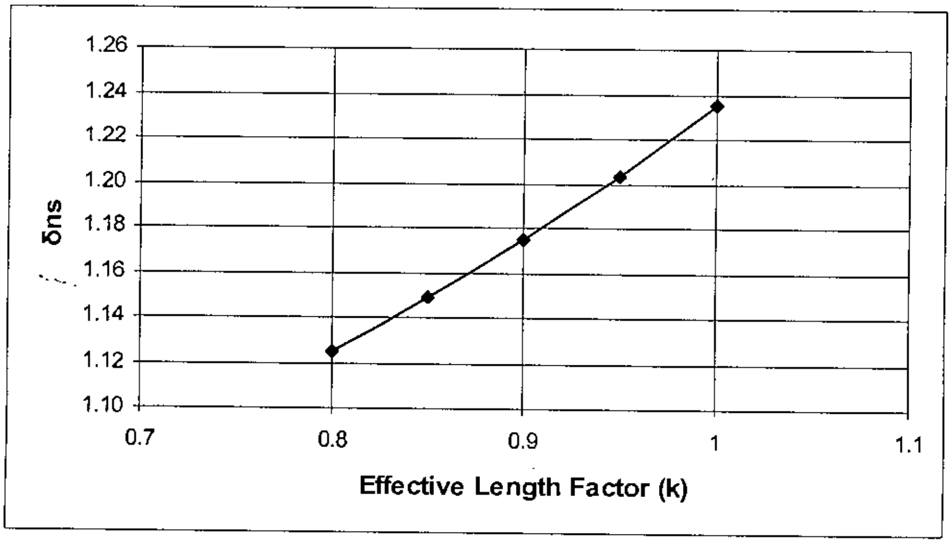


Figure 4.2: Variation of Nonsway Moment Magnification Factor with Effective Length Factor

As in Eq. (2.7), l_u is inversely proportional with P_c of column. This means when l_u increases then the P_c decreases. This indicates that the column shows more slender behavior because when critical load decreases column slenderness increases. The relation between l_u and P_c of column represents in Fig. 4.3. The buckling load capacity is larger in relatively short column. The variation of buckling load is very high with increasing of column length. From Fig. 4.3 this is observed that P_c decreases about 80% when unbraced length increases about 33%. So, very small increment of column length could decrease high buckling capacity of column. P_c of column is also depends on cross section of column. So, increment in length and decrement in cross section should be carefully done at the time of column design. Reduction of P_c should be carefully judged by the designer at the time of slender column analysis.

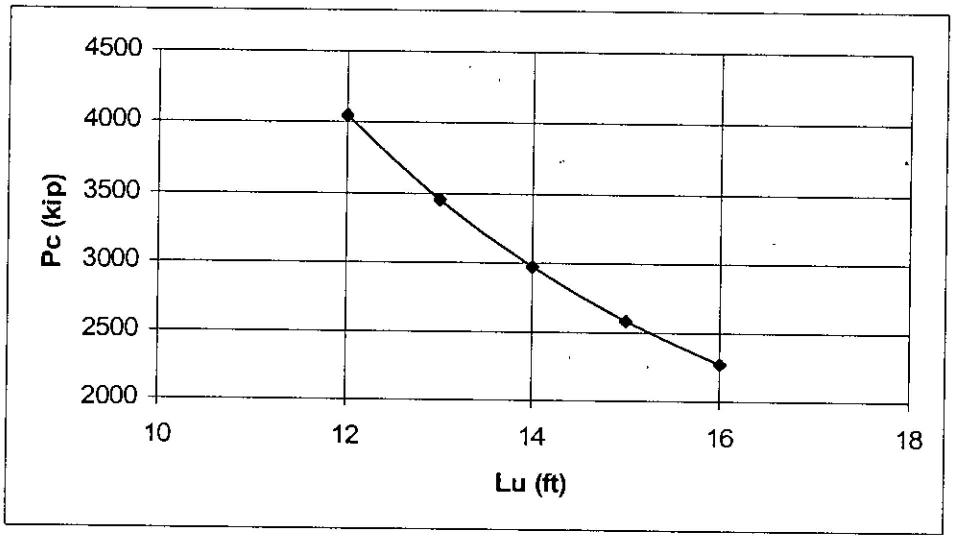


Figure 4.3: Variation of Critical Buckling Load with Unbraced Column Length

From Eq. (2.7) it is seen that l_u is proportional with δ_{ns}. Which means when column length increases the δ_{ns} increases subsequently the moment increases. The relation between l_u and δ_{ns} of column is presented in Fig. 4.4. As column length increases the buckling capacity decreases which is described in preceding section. Again when buckling capacity decreases then δ_{ns} increases. Also by combining this two relation this could be concluded that when l_u increases, δ_{ns} increases. From Fig. 4.4 it is seen that δ_{ns} increases about 25% when length increases about 33%.

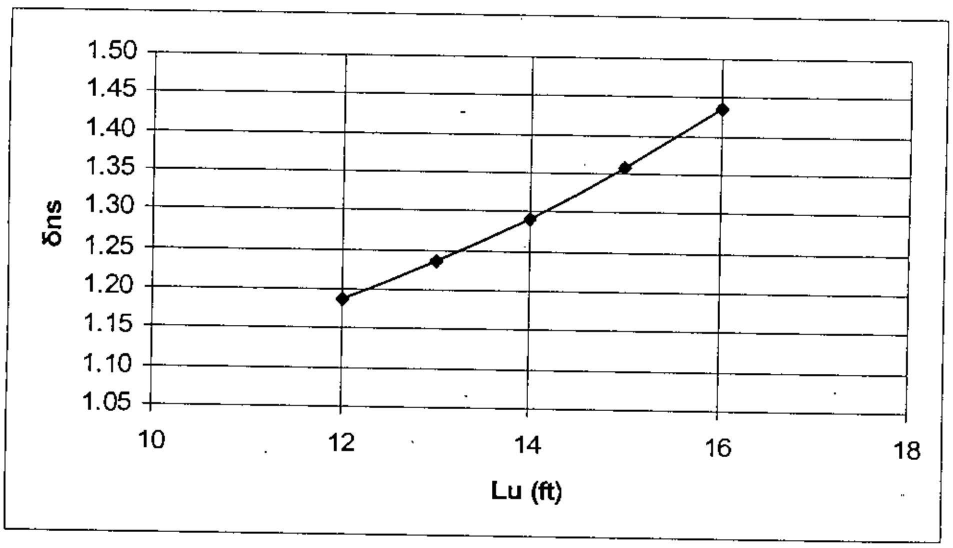


Figure 4.4: Variation of Nonsway Moment Magnification Factor with Unbraced Column Length

4.3 Model Development of Beam-Column Structure for Parametric Study

The preliminary steps of model generation has been described in Chapter 3 and graphically presented in Appendix-A.1. In this parametric study 15 models are generated for beam-column structure with a tube shape shear wall in core of the structure. 5 models have 15'x15' slab panel sizes, other 5 models have 20'x20' slab panel sizes and another 5 models have 25'x25' slab panel. Each floor has 3 slab panels in each direction. All the models are of 10 stories. The foundations for columns and shear walls are assigned as fixed support. For each model, 1 corner column, 1 edge column and 1 inner column in ground floor has been selected for this parametric study. So, among these 15 beam-column frame structures, total 45 ground floor columns are considered for slender column behavior analysis. The ground floor is increased from 10' to 20' height with an increment of 2.5'. The other storey height is 10' and kept unchanged in all structures. The column dimensions are determined in such a way that the steel ratio does not exceed 4% in most critical condition. The beam dimensions are chosen such a way that they do not fail for the most critical situation. The floor slabs are 6" thick. The imposed dead load and live load is 60 psf. The breakdown of dead load that has been considered in Chapter-3 is kept unchanged for this parametric study. The dead load and live load pattern in roof is same like other floor. All the floors have been assigned as rigid diaphragm to reduce relative displacement. All the floors and shear wall have been assigned as auto mesh using 4' by 4' meshing in the boundary line of slab and beam. The program automatically calculates the earthquake and wind force according to load parameters input. The earthquake load and wind load data are input as same as described in section 3.2.2 of Chapter 3. The special seismic force is not considered in calculation and the mass source is from all dead loads only.

Figure 4.5 represents the plan view of ground floor of one model consisting 15'x15' slab panel. Total bay width in either direction is 45'. The tube shape shear wall which is placed in core of the structure is 5'x5' in dimension and 9" thick. The shear wall is connected with main frame of the structure by grade beams. The grade beam is 20"x12" in depth. For all models the grade beam dimension is kept identical and all satisfy the most critical design condition. The column numbered according to grid line. Column 1-A, 1-B and 2-B is corner, edge and inner column, respectively.

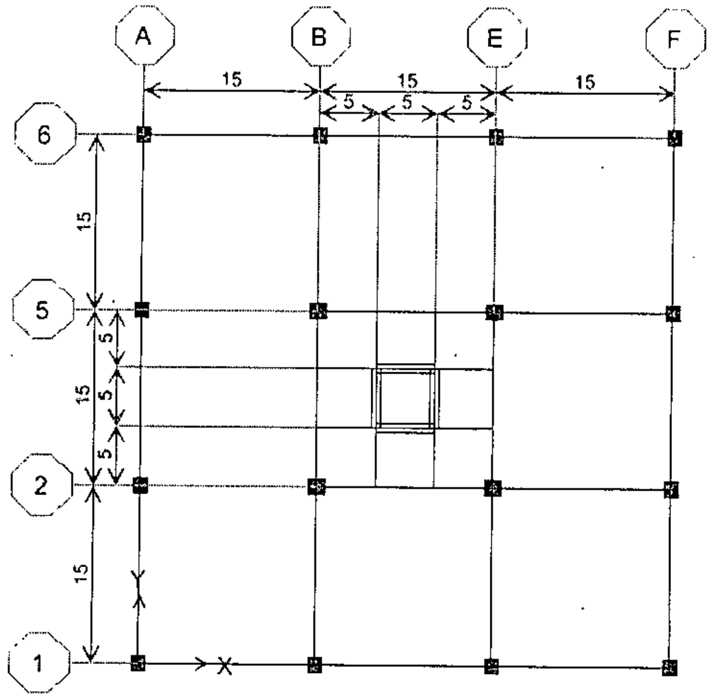


Figure 4.5: Plan view of ground floor in 15'x15' floor panel model

Figure 4.6 represents the plan view of ground floor of one model consisting 20'x20' slab panel. The total bay width in either direction is 60'. The tube shape shear wall which is placed in core of the structure is 10'x10' in dimension and 9" thick.

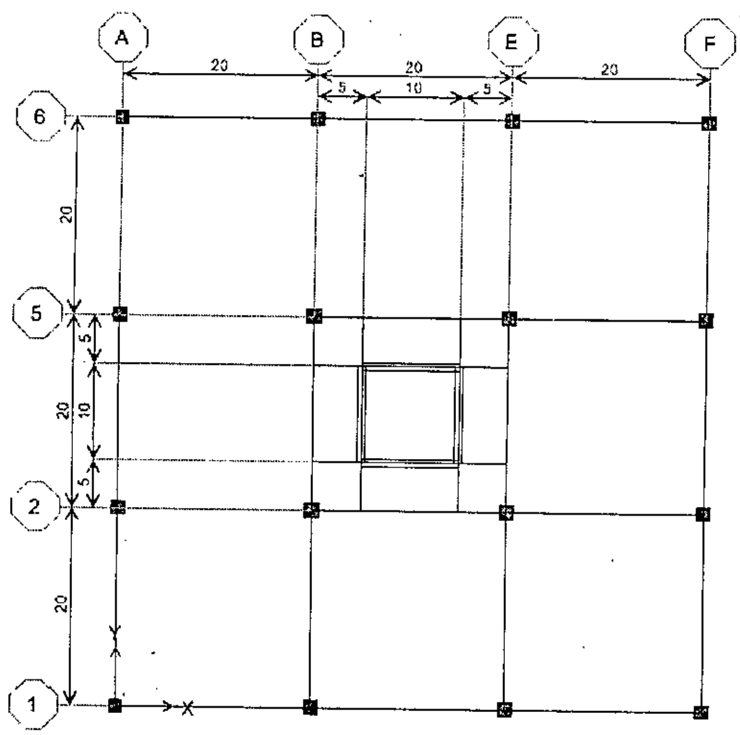


Figure 4.6: Plan view of ground floor in 20'x20' floor panel model

Figure 4.7 represents the plan view of ground floor of one model consisting 25'x25' slab panel. The total bay width in either direction is 75'. The tube shape shear wall which is placed in core of the structure is 10'x10' in dimension and 9" thick.

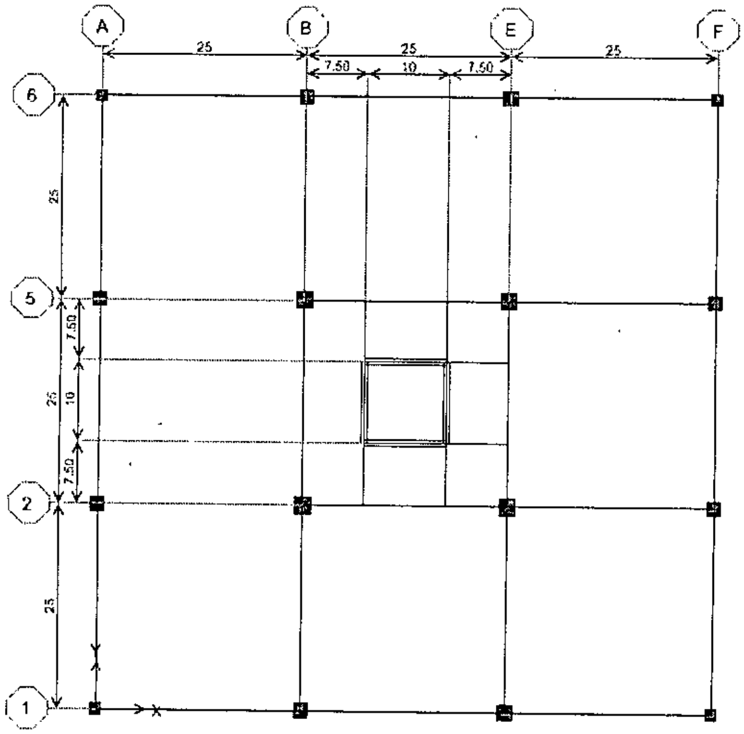


Figure 4.7: Plan view of ground floor in 25'x25' floor panel model

Table 4.1 shows different cases of studies. The case has been named according to slab panel size and length of ground floor column. The cross section of columns and beams are selected from practical point of view. For reinforced concrete column structure, slab panel greater than 25' in length is not common. The slab panel larger than 25' needs special design requirements. Slab panel smaller than 15' is not frequent for commercial building because of ground floor car parking point of view. Beam dimension kept same for all model because of consistency in analysis. After analysis each structure by ETABS, the moment values of column end and design load for each targeted columns are extracted from software and database is prepared using Microsoft Excel 2003. All the parameters are calculated using Excel datasheet and crosschecked with ETABS design output. Using the Excel output the graphs for selected parameters and functions are drawn.

Table 4.1: Parameters for beam-column structures

SN	Floor Panel Size (Feet)	Column Position	Case No. = Column Length AT Ground Level (Feet)	Column Dimension (Inches)	Beam Dimension (Inches)
1	15'x15'	Corner Column	Case 1.0 = 10.0'	14"x14"	18"x10"
			Case 2.0 = 12.5'		
			Case 3.0 = 15.0'		
			Case 4.0 = 17.5'		
			Case 5.0 = 20.0'		
		Edge Column	Case 1.0 = 10.0'	15"x15"	
			Case 2.0 = 12.5'		
			Case 3.0 = 15.0'		
			Case 4.0 = 17.5'		
			Case 5.0 = 20.0'		
		Inner Column	Case 1.0 = 10.0'	16"x16"	
			Case 2.0 = 12.5'		
			Case 3.0 = 15.0'		
			Case 4.0 = 17.5'		
			Case 5.0 = 20.0'		
2	20'x20'	Corner Column	Case 1.1 = 10.0'	15"x15"	18"x10"
			Case 2.1 = 12.5'		
			Case 3.1 = 15.0'		
			Case 4.1 = 17.5'		
			Case 5.1 = 20.0'		
		Edge Column	Case 1.1 = 10.0'	17"x17"	
			Case 2.1 = 12.5'		
			Case 3.1 = 15.0'		
			Case 4.1 = 17.5'		
			Case 5.1 = 20.0'		

Table 4.1 (Contd.)

		Inner Column	Case 1.1 = 10.0'	18"x18"	
			Case 2.1 = 12.5'		
			Case 3.1 = 15.0'		
			Case 4.1 = 17.5'		
			Case 5.1 = 20.0'		
3	25'x25'	Corner Column	Case 1.2 = 10.0'	16"x16"	18"x10"
			Case 2.2 = 12.5'		
			Case 3.2 = 15.0'		
			Case 4.2 = 17.5'		
			Case 5.2 = 20.0'		
		Edge Column	Case 1.2 = 10.0'	20"x20"	
			Case 2.2 = 12.5'		
			Case 3.2 = 15.0'		
			Case 4.2 = 17.5'		
			Case 5.2 = 20.0'		
		Inner Column	Case 1.2 = 10.0'	24"x24"	
			Case 2.2 = 12.5'		
			Case 3.2 = 15.0'		
			Case 4.2 = 17.5'		
			Case 5.2 = 20.0'		

4.4 Study on Corner Column for Nonsway Moment Magnification Factor (δ_{ns})

To calculate δ_{ns} for 15 columns of 5 different lengths and 3 different slab panels, the moment as well as load data are taken from ETABS solution and put into the prepared Excel 2003 datasheet. The limiting value of Slenderness Ratio has been checked with the recommended ACI Code guideline. According to the guideline when the value of Slenderness Ratio is greater than 34-12M_1/M_2, the column should be treated as slender column. The ratio M_1/M_2 depends on curvature type. The curvature type could be recognized by reading the sign of end moments. The maximum value of 34-12M_1/M_2 that is limited by ACI Code is 40.0. This is observed that for 15 corner columns in beam-column frame structure the value of 34-12M_1/M_2 is always greater than 40.0.

From the Fig. 4.8 below it is seen that among 15 columns Slenderness Ratio is above or equals to 40 in 6 columns. But, from Appendix-B.1 it is seen that only 3 columns experiences slenderness effect, which means δ_{ns} is greater than 1. The P_c value decreases around 25.0% when column length increment is 2.5'.

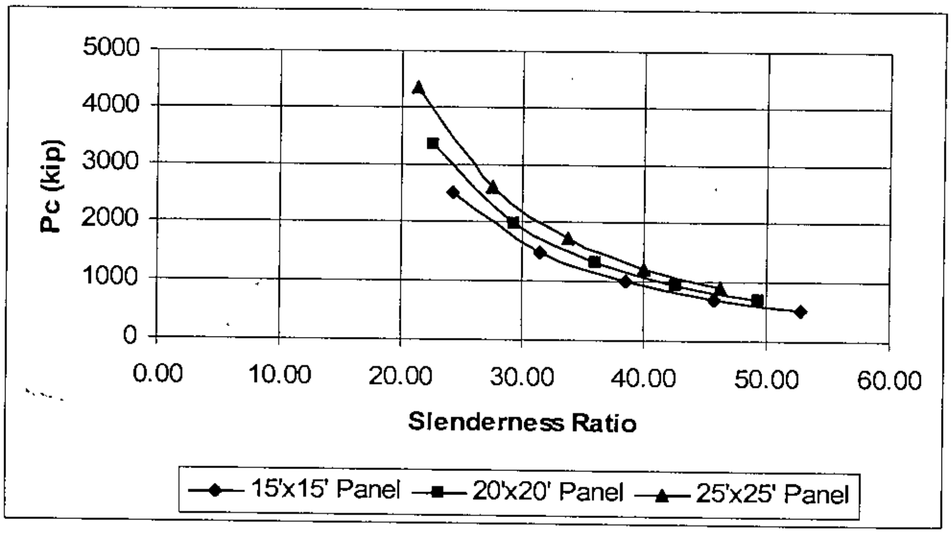


Figure 4.8: Comparison of Critical Buckling Load with Slenderness Ratio in 3 panels of different height for corner columns

To understand the influence of loads in corner column, Fig. 4.9 represents the ratio of P_u/P_c with Slenderness Ratio. When P_u increases about 30% from P_c in the corner column then the designer should check the column for slenderness effect.

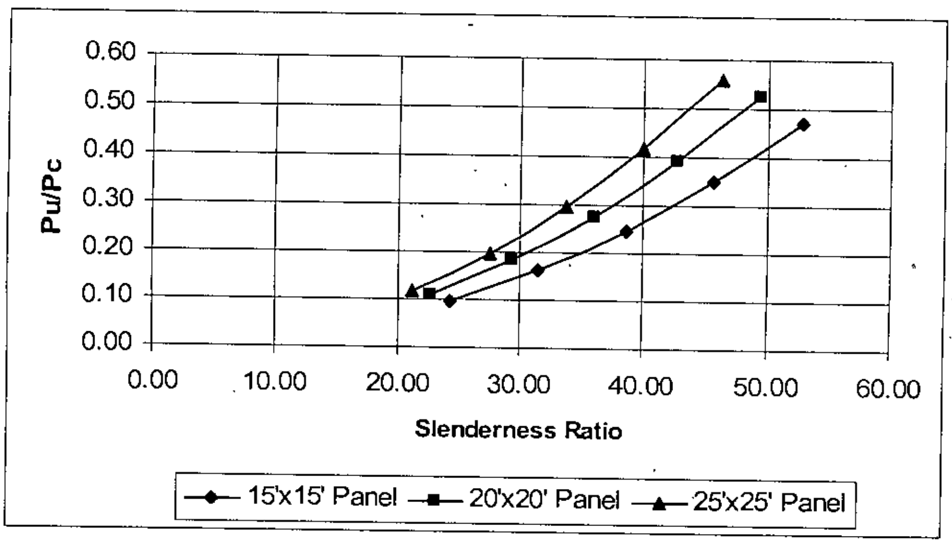


Figure 4.9: Comparison of ratio of Column Design Load to Critical Buckling Load with Slenderness Ratio in 3 panels of different height for corner columns

From Fig. 4.10 this is seen that for 15'x15' slab panel size the δ_{ns} value increased about 43.0% when column length increased from 17.5' to 20.0'. This value increased about 25.0% when column length increased from 15.0' to 17.5'. For 20'x20' slab panel size δ_{ns} increase about 59.0% for same increment (17.5' to 20.0') in column length and for 25'x25' panel size this increase about 72.0%. So, there is a drastically change observed when column length increase from 17.5' to 20.0'. So, for a double height column the designer should need some extra attention. According to ACI Code approach the 17.5' column should be considered as slender column but ETABS does not consider it as slender column as δ_{ns} is lower than 1. This is seen that δ_{ns} for corner column exceeds 1 only for three columns even from slenderness point of view 6 columns should design as slender column. From this statement this is concluded that ACI Code design approach is more conservative in this case.

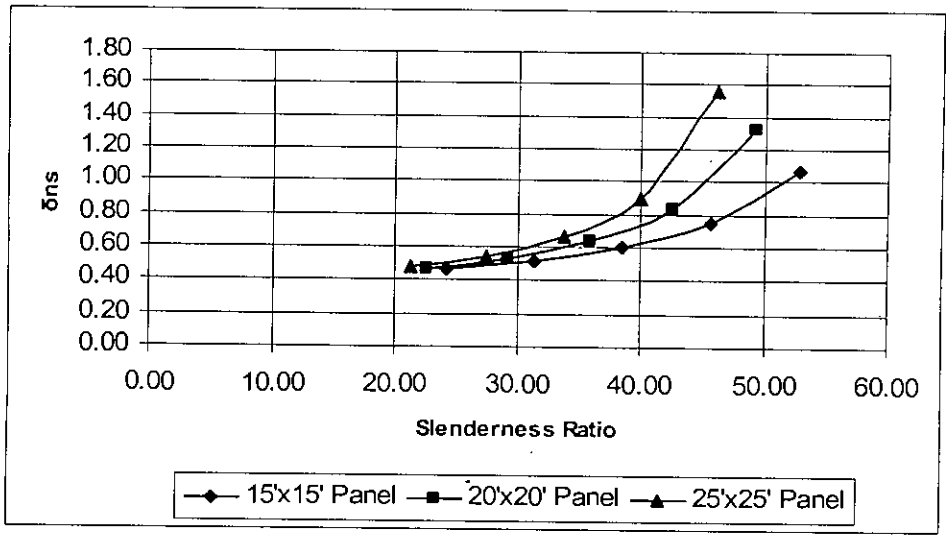


Figure 4.10: Comparison of Nonsway Moment Magnification Factor with Slenderness Ratio in 3 panels of different height for corner columns

4.5 Study on Edge Column for Nonsway Moment Magnification Factor (δ_{ns})

Critical Buckling Load Verses Slenderness Ratio has been drawn in Fig. 4.11. From Appendix-2(A), it is seen that only for three cases the Slenderness Ratio crosses the limit of 34-12M_1/M_2. For 25'x25' panel size it is seen that no Slenderness Ratio is greater than 34-12M_1/M_2 but in 20' column of this panel has slenderness effect. The edge column size in 25'x25' panel is 17"x17". If the column size reduces the steel ratio increases more than 8% in some cases. For this reason the column did not made too slender. The Slenderness Ratio of one column in 15'x15' panel size is greater than 34-

$12M_1/M_2$ but according to ETABS calculation the δ_{ns} value is less than 1. In case of three panels, even the decrement ratio of P_c is more or less same but δ_{ns} increases suddenly.

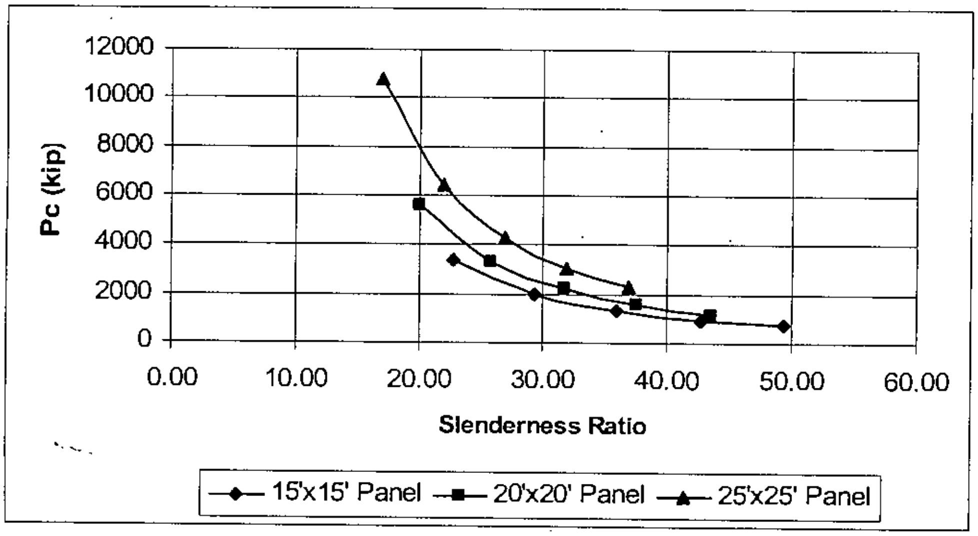


Figure 4.11: Comparison of Critical Buckling Load with Slenderness Ratio in 3 panels of different height for edge columns

To understand the influence of loads in edge column, Fig. 4.12 represents the graph of ratio of column design load to critical buckling load with Slenderness Ratio..When P_u increases about 40% from P_c, then the edge column need extra attention for slenderness consideration.

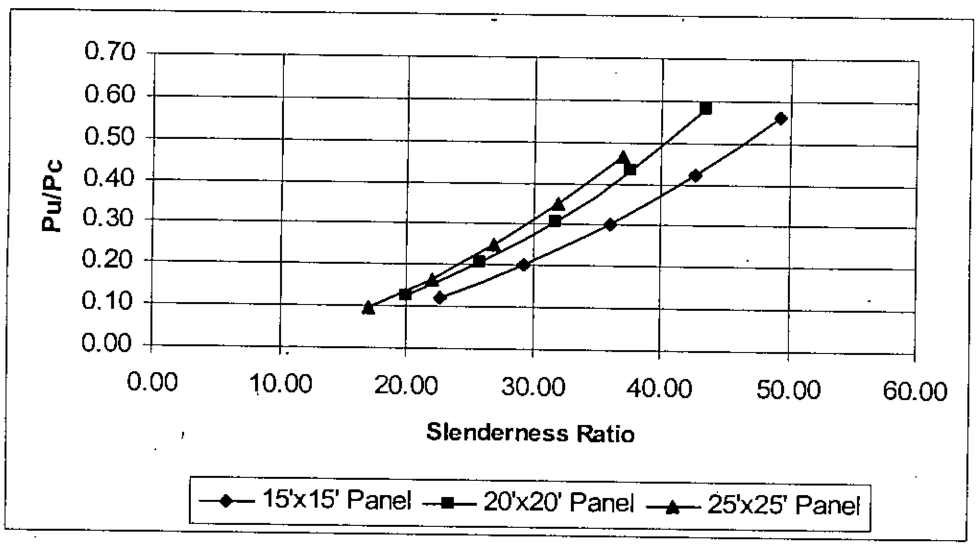


Figure 4.12: Comparison of ratio of Column Design Load to Critical Buckling Load with Slenderness Ratio in 3 panels of different height for edge columns

Figure 4.13 is drawn for δ_{ns} against Slenderness Ratio for edge columns of 3 panels. For edge column in 25'x25' panel the δ_{ns} value is not significant comparing with the other two panels. For 15'x15' panel size the δ_{ns} value increased about 74.0% when column length increased from 17.5' to 20.0'. For 20'x20' slab panel size δ_{ns} increase about 84.0% for column length 17.5' to 20.0' and for 25'x25' panel size this increment is about 36.0%. Like corner column of beam-column frame structure there is a drastically change observed when column length increase from 17.5' to 20.0'. So, for a double height column the designer should need some extra care.

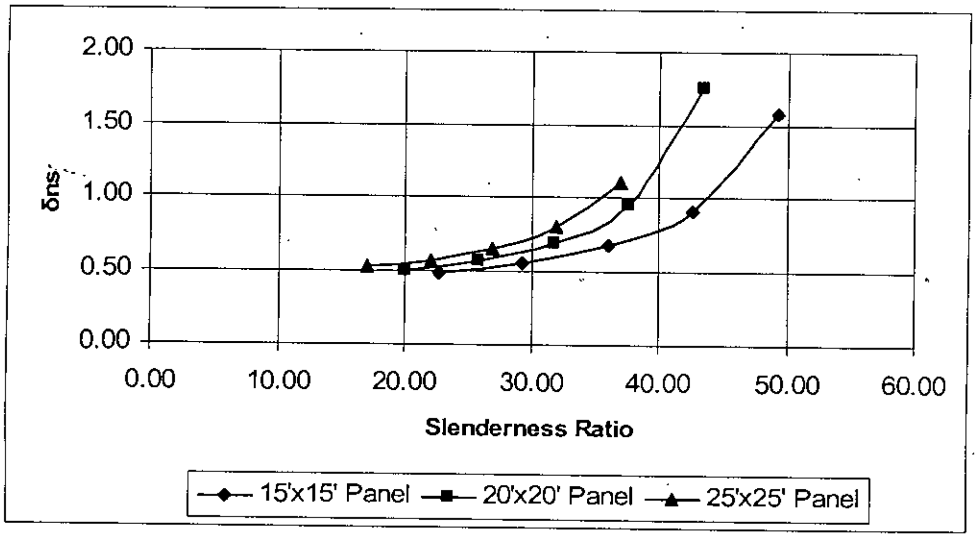


Figure 4.13: Comparison of Nonsway Moment Magnification Factor with Slenderness Ratio in 3 panels of different height for edge columns

4.6 Study on Inner Column for Nonsway Moment Magnification Factor (δ_{ns})

From Excel datasheet, this is seen that for 15 inner columns only three columns show slenderness effect. Figure 4.14 below shows the P_c of different column length with corresponding Slenderness Ratio. The curve for 25'x25' panel is significantly separate in this case. No inner columns of 25'x25' slab panel shows slenderness behavior. If the column dimension reduces in 25'x25' slab panel the steel ratio increases so high that the column fails due to over reinforcement. To keep the steel ratio in tolerable limit the column section increases. For this reason the column of this slab panel does not shows too slenderness. The variation in P_c value with respect to change in column height is fairly uniform. In case of three panels, for column 17.5' to 20.0' the P_c value decrease around 25.0%. But the δ_{ns} value increased about 20.0% to 45.0% which means drastically change in magnification factor. So, it will not be wise to judge for considering slenderness by only variation in Column Buckling Load.

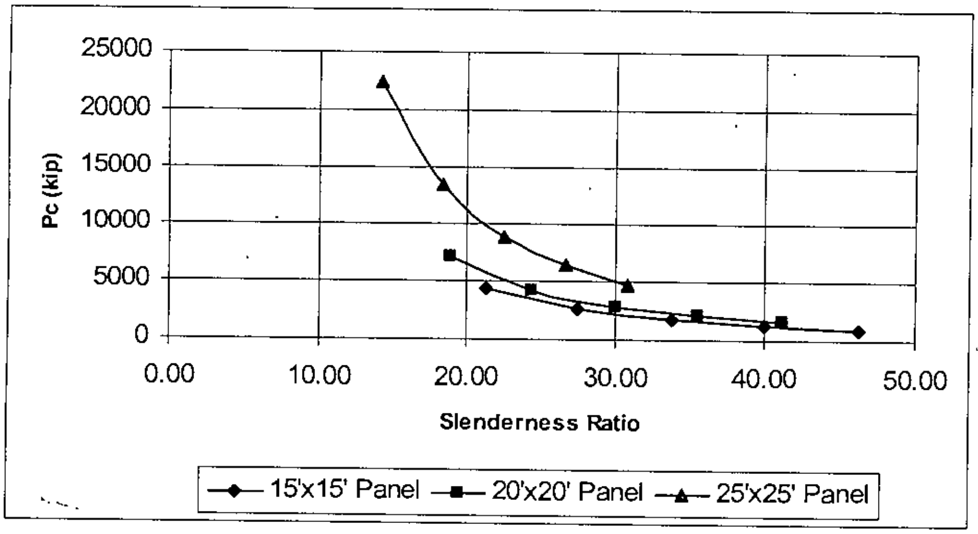


Figure 4.14: Comparison of Critical Buckling Load with Slenderness Ratio in 3 panels of different height for inner columns

To understand the influence of loads in slenderness behavior, Fig. 4.15 has drawn for the ratio of Column Design Load to Critical Buckling Load verses Slenderness Ratio. When P_u increases about 40% than P_c then columns shows slenderness behavior.

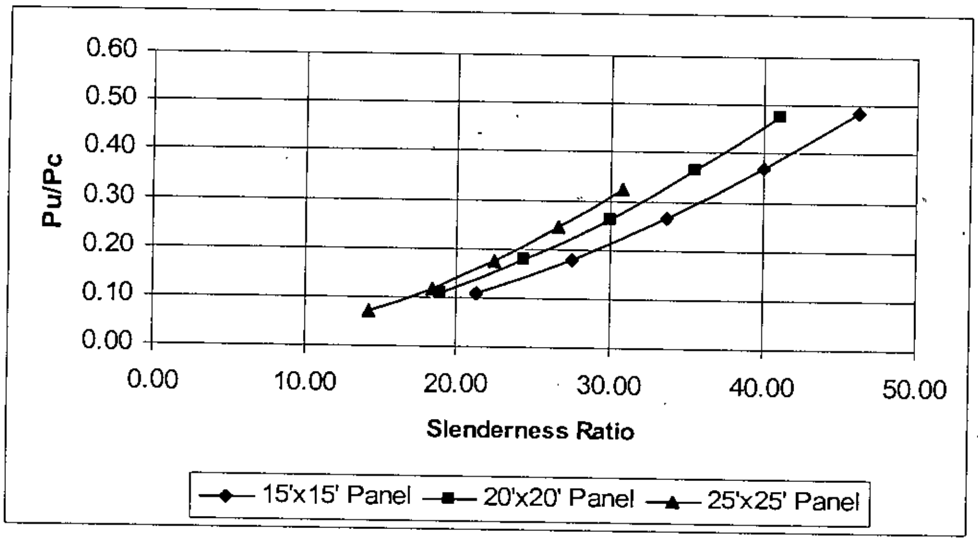


Figure 4.15: Comparison of ratio of Column Design Load to Critical Buckling Load with Slenderness Ratio in 3 panels of different height for inner columns

Figure 4.16 is drawn with Excel datasheet for δ_{ns} with Slenderness Ratio of corresponding columns. For 15'x15' panel size the δ_{ns} value increased about 45.0%

when column length increased from 17.5' to 20.0'. For 20'x20' slab panel size δ_{ns} increase about 43.0% for column length 17.5' to 20.0' and for 25'x25' panel size this increase about 20.0%. Comparing to other two locations of column i.e. corner and edge the value of δ_{ns} is not higher in inner column for all the panel size. This is happen because inner column is near to shear wall than corner and edge column. Shear wall is stiff structure which influences the slenderness of inner column.

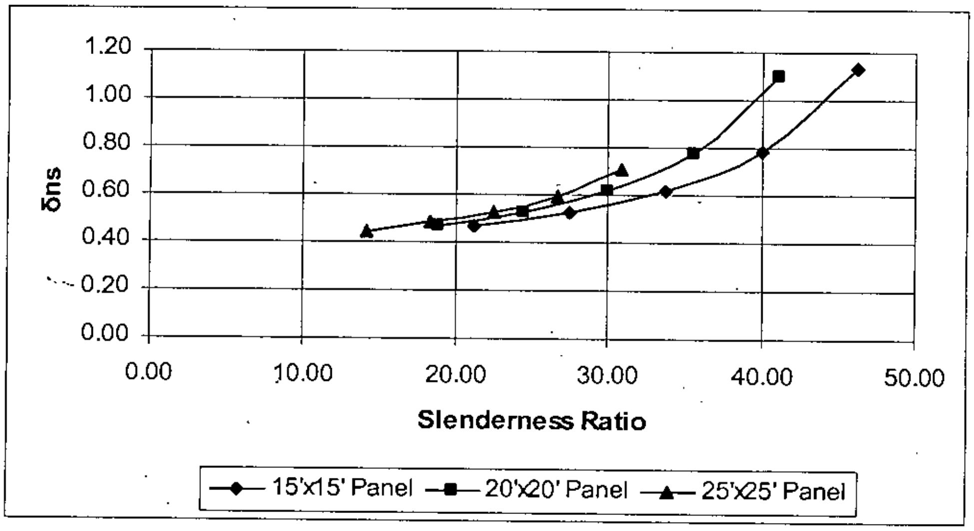


Figure 4.16: Comparison of Nonsway Moment Magnification Factor with Slenderness Ratio in 3 panels of different height for inner columns

4.7 Study on Corner Column for Sway Moment Magnification Factor (δ_s)

To calculate δ_s for 15 columns of 5 different lengths and 3 different slab panels, the load as well as shear force data of Load Combination-18 (DCON18) are taken from ETABS solution and put into Microsoft Excel 2003 datasheet. From ETABS analysis it is seen that the majority of 15 columns are designed for DCON18 load combination which means this is the most critical load case for design of column for sway condition. This should be noted that in ETABS there is no straight forward procedure for design of sway column considering δ_s. For sway frame the magnified moment in ETABS found after P-Δ analysis. Even in software the k value 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 nonsway frame. But this is not correct for Sway Moment Magnification analysis, because for sway frame k value ranges from 1.0 to infinity. There are two processes for calculating k value which has been described in Chapter 2. For this parametric study k value is calculated by Eqs. (2.17) and (2.18). PCACOL use this equation to calculate k value. In this parametric study, Eq. (2.25) is used to find out δ_s. Because in Chapter 3 it is seen that the magnified moment found using Eq. (2.25) have close matched with the magnified moment calculated by P-Δ analysis of ETABS.

The limiting value of Slenderness Ratio checked with the recommended ACI Code guideline. In case of sway frame according to ACI Code when the value of Slenderness Ratio is greater than 22, the column should be treated as slender column. This is observed that for 15 corner columns in beam-column frame structure the value of Slenderness Ratio is always greater than 22.0. So, the columns which are neglected due to lower value (less than 1.0) of δ_{ns} must be consider carefully for sway moment effect even the value is low. The variation in δ_s values of different column in a single panel is regular. There is no radically jump in δ_s values which is observed in δ_{ns} in some case. Among these 15 columns Stability Index (Q) is lower than 5.0% for 5 columns. But from Fig. 4.17 it is seen that even they are in nonsway frame but have slenderness effect. For 25'x25' slab panel and 20' column, the Slenderness Ratio is near about 100. That means this column is a very slender and need second order computer analysis. So, this could be concluded that sway slenderness effect occurred in all cases for corner column.

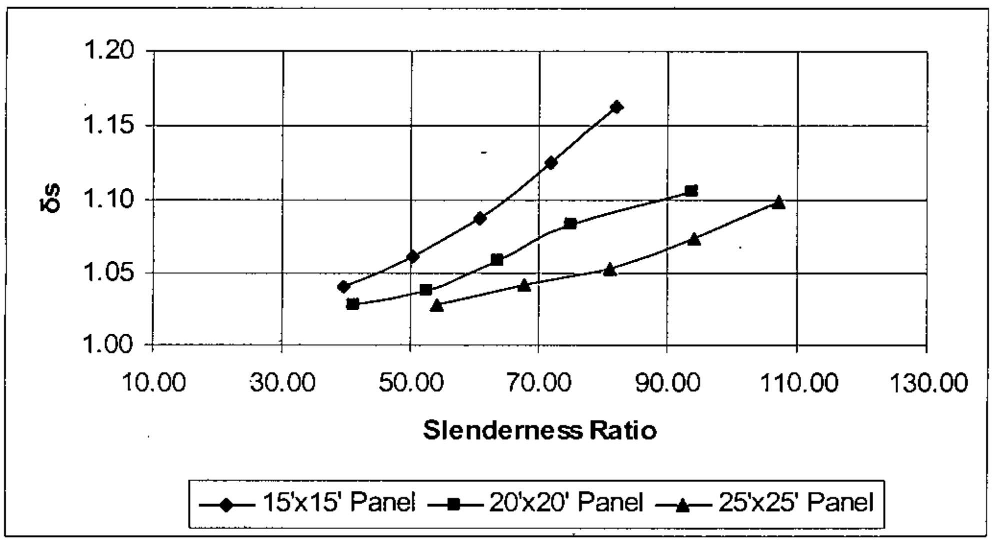


Figure 4.17: Comparison of Sway Moment Magnification Factor with Slenderness Ratio in 3 panels of different height for corner columns

4.8 Study on Edge Column for Sway Moment Magnification Factor (δ_s)

Figure 4.18 represents the δ_s of edge columns are plotted with Slenderness Ratio of 15 columns in 3 panels. In edge column all the members of different height of different panel have sway effect like corner columns. Column height of 20' in 25'x25' panel, Slenderness Ratio is near about 100.0. So, the column needs extra attention at the time of design. From comparing with Fig. 4.17 and 4.18 it is seen that the value of δ_s varies from 1.0 to 1.18. But for both corner and edge column in slab panel 25'x25', the δ_s value is more noticeable. For 25'x25' panel the Slenderness Ratio values extended from other two panels.

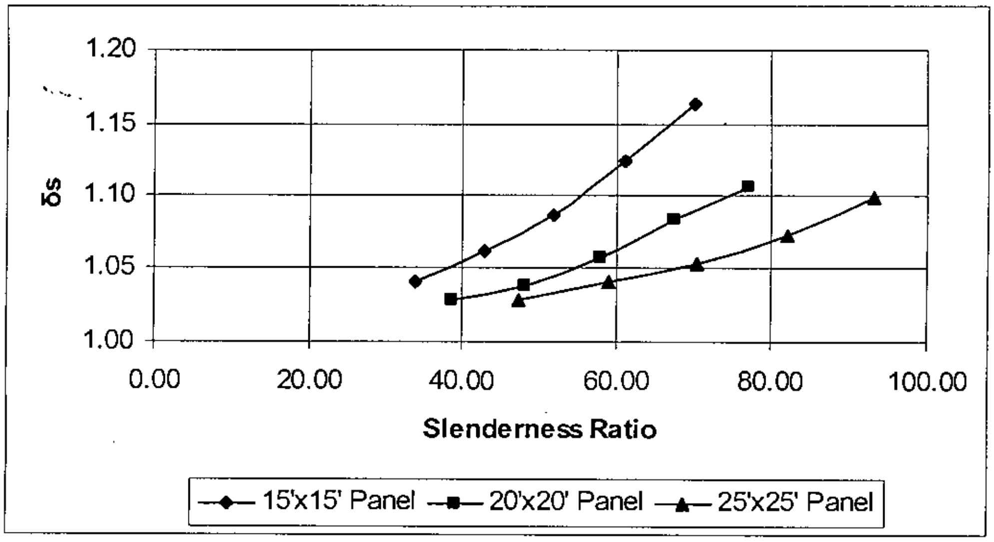


Figure 4.18: Comparison of Sway Moment Magnification Factor with Slenderness Ratio in 3 panels of different height for edge columns

4.9 Study on Inner Column for Sway Moment Magnification Factor (δ_s)

In Fig. 4.19 δ_s of inner columns are plotted against Slenderness Ratio for inner column using Microsoft Excel. The variation in δ_s value is regular in column length increment. No Slenderness Ratio exceeds 100.0 but every column has slenderness effect. The increment of δ_s in column of 15'x15' slab panel is steeper than other two panels. From the preceding description of δ_{ns} it was concluded that slenderness effect in inner column is not very much noticeable comparing with corner and edge column. This statement is also true for δ_s analysis of inner column. The reason is same that is described in δ_{ns} calculation. Shear walls influences the effect of slenderness in inner column because inner column is near to shear wall than corner and edge column.

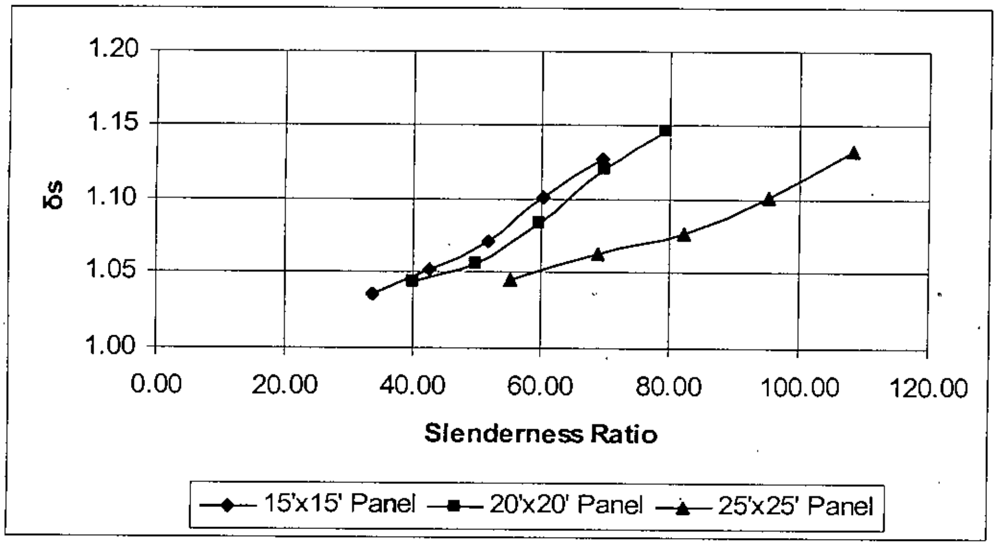


Figure 4.19: Comparison of Sway Moment Magnification Factor with Slenderness Ratio in 3 panels of different height for inner columns

4.10 Model Development of Flat-Plate Structure for Parametric Study

The preliminary steps of model generation has described in Chapter 3 and graphically presented in Appendix-A.1. In this parametric study 15 models are generated for flat-plate structure with a tube shape shear wall in core of the structure. 5 models are 15'x15' slab panel sizes, other 5 models are 20'x20' slab panel sizes and another 5 models are 25'x25' slab panel sizes. Every floor consists of three panels in each direction. The foundations for columns and shear walls are assigned as fixed support. For each model 1 corner column, 1 edge column and 1 inner column in ground floor has been selected for this parametric study. So, among these 15 flat-plates frame structure total 45 ground floor columns have been considered for slender column behavior analysis. All the models are of 10 stories. The ground floor is increased from 10' to 20' height with an increment of 2.5'. The other storey height is 10' and kept unchanged in all structures. The column dimensions are kept identical as beam-column frame structure. The periphery beam is matched with beam size of beam-column frame structure. All the columns and beams are satisfied for most critical condition. The floor slabs are 8.5" thick which confirms the minimum thickness for flat slab with periphery beam. No drop panels or column capitals are provided in flat-plate model. The imposed dead load and live load is 60 psf. The breakdown of dead load that consider in Chapter 3 kept unchanged for this parametric study. The dead load and live load pattern in roof is same like other floor. All the floors have been assigned as rigid diaphragm to reduce relative displacement. All the floors and shear wall have been assigned as auto mesh using 4' by 4' meshing in the boundary line of slab and beam. The program automatically calculates the earthquake and wind force according to load parameters input. The earthquake load and wind load data are input as same as described in Chapter 3. The special seismic force is not considered in calculation and the mass source is from all dead loads only.

Figure 4.20 represents the plan view of ground floor of one model consisting 15'x15' slab panel. Total bay width in either direction is 45'. The tube shape shear wall which is placed in core of the structure is 5'x5' in dimension and 9" thick. The shear wall is connected with main frame of the structure by grade beam in all models. The grade beam is 20"x12" in depth. For all models the grade beam dimension is kept identical.

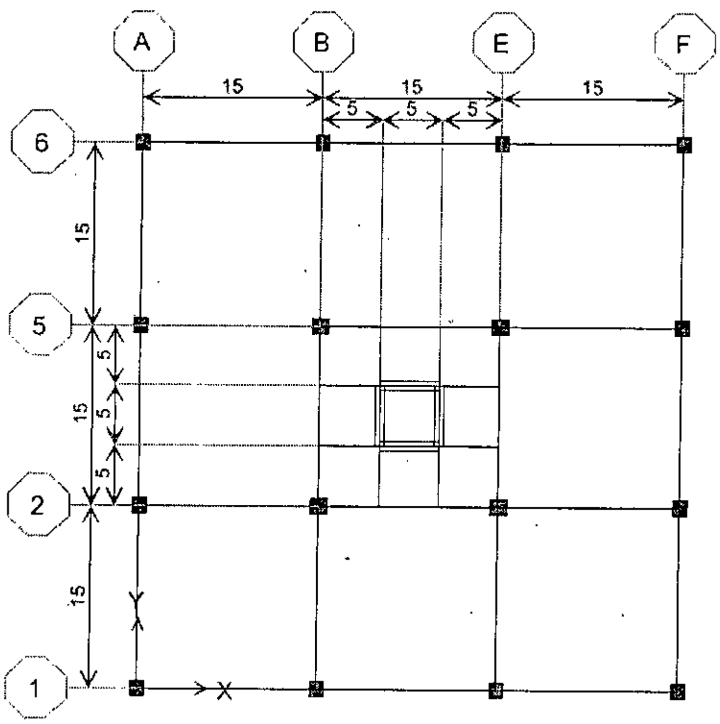


Figure 4.20: Plan view of ground floor in 15'x15' floor panel model

Figure 4.21 represents the plan view of ground floor of one model consisting 20'x20' slab panel. The total bay width in either direction is 60'. The tube shape shear wall which is placed in core of the structure is 10'x10' in dimension and 9" thick.

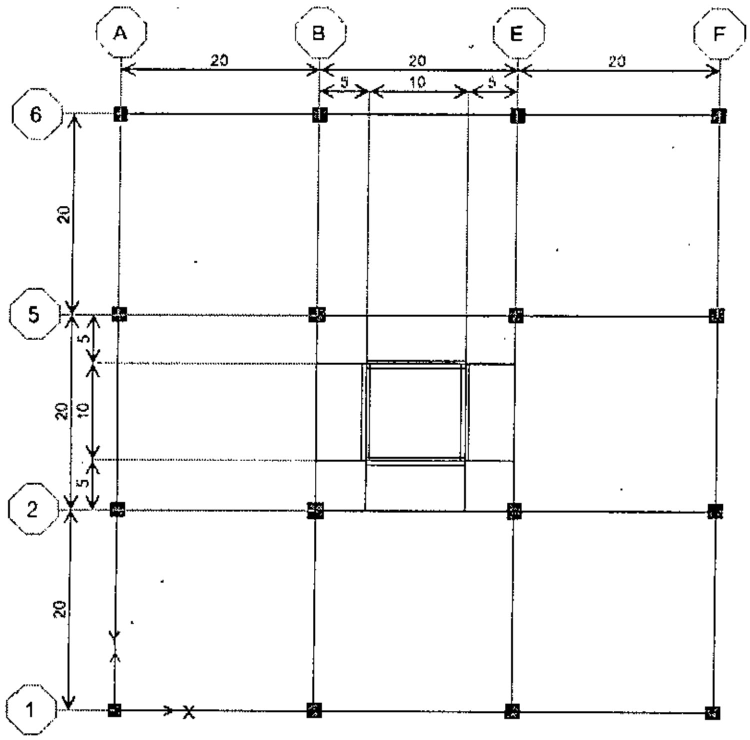


Figure 4.21: Plan view of ground floor in 20'x20' floor panel model

Figure 4.22 represent the plan view of ground floor of one model consisting 25'x25' slab panel. The total bay width in either direction is 75'. The tube shape shear wall which is placed in core of the structure is 10'x10' in dimension and 9" thick.

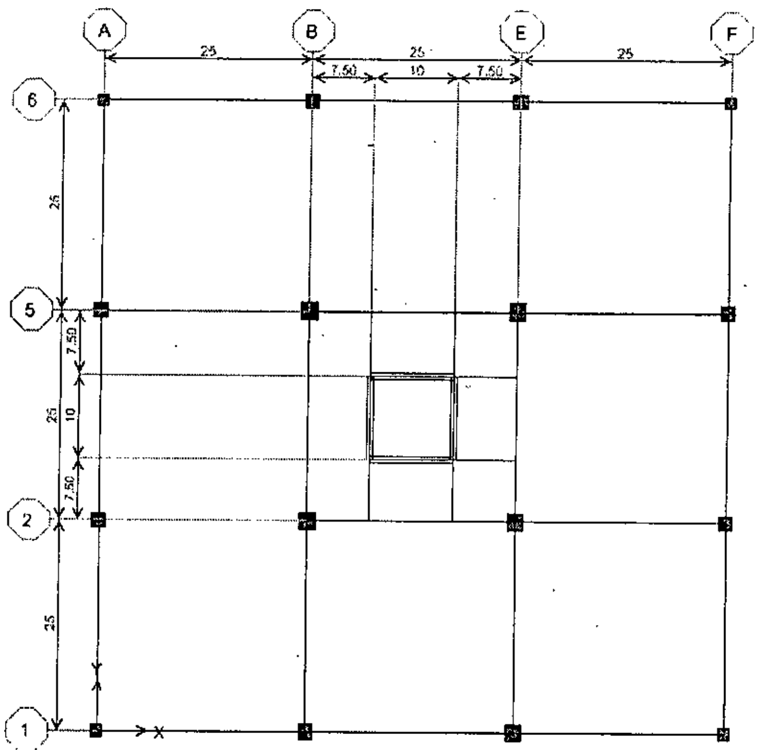


Figure 4.22: Plan view of ground floor in 25'x25' floor panel model

Table 4.2 shows different cases of studies. The case named according to slab panel size and length of ground floor column. Periphery beam dimension kept same for all model because of consistency in analysis. After analysis each structure by ETABS, the moment values of column end and design load for each targeted columns are extracted from software and made a database using Microsoft Excel. All the parameters are calculated using Excel datasheet and crosschecked each time with ETABS design output. Using the Excel output the graphs for selected parameters are drawn. For flat-plate structure, both δ_{ns} and δ_s are determined and for each case relative graphs are plotted by using Excel to describe the behavior of slender column in different conditions.

Table 4.2: Parameters for flat-plate structures

Sn.	**Floor Panel Size (Feet)**	**Column Positions**	**Case No. = Column Length @ Ground Level (Feet)**	**Column Dimension (Inches)**	**Periphery Beam Dimcnsion (Inches)**
1	15'x15'	Corner Column	Case 1.0 = 10.0'	14"x14"	18"x10"
			Case 2.0 = 12.5'		
			Case 3.0 = 15.0'		
			Case 4.0 = 17.5'		
			Case 5.0 = 20.0'		
		Edge Column	Case 1.0 = 10.0'	15"x15"	
			Case 2.0 = 12.5'		
			Case 3.0 = 15.0'		
			Case 4.0 = 17.5'		
			Case 5.0 = 20.0'		
		Inner Column	Case 1.0 = 10.0'	16"x16"	
			Case 2.0 = 12.5'		
			Case 3.0 = 15.0'		
			Case 4.0 = 17.5'		
			Case 5.0 = 20.0'		

Table 4.2 (Contd.)

2	20'x20'	Corner Column	Case 1.1 = 10.0'	15"x15"	18"x10"
			Case 2.1 = 12.5'		
			Case 3.1 = 15.0'		
			Case 4.1 = 17.5'		
			Case 5.1 = 20.0'		
		Edge Column	Case 1.1 = 10.0'	17"x17"	
			Case 2.1 = 12.5'		
			Case 3.1 = 15.0'		
			Case 4.1 = 17.5'		
			Case 5.1 = 20.0'		
		Inner Column	Case 1.1 = 10.0'	18"x18"	
			Case 2.1 = 12.5'		
			Case 3.1 = 15.0'		
			Case 4.1 = 17.5'		
			Case 5.1 = 20.0'		
3	25'x25'	Corner Column	Case 1.2 = 10.0'	16"x16"	18"x10"
			Case 2.2 = 12.5'		
			Case 3.2 = 15.0'		
			Case 4.2 = 17.5'		
			Case 5.2 = 20.0'		
		Edge Column	Case 1.2 = 10.0'	20"x20"	
			Case 2.2 = 12.5'		
			Case 3.2 = 15.0'		
			Case 4.2 = 17.5'		
			Case 5.2 = 20.0'		
		Inner Column	Case 1.2 = 10.0'	24"x24"	
			Case 2.2 = 12.5'		
			Case 3.2 = 15.0'		
			Case 4.2 = 17.5'		
			Case 5.2 = 20.0'		

4.11 Study on Corner Column for Nonsway Moment Magnification Factor (δ_{ns})

To calculate δ_{ns} for 15 columns of 5 different lengths and 3 different slab panels, the moment as well as load data are taken from ETABS solution and put into Microsoft Excel datasheet. The limiting value of Slenderness Ratio has been checked with the recommended ACI Code guideline. According to ACI Code when the value of Slenderness Ratio is greater than 34-12M_1/M_2, the column should be treated as slender column. M_1 and M_2 is the top and bottom end moment of a column respectively and M_1 is less than M_2. The maximum value of 34-12M_1/M_2 that is limiting ACI Code is 40.0. From the Fig. 4.23 below it is seen that among 15 columns Slenderness Ratio are above or equals to 40 in 8 columns. But, from Appendix-B.1 it is seen that only 5 columns are experience slenderness effect, which means δ_{ns} is greater than 1. In case of three panels, for column 17.5' to 20.0' the P_c value decrease around 25.0%. But the δ_{ns} value increased about 67.0% to 244.0%.

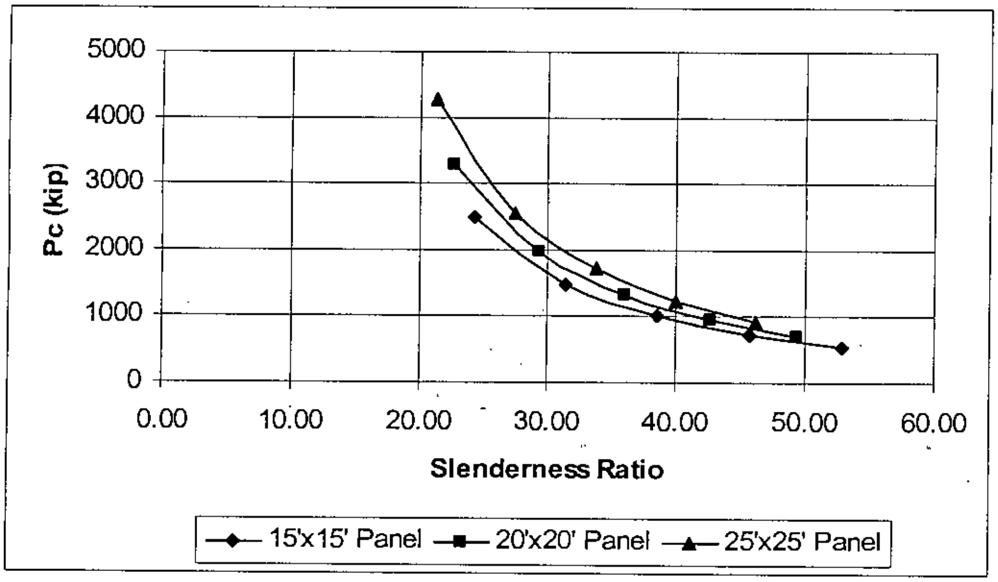


Figure 4.23: Comparison of Critical Buckling Load with Slenderness Ratio in 3 panels of different height for corner columns

To understand the influence of loads in slender column, Fig. 4.24 represents the ratio of Column Design Load to Critical Buckling Load. As the P_u increases about 40% from P_c then the corner column shows slenderness behavior.

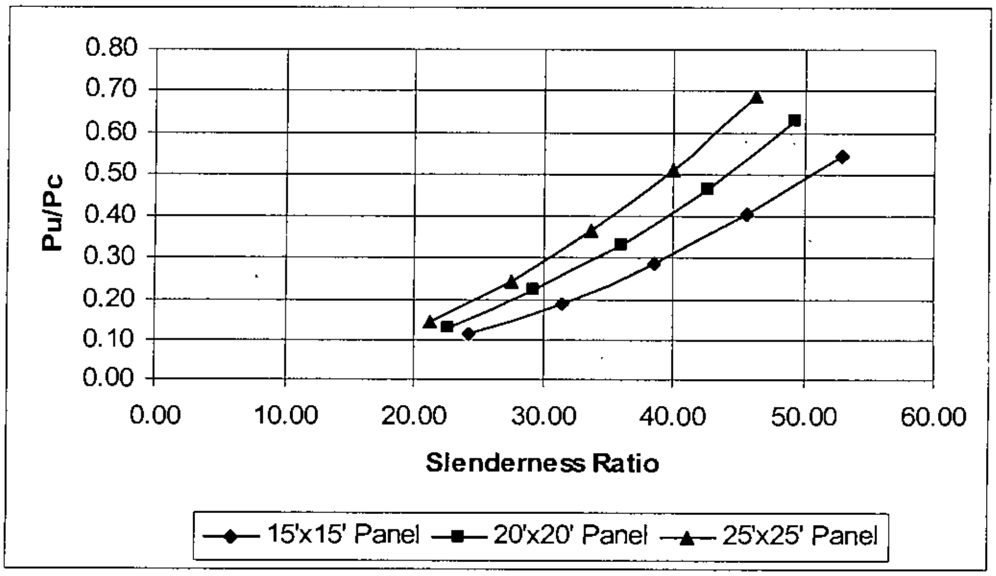


Figure 4.24: Comparison of ratio of Column Design Load to Critical Buckling Load with Slenderness Ratio in 3 panels of different height for corner columns

The noticeable thing is that for 15'x15' slab panel size δ_{ns} value increased about 67.0% when column length increased from 17.5' to 20.0'. δ_{ns} increased about 25% when column length increased from 15.0' to 17.5'. This value increased about 43% for same column height increment of beam-column structure. For 20'x20' slab panel size δ_{ns} increase about 132.0% for same increment (17.5' to 20.0') for column length and for 25'x25' panel size this increase is about 244.0%. So, there is a drastically change observed when column length increase from 17.5' to 20.0' and change in frame type. So, in flat-plate frame structure and to design a double height column the designer should need some extra attention. According to ACI Code guideline the 17.5' column should consider as slender column but for 15'x15' slab panel ETABS does not consider it as slender column as δ_{ns} is lower than 1. From this statement this is understand that ACI Code design approach is more conservative which is also confirmed in beam-column frame structure. From Fig. 4.27 below it is seen that the curve of three panels are more divertive from each other when the column length increases. The corner column of large panel is in esteemed situation. Even the Slenderness Ratio is not so high from the preceding one but δ_{ns} value increases a lot. Almost same situation observed in case of 20'x20' panel. Moderate change is caused in case of 15'x15' slab panel with respect to other slab size. A corner column of 25'x25' panel needs more attention then for a same column in 15'x15' panel.

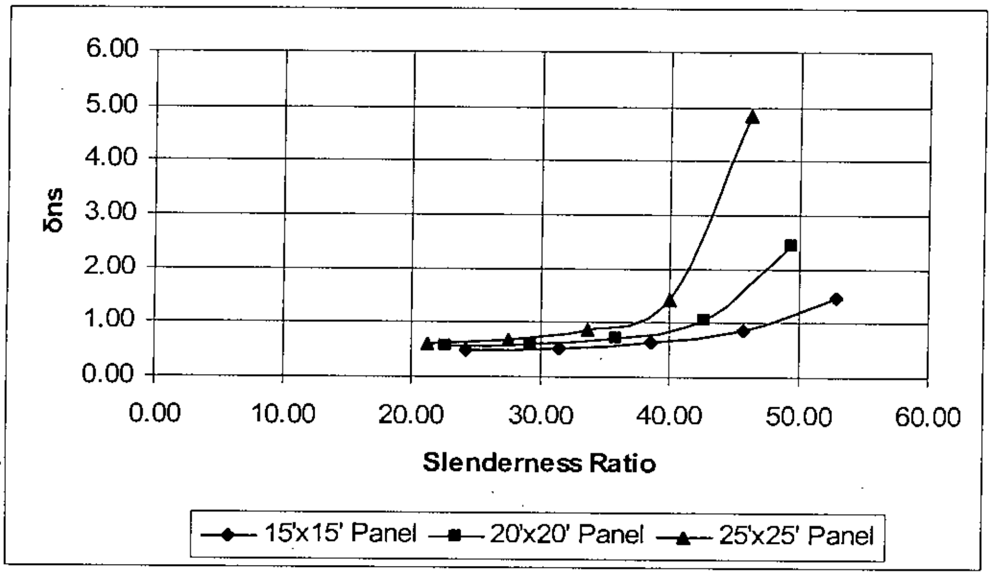


Figure 4.25: Comparison of Nonsway Moment Magnification Factor with Slenderness Ratio in 3 panels of different height for corner columns

4.12 Study on Edge Column for Nonsway Moment Magnification Factor (δ_{ns})

P_c verses Slenderness Ratio is drawn in Fig. 4.26. In case of three panels, for column 17.5' to 20.0' the P_c value decrease around 25.0%. But the δ_{ns} value increased about 58.0% to 164.0%. In flat-plate structure C_m is influenced by member end moment direction. The edge column shows single curvature in every case of this study. By Eq. (2.12) the moment component is positive in this case. This is the reason for high C_m value. C_m value ranges from 0.53 to 0.91 for edge column in flat-plate structures.

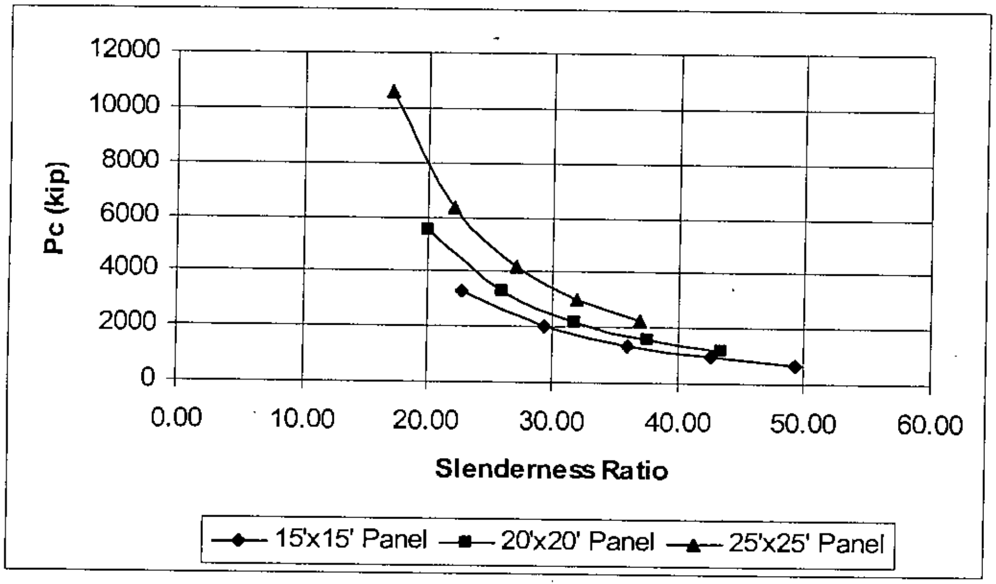


Figure 4.26: Comparison of Critical Buckling Load with Slenderness Ratio in 3 panels of different height for edge columns

Figure 4.27 represents the ratio of Column Design Load to Critical Load of Column with Slenderness Ratio. When the P_u increases about 40% from P_c the edge column shows slender behavior.

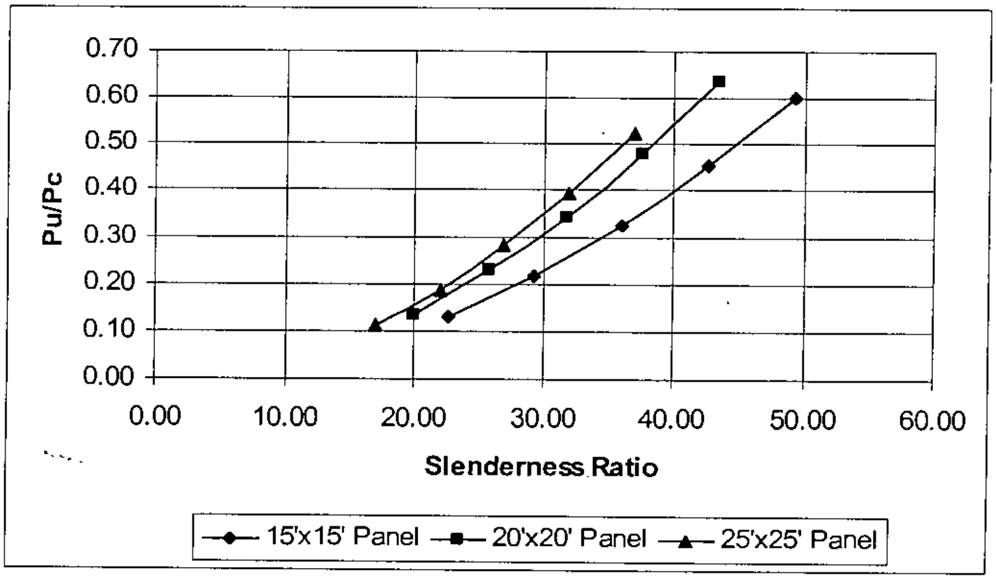


Figure 4.27: Comparison of ratio of Column Design Load to Critical Buckling Load with Slenderness Ratio in 3 panels of different height for edge columns

From Fig. 4.28 it is seen that 10 column experience slenderness effect. δ_{ns} ranges from 1.02 to 5.84. The δ_{ns} of corner column for 12.5' height in 15'x15' panel is 1.14. Four columns of different height out of five columns of 15'x15' panel experience slenderness effect. The same situation occurred in 20'x20' panel size. Two columns of 25'x25' slab panel size experience slenderness effect. For 20'x20' panel the extent of δ_{ns} is remarkable. The δ_{ns} value increases about 164.0% when column length increases from 17.5' to 20.0'. The δ_{ns} value increases about 61.0% when column length increases from 15.0' to 17.5'. in this case one thing is noticeable that for 25'x25' panel and 17.5' column the Slenderness Ratio value is less than 34-12M_1/M_2, which means no need to consider slenderness in this case but ETABS shows δ_{ns} is equals to 1.12. And for 17.5' column the δ_{ns} value is increased about 32.0% from 15.0' column. So, this should not be negligible. From Fig. 4.28 it is seen that the curve for 20'x20' slab panel and for 15'x15' slab panel is much extended and get steeper when it reaches from 17.5' to 20.0' then curve of 25'x25' slab panels.

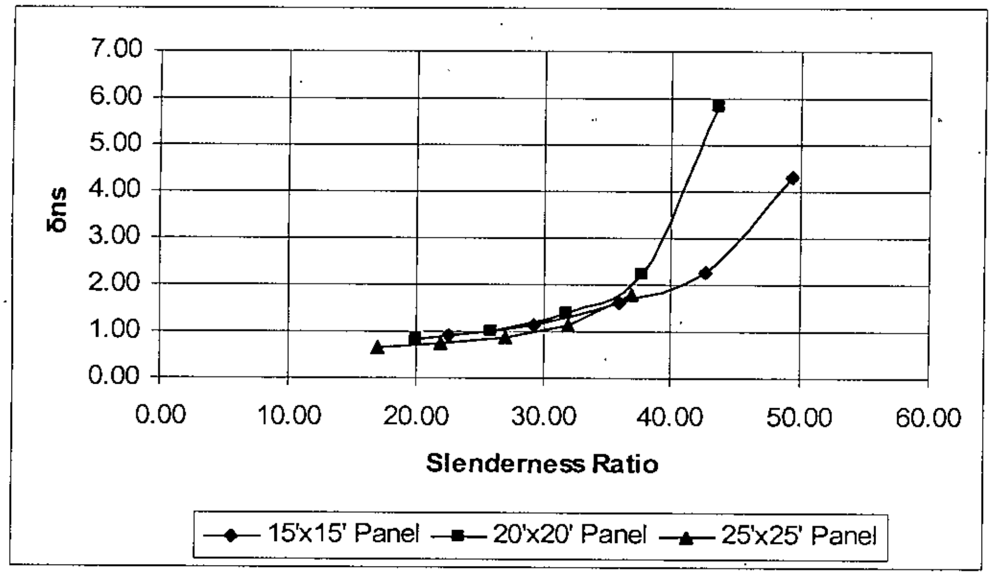


Figure 4.28: Comparison of Nonsway Moment Magnification Factor with Slenderness Ratio in 3 panels of different height for edge columns

4.13 Study on Inner Column for Nonsway Moment Magnification Factor (δ_{ns})

The graph is drawn from Excel datasheet represents in Fig. 4.29. This is seen that only two columns show slenderness effect. The Fig. 4.29 below shows the P_c of different column length with corresponding Slenderness Ratio. Only three Slenderness Ratio value is greater than 34-12M_1/M_2 which means only three column length experience slenderness effect. In case of three panels, for column 17.5' to 20.0' the P_c value decrease around 25.0%. But the δ_{ns} value increased about 38.0% to 44.0%. By ETABS for inner column of flat-plate panel the clear distance is equals to center to center distance of column. ETABS can not consider the slab thickness for determining l_c if there is no beam presents. For corner and edge column the presence of periphery beam is consider for determining l_c, but in case of inner column this is not happened. This is one of the major differences from beam column frame structure and flat plate frame structure analysis using ETABS. But the noticeable thing is even the l_c of column is increased due to not considering the slab thickness but the slenderness value does not cross 1.0 for 25'x25' panel. The value of l_c can be determined by overwrite but that does not done due to use the automatic feature of ETABS as much as possible. this effects also explain the effect of shear walls in inner columns.

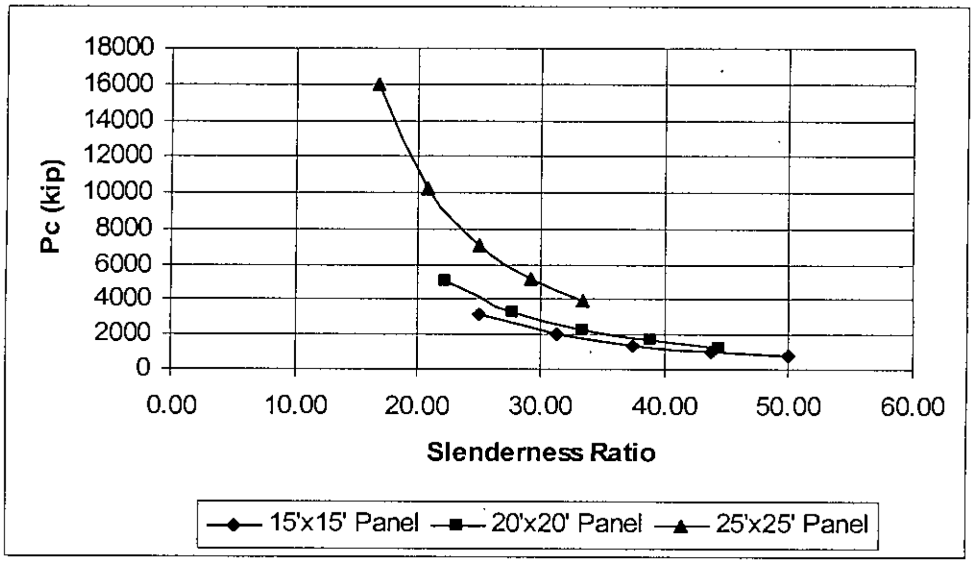


Figure 4.29: Comparison of Critical Buckling Load with Slenderness Ratio in 3 panels of different height for inner columns

To understand the influence of loads in slenderness behavior Fig. 4.30 represents the ratio of column design load to critical buckling load verses Slenderness Ratio. When P_u increases about 40% than P_c then columns shows slenderness behavior.

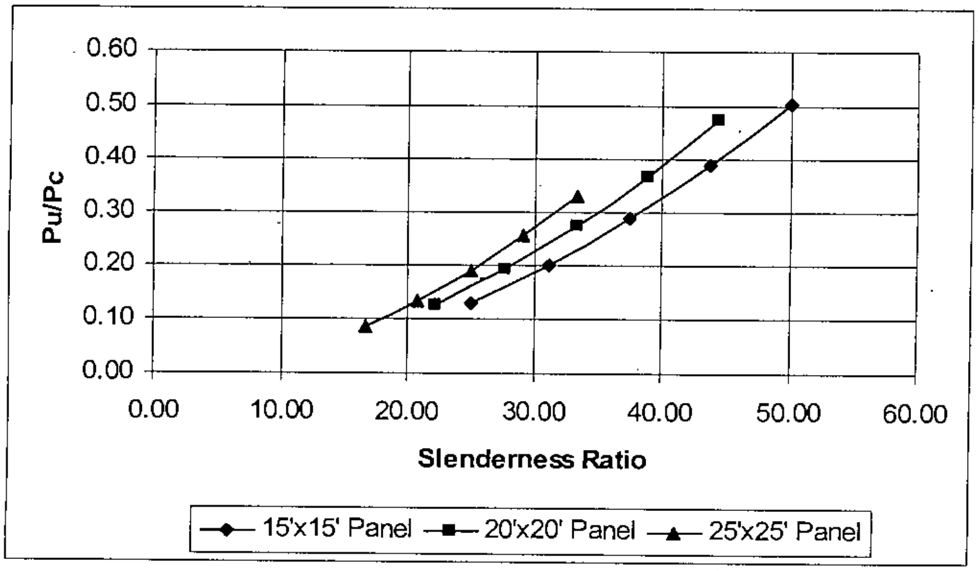


Figure 4.30: Comparison of ratio of Column Design Load to Critical Buckling Load with Slenderness Ratio in 3 panels of different height for inner columns

For inner column in 25'x25' panel the δ_{ns} value is not significant comparing with the other two panels. For 15'x15' panel size the δ_{ns} value increased about 44.0% when column length increased from 17.5' to 20.0'. For 20'x20' slab panel size δ_{ns} increase about 38.0% for column length 17.5' to 20.0' and for 25'x25' panel size this increase about 16.0%. According to ACI Code design approach the inner column of 25'x25' slab panel is not consider as slender column and from ETABS calculation the value of δ_{ns} is become less than 1.0 which is also checked by manual calculation. So, for larger panel size the inner column is not venerable by slenderness effect if shear wall is present in core of structure. Comparing to other two location of column i.e. corner and edge the value of δ_{ns} is not much higher in inner column for all the panel size. So, like beam-column frame structure, the inner column slenderness behavior is not much considerable in flat-plate frame structure due to presence of shear wall in structure.

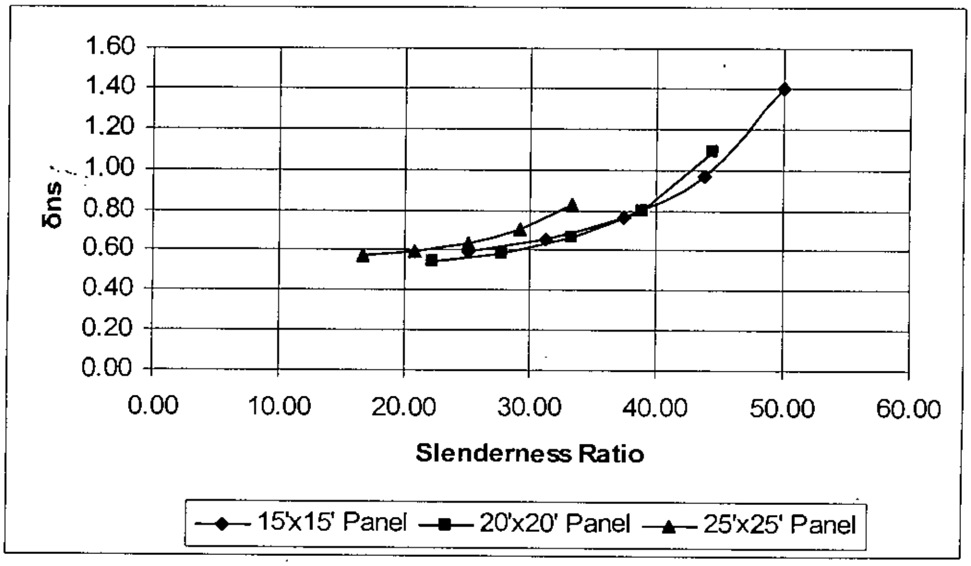


Figure 4.31: Comparison of Nonsway Moment Magnification Factor with Slenderness Ratio in 3 panels of different height for inner columns

4.14 Study on Corner Column for Sway Moment Magnification Factor (δ_s)

To calculate δ_s for 15 columns of 5 different lengths and 3 different slab panels, the load as well as shear force data of Load Combination-18 (DCON18) are taken from ETABS solution and put into Microsoft Excel datasheet. The limiting value of Slenderness Ratio checked with the recommended ACI Code guideline. In case of sway frame, according to ACI Code when the value of Slenderness Ratio is greater than 22 the column should be treated as slender column. This is observed that for 15 corner columns in flat-plate frame structure the value of Slenderness Ratio is always greater than 22.0. So, the columns which are neglected due to lower value (less than 1.0) of δ_{ns} must be consider carefully for sway moment effect even the value is low. The variation in δ_s values of different column in a single panel is regular. Among these 15 columns Stability Index is always equals to or greater than 5.0%. So, flat-plate structure is less stiff than beam-column structure in ground level. For 25'x25' slab panel and 20' column, the Slenderness Ratio is greater than 100. That means this column is a very slender member and need second order computer analysis. So, this could conclude that for flat-plate structure slenderness effect occurred in all cases for corner column of various extents and the effect is more vulnerable than beam-column.

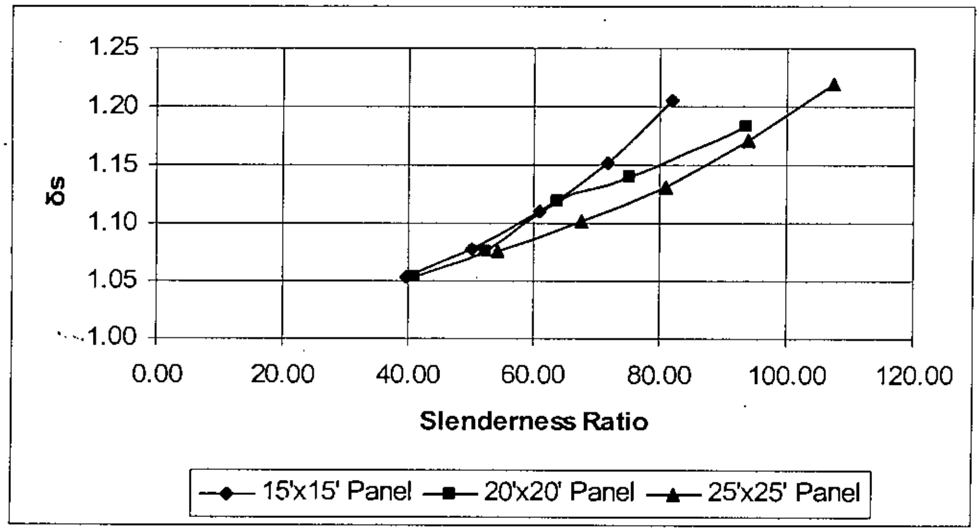


Figure 4.32: Comparison of Sway Moment Magnification Factor with Slenderness Ratio in 3 panels of different height for corner columns

4.15 Study on Edge Column for Sway Moment Magnification Factor (δ_s)

Figure 4.33 represents the δ_s with Slenderness Ratio of 15 columns in 3 panels. In edge column all the members of different height of different panel have sway effect like corner columns. Column height of 20' in 25'x25' panel, Slenderness Ratio is more than 100.0. According to ACI Code, the column needs second order computer analysis if k is greater than 100.0. From comparing with Fig. 4.32 and 4.33 it is seen that the value of δ_s is varies from 1.03 to 1.22. But for both corner and edge column in slab panel 25'x25' the Slenderness Ratio is more noticeable because increment in δ_s is not so high but Slenderness Ratio increase a lot.

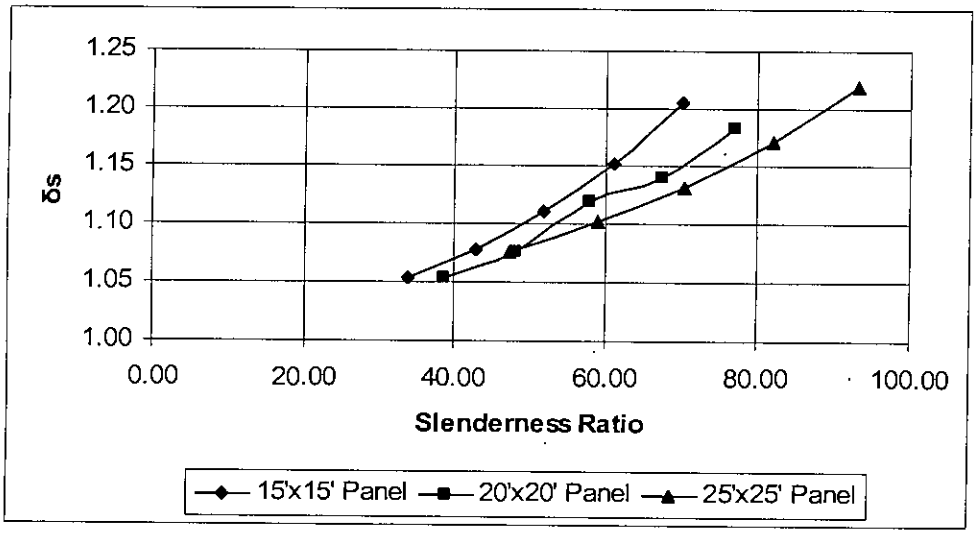


Figure 4.33: Comparison of Sway Moment Magnification Factor with Slenderness Ratio in 3 panels of different height for edge columns

4.16 Study on Inner Column for Sway Moment Magnification Factor (δ_s)

The value of δ_s for inner column is less then corner and edge column. The difference in Unbraced Length from corner and edge column is stated earlier. Total 15 columns experience the slenderness effect of various extents. The curves are very regular in shape. Four frames are nonsway frame i.e. Q is less than 0.05. Two of them are in 15'x15' panel and other two are in 20'x20' panel. This Q values less than 0.05 is presents for low height column. Increment in Slenderness Ratio is larger in larger span but δ_s are larger in smaller span.

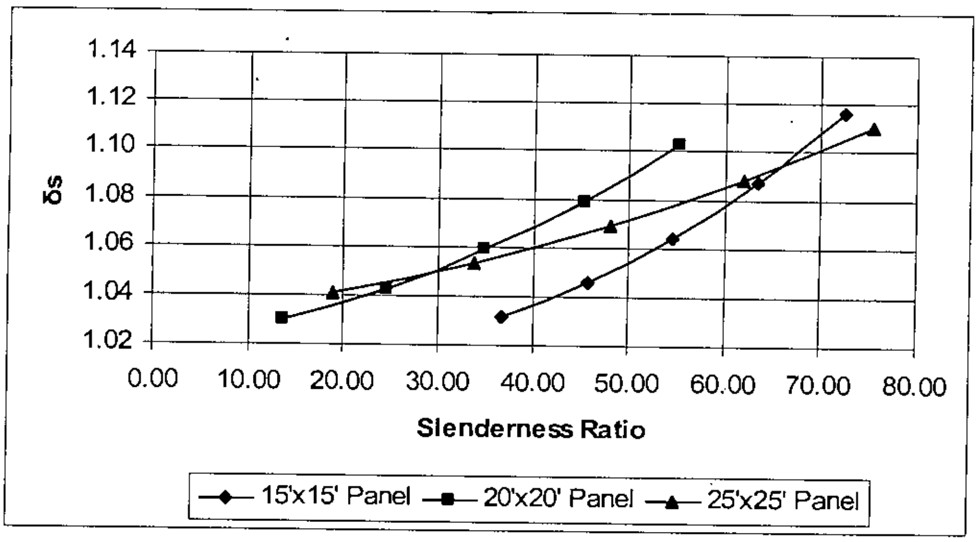


Figure 4.34: Comparison of Sway Moment Magnification Factor in 3 panels of different height for inner columns

4.17 Checking of ACI Code Guideline for Neglecting Nonsway Slenderness Effects

The first thing that a designer should check whether there is any slenderness effect in column is present or not. According to ACI Code, for compression member in nonsway 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 greater than 40. So, if $(kl_u/r)/(34\text{-}12M_1/M_2)$ is less than 1.0 than the column will not show any slender effect. Figure 4.35 is plotted against Nonsway Moment Magnification Factor against $(kl_u/r)/(34\text{-}12M_1/M_2)$ for beam-column frame structure and Fig. 4.36 is plotted for flat-plate frame structure . Both the value's limit is 1.0. From Fig. 4.35 this is seen that 9 columns and in Fig. 4.36 for thirteen columns, it is exceeded the limit bounded by ACI Code. The column length greater than 20' is at risk for high slenderness effect. Column length ranges from 17.5' to 20' should be substantially considered as slender column. In Fig. 4.35 only one column's δ_{ns} value is greater than 1 but the $(kl_u/r)/(34\text{-}12M_1/M_2)$ is less than 1 and for 2 columns the ratio of $(kl_u/r)/(34\text{-}12M_1/M_2)$ is greater than 1 but δ_{ns} value is less than 1. In Fig. 4.36 only one column's δ_{ns} value is greater than 1 but the $(kl_u/r)/(34\text{-}12M_1/M_2)$ is less than 1 and for 1 column the ratio of $(kl_u/r)/(34\text{-}12M_1/M_2)$ is greater than 1 but δ_{ns} value is less than 1. Considering 90 columns result this 5 cases are not very significant. This value could be considered as outlier. So, from figure this can be concluded that, the ACI guideline for considering Nonsway Moment Magnification Factor is correct.

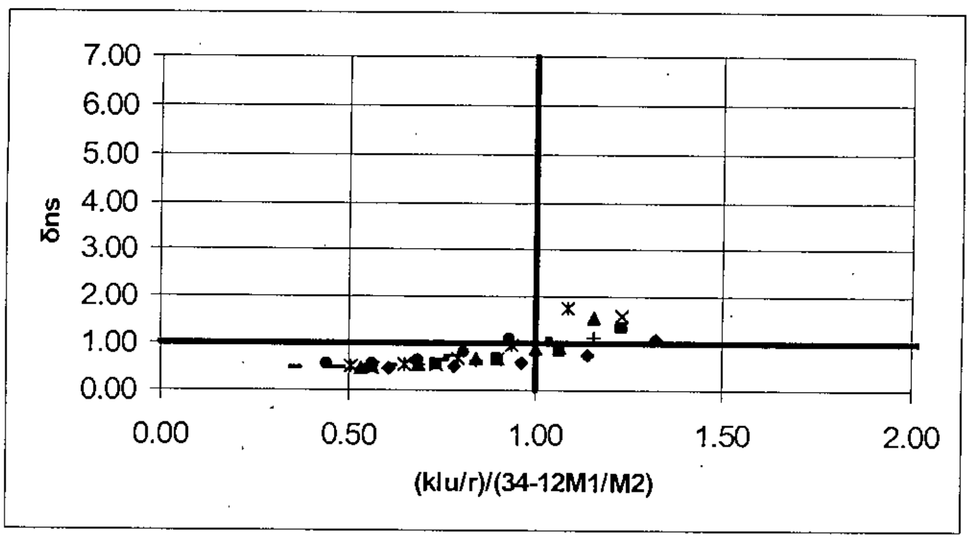


Figure 4.35: ACI Code limit for Nonsway Moment Magnification Factor in beam-column frame structure

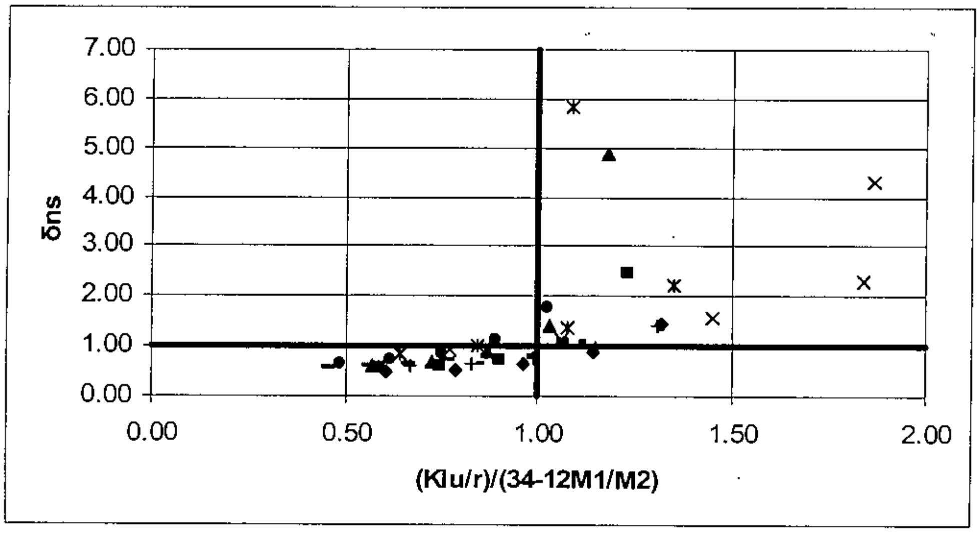


Figure 4.36: ACI Code limit for Nonsway Moment Magnification Factor in flat-plate frame structure

4.18 Checking of ACI Code Guideline for Neglecting Sway Slenderncss Effects

From articles 4.14 and 4.16 this is seen that for 45 columns of 15 models have sway slenderness effect. According to ACI Code, for compression member in sway frames, the effect of slenderness may be neglected when kl_u/r is less than 22. So, if $(kl_u/r)/22$ is less than 1 than the column will not show any slenderness effect. Fig. 4.37 is plotted against Sway Moment Magnification Factor against $(kl_u/r)/22$ for beam-column frame structure. Both the value's limit is 1. From figure this is seen that 45 columns are exceeded the limit bounded by ACI Code. The column length greater than 20' is at risk for slenderness effect. Column length ranges from 17.5' to 20' is substantially consider as slender column. Figure 4.38 is plotted against Sway Moment Magnification Factor against $(kl_u/r)/22$ for flat-plate frame structure. In Fig. 4.40 this is seen that 2 columns are less than $(kl_u/r)/22$ but have δ_s effect. Considering 45 columns these 2 values are outlier. Fig. 4.37 shows more dispersion than Fig. 4.38 but δ_s are higher in Fig. 4.38. So, from figure this can be concluded that, the guideline for considering Sway Moment Magnification Factor is correct and every column is at risk for sway effect even it may not be in danger for nonsway effect. So, the designer should check a column both for nonsway and sway effect.

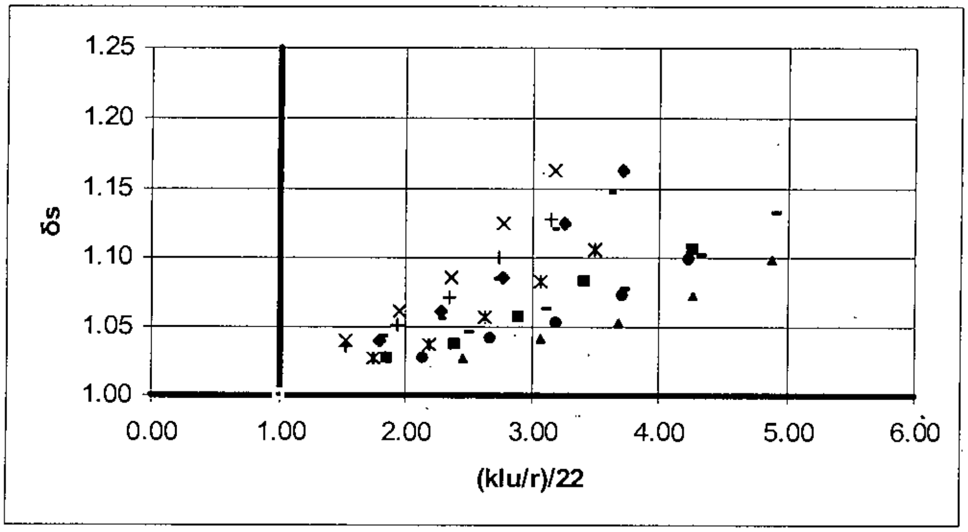


Figure 4.37: ACI Code limit for Sway Moment Magnification Factor in beam-column frame structure

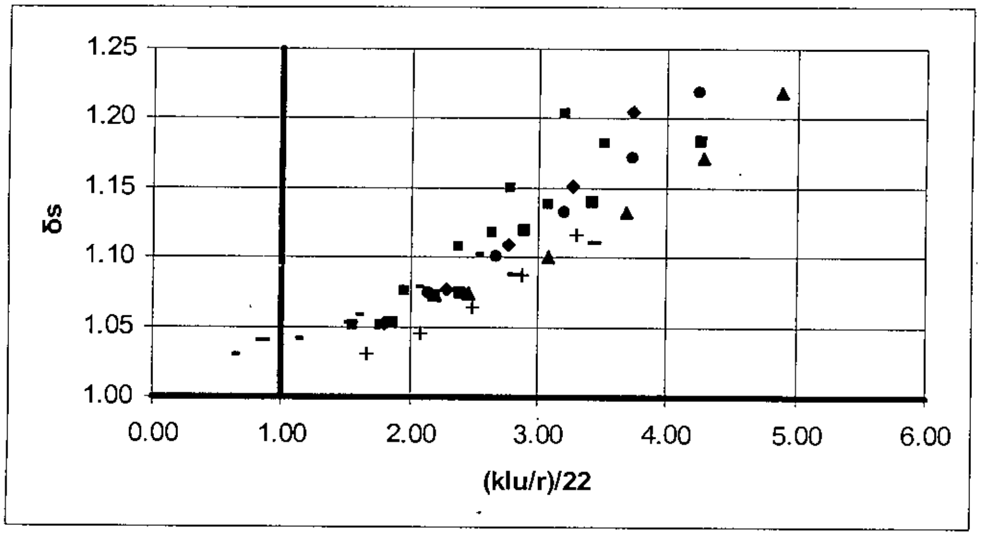


Figure 4.38: ACI Code limit for Sway Moment Magnification Factor in flat-plate frame structure

4.19 Variation in Steel Ratio Due to Before and After P-Δ Analysis

The increment of steel ratio due to secondary moment caused by *P*-Δ or sway effect is important for designer. In this parametric study the steel ratio of 90 columns are found by ETABS for both first order and second order computer analysis. The difference in steel ratio between the first order and second order analysis is not significant. This may due to presence of thick shear walls, because shear walls taking the majority of lateral load and moments. One case study has been done by decreasing the thickness of shear wall of a particular model that is described in previous section. In beam-column frame structure and for 15'x15' slab panel the edge column is tested by reducing the shear wall thickness by 3". Before reducing the shear wall thickness the design moment, design load case and steel ratio has been recoded for first order analysis and second order analysis. When the shear wall thickness is 9", the column is satisfied for DCON18 but when the wall thickness reduces the column satisfied for DCON6. The result of two different cases is present in Table 4.3. When the shear wall is thick than the steel ratio is in range of minimum requirement but after reducing the thickness the increase in moment value is noticeable. The moment value increases more than three times. The difference between the steel ratio before first order and second order analysis of thinner shear wall is about 20.0%. So, this could be concluded that the presence of shear wall influence the moment and steel ratio of column. So, in the 30 models of beam-column and flat-plate structure the steel ratio due to second order analysis does not increase noticeably due to presence of thick shear walls.

Table 4.3: Influence of shear walls in column moment

Slab Panel Size, Location and Size of Column	**Shear wall Thickness (Inches)**	**First order analysis moment (kip-ft)**	**Second order analysis moment (kip-ft)**	**Steel % after 1st order analysis (Steel % after 2nd order analysis)**
15'x15', Edge Column, 15"x15"	9"	*M2*=24.62 *M3*=22.66	*M2*=26.01 *M3*=22.97	1.0% (1.0%)
15'x15', Edge Column, 15"x15"	6"	*M2*=79.61 *M3*=43.27	*M2*=84.84 *M3*=44.58	1.5% (1.8%)

4.20 Conclusion

From this parametric study this is seen that when a column has slenderness effect then its design moment increases. Sometimes there is a sudden increment in moment value. So, before starting column design this is essential to check every column for slenderness effect. Slenderness effect is always present in column and always need to be considered. For a ten story structure in this story every column in ground floor has slenderness effect. Each column has either Nonsway Moment Magnification Factor or Sway Moment Magnification Factor or both. The ACI Code guideline for neglecting slenderness effect is accurate and conservative. The designer should follow ACI Code guideline for slender column design.

For longer column the magnification factor δ_{ns} is greater than δ_s. Sometimes the value of δ_{ns} is extremely higher then δ_s. In this study column length greater than 20' has extreme slenderness effect. The variation in δ_s is regular when the column length increment is constant but for some column even the length increment is steady, δ_{ns} increases suddenly and the steel ratio rises extremely. Panel size more than 20'x20' influences the slenderness of column. δ_{ns} value is larger in larger slab panel.

The column has slenderness effect when P_u varies from 30.0% to 40.0% of P_c. The column in flat-plate structure is more vulnerable then beam-column structure. By location, corner and edge column should be carefully judged at the time of calculating magnification factors. Especially edge column of flat-plate structure has shown maximum slenderness values in this study.

Most of the inner column of 30 models does not show any slender effect. Column closer to shear wall has less slenderness effect than column in outer frame of structure. Inner columns of these 30 models show less slenderness effect than corner and edge column because of influence of shear walls.

There is one limitation of ETABS for considering l_c in inner column of flat-plate structure. That is considering l_c from center to center of column instead of face of slab. ETABS does consider slab thickness in inner column of flat-plate structure.

The presence of shear walls reduces the effect of moment in column. Consequently after second order analysis the effect of sway is not prominent in column. For this reason the steel ratio does not increase rapidly after P-Δ analysis.

CHAPTER 5

CONCLUSION

5.1 General

Column is the most important element of structure as it transfers all loads to the foundation. Failure in column might cause total collapse of a building. Even being the most vital part of a structure, slenderness effect are often not considered by the designers due to the lack of appreciation of the philosophy behind this effect. Building systems as well as individual members are now being analyzed and designed widely by user friendly commercial software. Use of these software for analysis and design of structure make the work of designer easier particularly the complicated and time consuming procedure of column design involving slenderness effect becomes much straight forward if software are used. But use of these software in design offices without validations against the Code procedure is questionable. This study has been carried out to appreciate the provisions of ACI Code 318 (1999) for slender column design and to check the incorporation of these design procedures in the commercially available software. ACI Code guidelines have been reviewed for slender column design by manual calculation and the design by ETABS (CSI, 2003) and PCACOL (PCA, 1999) software was studied. The ETABS design output has also been checked by software. A limited parametric study has also been carried out to identify the influencing factors that affect slender column design and the parameters which should be considered with extra care by the designer have been pointed out.

5.2 Findings

The findings of the study are discussed in the following sections:

5.2.1 ACI Code review

An effect of slenderness in column is very obvious for high-rise structure. The designer should check every column of structure by following ACI Code guideline before starting design. The ACI criteria for neglecting slenderness effect in structure has been checked and found reasonable in the study. Among 90 columns of 30 models, 85 columns satisfy ACI Code guideline for neglecting slenderness effect. The deviations found in other 5 columns are very minimal.

5.2.2 Software validation

The manual calculation using ACI Code guideline for finding Nonsway Moment Magnification Factor (δ_{ns}) and Sway Moment Magnification Factor (δ_s) closely matched with ETABS and PCACOL calculation output. Thus calculation processes of these software have been validated.

The methods for determining δ_{ns} in ETABS and PCACOL are different because of considering two different calculation methods. These two methods are described in Chapter 2. ETABS neglects reinforcement involvement in δ_{ns} calculation. For this reason the value of δ_{ns} in ETABS is larger than PCACOL δ_{ns} value. Larger δ_{ns} value increases design moment. So, for calculating δ_{ns}, ETABS is more conservative than PCACOL. The designer should appreciate this fact while designing slender column using these two software.

To incorporate the sway effect involvement in slender column design, P-Δ should always be performed in ETABS software. In ETABS, P-Δ analysis does not consider Effective Length Factor (k) for magnified moment calculation. k in sway column is greater than 1.0. So, the result output in P-Δ analysis is not much magnified compare to the magnified moment found using k with Eq. (2.7). But for PCACOL as individual k value is needed to determine Sway Moment Magnification Factor (δ_s), the sway magnified moment is higher in PCACOL than ETABS. So, for calculating δ_s PCACOL is more conservative than ETABS.

5.2.3 Parametric study

Every frame in concrete structure needs to be treated as sway frame. Because every ground floor column in concrete structure has δ_s value greater than 1.0. So, it is necessary to run with P-Δ analysis option with specified load combination.

In the current study, the values of δ_{ns} are found greater than δ_s. The column taller than or equals to 17.5' is vulnerable due to higher value of δ_{ns}. The variation in δ_{ns}

sometimes increases rapidly when column length increases from 17.5' to 20.0'. From Microsoft Excel database this is noted that, rapid increment of δ_{ns} value is observed when Slenderness Ratio is greater than 40.0. This sudden increment in δ_{ns} values effects on steel ratio of column. Sometimes steel ratio increases from nominal value to maximum value or crosses the specified limit when the decrement in column section is very minimal.

Slenderness effect is more in flat-plate frame structure than beam-column frame structure. Corner and edge column are found more critical due to slenderness. Particularly edge column of flat-plate structure shows maximum effect due slenderness. Columns in these two positions need more attention while designing a building. The inner column does not show much slenderness effect in this study because inner column is nearer to shear wall. The column close to shear wall has less slenderness effect due to influence of shear wall in sway behavior of column.

Columns are designed conservatively in ETABS for flat-plate structure because l_c is taken from centre to centre.

When the Design Load (P_u) increases more than 30.0% from Critical Buckling Load (P_c) of a column, the designer should take precautions in the design.

5.3 Limitations

The structural model that has been developed using ETABS is a square shaped building. Practically this square shape building is not very common where as rectangular or irregular shape is prominent.

P-Δ analysis does not show remarkable change in steel ratio of column. This is due to shear wall influence in column buckling phenomena. One case study has been done by reducing shear wall thickness. By this, it is seen that shear wall influences much in second order analysis as well as steel ratio increases distinctly. The influence of shear wall has not been done thoroughly. The shape, size, placing of shear wall has not been studied in this work.

Shear walls have been considered in this study because in Dhaka city this is now very common phenomena to having a shear wall lift core in high-rise buildings. In this project, if the buildings have been modeled without the shear walls then the columns have to carry larger loads and moments and the design section will be large enough that the slenderness ratio will be low. For 10 stories buildings consideration of shear walls is practical but for low rise building effects of slenderness in columns without shear walls need a careful study.

Partition walls have not been considered in the study. Incorporation of partitions will reduce the sway effect.

The wind load has been calculated using ETABS's automatic analysis feature using UBC-1994 code. ETABS has option to provide user define loads. The wind load can be calculated by the BNBC guidelines and can be used in ETABS.

5.4 Recommendations

Influence of shear walls in slenderness behavior of column should be studied thoroughly. Because presence of shear walls in high-rise structure is very common. Shear wall size, shape and placing could influence in slender column behavior.

The 30 models that have been developed by ETABS are square in shape and all the columns are square in these models. Rectangular and irregular shape of building with different shape of columns needs to be studied.

The study has been done considering increment of all columns of a particular storey. This is observed that in Bangladesh only columns in one or two frames, double height are used while other columns kept as single height. So, the influence of slenderness in this type of building needs a through study.

The structural models have been developed considering that the building is located in moderate earthquake influence zone and occupancy as commercial building. In Bangladesh as other two earthquakes influence zones are present, so it is needed to study the effect of slenderness in columns in other earthquake zones and other types of occupancy i.e. residential etc.

By summarizing all the study stated in above recommendations, a more comprehensive guideline for slender column design in respect of Bangladesh could be established.

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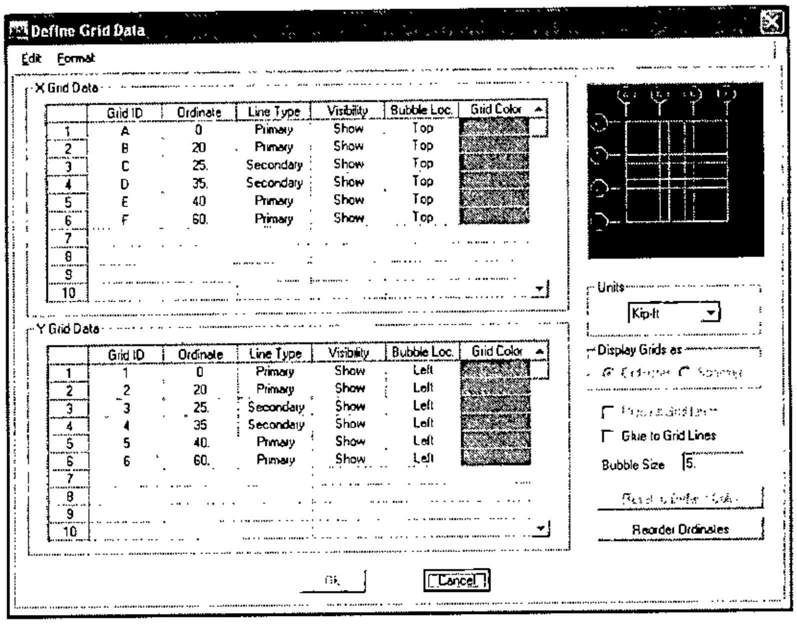
APPENDIX-A

A.1 MODEL GENERATION & STEPS FOR CALCULATING NON SWAY MOMENTS ON ETABS VERSION 8.4.6

1. Grid Data



X Grid Data

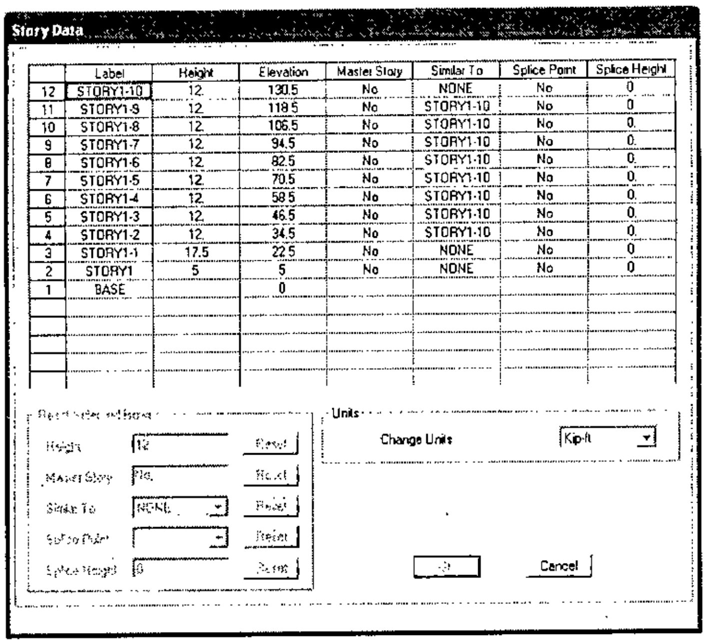
	Grid ID	Ordinate	Line Type	Visibility	Bubble Loc.
1	A	0	Primary	Show	Top
2	B	20	Primary	Show	Top
3	C	25	Secondary	Show	Top
4	D	35	Secondary	Show	Top
5	E	40	Primary	Show	Top
6	F	60	Primary	Show	Top

Y Grid Data

	Grid ID	Ordinate	Line Type	Visibility	Bubble Loc.
1	1	0	Primary	Show	Left
2	2	20	Primary	Show	Left
3	3	25	Secondary	Show	Left
4	4	35	Secondary	Show	Left
5	5	40	Primary	Show	Left
6	6	60	Primary	Show	Left

Figure 1: Input grid data for model generation

2. Story Data



	Label	Height	Elevation	Master Story	Similar To	Splice Point	Splice Height
12	STORY1-10	12	130.5	No	NONE	No	0
11	STORY1-9	12	118.5	No	STORY1-10	No	0
10	STORY1-8	12	106.5	No	STORY1-10	No	0
9	STORY1-7	12	94.5	No	STORY1-10	No	0
8	STORY1-6	12	82.5	No	STORY1-10	No	0
7	STORY1-5	12	70.5	No	STORY1-10	No	0
6	STORY1-4	12	58.5	No	STORY1-10	No	0
5	STORY1-3	12	46.5	No	STORY1-10	No	0
4	STORY1-2	12	34.5	No	STORY1-10	No	0
3	STORY1-1	17.5	22.5	No	NONE	No	0
2	STORY1	5	5	No	NONE	No	0
1	BASE		0				

Figure 2: Input story data for model generation

3. Grade Beam Layout Plan

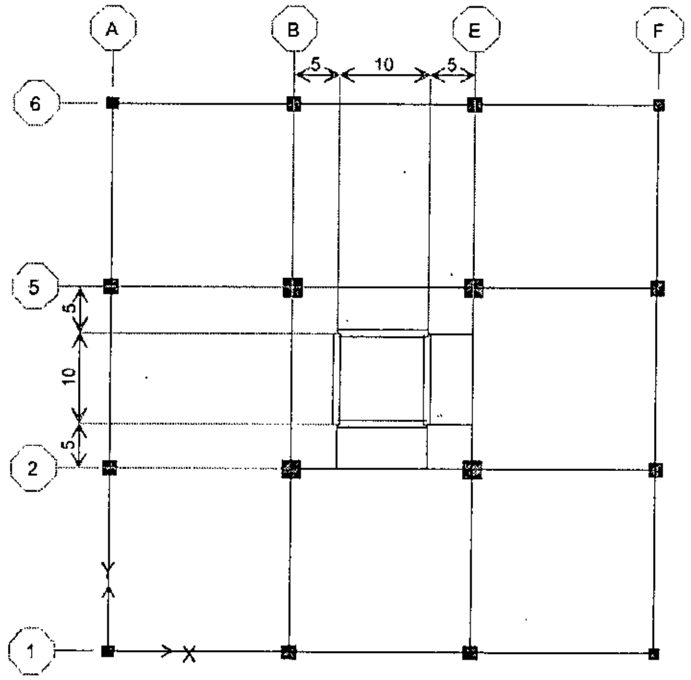


Figure 3: Grade beam layout plan of model

4. 3-D View of Model

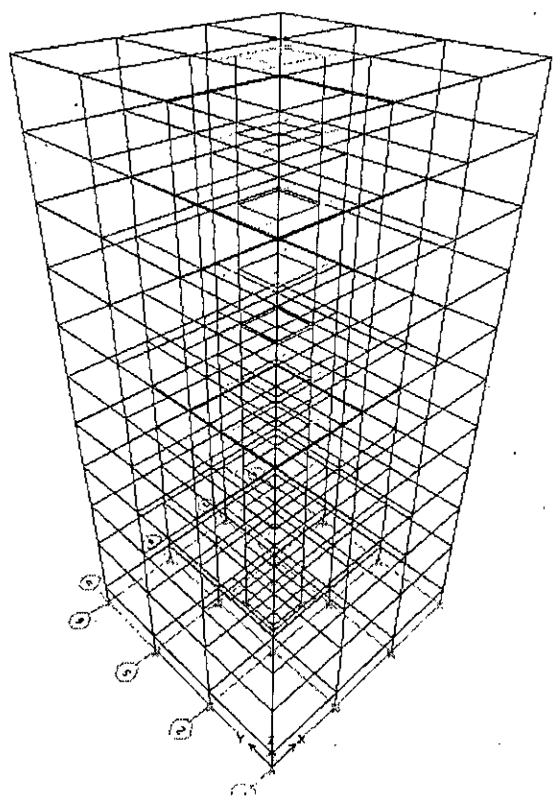


Figure 4: 3-D view of model after generation

5. Wind Load Data

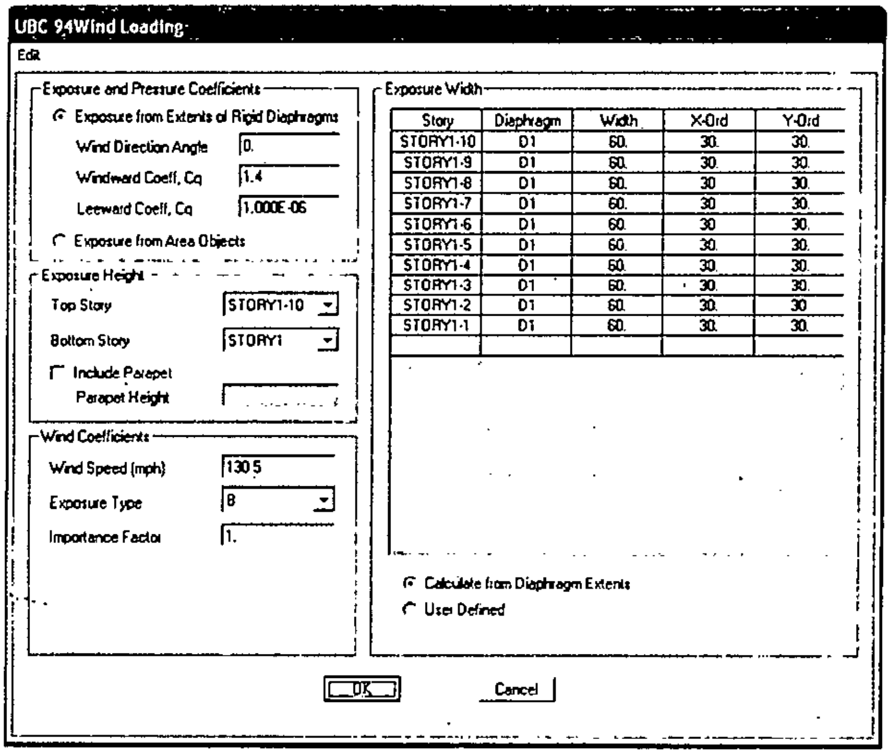


Figure 5: Data input for automated wind load generation on structure

6. Earthquake Load Data

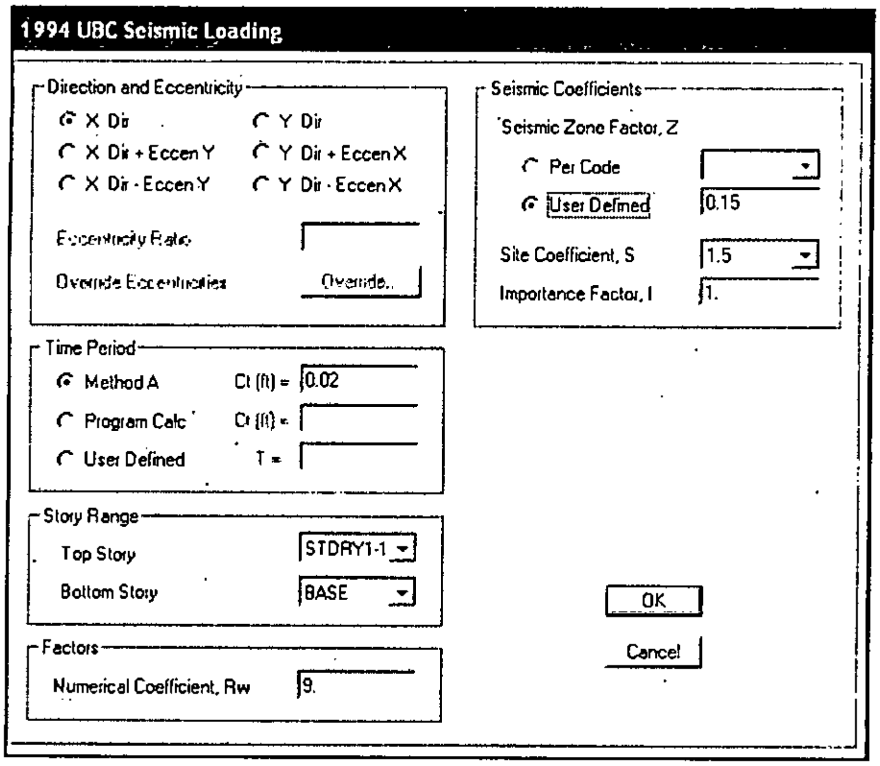


Figure 6: Data input for automated earthquake load generation on structure

7. Story Drift Output Data

DISPLACEMENTS AND DRIFTS AT POINT OBJECT 4

File

STORY	DISP-X	DISP-Y	DRIFT-X	DRIFT-Y
STORY1-10	2.226175	0.000000	0.001292	0.000000
STORY1-9	2.040198	0.000000	0.001371	0.000000
STORY1-8	1.842799	0.000000	0.001466	0.000000
STORY1-7	1.631675	0.000000	0.001565	0.000000
STORY1-6	1.406370	0.000000	0.001647	0.000000
STORY1-5	1.169250	0.000000	0.001695	0.000000
STORY1-4	0.925235	0.000000	0.001689	0.000000
STORY1-3	0.681980	0.000000	0.001611	0.000000
STORY1-2	0.450014	0.000000	0.001434	0.000000
STORY1-1	0.243470	0.000000	0.001102	0.000000
STORY1	0.012140	0.000019	0.000202	0.000000

Figure 7: Story drifts output of structure

8. Share Force Check for Earthquake Load

Support Reactions

Edit View

Support Reactions

Story	Point	Load	FX	FY	FZ
Summation	0, 0, Base	DEAD	0.00	0.00	7086.09
Summation	0, 0, Base	LIVE	0.00	0.00	2103.00
Summation	0, 0, Base	EQX	-259.42	0.00	0.00
Summation	0, 0, Base	EQY	0.00	-259.42	0.00

OK

Figure 8: Total DL of model and base share force due to EQ load

9. Result of δ_{ns} for C5 Column

Concrete Design Information ACI 318-99

File Drawing

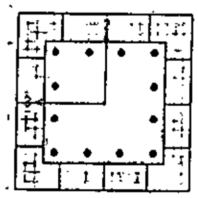
ACI 318-99 COLUMN SECTION DESIGN Type: Sway Intermediate Units: Kip-ft (Flexural Details) Units Kip-ft

Level : STORY1-1	L=17.500		
Element : C5	B=1.333	D=1.333	dc=0.125
Station Loc : 16.000	E=518400.000	fc=576.000	Lt.Wt. Fac.=1.000
Section ID : COL-2	fy=8640.000	fys=8640.000	
Combo ID : DCON2	RLLF=0.400		

Phi(Compression-Spiral):	0.750	Overstrength Factor: 1.25
Phi(Compression-Tied):	0.700	
Phi(Tension):	0.900	
Phi(Bending):	0.900	
Phi(Shear/Torsion):	0.850	

AXIAL FORCE & BIAXIAL MOMENT DESIGN FOR PU, M2, M3

Rebar Area	Rebar %	Design Pu	Design Mu2	Design Mu3
0.072	4.078	684.785	56.386	-111.057



Factored & Minimum Biaxial Moments

	Non-Sway Mns	Sway Ms	Factored Mu	Minimum Mmin	Minimum Eccentrcty
Major Bending(M3)	-2.367	0.000	-2.367	61.631	0.090
Minor Bending(M2)	36.017	0.000	36.017	61.631	0.090

Axial Force & Biaxial Moment Factors

	Cm Factor	Delta_ns Factor	Delta_s Factor	K Factor	L Length
Major Bending(M3)	0.460	1.802	1.000	1.000	16.000
Minor Bending(M2)	0.400	1.566	1.000	1.000	16.000

Figure 9: Result of C5 column for DCON2

A.2 STEPS FOR CALCULATING NON SWAY MOMENT MAGNIFICATION FACTOR ON PCACOL VERSION 3.0

1. **Step-1**

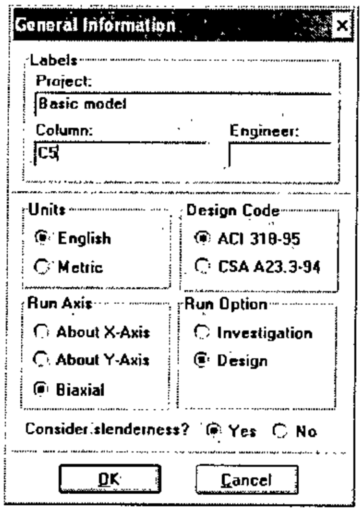


Figure 1: Input general information for slender column design

2. **Step-2**

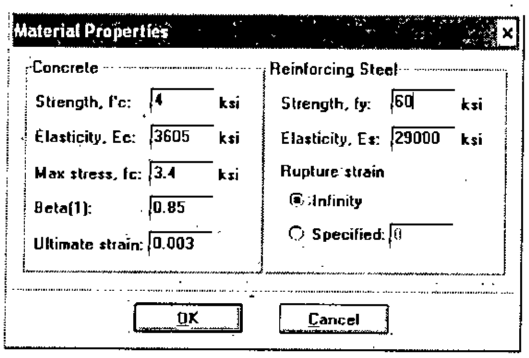


Figure 2: Input material properties for slender column design

3. **Step-3**

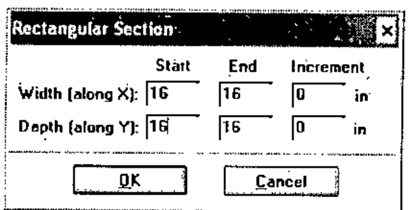


Figure 3: Sectional properties requirement

4. **Step-4**

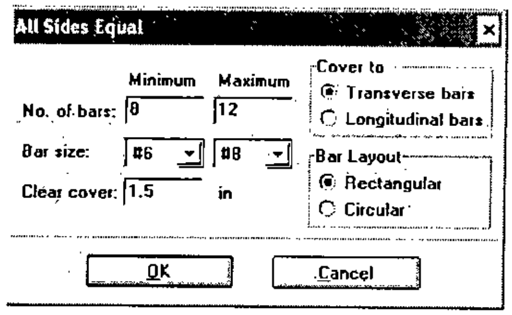


Figure 4: Transverse bar specifications of column design

5. **Step-5**

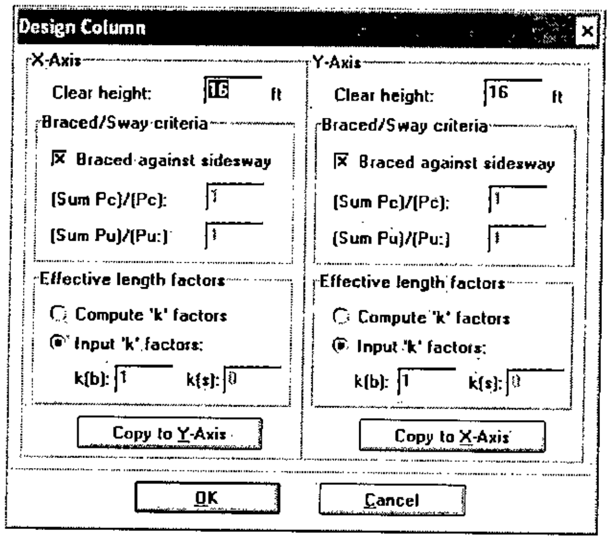


Figure 5: Slender column design parameter requirements for nonsway column

6. **Step-6**

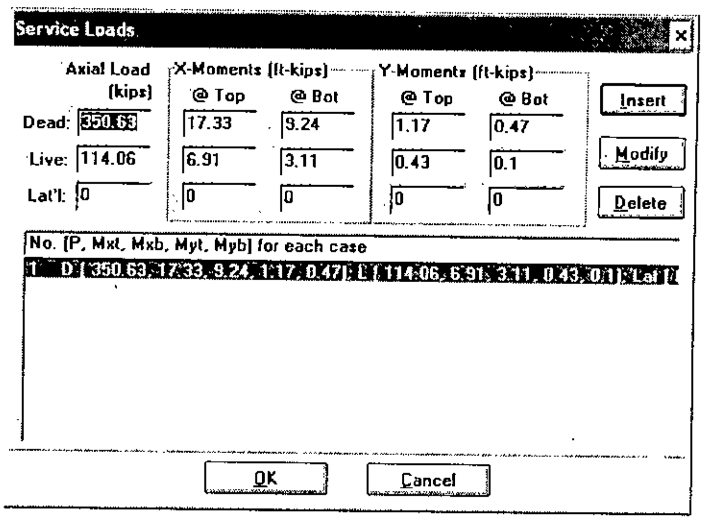


Figure 6: Applied load to column exported from ETABS analysis for nonsway column design

7. **Step-7**

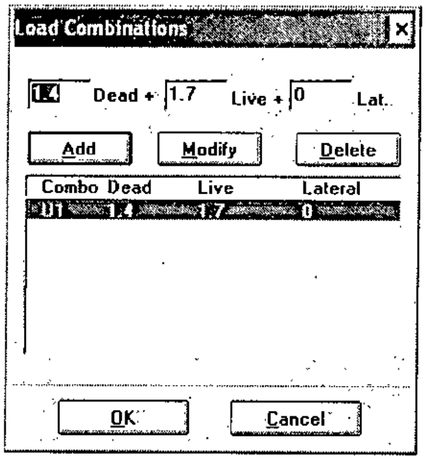


Figure 7: Specify load combination data

A.3 RESULTS OF NONSWAY MOMENT MAGNIFICATION FACTOR BY PCACOL

```
General Information:
===================
   File Name: C:\DOCUME~1\IMRAN\MYDOCU~1\THESIS~1\BASICM~1\C5.COL
   Project:  Basic model
   Column:   C5                             Engineer:
   Code:     ACI 318-95                     Units: English

   Run Option: Design                       Slenderness: Considered
   Run Axis:   Biaxial                      Column Type: Structural

Material Properties:
===================
   f'c   = 4 ksi                            fy   = 60 ksi
   Ec    = 3605 ksi                         Es   = 29000 ksi
   fc    = 3.4 ksi                          Rupture strain = Infinity
   Ultimate strain = 0.003 in/in
   Beta1 = 0.85

Section:
========
   Rectangular: Width = 16 in               Depth = 16 in

   Gross section area, Ag =  256 in^2
   Ix =  5461.33 in^4                       Iy =  5461.33 in^4
   Xo =  0 in                               Yo =  0 in

      Confinement: Tied; #3 ties with #10 bars,  #4 with larger bars.
   phi(a) = 0.8,  phi(b) = 0.9,  phi(c) = 0.7
   Layout: Rectangular
   Pattern: All Sides Equal  (Cover to transverse reinforcement)
   Total steel area, As = 9.48 in^2 at 3.70%
   12 #8   Cover = 1.5 in

Slenderness:
============
   Sway Criteria:
   --------------
   X-axis: Braced column.
   Y-axis: Braced column.
```

Column	Axis	Height ft	Width in	Depth in	I in^4	f'c ksi	Ec ksi
Design	X	16	16	16	5461.33	4	3605
	Y	16	16	16	5461.33	4	3605
Above	X	(no column specified...)					
	Y	(no column specified...)					
Below	X	(no column specified...)					
	Y	(no column specified...)					

X-Beams Location	Length ft	Width in	Depth in	I in^4	f'c ksi	Ec ksi
Above Left	(no beam specified...)					
Above Right	(no beam specified...)					
Below Left	(no beam specified...)					
Below Right	(no beam specified...)					

Y-Beams Location	Length ft	Width in	Depth in	I in^4	f'c ksi	Ec ksi
Above Left	(no beam specified...)					

```
Above Right  (no beam specified...)
Below Left   (no beam specified...)
Below Right  (no beam specified...)
```

Effective Length Factors:

Axis	Psi(top)	Psi(bot)	k(Braced)	k(Sway)	klu/r
X	0.000	0.000	1.000	(N/A)	41.57
Y	0.000	0.000	1.000	(N/A)	41.57

Moment Magnification Factors:

```
   Stiffness reduction factor, phi(K) = 0.75
   Cracked-section coefficients: cI(beams) =  0.35; cI(columns) = 0.7

   0.2*Ec*Ig + Es*Ise (X-axis) =  1.01e+007 kip-in^2
   0.2*Ec*Ig + Es*Ise (Y-axis) =  1.01e+007 kip-in^2
```

X-axis	Braced				Sway	
Ld/Comb	Pc(kip)	Betad	Cm	Delta	Pc(kip)	Delta
1 U1	1569	0.717	0.400	1.000	--- N/A ---	

Y-axis	Braced				Sway	
Ld/Comb	Pc(kip)	Betad	Cm	Delta	Pc(kip)	Delta
1 U1	1569	0.717	0.460	1.101	--- N/A ---	

Load Combinations:

```
   U1  = 1.400*Dead + 1.700*Live + 0.000*Lateral
```

Service Loads:

No.	Load Case	Axial Load kip	Mx @ Top k-ft	Mx @ Bot k-ft	My @ Top k-ft	My @ Bot k-ft
1	Dead	350.6	17.3	9.2	1.2	0.5
	Live	114.1	6.9	3.1	0.4	0.1
	Latl	0.0	0.0	0.0	0.0	0.0

Factored Loads and Moments with Corresponding Capacities: (see user's manual for notation)

```
   NOTE: Each loading combination includes the following cases:
         First line  - Pu, Mux, Muy (at column top)
         Second line - Pu, Mux, Muy (at column bottom)
         Third line  - Pu, Mx_min, Muy (larger of top or bottom)
         Fourth line - Pu, Mux, My_min (larger of top or bottom)
```

Load Combo	Pu kip	Mux k-ft	Muy k-ft	fMnx k-ft	fMny k-ft	fMn/Mu
1 U1	684.8	36.0	2.6	128.5	9.3	3.569
	684.8	-18.2	-0.9	-129.0	-6.4	7.080
	684.8	61.6	2.6	129.2	5.4	2.096
	684.8	36.0	67.9	56.4	106.6	1.569

A.4 RESULTS OF SWAY MOMENT MAGNIFICATION FACTOR BY ETABS VERSION 8.4.6

1. Sway Moment Magnification Factor Overwrite

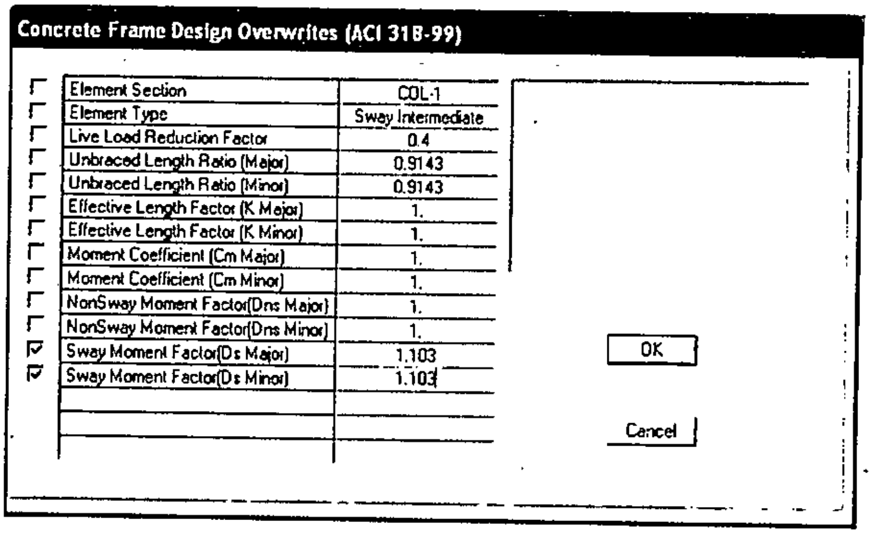


Figure 1: δ_s factor overwrite to achieve magnified moment

2. Results of Moment After Overwrite δ_s Factor

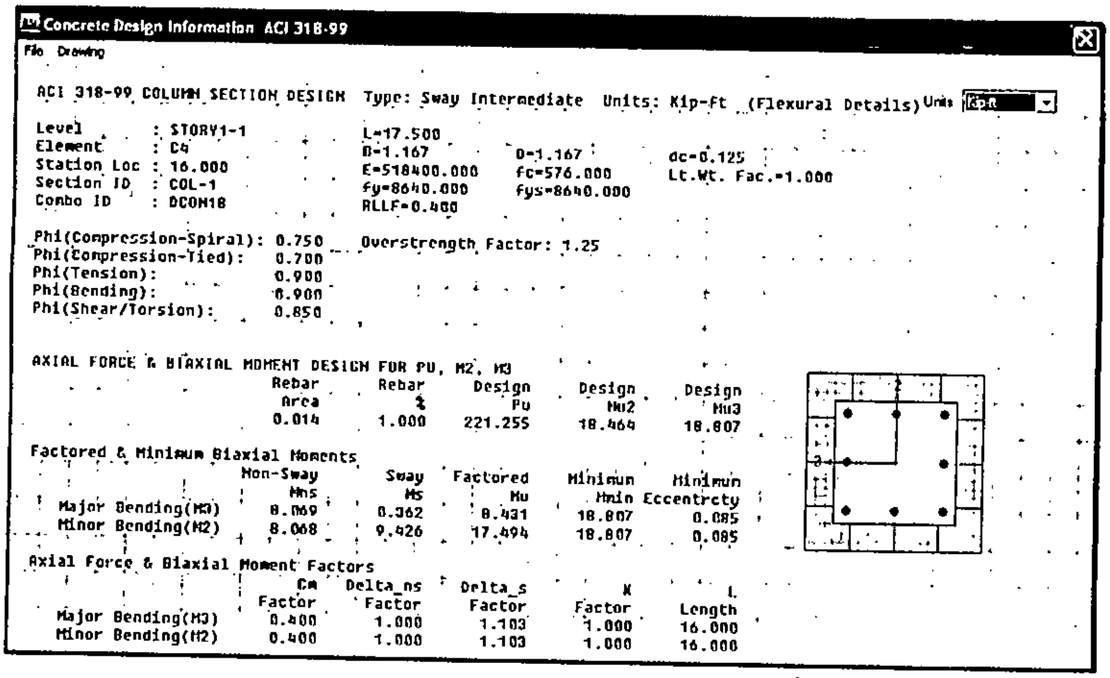


Figure 2: Result of C4 column after overwrite δ_s

3. Step for *P*-Δ Analysis

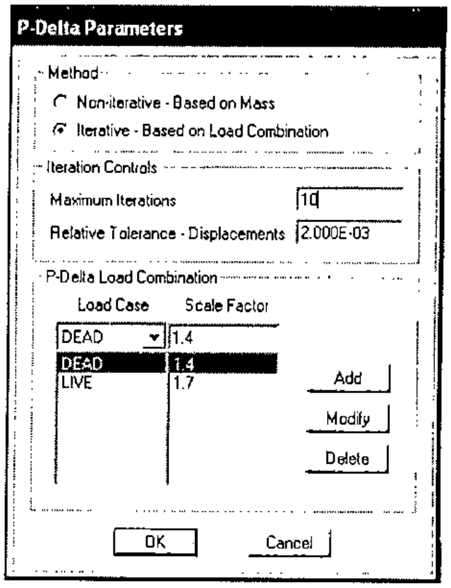


Figure 3: *P*-Δ analysis parameters input

4. Result of *P*-Δ Analysis

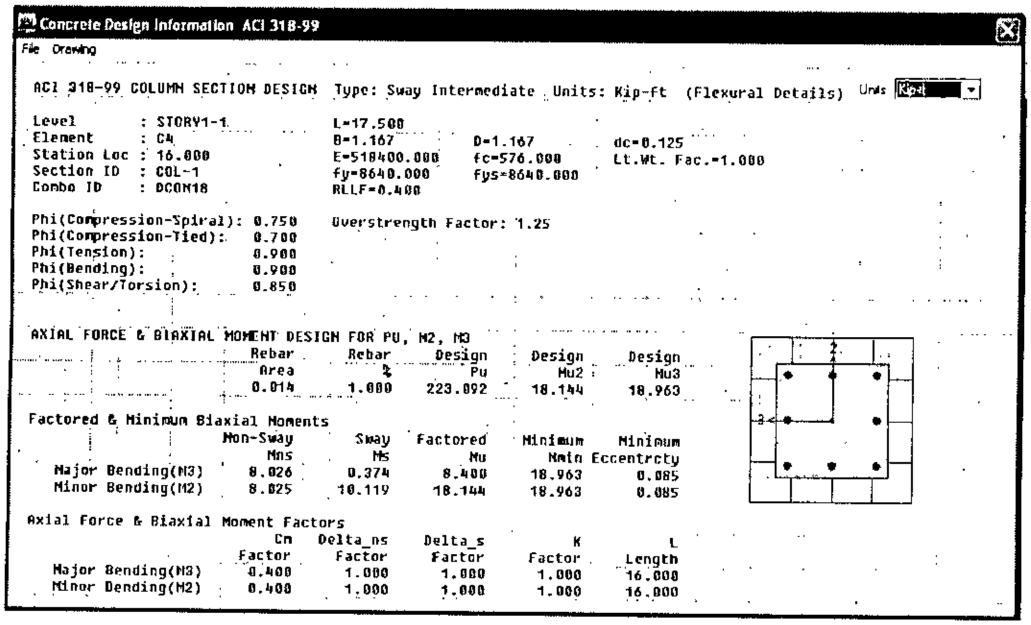


Figure 4: Result of C4 column after *P*-Δ analysis

A.5 STEPS FOR CALCULATING SWAY MOMENT MAGNIFICATION FACTOR ON PCACOL VERSION 3.0

THE STEPS FOR DESIGN OF COLUMN CONSIDERING SWAY MOMENT MAGNIFICATION FACTOR IS SAME FROM STEP-1 TO STEP-4 OF NONSWAY MOMENT MAGNIFICATION FACTOR DETERMINATION. FOR THIS REASON THE GRAPHICAL PRESENTATION START FROM STEP-5.

1. **Step-5**

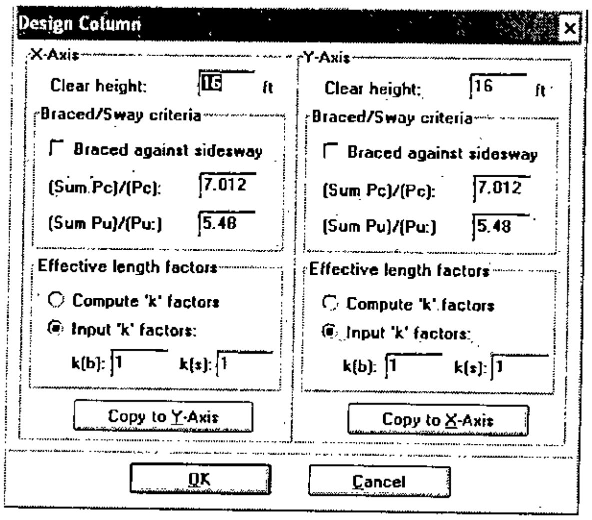


Figure 1: Slender column design parameter requirements for sway column

2. **Step-6**

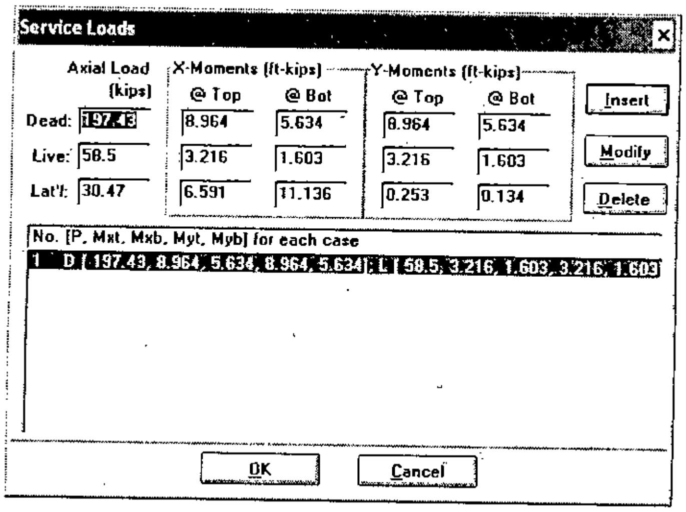


Figure 2: Applied load to column exported from ETABS analysis for sway column design

3. Step-7

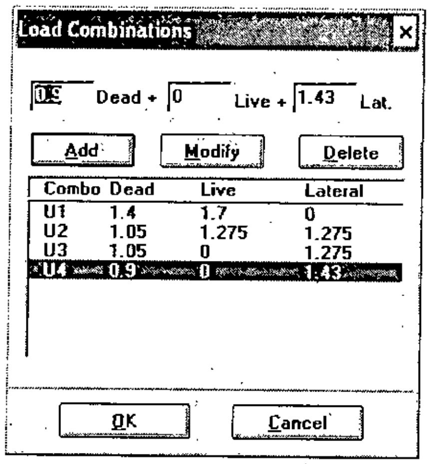


Figure 3: Specify load combination data

A.6 RESULTS OF SWAY MOMENT MAGNIFICATION FACTOR BY PCACOL VERSION 3.0

```
General Information:
   ====================
      File Name: C:\DOCUME~1\IMRAN\MYDOCU~1\THESIS~1\BASICM~1\C4.COL
      Project:   Basic Model
      Column:    C4                              Engineer:
      Code:      ACI 318-95                      Units: English

      Run Option: Design                         Slenderness: Considered
      Run Axis:   Biaxial                        Column Type: Structural

   Material Properties:
   ====================
      f'c    = 4 ksi                             fy    = 60 ksi
      Ec     = 3605 ksi                          Es    = 29000 ksi
      fc     = 3.4 ksi                           Rupture strain = Infinity
      Ultimate strain = 0.003 in/in
      Beta1 = 0.85

   Section:
   ========
      Rectangular: Width = 14 in                 Depth = 14 in

      Gross section area, Ag =  196 in^2
      Ix =  3201.33 in^4                         Iy =  3201.33 in^4
      Xo =  0 in                                 Yo =  0 in

   Slenderness:
   ============
      Sway Criteria:
      --------------
      X-axis: Unbraced column. Sum of Pc = 7.01*Pc   Sum of Pu = 5.48*Pu
      Y-axis: Unbraced column. Sum of Pc = 7.01*Pc   Sum of Pu = 5.48*Pu

              Height      Width      Depth          I       f'c          Ec
Column Axis     ft          in         in         in^4      ksi          ksi
------ ---- -------- ---------- ---------- ------------ -------- -----------
Design  X       16         14         14      3201.33        4         3605
        Y       16         14         14      3201.33        4         3605
Above   X   (no column specified...)
        Y   (no column specified...)
Below   X   (no column specified...)
        Y   (no column specified...)

X-Beams       Length      Width      Depth          I       f'c          Ec
Location        ft          in         in         in^4      ksi          ksi
----------- -------- ---------- ---------- ------------ -------- -----------
  Above Left   (no beam specified...)
  Above Right  (no beam specified...)
  Below Left   (no beam specified...)
  Below Right  (no beam specified...)

Y-Beams       Length      Width      Depth          I       f'c          Ec
Location        ft          in         in         in^4      ksi          ksi
----------- -------- ---------- ---------- ------------ -------- -----------
  Above Left   (no beam specified...)
  Above Right  (no beam specified...)
  Below Left   (no beam specified...)
  Below Right  (no beam specified...)

      Effective Length Factors:
      -------------------------
      Axis     Psi(top)     Psi(bot)    k(Braced)      k(Sway)        klu/r
      ---- ------------ ------------ ------------ ------------ ------------
```

X	0.000	0.000	1.000	1.000	47.51
Y	0.000	0.000	1.000	1.000	47.51

Moment Magnification Factors:
============================

Stiffness reduction factor, phi(K) = 0.75
Cracked-section coefficients: cI(beams) = 0.35; cI(columns) = 0.7

0.2*Ec*Ig + Es*Ise (X-axis) = 4.04e+006 kip-in^2
0.2*Ec*Ig + Es*Ise (Y-axis) = 4.04e+006 kip-in^2

X-axis Ld/Comb	Braced Pc(kip)	Braced Betad	Braced Cm	Braced Delta	Sway Pc(kip)	Sway Delta
1 U4	480	1.249	0.400	1.037	1080	1.271

Y-axis Ld/Comb	Braced Pc(kip)	Braced Betad	Braced Cm	Braced Delta	Sway Pc(kip)	Sway Delta
1 U4	480	1.249	0.400	1.037	1080	1.271

Load Combinations:
================

U4 = 0.900*Dead + 0.000*Live + 1.430*Lateral

Service Loads:
=============

No.	Load Case	Axial Load kip	Mx @ Top k-ft	Mx @ Bot k-ft	My @ Top k-ft	My @ Bot k-ft
1	Dead	197.4	9.0	5.6	9.0	5.6
	Live	58.5	3.2	1.6	3.2	1.6
	Latl	30.5	6.6	11.1	0.3	0.1

Factored Loads and Moments with Corresponding Capacities: (see user's manual for notation)
===

NOTE: Each loading combination includes the following cases:
First line - Pu, Mux, Muy (at column top)
Second line - Pu, Mux, Muy (at column bottom)
Third line - Pu, Mx_min, Muy (larger of top or bottom)
Fourth line - Pu, Mux, My_min (larger of top or bottom)

Load Combo	Pu kip	Mux k-ft	Muy k-ft	fMnx k-ft	fMny k-ft	fMn/Mu
1 U4	221.3	20.0	8.5	85.7	36.5	4.276
	221.3	-25.3	-5.3	-95.7	-20.1	3.782
	221.3	-18.8	-5.3	-92.3	-26.1	4.908
	221.3	20.0	18.8	65.4	61.3	3.261

APPENDIX-B

B.1 NONSWAY MOMENT MAGNIFICATION FACTOR DETERMINATION FOR BEAM COLUMN STRUCTURE

Case 1.0

Panel Size 15 Feet x 15 Feet **Column Length** = 10'

Ground Floor									
Location	**Dim**	**L (')**	**M1 (k-')**	**M2 (k-')**	**M1/M2**	**ABS (M1/M2)**	**Load Case**	**Steel %**	**Steel % (P-Del)**
Corner	14x14	10	-6.57	9.63	-0.68	0.68	2	1.0%	1.0%
Edge	15x15	10	-9.50	17.17	-0.55	0.55	2	1.0%	1.0%
Interior	16x16	10	7.72	-10.95	-0.71	0.71	2	1.0%	1.0%

Case 2.0

Panel Size 15 Feet x 15 Feet **Column Length** = 12.5'

Ground Floor									
Location	**Dim**	**L (')**	**M1 (k-')**	**M2 (k-')**	**M1/M2**	**ABS (M1/M2)**	**Load Case**	**Steel %**	**Steel % (P-Del)**
Corner	14x14	13	-6.02	9.07	-0.66	0.66	2	1.0%	1.0%
Edge	15x15	13	-8.80	15.85	-0.56	0.56	2	1.0%	1.0%
Interior	16x16	13	7.06	-10.61	-0.67	0.67	2	1.0%	1.0%

Case 3.0

Panel Size 15 Feet x 15 Feet **Column Length** = 15'

Ground Floor									
Location	**Dim**	**L (')**	**M1 (k-')**	**M2 (k-')**	**M1/M2**	**ABS (M1/M2)**	**Load Case**	**Steel %**	**Steel % (P-Del)**
Corner	14x14	15	-5.53	8.49	-0.65	0.65	2	1.0%	1.0%
Edge	15x15	15	-8.10	14.57	-0.56	0.56	2	1.0%	1.0%
Interior	16x16	15	6.56	-10.19	-0.64	0.64	2	1.0%	1.0%

Case 4.0

Panel Size 15 Feet x 15 Feet **Column Length** = 17.5'

Ground Floor									
Location	**Dim**	**L (')**	**M1 (k-')**	**M2 (k-')**	**M1/M2**	**ABS (M1/M2)**	**Load Case**	**Steel %**	**Steel % (P-Del)**
Corner	14x14	18	-5.10	7.95	-0.64	0.64	2	1.0%	1.0%
Edge	15x15	18	-7.44	13.41	-0.55	0.55	2	1.0%	1.0%
Interior	16x16	18	6.09	-9.76	-0.62	0.62	2	1.0%	1.0%

Case 5.0

Panel Size 15 Feet x 15 Feet **Column Length** = 20'

Ground Floor									
Location	**Dim**	**L (')**	**M1 (k-')**	**M2 (k-')**	**M1/M2**	**ABS (M1/M2)**	**Load Case**	**Steel %**	**Steel % (P-Del)**
Corner	14x14	20	-4.73	7.46	-0.63	0.63	2	1.0%	1.0%
Edge	15x15	20	-6.87	12.39	-0.55	0.55	2	1.0%	1.0%
Interior	16x16	20	5.72	-9.35	-0.61	0.61	2	1.0%	1.0%

Case 1.1
Panel Size 20 Feet x 20 Feet **Column Length** = 10'

Ground Floor

Location	Dim	L (')	M1 (k-')	M2 (k-')	M1/M2	ABS (M1/M2)	Load Case	Steel %	Steel % (P-Del)
Corner	15x15	10	-12.95	21.96	-0.59	0.59	2	1.0%	1.0%
Edge	17x17	10	-20.93	45.59	-0.46	0.46	2	1.9%	1.9%
Interior	18x18	10	16.88	-29.12	-0.58	0.58	2	1.8%	1.8%

Case 2.1
Panel Size 20 Feet x 20 Feet **Column Length** = 12.5'

Ground Floor

Location	Dim	L (')	M1 (k-')	M2 (k-')	M1/M2	ABS (M1/M2)	Load Case	Steel %	Steel % (P-Del)
Corner	15x15	13	-12.23	20.66	-0.59	0.59	2	1.0%	1.0%
Edge	17x17	13	-20.47	42.55	-0.48	0.48	2	1.9%	1.9%
Interior	18x18	13	15.77	-28.08	-0.56	0.56	2	1.6%	1.7%

Case 3.1
Panel Size 20 Feet x 20 Feet **Column Length** = 15'

Ground Floor

Location	Dim	L (')	M1 (k-')	M2 (k-')	M1/M2	ABS (M1/M2)	Load Case	Steel %	Steel % (P-Del)
Corner	15x15	15	-11.42	19.32	-0.59	0.59	2	1.0%	1.0%
Edge	17x17	15	-19.51	39.49	-0.49	0.49	2	1.8%	1.9%
Interior	18x18	15	14.87	-26.89	-0.55	0.55	2	1.5%	1.6%

Case 4.1
Panel Size 20 Feet x 20 Feet **Column Length** = 17.5'

Ground Floor

Location	Dim	L (')	M1 (k-')	M2 (k-')	M1/M2	ABS (M1/M2)	Load Case	Steel %	Steel % (P-Del)
Corner	15x15	18	-10.60	17.96	-0.59	0.59	2	1.0%	1.0%
Edge	17x17	18	-18.25	36.51	-0.50	0.50	2	1.8%	2.0%
Interior	18x18	18	14.10	-25.63	-0.55	0.55	2	1.4%	1.5%

Case 5.1
Panel Size 20 Feet x 20 Feet **Column Length** = 20'

Ground Floor

Location	Dim	L (')	M1 (k-')	M2 (k-')	M1/M2	ABS (M1/M2)	Load Case	Steel %	Steel % (P-Del)
Corner	15x15	20	-9.96	16.91	-0.59	0.59	2	1.0%	1.0%
Edge	17x17	20	-17.30	34.08	-0.51	0.51	2	3.2%	3.0%
Interior	18x18	20	13.25	-24.60	-0.54	0.54	2	1.3%	1.4%

Case 1.2
Panel Size 25 Feet x 25 Feet **Column Length** = 10'

Ground Floor									
Location	**Dim**	**L (')**	**M1 (k-')**	**M2 (k-')**	**M1/M2**	**ABS (M1/M2)**	**Load Case**	**Steel %**	**Steel % (P-Del)**
Corner	16x16	10	-21.72	41.99	-0.52	0.52	2	1.0%	1.0%
Edge	20x20	10	-34.45	95.91	-0.36	0.36	2	2.8%	2.8%
Interior	24x24	10	26.4	-53.77	-0.49	0.49	2	3.1%	3.1%

Case 2.2
Panel Size 25 Feet x 25 Feet **Column Length** = 12.5'

Ground Floor									
Location	**Dim**	**L (')**	**M1 (k-')**	**M2 (k-')**	**M1/M2**	**ABS (M1/M2)**	**Load Case**	**Steel %**	**Steel % (P-Del)**
Corner	16x16	13	-21.11	39.47	-0.53	0.53	2	1.0%	1.0%
Edge	20x20	13	-36.55	90.53	-0.40	0.40	2	2.8%	2.8%
Interior	24x24	13	24.96	-51.41	-0.49	0.49	2	3.1%	3.1%

Case 3.2
Panel Size 25 Feet x 25 Feet **Column Length** = 15'

Ground Floor									
Location	**Dim**	**L (')**	**M1 (k-')**	**M2 (k-')**	**M1/M2**	**ABS (M1/M2)**	**Load Case**	**Steel %**	**Steel % (P-Del)**
Corner	16x16	15	-20.06	36.86	-0.54	0.54	2	1.0%	1.0%
Edge	20x20	15	-36.59	84.91	-0.43	0.43	2	2.8%	2.8%
Interior	24x24	15	24.96	-49.95	-0.50	0.50	2	3.0%	3.0%

Case 4.2
Panel Size 25 Feet x 25 Feet **Column Length** = 17.5'

Ground Floor									
Location	**Dim**	**L (')**	**M1 (k-')**	**M2 (k-')**	**M1/M2**	**ABS (M1/M2)**	**Load Case**	**Steel %**	**Steel % (P-Del)**
Corner	16x16	18	-18.92	34.41	-0.55	0.55	2	1.0%	1.0%
Edge	20x20	18	-35.76	79.58	-0.45	0.45	2	2.8%	2.8%
Interior	24x24	18	23.92	-47.82	-0.50	0.50	2	2.9%	3.0%

Case 5.2
Panel Size 25 Feet x 25 Feet **Column Length** = 20'

Ground Floor									
Location	**Dim**	**L (')**	**M1 (k-')**	**M2 (k-')**	**M1/M2**	**ABS (M1/M2)**	**Load Case**	**Steel %**	**Steel % (P-Del)**
Corner	16x16	20	-17.82	32.19	-0.55	0.55	2	1.3%	1.2%
Edge	20x20	20	-34.53	74.66	-0.46	0.46	2	2.6%	2.9%
Interior	24x24	20	23.02	-45.82	-0.50	0.50	2	2.8%	2.9%

Panel Size, 15'x15' & Corner Column

K	Lc (')	Lu (")	r	Klu/r	34-12xM1/M2	34-12xM1/M2	Effect of Slenderness	Dead Load	Live Load	Max Fac. Axial Sus. Load
1	10	102	4.2	24.29	42.19	40.00	No	129.7	36.51	181.58
1	12.5	132	4.2	31.43	41.96	40.00	No	130.4	36.79	182.546
1	15	162	4.2	38.57	41.82	40.00	No	131.1	37.05	183.47
1	17.5	192	4.2	45.71	41.70	40.00	Yes	131.7	37.31	184.352
1	20	222	4.2	52.86	41.61	40.00	Yes	132.3	37.45	185.206

Max Fac. Axial Load	Bd	Ig	Ec	EI	Pc (Kip)	Cm	Cm	Pu (kip)	Del (Non Sway)	Pu/Pc	(klu/r)/ (34-12M1/M2)
243.647	0.75	3201	3.60E+06	2.65E+09	2509.2	0.33	0.40	243.65	0.46	0.10	0.61
245.089	0.74	3201	3.60E+06	2.65E+09	1498.6	0.33	0.40	245.09	0.51	0.16	0.79
246.455	0.74	3201	3.60E+06	2.65E+09	995.2	0.34	0.40	246.46	0.60	0.25	0.96
247.779	0.74	3201	3.60E+06	2.65E+09	708.7	0.34	0.40	247.78	0.75	0.35	1.14
248.871	0.74	3201	3.60E+06	2.65E+09	530.0	0.35	0.40	248.87	1.07	0.47	1.32

Panel Size, 20'x20' & Corner Column

K	Lc (')	Lu (")	r	Klu/r	34-12xM1/M2	34-12xM1/M2	Effect of Slenderness	Dead Load	Live Load	Max Fac. Axial Sus. Load
1	10	102	4.5	22.67	41.08	40.00	No	189.1	58.12	264.768
1	12.5	132	4.5	29.33	41.10	40.00	No	189.8	58.39	265.678
1	15	162	4.5	36.00	41.09	40.00	No	190.4	58.65	266.574
1	17.5	192	4.5	42.67	41.08	40.00	Yes	191	58.91	267.442
1	20	222	4.5	49.33	41.07	40.00	Yes	191.6	59.17	268.296

Max Fac. Axial Load	Bd	Ig	Ec	EI	Pc (Kip)	Cm	Cm	Pu (kip)	Del (Non Sway)	Pu/Pc	(klu/r)/ (34-12M1/M2)
363.572	0.73	4219	3.60E+06	3.52E+09	3339.2	0.36	0.40	363.57	0.47	0.11	0.57
364.941	0.73	4219	3.60E+06	3.52E+09	1994.1	0.36	0.40	364.94	0.53	0.18	0.73
366.279	0.73	4219	3.60E+06	3.52E+09	1324.1	0.36	0.40	366.28	0.63	0.28	0.90
367.589	0.73	4219	3.60E+06	3.52E+09	942.8	0.36	0.40	367.59	0.83	0.39	1.07
368.885	0.73	4219	3.60E+06	3.52E+09	705.3	0.36	0.40	368.89	1.32	0.52	1.23

Panel Size, 25'x25' & Corner Column

K	Lc (')	Lu (")	r	Klu/r	34-12xM1/M2	34-12xM1/M2	Effect of Slenderness	Dead Load	Live Load	Max Fac. Axial Sus. Load
1	10	102	4.8	21.25	40.21	40.00	No	260.2	84.79	364.308
1	12.5	132	4.8	27.50	40.42	40.00	No	260.6	84.93	364.77
1	15	162	4.8	33.75	40.60	40.00	No	260.9	85.1	365.246
1	17.5	192	4.8	40.00	40.60	40.00	Yes	261.2	85.25	365.736
1	20	222	4.8	46.25	40.64	40.00	Yes	261.6	85.41	366.226

Max Fac. Axial Load	Bd	Ig	Ec	EI	Pc (Kip)	Cm	Cm	Pu (kip)	Del (Non Sway)	Pu/Pc	(klu/r)/(34-12M1/M2)
508.451	0.72	5461	3.60E+06	4.59E+09	4352.3	0.39	0.40	508.45	0.47	0.12	0.53
509.151	0.72	5461	3.60E+06	4.59E+09	2598.9	0.39	0.40	509.15	0.54	0.20	0.69
509.916	0.72	5461	3.60E+06	4.59E+09	1725.6	0.38	0.40	509.92	0.66	0.30	0.84
510.661	0.72	5461	3.60E+06	4.59E+09	1228.5	0.38	0.40	510.66	0.90	0.42	1.00
511.423	0.72	5461	3.60E+06	4.59E+09	919.0	0.38	0.40	511.42	1.55	0.56	1.16

Panel Size, 15'x15' & Edge Column

K	Lc (')	Lu (")	r	Klu/r	34-12xM1/M2	34-12xM1/M2	Effect of Slenderness	Dead Load	Live Load	Max Fac. Axial Sus. Load
1	10	102	4.5	22.67	40.64	40.00	No	207.1	64.67	289.94
1	12.5	132	4.5	29.33	40.66	40.00	No	206.5	64.49	289.156
1	15	162	4.5	36.00	40.67	40.00	No	206	64.3	288.372
1	17.5	192	4.5	42.67	40.66	40.00	Yes	205.4	64.12	287.602
1	20	222	4.5	49.33	40.65	40.00	Yes	204.9	63.97	286.818

Max Fac. Axial Load	Bd	Ig	Ec	EI	Pc (Kip)	Cm	Cm	Pu (kip)	Del (Non Sway)	Pu/Pc	(klu/r)/(34-12M1/M2)
399.879	0.73	4219	3.60E+06	3.53E+09	3345.3	0.38	0.40	399.88	0.48	0.12	0.57
398.789	0.73	4219	3.60E+06	3.53E+09	1997.5	0.38	0.40	398.79	0.55	0.20	0.73
397.682	0.73	4219	3.60E+06	3.53E+09	1326.2	0.38	0.40	397.68	0.67	0.30	0.90
396.606	0.73	4219	3.60E+06	3.53E+09	944.1	0.38	0.40	396.61	0.91	0.42	1.07
395.567	0.73	4219	3.60E+06	3.53E+09	706.2	0.38	0.40	395.57	1.58	0.56	1.23

Panel Size, 20'x20' & Edge Column

K	Lc (')	Lu ('')	r	Klu/r	34-12xM1/M2	34-12xM1/M2	Effect of Slenderness	Dead Load	Live Load	Max Fac. Axial Sus. Load
1	10	102	5.1	20.00	39.51	39.51	No	347.1	117.4	485.912
1	12.5	132	5.1	25.88	39.77	39.77	No	346.5	117.2	485.044
1	15	162	5.1	31.76	39.93	39.93	No	345.8	117	484.162
1	17.5	192	5.1	37.65	40.00	40.00	No	345.2	116.7	483.266
1	20	222	5.1	43.53	40.46	40.00	Yes	344.5	116.5	482.342

Max Fac. Axial Load	Bd	Ig	Ec	EI	Pc (Kip)	Cm	Cm	Pu (kip)	Del (Non Sway)	Pu/Pc	(klu/r)/ (34-12M1/M2)
685.526	0.71	6960	3.60E+06	5.87E+09	5571.6	0.42	0.42	685.53	0.50	0.12	0.51
684.284	0.71	6960	3.60E+06	5.87E+09	3326.8	0.41	0.41	684.28	0.56	0.21	0.65
682.994	0.71	6960	3.60E+06	5.87E+09	2208.7	0.40	0.40	682.99	0.68	0.31	0.80
681.707	0.71	6960	3.60E+06	5.87E+09	1572.4	0.40	0.40	681.71	0.95	0.43	0.94
680.375	0.71	6960	3.60E+06	5.87E+09	1176.1	0.40	0.40	680.38	1.75	0.58	1.09

Panel Size, 25'x25' & Edge Column

K	Lc (')	Lu ('')	r	Klu/r	34-12xM1/M2	34-12xM1/M2	Effect of Slenderness	Dead Load	Live Load	Max Fac. Axial Sus. Load
1	10	102	6	17.00	38.31	38.31	No	527	186	737.8
1	12.5	132	6	22.00	38.84	38.84	No	526.8	185.9	737.576
1	15	162	6	27.00	39.17	39.17	No	526.7	185.9	737.324
1	17.5	192	6	32.00	39.39	39.39	No	526.5	185.8	737.072
1	20	222	6	37.00	39.55	39.55	No	526.3	185.7	736.792

Max Fac. Axial Load	Bd	Ig	Ec	EI	Pc (Kip)	Cm	Cm	Pu (kip)	Del (Non Sway)	Pu/Pc	(klu/r)/ (34-12M1/M2)
1053.966	0.70	13333	3.60E+06	1.13E+10	10728.7	0.46	0.46	1053.97	0.53	0.10	0.44
1053.657	0.70	13333	3.60E+06	1.13E+10	6406.2	0.44	0.44	1053.66	0.56	0.16	0.57
1053.303	0.70	13333	3.60E+06	1.13E+10	4253.3	0.43	0.43	1053.30	0.64	0.25	0.69
1052.932	0.70	13333	3.60E+06	1.13E+10	3027.9	0.43	0.43	1052.93	0.80	0.35	0.81
1052.533	0.70	13333	3.60E+06	1.13E+10	2264.9	0.42	0.42	1052.53	1.09	0.46	0.94

Panel Size, 15'x15' & Inner Column

K	Lc (')	Lu (")	r	Klu/r	34-12xM1/M2	34-12xM1/M2	Effect of Slenderness	Dead Load	Live Load	Max Fac. Axial Sus. Load
1	10	102	4.8	21.25	42.46	40.00	No	238.9	79.09	334.418
1	12.5	132	4.8	27.50	41.98	40.00	No	235.7	77.88	330.008
1	15	162	4.8	33.75	41.73	40.00	No	232.9	76.78	325.99
1	17.5	192	4.8	40.00	41.49	40.00	Yes	230.2	75.76	322.28
1	20	222	4.8	46.25	41.34	40.00	Yes	227.7	75.04	318.822

Max Fac. Axial Load	Bd	Ig	Ec	EI	Pc (Kip)	Cm	Cm	Pu (kip)	Del (Non Sway)	Pu/Pc	(klu/r)/ (34-12M1/M2)
468.871	0.71	5461	3.60E+06	4.60E+09	4360.6	0.32	0.40	468.87	0.47	0.11	0.53
462.404	0.71	5461	3.60E+06	4.60E+09	2603.1	0.33	0.40	462.40	0.52	0.18	0.69
456.516	0.71	5461	3.60E+06	4.59E+09	1727.8	0.34	0.40	456.52	0.62	0.26	0.84
451.072	0.71	5461	3.60E+06	4.59E+09	1229.8	0.35	0.40	451.07	0.78	0.37	1.00
446.39	0.71	5461	3.60E+06	4.59E+09	920.0	0.36	0.40	446.39	1.13	0.49	1.16

Panel Size, 20'x20' & Inner Column

K	Lc (')	Lu (")	r	Klu/r	34-12xM1/M2	34-12xM1/M2	Effect of Slenderness	Dead Load	Live Load	Max Fac. Axial Sus. Load
1	10	102	5.4	18.89	40.96	40.00	No	376.9	133	527.674
1	12.5	132	5.4	24.44	40.74	40.00	No	371	130.7	519.456
1	15	162	5.4	30.00	40.64	40.00	No	365.5	128.5	511.742
1	17.5	192	5.4	35.56	40.60	40.00	No	360.4	126.5	504.574
1	20	222	5.4	41.11	40.46	40.00	Yes	355.5	124.6	497.756

Max Fac. Axial Load	Bd	Ig	Ec	EI	Pc (Kip)	Cm	Cm	Pu (kip)	Del (Non Sway)	Pu/Pc	(klu/r)/ (34-12M1/M2)
753.74	0.70	8748	3.60E+06	7.42E+09	7038.9	0.37	0.40	753.74	0.47	0.11	0.47
741.595	0.70	8748	3.60E+06	7.42E+09	4202.0	0.38	0.40	741.60	0.52	0.18	0.61
730.209	0.70	8748	3.60E+06	7.42E+09	2789.2	0.38	0.40	730.21	0.61	0.26	0.75
719.59	0.70	8748	3.60E+06	7.42E+09	1985.3	0.38	0.40	719.59	0.77	0.36	0.89
709.559	0.70	8748	3.60E+06	7.41E+09	1484.7	0.38	0.40	709.56	1.10	0.48	1.03

Panel Size, 25'x25' & Inner Column

K	Lc (')	Lu (")	r	Klu/r	34-12xM1/M2	34-12xM1/M2	Effect of Slenderness	Dead Load	Live Load	Max Fac. Axial Sus. Load
1	10	102	7.2	14.17	39.89	39.89	No	776.7	286.6	1087.352
1	12.5	132	7.2	18.33	39.83	39.83	No	771.5	290	1080.058
1	15	162	7.2	22.50	40.00	40.00	No	766.4	282.4	1072.96
1	17.5	192	7.2	26.67	40.00	40.00	No	761.5	280.4	1066.072
1	20	222	7.2	30.83	40.03	40.00	No	756.7	278.4	1059.366

Max Fac. Axial Load	Bd	Ig	Ec	EI	Pc (Kip)	Cm	Cm	Pu (kip)	Del (Non Sway)	Pu/Pc	(klu/r)/ (34-12M1/M2)
1574.487	0.69	27648	3.60E+06	2.36E+10	22371.0	0.40	0.40	1574.49	0.45	0.07	0.36
1573.126	0.69	27648	3.60E+06	2.36E+10	13389.9	0.41	0.41	1573.13	0.48	0.12	0.46
1553.006	0.69	27648	3.60E+06	2.36E+10	8867.1	0.40	0.40	1553.01	0.52	0.18	0.56
1542.735	0.69	27648	3.60E+06	2.36E+10	6312.1	0.40	0.40	1542.74	0.59	0.24	0.67
1532.714	0.69	27648	3.60E+06	2.36E+10	4721.0	0.40	0.40	1532.71	0.71	0.32	0.77

B.2 SWAY MOMENT MAGNIFICATION FACTOR DETERMINATION FOR BEAM COLUMN STRUCTURE

Panel Size, 15'x15' & Corner Column

K	Lc (')	Lu (")	r	Klu/r	Effect of Slenderness	SUM Pu	SUM Vu	DEL(in)	Q	Q	Del (Sway)	(Klu/r)/22
1.63	10	102	4.2	39.59	Yes	845.64	21.56	0.12	0.04	Nonsway Frame	1.04	1.80
1.6	12.5	132	4.2	50.29	Yes	851.46	17.7	0.18	0.06	Sway Frame	1.06	2.29
1.58	15	162	4.2	60.94	Yes	857.1	14.9	0.25	0.08	Sway Frame	1.09	2.77
1.57	17.5	192	4.2	71.77	Yes	852.5	12.46	0.34	0.11	Sway Frame	1.12	3.26
1.55	20	222	4.2	81.93	Yes	868.22	11.36	0.44	0.14	Sway Frame	1.16	3.72

Panel Size, 20'x20' & Corner Column

K	Lc (')	Lu (")	r	Klu/r	Effect of Slenderness	SUM Pu	SUM Vu	DEL(in)	Q	Q	Del (Sway)	(Klu/r)/22
1.81	10	102	4.5	41.03	Yes	1142.64	24.5	0.07	0.03	Nonsway Frame	1.03	1.86
1.79	12.5	132	4.5	52.51	Yes	1149.04	19.08	0.09	0.04	Nonsway Frame	1.04	2.39
1.77	15	162	4.5	63.72	Yes	1155.38	15.32	0.13	0.05	Sway Frame	1.06	2.90
1.76	17.5	192	4.5	75.09	Yes	1161.46	12.22	0.17	0.08	Sway Frame	1.08	3.41
1.9	20	222	4.5	93.73	Yes	1167.98	10.74	0.21	0.10	Sway Frame	1.11	4.26

Panel Size, 25'x25' & Corner Column

K	Lc (')	Lu (")	r	Klu/r	Effect of Slenderness	SUM Pu	SUM Vu	DEL(in)	Q	Q	Del (Sway)	(Klu/r)/22
2.55	10	102	4.8	54.19	Yes	1625.18	43.84	0.09	0.03	Nonsway Frame	1.03	2.46
2.46	12.5	132	4.8	67.65	Yes	1632.52	34.8	0.13	0.04	Nonsway Frame	1.04	3.08
2.4	15	162	4.8	81.00	Yes	1639.76	28.44	0.16	0.05	Sway Frame	1.05	3.68
2.35	17.5	192	4.8	94.00	Yes	1647.06	23.88	0.21	0.07	Sway Frame	1.07	4.27
2.32	20	222	4.8	107.30	Yes	1654.34	20.46	0.27	0.09	Sway Frame	1.10	4.88

Panel Size, 15'x15' & Edge Column

K	Lc (')	Lu (")	r	Klu/r	Effect of Slenderness	SUM Pu	SUM Vu	DEL(in)	Q	Q	Del (Sway)	(Klu/r)/22
1.49	10	102	4.5	33.77	Yes	845.64	21.56	0.12	0.04	Nonsway Frame	1.04	1.54
1.46	12.5	132	4.5	42.83	Yes	851.46	17.7	0.18	0.06	Sway Frame	1.06	1.95
1.44	15	162	4.5	51.84	Yes	857.1	14.9	0.25	0.08	Sway Frame	1.09	2.36
1.43	17.5	192	4.5	61.01	Yes	852.5	12.46	0.34	0.11	Sway Frame	1.12	2.77
1.42	20	222	4.5	70.05	Yes	868.22	11.36	0.44	0.14	Sway Frame	1.16	3.18

Panel Size, 20'x20' & Edge Column

K	Lc (')	Lu (")	r	Klu/r	Effect of Slenderness	SUM Pu	SUM Vu	DEL(in)	Q	Q	Del (Sway)	(Klu/r)/22
1.93	10	102	5.1	38.60	Yes	1142.64	24.5	0.07	0.03	Nonsway Frame	1.03	1.75
1.86	12.5	132	5.1	48.14	Yes	1149.04	19.08	0.09	0.04	Nonsway Frame	1.04	2.19
1.82	15	162	5.1	57.81	Yes	1155.38	15.32	0.13	0.05	Sway Frame	1.06	2.63
1.79	17.5	192	5.1	67.39	Yes	1161.46	12.22	0.17	0.08	Sway Frame	1.08	3.06
1.77	20	222	5.1	77.05	Yes	1167.98	10.74	0.21	0.10	Sway Frame	1.11	3.50

Panel Size, 25'x25' & Edge Column

K	Lc (')	Lu (")	r	Klu/r	Effect of Slenderness	SUM Pu	SUM Vu	DEL(in)	Q	Q	Del (Sway)	(Klu/r)/22
2.78	10	102	6	47.26	Yes	1625.18	43.84	0.09	0.03	Nonsway Frame	1.03	2.15
2.68	12.5	132	6	58.96	Yes	1632.52	34.8	0.13	0.04	Nonsway Frame	1.04	2.68
2.61	15	162	6	70.47	Yes	1639.76	28.44	0.16	0.05	Sway Frame	1.05	3.20
2.56	17.5	192	6	81.92	Yes	1647.06	23.88	0.21	0.07	Sway Frame	1.07	3.72
2.52	20	222	6	93.24	Yes	1654.34	20.46	0.27	0.09	Sway Frame	1.10	4.24

Panel Size, 15'x15' & Inner Column

K	Lc (')	Lu ('')	r	Klu/r	Effect of Slenderness	SUM Pu	SUM Vu	DEL(in)	Q	Q	Del (Sway)	(Klu/r)/22
1.58	10	102	4.8	33.58	Yes	953.66	27.34	0.12	0.03	Nonsway Frame	1.04	1.53
1.55	12.5	132	4.8	42.63	Yes	943.48	23.16	0.18	0.05	Nonsway Frame	1.05	1.94
1.53	15	162	4.8	51.64	Yes	939.58	19.6	0.25	0.07	Sway Frame	1.07	2.35
1.51	17.5	192	4.8	60.40	Yes	929.86	16.38	0.34	0.09	Sway Frame	1.10	2.75
1.5	20	222	4.8	69.38	Yes	932.72	15.12	0.44	0.11	Sway Frame	1.13	3.15

Panel Size, 20'x20' & Inner Column

K	Lc (')	Lu ('')	r	Klu/r	Effect of Slenderness	SUM Pu	SUM Vu	DEL(in)	Q	Q	Del (Sway)	(Klu/r)/22
2.11	10	102	5.4	39.86	Yes	1554.78	21.4	0.07	0.04	Nonsway Frame	1.04	1.81
2.04	12.5	132	5.4	49.87	Yes	1542.04	17.3	0.09	0.05	Sway Frame	1.06	2.27
1.99	15	162	5.4	59.70	Yes	1535.14	14.32	0.13	0.08	Sway Frame	1.08	2.71
1.96	17.5	192	5.4	69.69	Yes	1528.86	11.5	0.17	0.11	Sway Frame	1.12	3.17
1.93	20	222	5.4	79.34	Yes	1522.74	10.5	0.21	0.13	Sway Frame	1.15	3.61

Panel Size, 25'x25' & Inner Column

K	Lc (')	Lu ('')	r	Klu/r	Effect of Slenderness	SUM Pu	SUM Vu	DEL(in)	Q	Q	Del (Sway)	(Klu/r)/22
3.9	10	102	7.2	55.25	Yes	2619.8	42.98	0.09	0.04	Nonsway Frame	1.05	2.51
3.75	12.5	132	7.2	68.75	Yes	2617.62	36.76	0.13	0.06	Sway Frame	1.06	3.13
3.65	15	162	7.2	82.13	Yes	2615.66	31.76	0.16	0.07	Sway Frame	1.08	3.73
3.57	17.5	192	7.2	95.20	Yes	2613.76	27.98	0.21	0.09	Sway Frame	1.10	4.33
3.51	20	222	7.2	108.23	Yes	2625.96	24.92	0.27	0.12	Sway Frame	1.13	4.92

B.3 NONSWAY MOMENT MAGNIFICATION FACTOR DETERMINATION FOR FLAT PLATE STRUCTURE

Case 1.0

Panel Size 15 Feet x 15 Feet **Column Length** = 10'

Ground Floor									
Location	**Dim**	**L (')**	**M1 (k-')**	**M2 (k-')**	**M1/M2**	**ABS (M1/M2)**	**Load Case**	**Steel %**	**Steel % (P-Del)**
Corner	14x14	10	1.49	-2.85	-0.52	0.52	2	1.0%	1.0%
Edge	15x15	10	0.38	1.00	0.38	0.38	2	1.0%	1.0%
Interior	16x16	10	-1.19	4.21	-0.28	0.28	2	1.0%	1.0%

Case 2.0

Panel Size 15 Feet x 15 Feet **Column Length** = 12.5'

Ground Floor									
Location	**Dim**	**L (')**	**M1 (k-')**	**M2 (k-')**	**M1/M2**	**ABS (M1/M2)**	**Load Case**	**Steel %**	**Steel % (P-Del)**
Corner	14x14	12.5	1.56	-2.52	-0.62	0.62	2	1.0%	1.0%
Edge	15x15	12.5	0.41	0.77	0.53	0.53	2	1.0%	1.0%
Interior	16x16	12.5	-0.93	3.04	-0.31	0.31	2	1.0%	1.0%

Case 3.0

Panel Size 15 Feet x 15 Feet **Column Length** = 15'

Ground Floor									
Location	**Dim**	**L (')**	**M1 (k-')**	**M2 (k-')**	**M1/M2**	**ABS (M1/M2)**	**Load Case**	**Steel %**	**Steel % (P-Del)**
Corner	14x14	15	1.57	-2.25	-0.70	0.70	2	1.0%	1.0%
Edge	15x15	15	-0.45	-0.59	0.76	0.76	2	1.0%	1.0%
Interior	16x16	15	-0.70	2.18	-0.32	0.32	2	1.0%	1.0%

Case 4.0

Panel Size 15 Feet x 15 Feet **Column Length** = 17.5'

Ground Floor									
Location	**Dim**	**L (')**	**M1 (k-')**	**M2 (k-')**	**M1/M2**	**ABS (M1/M2)**	**Load Case**	**Steel %**	**Steel % (P-Del)**
Corner	14x14	17.5	1.56	-2.04	-0.76	0.76	2	1.0%	1.0%
Edge	15x15	17.5	-0.45	-0.50	0.90	0.90	2	1.5%	1.6%
Interior	16x16	17.5	-0.51	1.53	-0.33	0.33	2	1.0%	1.0%

Case 5.0

Panel Size 15 Feet x 15 Feet **Column Length** = 20'

Ground Floor									
Location	**Dim**	**L (')**	**M1 (k-')**	**M2 (k-')**	**M1/M2**	**ABS (M1/M2)**	**Load Case**	**Steel %**	**Steel % (P-Del)**
Corner	14x14	20	1.52	-1.87	-0.81	0.81	2	1.0%	1.0%
Edge	15x15	20	-0.34	-0.54	0.63	0.63	2	4.0%	4.3%
Interior	16x16	20	-0.35	1.02	-0.34	0.34	2	1.0%	1.0%

Case 1.1

Panel Size 20 Feet x 20 Feet **Column Length** = 10'

Ground Floor									
Location	**Dim**	**L (')**	**M1 (k-')**	**M2 (k-')**	**M1/M2**	**ABS (M1/M2)**	**Load Case**	**Steel %**	**Steel % (P-Del)**
Corner	15x15	10	1.64	-4.50	-0.36	0.36	2	1.0%	1.0%
Edge	17x17	10	-0.60	-2.61	0.23	0.23	2	2.6%	2.7%
Interior	18x18	10	-4.43	11.92	-0.37	0.37	2	1.0%	1.0%

Case 2.1

Panel Size 20 Feet x 20 Feet **Column Length** = 12.5'

Ground Floor									
Location	**Dim**	**L (')**	**M1 (k-')**	**M2 (k-')**	**M1/M2**	**ABS (M1/M2)**	**Load Case**	**Steel %**	**Steel % (P-Del)**
Corner	15x15	12.5	1.80	-4.03	-0.45	0.45	2	1.0%	1.0%
Edge	17x17	12.5	-0.60	-2.18	0.28	0.28	2	2.6%	2.6%
Interior	18x18	12.5	-3.79	9.11	-0.42	0.42	2	1.0%	1.0%

Case 3.1

Panel Size 20 Feet x 20 Feet **Column Length** = 15'

Ground Floor									
Location	**Dim**	**L (')**	**M1 (k-')**	**M2 (k-')**	**M1/M2**	**ABS (M1/M2)**	**Load Case**	**Steel %**	**Steel % (P-Del)**
Corner	15x15	15	1.85	-3.62	-0.51	0.51	2	1.0%	1.0%
Edge	17x17	15	-0.67	-1.80	0.37	0.37	2	2.8%	2.9%
Interior	18x18	15	-3.15	6.98	-0.45	0.45	2	1.0%	1.0%

Case 4.1

Panel Size 20 Feet x 20 Feet **Column Length** = 17.5'

Ground Floor									
Location	**Dim**	**L (')**	**M1 (k-')**	**M2 (k-')**	**M1/M2**	**ABS (M1/M2)**	**Load Case**	**Steel %**	**Steel % (P-Del)**
Corner	15x15	17.5	1.85	-3.29	-0.56	0.56	2	1.0%	1.0%
Edge	17x17	17.5	-0.76	-1.49	0.51	0.51	2	3.7%	3.8%
Interior	18x18	17.5	-2.59	5.32	-0.49	0.49	2	1.0%	1.0%

Case 5.1

Panel Size 20 Feet x 20 Feet **Column Length** = 20'

Ground Floor									
Location	**Dim**	**L (')**	**M1 (k-')**	**M2 (k-')**	**M1/M2**	**ABS (M1/M2)**	**Load Case**	**Steel %**	**Steel % (P-Del)**
Corner	15x15	20	1.82	-3.01	-0.60	0.60	2	1.8%	1.9%
Edge	17x17	20	-0.86	-1.23	0.70	0.70	2	8.0%	8.0%
Interior	18x18	20	-2.11	4.00	-0.53	0.53	2	1.0%	1.0%

Case 1.2

Panel Size 25 Feet x 25 Feet **Column Length** = 10'

Ground Floor

Location	Dim	L (')	M1 (k-')	M2 (k-')	M1/M2	ABS (M1/M2)	Load Case	Steel %	Steel % (P-Del)
Corner	16x16	10	1.58	-5.97	-0.26	0.26	2	2.0%	2.1%
Edge	20x20	10	0.48	-5.33	-0.09	0.09	2	3.9%	3.9%
Interior	24x24	10	-9.11	41.46	-0.22	0.22	2	1.8%	1.9%

Case 2.2

Panel Size 25 Feet x 25 Feet **Column Length** = 12.5'

Ground Floor

Location	Dim	L (')	M1 (k-')	M2 (k-')	M1/M2	ABS (M1/M2)	Load Case	Steel %	Steel % (P-Del)
Corner	16x16	12.5	1.82	-5.51	-0.33	0.33	2	2.0%	2.1%
Edge	20x20	12.5	0.72	-5.06	-0.14	0.14	2	3.8%	3.9%
Interior	24x24	12.5	-9.75	35.96	-0.27	0.27	2	1.8%	1.8%

Case 3.2

Panel Size 25 Feet x 25 Feet **Column Length** = 15'

Ground Floor

Location	Dim	L (')	M1 (k-')	M2 (k-')	M1/M2	ABS (M1/M2)	Load Case	Steel %	Steel % (P-Del)
Corner	16x16	15	1.88	-5.03	-0.37	0.37	2	2.0%	2.1%
Edge	20x20	15	0.78	-4.7	-0.17	0.17	2	3.8%	3.9%
Interior	24x24	15	-9.59	31.37	-0.31	0.31	2	1.7%	1.8%

Case 4.2

Panel Size 25 Feet x 25 Feet **Column Length** = 17.5'

Ground Floor

Location	Dim	L (')	M1 (k-')	M2 (k-')	M1/M2	ABS (M1/M2)	Load Case	Steel %	Steel % (P-Del)
Corner	16x16	17.5	1.87	-4.60	-0.41	0.41	2	2.2%	2.3%
Edge	20x20	17.5	0.74	-4.82	-0.15	0.15	2	3.8%	3.9%
Interior	24x24	17.5	-9.1	27.53	-0.33	0.33	2	1.7%	1.7%

Case 5.2

Panel Size 25 Feet x 25 Feet **Column Length** = 20'

Ground Floor

Location	Dim	L (')	M1 (k-')	M2 (k-')	M1/M2	ABS (M1/M2)	Load Case	Steel %	Steel % (P-Del)
Corner	16x16	20	1.83	-4.22	-0.43	0.43	2	7.3%	7.5%
Edge	20x20	20	0.66	-3.96	-0.17	0.17	2	4.2%	4.3%
Interior	24x24	20	-8.48	24.3	-0.35	0.35	2	1.6%	1.7%

Panel Size, 15'x15' & Corner Column

K	Lc (')	Lu ('')	r	Klu/r	34-12xM1/M2	34-12xM1/M2	Effect of Slenderness	Dead Load	Live Load	Max Fac. Axial Sus. Load
1	10	102	4.2	24.29	40.27	40.00	No	152.42	39.36	213.388
1	12.5	132	4.2	31.43	41.43	40.00	No	153.09	36.63	214.326
1	15	162	4.2	38.57	42.37	40.00	No	153.71	39.89	215.194
1	17.5	192	4.2	45.71	43.18	40.00	Yes	154.3	40.13	216.02
1	20	222	4.2	52.86	43.75	40.00	Yes	154.85	40.36	216.79

Max Fac. Axial Load	Bd	Ig	Ec	EI	Pc (Kip)	Cm	Cm	Pu (kip)	Del (Non Sway)	Pu/Pc	(klu/r)/ (34-12M1/M2)
280.3	0.76	3201	3.60E+06	2.62E+09	2486.37	0.39	0.40	280.30	0.47	0.11	0.61
276.597	0.77	3201	3.60E+06	2.60E+09	1473.27	0.35	0.40	276.60	0.53	0.19	0.79
283.007	0.76	3201	3.60E+06	2.62E+09	986.18	0.32	0.40	283.01	0.65	0.29	0.96
284.241	0.76	3201	3.60E+06	2.62E+09	702.24	0.29	0.40	284.24	0.87	0.40	1.14
285.402	0.76	3201	3.60E+06	2.62E+09	525.38	0.27	0.40	285.40	1.45	0.54	1.32

Panel Size, 20'x20' & Corner Column

K	Lc (')	Lu ('')	r	Klu/r	34-12xM1/M2	34-12xM1/M2	Effect of Slenderness	Dead Load	Live Load	Max Fac. Axial Sus. Load
1	10	102	4.5	22:67	38.37	38.37	No	231.07	63.58	323.498
1	12.5	132	4.5	29.33	39.36	39.36	No	231.76	63.86	324.464
1	15	162	4.5	36.00	40.13	40.00	No	232.41	64.13	325.374
1	17.5	192	4.5	42.67	40.75	40.00	Yes	233.05	60.4	326.27
1	20	222	4.5	49.33	41.26	40.00	Yes	233.65	64.65	327.11

Max Fac. Axial Load	Bd	Ig	Ec	EI	Pc (Kip)	Cm	Cm	Pu (kip)	Del (Non Sway)	Pu/Pc	(klu/r)/ (34-12M1/M2)
431.584	0.75	4219	3.60E+06	3.48E+09	3298.52	0.45	0.45	431.58	0.55	0.13	0.59
433.026	0.75	4219	3.60E+06	3.48E+09	1969.87	0.42	0.42	433.03	0.60	0.22	0.75
434.395	0.75	4219	3.60E+06	3.48E+09	1308.04	0.40	0.40	434.40	0.72	0.33	0.90
428.95	0.76	4219	3.60E+06	3.46E+09	925.08	0.38	0.40	428.95	1.05	0.46	1.07
437.015	0.75	4219	3.60E+06	3.48E+09	696.75	0.38	0.40	437.02	2.44	0.63	1.23

Panel Size, 25'x25' & Corner Column

K	Lc (')	Lu (")	r	Klu/r	34-12xM1/M2	34-12xM1/M2	Effect of Slenderness	Dead Load	Live Load	Max Fac. Axial Sus. Load
1	10	102	4.8	21.25	37.18	37.18	No	326.52	93.82	457.128
1	12.5	132	4.8	27.50	37.96	37.96	No	326.88	93.99	457.632
1	15	162	4.8	33.75	38.88	38.88	No	327.23	94.16	458.122
1	17.5	192	4.8	40.00	38.88	38.88	Yes	327.57	94.33	458.598
1	20	222	4.8	46.25	39.20	39.20	Yes	327.91	94.5	459.074

Max Fac. Axial Load	Bd	Ig	Ec	EI	Pc (Kip)	Cm	Cm	Pu (kip)	Del (Non Sway)	Pu/Pc	(klu/r)/ (34-12M1/M2)
616.622	0.74	5461	3.60E+06	4.52E+09	4290.21	0.49	0.49	616.62	0.61	0.14	0.57
617.415	0.74	5461	3.60E+06	4.52E+09	2561.92	0.47	0.47	617.42	0.69	0.24	0.72
618.194	0.74	5461	3.60E+06	4.52E+09	1701.05	0.45	0.45	618.19	0.87	0.36	0.87
618.959	0.74	5461	3.60E+06	4.52E+09	1211.11	0.45	0.45	618.96	1.41	0.51	1.03
619.724	0.74	5461	3.60E+06	4.52E+09	905.97	0.43	0.43	619.72	4.85	0.68	1.18

Panel Size, 15'x15' & Edge Column

K	Lc (')	Lu (")	r	Klu/r	34-12xM1/M2	34-12xM1/M2	Effect of Slenderness	Dead Load	Live Load	Max Fac. Axial Sus. Load
1	10	102	4.5	22.67	29.44	29.44	No	226.3	64.71	316.82
1	12.5	132	4.5	29.33	27.61	27.61	Yes	225.5	64.47	315.7
1	15	162	4.5	36:00	24.85	24.85	Yes	224.71	64.23	314.594
1	17.5	192	4.5	42.67	23.20	23.20	Yes	223.93	64	313.502
1	20	222	4.5	49.33	26.44	26.44	Yes	223.18	63.77	312.452

Max Fac. Axial Load	Bd	Ig	Ec	EI	Pc (Kip)	Cm	Cm	Pu (kip)	Del (Non Sway)	Pu/Pc	(klu/r)/ (34-12M1/M2)
426.827	0.74	4219	3.60E+06	3.49E+09	3312.33	0.75	0.75	426.83	0.91	0.13	0.77
425.299	0.74	4219	3.60E+06	3.49E+09	1977.78	0.81	0.81	425.30	1.14	0.22	1.06
423.785	0.74	4219	3.60E+06	3.49E+09	1313.06	0.91	0.91	423.79	1.59	0.32	1.45
422.302	0.74	4219	3.60E+06	3.49E+09	934.77	0.91	0.91	422.30	2.28	0.45	1.84
420.861	0.74	4219	3.60E+06	3.49E+09	699.18	0.85	0.85	420.86	4.31	0.60	1.87

Panel Size, 20'x20' & Edge Column

K	Lc (')	Lu ('')	r	Klu/r	34-12xM1/M2	34-12xM1/M2	Effect of Slenderness	Dead Load	Live Load	Max Fac. Axial Sus. Load
1	10	102	5.1	20.00	31.24	31.24	No	390.35	117.89	546.49
1	12.5	132	5.1	25.88	30.70	30.70	No	389.39	117.58	545.146
1	15	162	5.1	31.76	29.53	29.53	Yes	388.42	117.26	543.788
1	17.5	192	5.1	37.65	27.88	27.88	Yes	387.46	116.95	542.444
1	20	222	5.1	43.53	40.33	40.00	Yes	386.49	116.63	541.086

Max Fac. Axial Load	Bd	Ig	Ec	EI	Pc (Kip)	Cm	Cm	Pu (kip)	Del (Non Sway)	Pu/Pc	(klu/r)/ (34-12M1/M2)
746.903	0.73	6960	3.60E+06	5.80E+09	5498.10	0.69	0.69	746.90	0.85	0.14	0.64
745.032	0.73	6960	3.60E+06	5.80E+09	3282.89	0.71	0.71	745.03	1.02	0.23	0.84
743.13	0.73	6960	3.60E+06	5.80E+09	2179.53	0.75	0.75	743.13	1.37	0.34	1.08
741.259	0.73	6960	3.60E+06	5.80E+09	1551.61	0.80	0.80	741.26	2.21	0.48	1.35
739.357	0.73	6960	3.60E+06	5.80E+09	1160.56	0.88	0.88	739.36	5.84	0.64	1.09

Panel Size, 25'x25' & Edge Column

K	Lc (')	Lu ('')	r	Klu/r	34-12xM1/M2	34-12xM1/M2	Effect of Slenderness	Dead Load	Live Load	Max Fac. Axial Sus. Load
1	10	102	6	17.00	35.08	35.08	No	608.29	188.99	851.606
1	12.5	132	6	22.00	35.71	35.71	No	607.85	188.85	850.99
1	15	162	6	27.00	35.99	35.99	No	607.39	188.7	850.346
1	17.5	192	6	32.00	35.84	35.84	No	606.92	188.55	849.688
1	20	222	6	37.00	36.00	36.00	Yes	606.44	188.39	849.016

Max Fac. Axial Load	Bd	Ig	Ec	EI	Pc (Kip)	Cm	Cm	Pu (kip)	Del (Non Sway)	Pu/Pc	(klu/r)/ (34-12M1/M2)
1172.889	0.73	13333	3.60E+06	1.11E+10	10566.79	0.56	0.56	1172.89	0.66	0.11	0.48
1172.035	0.73	13333	3.60E+06	1.11E+10	6309.50	0.54	0.54	1172.04	0.72	0.19	0.62
1171.136	0.73	13333	3.60E+06	1.11E+10	4189.00	0.53	0.53	1171.14	0.85	0.28	0.75
1170.223	0.73	13333	3.60E+06	1.11E+10	2982.20	0.53	0.53	1170.22	1.12	0.39	0.89
1169.279	0.73	13333	3.60E+06	1.11E+10	2230.65	0.53	0.53	1169.28	1.77	0.52	1.03

Panel Size, 15'x15' & Inner Column

K	Lc (')	Lu (")	r	Klu/r	34-12xM1/M2	34-12xM1/M2	Effect of Slenderness	Dead Load	Live Load	Max Fac. Axial Sus. Load
1	10	120	4.8	25.00	37.39	37.39	No	211.47	64.98	296.058
1	12.5	150	4.8	31.25	37.67	37.67	No	209.42	64.2	293.188
1	15	180	4.8	37.50	37.85	37.85	No	207.56	63.48	290.584
1	17.5	210	4.8	43.75	38.00	38.00	Yes	205.87	62.83	288.218
1	20	240	4.8	50.00	38.12	38.12	Yes	204.32	62.24	286.048

Max Fac. Axial Load	Bd	Ig	Ec	EI	Pc (Kip)	Cm	Cm	Pu (kip)	Del (Non Sway)	Pu/Pc	(klu/r)/ (34-12M1/M2)
406.524	0.73	5461	3.60E+06	4.56E+09	3123.13	0.49	0.49	406.52	0.59	0.13	0.67
402.328	0.73	5461	3.60E+06	4.56E+09	1998.27	0.48	0.48	402.33	0.65	0.20	0.83
398.5	0.73	5461	3.60E+06	4.55E+09	1387.31	0.47	0.47	398.50	0.76	0.29	0.99
395.029	0.73	5461	3.60E+06	4.55E+09	1019.00	0.47	0.47	395.03	0.97	0.39	1.15
391.856	0.73	5461	3.60E+06	4.55E+09	780.01	0.46	0.46	391.86	1.40	0.50	1.31

Panel Size, 20'x20' & Inner Column

K	Lc (')	Lu (")	r	Klu/r	34-12xM1/M2	34-12xM1/M2	Effect of Slenderness	Dead Load	Live Load	Max Fac. Axial Sus. Load
1	10	120	5.4	22.22	38.46	38.46	No	323.14	102.16	452.396
1	12.5	150	5.4	27.78	38.99	38.99	No	318.66	100.53	446.124
1	15	180	5.4	33.33	39.42	39.42	No	314.56	99.04	440.384
1	17.5	210	5.4	38.89	39.84	39.84	No	310.8	97.67	435.12
1	20	240	5.4	44.44	40.33	40.00	Yes	307.32	96.4	430.248

Max Fac. Axial Load	Bd	Ig	Ec	EI	Pc (Kip)	Cm	Cm	Pu (kip)	Del (Non Sway)	Pu/Pc	(klu/r)/ (34-12M1/M2)
626.068	0.72	8748	3.60E+06	7.32E+09	5019.11	0.45	0.45	626.07	0.54	0.12	0.58
617.025	0.72	8748	3.60E+06	7.32E+09	3211.44	0.43	0.43	617.03	0.58	0.19	0.71
608.752	0.72	8748	3.60E+06	7.32E+09	2229.65	0.42	0.42	608.75	0.66	0.27	0.85
601.159	0.72	8748	3.60E+06	7.32E+09	1637.75	0.41	0.41	601.16	0.79	0.37	0.98
594.128	0.72	8748	3.60E+06	7.32E+09	1253.64	0.39	0.40	594.13	1.09	0.47	1.11

Panel Size, 25'x25' & Inner Column

K	Lc (')	Lu (")	r	Klu/r	34-12xM1/M2	34-12xM1/M2	Effect of Slenderness	Dead Load	Live Load	Max Fac. Axial Sus. Load
1	10	120	7.2	16.67	36.64	36.64	No	692.25	225.56	969.15
1	12.5	150	7.2	20.83	37.25	37.25	No	687.47	224.79	962.458
1	15	180	7.2	25.00	37.67	37.67	No	682.92	223.09	956.088
1	17.5	210	7.2	29.17	37.97	37.97	No	678.56	221.46	949.984
1	20	240	7.2	33.33	38.19	38.19	No	674.39	219.9	944.146

Max Fac. Axial Load	Bd	Ig	Ec	EI	Pc (Kip)	Cm	Cm	Pu (kip)	Del (Non Sway)	Pu/Pc	(klu/r)/(34-12M1/M2)
1352.602	0.72	27648	3.60E+06	2.32E+10	15919.16	0.51	0.51	1352.60	0.58	0.08	0.45
1344.601	0.72	27648	3.60E+06	2.32E+10	10192.50	0.49	0.49	1344.60	0.60	0.13	0.56
1335.341	0.72	27648	3.60E+06	2.32E+10	7077.32	0.48	0.48	1335.34	0.64	0.19	0.66
1326.466	0.72	27648	3.60E+06	2.32E+10	5199.09	0.47	0.47	1326.47	0.71	0.26	0.77
1317.976	0.72	27648	3.60E+06	2.32E+10	3980.13	0.46	0.46	1317.98	0.82	0.33	0.87

B.4 SWAY MOMENT MAGNIFICATION FACTOR DETERMINATION FOR FLAT PLATE STRUCTURE

Panel Size, 15'x15' & Corner Column

K	Lc (')	Lu (")	r	Klu/r	Effect of Slenderness	SUM Pu	SUM Vu	DEL(in)	Q	Q	Del (Sway)	(Klu/r)/22
1.63	10	102	4.2	39.59	Yes	942.98	16.42	0.11	0.05	Sway Frame	1.05	1.80
1.60	12.5	132	4.2	50.29	Yes	945.42	14.48	0.17	0.07	Sway Frame	1.08	2.29
1.58	15	162	4.2	60.94	Yes	947.70	12.78	0.24	0.10	Sway Frame	1.11	2.77
1.57	17.5	192	4.2	71.77	Yes	949.90	11.32	0.33	0.13	Sway Frame	1.15	3.26
1.55	20	222	4.2	81.93	Yes	959.66	10.20	0.43	0.17	Sway Frame	1.20	3.72

Panel Size, 20'x20' & Corner Column

K	Lc (')	Lu (")	r	Klu/r	Effect of Slenderness	SUM Pu	SUM Vu	DEL(in)	Q	Q	Del (Sway)	(Klu/r)/22
1.81	10	102	4.5	41.03	Yes	1335.58	12.22	0.06	0.05	Sway Frame	1.05	1.86
1.79	12.5	132	4.5	52.51	Yes	1339.60	10.62	0.08	0.07	Sway Frame	1.08	2.39
1.77	15	162	4.5	63.72	Yes	1343.48	9.18	0.13	0.11	Sway Frame	1.12	2.90
1.76	17.5	192	4.5	75.09	Yes	1347.24	8.00	0.15	0.12	Sway Frame	1.14	3.41
1.90	20	222	4.5	93.73	Yes	1358.00	7.10	0.19	0.15	Sway Frame	1.18	4.26

Panel Size, 25'x25' & Corner Column

K	Lc (')	Lu (")	r	Klu/r	Effect of Slenderness	SUM Pu	SUM Vu	DEL(in)	Q	Q	Del (Sway)	(Klu/r)/22
2.55	10	102	4.8	54.19	Yes	1951.34	16.24	0.07	0.07	Sway Frame	1.08	2.46
2.46	12.5	132	4.8	67.65	Yes	1955.76	14.50	0.10	0.09	Sway Frame	1.10	3.08
2.40	15	162	4.8	81.00	Yes	1960.10	13.00	0.14	0.12	Sway Frame	1.13	3.68
2.35	17.5	192	4.8	94.00	Yes	1964.40	11.70	0.18	0.15	Sway Frame	1.17	4.27
2.32	20	222	4.8	107.30	Yes	1977.04	10.70	0.23	0.18	Sway Frame	1.22	4.88

Panel Size, 15'x15' & Edge Column

K	Lc (')	Lu (")	r	Klu/r	Effect of Slenderness	SUM Pu	SUM Vu	DEL(in)	Q	Q	Del (Sway)	(Klu/r)/22
1.49	10	102	4.5	33.77	Yes	942.98	16.42	0.11	0.05	Sway Frame	1.05	1.54
1.46	12.5	132	4.5	42.83	Yes	945.42	14.48	0.17	0.07	Sway Frame	1.08	1.95
1.44	15	162	4.5	51.84	Yes	947.70	12.78	0.24	0.10	Sway Frame	1.11	2.36
1.43	17.5	192	4.5	61.01	Yes	949.90	11.32	0.33	0.13	Sway Frame	1.15	2.77
1.42	20	222	4.5	70.05	Yes	959.66	10.20	0.43	0.17	Sway Frame	1.20	3.18

Panel Size, 20'x20' & Edge Column

K	Lc (')	Lu (")	r	Klu/r	Effect of Slenderness	SUM Pu	SUM Vu	DEL(in)	Q	Q	Del (Sway)	(Klu/r)/22
1.93	10	102	5.1	38.60	Yes	1335.58	12.22	0.06	0.05	Sway Frame	1.05	1.75
1.86	12.5	132	5.1	48.14	Yes	1339.60	10.62	0.08	0.07	Sway Frame	1.08	2.19
1.82	15	162	5.1	57.81	Yes	1343.48	9.18	0.13	0.11	Sway Frame	1.12	2.63
1.79	17.5	192	5.1	67.39	Yes	1347.24	8.00	0.15	0.12	Sway Frame	1.14	3.06
1.77	20	222	5.1	77.05	Yes	1358.00	7.10	0.19	0.15	Sway Frame	1.18	3.50

Panel Size, 25'x25' & Edge Column

K	Lc (')	Lu (")	r	Klu/r	Effect of Slenderness	SUM Pu	SUM Vu	DEL(in)	Q	Q	Del (Sway)	(Klu/r)/22
2.78	10	102	6	47.26	Yes	1951.34	16.24	0.07	0.07	Sway Frame	1.08	2.15
2.68	12.5	132	6	58.96	Yes	1955.76	14.50	0.10	0.09	Sway Frame	1.10	2.68
2.61	15	162	6	70.47	Yes	1960.10	13.00	0.14	0.12	Sway Frame	1.13	3.20
2.56	17.5	192	6	81.92	Yes	1964.40	11.70	0.18	0.15	Sway Frame	1.17	3.72
2.52	20	222	6	93.24	Yes	1977.04	10.70	0.23	0.18	Sway Frame	1.22	4.24

Panel Size, 15'x15' & Inner Column

K	Lc (')	Lu (")	r	Klu/r	Effect of Slenderness	SUM Pu	SUM Vu	DEL(in)	Q	Q	Del (Sway)	(Klu/r)/22
1.58	10	111	4.8	36.70	Yes	960.20	30.48	0.12	0.03	Nonsway Frame	1.03	1.67
1.55	12.5	141	4.8	45.69	Yes	954.82	25.44	0.18	0.04	Nonsway Frame	1.05	2.08
1.53	15	171	4.8	54.66	Yes	950.02	21.64	0.25	0.06	Sway Frame	1.06	2.48
1.51	17.5	201	4.8	63.38	Yes	945.66	18.70	0.33	0.08	Sway Frame	1.09	2.88
1.50	20	231	4.8	72.34	Yes	945.78	16.48	0.43	0.10	Sway Frame	1.12	3.29

Panel Size, 20'x20' & Inner Column

K	Lc (')	Lu (")	r	Klu/r	Effect of Slenderness	SUM Pu	SUM Vu	DEL(in)	Q	Q	Del (Sway)	(Klu/r)/22
2.11	10	35	5.4	13.60	No	1565.82	27.66	0.06	0.03	Nonsway Frame	1.03	0.62
2.04	12.5	65	5.4	24.48	Yes	1555.84	22.48	0.09	0.04	Nonsway Frame	1.04	1.11
1.99	15	95	5.4	34.94	Yes	1546.68	18.64	0.12	0.06	Sway Frame	1.06	1.59
1.96	17.5	125	5.4	45.30	Yes	1538.22	15.72	0.16	0.07	Sway Frame	1.08	2.06
1.93	20	155	5.4	55.33	Yes	1537.8	13.58	0.20	0.09	Sway Frame	1.10	2.51

Panel Size, 25'x25' & Inner Column

K	Lc (')	Lu (")	r	Klu/r	Effect of Slenderness	SUM Pu	SUM Vu	DEL(in)	Q	Q	Del (Sway)	(Klu/r)/22
3.90	10	35	7.2	18.85	No	2682.02	42.70	0.07	0.04	Nonsway Frame	1.04	0.86
3.75	12.5	65	7.2	33.75	Yes	2675.70	36.98	0.11	0.05	Sway Frame	1.05	1.53
3.65	15	95	7.2	48.06	Yes	2669.68	32.46	0.14	0.06	Sway Frame	1.07	2.18
3.57	17.5	125	7.2	61.88	Yes	2663.94	28.80	0.18	0.08	Sway Frame	1.09	2.81
3.51	20	155	7.2	75.47	Yes	2668.46	25.98	0.23	0.10	Sway Frame	1.11	3.43