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# Column Capacity Evaluation For Low-Rise Moment Frame Steel Buildings In Mississippi Subject To Wind And Seismic Hazards

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COLUMN CAPACITY EVALUATION FOR LOW-RISE MOMENT FRAME STEEL BUILDINGS IN MISSISSIPPI SUBJECT TO WIND AND SEISMIC HAZARDS

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ABSTRACT

For the analysis of a structure, designers must choose an appropriate model and analysis procedure to reflect the system's response to the applied load. When extreme lateral forces are applied to structure, the second-order effects should be considered in building design. This thesis mainly focuses on so-called P-delta effects and their influence on critical column capacity evaluation under multi-hazard conditions in low rise steel frame buildings characteristic of locations throughout the state of Mississippi.

For the design of steel frame structures, AISC 360-05 Specification outlines specification-based approaches of second- order analysis. In this study, two of these approaches are considered: 1) Amplified First-Order Analysis Method and 2) Direct Analysis Method. The methods were studied using a commercially available software package known as SAP2000. Amplification of moments in a two-story plane frame subject to a representative wind load case is computed using both methods and shown to be of comparable magnitude for small deformations.

ASCE/SEI 7 and IBC provisions were used to develop lateral design load conditions for the frames. Wind and earthquake are two common natural hazards influencing building design in Mississippi. In this study, a hypothetical building was assumed to be constructed in four different locations (Southaven, Batesville, Jackson and Gulfport) in Mississippi capturing a range of relative importance for these two hazards.

For a three-story moment resisting frame building, second-order effects were developed for each column using the Amplified First Order Analysis Method. Interaction ratios were then used as the basis for establishing capacity of each column and thus critical members and dominant hazard at a particular location. As expected, Southaven and Batesville are shown to be dominated by the earthquake hazard and Gulfport by the wind hazard. Jackson is roughly equally influenced by both hazards. Jackson may thus be considered a boundary for dividing the state into seismic dominated versus wind dominated zones. This evaluation may give insight to building designers in preliminary design.

# DEDICATION

This thesis is dedicated to my husband, Yuanqing Ding and my parents, Youfu Shi and Xiuying Li, for their support and motivation. In addition, this thesis is dedicated to my children, Joanna and Joshua, for the happiness they gave to me in the past two years.

LIST OF ABBREVIATIONS AND SYMBOLS

$B_1$ An amplifier to account for second order effect caused by displacements between brace points

$B_2$ An amplifier to account for second order effect caused by displacements of braced points

$C_m$ A coefficient assuming no lateral translation of the frame

$C_s$ Seismic response coefficient

$C_t$ Building period coefficient

$C_{vx}$ Vertical distribution factor

D Dead load

E Eartthquake load

E Modulus of elasticity

EI*, EA* Reduced flexural and axial stiffness

$F_a$ Short period site coefficient

$F_v$ Long-period site coefficient

$F_x$ Portion of the seismic base shear

$h_n$ Structure height

I Moment of inertia in the plane of bending

I The important factor

k Distribution exponent

$K_1$ Effective length factor in the plane of bending, calculated based on the assumption of no lateral translation

$K_2$ Effective length factor in the plane of bending, calculated based on a side-sway buckling analysis

$K_{zt}$ Topographic factor

L Live load

L Length of column

LR Roof live load

$M_1$, $M_2$ Smaller and larger first-order moments at the ends of that portion of the member unbraced in the plane of bending under consideration.

$M_{cx}$ Available flexural strength to strong axis bending

$M_{cy}$ Available flexural strength to weak axis bending

$M_{lt}$ First-order moment using LRFD load combinations caused by lateral translation of the frame only

$M_{nt}$ First-order moment using LRFD load combinations, assuming there is no lateral translation of the frame

$M_r$ Required second-order flexural strength using LRFD load combinations

$M_{rx}$ Required flexural strength to strong axis bending

$M_{ry}$ Required flexural strength to weak axis bending

$N_i$ Nominal loads

$P_c$ Available axial compressive strength

$P_{e1}$ Elastic critical buckling resistance of the member in the plane of bending

$P_{lt}$ First-order axial force using LRFD load combinations caused by lateral translation of the frame only

$P_{net}$ Net design wind pressure on a parapet

$P_{net30}$ Simplified design wind pressure for exposure B at h=30 ft and I=1.0

$P_{nt}$ First-order axial force using LRFD load combinations, assuming there is no lateral translation of the frame

$P_r$ Required axial compressive strength

$P_r$ Required second-order axial strength using LRFD load combination

$P_r$ Required second-order axial strength using LRFD load combinations

$P_u$ Required concentrated beam load using LRFD load combinations

R Response modification coefficient

$S_1$ 5 percent damped spectral response acceleration parameter at a period of 1s

$S_{D1}$ Design, 5 percent damped, spectral response acceleration parameter at a period of 1 s

$S_{DS}$ Design, 5 percent damped, spectral response acceleration parameter at short periods

$S_{M1}$ 5 percent damped, spectral response acceleration parameter at a period of 1 s adjusted for site class effects

$S_{MS}$ 5 percent damped, spectral response acceleration parameter at short period adjusted for site class effects

$S_S$ 5 percent damped spectral response acceleration parameter at short period

$T_a$ Approximated fundamental periods of the building

v The basic wind speed

V Total design lateral force or shear at the base

W Wind load

$W$ Effective seismic weight of the building

$x$ Design level

$Y_1$ Distance from top of the steel beam to plastic neutral axis

$Y_2$ Distance from top of the steel beam to concrete flange force in a composite beam

$Y_i$ Design gravity load at each level

$\alpha$ Effective concrete flange thickness of a composite beam

$\lambda$ Adjustment factor for building height and exposure

$\Sigma H$ Story shear produced by the lateral forces

$\Sigma P_{e2}$ Elastic critical buckling resistance of the member in the plane of bending

$\Sigma P_{nt}$ Total vertical load supported by the story using LRFD load combinations, including gravity column loads

$\Phi P_n$ Design compressive strength

$\Delta H$ First-order interstory drift due to lateral forces

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# CHAPTER I

# INTRODUCTION

## 1.1 Background and Motivations

For the analysis of a structure, designers must choose an appropriate model and analysis procedure to reflect the system's response to the applied load. Because the common elastic methods of structural analysis assume that all deformations are small, results of these elastic analyses are referred as first-order forces. A first-order elastic analysis is performed based on the un-deformed configuration of the structure and material of the structure is assumed as linear-elastic. This method is simple to perform but is not enough to reflect the actual response of the structure. When extreme lateral loads, such as a wind load, an earthquake or a blast load, are applied to the structure, the second-order effects should be considered in the building design. In the second-order analysis, the equilibrium is calculated on the deformed geometry of the structure. The second-order analysis is "always necessary for the stability consideration of structures" [2].

For the design of steel moment frame structures, the American Institute of Steel Construction (AISC) outlines specification-based approaches of the second- order analysis both in the 2005 Load Resistance Factor Design (LRFD) and Allowable Stress Design (ASD) Specification, Standard AISC 360-05 [1] for stability assessment. This report will focus on the column design of a moment-resisting steel frame building which is subjected to both wind and earthquake loadings.

Because wind and earthquakes are the two common natural hazards in the state of Mississippi, a hyperthetical steel frame building assumed to be constructed in four locations in Mississippi is used to capture a range of relative importance for wind and seismic loading effects. North Mississippi is located near the termination point of the New Madrid fault while the southern border of the state terminates at the Gulf of Mexico. That is, the state is not dominated by a single hazard and is exposed to both severe wind and earthquake. Current analysis and design procedures are only specialized to individual hazards. Multiple hazards evaluation in this thesis will more effectively identify the balance of two load cases in specified locations and meet a specific hazard only in design.

## 1.2 Literature Review

### 1.2.1 Background of Second-order Effects

Mashary and Chen [9] introduced several second-order effects including P-delta effects, column axial shortening effects, semi-rigid behavior of connections, panel-zone effect, non-uniform temperature effects, out-of-straightness and out-of-plumbness effects, residual stresses and imperfections, and the redistribution effect. Note that this thesis mainly focuses on the P-delta effects. In the following content, second-order effects will refer to P-delta effects only. Out-of-straightness and out-of-plumbness effects, residual stresses, and imperfections will be considered in one of analysis methods.

### 1.2.2 Definition of P-delta Effects

According to Chen & Lui [2], there are two types of secondary effects: the P- δ (P-small delta) effect and P-Δ (P-big delta) effect. These effects cause the member to deform more and induce additional stresses in the member. As a result, they have a weakening effect on the structure.

P-delta effects are also defined in AISC 360-05 Specification [1]. P- δ is the effect of axial loads(P) acting on the deflected shape of a member between joints or nodes. The magnitude of this additional second-order moment depends upon the properties of the member itself. P-Δ is the effect of axial loads (P) acting on the deformed location of joints or nodes in a structure (Figure1.1). Both of these second-order effects must be considered in the frame structures design.

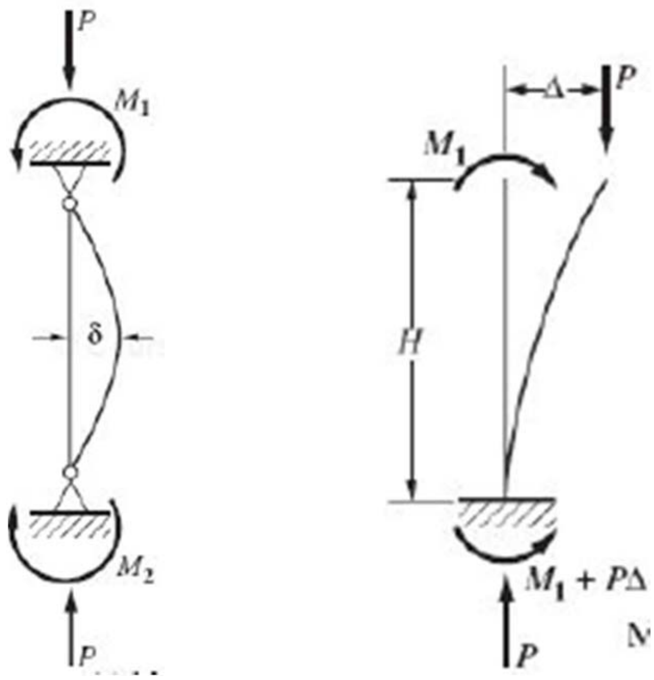


Figure 1.1 P- δ and P-Δ Effects [7]

### 1.2.3 Simplified Second-order Analysis Procedures

Chen and Lui [2] develop several simplified methods to approximate the second-order solution and make the analysis more efficient. The first one is the Two Cycles Iterative Method. In this method, first-order axial loads are first obtained and the stiffness matrix is modified to obtain the second-order effects. This method is time-consuming for multi-member frames.

The second method is the Fictitious Lateral Load Method that includes only frame (P-Δ) effect. In this method, first-order deflections are determined first. Fictitious loads are calculated next and are applied to the frame structure to simulate P-Δ effects and then another first-order analysis is performed to obtain new deflections. The procedure repeats until the moment of member do not change apparently. This method is simple since the stiffness matrix does not change. However, this method does not consider P-δ effects.

The third one is the Iterative Gravity Load Method that also considers P-Δ effects only. In this method, axial and lateral loads are applied to the frame to determine the lateral deflection and moment caused by the deflection. Based on the deflection, the local coordinates are changed and the frame is re-analyzed. The procedure repeats until the difference of deflection becomes very small and can be neglected. This method is simplified by creating a fictitious bay which is composed of axially rigid columns with zero flexural rigidity. Finally, only fictitious member coordinates need to be updated.

Another method is the Negative Stiffness Method. In this method, P-Δ effects are performed by reducing the lateral stiffness of the frame. Above simplified methods can complete the second-order by hand. However, most of them only include P-Δ effects.

### 1.2.4 Analysis Methods Based upon AISC 360-05 Specification

Galambos and Surovek [4] introduced some specification-based applications of stability in steel structural design. They focus on the recent AISC 360-05 Specification [1] and provide the design equations included in the specification. The first approach is called first-order elastic analysis with amplified factors. The use of this method is limited to the side-sway amplification less than 1.1. Specifically, this approach must satisfied the following criteria: (1) the required axial load must be less than or equal to one-half of the yield strength of the member; (2) all load combinations must include an additional load at each level; (3) the $B_1$ moment amplification factor must be used with the total moments in order to account for non-sway members. This method assumes that the three criteria must apply to all members and all stories in the structure.

The other method called Direct Second-order Analysis Method specified in AISC 360-05 Specification Appendix 7 becomes more common to address stability requirements in steel framing system. The new edition of AISC 360-10 Specification will move the direct analysis method from Appendix to Chapter C, whereas other two methods, effective length method and first-order analysis method will be relegated to Appendix. AISC committee members suggested this change because direct analysis method is a more accurate determination of the load effects in the structure [14]. This reorganization indicates the significance of direct analysis method in stability design in steel structures.

Halvorson, Warner and Lang [15] introduced an application of direct analysis method for addressing stability in a Russia Tower design. In the column stability design of this tallest building in Europe, the software package ETABs was used to perform the direct analysis method. The iterative approximated P-delta method in ETABs can capture deformations and secondary forces. Flexure, axial and shear deformation were also accounted for by ETABs analysis. Since columns were highly stressed members, notional loads which are a proportion of the gravity load applied laterally were defined as 0.3% gravity loads instead of 0.2% gravity load to address imperfections and out-of-straightness. Material non-linearity and residual stresses were considered by applying modifiers to members and material. The modifier for this case was defined as 75% of column moments of inertias. Using this method, the final column designs represent the most conservative design for each member.

Nair [22] discussed a model specification for stability design by the direct analysis method. This paper expanded some of the provisions beyond what is in the current AISC 360-05 Specification. Geometric imperfections, stiffness reduction due to inelasticity, and uncertainty in strength and stiffness were specified in greater detail. A design that conformed to this model specification would also conform to the stability provision of the AISC Specification 2005.

Muir and Duncan [14] introduced the new edition of AISC Specification, AISC 2010 Specification 14$^{th}$ edition. It will be available in the summer of 2011. In the new specification, direct analysis method will move from Appendix 7 to main section, Chapter C "Stability Analysis and Design" as a method of stability design. An additional requirement of the uncertainty in stiffness and strength will be considered in direct analysis method.

## 1.3 Objectives and Major Tasks

The purpose of this study is to demonstrate the column design implications of AISC second-order analysis procedures for typical low-rise steel frame buildings in Mississippi under multi-hazard exposures. Earthquake and wind are two major hazards in Mississippi. Both of which generate severe lateral load. During an earthquake or wind event (including thunderstorm, hurricane, and tornado), columns are the most directly affected element in a moment frame structural system.

Seismic design loads are computed in this study using IBC2006 [5] and new post-hurricane Katrina provisions ASCE/SEI 7-10 [6] is used to compute wind design loads. Required strengths are calculated member by member for each hazard specific load combination. AISC interaction equations are used to evaluate the capacity of each column. The effect of location within the state on the capacity of critical 1$^{st}$ floor columns is demonstrated four locations representing distinct hazard exposure.

A building model is developed using commercially available finite element software [10].

## 1.4 Scope of Work and Organization of Report

Chapter 2 presents two specification-based methods to calculate second-order effects. The first method called the Amplified First-order Elastic Analysis Method which is presented in AISC 360-05 Specification Chapter C, uses easily magnification factors to increase the first-order axial force and moments in the members. This is an approximate method which will soon be removed from the main part of AISC Specification [14]. The second one called the Direct Analysis Method which is presented in AISC 360-05 Specification Appendix requires in general use of advanced commercial or academic software to directly calculate second-order forces and moments. The calculated second-order required axial forces and bending moments can be used in steel frame stability design. Two simple examples will be introduced to demonstrate the calculation procedures of second-order effects using two methods.

Chapter 3 focuses on multi-hazard design of low-rise steel moment frame building columns in Mississippi. The hypothetical steel frame building assumed constructed in different four locations in Mississippi will be used to evaluate seismic and wind loading effects. As required in AISC 360-05 Specification, second-order effects must be considered in stability design for frame building. In this chapter, when the building is subjected to seismic and wind loads, the amplified first-order method will be used to calculate the second-order effects for each column. Then interaction ratios will be calculated to check the system stability. From interaction ratios of each column, designer can identify the relative importance of seismic and wind load for this type of frame building in different locations of Mississippi.

Chapter 4 will summarize the conclusions of this study and provide recommendations for future work.

# CHAPTER II

# CALCULATION METHODS OF SECOND-ORDER EFFECTS

## 2.1 Background

For the design of steel frame moment structures, AISC 2005 Specification requires that second-order effects must be considered in stability analysis and design. The second-order effects on a structure are taken into account for combination of P-Δ effects, which correspond to the structure, and P-δ effect, which correspond to individual members within the structure. Since these two effects contribute to the deformation of the structure, it is important to consider their combined effect.

Stability becomes a major concern for moment-resisting frame building design. Structural instability occurs when a structure or a structure component is unable to resist applied loads in the deformed state. The first-order analysis is simple but is not sufficient to design for stability. Thus, a second-order nonlinear analysis is required. When extreme lateral loads, such as earthquake or wind, are applied to the moment frame building, the lateral stability is provided by the flexural stiffness of connected beams and columns [1]. Columns will be subject to both axial loads (dead or live loads) and lateral loads (seismic or wind loads). This type of members can be defined as beam-columns. Their behavior falls between that of a pure, axially loaded column and a beam with only moments applied [7].

## 2.2 Interaction Equations

Second-order effects are considered in beam-columns by using the strength interaction equations that express the combinations of axial loads and bending moments that the member can support.

The interaction of axial load and bending moment of a beam-column can be developed through the techniques of superposition. This approach is normally considered in elementary strength of materials where the normal stress due to an axial force is added to the normal stress due to a bending moment [7].

Three-dimensional interaction equations for doubly and singly symmetric members subjected to flexure and axial forces are provided in Chapter H of the AISC 360-05 Specification. These equations are given by:

For $\frac{P_r}{P_c} \geq 0.2$:

$$\frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0 \qquad \text{(AISC H1-1a)}$$

For $\frac{P_r}{P_c} < 0.2$:

$$\frac{P_r}{2P_c} + \left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0 \qquad \text{(AISC H1-1b)}$$

where

$P_r$ and $M_r$ are required second-order axial compressive and flexural strength

$P_c$ and $M_c$ are available second-order axial compressive and flexural strength

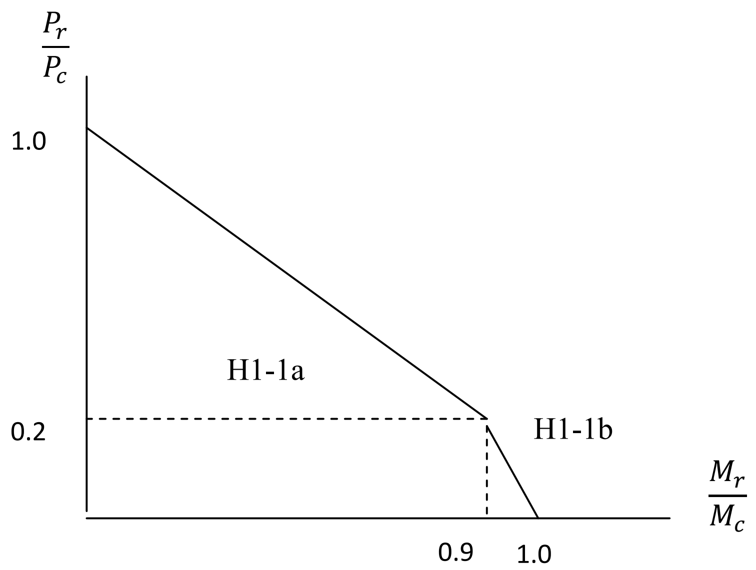


Figure 2.1 Interaction Equations AISC H1-1a and H1-1b

The equations are plotted in Figure 2.1. The interaction equations in AISC Specification consider bending about both principal axes, whereas the plot in Figure 2.1 is for single axis bending [7].

These two interaction equations involve in sums of ratio of required strengths to the available strengths. In applying the interaction equations, the axial capacity and moment capacity should be calculated for each individual member.

## 2.3 Second-order Analysis Methods

In this thesis, the second-order effects defined in AISC 360-05 Specification will be calculated for each column member. Then interaction checks will be performed to evaluate the capacity of each column. If all columns in the frame meet the strength criteria, the structure is considered to be stable.

According to AISC 360-05 Specification, the second-order required strengths can be calculated by two methods. The first method, which is called the Amplified First-order Elastic Analysis Method presented in AISC 360-05 Specification Chapter C, uses magnification factors to increase the first-order axial forces and moments in the members. The second one, which is called Direct Analysis Method presented in AISC 360-05 Specification Appendix 7, can use commercial or academic software that directly calculates second-order forces and moments.

### 2.3.1 the Amplified First-Order Elastic Analysis Method

The Amplified First-Order Elastic Analysis Method is also called $B_1$-$B_2$ Method which is presented in AISC 360-05 Specification Chapter C. In this procedure, amplification factors $B_1$ and $B_2$ are applied to the first-order moments and axial forces in members to obtain an estimate of the second-order forces. The $B_1$ and $B_2$ factors are the P-$\delta$ and P-$\Delta$ moment amplification factors respectively, which account for the displaced geometry of the frame [1]. The following equations are specified in AISC 360-05 to consider second-order effects.

$$\mathrm{M_r} = \mathrm{B_1 M_{nt}} + \mathrm{B_2 M_{lt}}$$

$$\mathrm{P_r} = \mathrm{P_{nt}} + \mathrm{B_2 P_{lt}}$$

Second-order moments and axial loads are considered under two conditions: no translation and lateral translation (NT and LT) separately. $M_{nt}$ is the maximum moment and $P_{nt}$ is the maximum axial load assuming that story sidesway is prevented. $M_{lt}$ is the maximum moment and $P_{lt}$ is the maximum axial load caused by side-sway from lateral loads [4].

It is noted that Load and Resistance Factor Design (LRFD) defined in AISC 2005 Specification will be used throughout the study. The $B_1$ coefficient accounts for amplification of moments due to P-$\delta$ effects or the amplification caused by displacements between braced points [1] which is also known as P-$\delta$ amplification factor . The equation is given by :

$$B_1 = \frac{C_m}{1 - \frac{\alpha P_r}{P_{e1}}} \geq 1.$$

where $\alpha$=1.0 for LRFD .

$B_1$ is directly proportional to the axial load level that is represented by the term $P_r/P_{e1}$. $C_m$ is a coefficient for members braced against joint translation with transverse loading between supports. It is also referred as for equivalent moment factor for members subjected to end moments only [1]. The following equation is presented for the beam-columns subjected to end moment without transverse loading between supports in the plane of bending:

$$C_m = 0.6 - 0.4 \left(\frac{M_1}{M_2}\right)$$

where $M_1$ and $M_2$, calculated from a first-order analysis, are the smaller and larger end moments, respectively, at the ends of that portion of the member un-braced in the plane of bending under consideration. The ratio $\frac{M_1}{M_2}$ is positive for double curvature bending and negative for single curvature [1]. For beam-columns subjected to transverse loading between supports, $C_m$ may be conservatively taken equal to 1.0.

It is possible for $C_m$ less than 1.0 and $B_1$ amplification factor is less than 1.0. This indicates that the combination of the P-$\delta$ effects and the non-uniform moment gradient result in a moment less than the maximum moment on the beam-column from first-order effects. In this case, the amplification factor $B_1$= 1.0.

$P_{e1}$ is the elastic Euler buckling load of the member and is given by [1]:

$$P_{e1} = \frac{\pi^2 EI}{(K_1 L)^2}.$$

where $K_1$ is Effective Length Factor. It can be calculated based upon the assumption of no side-sway which is like a braced frame. In braced frame, the possible K-factor range from 0.5 to 1.0. For a conservative approximation and simplified design, K is often taken as 1.0 [4, 7].

$B_2$ is amplifier to account for second-order effects due to P-Δ effects. When lateral forces, $\sum H$ apply to a frame, the frame will deform laterally until the equilibrium is reached. The corresponding lateral deflection Δ1 is calculated based upon the un-deformed shape of the frame. If the additional vertical forces $\sum P$ are acting on the frame, these vertical forces will interact with the first-order lateral displacement Δ1 to deflect the frame further until the new equilibrium is reached. The lateral deflection Δ corresponds to the new equilibrium position. This phenomenon is called P-Δ effects. The results of this effect are an increase in drift and an increase in over-turning moment [1]. $B_2$ is also called P-Δ amplification factor which is given by [1]:

$$B_2 = \frac{1}{1 - \frac{\alpha \sum P_{nt}}{\sum P_{e2}}} \geq 1.0$$

where $\sum P_{e2}$ is the elastic critical buckling resistance of the story and it can be calculated using the following equation:

$$\sum P_{e2} = \sum \frac{\pi^2 EI}{(K_2 L)^2} = R_M \frac{\sum HL}{\Delta_H}$$

where $R_M$ is 0.85 for moment frames to account for the influence of the member effect on the side-sway displacement. $\sum H$ is the total story shear and $\Delta_H$ is story drift from a first-order analysis due to the lateral load.

It is useful to note that $B_1$ is based on member properties and $B_2$ is a story-based stiffness. Both of them are greater than or equal to 1. In moment frames, the side-sway instability is modeled as a story phenomenon rather than a member phenomenon. The basis of the stability provisions for sway frames is that no single column can buckle in a side-sway mode; instead, all columns in a story buckle simultaneously. Thus, $B_1$ must be calculated for each member and $B_2$ is calculated at each story level [4]. If the load combinations do not include lateral loads, the amplification factor $B_2$ can be set to 0 and first-order analysis should be sufficient.

The second-order amplification affects not only the beam-column, but also the moment in any adjoining members and connections as required by equilibrium.

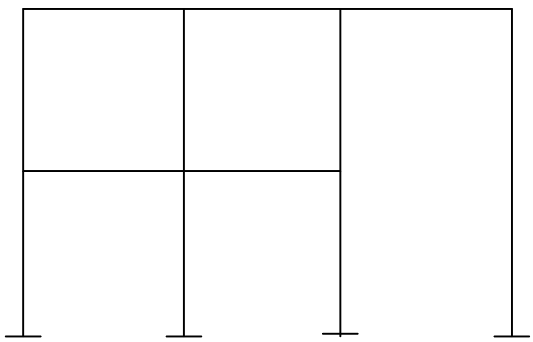

Figure 2.2 Framing System with Non-uniform Story

Unfortunately, the $B_1$ and $B_2$ magnification factors are accurate only if the structure behaves in the same manner as the behavior of the simply structure. For example, if the frame stories do not line up as shown in the Figure 2.2, the $B_2$ factor for the right side column cannot be calculated from the specification approach. Or in some instances, where $B_2$ is greater than 1.2, the direct second-order analysis is recommended in AISC 360-05 Specification.

### 2.3.2 Direct Second-order Analysis

The direct second-order analysis is the most generally applicable method of accounting for P-Δ and P- δ effects, imperfections, and residual stresses. It is a more rigorous analysis method capable of more accurate stability of steel frame structure. This method eliminates K and can be used for all types of steel structures including braced, moment, and combined framing systems.

Using direct second-order analysis to calculate the second-order effects has three features. The first one is that P-Δ and P- δ effects are accounted for through second-order analysis without separated.

The second one requires that geometric imperfections must be account for either by directly modeling these imperfections or by the application of notional loads base on the nominal geometry of the structure. The imperfection can be directly modeled as an L/500 out-of-plumbness. Nominal loads applied as a lateral load are a proportion of the gravity loads which can be expressed as:

$$N_i = 0.002Y_i$$

where $Y_i$ is the gravity load from the appropriate load combinations applied at level $i$.

The third feature is that the analysis can be conducted using reduced stiffness. Stiffness reductions due to residual stresses are accounted for by reducing the flexural stiffness and axial stiffness of members. Reduced flexural and axial stiffness ($EI^* \ and \ EA^*$ ) shall be used for all members whose flexural and axial stiffness is considered to contribute to the lateral stability of the structure [1]. Applying stiffness reduction to only some of members can cause artificial distortions of structure due to given loads. In order to avoid this, stiffness reductions must be applied to all members of structure.

The direct second-order effects can be obtained from commercial or academic software directly. Table 2.1 provides a comparison of three analysis methods.

Table 2.1 Comparison of Analysis Methods

| | Direct analysis method | Amplified first-order elastic analysis | First-order analysis |
|---|---|---|---|
| Specification reference | Appendix 7 | Chapter C. 2. 1b | Chapter C. 2. 2 b |
| Limit on applicability | No | Yes | Yes |
| Type of analysis | Second-order | Second-order | First-order |
| Member stiffness | Reduced EI and EA | Nominal EI and EA | Nominal EI and EA |
| Notional lateral load | Yes | Yes | Yes |
| Column effective length | K=1 | Side-sway buckling analysis | K=1 |

### 2.4 Examples Using Two Methods and Comparison of the Results

#### 2.4.1 Simple Portal Frame

Galambos and Surovek [4] provided an example in their text book: Structure Stability of Steel. A simple portal frame is a regular, one-bay, one-story plane frame as shown in Figure 2.3 a; the given loads are factored loads shown. The second-order forces were calculated using $B_1$-$B_2$ method and the direct second-order analysis method.

This work needs to first determine the second-order effects in the columns using $B_1$-$B_2$ method and then compare the results to the second-order forces and moments in the system using the direct second-order analysis method. The frame model and first-order analysis results as well as the direct second-order analysis results will be obtained from the structure analysis software, SAP2000.

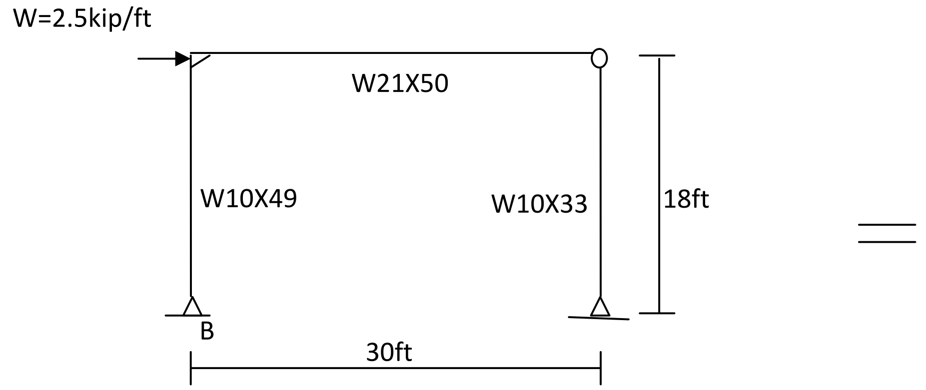


Figure 2.3 a Portal Frame Example

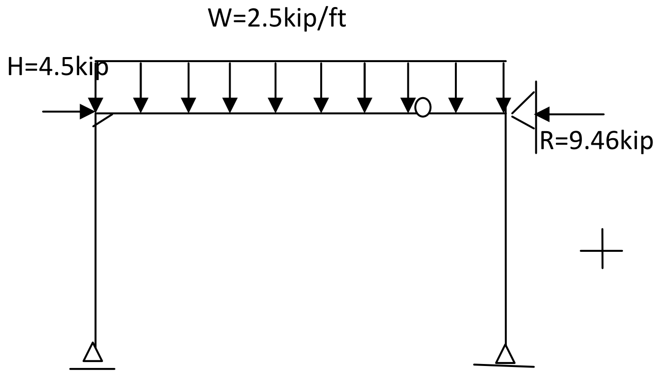



Figure 2.3 b NT Analysis Model

Figure 2.3 c LT Analysis Model

To applying amplified first-order analysis, NT-LT first-order analyses are required. In NT analysis, artificial support is introduced to against lateral translation and the factored loads as shown in Figure 2.3 b. After the first-order analysis is run, the moments and axial loads in the columns and the reaction force at joint C can be obtained. The forces $P_{nt}$ and the moments $M_{nt}$ are recorded in Table 2.2, and the reaction force R=9.46kips.

Table 2.2 NT-LT Analysis Results for Example 2.4.1

| Column No. | NT analysis | | LT analysis | |
|---|---|---|---|---|
| | $P_{nt}$ (kip) | $M_{nt}$ (in-kip) | $P_{lt}$ (kip) | $M_{lt}$ (in-kip) |
| Column AB | -41.228 | 1071.826 | 5.676 | -2043.36 |
| Column CD | -35.273 | 0 | -5.676 | 0 |

For the LT analysis, the reaction R induced in the artificial support is applied to the node C in the reverse direction as shown in Figure 2.3 c. Moments $M_{lt}$ and axial forces $P_{lt}$ obtained from this analysis are recorded in Table 2.2.

Then the $B_1$ and $B_2$ factors are calculated for Column AB. $B_1$ is given by :

$$B_1 = \frac{C_m}{1 - \frac{\alpha P_r}{P_{e1}}} \geq 1.0$$

where

$$C_m = 0.6 - 0.4\left(\frac{M_1}{M_2}\right) = 0.6 - 0.4\left(\frac{0}{1071.826}\right) = 0.6$$

and

$$P_{e1} = \frac{\pi^2 EI}{(K_1 L)^2} = \frac{\pi^2 (29000)(272)}{(1 \times 18 \times 12)^2} = 1670\ kips$$

$K_1$ is taken as 1.0 for a conservative approximation.

$$P_r = P_{nt} + P_{lt} = -41.228 + 5.676 = -35.552kips$$

$$B_1 = \frac{C_m}{1 - \frac{\alpha P_r}{P_{e1}}} = \frac{0.6}{1 - \frac{1 \times 35.552}{1670}} = 0.61 \to B_1 = 1$$

The value of $B_1$<1 indicates that no amplification of the $M_{nt}$ moment is required.

Next, the second-order amplification of the sway moment is determined by calculating $B_2 = \frac{1}{1 - \frac{\alpha \sum P_{nt}}{\sum P_{e2}}} \geq 1.0$, where $\alpha = 1.0$ for LRFD.

$$\sum P_{nt} = 41.228 + 35.273 = 76.501kips$$

$$\sum P_{e2} = \sum \frac{\pi^2 EI}{(K_2 L)^2} = R_m \frac{\sum HL}{\Delta_H} = 0.85 \frac{9.46 \times 18 \times 12}{5.957} = 291.565kips$$

$$B_2 = \frac{1}{1 - \frac{\alpha \sum P_{nt}}{\sum P_{e2}}} = \frac{1}{1 - \frac{76.5}{291.565}} = 1.3557$$

The final forces are given by

$$M_r = B_1 M_{nt} + B_2 M_{lt} = 1071.826 - 1.3557 \times 2043.36 = 1698.36kips$$

$$P_r = P_{nt} + B_2 P_{lt} = -41.228 + 1.3557 \times 5.676 = -33.531kip$$

Same procedure is used in Column CD to obtain the second-order moment and axial force recorded in Table 2.3.

Table 2.3 Second-order Moment and Axial Load Values for Example 2.4.1

| Column No. | Amplified first-order analysis | | Direct second order analysis | |
|---|---|---|---|---|
| | $P_r$ (kip) | $M_r$ (in-kip) | $P_r$ (kip) | $M_r$ (in-kip) |
| Column AB | -33.531 | 1698.35 | -34.753 | 1576.468 |
| Column CD | -42.97 | 0 | -42.629 | 0 |

For direct second-order analysis, SAP2000 automatically includes the structure and member P-Δ and P- δ effects that are required by the direct second-order analysis. The program can automatically generate all notional loads and associated vertical and lateral load combinations to consider geometric imperfections such as the out-of –plumbness effects on a structure. Finally, using the reduced stiffness of 0.8EI to account for residual stresses, SAP2000 can produce the direct analysis results. The second-order forces and moments for each column can be obtained directly from running P-delta analysis. The results are recorded in Table 2.3.

From the results of two methods, the results of amplified first-order analysis are relatively conservative compared to the results of the direct second-order analysis. These results also verify that direct second-order analysis is more accurate.

Table 2.4 shows the results from running a SAP2000 model and the results from reference of Galambos [4].

The results from running SAP2000 and the reference showed in Table 2.4 are very close (less than 3%). This indicates that SAP2000 is an acceptable software package to conduct second-order analysis.

Table 2.4 Comparison of Results from SAP2000 and Text Book for Example 2.4.1

| Column No. | Amplified first-order analysis | | | | Direct second-order analysis | | | |
|---|---|---|---|---|---|---|---|---|
| | SAP2000 | | Reference | | SAP2000 | | Reference | |
| | $P_r$ (kip) | $M_r$ (in-kip) | $P_r$ (kip) | $M_r$ (in-kip) | $P_r$ (kip) | $M_r$ (in-kip) | $P_r$ (kip) | $M_r$ (in-kip) |
| Column AB | -33.5 | 1698 | -33 | 1664 | -35 | 1576 | -33 | 1534 |
| Column CD | -43 | 0 | 42 | 0 | -43 | 0 | -42 | 0 |

### 2.4.2 Example of Two-dimensional Steel Moment Frame

This frame is a regular, two-bay, two-story 2-D frame [12]. Each bay is 30 feet with a total width of 60 feet. The story height is 15ft and the total height is 30 ft. The column ends are pinned and the beams are connected with full moment connections. A dead load of 0.36k/feet is applied to each level's beams. A live load of 2.88k/feet is applied to first level and a roof live load of 1.92k/feet is applied to second level. A 16-kip wind load is applied to left corner of second level, and another 8-kip wind load is applied to left node of first level. All columns are W12x79, and beams are W18x50. The frame is shown in Figure 2.4.

As defined in AISC 2005, load combination 1.2D+0.5L+0.5LR+1.6W will be used in this analysis where D is the dead load, L is live load, LR is roof live load, and W is wind load. Only the columns in lower level will be evaluated. All analyses are run on SAP2000. First no translation results are obtained by applying gravity loads (1.2D+0.5L+0.5LR) only. Next, lateral translation results are obtained by applying lateral loads (1.6W) only. $B_1$ for three columns are calculated as 1.0. $B_2$ for lower level is calculated as 1.035. Results are showed in Table 2.5.

2.4).

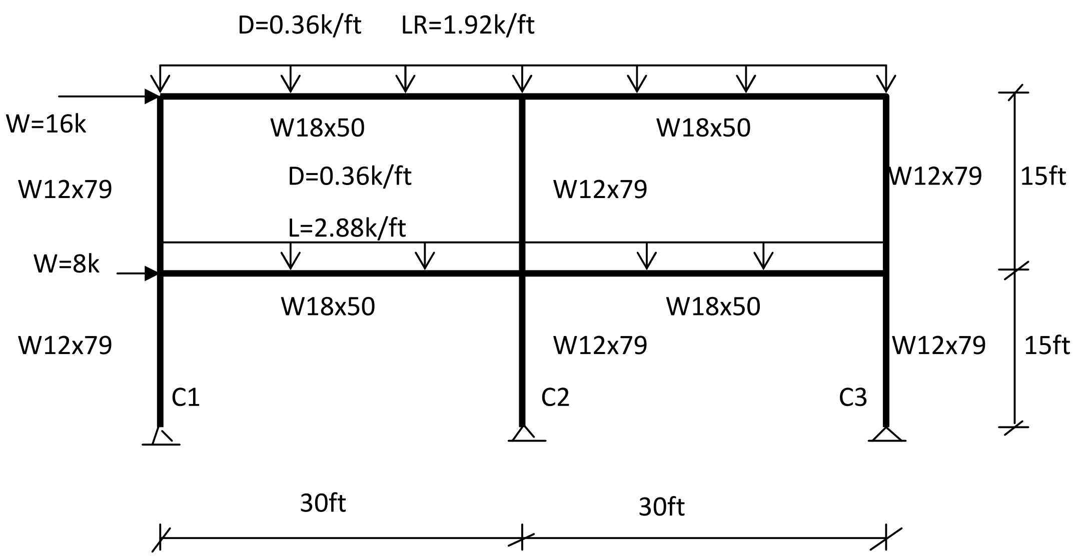


Figure 2.4 2-D Steel Moment Frame

Table 2.5 NT-LT Analysis Results for Example 2.4.2

| Column No. | NT analysis (1.2D+0.5LL+0.5LR) | | LT analysis (1.6W) | | $B_1$ | $B_2$ |
|---|---|---|---|---|---|---|
| | $P_{nt}$ (kip) | $M_{nt}$ (ft-kip) | $P_{lt}$ (kip) | $M_{lt}$ (ft-kip) | | |
| Column 1 | -49 | 41 | 16 | -184 | 1 | 1.035 |
| Column 2 | -109 | 0 | -0.09 | -239 | 1 | 1.035 |
| Column 3 | -49 | -41 | -16 | -184 | 1 | 1.035 |

Then using equations of amplified first-order analysis method, the second-order required strengths can be calculated. Then second-order results directly obtained from SAP2000 nonlinear P-delta analysis using 0.8EI reduced stiffness will be compared to the calculated results (Table 2.6).

Table 2.6 Second-Order Axial Forces and Moments for Example 2.4.2

| Column No. | the Amplified First-order Analysis Results | | SAP2000 Direct Analysis Method Results | | Comparison of the Results | |
|---|---|---|---|---|---|---|
| | $P_r$(kip) | $M_r$ (ft-kip) | $P_r$ (kip) | $M_r$ (ft-kip) | $P_r$ | $M_r$ |
| Column 1 | -32 | -149 | -32 | -151 | 0 | -1.3% |
| Column 2 | -109 | -247 | -109 | -249 | 0 | -0.5% |
| Column 3 | -65 | -231 | -66 | -232 | 0 | 0.25% |

Finally, interaction checks are conducted to check stability of the structure. For column W12x79, available compressive strength $P_c$ is equal to 809 kips and available flexural strength $M_c$ is equal to 446 kip-ft. By using the interaction equations (AISC H1-1a and AISC H1-1b) mentioned in Section 2.2, interaction ratios for each column are calculated. Results are recorded in Table 2.7

Table 2.7 Interaction Checks for 2-D Steel Frame

| Column No. | the Amplified First-order Analysis Results | |
|---|---|---|
| | $P_r/P_c$ | Interaction equation (Eq. H1-1a AISC 2005) |
| Column 1 | 0.04<0.2 | 0.35<1 |
| Column 2 | 0.135<0.2 | 0.617<1 |
| Column 3 | 0.08<0.2 | 0.552<1 |

The interaction ratios for all columns are less than 1.0. These results indicate that all columns meet stability requirements. The system has sufficient strength and stability to withstand the given loadings. The interaction ratio of interior column is the biggest one. This column is the critical column.

From table 2.7, the results from two methods are almost identical. For regular moment frames in which all members are perpendicular and there are no missing members, amplified first-order analysis method predicts the results as accurate as that of direct analysis method as expected. Therefore, amplified first-order method can be used in the regular frame building stability design in next chapter. Biaxial bending moments will be considered in that case.

# CHAPTER III

# MULTI-HAZARD DESIGN FOR STEEL MOMENT FRAME BUILDING COLUMNS

## 3.1 Introductions

The previous chapter introduces methods of second-order analysis: a simple two-dimensional steel frame subjected to both gravity and lateral load was assessed using second-order analysis methods. This chapter will focus on multi-hazard design of low-rise steel moment frame building columns in Mississippi. A three-dimensional steel frame building will be designed to satisfy the given loads and associated load combinations provided in the IBC2006 and ASCE/SEI 7-10. The same building in different four locations in Mississippi will be used to evaluate seismic and wind loading effects. As required in AISC 360-05 Specification, second-order effects must be considered in stability design of frame building and each member. In this chapter, the amplified first-order method will be used to calculate the second-order effects of each column when the building is subjected to seismic and wind loads. Then interaction ratios will be calculated to check the system's stability and determine the governing load case and the critical column.

## 3.2 Brief Introduction of Original Preliminary Design

The existing moment frame building [3] has 3 stories with three 40-foot longitudinal bays and two-20 foot bays through the depth. The story height is 12 feet, and the total height is 36 feet (Figures 3.1a and 3.1 b).

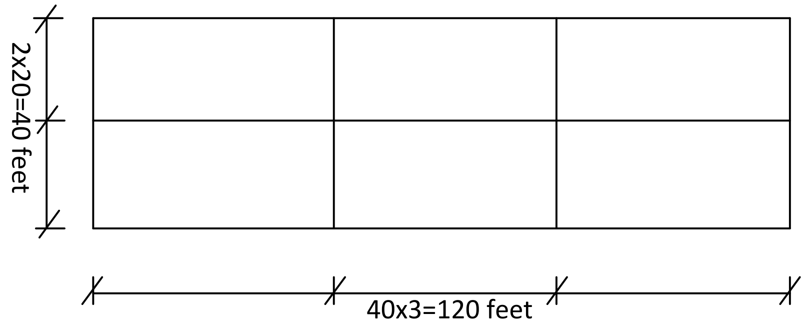


Figure 3.1a Floor Plan of 3-D Moment Frame Building

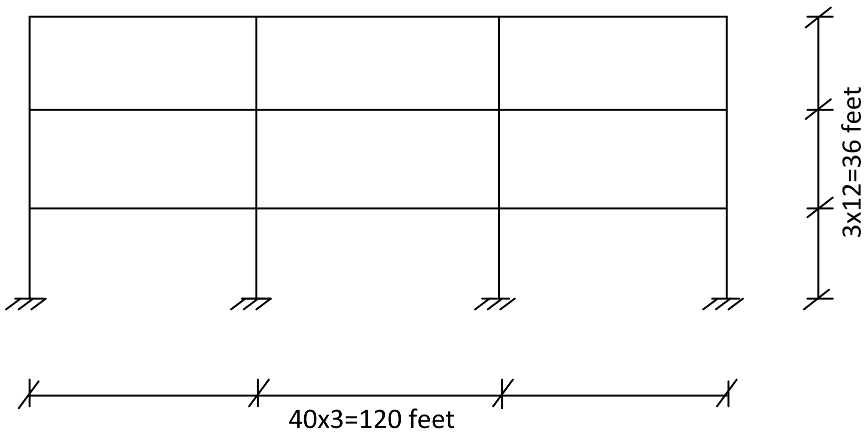


Figure 3.1 b Elevation of 3-D Moment Frame Building

The International Code Council's International Building Code 2006 (IBC 2006) and Minimum Design Loads for Buildings and Other Structures (ASCE/SEI 7-10) were used to develop all design loads and were followed through the entire design process. The building model was developed using commercial software [10] and the three-dimensional model is shown in Figure 3.2.

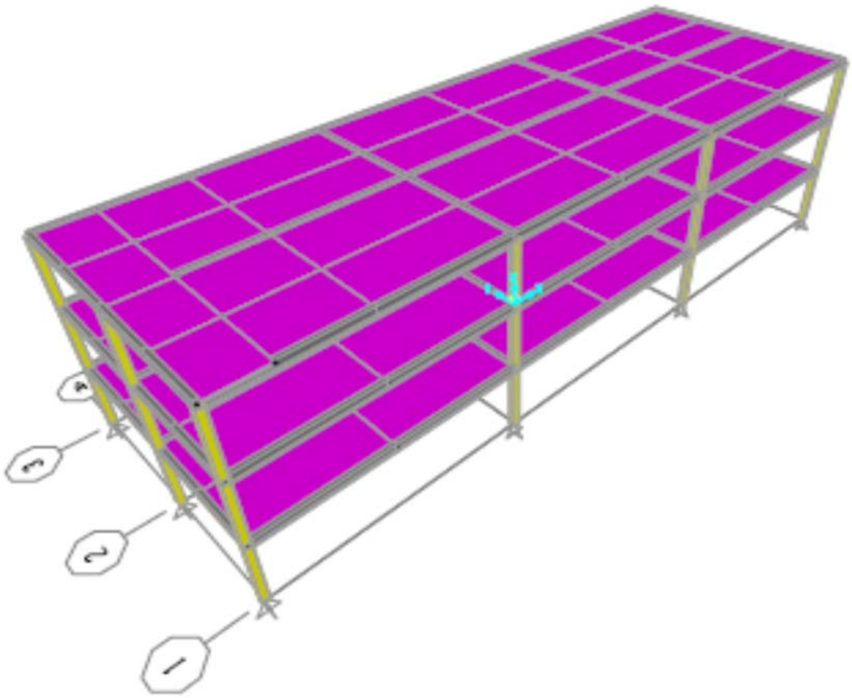


Figure 3.2 3-D Steel Moment Frame Building Model

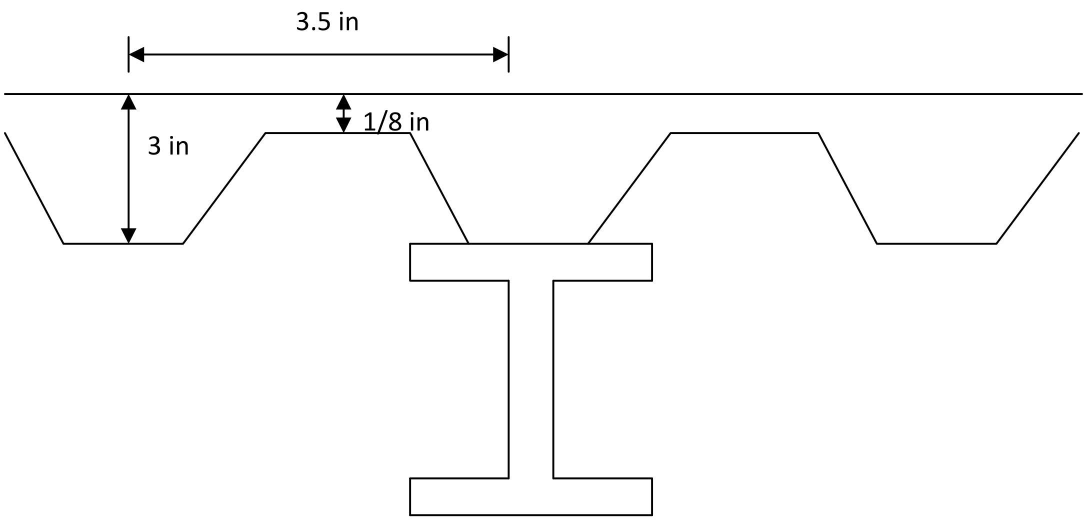


Figure 3.3 Composite Slab

3.2.1 Slab design

The slabs were chosen to be composite concrete with steel decking. The live load is chosen to be 50 pounds per square foot which is recommended for "office use" by IBC 2006. The roof live load is 20 pounds per square foot which is used for "ordinary flat" roof. From these basic loads, ACI 310-05 was referenced for the slab design. A three-inch thickness with 1/8 inch steel plate and 3.5 inch spans was used for the slab. Figure 3.3 shows a simple sketch of the floor slab. From IBC 2006, the largest load case was: $1.2D + 1.6LR + f_1 L$, where D is dead load, LR is the roof live load and L is live load. The $f_1$ coefficient is assumed to be 1 for "floors in places of public assembly".

3.2.2 Composite Beam Design

Beam selection was based on AISC 360-05 and Structural Steel Design by McCormac [23]. A W18x50 beam was selected from AISC 360-05 Table 3-19 and $Y_1$, $Y_2$, $\alpha$ and $\sum Q_n$ are given.

3.2.3 Column Design

Since the response of 3-D frame is base upon column response, column design is critical for this study. From existing building [3], a section size W8x31 was selected following the steps in the former mentioned-McCormac [23] and also Steel Structures: Behavior and Design by Salmon, *et al* [12]. For the top floor, the comparison of $\Phi P_n$ to $P_u$ shows that the W8x31 is only being used for 30% of its capacity. The actual cross-sectional area is 3 times greater than the required area. At the lowest floor, the column was using 75% of its capacity. This means the section of W8x31 is sufficient for use through the entire three-story steel frame design. The same section size was used for all columns in the frame building.

### 3.3 Multi-hazard Design

For this case, wind and earthquake will be considered in design. Wind and earthquake are two common natural hazards in state of Mississippi. North Mississippi is located near the termination point of New Madrid fault and the south end of Mississippi has a narrow coast of Gulf. Apparently the state is exposed under both severe wind and earthquake. Multi-hazard evaluation for building design is an important major in Mississippi. The frame building will be located in several locations to assess the effects of wind and earthquake. Four locations were chosen for this study: Southaven, Batesville, Jackson, and Gulfport. Southaven and Batesville are located in north Mississippi where is near the southern end of the New Madrid Seismic Zone. Gulfport is located on the Gulf of Mexico Coast. The city was hit by Hurricane Katrina in 2005. Much of the city was flooded or destroyed by the strong hurricane-force winds. Jackson is fairly centrally located between Batesville and Gulfport.

#### 3.3.1 Seismic Design

According to the IBC 2006 [5], the seismic design load is calculated as a static equivalent. Following the procedure described in IBC2006 Section 1613, earthquake loads, spectral accelerations $S_S$ and $S_1$ of different geographic locations are first determined. For this consideration, the IBC2006 provides historically-based maps with isoline defining regions subjected to varying levels of seismic accelerations. Short period (0.2-sec or 5 Hz) $S_S$ and long period (1-sec or 1 Hz) $S_1$ accelerations are mapped separately. $S_S$ and $S_1$ can also be determined using an internet tool [13] provided by the United States Geological Survey. It allows input of the building code used and zip code, and the USGS code will output the spectral response accelerations. $S_S$ and $S_1$ are 1.031 and 0.288 for Southaven, 0.588 and 0.188 for Batesville, 0.193

and 0.085 for Jackson, and 0.122 and 0.053 for Gulfport respectively. The next step is to assess the characteristics of the rock and soil of sites to determine the appropriate site class. For the current application, a site class "D" is used for three locations. Using the site class of D and the previously determined mapped acceleration parameters of $S_S$ and $S_1$, the IBC2006 then provides Table 1613.5.3 for determining a site coefficient for each of these periods. The IBC2006 notation for these coefficients is $F_a$ for the short-period ("a" indicating acceleration dominant) and $F_v$ for the 1-second-period ("v" indicating velocity dominant). The $F_a$ and $F_v$ values can be taken directly from the table, respectively. The 5%-damped design spectral response accelerations for short period and long periods are then defined by the IBC2006 using the following equations.

$$S_{DS} = \frac{2}{3} F_a S_s$$

$$S_{D1} = \frac{2}{3} F_v S_1$$

The IBC2006 then requires the definition of a seismic design category based on the design spectral response acceleration parameters ($S_{DS}$ and $S_{D1}$) and the occupancy category of the building. Occupancy Category II was chosen first [3]. For locations of Southaven and Batesville, this structure can be design under design category "D". For location of Gulfport, the structure can be designed under design category "A" or "B". The higher level of two conditions "B" should be chosen because "B" is the more conservative approach. For Jackson, seismic design category is "C".

Then the fundamental period is calculated by using:

$$T_a = C_t {h_n}^x$$

where $C_t$ and $x$ are values of period of parameters and $h_n$ the total height of the structure. For steel moment-resisting frame, $C_t$ is equal to 0.028 and $x$ is equal to 0.8. For three-story building, if the fundamental period is greater than 3.5 times $T_s$, dynamic analysis may be required. $T_s$ is the product of the spectral response acceleration of 1-second period $S_{D1}$ divided by the spectral response acceleration for short period $S_{DS}$. For these three locations, $T_a$ are less than 3.5 $T_s$; therefore, the static analysis is sufficient and can be continued.

The seismic base shear, V, in a given direction is determined in accordance with the following equation:

$$V = C_s W$$

where Cs is the seismic response coefficient, which, when multiplied by the weight of the structure, gives the base shear force for seismic design. To derive the seismic response coefficient for the short-period accelerations, ASCE 7-10 section 12.8 provides the following formulas:

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I}\right)}$$

where R is the response modification coefficient and I is an importance factor, equal to 2 and 1 for this building [5].

The total base shear is then vertically distributed to each floor, or the location of concentrated masses. This can be determined by following equation:

$$F_X = C_{VX} \cdot V$$

where

$$C_{VX} = \frac{W_x \cdot h_x^k}{\sum W_i h_i^k}$$

where $W_i$ and $W_x$ are the portion of the total effective seismic weight of the structure (W) located or assigned to Level i or x. $h_i$ and $h_x$ are the height (feet) from the base to Level i or x. *k* is an exponent related to the structure period. For these four cases, *k* is equal to 1.

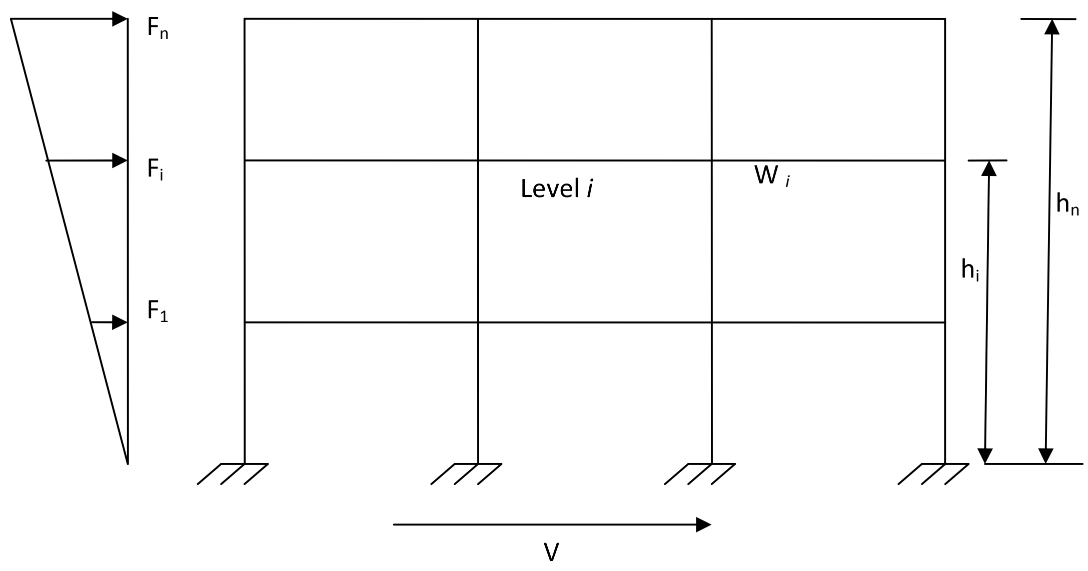


Figure 3.4 Distribution of Total Seismic Base Shear

Table 3.1 shows the results of seismic design for four different locations.

With the base shear distributed through the height of the structure, those results can be distributed on three-dimensional structure based on the relative stiffness of the frame.

Table 3.1 Results of Seismic Design

| | Southaven | Batesville | Jackson | Gulfport |
|---|---|---|---|---|
| $S_s$ = | 1.031 | 0.588 | 0.193 | 0.122 |
| $S_1$ = | 0.288 | 0.188 | 0.085 | 0.053 |
| $F_a$ = | 1.088 | 1.33 | 1.6 | 1.6 |
| $F_v$ = | 1.823 | 2.049 | 2.4 | 2.4 |
| $S_{MS}$ = | 1.121728 | 0.78204 | 0.3088 | 0.1952 |
| $S_{M1}$ = | 0.525024 | 0.385212 | 0.204 | 0.1272 |
| $S_{DS}$ = | 0.747818667 | 0.52136 | 0.206 | 0.13013333 |
| $S_{D1}$ = | 0.350016 | 0.256808 | 0.136 | 0.0848 |
| Occupancy Category | II | II | II | II |
| SDC | D | D | C | B |
| $h_n$ (ft) = | 36 | 36 | 36 | 36 |
| $C_t$ = | 0.028 | 0.028 | 0.028 | 0.028 |
| x = | 0.8 | 0.8 | 0.8 | 0.8 |
| $T_a$ = | 0.492266217 | 0.492266217 | 0.49226622 | 0.49226622 |
| $3.5T_s$ | 1.638 | 1.724 | 2.312 | 2.281 |
| R = | 2 | 2 | 2 | 2 |
| I | 1 | 1 | 1 | 1 |
| $C_s$= | 0.355514951 | 0.26068 | 0.065 | 0.06506667 |
| Total Weight (W) (kips)= | 748.032 | 748.032 | 748.32 | 748.032 |
| V (kips) = | 265.937 | 194.997 | 103.331 | 48.672 |
| $F_1$ (kips) = | 44.323 | 32.499 | 17.222 | 8.112 |
| $F_2$ (kips) = | 88.646 | 64.999 | 34.444 | 16.224 |
| $F_3$ (kips) = | 132.968 | 97.498 | 51.665 | 24.336 |

### 3.3.2 Wind Design

Since this is a low-rise (H<60 feet) simple diaphragm building with flat roof, the simplified procedure can be used to design the wind load (transverse or longitude) for this building. The basic wind speed v should be determined based on the occupancy category and the location of the structure first. For current case, occupancy category II is used for four locations (Southaven, Batesville, Jackson and Gulfport). The basic wind speed v can be determined by using ASCE/SEI 7-10 [6] Figure 26.5-A. The basic wind speed is 115 miles per hour for Southaven, Jackson and Batesville and 160 miles per hour for Gulfport. The important factor "I", then, can be defined by 1.15 in according with ASCE/SEI 7-10 [6] Table 1-1. The following step is to determine the exposure category. For this case, exposure category B (urban and suburban area) is used. The adjustment factor $\lambda$ can be determined by the mean roof height and the exposure category in accordance with IBC Section 1609.4. The topographic factor $K_{zt}$ of 1 can be determined using ASCE/SEI 7-10 [6] Figure 26.8-1. $P_{S30}$ is the simplified wind pressure for category B and I equal to 1 at height equal to 30 feet. For different zones, the values of $P_{S30}$ are different. This can be found according to their zone categories (Figure 3.5), and the basic wind speed.

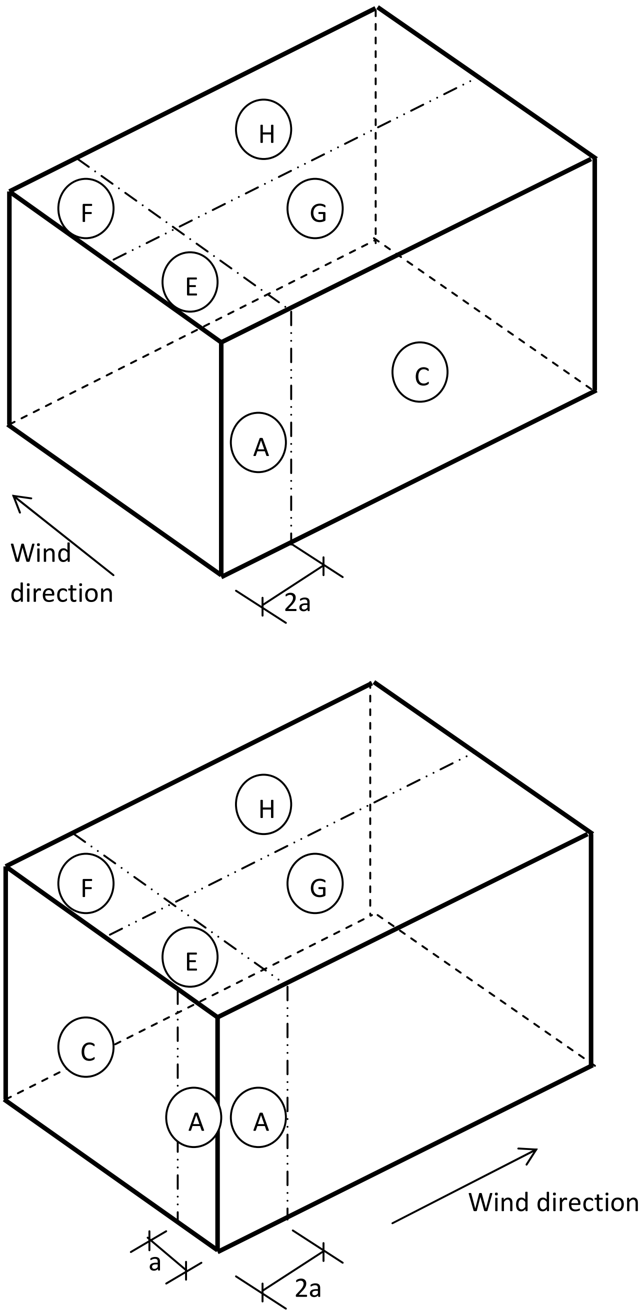


Figure 3.5 Wind Pressure Zone Categories

Where “a” is 10 percent of least horizontal dimension or 0.4h, which is smaller. For this case, “a” is equal to 4 feet.

The net adjusted simplified design wind pressures $P_{net}$ can be determined using the following equation for different zones of the building.

$$P_{net} = \lambda K_{zt} I P_{S30}$$

$P_{net}$ represents the net pressure (sum of internal and external) to be applied to the vertical and horizontal projections of building surfaces. For horizontal pressures, $P_{net}$ is the combination of the windward and leeward net pressures.

Table 3.2 Wind Pressures of the frame building for Southaven, Batesville, and Jackson

| Zone | λ | $K_{zt}$ | I | $P_{net30}$ (psf) | $P_{net}$ (psf) |
|---|---|---|---|---|---|
| A | 1.05 | 1 | 1.15 | 21 | 25.36 |
| C | 1.05 | 1 | 1.15 | 13.9 | 16.78 |
| E | 1.05 | 1 | 1.15 | -25.2 | -30.43 |
| F | 1.05 | 1 | 1.15 | -14.3 | -17.27 |
| G | 1.05 | 1 | 1.15 | -17.5 | -21.13 |
| H | 1.05 | 1 | 1.15 | -11.1 | -13.40 |

Table 3.3 Wind Pressures of the frame building for Gulfport

| Zone | λ | $K_{zt}$ | I | $P_{net30}$ (psf) | $P_{net}$ (psf) |
|---|---|---|---|---|---|
| A | 1.05 | 1 | 1.15 | 40.6 | 49.02 |
| C | 1.05 | 1 | 1.15 | 26.9 | 32.48 |
| E | 1.05 | 1 | 1.15 | -48.8 | -58.93 |
| F | 1.05 | 1 | 1.15 | -27.7 | -33.45 |
| G | 1.05 | 1 | 1.15 | -34 | -41.06 |
| H | 1.05 | 1 | 1.15 | -21.5 | -25.96 |

From above, the wind pressure for different zones can be calculated as shown in Table 3.2 and Table 3.3. The horizontal pressures can be distributed to the frame building.

## 3.4 Amplified First-order Analysis

Once the seismic and wind are determined, linear elastic structural analysis is used to determine the axial forces and bending moments on the structure. The P-delta effects will be taken into account by determining the first-order amplified moments. These moments will be used to finalize the seismic and wind design of the members.

### 3.4.1 Load Combinations

Under normal operating conditions, two or more load types will act on a structure at any given time. The load types combine to produce more severe conditions than if only single loads were to act. Therefore, the appropriate load combination is significant for building design. AISC 360-05 Specification provides six types of design load combination for LRFD method. For the building subjected to wind and seismic loads, the following two load combinations can be used in design.

$$1.2D + 1.6W + 0.5L + 0.5\,LR$$

$$1.2D + 1.0E + 0.5L$$

where D is dead load, L is live load, LR is roof live load, W is wind load and E is earthquake load. These load combinations are a natural choice for the combined effects of gravity and lateral load. The first one is the most common loading system for structures where lateral load is taken into account. It plays the primary role when P-delta effects are considered. The second load combination deals with the behavior of the frame under seismic action. In the real world, earthquake and wind forces may come from any directions. Therefore, both transverse (X direction) and longitude (Y direction) earthquake and wind forces will be considered. In other words, four types of load combinations will be used to evaluate the wind and seismic effects for each case.

### 3.4.2 Amplified First-order Analysis for Columns

In the analysis and design of a structure, the governing load effect controls the sizes of members and connections. Columns are the most directly affected element in a structure during an earthquake or tornado. A second-order analysis will be used to determine the governing load effect and check the capacity of columns.

Firstly, interaction equations mentioned in Chapter 2, Section 2.1 are used to check the stability of originally designed frame building. For column section W8x31, the available second-order axial compressive strength $P_c$ and the available second-order flexural strength for strong and weak axis $M_{cx}$ and $M_{cx}$ can be directly obtained from AISC 360-05 Specification Table4-1, Table 3-2, and Table 3-3 respectively. $P_r$ and $M_r$ are the required second-order axial compressive strength and required second-order flexural strength; these quantities are needed to be calculated for individual members using second-order analysis methods mentioned in chapter 2.

Because this is a regular plane frame system with fixed column support and full moment connection throughout the structure, second-order effects can be calculated using the amplified first-order analysis method. The procedures will follow the calculation steps of 2-D frame example in Chapter 2, Section 2.4.2. The dead load, live load, roof live load and wind load that applied to the roof can be considered as uniformly distributed gravity load. The seismic load and the wind load can be regarded as the lateral load. The original column orientation [3] and assigned column numbers are shown in Figure 3.6.

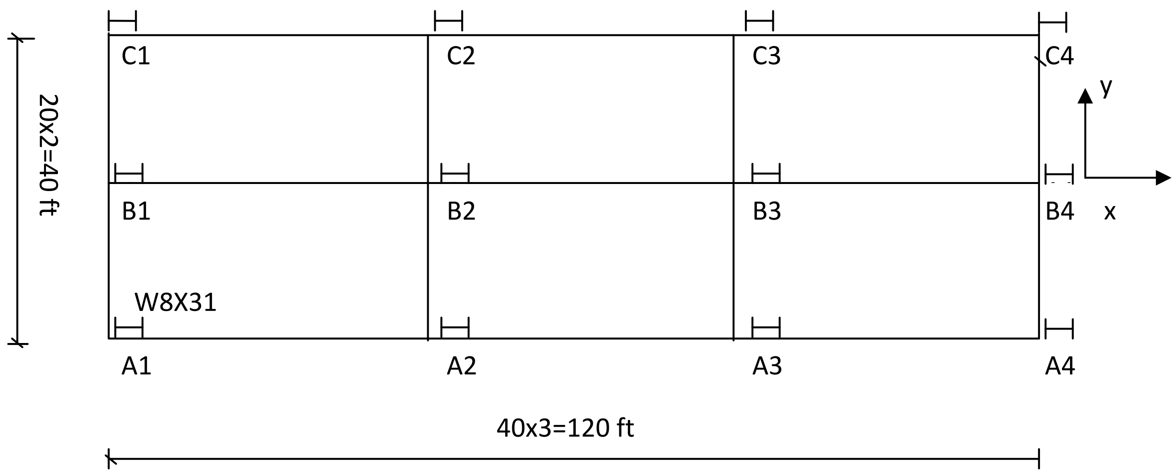


Figure 3.6 Original Column Orientation and Assigned Column Numbers

The amplified first-order analysis method is performed by amplifying the first-order elastic analysis results with amplified coefficients $B_1$ and $B_2$. Therefore, first-order analysis is needed to be completed first. As mentioned before, the first-order moments and the axial loads should be found for two conditions: no translation and lateral translation (NT and LT, respectively). For seismic loads, the load combination of 1.2D+0.5L can be performed by SAP 2000 static elastic analysis to obtain the NT first-order results. Then running load cases EX (transverse earthquake loads) and EY (longitude earthquake loads) can obtain LT results. Similarly, for the wind load, NT and LT results can be obtained by running the load case of 1.2D+0.5L+0.5LR and1.6 WX or 1.6 WY. Table 3.4 shows NT and LT elastic static axial forces and bending moments of lowest level for load combination $1.2D + 1.0EY + 0.5L$ in Southaven performed by SAP2000.

The $B_1$ coefficients are taken as 1.0 despite the calculated $B_1$ for each member are all smaller than 1. Then using the equation discussed in Chapter 2, the results for $B_2$ are presented in Table 3.5. The next step is to use the following equations to calculate the second-order moments and axial loads for each column member. Results are shown in Table 3.5.

$$M_r = B_1 M_{nt} + B_2 M_{lt}$$

$$P_r = P_{nt} + B_2 P_{lt}$$

Table 3.4 First-order Analysis Results of Lowest Level in Southaven

| Column No. | 1.2D+0.5L+EY | | | | | |
|---|---|---|---|---|---|---|
| | NT (1.2D+0.5L) | | | LT(EY) | | |
| | $P_{nt}$ (kip) | $M_{ntx}$ (kip-ft) | $M_{nty}$ (kip-ft) | $P_{lt}$ (kip) | $M_{ltx}$ (kip-ft) | $M_{lty}$ (kip-ft) |
| A1 | -34.153 | 9.518 | 1.405 | 44.79 | -0.0277 | -250.1572 |
| A2 | -74.791 | -1.7976 | 2.751 | 44.794 | -0.0186 | -250.1884 |
| A3 | -74.791 | 1.7764 | 2.7513 | 44.794 | 0.0186 | -250.1884 |
| A4 | -34.153 | -9.5115 | 1.4217 | 44.79 | 0.0278 | -250.1572 |
| B1 | -74.926 | 17.5614 | 0 | 0 | 0 | -267.4494 |
| B2 | -164.855 | -3.2636 | 0 | 0 | 0 | -267.5333 |
| B3 | -164.855 | -3.2409 | 0 | 0 | 0 | -267.5333 |
| B4 | -74.926 | -17.5093 | 0 | 0 | 0 | -267.4494 |
| C1 | -34.153 | 9.518 | -1.4045 | -44.79 | 0.0277 | -250.1572 |
| C2 | -74.791 | -1.7976 | -2.751 | -44.794 | 0.0186 | -250.1884 |
| C3 | -74.791 | 1.7764 | 2.7513 | -44.794 | -0.0186 | -250.1884 |
| C4 | -34.153 | -9.5115 | 1.4217 | -44.79 | -0.0277 | -250.1572 |

Finally, stability checks for each column are performed using interaction equations. From Table 3.5, all interaction ratios are larger than 1.0. This means all columns are overstressed. Thus, W8x31 is not able to meet strength criteria. Columns should be redesigned to satisfy the stability requirement.

Table 3.5 Second-Order Analysis Results

| column No. | 1.2D+0.5L+EY | | | | | | |
|---|---|---|---|---|---|---|---|
| | $B_1$ | $B_2$ | $P_r$(kip) | $M_{rx}$(kip-ft) | $M_{ry}$(kip-ft) | $P_r/\Phi P_n$ | Interaction ratio examination |
| A1 | 1 | 1.288 | 25.2142 | 9.4813 | -330.1684 | 0.089<0.2 | 6.38>1 overstressed |
| A2 | 1 | 1.288 | -11.3876 | -1.8239 | -351.3768 | 0.04<0.2 | 6.69>1 overstressed |
| A3 | 1 | 1.288 | -11.3875 | -1.8027 | -351.3765 | 0.04<0.2 | 6.69>1 overstressed |
| A4 | 1 | 1.288 | 25.2142 | 9.4746 | -330.1512 | 0.089<0.2 | 6.38>1 overstressed |
| B1 | 1 | 1.288 | -74.926 | 17.5614 | -378.6306 | 0.264>0.2 | 6.776>1 overstressed |
| B2 | 1 | 1.288 | -164.855 | -3.2636 | -378.6786 | 0.58>0.2 | 6.983>1 overstressed |
| B3 | 1 | 1.288 | -164.855 | -3.2409 | -378.6786 | 0.58>0.2 | 6.983>1 overstressed |
| B4 | 1 | 1.288 | -74.926 | -17.5093 | -378.6306 | 0.264>0.2 | 6.776>1 overstressed |
| C1 | 1 | 1.288 | -93.5202 | 9.5447 | -332.9774 | 0.33>0.2 | 6.01>1 overstressed |
| C2 | 1 | 1.288 | -138.1944 | -1.7711 | -356.8788 | 0.488>0.2 | 6.51>1 overstressed |
| C3 | 1 | 1.288 | -138.1944 | 1.75 | -351.3765 | 0.488>0.2 | 6.42>1 overstressed |
| C4 | 1 | 1.288 | -93.5202 | -9.5483 | -332.9946 | 0.33>0.2 | 6.01>1 overstressed |

### 3.4.3 Column Design

First, the W-shape column has a strong or major axis (x-x) and a weak or minor axis (y-y) (Figure 3.7). The strong axis stiffness is much larger than the weak axis stiffness. The Column should be oriented their strong bending axis in response to major bending forces that are combined with the column axial load. From the original building model, all columns are oriented to support transverse lateral forces (Figure 3.7). However, in a real situation, the direction of the lateral load from wind or an earthquake cannot be estimated. Biaxial bending moments should be considered in the building design.

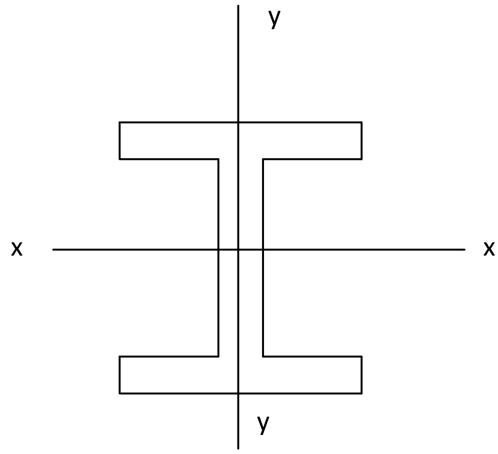


Figure 3.7 Strong and Weak Axis of W shape Column

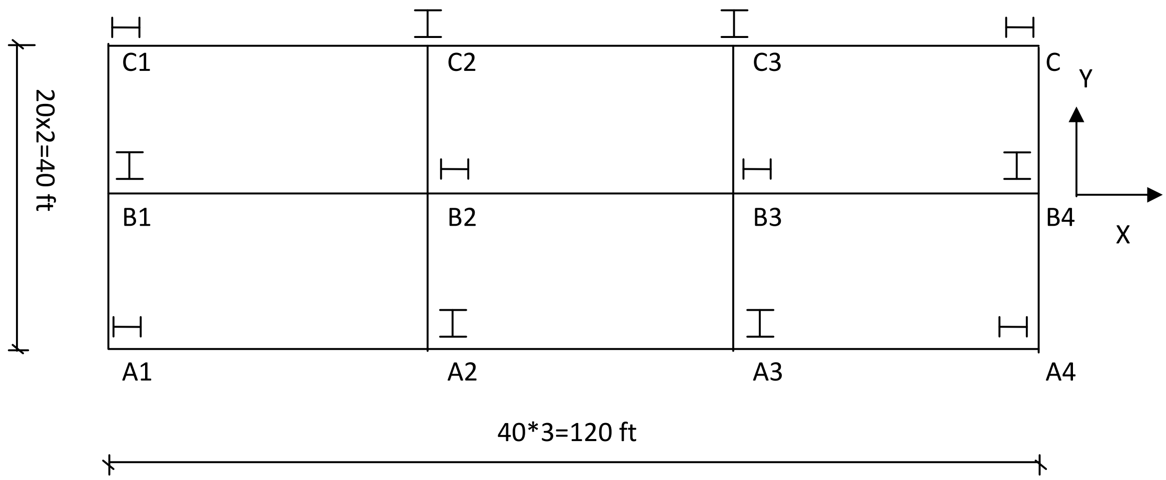


Figure 3.8 Column Orientation

As a result, the column orientation was changed to support both directions' bending force as shows in Figure 3.8. Columns A1, A4, B2, B3, C1, and C4 are mainly designed to support the transverse lateral forces. Columns A2, A3, B1, B4, C1, and C4 are designed to support the longitudinal lateral forces.

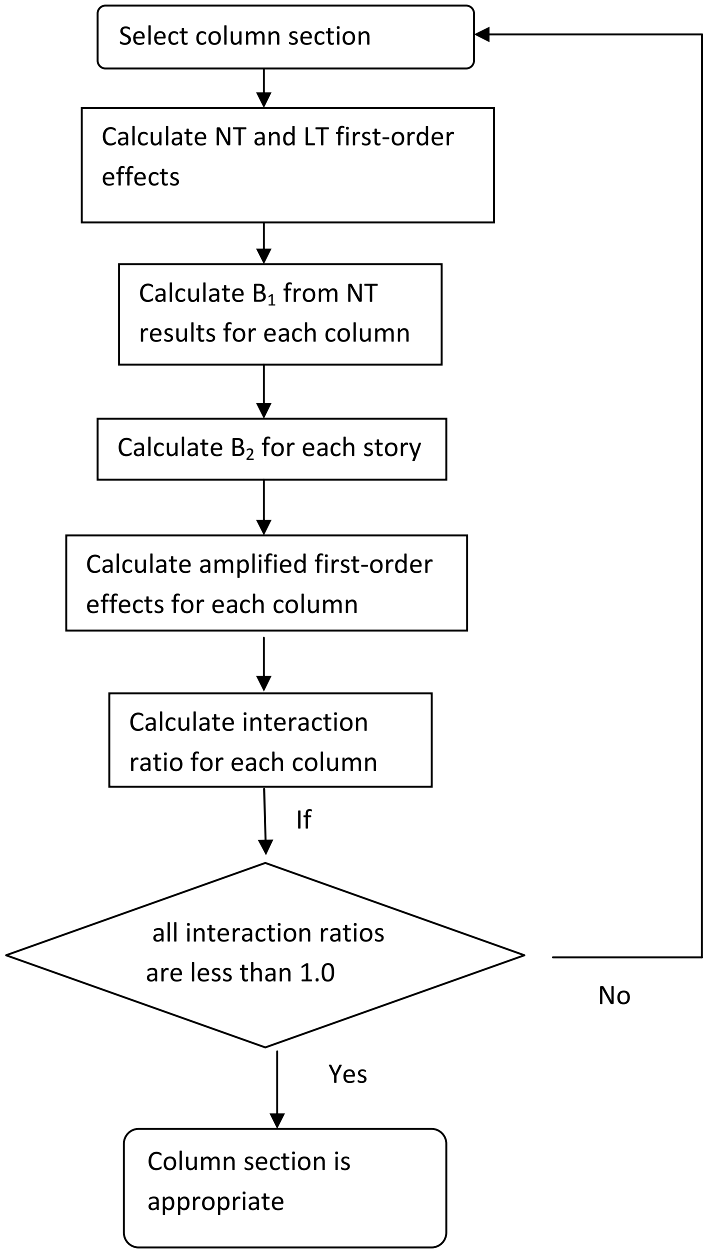


Figure 3.9 Procedures for Column Design

The next step is to determine the column section. Figure 3.9 shows the procedure to choose the appropriate column section size.

After several attempts, including using section W10x88, W12x72, W12x79, W14x82, a final section size of W12x96 was chosen to be used in building for all cases. SAP 2000 was used to develop all results of static elastic analysis. By applying gravity loads and the lateral load to the frame separately, NT and LT first-order elastic axial forces and moments for each column were obtained first. Then based upon the NT elastic results, the $B_1$ coefficient can be calculated for each column. Next, the $B_2$ amplifier can be calculated for each story. Using $B_1$ and $B_2$ coefficients to amplify first-order elastic axial forces and moments, the second-order moments and the axial compressive strength can be obtained. Afterward using the interaction equations, the structural stability is checked member by member. Results of the interaction ratios are shown in Table 3.6 through Table 3.9.

Table 3.6 Stability Checks for Southaven

| Column No. | Interaction Ratios for Different Load Combinations | | | |
|---|---|---|---|---|
| | Earthquake Case | | Wind Case | |
| | 1.2D+0.5L+EX | 1.2D+0.5L+EY | 1.2D+0.5L+0.5LR+1.6WX | 1.2D+0.5L+0.5LR+1.6WY |
| A1 | 0.5756 | 0.6987 | 0.0998 | 0.2883 |
| A2 | 0.9172 | 0.5868 | 0.1395 | 0.2345 |
| A3 | 0.905 | 0.5867 | 0.1399 | 0.2312 |
| A4 | 0.6625 | 0.6977 | 0.1494 | 0.2545 |
| B1 | 0.5951 | 0.9267 | 0.1112 | 0.43654 |
| B2 | 0.8606 | 0.9454 | 0.2224 | 0.391 |
| B3 | 0.8458 | 0.9451 | 0.1767 | 0.3795 |
| B4 | 0.8099 | 0.9278 | 0.2302 | 0.4052 |
| C1 | 0.5756 | 0.7267 | 0.1541 | 0.3314 |
| C2 | 0.9172 | 0.6745 | 0.218 | 0.3036 |
| C3 | 0.905 | 0.6757 | 0.1887 | 0.2892 |
| C4 | 0.6625 | 0.7684 | 0.1454 | 0.3114 |

Table 3.7 Stability Checks for Batesville

| Column No. | Interaction Ratios for Different Load Combinations | | | |
|---|---|---|---|---|
| | Earthquake Case | | Wind Case | |
| | 1.2D+0.5L+EX | 1.2D+0.5L+EY | 1.2D+0.5L+0.5LR+1.6WX | 1.2D+0.5L+0.5LR+1.6WY |
| A1 | 0.4234 | 0.5132 | 0.0998 | 0.2883 |
| A2 | 0.6899 | 0.4342 | 0.1395 | 0.2345 |
| A3 | 0.6795 | 0.434 | 0.1399 | 0.2312 |
| A4 | 0.5044 | 0.5135 | 0.1494 | 0.2545 |
| B1 | 0.4202 | 0.7143 | 0.1112 | 0.43654 |
| B2 | 0.6525 | 0.7187 | 0.2224 | 0.391 |
| B3 | 0.6373 | 0.7185 | 0.1767 | 0.3795 |
| B4 | 0.6291 | 0.7155 | 0.2302 | 0.4052 |
| C1 | 0.4134 | 0.5464 | 0.1541 | 0.3314 |
| C2 | 0.6399 | 0.5104 | 0.218 | 0.3036 |
| C3 | 0.6795 | 0.517 | 0.1887 | 0.2892 |
| C4 | 0.5044 | 0.5819 | 0.1454 | 0.3114 |

Table 3.8 Stability Checks for Gulfport

| Column No. | Interaction Ratios for Different Load Combinations | | | |
|---|---|---|---|---|
| | Earthquake Case | | Wind Case | |
| | 1.2D+0.5L+EX | 1.2D+0.5L+EY | 1.2D+0.5L+0.5LR+1.6WX | 1.2D+0.5L+0.5LR+1.6WY |
| A1 | 0.1093 | 0.1516 | 0.2109 | 0.5438 |
| A2 | 0.221 | 0.1194 | 0.2542 | 0.4472 |
| A3 | 0.2143 | 0.1271 | 0.2131 | 0.422 |
| A4 | 0.1781 | 0.1519 | 0.2228 | 0.4141 |
| B1 | 0.0592 | 0.2764 | 0.2407 | 0.7321 |
| B2 | 0.2232 | 0.2387 | 0.3521 | 0.6691 |
| B3 | 0.2073 | 0.2425 | 0.2786 | 0.6236 |
| B4 | 0.256 | 0.2776 | 0.3195 | 0.6086 |
| C1 | 0.1093 | 0.1967 | 0.2924 | 0.5796 |
| C2 | 0.221 | 0.1779 | 0.3586 | 0.5273 |
| C3 | 0.2143 | 0.1895 | 0.3071 | 0.4887 |
| C4 | 0.1781 | 0.1974 | 0.2237 | 0.4984 |

Table 3.9 Stability Checks for Jackson

| Column No. | Interaction Ratios for Different Load Combinations | | | |
|---|---|---|---|---|
| | Earthquake Case | | Wind Case | |
| | 1.2D+0.5L+EX | 1.2D+0.5L+EY | 1.2D+0.5L+0.5LR+1.6WX | 1.2D+0.5L+0.5LR+1.6WY |
| A1 | 0.2266 | 0.2867 | 0.0998 | 0.2883 |
| A2 | 0.3961 | 0.2370 | 0.1395 | 0.2345 |
| A3 | 0.3880 | 0.2373 | 0.1399 | 0.2312 |
| A4 | 0.2999 | 0.2870 | 0.1494 | 0.2545 |
| B1 | 0.1940 | 0.4400 | 0.1112 | 0.43654 |
| B2 | 0.3835 | 0.4165 | 0.2224 | 0.391 |
| B3 | 0.3679 | 0.4162 | 0.1767 | 0.3795 |
| B4 | 0.3953 | 0.4411 | 0.2302 | 0.4052 |
| C1 | 0.2129 | 0.3135 | 0.1541 | 0.3314 |
| C2 | 0.3460 | 0.2983 | 0.218 | 0.3036 |
| C3 | 0.3880 | 0.3044 | 0.1887 | 0.2892 |
| C4 | 0.2999 | 0.3411 | 0.1454 | 0.3114 |

The results in Table 3.6 through 3.9 indicate that the interaction ratios of all columns are less than 1.0. Therefore, W12x96 can be used in all cases. This structure can be considered to have sufficient strength and stability to withstand the given loadings. The more the column interaction ratios are close to 1, the more the column reaches its capacity.

The results also show that in Southaven, when the building is under longitudinal seismic lateral loadings, the interaction ratio of column B2 is 0.9245 which is the largest value. This means that column B2 is the critical column member for this building during an earthquake. This column is almost used at 93% of its capacity. If the building is subjected to earthquake and severe wind simultaneously or more extreme lateral loads, such as an explosion, it may lose its stability. The column section should be increased to endure more extreme case. In Batesville, results are the same as in Southaven but the building is evaluated as more conservative because the critical column B2 is only used at 72% of its capacity. In Jackson, the interaction ratio of column B4 is greatest, which means that B4 is the most dangerous member during an earthquake. Nevertheless, the most dangerous column is only using 45% of its capacity, so the section of columns can be reduced for economic purposes. In Gulfport, when the building is subjected to the wind load, column B1 is the most critical member based on the largest interaction ratio value. The critical column will reach 74% of its capacity.

# CHAPTER IV

# CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE WORK

## 4.1 Conclusions

For evaluating a multi-hazard in Mississippi, the four cities chosen in this study were typical locations for earthquake and wind assessment. Southaven is the nearest location from the New Madrid Seismic Zone. The interaction ratios of columns B2 and B3 are the largest for the load combination 1.2D+0.5L+EY. This indicates that the longitudinal earthquake is the governing load case and the interior columns are the most dangerous members during an earthquake. The seismic forces will apparently control the building design. When the building moves south towards Batesville, the seismic load was still found to control over the wind load even though it is located in a relatively low seismic region. When the building moves to Gulfport, which is located on the Gulf coast, the wind load should be mainly considered in design and the seismic load can be neglected. The longitudinal wind load is the governing load case because the larger area of the building will support larger wind pressure. If the prevailing wind direction is known, buildings should be constructed along the wind direction. In this study, interaction ratios are used as the basis for establishing capacity of each column and dominant hazard at a particular location. As expected, Southaven and Batesville are show to be dominated by the earthquake hazard and Gulfport by the wind load.

The results shown in Table 3-7 indicate that Jackson, the capital of Mississippi, which is located between Batesville and Gulfport, is a significant place to evaluate the seismic and wind interactivities. The results of the interaction ratio checks show that the seismic force is still dominated in design in Jackson. However, interaction ratios from wind load are very close to the results of the seismic load. That means Jackson is roughly equally influenced by both hazards. Jackson may thus be considered as a boundary for dividing the state into seismic dominated zone and the wind dominated zone. In the region in north of Jackson, seismic should be mainly considered in design. In the region south of Jackson, the designer should pay more attention on the wind damage of a tornado. This evaluation may give insight for building designers in preliminary design.

## 4.2 Recommendations for Future Work

For the regular frame moment buildings, amplified first-order method is an applicable method to calculated second-order effects. The calculation procedures in Chapter 2 and Chapter 3 can be followed by building designers to determine second-order axial loads and bending moments in the similar steel frame building cases. More steel building types need to be evaluated to more fully assess the influence of the wind and seismic hazard in Mississippi.

In more complicated building frame configurations, the direct analysis method is expected to be more accurate and should be used. Future AISC editions will be recommended this for all types. However, in this study, the software used was not able to perform the required P-Delta analysis for the 3-story building. Therefore, use of more general purpose software than the one used here in proper use of nonlinear solution techniques is recommended for general application the direct analysis method.

More severe load cases are suggested for a second-order analysis of the frame system in order to advance further the knowledge of multi-hazard structures

A more in-depth computer model is also encouraged. Connections of beam and column should be considered in design. A dynamic analysis is also encouraged due to a more accurate evaluation of damage.

LIST OF REFERENCES

## LIST OF REFERENCES


1. American Institute of Steel Construction, Inc., Steel Construction Manual. $13^{th}$, 2005.
2. Chen, W. F. and Lui, E. M., Stability Design of Steel Frames, 1991, CRC Press, Inc., Boca Raton, FL.
3. Charlie A. Burchfield, Column Capacity Analysis for Determining Blast Resistance of Steel Structure in Multi-hazard Design and Evaluation, 2009, University of Mississippi, Oxford, MS.
4. Galambos, T. V. and A.E. Surovek, Structural Stability of Steel: Concept and Application for Structure Engineers, 2008, John Wiley & Sons, Inc., Hoboken, New Jersey.
5. International Code Council, Inc., 2006 International Building Code, 2006.
6. American Society of Civil Engineers, ASCE 7-10: Minimum Design Loads for Building and Other Structures, 2010.
7. L.F. Geschwindner, Unified Design of Steel Structures, 2008, John Wiley & Sons, Inc., Hoboken, New Jersey.
8. Chen, W. F. and Lui, E. M., Structural Stability, 1987, Theory and Implementation, Elsevier Science Publishing Company Inc., New York, NY.
9. Mashary, A.M., Chen, W.F., Elastic Second-Order Analysis for Frame Design. Journal of Construction Steel Research, 15, P303-322
10. Computers and Structures, Inc. SAP2000 Version 14 and User's Manual. 2009, Berkeley, CA.
11. Gere, J.M., Mechanics of Materials. $6^{th}$ ed. 2004, Brooks/Cole-Thomson Learning, Belmont, CA.
12. Salmon, C.G., J.E. Johnson, and F.A. Malhas, Steel Structures Design and Behavior, 5th ed.,

2009, Pearson Prentice Hall, Upper Saddle River, NJ.

13. United States Geological Survey. http://earthquake.usgs.gov/hazards/designmaps/

14. Larry Muir and Cynthia J. Duncan, The AISC 2010 Specification and the14th Edition Steel Construction Manual, in Structures Congress 2011, P661-675.

15. Halvorson, R. A., Warner, C. and Lang, A., Direct Analysis Method Case Study-Addressing Stability for the Russia Tower, in Structures Congress 2009, P609-618.

16. Okazaki, T., Parkolap, M., and Fahnestock, L. A., Interface of the Direct Analysis Method and Seismic Design, in Structures Congress 2009, P619-626.

17. Charney, F. A., Use of Nonlinear Analysis in the Context of the ASCE 7 Seismic Load Provision, in Structures Congress 2011, P782-790.

18. Ambrose, J. and Vergun, D., Simplified Building Design for Wind and Earthquake Forces, $3^{th}$, 1995 John Wiley & Son Inc, New York, NY.

19. Surovek, A. E., Alemdar, B. N., and Hartwell, A. J., Three-dimensional Verification and Application of the Direct Analysis Approach, Structure Congress 2009, P627-631.

20. Galambos, T. V., Guide to stability design criteria for metal structures, $4^{th}$ ed., 1988, John Wiley& Son Inc., New York, NY.

21. Gambhir, M.L., Stability analysis and design of structures, 2004, Springer, Berlin, NY

22. Nair, R. S., A Model Specification for Stability Design by Direct Analysis, Engineering Journal No. 4, 2009, P29-38.

23. McCormac, J.C., Structural Steel Design, Upper Saddle River, NJ, 2008

LIST OF APPENDICES

# APPENDIX A

# SEISMIC ANALYSIS RESULTS

Table A.1 First-order Results of Lowest Level in Southaven (Y-direction)

| Column No. | 1.2D+0.5L+EY | | | | | |
|---|---|---|---|---|---|---|
| | NT (1.2D+0.5L) | | | LT(EY) | | |
| | $P_{nt}$ (kip) | $M_{ntx}$ (kip-ft) | $M_{nty}$ (kip-ft) | $P_{lt}$ (kip) | $M_{ltx}$ (kip-ft) | $M_{lty}$ (kip-ft) |
| A1 | -39.006 | 17.6338 | 4.7325 | 48.146 | 1.2264 | -165.337 |
| A2 | -79.845 | 13.862 | 0.735 | 44.932 | -315.661 | -0.162 |
| A3 | -80.006 | -0.669 | 13.862 | 44.932 | -315.66 | 0.162 |
| A4 | -40.412 | -17.560 | 4.794 | 48.147 | -1.226 | -165.337 |
| B1 | -75.473 | 0 | -24.24 | 0 | -422.689 | 0 |
| B2 | -155.557 | -4.603 | 0 | 0 | 0 | -210.946 |
| B3 | -155.842 | 4.386 | 0 | 0 | 0 | -210.946 |
| B4 | -79.628 | 0 | 24.04 | 0 | -422.699 | 0 |
| C1 | -39.006 | 17.634 | -4.773 | -48.146 | -1.226 | -165.337 |
| C2 | -79.845 | -13.862 | 0.735 | -44.932 | 0.162 | -315.661 |
| C3 | -80.006 | -0.669 | 13.862 | 44.932 | -315.66 | 0.162 |
| C4 | -40.412 | -17.560 | -4.794 | -48.147 | 1.226 | -165.337 |

Table A.2 Amplified First-order Results of Lowest Level in Southaven (Y-direction)

| Column No. | 1.2D+0.5L+EY | | | | | | |
|---|---|---|---|---|---|---|---|
| | $B_1$ | $B_2$ | $P_r$(kip) | $M_{rx}$(kip- | $M_{ry}$(kip-ft) | $P_r/\Phi P_n$ | Interaction ratio |
| A1 | 1 | 1.038 | 10.946 | 18.906 | -166.8 | 0.010 | 0.699<1 OK |
| A2 | 1 | 1.038 | -33.277 | -313.643 | 0.567 | 0.031 | 0.587<1 OK |
| A3 | 1 | 1.038 | -33.388 | -313.642 | -0.501 | 0.031 | 0.587<1 OK |
| A4 | 1 | 1.038 | 9.541 | -18.832 | -166.747 | 0.009 | 0.698<1 OK |
| B1 | 1 | 1.038 | -75.473 | -438.558 | -24.24 | 0.07 | 0.927<1 OK |
| B2 | 1 | 1.038 | -155.557 | -4.603 | -218.861 | 0.144 | 0.945<1 OK |
| B3 | 1 | 1.038 | -155.842 | 4.386 | -218.861 | 0.144 | 0.945<1 OK |
| B4 | 1 | 1.038 | -79.628 | -438.559 | 24.04 | 0.074 | 0.928<1 OK |
| C1 | 1 | 1.038 | -88.958 | 16.361 | -176.273 | 0.082 | 0.768<1 OK |
| C2 | 1 | 1.038 | -126.463 | -341.366 | 0.902 | 0.117 | 0.675<1 OK |
| C3 | 1 | 1.038 | -126.624 | -341.365 | -0.836 | 0.117 | 0.681<1 OK |
| C4 | 1 | 1.038 | -90.365 | -16.287 | -176.334 | 0.084 | 0.768<1 OK |

Table A.3 First-order Results of Lowest Level in Southaven (X-direction)

| Column No. | 1.2D+0.5L+EX | | | | | |
|---|---|---|---|---|---|---|
| | NT (1.2D+0.5L) | | | LT(EX) | | |
| | $P_{nt}$ (kip) | $M_{ntx}$ (kip-ft) | $M_{nty}$ (kip-ft) | $P_{lt}$ (kip) | $M_{ltx}$ (kip-ft) | $M_{lty}$ (kip-ft) |
| A1 | -39.006 | 17.6338 | 4.733 | 21.891 | -306.397 | -0.109 |
| A2 | -79.845 | 13.862 | 0.735 | -7.342 | -0.099 | 205.201 |
| A3 | -80.006 | 13.862 | -0.669 | 7.342 | 0.099 | 205.201 |
| A4 | -40.412 | -17.56 | 4.794 | -21.891 | -306.398 | 0.109 |
| B1 | -75.473 | 0 | -24.24 | 22.695 | 0 | 161.263 |
| B2 | -155.557 | -4.603 | 0 | 1.412 | -411.183 | 0 |
| B3 | -155.842 | 4.386 | 0 | -1.412 | -411.183 | 0 |
| B4 | -79.628 | 0 | 24.04 | -22.695 | 0 | 161.262 |
| C1 | -39.006 | 17.634 | -4.733 | 21.891 | -306.397 | 0.109 |
| C2 | -79.845 | -13.862 | 0.735 | -7.342 | 0.099 | 205.210 |
| C3 | -80.006 | -13.862 | -0.669 | 7.342 | -0.099 | 205.201 |
| C4 | -40.412 | -17.576 | -4.794 | -21.891 | -306.398 | -0.109 |

Table A.4 Amplified First-order Results of Lowest Level in Southaven (X-direction)

| Column No. | 1.2D+0.5L+EX | | | | | | |
|---|---|---|---|---|---|---|---|
| | $B_1$ | $B_2$ | $P_r$(kip) | $M_{rx}$(kip-ft) | $M_{ry}$(kip-ft) | $P_r/\Phi P_n$ | Interaction ratio |
| A1 | 1 | 1.0465 | -16.097 | -303.018 | 4.618 | 0.015 | 0.576<1 OK |
| A2 | 1 | 1.0465 | -87.529 | 13.758 | 215.482 | 0.081 | 0.917<1 OK |
| A3 | 1 | 1.0465 | -72.322 | 13.966 | 214.079 | 0.067 | 0.905<1 OK |
| A4 | 1 | 1.0465 | -63.321 | -338.212 | 4.908 | 0.059 | 0.663<1 OK |
| B1 | 1 | 1.0465 | -51.722 | 0 | 144.525 | 0.048 | 0.595<1 OK |
| B2 | 1 | 1.0465 | -154.079 | -434.915 | 0 | 0.143 | 0.861<1 OK |
| B3 | 1 | 1.0465 | -157.32 | 425.926 | 0 | 0.146 | 0.846<1 OK |
| B4 | 1 | 1.0465 | -103.379 | 0 | 192.805 | 0.096 | 0.81<1 OK |
| C1 | 1 | 1.0465 | -16.097 | -303.018 | -4.618 | 0.015 | 0.576<1 OK |
| C2 | 1 | 1.0465 | -87.529 | -13.758 | 215.482 | 0.081 | 0.917<1 OK |
| C3 | 1 | 1.0465 | -72.322 | -13.966 | 214.079 | 0.067 | 0.905<1 OK |
| C4 | 1 | 1.0465 | -63.321 | -338.212 | -4.908 | 0.059 | 0.663<1 OK |

Table A.5 First-order Results of Lowest Level in Batesville (Y-direction)

| Column No. | 1.2D+0.5L+EY | | | | | |
|---|---|---|---|---|---|---|
| | NT (1.2D+0.5L) | | | LT(EY) | | |
| | $P_{nt}$ (kip) | $M_{ntx}$ (kip-ft) | $M_{nty}$ (kip-ft) | $P_{lt}$ (kip) | $M_{ltx}$ (kip-ft) | $M_{lty}$ (kip-ft) |
| A1 | -39.006 | 17.6338 | 4.733 | 35.303 | 0.8992 | -121.2306 |
| A2 | -79.845 | 13.862 | 0.735 | 32.946 | -213.453 | -0.119 |
| A3 | -80.006 | 13.862 | -0.669 | 32.946 | -213.453 | 0.119 |
| A4 | -40.412 | -17.56 | 4.794 | 35.303 | -0.899 | -121.231 |
| B1 | -75.473 | 0 | -24.24 | 0 | -309.937 | 0 |
| B2 | -155.557 | -4.603 | 0 | 0 | 0 | -155.673 |
| B3 | -155.842 | 4.386 | 0 | 0 | 0 | -155.673 |
| B4 | -79.628 | 0 | 24.04 | 0 | -309.937 | 0 |
| C1 | -39.006 | 17.634 | -4.733 | -35.303 | -0.899 | -121.231 |
| C2 | -79.845 | -13.862 | 0.735 | -32.946 | -213.453 | 0.119 |
| C3 | -80.006 | -13.862 | -0.669 | -32.946 | -213.453 | -0.119 |
| C4 | -40.412 | -17.576 | -4.794 | -35.303 | 0.899 | -121.231 |

Table A.6 Amplified First-order Results of Lowest Level in Batesville (Y-direction)

| Column No. | 1.2D+0.5L+EY | | | | | | |
|---|---|---|---|---|---|---|---|
| | $B_1$ | $B_2$ | $P_r$(kip) | $M_{rx}$(kip-ft) | $M_{ry}$(kip-ft) | $P_r/\Phi P_n$ | Interaction ratio |
| A1 | 1 | 1.038 | -2.378 | 18.567 | -121.046 | 0.002 | 0.513<1 OK |
| A2 | 1 | 1.038 | -45.663 | -226.274 | 0.612 | 0.042 | 0.434<1 OK |
| A3 | 1 | 1.038 | -45.824 | -266.273 | -0.546 | 0.042 | 0.434<1 OK |
| A4 | 1 | 1.038 | -3.785 | -18.492 | -120.985 | 0.004 | 0.514<1 OK |
| B1 | 1 | 1.038 | -75.473 | -321.564 | -24.24 | 0.07 | 0.714<1 OK |
| B2 | 1 | 1.038 | -155.557 | -4.603 | -161.513 | 0.144 | 0.719<1 OK |
| B3 | 1 | 1.038 | -155.842 | 4.386 | -161.513 | 0.144 | 0.719<1 OK |
| B4 | 1 | 1.038 | -79.628 | -321.564 | 24.04 | 0.074 | 0.715<1 OK |
| C1 | 1 | 1.038 | -75.633 | 16.701 | -130.511 | 0.07 | 0.581<1 OK |
| C2 | 1 | 1.038 | -114.027 | -253.997 | 0.858 | 0.106 | 0.51<1 OK |
| C3 | 1 | 1.038 | -114.188 | -253.997 | -0.792 | 0.106 | 0.517<1 OK |
| C4 | 1 | 1.038 | -77.039 | -16.627 | -130.572 | 0.071 | 0.582<1 OK |

Table A.7 First-order Results of Lowest Level in Batesville (X-direction)

| Column No. | 1.2D+0.5L+EX | | | | | |
|---|---|---|---|---|---|---|
| | NT (1.2D+0.5L) | | | LT(EX) | | |
| | $P_{nt}$ (kip) | $M_{ntx}$ (kip-ft) | $M_{nty}$ (kip-ft) | $P_{lt}$ (kip) | $M_{ltx}$ (kip-ft) | $M_{lty}$ (kip-ft) |
| A1 | -39.006 | 17.6338 | 4.733 | 16.051 | -224.66 | -0.08 |
| A2 | -79.845 | 13.862 | 0.735 | -5.384 | -0.073 | 150.46 |
| A3 | -80.006 | 13.862 | -0.669 | 5.384 | 0.073 | 150.46 |
| A4 | -40.412 | -17.56 | 4.794 | -16.051 | -224.661 | 0.08 |
| B1 | -75.473 | 0 | -24.24 | 16.641 | 0 | 118.243 |
| B2 | -155.557 | -4.603 | 0 | 1.035 | -301.493 | 0 |
| B3 | -155.842 | 4.386 | 0 | -1.035 | -301.493 | 0 |
| B4 | -79.628 | 0 | 24.04 | -16.641 | 0 | 118.243 |
| C1 | -39.006 | 17.634 | -4.733 | 16.051 | -224.66 | 0.08 |
| C2 | -79.845 | -13.862 | 0.735 | -5.384 | -0.073 | 150.46 |
| C3 | -80.006 | -13.862 | -0.669 | 5.384 | -0.073 | 150.46 |
| C4 | -40.412 | -17.576 | -4.794 | -16.051 | -224.66 | -0.08 |

Table A.8 Amplified First-order Results of Lowest Level in Batesville (X-direction)

| Column No. | 1.2D+0.5L+EX | | | | | | |
|---|---|---|---|---|---|---|---|
| | $B_1$ | $B_2$ | $P_r$(kip) | $M_{rx}$(kip-ft) | $M_{ry}$(kip-ft) | $P_r/\Phi P_n$ | Interaction ratio |
| A1 | 1 | 1.047 | -22.208 | -217.489 | 4.649 | 0.021 | 0.423<1 OK |
| A2 | 1 | 1.047 | -85.48 | 13.785 | 158.201 | 0.079 | 0.69<1 OK |
| A3 | 1 | 1.047 | -74.371 | 13.938 | 156.798 | 0.069 | 0.679<1 OK |
| A4 | 1 | 1.047 | -57.21 | -252.683 | 4.877 | 0.053 | 0.504<1 OK |
| B1 | 1 | 1.047 | -58.057 | 0 | 99.51 | 0.054 | 0.42<1 OK |
| B2 | 1 | 1.047 | -154.474 | -320.136 | 0 | 0.143 | 0.653<1 OK |
| B3 | 1 | 1.047 | -156.925 | -311.147 | 0 | 0.145 | 0.637<1 OK |
| B4 | 1 | 1.047 | -97.044 | 0 | 147.79 | 0.09 | 0.629<1 OK |
| C1 | 1 | 1.047 | -22.208 | -217.489 | -4.649 | 0.021 | 0.423<1 OK |
| C2 | 1 | 1.047 | -85.48 | -13.785 | 158.201 | 0.079 | 0.69<1 OK |
| C3 | 1 | 1.047 | -74.371 | -13.938 | 156.798 | 0.069 | 0.679<1 OK |
| C4 | 1 | 1.047 | -57.21 | -252.683 | 4.877 | 0.053 | 0.504<1 OK |

Table A.9 First-order Results of Lowest Level in Jackson (Y-direction)

| Column No. | 1.2D+0.5L+EY | | | | | |
|---|---|---|---|---|---|---|
| | NT (1.2D+0.5L) | | | LT(EY) | | |
| | $P_{nt}$ (kip) | $M_{ntx}$ (kip-ft) | $M_{nty}$ (kip-ft) | $P_{lt}$ (kip) | $M_{ltx}$ (kip-ft) | $M_{lty}$ (kip-ft) |
| A1 | -39.006 | 17.6338 | 4.733 | 18.707 | 0.477 | -64.24 |
| A2 | -79.845 | 13.862 | 0.735 | 17.458 | -122.646 | -0.063 |
| A3 | -80.006 | 13.862 | -0.669 | 17.458 | -122.646 | 0.063 |
| A4 | -40.412 | -17.56 | 4.794 | 18.707 | -0.477 | -64.24 |
| B1 | -75.473 | 0 | -24.24 | 0 | -164.235 | 0 |
| B2 | -155.557 | -4.603 | 0 | 0 | 0 | -81.961 |
| B3 | -155.842 | 4.386 | 0 | 0 | 0 | -81.961 |
| B4 | -79.628 | 0 | 24.04 | 0 | -164.235 | 0 |
| C1 | -39.006 | 17.634 | -4.733 | -18.707 | -0.477 | -64.24 |
| C2 | -79.845 | -13.862 | 0.735 | -17.458 | -122.646 | 0.063 |
| C3 | -80.006 | -13.862 | -0.669 | -17.458 | -122.646 | -0.063 |
| C4 | -40.412 | -17.576 | -4.794 | -18.707 | 0.477 | -64.24 |

Table A.10 Amplified First-order Results of Lowest Level in Jackson (Y-direction)

| Column No. | 1.2D+0.5L+EY | | | | | | |
|---|---|---|---|---|---|---|---|
| | $B_1$ | $B_2$ | $P_r$(kip) | $M_{rx}$(kip-ft) | $M_{ry}$(kip-ft) | $P_r/\Phi P_n$ | Interaction ratio |
| A1 | 1 | 1.038 | -19.597 | 18.128 | -61.919 | 0.018 | 0.287<1 OK |
| A2 | 1 | 1.038 | -61.732 | -113.389 | 0.669 | 0.057 | 0.237<1 OK |
| A3 | 1 | 1.038 | -61.893 | -113.388 | -0.604 | 0.057 | 0.237<1 OK |
| A4 | 1 | 1.038 | -21.003 | -18.054 | -61.858 | 0.019 | 0.287<1 OK |
| B1 | 1 | 1.038 | -75.473 | -170.4 | -24.24 | 0.07 | 0.44<1 OK |
| B2 | 1 | 1.038 | -155.557 | -4.603 | -85.037 | 0.144 | 0.416<1 OK |
| B3 | 1 | 1.038 | -155.842 | 4.386 | -85.037 | 0.144 | 0.416<1 OK |
| B4 | 1 | 1.038 | -79.628 | -170.4 | 24.24 | 0.074 | 0.441<1 OK |
| C1 | 1 | 1.038 | -58.509 | 17.139 | -71.384 | 0.054 | 0.34<1 OK |
| C2 | 1 | 1.038 | -97.958 | -141.112 | 0.8 | 0.091 | 0.298<1 OK |
| C3 | 1 | 1.038 | -98.119 | -141.112 | -0.734 | 0.091 | 0.304<1 OK |
| C4 | 1 | 1.038 | -59.915 | -17.065 | -71.445 | 0.055 | 0.341<1 OK |

Table A.11 First-order Results of Lowest Level in Jackson (X-direction)

| Column No. | 1.2D+0.5L+EX | | | | | |
|---|---|---|---|---|---|---|
| | NT (1.2D+0.5L) | | | LT(EX) | | |
| | $P_{nt}$ (kip) | $M_{ntx}$ (kip-ft) | $M_{nty}$ (kip-ft) | $P_{lt}$ (kip) | $M_{ltx}$ (kip-ft) | $M_{lty}$ (kip-ft) |
| A1 | -39.006 | 17.6338 | 4.733 | 8.505 | -119.047 | -0.042 |
| A2 | -79.845 | 13.862 | 0.735 | -2.853 | -0.039 | 79.728 |
| A3 | -80.006 | 13.862 | -0.669 | 2.853 | 0.039 | 79.728 |
| A4 | -40.412 | -17.56 | 4.794 | -8.505 | -119.047 | 0.042 |
| B1 | -75.473 | 0 | -24.24 | 8.818 | 0 | 62.657 |
| B2 | -155.557 | -4.603 | 0 | 0.548 | -159.76 | 0 |
| B3 | -155.842 | 4.386 | 0 | -0.548 | -159.76 | 0 |
| B4 | -79.628 | 0 | 24.04 | -8.818 | 0 | 62.657 |
| C1 | -39.006 | 17.634 | -4.733 | 8.505 | -119.047 | 0.042 |
| C2 | -79.845 | -13.862 | 0.735 | -2.853 | 0.039 | 79.728 |
| C3 | -80.006 | -13.862 | -0.669 | 2.853 | -0.039 | 79.728 |
| C4 | -40.412 | -17.576 | -4.794 | -8.505 | -119.047 | -0.042 |

Table A.12 Amplified First-order Results of Lowest Level in Jackson (X-direction)

| Column No. | 1.2D+0.5L+EX | | | | | | |
|---|---|---|---|---|---|---|---|
| | $B_1$ | $B_2$ | $P_r$(kip) | $M_{rx}$(kip-ft) | $M_{ry}$(kip-ft) | $P_r/\Phi P_n$ | Interaction ratio |
| A1 | 1 | 1.047 | -30.105 | -106.954 | 4.688 | 0.028 | 0.277<1 OK |
| A2 | 1 | 1.047 | -82.831 | 13.821 | 84.174 | 0.077 | 0.396<1 OK |
| A3 | 1 | 1.047 | -77.02 | 13.902 | 82.77 | 0.071 | 0.388<1 OK |
| A4 | 1 | 1.047 | -49.313 | -142.148 | 4.838 | 0.046 | 0.3<1 OK |
| B1 | 1 | 1.047 | -66.245 | 0 | 41.333 | 0.061 | 0.194<1 OK |
| B2 | 1 | 1.047 | -154.983 | -171.799 | 0 | 0.144 | 0.384<1 OK |
| B3 | 1 | 1.047 | -156.417 | -162.81 | 0 | 0.145 | 0.368<1 OK |
| B4 | 1 | 1.047 | -88.856 | 0 | 89.613 | 0.082 | 0.395<1 OK |
| C1 | 1 | 1.047 | -30.105 | -106.954 | -4.688 | 0.028 | 0.277<1 OK |
| C2 | 1 | 1.047 | -82.831 | -13.821 | 84.174 | 0.077 | 0.396<1 OK |
| C3 | 1 | 1.047 | -77.02 | -13.902 | 82.77 | 0.071 | 0.388<1 OK |
| C4 | 1 | 1.047 | -49.313 | -142.148 | -4.838 | 0.046 | 0.3<1 OK |

Table A.13 First-order Results of Lowest Level in Gulfport (Y-direction)

| Column No. | 1.2D+0.5L+EY | | | | | |
|---|---|---|---|---|---|---|
| | NT (1.2D+0.5L) | | | LT(EY) | | |
| | $P_{nt}$ (kip) | $M_{ntx}$ (kip-ft) | $M_{nty}$ (kip-ft) | $P_{lt}$ (kip) | $M_{ltx}$ (kip-ft) | $M_{lty}$ (kip-ft) |
| A1 | -39.006 | 17.6338 | 4.733 | 8.812 | 0.225 | -30.26 |
| A2 | -79.845 | 13.862 | 0.735 | 8.244 | -57.772 | -0.03 |
| A3 | -80.006 | 13.862 | -0.669 | 8.244 | -57.772 | 0.03 |
| A4 | -40.412 | -17.56 | 4.794 | 8.812 | -0.224 | -30.26 |
| B1 | -75.473 | 0 | -24.24 | 0 | -77.362 | 0 |
| B2 | -155.557 | -4.603 | 0 | 0 | 0 | -38.607 |
| B3 | -155.842 | 4.386 | 0 | 0 | 0 | -38.607 |
| B4 | -79.628 | 0 | 24.04 | 0 | -77.362 | 0 |
| C1 | -39.006 | 17.634 | -4.733 | -8.812 | -0.225 | -30.26 |
| C2 | -79.845 | -13.862 | 0.735 | -8.244 | -57.772 | 0.03 |
| C3 | -80.006 | -13.862 | -0.669 | -8.244 | -57.772 | -0.03 |
| C4 | -40.412 | -17.576 | -4.794 | -8.812 | 0.225 | -30.26 |

Table A.14 Amplified First-order Results of Lowest Level in Gulfport (Y-direction)

| Column No. | 1.2D+0.5L+EY | | | | | | |
|---|---|---|---|---|---|---|---|
| | $B_1$ | $B_2$ | $P_r$(kip) | $M_{rx}$(kip-ft) | $M_{ry}$(kip-ft) | $P_r/\Phi P_n$ | Interaction ratio |
| A1 | 1 | 1.038 | -29.863 | 17.867 | -26.663 | 0.028 | 0.152<1 OK |
| A2 | 1 | 1.038 | -71.312 | -46.078 | 0.704 | 0.066 | 0.119<1 OK |
| A3 | 1 | 1.038 | -71.474 | -46.078 | -0.638 | 0.066 | 0.119<1 OK |
| A4 | 1 | 1.038 | -31.269 | -17.792 | -26.602 | 0.029 | 0.152<1 OK |
| B1 | 1 | 1.038 | -75.473 | -80.265 | -24.24 | 0.07 | 0.276<1 OK |
| B2 | 1 | 1.038 | -155.557 | -4.603 | -40.056 | 0.144 | 0.239<1 OK |
| B3 | 1 | 1.038 | -155.842 | 4.386 | -41.093 | 0.144 | 0.243<1 OK |
| B4 | 1 | 1.038 | -79.628 | -80.265 | 24.04 | 0.074 | 0.278<1 OK |
| C1 | 1 | 1.038 | -48.149 | 17.401 | -36.128 | 0.045 | 0.197<1 OK |
| C2 | 1 | 1.038 | -88.378 | -73.801 | 0.765 | 0.082 | 0.178<1 OK |
| C3 | 1 | 1.038 | -88.538 | -73.801 | -0.699 | 0.082 | 0.178<1 OK |
| C4 | 1 | 1.038 | -49.555 | -17.327 | -36.189 | 0.046 | 0.197<1 OK |

Table A.15 First-order Results of Lowest Level in Gulfport (X-direction)

| Column No. | 1.2D+0.5L+EX | | | | | |
|---|---|---|---|---|---|---|
| | NT (1.2D+0.5L) | | | LT(EX) | | |
| | $P_{nt}$ (kip) | $M_{ntx}$ (kip-ft) | $M_{nty}$ (kip-ft) | $P_{lt}$ (kip) | $M_{ltx}$ (kip-ft) | $M_{lty}$ (kip-ft) |
| A1 | -39.006 | 17.6338 | 4.733 | 4.006 | -56.076 | -0.02 |
| A2 | -79.845 | 13.862 | 0.735 | -1.344 | -0.018 | 37.556 |
| A3 | -80.006 | 13.862 | -0.669 | 1.344 | 0.018 | 37.556 |
| A4 | -40.412 | -17.56 | 4.794 | -4.006 | -56.076 | 0.02 |
| B1 | -75.473 | 0 | -24.24 | 4.154 | 0 | 29.514 |
| B2 | -155.557 | -4.603 | 0 | 0.258 | -75.254 | 0 |
| B3 | -155.842 | 4.386 | 0 | -0.258 | -75.254 | 0 |
| B4 | -79.628 | 0 | 24.04 | -4.154 | 0 | 29.514 |
| C1 | -39.006 | 17.634 | -4.733 | 4.006 | -56.076 | 0.02 |
| C2 | -79.845 | -13.862 | 0.735 | -1.344 | 0.018 | 37.556 |
| C3 | -80.006 | -13.862 | -0.669 | 1.344 | -0.018 | 37.556 |
| C4 | -40.412 | -17.576 | -4.794 | -4.006 | -56.076 | -0.02 |

Table A.16 Amplified First-order Results of Lowest Level in Gulfport (X-direction)

| Column No. | 1.2D+0.5L+EX | | | | | | |
|---|---|---|---|---|---|---|---|
| | $B_1$ | $B_2$ | $P_r$(kip) | $M_{rx}$(kip-ft) | $M_{ry}$(kip-ft) | $P_r/\Phi P_n$ | Interaction ratio |
| A1 | 1 | 1.047 | -34.813 | -41.057 | 4.712 | 0.032 | 0.109<1 OK |
| A2 | 1 | 1.047 | -81.252 | 13.842 | 40.041 | 0.075 | 0.221<1 OK |
| A3 | 1 | 1.047 | -78.599 | 13.881 | 38.638 | 0.073 | 0.214<1 OK |
| A4 | 1 | 1.047 | -44.065 | -76.251 | 4.815 | 0.041 | 0.178<1 OK |
| B1 | 1 | 1.047 | -71.125 | 0 | 6.65 | 0.066 | 0.059<1 OK |
| B2 | 1 | 1.047 | -155.287 | -83.365 | 0 | 0.144 | 0.223<1 OK |
| B3 | 1 | 1.047 | -156.112 | -74.376 | 0 | 0.145 | 0.207<1 OK |
| B4 | 1 | 1.047 | -83.976 | 0 | 54.93 | 0.078 | 0.256<1 OK |
| C1 | 1 | 1.047 | -34.813 | -41.057 | -4.712 | 0.032 | 0.109<1 OK |
| C2 | 1 | 1.047 | -81.252 | -13.842 | 40.041 | 0.075 | 0.221<1 OK |
| C3 | 1 | 1.047 | -78.599 | -13.881 | 38.638 | 0.073 | 0.214<1 OK |
| C4 | 1 | 1.047 | -44.065 | -76.251 | -4.815 | 0.041 | 0.178<1 OK |

APPENDIX B

WIND LOAD ANALYSIS RESULTS

Table B.1 First-order Results of Lowest Level in Southaven, Batesville and Jackson (X-direction)

| Column No. | 1.2D+0.5L+0.5LR+1.6WX | | | | | |
|---|---|---|---|---|---|---|
| | NT (1.2D+0.5L+0.5LR+1.6WR) | | | LT(1.6WX) | | |
| | $P_{nt}$ (kip) | $M_{ntx}$ (kip-ft) | $M_{nty}$ (kip-ft) | $P_{lt}$ (kip) | $M_{ltx}$ (kip-ft) | $M_{lty}$ (kip-ft) |
| A1 | -35.441 | 17.334 | 4.682 | 5.34 | -36.887 | 7.362 |
| A2 | -72.736 | 13.658 | 0.673 | 1.189 | -17.761 | 23.419 |
| A3 | -72.889 | 13.66 | -0.637 | 1.351 | -5.74 | 23.408 |
| A4 | -37.086 | -17.32 | 4.731 | -2.622 | -36.801 | 2.972 |
| B1 | -70.083 | 0.023 | -23.881 | 2.339 | -38.618 | 21.285 |
| B2 | -144.481 | -4.315 | 0.034 | 0.151 | -54.789 | -11.183 |
| B3 | -144.768 | 4.199 | 0.034 | -0.15 | -54.698 | -3.727 |
| B4 | -74.261 | 0.045 | 23.773 | -2.34 | 7.698 | 21.184 |
| C1 | -37.353 | 17.468 | -4.717 | -0.835 | -47.495 | -14.908 |
| C2 | -76.56 | -13.689 | 0.703 | -2.698 | -17.764 | 29.598 |
| C3 | -76.718 | -13.689 | -0.66 | 0.164 | -5.775 | 29.525 |
| C4 | -38.803 | -17.403 | -4.773 | -1.888 | -47.113 | 2.937 |

Table B.2 Amplified First-order Results of Lowest Level in Southaven, Batesville and Jackson

(X-direction)

| Column No.. | 1.2D+0.5L+0.5LR+1.6WX | | | | | | |
|---|---|---|---|---|---|---|---|
| | $B_1$ | $B_2$ | $P_r$(kip) | $M_{rx}$(kip-ft) | $M_{ry}$(kip-ft) | $P_r/\Phi P_n$ | Interaction ratio |
| A1 | 1 | 1.03 | -29.941 | -20.665 | 12.264 | 0.028 | 0.1<1 OK |
| A2 | 1 | 1.03 | -71.551 | -4.634 | 24.791 | 0.066 | 0.14<1 OK |
| A3 | 1 | 1.03 | -71.498 | 7.748 | 23.471 | 0.066 | 0.14<1 OK |
| A4 | 1 | 1.03 | -39.786 | -55.222 | 7.792 | 0.037 | 0.149<1 OK |
| B1 | 1 | 1.03 | -67.674 | -39.749 | -1.959 | 0.063 | 0.111<1 OK |
| B2 | 1 | 1.03 | -144.325 | -60.742 | -11.483 | 0.134 | 0.222<1 OK |
| B3 | 1 | 1.03 | -144.922 | -52.134 | -3.804 | 0.134 | 0.177<1 OK |
| B4 | 1 | 1.03 | -76.671 | 7.973 | 45.59 | 0.071 | 0.23<1 OK |
| C1 | 1 | 1.03 | -38.213 | -31.447 | -20.07 | 0.035 | 0.154<1 OK |
| C2 | 1 | 1.03 | -79.339 | -31.984 | 31.185 | 0.073 | 0.218<1 OK |
| C3 | 1 | 1.03 | -76.549 | -19.636 | 29.747 | 0.071 | 0.189<1 OK |
| C4 | 1 | 1.03 | -40.747 | -65.925 | -1.749 | 0.038 | 0.145<1 OK |

Table B.3 First-order Results of Lowest Level in Southaven, Batesville and Jackson (Y-direction)

| Column No. | 1.2D+0.5L+0.5LR+1.6WY | | | | | |
|---|---|---|---|---|---|---|
| | NT (1.2D+0.5L+0.5LR+1.6WR) | | | LT(1.6WY) | | |
| | $P_{nt}$ (kip) | $M_{ntx}$ (kip-ft) | $M_{nty}$ (kip-ft) | $P_{lt}$ (kip) | $M_{ltx}$ (kip-ft) | $M_{lty}$ (kip-ft) |
| A1 | -35.441 | 17.334 | 4.682 | 14.636 | 2.408 | -64.644 |
| A2 | -72.736 | 13.658 | 0.673 | 13.268 | -123.675 | -1.216 |
| A3 | -72.889 | 13.66 | -0.637 | 12.753 | -119.102 | -1.127 |
| A4 | -37.086 | -17.32 | 4.731 | 13.318 | 1.593 | -57.817 |
| B1 | -70.083 | 0.023 | -23.881 | 0 | -166.793 | 0 |
| B2 | -144.481 | -4.315 | 0.034 | 0 | 0.006 | -78.25 |
| B3 | -144.768 | 4.199 | 0.034 | 0 | -0.009 | -75.41 |
| B4 | -74.261 | 0.045 | 23.773 | 0 | -149.141 | 0.003 |
| C1 | -37.353 | 17.468 | -4.717 | -14.637 | -2.322 | -64.56 |
| C2 | -76.56 | -13.689 | 0.703 | -13.272 | -123.505 | 1.209 |
| C3 | -76.718 | -13.689 | -0.66 | -12.757 | -118.932 | 1.136 |
| C4 | -38.803 | -17.403 | -4.773 | -13.321 | -1.671 | -57.748 |

Table B.4 Amplified First-order Results of Lowest Level in Southaven, Batesville and Jackson

(Y-direction)

| Column No. | 1.2D+0.5L+0.5LR+1.6WY | | | | | | |
|---|---|---|---|---|---|---|---|
| | $B_1$ | $B_2$ | $P_r$(kip) | $M_{rx}$(kip-ft) | $M_{ry}$(kip-ft) | $P_r/\Phi P_n$ | Interaction ratio |
| A1 | 1 | 1.023 | -20.465 | 19.798 | -61.464 | 0.019 | 0.288<1 OK |
| A2 | 1 | 1.023 | -59.16 | -112.89 | -0.571 | 0.055 | 0.235<1 OK |
| A3 | 1 | 1.023 | -59.84 | -108.209 | -1.79 | 0.055 | 0.231<1 OK |
| A4 | 1 | 1.023 | -23.459 | -15.691 | -54.429 | 0.022 | 0.254<1 OK |
| B1 | 1 | 1.023 | -70.083 | -170.644 | -23.882 | 0.065 | 0.437<1 OK |
| B2 | 1 | 1.023 | -144.481 | -4.308 | -80.33 | 0.134 | 0.391<1 OK |
| B3 | 1 | 1.023 | -144.768 | 4.19 | 77.127 | 0.134 | 0.379<1 OK |
| B4 | 1 | 1.023 | -74.261 | -152.56 | 23.776 | 0.069 | 0.405<1 OK |
| C1 | 1 | 1.023 | -52.33 | 15.092 | -70.776 | 0.048 | 0.331<1 OK |
| C2 | 1 | 1.023 | -90.14 | -140.062 | 1.94 | 0.083 | 0.304<1 OK |
| C3 | 1 | 1.023 | -89.771 | -135.383 | 0.502 | 0.083 | 0.289<1 OK |
| C4 | 1 | 1.023 | -52.433 | -19.113 | -63.862 | 0.049 | 0.311<1 OK |

Table B.5 First-order Results of Lowest Level in Gulfport (X-direction)

| Column No. | 1.2D+0.5L+0.5LR+1.6WX | | | | | |
|---|---|---|---|---|---|---|
| | NT (1.2D+0.5L+0.5LR+1.6WR) | | | LT(1.6WX) | | |
| | $P_{nt}$ (kip) | $M_{ntx}$ (kip-ft) | $M_{nty}$ (kip-ft) | $P_{lt}$ (kip) | $M_{ltx}$ (kip-ft) | $M_{lty}$ (kip-ft) |
| A1 | -31.004 | 16.938 | 4.662 | 10.329 | -71.375 | -28.942 |
| A2 | -63.268 | 13.366 | 0.594 | 2.298 | -34.342 | 45.314 |
| A3 | -63.386 | 13.369 | -0.595 | 2.612 | -11.094 | 45.294 |
| A4 | -32.632 | -16.992 | 4.649 | -5.073 | -71.21 | 5.752 |
| B1 | -62.821 | 0.045 | -23.384 | 4.525 | -74.679 | 41.184 |
| B2 | -129.972 | -3.911 | 0.067 | 0.293 | -106.01 | -21.624 |
| B3 | -128.433 | 3.942 | 0.067 | -0.291 | -105.833 | -7.205 |
| B4 | -66.348 | 0.088 | 23.379 | -4.528 | 14.896 | 40.987 |
| C1 | -34.71 | 17.198 | -4.689 | -1.612 | -91.892 | -28.828 |
| C2 | -70.736 | -13.426 | 0.652 | -5.218 | -34.349 | 57.264 |
| C3 | -70.867 | -13.426 | -0.642 | 0.318 | -11.156 | 57.123 |
| C4 | -35.985 | -17.153 | -4.732 | -3.652 | -91.153 | 5.683 |

Table B.6 Amplified First-order Results of Lowest Level in Gulfport (X-direction)

| Column No. | 1.2D+0.5L+0.5LR+1.6WX | | | | | | |
|---|---|---|---|---|---|---|---|
| | $B_1$ | $B_2$ | $P_r$(kip) | $M_{rx}$(kip-ft) | $M_{ry}$(kip-ft) | $P_r/\Phi P_n$ | Interaction ratio |
| A1 | 1 | 1.027 | -20.399 | -56.343 | -25.093 | 0.019 | 0.211<1 OK |
| A2 | 1 | 1.027 | -60.909 | -21.893 | 47.118 | 0.056 | 0.254<1 OK |
| A3 | 1 | 1.027 | -60.704 | 1.979 | 45.908 | 0.056 | 0.213<1 OK |
| A4 | 1 | 1.027 | -37.84 | -90.103 | 10.554 | 0.035 | 0.223<1 OK |
| B1 | 1 | 1.027 | -58.175 | -76.628 | 18.9 | 0.054 | 0.241<1 OK |
| B2 | 1 | 1.027 | -129.671 | -112.751 | -22.134 | 0.12 | 0.352<1 OK |
| B3 | 1 | 1.027 | -128.732 | -104.716 | -7.33 | 0.119 | 0.279<1 OK |
| B4 | 1 | 1.027 | -70.997 | 15.382 | 65.461 | 0.066 | 0.32<1 OK |
| C1 | 1 | 1.027 | -36.365 | -77.147 | -34.287 | 0.034 | 0.224<1 OK |
| C2 | 1 | 1.027 | -76.093 | -48.692 | 59.445 | 0.07 | 0.359<1 OK |
| C3 | 1 | 1.027 | -70.541 | -24.879 | 58.007 | 0.065 | 0.307<1 OK |
| C4 | 1 | 1.027 | -39.735 | -110.74 | 1.103 | 0.037 | 0.224<1 OK |

Table B.7 First-order Results of Lowest Level in Gulfport (Y-direction)

| Column No. | 1.2D+0.5L+0.5LR+1.6WY | | | | | |
|---|---|---|---|---|---|---|
| | NT (1.2D+0.5L+0.5LR+1.6WR) | | | LT(1.6WY) | | |
| | $P_{nt}$ (kip) | $M_{ntx}$ (kip-ft) | $M_{nty}$ (kip-ft) | $P_{lt}$ (kip) | $M_{ltx}$ (kip-ft) | $M_{lty}$ (kip-ft) |
| A1 | -31.004 | 16.938 | 4.662 | 28.935 | 8.129 | -127.721 |
| A2 | -63.268 | 13.366 | 0.594 | 25.141 | -236.293 | -4.661 |
| A3 | -63.386 | 13.369 | -0.595 | 22.523 | -219.744 | -4.449 |
| A4 | -32.632 | -16.992 | 4.649 | 22.235 | 6.632 | -101.41 |
| B1 | -62.821 | 0.045 | -23.384 | 0 | -329.482 | -0.003 |
| B2 | -129.972 | -3.911 | 0.067 | 0 | 0.011 | -149.172 |
| B3 | -128.433 | 3.942 | 0.067 | 0 | -0.019 | -138.054 |
| B4 | -66.348 | 0.088 | 23.379 | 0.017 | -262.015 | 0.007 |
| C1 | -34.71 | 17.198 | -4.689 | -28.936 | -7.966 | -127.557 |
| C2 | -70.736 | -13.426 | 0.652 | -25.149 | -235.865 | 4.598 |
| C3 | -70.867 | -13.426 | -0.642 | -22.533 | -218.744 | 4.467 |
| C4 | -35.985 | -17.153 | -4.732 | -22.25 | -6.778 | -101.28 |

Table B.8 Amplified First-order Results of Lowest Level in Gulfport (Y-direction)

| Column No. | 1.2D+0.5L+0.5LR+1.6WY | | | | | | |
|---|---|---|---|---|---|---|---|
| | $B_1$ | $B_2$ | $P_r$(kip) | $M_{rx}$(kip-ft) | $M_{ry}$(kip-ft) | $P_r/\Phi P_n$ | Interaction ratio |
| A1 | 1 | 1.021 | -1.454 | 25.239 | -125.812 | 0.001 | 0.544<1 OK |
| A2 | 1 | 1.021 | -37.593 | -227.844 | -4.115 | 0.035 | 0.447<1 OK |
| A3 | 1 | 1.021 | -40.385 | -211.042 | -5.139 | 0.037 | 0.422<1 OK |
| A4 | 1 | 1.021 | -9.925 | 10.219 | -98.915 | 0.009 | 0.414<1 OK |
| B1 | 1 | 1.021 | -62.821 | -336.435 | -23.387 | 0.058 | 0.732<1 OK |
| B2 | 1 | 1.021 | -129.972 | -3.899 | -152.274 | 0.12 | 0.669<1 OK |
| B3 | 1 | 1.021 | -128.433 | 3.923 | -140.919 | 0.119 | 0.624<1 OK |
| B4 | 1 | 1.021 | -66.331 | -267.492 | 23.387 | 0.061 | 0.609<1 OK |
| C1 | 1 | 1.021 | -64.261 | 9.062 | -134.956 | 0.06 | 0.58<1 OK |
| C2 | 1 | 1.021 | -96.419 | -254.3 | 5.348 | 0.089 | 0.527<1 OK |
| C3 | 1 | 1.021 | -93.879 | -236.815 | 3.92 | 0.087 | 0.489<1 OK |

# VITA

Yihong Shi was born at December 03, 1975 in Beijing, China to Youfu Shi and Xiuying Li. She spent her childhood life and did her schooling in Beijing. She completed her under-graduation in Civil Engineering from Beijing Institute of Civil Engineering and Architecture in 1998. At the same year, she began her first work in Beijing Residence Construction Engineering Company. After four years, she began her second job as an estimator in Beijing Guohua Investment and Real Estate Company. In 2007, she came to the United State with her husband and began her new life in American.

In January, 2009, she began to pursue her Master Degree with emphasis in Civil Engineering with Dr. Christopher Mullen as her advisor.