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DESIGN AND EVALUATION OF RECTANGULAR CONCRETE FILLED TUBE (RCFT) FRAMES FOR SEISMIC DEMAND ASSESSMENT

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ABSTRACT

The research presented in this study is part of a larger ongoing research effort whose goal is to develop a performance-based design methodology for seismic engineering of low-rise and mid-rise buildings with lateral force resisting systems made of rectangular concrete-filled steel tubes (RCFT). One of the critical steps in developing a performance-based design methodology for RCFT columns, connections, and frames is the assessment of seismic demand. The demand assessment is considered sufficiently comprehensive when it is performed on a suite of buildings that exhibit a wide range of RCFT moment-resisting frame structural responses. The research presented in this report describes the process and methodologies that were used to develop this comprehensive suite of buildings suitable for a seismic demand assessment of RCFT frames.

This study involved the following two main tasks: 1) design a suite of buildings with lateral force resisting systems made of steel girders framing into RCFT columns with moment-resisting connections using linear static analysis methods, and 2) analyze the suite of buildings using nonlinear static pushover analysis methods to assess the comprehensiveness of the suite and to determine if the buildings cover the full range of possible composite behavior that is expected for low-rise and mid-rise RCFT buildings.

The first task involved designing the RCFT column and wide flange girder section sizes for every building of the suite. Linear static analysis methods incorporating second-order effects were used to design all of the moment-resisting frames. Thirteen three-dimensional structures make up the suite of buildings from which a series of thirteen two-dimensional frames were designed. Six of these buildings were specifically designed as described in this report, whereas the other seven were designed in prior research. The thirteen buildings in the suite varied between each other in their number of stories (i.e., 3-story, 9-story, and 18-story), bay spacing, the type of live loading (i.e., office building live load or warehouse live load), the column material design strengths (i.e., column yield strengths of 46 ksi and 80 ksi, and concrete compressive strengths of 4 ksi and 16 ksi), and the column d/t ratios [i.e., the maximum allowed d/t ratio was set at either the AISC (AISC, 2005) limit or a value of 80].

The second task involved using three methods to assess the comprehensiveness of the suite of buildings. The first method made a comparison of the final building seismic designs with the elastic seismic design spectrum. The results show that the major portions of the seismic design spectrum were used to design the suite of buildings. The second method involved developing an envelope of expected upper bound and lower bound system response curves for a pushover analysis and then comparing this envelope to the pushover curves of the suite of buildings. This assessment demonstrated that all of the buildings exhibited system responses that are within the envelope of possible system response. The third method utilized the rigidity ratio to verify the overstrength value of each building and to show that all of the buildings do not have too much inherent overstrength built into them. Together these three methods were able to demonstrate that the thirteen buildings chosen to make up the suite of buildings are comprehensive as a group, and that together they cover a wide range of expected performance of RCFT moment-resisting frames.

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Chapter 1

Introduction

Rectangular concrete-filled steel tube (RCFT) columns are becoming more popular to use in new building construction in the United States. However, bare steel or reinforced concrete columns are still used more extensively than RCFTs due to the lack of knowledge and experience that U.S. engineers have with RCFT structural systems. One part of the current research in RCFT design is the ongoing development of a performance-based design (PBD) methodology exclusively for RCFT columns, connections, and frames (e.g., Tort and Hajjar, 2004). When completed, this methodology will be able to provide a more detailed approach for RCFT building design and evaluation.

1.1 Background

RCFT columns have several advantages over both bare steel columns and reinforced concrete columns. RCFT columns are able to take advantage of the inherent strengths of both the steel and concrete. The outer steel "shell" of a RCFT column is able to utilize the tension strength of steel. By placing the steel farthest from the neutral axis of the column the steel is able to efficiently resist the flexural bending. The inner core of a RCFT column is then able to utilize the compressive strength of the concrete. The outer steel shell confines the concrete core, which further increases the compressive strength of the column. The concrete core will delay the local buckling of the outer steel shell, which provides further strength increases in the column section. By allowing the steel and concrete to compliment each other within the column cross section, the RCFT column becomes a very efficient structural section both in strength and in overall building costs.

Performance-based design (PBD) is not a new concept in the United States. However, a reliability-based PBD methodology for RCFT frame systems does not exist. One step in developing a PBD methodology for RCFT structural systems is the demand assessment that requires a transient dynamic analysis to be performed on multiple buildings made of RCFT structural systems. For the PBD methodology to be able to predict the performance of a RCFT structural system, these RCFT buildings need to represent a broad range of possible building system performances and composite behavior that would be expected to occur in any RCFT structural system.

1.2 Objectives and Scope of Work

There are two objectives of the research presented in this report. The first objective was to design a suite of buildings that are to be used in a performance-based design demand assessment exclusively for RCFT structural systems. The demand assessment is not a part of this study and so it will not be included in the research presented in this report. The second objective was to demonstrate that the suite of buildings is comprehensive enough to represent a wide range of expected structural system behavior of RCFT systems.

A number of design parameters were established to limit the scope of this study and to provide consistency within the suite of buildings. All of the buildings are made of cold formed hollow structural sections (HSS) filled with concrete. The lateral force resisting system (LFRS) of each building was considered to be an unbraced special moment frame (SMF). All of the girders in the moment frames are made of hot rolled U.S. wide flange steel sections. All of the buildings are designed as regular structures with no plan irregularities or torsional irregularities. It was assumed that the floor diaphragms transfer the wind and seismic shear loads to each lateral force resisting system in proportion to their respective rigidities. Each building was designed assuming that each first story column is rigidly fixed at its base. All of the buildings have a constant story height throughout the entire building and every building in the suite uses the same story height. Each building was assumed to be located at the same site in a region of the United States that is expected to experience high seismic activity.

This study included the design and assessment of low-rise and mid-rise RCFT buildings. Three building heights were chosen to represent this range of buildings – 3-story, 9-story, and 18-story. The 3-story building designs are from the work of La Fore and Hajjar (2005) while the 9-story and 18-story buildings were designed for this study as described in Chapter 3 of this report. Three of the 3-story buildings and all of the 9-story and 18-story buildings were designed using office-building live loads. In an effort to increase the axial load in the columns, the bay spacing varied between some of the buildings and four of the 3-story buildings were designed using warehouse type live loads. Therefore, a total of thirteen buildings were chosen to make up the final suite of buildings: (7) 3-story buildings, (3) 9-story buildings, and (3) 18-story buildings.

The major differences in design parameters between the thirteen buildings that make up the suite of buildings is the material design strengths of the columns, the allowed column d/t limits, and the number of stories in a building. The column HSSs in each building are made of ASTM A500 Grade B material, or equivalent. A constant design yield strength and design concrete compressive strength was used in each building. The buildings were designed using column yield strength values of 46 ksi or 80 ksi and design concrete compressive strength values of 4 ksi or 16 ksi. A concrete density of 145 lb/ft^3 was used in all of the buildings. The bare steel girders are made of ASTM A992 material with a design yield strength of 50 ksi. The maximum column d/t ratio ranges from the AISC limit allowed for RCFT columns (AISC, 2005) up to a maximum of 80. Each building used a constant 13-foot story height. Table 1.2.1 summarizes all of the design parameters that were used in each of the thirteen buildings that make of the final suite of buildings.

Design Parameter	3-Story Buildings							9-Story Buildings			18-Story Buildings		
	3A	3B	3C	3D	3E	3F	3G	9A	9B	9C	18A	18B	18C
Building (Live Load) Type	Office	Office	Office	Warehouse	Warehouse	Warehouse	Warehouse	Office	Office	Office	Office	Office	Office
Bay Spacing *[feet]*	30	30	30	30	30	20	20	30	30	30	20	20	20
Column F_{yc} *[ksi]*	46	80	80	46	80	46	80	46	80	50	46	80	50
Column f'_c *[ksi]*	4	16	16	4	16	4	16	4	16	16	4	16	16
Girder F_{yg} *[ksi]*	50	50	50	50	50	50	50	50	50	50	50	50	50
Target d/t	≤ AISC LIMIT	≤ AISC LIMIT	≤ 80	≤ AISC LIMIT	≤ AISC LIMIT	≤ AISC LIMIT	≤ AISC LIMIT	≤ AISC LIMIT	≤ AISC LIMIT	≤ 80	≤ AISC LIMIT	≤ AISC LIMIT	≤ 80

Table 1.2.1: Design Parameters For Each Building

All of the buildings in the suite were developed as three-dimensional structures, from which a series of two-dimensional (2D) moment-resisting frames were designed. This design approach was intended to be that of standard practices that are commonly available in a typical engineering office. The lateral force resisting system of each building was designed using linear methods with approximate second-order effects and the computer program VisualAnalysis. Equivalent static loads were used based on the building code design loads and LRFD load combinations. The wind and seismic design loads were calculated in accordance with the 2003 International Building Code while the RCFT design strengths are per the requirements of the 2005 AISC Specification.

Once the suite of thirteen buildings was determined, each building was analyzed using a nonlinear static pushover analysis to determine if the suite of buildings is comprehensive enough to provide a full range of possible composite system responses and behavior. The results of the pushover analysis were only used as a tool to provide insight into the behavior of each building. The nonlinear analysis program CFTMacro (Gourley and Hajjar, 1994) was used to perform the nonlinear static pushover analysis.

1.3 Organization of the Report

This report is divided into two major sections. The first section describes the process that was used to develop the suite of buildings and how each of the thirteen buildings was designed using the prescribed loads and member strengths. The second section summarizes the analysis process that was used to assess the suite of buildings and to determine if the suite is adequate in covering the full range of possible composite system behavior.

The first section of this report is composed of Chapter 2 and Chapter 3. Chapter 2 provides a brief summary of RCFT columns and general performance-based design methodology. Engineering demand parameters and how they relate to RCFT columns are described as well as the pushover analysis techniques and methods that were used in this study. Chapter 3 describes all of the design parameters and equations that were used in the second-order linear static analysis of each of the thirteen buildings. The actual calculations of the 9-story and 18-story buildings are included in the appendices at the end of this report.

The second section of this report is made of Chapter 4 through Chapter 6. Chapter 4 describes the design methods and parameters that were used in the nonlinear static pushover analysis for all thirteen buildings. Chapter 5 summarizes the results from both the second-order linear analysis and the nonlinear pushover analysis for each building. The linear analysis results mainly consist of the final column and girder section sizes for each building as well as the major system characteristics for each building such as the fundamental period, the elastic stiffness, and the interstory drifts. The nonlinear pushover analysis results consist of the pushover analysis curves and the system characteristics that were derived from these curves including the elastic stiffness, the fundamental period, the capacity, and the overstrength factor. Chapter 6 describes the methods and results of the building assessment process. Three methods were employed to determine if the suite of thirteen buildings was sufficient in providing the required composite behavior and performance that is needed for the seismic demand assessment. The first method included comparing the thirteen buildings with the design elastic seismic design spectrum while the second method compared the pushover analysis curve of each building with an envelope of expected pushover curve response. The third method verified the overstrength factor for each building through the introduction of a flexural rigidity ratio, η.

Chapter 7 provides a summary of all of the building designs and assessment findings followed by the final conclusions of the research that is presented in this report. Suggested future work is then presented for any companion studies or coincidental research in RCFT structural systems.

Chapter 2

Literature Review

In the last two decades performance-based building design has evolved in the United States to allow more decision variables to be included in the design process and to establish a reliability-based framework for the design procedures. This chapter summarizes some of the recent work in this field.

2.1 Performance-Based Design

Current building codes in the United States are based primarily on the requirement of "life safety," i.e. their primary goal is to have a building designed to prevent the loss of life of the occupants and those nearby by preventing building collapse (ATC, 2003). During the 1994 Northridge, California earthquake the building codes were put to the test and it was shown that they in large part fulfilled their goal. However, many buildings that were made of welded moment-resisting steel frames had brittle fracture damage in their connections. Even though these fractures did not cause the buildings to collapse or result in a loss of human life, they did cause a loss of millions of dollars for the business community due to repair costs and downtime for the buildings and their occupants.

The objective of the performance-based design methodology is to establish a criterion that will allow a building owner to select the expected building performance level for a specific hazard or event. Performance-based seismic engineering, PBSE, (also called performance-based earthquake engineering, PBEE) allows the building owner to choose how the building will perform during a seismic event, and to specify what kind of losses to expect after the prescribed seismic event has occurred. Performance levels then give the structural engineer a benchmark of the expected building response for a specific hazard level.

Numerous qualitative measures have been proposed by both FEMA 273 and the Vision 2000 Report to describe how a building will perform during an earthquake (ATC, 2003). Examples of the qualitative performance levels from FEMA 273 include immediate occupancy (IO), life safety (LS), and collapse prevention (CP). As an alternative to calibrating a major portion of a probability-based design methodology on engineering judgment, Wen et al., (2003) have suggested using quantitative methods to define building performance levels. Either nonlinear pushover analysis or incremental dynamic analysis techniques can then be used to rationally identify the expected performance of a building.

Figure 2.1.1 illustrates how a nonlinear pushover analysis might be used to identify three quantitative building system performance levels - first yield (FY) at the elastic limit, plastic mechanism initiation (PMI), and strength degradation (SD) which occurs just before collapse. Once these three quantitative performance levels are determined for a particular building they can be correlated to known qualitative performance levels, such as immediate occupancy (IO), life safety (LS), and collapse prevention (CP) respectively.

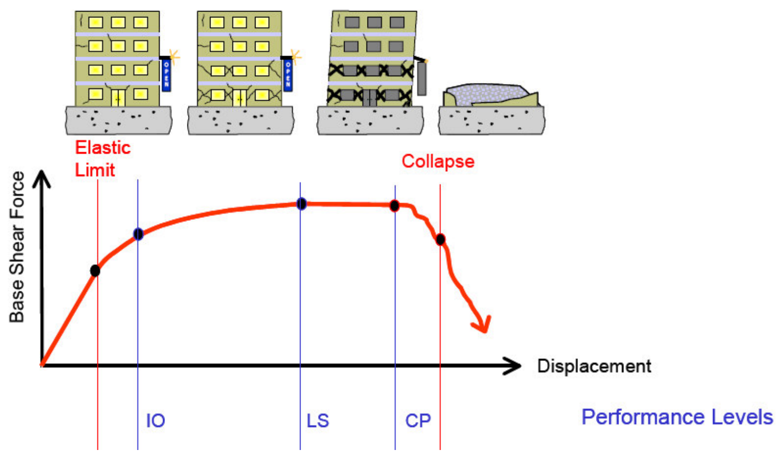


Figure 2.1.1: Determining Possible Performance Levels From a Pushover Analysis (from ATC, 2003)

FEMA 273 (ATC/BSSC, 1997) and the Vision 2000 Report (SEAOC, 1995) were one of the first national design guidelines to set discrete qualitative levels of performance for a building. Together they have set the benchmark for performance-based design methodologies in the United States. By utilizing current research the Applied Technology Council (ATC) has taken the performance levels from FEMA and Vision 2000 a step further by recommending the use of a continuum of levels rather than discrete performance levels. This continuum of performance levels, as illustrated in Figure 2.1.2, allows for a greater range of possible building designs for the building owner and structural engineer. A continuum also allows for the potential of further reductions in the cost of a building by almost any increment that the owner chooses to use. If the use of the continuum becomes too complex in the design of a building, ATC (2003) suggests that the design engineer can refer to the discrete levels of performance.

Four discrete levels of performance (four major categories in the continuum of performance levels) have been proposed by ATC (2003). These four proposed levels of performance (and their respective damage descriptions) are as follows: life safety (collapse prevention), interrupted occupancy and operations (significant or substantial damage), continued occupancy and interrupted operations (limited damage), and continued occupancy and operations (minimal to no damage). These four levels of performance are summarized in Table 2.1.1 along with their damage descriptions.

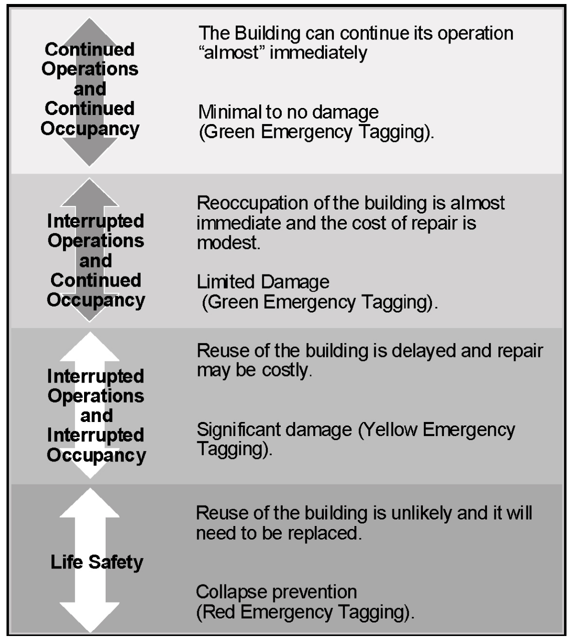


Figure 2.1.2: ATC (2003) Performance Level Continuum (from ATC, 2003)

Recommended Discrete Levels of Performance		
Performance Level	***Building Usability***	***Damage Description***
Life Safety	The building will most likely never be used again and it will need to be rebuilt	Collapse Prevention
Interrupted Occupancy and Operations	The building can be reused but repairs will be expensive	Significant or Substantial Damage
Continued Occupancy and Interrupted Operations	Reoccupation is almost immediate and the cost of repairs are moderate	Limited Damage
Continued Occupancy and Operations	The building is able to continue operations (almost) immediately with minimal to no repair	Minimal to No Damage

Table 2.1.1: ATC (2003) Recommended Discrete Levels of Performance (ATC, 2003)

To overcome shortcomings of earlier performance-based design approaches that were developed in the 1990's, the Pacific Earthquake Engineering Research Center (PEER) has developed a more scientifically based method of performance-based earthquake engineering (PBEE). Even though the use and development of this reliability-based PBEE is beyond the scope of this study, it has been summarized below to provide additional context to this study.

One of the improvements of the newer approaches to performance-based earthquake engineering (PBEE) compared to earlier PBEE approaches is that it allows for additional decision variables to be included in a building design. These new decision variables allow a building owner to evaluate the economics associated with constructing and maintaining a building or structure (ATC, 2003). ATC (2003) recommends that at least three more losses should be included in the design of a building in addition to the loss of life, which current building codes already take into account. Therefore in addition to preventing the loss of life through collapse prevention of the building, performance-based design now allows the structural engineer to determine the direct losses and downtime, as well as the indirect losses, associated with a specific design level earthquake. These four recommended losses are as follows:

- direct losses that include the repair costs,
- downtime associated with the building not being able to be used,
- indirect losses from the building not being able to be used,
- life loss and injuries to occupants and those near the building.

The newer approaches to PBEE are based upon a reliability-based performance-based design methodology since they have incorporated a computational method in all steps of the design process and risk evaluation. The triple integral, which is based on the total probability theorem and shown in Equation 2.1-1, is the basis for PBEE (ATC, 2004). This equation is the model that will be used in the development of a performance-based design methodology for RCFT column systems (Tort and Hajjar, 2004).

$$\mathrm{v(DV)} = \iiint \mathrm{G}\langle \mathrm{DV} \mid \mathrm{DM} \rangle \left| \mathrm{dG}\langle \mathrm{DM} \mid \mathrm{EDP} \rangle \right| \mathrm{dG}\langle \mathrm{EDP} \mid \mathrm{IM} \rangle \left| \mathrm{d}\lambda(\mathrm{IM}) \right. \qquad (2.1\text{-}1)$$

The development of a design methodology that is based on PBEE begins with defining a ground motion Intensity Measure (IM) that will define the ground motion hazard in a probabilistic manner. Next Engineering Demand Parameters (EDPs) are determined which will describe the response of the structural system in terms of a response parameter such as a displacement or force in a specific member. Damage Measures (DM) are then determined which describe the building and its components during the seismic event. Finally Decision Variables (DV) are developed that transform the damage into quantities that allow the owner to make an economical risk assessment of the building (Moehle and Deierlein, 2004). DV's allow for risk-related decisions to be a part of the initial building design process by including probable direct dollar losses, downtime, and potential casualties.

Typically the development of a probability-based performance-based methodology involves executing nonlinear analyses suitable for estimating the relevant range of EDP values for representative structures. With this new database of data, statistical relationships can be established that are often used to determine the probability that a specific EDP will exceed a set value for each known value of the IM (Moehle and Deierlein, 2004). An example of an annual probability curve for a maximum story drift EDP, generally established for a specific class of structure, is illustrated in Figure 2.1.3.

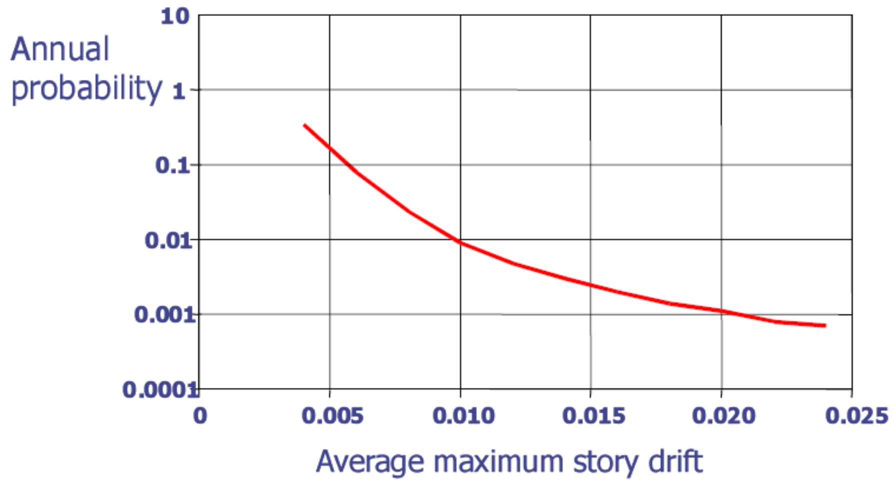


Figure 2.1.3: Example Annual Probability Curve For a Maximum Story Drift EDP (from Moehle and Deierlein, 2004)

2.2 Engineering Demand Parameters, EDPs

Engineering demand parameters are structural response values that are used to predict the damage in structural and nonstructural components of a building. Current building codes as well as the first generation of PBEE use EDPs in one form or another. The newer approaches to PBEE utilize EDPs in the triple integral of Equation 2.1-1 in the form of the EDP variable. This variable is a function of the ground motion (intensity measure, IM) whose statistical uncertainties are determined in the hazard analysis. The EDPs are then used to evaluate the decision variables, DVs, to determine the dollar loss or potential for collapse of the building (ATC, 2004).

EDPs are categorized as either direct or processed (ATC, 2004). Direct EDPs values are calculated by either direct structural analysis or through computational modeling of a building and are used in Equation 2.1-1. Traditionally direct EDPs were categorized as individual member forces and interstory displacements. The component forces (also known as component demands) were determined by second-order linear analysis of a building using the building code load pattern. Some typical direct EDPs are the axial force of a beam-column, the shear force in a beam-column, the plastic rotation angle in a beam-column or in the girders, and the plastic rotation in the connections (ATC, 2004).

Processed EDPs characterize damage limit states and structural performance of a component or system (ATC, 2004). A damage index (DI) is considered to be a processed EDP and comes directly from a damage function that is based on experimental results. Typically DIs are used to calibrate the performance of a structural component or system in terms of a number between 0 (no damage) and 1 (ultimate state or complete collapse).

2.3 RCFT Column Damage Functions

Tort and Hajjar (2004) have collected the most current damage functions that relate specifically to RCFT structural systems. Various damage functions are the result of research that has been performed in many countries, including the United States, Japan, Europe, and Australia. Their collection of damage functions forms the benchmark for processed EDPs in regards to RCFT column systems and will be used in the final stages of the development of the performance-based design methodology of RCFT columns.

Tort and Hajjar (2004) divided their database of known RCFT damage functions into two categories - deformation-based (D^{RCFT}) and energy-based (E^{RCFT}). The deformation-based damage functions are only appropriate for monotonic loaded test specimens and appear to give good results only when they are used to describe members that have a ductile response and strain hardening in the load-deflection curves. The basic deformation-based function is shown in Equation 2.3-1, where d equals the deformation at the local level.

$$D^{RCFT} = \frac{d}{d_o} \tag{2.3-1}$$

Energy-based damage functions were found to be good for all types of RCFT columns, including those that have softening or hardening responses and even for RCFT column systems that are subjected to cyclic loading (Tort and Hajjar, 2004). Energy-based damage functions are based on Equation 2.3-2, where E equals the total energy absorbed before the damage level is reached.

$$E^{RCFT} = \frac{E}{E_{total}} \tag{2.3-2}$$

A damage index (DI) is the result of a damage function. Damage functions are calibrated so that the resulting DIs range between 0 (representing no damage) and 1 (representing attainment of peak load for D^{RCFT} or attainment of final failure mode for E^{RCFT}). DIs in RCFT columns are used to describe damage levels of the column. The controlling damage level in a column is dependent on the type of loading (monotonically loaded versus cyclically loaded) that the column is undergoing. Common damage levels for a RCFT column are compression yielding of the steel tube, tension yielding of the steel tube, local buckling of steel tube web and flange, tension cracking of concrete, and concrete crushing (Tort and Hajjar, 2004). Damage levels in the steel girders include yielding of the girder flanges, plastic hinging, and local buckling of the flanges (Tort and Hajjar, 2004).

In general damage indices have some shortcomings that need to be kept in mind when they are being used in the PBEE process. All damage functions are calibrated against experimental data at the component level since not enough research has been conducted that would allow for entire system level damage functions to be developed. Typically the damage functions that are chosen are associated with damage that is only a concern to the structural engineer and not to the building community as a whole (ATC, 2004).

2.4 Pushover Analysis

A nonlinear static pushover analysis allows for an estimate of the post-elastic response of a building to be made as well as an estimate of the capacity of the overall building system. Since the response of a structure is extremely complicated, especially once it has exceeded its elastic limit and is still being subjected to random loading, such as during a seismic event, the actual point in time when the structural system is expected to collapse (i.e., the system capacity) can only be estimated.

A force-based pushover analysis applies a load distribution to the building in accordance with the design building code provisions and incrementally increases all of the loads by the same amount until the overall building stiffness reaches zero (within a tolerance). The building is considered to have lost its ability to retain load at this point in the analysis. In this approach there are a number of parameters that are not taken into account such as load redistribution or the effects of demand as the building stiffness changes (FEMA, 2000).

The force-based pushover analysis only provides response values for the first mode of a building. Higher mode effects are not considered in this type of analysis since only one mode shape can be estimated at a time when the loads are applied as static loads. Since the first mode shape is the most dominant mode shape, and the easiest to model with static loads, it is typically the only mode shape that is approximated with this kind of analysis.

Another shortcoming with this type of analysis method is that it will overestimate the displacements and underestimate the capacity. Right now the only way to overcome this problem and to consider higher modes of a building is to perform an incremental dynamic analysis (IDA) (Wen et al., 2003).

Jin and El-Tawil (2004) have shown that the load pattern specified by the building code will provide results that compare well to the results of a dynamic analysis, even thought they are not as good as one will obtain from a dynamic analysis. Their findings show that the best correlations between a dynamic analysis and a static pushover analysis were for tall buildings (16-story), and that the correlations decreased as the building height decreased. Only the load pattern specified by the building code was used in the pushover analysis, and higher mode effects were not taken into account in this study.

Due to the inherent differences between any two buildings with different numbers of stories, the results of a pushover analysis for taller buildings will be different from that of shorter buildings. Jin and El-Tawil (2004) have shown that the geometric nonlinearity effects (P-Δ effects) are more critical in affecting the stability of taller buildings than for smaller buildings. They also found that in a pushover analysis taller buildings will have a sharper transition between the elastic and inelastic portions of their pushover curve as well as have shorter yield plateaus compared to that of shorter buildings. And finally, the overstrength factor (Ω) will decrease as the building height increases.

Chapter 3

Linear Static Analysis

The first objective of this study was to design the suite of buildings using structural engineering design practices that are commonly available in a typical engineering office. Therefore, each building was designed by using a two-dimensional linear analysis model with equivalent static loads and moment magnification to account for second order effects. The nominal design loads and load combinations are in accordance with the 2003 International Building Code, and the column and girder design strengths are according to the requirements of the 2005 American Institute of Steel Construction (AISC) design specification.

3.1 Design Parameters

The suite of buildings is composed of buildings that are capable of resisting some of the largest seismic design loads in the United States. All of the 9-story and the 18-story buildings that are designed in this study are characterized as office buildings. They are simple diaphragm buildings made of special moment resisting frames. The columns are made of RCFT members, and the girders are made of hot rolled U.S. wide flange steel sections. The gravity and environmental design loads as well as the interstory drift limits are in accordance with the design building code. The column and girder design strengths are in accordance with the most current AISC design specification for each respective member type.

The columns and girders of each lateral force resisting system are made of materials that are commonly available in the United States. All of the girders are hot rolled wide flange sections made of ASTM A992 material and have a design yield strength of 50 ksi. The RCFT column HSSs are made of ASTM A500 Grade B material. The design yield strength ranged from 46 ksi in some buildings to 80 ksi in other buildings. The concrete design compressive strength ranged from 4 ksi in some buildings up to 16 ksi in other buildings. Reference Table 1.2.1 for a full summary of the material design strengths used in each building design.

3.1.1 Site Location

All of the buildings in this study were designed to resist environmental loads (wind and seismic loads) for a central location within the city of Los Angeles, California. The chosen site was selected primarily due to its known seismic activity, and the resulting large seismic design loads that the buildings would have to be designed to resist. Figure 3.1.1.1 indicates an approximate location where all of the buildings are assumed to be located.

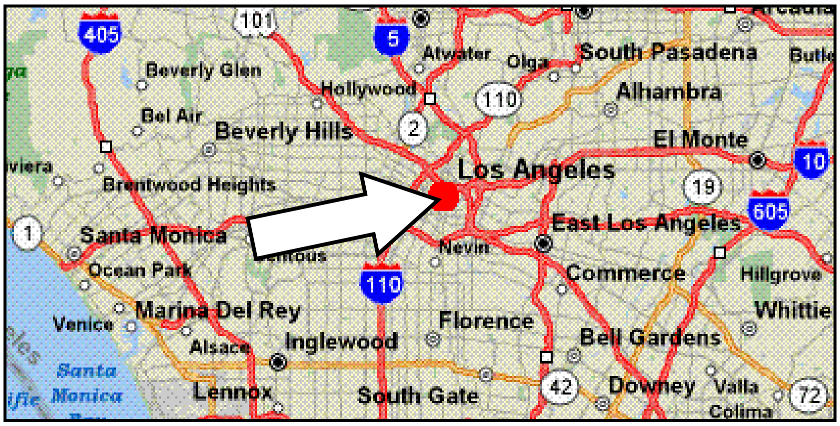


Figure 3.1.1.1: Site Location for the Buildings Designed in this Study (from LaFore and Hajjar, 2005)

3.1.2 Site Conditions

The site conditions are the design parameters that are required by the building code to be used in the structural analysis of each building. These design parameters are needed to determine the required design loads when used with the specified building code. For this study the 2003 International Building Code (IBC, 2003) and the ASCE 7 standard (ASCE, 2002) were used to determine the nominal and factored design loads. All of the buildings are categorized as being in "Occupancy Category II," so both wind and seismic loads have an importance factor of 1.00. The remaining specific site design parameters are as follows:

- Wind Loads
  - (3-sec gust) Basic Wind Speed, V = 85 mph
  - Exposure Category B

- Seismic Loads
  - Seismic Use Group I
  - Mapped Spectral Accelerations
    - $S_S = 1.5g$
    - $S_1 = 0.6g$
  - Site Class D
  - Seismic Design Category D

3.1.3 Building Layouts

The 9-story and the 18-story buildings have similar floor plans and frame layouts. All buildings have 13-foot center-to-center story heights, a 42 inch parapet around the roof perimeter, a 13-foot penthouse at the center of each building roof, RCFT columns, girders made of U.S. wide flange steel sections, 5 ½ inch thick concrete floors with cold formed steel decking, special moment frame detailing assuming rigid beam-to-column moment connections, and fully fixed column bases to the foundation at the first story level. No basement or lower floor levels were used at the base of the buildings. Each building has four lateral force resisting systems (LFRS) in each of the two principal directions. The columns that do not make up the LFRS are assumed to be leaner columns and only support gravity loads. The leaner columns were also not assumed to contribute to the stiffness of the buildings.

The primary differences between the 9-story and the 18-story buildings are the particular geometries of each building. Each 9-story building is a 5 bay x 5 bay building with equal 30-foot bay spacing. The beams between the girders are spaced 10-feet apart, center-to-center. The 18-story buildings are 6 bay x 6 bay buildings with equal 20-foot bay spacing. The beams are spaced 10-feet apart, center-to-center. The most heavily loaded frames were designed in each building. These moment-resisting frames were designated as moment frame "A2-F2" in the 9-story buildings and moment frame "A3-G3" in the 18-story buildings. Figures 3.1.3.1 through 3.1.3.4 illustrate each typical 9-story and 18-story building plan view and elevation view.

Each building designed in this study is considered to be a regular building since there are no plan irregularities or torsional irregularities. The composite floor decking is considered to be a diaphragm that is able to transfer all of the story wind and seismic shear loads to each of the LFRS at every story level. To simplify the analysis, the girders were designed to be bare steel sections during the seismic loading, based on the assumption that the concrete deck would be cracked. The girders were also treated as bare steel sections for the wind and gravity load combinations to keep the designs consistent. Each of the four LFRS in every building was made of the same structural sections at each story level. Therefore each moment frame that is a part of the LFRS had the same stiffness and rigidity at each story. This resulted in each frame being designed to resist a proportionate amount of wind and seismic story shear loads.

Only the 9-story and the 18-story buildings were designed in this portion of the study. All of the 3-story buildings that are referenced throughout the remaining sections of this study are from the work of LaFore and Hajjar (2005).

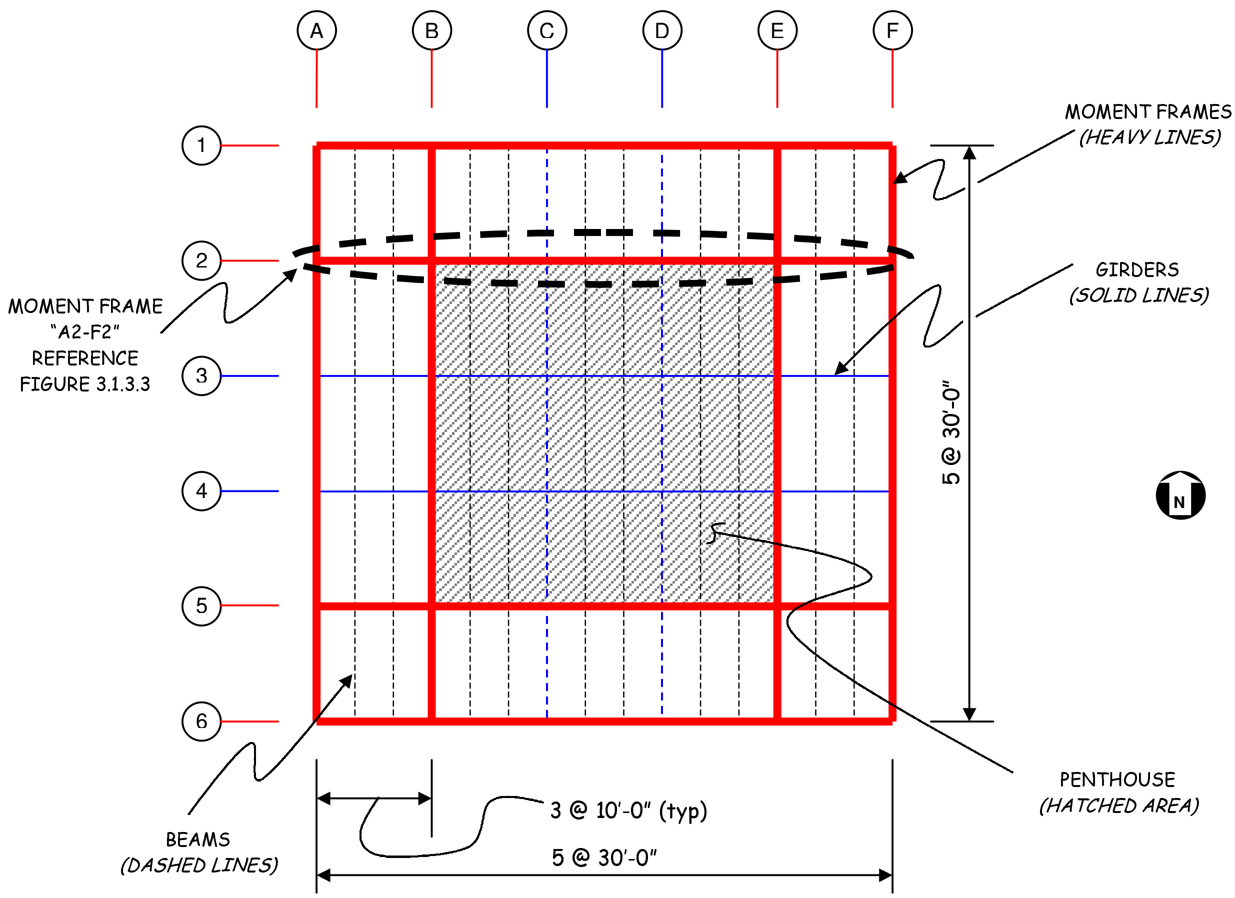


Figure 3.1.3.1: Typical 9-Story Building Plan View

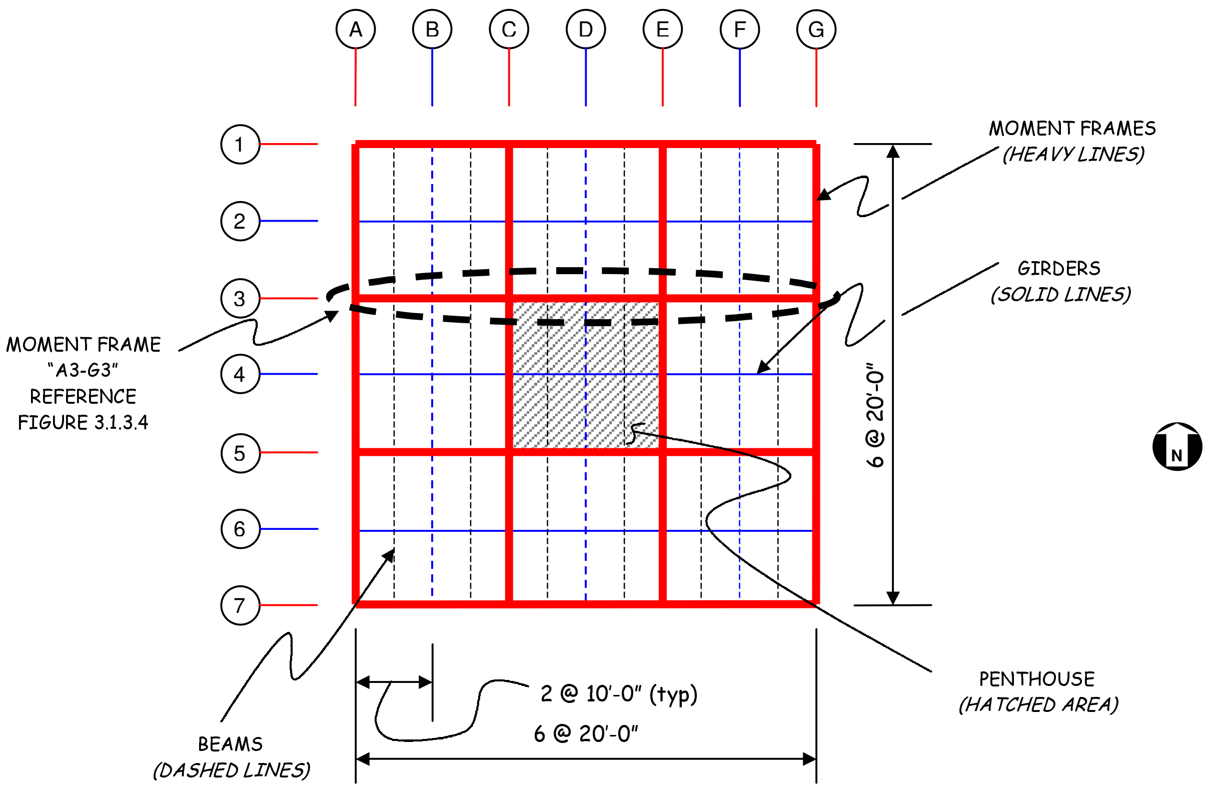


Figure 3.1.3.2: Typical 18-Story Building Plan View

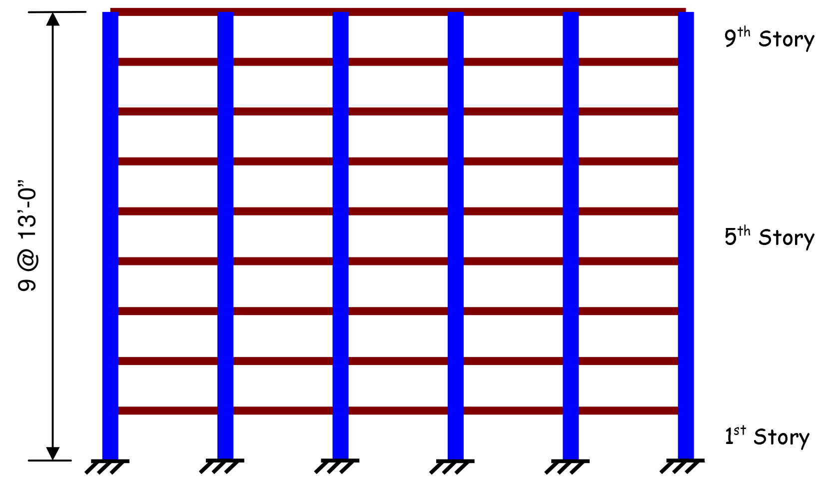


Figure 3.1.3.3: Typical 9-Story Building Elevation View (Moment Frame "A2-F2")

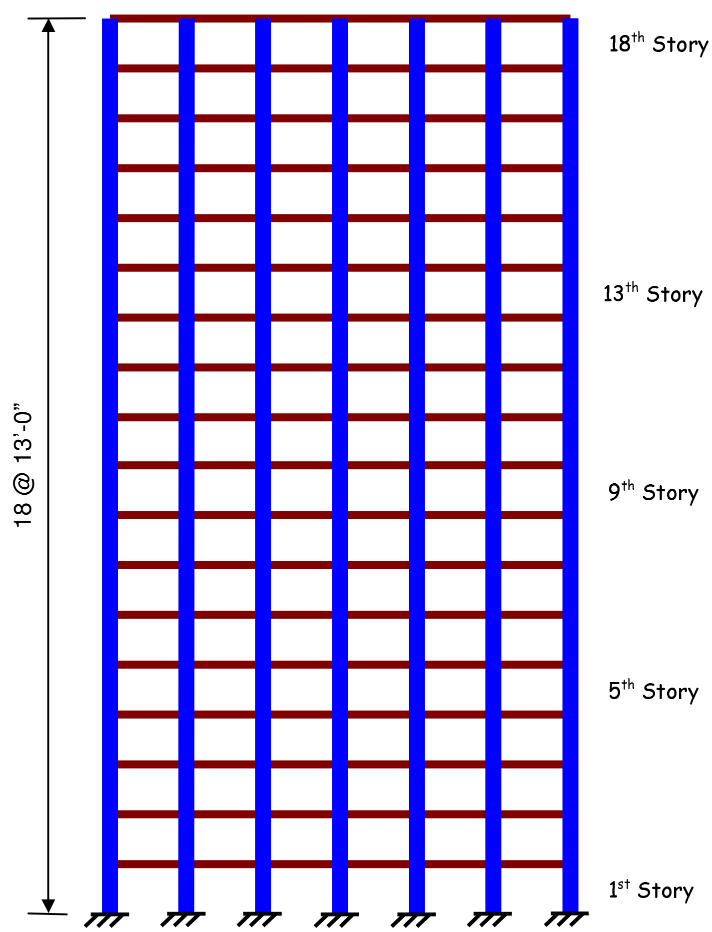


Figure 3.1.3.4: Typical 18-Story Building Elevation View (Moment Frame "A3-G3")

3.1.4 Nominal Loads

The nominal loads include unfactored dead loads, floor live loads for an office building, roof live loads, wind loads, and seismic loads. The dead and live loads were applied to the 2D models represented as unfactored point loads applied to the girders and to the nodes that connect the girders to the columns. The wind and seismic loads were modeled as point loads and applied directly at the node that connects two columns to a girder on one outside face of each frame.

3.1.4.1 Design Building Code

The buildings in this study were designed in accordance with the 2003 International Building Code (IBC, 2003). IBC 2003 specified both the live load and the environmental (wind and seismic) load requirements for each building. The environmental loads are called out in IBC 2003 to be designed in accordance with the ASCE 7-02 standard.

3.1.4.2 Dead Loads

The dead loads are the self-weight of the building. The same dead loads were used for each building type (i.e., the 9-story or the 18-story building) regardless of the particular column or girder sizes. Normal weight concrete with a density of 145 lb/ft^3 was used in all of the composite floor systems. Three categories of dead loads were used in the building designs – floor dead loads, roof dead loads, and penthouse dead loads.

- Floor dead loads include the following:
  - Columns, beams, girders, miscellaneous structural system — 20 lb/ft^2
  - Flooring — 1 lb/ft^2
  - Composite floor system (concrete + metal decking) — 50 lb/ft^2
  - Ceiling (from story below) + fireproofing — 2 lb/ft^2
  - HVAC + electrical (from story below) — 7 lb/ft^2
    - *Total floor dead loads:* — *80 lb/ft^2*
  - Exterior walls (applied to surface area of the wall) — 25 lb/ft^2

- Roof dead loads include the following:
  - Roofing — 7 lb/ft^2
  - Composite roof system (concrete + metal decking) — 50 lb/ft^2
  - (Roof) beams, girders, miscellaneous structural system — 20 lb/ft^2
  - Ceiling (from story below) + fireproofing — 2 lb/ft^2
  - HVAC + electrical (from story below) — 7 lb/ft^2
    - *Total roof dead loads:* — *86 lb/ft^2*
  - Parapet (applied to surface area of the wall) — 25 lb/ft^2

- Penthouse dead loads include the following:
    - Composite roof system (concrete + metal decking) — 50 lb/ft²
    - Ceiling + fireproofing — 2 lb/ft²
    - Columns, beams, girders, miscellaneous structural system — 20 lb/ft²
    - Mechanical equipment — 40 lb/ft²
    - Flooring — 1 lb/ft²
        - *Total penthouse dead loads:* — *113 lb/ft²*
    - Exterior walls (applied to surface area of the wall) — 25 lb/ft²

3.1.4.3 Live Loads

Live loads in the 9-story and 18-story buildings were determined by using the office building type live load requirements per IBC 2003. A moveable partition live load was also incorporated in the floor loading as well as a roof live load. The sections of the IBC 2003, with which the specific live loads are in accordance, are listed in Appendix A and Appendix E for the 9-story and 18-story buildings, respectively

- Building (floors)
    - Office building occupancy — 50 lb/ft²
    - Moveable partitions — 20 lb/ft²
        - *Total floor live load* — *70 lb/ft²*

- Building (roof)
    - Roof live load — 20 lb/ft²
        - *Total roof live load* — *20 lb/ft²*

- Penthouse
    - General live load — 20 lb/ft²
    - Roof live load — 0 lb/ft²
        - *Total penthouse live load* — *20 lb/ft²*

3.1.4.4 Wind Loads

Even though seismic loads controlled the final design of each building, wind loads were included in the design process. The wind loads were determined using the requirements of IBC 2003. An exposure category and a design (3-second gust) wind speed for the site location in Los Angeles were used to calculate the design wind loads. The gust effect factor was calculated using the maximum allowed (approximate) building period from the seismic design section of ASCE 7-02 standard. Therefore, all of the 9-story and 18-story buildings resulted in having design frequencies that categorized them as being flexible structures. The specific sections of the building code that were used in the wind design are listed in Appendix A and Appendix E for the 9-story and 18-story buildings, respectively.

The buildings were designed as "simple diaphragm" type buildings since the roof and floor diaphragms are assumed to be capable of transferring the wind loads to each LFRS. Therefore, there is the assumption that half of the total wind load that is going to a specific story is to be resisted by the floor above while the other half is resisted by the floor below.

The following wind design parameters were used in all of the building designs:

- (3-second gust) Basic Wind Speed, V = 85 mph
- Occupancy Category II
- Importance Factor, I = 1.00
- Exposure Category B
- Wind Directionality Factor, $K_d = 0.85$
- Topographic Factor, $K_{zt} = 1.0$
- Enclosure Classification = "Enclosed"
- Building Type = "Simple Diaphragm"

The ASCE 7-02 standard requires four wind cases to be analyzed - wind along only one principal axis of the building at a time, wind along both principal axis at the same time, torsional loads plus wind along one principal axis at a time, and wind along both principal axis plus torsional loads. A rigidity analysis was performed to account for torsional wind loads by assuming that each LFRS had the same stiffness (rigidity) at every story level. To account for the increase in design wind shear loads per story due to the torsional loads on some frames (while the frames on the opposite side of the building will have a decrease in their overall wind loads due to the torsional loads), a relationship was derived which increases the design wind shear per story by a scalar value. This relationship, as shown in Equation 3.1.4.4-1, is based on the assumption that all of the LFRSs of a building have the same stiffness at each story level.

$$V_{dsgn} = V_i\left(\frac{1}{n} + 0.002e\right) \qquad (3.1.4.4\text{-}1)$$

Where: V_{dsgn} = design wind shear at each story level with torsion included
V_i = wind shear at each story level without torsion included
n = number of LFRSs in the same direction as the frame being analyzed
e = eccentricity of the building per ASCE 7-02

3.1.4.5 Lateral Seismic Loads

The lateral seismic loads that were used in the elastic analysis of each building are in accordance with IBC 2003. The seismic design parameters are based on a location in central Los Angeles as shown in Figure 3.1.1.1. The design values of the spectral response accelerations, S_S and S_1, were taken from the NEHRP Maximum Considered Earthquake Map #5 and Map #6. The specific steps and equations that were used in the seismic design are listed in Appendix A and Appendix E for the 9-story and 18-story buildings, respectively.

The following seismic design parameters were used for all of the building designs:

- Mapped Spectral Accelerations
  - $S_S = 1.5g$
  - $S_1 = 0.6$ g
- Site Class D
- Seismic Design Category D
- Occupancy Category II
- Seismic Use Group I
- Occupancy Importance Factor, I = 1.00
- Site Coefficients
  - $F_a = 1.0$
  - $F_v = 1.5$
- Seismic-Force Resisting System = "Special Composite Moment Frames"
  - Response Modification Coefficient, R = 8
  - Deflection Amplification Factor, $C_d = 5.5$
- Equivalent Lateral Force Analysis Procedure

The fundamental period, T_a, was calculated by using Equation 3.1.4.5-1, which was taken directly from ASCE 7-02. This simplified the design process by not requiring a dynamic analysis to be performed on each building. Even though the frames are made of RCFT columns, it was assumed that the frames were moment resisting frame systems of steel per ASCE 7-02. Therefore, design values of 0.028 and 0.8 were used for C_t and x, respectively.

$$T_a = C_t h_r^x \qquad (3.1.4.5\text{-}1)$$

Where: h_r = building roof elevation

The design base shear is a percentage of the seismic weight, D′, of the building. Since the 9-story and the 18-story buildings are designed as office buildings, the seismic weight of each building includes the dead load plus the moveable partition live load.

The seismic loads were distributed along the height of each building in accordance with the method specified by ASCE 7-02. The load pattern that was used in the analysis of each building was in the form of an inverted triangle, with the largest load applied to the roof level, and subsequent floor loads decreased in value as they approached the second floor. Equations 3.1.4.5-2 and 3.1.4.5-3 were used in this step of the analysis and are taken directly from ASCE 7-02.

$$F_x = C_{vx} V \qquad (3.1.4.5\text{-}2)$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \qquad (3.1.4.5\text{-}3)$$

In accordance with IBC 2003, the redundancy coefficient, ρ, was calculated to be 1.00 for all of the 9-story and the 18-story buildings. The seismic loads were multiplied by a simplifying factor of 1.025 to account for the accidental torsional effects. To account for orthogonal loading, the weak axis (y-axis) of the columns that are shared between two perpendicular moment frames were designed to resist 30% of their strong axis (x-axis) design loads along their weak axis (y-axis), and 100% of the design loads along their strong axis (x-axis). The columns that are not shared between two adjacent moment frames were only designed to resist 100% of the design loads along their strong axis plus an increase in axial load due to the end shear of the pinned connection along their weak axis.

3.1.4.6 Vertical Seismic Loads

The vertical seismic loads that were used in the elastic analysis of each building are in accordance with IBC 2003. The same seismic design parameters that were used in the lateral seismic load calculations were used to determine the vertical seismic loads. The magnitude of the unfactored vertical seismic loads was equal to $0.2S_{DS}D'$, where D' is the same seismic weight as was used to calculate the lateral seismic loads.

3.2 Design Loads

The nominal loads were applied to each 2-D moment frame model using only one nominal basic load case at a time. The resulting deformations and forces for each member was then factored according to their respective load combination load factor and combined with the other resulting forces based on the principle of superposition. The design level (factored) loads were then compared to the allowable member strengths to determine the appropriate column and girder size necessary to resist the loads from the load combination. The specific design loads and locations of each load, and how they are applied to each 2-D moment frame, are shown in Appendix A and Appendix E for the 9-story and 18-story buildings, respectively.

3.2.1 Load Combinations

The load combinations that were used in the elastic analysis were taken directly from Section 1605 of the 2003 IBC. Each building was first analyzed using only one (unfactored) nominal basic load case at a time. The basic load cases consisted of the dead loads, live loads, roof live loads, seismic weight, wind loads, seismic lateral loads, and seismic vertical loads. By the principle of superposition the resulting member forces and displacements from each of the six basic load cases were then combined and factored according to the below listed load combinations. The result was design level (factored) forces and deformations for each column and girder in a moment-resisting frame. These factored forces were then used to design the final member sizes.

The six load combinations that were used in this study are as follows:

- 1.4D
- $1.2D + 1.6L + 0.5L_R$
- $1.2D + 1.6L_R + f_1L$
- $1.2D + 1.6L_R + 0.8W$
- $1.2D + 1.6W + f_1L + 0.5\ L_R$
- $1.2D + 1.0E + f_1L$

Where: $f_1 = 0.5$
$E = \rho Q_E + 0.2S_{DS}D'$
$\rho = 1.00$
D' = seismic weight

3.2.2 Stability Coefficient, θ

The stability coefficient, θ, was used to estimate the stability of each building. More specifically, the stability coefficient can approximate if geometric nonlinearities (P-Δ effects) should be anticipated or if they can be ignored. When θ is between 0 and 0.10, P-Δ effects can be ignored. If θ is between 0.10 and the maximum allowed, then it can be anticipated that P-Δ effects will start to affect the response of the building when the building is beyond its elastic limit. If θ is more than the maximum allowed limit, then the building is potentially unstable, and a redesign of the building is necessary. When calculating the maximum allowed stability coefficient, the ratio known as β, which is the ratio of the story shear demand, ΣV_u, to the story shear capacity, $\Sigma\phi_v V_n$, was calculated for each story using the shear strength equations for a HSS section using Chapter G of the 2005 AISC specification.

The stability coefficient for each story of a building is a function of the story stiffness, K_i, story height, H_i, and the total factored axial load on the story, ΣP_u, as shown in Equation 3.2.2-1. Since K_i and H_i are constant for each story (assuming the column and girder sections do not change beyond this point in a design) the only variable in determining θ is ΣP_u. Each load combination will have different values of factored gravity loads, making ΣP_u a function of the gravity load factors and the gravity loads. Therefore, the stability coefficient will vary for each story of a building as well as for each load combination. The maximum stability coefficient for a story will occur during the load combination that has the largest ΣP_u.

$$\theta_i = \frac{\sum P_u}{\sum V_i H_i \big/ \Delta_i} = \frac{\sum P_u}{K_i H_i} \qquad (3.2.2\text{-}1)$$

Where: ΣP_u = total factored axial load on all of the columns in a story
V_i = total horizontal force to story i
H_i = story (column) height
Δ_i = elastic interstory drift due to V_i
K_i = story stiffness

In addition to estimating the stability of a building, θ was also used to determine the column effective length factors, K_x and K_y, and the B_2 moment amplification factors, B_{2x} and B_{2y} (where the subscripts x and y denote the principle axis of the column). The specific design values for K_x, K_y, B_{2x}, and B_{2y} for each column are shown in the Appendices for each particular building design. The relationship between θ and the column effective length factors, K_x and K_y, and the "B_2 factors" B_{2x} and B_{2y} is shown in Equations 3.3.2.1-1 and 3.2.3-4, respectively.

3.2.3 Moment Amplification Factors, B_1 and B_2

The AISC approximate second-order analysis procedure requires the calculation of the moment amplification factors B_1 and B_2. Each of these parameters was calculated in accordance with the AISC specification (AISC, 2001) with some modifications for RCFT composite columns. B_1 was calculated by using Equation 3.2.3-1 and assuming an effective length factor, K, of 1.0 for each column. The factored axial load to each individual column, P_u, is the total factored load to that column. A separate value of B_1 was calculated for every load combination, ignoring any axial loads due to wind or seismic lateral loads should they be part of the load combination. P_{e1} was modified from the AISC specification so that it can be used with RCFT composite columns as shown in Equation 3.2.3-3.

$$B_1 = \frac{C_m}{\left(1 - \frac{P_u}{P_{e1}}\right)} \geq 1.0 \qquad (3.2.3\text{-}1)$$

$$C_m = 0.6 - 0.4\left(\frac{M_1}{M_2}\right) \qquad (3.2.3\text{-}2)$$

Where: M_1 = smaller end moment from a first-order analysis
M_2 = larger end moment from a first-order analysis

$$P_{e1} = \frac{\pi^2 (EI_{eff})}{(KH)^2} \qquad (3.2.3\text{-}3)$$

Where: K = column effective length factor
H = column height (length)

When the B_2 factor was required, every column in a story of a building used the same value of B_2. However, each load combination will result in a different B_2 factor for each story since ΣP_u will vary depending on the load factors and applied gravity loads. B_2 was calculated for each story by using the stability coefficient, θ, as shown in Equation 3.2.3-4.

$$B_2 = \frac{1}{1-\theta} \tag{3.2.3-4}$$

3.3 Member Design Strengths

The computer program VisualAnalysis was used to analyze each 2D building model using linear analysis methods. The resulting member forces were then compared to the allowed RCFT column strengths using the provisions of the 2005 American Institute of Steel Construction (AISC) Specification (although with the equations in a different format in this work) while the allowed girder strengths were in accordance with the 1999 AISC Specification. The strong-column-weak-beam (SC/WB) provisions of the 2002 AISC Seismic Provisions and the interstory drift limitations of IBC 2003 were also followed in determining the final column and girder sizes.

3.3.1 Girders

The girders were sized in accordance with the AISC Seismic Provisions strong-column-weak-beam (SC/WB) requirements as shown in Equation 3.3.1-1. The girders were designed as bare steel beams rather than compositely with the concrete floor slab since the shear studs are not allowed near the girders' plastic hinge (AISC, 2002) and that during cyclic loading the concrete slab will be in tension.

$$\frac{\sum M_{pc}}{\sum M_{pg}} > 1.0 \tag{3.3.1-1}$$

Where: $\Sigma M_{pc} = \Sigma Z_c(F_{yc} - P_{uc} / (A_c + A_s))$
$\Sigma M_{pg} = \Sigma(1.1 R_y F_{yg} Z_g + M_v)$
A_c = area of column concrete portion
A_s = area of column steel HSS portion
F_{yc} = specified minimum yield strength of column steel HSS
F_{yg} = specified minimum yield strength of the girder
P_{uc} = factored column axial compressive load
Z_g= plastic section modulus of the girder
Z_c = plastic section modulus of the steel HSS portion of the column
M_v = moment due to shear amplification between the plastic hinge in the girder and the centerline of the column. To simplify the analysis a value of zero was used in each building design.
R_y = 1.1 (girders assumed to be ASTM A992 material)

After initially sizing the girders to adhere to the SC/WB provisions, the AISC LRFD design member strengths were compared to the member design forces. All of the girders were designed with an unbraced length of ten feet since they support a beam every ten feet.

3.3.2 RCFT Columns

The RCFT columns were designed by calculating the allowable compressive strength and the allowable flexural strength for each column in accordance with Chapter I of the 2005 AISC Specification (although presented in a different format in this work). The interaction equations from the 2005 AISC Specification Chapter I Commentary were then used to select the final column sizes. Table 3.3.2.1 and Table 3.3.2.2 list the order of the AISC equations that were used to calculate the allowable compressive and flexural strengths, respectively. Table 3.3.2.3 lists the parameters that were used to derive the column interaction values.

Step No.	Parameter	AISC Section No.	Equation No.
1	P_o	I.2.2b	I2-15
2	$C_2 = 0.85$	I.2.2b	*RCFT Section*
3	EI_{eff}	I.2.2b	I2-16
4	C_3	I.2.2b	I2-17
5	P_e	I.2.1b	I2-4
6	α	I.2.1b	I2-2
7	Λ	I.2.1b	I2-8 or I2-9
8	$P_n = \Lambda P_o$	I.2.1b	I2-7
9	$\phi_c = 0.75$	I.4	*LRFD*
10	$\phi_c P_n$	I.4	---

Table 3.3.2.1: RCFT Compressive Strength Design Steps Using the 2005 AISC Specification

Step No.	Parameter	AISC Section No.	Equation No.
1	$M_n = ZF_{yc}$	I.3(b)	*Plastic stress distribution*
2	$\phi_b = 0.90$	I.4	*LRFD*

Table 3.3.2.2: RCFT Flexural Strength Design Steps Using the 2005 AISC Specification

Step No.	Parameter	AISC Section No.	Equation No.
1	$P_r = P_u$	Ch. I Commentary	*LRFD*
2	C	Table C-I1.1	---
3	$C_\lambda = \Lambda C$	Ch. I Commentary	*LRFD*
4	$C_d = \phi_c C_\lambda$	Ch. I Commentary	*LRFD*
5	$A_d = \phi_c P_n$	Ch. I Commentary	*LRFD*
6	$M_{rx} = M_{ux}$	Ch. I Commentary	*LRFD*
7	$M_{ry} = M_{uy}$	Ch. I Commentary	*LRFD*
8	M_{cx} & M_{cy}	Ch. I Commentary	*LRFD*
9	Interaction Value	Ch. I Commentary	C-I4-1a or C-I4-1b

Table 3.3.2.3: RCFT Interaction Value Design Steps Using the 2005 AISC Specification

3.3.2.1 Effective Length Factors, K_x and K_y

ASCE (ASCE, 1997) has shown that the effective length factors for a column may be calculated as functions of constant column parameters (i.e., EI_{eff} and H) as well as varying parameters like P_u and the stability coefficient, θ. This relationship is illustrated in Equation 3.3.2.1-1. Since the two varying parameters P_u and θ are dependant on the load factors of the particular load combination, the effective length factors for a column are a function of the particular load combination. Therefore, just as a different value of θ is calculated in a story for each load combination, a different set of effective length factors, K_x and K_y, is calculated for each column for every load combination. The value of K_y is calculated by using θ_y instead of θ_x in Equation 3.3.2.1-1.

$$K_x = \sqrt{\left(\frac{\pi^2 (EI_{eff})}{H^2}\right)\left(\frac{\theta_x}{0.85P_u}\right)} \tag{3.3.2.1-1}$$

Where: P_u = factored column axial compressive load
H = column height (length)

3.3.2.2 Cross Sectional Properties

Both the girder and the RCFT column cross sectional properties were calculated in order for the elastic analysis to be performed properly. The girder cross sectional properties were taken directly from AISC, since all of the girders that were used in this study are hot rolled U.S. wide flange sections. The RCFT columns required calculations to be performed to determine the cross sectional properties since most of the columns are larger than the HSS sections that are listed in the 2001 AISC.

3.3.2.2.1 Steel HSS

The HSS members that are listed in the AISC steel manuals are sections that have a perimeter less than or equal to 64 inches. This is due to the fact that the ASTM A500 specification, with which all of these listed HSS sections are made in accordance, specifies that the largest perimeter allowed for this particular ASTM specification is 64 inches. Therefore, the HSS sections that have perimeters larger than 64 inches required the cross sectional properties to be calculated for this study. These cross sectional properties include the area, moment of inertia, radius of gyration, and plastic modulus. The Steel Tube Institute (STI, 1996) provided equations for this study that were used to calculate these four cross sectional properties. These equations are listed in Appendix K.

When using the STI equations, it is important to understand the manufacturing process of the particular HSS that is being analyzed so that accurate design values of the outside corner radii and wall thickness are used to calculate the cross sectional properties. The electric-resistance welding (ERW) process is used to manufacture HSS with perimeters smaller than or equal to 64 inches. HSS with perimeters greater than 64 inches are manufactured by using the submerged arc welding (SAW) process. Because of these different welding and manufacturing practices, the outside corner radii could vary from one HSS to another with a different perimeter. The outside corner radii equals 2.0 times the design wall thickness for ERW HSS (i.e., for perimeters ≤ 64 inches). SAW HSS (perimeters > 64 inches) have an outside corner radii equal to 3.6 times the design wall thickness when the nominal wall thickness is $^5/_8$ inch, and 3.0 times the design wall thickness when the nominal wall thickness is either ½ inch or $^3/_8$ inch thick. The design wall thickness for ERW HSS equals 0.93 times the nominal wall thickness, and for SAW HSS the design wall thickness equals the nominal wall thickness.

3.3.2.2.2 Concrete Core

Equations for calculating the area, moment of inertia, and plastic modulus of the concrete core were derived for this study based on the outside corner radii and design wall thickness rules of the steel HSS. Figure 3.3.2.2.2.1 and Equations 3.3.2.2.2-1 through 3.3.2.2.2-3 illustrate the final forms of these equations.

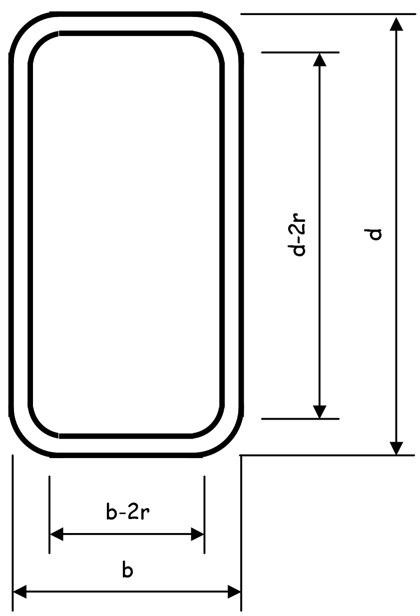


Figure 3.3.2.2.2.1: Typical HSS Cross Section

$$A_c = (d-2r)(b-2r) + 2(r-t)(b+d-4r) + \pi(r-t)^2 \qquad (3.3.2.2.2\text{-}1)$$

$$I_c = \frac{1}{12}(b-2t)(d-2r)^3 + 2\left[\frac{1}{12}(b-2r)(r-t)^3 + (b-2r)(r-t)\left[\frac{d-r-t}{2}\right]^2\right] + \ldots$$
$$\ldots + 2\left\{(r-t)^4\left(\frac{\pi}{8} - \frac{8}{9\pi}\right) + \left[\frac{\pi(r-t)^2}{2}\right]\left[\frac{d}{2} - t - (r-t)\left(1 - \frac{4}{3\pi}\right)\right]^2\right\} \qquad (3.3.2.2.2\text{-}2)$$

$$Z_c = \frac{(b-2r)(d-2t)^2}{4} + 2(r-t)\left(\frac{d}{2} - r\right)^2 + \pi(r-t)^2\left(\frac{4(r-t)}{3\pi} + \frac{d}{2} - r\right) \qquad (3.3.2.2.2\text{-}3)$$

Where: A_c = area of the concrete core
b = outside width
d = outside depth
I_c = moment of inertia of concrete core
r = outside corner radii
t = design wall thickness
Z_c = plastic modulus of concrete core

3.3.2.3 Elastic Design Values of E, A, and I

For an elastic analysis to be performed on a 2D moment frame, user defined modulus of elasticity, area, and moment of inertia values are needed for all of the columns and girders. These values are readily available for the hot rolled wide flange girders, but for RCFT column sections individual values of E, A, and I are not defined. Consequently, relative values of E, A, and I need to be calculated. Equation 3.3.2.3-1 and Equation 3.3.2.3-2 show how a modified area, A_e, and modified moment of inertia, I_e, are determined by assuming any value for the modulus of elasticity, E'. This method allows for any constant value of E' to be used to calculate a relative value of A_e and I_e that can then be used in the elastic analysis. The values of EI_{eff} and C_3, as defined below, are from the 2005 AISC Specification.

$$A_e = \frac{EA_{eff}}{E'} \qquad (3.3.2.3\text{-}1)$$

$$I_e = \frac{EI_{eff}}{E'} \qquad (3.3.2.3\text{-}2)$$

Where: E' = any constant value
$EA_{eff} = E_sA_s + E_cA_c$
$EI_{eff} = E_sI_s + C_3E_cI_c$
$C_3 = 0.6 + 2A_s/(A_c+A_s) \leq 0.9$

3.3.3 Interstory Drift Limits

One of the final steps in the second-order linear design process was to check the interstory drift of each story in a building to make sure that they remain within the allowable limits of the IBC 2003. For this study the maximum expected *inelastic* interstory drift, δ_x, could not exceed $0.02H_i$. The corresponding maximum *elastic* interstory drift allowed in the elastic analysis was determined by using Equation 3.3.3-1.

$$\Delta_e = \frac{\delta_x I}{C_d} \qquad (3.3.3\text{-}1)$$

Where: Δ_e = elastic interstory drift limit
δ_x = inelastic interstory drift limit
I = occupancy importance factor
C_d = deflection amplification factor

Chapter 4

Nonlinear Static Pushover Analysis

A force-based nonlinear static pushover analysis allows for an estimate of the post-elastic response of each building to be made, as well as provide an approximate value of the overall structural system capacity. These two quantities were determined for all of the buildings in this study using both the actual and the normalized pushover analysis curves of the roof drift versus base shear. This chapter summarizes the pushover analysis process and which analysis results were used in the assessment of the final suite of buildings.

4.1 Analysis Procedure

A force-based pushover analysis is a static analysis method whereby the structural system, in this study 2D moment frames (3-stories, 9-stories, and 18-stories), is analyzed using constant gravity loads and uniformly increasing lateral loads. The analysis of each building ends when the stiffness of the system reaches zero, or more specifically, when the stiffness matrix becomes indefinite (i.e., the eigenvalues become negative).

Two different methods were used to compare the load-deformation relationship for each building. The first method makes a direct comparison between each building using the applied base shear and corresponding roof drift. The second method uses normalized values and compares the applied base shear divided by the design base shear and the roof drift divided by the roof height. The applied force and displacement values were used to determine an elastic stiffness, K_e, the system capacity, and the relative energy that is absorbed by the system. The normalized force-displacement values were used to compare the system response of each building with respect to each other and to show how the overstrength factor, Ω, varies between buildings. By normalizing the pushover analysis values for each building, a clear comparison can be made between two buildings, regardless of the number of stories in each building.

Factored gravity and lateral (seismic) loads were used in each pushover analysis. The gravity loads were applied to each 2D moment frame at the same locations and with the same magnitudes as was done in the earthquake load combination of the original elastic analysis. The lateral loads were applied to the model using the same vertical distribution of forces as was used in the original elastic analysis model except they initially had a magnitude of zero and were then uniformly increased until the analysis ended.

The vertical distribution of lateral forces is based on the building code period dependent load pattern. As with the elastic analysis, the largest load is applied at the roof, while the loads decrease in magnitude as the floor levels decrease in elevation.

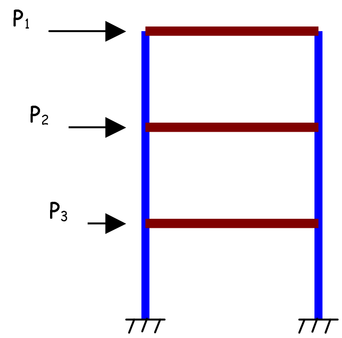


Figure 4.1.1: Example of How Lateral Loads are Applied With $P_1 > P_2 > P_3$

CFTMacro (Gourley and Hajjar, 1994) was the analysis program used to perform the nonlinear static pushover analysis of each 2D moment frame. CFTMacro was developed to analyze RCFT frames. A stress-resultant bounding-surface formulation is used whereby member inelasticity is tracked at the ends of each beam finite element. Default parameters for the constitutive models were used as reported in Hajjar et al. (1997).

4.2 Analysis Results

The main parameters that are measured in a pushover analysis are the base shear, V, and the corresponding roof drift, Δ. From these two parameters, a number of characteristics can be determined for each building, as shown in Figure 4.2.1. These characteristics include the elastic stiffness, K_e, the capacity of the building, the relative energy absorbed by the building, and the overstrength factor, Ω.

Some key points that are measured during the analysis are the design base shear, V_{design}, the base shear at the yield point of the structure, V_{yield}, and the ultimate base shear, V_{ult}. The design base shear corresponds to the base shear that was used in the elastic analysis. The yielding base shear is the base shear at the point in the analysis when the stiffness begins to decrease. The ultimate base shear is the largest base shear value in the analysis.

An approximate elastic stiffness, K_e, of a building is measured by taking the base shear force along the elastic portion of the pushover curve and dividing it by the corresponding roof drift. Since the elastic portion of a pushover analysis is not perfectly elastic, only the approximate elastic stiffness of the building can be calculated.

The capacity curve of a building is a relative measure of the amount of force that the building can resist before collapse. The capacity curve is calculated by dividing the relative energy that the building has absorbed by the corresponding displacement (Guo and Gilsanz, 2003).

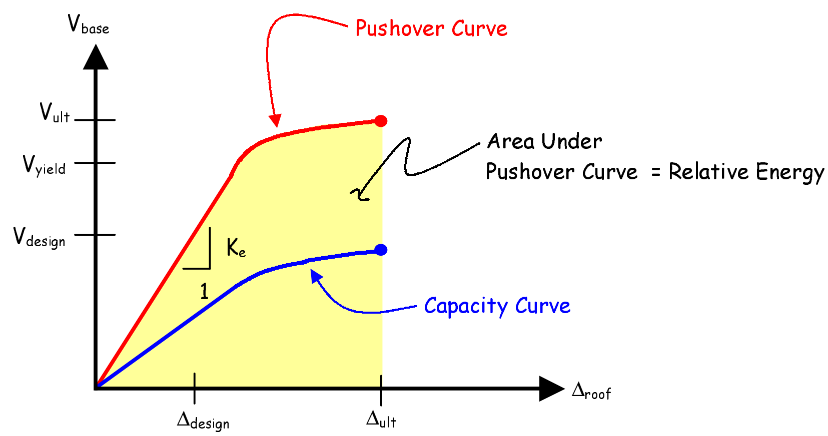


Figure 4.2.1: Building Response Parameters that are Derived From a Pushover Analysis

Relative energy is used to describe the amount of energy absorbed by each building, and corresponds to the area under the pushover curve between two points. Since the force-based pushover analysis is very sensitive to where the analysis terminates, it is difficult to obtain accurate values of the total energy that is absorbed by a building. Therefore, it becomes difficult to make accurate comparisons between two buildings. To account for these inaccuracies relative termination points were used to determine when to stop measuring the energy absorbed for a building.

The energy calculations that are presented in this study have been categorized as relative energy values and not total energy values. This is because the area under a roof drift versus base shear pushover curve is not the true energy absorbed by the building. For example, a more accurate way to measure the total energy absorbed by a building is determined by calculating the energy absorbed at each story level and then summing the energy values from all stories.

Relative energy should also not be confused with the energy of a building when it is subjected to cyclic loading. A pushover analysis only estimates energy values due to a monotonic loading. A building will respond differently when it is subjected to monotonic loading compared to cyclic loading. Therefore, the energy values calculated using the pushover analyses in this study are only used as relative measures so that a relative comparison can be made between any two buildings in this study.

Chapter 5

Design and Analysis Results

A two-dimensional (2D) linear static analysis that included approximate second-order effects was used to design the suite of thirteen buildings. After each building was designed elastically, a nonlinear static pushover analysis was performed so that an estimate of the post-elastic response of each building could be determined. This chapter summarizes all of the design and analysis results from the work that was performed in this study.

5.1 Linear Static Analysis Results

The column and girder sections of each moment frame were sized according to current material design specifications and building code requirements using a 2D linear static analysis. The member strengths of each column and girder are in accordance with the provisions of the American Institute of Steel Construction (AISC) 2005 Specification, while the gravity, wind, and seismic loads follow the requirements of the 2003 International Building Code (IBC, 2003). Since this study only focused on the elastic design of the 9-story and 18-story buildings, the 3-story building design results that are presented in this section have been reproduced from the work of LaFore and Hajjar (2005).

Two kinds of environment loads were used to design each building – wind loads and seismic loads. The nominal seismic shear at the base of each building and the major design parameters for the seismic loading are summarized in Table 5.1.1. The design value of the seismic response coefficient varied between 0.125g for the 3-story buildings, 0.06g for the 9-story buildings, and 0.044g for the 18-story buildings. By comparing the design fundamental period, the seismic weight, and the final seismic base shear of each building it becomes evident that the values of C_s, and ultimately the base shear, follow a typical pattern of an elastic design spectrum whereby the design period is inversely proportional to C_s. This is best illustrated by taking building design 3E and 18A, which have a design base shear of 282 kips and 316 kips, and a seismic weight of 9,038 kips and 28,722 kips, respectively. Even though building 18A is almost 3.2 times heavier than building 3E, it only has to resist 12% more seismic force than building 3E.

By comparing the wind shear with the seismic shear, seismic loads initially appeared to control all of the column and girder designs in each building. For most of the column and girder sections, the seismic load combinations did control their design. In some cases for the

18-story buildings the wind load combinations controlled the designs of some columns on the windward side of the moment frames. However, the final column sections that are presented in this study are sized to resist the seismic load combinations since the columns on the leeward side of each moment frame will have to resist more axial load and bending moment due to seismic loading than the windward side columns have to resist due to wind loading. Therefore, since the frames are symmetrical in geometry and in column section size, the seismic loading controlled the design of the final column and girder sections.

Linear Elastic Design Values	3-Story Buildings							9-Story Buildings			18-Story Buildings		
	3A	3B	3C	3D	3E	3F	3G	9A	9B	9C	18A	18B	18C
Design Fundamental Period for Seismic Loading T_a *[seconds]*	0.525	0.525	0.525	0.525	0.525	0.525	0.525	1.264	1.264	1.264	2.201	2.201	2.201
Seismic Response Coefficient C_s *[g]*	0.125	0.125	0.125	0.125	0.125	0.125	0.125	0.060	0.060	0.060	0.044	0.044	0.044
Nominal (Building) Seismic Weight W_s *[kips]*	7,202	7,202	7,202	9,038	9,038	7,860	7,860	22,677	22,677	22,677	28,722	28,722	28,722
2D Moment Frame Seismic (Design) Base Shear V_{design} *[kips]*	225	225	225	282	282	246	246	340	340	340	316	316	316
2D Moment Frame Seismic Base Shear Plus 2.5% (Approx.) Accidental Torsion Shear *[kips]*	231	231	231	289	289	252	252	349	349	349	324	324	324
2D Moment Frame Wind Base Shear V_{wind} *[kips]*	16	16	16	16	16	16	16	79	79	79	134	134	134

Table 5.1.1: Linear Static Analysis Design Values for Each Building

The final column and girder sections for each building are listed in Tables 5.1.2 through 5.1.7. Even though three different building heights were designed in this study, the final column and subsequently the final girder sections are all within a relatively small range of member sizes. The column HSSs ranged in size from 16 inches to 22 inches, while the girders were between 12 inches and 30 inches in nominal depth. Only square tube cross sections were chosen for the columns, since most of the columns in each moment frame have been designed to resist loading along both their local x-axis and y-axis. The columns that only resist bending moment along one axis were also made of square HSSs so that all of the columns in a story would be made of the same section size. This was done assuming that the typical structural engineer will follow a similar practice of economy of scale in an effort to reduce overall project costs.

Besides varying the gravity loading and the material strengths between buildings, exceptionally large d/t ratios were used in Designs 3C, 9C, and 18C. This resulted in column sections that are up to 27 inches in depth and d/t ratios that range between 61 and 80. Although tube sections of this size are available for purchase, the 2005 AISC Specification does not allow sections with such large d/t ratios to be used for design. These sections were included in this study so that the response of a building with large d/t ratios could be measured in the inelastic pushover analysis and later in the demand assessment. Their responses in each analysis will be compared to the buildings with d/t ratios that are within the allowed limits to determine how the d/t ratio affects the overall behavior of a structural building system.

The controlling design parameter for the column and girder sections in all thirteen buildings was the drift limits required by IBC 2003. The maximum inelastic interstory drift, δ_x, was not to exceed 2% of the story height ($0.02H_i$) based on the RCFT moment frames being categorized as "All other structures." Therefore, the maximum permissible elastic interstory drift used in each elastic analysis is equal to 0.567 inches. This value is based on the constant story height of 13-feet in all of the buildings. Due to these drift limits, all of the column and girder sections resulted in being stiffer (and stronger) than what would be needed if they were only sized based on the 2005 AISC Specification member strengths. Therefore, all of the column interaction values ranged between 0.45 and 0.95, which are less than the maximum allowed value of 1.0 per AISC (AISC, 2005).

The elastic drift of a building is necessary for determining the elastic stiffness of that building. A value of 5.5 was used for the deflection amplification factor, C_d, in determining the maximum permissible elastic roof drift per Equation 3.3.3-1. The elastic drift at the center of gravity of each building was approximated through linear interpolation using the known elastic drift values at each floor level above and below the center of gravity. An approximate building elastic stiffness, K_e, was calculated for each building by dividing the design base shear by the elastic drift of the center of gravity. The 3-story buildings have an average stiffness of 1,273 kips/in while the 9-story and 18-story buildings have an average stiffness of 520 kips/in and 242 kips/in, respectively.

A key reason for the difference in building stiffness values between two buildings with different number of stories is due to the geometry of each building. The interstory drift in short buildings is mainly due to the end rotations of the beams and columns, otherwise known as bent action (Naeim, 2001), in addition to the flexure of the columns. In taller buildings the axial reaction due to the overturning moment plays a larger role in the overall drift of the building because this increase in axial load shortens the lower level columns, as well as makes them unstable sooner than when compared to the lower columns in a shorter building. Short buildings are mainly designed to resist shear loads while tall buildings are mainly designed to resist axial loads due to the overturning moment. Since shorter buildings are mainly designed to resist shear loads they need to have stiffer beam-columns compared to taller buildings, which need columns that are designed to mainly resist axial loads. This difference in the role of the columns between buildings of different heights becomes evident when the building capacity values are calculated and compared.

Story	3-Story Buildings						
	3A	3B	3C	3D	3E	3F	3G
1	HSS 19x19x3/8	HSS 18x18x1/2	HSS 21x21x5/16	HSS 22x22x5/8	HSS 21x21x3/8	HSS 17x17x5/8	HSS 16x16x1/2
2	HSS 19x19x3/8	HSS 18x18x1/2	HSS 21x21x5/16	HSS 22x22x5/8	HSS 21x21x3/8	HSS 17x17x5/8	HSS 16x16x1/2
3	HSS 19x19x3/8	HSS 18x18x1/2	HSS 21x21x5/16	HSS 22x22x5/8	HSS 21x21x3/8	HSS 17x17x5/8	HSS 16x16x1/2

Table 5.1.2: Exterior Column Sections for the 3-Story Buildings

Story	9-Story Buildings			18-Story Buildings		
	9A	9B	9C	18A	18B	18C
1	HSS 22x22x5/8	HSS 18x18x5/8	HSS 27x27x5/16	HSS 20x20x1/2	HSS 16x16x5/8	HSS 24x24x5/16
2	HSS 22x22x5/8	HSS 18x18x5/8	HSS 27x27x5/16	HSS 20x20x1/2	HSS 16x16x5/8	HSS 24x24x5/16
3	HSS 22x22x5/8	HSS 18x18x5/8	HSS 27x27x5/16	HSS 20x20x1/2	HSS 16x16x5/8	HSS 24x24x5/16
4	HSS 22x22x5/8	HSS 18x18x5/8	HSS 27x27x5/16	HSS 20x20x1/2	HSS 16x16x5/8	HSS 24x24x5/16
5	HSS 22x22x1/2	HSS 18x18x1/2	HSS 25x25x5/16	HSS 20x20x1/2	HSS 16x16x5/8	HSS 24x24x5/16
6	HSS 22x22x1/2	HSS 18x18x1/2	HSS 25x25x5/16	HSS 20x20x1/2	HSS 16x16x5/8	HSS 24x24x5/16
7	HSS 20x20x5/8	HSS 16x16x5/8	HSS 22x22x3/8	HSS 20x20x1/2	HSS 16x16x5/8	HSS 24x24x5/16
8	HSS 20x20x1/2	HSS 16x16x1/2	HSS 22x22x5/16	HSS 20x20x1/2	HSS 16x16x5/8	HSS 24x24x5/16
9	HSS 20x20x1/2	HSS 16x16x1/2	HSS 22x22x5/16	HSS 20x20x1/2	HSS 16x16x5/8	HSS 22x22x5/16
10	---	---	---	HSS 20x20x1/2	HSS 16x16x5/8	HSS 22x22x5/16
11	---	---	---	HSS 18x18x5/8	HSS 16x16x1/2	HSS 22x22x5/16
12	---	---	---	HSS 18x18x5/8	HSS 16x16x1/2	HSS 22x22x5/16
13	---	---	---	HSS 18x18x1/2	HSS 16x16x1/2	HSS 21x21x5/16
14	---	---	---	HSS 18x18x1/2	HSS 14x14x3/4	HSS 21x21x5/16
15	---	---	---	HSS 18x18x1/2	HSS 14x14x3/4	HSS 21x21x5/16
16	---	---	---	HSS 16x16x3/4	HSS 12x12x3/4	HSS 18x18x1/4
17	---	---	---	HSS 16x16x3/4	HSS 12x12x3/4	HSS 18x18x1/4
18	---	---	---	HSS 16x16x3/4	HSS 12x12x3/4	HSS 18x18x1/4

Table 5.1.3: Exterior Column Sections for the 9-Story and 18-Story Buildings

Story	3-Story Buildings						
	3A	3B	3C	3D	3E	3F	3G
1	HSS 22x22x5/8	HSS 18x18x1/2	HSS 21x21x5/16	HSS 22x22x5/8	HSS 21x21x3/8	HSS 17x17x5/8	HSS 16x16x1/2
2	HSS 22x22x5/8	HSS 18x18x1/2	HSS 21x21x5/16	HSS 22x22x5/8	HSS 21x21x3/8	HSS 17x17x5/8	HSS 16x16x1/2
3	HSS 22x22x5/8	HSS 18x18x1/2	HSS 21x21x5/16	HSS 22x22x5/8	HSS 21x21x3/8	HSS 17x17x5/8	HSS 16x16x1/2

Table 5.1.4: Interior Column Sections for the 3-Story Buildings

Story	9-Story Buildings			18-Story Buildings		
	9A	9B	9C	18A	18B	18C
1	HSS 22x22x5/8	HSS 18x18x5/8	HSS 27x27x5/16	HSS 20x20x1/2	HSS 16x16x5/8	HSS 24x24x5/16
2	HSS 22x22x5/8	HSS 18x18x5/8	HSS 27x27x5/16	HSS 20x20x1/2	HSS 16x16x5/8	HSS 24x24x5/16
3	HSS 22x22x5/8	HSS 18x18x5/8	HSS 27x27x5/16	HSS 20x20x1/2	HSS 16x16x5/8	HSS 24x24x5/16
4	HSS 22x22x5/8	HSS 18x18x5/8	HSS 27x27x5/16	HSS 20x20x1/2	HSS 16x16x5/8	HSS 24x24x5/16
5	HSS 22x22x1/2	HSS 18x18x1/2	HSS 25x25x5/16	HSS 20x20x1/2	HSS 16x16x5/8	HSS 24x24x5/16
6	HSS 22x22x1/2	HSS 18x18x1/2	HSS 25x25x5/16	HSS 20x20x1/2	HSS 16x16x5/8	HSS 24x24x5/16
7	HSS 20x20x5/8	HSS 16x16x5/8	HSS 22x22x3/8	HSS 20x20x1/2	HSS 16x16x5/8	HSS 24x24x5/16
8	HSS 20x20x1/2	HSS 16x16x1/2	HSS 22x22x5/16	HSS 20x20x1/2	HSS 16x16x5/8	HSS 24x24x5/16
9	HSS 20x20x1/2	HSS 16x16x1/2	HSS 22x22x5/16	HSS 20x20x1/2	HSS 16x16x5/8	HSS 22x22x5/16
10	---	---	---	HSS 20x20x1/2	HSS 16x16x5/8	HSS 22x22x5/16
11	---	---	---	HSS 18x18x5/8	HSS 16x16x1/2	HSS 22x22x5/16
12	---	---	---	HSS 18x18x5/8	HSS 16x16x1/2	HSS 22x22x5/16
13	---	---	---	HSS 18x18x1/2	HSS 16x16x1/2	HSS 21x21x5/16
14	---	---	---	HSS 18x18x1/2	HSS 14x14x3/4	HSS 21x21x5/16
15	---	---	---	HSS 18x18x1/2	HSS 14x14x3/4	HSS 21x21x5/16
16	---	---	---	HSS 16x16x3/4	HSS 12x12x3/4	HSS 18x18x1/4
17	---	---	---	HSS 16x16x3/4	HSS 12x12x3/4	HSS 18x18x1/4
18	---	---	---	HSS 16x16x3/4	HSS 12x12x3/4	HSS 18x18x1/4

Table 5.1.5: Interior Column Sections for the 9-Story and 18-Story Buildings

Floor	3-Story Buildings						
	3A	3B	3C	3D	3E	3F	3G
1	---	---	---	---	---	---	---
2	W18x119	W27x84	W24x94	W21x122	W21x122	W21x68	W12x136
3	W18x119	W27x84	W24x94	W21x122	W21x122	W21x68	W12x136
Roof	W24x55	W24x55	W21x57	W24x62	W24x62	W18x40	W12x72

Table 5.1.6: Girder Sections for the 3-Story Buildings

Floor	9-Story Buildings			18-Story Buildings		
	9A	9B	9C	18A	18B	18C
1	---	---	---	---	---	---
2	W30x90	W30x99	W27x84	W24x76	W24x84	W24x68
3	W30x90	W30x99	W27x84	W24x76	W24x84	W24x68
4	W30x90	W30x99	W27x84	W24x76	W24x84	W24x68
5	W27x84	W30x90	W27x84	W24x76	W24x84	W24x68
6	W27x84	W30x90	W24x84	W24x76	W24x84	W24x68
7	W27x84	W27x84	W24x76	W24x76	W24x84	W24x68
8	W24x84	W24x84	W24x68	W24x76	W24x84	W24x68
9	W24x76	W24x76	W24x68	W24x76	W24x84	W21x68
10	---	---	---	W24x76	W24x84	W21x68
11	---	---	---	W24x68	W24x76	W21x68
12	---	---	---	W24x68	W24x76	W21x68
13	---	---	---	W24x68	W24x68	W21x68
14	---	---	---	W21x68	W24x68	W21x68
15	---	---	---	W21x68	W24x68	W18x50
16	---	---	---	W21x62	W21x62	W18x50
17	---	---	---	W18x55	W21x62	W18x50
18	---	---	---	W18x55	W21x62	W18x50
Roof	W24x76	W24x76	W24x68	W18x55	W21x62	W18x50

Table 5.1.7: Girder Sections for the 9-Story and 18-Story Buildings

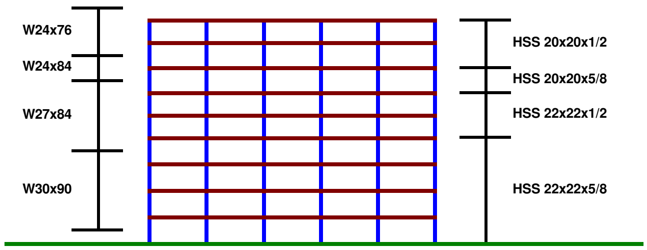


Figure 5.1.1: Design 9A Section Sizes

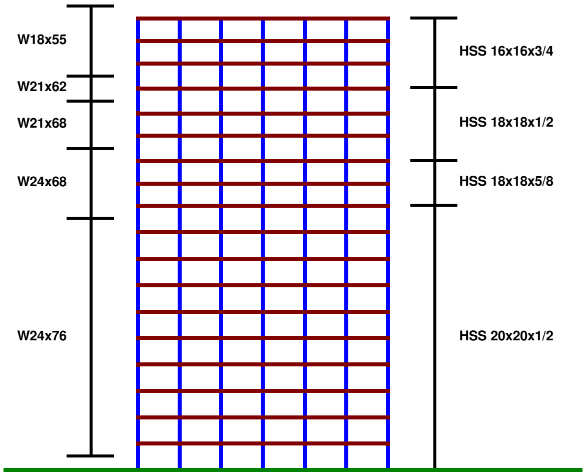


Figure 5.1.2: Design 18A Section Sizes

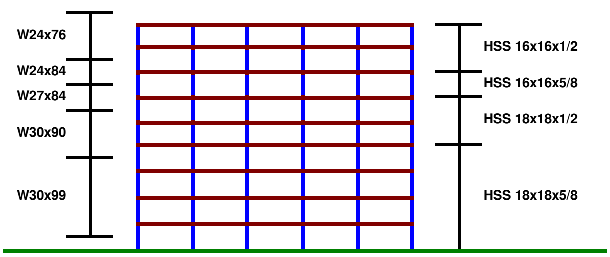


Figure 5.1.3: Design 9B Section Sizes

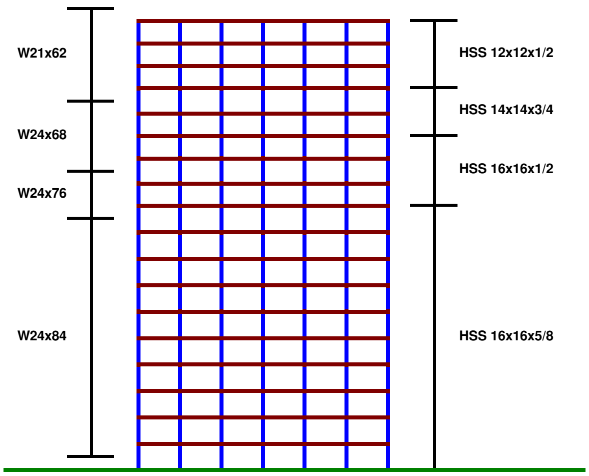


Figure 5.1.4: Design 18B Section Sizes

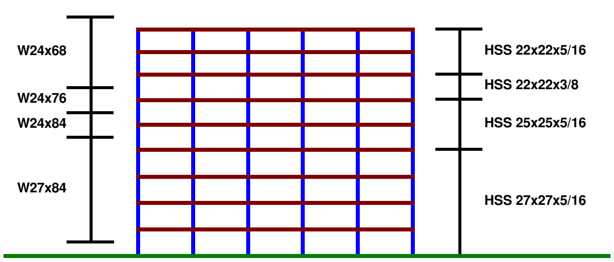


Figure 5.1.5: Design 9C Section Sizes

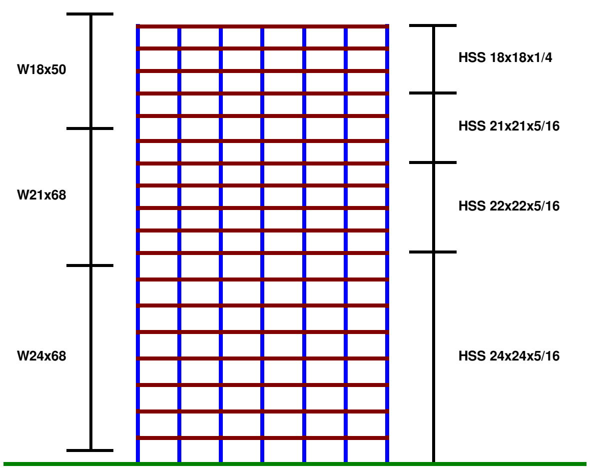


Figure 5.1.6: Design 18C Section Sizes

Table 5.1.8 summarizes the maximum value of the stability coefficient, θ, for each building. This comparison illustrates that the 3-story buildings are very stable since their θ values are well below 0.10. The 9-story buildings are also stable, but since their θ values are increasing, they are not considered as stable as the 3-story buildings. The 18-story buildings have θ values at or near 0.10, indicating that P-δ effects will start to affect the response of these buildings when they exceed their elastic limit.

Linear Elastic Analysis Results	3-Story Buildings							9-Story Buildings			18-Story Buildings		
	3A	3B	3C	3D	3E	3F	3G	9A	9B	9C	18A	18B	18C
Maximum d / t	45	30	61	28	50	20	30	38	30	80	34	30	71
Maximum Interaction Value	0.70	0.47	0.54	0.61	0.58	0.56	0.45	0.78	0.47	0.91	0.95	0.47	0.81
Maximum Stability Coefficient θ	0.032	0.031	0.029	0.032	0.033	0.032	0.035	0.076	0.083	0.075	0.112	0.123	0.103
Roof Drift at Design Base Shear Δ_{design} *[inches]*	1.41	1.39	1.28	1.28	1.35	1.30	1.42	4.39	4.67	4.50	9.17	9.69	9.25
Maximum Allowed Elastic Roof Drift *[inches]*	1.70	1.70	1.70	1.70	1.70	1.70	1.70	5.11	5.11	5.11	10.21	10.21	10.21
Expected Maximum Inelastic Roof Drift Δ_I *[inches]*	7.76	7.65	7.04	7.04	7.43	7.15	7.81	24.15	25.69	24.75	50.44	53.30	50.88
Building Center of Gravity Elevation *[feet]*	26.2	26.2	26.2	20.5	20.5	24.5	24.5	66.5	66.5	66.5	123.5	123.5	123.5
Elastic Drift at Building Center of Gravity Δ_{CG} *[inches]*	0.90	0.91	0.81	0.60	0.67	0.84	0.92	2.60	2.71	2.54	5.17	5.52	4.99
Building Elastic Stiffness at Center of Gravity K_e *[kips/in]*	1,000	989	1,111	1,883	1,686	1,170	1,068	523	502	535	244	229	253

Table 5.1.8: Linear Static Analysis Results for Each Building

Equations 5.1-1 and 5.1-2 were developed for this study to check the approximate design period, T_a, which was used in each building design. Equation 5.1-1 was derived using the same method that is illustrated by Equations 6.2-2 through 6.2-6. T_1 (and T_2) result in period values that are approximately 1.6 times larger than the design period, T_a, in all thirteen building designs. This indicates that a dynamic analysis of each building could result in smaller column and girder sections, since lower seismic loads would typically result from such an analysis. Since there is a 60% difference in values of the fundamental period between these two methods the use of the maximum allowed period by ASCE 7-02 might result in a more accurate value of C_s than by just using the minimum value of T_a. However the intent of this study was to design the columns and girders of each building without using relatively complex methods of analysis. Therefore, the design period of each building was based on T_a and the values of T_1 (and T_2) are only for reference.

$$T_1 = 2\pi\sqrt{\frac{V_{design}/C_s}{gK_e}} \tag{5.1-1}$$

$$T_2 = 2\pi\sqrt{\frac{\Delta_{CG}}{gC_s}} \tag{5.1-2}$$

Where: V_{design} = building total design seismic base shear
C_s = design seismic response coefficient
Δ_{CG} = elastic drift at the building center of gravity
g = acceleration of gravity
K_e = building elastic stiffness = V_{design} / Δ_{CG}

Since the fundamental period is needed to calculate C_s for each building, T_1 (and T_2) are best used for verifying the design period that is used in the seismic design. If T_a is too conservative (i.e., it is smaller than what would result from a more substantiated rational analysis such as Rayleigh's Method) C_s would be too large which will lead to an uneconomical building design. By estimating how conservative the design period is, a decision can be made by the structural engineer to determine if a more exact method for calculating the design period is required, or if the current design value is sufficient. The values of T_a that were used for each building design are listed in Table 5.1.1 while the values of T_1 (and T_2) that are based on the elastic analysis results are listed in Table 5.1.9.

To verify the accuracy of Equation 5.1-1 and Equation 5.1-2, a dynamic (eigenvalue) analysis was performed on each 9-story and 18-story building so that a more rational fundamental period, T_r, for each building could be calculated. These values of T_r for each building are listed in Table 5.1.9. Since there is only a 5% difference between the values of T_1 (and T_2) and T_r for each building, Equation 5.1-1 and Equation 5.1-2 are considered to be able to provide an accurate method for calculating an approximate fundamental period of a building.

Fundamental Period	9-Story Buildings			18-Story Buildings		
	9A	9B	9C	18A	18B	18C
Building Design Period T_a *[seconds]*	1.264	1.264	1.264	2.201	2.201	2.201
Building Approximated Period T_1 (and T_2) *[seconds]*	2.10	2.15	2.08	3.46	3.58	3.40
Building Period by Rational Analysis T_r *[seconds]*	2.05	2.10	2.05	3.28	3.37	3.24

Table 5.1.9: Approximate and Rationally Calculated Fundamental Periods of Each 9-Story and 18-Story Building

5.2 Nonlinear Static Pushover Analysis Results

Once each building was designed elastically and the column and girder section sizes were finalized, a force-based nonlinear static pushover analysis was performed on each building to determine its lateral strength and inelastic (post-yielding) response. The main parameters that were measured and used in this portion of the study were the base shear, V_{PO}, and the corresponding roof drift, Δ_{roof}. Using these two parameters, a number of system characteristics were determined for each building including the elastic stiffness, K_e, the capacity of the building, the relative energy absorbed by the building, and the overstrength factor, Ω.

Figure 5.2.1 shows the base shear and roof drift relationship for all thirteen buildings. As shown in Table 5.2.1.1 and illustrated in Figure 5.2.1, the 3-story buildings are much stiffer than the 9-story and 18-story buildings by a factor of 3 and 4, respectively. However, the average maximum (ultimate) base shear is not much larger for the 3-story buildings compared to the 9-story and 18-story buildings. The 3-story building average maximum base shear is just over 1.0 times larger than the 9-story buildings and 1.3 times larger than the 18-story buildings.

Figure 5.2.2 illustrates the normalized base shear and the normalized roof drift for each building. By normalizing the force and displacement values for each building, a comparison can be made between any two building systems regardless of the number of stories. All of the buildings are bunched together in a relatively tight bandwidth in the elastic portion of the curves. Only after the values start to exceed their elastic limit do they start to spread out from each other and show their individual characteristics. The 3-story buildings have the largest normalized ultimate force with an average value of 3.8, while the 18-story buildings have the smallest at 2.3.

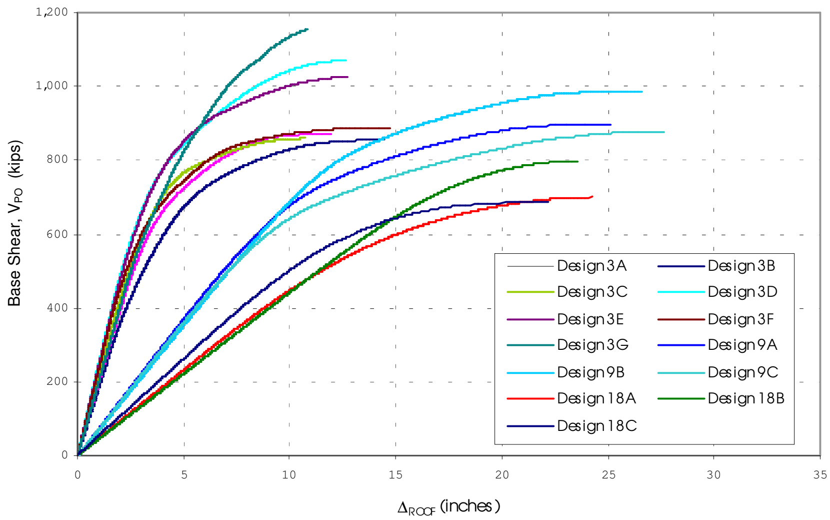


Figure 5.2.1: Pushover Analysis Curves For All Thirteen buildings of This Study

The normalized curves, as shown in Figure 5.2.2, illustrate how there is not much of a difference in system overstrength between buildings that have the same roof elevation (or with the same design period), but will vary significantly between buildings with different roof elevations (or with significantly different design periods).

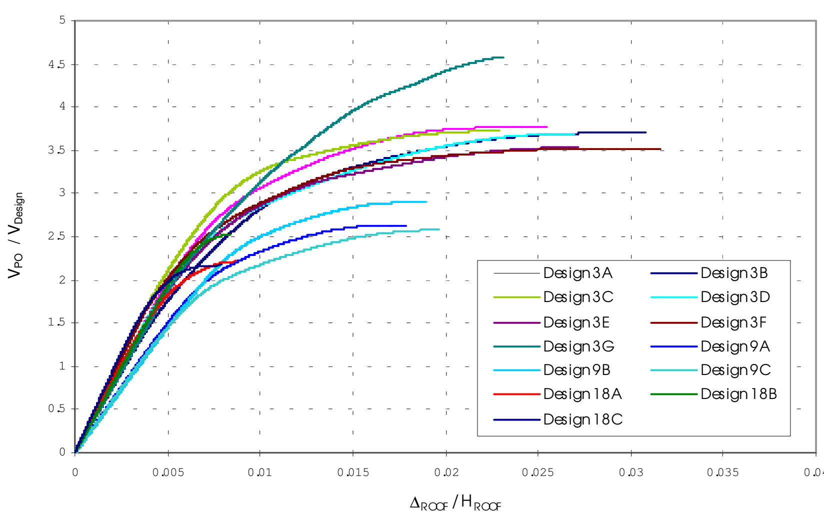


Figure 5.2.2: Normalized Pushover Analysis Curves For All Thirteen buildings of This Study

5.2.1 Elastic Stiffness, Capacity, and Relative Energy

Even though each building is a multiple degree of freedom (MDOF) system, a single degree of freedom (SDOF) system approximation was used to calculate the elastic stiffness, as was done in the elastic analysis. The elastic stiffness of each building was determined by dividing the base shear force with the displacement at the center of gravity of the building. Since the pushover analysis results do not yield an exact elastic limit, the elastic limit drift value was designated as the point in the pushover analysis when the change between two consecutive stiffness points was less than -0.02. Using the base shear and displacement at this point in the analysis, the elastic stiffness at the center of gravity was determined. The 3-story buildings had an average building stiffness of 1,465 kips/in while the 9-story and 18-story buildings had an average stiffness of 498 kips/in and 342 kips/in, respectively.

The relative termination point for the energy calculation of each building depended on the design period, T_a, and the corresponding k-value that was used to distribute the seismic loads vertically along the height of the building, as shown in Equation 3.1.4.5-3. The pushover curves for each building were distributed in such a way as to suggest that the period of the building was contributing where the pushover curve for a building will be located with respect to the other building curves. Therefore, the k-value was used in determining the end value of the energy calculations, since it is a function of the period of the building. The k-value was also found to be able to terminate the energy calculations for each building at a point in their analysis that is near their actual ending point.

The energy values were calculated by determining the area under the pushover curve from the start of the analysis to the roof drift that corresponded to $\Delta/h_r = 0.02/k$ on the normalized curve. The value of k equals 1.01, 1.38, and 1.85 for the 3-story, 9-story, and 18-story buildings, respectively. The constant 0.02 is used since when it is divided by the k-value the result is a maximum Δ/h_r value that corresponds with the pushover curves for each building. The k-value, although dependent on the design period, T_a, is indirectly dependent of the roof height, h_r, since T_a is a function of the roof height.

Figure 5.2.1.1 illustrates the relationship between the Δ/h_r and the roof height of a building. This relationship is similar to the relationship shown in a typical elastic design spectrum between the design period and the seismic base shear. As the roof height (or design period) increases, the base shear decreases, as does Δ/h_r and the k-value. This decreasing trend is also found in the pushover curves.

As shown in Figure 5.2.1, increases in a building roof height correlate to both a decrease in the maximum base shear force and in the overall roof drift. Therefore, for a variable to describe the end point of the energy calculations near the actual analysis termination points, it has to have similar trends as the pushover curves, and it has to vary from building to building. The k-value fits the data points well and terminates the relative energy analysis for each building at or near their actual analysis end points.

Pushover Analysis Results	3-Story Buildings							9-Story Buildings			18-Story Buildings		
	3A	3B	3C	3D	3E	3F	3G	9A	9B	9C	18A	18B	18C
2D Moment Frame Seismic (Design) Base Shear V_{design} *[kips]*	225	225	225	282	282	246	246	340	340	340	316	316	316
2D Moment Frame "Elastic Limit" Base Shear V_e *[kips]*	349	437	751	444	419	387	497	546	623	543	622	757	433
Maximum 2D Moment Frame Base Shear V_{max} *[kips]*	856	872	859	1,070	1,025	888	1,154	897	986	878	700	799	687
Roof Drift at Design Base Shear Δ_{design} *[inches]*	1.26	1.17	1.05	1.17	1.19	1.07	1.25	4.60	4.83	4.72	6.85	7.14	6.05
Roof Drift at "Elastic Limit" Δ_e *[inches]*	1.94	2.25	4.69	1.81	1.75	1.65	2.49	7.57	8.96	7.86	16.17	19.05	8.43
Maximum Inelastic Roof Drift Δ_{max} *[inches]*	14.43	11.92	10.74	12.64	12.73	14.77	10.81	25.13	26.64	27.63	24.28	23.53	22.14
Building Center of Gravity (C.G.) Elevation *[feet]*	26.2	26.2	26.2	20.5	20.5	24.5	24.5	66.5	66.5	66.5	123.5	123.5	123.5
Elastic Drift at Building C.G. Δ_{CG} *[inches]*	0.80	0.77	0.66	0.55	0.59	0.69	0.81	2.72	2.80	2.66	3.86	4.07	3.26
Elastic Stiffness at Building C.G. K_e *[kips/in]*	1,119	1,175	1,355	2,060	1,913	1,421	1,213	499	485	510	327	311	387
Building Fundamental Period T_1 *[seconds]*	0.81	0.79	0.74	0.67	0.70	0.75	0.81	2.15	2.18	2.13	2.99	3.07	2.75
Building Fundamental Period T_2 *[seconds]*	0.81	0.79	0.74	0.67	0.70	0.75	0.81	2.15	2.18	2.13	2.99	3.07	2.75

Table 5.2.1.1: Nonlinear Static Pushover Analysis Results for Each Building

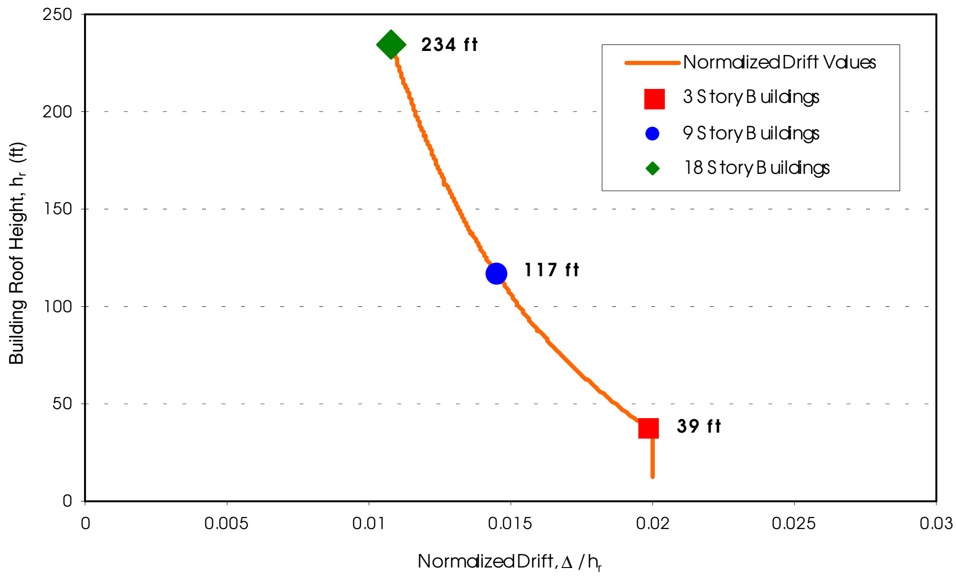


Figure 5.2.1.1: Normalized Drift, Δ/h_r, Used to End the Relative Energy Calculations

The capacity curve of a 2D moment frame is a relative measure of the amount of force that the frame can resist before it reaches its limit state of collapse. The capacity curve is calculated by dividing the energy that the 2D moment frame has absorbed by the corresponding displacement (Guo and Gilsanz, 2003). As shown in Table 5.2.1.2, the 3-story moment frames have an average capacity that is 15% greater than the average 9-story frame capacity and 40% greater than the average 18-story frame capacity.

Pushover Analysis Results	3-Story Buildings							9-Story Buildings			18-Story Buildings		
	3A	3B	3C	3D	3E	3F	3G	9A	9B	9C	18A	18B	18C
Normalized Roof Displacement at Energy Calculation Termination Δ / h_r	0.020	0.020	0.020	0.020	0.020	0.020	0.020	0.014	0.014	0.014	0.011	0.011	0.011
Roof Displacement at Energy Calculation Termination *[inches]*	9.35	9.35	9.36	9.35	9.36	9.36	9.36	19.57	19.65	19.62	24.67	23.25	21.96
Relative Energy Absorbed *[kip-ft]*	441	474	497	561	552	499	553	935	971	894	948	924	846
Capacity *[kips]*	566	608	638	720	709	641	709	573	593	547	461	477	462

Table 5.2.1.2: Capacity and Relative Energy Absorbed for Each Building

This difference in building capacity values is attributed to the fact that a greater percentage of the force that the columns in shorter buildings are designed to resist is shear loads, while a larger percentage of the force in taller building columns are axial loads. This trend results in the shorter buildings having stiffer columns relative to the overall building mass compared to the taller buildings. Just as was demonstrated in the elastic stiffness discussion, this phenomenon of varying capacity values is expected to occur between any two buildings that have different roof heights as well as different fundamental periods.

5.2.1.1 Ramberg-Osgood Equation Approximation

The Ramberg-Osgood model (Ramberg and Osgood, 1943), as shown in Equation 5.2.1.1-1, was used to approximate the pushover curve for each building in this study. This model was chosen to describe the pushover curves since it is able to provide a good approximation of the curves with only three variables per building. Appendix N describes how the constants G and s were calibrated for each building curve. Once an equation was derived for a pushover curve the area under the curve (relative energy absorbed) was calculated by integrating each equation from time zero to the time at which the roof drift corresponded to when Δ/h_r equaled 0.02/k on the normalized pushover curve.

$$\Delta_{roof} = \frac{V_{PO}}{K_{roof}} + G\left(\frac{V_{PO}}{K_{roof}}\right)^{s} \qquad (5.2.1.1\text{-}1)$$

Where: Δ_{roof} = roof drift
V_{PO} = shear at the base of the 2D moment frame
K_{roof} = elastic stiffness of the 2D moment frame using roof drift
G = constant for each 2D moment frame
s = constant for each 2D moment frame

Pushover Analysis Results	3-Story Buildings							9-Story Buildings			18-Story Buildings		
	3A	3B	3C	3D	3E	3F	3G	9A	9B	9C	18A	18B	18C
Elastic Stiffness at the Roof K_{roof} *[kips/in]*	183	198	219	248	242	236	202	74	71	72	46	44	52
G	5.0 E-04	1.3 E-04	1.1 E-06	6.0 E-04	1.0 E-04	1.0 E-04	4.4 E-04	1.9 E-08	1.0 E-08	3.3 E-08	5.0 E-12	1.0 E-17	1.0 E-14
s	6.10	7.13	11.29	6.41	7.75	8.40	5.34	8.01	7.75	7.93	10.40	14.10	13.35
Ending V_{PO} [kips]	825	868	857	1,022	987	868	1,105	877	957	827	700	799	687
Ending Δ_{roof} [inches]	9.35	9.35	9.36	9.35	9.36	9.36	9.36	19.57	19.65	19.62	24.67	23.25	21.96

Table 5.2.1.1.1: Ramberg-Osgood Equation Parameters and Constants

Table 5.2.1.1.1 lists the constants G and s that were derived for each building along with the corresponding constant elastic stiffness value, K_{roof}. Since the independent variable in each equation is V_{PO}, the area under the curve was determined by integrating each equation and subtracting it from the value computed from multiplying the last base shear and the corresponding roof drift values used in the energy calculations, $(V_{PO})_{ending}$ x $(\Delta_{roof})_{ending}$.

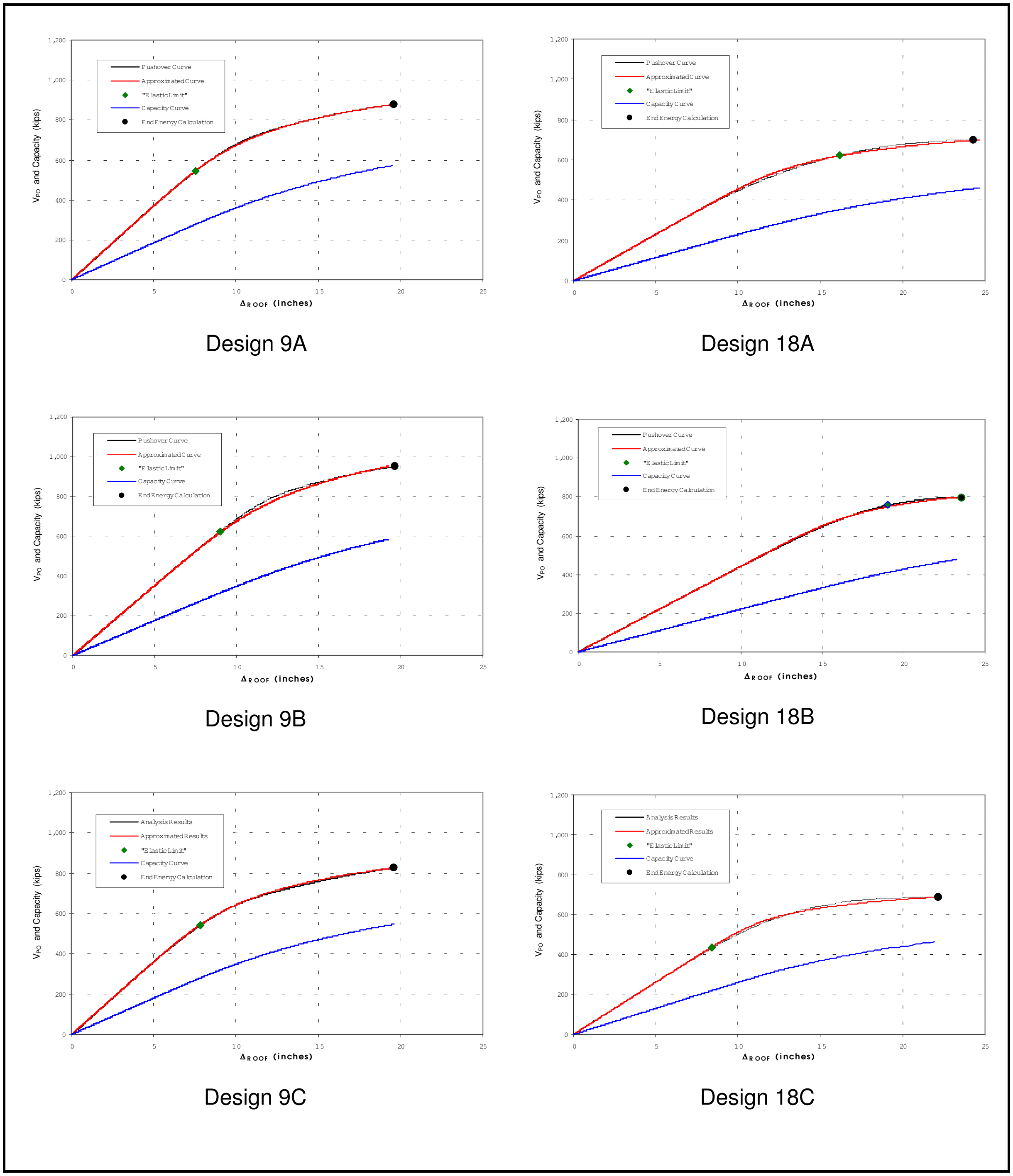


Figure 5.2.1.1.1: 9-Story and 18-Story Building Pushover Curves, Capacity Curves, and the Ramberg-Osgood Equation Approximated Curves

Figure 5.2.1.1.1 illustrates three major curves for each 9-story and 18-story building. These three curves include the actual pushover curve for each building, the approximated curve that was derived using the Ramberg-Osgood Equation, and the capacity curve for each building. These plots demonstrate how the capacity curve varies between buildings of different heights and how the Ramberg-Osgood Equation is able to model a good approximation of the actual pushover curve for each building.

5.2.2 System Overstrength Factor, Ω

The system overstrength factor, Ω, represents the ratio between the maximum base shear from the pushover analysis, V_{PO}, to the design seismic base shear, V_{design}. By using Equation 5.2.2-1 to determine the overstrength factor for each of the thirteen buildings of this study, it was determined that the value of Ω for RCFT structural systems is actually dependent on one or more system characteristics, rather than being constant for any building with a particular structural system (i.e., special moment RCFT frame).

$$\Omega = \frac{V_{PO}}{V_{design}} \tag{5.2.2-1}$$

Three system characteristics were found to help predict Ω. These three system characteristics are the number of stories of a building, n, the roof elevation, h_r, and the fundamental design period, T_a, that was used to calculate the seismic base shear coefficient. Three different second order polynomial equations were derived to estimate the system overstrength factor for a RCFT building when a particular system characteristic is known. These three system relationships are shown in Equations 5.2.2-2 through 5.2.2-4 and in Figures 5.2.2.1 through 5.2.2.3.

$$\Omega = \left(\frac{n}{11.5}\right)^2 - \frac{n}{4} + 4.5 \tag{5.2.2-2}$$

$$\Omega = \left(\frac{T_a}{1.55}\right)^2 - 2T_a + 4.7 \tag{5.2.2-3}$$

$$\Omega = \left(\frac{h_r}{160}\right)^2 - \frac{h_r}{55} + 4.5 \tag{5.2.2-4}$$

The development of Equations 5.2.2-2 through 5.2.2-4 required three anchor points to be calculated from the pushover analysis results. These anchor points are the mean values of the overstrength factor that was calculated from the pushover analysis for each building according to the number of stories in the building. The 3-story buildings have a mean value of 3.79 while the 9-story and 18-story buildings that have mean values of 2.70 and 2.31, respectively. The value of each anchor point is listed in Table 5.2.2.1.

Building Designation	Ω	Number of Stories n	Design Period T_a	Roof Height h_r
3 Story	3.79	3	0.525 sec	39 ft
9 Story	2.70	9	1.264 sec	117 ft
18 Story	2.31	18	2.201 sec	234 ft

Table 5.2.2.1: Anchor Points Used in Figure 5.2.2.1 Through Figure 5.2.2.3

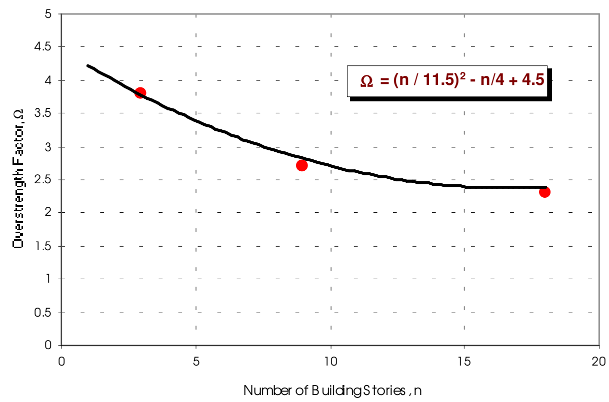


Figure 5.2.2.1: Overstrength Factor, Ω, as a Function of the Number of Stories, n

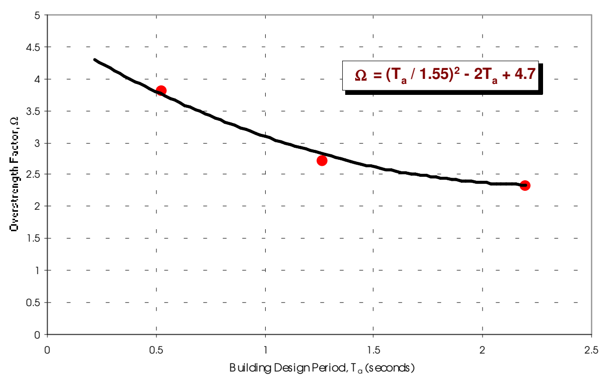


Figure 5.2.2.2: Overstrength Factor, Ω, as a Function of the Building Design Period, T_a

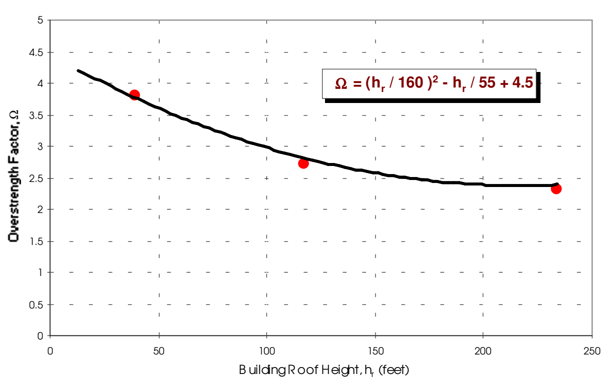


Figure 5.2.2.3: Overstrength Factor, Ω, as a Function of the Roof Height, h_r

Chapter 6

Assessment of the Final Suite of Buildings

One of the critical steps in calibrating a reliability based performance-based design methodology for RCFT columns is to perform a seismic demand assessment. The demand assessment is considered sufficiently inclusive when it is performed on a suite of buildings that cover a wide range of structural system responses within the limits of the methodology. This chapter describes how the thirteen buildings that make up the suite of buildings was assessed for this study.

Three methods were developed to verify that the buildings that were designed in this study provide a comprehensive set of system responses that can be used in the demand assessment. The first method used the elastic design spectrum to show that together all thirteen buildings cover a well-dispersed set of possible design base shear values within the continuum of possibilities. The second method set up an envelope of maximum and minimum possible pushover analysis curves and showed that the buildings used in this study fall within this envelope of possible responses. The third method utilized the rigidity ratio concept to show that the overstrength value of each building is at or near its expected value.

6.1 Method 1: *Elastic Seismic Design Spectrum Comparison*

Modern building codes allow for the structural engineer to use an equivalent static analysis method to design a building to resist seismic loads. The equivalent static analysis method is based on the concept of using an elastic seismic design spectrum. By determining the fundamental period of a building, either through approximate methods or by a more rigorous rational analysis, the design base shear coefficient is determined by using the code specified seismic design spectrum. Since the design spectrum includes an infinite number of possible values of the base shear coefficient based on an infinite number of possible fundamental periods, it is not practical to design a different building for every possible design base shear value on the spectrum. However, it is possible to design a building at some key points along the elastic seismic design spectrum.

As with all seismic design spectrums, a portion of the spectrum curve is made up of constant values of the base shear coefficient. In the seismic design spectrum that was used in this study, the constant region occurs for period values that range from zero seconds to just over 0.6 seconds. As the design periods increase in value, the base shear coefficient decreases nonlinearly. When the period reaches approximately 1.75 seconds, the spectrum flattens out again and remains constant for the remaining design periods.

A comparison of the design values of the base shear coefficient, C_s, used in each of the thirteen buildings in this study to the design spectrum indicates that together the buildings have captured values of C_s at three major portions of the design spectrum. The design values of C_s were determined by using the approximate period, T_a, per Equation 3.1.4.5-1.

As Figure 6.1.1 illustrates, the 3-story buildings have a design base shear coefficient in the first plateau region of the design spectrum with a design period of 0.525 seconds and a C_s value of 0.125. The 9-story buildings fall into the middle portion of the nonlinear range of the spectrum and have a design period of 1.26 seconds and C_s equal to 0.06. The 18-story buildings are in the lower plateau region of the design spectrum and have a design period of 2.2 seconds and C_s equal to 0.044.

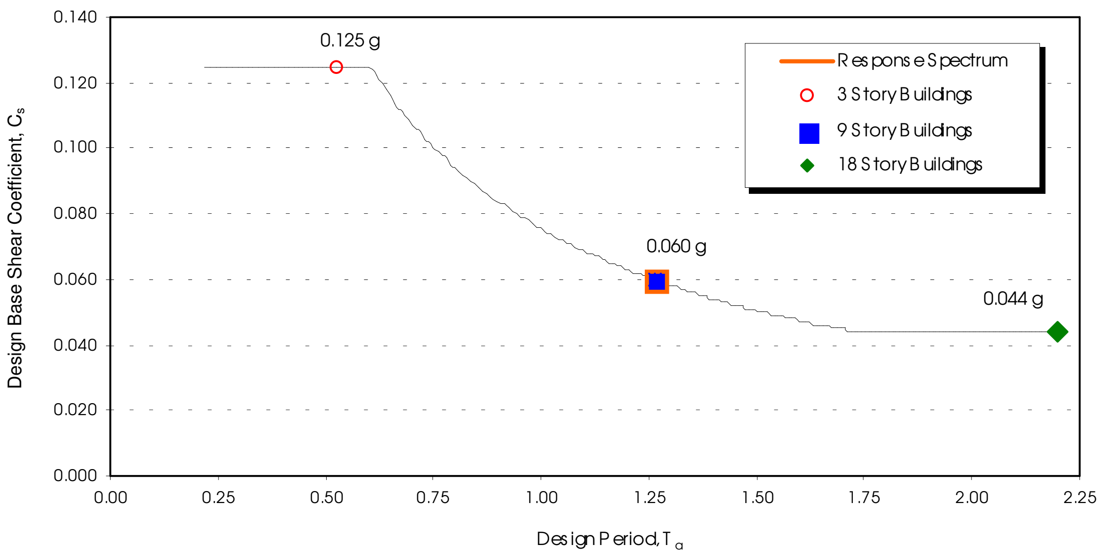


Figure 6.1.1: Design Values of C_s for the Thirteen buildings of This Study On the ASCE 7-02 Elastic Seismic Design Spectrum Using Design Values of the Fundamental Period, T_a

A further comparison of the thirteen buildings with the design spectrum indicates that when a more rigorous method is used to calculate their fundamental periods, the buildings cover more portions of the design spectrum than what was shown in the first comparison. This second comparison is based on using a fundamental period that is calculated by using the stiffness values from the pushover analysis (reference Table 5.2.1.1 and Equations 5.1-1 and 5.1-2) rather than the building code specified minimum fundamental period, T_a. As Figure 6.1.2 illustrates this second comparison shows that the buildings cover a wider range of possible C_s values along the design spectrum than the first comparison showed.

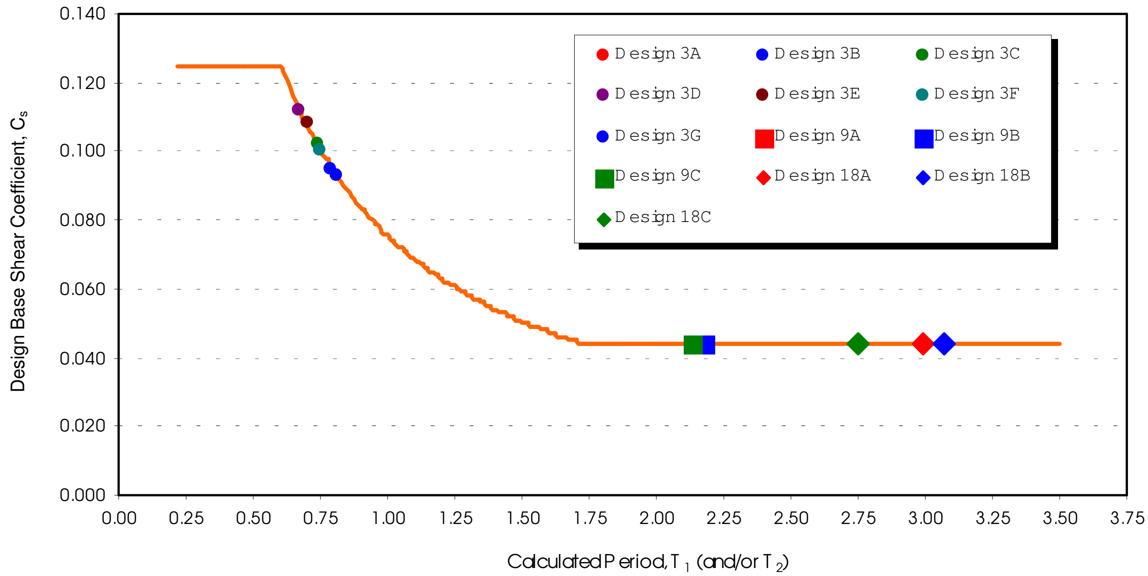


Figure 6.1.2: Values of C_s for the Thirteen buildings of this Study On the ASCE 7-02 Elastic Seismic Design Spectrum Using Calculated Values of the Fundamental Period, T_1 (and T_2), From Table 5.2.1.1

6.2 Method 2: *Pushover Curve Envelope*

The second method that was used to assess the suite of thirteen buildings made a comparison between the pushover analysis curve of each building to an upper bound and a lower bound envelope of idealized system response based on the height of the building. By showing that the actual pushover analysis curve for each of the thirteen buildings falls within a range of expected idealized response curves it can be demonstrated that the buildings are behaving as expected and they are suitable to be a part of the final suite of buildings. A second comparison was then made between all thirteen response curves to the largest upper bound and the smallest lower bound limits of expected system response by taking the largest and the smallest expected system responses from eighteen idealized building systems ranging between 1-story up through 18-stories. This comparison allowed for the system responses of the thirteen buildings in the suite to be compared to the building system range of this study.

As shown in Figure 6.2.1, both the upper bound and the lower bound limits of each system response envelope are comprised of two parts – an initial segment that has a slope representative of the elastic stiffness of the building, and a plateau region that represents a constant base shear force after the onset of nonlinear response. For an idealized upper bound and lower bound stiffness to be determined for each building height, a building mass and a maximum and minimum fundamental period needed to be calculated.

The estimates of the upper bound and lower bound base shear forces are dependent on the design seismic base shear and upper bound and lower bound overstrength factors. Once these parameters are known for a particular building height, the upper bound and lower bound envelope of expected system response was determined by calculating the following parameters:

- Approximate building seismic weight, W
- Maximum and minimum fundamental periods, T_{max} and T_{min}
- Maximum and minimum elastic stiffness, k_{max} and k_{min}
- Maximum and minimum overstrength factors, Ω_{max} and Ω_{min}
- Upper bound and lower bound base shear, V_{max} and V_{min}

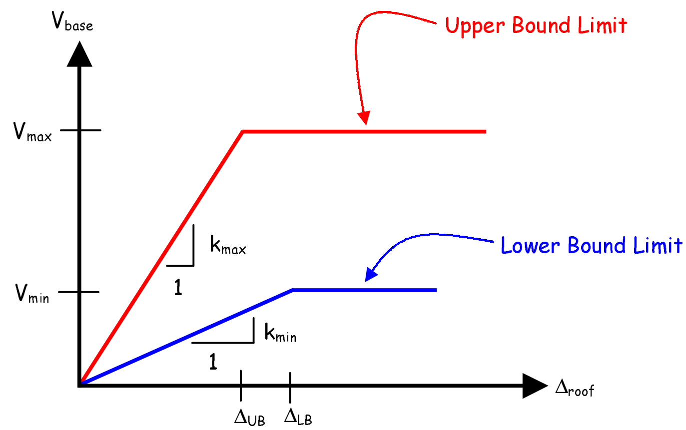


Figure 6.2.1: Idealized Building System Envelope

The first step of this assessment process involved approximating the building weight for each idealized building from 1-story through 18-stories. During the structural design of the thirteen buildings that make up the suite of buildings, the 3-story, 9-story, and 18-story building weights were calculated and recorded. However, in an attempt to develop upper and lower bound curves for eighteen separate idealized buildings that range between 1-story and 18-stories, the fifteen remaining building heights that were not designed for this study needed to have their weights approximated. By using the known weights of the 3-story, 9-story, and 18-story buildings three anchor points were established, as shown in Figure 6.2.2, which then allowed for the remaining building weights to be approximated.

When the thirteen buildings in the suite were designed, the largest member forces in the buildings resulted from the LRFD load combination that included seismic loads. Therefore, the building weight that was used to calculate the building stiffness was based on the building code specified seismic weight. The office building gravity loading was used to calculate the seismic weight based on the assumption that most buildings that range between 1-story and 18-stories will be designed to support office building type gravity loads rather than industrial or warehouse type gravity loads.

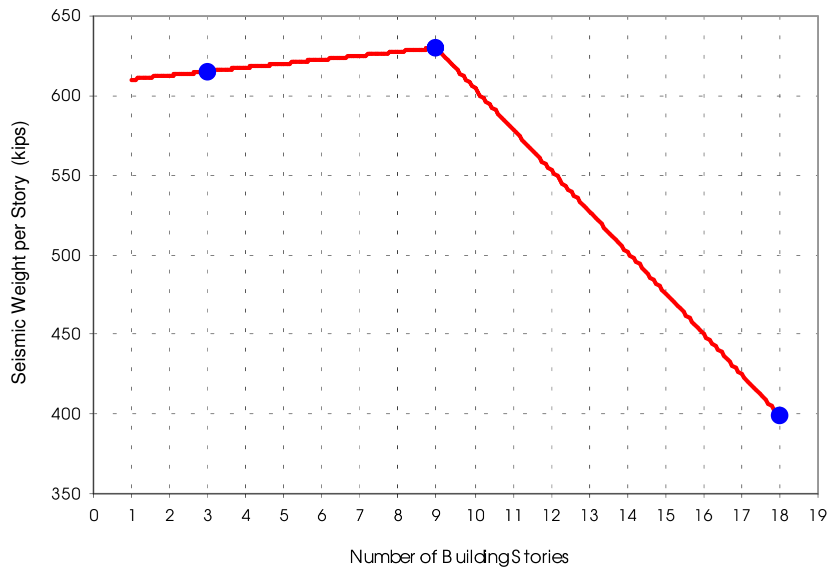


Figure 6.2.2: Approximate Building Seismic Weight per Story

The elastic stiffness of each idealized building is dependent on the fundamental period of the building. The fundamental period of the upper bound curve has been designated as the minimum period, T_{min}, while the fundamental period of the lower bound curve has been called the maximum period, T_{max}. The minimum period represents the period of the smallest (i.e., stiffest) building that would be expected to be designed for a particular building height. The largest period represents the period of the most flexible building that would be expected to be designed for a particular building height.

The minimum period, T_{min}, was determined by multiplying Equation 3.1.4.5-1 by the building code upper bound coefficient C_u, as shown in Equation 6.2-1. The upper limit on the seismic design approximate period, T_a, was used for calculating T_{min} because of the general consensus that the value of the actual fundamental period of most structures will be larger than the seismic design approximate period that is typically used to calculate the static seismic design loads. This approach for calculating a minimum fundamental period for an idealized building is shown in Equation 6.2-1.

$$T_{min} = C_u C_t h_r^x \tag{6.2-1}$$

Where:

$C_u = 1.4$
$C_t = 0.028$
$x = 0.8$
h_r = building roof elevation (assuming all story heights are 13-feet)

The maximum period, T_{max}, represents the largest fundamental period that would be expected to be calculated for a particular building height. Since the largest expected fundamental period for any building system would be its actual period value, this step involved using an approximate method to estimate the period of each idealized building. Based on previous findings of this study, Equation 5.1-2 was shown to provide a good approximation of the actual fundamental period of a building.

In an effort to increase the period by a small margin the elastic roof displacement was used instead of the elastic displacement of the center of gravity of the building. The value of the roof displacement was set equal to the maximum allowed building drift that the building code has established. For this study, the limit on the elastic interstory drift was equal to 0.567 inches per story, as per Equation 3.3.3-1. For example, the limit on the elastic roof drift of an idealized 12-story building would be 6.8 inches. The equation for T_{max} is shown in Equation 6.2-2 followed by how it was derived using known relationships for an SDOF system.

$$T_{max} = 2\pi\sqrt{\frac{\Delta_{roof}}{gC_s}} \tag{6.2-2}$$

Where: Δ_{roof} = the elastic roof displacement = 0.567 x No. Stories
C_s = seismic design coefficient per ASCE 7-02
g = acceleration of gravity

$$\text{Period}, T = 2\pi\sqrt{\frac{m}{k}} \tag{6.2-3}$$

$$\text{Mass}, m = W / g \tag{6.2-4}$$

$$\text{Weight}, W = V_{base} / C_s \tag{6.2-5}$$

$$\text{Stiffness}, k = V_{base} / \Delta_{roof} \tag{6.2-6}$$

$$\text{Therefore, } T_{max} = 2\pi\sqrt{\frac{m}{k}} = 2\pi\sqrt{\frac{W\Delta_{roof}}{gV_{base}}} = 2\pi\sqrt{\frac{V_{base}\Delta_{roof}}{gC_sV_{base}}} = 2\pi\sqrt{\frac{\Delta_{roof}}{gC_s}}$$

Once T_{min}, T_{max}, and the seismic weight (and mass) were calculated for each idealized building between 1-story and 18-stories, their respective maximum and minimum elastic stiffness values were calculated. The stiffness values, k, were determined by using Equation 6.2-7 where T_{min} was used to calculate k_{max}, and T_{max} was used to calculate k_{min}.

$$k_{max/min} = m\left(\frac{2\pi}{T_{min/max}}\right)^2 \tag{6.2-7}$$

The upper bound and lower bound plateaus of the envelope curves for each idealized building system was determined by first calculating a maximum and minimum overstrength factor, Ω. The results of the pushover analysis curves for the thirteen buildings that make up the final suite of buildings have been demonstrated in this study to show that the maximum overstrength factor is more likely a function of a particular system characteristic (i.e., the design period or number of stories) rather than a constant value for a particular type of structural system (i.e., special moment frame system). Therefore, Equation 3.1.4.5-1 and Equation 5.2.2-3 were used to estimate a maximum overstrength factor, Ω_{max}, for each of the eighteen idealized buildings that range between 1-story through 18-stories. A value of 1.0 was used for the minimum overstrength factor, Ω_{min}, so that the lower limit of each envelope would be equal to the design value of the seismic base shear for each idealized building.

The upper bound base shear, V_{max}, and the lower bound base shear, V_{min}, were calculated for each idealized building system by multiplying their respective overstrength factor by the seismic design base shear value for the building. The seismic design base shear was determined for each of the eighteen idealized buildings by calculating a seismic response coefficient, C_s, from the building code seismic design provisions. Then this response coefficient was multiplied by the seismic weight of that building.

The roof displacement value, on each upper bound and lower bound envelope where the plateau portion begins and the sloped portion ends, was calculated by modifying Equation 6.2-6 and using the upper and lower bound base shear values and the upper and lower bound stiffness values for each idealized building.

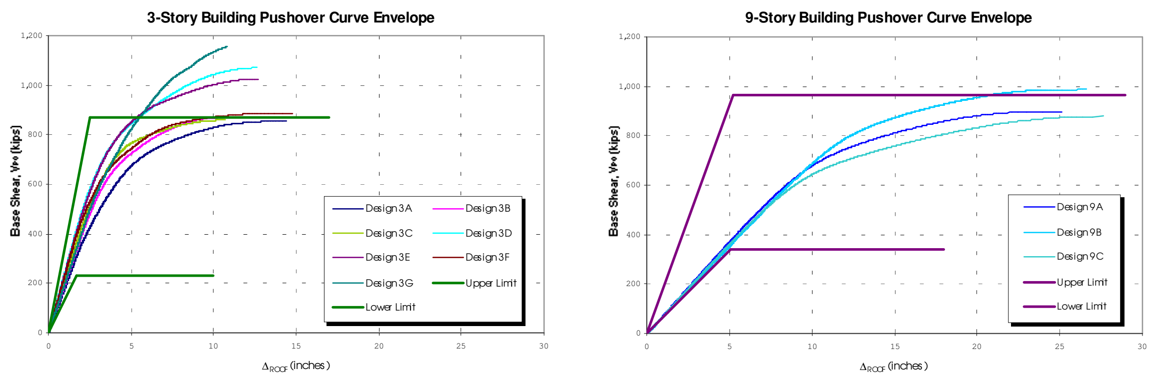
Once the maximum and minimum values of the four parameters (T, k, Ω, and V) were determined for each of the eighteen idealized building systems, an upper bound and lower bound curve was developed for the original three building heights that were used in this study – 3-story, 9-story, and 18-story. Table 6.2.1 lists the four parameters that were used in these three building height envelopes. As shown in Figure 6.2.3, these three envelopes allowed for a direct comparison to be made between the actual pushover curves (from Figure 5.2.1) of the thirteen buildings that make up the suite of buildings and their respective idealized envelope.

These three plots show how all of the thirteen pushover analysis curves fall within their individual idealized envelope of upper and lower bound limits. In most of the buildings it has been shown that the actual curves are closer to the lower bound sloped curve rather than the upper bound sloped curve. This demonstrates that these buildings were designed appropriately whereby their interstory drifts are near the code specified limit, which results in these curves being closer to the lower bound stiffness curve.

Pushover Curve Envelope Parameters	**3-Story Buildings**	**9-Story Buildings**	**18-Story Buildings**
Maximum Fundamental Period T_{max} *[seconds]*	1.18	2.95	4.87
Minimum Fundamental Period T_{min} *[seconds]*	0.74	1.77	3.08
Maximum Elastic Stiffness K_{max} *[kips/in]*	349	185	77
Minimum Elastic Stiffness K_{min} *[kips/in]*	136	67	***31***
Maximum Overstrength Factor Ω_{max}	3.77	2.84	2.31
Minimum Overstrength Factor Ω_{min}	1.00	1.00	1.00
Maximum Base Shear V_{max} *[kips]*	869	965	732
Minimum Base Shear V_{min} *[kips]*	231	340	316
Upper Bound Roof Drift Transition Point Δ_{max} *[inches]*	2.49	5.22	9.46
Lower Bound Roof Drift Transition Point Δ_{min} *[inches]*	1.70	5.11	10.21

Table 6.2.1: 3-Story, 9-Story, and 18-Story Upper and Lower Bound Envelope Curve Data Points Used in Figure 6.2.3

Note: Maximum and Minimum data points in Figure 6.2.6 have been Italicized and highlighted in Table 6.2.1 and Table 6.2.2.



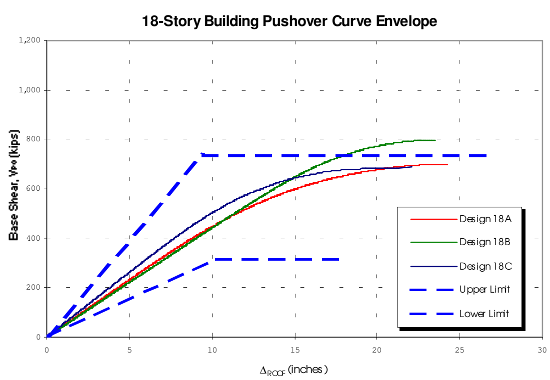


Figure 6.2.3: Actual Pushover Curves and Their Idealized Upper and Lower Limit Envelope for the 3-Story, 9-Story, and 18-Story Buildings

When all of the idealized envelopes are combined in one plot and the overall maximum upper bound limits and minimum lower bound limits were determined, an overall envelope of maximum and minimum limits was able to be set up to cover the building height limits of this study – low-rise to mid-rise buildings ranging between 1-story through 18-stories. The values of the four parameters (T, k, Ω, and V) that were used to construct the remaining fifteen idealized building envelope curves are listed in Table 6.2.2.

Figure 6.2.4 overlays some of the idealized building envelope curves for comparison purposes while the remaining curves were left out for clarity purposes. As can be see from this figure, there is a general trend in each idealized upper and lower bound limit that starts with the 1-story curves, it then increases until the 6-story curve produces the largest upper bound limit, and then the trend gradually decreases down to the 18-story curves.

The final overall envelope that approximates the maximum expected pushover analysis curve and the minimum expected pushover analysis curve for this study is a result of combining the maximum elastic stiffness with the maximum base shear and combining the minimum elastic stiffness with the minimum base shear from all eighteen idealized building systems (i.e., using the maximum and minimum data points from Table 6.2.1 and Table 6.2.2). The maximum base shear was calculated to be 1,003 kips (from the 4-story building) while the minimum base shear was 76 kips (from the 1-story building). The maximum elastic stiffness was 670 kips/in (from the 1-story building) while the minimum elastic stiffness was 31 kips/in (from the 18-story building). Using these maximum and minimum values, the transition point in the upper bound curve is located at a roof drift of 1.50 inches while the transition point in the lower bound curve is located at a roof drift of 2.45 inches. Figure 6.2.5 shows how these final data points were used to develop the overall envelope curves.

Pushover Curve Envelope Parameters	1-Story	2-Story	4-Story	5-Story	6-Story	7-Story	8-Story	10-Story	11-Story	12-Story	13-Story	14-Story	15-Story	16-Story	17-Story
Maximum Fundamental Period T_{max} ***[seconds]***	0.68	0.96	1.43	1.75	2.05	2.36	2.65	3.25	3.54	3.81	4.09	4.29	4.45	4.59	4.73
Minimum Fundamental Period T_{min} ***[seconds]***	0.31	0.53	0.93	1.11	1.28	1.45	1.61	1.93	2.08	2.23	2.38	2.52	2.66	2.80	2.94
Maximum Elastic Stiffness K_{max} ***[kips/in]***	***670***	444	295	259	233	213	198	167	151	137	124	113	103	94	85
Minimum Elastic Stiffness K_{min} ***[kips/in]***	134	135	124	104	91	80	73	59	52	47	42	39	37	35	33
Maximum Overstrength Factor Ω_{max}	4.28	4.00	3.56	3.38	3.22	3.08	2.95	2.74	2.65	2.57	2.51	2.45	2.40	2.36	2.34
Minimum Overstrength Factor Ω_{min}	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Maximum Base Shear V_{max} ***[kips]***	327	613	***1,003***	996	998	983	978	910	860	819	773	757	755	750	742
Minimum Base Shear V_{min} ***[kips]***	***76***	153	282	295	310	319	331	332	325	319	309	309	314	317	318
Upper Bound Roof Drift Transition Point Δ_{max} ***[inches]***	0.49	1.38	3.40	3.84	4.28	4.61	4.94	5.46	5.71	5.99	6.22	6.70	7.33	8.00	8.71
Lower Bound Roof Drift Transition Point Δ_{min} ***[inches]***	0.57	1.14	2.27	2.84	3.40	3.97	4.54	5.67	6.24	6.81	7.38	7.94	8.51	9.08	9.64

Table 6.2.2: Idealized Building Upper and Lower Bound Envelope Curve Data Points (the 3-Story, 9-Story, and 18-Story Data Points are in Table 6.2.1)

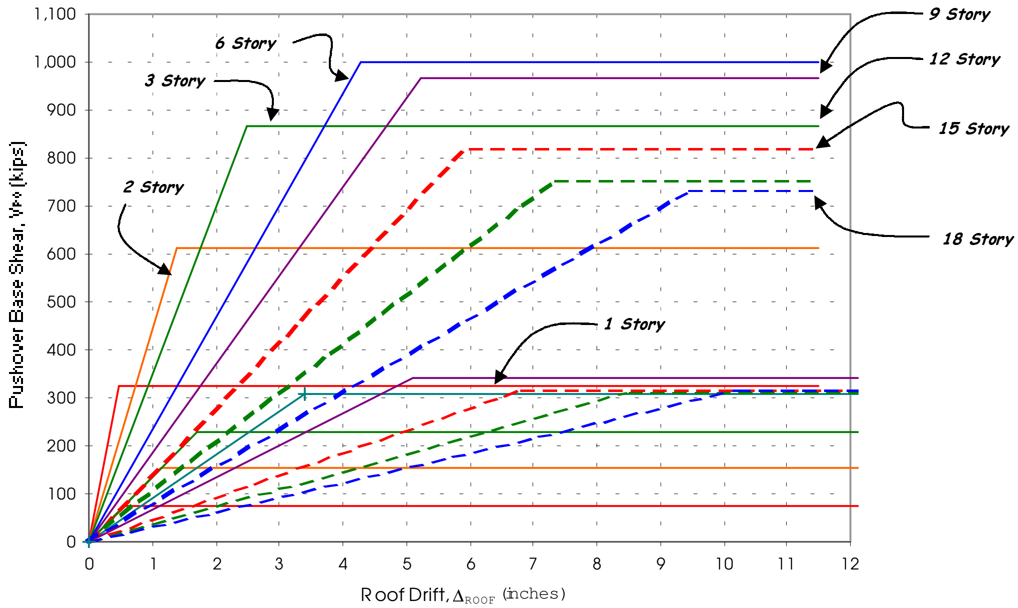


Figure 6.2.4: Upper and Lower Bound Envelopes for the 1, 2, 3, 6, 9, 12, 15, and 18-Story Idealized Building Systems (the Remaining Idealized Building System Envelopes Were Omitted for Clarity)

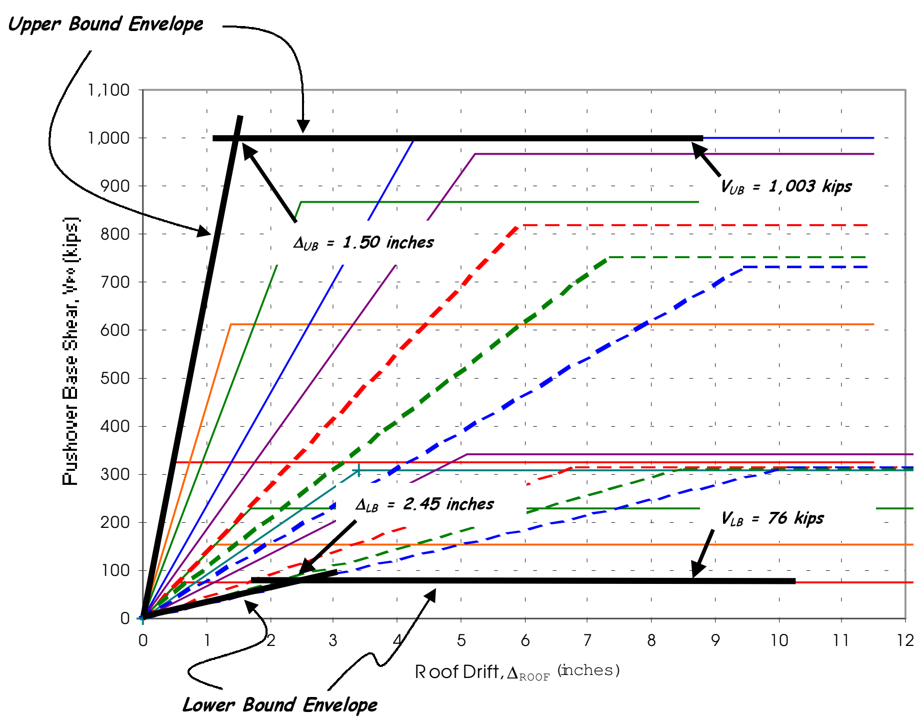


Figure 6.2.5: Final Upper and Lower Bound Envelope Compared to the Idealized Envelopes of the 1, 2, 3, 6, 9, 12, 15, and 18-Story Idealized Building Systems (the Remaining Idealized Building Systems Were Omitted for Clarity)

When the final overall envelope is put on the same plot as the pushover analysis curves for the actual 3-story, 9-story, and 18-story buildings, the thirteen building responses are shown to be located within this maximum and minimum envelope (reference Figure 6.2.6).

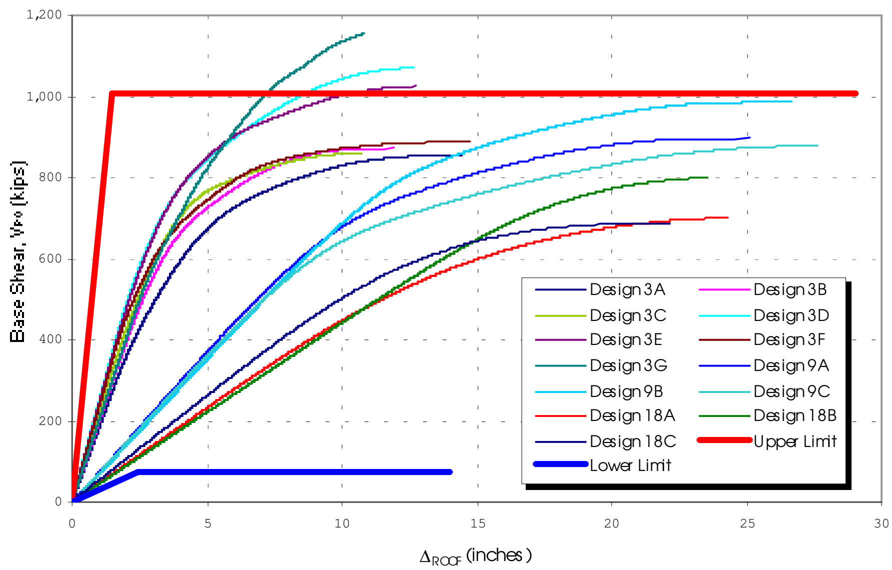


Figure 6.2.6: Final Upper and Lower Bound Envelopes With the Actual Pushover Analysis Curves of the 3-Story, 9-Story, and 18–Story Buildings

An upper bound and a lower bound envelope of idealized system response based on the height of the building was then determined for the thirteen normalized pushover curves. The normalized envelope is similar to the actual envelope of Figure 6.2.6, but there are a few differences in the parameters that were used to develop this envelope. These differences include the value of the slope of the sloped portion of each limit, k′, as well as the normalized force value of each curve.

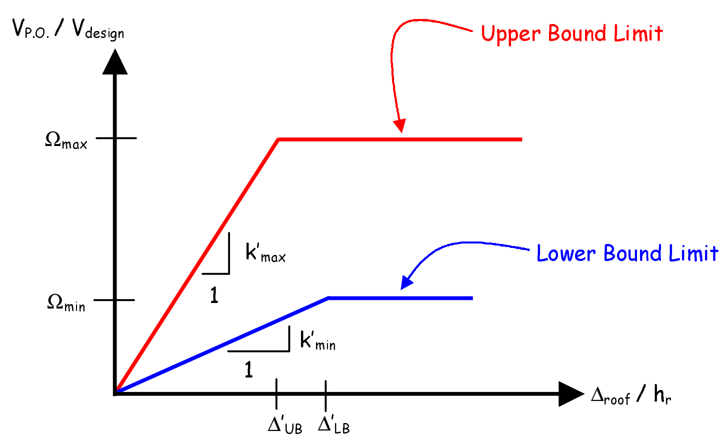


Figure 6.2.7: Idealized Building System Normalized Envelope

The normalized stiffness value, k′, of each curve is based on modifying the corresponding actual stiffness value, k, that was used in the envelope curves from Figure 6.2.1 and Equation 6.2-7, as shown in Equation 6.2-8.

$$k'_{max/min} = \left(k_{max/min}\right)\left(\frac{h_r}{V_{design}}\right) \tag{6.2-8}$$

When Equation 6.2-8 is used to calculate k'_{min} all buildings end up having the same value of normalized minimum stiffness. This is due to the fact that Equation 6.2-2 was used to calculate $T_{max,}$ which results in k'_{min} being equal to h_r divided by Δ_{roof}. Since the maximum allowed elastic roof drift is equal to $0.02h_r$ divided by C_d (as described in Section 3.3.3 of this study) k'_{min} is equal to 275 for all building systems in this study.

The plateau portion of each limit curve is the value of the normalized base shear, which is also known as the overstrength value. The value of the upper plateau of each idealized system is the value of the maximum overstrength value from Table 6.2.1 and Table 6.2.2. The value of the lower plateau is equal to an overstrength value of 1.0 for all building systems. Therefore, since the lower plateau is equal to 1.0 and k'_{min} is equal to 275 for all idealized building systems, there is only one normalized lower bound curve that is shared by all of the idealized building systems.

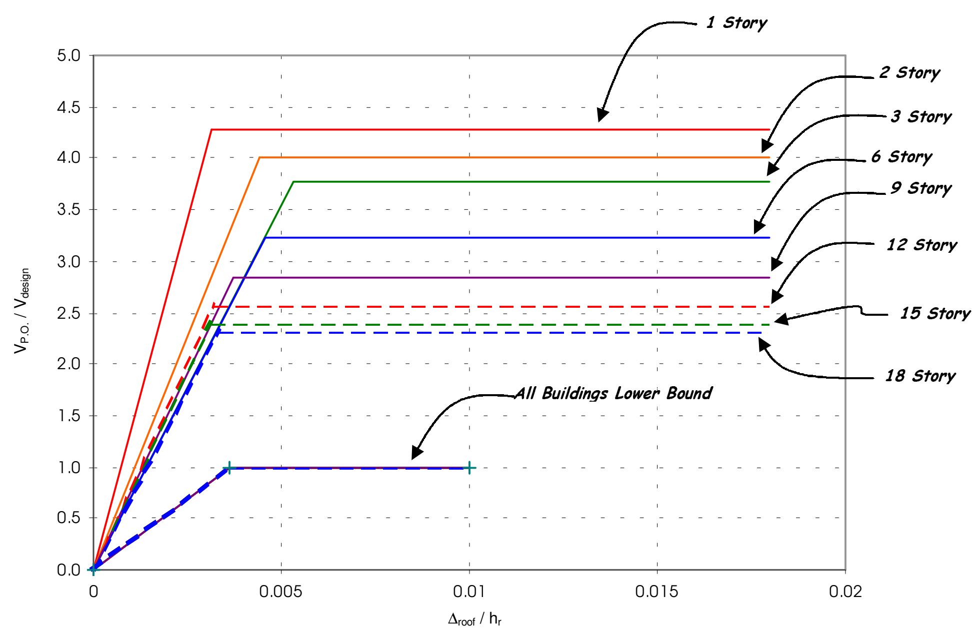


Figure 6.2.8: Upper and Lower Bound Normalized Envelopes for the 1, 2, 3, 6, 9, 12, 15, and 18-Story Idealized Building Systems (the Remaining Idealized Building System Envelopes Were Omitted for Clarity)

Figure 6.2.8 overlays some of the eighteen idealized normalized building curves for comparison purposes while the remaining curves were left out for clarity purposes. As can be seen from this figure, there is a general trend in each idealized upper and lower bound limit that starts with the 1-story curve and then decreases until the 18-story curve is shown.

The final overall normalized envelope shown in Figure 6.2.9 approximates the maximum expected normalized pushover analysis curve and the minimum expected normalized pushover analysis curve for this study. This envelope is a result of combining the maximum normalized elastic stiffness with the maximum overstrength value and combining the minimum normalized elastic stiffness with the minimum overstrength from all eighteen idealized building systems as shown in Figure 6.2.8. The maximum elastic normalized stiffness was 1,370 (from the 1-story building) while all buildings had a minimum normalized stiffness value of 275. Using these maximum and minimum values, the transition point in the upper bound curve is located at a normalized roof drift of 0.0031 while the transition point in the lower bound curve is located at a normalized roof drift of 0.0036.

Figure 6.2.9 illustrates how these final data points were used to develop the overall normalized envelope curves. By comparing the actual pushover analysis curves of each of the thirteen buildings with their actual and normalized envelope curves, it can be seen that overall these thirteen buildings are able to provide a wide range of building responses.

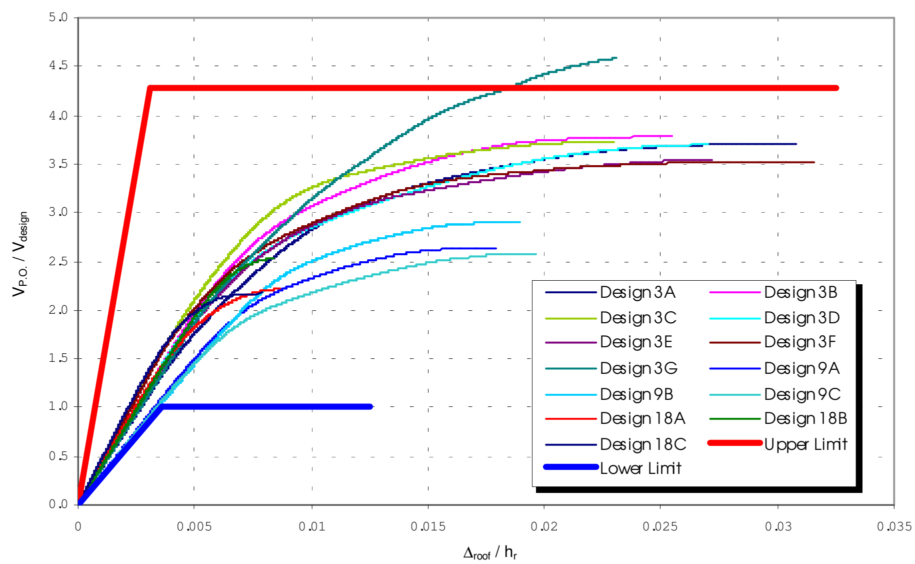


Figure 6.2.9: Final Upper and Lower Bound Normalized Envelopes With the Actual Normalized Pushover Analysis Curves of the 3-Story, 9-Story, and 18–Story Buildings

6.3 Method 3: *System Overstrength Factor Verification*

The buildings in this study were designed to be as economical as possible (i.e., they have the smallest column and girder sections) while maintaining the building code interstory drift limits, the AISC material design strength limits, the d/t limits, and the AISC Seismic Provisions SC/WB limit. However, each building will have an inherent system capacity that will be larger than what would result if the buildings were designed to precisely meet the minimum design limits. This additional capacity is due to the availability of member sizes, actual material strengths will be larger than the actual design values, redundancies within the building system are not taken into account in the 2D pushover analysis, etc.

If the buildings that were used to develop the overstrength factors of Equations 5.2.2-2 through 5.2.2-4 were inadvertently designed with too much inherent strength, then the overstrength factor that was calculated for each building would be too large. This would affect the pushover analysis curves and capacity curves in that they would be shifted up in value, and the pushover analysis curve envelope would not be able to provide an accurate assessment of the buildings since it would also be based on inaccurate overstrength values. Therefore, the flexural rigidity ratio, η, was developed so that a relative measurement could be made between each building system in relation to a baseline set of building systems. The rigidity ratio was not correlated to the overstrength factor in this study, but rather it allows for consistency to be shown in each of the thirteen building designs when they are compared to their baseline building system.

The flexural rigidity ratio provides a way for comparing a RCFT moment-resisting frame to a set of baseline RCFT moment-resisting frames whose members were sized by ignoring availability of section sizes, and by not taking into account the idea of economy of scale. This allowed for these baseline RCFT building systems to have smaller and lighter sections than the thirteen buildings of this study, but still be within the same design limits.

To determine if a RCFT building is over designed, a comparison is made between its value of η and that of its baseline value of η. If the actual value of η is the same or slightly larger than its baseline value, the building design is considered appropriate to use in this study. If the value of η is smaller than its baseline value the building system is considered under designed, and if η is too large the building system is considered over designed.

The first step in using the flexural rigidity ratio is to determine the building-based value of η for the moment-resisting frame that is being checked. η can be calculated in numerous ways, nine of which are presented in this study. Equation 6.3-1 shows how η is first calculated for every story, i, of a building. A single value of η is then calculated for the building by taking the mean value from all of the story-based values.

Equation 6.3-2 provides a second method for calculating a building-based value of η by ignoring the specific story-based values and using mean values from every story in the building. These two approaches result in a difference of 5% in building-based values of η and are the basic ways to calculate η. The specific story-based values of η_1 and the total building-based value of η_1 as well as the building-based value of η_2 are listed in Table 6.3.1 and Table 6.3.2, respectively, for each of the thirteen buildings in this study.

$$\eta_1 = \left[\frac{\Sigma EI_{eff}}{(\Sigma EI)_a + (\Sigma EI)_b} \right]_i \tag{6.3-1}$$

$$\eta_2 = \left[\frac{(\Sigma EI_{eff})_{ave}}{2(\Sigma EI)_{ave}} \right]_{bldg} \tag{6.3-2}$$

Where: i = story number
EI_{eff} = flexural rigidity of a composite column
$(\Sigma EI)_a$ = total flexural rigidity of the girders *above* story i
$(\Sigma EI)_b$ = total flexural rigidity of the girders *below* story i
$(\Sigma EI)_{ave}$ = average of all of the story summations

Equations 6.3-3 through 6.3-9 were developed with the notion that if Equations 6.3-1 and 6.3–2 were broken out into more basic components of the RCFT structural system [i.e., individual geometries (d_c, d_g, L, H, etc.), and material properties (F_y, E_s, E_c, f'_c, etc.)] a more accurate method for determining η could be developed. Overall the value of η did not vary significantly between these nine different methods. By providing numerous methods for calculating η a check of the structural system can be made at anytime during the structural design process since any number of design parameters can now be used to calculate η, rather than limiting the process to only Equation 6.3-1 or Equation 6.3-2.

The design parameters that are used in Equation 6.3-1 through Equation 6.3-9 are as follows:

B	number of bays in the story
d_c	column depth
d_g	girder depth
d_{ga}	depth of girder *above* the story
d_{gb}	depth of girder *below* the story
E_s	modulus of elasticity of steel = 29,000 ksi
E_c	modulus of elasticity of concrete = $w_c^{1.5}\sqrt{f'_c}$
E'_c	modified modulus of elasticity of concrete
EI_{eff}	effective flexural rigidity of a composite column
f'_c	minimum concrete compressive strength
F_{yg}	minimum girder yield strength
F_{yc}	minimum column yield strength
H	story height; column length
I_g	girder moment of inertia
I_{c_s}	column moment of inertia of the steel HSS portion
K	story stiffness
ΣK_{col}	$\Sigma(EI_{eff} / H)$
L	girder length
M_{pc}	column plastic moment
M_{pg}	girder plastic moment
R_y	expected yield strength factor
t	nominal thickness of the HSS column wall
w_c	density of concrete = 145 lb/ft³

Flexural Rigidity Ratio η_1													
Story	**Building Design Number**												
	3A	**3B**	**3C**	**3D**	**3E**	**3F**	**3G**	**9A**	**9B**	**9C**	**18A**	**18B**	**18C**
18	---	---	---	---	---	---	---	---	---	---	1.32	0.38	1.64
17	---	---	---	---	---	---	---	---	---	---	1.32	0.38	1.64
16	---	---	---	---	---	---	---	---	---	---	1.06	0.38	1.64
15	---	---	---	---	---	---	---	---	---	---	0.99	0.55	3.13
14	---	---	---	---	---	---	---	---	---	---	0.94	0.48	2.19
13	---	---	---	---	---	---	---	---	---	---	0.84	0.63	1.69
12	---	---	---	---	---	---	---	---	---	---	0.87	0.59	2.00
11	---	---	---	---	---	---	---	---	---	---	0.87	0.55	2.00
10	---	---	---	---	---	---	---	---	---	---	1.01	0.58	2.00
9	---	---	---	---	---	---	---	0.98	0.57	1.66	0.95	0.55	2.00
8	---	---	---	---	---	---	---	0.92	0.53	1.66	0.95	0.55	2.44
7	---	---	---	---	---	---	---	0.90	0.51	1.67	0.95	0.55	2.21
6	---	---	---	---	---	---	---	0.99	0.57	2.16	0.95	0.55	2.21
5	---	---	---	---	---	---	---	0.99	0.51	1.85	0.95	0.55	2.21
4	---	---	---	---	---	---	---	1.00	0.54	2.23	0.95	0.55	2.21
3	1.48	0.92	1.39	1.49	1.29	1.25	1.26	0.89	0.52	2.23	0.95	0.55	2.21
2	1.20	0.68	0.99	1.14	0.98	0.89	0.93	0.89	0.52	2.23	0.95	0.55	2.21
1	1.20	0.68	0.99	1.14	0.98	0.89	0.93	0.89	0.52	2.23	0.95	0.55	2.21
AVE	***1.29***	***0.76***	***1.12***	***1.26***	***1.08***	***1.01***	***1.04***	***0.94***	***0.53***	***1.99***	***0.99***	***0.53***	***2.10***

Table 6.3.1: Values of η_1 per Story in Each Building

Flexural Rigidity Ratio η_2													
Story	**Building Design Number**												
	3A	**3B**	**3C**	**3D**	**3E**	**3F**	**3G**	**9A**	**9B**	**9C**	**18A**	**18B**	**18C**
AVE	***1.37***	***0.82***	***1.22***	***1.35***	***1.16***	***1.10***	***1.13***	***0.96***	***0.55***	***2.08***	***0.98***	***0.54***	***2.16***

Table 6.3.2: Values of η_2 per Building

$$\eta_3 = \left(\frac{B+1}{B}\right)\left[\left(\frac{d_c}{d_g}\right)\left(\frac{R_y F_{yg}}{F_{yc}}\right)\left(\frac{0.6EI_{eff}}{E_s I_{c\_s}}\right)\right]_i \qquad (6.3\text{-}3)$$

Where: d_c = average column depth for story i
d_g = average girder depth for story i = average of $\Sigma d_{g\_a} + \Sigma d_{g\_b}$
EI_{eff} = average effective flexural rigidity for story i
I_{c_s} = average column moment of inertia for story i

Flexural Rigidity Ratio η_3													
Story	Building Design Number												
	3A	3B	3C	3D	3E	3F	3G	9A	9B	9C	18A	18B	18C
18	---	---	---	---	---	---	---	---	---	---	0.93	0.38	1.90
17	---	---	---	---	---	---	---	---	---	---	0.93	0.38	1.90
16	---	---	---	---	---	---	---	---	---	---	0.86	0.38	1.90
15	---	---	---	---	---	---	---	---	---	---	1.01	0.44	2.14
14	---	---	---	---	---	---	---	---	---	---	1.00	0.41	1.97
13	---	---	---	---	---	---	---	---	---	---	0.95	0.57	1.82
12	---	---	---	---	---	---	---	---	---	---	0.86	0.57	1.96
11	---	---	---	---	---	---	---	---	---	---	0.86	0.57	1.96
10	---	---	---	---	---	---	---	---	---	---	1.01	0.52	1.96
9	---	---	---	---	---	---	---	1.04	0.58	1.79	1.01	0.52	1.96
8	---	---	---	---	---	---	---	1.04	0.58	1.79	1.01	0.52	2.11
7	---	---	---	---	---	---	---	0.94	0.51	1.63	1.01	0.52	1.99
6	---	---	---	---	---	---	---	1.05	0.57	2.16	1.01	0.52	1.99
5	---	---	---	---	---	---	---	1.05	0.55	2.04	1.01	0.52	1.99
4	---	---	---	---	---	---	---	0.95	0.51	2.19	1.01	0.52	1.99
3	1.28	0.67	1.14	1.22	1.04	0.98	1.06	0.90	0.51	2.19	1.01	0.52	1.99
2	1.43	0.63	1.06	1.28	1.09	0.90	1.01	0.90	0.51	2.19	1.01	0.52	1.99
1	1.43	0.63	1.06	1.28	1.09	0.90	1.01	0.90	0.51	2.19	1.01	0.52	1.99
AVE	***1.38***	***0.64***	***1.09***	***1.26***	***1.07***	***0.93***	***1.02***	***0.97***	***0.54***	***2.02***	***0.97***	***0.49***	***1.97***

Table 6.3.3: Values of η_3 per Story in Each Building

$$\eta_4 = \left(\Sigma K_{col}\right)_i \left(\frac{6}{KHL_{ave}}\right)_i - \frac{1}{2}\left(\frac{H}{L_{ave}}\right)_i \tag{6.3-4}$$

Where: $\Sigma K_{col} = \Sigma(EI_{eff} / H)$

Flexural Rigidity Ratio η_4													
Story	**Building Design Number**												
	3A	**3B**	**3C**	**3D**	**3E**	**3F**	**3G**	**9A**	**9B**	**9C**	**18A**	**18B**	**18C**
18	---	---	---	---	---	---	---	---	---	---	1.94	0.57	2.21
17	---	---	---	---	---	---	---	---	---	---	1.44	0.50	1.76
16	---	---	---	---	---	---	---	---	---	---	1.18	0.44	1.66
15	---	---	---	---	---	---	---	---	---	---	1.16	0.66	2.95
14	---	---	---	---	---	---	---	---	---	---	1.06	0.56	2.30
13	---	---	---	---	---	---	---	---	---	---	0.97	0.73	1.90
12	---	---	---	---	---	---	---	---	---	---	1.01	0.68	2.15
11	---	---	---	---	---	---	---	---	---	---	0.96	0.64	2.09
10	---	---	---	---	---	---	---	---	---	---	1.13	0.66	2.07
9	---	---	---	---	---	---	---	1.07	0.58	1.75	1.07	0.63	2.00
8	---	---	---	---	---	---	---	0.86	0.53	1.58	1.07	0.61	2.55
7	---	---	---	---	---	---	---	0.91	0.51	1.58	1.03	0.62	2.37
6	---	---	---	---	---	---	---	1.02	0.60	2.05	1.02	0.60	2.27
5	---	---	---	---	---	---	---	0.96	0.53	1.82	1.03	0.60	2.28
4	---	---	---	---	---	---	---	1.02	0.56	2.22	1.02	0.59	2.26
3	1.02	0.76	0.92	1.19	1.10	1.42	1.36	0.90	0.52	2.02	0.99	0.57	2.11
2	0.96	0.63	0.87	0.93	0.83	0.86	0.83	0.79	0.49	1.66	0.89	0.56	1.87
1	---	---	---	---	---	---	---	---	---	---	---	---	---
AVE	***0.99***	***0.69***	***0.89***	***1.06***	***0.96***	***1.14***	***1.10***	***0.94***	***0.54***	***1.84***	***1.11***	***0.60***	***2.17***

Table 6.3.4: Values of η_4 per Story in Each Building

$$\eta_5 = \left(\frac{B+1}{B}\right)\left[\left(\frac{E_c^{'}}{E_s}\right)\left(\frac{d_c^4}{23I_g}\right)+\left(\frac{I_{c\_s}}{2I_g}\right)\left(1-\frac{E_c^{'}}{E_s}\right)\right]_i \qquad (6.3\text{-}5)$$

Where: $E'_c = \min[\ (0.6 + 8t/d_c)$ or $0.9\ ]E_c$
d_c = average column depth for story i
I_{c_s} = average column moment of inertia for story i
I_g = average girder moment of inertia above and below story i

Flexural Rigidity Ratio η_5													
Story	**Building Design Number**												
	3A	**3B**	**3C**	**3D**	**3E**	**3F**	**3G**	**9A**	**9B**	**9C**	**18A**	**18B**	**18C**
18	---	---	---	---	---	---	---	---	---	---	1.34	0.39	1.69
17	---	---	---	---	---	---	---	---	---	---	1.34	0.39	1.69
16	---	---	---	---	---	---	---	---	---	---	1.07	0.39	1.69
15	---	---	---	---	---	---	---	---	---	---	1.01	0.57	3.22
14	---	---	---	---	---	---	---	---	---	---	0.96	0.49	2.26
13	---	---	---	---	---	---	---	---	---	---	0.86	0.66	1.74
12	---	---	---	---	---	---	---	---	---	---	0.88	0.61	2.06
11	---	---	---	---	---	---	---	---	---	---	0.88	0.57	2.06
10	---	---	---	---	---	---	---	---	---	---	1.03	0.60	2.06
9	---	---	---	---	---	---	---	0.99	0.59	1.71	0.97	0.57	2.06
8	---	---	---	---	---	---	---	0.93	0.55	1.71	0.97	0.57	2.52
7	---	---	---	---	---	---	---	0.91	0.53	1.72	0.97	0.57	2.28
6	---	---	---	---	---	---	---	1.01	0.59	2.22	0.97	0.57	2.28
5	---	---	---	---	---	---	---	1.01	0.53	1.90	0.97	0.57	2.28
4	---	---	---	---	---	---	---	1.01	0.56	2.30	0.97	0.57	2.28
3	1.07	0.94	1.43	1.52	1.32	1.27	1.31	0.91	0.53	2.30	0.97	0.57	2.28
2	0.87	0.70	1.02	1.15	1.01	0.90	0.97	0.91	0.53	2.30	0.97	0.57	2.28
1	0.87	0.70	1.02	1.15	1.01	0.90	0.97	0.91	0.53	2.30	0.97	0.57	2.28
AVE	***0.94***	***0.78***	***1.16***	***1.28***	***1.11***	***1.02***	***1.08***	***0.95***	***0.55***	***2.05***	***1.00***	***0.54***	***2.17***

Table 6.3.5: Values of η_5 per Story in Each Building

$$\eta_6 = \left(\frac{B+1}{B}\right)\left\{\left(\frac{M_{pc}}{M_{pg}}\right)\left(\frac{R_y F_{yg}}{F_{yc}}\right)\left(\frac{1}{2E_s d_g}\right)\left[\frac{E_c^{'} M_{pc}}{5(1.4t)^2 F_{yc}} + d_c\left(E_s - E_c^{'}\right)\right]\right\}_i (RF) \qquad (6.3\text{-}6)$$

Where: d_c = average column depth for story I
d_g = average girder depth for story i = average of $\Sigma d_{g\_a} + \Sigma d_{g\_b}$
$E'_c = \min[\ (0.6 + 8t/d_c)$ or $0.9\ ]E_c$
RF = reduction factor per Table 6.3.6

F_{yc}	f'_c	RF
46	4	0.20
80	16	0.30
50	16	0.35

Table 6.3.6: Reduction Factor, RF, per Material Property

Flexural Rigidity Ratio η_6													
Story	Building Design Number												
	3A	3B	3C	3D	3E	3F	3G	9A	9B	9C	18A	18B	18C
18	---	---	---	---	---	---	---	---	---	---	0.83	0.35	2.00
17	---	---	---	---	---	---	---	---	---	---	0.83	0.35	2.00
16	---	---	---	---	---	---	---	---	---	---	0.83	0.35	2.00
15	---	---	---	---	---	---	---	---	---	---	1.02	0.45	2.22
14	---	---	---	---	---	---	---	---	---	---	1.01	0.40	2.22
13	---	---	---	---	---	---	---	---	---	---	1.01	0.64	1.89
12	---	---	---	---	---	---	---	---	---	---	0.82	0.64	2.05
11	---	---	---	---	---	---	---	---	---	---	0.82	0.63	2.05
10	---	---	---	---	---	---	---	---	---	---	1.05	0.56	2.05
9	---	---	---	---	---	---	---	1.00	0.58	1.74	1.04	0.56	2.05
8	---	---	---	---	---	---	---	1.00	0.58	1.74	1.04	0.56	2.37
7	---	---	---	---	---	---	---	0.91	0.51	1.54	1.04	0.56	2.11
6	---	---	---	---	---	---	---	1.04	0.60	2.14	1.04	0.56	2.11
5	---	---	---	---	---	---	---	1.04	0.54	2.12	1.04	0.56	2.11
4	---	---	---	---	---	---	---	0.94	0.49	2.18	1.04	0.56	2.11
3	1.12	0.82	1.24	1.17	1.00	1.13	1.10	0.85	0.49	2.18	1.04	0.56	2.11
2	1.39	0.72	1.07	1.29	1.10	0.96	1.01	0.85	0.49	2.18	1.04	0.56	2.11
1	1.39	0.72	1.07	1.29	1.10	0.96	1.01	0.85	0.49	2.18	1.04	0.56	2.11
AVE	***1.30***	***0.76***	***1.13***	***1.25***	***1.06***	***1.01***	***1.04***	***0.94***	***0.53***	***2.00***	***0.98***	***0.52***	***2.09***

Table 6.3.7: Values of η_6 per Story in Each Building

$$\eta_7 = (B+1)\left[d_c^{\,3}\left(E_c^{'}\left(\frac{d_c}{12}\right) + 0.6t\left(E_s - E_c^{'}\right)\right)\left(\frac{6}{KH^2 L_{ave}}\right)\right]_i - \frac{1}{2}\left(\frac{H}{L_{ave}}\right)_i \qquad (6.3\text{-}7)$$

Where: $E'_c = \min[\,(0.6 + 8t/d_c)\text{ or } 0.9\,]E_c$
d_c = average column depth for story i

Flexural Rigidity Ratio η_7													
Story	**Building Design Number**												
	3A	**3B**	**3C**	**3D**	**3E**	**3F**	**3G**	**9A**	**9B**	**9C**	**18A**	**18B**	**18C**
18	---	---	---	---	---	---	---	---	---	---	2.20	0.70	2.18
17	---	---	---	---	---	---	---	---	---	---	1.64	0.61	1.73
16	---	---	---	---	---	---	---	---	---	---	1.35	0.54	1.64
15	---	---	---	---	---	---	---	---	---	---	1.19	0.77	2.92
14	---	---	---	---	---	---	---	---	---	---	1.09	0.65	2.27
13	---	---	---	---	---	---	---	---	---	---	0.99	0.78	1.88
12	---	---	---	---	---	---	---	---	---	---	1.08	0.74	2.12
11	---	---	---	---	---	---	---	---	---	---	1.03	0.69	2.07
10	---	---	---	---	---	---	---	---	---	---	1.15	0.73	2.04
9	---	---	---	---	---	---	---	1.08	0.62	1.73	1.08	0.70	1.97
8	---	---	---	---	---	---	---	0.88	0.57	1.56	1.08	0.68	2.51
7	---	---	---	---	---	---	---	0.95	0.57	1.56	1.04	0.69	2.34
6	---	---	---	---	---	---	---	1.03	0.61	2.02	1.04	0.67	2.24
5	---	---	---	---	---	---	---	0.96	0.54	1.79	1.04	0.67	2.25
4	---	---	---	---	---	---	---	1.06	0.59	2.19	1.03	0.66	2.22
3	1.04	0.77	0.91	1.23	1.09	1.53	1.45	0.93	0.55	1.99	1.00	0.64	2.07
2	0.98	0.65	0.86	0.97	0.82	0.94	0.89	0.82	0.52	1.63	0.91	0.62	1.84
1	0.73	0.43	0.65	0.63	0.50	0.48	0.43	0.37	0.25	0.70	0.40	0.29	0.76
AVE	***0.92***	***0.61***	***0.80***	***0.94***	***0.81***	***0.98***	***0.93***	***0.90***	***0.54***	***1.69***	***1.13***	***0.66***	***2.06***

Table 6.3.8: Values of η_7 per Story in Each Building

$$\eta_8 = \left(\frac{6\Sigma EI_{eff} - KH^3/2}{KH^2L_{ave}} \right)_i \qquad (6.3\text{-}8)$$

Where: EI_{eff} = average effective flexural rigidity for story i

Flexural Rigidity Ratio η_8													
Story	Building Design Number												
	3A	3B	3C	3D	3E	3F	3G	9A	9B	9C	18A	18B	18C
18	---	---	---	---	---	---	---	---	---	---	1.94	0.57	2.21
17	---	---	---	---	---	---	---	---	---	---	1.44	0.50	1.76
16	---	---	---	---	---	---	---	---	---	---	1.18	0.44	1.66
15	---	---	---	---	---	---	---	---	---	---	1.16	0.66	2.95
14	---	---	---	---	---	---	---	---	---	---	1.06	0.56	2.30
13	---	---	---	---	---	---	---	---	---	---	0.97	0.73	1.90
12	---	---	---	---	---	---	---	---	---	---	1.01	0.68	2.15
11	---	---	---	---	---	---	---	---	---	---	0.96	0.64	2.09
10	---	---	---	---	---	---	---	---	---	---	1.13	0.66	2.07
9	---	---	---	---	---	---	---	1.07	0.58	1.75	1.07	0.63	2.00
8	---	---	---	---	---	---	---	0.86	0.53	1.58	1.07	0.61	2.55
7	---	---	---	---	---	---	---	0.91	0.51	1.58	1.03	0.62	2.37
6	---	---	---	---	---	---	---	1.02	0.60	2.05	1.02	0.60	2.27
5	---	---	---	---	---	---	---	0.96	0.53	1.82	1.03	0.60	2.28
4	---	---	---	---	---	---	---	1.02	0.56	2.22	1.02	0.59	2.26
3	1.02	0.76	0.92	1.19	1.10	1.42	1.36	0.90	0.52	2.02	0.99	0.57	2.11
2	0.96	0.63	0.87	0.93	0.83	0.86	0.83	0.79	0.49	1.66	0.89	0.56	1.87
1	---	---	---	---	---	---	---	---	---	---	---	---	---
AVE	***0.99***	***0.69***	***0.89***	***1.06***	***0.96***	***1.14***	***1.10***	***0.94***	***0.54***	***1.84***	***1.11***	***0.60***	***2.17***

Table 6.3.9: Values of η_8 per Story in Each Building

$$\eta_9 = \left(\frac{B+1}{B}\right)\left\{\left(\frac{0.6}{E_s}\right)\left(\frac{R_y F_{yg}}{F_{yc}}\right)\left(\frac{d_c}{d_g}\right)\left[\frac{E_c'}{7}\left(\frac{d_c}{t}\right)+\left(E_s - E_c'\right)\right]\right\}_i \tag{6.3-9}$$

Where: $E'_c = \min[\ (0.6 + 8t/d_c)\text{ or } 0.9\ ]E_c$
d_c = average column depth for story i
d_g = average girder depth for story i

Flexural Rigidity Ratio η_9													
Story	Building Design Number												
	3A	3B	3C	3D	3E	3F	3G	9A	9B	9C	18A	18B	18C
18	---	---	---	---	---	---	---	---	---	---	0.90	0.35	2.00
17	---	---	---	---	---	---	---	---	---	---	0.90	0.35	2.00
16	---	---	---	---	---	---	---	---	---	---	0.90	0.35	2.00
15	---	---	---	---	---	---	---	---	---	---	1.01	0.44	2.24
14	---	---	---	---	---	---	---	---	---	---	1.01	0.39	2.24
13	---	---	---	---	---	---	---	---	---	---	1.01	0.56	1.90
12	---	---	---	---	---	---	---	---	---	---	0.84	0.56	2.05
11	---	---	---	---	---	---	---	---	---	---	0.84	0.56	2.05
10	---	---	---	---	---	---	---	---	---	---	1.03	0.51	2.05
9	---	---	---	---	---	---	---	1.05	0.57	1.88	1.02	0.50	2.05
8	---	---	---	---	---	---	---	1.05	0.57	1.88	1.02	0.50	2.35
7	---	---	---	---	---	---	---	0.98	0.52	1.70	1.02	0.50	2.10
6	---	---	---	---	---	---	---	1.06	0.61	2.28	1.02	0.50	2.10
5	---	---	---	---	---	---	---	1.06	0.55	2.27	1.02	0.50	2.10
4	---	---	---	---	---	---	---	0.99	0.50	2.32	1.02	0.50	2.10
3	1.32	0.72	1.28	1.16	1.03	1.04	1.09	0.90	0.50	2.32	1.02	0.50	2.10
2	1.32	0.63	1.11	1.28	1.12	0.88	0.99	0.90	0.50	2.32	1.02	0.50	2.10
1	1.32	0.63	1.11	1.28	1.12	0.88	0.99	0.90	0.50	2.32	1.02	0.50	2.10
AVE	***1.32***	***0.66***	***1.17***	***1.24***	***1.09***	***0.94***	***1.03***	***0.99***	***0.54***	***2.14***	***0.98***	***0.48***	***2.09***

Table 6.3.10: Values of η_9 per Story in Each Building

The mean value of η from all nine methods of calculation has been listed in Table 6.3.11 for all thirteen buildings in this study. Table 6.3.11 illustrates the relationship between the mean values of η and the design values of F_{yc} and f'_c. The 3-story buildings have larger values of η compared to the 9-story and the 18-story buildings. Table 6.3.12 provides an overall value of η based on the number of stories in a building, and based on the material design strengths of the RCFT columns. The mean value of η is 0.78 in all of the high strength buildings (i.e., F_{yc} = 80 ksi and f'_c = 16 ksi), and 1.07 in all of the low strength buildings (i.e., F_{yc} = 46 ksi and f'_c = 4 ksi).

d/t	Material Strength (ksi)		Values of η												
	F_{yc}	f'_c	3A	3B	3C	3D	3E	3F	3G	9A	9B	9C	18A	18B	18C
LOW	46	4	1.17	---	---	1.19	---	1.03	---	0.95	---	---	1.03	---	---
	80	16	---	0.71	---	---	1.04	---	1.05	---	0.54	---	---	0.55	---
HIGH	50	16	---	---	---	---	---	---	---	---	---	1.96	---	---	2.11
	80	16	---	---	1.05	---	---	---	---	---	---	---	---	---	---

Table 6.3.11: Mean Flexural Rigidity Ratio, η, For Each Building Using Equation 6.3-1 Through Equation 6.3-9

Material Stress (ksi)		Values of η			
F_{yc}	f'_c	3-Story	9-Story	18-Story	***Mean Value***
46	4	1.13	0.95	1.03	***1.07***
80	16	0.93	0.54	0.55	***0.78***

Table 6.3.12: Mean Flexural Rigidity Ratio, η, According to the Number of Stories in the Building

The reason for the buildings made of high strength material having lower values of η compared to the buildings made of low strength materials is due to the behavior of a few of the design parameters, as well as due to the nature of the HSS steel column sections. When the material design strength of a column is increased, the column depth and or wall thickness can be reduced compared to what is required for a column made of lower strength materials. However, since the buildings in this study were controlled by interstory drift limits rather than member strength limits, the buildings with larger material strengths resulted in having smaller column sections even though they still needed to be as stiff as the lower strength buildings with the same number of stories. The only way to increase the stiffness of a building made of moment-resisting frames is to increase the column depth, HSS wall thickness, or to increase the girder sizes. Since the columns have already been made smaller with their increase in material strengths, the girders were increased in size. This resulted in the girder bending strength and flexural rigidities, EI_a and EI_b, to increase in value. Therefore, the buildings with higher strength column materials ended up having smaller column sizes and larger girder sizes compared to the lower strength buildings, which have larger column sizes and smaller girder sizes. The result is higher values of η for the lower strength buildings and lower values of η for the higher strength buildings.

After analyzing all thirteen buildings in this study and calculating a value of η for each building, it was determined that η does not vary significantly between any two buildings that have the same column design yield strength, F_{yc}, and concrete compressive strength, f'_c, even when they have a different number of stories. η only varies between buildings when the material design strengths are different regardless of the number of stories in each building. Therefore, the best way to categorize η between any two buildings is to use the design values F_{yc} and f'_c of the RCFT columns.

The second step in the flexural rigidity ratio analysis was to set up a database of baseline values of η so that the actual value of η could be compared against a corresponding baseline (i.e., target) value of η. A second set of moment-resisting frames was used to calculate the baseline values of η. The columns and girders in these frames were sized through an approximating method that allowed for each frame to adhere to the interstory drift limits and the AISC SC/WB provisions (i.e., the two main controlling factors in the original thirteen building designs). The column depths were also only allowed to vary by one-inch increments rather than in larger increments as the original thirteen building designs used, and there was no allowance for economy of scale in these frames.

Even though these moment frames were not analyzed using elastic analysis methods and AISC design strengths, a few representative buildings were analyzed so that this approximation method could be verified. These representative frames had AISC interaction values at or just less than 1.0, and their interstory drifts were within the interstory drift limits of the building code. Therefore, even though all of these moment frames were not designed as the original thirteen buildings were designed, this representative group of frames were checked and verified to pass the design limits and AISC provisions of this study.

Base Line Number	F_{yc} *[ksi]*	f'_c *[ksi]*
1	46	4
2	46	10
3	46	16
4	63	4
5	63	10
6	63	16
7	80	4
8	80	10
9	80	16

Table 6.3.13: Design Parameters for Each Baseline Value of η

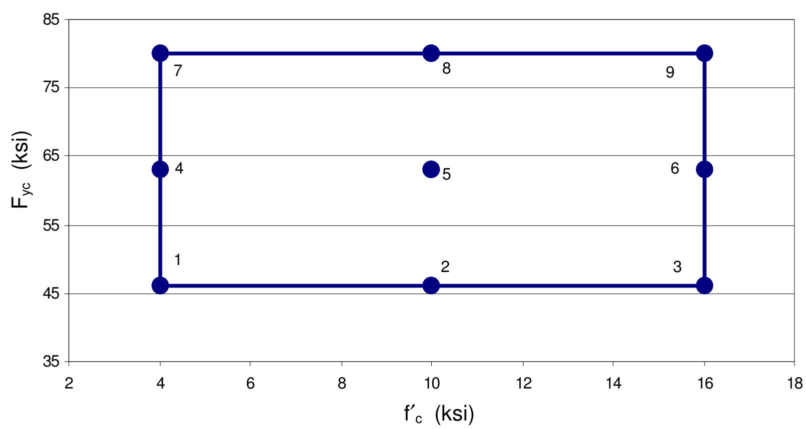


Figure 6.3.1: The 9 Baseline η Values in Relation to the Envelope of Possible Design Parameters F_{yc} and f'_c

The baseline set of moment-resisting frames used nine different combinations of column material strengths. These nine different pairs of material strengths, as shown in Table 6.3.13 and in Figure 6.3.1, represent the eight outer limit values plus the center value of the spectrum of possible material strengths that this study has been limited to use.

The columns and girders in each baseline moment-resisting frame were sized by approximating column and girder sizes for five different roof heights (three 3-story, one 9-story, and one 18-story) for each of the nine pairs of F_{yc} and f'_c material strengths. Then the mean value of the results of Equations 6.3-1 through 6.3-9 for each of these five buildings was calculated and used as the baseline value of η for each material strength pair. This process was repeated for all nine baseline pairs of F_{yc} and f'_c using low d/t ratios. The process was repeated a second time for d/t ratios near the AISC limit, and then a third time for d/t ratios just less than 80. These three d/t categories are the same ones that were used in the design of the original thirteen buildings of this study. For each of these three d/t categories 45 moment-resisting frame were sized. Overall 135 moment-resisting frames were sized so that each of the 27 baseline values of η could be calculated. Appendix M summarizes the steps that were followed to determine the final column and girder section sizes for each of these 135 moment-resisting frames.

By comparing the value of η from each of the thirteen original buildings to their respective baseline value, the inherent overstrength of each original building was estimated. With a way to measure relatively the potential inherent overstrength of each building, the overstrength factor values that were calculated using Equations 5.2.2-2 through 5.2.2-4 were evaluated to see if they were too conservative, or if they are appropriate overstrength factor design values to use in this study.

Table 6.3.14 shows the baseline values of η that were calculated using the 135 estimated moment-resisting frames. Three categories were set up to separate the rigidity ratios – 1) columns with low d/t; 2) columns with $d/t \leq 2.26\sqrt{(E/F_{yc})}$, the AISC limit; and 3) columns with $d/t \approx 80$.

LOW d/t			**$d/t \leq 2.26\sqrt{(E/F_{yc})}$**			**$d/t \approx 80$**		
F_{yc} *[ksi]*	**f'_c *[ksi]***	**AVE η**	**F_{yc} *[ksi]***	**f'_c *[ksi]***	**AVE η**	**F_{yc} *[ksi]***	**f'_c *[ksi]***	**AVE η**
46	4	**0.7**	46	4	**2.0**	46	4	**2.5**
46	10	**0.8**	46	10	**2.3**	46	10	**3.0**
46	16	**0.9**	46	16	**2.6**	46	16	**3.4**
63	4	**0.5**	63	4	**1.2**	63	4	**1.7**
63	10	**0.54**	63	10	**1.35**	63	10	**2.0**
63	16	**0.56**	63	16	**1.5**	63	16	**2.3**
80	4	**0.38**	80	4	**0.8**	80	4	**1.2**
80	10	**0.4**	80	10	**0.9**	80	10	**1.45**
80	16	**0.41**	80	16	**1.0**	80	16	**1.7**

Table 6.3.14: Baseline Values of η

Even though the same general pattern of decreasing values of η exists as the material strengths are increased, a new pattern emerged from the data points in Table 6.3.14. Within each of the three categories of rigidity ratios, whenever F_{yc} is kept constant and only f'_c is varied, the baseline value of η will also vary, but only in small increments. This pattern can be attributed to the fact that since f'_c does not affect the bending strength of the columns significantly and the girders are not affected when f'_c increases (or decreases) in strength. Therefore, the only effect of increasing f'_c while keeping F_{yc} constant is that the column flexural rigidity increases because EI_{eff} is increasing. However, η will only increase in small increments as f'_c increases since the rigidity of the girders does not change.

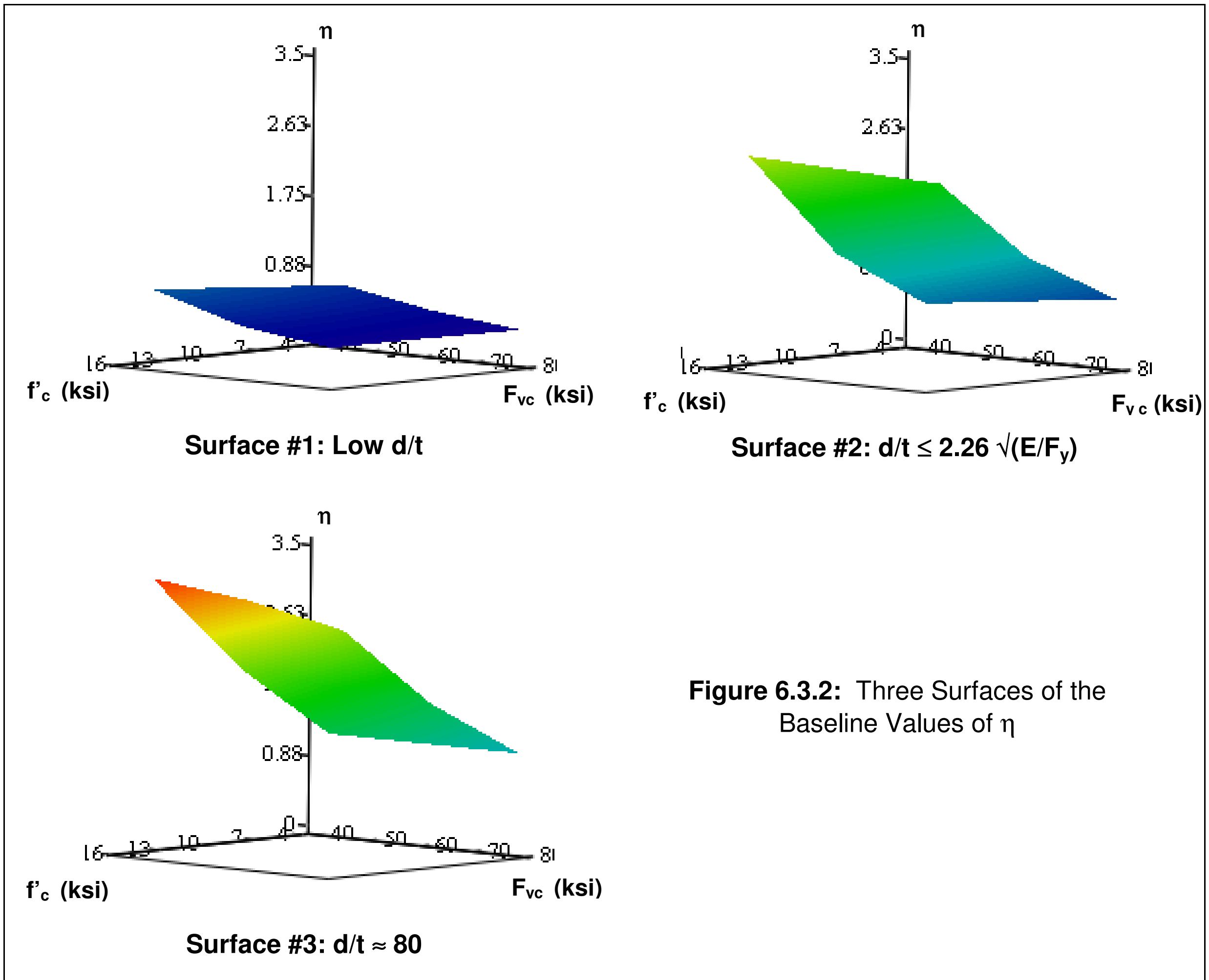


Figure 6.3.2: Three Surfaces of the Baseline Values of η

Figure 6.3.2 illustrates how the three baseline d/t categories of low d/t, maximum allowed d/t, and d/t ≈ 80 can be represented as surfaces in three-dimensional space by plotting the nine data points of each d/t category from Table 6.3.14. Surface #1 (low d/t) represents the most efficient building designs since the columns in these buildings are generally the smallest and lightest columns of the three d/t categories. Surface #2 (the maximum allowed d/t) represents values of η of buildings with d/t ratios that are at or just under the AISC d/t limit. Surface #3 (d/t ≈ 80) is made of η values from the buildings with large d/t ratios from this study. Equation 6.3-10 through Equation 6.3-12 describe these three 3D surfaces and were derived by curve fitting the data points of Table 6.3.14.

Low d/t ratios:

$$\begin{aligned} \eta &= a + b\left(f_c'\right) + c\left(f_c'^2\right) + d\left(F_{yc}\right) + e\left(F_{yc} f_c'\right) + f\left(F_{yc} f_c'^2\right) + \dots \\ &\dots + g\left(F_{yc}^2\right) + h\left(F_{yc}^2 f'\right)_c + i\left(F_{yc}^2 f_c'^2\right) \end{aligned} \tag{6.3-10}$$

Where:	a = 1.4473087273654830E+00	b = 3.7380815083001562E-02
	c = 2.8407343323402119E-03	d = -2.3266051516881974E-02
	e = -5.1662822029510842E-04	f = -9.4915417127780120E-05
	g = 1.2110726642213743E-04	h = 1.4417531747580099E-06
	i = 7.2087658578129920E-07	

d/t ≤ 2.26 √(E/F$_y$):

$$\eta = a + b\left(f_c'\right) + c\left(F_{yc}\right) + d\left(F_{yc} f_c'\right) + e\left(F_{yc}^2\right) + f\left(F_{yc}^2 f_c'\right) \tag{6.3-11}$$

Where:	a = 5.3653979238771816E+00	b = 2.0121107266420846E-01
	c = -1.0403690888124810E-01	d = -4.6136101499386212E-03
	e = 5.7670126874282898E-04	f = 2.8835063437135838E-05

d/t ≈ 80:

$$\begin{aligned} \eta &= a + b\left(f_c'\right) + c\left(f_c'^2\right) + d\left(F_{yc}\right) + e\left(F_{yc} f_c'\right) + f\left(F_{yc} f_c'^2\right) + \dots \\ &\dots + g\left(F_{yc}^2\right) + h\left(F_{yc}^2 f_c'\right) + i\left(F_{yc}^2 f_c'^2\right) \end{aligned} \tag{6.3-12}$$

Where:	a = 4.4889273350525487E+00	b = 4.6842560568455516E-01
	c = -1.2110726651196217E-02	d = -6.3187235656542093E-02
	e = -1.1485966940532069E-02	f = 3.4361783953559236E-04
	g = 2.4990388294893261E-04	h = 7.6893502535259090E-05
	i = -2.4029219549341513E-06	

Equation 6.3-13 is a simplified version of Equations 6.3-10 through 6.3-12. This equation is possible because the values of η do not vary significantly when F_{yc} is held constant. Therefore, by taking average values of η from Table 6.3.14 for each value of the design yield strength F_{yc}, an estimated value of η can be determined for a RCFT moment-resisting frame depending on how large of a d/t ratio is required.

$$\eta = a\, F_{yc}^2 + b\left[\ln\left(F_{yc}\right)\right] + \frac{c}{F_{yc}^2} \qquad (6.3\text{-}13)$$

Low d/t ratios: a = -3.19610427886004E-05 b = 1.01948679817606E-01
c = 1.00997659850529E+03

d/t ≤ 2.26 $\sqrt{(E/F_y)}$: a = -2.9533313486192E-05 b = 9.88113400161356E-02
c = 4.19852340833987E+03

d/t ≈ 80: a = -9.12010516359736E-05 b = 3.13847915123324E-01
c = 4.21373956584290E+03

Another parameter that does not vary significantly when F_{yc} is held constant is the average value of d/t for all of the columns in the moment-resisting frame. Based on the 135 moment-resisting frames that were used to calibrate the baseline values of η in this study, a relationship was established between the average d/t ratio for a moment frame and the design yield strength of the columns.

The values of d/t that were used to derive Equation 6.3-14 are shown in Table 6.3.15. Equation 6.3-14, which is based on the Ramberg-Osgood model, is able to estimate the required d/t ratio for any column in a moment frame depending on what the target value of the overall building value of η has been set to. If the building is being designed to have an overall η value at or just above Surface #1 (per Figure 6.3.2) then the variable "a", "b", and "c" for the "low d/t" category shall be used in Equation 6.3-14.

LOW d/t		**d/t ≤ 2.26 √(E/F$_{yc}$)**		**d/t ≈ 80**	
F_{yc} *[ksi]*	**AVE. d/t**	**F_{yc} *[ksi]***	**AVE. d/t**	**F_{yc} *[ksi]***	**AVE. d/t**
46	**22**	46	**52**	46	**73**
63	**20.5**	63	**45**	63	**69**
80	**20**	80	**40**	80	**68**

Table 6.3.15: Mean Values of d/t

$$\frac{d}{t} = \frac{F_{yc}}{a} + \left(\frac{F_{yc}}{b}\right)^{c} \tag{6.3-14}$$

Low d/t ratios: a = 1.23159224083901E+01 b = 9.43499630252338E+03
c = -5.45687025577683E-01

$d/t \leq 2.26\ \sqrt{(E/F_y)}$: a = -2.33848326894338E+01 b = 1.17819836272101E+06
c = -3.92910165308971E-01

$d/t \approx 80$: a = 3.5240366710384E+00 b = 1.49301458159099E+05
c = -5.0629760078164E-01

Another way to measure the value of η for a particular building is to plot its building value of η onto the three 3D surfaces in Figure 6.3.3. These 3D surfaces are the three surfaces from Figure 6.3.2. The space between each surface has been designated as a particular zone. Zone 1 is the space between Surface #1 and #2 while Zone 2 is the space between Surface #2 and #3. The lower a value of η is in a particular zone, the more the building is considered to be designed appropriately for its specified target value of d/t. Zone 1 represents RCFT moment frames that are to be designed in accordance with the AISC d/t limit. Zone 2 was only set up for this study. It was not intended for a real design to generate a building-based value of η that would fall into Zone 2 since it would result in d/t values larger than the AISC limit.

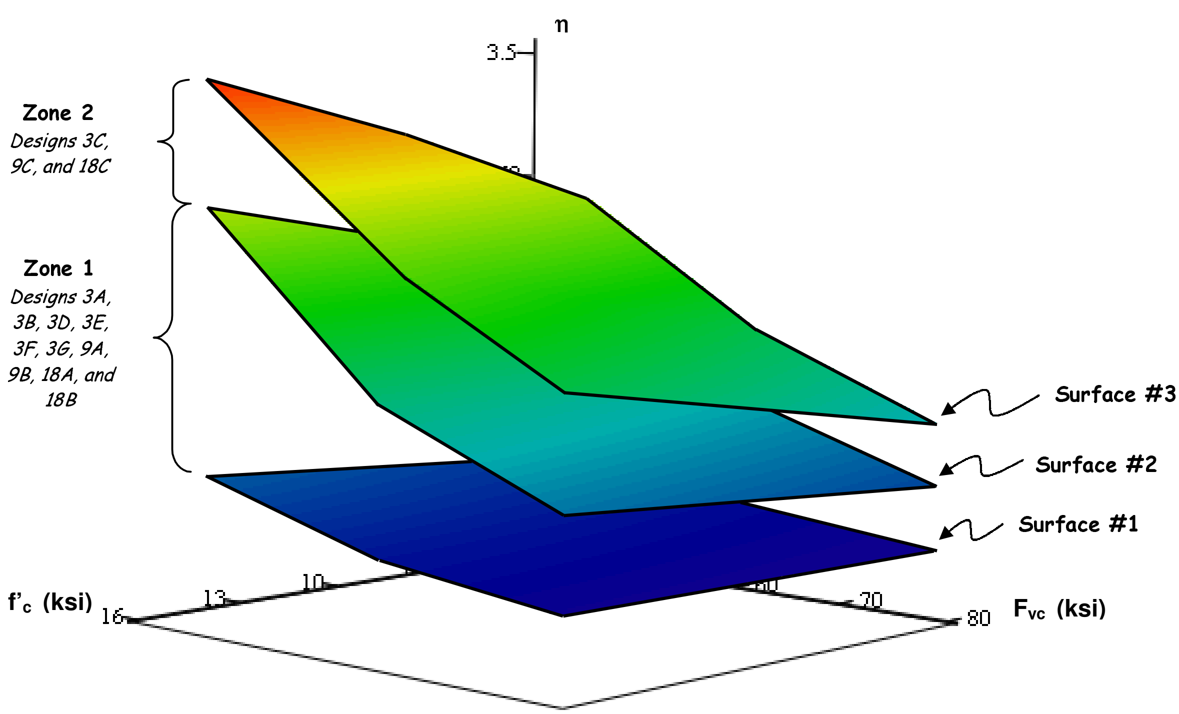


Figure 6.3.3: All Three Surfaces of the Baseline Values of η and the Two Zones Between Them

As shown in Figure 6.3.3, only the three designs that were intended to have large d/t ratios (Design 3C, 9C, and 18C) have values of η that are in Zone 2. All of the remaining 10 building designs have values of η that are within Zone 1.

Since the flexural rigidity ratio was developed to verify the overstrength of the buildings that were designed in this study, a building is considered to be appropriate to use in calibrating the overstrength factor equations (Equations 5.2.2-2 through 5.2.2-4) if its flexural rigidity ratio, η, is not smaller nor significantly larger than its corresponding baseline value. By comparing the actual values of η from Table 6.3.12 to their baseline values of η in Table 6.3.14, the low strength buildings have actual η values of 1.13, 0.95, and 1.03 for the 3-story, 9-story, and 18-story buildings, respectively. The baseline values of the rigidity ratio shall be between 0.7 and 2.0 for the low strength buildings. The high strength buildings have actual η values of 0.93, 0.54, 0.55 for the 3-story, 9-story, and 18-story buildings, respectively, while their baseline values shall be between 0.41 and 1.0.

Ten buildings (designs 3A, 3B, 3D, 3E, 3F, 3G, 9A, 9B, 18A, and 18B) have values of η that are within the baseline limits for d/t ratios which are less than or equal to the AISC limit. The 3-story buildings have values of η near the upper baseline limits for both the low strength and high strength buildings. The 9-story and 18-story buildings have η values that are near the lower baseline limits for both the low strength and the high strength buildings. Therefore, the 9-story and 18-story buildings are considered efficient building designs while the 3-story buildings are considered a little conservative in their design since they are near the top end of the spectrum.

Even though the 3-story buildings are not as efficient (i.e., they are stronger and stiffer than required by the building code) compared to the 9-story and 18-story buildings, their values of η are still within the baseline limits, and so they are considered appropriate to use in determining the overstrength factor. Therefore, the overstrength factors that were developed in Equations 5.2.2-2 through 5.2.2-4 represent an appropriate method for determining the design overstrength factor, Ω, for RCFT buildings that range in height from 1-story up to 18-stories.

Three methods have been developed in this study to assess the suite of buildings. The first method compared the suite of buildings with the elastic seismic design spectrum and the second method compared the buildings with a pushover curve envelope. Both of these methods were able to demonstrate that the thirteen buildings chosen for this study provide a representative suite of buildings within the limits of the study. The third method was able to demonstrate, through the use of the rigidity ratio, that all thirteen buildings in this study were not over designed, and their respective overstrength values were within expected values. Therefore, the thirteen buildings that were designed and assessed in this study can be used in the seismic demand assessment and ultimately in the calibration of the reliability-based performance-based design methodology for RCFT frames.

Chapter 7

Summary and Conclusions

A comprehensive suite of buildings is necessary for the RCFT performance-based design seismic demand assessment to be performed. The work presented in this report describes the process that was used to design and assess all thirteen buildings that were chosen to make up this comprehensive suite of buildings. Based on the findings of this study, this suite of buildings provides a representative set of building system performances and composite behavior that would be expected to occur in low-rise through mid-rise RCFT moment-resisting frame structural systems.

7.1 Summary of Results

The development of a suite of buildings for use in the RCFT column performance-based design demand assessment involved two research objectives that have been presented in this report. The first objective involved the linear design and analysis of each building so that the column and girder section sizes of the moment-resisting frames could be determined. The second objective involved the assessment of each building to determine the comprehensiveness of the suite of buildings and to show that the suite provides a full range of expected behavior of RCFT systems.

The first objective of this research was to design each of the thirteen buildings that make up the final suite of buildings using the most current building code loading and material design strengths specification. The wind and seismic loads are in accordance with the 2003 International Building Code requirements for a building located in central Los Angeles, California. The column strengths are in accordance with the 2005 AISC specification. A range of column steel and concrete material design strengths as well as different d/t limits were used in this study to account for various combinations of material strengths and d/t ratios that could be used in a building. The envelope of column HSS steel yield strengths ranged between 46 ksi and 80 ksi while the concrete compressive strengths envelope ranged between 4 ksi and 16 ksi. Two maximum allowed d/t ratios were used that ranged from the AISC limit of $2.26\sqrt{(E/F_{yc})}$ to a d/t equal to or less than 80.

The final column and girder sizes that were chosen for each building were based on a combination of the seismic interstory drift limitations of the 2003 IBC, and the strong column-weak beam requirement of the AISC Seismic Provisions. Column or girder design strengths did not control any of the building designs in this study.

The second objective of this study involved assessing the suite of thirteen buildings. This objective was broken down into two phases. The first phase analyzed each building using nonlinear static pushover analysis methods. The results from each building analysis were then used to determine if the building was behaving as expected regardless of its structural system or material strengths. The second phase involved assessing each building system response from its nonlinear static pushover analysis, and comparing the global results to what is expected for idealized RCFT structural systems.

The first phase of the second objective involved performing a nonlinear analysis of each building and then analyzing the results for each building. This phase of the study allowed for an estimate of the post-yield response of each building to be made as well as for relative comparisons to be made between any two buildings. The elastic stiffness, relative energy absorbed, system capacity, and the system overstrength factor were all determined for each building based on their individual pushover analysis curve.

An important characteristic of a building that is dependent on its elastic stiffness is the fundamental period. Two equations were developed for this study that are able to estimate the fundamental period of a building using parameters readily available in an elastic analysis. These equations (Equation 5.1-1 and Equation 5.1-2) were shown to calculate values of the fundamental period for each building within 5% of period values that were determined from a dynamic analysis. If T_a is too conservative (i.e., it is smaller than what would result from a more substantiated rational analysis) C_s would be too large which will lead to an uneconomical building design. Therefore, Equation 5.1-1 and Equation 5.1-2 allowed for the design period from the building code to be checked to see if a more rational method for calculating the design period is required, or if the current design value is sufficient.

When using Equation 5.1-1 and Equation 5.1-2 and the elastic analysis results, the 3-story buildings were shown to have an average fundamental period of 0.81 seconds while the 9-story and 18-story buildings had an average fundamental period of 2.11 seconds and 3.48 seconds, respectively. The inelastic pushover analysis resulted in the 3-story buildings having an average fundamental period of 0.75 seconds while the 9-story and 18-story buildings had an average fundamental period of 2.15 seconds and 2.94 seconds, respectively.

A second system characteristic that was determined from the pushover analysis was the relative energy that was absorbed by each building. This energy value allows for a comparison to be made between any two buildings to estimate their overall system responses. The relative energy absorbed by a building is measured by calculating the area under each pushover analysis curve. An accurate measurement of the area under a curve requires that the equation of the curve be integrated between two points along the curve. A method was developed in this study that used the Ramberg-Osgood Equation model so that an equation could be derived that describes the pushover analysis curve for each building.

To compare the relative energy values between any two buildings, a method was developed that allowed for a consistent stopping point of each pushover analysis curve. This method employed the ASCE 7-02 k-value that was is used to vertically distribute the seismic story shear loads. The k-value was chosen because it is able to locate the end point of the energy calculations near the actual analysis termination points for each building. It has similar trends as the pushover curves in relation to the building fundamental period, and it varies between each building. By using the k-value, the effects of the building period on the pushover curve are able to be included in determining when to end the energy calculation. The k-value also allows for the analysis termination points to end at a point that corresponds with the actual data points of each normalized pushover analysis curve. By using the Ramberg-Osgood model to derive an equation of each pushover curve and then integrating each equation, a relative energy value was determined for each building.

The third system characteristic that was derived from the pushover analysis is the capacity of each building. The capacity value provides a way to compare two buildings and to determine if their system responses to the lateral loads are what would be expected for each building based on its number of stories and roof height. On average the 3-story buildings have a capacity that is 15% larger than the 9-story buildings and 40% larger than the 18-story buildings. This demonstrates that even though the shorter buildings did not absorb as much energy (as demonstrated in the previous discussion) they were able to resist larger external forces before they were expected to collapse.

This study demonstrated that there are at least three system characteristics that can be used to estimate the system overstrength factor for a regular RCFT building system. These three characteristics are the number of stories, the design fundamental period, and the roof height. The period dependent model was determined to be the most appropriate to use for a RCFT moment-resisting frame system based on the assumption that no matter if a building is considered regular or irregular, or how many stories it has, or how tall the stories are, the period dependent model would still result in appropriate estimates of the system overstrength factor. Overall the overstrength factors for the 3-story, 9-story, and 18-story buildings have the same relative pattern as the design base shear coefficients in that the shorter buildings have larger overstrength factors. The overstrength factors calculated in this study were 3.79 for the 3-story buildings, 2.70 for the 9-story buildings, and 2.31 for the 18-story buildings.

Three methods were used in the second phase of the second objective to assess the suite of buildings. The first method compared the base shear seismic coefficient that was used to design each building with the entire elastic seismic design spectrum. The second method compared the pushover analysis curve for each building to an envelope of possible curves to determine if the buildings in the suite are a good representation of the spectrum of possible building responses. The third method that was performed in this study on the suite of buildings was to determine how much each building was over designed in relation to the applicable design limits for RCFT moment-resisting frames. To verify that the overstrength factor estimate (which was developed in a previous discussion) is accurate, and to prove that the suite of buildings is not representing buildings that are too strong (i.e., too conservative), the flexural rigidity ratio, η, was developed for this study.

The results of the first method of the suite assessment showed that the buildings in the suite use base shear coefficients from the three major portions of the elastic seismic design spectrum - the upper plateau, the mid-point of the curve, and the lower plateau. Therefore, a good representative sample of possible base shear coefficients was used to design these buildings. This method also demonstrated that if an estimated fundamental period, T_1, was used to calculate the base shear coefficient instead of T_a, the base shear coefficients would spread out a little more along the seismic design spectrum. This further demonstrated that the thirteen buildings that have been chosen to make up the suite represent a well-dispersed sample of possible building responses.

The second method that was used to assess the suite developed an envelope of maximum and minimum idealized pushover analysis curves. This envelope encompasses all of the system responses that would be expected to occur for buildings between 1-story and 18-stories. Using this envelope of possible minimum and maximum building responses, all thirteen buildings were shown to fall within the limits of the overall idealized envelope curve. Therefore, the buildings provide a good representation of possible RCFT building responses.

The third method used to assess the suite of buildings used the flexural rigidity ratio so that a relative measurement of the inherent overstrength of each RCFT building could be made. This relative measurement was made by comparing the rigidity ratio of a particular building to a target value based on three RCFT column design parameters. These three parameters are the column yield strength, the concrete compressive strength, and the d/t ratio. Once these three parameters are known, a target value of η is determined by using a database of values that was developed for this study. The actual η value is then calculated for the building and compared to its target value. If the actual value is near or just greater than its target value, the building is considered to be designed appropriately. If the actual value of η is less than its target value then the building is most likely not designed in accordance with the building code drift limits or the AISC member strength limits and will need to be redesigned. If the actual value of η is much greater than its target value, the building is considered over designed and will need to be further optimized.

Nine equations were developed to calculate η for a particular building. 135 additional moment-resisting frames were then approximated so that representative baseline values (i.e., target values) of η could be calculated. By comparing the baseline values with the actual building values, all thirteen buildings of this study were shown to have η values that are closer to the lower end of the range of target values rather than at the upper end. Therefore, all of the buildings that make up the suite are considered to have been designed appropriately and they do not have too much overstrength incorporated into the their structural systems.

7.2 Conclusions

The research presented in this report describes the process and methodologies that were used to develop a comprehensive suite of buildings suitable for conducting a seismic demand assessment of RCFT frames. Once each of the thirteen buildings was designed and analyzed their individual system responses were assessed using a number of analysis techniques including linear analysis, nonlinear analysis, as well as some approximating methods of analysis. From this assessment process each building was shown to exhibit an overall structural system behavior that is expected to occur within the limits of this study based on their number of stories, roof height, fundamental period, and material design strengths. All thirteen buildings were designed to be within the limits of the design building code as well as within the AISC strength limits and Seismic Provisions. All thirteen buildings were shown to cover a wide sample of possible seismic base shear coefficients compared to the elastic seismic design spectrum. Each building was also demonstrated to behave within the limits of an idealized envelope of upper and lower bound pushover analysis curves. As a final assessment, the overstrength of each building was verified by showing that all of their building-based flexural rigidity ratios were within the expected limits for each building.

All of the analysis and assessment methods used in this study were able to demonstrate that the thirteen buildings chosen to make up the suite of buildings are, as a group, comprehensive. The structural systems that make up the buildings in this suite were shown to exhibit a wide range of expected performance of RCFT moment-resisting frames. Therefore, these thirteen buildings are suitable to be used in a reliability-based performance-based design seismic demand assessment of RCFT frames.

7.3 Future Work

Additional research is needed to understand other areas of the behavior and response of RCFT moment-resisting frame systems that were not investigated in this study. It is recommended that the actual fundamental period of a building be used in the period dependent model for calculating the overstrength factor to see if it allows for a more accurate estimate of the system overstrength factor. It is also suggested that the composite floor deck, or other contributions to stiffness and strength, be included in the analysis to determine how they affect the overall system response to seismic loading and how they affect the system overstrength factor for composite systems. Finally, it is recommended that a method be developed that relates the flexural rigidity ratio to the overstrength of a RCFT moment-resisting frame system.

REFERENCES

Al-Khafaji, A. and Tooley, J. (1986). *Numerical Methods in Engineering Practice*, CBS College Publishing, New York, New York.

American Institute of Steel Construction, Inc. (AISC) (1994). *Manual of Steel Construction: Load and Resistance Factor Design,* 2nd Edition, American Institute of Steel Construction, Inc., Chicago, Illinois.

American Institute of Steel Construction, Inc. (AISC) (2001). *Manual of Steel Construction: Load and Resistance Factor Design*, 3rd Edition, American Institute of Steel Construction, Inc., Chicago, Illinois.

American Institute of Steel Construction, Inc. (AISC) (2002). *Seismic Provisions for Structural Steel Buildings*, American Institute of Steel Construction, Inc., Chicago, Illinois.

American Institute of Steel Construction, Inc. (AISC) (2005). *Specification for Structural Steel Buildings*, American Institute of Steel Construction, Inc., Chicago, Illinois.

American Society of Civil Engineers (ASCE) (2003). *Minimum Design Loads for Buildings and Other Structures*, ASCE Standard 7-02, American Society of Civil Engineers, New York, New York.

Applied Technology Council (ATC) (2003). "Preliminary Evaluation of Methods for Defining Performance," ATC-58-2, Applied Technology Council, Redwood City, California.

Applied Technology Council (ATC) (2004). "Engineering Demand Parameters for Structural Framing Systems," ATC-58 Project Task Report, Phase 2, Task 2.2, Applied Technology Council, Redwood City, California.

ATC/BSSC (1997). "NEHRP Guidelines for the Seismic Rehabilitation of Buildings," Applied Technology Council for Building Seismic Safety Council, Federal Emergency Management Agency (FEMA Report 273), Washington, D.C..

Federal Emergency Management Agency (FEMA) (2000). "Sate of the Art Report on Systems Performance of Steel Moment-Frames Subject to Earthquake Ground Shaking," SAC Joint Venture (FEMA Report 355C), Washington, D.C..

Federal Emergency Management Agency (FEMA) (2000). "State of the Art Report on Past Performance of Steel Moment-Frame Buildings in Earthquakes," SAC Joint Venture (FEMA Report 355E), Washington, D.C..

Federal Emergency Management Agency (FEMA) (2000). "State of the Art Report on Performance Prediction and Evaluation of Steel Moment-Frame Buildings," SAC Joint Venture, (FEMA Report 355F), Washington, D.C..

Gourley, B. C. and Hajjar, J. F. (1994). "Cyclic Nonlinear Analysis of Three-Dimensional Concrete-Filled Steel Tube Beam-Columns and Composite Frames," Structural Engineering Report No. ST-94-3, Department of Civil Engineering, University of Minnesota, Minneapolis, Minnesota, November, 216 pp.

Gourley, B.C., Tort, C., Hajjar, J.F., and Schiller, P.H. (2001). "A Synopsis of Studies of the Monotonic and Cyclic Behavior of Concrete-Filled Steel Tube Beam-Columns," Structural Engineering Report No. ST-01-04, Version 3.0, Department of Civil Engineering, University of Minnesota, Minneapolis, Minnesota.

Guo, W. and Gilsanz, R. (2003). "Simple Nonlinear Static Analysis Procedure for Progressive Collapse Evaluation," American Institute of Steel Construction, Inc (AISC) and The Steel Institute of New York (SINY) *Steel Building Symposium: Blast and Progressive Collapse Resistance,* New York, New York.

Hajjar, J. F., Gourley, B. C., and Olson, M. C. (1997). "A Cyclic Nonlinear Model for Concrete-Filled Tubes. I. Formulation. II. Verification," *Journal of Structural Engineering*, ASCE, Vol. 123, No. 6, pp. 736-754.

Hajjar, J., White, D., Clarke, M., Bridge, R., Lui, E., Leon, R., and Sheikh, T. (1997). "Effective Length and Notional Load Approaches for Assessing Frame Stability: Implications for American Steel Design," Committee Report from the American Society of Civil Engineers (ASCE) Structural Engineering Institute (SEI) Task Committee on Effective Length and the ASCE SEI Technical Committee on Load and Resistance Factor Design, ASCE, New York, New York.

Housner, G.W. and Jennings, P.C. (1982). *Earthquake Design Criteria*, Earthquake Engineering Research Institute, Oakland, California.

International Code Council (2002). *2003 International Building Code*, International Code Council, Inc., Falls Church, Virginia.

Jin, J and El-Tawil, S. (2004). "Seismic Performance of Steel Frames with Reduced Beam Section Connections," *Journal of Constructional Steel Research*, pp. 453-471.

La Fore, S. and Hajjar, J.F. (2005). "Design of Concrete-Filled Steel Tube Frames for Assessment Under Seismic Loading," Structural Engineering Report No. ST-05-01, Department of Civil Engineering, University of Minnesota, Minneapolis, Minnesota.

Mehta, K. and Marshall, R. (1998). *Guide to the Use of the Wind Load Provisions of ASCE 7-95*, ASCE Press, Reston, Virginia.

Moehle, J. and Deierlein, G. (2004). "A Framework Methodology for Performance-Based Earthquake Engineering," Paper No. 679, 13th World Conference on Earthquake Engineering, Vancouver, B.C..

Muvdi, B.B. and McNabb, J.W. (1991). *Engineering Mechanics of Materials*, 3rd Edition, Springer-Verlag, New York, New York.

Naeim, Farzad. (2001). *The Seismic Design Handbook*, Second Edition, Kluwer Academic Publishers, Norwell, Massachusetts.

Newmark, N.M. and Hall, W.J. (1982). *Earthquake Spectra and Design*. Earthquake Engineering Research Institute, Oakland, California.

Nowak, A. and Collins, K. (2000). *Reliability of Structures,* McGraw-Hill, New York, New York.

Ramberg, W. and Osgood, W. (1943). "Description of Stress-Strain Curves by Three Parameters," National Advisory Committee for Aeronautics (NACA), Technical Note No. 902, Washington, D.C..

Schultz, A.E. (1992). "Approximating Lateral Stiffness of Stories in Elastic Frames," *Journal of Structural Engineering*, ASCE, Vol. 118, No. 1, January, pp. 243 – 263.

SEAOC (1995). "Performance Based Seismic Engineering of Buildings," Vision 2000 Report, Structural Engineers Association of California, Volumes I and II, Sacramento, California.

Steel Tube Institute (STI) (2004). "Hollow Structural Sections: Dimensions and Section Properties," Steel Tube Institute of North America, Coral Gables, Florida.

Steel Tube Institute (STI) (1996). "Summary of Formats and Procedures for Calculating Properties of Square and Rectangular HSS / Structural Steel Tubing," Draft Copy, Steel Tube Institute of North America, Coral Gables, Florida.

Taranath, B. (1998). *Steel, Concrete, and Composite Design of Tall Buildings*, McGraw-Hill, New York, New York.

Tort, C. and Hajjar, J.F. (2003). "Damage Assessment of Concrete-Filled Steel Tube Members and Frames for Performance-Based Design," Structural Engineering Report No. ST-03-01, Department of Civil Engineering, University of Minnesota, Minneapolis, Minnesota.

Tort, C. and Hajjar, J. F. (2004). "Damage Assessment of Rectangular Concrete-Filled Steel Tubes for Performance-Based Design," *Earthquake Spectra*, Vol. 20, No. 4, pp. 1317-1348.

VisualAnalysis, Version 4.01.015 (2000), Computer Software, Integrated Engineering Software, Inc., Bozeman, Montana.

Wen, Y.K., Ellingwood, B.R., Venexiano, D., and Bracci, J. (2003). "Uncertainty Modeling in Earthquake Engineering," MAE Center Project FD-2 Report, Mid-America Earthquake Center, University of Illinois at Urbana-Champaign, Urbana, Illinois.

Appendix A

9-Story Building Nominal Loads

The first step in the linear design of the 9-story buildings was to determine the nominal (unfactored) loads that each building needed to be designed to resist. Once the building layout and geometries were determined the gravity loads (dead and live loads) were calculated followed by the environmental (wind and seismic) loads. This appendix shows the design calculations that determined the nominal loads for the 9-story building designs.

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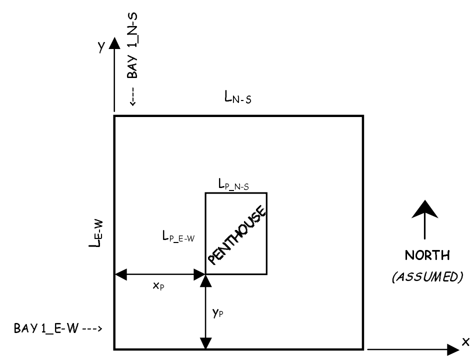
SUBJECT

2-D MOMENT FRAME [MF A2 - F2] ANALYSIS LOAD SUMMARY

[1] BUILDING GEOMETRY

- NUMBER OF STORIES, N_S — 9 STORIES
- NUMBER OF BAYS:
  - ALONG THE N-S *FACE* OF THE BUILDING, $N_{B\_N\text{-}S}$ — 5 BAYS
  - ALONG THE E-W *FACE* OF THE BUILDING, $N_{B\_E\text{-}W}$ — 5 BAYS
- BUILDING LENGTH (CTR-TO-CTR OF COLUMNS):
  - ALONG THE N-S *FACE* OF THE BUILDING, $L_{N\text{-}S}$ — 150.0 ft
  - ALONG THE E-W *FACE* OF THE BUILDING, $L_{E\text{-}W}$ — 150.0 ft
- CTR-TO-CTR DISTANCE (SPACING) OF THE FOLLOWING ITEM(S):
  - BEAMS — 10.0 ft
- PARAPET HEIGHT — 3.5 ft
- PENTHOUSE:
  - HEIGHT — 13.0 ft
  - LENGTH ALONG THE N-S *FACE* OF THE BUILDING, $L_{P\_N\text{-}S}$ — 90.0 ft
  - LENGTH ALONG THE E-W *FACE* OF THE BUILDING, $L_{P\_E\text{-}W}$ — 90.0 ft
- DISTANCE OF THE *SOUTHWEST* CORNER (OF THE PENTHOUSE):
  - x_P — 30.0 ft
  - y_P — 30.0 ft



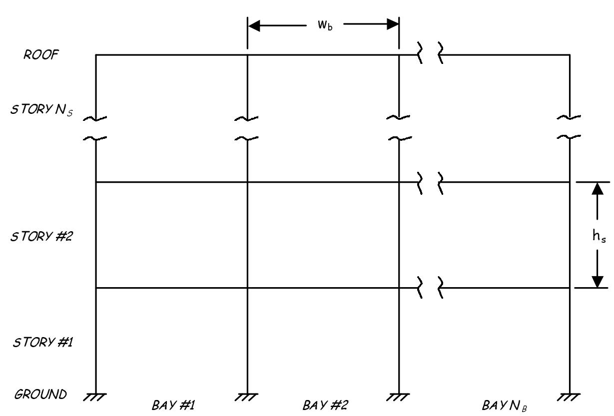
PLAN VIEW

- DIRECTION THAT THE *COMPOSITE FLOOR SYSTEM* SPANS — E-W

THEREFORE, THE BEAMS SPAN IN THE N-S DIRECTION

STORY NUMBER	STORY HEIGHT, h_s *(CTR-TO-CTR OF GIRDERS)*
9	13.0 ft
8	13.0 ft
7	13.0 ft
6	13.0 ft
5	13.0 ft
4	13.0 ft
3	13.0 ft
2	13.0 ft
1	13.0 ft

BAY WIDTH, w_b ALONG EACH *FACE* OF THE BUILDING			
ALONG THE N-S *FACE* (GIRDERS ARE PARALLEL TO THE E-W DIRECTION)		ALONG THE E-W *FACE* (GIRDERS ARE PARALLEL TO THE N-S DIRECTION)	
BAY NUMBER	WIDTH, w_b *CTR-TO-CTR OF COLUMNS*	BAY NUMBER	WIDTH, w_b *CTR-TO-CTR OF COLUMNS*
1_N-S	30.0 ft	1_E-W	30.0 ft
2_N-S	30.0 ft	2_E-W	30.0 ft
3_N-S	30.0 ft	3_E-W	30.0 ft
4_N-S	30.0 ft	4_E-W	30.0 ft
5_N-S	30.0 ft	5_E-W	30.0 ft



ELEVATION VIEW

GG Engineering	JOB NO. 9-STORY BUILDINGS	BY SMG	DATE 9/16/04	SHEET NO.
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SUBJECT

2-D MOMENT FRAME [MF A2 - F2] ANALYSIS LOAD SUMMARY

[2] 2-D MOMENT FRAME GEOMETRY

- TYPE OF FRAME: MOMENT FRAME
- DIRECTION THAT THE MOMENT FRAME RUNS PARALLEL WITH: E-W — *(THEREFORE, THIS MOMENT FRAME WILL RESIST SEISMIC LOADS IN THE E-W DIRECTION)*
- MOMENT FRAME NAME: MF A2 - F2
- DISTANCE FROM THE CLOSEST COLUMN STACK IN THE MOMENT FRAME TO THE SOUTHWEST CORNER OF THE BUILDING, Yc: 120.0 ft — *THIS FRAME IS LOCATED IN THE INTERIOR OF THE BUILDING AND THIS FRAME SUPPORTS PART OF THE PENTHOUSE.*
- NUMBER OF BAYS IN THIS MOMENT FRAME: 5 BAYS
- THE FIRST BAY IN THIS MOMENT FRAME IS THE SAME BAY AS WHICH BAY NUMBER IN THE BUILDING LAYOUT? 1_N-S — *THIS MOMENT FRAME HAS A TOTAL LENGTH OF 150ft.*
- DISTANCE TO THE CLOSEST (GRAVITY/MOMENT) FRAME: 30.0 ft
- DISTANCE TO THE CLOSEST (GRAVITY/MOMENT) FRAME ON THE OTHER SIDE: 30.0 ft
- JOINT COORDINATES WITH RESPECT TO THE SOUTHWEST CORNER OF THE BUILDING:

Coordinates of the MOMENT FRAME:

End Column Location:	X	Y
Farthest West	0.0 ft	120.0 ft
Farthest East	150.0 ft	120.0 ft

Coordinates of the Penthouse:

Corner Location	X	Y
Southwest	30.0 ft	30.0 ft
Northwest	30.0 ft	120.0 ft
Northeast	120.0 ft	120.0 ft
Southeast	120.0 ft	30.0 ft

COORDINATES OF THE MOMENT FRAME COLUMN STACKS WRT THE SOUTHWEST CORNER OF THE BUILDING

COLUMN NUMBER	1		2		3		4		5		6											
	x	y	x	y	x	y	x	y	x	y	x	y	x	y	x	y	x	y	x	y	x	y
	0	120	30	120	60	120	90	120	120	120	150	120										

- ASSUMING THAT THE PENTHOUSE PERIMETER IS ALWAYS LOCATED OVER A GRAVITY/MOMENT FRAME, THE FOLLOWING IS TRUE FOR THIS MOMENT FRAME:

COLUMN (STACK) NO.	1	2	3	4	5	6					
COLUMN (STACK) IS SUPPORTING WHICH PART OF THE PENTHOUSE?	NONE	EXTERIOR	EXTERIOR	EXTERIOR	EXTERIOR	NONE					

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SUBJECT

2-D MOMENT FRAME [MF A2 - F2] ANALYSIS LOAD SUMMARY

[3] BUILDING DEAD LOAD

o BUILDING (FLOORS):
- *COLUMNS, BEAMS, GIRDERS, MISC. STRUCTURAL SYSTEM COMPONENTS* 20 lb/ft²
- *EXTERIOR WALLS* 25 lb/ft² (Applied to Surface Area of the *WALL*)
- *FLOORING* 1 lb/ft²
- *COMPOSITE FLOOR SYSTEM (CONCRETE + METAL DECKING)* 50 lb/ft²
- *CEILING (FROM STORY BELOW) + FIREPROOFING* 2 lb/ft²
- *HVAC + ELECTRICAL (FROM STORY BELOW)* 7 lb/ft²

o BUILDING (ROOF):
- *PARAPET* 25 lb/ft² (Applied to Surface Area of the *WALL*)
- *ROOFING* 7 lb/ft²
- *COMPOSITE ROOF SYSTEM (CONCRETE + METAL DECKING)* 50 lb/ft²
- *(ROOF) BEAMS, GIRDERS, MISC. STRUCTURAL SYSTEM COMPONENTS* 20 lb/ft²
- *CEILING (FROM STORY BELOW) + FIREPROOFING* 2 lb/ft²
- *HVAC + ELECTRICAL (FROM STORY BELOW)* 7 lb/ft²

o PENTHOUSE:
- *COMPOSITE ROOF SYSTEM (CONCRETE + METAL DECKING)* 50 lb/ft²
- *CEILING + FIREPROOFING* 2 lb/ft²
- *EXTERIOR WALLS* 25 lb/ft² (Applied to Surface Area of the *WALL*)
- *COLUMNS, BEAMS, GIRDERS, MISC. STRUCTURAL SYSTEM COMPONENTS* 20 lb/ft²
- *MECHANICAL EQUIPMENT* 40 lb/ft²
- *FLOORING* 1 lb/ft²

NUMBER		(STORY)	D.L. (SURFACE) AREA		DEAD LOAD		TOTAL DEAD LOAD		TOTAL STORY DL
STORY	FLOOR	HEIGHT	ROOF / FLOOR	PENTHOUSE	ROOF / FLOOR	PENTHOUSE	ROOF / FLOOR	PENTHOUSE	ΣP_i
---	---	---	---	8,100 ft²	---	127.44 lb/ft²	---	1,032.26 kips	---
---	ROOF	---	22,500 ft²	---	88.33 lb/ft²	---	1,987.43 kips	---	---
9	9	13.0 ft	22,500 ft²	---	88.67 lb/ft²	---	1,995.08 kips	---	3,020 kips
8	8	13.0 ft	22,500 ft²	---	88.67 lb/ft²	---	1,995.08 kips	---	5,015 kips
7	7	13.0 ft	22,500 ft²	---	88.67 lb/ft²	---	1,995.08 kips	---	7,010 kips
6	6	13.0 ft	22,500 ft²	---	88.67 lb/ft²	---	1,995.08 kips	---	9,005 kips
5	5	13.0 ft	22,500 ft²	---	88.67 lb/ft²	---	1,995.08 kips	---	11,000 kips
4	4	13.0 ft	22,500 ft²	---	88.67 lb/ft²	---	1,995.08 kips	---	12,995 kips
3	3	13.0 ft	22,500 ft²	---	88.67 lb/ft²	---	1,995.08 kips	---	14,990 kips
2	2	13.0 ft	22,500 ft²	---	88.67 lb/ft²	---	1,995.08 kips	---	16,985 kips
1	GROUND	13.0 ft	N.A.	---	N.A.	---	N.A.	---	18,980 kips
								Building Total Dead Load (Ground Floor + 1st Story Dead Load NOT Included) =	*18,980 kips*

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SUBJECT

2-D MOMENT FRAME [MF A2 - F2] ANALYSIS LOAD SUMMARY

[4] BUILDING LIVE LOAD

o BUILDING (FLOORS):
- *OFFICE BUILDING OCCUPANCY PER IBC 2003, TABLE 1607.1* — 50 lb/ft²
- *(MOVEABLE) PARTITIONS PER IBC 2003, SECTION 1607.5* — 20 lb/ft²

o BUILDING (ROOF):
- *MINIMUM ROOF LL PER IBC 2003, SECTION 1607.11* — 20 lb/ft² *(ROOF LIVE LOAD, Lr)*

o PENTHOUSE:
- *GENERAL PENTHOUSE (INTERIOR) LIVE LOAD* — 20 lb/ft² *(TREATED AS ROOF LIVE LOAD, Lr)*
- *PENTHOUSE (ROOF) LIVE LOAD* — 0 lb/ft² *(ROOF LIVE LOAD, Lr)*

NUMBER		(STORY) HEIGHT	L.L. (SURFACE) AREA		LIVE LOAD		TOTAL LIVE LOAD		TOTAL STORY LL, ΣP_i	
STORY	FLOOR		ROOF / FLOOR	PENTHOUSE	ROOF / FLOOR	PENTHOUSE	ROOF / FLOOR	PENTHOUSE	*FLOOR LL*	*ROOF LL*
---	---	---	---	8,100 ft²	---	20 lb/ft²	---	162.0 kips	---	---
---	ROOF	---	14,400 ft²	---	20 lb/ft²	---	288.0 kips	---	---	---
9	9	13.0 ft	22,500 ft²	---	70 lb/ft²	---	1,575.0 kips	---	*0 kips*	*450 kips*
8	8	13.0 ft	22,500 ft²	---	70 lb/ft²	---	1,575.0 kips	---	1,575 kips	450 kips
7	7	13.0 ft	22,500 ft²	---	70 lb/ft²	---	1,575.0 kips	---	3,150 kips	450 kips
6	6	13.0 ft	22,500 ft²	---	70 lb/ft²	---	1,575.0 kips	---	4,725 kips	450 kips
5	5	13.0 ft	22,500 ft²	---	70 lb/ft²	---	1,575.0 kips	---	6,300 kips	450 kips
4	4	13.0 ft	22,500 ft²	---	70 lb/ft²	---	1,575.0 kips	---	7,875 kips	450 kips
3	3	13.0 ft	22,500 ft²	---	70 lb/ft²	---	1,575.0 kips	---	9,450 kips	450 kips
2	2	13.0 ft	22,500 ft²	---	70 lb/ft²	---	1,575.0 kips	---	11,025 kips	450 kips
1	GROUND	13.0 ft	N.A.	---	N.A.	---	N.A.	---	12,600 kips	450 kips
								Building Total Live Load and Roof Live Load =	12,600 kips	450 kips

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2-D MOMENT FRAME [MF A2 - F2] ANALYSIS LOAD SUMMARY

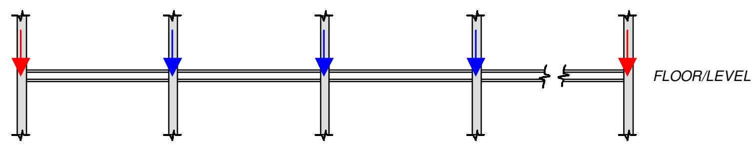
[5] MOMENT FRAME DEAD LOAD

o SUMMARY:

NUMBER OF STORIES, N_S	9 STORIES
NUMBER OF BAYS, N_B	5 BAYS
DIRECTION THAT THE *MOMENT FRAME* RUNS PARALLEL WITH:	E-W
LOCATION OF THE MOMENT FRAME WRT THE BUILDING PERIMETER:	INTERIOR
DOES THIS FRAME SUPPORT PART OF THE PENTHOUSE GRAVITY LOADS?	YES
DISTANCE TO THE CLOSEST (GRAVITY/MOMENT) FRAME:	30.0 ft
DISTANCE TO THE CLOSEST (GRAVITY/MOMENT) FRAME ON THE OTHER SIDE:	30.0 ft

o THEREFORE: DEAD LOAD TRIBUTARY WIDTH TO THIS MOMENT FRAME: 30.0 ft

NOTE: *PARAPET DEAD LOAD (PER ft² OF ROOF SURFACE AREA)* *2.33 lb/ft²*
PARAPET DEAD LOAD (PER FOOT OF PARAPET LENGTH) *87.5 lb/ft*



COLUMN (STACK) NO.	1	2	3	4	5	6					
PENTHOUSE LOADS TO THE COLUMNS	*0.0 kips*	*9.6 kips*	*19.1 kips*	*19.1 kips*	*9.6 kips*	*0.0 kips*					
FLOOR NUMBER	**UNFACTORED (NOMINAL) DEAD LOAD BEAM END REACTIONS TO EACH COLUMN OF THE MOMENT FRAME AT EVERY FLOOR/LEVEL**										
ROOF	15.5 kips	35.4 kips	44.9 kips	44.9 kips	35.4 kips	15.5 kips					
9	13.3 kips	26.6 kips	26.6 kips	26.6 kips	26.6 kips	13.3 kips					
8	13.3 kips	26.6 kips	26.6 kips	26.6 kips	26.6 kips	13.3 kips					
7	13.3 kips	26.6 kips	26.6 kips	26.6 kips	26.6 kips	13.3 kips					
6	13.3 kips	26.6 kips	26.6 kips	26.6 kips	26.6 kips	13.3 kips					
5	13.3 kips	26.6 kips	26.6 kips	26.6 kips	26.6 kips	13.3 kips					
4	13.3 kips	26.6 kips	26.6 kips	26.6 kips	26.6 kips	13.3 kips					
3	13.3 kips	26.6 kips	26.6 kips	26.6 kips	26.6 kips	13.3 kips					
2	13.3 kips	26.6 kips	26.6 kips	26.6 kips	26.6 kips	13.3 kips					
GROUND	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.					

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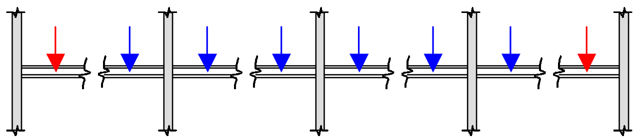
SUBJECT

2-D MOMENT FRAME [MF A2 - F2] ANALYSIS LOAD SUMMARY

[6] MOMENT FRAME DEAD LOAD (CONTINUED)

o DEAD LOAD TRIBUTARY WIDTH TO THIS MOMENT FRAME: 30.0 ft



FLOOR/LEVEL

BAY NUMBER	**1_N-S**	**2_N-S**	**3_N-S**	**4_N-S**	**5_N-S**					
BAY WIDTH	*30.0 ft*	*30.0 ft*	*30.0 ft*	*30.0 ft*	*30.0 ft*					
NO. OF BEAMS PER BAY	**2**	**2**	**2**	**2**	**2**					
BEAM SPACING (*BETWEEN BEAMS)*	*10.0 ft*	*10.0 ft*	*10.0 ft*	*10.0 ft*	*10.0 ft*					
PENTHOUSE LOADS TO THE ROOF GIRDERS	*0.0 kips*	*19.1 kips*	*19.1 kips*	*19.1 kips*	*0.0 kips*					
FLOOR NUMBER	**UNFACTORED (NOMINAL) DEAD LOAD BEAM END REACTIONS TO EACH GIRDER OF THE MOMENT FRAME AT EVERY FLOOR/LEVEL**									
ROOF	25.8 kips	44.9 kips	44.9 kips	44.9 kips	25.8 kips					
9	26.6 kips	26.6 kips	26.6 kips	26.6 kips	26.6 kips					
8	26.6 kips	26.6 kips	26.6 kips	26.6 kips	26.6 kips					
7	26.6 kips	26.6 kips	26.6 kips	26.6 kips	26.6 kips					
6	26.6 kips	26.6 kips	26.6 kips	26.6 kips	26.6 kips					
5	26.6 kips	26.6 kips	26.6 kips	26.6 kips	26.6 kips					
4	26.6 kips	26.6 kips	26.6 kips	26.6 kips	26.6 kips					
3	26.6 kips	26.6 kips	26.6 kips	26.6 kips	26.6 kips					
2	26.6 kips	26.6 kips	26.6 kips	26.6 kips	26.6 kips					
GROUND	N.A.	N.A.	N.A.	N.A.	N.A.					

GG Engineering	JOB NO. 9-STORY BUILDINGS	BY SMG	DATE 9/16/04	SHEET NO. ___ OF ___
	CUSTOMER RCFT PARAMETRIC STUDY	CKD	DATE	

SUBJECT

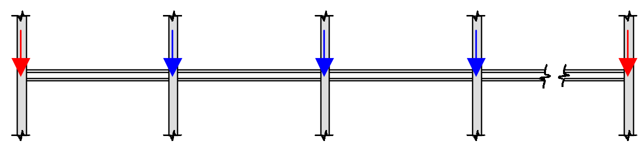
2-D MOMENT FRAME [MF A2 - F2] ANALYSIS LOAD SUMMARY

[7] MOMENT FRAME LIVE LOAD

o SUMMARY:

NUMBER OF STORIES, N_S	9 STORIES
NUMBER OF BAYS, N_B	5 BAYS
DIRECTION THAT THE *MOMENT FRAME* RUNS PARALLEL WITH:	E-W
LOCATION OF THE MOMENT FRAME WRT THE BUILDING PERIMETER:	INTERIOR
DOES THIS FRAME SUPPORT PART OF THE PENTHOUSE GRAVITY LOADS?	YES
DISTANCE TO THE CLOSEST (GRAVITY/MOMENT) FRAME:	30.0 ft
DISTANCE TO THE CLOSEST (GRAVITY/MOMENT) FRAME ON THE OTHER SIDE:	30.0 ft

o THEREFORE: LIVE LOAD TRIBUTARY WIDTH TO THIS MOMENT FRAME: 30.0 ft



COLUMN (STACK) NO.	1	2	3	4	5	6					
PENTHOUSE LOADS TO THE COLUMNS	*0.0 kips*	*1.5 kips*	*3.0 kips*	*3.0 kips*	*1.5 kips*	*0.0 kips*					
FLOOR NUMBER	**UNFACTORED (NOMINAL) LIVE LOAD BEAM END REACTIONS TO EACH COLUMN OF THE MOMENT FRAME AT EVERY FLOOR/LEVEL**										
ROOF	3.0 kips	7.5 kips	9.0 kips	9.0 kips	7.5 kips	3.0 kips					
9	10.5 kips	21.0 kips	21.0 kips	21.0 kips	21.0 kips	10.5 kips					
8	10.5 kips	21.0 kips	21.0 kips	21.0 kips	21.0 kips	10.5 kips					
7	10.5 kips	21.0 kips	21.0 kips	21.0 kips	21.0 kips	10.5 kips					
6	10.5 kips	21.0 kips	21.0 kips	21.0 kips	21.0 kips	10.5 kips					
5	10.5 kips	21.0 kips	21.0 kips	21.0 kips	21.0 kips	10.5 kips					
4	10.5 kips	21.0 kips	21.0 kips	21.0 kips	21.0 kips	10.5 kips					
3	10.5 kips	21.0 kips	21.0 kips	21.0 kips	21.0 kips	10.5 kips					
2	10.5 kips	21.0 kips	21.0 kips	21.0 kips	21.0 kips	10.5 kips					
GROUND	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.					

GG Engineering	JOB NO. 9-STORY BUILDINGS	BY SMG	DATE 9/16/04	SHEET NO.
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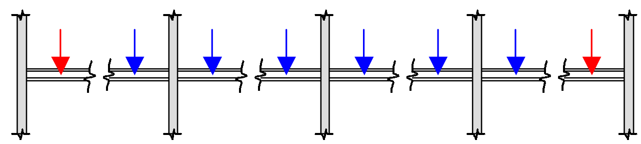
SUBJECT

2-D MOMENT FRAME [MF A2 - F2] ANALYSIS LOAD SUMMARY

[8] MOMENT FRAME LIVE LOAD (CONTINUED)

- LIVE LOAD TRIBUTARY WIDTH TO THIS MOMENT FRAME: 30.0 ft



BAY NUMBER	**1_N-S**	**2_N-S**	**3_N-S**	**4_N-S**	**5_N-S**					
BAY WIDTH	*30.0 ft*	*30.0 ft*	*30.0 ft*	*30.0 ft*	*30.0 ft*					
NO. OF BEAMS PER BAY	**2**	**2**	**2**	**2**	**2**					
BEAM SPACING (*BETWEEN BEAMS*)	*10.0 ft*	*10.0 ft*	*10.0 ft*	*10.0 ft*	*10.0 ft*					
PENTHOUSE LOADS TO THE ROOF GIRDERS	*0.0 kips*	*3.0 kips*	*3.0 kips*	*3.0 kips*	*0.0 kips*					
FLOOR NUMBER	**UNFACTORED (NOMINAL) LIVE LOAD BEAM END REACTIONS TO EACH GIRDER OF THE MOMENT FRAME AT EVERY FLOOR/LEVEL**									
ROOF	6.0 kips	9.0 kips	9.0 kips	9.0 kips	6.0 kips					
9	21.0 kips	21.0 kips	21.0 kips	21.0 kips	21.0 kips					
8	21.0 kips	21.0 kips	21.0 kips	21.0 kips	21.0 kips					
7	21.0 kips	21.0 kips	21.0 kips	21.0 kips	21.0 kips					
6	21.0 kips	21.0 kips	21.0 kips	21.0 kips	21.0 kips					
5	21.0 kips	21.0 kips	21.0 kips	21.0 kips	21.0 kips					
4	21.0 kips	21.0 kips	21.0 kips	21.0 kips	21.0 kips					
3	21.0 kips	21.0 kips	21.0 kips	21.0 kips	21.0 kips					
2	21.0 kips	21.0 kips	21.0 kips	21.0 kips	21.0 kips					
GROUND	N.A.	N.A.	N.A.	N.A.	N.A.					

GG Engineering	JOB NO. 9-STORY BUILDINGS	BY SMG	DATE 9/16/04	SHEET NO. ___ OF ___
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SUBJECT

2-D MOMENT FRAME [MF A2 - F2] ANALYSIS LOAD SUMMARY

[9] MOMENT FRAME SEISMIC WEIGHT (DEAD LOAD + PARTITION LIVE LOAD)

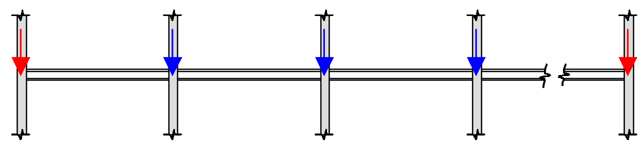
o SUMMARY:

NUMBER OF STORIES, N_S	9 STORIES
NUMBER OF BAYS, N_B	5 BAYS
DIRECTION THAT THE *MOMENT FRAME* RUNS PARALLEL WITH:	E-W
LOCATION OF THE MOMENT FRAME WRT THE BUILDING PERIMETER:	INTERIOR
DOES THIS FRAME SUPPORT PART OF THE PENTHOUSE GRAVITY LOADS?	YES
DISTANCE TO THE CLOSEST (GRAVITY/MOMENT) FRAME:	30.0 ft
DISTANCE TO THE CLOSEST (GRAVITY/MOMENT) FRAME ON THE OTHER SIDE:	30.0 ft

o THEREFORE: SEISMIC WEIGHT TRIBUTARY WIDTH TO THIS MOMENT FRAME: 30.0 ft

NOTE:

PARAPET SEISMIC WEIGHT (PER ft² OF ROOF SURFACE AREA)	*2.33 lb/ft²*
PARAPET SEISMIC WEIGHT (PER FOOT OF PARAPET LENGTH)	*87.5 lb/ft*
PARTITION LIVE LOAD	*20 lb/ft²*



COLUMN (STACK) NO.	1	2	3	4	5	6
PENTHOUSE LOADS TO THE COLUMNS	*0.0 kips*	*9.6 kips*	*19.1 kips*	*19.1 kips*	*9.6 kips*	*0.0 kips*
FLOOR NUMBER	**UNFACTORED (NOMINAL) SEISMIC WEIGHT BEAM END REACTIONS TO EACH COLUMN OF THE MOMENT FRAME AT EVERY FLOOR/LEVEL**					
ROOF	15.5 kips	35.4 kips	44.9 kips	44.9 kips	35.4 kips	15.5 kips
9	16.3 kips	32.6 kips	32.6 kips	32.6 kips	32.6 kips	16.3 kips
8	16.3 kips	32.6 kips	32.6 kips	32.6 kips	32.6 kips	16.3 kips
7	16.3 kips	32.6 kips	32.6 kips	32.6 kips	32.6 kips	16.3 kips
6	16.3 kips	32.6 kips	32.6 kips	32.6 kips	32.6 kips	16.3 kips
5	16.3 kips	32.6 kips	32.6 kips	32.6 kips	32.6 kips	16.3 kips
4	16.3 kips	32.6 kips	32.6 kips	32.6 kips	32.6 kips	16.3 kips
3	16.3 kips	32.6 kips	32.6 kips	32.6 kips	32.6 kips	16.3 kips
2	16.3 kips	32.6 kips	32.6 kips	32.6 kips	32.6 kips	16.3 kips
GROUND	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.

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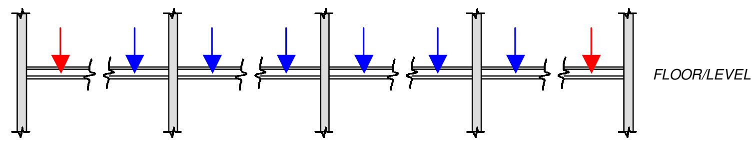
2-D MOMENT FRAME [MF A2 - F2] ANALYSIS LOAD SUMMARY

[10] MOMENT FRAME SEISMIC WEIGHT (CONTINUED)

o SEISMIC WEIGHT TRIBUTARY WIDTH TO THIS MOMENT FRAME: 30.0 ft



BAY NUMBER	1_N-S	2_N-S	3_N-S	4_N-S	5_N-S
BAY WIDTH	*30.0 ft*	*30.0 ft*	*30.0 ft*	*30.0 ft*	*30.0 ft*
NO. OF BEAMS PER BAY	**2**	**2**	**2**	**2**	**2**
BEAM SPACING (*BETWEEN BEAMS)*	*10.0 ft*	*10.0 ft*	*10.0 ft*	*10.0 ft*	*10.0 ft*
PENTHOUSE LOADS TO THE ROOF GIRDERS	*0.0 kips*	*19.1 kips*	*19.1 kips*	*19.1 kips*	*0.0 kips*
FLOOR NUMBER	**UNFACTORED (NOMINAL) SEISMIC WEIGHT BEAM END REACTIONS TO EACH GIRDER OF THE MOMENT FRAME AT EVERY FLOOR/LEVEL**				
ROOF	25.8 kips	44.9 kips	44.9 kips	44.9 kips	25.8 kips
9	32.6 kips	32.6 kips	32.6 kips	32.6 kips	32.6 kips
8	32.6 kips	32.6 kips	32.6 kips	32.6 kips	32.6 kips
7	32.6 kips	32.6 kips	32.6 kips	32.6 kips	32.6 kips
6	32.6 kips	32.6 kips	32.6 kips	32.6 kips	32.6 kips
5	32.6 kips	32.6 kips	32.6 kips	32.6 kips	32.6 kips
4	32.6 kips	32.6 kips	32.6 kips	32.6 kips	32.6 kips
3	32.6 kips	32.6 kips	32.6 kips	32.6 kips	32.6 kips
2	32.6 kips	32.6 kips	32.6 kips	32.6 kips	32.6 kips
GROUND	N.A.	N.A.	N.A.	N.A.	N.A.

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2-D MOMENT FRAME [MF A2 - F2] ANALYSIS LOAD SUMMARY

[11] SEISMIC LOADS PER THE INTERNATIONAL BUILDING CODE (IBC) 2003 [ASCE 7]

o *PER THE EXCEPTION OF SECTION 1614 OF THE IBC 2003, ASCE 7 IS PERMITTED TO BE USED TO DETERMINE THE SEISMIC LOADS. THEREFORE, THE FOLLOWING DESIGN PARAMETERS ARE TAKEN FROM ASCE 7-02.*		Section 1614	(p. 302)
o OCCUPANCY CATEGORY	II	Table 1-1	(p. 4)
o SEISMIC USE GROUP	I	Section 9.1.3 Table 9.1.3	(p. 96) (p. 96)
o OCCUPANCY IMPORTANCE FACTOR, I_E	1.00	Section 9.1.4 Table 9.1.4	(p. 96) (p. 97)
o THE MAPPED SPECTRAL ACCELERATIONS:	FOR SHORT PERIODS, S_S = 1.5 g FOR A 1-SECOND PERIOD, S_1 = 0.6 g	Section 9.4.1.2 Figure 9.4.1.1(c) & (d)	(p. 107)
o SITE CLASS	D	Section 9.4.1.2.1 *Note the "Exception"*	(p. 108)
o SITE COEFFICIENTS:	F_a = 1.0 F_v = 1.5	Table 9.4.1.2.4a Table 9.4.1.2.4b	(p. 129) (p. 130)
o MAX. CONSIDERED SPECTRAL RESPONSE ACCELERATIONS:	FOR SHORT PERIODS, S_{MS} = 1.5 g FOR A 1-SECOND PERIOD, S_{M1} = 0.9 g	Equation 9.4.1.2.4-1 Equation 9.4.1.2.4-2	(p. 129) (p. 129)
o DESIGN SPECTRAL RESPONSE ACCELERATIONS:	FOR SHORT PERIODS, S_{DS} = 1.0 g FOR A 1-SECOND PERIOD, S_{D1} = 0.6 g	Equation 9.4.1.2.5-1 Equation 9.4.1.2.5-2	(p. 129) (p. 129)
o SEISMIC DESIGN CATEGORY	D	Table 9.4.2.1a Table 9.4.2.1b	(p. 131) (p. 132)
o BASIC SEISMIC-FORCE RESISTING SYSTEM	*"SPECIAL COMPOSITE MOMENT FRAMES"*	Table 9.5.2.2	(p. 134)
RESPONSE MODIFICATION COEFFICIENT, R	8	Table 9.5.2.2	(p. 134)
DEFLECTION AMPLIFICATION FACTOR, C_d	5.5	Table 9.5.2.2	(p. 134)
o FUNDAMENTAL PERIOD, T		Section 9.5.5.3	(p. 147)
BUILDING PERIOD COEFFICIENTS:	C_T = 0.028 x = 0.8	Table 9.5.5.3.2 Table 9.5.5.3.2	(p. 147) (p. 147)
ELEVATION OF THE BUILDING ROOF ABOVE THE BASE, h_n	117 ft		
COEFFICIENT FOR UPPER LIMIT ON CALCULATED PERIOD, C_U	1.4	Table 9.5.5.3.1	(p. 147)
APPROXIMATE FUNDAMENTAL PERIOD, T_a	1.264 sec	Equation 9.5.5.3.2-1	(p. 147)
PERIOD FROM RATIONAL ANALYSIS, T_R	*NONE CALCULATED*		
MAXIMUM ALLOWED PERIOD, C_UT_a	1.770 sec	Section 9.5.5.3	(p. 147)
DESIGN PERIOD, T	1.264 sec	Section 9.5.5.3	(p. 147)
o ANALYSIS PROCEDURE	*EQUIVALENT LATERAL FORCE ANALYSIS*	Section 9.5.2.5.1 Table 9.5.2.5.1	(p. 139) (p. 140)
o SEISMIC RESPONSE COEFFICIENT, C_S	C_S = 0.125 g	Equation 9.5.5.2.1-1	(p. 146)
	$C_S \leq$ 0.060 g	Equation 9.5.5.2.1-2	(p. 146)
	$C_S \geq$ 0.044 g	Equation 9.5.5.2.1-3	(p. 146)
	DESIGN C_S = 0.060 g		
o SEISMIC (DESIGN) BASE SHEAR, V	***V = 0.060 W***	Equation 9.5.5.2-1	(p. 146)

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2-D MOMENT FRAME [MF A2 - F2] ANALYSIS LOAD SUMMARY

[12] VERTICAL DISTRIBUTION OF THE SEISMIC LOADS PER ASCE 7-02 [IBC 2003]

Section 9.5.5.4 (p. 148)
Equation 9.5.5.4-1 (p. 148)
Equation 9.5.5.4-2 (p. 148)

- o BUILDING FUNDAMENTAL (DESIGN) PERIOD, T — 1.264 sec
- o DISTRIBUTION EXPONENT, k — 1.38
- o DESIGN BASE SHEAR, V — 0.060 W
- o EFFECTIVE SEISMIC WEIGHT, W — 22,677.8 kips
- o DESIGN SHEAR AT THE BASE OF THE BUILDING, V — 1,360.7 kips

NUMBER		STORY HEIGHT	FLOOR ELEVATION, h_x	DEAD LOAD		(MOVEABLE PARTITION) L.L.		(SEISMIC) WEIGHT w_x	SEISMIC WT. TOTAL PER FLOOR	$w_x h_x^k$ (kip-ft)	C_{vx}	F_x
STORY	FLOOR			ROOF / FLR	PENTHOUSE	ROOF / FLR	PENTHOUSE					
---	---	---	---	---	---	---	---	---	---	---	---	---
---	ROOF	---	117.0 ft	2,084.93 kips	1,032.26 kips	0.0 kips	0.0 kips	3,117.19 kips	---	2.228E+06	0.2792	379.9 kips
9	9	13.0 ft	104.0 ft	1,995.08 kips	---	450.0 kips	---	2,445.08 kips	3,117.2 kips	1.485E+06	0.1861	253.2 kips
8	8	13.0 ft	91.0 ft	1,995.08 kips	---	450.0 kips	---	2,445.08 kips	5,562.3 kips	1.235E+06	0.1547	210.5 kips
7	7	13.0 ft	78.0 ft	1,995.08 kips	---	450.0 kips	---	2,445.08 kips	8,007.4 kips	9.986E+05	0.1251	170.2 kips
6	6	13.0 ft	65.0 ft	1,995.08 kips	---	450.0 kips	---	2,445.08 kips	10,452.5 kips	7.765E+05	0.0973	132.4 kips
5	5	13.0 ft	52.0 ft	1,995.08 kips	---	450.0 kips	---	2,445.08 kips	12,897.6 kips	5.707E+05	0.0715	97.3 kips
4	4	13.0 ft	39.0 ft	1,995.08 kips	---	450.0 kips	---	2,445.08 kips	15,342.7 kips	3.837E+05	0.0481	65.4 kips
3	3	13.0 ft	26.0 ft	1,995.08 kips	---	450.0 kips	---	2,445.08 kips	17,787.8 kips	2.193E+05	0.0275	37.4 kips
2	2	13.0 ft	13.0 ft	1,995.08 kips	---	450.0 kips	---	2,445.08 kips	20,232.9 kips	8.424E+04	0.0106	14.4 kips
1	GROUND	13.0 ft	0.0 ft	N.A.	---	N.A.	---	0.0 kips	22,678.0 kips	0.000E+00	0.0000	0.0 kips
				19,077.83 kips		3,600.0 kips		22,677.83 kips		7.981E+06		1,360.7 kips

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2-D MOMENT FRAME [MF A2 - F2] ANALYSIS LOAD SUMMARY

[13] REDUNDANCY COEFFICIENT, ρ, PER ASCE 7-02 [IBC 2003] Section 9.5.2.4 (p. 138)

- o SEISMIC DESIGN CATEGORY D
- o ARE THERE ONLY *SPECIAL* MOMENT FRAMES? YES
- o REDUNDANCY COEFFICIENT, *ρ* $r_i = 2 - \dfrac{20}{r_{max}\sqrt{A_i}}$ Equation 9.5.2.4.2-1 (p. 138)

$\rho_{max} =$ 0.19 *(FROM THE TABLE BELOW)*

- **o (DESIGN) REDUNDANCY COEFFICIENT, *ρ*** **$\rho =$ 1.00**

STORY NUMBER	AREA OF THE DIAPHRAGM	SEISMIC SHEAR IN EACH COLUMN OF THE MOMENT FRAME FOR EVERY STORY OF THE BUILDING											DESIGN STORY SHEAR	r_{max_i}	ρ_i
		COL 1	COL 2	COL 3	COL 4	COL 5	COL 6								
9	22,500 ft²	8 kips	20 kips	20 kips	20 kips	20 kips	8 kips						379.9 kips	0.0737	0.1909
8	22,500 ft²	17 kips	31 kips	31 kips	31 kips	31 kips	17 kips						633.1 kips	0.0686	0.0564
7	22,500 ft²	22 kips	42 kips	42 kips	42 kips	42 kips	22 kips						843.6 kips	0.0697	0.0870
6	22,500 ft²	27 kips	50 kips	50 kips	50 kips	50 kips	27 kips						1,013.8 kips	0.0690	0.0676
5	22,500 ft²	31 kips	56 kips	56 kips	56 kips	56 kips	31 kips						1,146.2 kips	0.0684	0.0507
4	22,500 ft²	32 kips	62 kips	62 kips	62 kips	62 kips	32 kips						1,243.5 kips	0.0698	0.0898
3	22,500 ft²	35 kips	65 kips	64 kips	64 kips	65 kips	35 kips						1,308.9 kips	0.0690	0.0676
2	22,500 ft²	38 kips	66 kips	65 kips	65 kips	66 kips	38 kips						1,346.3 kips	0.0681	0.0421
1	22,500 ft²	46 kips	62 kips	62 kips	62 kips	62 kips	46 kips						1,360.7 kips	0.0657	-0.0294

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2-D MOMENT FRAME [MF A2 - F2] ANALYSIS LOAD SUMMARY

[14] WIND LOADS PER THE INTERNATIONAL BUILDING CODE (IBC) 2003 [ASCE 7]

o *PER SECTION 1609.1.1 OF THE IBC 2003, SECTION 6 OF ASCE 7 SHALL BE USED TO DETERMINE THE WIND LOADS. THEREFORE, THE FOLLOWING DESIGN PARAMETERS ARE TAKEN FROM ASCE 7-02.*			Section 1609	(p. 283)
o OCCUPANCY CATEGORY	II		Table 1-1	(p. 4)
o IMPORTANCE FACTOR, I	1.00		Table 6-1	(p. 73)
o (3-SECOND GUST) BASIC WIND SPEED, V	85 mph		Figure 6-1	(p. 36)
o EXPOSURE CATEGORY	B		Section 6.5.6.3	(p. 28)
o WIND DIRECTIONALITY FACTOR, K_d, FOR MWFRS OF A BUILDING	0.85		Table 6-4	(p. 76)
o TOPOGRAPHIC FACTOR, K_{zt}	1.0		Section 6.5.7.2 Equation 6-3	(p. 30) (p. 30)
o ENCLOSURE CLASSIFICATION	*ENCLOSED* - SINCE NOT IN A HURICANE REGION AND THERE IS A SMALL CHANCE THAT WIND BORNE DEBRIS WILL PENETRATE THE WINDOWS AND CLADDING.		Section 6.5.9 Section 6.2	(p. 30) (p. 23)
o BUILDING TYPE	*SIMPLE DIAPHRAGM* - WIND LOADS ARE TRANSFERRED THROUGH THE ROOF AND FLOOR DIAPHRAGMS TO THE MWFRS (MOMENT FRAMES).		Section 6.2	(p. 24)
o APPROXIMATE BUILDING (MAX ALLOWED) PERIOD APPROXIMATE BUILDING FREQUENCY, n_1	1.770 sec 0.565 Hz	*(FROM SEISMIC CALCULATIONS)* *SINCE n1 < 1.0 THE BUILDING IS FLEXIBLE*	Section 6.2	(p. 24)
o DIRECTION THAT THE MOMENT FRAME RUNS PARALLEL WITH:	E-W			
o BUILDING WIDTH (DIMENSION PERPENDICULAR TO WIND DIRECTION),	150.0 ft	*ALONG THE E-W FACE*		
o BUILDING DEPTH (DIMENSION PARALLEL TO WIND DIRECTION), L	150.0 ft	*ALONG THE N-S FACE*		
o MEAN ROOF HEIGHT ABOVE GRADE, h	117.0 ft			
o VELOCITY PRESSURES	qz = 15.72 Kz lb/ft² qh = 16.19 lb/ft²		Section 6.5.10 Equation 6-15	(p. 31) (p. 31)
o GUST EFFECT FACTOR, G_f			Section 6.5.8.2	(p. 30)

From Table 6-2

c = 0.30	z_{min} = 30 ft
$\bar{\alpha}$ = 1/4	$\bar{\varepsilon}$ = 1/3
$\bar{b}$ = 0.45	ℓ = 320 ft
α = 7.0	z_g = 1200 ft

$g_Q = g_V$ = 3.4	Section 6.5.8.2
g_R = 4.051	Equation 6-9
$\bar{z}$ = 70.2 ft	Section 6.5.8.1
$I_{\bar{z}}$ = 0.265	Equation 6-5
$\bar{V}_{\bar{z}}$ = 67.75 ft/sec	Equation 6-14
$L_{\bar{z}}$ = 411.55	Equation 6-7

Equation 6-13

η_h = 4.488	R_h = 0.198
η_B = 5.754	R_B = 0.159
η_L = 5.754	R_L = 0.159

N_1 = 3.432	Equation 6-12
R_n = 0.064	Equation 6-11
Assumed Critical Damping Ratio, β = 0.05	
Resonant Response Factor, R = 0.156	Equation 6-10
Background Response, Q = 0.822	Equation 6-6
Gust Effect Factor, G_f = 0.837	Equation 6-8

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2-D MOMENT FRAME [MF A2 - F2] ANALYSIS LOAD SUMMARY

[15] WIND LOADS *CONTINUED*

o *WALL* EXTERNAL PRESSURE COEFFICIENTS, C_p

Plus signs signify pressures acting towards the surface.
Negative signs signify pressures acting away from the surface.

SURFACE	L / B	C_p
WINDWARD WALL	ALL	0.8
SIDE WALLS	1.00	-0.5
LEEWARD WALL	ALL	-0.7

Section 6.5.11.2.1 (p. 31)
Figure 6-6 (p. 51)

o *ROOF* EXTERNAL PRESSURE COEFFICIENT, C_p

h = 117.0 ft
L = 150.0 ft
h / L = 0.780 (Assume h/L > 1.0)
h / 2 = 58.5 ft

FOR: θ < 10°	
DISTANCE FROM LEADING EDGE	C_p
0 to h / 2	-1.3
> h / 2	-0.7

Section 6.5.11.2.1 (p. 31)
Figure 6-6 (p. 51)

o INTERNAL PRESSURE COEFFICIENT, GC_{pi}

GC_{pi} = ± 0.18

Section 6.5.11.1 (p. 31)
Figure 6-5 (p. 49)

		FLOOR LEVEL WIND LOADS PER EQUATION 6-19 AND PARAPET WIND LOADS PER EQUATION 6-20									*WINDWARD WALL*		*LEEWARD WALL*				
FLOOR NUMBER	ELEVATION	(BUILDING) WIND TRIBUTARY PARAMETERS					K_z	q_z (psf)	K_h	q_h (psf)	PRESSURE (psf) WITH		PRESSURE (psf) WITH		TOTAL WIND SHEAR WITH		DESIGN WIND SHEAR PER FLOOR
		TOP ELEV.	BOT. ELEV.	HEIGHT	WIDTH	AREA					($+GC_{pi}$)	($-GC_{pi}$)	($+GC_{pi}$)	($-GC_{pi}$)	($+GC_{pi}$)	($-GC_{pi}$)	
PENTHOUSE	---	130.0 ft	117.0 ft	13.0 ft	90.0 ft	1,170 ft²	1.07	16.82	1.03	16.19	10.0	14.18	-12.4	-10.0	26.2 kips	28.3 kips	28.3 kips
PARAPET	---	120.5 ft	117.0 ft	3.5 ft	150.0 ft	525 ft²	1.04	16.35	1.03	16.19	29.43		-17.99		24.9 kips	24.9 kips	24.9 kips
ROOF	117.0 ft	117.0 ft	110.5 ft	6.5 ft	150.0 ft	975 ft²	1.03	16.19	1.03	16.19	10.0	13.76	-12.4	-10.0	21.8 kips	23.2 kips	23.2 kips
9	104.0 ft	110.5 ft	97.5 ft	13.0 ft	150.0 ft	1,950 ft²	1.02	16.04	1.03	16.19	10.0	13.65	-12.4	-10.0	43.7 kips	46.1 kips	46.1 kips
8	91.0 ft	97.5 ft	84.5 ft	13.0 ft	150.0 ft	1,950 ft²	0.98	15.41	1.03	16.19	10.0	13.23	-12.4	-10.0	43.7 kips	45.3 kips	45.3 kips
7	78.0 ft	84.5 ft	71.5 ft	13.0 ft	150.0 ft	1,950 ft²	0.94	14.78	1.03	16.19	10.0	12.81	-12.4	-10.0	43.7 kips	44.5 kips	44.5 kips
6	65.0 ft	71.5 ft	58.5 ft	13.0 ft	150.0 ft	1,950 ft²	0.90	14.15	1.03	16.19	10.0	12.39	-12.4	-10.0	43.7 kips	43.7 kips	43.7 kips
5	52.0 ft	58.5 ft	45.5 ft	13.0 ft	150.0 ft	1,950 ft²	0.85	13.36	1.03	16.19	10.0	11.86	-12.4	-10.0	43.7 kips	42.6 kips	42.6 kips
4	39.0 ft	45.5 ft	32.5 ft	13.0 ft	150.0 ft	1,950 ft²	0.79	12.42	1.03	16.19	10.0	11.23	-12.4	-10.0	43.7 kips	41.4 kips	41.4 kips
3	26.0 ft	32.5 ft	19.5 ft	13.0 ft	150.0 ft	1,950 ft²	0.72	11.32	1.03	16.19	10.0	10.49	-12.4	-10.0	43.7 kips	40.0 kips	40.0 kips
2	13.0 ft	19.5 ft	6.5 ft	13.0 ft	150.0 ft	1,950 ft²	0.62	9.75	1.03	16.19	10.0	10.0	-12.4	-10.0	43.7 kips	39.0 kips	39.0 kips
GROUND	0.0 ft	6.5 ft	0.0 ft	6.5 ft	150.0 ft	975 ft²	0.57	8.96	1.03	16.19	10.0	10.0	-12.4	-10.0	21.8 kips	19.5 kips	N.A.
															444 kips	439 kips	

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2-D MOMENT FRAME [MF A2 - F2] ANALYSIS LOAD SUMMARY

[16] WIND LOADS *CONTINUED*

- o BUIDLING WIDTH (DIMENSION PERPENDICULAR TO WIND DIRECTION), B 150.0 ft *ALONG THE E-W FACE*
- o ECCENTRICITY ALONG THE WINDWARD FACE OF THE BUILDING, e_x ± 22.5 ft Figure 6-9 (p. 54)
- o BUIDLING DEPTH (DIMENSION PARALLEL TO WIND DIRECTION), L 150.0 ft *ALONG THE N-S FACE*
- o ECCENTRICITY ALONG THE SIDEWALL OF THE BUILDING, e_y ± 22.5 ft Figure 6-9 (p. 54)
- o ECCENTRICITY FOR FLEXIBLE STRUCTURES (e_x AND e_y) ± 39.38 ft Equation 6-21 (p. 33)
- o *TO SIMPLIFY THE TORSIONAL MOMENT CALCULATIONS, THE BUILDING PLAN IS ASSUMED TO BE SQUARE SO THAT THE WIND SHEAR LOADS CAN BE CALCULATED ONCE ALONG ONE PRINCIPAL DIRECTION AND THEN USED IN BOTH PRINCIPAL DIRECTIONS.*
- o TORSION LOADS ARE SIMPLIFIED SO THAT THE MAXIMUM SHEAR PER MOMENT FRAME = *STORY SHEAR x (1 / NO. OF MOMENT FRAMES + 0.002 x ECCENTRICITY)*

FLOOR NUMBER	ELEVATION	ASCE 7 Figure 6-9 ***CASE #1*** SHEAR LOADS DUE TO: SHEAR	TORSION	TOTAL STORY SHEAR	ASCE 7 Figure 6-9 ***CASE #2*** SHEAR LOADS DUE TO: SHEAR	TORSION	TOTAL STORY SHEAR	ASCE 7 Figure 6-9 ***CASE #3*** SHEAR LOADS DUE TO: SHEAR	TORSION	TOTAL STORY SHEAR	ASCE 7 Figure 6-9 ***CASE #4*** SHEAR LOADS DUE TO: SHEAR	TORSION	TOTAL STORY SHEAR
PENTHOUSE	---	28.3 kips	± 0.0 kips	28.3 kips	21.2 kips	± 0.0 kips	21.2 kips	21.2 kips	± 0.0 kips	21.2 kips	15.9 kips	± 0.0 kips	15.9 kips
PARAPET	---	24.9 kips	± 0.0 kips	24.9 kips	18.7 kips	± 1.5 kips	18.7 kips	18.7 kips	± 0.0 kips	18.7 kips	14.0 kips	± 1.1 kips	14.0 kips
ROOF	117.0 ft	23.2 kips	± 0.0 kips	23.2 kips	17.4 kips	± 1.4 kips	17.4 kips	17.4 kips	± 0.0 kips	17.4 kips	13.1 kips	± 1.0 kips	13.1 kips
9	104.0 ft	46.1 kips	± 0.0 kips	46.1 kips	34.6 kips	± 2.7 kips	34.6 kips	34.6 kips	± 0.0 kips	34.6 kips	26.0 kips	± 2.0 kips	26.0 kips
8	91.0 ft	45.3 kips	± 0.0 kips	45.3 kips	34.0 kips	± 2.7 kips	34.0 kips	34.0 kips	± 0.0 kips	34.0 kips	25.5 kips	± 2.0 kips	25.5 kips
7	78.0 ft	44.5 kips	± 0.0 kips	44.5 kips	33.4 kips	± 2.6 kips	33.4 kips	33.4 kips	± 0.0 kips	33.4 kips	25.1 kips	± 2.0 kips	25.1 kips
6	65.0 ft	43.7 kips	± 0.0 kips	43.7 kips	32.8 kips	± 2.6 kips	32.8 kips	32.8 kips	± 0.0 kips	32.8 kips	24.6 kips	± 1.9 kips	24.6 kips
5	52.0 ft	42.6 kips	± 0.0 kips	42.6 kips	32.0 kips	± 2.5 kips	32.0 kips	32.0 kips	± 0.0 kips	32.0 kips	24.0 kips	± 1.9 kips	24.0 kips
4	39.0 ft	41.4 kips	± 0.0 kips	41.4 kips	31.1 kips	± 2.4 kips	31.1 kips	31.1 kips	± 0.0 kips	31.1 kips	23.3 kips	± 1.8 kips	23.3 kips
3	26.0 ft	40.0 kips	± 0.0 kips	40.0 kips	30.0 kips	± 2.4 kips	30.0 kips	30.0 kips	± 0.0 kips	30.0 kips	22.5 kips	± 1.8 kips	22.5 kips
2	13.0 ft	39.0 kips	± 0.0 kips	39.0 kips	29.3 kips	± 2.3 kips	29.3 kips	29.3 kips	± 0.0 kips	29.3 kips	22.0 kips	± 1.7 kips	22.0 kips
GROUND	0.0 ft	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.
				419.0 kips			*314.5 kips*			*314.5 kips*			*236.0 kips*

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2-D MOMENT FRAME [MF A2 - F2] ANALYSIS LOAD SUMMARY

[17] 9-STORY BUILDINGS MOMENT FRAME ANALYSIS LOAD SUMMARY

o SUMMARY:

TOTAL NUMBER OF (IDENTICAL) MOMENT FRAMES ALONG THE E-W DIRECTION	4
NUMBER OF STORIES IN THE MOMENT FRAME, N_S	9 STORIES
NUMBER OF BAYS IN THE MOMENT FRAME, N_B	5 BAYS
DIRECTION THAT THE *MOMENT FRAME* RUNS PARALLEL WITH:	E-W
LOCATION OF THE MOMENT FRAME WRT THE BUILDING PERIMETER:	INTERIOR
DOES THIS FRAME SUPPORT PART OF THE PENTHOUSE GRAVITY LOADS?	YES
DISTANCE TO THE CLOSEST (GRAVITY/MOMENT) FRAME:	30.0 ft
DISTANCE TO THE CLOSEST (GRAVITY/MOMENT) FRAME ON THE OTHER SIDE:	30.0 ft
DESIGN SEISMIC BASE SHEAR	0.060 W
BASIC WIND SPEED	85 mph

o DESIGN ASSUMPTION: *BUILDING HAS "RIGID DIAPHRAGMS" (PER SEISMIC DESIGN), THE BUILDING IS CONSIDERED A "SIMPLE DIAPHRAGM" BUILDING (PER WIND DESIGN), AND ALL OF THE MOMENT FRAMES HAVE THE SAME RIGIDITY AT EACH FLOOR LEVEL. THEREFORE, ALL LATERAL LOADS ARE DISTRIBUTED EQUALLY AMONG ALL OF THE MOMENT FRAMES IN EACH PRINCIPAL DIRECTION OF THE BUILDING.*

NUMBER		STORY HEIGHT	FLOOR ELEVATION h_x	DESIGN SHEAR LOAD AT EACH FLOOR LEVEL PER MOMENT FRAME (THESE ARE THE POINT LOADS THAT SHALL BE USED IN THE 2-D MODEL)				
				SEISMIC LOAD	WIND LOAD			
STORY	FLOOR				CASE #1	CASE #2	CASE #3	CASE #4
---	---	---	---	---	---	---	---	---
---	ROOF	---	117.0 ft	95.0 kips	19.1 kips	17.2 kips	14.3 kips	12.9 kips
9	9	13.0 ft	104.0 ft	63.3 kips	11.5 kips	11.4 kips	8.7 kips	8.5 kips
8	8	13.0 ft	91.0 ft	52.6 kips	11.3 kips	11.2 kips	8.5 kips	8.4 kips
7	7	13.0 ft	78.0 ft	42.6 kips	11.1 kips	11.0 kips	8.4 kips	8.3 kips
6	6	13.0 ft	65.0 ft	33.1 kips	10.9 kips	10.8 kips	8.2 kips	8.1 kips
5	5	13.0 ft	52.0 ft	24.3 kips	10.7 kips	10.5 kips	8.0 kips	7.9 kips
4	4	13.0 ft	39.0 ft	16.4 kips	10.4 kips	10.2 kips	7.8 kips	7.6 kips
3	3	13.0 ft	26.0 ft	9.4 kips	10.0 kips	9.9 kips	7.5 kips	7.4 kips
2	2	13.0 ft	13.0 ft	3.6 kips	9.8 kips	9.6 kips	7.3 kips	7.2 kips
1	GROUND	13.0 ft	0.0 ft	N.A.	N.A.	N.A.	N.A.	N.A.
				340.3 kips	*104.8 kips*	*101.8 kips*	*78.7 kips*	*76.3 kips*

Appendix B

Building Design 9A Calculations

This appendix consists of the design calculations that were performed for building Design 9A which is the 9-story building that used low strength materials in the columns (F_{yc} = 46 ksi and f'_c = 4 ksi) and a relatively low column d/t ratio. The final RCFT column and wide flange girder sections are presented in Chapter 5. The linear elastic analysis consisted of taking the nominal loads that were generated in Appendix A and factoring them per the applicable LRFD load combination. The calculation for the stability coefficient, θ, and the moment magnification factor, B_2, were performed for each load combination that has lateral loads (wind and seismic load combinations #4, #5, and #6) and are included in this appendix. The maximum interaction value for each column is listed at the end of this appendix along with its respective load combination.

The load combinations that were used in this building design are listed below for reference:

1. $1.4D$
2. $1.2D + 1.6L + 0.5L_R$
3. $1.2D + 1.6L_R + f_1L$
4. $1.2D + 1.6L_R + 0.8W$
5. $1.2D + 1.6W + f_1L + 0.5L_R$
6. $1.2D + 1.0E + f_1L$

Where: $f_1 = 0.5$
$E = \rho Q_E + 0.2S_{DS}D'$
D' = seismic weight

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SUBJECT

DESIGN PARAMETERS SUMMARY

MOMENT FRAME
MF A2 - F2

o DESIGN INPUTS:

o TOAL NUMBER OF COLUMNS BEING ANALYZED 54

o YIELD STRENGTH: HSS, F_y = 46 ksi
CONCRETE REINFORCEMENT, F_{yr} = 0 ksi

o MODULUS OF ELASTICITY: HSS, E_s = 29,000 ksi
CONCRETE REINFORCEMENT, E_{cr} = 29,000 ksi

o MINIMUM CONCRETE COMPRESSIVE STRENGTH f'_c = 4.0 ksi

o CONCRETE DENSITY w = 145 lb/ft³

o CONCRETE REINFORCEMENT AREA, A_{sr} = 0.0 in²
I_{xxr} = 0.0 in^4
I_{yyr} = 0.0 in^4
Z_{xxr} = 0.0 in³
Z_{yyr} = 0.0 in³

o RESISTANCE FACTORS AXIAL COMPRESSION, ϕ_c = 0.75
FLEXURAL BENDING, ϕ_b = 0.90

o SEISMIC PARAMETERS REDUNDANCY COEFFICIENT, ρ = 1.00
VERTICAL SEISMIC "FACTOR," $0.2S_{DS}$ = 0.20
ORTHOGONAL LOAD FACTOR ALONG Y-AXIS OF SHARED COLUMNS = 0.30
FACTOR TO ACCOUNT FOR *5% ACCIDENTAL TORSION* ("SIMPLIFIED APPROACH"...) = 0.025

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SUBJECT

B2 CALCULATION - *FOR BENDING ALONG THE X-AXIS OF THE COLUMN*

MOMENT FRAME
MF A2 - F2

o ***LOAD COMBINATION =*** **1.2D + 1.6Lr + 0.8W** *(L.C. # 4)*

STORY NUMBER	LENGTH *(STORY HEIGHT)* L	DUE TO FORCES THAT CAUSE BENDING ALONG THE X-AXIS OF THE MOMENT FRAME COLUMNS: TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips): DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY — *LOAD FACTOR*: *D.L.* 1.2	*L.L.* 0	*ROOF L.L.* 1.6	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0	TOTAL ΣP_{ui} (kips)	**STORY $B_{2i\_X\text{-}AXIS}$**
9	13.0 ft	380 kips	0.3 in	3,020	0	450	3,118	3,624.0	0.0	720.0	0.0	4,344.0	**1.022**
8	13.0 ft	633 kips	0.42 in	5,015	1,575	450	5,563	6,018.0	0.0	720.0	0.0	6,738.0	**1.030**
7	13.0 ft	844 kips	0.51 in	7,010	3,150	450	8,008	8,412.0	0.0	720.0	0.0	9,132.0	**1.037**
6	13.0 ft	1,014 kips	0.56 in	9,005	4,725	450	10,453	10,806.0	0.0	720.0	0.0	11,526.0	**1.043**
5	13.0 ft	1,146 kips	0.6 in	11,000	6,300	450	12,898	13,200.0	0.0	720.0	0.0	13,920.0	**1.049**
4	13.0 ft	1,244 kips	0.6 in	12,995	7,875	450	15,342	15,594.0	0.0	720.0	0.0	16,314.0	**1.053**
3	13.0 ft	1,309 kips	0.57 in	14,990	9,450	450	17,788	17,988.0	0.0	720.0	0.0	18,708.0	**1.055**
2	13.0 ft	1,346 kips	0.53 in	16,985	11,025	450	20,233	20,382.0	0.0	720.0	0.0	21,102.0	**1.056**
1	13.0 ft	1,361 kips	0.3 in	18,980	12,600	450	22,678	22,776.0	0.0	720.0	0.0	23,496.0	**1.034**

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SUBJECT **B2 CALCULATION - *FOR BENDING ALONG THE Y-AXIS OF THE COLUMN*** — MOMENT FRAME MF A2 - F2

o ***LOAD COMBINATION =*** **1.2D + 1.6Lr + 0.8W**

		DUE TO FORCES THAT CAUSE BENDING ALONG THE Y-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					
								LOAD FACTOR					
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0	*ROOF L.L.* 1.6	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0	TOTAL ΣP_{ui} (kips)	**STORY $B_{2i\_Y\text{-}AXIS}$**
9	13.0 ft	380 kips	0.3 in	3,020	0	450	3,118	3,624.0	0.0	720.0	0.0	4,344.0	**1.022**
8	13.0 ft	633 kips	0.42 in	5,015	1,575	450	5,563	6,018.0	0.0	720.0	0.0	6,738.0	**1.030**
7	13.0 ft	844 kips	0.51 in	7,010	3,150	450	8,008	8,412.0	0.0	720.0	0.0	9,132.0	**1.037**
6	13.0 ft	1,014 kips	0.56 in	9,005	4,725	450	10,453	10,806.0	0.0	720.0	0.0	11,526.0	**1.043**
5	13.0 ft	1,146 kips	0.6 in	11,000	6,300	450	12,898	13,200.0	0.0	720.0	0.0	13,920.0	**1.049**
4	13.0 ft	1,244 kips	0.6 in	12,995	7,875	450	15,342	15,594.0	0.0	720.0	0.0	16,314.0	**1.053**
3	13.0 ft	1,309 kips	0.57 in	14,990	9,450	450	17,788	17,988.0	0.0	720.0	0.0	18,708.0	**1.055**
2	13.0 ft	1,346 kips	0.53 in	16,985	11,025	450	20,233	20,382.0	0.0	720.0	0.0	21,102.0	**1.056**
1	13.0 ft	1,361 kips	0.3 in	18,980	12,600	450	22,678	22,776.0	0.0	720.0	0.0	23,496.0	**1.034**

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SUBJECT

STABILITY COEFFICIENT ALONG COLUMN X-AXIS, θ_x

MOMENT FRAME
MF A2 - F2

- ***LOAD COMBINATION:*** 1.2D + 1.6Lr + 0.8W
- ***DEFLECTION AMPLIFICATION FACTOR:*** C_d = 5.5
- ***(SEISMIC) IMPORTANCE FACTOR*** I_E = 1.0
- ***MOMENT FRAME RESISTS WHAT % OF THE TOTAL SEISMIC SHEAR TO THE BUIDLING?*** 25%

		DUE TO FORCES THAT CAUSE BENDING ALONG THE X-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					STABILITY COEFFICIENT PER STORY θ_i
								LOAD FACTOR					
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO ANY HORIZONTAL LOAD ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0	*ROOF L.L.* 1.6	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0	TOTAL ΣP_{ui} (kips)	
9	13.0 ft	380 kips	0.3 in	3,020	0	450	3,118	3,624.0	0.0	720.0	0.0	4,344.0	**0.022**
8	13.0 ft	633 kips	0.42 in	5,015	1,575	450	5,563	6,018.0	0.0	720.0	0.0	6,738.0	**0.029**
7	13.0 ft	844 kips	0.51 in	7,010	3,150	450	8,008	8,412.0	0.0	720.0	0.0	9,132.0	**0.035**
6	13.0 ft	1,014 kips	0.56 in	9,005	4,725	450	10,453	10,806.0	0.0	720.0	0.0	11,526.0	**0.041**
5	13.0 ft	1,146 kips	0.6 in	11,000	6,300	450	12,898	13,200.0	0.0	720.0	0.0	13,920.0	**0.047**
4	13.0 ft	1,244 kips	0.6 in	12,995	7,875	450	15,342	15,594.0	0.0	720.0	0.0	16,314.0	**0.050**
3	13.0 ft	1,309 kips	0.57 in	14,990	9,450	450	17,788	17,988.0	0.0	720.0	0.0	18,708.0	**0.052**
2	13.0 ft	1,346 kips	0.53 in	16,985	11,025	450	20,233	20,382.0	0.0	720.0	0.0	21,102.0	**0.053**
1	13.0 ft	1,361 kips	0.3 in	18,980	12,600	450	22,678	22,776.0	0.0	720.0	0.0	23,496.0	**0.033**

GG Engineering	JOB NO. 9 - STORY BUILDINGS	BY SMG	DATE 9/16/04	SHEET NO. ___ OF ___
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SUBJECT

STABILITY COEFFICIENT ALONG COLUMN Y-AXIS, θ_y

MOMENT FRAME
MF A2 - F2

- ***LOAD COMBINATION:*** 1.2D + 1.6Lr + 0.8W
- ***DEFLECTION AMPLIFICATION FACTOR:*** C_d = 5.5
- ***(SEISMIC) IMPORTANCE FACTOR*** I_E = 1.0
- ***MOMENT FRAME RESISTS WHAT % OF THE TOTAL SEISMIC SHEAR TO THE BUIDLING?*** 25%

		DUE TO FORCES THAT CAUSE BENDING ALONG THE Y-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					STABILITY COEFFICIENT PER STORY θ_i
								LOAD FACTOR				TOTAL ΣP_{ui} (kips)	
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO ANY HORIZONTAL LOAD ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0	*ROOF L.L.* 1.6	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0		
9	13.0 ft	380 kips	0.3 in	3,020	0	450	3,118	3,624.0	0.0	720.0	0.0	4,344.0	**0.022**
8	13.0 ft	633 kips	0.42 in	5,015	1,575	450	5,563	6,018.0	0.0	720.0	0.0	6,738.0	**0.029**
7	13.0 ft	844 kips	0.51 in	7,010	3,150	450	8,008	8,412.0	0.0	720.0	0.0	9,132.0	**0.035**
6	13.0 ft	1,014 kips	0.56 in	9,005	4,725	450	10,453	10,806.0	0.0	720.0	0.0	11,526.0	**0.041**
5	13.0 ft	1,146 kips	0.6 in	11,000	6,300	450	12,898	13,200.0	0.0	720.0	0.0	13,920.0	**0.047**
4	13.0 ft	1,244 kips	0.6 in	12,995	7,875	450	15,342	15,594.0	0.0	720.0	0.0	16,314.0	**0.050**
3	13.0 ft	1,309 kips	0.57 in	14,990	9,450	450	17,788	17,988.0	0.0	720.0	0.0	18,708.0	**0.052**
2	13.0 ft	1,346 kips	0.53 in	16,985	11,025	450	20,233	20,382.0	0.0	720.0	0.0	21,102.0	**0.053**
1	13.0 ft	1,361 kips	0.3 in	18,980	12,600	450	22,678	22,776.0	0.0	720.0	0.0	23,496.0	**0.033**

GG Engineering	JOB NO. 9 - STORY BUILDINGS	BY SMG	DATE 9/16/04	SHEET NO. ___ OF ___
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SUBJECT

B2 CALCULATION - *FOR BENDING ALONG THE X-AXIS OF THE COLUMN*

MOMENT FRAME
MF A2 - F2

o ***LOAD COMBINATION =*** **1.2D + 0.5L + 0.5Lr + 1.6W** *(L.C. # 5)*

		DUE TO FORCES THAT CAUSE BENDING ALONG THE X-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					
								LOAD FACTOR					
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0.5	*ROOF L.L.* 0.5	*SEISMIC VERTICAL* $0.2S_{DS} = 0$	TOTAL ΣP_{ui} (kips)	**STORY $B_{2i\_X\text{-}AXIS}$**
9	13.0 ft	380 kips	0.3 in	3,020	0	450	3,118	3,624.0	0.0	225.0	0.0	3,849.0	**1.020**
8	13.0 ft	633 kips	0.42 in	5,015	1,575	450	5,563	6,018.0	787.5	225.0	0.0	7,030.5	**1.031**
7	13.0 ft	844 kips	0.51 in	7,010	3,150	450	8,008	8,412.0	1,575.0	225.0	0.0	10,212.0	**1.041**
6	13.0 ft	1,014 kips	0.56 in	9,005	4,725	450	10,453	10,806.0	2,362.5	225.0	0.0	13,393.5	**1.050**
5	13.0 ft	1,146 kips	0.6 in	11,000	6,300	450	12,898	13,200.0	3,150.0	225.0	0.0	16,575.0	**1.059**
4	13.0 ft	1,244 kips	0.6 in	12,995	7,875	450	15,342	15,594.0	3,937.5	225.0	0.0	19,756.5	**1.065**
3	13.0 ft	1,309 kips	0.57 in	14,990	9,450	450	17,788	17,988.0	4,725.0	225.0	0.0	22,938.0	**1.068**
2	13.0 ft	1,346 kips	0.53 in	16,985	11,025	450	20,233	20,382.0	5,512.5	225.0	0.0	26,119.5	**1.071**
1	13.0 ft	1,361 kips	0.3 in	18,980	12,600	450	22,678	22,776.0	6,300.0	225.0	0.0	29,301.0	**1.043**

GG Engineering	JOB NO. 9 - STORY BUILDINGS	BY SMG	DATE 9/16/04	SHEET NO. ___ OF ___
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SUBJECT **B2 CALCULATION - *FOR BENDING ALONG THE Y-AXIS OF THE COLUMN*** — **MOMENT FRAME MF A2 - F2**

o ***LOAD COMBINATION =*** **1.2D + 0.5L + 0.5Lr + 1.6W**

STORY NUMBER	LENGTH *(STORY HEIGHT)* L	DUE TO FORCES THAT CAUSE BENDING ALONG THE Y-AXIS OF THE MOMENT FRAME COLUMNS: TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips): DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY — *LOAD FACTOR*: *D.L.* 1.2	*L.L.* 0.5	*ROOF L.L.* 0.5	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0	TOTAL ΣP_{ui} (kips)	**STORY $B_{2i\_Y\text{-}AXIS}$**
9	13.0 ft	380 kips	0.3 in	3,020	0	450	3,118	3,624.0	0.0	225.0	0.0	3,849.0	**1.020**
8	13.0 ft	633 kips	0.42 in	5,015	1,575	450	5,563	6,018.0	787.5	225.0	0.0	7,030.5	**1.031**
7	13.0 ft	844 kips	0.51 in	7,010	3,150	450	8,008	8,412.0	1,575.0	225.0	0.0	10,212.0	**1.041**
6	13.0 ft	1,014 kips	0.56 in	9,005	4,725	450	10,453	10,806.0	2,362.5	225.0	0.0	13,393.5	**1.050**
5	13.0 ft	1,146 kips	0.6 in	11,000	6,300	450	12,898	13,200.0	3,150.0	225.0	0.0	16,575.0	**1.059**
4	13.0 ft	1,244 kips	0.6 in	12,995	7,875	450	15,342	15,594.0	3,937.5	225.0	0.0	19,756.5	**1.065**
3	13.0 ft	1,309 kips	0.57 in	14,990	9,450	450	17,788	17,988.0	4,725.0	225.0	0.0	22,938.0	**1.068**
2	13.0 ft	1,346 kips	0.53 in	16,985	11,025	450	20,233	20,382.0	5,512.5	225.0	0.0	26,119.5	**1.071**
1	13.0 ft	1,361 kips	0.3 in	18,980	12,600	450	22,678	22,776.0	6,300.0	225.0	0.0	29,301.0	**1.043**

GG Engineering	JOB NO. 9 - STORY BUILDINGS	BY SMG	DATE 9/16/04	SHEET NO. ___ OF ___
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SUBJECT

STABILITY COEFFICIENT ALONG COLUMN X-AXIS, θ_x

MOMENT FRAME
MF A2 - F2

- o ***LOAD COMBINATION:*** 1.2D + 0.5L + 0.5Lr + 1.6W
- o ***DEFLECTION AMPLIFICATION FACTOR:*** C_d = 5.5
- o ***(SEISMIC) IMPORTANCE FACTOR*** I_E = 1.0
- o ***MOMENT FRAME RESISTS WHAT % OF THE TOTAL SEISMIC SHEAR TO THE BUIDLING?*** 25%

		DUE TO FORCES THAT CAUSE BENDING ALONG THE X-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					STABILITY COEFFICIENT PER STORY θ_i
								LOAD FACTOR					
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO ANY HORIZONTAL LOAD ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0.5	*ROOF L.L.* 0.5	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0	TOTAL ΣP_{ui} (kips)	
9	13.0 ft	380 kips	0.3 in	3,020	0	450	3,118	3,624.0	0.0	225.0	0.0	3,849.0	**0.019**
8	13.0 ft	633 kips	0.42 in	5,015	1,575	450	5,563	6,018.0	787.5	225.0	0.0	7,030.5	**0.030**
7	13.0 ft	844 kips	0.51 in	7,010	3,150	450	8,008	8,412.0	1,575.0	225.0	0.0	10,212.0	**0.040**
6	13.0 ft	1,014 kips	0.56 in	9,005	4,725	450	10,453	10,806.0	2,362.5	225.0	0.0	13,393.5	**0.047**
5	13.0 ft	1,146 kips	0.6 in	11,000	6,300	450	12,898	13,200.0	3,150.0	225.0	0.0	16,575.0	**0.056**
4	13.0 ft	1,244 kips	0.6 in	12,995	7,875	450	15,342	15,594.0	3,937.5	225.0	0.0	19,756.5	**0.061**
3	13.0 ft	1,309 kips	0.57 in	14,990	9,450	450	17,788	17,988.0	4,725.0	225.0	0.0	22,938.0	**0.064**
2	13.0 ft	1,346 kips	0.53 in	16,985	11,025	450	20,233	20,382.0	5,512.5	225.0	0.0	26,119.5	**0.066**
1	13.0 ft	1,361 kips	0.3 in	18,980	12,600	450	22,678	22,776.0	6,300.0	225.0	0.0	29,301.0	**0.041**

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SUBJECT

STABILITY COEFFICIENT ALONG COLUMN Y-AXIS, θ_y

MOMENT FRAME
MF A2 - F2

- ***LOAD COMBINATION:*** 1.2D + 0.5L + 0.5Lr + 1.6W
- ***DEFLECTION AMPLIFICATION FACTOR:*** $C_d = 5.5$
- ***(SEISMIC) IMPORTANCE FACTOR*** $I_E = 1.0$
- ***MOMENT FRAME RESISTS WHAT % OF THE TOTAL SEISMIC SHEAR TO THE BUIDLING?*** 25%

		DUE TO FORCES THAT CAUSE BENDING ALONG THE Y-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					STABILITY COEFFICIENT PER STORY θ_i
								LOAD FACTOR					
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO ANY HORIZONTAL LOAD ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0.5	*ROOF L.L.* 0.5	*SEISMIC VERTICAL* $0.2S_{DS} = 0$	TOTAL ΣP_{ui} (kips)	
9	13.0 ft	380 kips	0.3 in	3,020	0	450	3,118	3,624.0	0.0	225.0	0.0	3,849.0	**0.019**
8	13.0 ft	633 kips	0.42 in	5,015	1,575	450	5,563	6,018.0	787.5	225.0	0.0	7,030.5	**0.030**
7	13.0 ft	844 kips	0.51 in	7,010	3,150	450	8,008	8,412.0	1,575.0	225.0	0.0	10,212.0	**0.040**
6	13.0 ft	1,014 kips	0.56 in	9,005	4,725	450	10,453	10,806.0	2,362.5	225.0	0.0	13,393.5	**0.047**
5	13.0 ft	1,146 kips	0.6 in	11,000	6,300	450	12,898	13,200.0	3,150.0	225.0	0.0	16,575.0	**0.056**
4	13.0 ft	1,244 kips	0.6 in	12,995	7,875	450	15,342	15,594.0	3,937.5	225.0	0.0	19,756.5	**0.061**
3	13.0 ft	1,309 kips	0.57 in	14,990	9,450	450	17,788	17,988.0	4,725.0	225.0	0.0	22,938.0	**0.064**
2	13.0 ft	1,346 kips	0.53 in	16,985	11,025	450	20,233	20,382.0	5,512.5	225.0	0.0	26,119.5	**0.066**
1	13.0 ft	1,361 kips	0.3 in	18,980	12,600	450	22,678	22,776.0	6,300.0	225.0	0.0	29,301.0	**0.041**

GG Engineering	JOB NO. 9 - STORY BUILDINGS	BY SMG	DATE 9/16/04	SHEET NO. ___ OF ___
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SUBJECT **B2 CALCULATION - *FOR BENDING ALONG THE X-AXIS OF THE COLUMN*** ***MOMENT FRAME MF A2 - F2***

o ***LOAD COMBINATION =*** **1.2D + 0.5L + 1.0E** *(L.C. # 6)*

		DUE TO FORCES THAT CAUSE BENDING ALONG THE X-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					
								LOAD FACTOR					
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0.5	*ROOF L.L.* 0	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0.2	TOTAL ΣP_{ui} (kips)	**STORY $B_{2i\_X\text{-}AXIS}$**
9	13.0 ft	380 kips	0.3 in	3,020	0	450	3,118	3,624.0	0.0	0.0	623.6	4,247.6	**1.022**
8	13.0 ft	633 kips	0.42 in	5,015	1,575	450	5,563	6,018.0	787.5	0.0	1,112.6	7,918.1	**1.035**
7	13.0 ft	844 kips	0.51 in	7,010	3,150	450	8,008	8,412.0	1,575.0	0.0	1,601.6	11,588.6	**1.047**
6	13.0 ft	1,014 kips	0.56 in	9,005	4,725	450	10,453	10,806.0	2,362.5	0.0	2,090.6	15,259.1	**1.057**
5	13.0 ft	1,146 kips	0.6 in	11,000	6,300	450	12,898	13,200.0	3,150.0	0.0	2,579.6	18,929.6	**1.068**
4	13.0 ft	1,244 kips	0.6 in	12,995	7,875	450	15,342	15,594.0	3,937.5	0.0	3,068.4	22,599.9	**1.075**
3	13.0 ft	1,309 kips	0.57 in	14,990	9,450	450	17,788	17,988.0	4,725.0	0.0	3,557.6	26,270.6	**1.079**
2	13.0 ft	1,346 kips	0.53 in	16,985	11,025	450	20,233	20,382.0	5,512.5	0.0	4,046.6	29,941.1	**1.082**
1	13.0 ft	1,361 kips	0.3 in	18,980	12,600	450	22,678	22,776.0	6,300.0	0.0	4,535.6	33,611.6	**1.050**

GG Engineering	JOB NO. 9 - STORY BUILDINGS	BY SMG	DATE 9/16/04	SHEET NO. ___ OF ___
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SUBJECT **B2 CALCULATION - *FOR BENDING ALONG THE Y-AXIS OF THE COLUMN*** **MOMENT FRAME MF A2 - F2**

o ***LOAD COMBINATION =*** **1.2D + 0.5L + 1.0E**

		DUE TO FORCES THAT CAUSE BENDING ALONG THE Y-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					
								LOAD FACTOR					
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0.5	*ROOF L.L.* 0	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0.2	TOTAL ΣP_{ui} (kips)	**STORY $B_{2i\_Y\text{-}AXIS}$**
9	13.0 ft	380 kips	0.3 in	3,020	0	450	3,118	3,624.0	0.0	0.0	623.6	4,247.6	**1.022**
8	13.0 ft	633 kips	0.42 in	5,015	1,575	450	5,563	6,018.0	787.5	0.0	1,112.6	7,918.1	**1.035**
7	13.0 ft	844 kips	0.51 in	7,010	3,150	450	8,008	8,412.0	1,575.0	0.0	1,601.6	11,588.6	**1.047**
6	13.0 ft	1,014 kips	0.56 in	9,005	4,725	450	10,453	10,806.0	2,362.5	0.0	2,090.6	15,259.1	**1.057**
5	13.0 ft	1,146 kips	0.6 in	11,000	6,300	450	12,898	13,200.0	3,150.0	0.0	2,579.6	18,929.6	**1.068**
4	13.0 ft	1,244 kips	0.6 in	12,995	7,875	450	15,342	15,594.0	3,937.5	0.0	3,068.4	22,599.9	**1.075**
3	13.0 ft	1,309 kips	0.57 in	14,990	9,450	450	17,788	17,988.0	4,725.0	0.0	3,557.6	26,270.6	**1.079**
2	13.0 ft	1,346 kips	0.53 in	16,985	11,025	450	20,233	20,382.0	5,512.5	0.0	4,046.6	29,941.1	**1.082**
1	13.0 ft	1,361 kips	0.3 in	18,980	12,600	450	22,678	22,776.0	6,300.0	0.0	4,535.6	33,611.6	**1.050**

GG Engineering	JOB NO. 9 - STORY BUILDINGS	BY SMG	DATE 9/16/04	SHEET NO. ___ OF ___
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SUBJECT

STABILITY COEFFICIENT ALONG COLUMN X-AXIS, θ_x

MOMENT FRAME
MF A2 - F2

- o ***LOAD COMBINATION:*** 1.2D + 0.5L + 1.0E
- o ***DEFLECTION AMPLIFICATION FACTOR:*** C_d = 5.5
- o ***(SEISMIC) IMPORTANCE FACTOR*** I_E = 1.0
- o ***MOMENT FRAME RESISTS WHAT % OF THE TOTAL SEISMIC SHEAR TO THE BUIDLING?*** 25%

		DUE TO FORCES THAT CAUSE BENDING ALONG THE X-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					STABILITY COEFFICIENT PER STORY θ_i
								LOAD FACTOR					
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO ANY HORIZONTAL LOAD ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0.5	*ROOF L.L.* 0	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0.2	TOTAL ΣP_{ui} (kips)	
9	13.0 ft	380 kips	0.3 in	3,020	0	450	3,118	3,624.0	0.0	0.0	623.6	4,247.6	**0.021**
8	13.0 ft	633 kips	0.42 in	5,015	1,575	450	5,563	6,018.0	787.5	0.0	1,112.6	7,918.1	**0.034**
7	13.0 ft	844 kips	0.51 in	7,010	3,150	450	8,008	8,412.0	1,575.0	0.0	1,601.6	11,588.6	**0.045**
6	13.0 ft	1,014 kips	0.56 in	9,005	4,725	450	10,453	10,806.0	2,362.5	0.0	2,090.6	15,259.1	**0.054**
5	13.0 ft	1,146 kips	0.6 in	11,000	6,300	450	12,898	13,200.0	3,150.0	0.0	2,579.6	18,929.6	**0.064**
4	13.0 ft	1,244 kips	0.6 in	12,995	7,875	450	15,342	15,594.0	3,937.5	0.0	3,068.4	22,599.9	**0.070**
3	13.0 ft	1,309 kips	0.57 in	14,990	9,450	450	17,788	17,988.0	4,725.0	0.0	3,557.6	26,270.6	**0.073**
2	13.0 ft	1,346 kips	0.53 in	16,985	11,025	450	20,233	20,382.0	5,512.5	0.0	4,046.6	29,941.1	**0.076**
1	13.0 ft	1,361 kips	0.3 in	18,980	12,600	450	22,678	22,776.0	6,300.0	0.0	4,535.6	33,611.6	**0.047**

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SUBJECT

STABILITY COEFFICIENT ALONG COLUMN X-AXIS, θ_x

MOMENT FRAME
MF A2 - F2

- ***LOAD COMBINATION:*** 1.2D + 0.5L + 1.0E
- ***DEFLECTION AMPLIFICATION FACTOR:*** C_d = 5.5

		DUE TO FORCES THAT CAUSE BENDING ALONG THE X-AXIS OF THE MOMENT FRAME COLUMNS						
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH_i	INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	TOTAL STORY SHEAR CAPACITY (OF ALL OF THE SEISMIC RESISTING MOMENT FRAMES)	RATIO OF SHEAR DEMAND / SHEAR CAPACITY PER STORY β	MAXIMUM ALLOWED STABILITY COEFFICIENT PER STORY θ_{i_max}	STABILITY COEFFICIENT PER STORY θ_i	***COMMENT***
9	13.0 ft	380 kips	0.3 in	11,029 kips	0.0345	0.250	0.021	***OK***
8	13.0 ft	633 kips	0.42 in	11,029 kips	0.0574	0.250	0.034	***OK***
7	13.0 ft	844 kips	0.51 in	13,507 kips	0.0625	0.250	0.045	***OK***
6	13.0 ft	1,014 kips	0.56 in	12,221 kips	0.0830	0.250	0.054	***OK***
5	13.0 ft	1,146 kips	0.6 in	12,221 kips	0.0938	0.250	0.064	***OK***
4	13.0 ft	1,244 kips	0.6 in	14,997 kips	0.0829	0.250	0.070	***OK***
3	13.0 ft	1,309 kips	0.57 in	14,997 kips	0.0873	0.250	0.073	***OK***
2	13.0 ft	1,346 kips	0.53 in	14,997 kips	0.0898	0.250	0.076	***OK***
1	13.0 ft	1,361 kips	0.3 in	14,997 kips	0.0908	0.250	0.047	***OK***

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SUBJECT

STABILITY COEFFICIENT ALONG COLUMN Y-AXIS, θ_y

MOMENT FRAME
MF A2 - F2

- ***LOAD COMBINATION:*** 1.2D + 0.5L + 1.0E
- ***DEFLECTION AMPLIFICATION FACTOR:*** C_d = 5.5
- ***(SEISMIC) IMPORTANCE FACTOR*** I_E = 1.0
- ***MOMENT FRAME RESISTS WHAT % OF THE TOTAL SEISMIC SHEAR TO THE BUIDLING?*** 25%

		DUE TO FORCES THAT CAUSE BENDING ALONG THE Y-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					
								LOAD FACTOR					
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO ANY HORIZONTAL LOAD ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0.5	*ROOF L.L.* 0	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0.2	TOTAL ΣP_{ui} (kips)	STABILITY COEFFICIENT PER STORY θ_i
9	13.0 ft	380 kips	0.3 in	3,020	0	450	3,118	3,624.0	0.0	0.0	623.6	4,247.6	**0.021**
8	13.0 ft	633 kips	0.42 in	5,015	1,575	450	5,563	6,018.0	787.5	0.0	1,112.6	7,918.1	**0.034**
7	13.0 ft	844 kips	0.51 in	7,010	3,150	450	8,008	8,412.0	1,575.0	0.0	1,601.6	11,588.6	**0.045**
6	13.0 ft	1,014 kips	0.56 in	9,005	4,725	450	10,453	10,806.0	2,362.5	0.0	2,090.6	15,259.1	**0.054**
5	13.0 ft	1,146 kips	0.6 in	11,000	6,300	450	12,898	13,200.0	3,150.0	0.0	2,579.6	18,929.6	**0.064**
4	13.0 ft	1,244 kips	0.6 in	12,995	7,875	450	15,342	15,594.0	3,937.5	0.0	3,068.4	22,599.9	**0.070**
3	13.0 ft	1,309 kips	0.57 in	14,990	9,450	450	17,788	17,988.0	4,725.0	0.0	3,557.6	26,270.6	**0.073**
2	13.0 ft	1,346 kips	0.53 in	16,985	11,025	450	20,233	20,382.0	5,512.5	0.0	4,046.6	29,941.1	**0.076**
1	13.0 ft	1,361 kips	0.3 in	18,980	12,600	450	22,678	22,776.0	6,300.0	0.0	4,535.6	33,611.6	**0.047**

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SUBJECT

STABILITY COEFFICIENT ALONG COLUMN Y-AXIS, θ_y

MOMENT FRAME MF A2 - F2

- ***LOAD COMBINATION:*** 1.2D + 0.5L + 1.0E
- ***DEFLECTION AMPLIFICATION FACTOR:*** C_d = 5.5

		DUE TO FORCES THAT CAUSE BENDING ALONG THE Y-AXIS OF THE MOMENT FRAME COLUMNS						
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH_i	INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	TOTAL STORY SHEAR CAPACITY (OF ALL OF THE SEISMIC RESISTING MOMENT FRAMES)	RATIO OF SHEAR DEMAND / SHEAR CAPACITY PER STORY β	MAXIMUM ALLOWED STABILITY COEFFICIENT PER STORY θ_{i_max}	STABILITY COEFFICIENT PER STORY θ_i	***COMMENT***
9	13.0 ft	380 kips	0.3 in	11,029 kips	0.0345	0.250	0.021	***OK***
8	13.0 ft	633 kips	0.42 in	11,029 kips	0.0574	0.250	0.034	***OK***
7	13.0 ft	844 kips	0.51 in	13,507 kips	0.0625	0.250	0.045	***OK***
6	13.0 ft	1,014 kips	0.56 in	12,221 kips	0.0830	0.250	0.054	***OK***
5	13.0 ft	1,146 kips	0.6 in	12,221 kips	0.0938	0.250	0.064	***OK***
4	13.0 ft	1,244 kips	0.6 in	14,997 kips	0.0829	0.250	0.070	***OK***
3	13.0 ft	1,309 kips	0.57 in	14,997 kips	0.0873	0.250	0.073	***OK***
2	13.0 ft	1,346 kips	0.53 in	14,997 kips	0.0898	0.250	0.076	***OK***
1	13.0 ft	1,361 kips	0.3 in	14,997 kips	0.0908	0.250	0.047	***OK***

COLUMN		MAXIMUM INTERACTION VALUE	CONTROLLING LOAD COMBINATION	BUILDING MAX.	COMMENTS FOR: DESIGN 9A	
NAME	MEMBER SIZE			INTERACTION 0.7799	COLUMN STEEL AREA CHECK	COLUMN COMPACTNESS CHECK
A2-1	HSS 22 x 22 x 0.625	0.483700765	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
A2-2	HSS 22 x 22 x 0.625	0.224187253	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
A2-3	HSS 22 x 22 x 0.625	0.171919302	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
A2-4	HSS 22 x 22 x 0.625	0.168485888	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
A2-5	HSS 22 x 22 x 0.5	0.202176874	2		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
A2-6	HSS 22 x 22 x 0.5	0.210225303	2		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
A2-7	HSS 20 x 20 x 0.625	0.209005156	2		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
A2-8	HSS 20 x 20 x 0.5	0.269076149	2		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
A2-9	HSS 20 x 20 x 0.5	0.281557549	3		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
B2-1	HSS 22 x 22 x 0.625	0.778328448	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
B2-2	HSS 22 x 22 x 0.625	0.616403889	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
B2-3	HSS 22 x 22 x 0.625	0.489984076	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
B2-4	HSS 22 x 22 x 0.625	0.438191867	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
B2-5	HSS 22 x 22 x 0.5	0.468062264	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
B2-6	HSS 22 x 22 x 0.5	0.430823138	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
B2-7	HSS 20 x 20 x 0.625	0.349528014	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
B2-8	HSS 20 x 20 x 0.5	0.334463065	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
B2-9	HSS 20 x 20 x 0.5	0.253051252	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
C2-1	HSS 22 x 22 x 0.625	0.662124876	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
C2-2	HSS 22 x 22 x 0.625	0.520351324	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
C2-3	HSS 22 x 22 x 0.625	0.407621386	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
C2-4	HSS 22 x 22 x 0.625	0.332593718	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
C2-5	HSS 22 x 22 x 0.5	0.357601207	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
C2-6	HSS 22 x 22 x 0.5	0.327835698	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
C2-7	HSS 20 x 20 x 0.625	0.267926817	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
C2-8	HSS 20 x 20 x 0.5	0.24715319	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
C2-9	HSS 20 x 20 x 0.5	0.179404562	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
D2-1	HSS 22 x 22 x 0.625	0.665496286	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
D2-2	HSS 22 x 22 x 0.625	0.519557385	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
D2-3	HSS 22 x 22 x 0.625	0.407308327	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
D2-4	HSS 22 x 22 x 0.625	0.332634097	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
D2-5	HSS 22 x 22 x 0.5	0.359089469	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
D2-6	HSS 22 x 22 x 0.5	0.328760813	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
D2-7	HSS 20 x 20 x 0.625	0.266981664	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
D2-8	HSS 20 x 20 x 0.5	0.27054573	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
D2-9	HSS 20 x 20 x 0.5	0.197241414	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
E2-1	HSS 22 x 22 x 0.625	0.7798745	6	***<---CONTROLS!***	OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
E2-2	HSS 22 x 22 x 0.625	0.607680457	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
E2-3	HSS 22 x 22 x 0.625	0.494294262	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
E2-4	HSS 22 x 22 x 0.625	0.449422415	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
E2-5	HSS 22 x 22 x 0.5	0.490109411	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
E2-6	HSS 22 x 22 x 0.5	0.452287319	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
E2-7	HSS 20 x 20 x 0.625	0.37594891	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
E2-8	HSS 20 x 20 x 0.5	0.355309501	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
E2-9	HSS 20 x 20 x 0.5	0.457723031	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
F2-1	HSS 22 x 22 x 0.625	0.598194317	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
F2-2	HSS 22 x 22 x 0.625	0.455036186	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
F2-3	HSS 22 x 22 x 0.625	0.404502813	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
F2-4	HSS 22 x 22 x 0.625	0.405054719	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
F2-5	HSS 22 x 22 x 0.5	0.461949587	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
F2-6	HSS 22 x 22 x 0.5	0.43764115	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
F2-7	HSS 20 x 20 x 0.625	0.389013979	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
F2-8	HSS 20 x 20 x 0.5	0.44290552	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
F2-9	HSS 20 x 20 x 0.5	0.418925396	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT

Appendix C

Building Design 9B Calculations

This appendix consists of the design calculations that were performed for building Design 9B which is the 9-story building that used high strength materials in the columns (F_{yc} = 80 ksi and f'_c = 16 ksi) and a relatively low column d/t ratio. The final RCFT column and wide flange girder sections are presented in Chapter 5. The linear elastic analysis consisted of taking the nominal loads that were generated in Appendix A and factoring them per the applicable LRFD load combination. The calculation for the stability coefficient, θ, and the moment magnification factor, B_2, were performed for each load combination that has lateral loads (wind and seismic load combinations #4, #5, and #6) and are included in this appendix. The maximum interaction value for each column is listed at the end of this appendix along with its respective load combination.

The load combinations that were used in this building design are listed below for reference:

1. $1.4D$
2. $1.2D + 1.6L + 0.5L_R$
3. $1.2D + 1.6L_R + f_1L$
4. $1.2D + 1.6L_R + 0.8W$
5. $1.2D + 1.6W + f_1L + 0.5L_R$
6. $1.2D + 1.0E + f_1L$

Where: $f_1 = 0.5$
$E = \rho Q_E + 0.2S_{DS}D'$
D' = seismic weight

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SUBJECT

DESIGN PARAMETERS SUMMARY

MOMENT FRAME
MF A2 - F2

o DESIGN INPUTS:

o TOAL NUMBER OF COLUMNS BEING ANALYZED 54

o YIELD STRENGTH:
HSS, F_y = 80 ksi
CONCRETE REINFORCEMENT, F_{yr} = 0 ksi

o MODULUS OF ELASTICITY:
HSS, E_s = 29,000 ksi
CONCRETE REINFORCEMENT, E_{cr} = 29,000 ksi

o MINIMUM CONCRETE COMPRESSIVE STRENGTH f'_c = 16.0 ksi

o CONCRETE DENSITY w = 145 lb/ft³

o CONCRETE REINFORCEMENT
AREA, A_{sr} = 0.0 in²
I_{xr} = 0.0 in^4
I_{yr} = 0.0 in^4
Z_{xr} = 0.0 in³
Z_{yr} = 0.0 in³

o RESISTANCE FACTORS
AXIAL COMPRESSION, ϕ_c = 0.75
FLEXURAL BENDING, ϕ_b = 0.90

o SEISMIC PARAMETERS
REDUNDANCY COEFFICIENT, ρ = 1.00
VERTICAL SEISMIC "FACTOR," $0.2S_{DS}$ = 0.20
ORTHOGONAL LOAD FACTOR ALONG Y-AXIS OF SHARED COLUMNS = 0.30
FACTOR TO ACCOUNT FOR *5% ACCIDENTAL TORSION* ("SIMPLIFIED APPROACH"...) = 0.025

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SUBJECT **B2 CALCULATION - *FOR BENDING ALONG THE X-AXIS OF THE COLUMN*** — **MOMENT FRAME *MF A2 - F2***

o ***LOAD COMBINATION =*** 1.2D + 1.6Lr + 0.8W *(L.C. # 4)*

		DUE TO FORCES THAT CAUSE BENDING ALONG THE X-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					**STORY $B_{2i\_X\text{-}AXIS}$**
								LOAD FACTOR					
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0	*ROOF L.L.* 1.6	*SEISMIC VERTICAL* $0.2S_{DS} = 0$	TOTAL ΣP_{ui} (kips)	
9	13.0 ft	380 kips	0.32 in	3,020	0	450	3,118	3,624.0	0.0	720.0	0.0	4,344.0	**1.024**
8	13.0 ft	633 kips	0.5 in	5,015	1,575	450	5,563	6,018.0	0.0	720.0	0.0	6,738.0	**1.035**
7	13.0 ft	844 kips	0.58 in	7,010	3,150	450	8,008	8,412.0	0.0	720.0	0.0	9,132.0	**1.042**
6	13.0 ft	1,014 kips	0.56 in	9,005	4,725	450	10,453	10,806.0	0.0	720.0	0.0	11,526.0	**1.043**
5	13.0 ft	1,146 kips	0.58 in	11,000	6,300	450	12,898	13,200.0	0.0	720.0	0.0	13,920.0	**1.047**
4	13.0 ft	1,244 kips	0.59 in	12,995	7,875	450	15,342	15,594.0	0.0	720.0	0.0	16,314.0	**1.052**
3	13.0 ft	1,309 kips	0.59 in	14,990	9,450	450	17,788	17,988.0	0.0	720.0	0.0	18,708.0	**1.057**
2	13.0 ft	1,346 kips	0.58 in	16,985	11,025	450	20,233	20,382.0	0.0	720.0	0.0	21,102.0	**1.062**
1	13.0 ft	1,361 kips	0.37 in	18,980	12,600	450	22,678	22,776.0	0.0	720.0	0.0	23,496.0	**1.043**

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SUBJECT **B2 CALCULATION - *FOR BENDING ALONG THE Y-AXIS OF THE COLUMN*** — **MOMENT FRAME MF A2 - F2**

o ***LOAD COMBINATION =*** **1.2D + 1.6Lr + 0.8W**

		DUE TO FORCES THAT CAUSE BENDING ALONG THE Y-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					
								LOAD FACTOR					
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0	*ROOF L.L.* 1.6	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0	TOTAL ΣP_{ui} (kips)	**STORY $B_{2i\_Y\text{-}AXIS}$**
9	13.0 ft	380 kips	0.32 in	3,020	0	450	3,118	3,624.0	0.0	720.0	0.0	4,344.0	**1.024**
8	13.0 ft	633 kips	0.5 in	5,015	1,575	450	5,563	6,018.0	0.0	720.0	0.0	6,738.0	**1.035**
7	13.0 ft	844 kips	0.58 in	7,010	3,150	450	8,008	8,412.0	0.0	720.0	0.0	9,132.0	**1.042**
6	13.0 ft	1,014 kips	0.56 in	9,005	4,725	450	10,453	10,806.0	0.0	720.0	0.0	11,526.0	**1.043**
5	13.0 ft	1,146 kips	0.58 in	11,000	6,300	450	12,898	13,200.0	0.0	720.0	0.0	13,920.0	**1.047**
4	13.0 ft	1,244 kips	0.59 in	12,995	7,875	450	15,342	15,594.0	0.0	720.0	0.0	16,314.0	**1.052**
3	13.0 ft	1,309 kips	0.59 in	14,990	9,450	450	17,788	17,988.0	0.0	720.0	0.0	18,708.0	**1.057**
2	13.0 ft	1,346 kips	0.58 in	16,985	11,025	450	20,233	20,382.0	0.0	720.0	0.0	21,102.0	**1.062**
1	13.0 ft	1,361 kips	0.37 in	18,980	12,600	450	22,678	22,776.0	0.0	720.0	0.0	23,496.0	**1.043**

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SUBJECT

STABILITY COEFFICIENT ALONG COLUMN X-AXIS, θ_x

MOMENT FRAME
MF A2 - F2

- ***LOAD COMBINATION:*** 1.2D + 1.6Lr + 0.8W
- ***DEFLECTION AMPLIFICATION FACTOR:*** C_d = 5.5
- ***(SEISMIC) IMPORTANCE FACTOR*** I_E = 1.0
- ***MOMENT FRAME RESISTS WHAT % OF THE TOTAL SEISMIC SHEAR TO THE BUIDLING?*** 25%

		DUE TO FORCES THAT CAUSE BENDING ALONG THE X-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					STABILITY COEFFICIENT PER STORY θ_i
								LOAD FACTOR				TOTAL ΣP_{ui} (kips)	
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO ANY HORIZONTAL LOAD ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0	*ROOF L.L.* 1.6	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0		
9	13.0 ft	380 kips	0.32 in	3,020	0	450	3,118	3,624.0	0.0	720.0	0.0	4,344.0	**0.023**
8	13.0 ft	633 kips	0.5 in	5,015	1,575	450	5,563	6,018.0	0.0	720.0	0.0	6,738.0	**0.034**
7	13.0 ft	844 kips	0.58 in	7,010	3,150	450	8,008	8,412.0	0.0	720.0	0.0	9,132.0	**0.040**
6	13.0 ft	1,014 kips	0.56 in	9,005	4,725	450	10,453	10,806.0	0.0	720.0	0.0	11,526.0	**0.041**
5	13.0 ft	1,146 kips	0.58 in	11,000	6,300	450	12,898	13,200.0	0.0	720.0	0.0	13,920.0	**0.045**
4	13.0 ft	1,244 kips	0.59 in	12,995	7,875	450	15,342	15,594.0	0.0	720.0	0.0	16,314.0	**0.050**
3	13.0 ft	1,309 kips	0.59 in	14,990	9,450	450	17,788	17,988.0	0.0	720.0	0.0	18,708.0	**0.054**
2	13.0 ft	1,346 kips	0.58 in	16,985	11,025	450	20,233	20,382.0	0.0	720.0	0.0	21,102.0	**0.058**
1	13.0 ft	1,361 kips	0.37 in	18,980	12,600	450	22,678	22,776.0	0.0	720.0	0.0	23,496.0	**0.041**

GG Engineering	JOB NO. 9 - STORY BUILDINGS	BY SMG	DATE 9/16/04	SHEET NO. ___ OF ___
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SUBJECT

STABILITY COEFFICIENT ALONG COLUMN Y-AXIS, θ_y

MOMENT FRAME
MF A2 - F2

- ***LOAD COMBINATION:*** 1.2D + 1.6Lr + 0.8W
- ***DEFLECTION AMPLIFICATION FACTOR:*** C_d = 5.5
- ***(SEISMIC) IMPORTANCE FACTOR*** I_E = 1.0
- ***MOMENT FRAME RESISTS WHAT % OF THE TOTAL SEISMIC SHEAR TO THE BUIDLING?*** 25%

		DUE TO FORCES THAT CAUSE BENDING ALONG THE Y-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					STABILITY COEFFICIENT PER STORY θ_i
								LOAD FACTOR					
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO ANY HORIZONTAL LOAD ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0	*ROOF L.L.* 1.6	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0	TOTAL ΣP_{ui} (kips)	
9	13.0 ft	380 kips	0.32 in	3,020	0	450	3,118	3,624.0	0.0	720.0	0.0	4,344.0	**0.023**
8	13.0 ft	633 kips	0.5 in	5,015	1,575	450	5,563	6,018.0	0.0	720.0	0.0	6,738.0	**0.034**
7	13.0 ft	844 kips	0.58 in	7,010	3,150	450	8,008	8,412.0	0.0	720.0	0.0	9,132.0	**0.040**
6	13.0 ft	1,014 kips	0.56 in	9,005	4,725	450	10,453	10,806.0	0.0	720.0	0.0	11,526.0	**0.041**
5	13.0 ft	1,146 kips	0.58 in	11,000	6,300	450	12,898	13,200.0	0.0	720.0	0.0	13,920.0	**0.045**
4	13.0 ft	1,244 kips	0.59 in	12,995	7,875	450	15,342	15,594.0	0.0	720.0	0.0	16,314.0	**0.050**
3	13.0 ft	1,309 kips	0.59 in	14,990	9,450	450	17,788	17,988.0	0.0	720.0	0.0	18,708.0	**0.054**
2	13.0 ft	1,346 kips	0.58 in	16,985	11,025	450	20,233	20,382.0	0.0	720.0	0.0	21,102.0	**0.058**
1	13.0 ft	1,361 kips	0.37 in	18,980	12,600	450	22,678	22,776.0	0.0	720.0	0.0	23,496.0	**0.041**

GG Engineering	JOB NO. 9 - STORY BUILDINGS	BY SMG	DATE 9/16/04	SHEET NO. ___ OF ___
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SUBJECT **B2 CALCULATION - *FOR BENDING ALONG THE X-AXIS OF THE COLUMN*** **MOMENT FRAME** ***MF A2 - F2***

o ***LOAD COMBINATION =*** **1.2D + 0.5L + 0.5Lr + 1.6W** *(L.C. # 5)*

		DUE TO FORCES THAT CAUSE BENDING ALONG THE X-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					**STORY $B_{2i\_X\text{-}AXIS}$**
								LOAD FACTOR					
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0.5	*ROOF L.L.* 0.5	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0	TOTAL ΣP_{ui} (kips)	
9	13.0 ft	380 kips	0.32 in	3,020	0	450	3,118	3,624.0	0.0	225.0	0.0	3,849.0	**1.021**
8	13.0 ft	633 kips	0.5 in	5,015	1,575	450	5,563	6,018.0	787.5	225.0	0.0	7,030.5	**1.037**
7	13.0 ft	844 kips	0.58 in	7,010	3,150	450	8,008	8,412.0	1,575.0	225.0	0.0	10,212.0	**1.047**
6	13.0 ft	1,014 kips	0.56 in	9,005	4,725	450	10,453	10,806.0	2,362.5	225.0	0.0	13,393.5	**1.050**
5	13.0 ft	1,146 kips	0.58 in	11,000	6,300	450	12,898	13,200.0	3,150.0	225.0	0.0	16,575.0	**1.057**
4	13.0 ft	1,244 kips	0.59 in	12,995	7,875	450	15,342	15,594.0	3,937.5	225.0	0.0	19,756.5	**1.064**
3	13.0 ft	1,309 kips	0.59 in	14,990	9,450	450	17,788	17,988.0	4,725.0	225.0	0.0	22,938.0	**1.071**
2	13.0 ft	1,346 kips	0.58 in	16,985	11,025	450	20,233	20,382.0	5,512.5	225.0	0.0	26,119.5	**1.078**
1	13.0 ft	1,361 kips	0.37 in	18,980	12,600	450	22,678	22,776.0	6,300.0	225.0	0.0	29,301.0	**1.054**

GG Engineering	JOB NO. 9 - STORY BUILDINGS	BY SMG	DATE 9/16/04	SHEET NO.
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SUBJECT **B2 CALCULATION - *FOR BENDING ALONG THE Y-AXIS OF THE COLUMN*** **MOMENT FRAME MF A2 - F2**

o ***LOAD COMBINATION =*** **1.2D + 0.5L + 0.5Lr + 1.6W**

STORY NUMBER	LENGTH *(STORY HEIGHT)* L	DUE TO FORCES THAT CAUSE BENDING ALONG THE Y-AXIS OF THE MOMENT FRAME COLUMNS: TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips): DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY — *LOAD FACTOR*: *D.L.* 1.2	*L.L.* 0.5	*ROOF L.L.* 0.5	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0	TOTAL ΣP_{ui} (kips)	**STORY $B_{2i\_Y\text{-}AXIS}$**
9	13.0 ft	380 kips	0.32 in	3,020	0	450	3,118	3,624.0	0.0	225.0	0.0	3,849.0	**1.021**
8	13.0 ft	633 kips	0.5 in	5,015	1,575	450	5,563	6,018.0	787.5	225.0	0.0	7,030.5	**1.037**
7	13.0 ft	844 kips	0.58 in	7,010	3,150	450	8,008	8,412.0	1,575.0	225.0	0.0	10,212.0	**1.047**
6	13.0 ft	1,014 kips	0.56 in	9,005	4,725	450	10,453	10,806.0	2,362.5	225.0	0.0	13,393.5	**1.050**
5	13.0 ft	1,146 kips	0.58 in	11,000	6,300	450	12,898	13,200.0	3,150.0	225.0	0.0	16,575.0	**1.057**
4	13.0 ft	1,244 kips	0.59 in	12,995	7,875	450	15,342	15,594.0	3,937.5	225.0	0.0	19,756.5	**1.064**
3	13.0 ft	1,309 kips	0.59 in	14,990	9,450	450	17,788	17,988.0	4,725.0	225.0	0.0	22,938.0	**1.071**
2	13.0 ft	1,346 kips	0.58 in	16,985	11,025	450	20,233	20,382.0	5,512.5	225.0	0.0	26,119.5	**1.078**
1	13.0 ft	1,361 kips	0.37 in	18,980	12,600	450	22,678	22,776.0	6,300.0	225.0	0.0	29,301.0	**1.054**

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SUBJECT

STABILITY COEFFICIENT ALONG COLUMN X-AXIS, θ_x

MOMENT FRAME
MF A2 - F2

- ***LOAD COMBINATION:*** 1.2D + 0.5L + 0.5Lr + 1.6W
- ***DEFLECTION AMPLIFICATION FACTOR:*** C_d = 5.5
- ***(SEISMIC) IMPORTANCE FACTOR*** I_E = 1.0
- ***MOMENT FRAME RESISTS WHAT % OF THE TOTAL SEISMIC SHEAR TO THE BUIDLING?*** 25%

		DUE TO FORCES THAT CAUSE BENDING ALONG THE X-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					STABILITY COEFFICIENT PER STORY θ_i
								LOAD FACTOR				TOTAL ΣP_{ui} (kips)	
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO ANY HORIZONTAL LOAD ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0.5	*ROOF L.L.* 0.5	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0		
9	13.0 ft	380 kips	0.32 in	3,020	0	450	3,118	3,624.0	0.0	225.0	0.0	3,849.0	**0.021**
8	13.0 ft	633 kips	0.5 in	5,015	1,575	450	5,563	6,018.0	787.5	225.0	0.0	7,030.5	**0.036**
7	13.0 ft	844 kips	0.58 in	7,010	3,150	450	8,008	8,412.0	1,575.0	225.0	0.0	10,212.0	**0.045**
6	13.0 ft	1,014 kips	0.56 in	9,005	4,725	450	10,453	10,806.0	2,362.5	225.0	0.0	13,393.5	**0.047**
5	13.0 ft	1,146 kips	0.58 in	11,000	6,300	450	12,898	13,200.0	3,150.0	225.0	0.0	16,575.0	**0.054**
4	13.0 ft	1,244 kips	0.59 in	12,995	7,875	450	15,342	15,594.0	3,937.5	225.0	0.0	19,756.5	**0.060**
3	13.0 ft	1,309 kips	0.59 in	14,990	9,450	450	17,788	17,988.0	4,725.0	225.0	0.0	22,938.0	**0.066**
2	13.0 ft	1,346 kips	0.58 in	16,985	11,025	450	20,233	20,382.0	5,512.5	225.0	0.0	26,119.5	**0.072**
1	13.0 ft	1,361 kips	0.37 in	18,980	12,600	450	22,678	22,776.0	6,300.0	225.0	0.0	29,301.0	**0.051**

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SUBJECT

STABILITY COEFFICIENT ALONG COLUMN Y-AXIS, θ_y

MOMENT FRAME
MF A2 - F2

- o ***LOAD COMBINATION:*** 1.2D + 0.5L + 0.5Lr + 1.6W
- o ***DEFLECTION AMPLIFICATION FACTOR:*** C_d = 5.5
- o ***(SEISMIC) IMPORTANCE FACTOR*** I_E = 1.0
- o ***MOMENT FRAME RESISTS WHAT % OF THE TOTAL SEISMIC SHEAR TO THE BUIDLING?*** 25%

		DUE TO FORCES THAT CAUSE BENDING ALONG THE Y-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					STABILITY COEFFICIENT PER STORY θ_i
								LOAD FACTOR				TOTAL ΣP_{ui} (kips)	
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO ANY HORIZONTAL LOAD ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0.5	*ROOF L.L.* 0.5	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0		
9	13.0 ft	380 kips	0.32 in	3,020	0	450	3,118	3,624.0	0.0	225.0	0.0	3,849.0	**0.021**
8	13.0 ft	633 kips	0.5 in	5,015	1,575	450	5,563	6,018.0	787.5	225.0	0.0	7,030.5	**0.036**
7	13.0 ft	844 kips	0.58 in	7,010	3,150	450	8,008	8,412.0	1,575.0	225.0	0.0	10,212.0	**0.045**
6	13.0 ft	1,014 kips	0.56 in	9,005	4,725	450	10,453	10,806.0	2,362.5	225.0	0.0	13,393.5	**0.047**
5	13.0 ft	1,146 kips	0.58 in	11,000	6,300	450	12,898	13,200.0	3,150.0	225.0	0.0	16,575.0	**0.054**
4	13.0 ft	1,244 kips	0.59 in	12,995	7,875	450	15,342	15,594.0	3,937.5	225.0	0.0	19,756.5	**0.060**
3	13.0 ft	1,309 kips	0.59 in	14,990	9,450	450	17,788	17,988.0	4,725.0	225.0	0.0	22,938.0	**0.066**
2	13.0 ft	1,346 kips	0.58 in	16,985	11,025	450	20,233	20,382.0	5,512.5	225.0	0.0	26,119.5	**0.072**
1	13.0 ft	1,361 kips	0.37 in	18,980	12,600	450	22,678	22,776.0	6,300.0	225.0	0.0	29,301.0	**0.051**

GG Engineering	JOB NO. 9 - STORY BUILDINGS	BY SMG	DATE 9/16/04	SHEET NO. ___ OF ___
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SUBJECT **B2 CALCULATION - *FOR BENDING ALONG THE X-AXIS OF THE COLUMN*** — **MOMENT FRAME *MF A2 - F2***

o ***LOAD COMBINATION =*** **1.2D + 0.5L + 1.0E** *(L.C. # 6)*

STORY NUMBER	LENGTH *(STORY HEIGHT)* L	DUE TO FORCES THAT CAUSE BENDING ALONG THE X-AXIS OF THE MOMENT FRAME COLUMNS: TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips): DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY — *LOAD FACTOR*: *D.L.* 1.2	*L.L.* 0.5	*ROOF L.L.* 0	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0.2	TOTAL ΣP_{ui} (kips)	**STORY $B_{2i\_X\text{-}AXIS}$**
9	13.0 ft	380 kips	0.32 in	3,020	0	450	3,118	3,624.0	0.0	0.0	623.6	4,247.6	**1.023**
8	13.0 ft	633 kips	0.5 in	5,015	1,575	450	5,563	6,018.0	787.5	0.0	1,112.6	7,918.1	**1.042**
7	13.0 ft	844 kips	0.58 in	7,010	3,150	450	8,008	8,412.0	1,575.0	0.0	1,601.6	11,588.6	**1.054**
6	13.0 ft	1,014 kips	0.56 in	9,005	4,725	450	10,453	10,806.0	2,362.5	0.0	2,090.6	15,259.1	**1.057**
5	13.0 ft	1,146 kips	0.58 in	11,000	6,300	450	12,898	13,200.0	3,150.0	0.0	2,579.6	18,929.6	**1.065**
4	13.0 ft	1,244 kips	0.59 in	12,995	7,875	450	15,342	15,594.0	3,937.5	0.0	3,068.4	22,599.9	**1.074**
3	13.0 ft	1,309 kips	0.59 in	14,990	9,450	450	17,788	17,988.0	4,725.0	0.0	3,557.6	26,270.6	**1.082**
2	13.0 ft	1,346 kips	0.58 in	16,985	11,025	450	20,233	20,382.0	5,512.5	0.0	4,046.6	29,941.1	**1.090**
1	13.0 ft	1,361 kips	0.37 in	18,980	12,600	450	22,678	22,776.0	6,300.0	0.0	4,535.6	33,611.6	**1.062**

GG Engineering	JOB NO. 9 - STORY BUILDINGS	BY SMG	DATE 9/16/04	SHEET NO. ___ OF ___
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SUBJECT **B2 CALCULATION - *FOR BENDING ALONG THE Y-AXIS OF THE COLUMN*** — **MOMENT FRAME MF A2 - F2**

o ***LOAD COMBINATION =*** **1.2D + 0.5L + 1.0E**

		DUE TO FORCES THAT CAUSE BENDING ALONG THE Y-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					
								LOAD FACTOR					
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0.5	*ROOF L.L.* 0	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0.2	TOTAL ΣP_{ui} (kips)	**STORY $B_{2i\_Y\text{-}AXIS}$**
9	13.0 ft	380 kips	0.32 in	3,020	0	450	3,118	3,624.0	0.0	0.0	623.6	4,247.6	**1.023**
8	13.0 ft	633 kips	0.5 in	5,015	1,575	450	5,563	6,018.0	787.5	0.0	1,112.6	7,918.1	**1.042**
7	13.0 ft	844 kips	0.58 in	7,010	3,150	450	8,008	8,412.0	1,575.0	0.0	1,601.6	11,588.6	**1.054**
6	13.0 ft	1,014 kips	0.56 in	9,005	4,725	450	10,453	10,806.0	2,362.5	0.0	2,090.6	15,259.1	**1.057**
5	13.0 ft	1,146 kips	0.58 in	11,000	6,300	450	12,898	13,200.0	3,150.0	0.0	2,579.6	18,929.6	**1.065**
4	13.0 ft	1,244 kips	0.59 in	12,995	7,875	450	15,342	15,594.0	3,937.5	0.0	3,068.4	22,599.9	**1.074**
3	13.0 ft	1,309 kips	0.59 in	14,990	9,450	450	17,788	17,988.0	4,725.0	0.0	3,557.6	26,270.6	**1.082**
2	13.0 ft	1,346 kips	0.58 in	16,985	11,025	450	20,233	20,382.0	5,512.5	0.0	4,046.6	29,941.1	**1.090**
1	13.0 ft	1,361 kips	0.37 in	18,980	12,600	450	22,678	22,776.0	6,300.0	0.0	4,535.6	33,611.6	**1.062**

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SUBJECT

STABILITY COEFFICIENT ALONG COLUMN X-AXIS, θ_x

MOMENT FRAME
MF A2 - F2

- o ***LOAD COMBINATION:*** 1.2D + 0.5L + 1.0E
- o ***DEFLECTION AMPLIFICATION FACTOR:*** C_d = 5.5
- o ***(SEISMIC) IMPORTANCE FACTOR*** I_E = 1.0
- o ***MOMENT FRAME RESISTS WHAT % OF THE TOTAL SEISMIC SHEAR TO THE BUIDLING?*** 25%

		DUE TO FORCES THAT CAUSE BENDING ALONG THE X-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					STABILITY COEFFICIENT PER STORY θ_i
								LOAD FACTOR					
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO ANY HORIZONTAL LOAD ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0.5	*ROOF L.L.* 0	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0.2	TOTAL ΣP_{ui} (kips)	
9	13.0 ft	380 kips	0.32 in	3,020	0	450	3,118	3,624.0	0.0	0.0	623.6	4,247.6	**0.023**
8	13.0 ft	633 kips	0.5 in	5,015	1,575	450	5,563	6,018.0	787.5	0.0	1,112.6	7,918.1	**0.040**
7	13.0 ft	844 kips	0.58 in	7,010	3,150	450	8,008	8,412.0	1,575.0	0.0	1,601.6	11,588.6	**0.051**
6	13.0 ft	1,014 kips	0.56 in	9,005	4,725	450	10,453	10,806.0	2,362.5	0.0	2,090.6	15,259.1	**0.054**
5	13.0 ft	1,146 kips	0.58 in	11,000	6,300	450	12,898	13,200.0	3,150.0	0.0	2,579.6	18,929.6	**0.061**
4	13.0 ft	1,244 kips	0.59 in	12,995	7,875	450	15,342	15,594.0	3,937.5	0.0	3,068.4	22,599.9	**0.069**
3	13.0 ft	1,309 kips	0.59 in	14,990	9,450	450	17,788	17,988.0	4,725.0	0.0	3,557.6	26,270.6	**0.076**
2	13.0 ft	1,346 kips	0.58 in	16,985	11,025	450	20,233	20,382.0	5,512.5	0.0	4,046.6	29,941.1	**0.083**
1	13.0 ft	1,361 kips	0.37 in	18,980	12,600	450	22,678	22,776.0	6,300.0	0.0	4,535.6	33,611.6	**0.059**

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SUBJECT

STABILITY COEFFICIENT ALONG COLUMN X-AXIS, θ_x

MOMENT FRAME MF A2 - F2

- o ***LOAD COMBINATION:*** 1.2D + 0.5L + 1.0E
- o ***DEFLECTION AMPLIFICATION FACTOR:*** C_d = 5.5

		DUE TO FORCES THAT CAUSE BENDING ALONG THE X-AXIS OF THE MOMENT FRAME COLUMNS						
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH_i	INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	TOTAL STORY SHEAR CAPACITY (OF ALL OF THE SEISMIC RESISTING MOMENT FRAMES)	RATIO OF SHEAR DEMAND / SHEAR CAPACITY PER STORY β	MAXIMUM ALLOWED STABILITY COEFFICIENT PER STORY θ_{i_max}	STABILITY COEFFICIENT PER STORY θ_i	***COMMENT***
9	13.0 ft	380 kips	0.32 in	14,082 kips	0.0270	0.250	0.023	***OK***
8	13.0 ft	633 kips	0.5 in	14,082 kips	0.0449	0.250	0.040	***OK***
7	13.0 ft	844 kips	0.58 in	17,183 kips	0.0491	0.250	0.051	***OK***
6	13.0 ft	1,014 kips	0.56 in	17,107 kips	0.0593	0.250	0.054	***OK***
5	13.0 ft	1,146 kips	0.58 in	17,107 kips	0.0670	0.250	0.061	***OK***
4	13.0 ft	1,244 kips	0.59 in	20,898 kips	0.0595	0.250	0.069	***OK***
3	13.0 ft	1,309 kips	0.59 in	20,898 kips	0.0626	0.250	0.076	***OK***
2	13.0 ft	1,346 kips	0.58 in	20,898 kips	0.0644	0.250	0.083	***OK***
1	13.0 ft	1,361 kips	0.37 in	20,898 kips	0.0651	0.250	0.059	***OK***

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SUBJECT

STABILITY COEFFICIENT ALONG COLUMN Y-AXIS, θ_y

MOMENT FRAME
MF A2 - F2

- ***LOAD COMBINATION:*** 1.2D + 0.5L + 1.0E
- ***DEFLECTION AMPLIFICATION FACTOR:*** $C_d = 5.5$
- ***(SEISMIC) IMPORTANCE FACTOR*** $I_E = 1.0$
- ***MOMENT FRAME RESISTS WHAT % OF THE TOTAL SEISMIC SHEAR TO THE BUIDLING?*** 25%

		DUE TO FORCES THAT CAUSE BENDING ALONG THE Y-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					STABILITY COEFFICIENT PER STORY θ_i
								LOAD FACTOR				TOTAL ΣP_{ui} (kips)	
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO ANY HORIZONTAL LOAD ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0.5	*ROOF L.L.* 0	*SEISMIC VERTICAL* $0.2S_{DS} = 0.2$		
9	13.0 ft	380 kips	0.32 in	3,020	0	450	3,118	3,624.0	0.0	0.0	623.6	4,247.6	**0.023**
8	13.0 ft	633 kips	0.5 in	5,015	1,575	450	5,563	6,018.0	787.5	0.0	1,112.6	7,918.1	**0.040**
7	13.0 ft	844 kips	0.58 in	7,010	3,150	450	8,008	8,412.0	1,575.0	0.0	1,601.6	11,588.6	**0.051**
6	13.0 ft	1,014 kips	0.56 in	9,005	4,725	450	10,453	10,806.0	2,362.5	0.0	2,090.6	15,259.1	**0.054**
5	13.0 ft	1,146 kips	0.58 in	11,000	6,300	450	12,898	13,200.0	3,150.0	0.0	2,579.6	18,929.6	**0.061**
4	13.0 ft	1,244 kips	0.59 in	12,995	7,875	450	15,342	15,594.0	3,937.5	0.0	3,068.4	22,599.9	**0.069**
3	13.0 ft	1,309 kips	0.59 in	14,990	9,450	450	17,788	17,988.0	4,725.0	0.0	3,557.6	26,270.6	**0.076**
2	13.0 ft	1,346 kips	0.58 in	16,985	11,025	450	20,233	20,382.0	5,512.5	0.0	4,046.6	29,941.1	**0.083**
1	13.0 ft	1,361 kips	0.37 in	18,980	12,600	450	22,678	22,776.0	6,300.0	0.0	4,535.6	33,611.6	**0.059**

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SUBJECT

STABILITY COEFFICIENT ALONG COLUMN Y-AXIS, θ_y

MOMENT FRAME
MF A2 - F2

- ***LOAD COMBINATION:*** 1.2D + 0.5L + 1.0E
- ***DEFLECTION AMPLIFICATION FACTOR:*** C_d = 5.5

		DUE TO FORCES THAT CAUSE BENDING ALONG THE Y-AXIS OF THE MOMENT FRAME COLUMNS						
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH_i	INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	TOTAL STORY SHEAR CAPACITY (OF ALL OF THE SEISMIC RESISTING MOMENT FRAMES)	RATIO OF SHEAR DEMAND / SHEAR CAPACITY PER STORY β	MAXIMUM ALLOWED STABILITY COEFFICIENT PER STORY θ_{i_max}	STABILITY COEFFICIENT PER STORY θ_i	***COMMENT***
9	13.0 ft	380 kips	0.32 in	14,082 kips	0.0270	0.250	0.023	***OK***
8	13.0 ft	633 kips	0.5 in	14,082 kips	0.0449	0.250	0.040	***OK***
7	13.0 ft	844 kips	0.58 in	17,183 kips	0.0491	0.250	0.051	***OK***
6	13.0 ft	1,014 kips	0.56 in	17,107 kips	0.0593	0.250	0.054	***OK***
5	13.0 ft	1,146 kips	0.58 in	17,107 kips	0.0670	0.250	0.061	***OK***
4	13.0 ft	1,244 kips	0.59 in	20,898 kips	0.0595	0.250	0.069	***OK***
3	13.0 ft	1,309 kips	0.59 in	20,898 kips	0.0626	0.250	0.076	***OK***
2	13.0 ft	1,346 kips	0.58 in	20,898 kips	0.0644	0.250	0.083	***OK***
1	13.0 ft	1,361 kips	0.37 in	20,898 kips	0.0651	0.250	0.059	***OK***

COLUMN		MAXIMUM INTERACTION VALUE	CONTROLLING LOAD COMBINATION	BUILDING MAX. INTERACTION 0.4703	COMMENTS FOR: DESIGN 9B	
NAME	MEMBER SIZE				COLUMN STEEL AREA CHECK	COLUMN COMPACTNESS CHECK
A2-1	HSS 18 x 18 x 0.625	0.373119985	6		*OK - STEEL AREA IS > 1% OF TOTAL AREA*	*OK - STEEL HSS IS COMPACT*
A2-2	HSS 18 x 18 x 0.625	0.182245565	6		*OK - STEEL AREA IS > 1% OF TOTAL AREA*	*OK - STEEL HSS IS COMPACT*
A2-3	HSS 18 x 18 x 0.625	0.155019405	6		*OK - STEEL AREA IS > 1% OF TOTAL AREA*	*OK - STEEL HSS IS COMPACT*
A2-4	HSS 18 x 18 x 0.625	0.139151161	2		*OK - STEEL AREA IS > 1% OF TOTAL AREA*	*OK - STEEL HSS IS COMPACT*
A2-5	HSS 18 x 18 x 0.5	0.166926121	2		*OK - STEEL AREA IS > 1% OF TOTAL AREA*	*OK - STEEL HSS IS COMPACT*
A2-6	HSS 18 x 18 x 0.5	0.17765688	2		*OK - STEEL AREA IS > 1% OF TOTAL AREA*	*OK - STEEL HSS IS COMPACT*
A2-7	HSS 16 x 16 x 0.625	0.182294852	2		*OK - STEEL AREA IS > 1% OF TOTAL AREA*	*OK - STEEL HSS IS COMPACT*
A2-8	HSS 16 x 16 x 0.5	0.236061185	2		*OK - STEEL AREA IS > 1% OF TOTAL AREA*	*OK - STEEL HSS IS COMPACT*
A2-9	HSS 16 x 16 x 0.5	0.232651109	3		*OK - STEEL AREA IS > 1% OF TOTAL AREA*	*OK - STEEL HSS IS COMPACT*
B2-1	HSS 18 x 18 x 0.625	0.465950292	6		*OK - STEEL AREA IS > 1% OF TOTAL AREA*	*OK - STEEL HSS IS COMPACT*
B2-2	HSS 18 x 18 x 0.625	0.410778673	6		*OK - STEEL AREA IS > 1% OF TOTAL AREA*	*OK - STEEL HSS IS COMPACT*
B2-3	HSS 18 x 18 x 0.625	0.383318874	6		*OK - STEEL AREA IS > 1% OF TOTAL AREA*	*OK - STEEL HSS IS COMPACT*
B2-4	HSS 18 x 18 x 0.625	0.36405753	6		*OK - STEEL AREA IS > 1% OF TOTAL AREA*	*OK - STEEL HSS IS COMPACT*
B2-5	HSS 18 x 18 x 0.5	0.403383965	6		*OK - STEEL AREA IS > 1% OF TOTAL AREA*	*OK - STEEL HSS IS COMPACT*
B2-6	HSS 18 x 18 x 0.5	0.358018225	6		*OK - STEEL AREA IS > 1% OF TOTAL AREA*	*OK - STEEL HSS IS COMPACT*
B2-7	HSS 16 x 16 x 0.625	0.319347497	6		*OK - STEEL AREA IS > 1% OF TOTAL AREA*	*OK - STEEL HSS IS COMPACT*
B2-8	HSS 16 x 16 x 0.5	0.30700674	6		*OK - STEEL AREA IS > 1% OF TOTAL AREA*	*OK - STEEL HSS IS COMPACT*
B2-9	HSS 16 x 16 x 0.5	0.218621347	6		*OK - STEEL AREA IS > 1% OF TOTAL AREA*	*OK - STEEL HSS IS COMPACT*
C2-1	HSS 18 x 18 x 0.625	0.355199293	6		*OK - STEEL AREA IS > 1% OF TOTAL AREA*	*OK - STEEL HSS IS COMPACT*
C2-2	HSS 18 x 18 x 0.625	0.305806159	6		*OK - STEEL AREA IS > 1% OF TOTAL AREA*	*OK - STEEL HSS IS COMPACT*
C2-3	HSS 18 x 18 x 0.625	0.288619505	6		*OK - STEEL AREA IS > 1% OF TOTAL AREA*	*OK - STEEL HSS IS COMPACT*
C2-4	HSS 18 x 18 x 0.625	0.273824655	6		*OK - STEEL AREA IS > 1% OF TOTAL AREA*	*OK - STEEL HSS IS COMPACT*
C2-5	HSS 18 x 18 x 0.5	0.304948557	6		*OK - STEEL AREA IS > 1% OF TOTAL AREA*	*OK - STEEL HSS IS COMPACT*
C2-6	HSS 18 x 18 x 0.5	0.269548847	6		*OK - STEEL AREA IS > 1% OF TOTAL AREA*	*OK - STEEL HSS IS COMPACT*
C2-7	HSS 16 x 16 x 0.625	0.24248198	6		*OK - STEEL AREA IS > 1% OF TOTAL AREA*	*OK - STEEL HSS IS COMPACT*
C2-8	HSS 16 x 16 x 0.5	0.226296761	6		*OK - STEEL AREA IS > 1% OF TOTAL AREA*	*OK - STEEL HSS IS COMPACT*
C2-9	HSS 16 x 16 x 0.5	0.155238516	6		*OK - STEEL AREA IS > 1% OF TOTAL AREA*	*OK - STEEL HSS IS COMPACT*
D2-1	HSS 18 x 18 x 0.625	0.357954132	6		*OK - STEEL AREA IS > 1% OF TOTAL AREA*	*OK - STEEL HSS IS COMPACT*
D2-2	HSS 18 x 18 x 0.625	0.305743453	6		*OK - STEEL AREA IS > 1% OF TOTAL AREA*	*OK - STEEL HSS IS COMPACT*
D2-3	HSS 18 x 18 x 0.625	0.288674669	6		*OK - STEEL AREA IS > 1% OF TOTAL AREA*	*OK - STEEL HSS IS COMPACT*
D2-4	HSS 18 x 18 x 0.625	0.273692435	6		*OK - STEEL AREA IS > 1% OF TOTAL AREA*	*OK - STEEL HSS IS COMPACT*
D2-5	HSS 18 x 18 x 0.5	0.305688344	6		*OK - STEEL AREA IS > 1% OF TOTAL AREA*	*OK - STEEL HSS IS COMPACT*
D2-6	HSS 18 x 18 x 0.5	0.269066443	6		*OK - STEEL AREA IS > 1% OF TOTAL AREA*	*OK - STEEL HSS IS COMPACT*
D2-7	HSS 16 x 16 x 0.625	0.241870121	6		*OK - STEEL AREA IS > 1% OF TOTAL AREA*	*OK - STEEL HSS IS COMPACT*
D2-8	HSS 16 x 16 x 0.5	0.242194353	6		*OK - STEEL AREA IS > 1% OF TOTAL AREA*	*OK - STEEL HSS IS COMPACT*
D2-9	HSS 16 x 16 x 0.5	0.167897718	6		*OK - STEEL AREA IS > 1% OF TOTAL AREA*	*OK - STEEL HSS IS COMPACT*
E2-1	HSS 18 x 18 x 0.625	0.470271887	6	***<---CONTROLS!***	*OK - STEEL AREA IS > 1% OF TOTAL AREA*	*OK - STEEL HSS IS COMPACT*
E2-2	HSS 18 x 18 x 0.625	0.414101564	6		*OK - STEEL AREA IS > 1% OF TOTAL AREA*	*OK - STEEL HSS IS COMPACT*
E2-3	HSS 18 x 18 x 0.625	0.388237117	6		*OK - STEEL AREA IS > 1% OF TOTAL AREA*	*OK - STEEL HSS IS COMPACT*
E2-4	HSS 18 x 18 x 0.625	0.369515169	6		*OK - STEEL AREA IS > 1% OF TOTAL AREA*	*OK - STEEL HSS IS COMPACT*
E2-5	HSS 18 x 18 x 0.5	0.419431515	6		*OK - STEEL AREA IS > 1% OF TOTAL AREA*	*OK - STEEL HSS IS COMPACT*
E2-6	HSS 18 x 18 x 0.5	0.368192813	6		*OK - STEEL AREA IS > 1% OF TOTAL AREA*	*OK - STEEL HSS IS COMPACT*
E2-7	HSS 16 x 16 x 0.625	0.334915762	6		*OK - STEEL AREA IS > 1% OF TOTAL AREA*	*OK - STEEL HSS IS COMPACT*
E2-8	HSS 16 x 16 x 0.5	0.313884307	6		*OK - STEEL AREA IS > 1% OF TOTAL AREA*	*OK - STEEL HSS IS COMPACT*
E2-9	HSS 16 x 16 x 0.5	0.367807241	6		*OK - STEEL AREA IS > 1% OF TOTAL AREA*	*OK - STEEL HSS IS COMPACT*
F2-1	HSS 18 x 18 x 0.625	0.462781031	6		*OK - STEEL AREA IS > 1% OF TOTAL AREA*	*OK - STEEL HSS IS COMPACT*
F2-2	HSS 18 x 18 x 0.625	0.378689519	6		*OK - STEEL AREA IS > 1% OF TOTAL AREA*	*OK - STEEL HSS IS COMPACT*
F2-3	HSS 18 x 18 x 0.625	0.351458958	6		*OK - STEEL AREA IS > 1% OF TOTAL AREA*	*OK - STEEL HSS IS COMPACT*
F2-4	HSS 18 x 18 x 0.625	0.338454655	6		*OK - STEEL AREA IS > 1% OF TOTAL AREA*	*OK - STEEL HSS IS COMPACT*
F2-5	HSS 18 x 18 x 0.5	0.393112417	6		*OK - STEEL AREA IS > 1% OF TOTAL AREA*	*OK - STEEL HSS IS COMPACT*
F2-6	HSS 18 x 18 x 0.5	0.35386424	6		*OK - STEEL AREA IS > 1% OF TOTAL AREA*	*OK - STEEL HSS IS COMPACT*
F2-7	HSS 16 x 16 x 0.625	0.351983989	6		*OK - STEEL AREA IS > 1% OF TOTAL AREA*	*OK - STEEL HSS IS COMPACT*
F2-8	HSS 16 x 16 x 0.5	0.3912759	6		*OK - STEEL AREA IS > 1% OF TOTAL AREA*	*OK - STEEL HSS IS COMPACT*
F2-9	HSS 16 x 16 x 0.5	0.357484129	6		*OK - STEEL AREA IS > 1% OF TOTAL AREA*	*OK - STEEL HSS IS COMPACT*

Appendix D

Building Design 9C Calculations

This appendix consists of the design calculations that were performed for building Design 9C which is the 9-story building that used low strength steel and high strength concrete in the columns (F_{yc} = 50 ksi and f'_c = 16 ksi) and a high column d/t ratio. The final RCFT column and wide flange girder sections are presented in Chapter 5. The linear elastic analysis consisted of taking the nominal loads that were generated in Appendix A and factoring them per the applicable LRFD load combination. The calculation for the stability coefficient, θ, and the moment magnification factor, B_2, were performed for each load combination that has lateral loads (wind and seismic load combinations #4, #5, and #6) and are included in this appendix. The maximum interaction value for each column is listed at the end of this appendix along with its respective load combination.

The load combinations that were used in this building design are listed below for reference:

1. $1.4D$
2. $1.2D + 1.6L + 0.5L_R$
3. $1.2D + 1.6L_R + f_1L$
4. $1.2D + 1.6L_R + 0.8W$
5. $1.2D + 1.6W + f_1L + 0.5L_R$
6. $1.2D + 1.0E + f_1L$

Where: $f_1 = 0.5$
$E = \rho Q_E + 0.2S_{DS}D'$
D' = seismic weight

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SUBJECT

DESIGN PARAMETERS SUMMARY

MOMENT FRAME
MF A2 - F2

o DESIGN INPUTS:

o TOAL NUMBER OF COLUMNS BEING ANALYZED 54

o YIELD STRENGTH:
HSS, F_y = 50 ksi
CONCRETE REINFORCEMENT, F_{yr} = 0 ksi

o MODULUS OF ELASTICITY:
HSS, E_s = 29,000 ksi
CONCRETE REINFORCEMENT, E_{cr} = 29,000 ksi

o MINIMUM CONCRETE COMPRESSIVE STRENGTH f'_c = 16.0 ksi

o CONCRETE DENSITY w = 145 lb/ft^3

o CONCRETE REINFORCEMENT
AREA, A_{sr} = 0.0 in^2
I_{xxr} = 0.0 in^4
I_{yyr} = 0.0 in^4
Z_{xxr} = 0.0 in^3
Z_{yyr} = 0.0 in^3

o RESISTANCE FACTORS
AXIAL COMPRESSION, ϕ_c = 0.75
FLEXURAL BENDING, ϕ_b = 0.90

o SEISMIC PARAMETERS
REDUNDANCY COEFFICIENT, ρ = 1.00
VERTICAL SEISMIC "FACTOR," $0.2S_{DS}$ = 0.20
ORTHOGONAL LOAD FACTOR ALONG Y-AXIS OF SHARED COLUMNS = 0.30
FACTOR TO ACCOUNT FOR *5% ACCIDENTAL TORSION* ("SIMPLIFIED APPROACH"...) = 0.025

GG Engineering	JOB NO. 9 - STORY BUILDINGS	BY SMG	DATE 9/16/04	SHEET NO. ___ OF ___
	CUSTOMER DESIGN 9C	CKD	DATE	

SUBJECT **B2 CALCULATION - *FOR BENDING ALONG THE X-AXIS OF THE COLUMN*** — **MOMENT FRAME** ***MF A2 - F2***

o ***LOAD COMBINATION =*** **1.2D + 1.6Lr + 0.8W** *(L.C. # 4)*

STORY NUMBER	LENGTH *(STORY HEIGHT)* L	DUE TO FORCES THAT CAUSE BENDING ALONG THE X-AXIS OF THE MOMENT FRAME COLUMNS: TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips): DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY — *LOAD FACTOR*: *D.L.* 1.2	*L.L.* 0	*ROOF L.L.* 1.6	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0	TOTAL ΣP_{ui} (kips)	**STORY $B_{2i\_X\text{-}AXIS}$**
9	13.0 ft	380 kips	0.31 in	3,020	0	450	3,118	3,624.0	0.0	720.0	0.0	4,344.0	**1.023**
8	13.0 ft	633 kips	0.47 in	5,015	1,575	450	5,563	6,018.0	0.0	720.0	0.0	6,738.0	**1.033**
7	13.0 ft	844 kips	0.58 in	7,010	3,150	450	8,008	8,412.0	0.0	720.0	0.0	9,132.0	**1.042**
6	13.0 ft	1,014 kips	0.6 in	9,005	4,725	450	10,453	10,806.0	0.0	720.0	0.0	11,526.0	**1.046**
5	13.0 ft	1,146 kips	0.61 in	11,000	6,300	450	12,898	13,200.0	0.0	720.0	0.0	13,920.0	**1.050**
4	13.0 ft	1,244 kips	0.6 in	12,995	7,875	450	15,342	15,594.0	0.0	720.0	0.0	16,314.0	**1.053**
3	13.0 ft	1,309 kips	0.58 in	14,990	9,450	450	17,788	17,988.0	0.0	720.0	0.0	18,708.0	**1.056**
2	13.0 ft	1,346 kips	0.5 in	16,985	11,025	450	20,233	20,382.0	0.0	720.0	0.0	21,102.0	**1.053**
1	13.0 ft	1,361 kips	0.25 in	18,980	12,600	450	22,678	22,776.0	0.0	720.0	0.0	23,496.0	**1.028**

GG Engineering	JOB NO. 9 - STORY BUILDINGS	BY SMG	DATE 9/16/04	SHEET NO. ___ OF ___
	CUSTOMER DESIGN 9C	CKD	DATE	
SUBJECT	**B2 CALCULATION - *FOR BENDING ALONG THE Y-AXIS OF THE COLUMN***			**MOMENT FRAME MF A2 - F2**

o ***LOAD COMBINATION =*** **1.2D + 1.6Lr + 0.8W**

STORY NUMBER	LENGTH *(STORY HEIGHT)* L	DUE TO FORCES THAT CAUSE BENDING ALONG THE Y-AXIS OF THE MOMENT FRAME COLUMNS: TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH_i	DUE TO FORCES THAT CAUSE BENDING ALONG THE Y-AXIS OF THE MOMENT FRAME COLUMNS: ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips): DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY — *LOAD FACTOR*: *D.L.* 1.2	*L.L.* 0	*ROOF L.L.* 1.6	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0	TOTAL ΣP_{ui} (kips)	**STORY $B_{2i\_Y\text{-}AXIS}$**
9	13.0 ft	380 kips	0.31 in	3,020	0	450	3,118	3,624.0	0.0	720.0	0.0	4,344.0	**1.023**
8	13.0 ft	633 kips	0.47 in	5,015	1,575	450	5,563	6,018.0	0.0	720.0	0.0	6,738.0	**1.033**
7	13.0 ft	844 kips	0.58 in	7,010	3,150	450	8,008	8,412.0	0.0	720.0	0.0	9,132.0	**1.042**
6	13.0 ft	1,014 kips	0.6 in	9,005	4,725	450	10,453	10,806.0	0.0	720.0	0.0	11,526.0	**1.046**
5	13.0 ft	1,146 kips	0.61 in	11,000	6,300	450	12,898	13,200.0	0.0	720.0	0.0	13,920.0	**1.050**
4	13.0 ft	1,244 kips	0.6 in	12,995	7,875	450	15,342	15,594.0	0.0	720.0	0.0	16,314.0	**1.053**
3	13.0 ft	1,309 kips	0.58 in	14,990	9,450	450	17,788	17,988.0	0.0	720.0	0.0	18,708.0	**1.056**
2	13.0 ft	1,346 kips	0.5 in	16,985	11,025	450	20,233	20,382.0	0.0	720.0	0.0	21,102.0	**1.053**
1	13.0 ft	1,361 kips	0.25 in	18,980	12,600	450	22,678	22,776.0	0.0	720.0	0.0	23,496.0	**1.028**

GG Engineering	JOB NO. 9 - STORY BUILDINGS	BY SMG	DATE 9/16/04	SHEET NO. ___ OF ___
	CUSTOMER DESIGN 9C	CKD	DATE	

SUBJECT

STABILITY COEFFICIENT ALONG COLUMN X-AXIS, θ_x

MOMENT FRAME
MF A2 - F2

- ***LOAD COMBINATION:*** 1.2D + 1.6Lr + 0.8W
- ***DEFLECTION AMPLIFICATION FACTOR:*** C_d = 5.5
- ***(SEISMIC) IMPORTANCE FACTOR*** I_E = 1.0
- ***MOMENT FRAME RESISTS WHAT % OF THE TOTAL SEISMIC SHEAR TO THE BUIDLING?*** 25%

		DUE TO FORCES THAT CAUSE BENDING ALONG THE X-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					STABILITY COEFFICIENT PER STORY θ_i
								LOAD FACTOR					
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO ANY HORIZONTAL LOAD ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0	*ROOF L.L.* 1.6	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0	TOTAL ΣP_{ui} (kips)	
9	13.0 ft	380 kips	0.31 in	3,020	0	450	3,118	3,624.0	0.0	720.0	0.0	4,344.0	**0.023**
8	13.0 ft	633 kips	0.47 in	5,015	1,575	450	5,563	6,018.0	0.0	720.0	0.0	6,738.0	**0.032**
7	13.0 ft	844 kips	0.58 in	7,010	3,150	450	8,008	8,412.0	0.0	720.0	0.0	9,132.0	**0.040**
6	13.0 ft	1,014 kips	0.6 in	9,005	4,725	450	10,453	10,806.0	0.0	720.0	0.0	11,526.0	**0.044**
5	13.0 ft	1,146 kips	0.61 in	11,000	6,300	450	12,898	13,200.0	0.0	720.0	0.0	13,920.0	**0.047**
4	13.0 ft	1,244 kips	0.6 in	12,995	7,875	450	15,342	15,594.0	0.0	720.0	0.0	16,314.0	**0.050**
3	13.0 ft	1,309 kips	0.58 in	14,990	9,450	450	17,788	17,988.0	0.0	720.0	0.0	18,708.0	**0.053**
2	13.0 ft	1,346 kips	0.5 in	16,985	11,025	450	20,233	20,382.0	0.0	720.0	0.0	21,102.0	**0.050**
1	13.0 ft	1,361 kips	0.25 in	18,980	12,600	450	22,678	22,776.0	0.0	720.0	0.0	23,496.0	**0.028**

GG Engineering	JOB NO. 9 - STORY BUILDINGS	BY SMG	DATE 9/16/04	SHEET NO.
	CUSTOMER DESIGN 9C	CKD	DATE	OF

SUBJECT

STABILITY COEFFICIENT ALONG COLUMN Y-AXIS, θ_y

MOMENT FRAME
MF A2 - F2

- ***LOAD COMBINATION:*** 1.2D + 1.6Lr + 0.8W
- ***DEFLECTION AMPLIFICATION FACTOR:*** C_d = 5.5
- ***(SEISMIC) IMPORTANCE FACTOR*** I_E = 1.0
- ***MOMENT FRAME RESISTS WHAT % OF THE TOTAL SEISMIC SHEAR TO THE BUIDLING?*** 25%

		DUE TO FORCES THAT CAUSE BENDING ALONG THE Y-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					STABILITY COEFFICIENT PER STORY θ_i
								LOAD FACTOR					
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO ANY HORIZONTAL LOAD ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0	*ROOF L.L.* 1.6	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0	TOTAL ΣP_{ui} (kips)	
9	13.0 ft	380 kips	0.31 in	3,020	0	450	3,118	3,624.0	0.0	720.0	0.0	4,344.0	**0.023**
8	13.0 ft	633 kips	0.47 in	5,015	1,575	450	5,563	6,018.0	0.0	720.0	0.0	6,738.0	**0.032**
7	13.0 ft	844 kips	0.58 in	7,010	3,150	450	8,008	8,412.0	0.0	720.0	0.0	9,132.0	**0.040**
6	13.0 ft	1,014 kips	0.6 in	9,005	4,725	450	10,453	10,806.0	0.0	720.0	0.0	11,526.0	**0.044**
5	13.0 ft	1,146 kips	0.61 in	11,000	6,300	450	12,898	13,200.0	0.0	720.0	0.0	13,920.0	**0.047**
4	13.0 ft	1,244 kips	0.6 in	12,995	7,875	450	15,342	15,594.0	0.0	720.0	0.0	16,314.0	**0.050**
3	13.0 ft	1,309 kips	0.58 in	14,990	9,450	450	17,788	17,988.0	0.0	720.0	0.0	18,708.0	**0.053**
2	13.0 ft	1,346 kips	0.5 in	16,985	11,025	450	20,233	20,382.0	0.0	720.0	0.0	21,102.0	**0.050**
1	13.0 ft	1,361 kips	0.25 in	18,980	12,600	450	22,678	22,776.0	0.0	720.0	0.0	23,496.0	**0.028**

GG Engineering	JOB NO. 9 - STORY BUILDINGS	BY SMG	DATE 9/16/04	SHEET NO.
	CUSTOMER DESIGN 9C	CKD	DATE	___ OF ___

SUBJECT **B2 CALCULATION - *FOR BENDING ALONG THE X-AXIS OF THE COLUMN*** — **MOMENT FRAME *MF A2 - F2***

o ***LOAD COMBINATION =*** **1.2D + 0.5L + 0.5Lr + 1.6W** *(L.C. # 5)*

		DUE TO FORCES THAT CAUSE BENDING ALONG THE X-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					
								LOAD FACTOR					
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0.5	*ROOF L.L.* 0.5	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0	TOTAL ΣP_{ui} (kips)	**STORY $B_{2i\_X\text{-}AXIS}$**
9	13.0 ft	380 kips	0.31 in	3,020	0	450	3,118	3,624.0	0.0	225.0	0.0	3,849.0	**1.021**
8	13.0 ft	633 kips	0.47 in	5,015	1,575	450	5,563	6,018.0	787.5	225.0	0.0	7,030.5	**1.035**
7	13.0 ft	844 kips	0.58 in	7,010	3,150	450	8,008	8,412.0	1,575.0	225.0	0.0	10,212.0	**1.047**
6	13.0 ft	1,014 kips	0.6 in	9,005	4,725	450	10,453	10,806.0	2,362.5	225.0	0.0	13,393.5	**1.054**
5	13.0 ft	1,146 kips	0.61 in	11,000	6,300	450	12,898	13,200.0	3,150.0	225.0	0.0	16,575.0	**1.060**
4	13.0 ft	1,244 kips	0.6 in	12,995	7,875	450	15,342	15,594.0	3,937.5	225.0	0.0	19,756.5	**1.065**
3	13.0 ft	1,309 kips	0.58 in	14,990	9,450	450	17,788	17,988.0	4,725.0	225.0	0.0	22,938.0	**1.070**
2	13.0 ft	1,346 kips	0.5 in	16,985	11,025	450	20,233	20,382.0	5,512.5	225.0	0.0	26,119.5	**1.066**
1	13.0 ft	1,361 kips	0.25 in	18,980	12,600	450	22,678	22,776.0	6,300.0	225.0	0.0	29,301.0	**1.036**

GG Engineering	JOB NO. 9 - STORY BUILDINGS	BY SMG	DATE 9/16/04	SHEET NO. ___ OF ___
	CUSTOMER DESIGN 9C	CKD	DATE	

SUBJECT **B2 CALCULATION - *FOR BENDING ALONG THE Y-AXIS OF THE COLUMN*** — **MOMENT FRAME MF A2 - F2**

o ***LOAD COMBINATION =*** **1.2D + 0.5L + 0.5Lr + 1.6W**

		DUE TO FORCES THAT CAUSE BENDING ALONG THE Y-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					
								LOAD FACTOR					
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0.5	*ROOF L.L.* 0.5	*SEISMIC VERTICAL* $0.2S_{DS} = 0$	TOTAL ΣP_{ui} (kips)	**STORY $B_{2i\_Y\text{-}AXIS}$**
9	13.0 ft	380 kips	0.31 in	3,020	0	450	3,118	3,624.0	0.0	225.0	0.0	3,849.0	**1.021**
8	13.0 ft	633 kips	0.47 in	5,015	1,575	450	5,563	6,018.0	787.5	225.0	0.0	7,030.5	**1.035**
7	13.0 ft	844 kips	0.58 in	7,010	3,150	450	8,008	8,412.0	1,575.0	225.0	0.0	10,212.0	**1.047**
6	13.0 ft	1,014 kips	0.6 in	9,005	4,725	450	10,453	10,806.0	2,362.5	225.0	0.0	13,393.5	**1.054**
5	13.0 ft	1,146 kips	0.61 in	11,000	6,300	450	12,898	13,200.0	3,150.0	225.0	0.0	16,575.0	**1.060**
4	13.0 ft	1,244 kips	0.6 in	12,995	7,875	450	15,342	15,594.0	3,937.5	225.0	0.0	19,756.5	**1.065**
3	13.0 ft	1,309 kips	0.58 in	14,990	9,450	450	17,788	17,988.0	4,725.0	225.0	0.0	22,938.0	**1.070**
2	13.0 ft	1,346 kips	0.5 in	16,985	11,025	450	20,233	20,382.0	5,512.5	225.0	0.0	26,119.5	**1.066**
1	13.0 ft	1,361 kips	0.25 in	18,980	12,600	450	22,678	22,776.0	6,300.0	225.0	0.0	29,301.0	**1.036**

GG Engineering	JOB NO. 9 - STORY BUILDINGS	BY SMG	DATE 9/16/04	SHEET NO.
	CUSTOMER DESIGN 9C	CKD	DATE	___ OF ___

SUBJECT

STABILITY COEFFICIENT ALONG COLUMN X-AXIS, θ_x

MOMENT FRAME
MF A2 - F2

- ***LOAD COMBINATION:*** 1.2D + 0.5L + 0.5Lr + 1.6W
- ***DEFLECTION AMPLIFICATION FACTOR:*** C_d = 5.5
- ***(SEISMIC) IMPORTANCE FACTOR*** I_E = 1.0
- ***MOMENT FRAME RESISTS WHAT % OF THE TOTAL SEISMIC SHEAR TO THE BUIDLING?*** 25%

		DUE TO FORCES THAT CAUSE BENDING ALONG THE X-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					
								LOAD FACTOR					
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO ANY HORIZONTAL LOAD ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0.5	*ROOF L.L.* 0.5	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0	TOTAL ΣP_{ui} (kips)	STABILITY COEFFICIENT PER STORY θ_i
9	13.0 ft	380 kips	0.31 in	3,020	0	450	3,118	3,624.0	0.0	225.0	0.0	3,849.0	**0.020**
8	13.0 ft	633 kips	0.47 in	5,015	1,575	450	5,563	6,018.0	787.5	225.0	0.0	7,030.5	**0.033**
7	13.0 ft	844 kips	0.58 in	7,010	3,150	450	8,008	8,412.0	1,575.0	225.0	0.0	10,212.0	**0.045**
6	13.0 ft	1,014 kips	0.6 in	9,005	4,725	450	10,453	10,806.0	2,362.5	225.0	0.0	13,393.5	**0.051**
5	13.0 ft	1,146 kips	0.61 in	11,000	6,300	450	12,898	13,200.0	3,150.0	225.0	0.0	16,575.0	**0.057**
4	13.0 ft	1,244 kips	0.6 in	12,995	7,875	450	15,342	15,594.0	3,937.5	225.0	0.0	19,756.5	**0.061**
3	13.0 ft	1,309 kips	0.58 in	14,990	9,450	450	17,788	17,988.0	4,725.0	225.0	0.0	22,938.0	**0.065**
2	13.0 ft	1,346 kips	0.5 in	16,985	11,025	450	20,233	20,382.0	5,512.5	225.0	0.0	26,119.5	**0.062**
1	13.0 ft	1,361 kips	0.25 in	18,980	12,600	450	22,678	22,776.0	6,300.0	225.0	0.0	29,301.0	**0.035**

GG Engineering	JOB NO. 9 - STORY BUILDINGS	BY SMG	DATE 9/16/04	SHEET NO. ___ OF ___
	CUSTOMER DESIGN 9C	CKD	DATE	

SUBJECT

STABILITY COEFFICIENT ALONG COLUMN Y-AXIS, θ_y

MOMENT FRAME
MF A2 - F2

- ***LOAD COMBINATION:*** 1.2D + 0.5L + 0.5Lr + 1.6W
- ***DEFLECTION AMPLIFICATION FACTOR:*** C_d = 5.5
- ***(SEISMIC) IMPORTANCE FACTOR*** I_E = 1.0
- ***MOMENT FRAME RESISTS WHAT % OF THE TOTAL SEISMIC SHEAR TO THE BUIDLING?*** 25%

		DUE TO FORCES THAT CAUSE BENDING ALONG THE Y-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					STABILITY COEFFICIENT PER STORY θ_i
								LOAD FACTOR					
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO ANY HORIZONTAL LOAD ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0.5	*ROOF L.L.* 0.5	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0	TOTAL ΣP_{ui} (kips)	
9	13.0 ft	380 kips	0.31 in	3,020	0	450	3,118	3,624.0	0.0	225.0	0.0	3,849.0	**0.020**
8	13.0 ft	633 kips	0.47 in	5,015	1,575	450	5,563	6,018.0	787.5	225.0	0.0	7,030.5	**0.033**
7	13.0 ft	844 kips	0.58 in	7,010	3,150	450	8,008	8,412.0	1,575.0	225.0	0.0	10,212.0	**0.045**
6	13.0 ft	1,014 kips	0.6 in	9,005	4,725	450	10,453	10,806.0	2,362.5	225.0	0.0	13,393.5	**0.051**
5	13.0 ft	1,146 kips	0.61 in	11,000	6,300	450	12,898	13,200.0	3,150.0	225.0	0.0	16,575.0	**0.057**
4	13.0 ft	1,244 kips	0.6 in	12,995	7,875	450	15,342	15,594.0	3,937.5	225.0	0.0	19,756.5	**0.061**
3	13.0 ft	1,309 kips	0.58 in	14,990	9,450	450	17,788	17,988.0	4,725.0	225.0	0.0	22,938.0	**0.065**
2	13.0 ft	1,346 kips	0.5 in	16,985	11,025	450	20,233	20,382.0	5,512.5	225.0	0.0	26,119.5	**0.062**
1	13.0 ft	1,361 kips	0.25 in	18,980	12,600	450	22,678	22,776.0	6,300.0	225.0	0.0	29,301.0	**0.035**

GG Engineering	JOB NO. 9 - STORY BUILDINGS	BY SMG	DATE 9/16/04	SHEET NO. ___ OF ___
	CUSTOMER DESIGN 9C	CKD	DATE	

SUBJECT **B2 CALCULATION - *FOR BENDING ALONG THE X-AXIS OF THE COLUMN*** **MOMENT FRAME** ***MF A2 - F2***

o ***LOAD COMBINATION =*** **1.2D + 0.5L + 1.0E** *(L.C. # 6)*

		DUE TO FORCES THAT CAUSE BENDING ALONG THE X-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					
								LOAD FACTOR					
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0.5	*ROOF L.L.* 0	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0.2	TOTAL ΣP_{ui} (kips)	**STORY $B_{2i\_X\text{-}AXIS}$**
9	13.0 ft	380 kips	0.31 in	3,020	0	450	3,118	3,624.0	0.0	0.0	623.6	4,247.6	**1.023**
8	13.0 ft	633 kips	0.47 in	5,015	1,575	450	5,563	6,018.0	787.5	0.0	1,112.6	7,918.1	**1.039**
7	13.0 ft	844 kips	0.58 in	7,010	3,150	450	8,008	8,412.0	1,575.0	0.0	1,601.6	11,588.6	**1.054**
6	13.0 ft	1,014 kips	0.6 in	9,005	4,725	450	10,453	10,806.0	2,362.5	0.0	2,090.6	15,259.1	**1.061**
5	13.0 ft	1,146 kips	0.61 in	11,000	6,300	450	12,898	13,200.0	3,150.0	0.0	2,579.6	18,929.6	**1.069**
4	13.0 ft	1,244 kips	0.6 in	12,995	7,875	450	15,342	15,594.0	3,937.5	0.0	3,068.4	22,599.9	**1.075**
3	13.0 ft	1,309 kips	0.58 in	14,990	9,450	450	17,788	17,988.0	4,725.0	0.0	3,557.6	26,270.6	**1.081**
2	13.0 ft	1,346 kips	0.5 in	16,985	11,025	450	20,233	20,382.0	5,512.5	0.0	4,046.6	29,941.1	**1.077**
1	13.0 ft	1,361 kips	0.25 in	18,980	12,600	450	22,678	22,776.0	6,300.0	0.0	4,535.6	33,611.6	**1.041**

GG Engineering	JOB NO. 9 - STORY BUILDINGS	BY SMG	DATE 9/16/04	SHEET NO. ___ OF ___
	CUSTOMER DESIGN 9C	CKD	DATE	

SUBJECT **B2 CALCULATION - *FOR BENDING ALONG THE Y-AXIS OF THE COLUMN*** — **MOMENT FRAME MF A2 - F2**

- ***LOAD COMBINATION =*** **1.2D + 0.5L + 1.0E**

STORY NUMBER	LENGTH *(STORY HEIGHT)* L	DUE TO FORCES THAT CAUSE BENDING ALONG THE Y-AXIS OF THE MOMENT FRAME COLUMNS: TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips): DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY — *LOAD FACTOR*: *D.L.* 1.2	*L.L.* 0.5	*ROOF L.L.* 0	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0.2	TOTAL ΣP_{ui} (kips)	**STORY $B_{2i\_Y\text{-}AXIS}$**
9	13.0 ft	380 kips	0.31 in	3,020	0	450	3,118	3,624.0	0.0	0.0	623.6	4,247.6	**1.023**
8	13.0 ft	633 kips	0.47 in	5,015	1,575	450	5,563	6,018.0	787.5	0.0	1,112.6	7,918.1	**1.039**
7	13.0 ft	844 kips	0.58 in	7,010	3,150	450	8,008	8,412.0	1,575.0	0.0	1,601.6	11,588.6	**1.054**
6	13.0 ft	1,014 kips	0.6 in	9,005	4,725	450	10,453	10,806.0	2,362.5	0.0	2,090.6	15,259.1	**1.061**
5	13.0 ft	1,146 kips	0.61 in	11,000	6,300	450	12,898	13,200.0	3,150.0	0.0	2,579.6	18,929.6	**1.069**
4	13.0 ft	1,244 kips	0.6 in	12,995	7,875	450	15,342	15,594.0	3,937.5	0.0	3,068.4	22,599.9	**1.075**
3	13.0 ft	1,309 kips	0.58 in	14,990	9,450	450	17,788	17,988.0	4,725.0	0.0	3,557.6	26,270.6	**1.081**
2	13.0 ft	1,346 kips	0.5 in	16,985	11,025	450	20,233	20,382.0	5,512.5	0.0	4,046.6	29,941.1	**1.077**
1	13.0 ft	1,361 kips	0.25 in	18,980	12,600	450	22,678	22,776.0	6,300.0	0.0	4,535.6	33,611.6	**1.041**

GG Engineering	JOB NO. 9 - STORY BUILDINGS	BY SMG	DATE 9/16/04	SHEET NO.
	CUSTOMER DESIGN 9C	CKD	DATE	___ OF ___

SUBJECT

STABILITY COEFFICIENT ALONG COLUMN X-AXIS, θ_x

MOMENT FRAME
MF A2 - F2

- ***LOAD COMBINATION:*** 1.2D + 0.5L + 1.0E
- ***DEFLECTION AMPLIFICATION FACTOR:*** C_d = 5.5
- ***(SEISMIC) IMPORTANCE FACTOR*** I_E = 1.0
- ***MOMENT FRAME RESISTS WHAT % OF THE TOTAL SEISMIC SHEAR TO THE BUIDLING?*** 25%

		DUE TO FORCES THAT CAUSE BENDING ALONG THE X-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					STABILITY COEFFICIENT PER STORY θ_i
								LOAD FACTOR					
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO ANY HORIZONTAL LOAD ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0.5	*ROOF L.L.* 0	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0.2	TOTAL ΣP_{ui} (kips)	
9	13.0 ft	380 kips	0.31 in	3,020	0	450	3,118	3,624.0	0.0	0.0	623.6	4,247.6	**0.022**
8	13.0 ft	633 kips	0.47 in	5,015	1,575	450	5,563	6,018.0	787.5	0.0	1,112.6	7,918.1	**0.038**
7	13.0 ft	844 kips	0.58 in	7,010	3,150	450	8,008	8,412.0	1,575.0	0.0	1,601.6	11,588.6	**0.051**
6	13.0 ft	1,014 kips	0.6 in	9,005	4,725	450	10,453	10,806.0	2,362.5	0.0	2,090.6	15,259.1	**0.058**
5	13.0 ft	1,146 kips	0.61 in	11,000	6,300	450	12,898	13,200.0	3,150.0	0.0	2,579.6	18,929.6	**0.065**
4	13.0 ft	1,244 kips	0.6 in	12,995	7,875	450	15,342	15,594.0	3,937.5	0.0	3,068.4	22,599.9	**0.070**
3	13.0 ft	1,309 kips	0.58 in	14,990	9,450	450	17,788	17,988.0	4,725.0	0.0	3,557.6	26,270.6	**0.075**
2	13.0 ft	1,346 kips	0.5 in	16,985	11,025	450	20,233	20,382.0	5,512.5	0.0	4,046.6	29,941.1	**0.071**
1	13.0 ft	1,361 kips	0.25 in	18,980	12,600	450	22,678	22,776.0	6,300.0	0.0	4,535.6	33,611.6	**0.040**

GG Engineering	JOB NO. 9 - STORY BUILDINGS	BY SMG	DATE 9/16/04	SHEET NO.
	CUSTOMER DESIGN 9C	CKD	DATE	___ OF ___

SUBJECT

STABILITY COEFFICIENT ALONG COLUMN X-AXIS, θ_x

MOMENT FRAME
MF A2 - F2

- ***LOAD COMBINATION:*** 1.2D + 0.5L + 1.0E
- ***DEFLECTION AMPLIFICATION FACTOR:*** C_d = 5.5

		DUE TO FORCES THAT CAUSE BENDING ALONG THE X-AXIS OF THE MOMENT FRAME COLUMNS						
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH_i	INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	TOTAL STORY SHEAR CAPACITY (OF ALL OF THE SEISMIC RESISTING MOMENT FRAMES)	RATIO OF SHEAR DEMAND / SHEAR CAPACITY PER STORY β	MAXIMUM ALLOWED STABILITY COEFFICIENT PER STORY θ_{i_max}	STABILITY COEFFICIENT PER STORY θ_i	***COMMENT***
9	13.0 ft	380 kips	0.31 in	57,421 kips	0.0066	0.250	0.022	***OK***
8	13.0 ft	633 kips	0.47 in	57,421 kips	0.0110	0.250	0.038	***OK***
7	13.0 ft	844 kips	0.58 in	10,145 kips	0.0832	0.250	0.051	***OK***
6	13.0 ft	1,014 kips	0.6 in	6,668 kips	0.1521	0.250	0.058	***OK***
5	13.0 ft	1,146 kips	0.61 in	6,668 kips	0.1719	0.250	0.065	***OK***
4	13.0 ft	1,244 kips	0.6 in	6,192 kips	0.2009	0.250	0.070	***OK***
3	13.0 ft	1,309 kips	0.58 in	6,192 kips	0.2114	0.250	0.075	***OK***
2	13.0 ft	1,346 kips	0.5 in	6,192 kips	0.2174	0.250	0.071	***OK***
1	13.0 ft	1,361 kips	0.25 in	6,192 kips	0.2198	0.250	0.040	***OK***

GG Engineering	JOB NO. 9 - STORY BUILDINGS	BY SMG	DATE 9/16/04	SHEET NO. ___ OF ___
	CUSTOMER DESIGN 9C	CKD	DATE	

SUBJECT

STABILITY COEFFICIENT ALONG COLUMN Y-AXIS, θ_y

MOMENT FRAME
MF A2 - F2

- o *LOAD COMBINATION:* 1.2D + 0.5L + 1.0E
- o *DEFLECTION AMPLIFICATION FACTOR:* $C_d =$ 5.5
- o *(SEISMIC) IMPORTANCE FACTOR* $I_E =$ 1.0
- o *MOMENT FRAME RESISTS WHAT % OF THE TOTAL SEISMIC SHEAR TO THE BUIDLING?* 25%

		DUE TO FORCES THAT CAUSE BENDING ALONG THE Y-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					STABILITY COEFFICIENT PER STORY θ_i
								LOAD FACTOR				TOTAL ΣP_{ui} (kips)	
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO ANY HORIZONTAL LOAD ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0.5	*ROOF L.L.* 0	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0.2		
9	13.0 ft	380 kips	0.31 in	3,020	0	450	3,118	3,624.0	0.0	0.0	623.6	4,247.6	**0.022**
8	13.0 ft	633 kips	0.47 in	5,015	1,575	450	5,563	6,018.0	787.5	0.0	1,112.6	7,918.1	**0.038**
7	13.0 ft	844 kips	0.58 in	7,010	3,150	450	8,008	8,412.0	1,575.0	0.0	1,601.6	11,588.6	**0.051**
6	13.0 ft	1,014 kips	0.6 in	9,005	4,725	450	10,453	10,806.0	2,362.5	0.0	2,090.6	15,259.1	**0.058**
5	13.0 ft	1,146 kips	0.61 in	11,000	6,300	450	12,898	13,200.0	3,150.0	0.0	2,579.6	18,929.6	**0.065**
4	13.0 ft	1,244 kips	0.6 in	12,995	7,875	450	15,342	15,594.0	3,937.5	0.0	3,068.4	22,599.9	**0.070**
3	13.0 ft	1,309 kips	0.58 in	14,990	9,450	450	17,788	17,988.0	4,725.0	0.0	3,557.6	26,270.6	**0.075**
2	13.0 ft	1,346 kips	0.5 in	16,985	11,025	450	20,233	20,382.0	5,512.5	0.0	4,046.6	29,941.1	**0.071**
1	13.0 ft	1,361 kips	0.25 in	18,980	12,600	450	22,678	22,776.0	6,300.0	0.0	4,535.6	33,611.6	**0.040**

GG Engineering	JOB NO. 9 - STORY BUILDINGS	BY SMG	DATE 9/16/04	SHEET NO. ___ OF ___
	CUSTOMER DESIGN 9C	CKD	DATE	

SUBJECT

STABILITY COEFFICIENT ALONG COLUMN Y-AXIS, θ_y

MOMENT FRAME
MF A2 - F2

- ***LOAD COMBINATION:*** 1.2D + 0.5L + 1.0E
- ***DEFLECTION AMPLIFICATION FACTOR:*** C_d = 5.5

		DUE TO FORCES THAT CAUSE BENDING ALONG THE Y-AXIS OF THE MOMENT FRAME COLUMNS						
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH_i	INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	TOTAL STORY SHEAR CAPACITY (OF ALL OF THE SEISMIC RESISTING MOMENT FRAMES)	RATIO OF SHEAR DEMAND / SHEAR CAPACITY PER STORY β	MAXIMUM ALLOWED STABILITY COEFFICIENT PER STORY θ_{i_max}	STABILITY COEFFICIENT PER STORY θ_i	***COMMENT***
9	13.0 ft	380 kips	0.31 in	57,421 kips	0.0066	0.250	0.022	***OK***
8	13.0 ft	633 kips	0.47 in	57,421 kips	0.0110	0.250	0.038	***OK***
7	13.0 ft	844 kips	0.58 in	10,145 kips	0.0832	0.250	0.051	***OK***
6	13.0 ft	1,014 kips	0.6 in	6,668 kips	0.1521	0.250	0.058	***OK***
5	13.0 ft	1,146 kips	0.61 in	6,668 kips	0.1719	0.250	0.065	***OK***
4	13.0 ft	1,244 kips	0.6 in	6,192 kips	0.2009	0.250	0.070	***OK***
3	13.0 ft	1,309 kips	0.58 in	6,192 kips	0.2114	0.250	0.075	***OK***
2	13.0 ft	1,346 kips	0.5 in	6,192 kips	0.2174	0.250	0.071	***OK***
1	13.0 ft	1,361 kips	0.25 in	6,192 kips	0.2198	0.250	0.040	***OK***

COLUMN		MAXIMUM INTERACTION VALUE	CONTROLLING LOAD COMBINATION	BUILDING MAX. INTERACTION 0.9104	COMMENTS FOR: DESIGN 9C	
NAME	MEMBER SIZE				COLUMN STEEL AREA CHECK	COLUMN COMPACTNESS CHECK
A2-1	HSS 27 x 27 x 0.3125	0.767832739	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
A2-2	HSS 27 x 27 x 0.3125	0.377304752	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
A2-3	HSS 27 x 27 x 0.3125	0.226830201	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
A2-4	HSS 27 x 27 x 0.3125	0.189153185	2		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
A2-5	HSS 25 x 25 x 0.3125	0.215737305	2		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
A2-6	HSS 25 x 25 x 0.3125	0.224119667	2		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
A2-7	HSS 22 x 22 x 0.375	0.23727231	2		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
A2-8	HSS 22 x 22 x 0.3125	0.31803858	2		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
A2-9	HSS 22 x 22 x 0.3125	0.344789777	4		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
B2-1	HSS 27 x 27 x 0.3125	0.896133118	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
B2-2	HSS 27 x 27 x 0.3125	0.645415472	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
B2-3	HSS 27 x 27 x 0.3125	0.531147114	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
B2-4	HSS 27 x 27 x 0.3125	0.474147412	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
B2-5	HSS 25 x 25 x 0.3125	0.516416029	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
B2-6	HSS 25 x 25 x 0.3125	0.461921239	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
B2-7	HSS 22 x 22 x 0.375	0.442588351	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
B2-8	HSS 22 x 22 x 0.3125	0.450584244	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
B2-9	HSS 22 x 22 x 0.3125	0.343905363	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
C2-1	HSS 27 x 27 x 0.3125	0.68990783	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
C2-2	HSS 27 x 27 x 0.3125	0.489855402	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
C2-3	HSS 27 x 27 x 0.3125	0.405922611	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
C2-4	HSS 27 x 27 x 0.3125	0.360326244	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
C2-5	HSS 25 x 25 x 0.3125	0.395223802	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
C2-6	HSS 25 x 25 x 0.3125	0.351835234	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
C2-7	HSS 22 x 22 x 0.375	0.338308469	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
C2-8	HSS 22 x 22 x 0.3125	0.33417182	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
C2-9	HSS 22 x 22 x 0.3125	0.239106672	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
D2-1	HSS 27 x 27 x 0.3125	0.693301826	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
D2-2	HSS 27 x 27 x 0.3125	0.48853368	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
D2-3	HSS 27 x 27 x 0.3125	0.405094667	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
D2-4	HSS 27 x 27 x 0.3125	0.359613472	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
D2-5	HSS 25 x 25 x 0.3125	0.395495864	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
D2-6	HSS 25 x 25 x 0.3125	0.350477978	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
D2-7	HSS 22 x 22 x 0.375	0.336370974	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
D2-8	HSS 22 x 22 x 0.3125	0.365604195	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
D2-9	HSS 22 x 22 x 0.3125	0.266419529	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
E2-1	HSS 27 x 27 x 0.3125	0.908047773	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
E2-2	HSS 27 x 27 x 0.3125	0.642706488	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
E2-3	HSS 27 x 27 x 0.3125	0.534358815	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
E2-4	HSS 27 x 27 x 0.3125	0.477832784	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
E2-5	HSS 25 x 25 x 0.3125	0.527075833	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
E2-6	HSS 25 x 25 x 0.3125	0.462768167	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
E2-7	HSS 22 x 22 x 0.375	0.455464359	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
E2-8	HSS 22 x 22 x 0.3125	0.474670317	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
E2-9	HSS 22 x 22 x 0.3125	0.598277317	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
F2-1	HSS 27 x 27 x 0.3125	0.910395868	6	***<---CONTROLS!***	OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
F2-2	HSS 27 x 27 x 0.3125	0.637488415	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
F2-3	HSS 27 x 27 x 0.3125	0.493472078	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
F2-4	HSS 27 x 27 x 0.3125	0.4305158	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
F2-5	HSS 25 x 25 x 0.3125	0.484989091	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
F2-6	HSS 25 x 25 x 0.3125	0.459299179	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
F2-7	HSS 22 x 22 x 0.375	0.482490899	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
F2-8	HSS 22 x 22 x 0.3125	0.581049534	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
F2-9	HSS 22 x 22 x 0.3125	0.524448535	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!

Appendix E

18-Story Building Nominal Loads

The first step in the linear design of the 18-story buildings was to determine the nominal (unfactored) loads that each building needed to be designed to resist. Once the building layout and geometries were determined the gravity loads (dead and live loads) were calculated followed by the environmental (wind and seismic) loads. This appendix shows the design calculations that determined the nominal loads for the 18-story building designs.

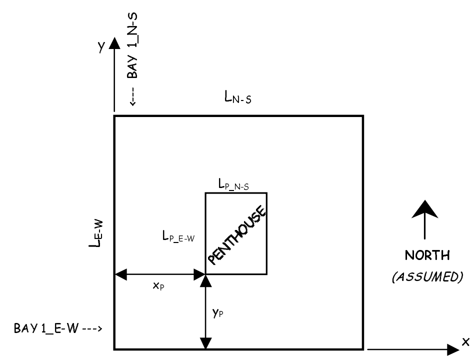
GG Engineering	JOB NO. 18-STORY BUILDINGS	BY SMG	DATE 4/22/05	SHEET NO. ___ OF ___
	CUSTOMER RCFT PARAMETRIC STUDY	CKD	DATE	

SUBJECT

2-D MOMENT FRAME [MF A3 - G3] ANALYSIS LOAD SUMMARY

[1] BUILDING GEOMETRY

- o NUMBER OF STORIES, N_S — 18 STORIES
- o NUMBER OF BAYS:
  - ALONG THE N-S *FACE* OF THE BUILDING, $N_{B\_N\text{-}S}$ — 6 BAYS
  - ALONG THE E-W *FACE* OF THE BUILDING, $N_{B\_E\text{-}W}$ — 6 BAYS
- o BUILDING LENGTH (CTR-TO-CTR OF COLUMNS):
  - ALONG THE N-S *FACE* OF THE BUILDING, $L_{N\text{-}S}$ — 120.0 ft
  - ALONG THE E-W *FACE* OF THE BUILDING, $L_{E\text{-}W}$ — 120.0 ft
- o CTR-TO-CTR DISTANCE (SPACING) OF THE FOLLOWING ITEM(S):
  - BEAMS — 10.0 ft
- o PARAPET HEIGHT — 3.5 ft
- o PENTHOUSE:
  - HEIGHT — 13.0 ft
  - LENGTH ALONG THE N-S *FACE* OF THE BUILDING, $L_{P\_N\text{-}S}$ — 40.0 ft
  - LENGTH ALONG THE E-W *FACE* OF THE BUILDING, $L_{P\_E\text{-}W}$ — 40.0 ft
- o DISTANCE OF THE *SOUTHWEST* CORNER (OF THE PENTHOUSE):
  - x_P — 40.0 ft
  - y_P — 40.0 ft



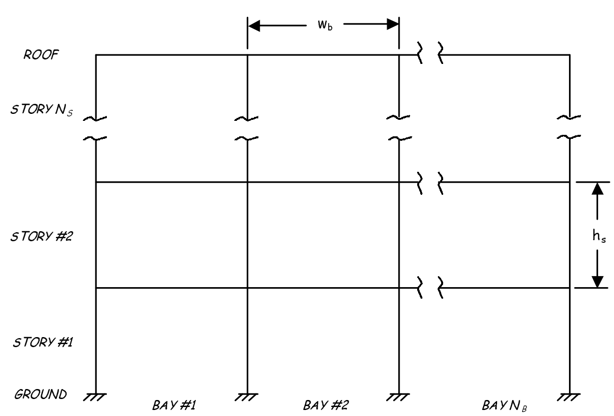
PLAN VIEW

- o DIRECTION THAT THE *COMPOSITE FLOOR SYSTEM* SPANS — E-W — *THEREFORE, THE BEAMS SPAN IN THE N-S DIRECTION*

STORY NUMBER	STORY HEIGHT, h_s (CTR-TO-CTR OF GIRDERS)
18	13.0 ft
17	13.0 ft
16	13.0 ft
15	13.0 ft
14	13.0 ft
13	13.0 ft
12	13.0 ft
11	13.0 ft
10	13.0 ft
9	13.0 ft
8	13.0 ft
7	13.0 ft
6	13.0 ft
5	13.0 ft
4	13.0 ft
3	13.0 ft
2	13.0 ft
1	13.0 ft

BAY WIDTH, w_b ALONG EACH *FACE* OF THE BUILDING

ALONG THE N-S *FACE* (GIRDERS ARE PARALLEL TO THE E-W DIRECTION)		ALONG THE E-W *FACE* (GIRDERS ARE PARALLEL TO THE N-S DIRECTION)	
BAY NUMBER	WIDTH, w_b *CTR-TO-CTR OF COLUMNS*	BAY NUMBER	WIDTH, w_b *CTR-TO-CTR OF COLUMNS*
1_N-S	20.0 ft	1_E-W	20.0 ft
2_N-S	20.0 ft	2_E-W	20.0 ft
3_N-S	20.0 ft	3_E-W	20.0 ft
4_N-S	20.0 ft	4_E-W	20.0 ft
5_N-S	20.0 ft	5_E-W	20.0 ft
6_N-S	20.0 ft	6_E-W	20.0 ft



ELEVATION VIEW

GG Engineering	JOB NO. 18-STORY BUILDINGS	BY SMG	DATE 4/22/05	SHEET NO.
	CUSTOMER RCFT PARAMETRIC STUDY	CKD	DATE	___ OF ___

SUBJECT

2-D MOMENT FRAME [MF A3 - G3] ANALYSIS LOAD SUMMARY

[2] 2-D MOMENT FRAME GEOMETRY

- o TYPE OF FRAME: MOMENT FRAME
- o DIRECTION THAT THE MOMENT FRAME RUNS PARALLEL WITH: E-W *(THEREFORE, THIS MOMENT FRAME WILL RESIST SEISMIC LOADS IN THE E-W DIRECTION)*
- o MOMENT FRAME NAME: MF A3 - G3
- o DISTANCE FROM THE CLOSEST COLUMN STACK IN THE MOMENT FRAME TO THE SOUTHWEST CORNER OF THE BUILDING, Yc: 80.0 ft *THIS FRAME IS LOCATED IN THE INTERIOR OF THE BUILDING AND THIS FRAME SUPPORTS PART OF THE PENTHOUSE.*
- o NUMBER OF BAYS IN THIS MOMENT FRAME: 6 BAYS
- o THE FIRST BAY IN THIS MOMENT FRAME IS THE SAME BAY AS WHICH BAY NUMBER IN THE BUILDING LAYOUT? 1_N-S *THIS MOMENT FRAME HAS A TOTAL LENGTH OF 120ft.*
- o DISTANCE TO THE CLOSEST (GRAVITY/MOMENT) FRAME: 20.0 ft
- o DISTANCE TO THE CLOSEST (GRAVITY/MOMENT) FRAME ON THE OTHER SIDE: 20.0 ft
- o JOINT COORDINATES WITH RESPECT TO THE SOUTHWEST CORNER OF THE BUILDING:

Coordinates of the MOMENT FRAME:		
End Column Location:	X	Y
Farthest West	0.0 ft	80.0 ft
Farthest East	120.0 ft	80.0 ft

Coordinates of the Penthouse:		
Corner Location	X	Y
Southwest	40.0 ft	40.0 ft
Northwest	40.0 ft	80.0 ft
Northeast	80.0 ft	80.0 ft
Southeast	80.0 ft	40.0 ft

COORDINATES OF THE MOMENT FRAME COLUMN STACKS WRT THE SOUTHWEST CORNER OF THE BUILDING

COLUMN NUMBER	1		2		3		4		5		6		7									
	x	y	x	y	x	y	x	y	x	y	x	y	x	y	x	y	x	y	x	y	x	y
	0	80	20	80	40	80	60	80	80	80	100	80	120	80								

- o ASSUMING THAT THE PENTHOUSE PERIMETER IS ALWAYS LOCATED OVER A GRAVITY/MOMENT FRAME, THE FOLLOWING IS TRUE FOR THIS MOMENT FRAME:

COLUMN (STACK) NO.	1	2	3	4	5	6	7				
COLUMN (STACK) IS SUPPORTING WHICH PART OF THE PENTHOUSE?	NONE	NONE	EXTERIOR	EXTERIOR	EXTERIOR	NONE	NONE				

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SUBJECT

2-D MOMENT FRAME [MF A3 - G3] ANALYSIS LOAD SUMMARY

[3] BUILDING DEAD LOAD

o BUILDING (FLOORS):
- *COLUMNS, BEAMS, GIRDERS, MISC. STRUCTURAL SYSTEM COMPONENTS* 20 lb/ft²
- *EXTERIOR WALLS* 25 lb/ft² (Applied to Surface Area of the *WALL*)
- *FLOORING* 1 lb/ft²
- *COMPOSITE FLOOR SYSTEM (CONCRETE + METAL DECKING)* 50 lb/ft²
- *CEILING (FROM STORY BELOW) + FIREPROOFING* 2 lb/ft²
- *HVAC + ELECTRICAL (FROM STORY BELOW)* 7 lb/ft²

o BUILDING (ROOF):
- *PARAPET* 25 lb/ft² (Applied to Surface Area of the *WALL*)
- *ROOFING* 7 lb/ft²
- *COMPOSITE ROOF SYSTEM (CONCRETE + METAL DECKING)* 50 lb/ft²
- *(ROOF) BEAMS, GIRDERS, MISC. STRUCTURAL SYSTEM COMPONENTS* 20 lb/ft²
- *CEILING (FROM STORY BELOW) + FIREPROOFING* 2 lb/ft²
- *HVAC + ELECTRICAL (FROM STORY BELOW)* 7 lb/ft²

o PENTHOUSE:
- *COMPOSITE ROOF SYSTEM (CONCRETE + METAL DECKING)* 50 lb/ft²
- *CEILING + FIREPROOFING* 2 lb/ft²
- *EXTERIOR WALLS* 25 lb/ft² (Applied to Surface Area of the *WALL*)
- *COLUMNS, BEAMS, GIRDERS, MISC. STRUCTURAL SYSTEM COMPONENTS* 20 lb/ft²
- *MECHANICAL EQUIPMENT* 40 lb/ft²
- *FLOORING* 1 lb/ft²

NUMBER		(STORY)	D.L. (SURFACE) AREA		DEAD LOAD		TOTAL DEAD LOAD		TOTAL STORY DL
STORY	FLOOR	HEIGHT	ROOF / FLOOR	PENTHOUSE	ROOF / FLOOR	PENTHOUSE	ROOF / FLOOR	PENTHOUSE	ΣP_i
---	---	---	---	1,600 ft²	---	145.50 lb/ft²	---	232.8 kips	---
---	ROOF	---	14,400 ft²	---	88.92 lb/ft²	---	1,280.45 kips	---	---
18	18	13.0 ft	14,400 ft²	---	90.83 lb/ft²	---	1,307.95 kips	---	1,513 kips
17	17	13.0 ft	14,400 ft²	---	90.83 lb/ft²	---	1,307.95 kips	---	2,821 kips
16	16	13.0 ft	14,400 ft²	---	90.83 lb/ft²	---	1,307.95 kips	---	4,129 kips
15	15	13.0 ft	14,400 ft²	---	90.83 lb/ft²	---	1,307.95 kips	---	5,437 kips
14	14	13.0 ft	14,400 ft²	---	90.83 lb/ft²	---	1,307.95 kips	---	6,745 kips
13	13	13.0 ft	14,400 ft²	---	90.83 lb/ft²	---	1,307.95 kips	---	8,053 kips
12	12	13.0 ft	14,400 ft²	---	90.83 lb/ft²	---	1,307.95 kips	---	9,361 kips
11	11	13.0 ft	14,400 ft²	---	90.83 lb/ft²	---	1,307.95 kips	---	10,669 kips
10	10	13.0 ft	14,400 ft²	---	90.83 lb/ft²	---	1,307.95 kips	---	11,977 kips
9	9	13.0 ft	14,400 ft²	---	90.83 lb/ft²	---	1,307.95 kips	---	13,285 kips
8	8	13.0 ft	14,400 ft²	---	90.83 lb/ft²	---	1,307.95 kips	---	14,593 kips
7	7	13.0 ft	14,400 ft²	---	90.83 lb/ft²	---	1,307.95 kips	---	15,901 kips
6	6	13.0 ft	14,400 ft²	---	90.83 lb/ft²	---	1,307.95 kips	---	17,209 kips
5	5	13.0 ft	14,400 ft²	---	90.83 lb/ft²	---	1,307.95 kips	---	18,517 kips
4	4	13.0 ft	14,400 ft²	---	90.83 lb/ft²	---	1,307.95 kips	---	19,825 kips
3	3	13.0 ft	14,400 ft²	---	90.83 lb/ft²	---	1,307.95 kips	---	21,133 kips
2	2	13.0 ft	14,400 ft²	---	90.83 lb/ft²	---	1,307.95 kips	---	22,441 kips
1	GROUND	13.0 ft	N.A.	---	N.A.	---	N.A.	---	23,749 kips
Building Total Dead Load (Ground Floor + 1st Story Dead Load NOT Included) =									*23,748 kips*

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SUBJECT

2-D MOMENT FRAME [MF A3 - G3] ANALYSIS LOAD SUMMARY

[4] BUILDING LIVE LOAD

- o BUILDING (FLOORS):
  - *OFFICE BUILDING OCCUPANCY PER IBC 2003, TABLE 1607.1* — 50 lb/ft^2
  - *(MOVEABLE) PARTITIONS PER IBC 2003, SECTION 1607.5* — 20 lb/ft^2
- o BUILDING (ROOF):
  - *MINIMUM ROOF LL PER IBC 2003, SECTION 1607.11* — 20 lb/ft^2 *(ROOF LIVE LOAD, Lr)*
- o PENTHOUSE:
  - *GENERAL PENTHOUSE (INTERIOR) LIVE LOAD* — 20 lb/ft^2 *(TREATED AS ROOF LIVE LOAD, Lr)*
  - *PENTHOUSE (ROOF) LIVE LOAD* — 0 lb/ft^2 *(ROOF LIVE LOAD, Lr)*

NUMBER		(STORY)	L.L. (SURFACE) AREA		LIVE LOAD		TOTAL LIVE LOAD		TOTAL STORY LL, ΣP_i	
STORY	FLOOR	HEIGHT	ROOF / FLOOR	PENTHOUSE	ROOF / FLOOR	PENTHOUSE	ROOF / FLOOR	PENTHOUSE	*FLOOR LL*	*ROOF LL*
---	---	---	---	1,600 ft^2	---	20 lb/ft^2	---	32.0 kips	---	---
---	ROOF	---	12,800 ft^2	---	20 lb/ft^2	---	256.0 kips	---	---	---
18	18	13.0 ft	14,400 ft^2	---	70 lb/ft^2	---	1,008.0 kips	---	*0 kips*	*288 kips*
17	17	13.0 ft	14,400 ft^2	---	70 lb/ft^2	---	1,008.0 kips	---	1,008 kips	288 kips
16	16	13.0 ft	14,400 ft^2	---	70 lb/ft^2	---	1,008.0 kips	---	2,016 kips	288 kips
15	15	13.0 ft	14,400 ft^2	---	70 lb/ft^2	---	1,008.0 kips	---	3,024 kips	288 kips
14	14	13.0 ft	14,400 ft^2	---	70 lb/ft^2	---	1,008.0 kips	---	4,032 kips	288 kips
13	13	13.0 ft	14,400 ft^2	---	70 lb/ft^2	---	1,008.0 kips	---	5,040 kips	288 kips
12	12	13.0 ft	14,400 ft^2	---	70 lb/ft^2	---	1,008.0 kips	---	6,048 kips	288 kips
11	11	13.0 ft	14,400 ft^2	---	70 lb/ft^2	---	1,008.0 kips	---	7,056 kips	288 kips
10	10	13.0 ft	14,400 ft^2	---	70 lb/ft^2	---	1,008.0 kips	---	8,064 kips	288 kips
9	9	13.0 ft	14,400 ft^2	---	70 lb/ft^2	---	1,008.0 kips	---	9,072 kips	288 kips
8	8	13.0 ft	14,400 ft^2	---	70 lb/ft^2	---	1,008.0 kips	---	10,080 kips	288 kips
7	7	13.0 ft	14,400 ft^2	---	70 lb/ft^2	---	1,008.0 kips	---	11,088 kips	288 kips
6	6	13.0 ft	14,400 ft^2	---	70 lb/ft^2	---	1,008.0 kips	---	12,096 kips	288 kips
5	5	13.0 ft	14,400 ft^2	---	70 lb/ft^2	---	1,008.0 kips	---	13,104 kips	288 kips
4	4	13.0 ft	14,400 ft^2	---	70 lb/ft^2	---	1,008.0 kips	---	14,112 kips	288 kips
3	3	13.0 ft	14,400 ft^2	---	70 lb/ft^2	---	1,008.0 kips	---	15,120 kips	288 kips
2	2	13.0 ft	14,400 ft^2	---	70 lb/ft^2	---	1,008.0 kips	---	16,128 kips	288 kips
1	GROUND	13.0 ft	N.A.	---	N.A.	---	N.A.	---	17,136 kips	288 kips
								Building Total Live Load and Roof Live Load =	17,136 kips	288 kips

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SUBJECT

2-D MOMENT FRAME [MF A3 - G3] ANALYSIS LOAD SUMMARY

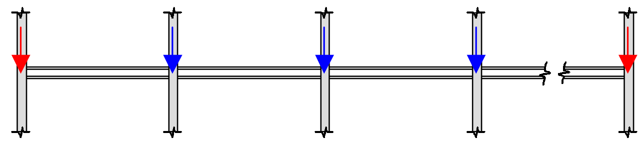
[5] MOMENT FRAME DEAD LOAD

o SUMMARY:

NUMBER OF STORIES, N_S	18 STORIES
NUMBER OF BAYS, N_B	6 BAYS
DIRECTION THAT THE *MOMENT FRAME* RUNS PARALLEL WITH:	E-W
LOCATION OF THE MOMENT FRAME WRT THE BUILDING PERIMETER:	INTERIOR
DOES THIS FRAME SUPPORT PART OF THE PENTHOUSE GRAVITY LOADS?	YES
DISTANCE TO THE CLOSEST (GRAVITY/MOMENT) FRAME:	20.0 ft
DISTANCE TO THE CLOSEST (GRAVITY/MOMENT) FRAME ON THE OTHER SIDE:	20.0 ft

o THEREFORE: DEAD LOAD TRIBUTARY WIDTH TO THIS MOMENT FRAME: 20.0 ft

NOTE: *PARAPET DEAD LOAD (PER ft² OF ROOF SURFACE AREA)* *2.92 lb/ft²*
PARAPET DEAD LOAD (PER FOOT OF PARAPET LENGTH) *87.5 lb/ft*



COLUMN (STACK) NO.	1	2	3	4	5	6	7				
PENTHOUSE LOADS TO THE COLUMNS	*0.0 kips*	*0.0 kips*	*7.3 kips*	*14.6 kips*	*7.3 kips*	*0.0 kips*	*0.0 kips*				
FLOOR NUMBER	**UNFACTORED (NOMINAL) DEAD LOAD BEAM END REACTIONS TO EACH COLUMN OF THE MOMENT FRAME AT EVERY FLOOR/LEVEL**										
ROOF	10.4 kips	17.2 kips	24.5 kips	31.8 kips	24.5 kips	17.2 kips	10.4 kips				
18	9.1 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips	9.1 kips				
17	9.1 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips	9.1 kips				
16	9.1 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips	9.1 kips				
15	9.1 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips	9.1 kips				
14	9.1 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips	9.1 kips				
13	9.1 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips	9.1 kips				
12	9.1 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips	9.1 kips				
11	9.1 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips	9.1 kips				
10	9.1 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips	9.1 kips				
9	9.1 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips	9.1 kips				
8	9.1 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips	9.1 kips				
7	9.1 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips	9.1 kips				
6	9.1 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips	9.1 kips				
5	9.1 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips	9.1 kips				
4	9.1 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips	9.1 kips				
3	9.1 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips	9.1 kips				
2	9.1 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips	9.1 kips				
GROUND	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.				

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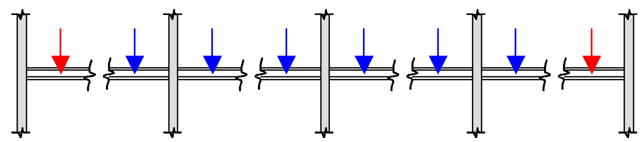
SUBJECT

2-D MOMENT FRAME [MF A3 - G3] ANALYSIS LOAD SUMMARY

[6] MOMENT FRAME DEAD LOAD (CONTINUED)

o DEAD LOAD TRIBUTARY WIDTH TO THIS MOMENT FRAME: 20.0 ft



FLOOR/LEVEL

BAY NUMBER	1_N-S	2_N-S	3_N-S	4_N-S	5_N-S	6_N-S				
BAY WIDTH	*20.0 ft*	*20.0 ft*	*20.0 ft*	*20.0 ft*	*20.0 ft*	*20.0 ft*				
NO. OF BEAMS PER BAY	1	1	1	1	1	1				
BEAM SPACING (*BETWEEN BEAMS*)	*10.0 ft*	*10.0 ft*	*10.0 ft*	*10.0 ft*	*10.0 ft*	*10.0 ft*				
PENTHOUSE LOADS TO THE ROOF GIRDERS	*0.0 kips*	*0.0 kips*	*14.6 kips*	*14.6 kips*	*0.0 kips*	*0.0 kips*				
FLOOR NUMBER	**UNFACTORED (NOMINAL) DEAD LOAD BEAM END REACTIONS TO EACH GIRDER OF THE MOMENT FRAME AT EVERY FLOOR/LEVEL**									
ROOF	17.2 kips	17.2 kips	31.8 kips	31.8 kips	17.2 kips	17.2 kips				
18	18.2 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips				
17	18.2 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips				
16	18.2 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips				
15	18.2 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips				
14	18.2 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips				
13	18.2 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips				
12	18.2 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips				
11	18.2 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips				
10	18.2 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips				
9	18.2 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips				
8	18.2 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips				
7	18.2 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips				
6	18.2 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips				
5	18.2 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips				
4	18.2 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips				
3	18.2 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips				
2	18.2 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips				
GROUND	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.				

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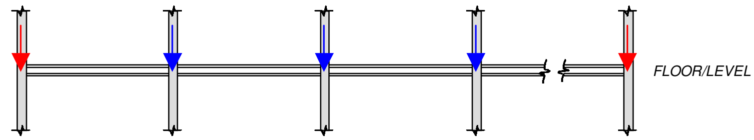
SUBJECT

2-D MOMENT FRAME [MF A3 - G3] ANALYSIS LOAD SUMMARY

[7] MOMENT FRAME LIVE LOAD

o SUMMARY:	NUMBER OF STORIES, N_S	18 STORIES
	NUMBER OF BAYS, N_B	6 BAYS
	DIRECTION THAT THE *MOMENT FRAME* RUNS PARALLEL WITH:	E-W
	LOCATION OF THE MOMENT FRAME WRT THE BUILDING PERIMETER:	INTERIOR
	DOES THIS FRAME SUPPORT PART OF THE PENTHOUSE GRAVITY LOADS?	YES
	DISTANCE TO THE CLOSEST (GRAVITY/MOMENT) FRAME:	20.0 ft
	DISTANCE TO THE CLOSEST (GRAVITY/MOMENT) FRAME ON THE OTHER SIDE:	20.0 ft
o THEREFORE:	LIVE LOAD TRIBUTARY WIDTH TO THIS MOMENT FRAME:	20.0 ft



COLUMN (STACK) NO.	1	2	3	4	5	6	7				
PENTHOUSE LOADS TO THE COLUMNS	*0.0 kips*	*0.0 kips*	*1.0 kips*	*2.0 kips*	*1.0 kips*	*0.0 kips*	*0.0 kips*				
FLOOR NUMBER	**UNFACTORED (NOMINAL) LIVE LOAD BEAM END REACTIONS TO EACH COLUMN OF THE MOMENT FRAME AT EVERY FLOOR/LEVEL**										
ROOF	2.0 kips	4.0 kips	5.0 kips	6.0 kips	5.0 kips	4.0 kips	2.0 kips				
18	7.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips	7.0 kips				
17	7.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips	7.0 kips				
16	7.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips	7.0 kips				
15	7.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips	7.0 kips				
14	7.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips	7.0 kips				
13	7.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips	7.0 kips				
12	7.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips	7.0 kips				
11	7.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips	7.0 kips				
10	7.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips	7.0 kips				
9	7.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips	7.0 kips				
8	7.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips	7.0 kips				
7	7.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips	7.0 kips				
6	7.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips	7.0 kips				
5	7.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips	7.0 kips				
4	7.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips	7.0 kips				
3	7.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips	7.0 kips				
2	7.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips	7.0 kips				
GROUND	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.				

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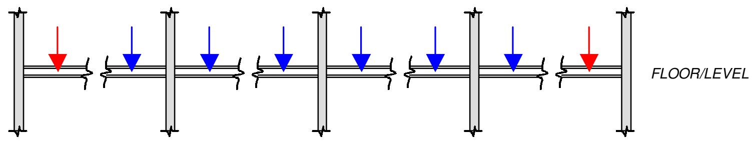
SUBJECT

2-D MOMENT FRAME [MF A3 - G3] ANALYSIS LOAD SUMMARY

[8] MOMENT FRAME LIVE LOAD (CONTINUED)

o LIVE LOAD TRIBUTARY WIDTH TO THIS MOMENT FRAME: 20.0 ft



BAY NUMBER	**1_N-S**	**2_N-S**	**3_N-S**	**4_N-S**	**5_N-S**	**6_N-S**				
BAY WIDTH	*20.0 ft*	*20.0 ft*	*20.0 ft*	*20.0 ft*	*20.0 ft*	*20.0 ft*				
NO. OF BEAMS PER BAY	1	1	1	1	1	1				
BEAM SPACING (*BETWEEN BEAMS*)	*10.0 ft*	*10.0 ft*	*10.0 ft*	*10.0 ft*	*10.0 ft*	*10.0 ft*				
PENTHOUSE LOADS TO THE ROOF GIRDERS	*0.0 kips*	*0.0 kips*	*2.0 kips*	*2.0 kips*	*0.0 kips*	*0.0 kips*				
FLOOR NUMBER	**UNFACTORED (NOMINAL) LIVE LOAD BEAM END REACTIONS TO EACH GIRDER OF THE MOMENT FRAME AT EVERY FLOOR/LEVEL**									
ROOF	4.0 kips	4.0 kips	6.0 kips	6.0 kips	4.0 kips	4.0 kips				
18	14.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips				
17	14.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips				
16	14.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips				
15	14.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips				
14	14.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips				
13	14.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips				
12	14.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips				
11	14.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips				
10	14.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips				
9	14.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips				
8	14.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips				
7	14.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips				
6	14.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips				
5	14.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips				
4	14.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips				
3	14.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips				
2	14.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips				
GROUND	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.				

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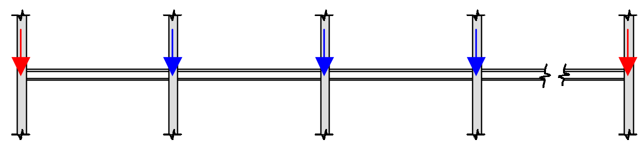
SUBJECT

2-D MOMENT FRAME [MF A3 - G3] ANALYSIS LOAD SUMMARY

[9] MOMENT FRAME SEISMIC WEIGHT (DEAD LOAD + PARTITION LIVE LOAD)

o SUMMARY:	NUMBER OF STORIES, N_S	18 STORIES
	NUMBER OF BAYS, N_B	6 BAYS
	DIRECTION THAT THE *MOMENT FRAME* RUNS PARALLEL WITH:	E-W
	LOCATION OF THE MOMENT FRAME WRT THE BUILDING PERIMETER:	INTERIOR
	DOES THIS FRAME SUPPORT PART OF THE PENTHOUSE GRAVITY LOADS?	YES
	DISTANCE TO THE CLOSEST (GRAVITY/MOMENT) FRAME:	20.0 ft
	DISTANCE TO THE CLOSEST (GRAVITY/MOMENT) FRAME ON THE OTHER SIDE:	20.0 ft
o THEREFORE:	SEISMIC WEIGHT TRIBUTARY WIDTH TO THIS MOMENT FRAME:	20.0 ft

NOTE:	*PARAPET SEISMIC WEIGHT (PER ft² OF ROOF SURFACE AREA)*	*2.92 lb/ft²*
	PARAPET SEISMIC WEIGHT (PER FOOT OF PARAPET LENGTH)	*87.5 lb/ft*
	PARTITION LIVE LOAD	*20 lb/ft²*



COLUMN (STACK) NO.	**1**	**2**	**3**	**4**	**5**	**6**	**7**				
PENTHOUSE LOADS TO THE COLUMNS	*0.0 kips*	*0.0 kips*	*7.3 kips*	*14.6 kips*	*7.3 kips*	*0.0 kips*	*0.0 kips*				
FLOOR NUMBER	**UNFACTORED (NOMINAL) SEISMIC WEIGHT BEAM END REACTIONS TO EACH COLUMN OF THE MOMENT FRAME AT EVERY FLOOR/LEVEL**										
ROOF	10.4 kips	17.2 kips	24.5 kips	31.8 kips	24.5 kips	17.2 kips	10.4 kips				
18	11.1 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips	11.1 kips				
17	11.1 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips	11.1 kips				
16	11.1 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips	11.1 kips				
15	11.1 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips	11.1 kips				
14	11.1 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips	11.1 kips				
13	11.1 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips	11.1 kips				
12	11.1 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips	11.1 kips				
11	11.1 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips	11.1 kips				
10	11.1 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips	11.1 kips				
9	11.1 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips	11.1 kips				
8	11.1 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips	11.1 kips				
7	11.1 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips	11.1 kips				
6	11.1 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips	11.1 kips				
5	11.1 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips	11.1 kips				
4	11.1 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips	11.1 kips				
3	11.1 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips	11.1 kips				
2	11.1 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips	11.1 kips				
GROUND	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.				

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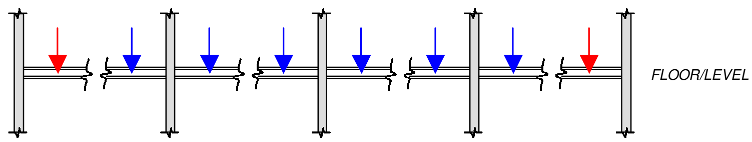
2-D MOMENT FRAME [MF A3 - G3] ANALYSIS LOAD SUMMARY

[10] MOMENT FRAME SEISMIC WEIGHT (CONTINUED)

o SEISMIC WEIGHT TRIBUTARY WIDTH TO THIS MOMENT FRAME: 20.0 ft



BAY NUMBER	**1_N-S**	**2_N-S**	**3_N-S**	**4_N-S**	**5_N-S**	**6_N-S**				
BAY WIDTH	*20.0 ft*	*20.0 ft*	*20.0 ft*	*20.0 ft*	*20.0 ft*	*20.0 ft*				
NO. OF BEAMS PER BAY	**1**	**1**	**1**	**1**	**1**	**1**				
BEAM SPACING (*BETWEEN BEAMS*)	*10.0 ft*	*10.0 ft*	*10.0 ft*	*10.0 ft*	*10.0 ft*	*10.0 ft*				
PENTHOUSE LOADS TO THE ROOF GIRDERS	*0.0 kips*	*0.0 kips*	*14.6 kips*	*14.6 kips*	*0.0 kips*	*0.0 kips*				
FLOOR NUMBER	**UNFACTORED (NOMINAL) SEISMIC WEIGHT BEAM END REACTIONS TO EACH GIRDER OF THE MOMENT FRAME AT EVERY FLOOR/LEVEL**									
ROOF	17.2 kips	17.2 kips	31.8 kips	31.8 kips	17.2 kips	17.2 kips				
18	22.2 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips				
17	22.2 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips				
16	22.2 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips				
15	22.2 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips				
14	22.2 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips				
13	22.2 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips				
12	22.2 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips				
11	22.2 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips				
10	22.2 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips				
9	22.2 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips				
8	22.2 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips				
7	22.2 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips				
6	22.2 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips				
5	22.2 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips				
4	22.2 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips				
3	22.2 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips				
2	22.2 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips				
GROUND	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.				

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2-D MOMENT FRAME [MF A3 - G3] ANALYSIS LOAD SUMMARY

[11] SEISMIC LOADS PER THE INTERNATIONAL BUILDING CODE (IBC) 2003 [ASCE 7]

o *PER THE EXCEPTION OF SECTION 1614 OF THE IBC 2003, ASCE 7 IS PERMITTED TO BE USED TO DETERMINE THE SEISMIC LOADS. THEREFORE, THE FOLLOWING DESIGN PARAMETERS ARE TAKEN FROM ASCE 7-02.*		Section 1614	(p. 302)
o OCCUPANCY CATEGORY	II	Table 1-1	(p. 4)
o SEISMIC USE GROUP	I	Section 9.1.3 Table 9.1.3	(p. 96) (p. 96)
o OCCUPANCY IMPORTANCE FACTOR, I_E	1.00	Section 9.1.4 Table 9.1.4	(p. 96) (p. 97)
o THE MAPPED SPECTRAL ACCELERATIONS:	FOR SHORT PERIODS, S_S = 1.5 g FOR A 1-SECOND PERIOD, S_1 = 0.6 g	Section 9.4.1.2 Figure 9.4.1.1(c) & (d)	(p. 107)
o SITE CLASS	D	Section 9.4.1.2.1 *Note the "Exception"*	(p. 108)
o SITE COEFFICIENTS:	F_a = 1.0 F_v = 1.5	Table 9.4.1.2.4a Table 9.4.1.2.4b	(p. 129) (p. 130)
o MAX. CONSIDERED SPECTRAL RESPONSE ACCELERATIONS:	FOR SHORT PERIODS, S_{MS} = 1.5 g FOR A 1-SECOND PERIOD, S_{M1} = 0.9 g	Equation 9.4.1.2.4-1 Equation 9.4.1.2.4-2	(p. 129) (p. 129)
o DESIGN SPECTRAL RESPONSE ACCELERATIONS:	FOR SHORT PERIODS, S_{DS} = 1.0 g FOR A 1-SECOND PERIOD, S_{D1} = 0.6 g	Equation 9.4.1.2.5-1 Equation 9.4.1.2.5-2	(p. 129) (p. 129)
o SEISMIC DESIGN CATEGORY	D	Table 9.4.2.1a Table 9.4.2.1b	(p. 131) (p. 132)
o BASIC SEISMIC-FORCE RESISTING SYSTEM	*"SPECIAL COMPOSITE MOMENT FRAMES"*	Table 9.5.2.2	(p. 134)
RESPONSE MODIFICATION COEFFICIENT, R	8	Table 9.5.2.2	(p. 134)
DEFLECTION AMPLIFICATION FACTOR, C_d	5.5	Table 9.5.2.2	(p. 134)
o FUNDAMENTAL PERIOD, T		Section 9.5.5.3	(p. 147)
BUILDING PERIOD COEFFICIENTS:	C_T = 0.028 x = 0.8	Table 9.5.5.3.2 Table 9.5.5.3.2	(p. 147) (p. 147)
ELEVATION OF THE BUILDING ROOF ABOVE THE BASE, h_n	234 ft		
COEFFICIENT FOR UPPER LIMIT ON CALCULATED PERIOD, C_U	1.4	Table 9.5.5.3.1	(p. 147)
APPROXIMATE FUNDAMENTAL PERIOD, T_a	2.201 sec	Equation 9.5.5.3.2-1	(p. 147)
PERIOD FROM RATIONAL ANALYSIS, T_R	*NONE CALCULATED*		
MAXIMUM ALLOWED PERIOD, $C_U T_a$	3.081 sec	Section 9.5.5.3	(p. 147)
DESIGN PERIOD, T	2.201 sec	Section 9.5.5.3	(p. 147)
o ANALYSIS PROCEDURE	*EQUIVALENT LATERAL FORCE ANALYSIS*	Section 9.5.2.5.1 Table 9.5.2.5.1	(p. 139) (p. 140)
o SEISMIC RESPONSE COEFFICIENT, C_S	C_S = 0.125 g	Equation 9.5.5.2.1-1	(p. 146)
	$C_S \leq$ 0.035 g	Equation 9.5.5.2.1-2	(p. 146)
	$C_S \geq$ 0.044 g	Equation 9.5.5.2.1-3	(p. 146)
	DESIGN C_S = 0.044 g		
o SEISMIC (DESIGN) BASE SHEAR, V	***V = 0.044 W***	Equation 9.5.5.2-1	(p. 146)

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2-D MOMENT FRAME [MF A3 - G3] ANALYSIS LOAD SUMMARY

[12] VERTICAL DISTRIBUTION OF THE SEISMIC LOADS PER ASCE 7-02 [IBC 2003]

Section 9.5.5.4 (p. 148)
Equation 9.5.5.4-1 (p. 148)
Equation 9.5.5.4-2 (p. 148)

- o BUILDING FUNDAMENTAL (DESIGN) PERIOD, T — 2.201 sec
- o DISTRIBUTION EXPONENT, k — 1.85
- o DESIGN BASE SHEAR, V — 0.044 W
- o EFFECTIVE SEISMIC WEIGHT, W — 28,722.4 kips
- o DESIGN SHEAR AT THE BASE OF THE BUILDING, V — 1,263.8 kips

NUMBER		STORY HEIGHT	FLOOR ELEVATION, h_x	DEAD LOAD		(MOVEABLE PARTITION) L.L.		(SEISMIC) WEIGHT w_x	SEISMIC WT. TOTAL PER FLOOR	$w_x h_x^k$ (kip-ft)	C_{vx}	F_x
STORY	FLOOR			ROOF / FLR	PENTHOUSE	ROOF / FLR	PENTHOUSE					
---	---	---	---	---	---	---	---	---	---	---	---	---
---	ROOF	---	234.0 ft	1,358.45 kips	232.8 kips	0.0 kips	0.0 kips	1,591.25 kips	---	3.844E+07	0.1462	184.8 kips
18	18	13.0 ft	221.0 ft	1,307.95 kips	---	288.0 kips	---	1,595.95 kips	1,591.3 kips	3.469E+07	0.1319	166.7 kips
17	17	13.0 ft	208.0 ft	1,307.95 kips	---	288.0 kips	---	1,595.95 kips	3,187.3 kips	3.101E+07	0.1179	149.0 kips
16	16	13.0 ft	195.0 ft	1,307.95 kips	---	288.0 kips	---	1,595.95 kips	4,783.3 kips	2.752E+07	0.1046	132.2 kips
15	15	13.0 ft	182.0 ft	1,307.95 kips	---	288.0 kips	---	1,595.95 kips	6,379.3 kips	2.422E+07	0.0921	116.4 kips
14	14	13.0 ft	169.0 ft	1,307.95 kips	---	288.0 kips	---	1,595.95 kips	7,975.3 kips	2.112E+07	0.0803	101.5 kips
13	13	13.0 ft	156.0 ft	1,307.95 kips	---	288.0 kips	---	1,595.95 kips	9,571.3 kips	1.821E+07	0.0692	87.5 kips
12	12	13.0 ft	143.0 ft	1,307.95 kips	---	288.0 kips	---	1,595.95 kips	11,167.3 kips	1.550E+07	0.0589	74.4 kips
11	11	13.0 ft	130.0 ft	1,307.95 kips	---	288.0 kips	---	1,595.95 kips	12,763.3 kips	1.300E+07	0.0494	62.4 kips
10	10	13.0 ft	117.0 ft	1,307.95 kips	---	288.0 kips	---	1,595.95 kips	14,359.3 kips	1.069E+07	0.0406	51.3 kips
9	9	13.0 ft	104.0 ft	1,307.95 kips	---	288.0 kips	---	1,595.95 kips	15,955.3 kips	8.601E+06	0.0327	41.3 kips
8	8	13.0 ft	91.0 ft	1,307.95 kips	---	288.0 kips	---	1,595.95 kips	17,551.3 kips	6.718E+06	0.0255	32.2 kips
7	7	13.0 ft	78.0 ft	1,307.95 kips	---	288.0 kips	---	1,595.95 kips	19,147.3 kips	5.051E+06	0.0192	24.3 kips
6	6	13.0 ft	65.0 ft	1,307.95 kips	---	288.0 kips	---	1,595.95 kips	20,743.3 kips	3.605E+06	0.0137	17.3 kips
5	5	13.0 ft	52.0 ft	1,307.95 kips	---	288.0 kips	---	1,595.95 kips	22,339.3 kips	2.386E+06	0.0091	11.5 kips
4	4	13.0 ft	39.0 ft	1,307.95 kips	---	288.0 kips	---	1,595.95 kips	23,935.3 kips	1.401E+06	0.0053	6.7 kips
3	3	13.0 ft	26.0 ft	1,307.95 kips	---	288.0 kips	---	1,595.95 kips	25,531.3 kips	6.618E+05	0.0025	3.2 kips
2	2	13.0 ft	13.0 ft	1,307.95 kips	---	288.0 kips	---	1,595.95 kips	27,127.3 kips	1.836E+05	0.0007	0.9 kips
1	GROUND	13.0 ft	0.0 ft	N.A.	---	N.A.	---	0.0 kips	28,723.3 kips	0.000E+00	0.0000	0.0 kips
				23,826.4 kips		4,896.0 kips		28,722.4 kips		2.630E+08		1,263.6 kips

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2-D MOMENT FRAME [MF A3 - G3] ANALYSIS LOAD SUMMARY

[13] REDUNDANCY COEFFICIENT, ρ, PER ASCE 7-02 [IBC 2003] Section 9.5.2.4 (p. 138)

- o SEISMIC DESIGN CATEGORY D
- o ARE THERE ONLY *SPECIAL* MOMENT FRAMES? YES
- o REDUNDANCY COEFFICIENT, ρ $r_i = 2 - \dfrac{20}{r_{max_i}\sqrt{A_i}}$ Equation 9.5.2.4.2-1 (p. 138)

$\rho_{max} =$ -0.20 *(FROM THE TABLE BELOW)*

- **o (DESIGN) REDUNDANCY COEFFICIENT, ρ** $\rho =$ **1.00**

STORY NUMBER	AREA OF THE DIAPHRAGM	SEISMIC SHEAR IN EACH COLUMN OF THE MOMENT FRAME FOR EVERY STORY OF THE BUILDING											DESIGN STORY SHEAR	r_{max_i}	ρ_i
		COL 1	COL 2	COL 3	COL 4	COL 5	COL 6	COL 7							
18	14,400 ft²	1 kips	7 kips	10 kips	10 kips	10 kips	7 kips	1 kips					184.8 kips	0.0758	-0.1988
17	14,400 ft²	6 kips	14 kips	16 kips	16 kips	16 kips	14 kips	6 kips					351.5 kips	0.0637	-0.6164
16	14,400 ft²	10 kips	20 kips	22 kips	22 kips	22 kips	20 kips	10 kips					500.5 kips	0.0615	-0.7100
15	14,400 ft²	12 kips	26 kips	28 kips	28 kips	28 kips	26 kips	12 kips					632.7 kips	0.0620	-0.6882
14	14,400 ft²	15 kips	30 kips	33 kips	33 kips	33 kips	30 kips	15 kips					749.1 kips	0.0617	-0.7012
13	14,400 ft²	15 kips	35 kips	38 kips	38 kips	38 kips	35 kips	15 kips					850.6 kips	0.0625	-0.6667
12	14,400 ft²	18 kips	38 kips	41 kips	41 kips	41 kips	38 kips	18 kips					938.1 kips	0.0612	-0.7233
11	14,400 ft²	19 kips	42 kips	44 kips	44 kips	44 kips	42 kips	19 kips					1,012.5 kips	0.0608	-0.7412
10	14,400 ft²	20 kips	44 kips	47 kips	47 kips	47 kips	44 kips	20 kips					1,074.9 kips	0.0612	-0.7233
9	14,400 ft²	26 kips	45 kips	47 kips	47 kips	47 kips	45 kips	26 kips					1,126.2 kips	0.0584	-0.8539
8	14,400 ft²	25 kips	48 kips	49 kips	49 kips	49 kips	48 kips	25 kips					1,167.5 kips	0.0588	-0.8345
7	14,400 ft²	26 kips	49 kips	50 kips	50 kips	50 kips	49 kips	26 kips					1,199.7 kips	0.0583	-0.8588
6	14,400 ft²	25 kips	51 kips	52 kips	52 kips	52 kips	51 kips	25 kips					1,224.0 kips	0.0595	-0.8011
5	14,400 ft²	26 kips	51 kips	52 kips	52 kips	52 kips	51 kips	26 kips					1,241.3 kips	0.0586	-0.8441
4	14,400 ft²	25 kips	52 kips	53 kips	53 kips	53 kips	52 kips	25 kips					1,252.8 kips	0.0592	-0.8153
3	14,400 ft²	27 kips	52 kips	52 kips	52 kips	52 kips	52 kips	27 kips					1,259.5 kips	0.0578	-0.8835
2	14,400 ft²	30 kips	51 kips	51 kips	51 kips	51 kips	51 kips	30 kips					1,262.7 kips	0.0565	-0.9499
1	14,400 ft²	36 kips	48 kips	49 kips	49 kips	49 kips	48 kips	36 kips					1,263.6 kips	0.0551	-1.0248

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2-D MOMENT FRAME [MF A3 - G3] ANALYSIS LOAD SUMMARY

[14] WIND LOADS PER THE INTERNATIONAL BUILDING CODE (IBC) 2003 [ASCE 7]

o *PER SECTION 1609.1.1 OF THE IBC 2003, SECTION 6 OF ASCE 7 SHALL BE USED TO DETERMINE THE WIND LOADS. THEREFORE, THE FOLLOWING DESIGN PARAMETERS ARE TAKEN FROM ASCE 7-02.* — Section 1609 (p. 283)

o OCCUPANCY CATEGORY	II		Table 1-1	(p. 4)
o IMPORTANCE FACTOR, I	1.00		Table 6-1	(p. 73)
o (3-SECOND GUST) BASIC WIND SPEED, V	85 mph		Figure 6-1	(p. 36)
o EXPOSURE CATEGORY	B		Section 6.5.6.3	(p. 28)
o WIND DIRECTIONALITY FACTOR, K_d, FOR MWFRS OF A BUILDING	0.85		Table 6-4	(p. 76)
o TOPOGRAPHIC FACTOR, K_{zt}	1.0		Section 6.5.7.2 Equation 6-3	(p. 30) (p. 30)
o ENCLOSURE CLASSIFICATION		*ENCLOSED* - SINCE NOT IN A HURICANE REGION AND THERE IS A SMALL CHANCE THAT WIND BORNE DEBRIS WILL PENETRATE THE WINDOWS AND CLADDING.	Section 6.5.9 Section 6.2	(p. 30) (p. 23)
o BUILDING TYPE		*SIMPLE DIAPHRAGM* - WIND LOADS ARE TRANSFERRED THROUGH THE ROOF AND FLOOR DIAPHRAGMS TO THE MWFRS (MOMENT FRAMES).	Section 6.2	(p. 24)
o APPROXIMATE BUILDING (MAX ALLOWED) PERIOD APPROXIMATE BUILDING FREQUENCY, n_1	3.081 sec 0.325 Hz	*(FROM SEISMIC CALCULATIONS)* *SINCE n1 < 1.0 THE BUILDING IS FLEXIBLE*	Section 6.2	(p. 24)
o DIRECTION THAT THE MOMENT FRAME RUNS PARALLEL WITH:	E-W			
o BUILDING WIDTH (DIMENSION PERPENDICULAR TO WIND DIRECTION),	120.0 ft	*ALONG THE E-W FACE*		
o BUILDING DEPTH (DIMENSION PARALLEL TO WIND DIRECTION), L	120.0 ft	*ALONG THE N-S FACE*		
o MEAN ROOF HEIGHT ABOVE GRADE, h	234.0 ft			
o VELOCITY PRESSURES	qz = 15.72 Kz lb/ft² qh = 19.81 lb/ft²		Section 6.5.10 Equation 6-15	(p. 31) (p. 31)
o GUST EFFECT FACTOR, G_f			Section 6.5.8.2	(p. 30)

From Table 6-2

c = 0.30	z_{min} = 30 ft
$\bar{\alpha}$ = 1/4	$\bar{\varepsilon}$ = 1/3
$\bar{b}$ = 0.45	ℓ = 320 ft
α = 7.0	z_g = 1200 ft

$g_Q = g_V =$ 3.4	Section 6.5.8.2
$g_R =$ 3.912	Equation 6-9
$\bar{z} =$ 140.4 ft	Section 6.5.8.1
$I_{\bar{z}} =$ 0.236	Equation 6-5
$\bar{V}_{\bar{z}} =$ 80.57 ft/sec	Equation 6-14
$L_{\bar{z}} =$ 518.52	Equation 6-7

Equation 6-13

η_h = 4.342	R_h = 0.204
η_B = 2.227	R_B = 0.349
η_L = 2.227	R_L = 0.349

$N_1 =$ 2.092	Equation 6-12
$R_n =$ 0.087	Equation 6-11
Assumed Critical Damping Ratio, β = 0.05	
Resonant Response Factor, R = 0.293	Equation 6-10
Background Response, Q = 0.818	Equation 6-6
Gust Effect Factor, G_f = 0.863	Equation 6-8

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2-D MOMENT FRAME [MF A3 - G3] ANALYSIS LOAD SUMMARY

[15] WIND LOADS *CONTINUED*

o *WALL* EXTERNAL PRESSURE COEFFICIENTS, C_p

Plus signs signify pressures acting towards the surface.
Negative signs signify pressures acting away from the surface.

SURFACE	L / B	C_p
WINDWARD WALL	ALL	0.8
SIDE WALLS	1.00	-0.5
LEEWARD WALL	ALL	-0.7

Section 6.5.11.2.1 (p. 31)
Figure 6-6 (p. 51)

o *ROOF* EXTERNAL PRESSURE COEFFICIENT, C_p

h = 234.0 ft
L = 120.0 ft
h / L = 1.950 (Assume h/L > 1.0)
h / 2 = 117.0 ft

FOR: θ < 10°	
DISTANCE FROM LEADING EDGE	C_p
0 to h / 2	-1.3
> h / 2	-0.7

Section 6.5.11.2.1 (p. 31)
Figure 6-6 (p. 51)

o INTERNAL PRESSURE COEFFICIENT, GC_{pi}

GC_{pi} = ± 0.18

Section 6.5.11.1 (p. 31)
Figure 6-5 (p. 49)

FLOOR LEVEL WIND LOADS PER EQUATION 6-19 AND PARAPET WIND LOADS PER EQUATION 6-20											*WINDWARD WALL*		*LEEWARD WALL*				
FLOOR NUMBER	ELEVATION	(BUILDING) WIND TRIBUTARY PARAMETERS					K_z	q_z (psf)	K_h	q_h (psf)	PRESSURE (psf) WITH		PRESSURE (psf) WITH		TOTAL WIND SHEAR WITH		DESIGN WIND SHEAR PER FLOOR
		TOP ELEV.	BOT. ELEV.	HEIGHT	WIDTH	AREA					($+GC_{pi}$)	($-GC_{pi}$)	($+GC_{pi}$)	($-GC_{pi}$)	($+GC_{pi}$)	($-GC_{pi}$)	
PENTHOUSE	---	247.0 ft	234.0 ft	13.0 ft	40.0 ft	520 ft²	1.28	20.12	1.26	19.81	10.33	17.46	-15.53	-10.0	13.4 kips	14.3 kips	14.3 kips
PARAPET	---	237.5 ft	234.0 ft	3.5 ft	120.0 ft	420 ft²	1.27	19.97	1.26	19.81	35.95		-21.97		24.3 kips	24.3 kips	24.3 kips
ROOF	234.0 ft	234.0 ft	227.5 ft	6.5 ft	120.0 ft	780 ft²	1.26	19.81	1.26	19.81	10.11	17.24	-15.53	-10.0	20.0 kips	21.2 kips	21.2 kips
18	221.0 ft	227.5 ft	214.5 ft	13.0 ft	120.0 ft	1,560 ft²	1.25	19.65	1.26	19.81	10.0	17.13	-15.53	-10.0	39.8 kips	42.3 kips	42.3 kips
17	208.0 ft	214.5 ft	201.5 ft	13.0 ft	120.0 ft	1,560 ft²	1.23	19.34	1.26	19.81	10.0	16.92	-15.53	-10.0	39.8 kips	42.0 kips	42.0 kips
16	195.0 ft	201.5 ft	188.5 ft	13.0 ft	120.0 ft	1,560 ft²	1.21	19.02	1.26	19.81	10.0	16.7	-15.53	-10.0	39.8 kips	41.7 kips	41.7 kips
15	182.0 ft	188.5 ft	175.5 ft	13.0 ft	120.0 ft	1,560 ft²	1.18	18.55	1.26	19.81	10.0	16.37	-15.53	-10.0	39.8 kips	41.1 kips	41.1 kips
14	169.0 ft	175.5 ft	162.5 ft	13.0 ft	120.0 ft	1,560 ft²	1.16	18.24	1.26	19.81	10.0	16.16	-15.53	-10.0	39.8 kips	40.8 kips	40.8 kips
13	156.0 ft	162.5 ft	149.5 ft	13.0 ft	120.0 ft	1,560 ft²	1.14	17.92	1.26	19.81	10.0	15.94	-15.53	-10.0	39.8 kips	40.5 kips	40.5 kips
12	143.0 ft	149.5 ft	136.5 ft	13.0 ft	120.0 ft	1,560 ft²	1.11	17.45	1.26	19.81	10.0	15.61	-15.53	-10.0	39.8 kips	40.0 kips	40.0 kips
11	130.0 ft	136.5 ft	123.5 ft	13.0 ft	120.0 ft	1,560 ft²	1.08	16.98	1.26	19.81	10.0	15.29	-15.53	-10.0	39.8 kips	39.5 kips	39.5 kips
10	117.0 ft	123.5 ft	110.5 ft	13.0 ft	120.0 ft	1,560 ft²	1.05	16.51	1.26	19.81	10.0	14.96	-15.53	-10.0	39.8 kips	38.9 kips	38.9 kips
9	104.0 ft	110.5 ft	97.5 ft	13.0 ft	120.0 ft	1,560 ft²	1.02	16.04	1.26	19.81	10.0	14.64	-15.53	-10.0	39.8 kips	38.4 kips	38.4 kips
8	91.0 ft	97.5 ft	84.5 ft	13.0 ft	120.0 ft	1,560 ft²	0.98	15.41	1.26	19.81	10.0	14.2	-15.53	-10.0	39.8 kips	37.8 kips	37.8 kips
7	78.0 ft	84.5 ft	71.5 ft	13.0 ft	120.0 ft	1,560 ft²	0.94	14.78	1.26	19.81	10.0	13.77	-15.53	-10.0	39.8 kips	37.1 kips	37.1 kips
6	65.0 ft	71.5 ft	58.5 ft	13.0 ft	120.0 ft	1,560 ft²	0.90	14.15	1.26	19.81	10.0	13.33	-15.53	-10.0	39.8 kips	36.4 kips	36.4 kips
5	52.0 ft	58.5 ft	45.5 ft	13.0 ft	120.0 ft	1,560 ft²	0.85	13.36	1.26	19.81	10.0	12.79	-15.53	-10.0	39.8 kips	35.6 kips	35.6 kips
4	39.0 ft	45.5 ft	32.5 ft	13.0 ft	120.0 ft	1,560 ft²	0.79	12.42	1.26	19.81	10.0	12.14	-15.53	-10.0	39.8 kips	34.5 kips	34.5 kips
3	26.0 ft	32.5 ft	19.5 ft	13.0 ft	120.0 ft	1,560 ft²	0.72	11.32	1.26	19.81	10.0	11.38	-15.53	-10.0	39.8 kips	33.4 kips	33.4 kips
2	13.0 ft	19.5 ft	6.5 ft	13.0 ft	120.0 ft	1,560 ft²	0.62	9.75	1.26	19.81	10.0	10.3	-15.53	-10.0	39.8 kips	31.7 kips	31.7 kips
GROUND	0.0 ft	6.5 ft	0.0 ft	6.5 ft	120.0 ft	780 ft²	0.57	8.96	1.26	19.81	10.0	10.0	-15.53	-10.0	19.9 kips	15.6 kips	N.A.
															754 kips	727 kips	

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2-D MOMENT FRAME [MF A3 - G3] ANALYSIS LOAD SUMMARY

[16] WIND LOADS *CONTINUED*

- o BUIDLING WIDTH (DIMENSION PERPENDICULAR TO WIND DIRECTION), B 120.0 ft *ALONG THE E-W FACE*
- o ECCENTRICITY ALONG THE WINDWARD FACE OF THE BUILDING, e_x ± 18.0 ft Figure 6-9 (p. 54)
- o BUIDLING DEPTH (DIMENSION PARALLEL TO WIND DIRECTION), L 120.0 ft *ALONG THE N-S FACE*
- o ECCENTRICITY ALONG THE SIDEWALL OF THE BUILDING, e_y ± 18.0 ft Figure 6-9 (p. 54)
- o ECCENTRICITY FOR FLEXIBLE STRUCTURES (e_x AND e_y) ± 31.56 ft Equation 6-21 (p. 33)
- o *TO SIMPLIFY THE TORSIONAL MOMENT CALCULATIONS, THE BUILDING PLAN IS ASSUMED TO BE SQUARE SO THAT THE WIND SHEAR LOADS CAN BE CALCULATED ONCE ALONG ONE PRINCIPAL DIRECTION AND THEN USED IN BOTH PRINCIPAL DIRECTIONS.*
- o TORSION LOADS ARE SIMPLIFIED SO THAT THE MAXIMUM SHEAR PER MOMENT FRAME = *STORY SHEAR x (1 / NO. OF MOMENT FRAMES + 0.002 x ECCENTRICITY)*

FLOOR NUMBER	ELEVATION	ASCE 7 Figure 6-9 ***CASE #1*** SHEAR LOADS DUE TO: SHEAR	TORSION	TOTAL STORY SHEAR	ASCE 7 Figure 6-9 ***CASE #2*** SHEAR LOADS DUE TO: SHEAR	TORSION	TOTAL STORY SHEAR	ASCE 7 Figure 6-9 ***CASE #3*** SHEAR LOADS DUE TO: SHEAR	TORSION	TOTAL STORY SHEAR	ASCE 7 Figure 6-9 ***CASE #4*** SHEAR LOADS DUE TO: SHEAR	TORSION	TOTAL STORY SHEAR
PENTHOUSE	---	14.3 kips	± 0.0 kips	14.3 kips	10.7 kips	± 0.0 kips	10.7 kips	10.7 kips	± 0.0 kips	10.7 kips	8.1 kips	± 0.0 kips	8.1 kips
PARAPET	---	24.3 kips	± 0.0 kips	24.3 kips	18.2 kips	± 1.1 kips	18.2 kips	18.2 kips	± 0.0 kips	18.2 kips	13.7 kips	± 0.9 kips	13.7 kips
ROOF	234.0 ft	21.2 kips	± 0.0 kips	21.2 kips	15.9 kips	± 1.0 kips	15.9 kips	15.9 kips	± 0.0 kips	15.9 kips	11.9 kips	± 0.8 kips	11.9 kips
18	221.0 ft	42.3 kips	± 0.0 kips	42.3 kips	31.7 kips	± 2.0 kips	31.7 kips	31.7 kips	± 0.0 kips	31.7 kips	23.8 kips	± 1.5 kips	23.8 kips
17	208.0 ft	42.0 kips	± 0.0 kips	42.0 kips	31.5 kips	± 2.0 kips	31.5 kips	31.5 kips	± 0.0 kips	31.5 kips	23.6 kips	± 1.5 kips	23.6 kips
16	195.0 ft	41.7 kips	± 0.0 kips	41.7 kips	31.3 kips	± 2.0 kips	31.3 kips	31.3 kips	± 0.0 kips	31.3 kips	23.5 kips	± 1.5 kips	23.5 kips
15	182.0 ft	41.1 kips	± 0.0 kips	41.1 kips	30.8 kips	± 1.9 kips	30.8 kips	30.8 kips	± 0.0 kips	30.8 kips	23.1 kips	± 1.5 kips	23.1 kips
14	169.0 ft	40.8 kips	± 0.0 kips	40.8 kips	30.6 kips	± 1.9 kips	30.6 kips	30.6 kips	± 0.0 kips	30.6 kips	23.0 kips	± 1.5 kips	23.0 kips
13	156.0 ft	40.5 kips	± 0.0 kips	40.5 kips	30.4 kips	± 1.9 kips	30.4 kips	30.4 kips	± 0.0 kips	30.4 kips	22.8 kips	± 1.4 kips	22.8 kips
12	143.0 ft	40.0 kips	± 0.0 kips	40.0 kips	30.0 kips	± 1.9 kips	30.0 kips	30.0 kips	± 0.0 kips	30.0 kips	22.5 kips	± 1.4 kips	22.5 kips
11	130.0 ft	39.5 kips	± 0.0 kips	39.5 kips	29.6 kips	± 1.9 kips	29.6 kips	29.6 kips	± 0.0 kips	29.6 kips	22.2 kips	± 1.4 kips	22.2 kips
10	117.0 ft	38.9 kips	± 0.0 kips	38.9 kips	29.2 kips	± 1.8 kips	29.2 kips	29.2 kips	± 0.0 kips	29.2 kips	21.9 kips	± 1.4 kips	21.9 kips
9	104.0 ft	38.4 kips	± 0.0 kips	38.4 kips	28.8 kips	± 1.8 kips	28.8 kips	28.8 kips	± 0.0 kips	28.8 kips	21.6 kips	± 1.4 kips	21.6 kips
8	91.0 ft	37.8 kips	± 0.0 kips	37.8 kips	28.4 kips	± 1.8 kips	28.4 kips	28.4 kips	± 0.0 kips	28.4 kips	21.3 kips	± 1.3 kips	21.3 kips
7	78.0 ft	37.1 kips	± 0.0 kips	37.1 kips	27.8 kips	± 1.8 kips	27.8 kips	27.8 kips	± 0.0 kips	27.8 kips	20.9 kips	± 1.3 kips	20.9 kips
6	65.0 ft	36.4 kips	± 0.0 kips	36.4 kips	27.3 kips	± 1.7 kips	27.3 kips	27.3 kips	± 0.0 kips	27.3 kips	20.5 kips	± 1.3 kips	20.5 kips
5	52.0 ft	35.6 kips	± 0.0 kips	35.6 kips	26.7 kips	± 1.7 kips	26.7 kips	26.7 kips	± 0.0 kips	26.7 kips	20.0 kips	± 1.3 kips	20.0 kips
4	39.0 ft	34.5 kips	± 0.0 kips	34.5 kips	25.9 kips	± 1.6 kips	25.9 kips	25.9 kips	± 0.0 kips	25.9 kips	19.4 kips	± 1.2 kips	19.4 kips
3	26.0 ft	33.4 kips	± 0.0 kips	33.4 kips	25.1 kips	± 1.6 kips	25.1 kips	25.1 kips	± 0.0 kips	25.1 kips	18.8 kips	± 1.2 kips	18.8 kips
2	13.0 ft	31.7 kips	± 0.0 kips	31.7 kips	23.8 kips	± 1.5 kips	23.8 kips	23.8 kips	± 0.0 kips	23.8 kips	17.8 kips	± 1.1 kips	17.8 kips
GROUND	0.0 ft	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.
				711.5 kips			*533.7 kips*			*533.7 kips*			*400.4 kips*

GG Engineering	JOB NO. 18-STORY BUILDINGS	BY SMG	DATE 4/22/05	SHEET NO. ___ OF ___
	CUSTOMER RCFT PARAMETRIC STUDY	CKD	DATE	

SUBJECT

2-D MOMENT FRAME [MF A3 - G3] ANALYSIS LOAD SUMMARY

[17] 18-STORY BUILDINGS MOMENT FRAME ANALYSIS LOAD SUMMARY

o SUMMARY:

TOTAL NUMBER OF (IDENTICAL) MOMENT FRAMES ALONG THE E-W DIRECTION	4
NUMBER OF STORIES IN THE MOMENT FRAME, N_S	18 STORIES
NUMBER OF BAYS IN THE MOMENT FRAME, N_B	6 BAYS
DIRECTION THAT THE *MOMENT FRAME* RUNS PARALLEL WITH:	E-W
LOCATION OF THE MOMENT FRAME WRT THE BUILDING PERIMETER:	INTERIOR
DOES THIS FRAME SUPPORT PART OF THE PENTHOUSE GRAVITY LOADS?	YES
DISTANCE TO THE CLOSEST (GRAVITY/MOMENT) FRAME:	20.0 ft
DISTANCE TO THE CLOSEST (GRAVITY/MOMENT) FRAME ON THE OTHER SIDE:	20.0 ft
DESIGN SEISMIC BASE SHEAR	0.044 W
BASIC WIND SPEED	85 mph

o DESIGN ASSUMPTION: *BUILDING HAS "RIGID DIAPHRAGMS" (PER SEISMIC DESIGN), THE BUILDING IS CONSIDERED A "SIMPLE DIAPHRAGM" BUILDING (PER WIND DESIGN), AND ALL OF THE MOMENT FRAMES HAVE THE SAME RIGIDITY AT EACH FLOOR LEVEL. THEREFORE, ALL LATERAL LOADS ARE DISTRIBUTED EQUALLY AMONG ALL OF THE MOMENT FRAMES IN EACH PRINCIPAL DIRECTION OF THE BUILDING.*

NUMBER		STORY HEIGHT	FLOOR ELEVATION h_x	DESIGN SHEAR LOAD AT EACH FLOOR LEVEL PER MOMENT FRAME (THESE ARE THE POINT LOADS THAT SHALL BE USED IN THE 2-D MODEL)				
				SEISMIC LOAD	WIND LOAD			
STORY	FLOOR				CASE #1	CASE #2	CASE #3	CASE #4
---	---	---	---	---	---	---	---	---
---	ROOF	---	234.0 ft	46.2 kips	15.0 kips	13.3 kips	11.2 kips	10.1 kips
18	18	13.0 ft	221.0 ft	41.7 kips	10.6 kips	9.9 kips	7.9 kips	7.5 kips
17	17	13.0 ft	208.0 ft	37.3 kips	10.5 kips	9.9 kips	7.9 kips	7.4 kips
16	16	13.0 ft	195.0 ft	33.1 kips	10.4 kips	9.8 kips	7.8 kips	7.4 kips
15	15	13.0 ft	182.0 ft	29.1 kips	10.3 kips	9.6 kips	7.7 kips	7.3 kips
14	14	13.0 ft	169.0 ft	25.4 kips	10.2 kips	9.6 kips	7.7 kips	7.3 kips
13	13	13.0 ft	156.0 ft	21.9 kips	10.1 kips	9.5 kips	7.6 kips	7.1 kips
12	12	13.0 ft	143.0 ft	18.6 kips	10.0 kips	9.4 kips	7.5 kips	7.0 kips
11	11	13.0 ft	130.0 ft	15.6 kips	9.9 kips	9.3 kips	7.4 kips	7.0 kips
10	10	13.0 ft	117.0 ft	12.8 kips	9.7 kips	9.1 kips	7.3 kips	6.9 kips
9	9	13.0 ft	104.0 ft	10.3 kips	9.6 kips	9.0 kips	7.2 kips	6.8 kips
8	8	13.0 ft	91.0 ft	8.1 kips	9.5 kips	8.9 kips	7.1 kips	6.6 kips
7	7	13.0 ft	78.0 ft	6.1 kips	9.3 kips	8.8 kips	7.0 kips	6.5 kips
6	6	13.0 ft	65.0 ft	4.3 kips	9.1 kips	8.5 kips	6.8 kips	6.4 kips
5	5	13.0 ft	52.0 ft	2.9 kips	8.9 kips	8.4 kips	6.7 kips	6.3 kips
4	4	13.0 ft	39.0 ft	1.7 kips	8.6 kips	8.1 kips	6.5 kips	6.1 kips
3	3	13.0 ft	26.0 ft	0.8 kips	8.4 kips	7.9 kips	6.3 kips	5.9 kips
2	2	13.0 ft	13.0 ft	0.2 kips	7.9 kips	7.5 kips	6.0 kips	5.6 kips
1	GROUND	13.0 ft	0.0 ft	N.A.	N.A.	N.A.	N.A.	N.A.
				316.1 kips	*178.0 kips*	*166.5 kips*	*133.6 kips*	*125.2 kips*

Appendix F

Building Design 18A Calculations

This appendix consists of the design calculations that were performed for building Design 18A which is the 18-story building that used low strength materials in the columns (F_{yc} = 46 ksi and f'_c = 4 ksi) and a relatively low column d/t ratio. The final RCFT column and wide flange girder sections are presented in Chapter 5. The linear-elastic analysis consisted of taking the nominal loads that were generated in Appendix E and factoring them per the applicable LRFD load combination. The calculation for the stability coefficient, θ, and the moment magnification factor, B_2, were performed for each load combination that has lateral loads (wind and seismic load combinations #4, #5, and #6) and are included in this appendix. The maximum interaction value for each column is listed at the end of this appendix along with its respective load combination.

The load combinations that were used in this building design are listed below for reference:

1. 1.4D
2. $1.2D + 1.6L + 0.5L_R$
3. $1.2D + 1.6L_R + f_1L$
4. $1.2D + 1.6L_R + 0.8W$
5. $1.2D + 1.6W + f_1L + 0.5L_R$
6. $1.2D + 1.0E + f_1L$

Where: $f_1 = 0.5$
$E = \rho Q_E + 0.2S_{DS}D'$
D' = seismic weight

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SUBJECT

DESIGN PARAMETERS SUMMARY

MOMENT FRAME
MF A3 - G3

o DESIGN INPUTS:

o TOAL NUMBER OF COLUMNS BEING ANALYZED 126

o YIELD STRENGTH:
HSS, F_y = 46 ksi
CONCRETE REINFORCEMENT, F_{yr} = 0 ksi

o MODULUS OF ELASTICITY:
HSS, E_s = 29,000 ksi
CONCRETE REINFORCEMENT, E_{cr} = 29,000 ksi

o MINIMUM CONCRETE COMPRESSIVE STRENGTH f'_c = 4.0 ksi

o CONCRETE DENSITY w = 145 lb/ft³

o CONCRETE REINFORCEMENT
AREA, A_{sr} = 0.0 in²
I_{xxr} = 0.0 in^4
I_{yyr} = 0.0 in^4
Z_{xxr} = 0.0 in³
Z_{yyr} = 0.0 in³

o RESISTANCE FACTORS
AXIAL COMPRESSION, ϕ_c = 0.75
FLEXURAL BENDING, ϕ_b = 0.90

o SEISMIC PARAMETERS
REDUNDANCY COEFFICIENT, ρ = 1.00
VERTICAL SEISMIC "FACTOR," $0.2S_{DS}$ = 0.20
ORTHOGONAL LOAD FACTOR ALONG Y-AXIS OF SHARED COLUMNS = 0.30
FACTOR TO ACCOUNT FOR *5% ACCIDENTAL TORSION* ("SIMPLIFIED APPROACH"…) = 0.025

GG Engineering	JOB NO. 18 - STORY BUILDINGS	BY SMG	DATE 4/22/05	SHEET NO. ___ OF ___
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SUBJECT

B2 CALCULATION - *FOR BENDING ALONG THE X-AXIS OF THE COLUMN*

MOMENT FRAME ***MF A3 - G3***

o ***LOAD COMBINATION =*** 1.2D + 1.6Lr + 0.8W *(L.C. # 4)*

STORY NUMBER	LENGTH *(STORY HEIGHT)* L	DUE TO FORCES THAT CAUSE BENDING ALONG THE X-AXIS OF THE MOMENT FRAME COLUMNS: TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips): DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY — *LOAD FACTOR*: *D.L.* 1.2	*L.L.* 0	*ROOF L.L.* 1.6	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0	TOTAL ΣP_{ui} (kips)	**STORY $B_{2i\_X\text{-}AXIS}$**
18	13.0 ft	185 kips	0.25 in	1,513	0	288	1,591	1,815.6	0.0	460.8	0.0	2,276.4	**1.020**
17	13.0 ft	352 kips	0.37 in	2,821	1,008	288	3,187	3,385.2	0.0	460.8	0.0	3,846.0	**1.027**
16	13.0 ft	501 kips	0.45 in	4,129	2,016	288	4,783	4,954.8	0.0	460.8	0.0	5,415.6	**1.032**
15	13.0 ft	633 kips	0.47 in	5,437	3,024	288	6,379	6,524.4	0.0	460.8	0.0	6,985.2	**1.034**
14	13.0 ft	749 kips	0.52 in	6,745	4,032	288	7,975	8,094.0	0.0	460.8	0.0	8,554.8	**1.040**
13	13.0 ft	851 kips	0.55 in	8,053	5,040	288	9,571	9,663.6	0.0	460.8	0.0	10,124.4	**1.044**
12	13.0 ft	938 kips	0.55 in	9,361	6,048	288	11,167	11,233.2	0.0	460.8	0.0	11,694.0	**1.046**
11	13.0 ft	1,013 kips	0.57 in	10,669	7,056	288	12,763	12,802.8	0.0	460.8	0.0	13,263.6	**1.050**
10	13.0 ft	1,075 kips	0.55 in	11,977	8,064	288	14,359	14,372.4	0.0	460.8	0.0	14,833.2	**1.051**
9	13.0 ft	1,126 kips	0.55 in	13,285	9,072	288	15,955	15,942.0	0.0	460.8	0.0	16,402.8	**1.054**
8	13.0 ft	1,168 kips	0.57 in	14,593	10,080	288	17,551	17,511.6	0.0	460.8	0.0	17,972.4	**1.060**
7	13.0 ft	1,200 kips	0.57 in	15,901	11,088	288	19,147	19,081.2	0.0	460.8	0.0	19,542.0	**1.063**
6	13.0 ft	1,224 kips	0.58 in	17,209	12,096	288	20,743	20,650.8	0.0	460.8	0.0	21,111.6	**1.069**
5	13.0 ft	1,241 kips	0.59 in	18,517	13,104	288	22,339	22,220.4	0.0	460.8	0.0	22,681.2	**1.074**
4	13.0 ft	1,253 kips	0.59 in	19,825	14,112	288	23,935	23,790.0	0.0	460.8	0.0	24,250.8	**1.079**
3	13.0 ft	1,260 kips	0.58 in	21,133	15,120	288	25,531	25,359.6	0.0	460.8	0.0	25,820.4	**1.082**
2	13.0 ft	1,263 kips	0.54 in	22,441	16,128	288	27,127	26,929.2	0.0	460.8	0.0	27,390.0	**1.081**
1	13.0 ft	1,264 kips	0.32 in	23,749	17,136	288	28,723	28,498.8	0.0	460.8	0.0	28,959.6	**1.049**

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SUBJECT **B2 CALCULATION - *FOR BENDING ALONG THE Y-AXIS OF THE COLUMN*** — **MOMENT FRAME MF A3 - G3**

o ***LOAD COMBINATION =*** **1.2D + 1.6Lr + 0.8W**

		DUE TO FORCES THAT CAUSE BENDING ALONG THE Y-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					**STORY $B_{2i\_Y\text{-}AXIS}$**
								LOAD FACTOR					
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0	*ROOF L.L.* 1.6	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0	TOTAL ΣP_{ui} (kips)	
18	13.0 ft	185 kips	0.25 in	1,513	0	288	1,591	1,815.6	0.0	460.8	0.0	2,276.4	**1.020**
17	13.0 ft	352 kips	0.37 in	2,821	1,008	288	3,187	3,385.2	0.0	460.8	0.0	3,846.0	**1.027**
16	13.0 ft	501 kips	0.45 in	4,129	2,016	288	4,783	4,954.8	0.0	460.8	0.0	5,415.6	**1.032**
15	13.0 ft	633 kips	0.47 in	5,437	3,024	288	6,379	6,524.4	0.0	460.8	0.0	6,985.2	**1.034**
14	13.0 ft	749 kips	0.52 in	6,745	4,032	288	7,975	8,094.0	0.0	460.8	0.0	8,554.8	**1.040**
13	13.0 ft	851 kips	0.55 in	8,053	5,040	288	9,571	9,663.6	0.0	460.8	0.0	10,124.4	**1.044**
12	13.0 ft	938 kips	0.55 in	9,361	6,048	288	11,167	11,233.2	0.0	460.8	0.0	11,694.0	**1.046**
11	13.0 ft	1,013 kips	0.57 in	10,669	7,056	288	12,763	12,802.8	0.0	460.8	0.0	13,263.6	**1.050**
10	13.0 ft	1,075 kips	0.55 in	11,977	8,064	288	14,359	14,372.4	0.0	460.8	0.0	14,833.2	**1.051**
9	13.0 ft	1,126 kips	0.55 in	13,285	9,072	288	15,955	15,942.0	0.0	460.8	0.0	16,402.8	**1.054**
8	13.0 ft	1,168 kips	0.57 in	14,593	10,080	288	17,551	17,511.6	0.0	460.8	0.0	17,972.4	**1.060**
7	13.0 ft	1,200 kips	0.57 in	15,901	11,088	288	19,147	19,081.2	0.0	460.8	0.0	19,542.0	**1.063**
6	13.0 ft	1,224 kips	0.58 in	17,209	12,096	288	20,743	20,650.8	0.0	460.8	0.0	21,111.6	**1.069**
5	13.0 ft	1,241 kips	0.59 in	18,517	13,104	288	22,339	22,220.4	0.0	460.8	0.0	22,681.2	**1.074**
4	13.0 ft	1,253 kips	0.59 in	19,825	14,112	288	23,935	23,790.0	0.0	460.8	0.0	24,250.8	**1.079**
3	13.0 ft	1,260 kips	0.58 in	21,133	15,120	288	25,531	25,359.6	0.0	460.8	0.0	25,820.4	**1.082**
2	13.0 ft	1,263 kips	0.54 in	22,441	16,128	288	27,127	26,929.2	0.0	460.8	0.0	27,390.0	**1.081**
1	13.0 ft	1,264 kips	0.32 in	23,749	17,136	288	28,723	28,498.8	0.0	460.8	0.0	28,959.6	**1.049**

GG Engineering	JOB NO. 18 - STORY BUILDINGS	BY SMG	DATE 4/22/05	SHEET NO. ___ OF ___
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SUBJECT

STABILITY COEFFICIENT ALONG COLUMN X-AXIS, θ_x

MOMENT FRAME
MF A3 - G3

- o ***LOAD COMBINATION:*** 1.2D + 1.6Lr + 0.8W
- o ***DEFLECTION AMPLIFICATION FACTOR:*** C_d = 5.5
- o ***(SEISMIC) IMPORTANCE FACTOR*** I_E = 1.0
- o ***MOMENT FRAME RESISTS WHAT % OF THE TOTAL SEISMIC SHEAR TO THE BUIDLING?*** 25%

		DUE TO FORCES THAT CAUSE BENDING ALONG THE X-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					STABILITY COEFFICIENT PER STORY θ_i
								LOAD FACTOR					
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO ANY HORIZONTAL LOAD ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0	*ROOF L.L.* 1.6	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0	TOTAL ΣP_{ui} (kips)	
18	13.0 ft	185 kips	0.25 in	1,513	0	288	1,591	1,815.6	0.0	460.8	0.0	2,276.4	**0.020**
17	13.0 ft	352 kips	0.37 in	2,821	1,008	288	3,187	3,385.2	0.0	460.8	0.0	3,846.0	**0.026**
16	13.0 ft	501 kips	0.45 in	4,129	2,016	288	4,783	4,954.8	0.0	460.8	0.0	5,415.6	**0.031**
15	13.0 ft	633 kips	0.47 in	5,437	3,024	288	6,379	6,524.4	0.0	460.8	0.0	6,985.2	**0.033**
14	13.0 ft	749 kips	0.52 in	6,745	4,032	288	7,975	8,094.0	0.0	460.8	0.0	8,554.8	**0.038**
13	13.0 ft	851 kips	0.55 in	8,053	5,040	288	9,571	9,663.6	0.0	460.8	0.0	10,124.4	**0.042**
12	13.0 ft	938 kips	0.55 in	9,361	6,048	288	11,167	11,233.2	0.0	460.8	0.0	11,694.0	**0.044**
11	13.0 ft	1,013 kips	0.57 in	10,669	7,056	288	12,763	12,802.8	0.0	460.8	0.0	13,263.6	**0.048**
10	13.0 ft	1,075 kips	0.55 in	11,977	8,064	288	14,359	14,372.4	0.0	460.8	0.0	14,833.2	**0.049**
9	13.0 ft	1,126 kips	0.55 in	13,285	9,072	288	15,955	15,942.0	0.0	460.8	0.0	16,402.8	**0.051**
8	13.0 ft	1,168 kips	0.57 in	14,593	10,080	288	17,551	17,511.6	0.0	460.8	0.0	17,972.4	**0.056**
7	13.0 ft	1,200 kips	0.57 in	15,901	11,088	288	19,147	19,081.2	0.0	460.8	0.0	19,542.0	**0.060**
6	13.0 ft	1,224 kips	0.58 in	17,209	12,096	288	20,743	20,650.8	0.0	460.8	0.0	21,111.6	**0.064**
5	13.0 ft	1,241 kips	0.59 in	18,517	13,104	288	22,339	22,220.4	0.0	460.8	0.0	22,681.2	**0.069**
4	13.0 ft	1,253 kips	0.59 in	19,825	14,112	288	23,935	23,790.0	0.0	460.8	0.0	24,250.8	**0.073**
3	13.0 ft	1,260 kips	0.58 in	21,133	15,120	288	25,531	25,359.6	0.0	460.8	0.0	25,820.4	**0.076**
2	13.0 ft	1,263 kips	0.54 in	22,441	16,128	288	27,127	26,929.2	0.0	460.8	0.0	27,390.0	**0.075**
1	13.0 ft	1,264 kips	0.32 in	23,749	17,136	288	28,723	28,498.8	0.0	460.8	0.0	28,959.6	**0.047**

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SUBJECT

STABILITY COEFFICIENT ALONG COLUMN Y-AXIS, θ_y

MOMENT FRAME
MF A3 - G3

- ***LOAD COMBINATION:*** 1.2D + 1.6Lr + 0.8W
- ***DEFLECTION AMPLIFICATION FACTOR:*** C_d = 5.5
- ***(SEISMIC) IMPORTANCE FACTOR*** I_E = 1.0
- ***MOMENT FRAME RESISTS WHAT % OF THE TOTAL SEISMIC SHEAR TO THE BUIDLING?*** 25%

		DUE TO FORCES THAT CAUSE BENDING ALONG THE Y-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					
								LOAD FACTOR					
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO ANY HORIZONTAL LOAD ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0	*ROOF L.L.* 1.6	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0	TOTAL ΣP_{ui} (kips)	STABILITY COEFFICIENT PER STORY θ_i
18	13.0 ft	185 kips	0.25 in	1,513	0	288	1,591	1,815.6	0.0	460.8	0.0	2,276.4	**0.020**
17	13.0 ft	352 kips	0.37 in	2,821	1,008	288	3,187	3,385.2	0.0	460.8	0.0	3,846.0	**0.026**
16	13.0 ft	501 kips	0.45 in	4,129	2,016	288	4,783	4,954.8	0.0	460.8	0.0	5,415.6	**0.031**
15	13.0 ft	633 kips	0.47 in	5,437	3,024	288	6,379	6,524.4	0.0	460.8	0.0	6,985.2	**0.033**
14	13.0 ft	749 kips	0.52 in	6,745	4,032	288	7,975	8,094.0	0.0	460.8	0.0	8,554.8	**0.038**
13	13.0 ft	851 kips	0.55 in	8,053	5,040	288	9,571	9,663.6	0.0	460.8	0.0	10,124.4	**0.042**
12	13.0 ft	938 kips	0.55 in	9,361	6,048	288	11,167	11,233.2	0.0	460.8	0.0	11,694.0	**0.044**
11	13.0 ft	1,013 kips	0.57 in	10,669	7,056	288	12,763	12,802.8	0.0	460.8	0.0	13,263.6	**0.048**
10	13.0 ft	1,075 kips	0.55 in	11,977	8,064	288	14,359	14,372.4	0.0	460.8	0.0	14,833.2	**0.049**
9	13.0 ft	1,126 kips	0.55 in	13,285	9,072	288	15,955	15,942.0	0.0	460.8	0.0	16,402.8	**0.051**
8	13.0 ft	1,168 kips	0.57 in	14,593	10,080	288	17,551	17,511.6	0.0	460.8	0.0	17,972.4	**0.056**
7	13.0 ft	1,200 kips	0.57 in	15,901	11,088	288	19,147	19,081.2	0.0	460.8	0.0	19,542.0	**0.060**
6	13.0 ft	1,224 kips	0.58 in	17,209	12,096	288	20,743	20,650.8	0.0	460.8	0.0	21,111.6	**0.064**
5	13.0 ft	1,241 kips	0.59 in	18,517	13,104	288	22,339	22,220.4	0.0	460.8	0.0	22,681.2	**0.069**
4	13.0 ft	1,253 kips	0.59 in	19,825	14,112	288	23,935	23,790.0	0.0	460.8	0.0	24,250.8	**0.073**
3	13.0 ft	1,260 kips	0.58 in	21,133	15,120	288	25,531	25,359.6	0.0	460.8	0.0	25,820.4	**0.076**
2	13.0 ft	1,263 kips	0.54 in	22,441	16,128	288	27,127	26,929.2	0.0	460.8	0.0	27,390.0	**0.075**
1	13.0 ft	1,264 kips	0.32 in	23,749	17,136	288	28,723	28,498.8	0.0	460.8	0.0	28,959.6	**0.047**

GG Engineering	JOB NO. 18 - STORY BUILDINGS	BY SMG	DATE 4/22/05	SHEET NO. ___ OF ___
	CUSTOMER DESIGN 18A	CKD	DATE	

SUBJECT

B2 CALCULATION - *FOR BENDING ALONG THE X-AXIS OF THE COLUMN*

MOMENT FRAME
MF A3 - G3

o ***LOAD COMBINATION =*** 1.2D + 0.5L + 0.5Lr + 1.6W *(L.C. # 5)*

STORY NUMBER	LENGTH *(STORY HEIGHT)* L	DUE TO FORCES THAT CAUSE BENDING ALONG THE X-AXIS OF THE MOMENT FRAME COLUMNS: TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips): DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY — *LOAD FACTOR*: *D.L.* 1.2	*L.L.* 0.5	*ROOF L.L.* 0.5	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0	TOTAL ΣP_{ui} (kips)	**STORY $B_{2i\_X\text{-}AXIS}$**
18	13.0 ft	185 kips	0.25 in	1,513	0	288	1,591	1,815.6	0.0	144.0	0.0	1,959.6	**1.017**
17	13.0 ft	352 kips	0.37 in	2,821	1,008	288	3,187	3,385.2	504.0	144.0	0.0	4,033.2	**1.028**
16	13.0 ft	501 kips	0.45 in	4,129	2,016	288	4,783	4,954.8	1,008.0	144.0	0.0	6,106.8	**1.036**
15	13.0 ft	633 kips	0.47 in	5,437	3,024	288	6,379	6,524.4	1,512.0	144.0	0.0	8,180.4	**1.041**
14	13.0 ft	749 kips	0.52 in	6,745	4,032	288	7,975	8,094.0	2,016.0	144.0	0.0	10,254.0	**1.048**
13	13.0 ft	851 kips	0.55 in	8,053	5,040	288	9,571	9,663.6	2,520.0	144.0	0.0	12,327.6	**1.054**
12	13.0 ft	938 kips	0.55 in	9,361	6,048	288	11,167	11,233.2	3,024.0	144.0	0.0	14,401.2	**1.057**
11	13.0 ft	1,013 kips	0.57 in	10,669	7,056	288	12,763	12,802.8	3,528.0	144.0	0.0	16,474.8	**1.063**
10	13.0 ft	1,075 kips	0.55 in	11,977	8,064	288	14,359	14,372.4	4,032.0	144.0	0.0	18,548.4	**1.065**
9	13.0 ft	1,126 kips	0.55 in	13,285	9,072	288	15,955	15,942.0	4,536.0	144.0	0.0	20,622.0	**1.069**
8	13.0 ft	1,168 kips	0.57 in	14,593	10,080	288	17,551	17,511.6	5,040.0	144.0	0.0	22,695.6	**1.076**
7	13.0 ft	1,200 kips	0.57 in	15,901	11,088	288	19,147	19,081.2	5,544.0	144.0	0.0	24,769.2	**1.082**
6	13.0 ft	1,224 kips	0.58 in	17,209	12,096	288	20,743	20,650.8	6,048.0	144.0	0.0	26,842.8	**1.089**
5	13.0 ft	1,241 kips	0.59 in	18,517	13,104	288	22,339	22,220.4	6,552.0	144.0	0.0	28,916.4	**1.097**
4	13.0 ft	1,253 kips	0.59 in	19,825	14,112	288	23,935	23,790.0	7,056.0	144.0	0.0	30,990.0	**1.103**
3	13.0 ft	1,260 kips	0.58 in	21,133	15,120	288	25,531	25,359.6	7,560.0	144.0	0.0	33,063.6	**1.108**
2	13.0 ft	1,263 kips	0.54 in	22,441	16,128	288	27,127	26,929.2	8,064.0	144.0	0.0	35,137.2	**1.107**
1	13.0 ft	1,264 kips	0.32 in	23,749	17,136	288	28,723	28,498.8	8,568.0	144.0	0.0	37,210.8	**1.064**

GG Engineering	JOB NO. 18 - STORY BUILDINGS	BY SMG	DATE 4/22/05	SHEET NO. ___ OF ___
	CUSTOMER DESIGN 18A	CKD	DATE	

SUBJECT **B2 CALCULATION - *FOR BENDING ALONG THE Y-AXIS OF THE COLUMN***

MOMENT FRAME MF A3 - G3

o ***LOAD COMBINATION =*** **1.2D + 0.5L + 0.5Lr + 1.6W**

		DUE TO FORCES THAT CAUSE BENDING ALONG THE Y-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					
								LOAD FACTOR					
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0.5	*ROOF L.L.* 0.5	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0	TOTAL ΣP_{ui} (kips)	**STORY $B_{2i\_Y\text{-}AXIS}$**
18	13.0 ft	185 kips	0.25 in	1,513	0	288	1,591	1,815.6	0.0	144.0	0.0	1,959.6	**1.017**
17	13.0 ft	352 kips	0.37 in	2,821	1,008	288	3,187	3,385.2	504.0	144.0	0.0	4,033.2	**1.028**
16	13.0 ft	501 kips	0.45 in	4,129	2,016	288	4,783	4,954.8	1,008.0	144.0	0.0	6,106.8	**1.036**
15	13.0 ft	633 kips	0.47 in	5,437	3,024	288	6,379	6,524.4	1,512.0	144.0	0.0	8,180.4	**1.041**
14	13.0 ft	749 kips	0.52 in	6,745	4,032	288	7,975	8,094.0	2,016.0	144.0	0.0	10,254.0	**1.048**
13	13.0 ft	851 kips	0.55 in	8,053	5,040	288	9,571	9,663.6	2,520.0	144.0	0.0	12,327.6	**1.054**
12	13.0 ft	938 kips	0.55 in	9,361	6,048	288	11,167	11,233.2	3,024.0	144.0	0.0	14,401.2	**1.057**
11	13.0 ft	1,013 kips	0.57 in	10,669	7,056	288	12,763	12,802.8	3,528.0	144.0	0.0	16,474.8	**1.063**
10	13.0 ft	1,075 kips	0.55 in	11,977	8,064	288	14,359	14,372.4	4,032.0	144.0	0.0	18,548.4	**1.065**
9	13.0 ft	1,126 kips	0.55 in	13,285	9,072	288	15,955	15,942.0	4,536.0	144.0	0.0	20,622.0	**1.069**
8	13.0 ft	1,168 kips	0.57 in	14,593	10,080	288	17,551	17,511.6	5,040.0	144.0	0.0	22,695.6	**1.076**
7	13.0 ft	1,200 kips	0.57 in	15,901	11,088	288	19,147	19,081.2	5,544.0	144.0	0.0	24,769.2	**1.082**
6	13.0 ft	1,224 kips	0.58 in	17,209	12,096	288	20,743	20,650.8	6,048.0	144.0	0.0	26,842.8	**1.089**
5	13.0 ft	1,241 kips	0.59 in	18,517	13,104	288	22,339	22,220.4	6,552.0	144.0	0.0	28,916.4	**1.097**
4	13.0 ft	1,253 kips	0.59 in	19,825	14,112	288	23,935	23,790.0	7,056.0	144.0	0.0	30,990.0	**1.103**
3	13.0 ft	1,260 kips	0.58 in	21,133	15,120	288	25,531	25,359.6	7,560.0	144.0	0.0	33,063.6	**1.108**
2	13.0 ft	1,263 kips	0.54 in	22,441	16,128	288	27,127	26,929.2	8,064.0	144.0	0.0	35,137.2	**1.107**
1	13.0 ft	1,264 kips	0.32 in	23,749	17,136	288	28,723	28,498.8	8,568.0	144.0	0.0	37,210.8	**1.064**

GG Engineering	JOB NO. 18 - STORY BUILDINGS	BY SMG	DATE 4/22/05	SHEET NO.
	CUSTOMER DESIGN 18A	CKD	DATE	___ OF ___

SUBJECT

STABILITY COEFFICIENT ALONG COLUMN X-AXIS, θ_x

MOMENT FRAME
MF A3 - G3

- ***LOAD COMBINATION:*** 1.2D + 0.5L + 0.5Lr + 1.6W
- ***DEFLECTION AMPLIFICATION FACTOR:*** C_d = 5.5
- ***(SEISMIC) IMPORTANCE FACTOR*** I_E = 1.0
- ***MOMENT FRAME RESISTS WHAT % OF THE TOTAL SEISMIC SHEAR TO THE BUIDLING?*** 25%

		DUE TO FORCES THAT CAUSE BENDING ALONG THE X-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					STABILITY COEFFICIENT PER STORY θ_i
								LOAD FACTOR					
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO ANY HORIZONTAL LOAD ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0.5	*ROOF L.L.* 0.5	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0	TOTAL ΣP_{ui} (kips)	
18	13.0 ft	185 kips	0.25 in	1,513	0	288	1,591	1,815.6	0.0	144.0	0.0	1,959.6	**0.017**
17	13.0 ft	352 kips	0.37 in	2,821	1,008	288	3,187	3,385.2	504.0	144.0	0.0	4,033.2	**0.027**
16	13.0 ft	501 kips	0.45 in	4,129	2,016	288	4,783	4,954.8	1,008.0	144.0	0.0	6,106.8	**0.035**
15	13.0 ft	633 kips	0.47 in	5,437	3,024	288	6,379	6,524.4	1,512.0	144.0	0.0	8,180.4	**0.039**
14	13.0 ft	749 kips	0.52 in	6,745	4,032	288	7,975	8,094.0	2,016.0	144.0	0.0	10,254.0	**0.046**
13	13.0 ft	851 kips	0.55 in	8,053	5,040	288	9,571	9,663.6	2,520.0	144.0	0.0	12,327.6	**0.051**
12	13.0 ft	938 kips	0.55 in	9,361	6,048	288	11,167	11,233.2	3,024.0	144.0	0.0	14,401.2	**0.054**
11	13.0 ft	1,013 kips	0.57 in	10,669	7,056	288	12,763	12,802.8	3,528.0	144.0	0.0	16,474.8	**0.059**
10	13.0 ft	1,075 kips	0.55 in	11,977	8,064	288	14,359	14,372.4	4,032.0	144.0	0.0	18,548.4	**0.061**
9	13.0 ft	1,126 kips	0.55 in	13,285	9,072	288	15,955	15,942.0	4,536.0	144.0	0.0	20,622.0	**0.065**
8	13.0 ft	1,168 kips	0.57 in	14,593	10,080	288	17,551	17,511.6	5,040.0	144.0	0.0	22,695.6	**0.071**
7	13.0 ft	1,200 kips	0.57 in	15,901	11,088	288	19,147	19,081.2	5,544.0	144.0	0.0	24,769.2	**0.075**
6	13.0 ft	1,224 kips	0.58 in	17,209	12,096	288	20,743	20,650.8	6,048.0	144.0	0.0	26,842.8	**0.082**
5	13.0 ft	1,241 kips	0.59 in	18,517	13,104	288	22,339	22,220.4	6,552.0	144.0	0.0	28,916.4	**0.088**
4	13.0 ft	1,253 kips	0.59 in	19,825	14,112	288	23,935	23,790.0	7,056.0	144.0	0.0	30,990.0	**0.094**
3	13.0 ft	1,260 kips	0.58 in	21,133	15,120	288	25,531	25,359.6	7,560.0	144.0	0.0	33,063.6	**0.098**
2	13.0 ft	1,263 kips	0.54 in	22,441	16,128	288	27,127	26,929.2	8,064.0	144.0	0.0	35,137.2	**0.096**
1	13.0 ft	1,264 kips	0.32 in	23,749	17,136	288	28,723	28,498.8	8,568.0	144.0	0.0	37,210.8	**0.060**

GG Engineering	JOB NO. 18 - STORY BUILDINGS	BY SMG	DATE 4/22/05	SHEET NO. ___ OF ___
	CUSTOMER DESIGN 18A	CKD	DATE	

SUBJECT

STABILITY COEFFICIENT ALONG COLUMN Y-AXIS, θ_y

MOMENT FRAME MF A3 - G3

- ***LOAD COMBINATION:*** 1.2D + 0.5L + 0.5Lr + 1.6W
- ***DEFLECTION AMPLIFICATION FACTOR:*** C_d = 5.5
- ***(SEISMIC) IMPORTANCE FACTOR*** I_E = 1.0
- ***MOMENT FRAME RESISTS WHAT % OF THE TOTAL SEISMIC SHEAR TO THE BUIDLING?*** 25%

		DUE TO FORCES THAT CAUSE BENDING ALONG THE Y-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					STABILITY COEFFICIENT PER STORY θ_i
								LOAD FACTOR					
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO ANY HORIZONTAL LOAD ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0.5	*ROOF L.L.* 0.5	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0	TOTAL ΣP_{ui} (kips)	
18	13.0 ft	185 kips	0.25 in	1,513	0	288	1,591	1,815.6	0.0	144.0	0.0	1,959.6	**0.017**
17	13.0 ft	352 kips	0.37 in	2,821	1,008	288	3,187	3,385.2	504.0	144.0	0.0	4,033.2	**0.027**
16	13.0 ft	501 kips	0.45 in	4,129	2,016	288	4,783	4,954.8	1,008.0	144.0	0.0	6,106.8	**0.035**
15	13.0 ft	633 kips	0.47 in	5,437	3,024	288	6,379	6,524.4	1,512.0	144.0	0.0	8,180.4	**0.039**
14	13.0 ft	749 kips	0.52 in	6,745	4,032	288	7,975	8,094.0	2,016.0	144.0	0.0	10,254.0	**0.046**
13	13.0 ft	851 kips	0.55 in	8,053	5,040	288	9,571	9,663.6	2,520.0	144.0	0.0	12,327.6	**0.051**
12	13.0 ft	938 kips	0.55 in	9,361	6,048	288	11,167	11,233.2	3,024.0	144.0	0.0	14,401.2	**0.054**
11	13.0 ft	1,013 kips	0.57 in	10,669	7,056	288	12,763	12,802.8	3,528.0	144.0	0.0	16,474.8	**0.059**
10	13.0 ft	1,075 kips	0.55 in	11,977	8,064	288	14,359	14,372.4	4,032.0	144.0	0.0	18,548.4	**0.061**
9	13.0 ft	1,126 kips	0.55 in	13,285	9,072	288	15,955	15,942.0	4,536.0	144.0	0.0	20,622.0	**0.065**
8	13.0 ft	1,168 kips	0.57 in	14,593	10,080	288	17,551	17,511.6	5,040.0	144.0	0.0	22,695.6	**0.071**
7	13.0 ft	1,200 kips	0.57 in	15,901	11,088	288	19,147	19,081.2	5,544.0	144.0	0.0	24,769.2	**0.075**
6	13.0 ft	1,224 kips	0.58 in	17,209	12,096	288	20,743	20,650.8	6,048.0	144.0	0.0	26,842.8	**0.082**
5	13.0 ft	1,241 kips	0.59 in	18,517	13,104	288	22,339	22,220.4	6,552.0	144.0	0.0	28,916.4	**0.088**
4	13.0 ft	1,253 kips	0.59 in	19,825	14,112	288	23,935	23,790.0	7,056.0	144.0	0.0	30,990.0	**0.094**
3	13.0 ft	1,260 kips	0.58 in	21,133	15,120	288	25,531	25,359.6	7,560.0	144.0	0.0	33,063.6	**0.098**
2	13.0 ft	1,263 kips	0.54 in	22,441	16,128	288	27,127	26,929.2	8,064.0	144.0	0.0	35,137.2	**0.096**
1	13.0 ft	1,264 kips	0.32 in	23,749	17,136	288	28,723	28,498.8	8,568.0	144.0	0.0	37,210.8	**0.060**

GG Engineering	JOB NO. 18 - STORY BUILDINGS	BY SMG	DATE 4/22/05	SHEET NO.
	CUSTOMER DESIGN 18A	CKD	DATE	OF

SUBJECT

B2 CALCULATION - *FOR BENDING ALONG THE X-AXIS OF THE COLUMN*

MOMENT FRAME
MF A3 - G3

o ***LOAD COMBINATION =*** 1.2D + 0.5L + 1.0E *(L.C. # 6)*

STORY NUMBER	LENGTH *(STORY HEIGHT)* L	DUE TO FORCES THAT CAUSE BENDING ALONG THE X-AXIS OF THE MOMENT FRAME COLUMNS: TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips): DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY — *LOAD FACTOR*: *D.L.* 1.2	*L.L.* 0.5	*ROOF L.L.* 0	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0.2	TOTAL ΣP_{ui} (kips)	**STORY $B_{2i\_X\text{-}AXIS}$**
18	13.0 ft	185 kips	0.25 in	1,513	0	288	1,591	1,815.6	0.0	0.0	318.2	2,133.8	**1.019**
17	13.0 ft	352 kips	0.37 in	2,821	1,008	288	3,187	3,385.2	504.0	0.0	637.4	4,526.6	**1.031**
16	13.0 ft	501 kips	0.45 in	4,129	2,016	288	4,783	4,954.8	1,008.0	0.0	956.6	6,919.4	**1.041**
15	13.0 ft	633 kips	0.47 in	5,437	3,024	288	6,379	6,524.4	1,512.0	0.0	1,275.8	9,312.2	**1.046**
14	13.0 ft	749 kips	0.52 in	6,745	4,032	288	7,975	8,094.0	2,016.0	0.0	1,595.0	11,705.0	**1.055**
13	13.0 ft	851 kips	0.55 in	8,053	5,040	288	9,571	9,663.6	2,520.0	0.0	1,914.2	14,097.8	**1.062**
12	13.0 ft	938 kips	0.55 in	9,361	6,048	288	11,167	11,233.2	3,024.0	0.0	2,233.4	16,490.6	**1.066**
11	13.0 ft	1,013 kips	0.57 in	10,669	7,056	288	12,763	12,802.8	3,528.0	0.0	2,552.6	18,883.4	**1.073**
10	13.0 ft	1,075 kips	0.55 in	11,977	8,064	288	14,359	14,372.4	4,032.0	0.0	2,871.8	21,276.2	**1.075**
9	13.0 ft	1,126 kips	0.55 in	13,285	9,072	288	15,955	15,942.0	4,536.0	0.0	3,191.0	23,669.0	**1.080**
8	13.0 ft	1,168 kips	0.57 in	14,593	10,080	288	17,551	17,511.6	5,040.0	0.0	3,510.2	26,061.8	**1.089**
7	13.0 ft	1,200 kips	0.57 in	15,901	11,088	288	19,147	19,081.2	5,544.0	0.0	3,829.4	28,454.6	**1.095**
6	13.0 ft	1,224 kips	0.58 in	17,209	12,096	288	20,743	20,650.8	6,048.0	0.0	4,148.6	30,847.4	**1.103**
5	13.0 ft	1,241 kips	0.59 in	18,517	13,104	288	22,339	22,220.4	6,552.0	0.0	4,467.8	33,240.2	**1.113**
4	13.0 ft	1,253 kips	0.59 in	19,825	14,112	288	23,935	23,790.0	7,056.0	0.0	4,787.0	35,633.0	**1.121**
3	13.0 ft	1,260 kips	0.58 in	21,133	15,120	288	25,531	25,359.6	7,560.0	0.0	5,106.2	38,025.8	**1.126**
2	13.0 ft	1,263 kips	0.54 in	22,441	16,128	288	27,127	26,929.2	8,064.0	0.0	5,425.4	40,418.6	**1.125**
1	13.0 ft	1,264 kips	0.32 in	23,749	17,136	288	28,723	28,498.8	8,568.0	0.0	5,744.6	42,811.4	**1.075**

GG Engineering	JOB NO. 18 - STORY BUILDINGS	BY SMG	DATE 4/22/05	SHEET NO. ___ OF ___
	CUSTOMER DESIGN 18A	CKD	DATE	

SUBJECT **B2 CALCULATION - *FOR BENDING ALONG THE Y-AXIS OF THE COLUMN*** — **MOMENT FRAME MF A3 - G3**

o ***LOAD COMBINATION =*** **1.2D + 0.5L + 1.0E**

STORY NUMBER	LENGTH *(STORY HEIGHT)* L	DUE TO FORCES THAT CAUSE BENDING ALONG THE Y-AXIS OF THE MOMENT FRAME COLUMNS: TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips): DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY — *LOAD FACTOR*: *D.L.* 1.2	*L.L.* 0.5	*ROOF L.L.* 0	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0.2	TOTAL ΣP_{ui} (kips)	**STORY $B_{2i\_Y\text{-}AXIS}$**
18	13.0 ft	185 kips	0.25 in	1,513	0	288	1,591	1,815.6	0.0	0.0	318.2	2,133.8	**1.019**
17	13.0 ft	352 kips	0.37 in	2,821	1,008	288	3,187	3,385.2	504.0	0.0	637.4	4,526.6	**1.031**
16	13.0 ft	501 kips	0.45 in	4,129	2,016	288	4,783	4,954.8	1,008.0	0.0	956.6	6,919.4	**1.041**
15	13.0 ft	633 kips	0.47 in	5,437	3,024	288	6,379	6,524.4	1,512.0	0.0	1,275.8	9,312.2	**1.046**
14	13.0 ft	749 kips	0.52 in	6,745	4,032	288	7,975	8,094.0	2,016.0	0.0	1,595.0	11,705.0	**1.055**
13	13.0 ft	851 kips	0.55 in	8,053	5,040	288	9,571	9,663.6	2,520.0	0.0	1,914.2	14,097.8	**1.062**
12	13.0 ft	938 kips	0.55 in	9,361	6,048	288	11,167	11,233.2	3,024.0	0.0	2,233.4	16,490.6	**1.066**
11	13.0 ft	1,013 kips	0.57 in	10,669	7,056	288	12,763	12,802.8	3,528.0	0.0	2,552.6	18,883.4	**1.073**
10	13.0 ft	1,075 kips	0.55 in	11,977	8,064	288	14,359	14,372.4	4,032.0	0.0	2,871.8	21,276.2	**1.075**
9	13.0 ft	1,126 kips	0.55 in	13,285	9,072	288	15,955	15,942.0	4,536.0	0.0	3,191.0	23,669.0	**1.080**
8	13.0 ft	1,168 kips	0.57 in	14,593	10,080	288	17,551	17,511.6	5,040.0	0.0	3,510.2	26,061.8	**1.089**
7	13.0 ft	1,200 kips	0.57 in	15,901	11,088	288	19,147	19,081.2	5,544.0	0.0	3,829.4	28,454.6	**1.095**
6	13.0 ft	1,224 kips	0.58 in	17,209	12,096	288	20,743	20,650.8	6,048.0	0.0	4,148.6	30,847.4	**1.103**
5	13.0 ft	1,241 kips	0.59 in	18,517	13,104	288	22,339	22,220.4	6,552.0	0.0	4,467.8	33,240.2	**1.113**
4	13.0 ft	1,253 kips	0.59 in	19,825	14,112	288	23,935	23,790.0	7,056.0	0.0	4,787.0	35,633.0	**1.121**
3	13.0 ft	1,260 kips	0.58 in	21,133	15,120	288	25,531	25,359.6	7,560.0	0.0	5,106.2	38,025.8	**1.126**
2	13.0 ft	1,263 kips	0.54 in	22,441	16,128	288	27,127	26,929.2	8,064.0	0.0	5,425.4	40,418.6	**1.125**
1	13.0 ft	1,264 kips	0.32 in	23,749	17,136	288	28,723	28,498.8	8,568.0	0.0	5,744.6	42,811.4	**1.075**

GG Engineering	JOB NO. 18 - STORY BUILDINGS	BY SMG	DATE 4/22/05	SHEET NO. ___ OF ___
	CUSTOMER DESIGN 18A	CKD	DATE	

SUBJECT

STABILITY COEFFICIENT ALONG COLUMN X-AXIS, θ_x

MOMENT FRAME
MF A3 - G3

- o ***LOAD COMBINATION:*** 1.2D + 0.5L + 1.0E
- o ***DEFLECTION AMPLIFICATION FACTOR:*** C_d = 5.5
- o ***(SEISMIC) IMPORTANCE FACTOR*** I_E = 1.0
- o ***MOMENT FRAME RESISTS WHAT % OF THE TOTAL SEISMIC SHEAR TO THE BUIDLING?*** 25%

		DUE TO FORCES THAT CAUSE BENDING ALONG THE X-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					STABILITY COEFFICIENT PER STORY θ_i
								LOAD FACTOR				TOTAL ΣP_{ui} (kips)	
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO ANY HORIZONTAL LOAD ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0.5	*ROOF L.L.* 0	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0.2		
18	13.0 ft	185 kips	0.25 in	1,513	0	288	1,591	1,815.6	0.0	0.0	318.2	2,133.8	**0.018**
17	13.0 ft	352 kips	0.37 in	2,821	1,008	288	3,187	3,385.2	504.0	0.0	637.4	4,526.6	**0.031**
16	13.0 ft	501 kips	0.45 in	4,129	2,016	288	4,783	4,954.8	1,008.0	0.0	956.6	6,919.4	**0.040**
15	13.0 ft	633 kips	0.47 in	5,437	3,024	288	6,379	6,524.4	1,512.0	0.0	1,275.8	9,312.2	**0.044**
14	13.0 ft	749 kips	0.52 in	6,745	4,032	288	7,975	8,094.0	2,016.0	0.0	1,595.0	11,705.0	**0.052**
13	13.0 ft	851 kips	0.55 in	8,053	5,040	288	9,571	9,663.6	2,520.0	0.0	1,914.2	14,097.8	**0.058**
12	13.0 ft	938 kips	0.55 in	9,361	6,048	288	11,167	11,233.2	3,024.0	0.0	2,233.4	16,490.6	**0.062**
11	13.0 ft	1,013 kips	0.57 in	10,669	7,056	288	12,763	12,802.8	3,528.0	0.0	2,552.6	18,883.4	**0.068**
10	13.0 ft	1,075 kips	0.55 in	11,977	8,064	288	14,359	14,372.4	4,032.0	0.0	2,871.8	21,276.2	**0.070**
9	13.0 ft	1,126 kips	0.55 in	13,285	9,072	288	15,955	15,942.0	4,536.0	0.0	3,191.0	23,669.0	**0.074**
8	13.0 ft	1,168 kips	0.57 in	14,593	10,080	288	17,551	17,511.6	5,040.0	0.0	3,510.2	26,061.8	**0.082**
7	13.0 ft	1,200 kips	0.57 in	15,901	11,088	288	19,147	19,081.2	5,544.0	0.0	3,829.4	28,454.6	**0.087**
6	13.0 ft	1,224 kips	0.58 in	17,209	12,096	288	20,743	20,650.8	6,048.0	0.0	4,148.6	30,847.4	**0.094**
5	13.0 ft	1,241 kips	0.59 in	18,517	13,104	288	22,339	22,220.4	6,552.0	0.0	4,467.8	33,240.2	**0.101**
4	13.0 ft	1,253 kips	0.59 in	19,825	14,112	288	23,935	23,790.0	7,056.0	0.0	4,787.0	35,633.0	**0.108**
3	13.0 ft	1,260 kips	0.58 in	21,133	15,120	288	25,531	25,359.6	7,560.0	0.0	5,106.2	38,025.8	**0.112**
2	13.0 ft	1,263 kips	0.54 in	22,441	16,128	288	27,127	26,929.2	8,064.0	0.0	5,425.4	40,418.6	**0.111**
1	13.0 ft	1,264 kips	0.32 in	23,749	17,136	288	28,723	28,498.8	8,568.0	0.0	5,744.6	42,811.4	**0.069**

GG Engineering	JOB NO. 18 - STORY BUILDINGS	BY SMG	DATE 4/22/05	SHEET NO. ___ OF ___
	CUSTOMER DESIGN 18A	CKD	DATE	

SUBJECT

STABILITY COEFFICIENT ALONG COLUMN X-AXIS, θ_x

MOMENT FRAME
MF A3 - G3

- ***LOAD COMBINATION:*** 1.2D + 0.5L + 1.0E
- ***DEFLECTION AMPLIFICATION FACTOR:*** C_d = 5.5

		DUE TO FORCES THAT CAUSE BENDING ALONG THE X-AXIS OF THE MOMENT FRAME COLUMNS						
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH_i	INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	TOTAL STORY SHEAR CAPACITY (OF ALL OF THE SEISMIC RESISTING MOMENT FRAMES)	RATIO OF SHEAR DEMAND / SHEAR CAPACITY PER STORY β	MAXIMUM ALLOWED STABILITY COEFFICIENT PER STORY θ_{i_max}	STABILITY COEFFICIENT PER STORY θ_i	***COMMENT***
18	13.0 ft	185 kips	0.25 in	13,494 kips	0.0137	0.250	0.018	***OK***
17	13.0 ft	352 kips	0.37 in	13,494 kips	0.0261	0.250	0.031	***OK***
16	13.0 ft	501 kips	0.45 in	13,494 kips	0.0371	0.250	0.040	***OK***
15	13.0 ft	633 kips	0.47 in	11,476 kips	0.0552	0.250	0.044	***OK***
14	13.0 ft	749 kips	0.52 in	11,476 kips	0.0653	0.250	0.052	***OK***
13	13.0 ft	851 kips	0.55 in	11,476 kips	0.0742	0.250	0.058	***OK***
12	13.0 ft	938 kips	0.55 in	14,019 kips	0.0669	0.250	0.062	***OK***
11	13.0 ft	1,013 kips	0.57 in	14,019 kips	0.0723	0.250	0.068	***OK***
10	13.0 ft	1,075 kips	0.55 in	12,867 kips	0.0835	0.250	0.070	***OK***
9	13.0 ft	1,126 kips	0.55 in	12,867 kips	0.0875	0.250	0.074	***OK***
8	13.0 ft	1,168 kips	0.57 in	12,867 kips	0.0908	0.250	0.082	***OK***
7	13.0 ft	1,200 kips	0.57 in	12,867 kips	0.0933	0.250	0.087	***OK***
6	13.0 ft	1,224 kips	0.58 in	12,867 kips	0.0951	0.250	0.094	***OK***
5	13.0 ft	1,241 kips	0.59 in	12,867 kips	0.0964	0.250	0.101	***OK***
4	13.0 ft	1,253 kips	0.59 in	12,867 kips	0.0974	0.250	0.108	***OK***
3	13.0 ft	1,260 kips	0.58 in	12,867 kips	0.0979	0.250	0.112	***OK***
2	13.0 ft	1,263 kips	0.54 in	12,867 kips	0.0982	0.250	0.111	***OK***
1	13.0 ft	1,264 kips	0.32 in	12,867 kips	0.0982	0.250	0.069	***OK***

GG Engineering	JOB NO. 18 - STORY BUILDINGS	BY SMG	DATE 4/22/05	SHEET NO. ___ OF ___
	CUSTOMER DESIGN 18A	CKD	DATE	

SUBJECT

STABILITY COEFFICIENT ALONG COLUMN Y-AXIS, θ_y

MOMENT FRAME
MF A3 - G3

- ***LOAD COMBINATION:*** 1.2D + 0.5L + 1.0E
- ***DEFLECTION AMPLIFICATION FACTOR:*** C_d = 5.5
- ***(SEISMIC) IMPORTANCE FACTOR*** I_E = 1.0
- ***MOMENT FRAME RESISTS WHAT % OF THE TOTAL SEISMIC SHEAR TO THE BUIDLING?*** 25%

		DUE TO FORCES THAT CAUSE BENDING ALONG THE Y-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					STABILITY COEFFICIENT PER STORY
								LOAD FACTOR					
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO ANY HORIZONTAL LOAD ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0.5	*ROOF L.L.* 0	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0.2	TOTAL ΣP_{ui} (kips)	θ_i
18	13.0 ft	185 kips	0.25 in	1,513	0	288	1,591	1,815.6	0.0	0.0	318.2	2,133.8	**0.018**
17	13.0 ft	352 kips	0.37 in	2,821	1,008	288	3,187	3,385.2	504.0	0.0	637.4	4,526.6	**0.031**
16	13.0 ft	501 kips	0.45 in	4,129	2,016	288	4,783	4,954.8	1,008.0	0.0	956.6	6,919.4	**0.040**
15	13.0 ft	633 kips	0.47 in	5,437	3,024	288	6,379	6,524.4	1,512.0	0.0	1,275.8	9,312.2	**0.044**
14	13.0 ft	749 kips	0.52 in	6,745	4,032	288	7,975	8,094.0	2,016.0	0.0	1,595.0	11,705.0	**0.052**
13	13.0 ft	851 kips	0.55 in	8,053	5,040	288	9,571	9,663.6	2,520.0	0.0	1,914.2	14,097.8	**0.058**
12	13.0 ft	938 kips	0.55 in	9,361	6,048	288	11,167	11,233.2	3,024.0	0.0	2,233.4	16,490.6	**0.062**
11	13.0 ft	1,013 kips	0.57 in	10,669	7,056	288	12,763	12,802.8	3,528.0	0.0	2,552.6	18,883.4	**0.068**
10	13.0 ft	1,075 kips	0.55 in	11,977	8,064	288	14,359	14,372.4	4,032.0	0.0	2,871.8	21,276.2	**0.070**
9	13.0 ft	1,126 kips	0.55 in	13,285	9,072	288	15,955	15,942.0	4,536.0	0.0	3,191.0	23,669.0	**0.074**
8	13.0 ft	1,168 kips	0.57 in	14,593	10,080	288	17,551	17,511.6	5,040.0	0.0	3,510.2	26,061.8	**0.082**
7	13.0 ft	1,200 kips	0.57 in	15,901	11,088	288	19,147	19,081.2	5,544.0	0.0	3,829.4	28,454.6	**0.087**
6	13.0 ft	1,224 kips	0.58 in	17,209	12,096	288	20,743	20,650.8	6,048.0	0.0	4,148.6	30,847.4	**0.094**
5	13.0 ft	1,241 kips	0.59 in	18,517	13,104	288	22,339	22,220.4	6,552.0	0.0	4,467.8	33,240.2	**0.101**
4	13.0 ft	1,253 kips	0.59 in	19,825	14,112	288	23,935	23,790.0	7,056.0	0.0	4,787.0	35,633.0	**0.108**
3	13.0 ft	1,260 kips	0.58 in	21,133	15,120	288	25,531	25,359.6	7,560.0	0.0	5,106.2	38,025.8	**0.112**
2	13.0 ft	1,263 kips	0.54 in	22,441	16,128	288	27,127	26,929.2	8,064.0	0.0	5,425.4	40,418.6	**0.111**
1	13.0 ft	1,264 kips	0.32 in	23,749	17,136	288	28,723	28,498.8	8,568.0	0.0	5,744.6	42,811.4	**0.069**

GG Engineering	JOB NO. 18 - STORY BUILDINGS	BY SMG	DATE 4/22/05	SHEET NO.
	CUSTOMER DESIGN 18A	CKD	DATE	___ OF ___

SUBJECT

STABILITY COEFFICIENT ALONG COLUMN Y-AXIS, θ_y

MOMENT FRAME
MF A3 - G3

- ***LOAD COMBINATION:*** 1.2D + 0.5L + 1.0E
- ***DEFLECTION AMPLIFICATION FACTOR:*** C_d = 5.5

		DUE TO FORCES THAT CAUSE BENDING ALONG THE Y-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL STORY SHEAR CAPACITY (OF ALL OF THE SEISMIC RESISTING MOMENT FRAMES)	RATIO OF SHEAR DEMAND / SHEAR CAPACITY PER STORY β	MAXIMUM ALLOWED STABILITY COEFFICIENT PER STORY θ_{i_max}	STABILITY COEFFICIENT PER STORY θ_i	***COMMENT***
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH_i	INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i					
18	13.0 ft	185 kips	0.25 in	13,494 kips	0.0137	0.250	0.018	***OK***
17	13.0 ft	352 kips	0.37 in	13,494 kips	0.0261	0.250	0.031	***OK***
16	13.0 ft	501 kips	0.45 in	13,494 kips	0.0371	0.250	0.040	***OK***
15	13.0 ft	633 kips	0.47 in	11,476 kips	0.0552	0.250	0.044	***OK***
14	13.0 ft	749 kips	0.52 in	11,476 kips	0.0653	0.250	0.052	***OK***
13	13.0 ft	851 kips	0.55 in	11,476 kips	0.0742	0.250	0.058	***OK***
12	13.0 ft	938 kips	0.55 in	14,019 kips	0.0669	0.250	0.062	***OK***
11	13.0 ft	1,013 kips	0.57 in	14,019 kips	0.0723	0.250	0.068	***OK***
10	13.0 ft	1,075 kips	0.55 in	12,867 kips	0.0835	0.250	0.070	***OK***
9	13.0 ft	1,126 kips	0.55 in	12,867 kips	0.0875	0.250	0.074	***OK***
8	13.0 ft	1,168 kips	0.57 in	12,867 kips	0.0908	0.250	0.082	***OK***
7	13.0 ft	1,200 kips	0.57 in	12,867 kips	0.0933	0.250	0.087	***OK***
6	13.0 ft	1,224 kips	0.58 in	12,867 kips	0.0951	0.250	0.094	***OK***
5	13.0 ft	1,241 kips	0.59 in	12,867 kips	0.0964	0.250	0.101	***OK***
4	13.0 ft	1,253 kips	0.59 in	12,867 kips	0.0974	0.250	0.108	***OK***
3	13.0 ft	1,260 kips	0.58 in	12,867 kips	0.0979	0.250	0.112	***OK***
2	13.0 ft	1,263 kips	0.54 in	12,867 kips	0.0982	0.250	0.111	***OK***
1	13.0 ft	1,264 kips	0.32 in	12,867 kips	0.0982	0.250	0.069	***OK***

COLUMN		MAXIMUM INTERACTION VALUE	CONTROLLING LOAD COMBINATION	BUILDING MAX. INTERACTION 0.9533	COMMENTS FOR: DESIGN 18A	
NAME	MEMBER SIZE				COLUMN STEEL AREA CHECK	COLUMN COMPACTNESS CHECK
A3-1	HSS 20 x 20 x 0.5	0.594658933	5		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
A3-2	HSS 20 x 20 x 0.5	0.373804609	5		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
A3-3	HSS 20 x 20 x 0.5	0.33042483	5		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
A3-4	HSS 20 x 20 x 0.5	0.309536747	5		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
A3-5	HSS 20 x 20 x 0.5	0.284451789	5		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
A3-6	HSS 20 x 20 x 0.5	0.258771446	5		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
A3-7	HSS 20 x 20 x 0.5	0.240976587	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
A3-8	HSS 20 x 20 x 0.5	0.227481653	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
A3-9	HSS 20 x 20 x 0.5	0.207664471	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
A3-10	HSS 20 x 20 x 0.5	0.196859786	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
A3-11	HSS 18 x 18 x 0.625	0.193843995	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
A3-12	HSS 18 x 18 x 0.625	0.162215341	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
A3-13	HSS 18 x 18 x 0.5	0.175196748	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
A3-14	HSS 18 x 18 x 0.5	0.163914559	5		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
A3-15	HSS 18 x 18 x 0.5	0.153007797	5		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
A3-16	HSS 16 x 16 x 0.75	0.116530489	5		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
A3-17	HSS 16 x 16 x 0.75	0.115659902	5		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
A3-18	HSS 16 x 16 x 0.75	0.145725562	5		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
B3-1	HSS 20 x 20 x 0.5	0.692789413	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
B3-2	HSS 20 x 20 x 0.5	0.62942709	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
B3-3	HSS 20 x 20 x 0.5	0.549216082	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
B3-4	HSS 20 x 20 x 0.5	0.479712622	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
B3-5	HSS 20 x 20 x 0.5	0.409142142	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
B3-6	HSS 20 x 20 x 0.5	0.354502968	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
B3-7	HSS 20 x 20 x 0.5	0.339081234	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
B3-8	HSS 20 x 20 x 0.5	0.322271199	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
B3-9	HSS 20 x 20 x 0.5	0.301122753	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
B3-10	HSS 20 x 20 x 0.5	0.290128354	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
B3-11	HSS 18 x 18 x 0.625	0.274777645	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
B3-12	HSS 18 x 18 x 0.625	0.251792746	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
B3-13	HSS 18 x 18 x 0.5	0.266421529	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
B3-14	HSS 18 x 18 x 0.5	0.233494757	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
B3-15	HSS 18 x 18 x 0.5	0.188476717	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
B3-16	HSS 16 x 16 x 0.75	0.133627385	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
B3-17	HSS 16 x 16 x 0.75	0.098396789	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
B3-18	HSS 16 x 16 x 0.75	0.051721636	2		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
C3-1	HSS 20 x 20 x 0.5	0.941074818	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
C3-2	HSS 20 x 20 x 0.5	0.850182351	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
C3-3	HSS 20 x 20 x 0.5	0.78439105	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
C3-4	HSS 20 x 20 x 0.5	0.721631249	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
C3-5	HSS 20 x 20 x 0.5	0.658366746	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
C3-6	HSS 20 x 20 x 0.5	0.591783731	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
C3-7	HSS 20 x 20 x 0.5	0.523095724	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
C3-8	HSS 20 x 20 x 0.5	0.503692204	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
C3-9	HSS 20 x 20 x 0.5	0.485266514	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
C3-10	HSS 20 x 20 x 0.5	0.469990515	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
C3-11	HSS 18 x 18 x 0.625	0.451545974	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
C3-12	HSS 18 x 18 x 0.625	0.424882189	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
C3-13	HSS 18 x 18 x 0.5	0.456158104	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
C3-14	HSS 18 x 18 x 0.5	0.410103349	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
C3-15	HSS 18 x 18 x 0.5	0.348320143	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
C3-16	HSS 16 x 16 x 0.75	0.246633281	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
C3-17	HSS 16 x 16 x 0.75	0.204161055	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
C3-18	HSS 16 x 16 x 0.75	0.145588189	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
D3-1	HSS 20 x 20 x 0.5	0.809253203	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
D3-2	HSS 20 x 20 x 0.5	0.738170708	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
D3-3	HSS 20 x 20 x 0.5	0.677189951	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
D3-4	HSS 20 x 20 x 0.5	0.616114581	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
D3-5	HSS 20 x 20 x 0.5	0.554304345	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
D3-6	HSS 20 x 20 x 0.5	0.489444317	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
D3-7	HSS 20 x 20 x 0.5	0.42312766	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
D3-8	HSS 20 x 20 x 0.5	0.389836519	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
D3-9	HSS 20 x 20 x 0.5	0.375966175	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
D3-10	HSS 20 x 20 x 0.5	0.363792984	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
D3-11	HSS 18 x 18 x 0.625	0.350335797	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
D3-12	HSS 18 x 18 x 0.625	0.329366521	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
D3-13	HSS 18 x 18 x 0.5	0.354170445	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
D3-14	HSS 18 x 18 x 0.5	0.318772948	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
D3-15	HSS 18 x 18 x 0.5	0.270584393	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
D3-16	HSS 16 x 16 x 0.75	0.191781499	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
D3-17	HSS 16 x 16 x 0.75	0.159611424	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
D3-18	HSS 16 x 16 x 0.75	0.113680092	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
E3-1	HSS 20 x 20 x 0.5	0.953298091	6	***<---CONTROLS!***	OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
E3-2	HSS 20 x 20 x 0.5	0.859360128	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
E3-3	HSS 20 x 20 x 0.5	0.797222972	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT

COLUMN		MAXIMUM INTERACTION VALUE	CONTROLLING LOAD COMBINATION	BUILDING MAX. INTERACTION 0.9533	COMMENTS FOR: DESIGN 18A	
NAME	MEMBER SIZE				COLUMN STEEL AREA CHECK	COLUMN COMPACTNESS CHECK
E3-4	HSS 20 x 20 x 0.5	0.736573072	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
E3-5	HSS 20 x 20 x 0.5	0.676138478	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
E3-6	HSS 20 x 20 x 0.5	0.611429679	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
E3-7	HSS 20 x 20 x 0.5	0.544526211	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
E3-8	HSS 20 x 20 x 0.5	0.52143647	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
E3-9	HSS 20 x 20 x 0.5	0.504923015	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
E3-10	HSS 20 x 20 x 0.5	0.48892498	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
E3-11	HSS 18 x 18 x 0.625	0.473619953	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
E3-12	HSS 18 x 18 x 0.625	0.445743746	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
E3-13	HSS 18 x 18 x 0.5	0.482016763	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
E3-14	HSS 18 x 18 x 0.5	0.436507659	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
E3-15	HSS 18 x 18 x 0.5	0.371278505	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
E3-16	HSS 16 x 16 x 0.75	0.267883867	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
E3-17	HSS 16 x 16 x 0.75	0.215710554	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
E3-18	HSS 16 x 16 x 0.75	0.237834975	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
F3-1	HSS 20 x 20 x 0.5	0.761188788	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
F3-2	HSS 20 x 20 x 0.5	0.6995934	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
F3-3	HSS 20 x 20 x 0.5	0.639053114	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
F3-4	HSS 20 x 20 x 0.5	0.580777245	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
F3-5	HSS 20 x 20 x 0.5	0.521778386	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
F3-6	HSS 20 x 20 x 0.5	0.459343121	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
F3-7	HSS 20 x 20 x 0.5	0.412795382	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
F3-8	HSS 20 x 20 x 0.5	0.404705547	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
F3-9	HSS 20 x 20 x 0.5	0.391953306	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
F3-10	HSS 20 x 20 x 0.5	0.380216466	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
F3-11	HSS 18 x 18 x 0.625	0.36752777	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
F3-12	HSS 18 x 18 x 0.625	0.348625047	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
F3-13	HSS 18 x 18 x 0.5	0.375641524	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
F3-14	HSS 18 x 18 x 0.5	0.339589176	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
F3-15	HSS 18 x 18 x 0.5	0.290330208	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
F3-16	HSS 16 x 16 x 0.75	0.207577034	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
F3-17	HSS 16 x 16 x 0.75	0.178206181	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
F3-18	HSS 16 x 16 x 0.75	0.118810269	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
G3-1	HSS 20 x 20 x 0.5	0.842296464	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
G3-2	HSS 20 x 20 x 0.5	0.669712906	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
G3-3	HSS 20 x 20 x 0.5	0.587508006	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
G3-4	HSS 20 x 20 x 0.5	0.521606456	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
G3-5	HSS 20 x 20 x 0.5	0.457777195	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
G3-6	HSS 20 x 20 x 0.5	0.393743032	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
G3-7	HSS 20 x 20 x 0.5	0.389434655	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
G3-8	HSS 20 x 20 x 0.5	0.383578762	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
G3-9	HSS 20 x 20 x 0.5	0.37110927	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
G3-10	HSS 20 x 20 x 0.5	0.359705242	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
G3-11	HSS 18 x 18 x 0.625	0.360751341	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
G3-12	HSS 18 x 18 x 0.625	0.336154856	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
G3-13	HSS 18 x 18 x 0.5	0.371645708	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
G3-14	HSS 18 x 18 x 0.5	0.348450617	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
G3-15	HSS 18 x 18 x 0.5	0.298065692	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
G3-16	HSS 16 x 16 x 0.75	0.228382363	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
G3-17	HSS 16 x 16 x 0.75	0.214728577	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
G3-18	HSS 16 x 16 x 0.75	0.191914138	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT

Appendix G

Building Design 18B Calculations

This appendix consists of the design calculations that were performed for building Design 18B which is the 18-story building that used high strength materials in the columns (F_{yc} = 80 ksi and f'_c = 16 ksi) and a relatively low column d/t ratio. The final RCFT column and wide flange girder sections are presented in Chapter 5. The linear elastic analysis consisted of taking the nominal loads that were generated in Appendix E and factoring them per the applicable LRFD load combination. The calculation for the stability coefficient, θ, and the moment magnification factor, B_2, were performed for each load combination that has lateral loads (wind and seismic load combinations #4, #5, and #6) and are included in this appendix. The maximum interaction value for each column is listed at the end of this appendix along with its respective load combination.

The load combinations that were used in this building design are listed below for reference.

1. $1.4D$
2. $1.2D + 1.6L + 0.5L_R$
3. $1.2D + 1.6L_R + f_1L$
4. $1.2D + 1.6L_R + 0.8W$
5. $1.2D + 1.6W + f_1L + 0.5L_R$
6. $1.2D + 1.0E + f_1L$

Where: $f_1 = 0.5$
$E = \rho Q_E + 0.2S_{DS}D'$
D' = seismic weight

GG Engineering	JOB NO. 18 - STORY BUILDINGS	BY SMG	DATE 4/22/05	SHEET NO.
	CUSTOMER DESIGN 18B	CKD	DATE	___ OF ___

SUBJECT

DESIGN PARAMETERS SUMMARY

MOMENT FRAME
MF A3 - G3

o DESIGN INPUTS:

- o TOAL NUMBER OF COLUMNS BEING ANALYZED 126
- o YIELD STRENGTH: HSS, F_y = 80 ksi
  - CONCRETE REINFORCEMENT, F_{yr} = 0 ksi
- o MODULUS OF ELASTICITY: HSS, E_s = 29,000 ksi
  - CONCRETE REINFORCEMENT, E_{cr} = 29,000 ksi
- o MINIMUM CONCRETE COMPRESSIVE STRENGTH f'_c = 16.0 ksi
- o CONCRETE DENSITY w = 145 lb/ft^3
- o CONCRETE REINFORCEMENT AREA, A_{sr} = 0.0 in^2
  - I_{xxr} = 0.0 in^4
  - I_{yyr} = 0.0 in^4
  - Z_{xxr} = 0.0 in^3
  - Z_{yyr} = 0.0 in^3
- o RESISTANCE FACTORS AXIAL COMPRESSION, ϕ_c = 0.75
  - FLEXURAL BENDING, ϕ_b = 0.90
- o SEISMIC PARAMETERS REDUNDANCY COEFFICIENT, ρ = 1.00
  - VERTICAL SEISMIC "FACTOR," $0.2S_{DS}$ = 0.20
  - ORTHOGONAL LOAD FACTOR ALONG Y-AXIS OF SHARED COLUMNS = 0.30
  - FACTOR TO ACCOUNT FOR *5% ACCIDENTAL TORSION* ("SIMPLIFIED APPROACH"...) = 0.025

GG Engineering	JOB NO. 18 - STORY BUILDINGS	BY SMG	DATE 4/22/05	SHEET NO. ___ OF ___
	CUSTOMER DESIGN 18B	CKD	DATE	

SUBJECT

B2 CALCULATION - *FOR BENDING ALONG THE X-AXIS OF THE COLUMN*

MOMENT FRAME
MF A3 - G3

o ***LOAD COMBINATION =*** 1.2D + 1.6Lr + 0.8W *(L.C. # 4)*

STORY NUMBER	LENGTH *(STORY HEIGHT)* L	DUE TO FORCES THAT CAUSE BENDING ALONG THE X-AXIS OF THE MOMENT FRAME COLUMNS: TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips): DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY — *LOAD FACTOR*: *D.L.* 1.2	*L.L.* 0	*ROOF L.L.* 1.6	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0	TOTAL ΣP_{ui} (kips)	**STORY $B_{2i\_X\text{-}AXIS}$**
18	13.0 ft	185 kips	0.23 in	1,513	0	288	1,591	1,815.6	0.0	460.8	0.0	2,276.4	**1.018**
17	13.0 ft	352 kips	0.4 in	2,821	1,008	288	3,187	3,385.2	0.0	460.8	0.0	3,846.0	**1.029**
16	13.0 ft	501 kips	0.53 in	4,129	2,016	288	4,783	4,954.8	0.0	460.8	0.0	5,415.6	**1.038**
15	13.0 ft	633 kips	0.5 in	5,437	3,024	288	6,379	6,524.4	0.0	460.8	0.0	6,985.2	**1.037**
14	13.0 ft	749 kips	0.53 in	6,745	4,032	288	7,975	8,094.0	0.0	460.8	0.0	8,554.8	**1.040**
13	13.0 ft	851 kips	0.54 in	8,053	5,040	288	9,571	9,663.6	0.0	460.8	0.0	10,124.4	**1.043**
12	13.0 ft	938 kips	0.57 in	9,361	6,048	288	11,167	11,233.2	0.0	460.8	0.0	11,694.0	**1.048**
11	13.0 ft	1,013 kips	0.59 in	10,669	7,056	288	12,763	12,802.8	0.0	460.8	0.0	13,263.6	**1.052**
10	13.0 ft	1,075 kips	0.57 in	11,977	8,064	288	14,359	14,372.4	0.0	460.8	0.0	14,833.2	**1.053**
9	13.0 ft	1,126 kips	0.58 in	13,285	9,072	288	15,955	15,942.0	0.0	460.8	0.0	16,402.8	**1.057**
8	13.0 ft	1,168 kips	0.59 in	14,593	10,080	288	17,551	17,511.6	0.0	460.8	0.0	17,972.4	**1.062**
7	13.0 ft	1,200 kips	0.61 in	15,901	11,088	288	19,147	19,081.2	0.0	460.8	0.0	19,542.0	**1.068**
6	13.0 ft	1,224 kips	0.61 in	17,209	12,096	288	20,743	20,650.8	0.0	460.8	0.0	21,111.6	**1.072**
5	13.0 ft	1,241 kips	0.62 in	18,517	13,104	288	22,339	22,220.4	0.0	460.8	0.0	22,681.2	**1.078**
4	13.0 ft	1,253 kips	0.62 in	19,825	14,112	288	23,935	23,790.0	0.0	460.8	0.0	24,250.8	**1.083**
3	13.0 ft	1,260 kips	0.61 in	21,133	15,120	288	25,531	25,359.6	0.0	460.8	0.0	25,820.4	**1.087**
2	13.0 ft	1,263 kips	0.6 in	22,441	16,128	288	27,127	26,929.2	0.0	460.8	0.0	27,390.0	**1.091**
1	13.0 ft	1,264 kips	0.39 in	23,749	17,136	288	28,723	28,498.8	0.0	460.8	0.0	28,959.6	**1.061**

GG Engineering	JOB NO. 18 - STORY BUILDINGS	BY SMG	DATE 4/22/05	SHEET NO. ___ OF ___
	CUSTOMER DESIGN 18B	CKD	DATE	

SUBJECT **B2 CALCULATION - *FOR BENDING ALONG THE Y-AXIS OF THE COLUMN*** **MOMENT FRAME MF A3 - G3**

o ***LOAD COMBINATION =*** **1.2D + 1.6Lr + 0.8W**

		DUE TO FORCES THAT CAUSE BENDING ALONG THE Y-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					**STORY $B_{2i\_Y\text{-}AXIS}$**
								LOAD FACTOR				TOTAL ΣP_{ui} (kips)	
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0	*ROOF L.L.* 1.6	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0		
18	13.0 ft	185 kips	0.23 in	1,513	0	288	1,591	1,815.6	0.0	460.8	0.0	2,276.4	**1.018**
17	13.0 ft	352 kips	0.4 in	2,821	1,008	288	3,187	3,385.2	0.0	460.8	0.0	3,846.0	**1.029**
16	13.0 ft	501 kips	0.53 in	4,129	2,016	288	4,783	4,954.8	0.0	460.8	0.0	5,415.6	**1.038**
15	13.0 ft	633 kips	0.5 in	5,437	3,024	288	6,379	6,524.4	0.0	460.8	0.0	6,985.2	**1.037**
14	13.0 ft	749 kips	0.53 in	6,745	4,032	288	7,975	8,094.0	0.0	460.8	0.0	8,554.8	**1.040**
13	13.0 ft	851 kips	0.54 in	8,053	5,040	288	9,571	9,663.6	0.0	460.8	0.0	10,124.4	**1.043**
12	13.0 ft	938 kips	0.57 in	9,361	6,048	288	11,167	11,233.2	0.0	460.8	0.0	11,694.0	**1.048**
11	13.0 ft	1,013 kips	0.59 in	10,669	7,056	288	12,763	12,802.8	0.0	460.8	0.0	13,263.6	**1.052**
10	13.0 ft	1,075 kips	0.57 in	11,977	8,064	288	14,359	14,372.4	0.0	460.8	0.0	14,833.2	**1.053**
9	13.0 ft	1,126 kips	0.58 in	13,285	9,072	288	15,955	15,942.0	0.0	460.8	0.0	16,402.8	**1.057**
8	13.0 ft	1,168 kips	0.59 in	14,593	10,080	288	17,551	17,511.6	0.0	460.8	0.0	17,972.4	**1.062**
7	13.0 ft	1,200 kips	0.61 in	15,901	11,088	288	19,147	19,081.2	0.0	460.8	0.0	19,542.0	**1.068**
6	13.0 ft	1,224 kips	0.61 in	17,209	12,096	288	20,743	20,650.8	0.0	460.8	0.0	21,111.6	**1.072**
5	13.0 ft	1,241 kips	0.62 in	18,517	13,104	288	22,339	22,220.4	0.0	460.8	0.0	22,681.2	**1.078**
4	13.0 ft	1,253 kips	0.62 in	19,825	14,112	288	23,935	23,790.0	0.0	460.8	0.0	24,250.8	**1.083**
3	13.0 ft	1,260 kips	0.61 in	21,133	15,120	288	25,531	25,359.6	0.0	460.8	0.0	25,820.4	**1.087**
2	13.0 ft	1,263 kips	0.6 in	22,441	16,128	288	27,127	26,929.2	0.0	460.8	0.0	27,390.0	**1.091**
1	13.0 ft	1,264 kips	0.39 in	23,749	17,136	288	28,723	28,498.8	0.0	460.8	0.0	28,959.6	**1.061**

GG Engineering	JOB NO. 18 - STORY BUILDINGS	BY SMG	DATE 4/22/05	SHEET NO.
	CUSTOMER DESIGN 18B	CKD	DATE	OF

SUBJECT

STABILITY COEFFICIENT ALONG COLUMN X-AXIS, θ_x

MOMENT FRAME
MF A3 - G3

- ***LOAD COMBINATION:*** 1.2D + 1.6Lr + 0.8W
- ***DEFLECTION AMPLIFICATION FACTOR:*** C_d = 5.5
- ***(SEISMIC) IMPORTANCE FACTOR*** I_E = 1.0
- ***MOMENT FRAME RESISTS WHAT % OF THE TOTAL SEISMIC SHEAR TO THE BUIDLING?*** 25%

		DUE TO FORCES THAT CAUSE BENDING ALONG THE X-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					STABILITY COEFFICIENT PER STORY θ_i
								LOAD FACTOR				TOTAL ΣP_{ui} (kips)	
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO ANY HORIZONTAL LOAD ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0	*ROOF L.L.* 1.6	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0		
18	13.0 ft	185 kips	0.23 in	1,513	0	288	1,591	1,815.6	0.0	460.8	0.0	2,276.4	**0.018**
17	13.0 ft	352 kips	0.4 in	2,821	1,008	288	3,187	3,385.2	0.0	460.8	0.0	3,846.0	**0.028**
16	13.0 ft	501 kips	0.53 in	4,129	2,016	288	4,783	4,954.8	0.0	460.8	0.0	5,415.6	**0.037**
15	13.0 ft	633 kips	0.5 in	5,437	3,024	288	6,379	6,524.4	0.0	460.8	0.0	6,985.2	**0.035**
14	13.0 ft	749 kips	0.53 in	6,745	4,032	288	7,975	8,094.0	0.0	460.8	0.0	8,554.8	**0.039**
13	13.0 ft	851 kips	0.54 in	8,053	5,040	288	9,571	9,663.6	0.0	460.8	0.0	10,124.4	**0.041**
12	13.0 ft	938 kips	0.57 in	9,361	6,048	288	11,167	11,233.2	0.0	460.8	0.0	11,694.0	**0.046**
11	13.0 ft	1,013 kips	0.59 in	10,669	7,056	288	12,763	12,802.8	0.0	460.8	0.0	13,263.6	**0.050**
10	13.0 ft	1,075 kips	0.57 in	11,977	8,064	288	14,359	14,372.4	0.0	460.8	0.0	14,833.2	**0.050**
9	13.0 ft	1,126 kips	0.58 in	13,285	9,072	288	15,955	15,942.0	0.0	460.8	0.0	16,402.8	**0.054**
8	13.0 ft	1,168 kips	0.59 in	14,593	10,080	288	17,551	17,511.6	0.0	460.8	0.0	17,972.4	**0.058**
7	13.0 ft	1,200 kips	0.61 in	15,901	11,088	288	19,147	19,081.2	0.0	460.8	0.0	19,542.0	**0.064**
6	13.0 ft	1,224 kips	0.61 in	17,209	12,096	288	20,743	20,650.8	0.0	460.8	0.0	21,111.6	**0.067**
5	13.0 ft	1,241 kips	0.62 in	18,517	13,104	288	22,339	22,220.4	0.0	460.8	0.0	22,681.2	**0.073**
4	13.0 ft	1,253 kips	0.62 in	19,825	14,112	288	23,935	23,790.0	0.0	460.8	0.0	24,250.8	**0.077**
3	13.0 ft	1,260 kips	0.61 in	21,133	15,120	288	25,531	25,359.6	0.0	460.8	0.0	25,820.4	**0.080**
2	13.0 ft	1,263 kips	0.6 in	22,441	16,128	288	27,127	26,929.2	0.0	460.8	0.0	27,390.0	**0.083**
1	13.0 ft	1,264 kips	0.39 in	23,749	17,136	288	28,723	28,498.8	0.0	460.8	0.0	28,959.6	**0.057**

GG Engineering	JOB NO. 18 - STORY BUILDINGS	BY SMG	DATE 4/22/05	SHEET NO. ___ OF ___
	CUSTOMER DESIGN 18B	CKD	DATE	

SUBJECT

STABILITY COEFFICIENT ALONG COLUMN Y-AXIS, θ_y

MOMENT FRAME
MF A3 - G3

- ***LOAD COMBINATION:*** 1.2D + 1.6Lr + 0.8W
- ***DEFLECTION AMPLIFICATION FACTOR:*** C_d = 5.5
- ***(SEISMIC) IMPORTANCE FACTOR*** I_E = 1.0
- ***MOMENT FRAME RESISTS WHAT % OF THE TOTAL SEISMIC SHEAR TO THE BUIDLING?*** 25%

		DUE TO FORCES THAT CAUSE BENDING ALONG THE Y-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					STABILITY COEFFICIENT PER STORY θ_i
								LOAD FACTOR					
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO ANY HORIZONTAL LOAD ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0	*ROOF L.L.* 1.6	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0	TOTAL ΣP_{ui} (kips)	
18	13.0 ft	185 kips	0.23 in	1,513	0	288	1,591	1,815.6	0.0	460.8	0.0	2,276.4	**0.018**
17	13.0 ft	352 kips	0.4 in	2,821	1,008	288	3,187	3,385.2	0.0	460.8	0.0	3,846.0	**0.028**
16	13.0 ft	501 kips	0.53 in	4,129	2,016	288	4,783	4,954.8	0.0	460.8	0.0	5,415.6	**0.037**
15	13.0 ft	633 kips	0.5 in	5,437	3,024	288	6,379	6,524.4	0.0	460.8	0.0	6,985.2	**0.035**
14	13.0 ft	749 kips	0.53 in	6,745	4,032	288	7,975	8,094.0	0.0	460.8	0.0	8,554.8	**0.039**
13	13.0 ft	851 kips	0.54 in	8,053	5,040	288	9,571	9,663.6	0.0	460.8	0.0	10,124.4	**0.041**
12	13.0 ft	938 kips	0.57 in	9,361	6,048	288	11,167	11,233.2	0.0	460.8	0.0	11,694.0	**0.046**
11	13.0 ft	1,013 kips	0.59 in	10,669	7,056	288	12,763	12,802.8	0.0	460.8	0.0	13,263.6	**0.050**
10	13.0 ft	1,075 kips	0.57 in	11,977	8,064	288	14,359	14,372.4	0.0	460.8	0.0	14,833.2	**0.050**
9	13.0 ft	1,126 kips	0.58 in	13,285	9,072	288	15,955	15,942.0	0.0	460.8	0.0	16,402.8	**0.054**
8	13.0 ft	1,168 kips	0.59 in	14,593	10,080	288	17,551	17,511.6	0.0	460.8	0.0	17,972.4	**0.058**
7	13.0 ft	1,200 kips	0.61 in	15,901	11,088	288	19,147	19,081.2	0.0	460.8	0.0	19,542.0	**0.064**
6	13.0 ft	1,224 kips	0.61 in	17,209	12,096	288	20,743	20,650.8	0.0	460.8	0.0	21,111.6	**0.067**
5	13.0 ft	1,241 kips	0.62 in	18,517	13,104	288	22,339	22,220.4	0.0	460.8	0.0	22,681.2	**0.073**
4	13.0 ft	1,253 kips	0.62 in	19,825	14,112	288	23,935	23,790.0	0.0	460.8	0.0	24,250.8	**0.077**
3	13.0 ft	1,260 kips	0.61 in	21,133	15,120	288	25,531	25,359.6	0.0	460.8	0.0	25,820.4	**0.080**
2	13.0 ft	1,263 kips	0.6 in	22,441	16,128	288	27,127	26,929.2	0.0	460.8	0.0	27,390.0	**0.083**
1	13.0 ft	1,264 kips	0.39 in	23,749	17,136	288	28,723	28,498.8	0.0	460.8	0.0	28,959.6	**0.057**

GG Engineering	JOB NO. 18 - STORY BUILDINGS	BY SMG	DATE 4/22/05	SHEET NO. ___ OF ___
	CUSTOMER DESIGN 18B	CKD	DATE	

SUBJECT

B2 CALCULATION - *FOR BENDING ALONG THE X-AXIS OF THE COLUMN*

MOMENT FRAME *MF A3 - G3*

o ***LOAD COMBINATION =*** 1.2D + 0.5L + 0.5Lr + 1.6W *(L.C. # 5)*

STORY NUMBER	LENGTH *(STORY HEIGHT)* L	DUE TO FORCES THAT CAUSE BENDING ALONG THE X-AXIS OF THE MOMENT FRAME COLUMNS: TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips): DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY — *LOAD FACTOR*: *D.L.* 1.2	*L.L.* 0.5	*ROOF L.L.* 0.5	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0	TOTAL ΣP_{ui} (kips)	**STORY $B_{2i\_X\text{-}AXIS}$**
18	13.0 ft	185 kips	0.23 in	1,513	0	288	1,591	1,815.6	0.0	144.0	0.0	1,959.6	**1.016**
17	13.0 ft	352 kips	0.4 in	2,821	1,008	288	3,187	3,385.2	504.0	144.0	0.0	4,033.2	**1.030**
16	13.0 ft	501 kips	0.53 in	4,129	2,016	288	4,783	4,954.8	1,008.0	144.0	0.0	6,106.8	**1.043**
15	13.0 ft	633 kips	0.5 in	5,437	3,024	288	6,379	6,524.4	1,512.0	144.0	0.0	8,180.4	**1.043**
14	13.0 ft	749 kips	0.53 in	6,745	4,032	288	7,975	8,094.0	2,016.0	144.0	0.0	10,254.0	**1.049**
13	13.0 ft	851 kips	0.54 in	8,053	5,040	288	9,571	9,663.6	2,520.0	144.0	0.0	12,327.6	**1.053**
12	13.0 ft	938 kips	0.57 in	9,361	6,048	288	11,167	11,233.2	3,024.0	144.0	0.0	14,401.2	**1.059**
11	13.0 ft	1,013 kips	0.59 in	10,669	7,056	288	12,763	12,802.8	3,528.0	144.0	0.0	16,474.8	**1.066**
10	13.0 ft	1,075 kips	0.57 in	11,977	8,064	288	14,359	14,372.4	4,032.0	144.0	0.0	18,548.4	**1.067**
9	13.0 ft	1,126 kips	0.58 in	13,285	9,072	288	15,955	15,942.0	4,536.0	144.0	0.0	20,622.0	**1.073**
8	13.0 ft	1,168 kips	0.59 in	14,593	10,080	288	17,551	17,511.6	5,040.0	144.0	0.0	22,695.6	**1.079**
7	13.0 ft	1,200 kips	0.61 in	15,901	11,088	288	19,147	19,081.2	5,544.0	144.0	0.0	24,769.2	**1.088**
6	13.0 ft	1,224 kips	0.61 in	17,209	12,096	288	20,743	20,650.8	6,048.0	144.0	0.0	26,842.8	**1.094**
5	13.0 ft	1,241 kips	0.62 in	18,517	13,104	288	22,339	22,220.4	6,552.0	144.0	0.0	28,916.4	**1.102**
4	13.0 ft	1,253 kips	0.62 in	19,825	14,112	288	23,935	23,790.0	7,056.0	144.0	0.0	30,990.0	**1.109**
3	13.0 ft	1,260 kips	0.61 in	21,133	15,120	288	25,531	25,359.6	7,560.0	144.0	0.0	33,063.6	**1.114**
2	13.0 ft	1,263 kips	0.6 in	22,441	16,128	288	27,127	26,929.2	8,064.0	144.0	0.0	35,137.2	**1.120**
1	13.0 ft	1,264 kips	0.39 in	23,749	17,136	288	28,723	28,498.8	8,568.0	144.0	0.0	37,210.8	**1.079**

GG Engineering	JOB NO. 18 - STORY BUILDINGS	BY SMG	DATE 4/22/05	SHEET NO. ___ OF ___
	CUSTOMER DESIGN 18B	CKD	DATE	

SUBJECT

B2 CALCULATION - *FOR BENDING ALONG THE Y-AXIS OF THE COLUMN*

MOMENT FRAME MF A3 - G3

o ***LOAD COMBINATION =*** **1.2D + 0.5L + 0.5Lr + 1.6W**

		DUE TO FORCES THAT CAUSE BENDING ALONG THE Y-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					
								LOAD FACTOR					
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0.5	*ROOF L.L.* 0.5	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0	TOTAL ΣP_{ui} (kips)	**STORY $B_{2i\_Y\text{-}AXIS}$**
18	13.0 ft	185 kips	0.23 in	1,513	0	288	1,591	1,815.6	0.0	144.0	0.0	1,959.6	**1.016**
17	13.0 ft	352 kips	0.4 in	2,821	1,008	288	3,187	3,385.2	504.0	144.0	0.0	4,033.2	**1.030**
16	13.0 ft	501 kips	0.53 in	4,129	2,016	288	4,783	4,954.8	1,008.0	144.0	0.0	6,106.8	**1.043**
15	13.0 ft	633 kips	0.5 in	5,437	3,024	288	6,379	6,524.4	1,512.0	144.0	0.0	8,180.4	**1.043**
14	13.0 ft	749 kips	0.53 in	6,745	4,032	288	7,975	8,094.0	2,016.0	144.0	0.0	10,254.0	**1.049**
13	13.0 ft	851 kips	0.54 in	8,053	5,040	288	9,571	9,663.6	2,520.0	144.0	0.0	12,327.6	**1.053**
12	13.0 ft	938 kips	0.57 in	9,361	6,048	288	11,167	11,233.2	3,024.0	144.0	0.0	14,401.2	**1.059**
11	13.0 ft	1,013 kips	0.59 in	10,669	7,056	288	12,763	12,802.8	3,528.0	144.0	0.0	16,474.8	**1.066**
10	13.0 ft	1,075 kips	0.57 in	11,977	8,064	288	14,359	14,372.4	4,032.0	144.0	0.0	18,548.4	**1.067**
9	13.0 ft	1,126 kips	0.58 in	13,285	9,072	288	15,955	15,942.0	4,536.0	144.0	0.0	20,622.0	**1.073**
8	13.0 ft	1,168 kips	0.59 in	14,593	10,080	288	17,551	17,511.6	5,040.0	144.0	0.0	22,695.6	**1.079**
7	13.0 ft	1,200 kips	0.61 in	15,901	11,088	288	19,147	19,081.2	5,544.0	144.0	0.0	24,769.2	**1.088**
6	13.0 ft	1,224 kips	0.61 in	17,209	12,096	288	20,743	20,650.8	6,048.0	144.0	0.0	26,842.8	**1.094**
5	13.0 ft	1,241 kips	0.62 in	18,517	13,104	288	22,339	22,220.4	6,552.0	144.0	0.0	28,916.4	**1.102**
4	13.0 ft	1,253 kips	0.62 in	19,825	14,112	288	23,935	23,790.0	7,056.0	144.0	0.0	30,990.0	**1.109**
3	13.0 ft	1,260 kips	0.61 in	21,133	15,120	288	25,531	25,359.6	7,560.0	144.0	0.0	33,063.6	**1.114**
2	13.0 ft	1,263 kips	0.6 in	22,441	16,128	288	27,127	26,929.2	8,064.0	144.0	0.0	35,137.2	**1.120**
1	13.0 ft	1,264 kips	0.39 in	23,749	17,136	288	28,723	28,498.8	8,568.0	144.0	0.0	37,210.8	**1.079**

GG Engineering	JOB NO. 18 - STORY BUILDINGS	BY SMG	DATE 4/22/05	SHEET NO. ___ OF ___
	CUSTOMER DESIGN 18B	CKD	DATE	

SUBJECT

STABILITY COEFFICIENT ALONG COLUMN X-AXIS, θ_x

MOMENT FRAME
MF A3 - G3

- ***LOAD COMBINATION:*** 1.2D + 0.5L + 0.5Lr + 1.6W
- ***DEFLECTION AMPLIFICATION FACTOR:*** C_d = 5.5
- ***(SEISMIC) IMPORTANCE FACTOR*** I_E = 1.0
- ***MOMENT FRAME RESISTS WHAT % OF THE TOTAL SEISMIC SHEAR TO THE BUIDLING?*** 25%

		DUE TO FORCES THAT CAUSE BENDING ALONG THE X-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					STABILITY COEFFICIENT PER STORY
								LOAD FACTOR					
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO ANY HORIZONTAL LOAD ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0.5	*ROOF L.L.* 0.5	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0	TOTAL ΣP_{ui} (kips)	θ_i
18	13.0 ft	185 kips	0.23 in	1,513	0	288	1,591	1,815.6	0.0	144.0	0.0	1,959.6	**0.016**
17	13.0 ft	352 kips	0.4 in	2,821	1,008	288	3,187	3,385.2	504.0	144.0	0.0	4,033.2	**0.029**
16	13.0 ft	501 kips	0.53 in	4,129	2,016	288	4,783	4,954.8	1,008.0	144.0	0.0	6,106.8	**0.041**
15	13.0 ft	633 kips	0.5 in	5,437	3,024	288	6,379	6,524.4	1,512.0	144.0	0.0	8,180.4	**0.041**
14	13.0 ft	749 kips	0.53 in	6,745	4,032	288	7,975	8,094.0	2,016.0	144.0	0.0	10,254.0	**0.047**
13	13.0 ft	851 kips	0.54 in	8,053	5,040	288	9,571	9,663.6	2,520.0	144.0	0.0	12,327.6	**0.050**
12	13.0 ft	938 kips	0.57 in	9,361	6,048	288	11,167	11,233.2	3,024.0	144.0	0.0	14,401.2	**0.056**
11	13.0 ft	1,013 kips	0.59 in	10,669	7,056	288	12,763	12,802.8	3,528.0	144.0	0.0	16,474.8	**0.062**
10	13.0 ft	1,075 kips	0.57 in	11,977	8,064	288	14,359	14,372.4	4,032.0	144.0	0.0	18,548.4	**0.063**
9	13.0 ft	1,126 kips	0.58 in	13,285	9,072	288	15,955	15,942.0	4,536.0	144.0	0.0	20,622.0	**0.068**
8	13.0 ft	1,168 kips	0.59 in	14,593	10,080	288	17,551	17,511.6	5,040.0	144.0	0.0	22,695.6	**0.073**
7	13.0 ft	1,200 kips	0.61 in	15,901	11,088	288	19,147	19,081.2	5,544.0	144.0	0.0	24,769.2	**0.081**
6	13.0 ft	1,224 kips	0.61 in	17,209	12,096	288	20,743	20,650.8	6,048.0	144.0	0.0	26,842.8	**0.086**
5	13.0 ft	1,241 kips	0.62 in	18,517	13,104	288	22,339	22,220.4	6,552.0	144.0	0.0	28,916.4	**0.093**
4	13.0 ft	1,253 kips	0.62 in	19,825	14,112	288	23,935	23,790.0	7,056.0	144.0	0.0	30,990.0	**0.098**
3	13.0 ft	1,260 kips	0.61 in	21,133	15,120	288	25,531	25,359.6	7,560.0	144.0	0.0	33,063.6	**0.103**
2	13.0 ft	1,263 kips	0.6 in	22,441	16,128	288	27,127	26,929.2	8,064.0	144.0	0.0	35,137.2	**0.107**
1	13.0 ft	1,264 kips	0.39 in	23,749	17,136	288	28,723	28,498.8	8,568.0	144.0	0.0	37,210.8	**0.074**

GG Engineering	JOB NO. 18 - STORY BUILDINGS	BY SMG	DATE 4/22/05	SHEET NO. ___ OF ___
	CUSTOMER DESIGN 18B	CKD	DATE	

SUBJECT

STABILITY COEFFICIENT ALONG COLUMN Y-AXIS, θ_y

MOMENT FRAME
MF A3 - G3

- ***LOAD COMBINATION:*** 1.2D + 0.5L + 0.5Lr + 1.6W
- ***DEFLECTION AMPLIFICATION FACTOR:*** C_d = 5.5
- ***(SEISMIC) IMPORTANCE FACTOR*** I_E = 1.0
- ***MOMENT FRAME RESISTS WHAT % OF THE TOTAL SEISMIC SHEAR TO THE BUIDLING?*** 25%

		DUE TO FORCES THAT CAUSE BENDING ALONG THE Y-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					STABILITY COEFFICIENT PER STORY θ_i
								LOAD FACTOR				TOTAL ΣP_{ui} (kips)	
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO ANY HORIZONTAL LOAD ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0.5	*ROOF L.L.* 0.5	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0		
18	13.0 ft	185 kips	0.23 in	1,513	0	288	1,591	1,815.6	0.0	144.0	0.0	1,959.6	**0.016**
17	13.0 ft	352 kips	0.4 in	2,821	1,008	288	3,187	3,385.2	504.0	144.0	0.0	4,033.2	**0.029**
16	13.0 ft	501 kips	0.53 in	4,129	2,016	288	4,783	4,954.8	1,008.0	144.0	0.0	6,106.8	**0.041**
15	13.0 ft	633 kips	0.5 in	5,437	3,024	288	6,379	6,524.4	1,512.0	144.0	0.0	8,180.4	**0.041**
14	13.0 ft	749 kips	0.53 in	6,745	4,032	288	7,975	8,094.0	2,016.0	144.0	0.0	10,254.0	**0.047**
13	13.0 ft	851 kips	0.54 in	8,053	5,040	288	9,571	9,663.6	2,520.0	144.0	0.0	12,327.6	**0.050**
12	13.0 ft	938 kips	0.57 in	9,361	6,048	288	11,167	11,233.2	3,024.0	144.0	0.0	14,401.2	**0.056**
11	13.0 ft	1,013 kips	0.59 in	10,669	7,056	288	12,763	12,802.8	3,528.0	144.0	0.0	16,474.8	**0.062**
10	13.0 ft	1,075 kips	0.57 in	11,977	8,064	288	14,359	14,372.4	4,032.0	144.0	0.0	18,548.4	**0.063**
9	13.0 ft	1,126 kips	0.58 in	13,285	9,072	288	15,955	15,942.0	4,536.0	144.0	0.0	20,622.0	**0.068**
8	13.0 ft	1,168 kips	0.59 in	14,593	10,080	288	17,551	17,511.6	5,040.0	144.0	0.0	22,695.6	**0.073**
7	13.0 ft	1,200 kips	0.61 in	15,901	11,088	288	19,147	19,081.2	5,544.0	144.0	0.0	24,769.2	**0.081**
6	13.0 ft	1,224 kips	0.61 in	17,209	12,096	288	20,743	20,650.8	6,048.0	144.0	0.0	26,842.8	**0.086**
5	13.0 ft	1,241 kips	0.62 in	18,517	13,104	288	22,339	22,220.4	6,552.0	144.0	0.0	28,916.4	**0.093**
4	13.0 ft	1,253 kips	0.62 in	19,825	14,112	288	23,935	23,790.0	7,056.0	144.0	0.0	30,990.0	**0.098**
3	13.0 ft	1,260 kips	0.61 in	21,133	15,120	288	25,531	25,359.6	7,560.0	144.0	0.0	33,063.6	**0.103**
2	13.0 ft	1,263 kips	0.6 in	22,441	16,128	288	27,127	26,929.2	8,064.0	144.0	0.0	35,137.2	**0.107**
1	13.0 ft	1,264 kips	0.39 in	23,749	17,136	288	28,723	28,498.8	8,568.0	144.0	0.0	37,210.8	**0.074**

GG Engineering	JOB NO. 18 - STORY BUILDINGS	BY SMG	DATE 4/22/05	SHEET NO.
	CUSTOMER DESIGN 18B	CKD	DATE	___ OF ___

SUBJECT

B2 CALCULATION - *FOR BENDING ALONG THE X-AXIS OF THE COLUMN*

MOMENT FRAME
MF A3 - G3

o ***LOAD COMBINATION =*** **1.2D + 0.5L + 1.0E** *(L.C. # 6)*

		DUE TO FORCES THAT CAUSE BENDING ALONG THE X-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					
								LOAD FACTOR					
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0.5	*ROOF L.L.* 0	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0.2	TOTAL ΣP_{ui} (kips)	**STORY $B_{2i\_X\text{-}AXIS}$**
18	13.0 ft	185 kips	0.23 in	1,513	0	288	1,591	1,815.6	0.0	0.0	318.2	2,133.8	**1.017**
17	13.0 ft	352 kips	0.4 in	2,821	1,008	288	3,187	3,385.2	504.0	0.0	637.4	4,526.6	**1.034**
16	13.0 ft	501 kips	0.53 in	4,129	2,016	288	4,783	4,954.8	1,008.0	0.0	956.6	6,919.4	**1.049**
15	13.0 ft	633 kips	0.5 in	5,437	3,024	288	6,379	6,524.4	1,512.0	0.0	1,275.8	9,312.2	**1.049**
14	13.0 ft	749 kips	0.53 in	6,745	4,032	288	7,975	8,094.0	2,016.0	0.0	1,595.0	11,705.0	**1.056**
13	13.0 ft	851 kips	0.54 in	8,053	5,040	288	9,571	9,663.6	2,520.0	0.0	1,914.2	14,097.8	**1.061**
12	13.0 ft	938 kips	0.57 in	9,361	6,048	288	11,167	11,233.2	3,024.0	0.0	2,233.4	16,490.6	**1.069**
11	13.0 ft	1,013 kips	0.59 in	10,669	7,056	288	12,763	12,802.8	3,528.0	0.0	2,552.6	18,883.4	**1.076**
10	13.0 ft	1,075 kips	0.57 in	11,977	8,064	288	14,359	14,372.4	4,032.0	0.0	2,871.8	21,276.2	**1.078**
9	13.0 ft	1,126 kips	0.58 in	13,285	9,072	288	15,955	15,942.0	4,536.0	0.0	3,191.0	23,669.0	**1.085**
8	13.0 ft	1,168 kips	0.59 in	14,593	10,080	288	17,551	17,511.6	5,040.0	0.0	3,510.2	26,061.8	**1.092**
7	13.0 ft	1,200 kips	0.61 in	15,901	11,088	288	19,147	19,081.2	5,544.0	0.0	3,829.4	28,454.6	**1.102**
6	13.0 ft	1,224 kips	0.61 in	17,209	12,096	288	20,743	20,650.8	6,048.0	0.0	4,148.6	30,847.4	**1.109**
5	13.0 ft	1,241 kips	0.62 in	18,517	13,104	288	22,339	22,220.4	6,552.0	0.0	4,467.8	33,240.2	**1.119**
4	13.0 ft	1,253 kips	0.62 in	19,825	14,112	288	23,935	23,790.0	7,056.0	0.0	4,787.0	35,633.0	**1.127**
3	13.0 ft	1,260 kips	0.61 in	21,133	15,120	288	25,531	25,359.6	7,560.0	0.0	5,106.2	38,025.8	**1.134**
2	13.0 ft	1,263 kips	0.6 in	22,441	16,128	288	27,127	26,929.2	8,064.0	0.0	5,425.4	40,418.6	**1.140**
1	13.0 ft	1,264 kips	0.39 in	23,749	17,136	288	28,723	28,498.8	8,568.0	0.0	5,744.6	42,811.4	**1.093**

GG Engineering	JOB NO. 18 - STORY BUILDINGS	BY SMG	DATE 4/22/05	SHEET NO.
	CUSTOMER DESIGN 18B	CKD	DATE	___ OF ___

SUBJECT **B2 CALCULATION - *FOR BENDING ALONG THE Y-AXIS OF THE COLUMN*** **MOMENT FRAME MF A3 - G3**

o ***LOAD COMBINATION =*** **1.2D + 0.5L + 1.0E**

		DUE TO FORCES THAT CAUSE BENDING ALONG THE Y-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					**STORY $B_{2i\_Y\text{-}AXIS}$**
								LOAD FACTOR					
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0.5	*ROOF L.L.* 0	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0.2	TOTAL ΣP_{ui} (kips)	
18	13.0 ft	185 kips	0.23 in	1,513	0	288	1,591	1,815.6	0.0	0.0	318.2	2,133.8	**1.017**
17	13.0 ft	352 kips	0.4 in	2,821	1,008	288	3,187	3,385.2	504.0	0.0	637.4	4,526.6	**1.034**
16	13.0 ft	501 kips	0.53 in	4,129	2,016	288	4,783	4,954.8	1,008.0	0.0	956.6	6,919.4	**1.049**
15	13.0 ft	633 kips	0.5 in	5,437	3,024	288	6,379	6,524.4	1,512.0	0.0	1,275.8	9,312.2	**1.049**
14	13.0 ft	749 kips	0.53 in	6,745	4,032	288	7,975	8,094.0	2,016.0	0.0	1,595.0	11,705.0	**1.056**
13	13.0 ft	851 kips	0.54 in	8,053	5,040	288	9,571	9,663.6	2,520.0	0.0	1,914.2	14,097.8	**1.061**
12	13.0 ft	938 kips	0.57 in	9,361	6,048	288	11,167	11,233.2	3,024.0	0.0	2,233.4	16,490.6	**1.069**
11	13.0 ft	1,013 kips	0.59 in	10,669	7,056	288	12,763	12,802.8	3,528.0	0.0	2,552.6	18,883.4	**1.076**
10	13.0 ft	1,075 kips	0.57 in	11,977	8,064	288	14,359	14,372.4	4,032.0	0.0	2,871.8	21,276.2	**1.078**
9	13.0 ft	1,126 kips	0.58 in	13,285	9,072	288	15,955	15,942.0	4,536.0	0.0	3,191.0	23,669.0	**1.085**
8	13.0 ft	1,168 kips	0.59 in	14,593	10,080	288	17,551	17,511.6	5,040.0	0.0	3,510.2	26,061.8	**1.092**
7	13.0 ft	1,200 kips	0.61 in	15,901	11,088	288	19,147	19,081.2	5,544.0	0.0	3,829.4	28,454.6	**1.102**
6	13.0 ft	1,224 kips	0.61 in	17,209	12,096	288	20,743	20,650.8	6,048.0	0.0	4,148.6	30,847.4	**1.109**
5	13.0 ft	1,241 kips	0.62 in	18,517	13,104	288	22,339	22,220.4	6,552.0	0.0	4,467.8	33,240.2	**1.119**
4	13.0 ft	1,253 kips	0.62 in	19,825	14,112	288	23,935	23,790.0	7,056.0	0.0	4,787.0	35,633.0	**1.127**
3	13.0 ft	1,260 kips	0.61 in	21,133	15,120	288	25,531	25,359.6	7,560.0	0.0	5,106.2	38,025.8	**1.134**
2	13.0 ft	1,263 kips	0.6 in	22,441	16,128	288	27,127	26,929.2	8,064.0	0.0	5,425.4	40,418.6	**1.140**
1	13.0 ft	1,264 kips	0.39 in	23,749	17,136	288	28,723	28,498.8	8,568.0	0.0	5,744.6	42,811.4	**1.093**

GG Engineering	JOB NO. 18 - STORY BUILDINGS	BY SMG	DATE 4/22/05	SHEET NO. ___ OF ___
	CUSTOMER DESIGN 18B	CKD	DATE	

SUBJECT

STABILITY COEFFICIENT ALONG COLUMN X-AXIS, θ_x

MOMENT FRAME
MF A3 - G3

- ***LOAD COMBINATION:*** 1.2D + 0.5L + 1.0E
- ***DEFLECTION AMPLIFICATION FACTOR:*** C_d = 5.5
- ***(SEISMIC) IMPORTANCE FACTOR*** I_E = 1.0
- ***MOMENT FRAME RESISTS WHAT % OF THE TOTAL SEISMIC SHEAR TO THE BUIDLING?*** 25%

		DUE TO FORCES THAT CAUSE BENDING ALONG THE X-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					STABILITY COEFFICIENT PER STORY θ_i
								LOAD FACTOR					
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO ANY HORIZONTAL LOAD ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0.5	*ROOF L.L.* 0	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0.2	TOTAL ΣP_{ui} (kips)	
18	13.0 ft	185 kips	0.23 in	1,513	0	288	1,591	1,815.6	0.0	0.0	318.2	2,133.8	**0.017**
17	13.0 ft	352 kips	0.4 in	2,821	1,008	288	3,187	3,385.2	504.0	0.0	637.4	4,526.6	**0.033**
16	13.0 ft	501 kips	0.53 in	4,129	2,016	288	4,783	4,954.8	1,008.0	0.0	956.6	6,919.4	**0.047**
15	13.0 ft	633 kips	0.5 in	5,437	3,024	288	6,379	6,524.4	1,512.0	0.0	1,275.8	9,312.2	**0.047**
14	13.0 ft	749 kips	0.53 in	6,745	4,032	288	7,975	8,094.0	2,016.0	0.0	1,595.0	11,705.0	**0.053**
13	13.0 ft	851 kips	0.54 in	8,053	5,040	288	9,571	9,663.6	2,520.0	0.0	1,914.2	14,097.8	**0.057**
12	13.0 ft	938 kips	0.57 in	9,361	6,048	288	11,167	11,233.2	3,024.0	0.0	2,233.4	16,490.6	**0.064**
11	13.0 ft	1,013 kips	0.59 in	10,669	7,056	288	12,763	12,802.8	3,528.0	0.0	2,552.6	18,883.4	**0.071**
10	13.0 ft	1,075 kips	0.57 in	11,977	8,064	288	14,359	14,372.4	4,032.0	0.0	2,871.8	21,276.2	**0.072**
9	13.0 ft	1,126 kips	0.58 in	13,285	9,072	288	15,955	15,942.0	4,536.0	0.0	3,191.0	23,669.0	**0.078**
8	13.0 ft	1,168 kips	0.59 in	14,593	10,080	288	17,551	17,511.6	5,040.0	0.0	3,510.2	26,061.8	**0.084**
7	13.0 ft	1,200 kips	0.61 in	15,901	11,088	288	19,147	19,081.2	5,544.0	0.0	3,829.4	28,454.6	**0.093**
6	13.0 ft	1,224 kips	0.61 in	17,209	12,096	288	20,743	20,650.8	6,048.0	0.0	4,148.6	30,847.4	**0.099**
5	13.0 ft	1,241 kips	0.62 in	18,517	13,104	288	22,339	22,220.4	6,552.0	0.0	4,467.8	33,240.2	**0.106**
4	13.0 ft	1,253 kips	0.62 in	19,825	14,112	288	23,935	23,790.0	7,056.0	0.0	4,787.0	35,633.0	**0.113**
3	13.0 ft	1,260 kips	0.61 in	21,133	15,120	288	25,531	25,359.6	7,560.0	0.0	5,106.2	38,025.8	**0.118**
2	13.0 ft	1,263 kips	0.6 in	22,441	16,128	288	27,127	26,929.2	8,064.0	0.0	5,425.4	40,418.6	**0.123**
1	13.0 ft	1,264 kips	0.39 in	23,749	17,136	288	28,723	28,498.8	8,568.0	0.0	5,744.6	42,811.4	**0.085**

GG Engineering	JOB NO. 18 - STORY BUILDINGS	BY SMG	DATE 4/22/05	SHEET NO.
	CUSTOMER DESIGN 18B	CKD	DATE	___ OF ___

SUBJECT

STABILITY COEFFICIENT ALONG COLUMN X-AXIS, θ_x

MOMENT FRAME
MF A3 - G3

- ***LOAD COMBINATION:*** 1.2D + 0.5L + 1.0E
- ***DEFLECTION AMPLIFICATION FACTOR:*** C_d = 5.5

		DUE TO FORCES THAT CAUSE BENDING ALONG THE X-AXIS OF THE MOMENT FRAME COLUMNS						
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH_i	INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	TOTAL STORY SHEAR CAPACITY (OF ALL OF THE SEISMIC RESISTING MOMENT FRAMES)	RATIO OF SHEAR DEMAND / SHEAR CAPACITY PER STORY β	MAXIMUM ALLOWED STABILITY COEFFICIENT PER STORY θ_{i_max}	STABILITY COEFFICIENT PER STORY θ_i	***COMMENT***
18	13.0 ft	185 kips	0.23 in	16,718 kips	0.0111	0.250	0.017	***OK***
17	13.0 ft	352 kips	0.4 in	16,718 kips	0.0211	0.250	0.033	***OK***
16	13.0 ft	501 kips	0.53 in	16,718 kips	0.0300	0.250	0.047	***OK***
15	13.0 ft	633 kips	0.5 in	20,093 kips	0.0315	0.250	0.047	***OK***
14	13.0 ft	749 kips	0.53 in	20,093 kips	0.0373	0.250	0.053	***OK***
13	13.0 ft	851 kips	0.54 in	16,430 kips	0.0518	0.250	0.057	***OK***
12	13.0 ft	938 kips	0.57 in	16,430 kips	0.0571	0.250	0.064	***OK***
11	13.0 ft	1,013 kips	0.59 in	16,430 kips	0.0617	0.250	0.071	***OK***
10	13.0 ft	1,075 kips	0.57 in	20,047 kips	0.0536	0.250	0.072	***OK***
9	13.0 ft	1,126 kips	0.58 in	20,047 kips	0.0562	0.250	0.078	***OK***
8	13.0 ft	1,168 kips	0.59 in	20,047 kips	0.0583	0.250	0.084	***OK***
7	13.0 ft	1,200 kips	0.61 in	20,047 kips	0.0599	0.250	0.093	***OK***
6	13.0 ft	1,224 kips	0.61 in	20,047 kips	0.0611	0.250	0.099	***OK***
5	13.0 ft	1,241 kips	0.62 in	20,047 kips	0.0619	0.250	0.106	***OK***
4	13.0 ft	1,253 kips	0.62 in	20,047 kips	0.0625	0.250	0.113	***OK***
3	13.0 ft	1,260 kips	0.61 in	20,047 kips	0.0629	0.250	0.118	***OK***
2	13.0 ft	1,263 kips	0.6 in	20,047 kips	0.0630	0.250	0.123	***OK***
1	13.0 ft	1,264 kips	0.39 in	20,047 kips	0.0631	0.250	0.085	***OK***

GG Engineering	JOB NO. 18 - STORY BUILDINGS	BY SMG	DATE 4/22/05	SHEET NO. ___ OF ___
	CUSTOMER DESIGN 18B	CKD	DATE	

SUBJECT

STABILITY COEFFICIENT ALONG COLUMN Y-AXIS, θ_y

MOMENT FRAME
MF A3 - G3

- ***LOAD COMBINATION:*** 1.2D + 0.5L + 1.0E
- ***DEFLECTION AMPLIFICATION FACTOR:*** $C_d = 5.5$
- ***(SEISMIC) IMPORTANCE FACTOR*** $I_E = 1.0$
- ***MOMENT FRAME RESISTS WHAT % OF THE TOTAL SEISMIC SHEAR TO THE BUIDLING?*** 25%

		DUE TO FORCES THAT CAUSE BENDING ALONG THE Y-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					STABILITY COEFFICIENT PER STORY θ_i
								LOAD FACTOR					
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO ANY HORIZONTAL LOAD ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0.5	*ROOF L.L.* 0	*SEISMIC VERTICAL* $0.2S_{DS} = 0.2$	TOTAL ΣP_{ui} (kips)	
18	13.0 ft	185 kips	0.23 in	1,513	0	288	1,591	1,815.6	0.0	0.0	318.2	2,133.8	**0.017**
17	13.0 ft	352 kips	0.4 in	2,821	1,008	288	3,187	3,385.2	504.0	0.0	637.4	4,526.6	**0.033**
16	13.0 ft	501 kips	0.53 in	4,129	2,016	288	4,783	4,954.8	1,008.0	0.0	956.6	6,919.4	**0.047**
15	13.0 ft	633 kips	0.5 in	5,437	3,024	288	6,379	6,524.4	1,512.0	0.0	1,275.8	9,312.2	**0.047**
14	13.0 ft	749 kips	0.53 in	6,745	4,032	288	7,975	8,094.0	2,016.0	0.0	1,595.0	11,705.0	**0.053**
13	13.0 ft	851 kips	0.54 in	8,053	5,040	288	9,571	9,663.6	2,520.0	0.0	1,914.2	14,097.8	**0.057**
12	13.0 ft	938 kips	0.57 in	9,361	6,048	288	11,167	11,233.2	3,024.0	0.0	2,233.4	16,490.6	**0.064**
11	13.0 ft	1,013 kips	0.59 in	10,669	7,056	288	12,763	12,802.8	3,528.0	0.0	2,552.6	18,883.4	**0.071**
10	13.0 ft	1,075 kips	0.57 in	11,977	8,064	288	14,359	14,372.4	4,032.0	0.0	2,871.8	21,276.2	**0.072**
9	13.0 ft	1,126 kips	0.58 in	13,285	9,072	288	15,955	15,942.0	4,536.0	0.0	3,191.0	23,669.0	**0.078**
8	13.0 ft	1,168 kips	0.59 in	14,593	10,080	288	17,551	17,511.6	5,040.0	0.0	3,510.2	26,061.8	**0.084**
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6	13.0 ft	1,224 kips	0.61 in	17,209	12,096	288	20,743	20,650.8	6,048.0	0.0	4,148.6	30,847.4	**0.099**
5	13.0 ft	1,241 kips	0.62 in	18,517	13,104	288	22,339	22,220.4	6,552.0	0.0	4,467.8	33,240.2	**0.106**
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2	13.0 ft	1,263 kips	0.6 in	22,441	16,128	288	27,127	26,929.2	8,064.0	0.0	5,425.4	40,418.6	**0.123**
1	13.0 ft	1,264 kips	0.39 in	23,749	17,136	288	28,723	28,498.8	8,568.0	0.0	5,744.6	42,811.4	**0.085**

GG Engineering	JOB NO. 18 - STORY BUILDINGS	BY SMG	DATE 4/22/05	SHEET NO.
	CUSTOMER DESIGN 18B	CKD	DATE	___ OF ___

SUBJECT

STABILITY COEFFICIENT ALONG COLUMN Y-AXIS, θ_y

MOMENT FRAME
MF A3 - G3

- ***LOAD COMBINATION:*** 1.2D + 0.5L + 1.0E
- ***DEFLECTION AMPLIFICATION FACTOR:*** C_d = 5.5

		DUE TO FORCES THAT CAUSE BENDING ALONG THE Y-AXIS OF THE MOMENT FRAME COLUMNS						
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH_i	INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	TOTAL STORY SHEAR CAPACITY (OF ALL OF THE SEISMIC RESISTING MOMENT FRAMES)	RATIO OF SHEAR DEMAND / SHEAR CAPACITY PER STORY β	MAXIMUM ALLOWED STABILITY COEFFICIENT PER STORY θ_{i_max}	STABILITY COEFFICIENT PER STORY θ_i	***COMMENT***
18	13.0 ft	185 kips	0.23 in	16,718 kips	0.0111	0.250	0.017	***OK***
17	13.0 ft	352 kips	0.4 in	16,718 kips	0.0211	0.250	0.033	***OK***
16	13.0 ft	501 kips	0.53 in	16,718 kips	0.0300	0.250	0.047	***OK***
15	13.0 ft	633 kips	0.5 in	20,093 kips	0.0315	0.250	0.047	***OK***
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13	13.0 ft	851 kips	0.54 in	16,430 kips	0.0518	0.250	0.057	***OK***
12	13.0 ft	938 kips	0.57 in	16,430 kips	0.0571	0.250	0.064	***OK***
11	13.0 ft	1,013 kips	0.59 in	16,430 kips	0.0617	0.250	0.071	***OK***
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9	13.0 ft	1,126 kips	0.58 in	20,047 kips	0.0562	0.250	0.078	***OK***
8	13.0 ft	1,168 kips	0.59 in	20,047 kips	0.0583	0.250	0.084	***OK***
7	13.0 ft	1,200 kips	0.61 in	20,047 kips	0.0599	0.250	0.093	***OK***
6	13.0 ft	1,224 kips	0.61 in	20,047 kips	0.0611	0.250	0.099	***OK***
5	13.0 ft	1,241 kips	0.62 in	20,047 kips	0.0619	0.250	0.106	***OK***
4	13.0 ft	1,253 kips	0.62 in	20,047 kips	0.0625	0.250	0.113	***OK***
3	13.0 ft	1,260 kips	0.61 in	20,047 kips	0.0629	0.250	0.118	***OK***
2	13.0 ft	1,263 kips	0.6 in	20,047 kips	0.0630	0.250	0.123	***OK***
1	13.0 ft	1,264 kips	0.39 in	20,047 kips	0.0631	0.250	0.085	***OK***

COLUMN NAME	COLUMN MEMBER SIZE	MAXIMUM INTERACTION VALUE	CONTROLLING LOAD COMBINATION	BUILDING MAX. INTERACTION 0.4688	COMMENTS FOR: DESIGN 18B — COLUMN STEEL AREA CHECK	COMMENTS FOR: DESIGN 18B — COLUMN COMPACTNESS CHECK
A3-1	HSS 16 x 16 x 0.625	0.42236241	5		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
A3-2	HSS 16 x 16 x 0.625	0.287806618	5		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
A3-3	HSS 16 x 16 x 0.625	0.265785852	5		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
A3-4	HSS 16 x 16 x 0.625	0.246439441	5		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
A3-5	HSS 16 x 16 x 0.625	0.225823288	5		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
A3-6	HSS 16 x 16 x 0.625	0.208852497	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
A3-7	HSS 16 x 16 x 0.625	0.1986573	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
A3-8	HSS 16 x 16 x 0.625	0.186727762	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
A3-9	HSS 16 x 16 x 0.625	0.172469529	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
A3-10	HSS 16 x 16 x 0.625	0.160212812	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
A3-11	HSS 16 x 16 x 0.5	0.181390212	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
A3-12	HSS 16 x 16 x 0.5	0.159135209	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
A3-13	HSS 16 x 16 x 0.5	0.133297963	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
A3-14	HSS 14 x 14 x 0.75	0.111717638	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
A3-15	HSS 14 x 14 x 0.75	0.102016841	5		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
A3-16	HSS 12 x 12 x 0.75	0.10479312	5		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
A3-17	HSS 12 x 12 x 0.75	0.108161754	5		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
A3-18	HSS 12 x 12 x 0.75	0.123312383	5		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
B3-1	HSS 16 x 16 x 0.625	0.357266114	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
B3-2	HSS 16 x 16 x 0.625	0.335702986	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
B3-3	HSS 16 x 16 x 0.625	0.315944275	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
B3-4	HSS 16 x 16 x 0.625	0.306631255	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
B3-5	HSS 16 x 16 x 0.625	0.295648053	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
B3-6	HSS 16 x 16 x 0.625	0.28344332	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
B3-7	HSS 16 x 16 x 0.625	0.271102617	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
B3-8	HSS 16 x 16 x 0.625	0.25702472	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
B3-9	HSS 16 x 16 x 0.625	0.24161104	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
B3-10	HSS 16 x 16 x 0.625	0.229485162	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
B3-11	HSS 16 x 16 x 0.5	0.258278264	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
B3-12	HSS 16 x 16 x 0.5	0.234744351	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
B3-13	HSS 16 x 16 x 0.5	0.210993905	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
B3-14	HSS 14 x 14 x 0.75	0.166397463	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
B3-15	HSS 14 x 14 x 0.75	0.138862905	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
B3-16	HSS 12 x 12 x 0.75	0.155734556	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
B3-17	HSS 12 x 12 x 0.75	0.098342459	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
B3-18	HSS 12 x 12 x 0.75	0.042259928	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
C3-1	HSS 16 x 16 x 0.625	0.466203391	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
C3-2	HSS 16 x 16 x 0.625	0.430264289	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
C3-3	HSS 16 x 16 x 0.625	0.422729252	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
C3-4	HSS 16 x 16 x 0.625	0.420086133	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
C3-5	HSS 16 x 16 x 0.625	0.416255977	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
C3-6	HSS 16 x 16 x 0.625	0.410166776	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
C3-7	HSS 16 x 16 x 0.625	0.402566515	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
C3-8	HSS 16 x 16 x 0.625	0.39112462	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
C3-9	HSS 16 x 16 x 0.625	0.377480515	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
C3-10	HSS 16 x 16 x 0.625	0.362633924	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
C3-11	HSS 16 x 16 x 0.5	0.414558354	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
C3-12	HSS 16 x 16 x 0.5	0.383670669	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
C3-13	HSS 16 x 16 x 0.5	0.354909182	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
C3-14	HSS 14 x 14 x 0.75	0.285946833	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
C3-15	HSS 14 x 14 x 0.75	0.243463416	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
C3-16	HSS 12 x 12 x 0.75	0.265497018	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
C3-17	HSS 12 x 12 x 0.75	0.196256175	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
C3-18	HSS 12 x 12 x 0.75	0.124567046	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
D3-1	HSS 16 x 16 x 0.625	0.359493331	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
D3-2	HSS 16 x 16 x 0.625	0.331526828	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
D3-3	HSS 16 x 16 x 0.625	0.325778891	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
D3-4	HSS 16 x 16 x 0.625	0.323651762	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
D3-5	HSS 16 x 16 x 0.625	0.320904094	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
D3-6	HSS 16 x 16 x 0.625	0.31630084	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
D3-7	HSS 16 x 16 x 0.625	0.310561976	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
D3-8	HSS 16 x 16 x 0.625	0.301888677	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
D3-9	HSS 16 x 16 x 0.625	0.291508997	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
D3-10	HSS 16 x 16 x 0.625	0.279968899	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
D3-11	HSS 16 x 16 x 0.5	0.320500785	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
D3-12	HSS 16 x 16 x 0.5	0.296527279	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
D3-13	HSS 16 x 16 x 0.5	0.274505277	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
D3-14	HSS 14 x 14 x 0.75	0.221431264	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
D3-15	HSS 14 x 14 x 0.75	0.188101028	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
D3-16	HSS 12 x 12 x 0.75	0.205607781	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
D3-17	HSS 12 x 12 x 0.75	0.152201457	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
D3-18	HSS 12 x 12 x 0.75	0.09551832	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
E3-1	HSS 16 x 16 x 0.625	0.468769098	6	<---CONTROLS!	OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
E3-2	HSS 16 x 16 x 0.625	0.431599323	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
E3-3	HSS 16 x 16 x 0.625	0.426290768	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT

COLUMN		MAXIMUM INTERACTION VALUE	CONTROLLING LOAD COMBINATION	BUILDING MAX. INTERACTION	COMMENTS FOR: DESIGN 18B	
NAME	MEMBER SIZE			0.4688	COLUMN STEEL AREA CHECK	COLUMN COMPACTNESS CHECK
E3-4	HSS 16 x 16 x 0.625	0.425068083	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
E3-5	HSS 16 x 16 x 0.625	0.42317743	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
E3-6	HSS 16 x 16 x 0.625	0.418482937	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
E3-7	HSS 16 x 16 x 0.625	0.412266933	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
E3-8	HSS 16 x 16 x 0.625	0.402224544	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
E3-9	HSS 16 x 16 x 0.625	0.389747978	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
E3-10	HSS 16 x 16 x 0.625	0.374894832	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
E3-11	HSS 16 x 16 x 0.5	0.43090515	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
E3-12	HSS 16 x 16 x 0.5	0.399614395	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
E3-13	HSS 16 x 16 x 0.5	0.371515388	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
E3-14	HSS 14 x 14 x 0.75	0.30168382	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
E3-15	HSS 14 x 14 x 0.75	0.255696886	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
E3-16	HSS 12 x 12 x 0.75	0.282749619	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
E3-17	HSS 12 x 12 x 0.75	0.208270532	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
E3-18	HSS 12 x 12 x 0.75	0.18224406	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
F3-1	HSS 16 x 16 x 0.625	0.362400017	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
F3-2	HSS 16 x 16 x 0.625	0.341282504	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
F3-3	HSS 16 x 16 x 0.625	0.334507663	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
F3-4	HSS 16 x 16 x 0.625	0.333259254	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
F3-5	HSS 16 x 16 x 0.625	0.331313237	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
F3-6	HSS 16 x 16 x 0.625	0.327360698	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
F3-7	HSS 16 x 16 x 0.625	0.322305573	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
F3-8	HSS 16 x 16 x 0.625	0.314357734	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
F3-9	HSS 16 x 16 x 0.625	0.304670162	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
F3-10	HSS 16 x 16 x 0.625	0.293154508	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
F3-11	HSS 16 x 16 x 0.5	0.337018364	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
F3-12	HSS 16 x 16 x 0.5	0.312697285	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
F3-13	HSS 16 x 16 x 0.5	0.290869628	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
F3-14	HSS 14 x 14 x 0.75	0.236146248	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
F3-15	HSS 14 x 14 x 0.75	0.200956634	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
F3-16	HSS 12 x 12 x 0.75	0.219562758	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
F3-17	HSS 12 x 12 x 0.75	0.166994505	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
F3-18	HSS 12 x 12 x 0.75	0.09867154	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
G3-1	HSS 16 x 16 x 0.625	0.442940694	5		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
G3-2	HSS 16 x 16 x 0.625	0.353907033	5		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
G3-3	HSS 16 x 16 x 0.625	0.337557525	5		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
G3-4	HSS 16 x 16 x 0.625	0.325200471	5		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
G3-5	HSS 16 x 16 x 0.625	0.316610617	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
G3-6	HSS 16 x 16 x 0.625	0.313503757	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
G3-7	HSS 16 x 16 x 0.625	0.309501872	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
G3-8	HSS 16 x 16 x 0.625	0.303166808	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
G3-9	HSS 16 x 16 x 0.625	0.294212428	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
G3-10	HSS 16 x 16 x 0.625	0.282578965	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
G3-11	HSS 16 x 16 x 0.5	0.331230243	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
G3-12	HSS 16 x 16 x 0.5	0.305575964	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
G3-13	HSS 16 x 16 x 0.5	0.288974494	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
G3-14	HSS 14 x 14 x 0.75	0.245193028	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
G3-15	HSS 14 x 14 x 0.75	0.205583476	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
G3-16	HSS 12 x 12 x 0.75	0.251739321	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
G3-17	HSS 12 x 12 x 0.75	0.200910797	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
G3-18	HSS 12 x 12 x 0.75	0.163507997	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT

Appendix H

Building Design 18C Calculations

This appendix consists of the design calculations that were performed for building Design 18C which is the 18-story building that used low strength steel and high strength concrete in the columns (F_{yc} = 50 ksi and f'_c = 16 ksi) and a high column d/t ratio. The final RCFT column and wide flange girder sections are presented in Chapter 5. The linear elastic analysis consisted of taking the nominal loads that were generated in Appendix E and factoring them per the applicable LRFD load combination. The calculation for the stability coefficient, θ, and the moment magnification factor, B_2, were performed for each load combination that has lateral loads (wind and seismic load combinations #4, #5, and #6) and are included in this appendix. The maximum interaction value for each column is listed at the end of this appendix along with its respective load combination.

The load combinations that were used in this building design are listed below for reference:

1. $1.4D$
2. $1.2D + 1.6L + 0.5L_R$
3. $1.2D + 1.6L_R + f_1L$
4. $1.2D + 1.6L_R + 0.8W$
5. $1.2D + 1.6W + f_1L + 0.5L_R$
6. $1.2D + 1.0E + f_1L$

Where: $f_1 = 0.5$
$E = \rho Q_E + 0.2S_{DS}D'$
D' = seismic weight

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SUBJECT

DESIGN PARAMETERS SUMMARY

MOMENT FRAME
MF A3 - G3

o DESIGN INPUTS:

o TOAL NUMBER OF COLUMNS BEING ANALYZED 126

o YIELD STRENGTH: HSS, F_y = 50 ksi
CONCRETE REINFORCEMENT, F_{yr} = 0 ksi

o MODULUS OF ELASTICITY: HSS, E_s = 29,000 ksi
CONCRETE REINFORCEMENT, E_{cr} = 29,000 ksi

o MINIMUM CONCRETE COMPRESSIVE STRENGTH f'_c = 16.0 ksi

o CONCRETE DENSITY w = 145 lb/ft³

o CONCRETE REINFORCEMENT AREA, A_{sr} = 0.0 in²
I_{xxr} = 0.0 in^4
I_{yyr} = 0.0 in^4
Z_{xxr} = 0.0 in³
Z_{yyr} = 0.0 in³

o RESISTANCE FACTORS AXIAL COMPRESSION, ϕ_c = 0.75
FLEXURAL BENDING, ϕ_b = 0.90

o SEISMIC PARAMETERS REDUNDANCY COEFFICIENT, ρ = 1.00
VERTICAL SEISMIC "FACTOR," $0.2S_{DS}$ = 0.20
ORTHOGONAL LOAD FACTOR ALONG Y-AXIS OF SHARED COLUMNS = 0.30
FACTOR TO ACCOUNT FOR *5% ACCIDENTAL TORSION* ("SIMPLIFIED APPROACH"…) = 0.025

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SUBJECT

B2 CALCULATION - *FOR BENDING ALONG THE X-AXIS OF THE COLUMN*

MOMENT FRAME
MF A3 - G3

o ***LOAD COMBINATION =*** **1.2D + 1.6Lr + 0.8W** *(L.C. # 4)*

		DUE TO FORCES THAT CAUSE BENDING ALONG THE X-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					
								LOAD FACTOR					
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0	*ROOF L.L.* 1.6	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0	TOTAL ΣP_{ui} (kips)	**STORY $B_{2i\_X\text{-}AXIS}$**
18	13.0 ft	185 kips	0.25 in	1,513	0	288	1,591	1,815.6	0.0	460.8	0.0	2,276.4	**1.020**
17	13.0 ft	352 kips	0.39 in	2,821	1,008	288	3,187	3,385.2	0.0	460.8	0.0	3,846.0	**1.028**
16	13.0 ft	501 kips	0.53 in	4,129	2,016	288	4,783	4,954.8	0.0	460.8	0.0	5,415.6	**1.038**
15	13.0 ft	633 kips	0.58 in	5,437	3,024	288	6,379	6,524.4	0.0	460.8	0.0	6,985.2	**1.043**
14	13.0 ft	749 kips	0.55 in	6,745	4,032	288	7,975	8,094.0	0.0	460.8	0.0	8,554.8	**1.042**
13	13.0 ft	851 kips	0.53 in	8,053	5,040	288	9,571	9,663.6	0.0	460.8	0.0	10,124.4	**1.042**
12	13.0 ft	938 kips	0.55 in	9,361	6,048	288	11,167	11,233.2	0.0	460.8	0.0	11,694.0	**1.046**
11	13.0 ft	1,013 kips	0.58 in	10,669	7,056	288	12,763	12,802.8	0.0	460.8	0.0	13,263.6	**1.051**
10	13.0 ft	1,075 kips	0.61 in	11,977	8,064	288	14,359	14,372.4	0.0	460.8	0.0	14,833.2	**1.057**
9	13.0 ft	1,126 kips	0.62 in	13,285	9,072	288	15,955	15,942.0	0.0	460.8	0.0	16,402.8	**1.061**
8	13.0 ft	1,168 kips	0.58 in	14,593	10,080	288	17,551	17,511.6	0.0	460.8	0.0	17,972.4	**1.061**
7	13.0 ft	1,200 kips	0.56 in	15,901	11,088	288	19,147	19,081.2	0.0	460.8	0.0	19,542.0	**1.062**
6	13.0 ft	1,224 kips	0.55 in	17,209	12,096	288	20,743	20,650.8	0.0	460.8	0.0	21,111.6	**1.065**
5	13.0 ft	1,241 kips	0.56 in	18,517	13,104	288	22,339	22,220.4	0.0	460.8	0.0	22,681.2	**1.070**
4	13.0 ft	1,253 kips	0.56 in	19,825	14,112	288	23,935	23,790.0	0.0	460.8	0.0	24,250.8	**1.075**
3	13.0 ft	1,260 kips	0.53 in	21,133	15,120	288	25,531	25,359.6	0.0	460.8	0.0	25,820.4	**1.075**
2	13.0 ft	1,263 kips	0.48 in	22,441	16,128	288	27,127	26,929.2	0.0	460.8	0.0	27,390.0	**1.071**
1	13.0 ft	1,264 kips	0.24 in	23,749	17,136	288	28,723	28,498.8	0.0	460.8	0.0	28,959.6	**1.037**

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SUBJECT **B2 CALCULATION - *FOR BENDING ALONG THE Y-AXIS OF THE COLUMN*** **MOMENT FRAME MF A3 - G3**

o ***LOAD COMBINATION =*** **1.2D + 1.6Lr + 0.8W**

		DUE TO FORCES THAT CAUSE BENDING ALONG THE Y-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					
								LOAD FACTOR					
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0	*ROOF L.L.* 1.6	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0	TOTAL ΣP_{ui} (kips)	**STORY $B_{2i\_Y\text{-}AXIS}$**
18	13.0 ft	185 kips	0.25 in	1,513	0	288	1,591	1,815.6	0.0	460.8	0.0	2,276.4	**1.020**
17	13.0 ft	352 kips	0.39 in	2,821	1,008	288	3,187	3,385.2	0.0	460.8	0.0	3,846.0	**1.028**
16	13.0 ft	501 kips	0.53 in	4,129	2,016	288	4,783	4,954.8	0.0	460.8	0.0	5,415.6	**1.038**
15	13.0 ft	633 kips	0.58 in	5,437	3,024	288	6,379	6,524.4	0.0	460.8	0.0	6,985.2	**1.043**
14	13.0 ft	749 kips	0.55 in	6,745	4,032	288	7,975	8,094.0	0.0	460.8	0.0	8,554.8	**1.042**
13	13.0 ft	851 kips	0.53 in	8,053	5,040	288	9,571	9,663.6	0.0	460.8	0.0	10,124.4	**1.042**
12	13.0 ft	938 kips	0.55 in	9,361	6,048	288	11,167	11,233.2	0.0	460.8	0.0	11,694.0	**1.046**
11	13.0 ft	1,013 kips	0.58 in	10,669	7,056	288	12,763	12,802.8	0.0	460.8	0.0	13,263.6	**1.051**
10	13.0 ft	1,075 kips	0.61 in	11,977	8,064	288	14,359	14,372.4	0.0	460.8	0.0	14,833.2	**1.057**
9	13.0 ft	1,126 kips	0.62 in	13,285	9,072	288	15,955	15,942.0	0.0	460.8	0.0	16,402.8	**1.061**
8	13.0 ft	1,168 kips	0.58 in	14,593	10,080	288	17,551	17,511.6	0.0	460.8	0.0	17,972.4	**1.061**
7	13.0 ft	1,200 kips	0.56 in	15,901	11,088	288	19,147	19,081.2	0.0	460.8	0.0	19,542.0	**1.062**
6	13.0 ft	1,224 kips	0.55 in	17,209	12,096	288	20,743	20,650.8	0.0	460.8	0.0	21,111.6	**1.065**
5	13.0 ft	1,241 kips	0.56 in	18,517	13,104	288	22,339	22,220.4	0.0	460.8	0.0	22,681.2	**1.070**
4	13.0 ft	1,253 kips	0.56 in	19,825	14,112	288	23,935	23,790.0	0.0	460.8	0.0	24,250.8	**1.075**
3	13.0 ft	1,260 kips	0.53 in	21,133	15,120	288	25,531	25,359.6	0.0	460.8	0.0	25,820.4	**1.075**
2	13.0 ft	1,263 kips	0.48 in	22,441	16,128	288	27,127	26,929.2	0.0	460.8	0.0	27,390.0	**1.071**
1	13.0 ft	1,264 kips	0.24 in	23,749	17,136	288	28,723	28,498.8	0.0	460.8	0.0	28,959.6	**1.037**

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SUBJECT

STABILITY COEFFICIENT ALONG COLUMN X-AXIS, θ_x

MOMENT FRAME
MF A3 - G3

- ***LOAD COMBINATION:*** 1.2D + 1.6Lr + 0.8W
- ***DEFLECTION AMPLIFICATION FACTOR:*** C_d = 5.5
- ***(SEISMIC) IMPORTANCE FACTOR*** I_E = 1.0
- ***MOMENT FRAME RESISTS WHAT % OF THE TOTAL SEISMIC SHEAR TO THE BUIDLING?*** 25%

		DUE TO FORCES THAT CAUSE BENDING ALONG THE X-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					STABILITY COEFFICIENT PER STORY θ_i
								LOAD FACTOR					
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO ANY HORIZONTAL LOAD ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0	*ROOF L.L.* 1.6	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0	TOTAL ΣP_{ui} (kips)	
18	13.0 ft	185 kips	0.25 in	1,513	0	288	1,591	1,815.6	0.0	460.8	0.0	2,276.4	**0.020**
17	13.0 ft	352 kips	0.39 in	2,821	1,008	288	3,187	3,385.2	0.0	460.8	0.0	3,846.0	**0.027**
16	13.0 ft	501 kips	0.53 in	4,129	2,016	288	4,783	4,954.8	0.0	460.8	0.0	5,415.6	**0.037**
15	13.0 ft	633 kips	0.58 in	5,437	3,024	288	6,379	6,524.4	0.0	460.8	0.0	6,985.2	**0.041**
14	13.0 ft	749 kips	0.55 in	6,745	4,032	288	7,975	8,094.0	0.0	460.8	0.0	8,554.8	**0.040**
13	13.0 ft	851 kips	0.53 in	8,053	5,040	288	9,571	9,663.6	0.0	460.8	0.0	10,124.4	**0.040**
12	13.0 ft	938 kips	0.55 in	9,361	6,048	288	11,167	11,233.2	0.0	460.8	0.0	11,694.0	**0.044**
11	13.0 ft	1,013 kips	0.58 in	10,669	7,056	288	12,763	12,802.8	0.0	460.8	0.0	13,263.6	**0.049**
10	13.0 ft	1,075 kips	0.61 in	11,977	8,064	288	14,359	14,372.4	0.0	460.8	0.0	14,833.2	**0.054**
9	13.0 ft	1,126 kips	0.62 in	13,285	9,072	288	15,955	15,942.0	0.0	460.8	0.0	16,402.8	**0.058**
8	13.0 ft	1,168 kips	0.58 in	14,593	10,080	288	17,551	17,511.6	0.0	460.8	0.0	17,972.4	**0.057**
7	13.0 ft	1,200 kips	0.56 in	15,901	11,088	288	19,147	19,081.2	0.0	460.8	0.0	19,542.0	**0.058**
6	13.0 ft	1,224 kips	0.55 in	17,209	12,096	288	20,743	20,650.8	0.0	460.8	0.0	21,111.6	**0.061**
5	13.0 ft	1,241 kips	0.56 in	18,517	13,104	288	22,339	22,220.4	0.0	460.8	0.0	22,681.2	**0.066**
4	13.0 ft	1,253 kips	0.56 in	19,825	14,112	288	23,935	23,790.0	0.0	460.8	0.0	24,250.8	**0.069**
3	13.0 ft	1,260 kips	0.53 in	21,133	15,120	288	25,531	25,359.6	0.0	460.8	0.0	25,820.4	**0.070**
2	13.0 ft	1,263 kips	0.48 in	22,441	16,128	288	27,127	26,929.2	0.0	460.8	0.0	27,390.0	**0.067**
1	13.0 ft	1,264 kips	0.24 in	23,749	17,136	288	28,723	28,498.8	0.0	460.8	0.0	28,959.6	**0.035**

GG Engineering	JOB NO. 18 - STORY BUILDINGS	BY SMG	DATE 4/22/05	SHEET NO. ___ OF ___
	CUSTOMER DESIGN 18C	CKD	DATE	

SUBJECT

STABILITY COEFFICIENT ALONG COLUMN Y-AXIS, θ_y

MOMENT FRAME
MF A3 - G3

- ***LOAD COMBINATION:*** 1.2D + 1.6Lr + 0.8W
- ***DEFLECTION AMPLIFICATION FACTOR:*** C_d = 5.5
- ***(SEISMIC) IMPORTANCE FACTOR*** I_E = 1.0
- ***MOMENT FRAME RESISTS WHAT % OF THE TOTAL SEISMIC SHEAR TO THE BUIDLING?*** 25%

		DUE TO FORCES THAT CAUSE BENDING ALONG THE Y-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					
								LOAD FACTOR					
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO ANY HORIZONTAL LOAD ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0	*ROOF L.L.* 1.6	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0	TOTAL ΣP_{ui} (kips)	STABILITY COEFFICIENT PER STORY θ_i
18	13.0 ft	185 kips	0.25 in	1,513	0	288	1,591	1,815.6	0.0	460.8	0.0	2,276.4	**0.020**
17	13.0 ft	352 kips	0.39 in	2,821	1,008	288	3,187	3,385.2	0.0	460.8	0.0	3,846.0	**0.027**
16	13.0 ft	501 kips	0.53 in	4,129	2,016	288	4,783	4,954.8	0.0	460.8	0.0	5,415.6	**0.037**
15	13.0 ft	633 kips	0.58 in	5,437	3,024	288	6,379	6,524.4	0.0	460.8	0.0	6,985.2	**0.041**
14	13.0 ft	749 kips	0.55 in	6,745	4,032	288	7,975	8,094.0	0.0	460.8	0.0	8,554.8	**0.040**
13	13.0 ft	851 kips	0.53 in	8,053	5,040	288	9,571	9,663.6	0.0	460.8	0.0	10,124.4	**0.040**
12	13.0 ft	938 kips	0.55 in	9,361	6,048	288	11,167	11,233.2	0.0	460.8	0.0	11,694.0	**0.044**
11	13.0 ft	1,013 kips	0.58 in	10,669	7,056	288	12,763	12,802.8	0.0	460.8	0.0	13,263.6	**0.049**
10	13.0 ft	1,075 kips	0.61 in	11,977	8,064	288	14,359	14,372.4	0.0	460.8	0.0	14,833.2	**0.054**
9	13.0 ft	1,126 kips	0.62 in	13,285	9,072	288	15,955	15,942.0	0.0	460.8	0.0	16,402.8	**0.058**
8	13.0 ft	1,168 kips	0.58 in	14,593	10,080	288	17,551	17,511.6	0.0	460.8	0.0	17,972.4	**0.057**
7	13.0 ft	1,200 kips	0.56 in	15,901	11,088	288	19,147	19,081.2	0.0	460.8	0.0	19,542.0	**0.058**
6	13.0 ft	1,224 kips	0.55 in	17,209	12,096	288	20,743	20,650.8	0.0	460.8	0.0	21,111.6	**0.061**
5	13.0 ft	1,241 kips	0.56 in	18,517	13,104	288	22,339	22,220.4	0.0	460.8	0.0	22,681.2	**0.066**
4	13.0 ft	1,253 kips	0.56 in	19,825	14,112	288	23,935	23,790.0	0.0	460.8	0.0	24,250.8	**0.069**
3	13.0 ft	1,260 kips	0.53 in	21,133	15,120	288	25,531	25,359.6	0.0	460.8	0.0	25,820.4	**0.070**
2	13.0 ft	1,263 kips	0.48 in	22,441	16,128	288	27,127	26,929.2	0.0	460.8	0.0	27,390.0	**0.067**
1	13.0 ft	1,264 kips	0.24 in	23,749	17,136	288	28,723	28,498.8	0.0	460.8	0.0	28,959.6	**0.035**

GG Engineering	JOB NO. 18 - STORY BUILDINGS	BY SMG	DATE 4/22/05	SHEET NO. ___ OF ___
	CUSTOMER DESIGN 18C	CKD	DATE	

SUBJECT

B2 CALCULATION - *FOR BENDING ALONG THE X-AXIS OF THE COLUMN*

MOMENT FRAME
MF A3 - G3

o ***LOAD COMBINATION =*** **1.2D + 0.5L + 0.5Lr + 1.6W** *(L.C. # 5)*

STORY NUMBER	LENGTH *(STORY HEIGHT)* L	DUE TO FORCES THAT CAUSE BENDING ALONG THE X-AXIS OF THE MOMENT FRAME COLUMNS: TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH_i	DUE TO FORCES THAT CAUSE BENDING ALONG THE X-AXIS OF THE MOMENT FRAME COLUMNS: ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips): DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY — *LOAD FACTOR*: *D.L.* 1.2	*L.L.* 0.5	*ROOF L.L.* 0.5	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0	TOTAL ΣP_{ui} (kips)	STORY $B_{2i\_X\text{-}AXIS}$
18	13.0 ft	185 kips	0.25 in	1,513	0	288	1,591	1,815.6	0.0	144.0	0.0	1,959.6	**1.017**
17	13.0 ft	352 kips	0.39 in	2,821	1,008	288	3,187	3,385.2	504.0	144.0	0.0	4,033.2	**1.029**
16	13.0 ft	501 kips	0.53 in	4,129	2,016	288	4,783	4,954.8	1,008.0	144.0	0.0	6,106.8	**1.043**
15	13.0 ft	633 kips	0.58 in	5,437	3,024	288	6,379	6,524.4	1,512.0	144.0	0.0	8,180.4	**1.050**
14	13.0 ft	749 kips	0.55 in	6,745	4,032	288	7,975	8,094.0	2,016.0	144.0	0.0	10,254.0	**1.051**
13	13.0 ft	851 kips	0.53 in	8,053	5,040	288	9,571	9,663.6	2,520.0	144.0	0.0	12,327.6	**1.052**
12	13.0 ft	938 kips	0.55 in	9,361	6,048	288	11,167	11,233.2	3,024.0	144.0	0.0	14,401.2	**1.057**
11	13.0 ft	1,013 kips	0.58 in	10,669	7,056	288	12,763	12,802.8	3,528.0	144.0	0.0	16,474.8	**1.064**
10	13.0 ft	1,075 kips	0.61 in	11,977	8,064	288	14,359	14,372.4	4,032.0	144.0	0.0	18,548.4	**1.072**
9	13.0 ft	1,126 kips	0.62 in	13,285	9,072	288	15,955	15,942.0	4,536.0	144.0	0.0	20,622.0	**1.079**
8	13.0 ft	1,168 kips	0.58 in	14,593	10,080	288	17,551	17,511.6	5,040.0	144.0	0.0	22,695.6	**1.078**
7	13.0 ft	1,200 kips	0.56 in	15,901	11,088	288	19,147	19,081.2	5,544.0	144.0	0.0	24,769.2	**1.080**
6	13.0 ft	1,224 kips	0.55 in	17,209	12,096	288	20,743	20,650.8	6,048.0	144.0	0.0	26,842.8	**1.084**
5	13.0 ft	1,241 kips	0.56 in	18,517	13,104	288	22,339	22,220.4	6,552.0	144.0	0.0	28,916.4	**1.091**
4	13.0 ft	1,253 kips	0.56 in	19,825	14,112	288	23,935	23,790.0	7,056.0	144.0	0.0	30,990.0	**1.097**
3	13.0 ft	1,260 kips	0.53 in	21,133	15,120	288	25,531	25,359.6	7,560.0	144.0	0.0	33,063.6	**1.098**
2	13.0 ft	1,263 kips	0.48 in	22,441	16,128	288	27,127	26,929.2	8,064.0	144.0	0.0	35,137.2	**1.094**
1	13.0 ft	1,264 kips	0.24 in	23,749	17,136	288	28,723	28,498.8	8,568.0	144.0	0.0	37,210.8	**1.047**

GG Engineering	JOB NO. 18 - STORY BUILDINGS	BY SMG	DATE 4/22/05	SHEET NO. ___ OF ___
	CUSTOMER DESIGN 18C	CKD	DATE	

SUBJECT

B2 CALCULATION - *FOR BENDING ALONG THE Y-AXIS OF THE COLUMN*

MOMENT FRAME MF A3 - G3

o ***LOAD COMBINATION =*** **1.2D + 0.5L + 0.5Lr + 1.6W**

		DUE TO FORCES THAT CAUSE BENDING ALONG THE Y-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					
								LOAD FACTOR					
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0.5	*ROOF L.L.* 0.5	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0	TOTAL ΣP_{ui} (kips)	**STORY $B_{2i\_Y\text{-}AXIS}$**
18	13.0 ft	185 kips	0.25 in	1,513	0	288	1,591	1,815.6	0.0	144.0	0.0	1,959.6	**1.017**
17	13.0 ft	352 kips	0.39 in	2,821	1,008	288	3,187	3,385.2	504.0	144.0	0.0	4,033.2	**1.029**
16	13.0 ft	501 kips	0.53 in	4,129	2,016	288	4,783	4,954.8	1,008.0	144.0	0.0	6,106.8	**1.043**
15	13.0 ft	633 kips	0.58 in	5,437	3,024	288	6,379	6,524.4	1,512.0	144.0	0.0	8,180.4	**1.050**
14	13.0 ft	749 kips	0.55 in	6,745	4,032	288	7,975	8,094.0	2,016.0	144.0	0.0	10,254.0	**1.051**
13	13.0 ft	851 kips	0.53 in	8,053	5,040	288	9,571	9,663.6	2,520.0	144.0	0.0	12,327.6	**1.052**
12	13.0 ft	938 kips	0.55 in	9,361	6,048	288	11,167	11,233.2	3,024.0	144.0	0.0	14,401.2	**1.057**
11	13.0 ft	1,013 kips	0.58 in	10,669	7,056	288	12,763	12,802.8	3,528.0	144.0	0.0	16,474.8	**1.064**
10	13.0 ft	1,075 kips	0.61 in	11,977	8,064	288	14,359	14,372.4	4,032.0	144.0	0.0	18,548.4	**1.072**
9	13.0 ft	1,126 kips	0.62 in	13,285	9,072	288	15,955	15,942.0	4,536.0	144.0	0.0	20,622.0	**1.079**
8	13.0 ft	1,168 kips	0.58 in	14,593	10,080	288	17,551	17,511.6	5,040.0	144.0	0.0	22,695.6	**1.078**
7	13.0 ft	1,200 kips	0.56 in	15,901	11,088	288	19,147	19,081.2	5,544.0	144.0	0.0	24,769.2	**1.080**
6	13.0 ft	1,224 kips	0.55 in	17,209	12,096	288	20,743	20,650.8	6,048.0	144.0	0.0	26,842.8	**1.084**
5	13.0 ft	1,241 kips	0.56 in	18,517	13,104	288	22,339	22,220.4	6,552.0	144.0	0.0	28,916.4	**1.091**
4	13.0 ft	1,253 kips	0.56 in	19,825	14,112	288	23,935	23,790.0	7,056.0	144.0	0.0	30,990.0	**1.097**
3	13.0 ft	1,260 kips	0.53 in	21,133	15,120	288	25,531	25,359.6	7,560.0	144.0	0.0	33,063.6	**1.098**
2	13.0 ft	1,263 kips	0.48 in	22,441	16,128	288	27,127	26,929.2	8,064.0	144.0	0.0	35,137.2	**1.094**
1	13.0 ft	1,264 kips	0.24 in	23,749	17,136	288	28,723	28,498.8	8,568.0	144.0	0.0	37,210.8	**1.047**

GG Engineering	JOB NO. 18 - STORY BUILDINGS	BY SMG	DATE 4/22/05	SHEET NO.
	CUSTOMER DESIGN 18C	CKD	DATE	___ OF ___

SUBJECT

STABILITY COEFFICIENT ALONG COLUMN X-AXIS, θ_x

MOMENT FRAME
MF A3 - G3

- ***LOAD COMBINATION:*** 1.2D + 0.5L + 0.5Lr + 1.6W
- ***DEFLECTION AMPLIFICATION FACTOR:*** C_d = 5.5
- ***(SEISMIC) IMPORTANCE FACTOR*** I_E = 1.0
- ***MOMENT FRAME RESISTS WHAT % OF THE TOTAL SEISMIC SHEAR TO THE BUIDLING?*** 25%

		DUE TO FORCES THAT CAUSE BENDING ALONG THE X-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					STABILITY COEFFICIENT PER STORY θ_i
								LOAD FACTOR					
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO ANY HORIZONTAL LOAD ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0.5	*ROOF L.L.* 0.5	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0	TOTAL ΣP_{ui} (kips)	
18	13.0 ft	185 kips	0.25 in	1,513	0	288	1,591	1,815.6	0.0	144.0	0.0	1,959.6	**0.017**
17	13.0 ft	352 kips	0.39 in	2,821	1,008	288	3,187	3,385.2	504.0	144.0	0.0	4,033.2	**0.029**
16	13.0 ft	501 kips	0.53 in	4,129	2,016	288	4,783	4,954.8	1,008.0	144.0	0.0	6,106.8	**0.041**
15	13.0 ft	633 kips	0.58 in	5,437	3,024	288	6,379	6,524.4	1,512.0	144.0	0.0	8,180.4	**0.048**
14	13.0 ft	749 kips	0.55 in	6,745	4,032	288	7,975	8,094.0	2,016.0	144.0	0.0	10,254.0	**0.048**
13	13.0 ft	851 kips	0.53 in	8,053	5,040	288	9,571	9,663.6	2,520.0	144.0	0.0	12,327.6	**0.049**
12	13.0 ft	938 kips	0.55 in	9,361	6,048	288	11,167	11,233.2	3,024.0	144.0	0.0	14,401.2	**0.054**
11	13.0 ft	1,013 kips	0.58 in	10,669	7,056	288	12,763	12,802.8	3,528.0	144.0	0.0	16,474.8	**0.060**
10	13.0 ft	1,075 kips	0.61 in	11,977	8,064	288	14,359	14,372.4	4,032.0	144.0	0.0	18,548.4	**0.067**
9	13.0 ft	1,126 kips	0.62 in	13,285	9,072	288	15,955	15,942.0	4,536.0	144.0	0.0	20,622.0	**0.073**
8	13.0 ft	1,168 kips	0.58 in	14,593	10,080	288	17,551	17,511.6	5,040.0	144.0	0.0	22,695.6	**0.072**
7	13.0 ft	1,200 kips	0.56 in	15,901	11,088	288	19,147	19,081.2	5,544.0	144.0	0.0	24,769.2	**0.074**
6	13.0 ft	1,224 kips	0.55 in	17,209	12,096	288	20,743	20,650.8	6,048.0	144.0	0.0	26,842.8	**0.077**
5	13.0 ft	1,241 kips	0.56 in	18,517	13,104	288	22,339	22,220.4	6,552.0	144.0	0.0	28,916.4	**0.084**
4	13.0 ft	1,253 kips	0.56 in	19,825	14,112	288	23,935	23,790.0	7,056.0	144.0	0.0	30,990.0	**0.089**
3	13.0 ft	1,260 kips	0.53 in	21,133	15,120	288	25,531	25,359.6	7,560.0	144.0	0.0	33,063.6	**0.089**
2	13.0 ft	1,263 kips	0.48 in	22,441	16,128	288	27,127	26,929.2	8,064.0	144.0	0.0	35,137.2	**0.086**
1	13.0 ft	1,264 kips	0.24 in	23,749	17,136	288	28,723	28,498.8	8,568.0	144.0	0.0	37,210.8	**0.045**

GG Engineering	JOB NO. 18 - STORY BUILDINGS	BY SMG	DATE 4/22/05	SHEET NO. ___ OF ___
	CUSTOMER DESIGN 18C	CKD	DATE	

SUBJECT

STABILITY COEFFICIENT ALONG COLUMN Y-AXIS, θ_y

MOMENT FRAME
MF A3 - G3

- ***LOAD COMBINATION:*** 1.2D + 0.5L + 0.5Lr + 1.6W
- ***DEFLECTION AMPLIFICATION FACTOR:*** C_d = 5.5
- ***(SEISMIC) IMPORTANCE FACTOR*** I_E = 1.0
- ***MOMENT FRAME RESISTS WHAT % OF THE TOTAL SEISMIC SHEAR TO THE BUIDLING?*** 25%

		DUE TO FORCES THAT CAUSE BENDING ALONG THE Y-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					STABILITY COEFFICIENT PER STORY θ_i
								LOAD FACTOR					
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO ANY HORIZONTAL LOAD ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0.5	*ROOF L.L.* 0.5	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0	TOTAL ΣP_{ui} (kips)	
18	13.0 ft	185 kips	0.25 in	1,513	0	288	1,591	1,815.6	0.0	144.0	0.0	1,959.6	**0.017**
17	13.0 ft	352 kips	0.39 in	2,821	1,008	288	3,187	3,385.2	504.0	144.0	0.0	4,033.2	**0.029**
16	13.0 ft	501 kips	0.53 in	4,129	2,016	288	4,783	4,954.8	1,008.0	144.0	0.0	6,106.8	**0.041**
15	13.0 ft	633 kips	0.58 in	5,437	3,024	288	6,379	6,524.4	1,512.0	144.0	0.0	8,180.4	**0.048**
14	13.0 ft	749 kips	0.55 in	6,745	4,032	288	7,975	8,094.0	2,016.0	144.0	0.0	10,254.0	**0.048**
13	13.0 ft	851 kips	0.53 in	8,053	5,040	288	9,571	9,663.6	2,520.0	144.0	0.0	12,327.6	**0.049**
12	13.0 ft	938 kips	0.55 in	9,361	6,048	288	11,167	11,233.2	3,024.0	144.0	0.0	14,401.2	**0.054**
11	13.0 ft	1,013 kips	0.58 in	10,669	7,056	288	12,763	12,802.8	3,528.0	144.0	0.0	16,474.8	**0.060**
10	13.0 ft	1,075 kips	0.61 in	11,977	8,064	288	14,359	14,372.4	4,032.0	144.0	0.0	18,548.4	**0.067**
9	13.0 ft	1,126 kips	0.62 in	13,285	9,072	288	15,955	15,942.0	4,536.0	144.0	0.0	20,622.0	**0.073**
8	13.0 ft	1,168 kips	0.58 in	14,593	10,080	288	17,551	17,511.6	5,040.0	144.0	0.0	22,695.6	**0.072**
7	13.0 ft	1,200 kips	0.56 in	15,901	11,088	288	19,147	19,081.2	5,544.0	144.0	0.0	24,769.2	**0.074**
6	13.0 ft	1,224 kips	0.55 in	17,209	12,096	288	20,743	20,650.8	6,048.0	144.0	0.0	26,842.8	**0.077**
5	13.0 ft	1,241 kips	0.56 in	18,517	13,104	288	22,339	22,220.4	6,552.0	144.0	0.0	28,916.4	**0.084**
4	13.0 ft	1,253 kips	0.56 in	19,825	14,112	288	23,935	23,790.0	7,056.0	144.0	0.0	30,990.0	**0.089**
3	13.0 ft	1,260 kips	0.53 in	21,133	15,120	288	25,531	25,359.6	7,560.0	144.0	0.0	33,063.6	**0.089**
2	13.0 ft	1,263 kips	0.48 in	22,441	16,128	288	27,127	26,929.2	8,064.0	144.0	0.0	35,137.2	**0.086**
1	13.0 ft	1,264 kips	0.24 in	23,749	17,136	288	28,723	28,498.8	8,568.0	144.0	0.0	37,210.8	**0.045**

GG Engineering	JOB NO. 18 - STORY BUILDINGS	BY SMG	DATE 4/22/05	SHEET NO. ___ OF ___
	CUSTOMER DESIGN 18C	CKD	DATE	

SUBJECT

B2 CALCULATION - *FOR BENDING ALONG THE X-AXIS OF THE COLUMN*

MOMENT FRAME ***MF A3 - G3***

o ***LOAD COMBINATION =*** **1.2D + 0.5L + 1.0E** *(L.C. # 6)*

STORY NUMBER	LENGTH *(STORY HEIGHT)* L	DUE TO FORCES THAT CAUSE BENDING ALONG THE X-AXIS OF THE MOMENT FRAME COLUMNS: TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips): DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY — *LOAD FACTOR*: *D.L.* 1.2	*L.L.* 0.5	*ROOF L.L.* 0	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0.2	TOTAL ΣP_{ui} (kips)	**STORY $B_{2i\_X\text{-}AXIS}$**
18	13.0 ft	185 kips	0.25 in	1,513	0	288	1,591	1,815.6	0.0	0.0	318.2	2,133.8	**1.019**
17	13.0 ft	352 kips	0.39 in	2,821	1,008	288	3,187	3,385.2	504.0	0.0	637.4	4,526.6	**1.033**
16	13.0 ft	501 kips	0.53 in	4,129	2,016	288	4,783	4,954.8	1,008.0	0.0	956.6	6,919.4	**1.049**
15	13.0 ft	633 kips	0.58 in	5,437	3,024	288	6,379	6,524.4	1,512.0	0.0	1,275.8	9,312.2	**1.058**
14	13.0 ft	749 kips	0.55 in	6,745	4,032	288	7,975	8,094.0	2,016.0	0.0	1,595.0	11,705.0	**1.058**
13	13.0 ft	851 kips	0.53 in	8,053	5,040	288	9,571	9,663.6	2,520.0	0.0	1,914.2	14,097.8	**1.060**
12	13.0 ft	938 kips	0.55 in	9,361	6,048	288	11,167	11,233.2	3,024.0	0.0	2,233.4	16,490.6	**1.066**
11	13.0 ft	1,013 kips	0.58 in	10,669	7,056	288	12,763	12,802.8	3,528.0	0.0	2,552.6	18,883.4	**1.074**
10	13.0 ft	1,075 kips	0.61 in	11,977	8,064	288	14,359	14,372.4	4,032.0	0.0	2,871.8	21,276.2	**1.084**
9	13.0 ft	1,126 kips	0.62 in	13,285	9,072	288	15,955	15,942.0	4,536.0	0.0	3,191.0	23,669.0	**1.091**
8	13.0 ft	1,168 kips	0.58 in	14,593	10,080	288	17,551	17,511.6	5,040.0	0.0	3,510.2	26,061.8	**1.090**
7	13.0 ft	1,200 kips	0.56 in	15,901	11,088	288	19,147	19,081.2	5,544.0	0.0	3,829.4	28,454.6	**1.093**
6	13.0 ft	1,224 kips	0.55 in	17,209	12,096	288	20,743	20,650.8	6,048.0	0.0	4,148.6	30,847.4	**1.098**
5	13.0 ft	1,241 kips	0.56 in	18,517	13,104	288	22,339	22,220.4	6,552.0	0.0	4,467.8	33,240.2	**1.106**
4	13.0 ft	1,253 kips	0.56 in	19,825	14,112	288	23,935	23,790.0	7,056.0	0.0	4,787.0	35,633.0	**1.114**
3	13.0 ft	1,260 kips	0.53 in	21,133	15,120	288	25,531	25,359.6	7,560.0	0.0	5,106.2	38,025.8	**1.114**
2	13.0 ft	1,263 kips	0.48 in	22,441	16,128	288	27,127	26,929.2	8,064.0	0.0	5,425.4	40,418.6	**1.109**
1	13.0 ft	1,264 kips	0.24 in	23,749	17,136	288	28,723	28,498.8	8,568.0	0.0	5,744.6	42,811.4	**1.055**

GG Engineering	JOB NO. 18 - STORY BUILDINGS	BY SMG	DATE 4/22/05	SHEET NO.
	CUSTOMER DESIGN 18C	CKD	DATE	OF

SUBJECT

B2 CALCULATION - *FOR BENDING ALONG THE Y-AXIS OF THE COLUMN*

MOMENT FRAME
MF A3 - G3

o ***LOAD COMBINATION =*** **1.2D + 0.5L + 1.0E**

		DUE TO FORCES THAT CAUSE BENDING ALONG THE Y-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					
								LOAD FACTOR					
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0.5	*ROOF L.L.* 0	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0.2	TOTAL ΣP_{ui} (kips)	**STORY $B_{2i\_Y\text{-}AXIS}$**
18	13.0 ft	185 kips	0.25 in	1,513	0	288	1,591	1,815.6	0.0	0.0	318.2	2,133.8	**1.019**
17	13.0 ft	352 kips	0.39 in	2,821	1,008	288	3,187	3,385.2	504.0	0.0	637.4	4,526.6	**1.033**
16	13.0 ft	501 kips	0.53 in	4,129	2,016	288	4,783	4,954.8	1,008.0	0.0	956.6	6,919.4	**1.049**
15	13.0 ft	633 kips	0.58 in	5,437	3,024	288	6,379	6,524.4	1,512.0	0.0	1,275.8	9,312.2	**1.058**
14	13.0 ft	749 kips	0.55 in	6,745	4,032	288	7,975	8,094.0	2,016.0	0.0	1,595.0	11,705.0	**1.058**
13	13.0 ft	851 kips	0.53 in	8,053	5,040	288	9,571	9,663.6	2,520.0	0.0	1,914.2	14,097.8	**1.060**
12	13.0 ft	938 kips	0.55 in	9,361	6,048	288	11,167	11,233.2	3,024.0	0.0	2,233.4	16,490.6	**1.066**
11	13.0 ft	1,013 kips	0.58 in	10,669	7,056	288	12,763	12,802.8	3,528.0	0.0	2,552.6	18,883.4	**1.074**
10	13.0 ft	1,075 kips	0.61 in	11,977	8,064	288	14,359	14,372.4	4,032.0	0.0	2,871.8	21,276.2	**1.084**
9	13.0 ft	1,126 kips	0.62 in	13,285	9,072	288	15,955	15,942.0	4,536.0	0.0	3,191.0	23,669.0	**1.091**
8	13.0 ft	1,168 kips	0.58 in	14,593	10,080	288	17,551	17,511.6	5,040.0	0.0	3,510.2	26,061.8	**1.090**
7	13.0 ft	1,200 kips	0.56 in	15,901	11,088	288	19,147	19,081.2	5,544.0	0.0	3,829.4	28,454.6	**1.093**
6	13.0 ft	1,224 kips	0.55 in	17,209	12,096	288	20,743	20,650.8	6,048.0	0.0	4,148.6	30,847.4	**1.098**
5	13.0 ft	1,241 kips	0.56 in	18,517	13,104	288	22,339	22,220.4	6,552.0	0.0	4,467.8	33,240.2	**1.106**
4	13.0 ft	1,253 kips	0.56 in	19,825	14,112	288	23,935	23,790.0	7,056.0	0.0	4,787.0	35,633.0	**1.114**
3	13.0 ft	1,260 kips	0.53 in	21,133	15,120	288	25,531	25,359.6	7,560.0	0.0	5,106.2	38,025.8	**1.114**
2	13.0 ft	1,263 kips	0.48 in	22,441	16,128	288	27,127	26,929.2	8,064.0	0.0	5,425.4	40,418.6	**1.109**
1	13.0 ft	1,264 kips	0.24 in	23,749	17,136	288	28,723	28,498.8	8,568.0	0.0	5,744.6	42,811.4	**1.055**

GG Engineering	JOB NO. 18 - STORY BUILDINGS	BY SMG	DATE 4/22/05	SHEET NO. ___ OF ___
	CUSTOMER DESIGN 18C	CKD	DATE	

SUBJECT

STABILITY COEFFICIENT ALONG COLUMN X-AXIS, θ_x

MOMENT FRAME
MF A3 - G3

- ***LOAD COMBINATION:*** 1.2D + 0.5L + 1.0E
- ***DEFLECTION AMPLIFICATION FACTOR:*** C_d = 5.5
- ***(SEISMIC) IMPORTANCE FACTOR*** I_E = 1.0
- ***MOMENT FRAME RESISTS WHAT % OF THE TOTAL SEISMIC SHEAR TO THE BUIDLING?*** 25%

		DUE TO FORCES THAT CAUSE BENDING ALONG THE X-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					STABILITY COEFFICIENT PER STORY θ_i
								LOAD FACTOR				TOTAL ΣP_{ui} (kips)	
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO ANY HORIZONTAL LOAD ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0.5	*ROOF L.L.* 0	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0.2		
18	13.0 ft	185 kips	0.25 in	1,513	0	288	1,591	1,815.6	0.0	0.0	318.2	2,133.8	**0.018**
17	13.0 ft	352 kips	0.39 in	2,821	1,008	288	3,187	3,385.2	504.0	0.0	637.4	4,526.6	**0.032**
16	13.0 ft	501 kips	0.53 in	4,129	2,016	288	4,783	4,954.8	1,008.0	0.0	956.6	6,919.4	**0.047**
15	13.0 ft	633 kips	0.58 in	5,437	3,024	288	6,379	6,524.4	1,512.0	0.0	1,275.8	9,312.2	**0.055**
14	13.0 ft	749 kips	0.55 in	6,745	4,032	288	7,975	8,094.0	2,016.0	0.0	1,595.0	11,705.0	**0.055**
13	13.0 ft	851 kips	0.53 in	8,053	5,040	288	9,571	9,663.6	2,520.0	0.0	1,914.2	14,097.8	**0.056**
12	13.0 ft	938 kips	0.55 in	9,361	6,048	288	11,167	11,233.2	3,024.0	0.0	2,233.4	16,490.6	**0.062**
11	13.0 ft	1,013 kips	0.58 in	10,669	7,056	288	12,763	12,802.8	3,528.0	0.0	2,552.6	18,883.4	**0.069**
10	13.0 ft	1,075 kips	0.61 in	11,977	8,064	288	14,359	14,372.4	4,032.0	0.0	2,871.8	21,276.2	**0.077**
9	13.0 ft	1,126 kips	0.62 in	13,285	9,072	288	15,955	15,942.0	4,536.0	0.0	3,191.0	23,669.0	**0.084**
8	13.0 ft	1,168 kips	0.58 in	14,593	10,080	288	17,551	17,511.6	5,040.0	0.0	3,510.2	26,061.8	**0.083**
7	13.0 ft	1,200 kips	0.56 in	15,901	11,088	288	19,147	19,081.2	5,544.0	0.0	3,829.4	28,454.6	**0.085**
6	13.0 ft	1,224 kips	0.55 in	17,209	12,096	288	20,743	20,650.8	6,048.0	0.0	4,148.6	30,847.4	**0.089**
5	13.0 ft	1,241 kips	0.56 in	18,517	13,104	288	22,339	22,220.4	6,552.0	0.0	4,467.8	33,240.2	**0.096**
4	13.0 ft	1,253 kips	0.56 in	19,825	14,112	288	23,935	23,790.0	7,056.0	0.0	4,787.0	35,633.0	**0.102**
3	13.0 ft	1,260 kips	0.53 in	21,133	15,120	288	25,531	25,359.6	7,560.0	0.0	5,106.2	38,025.8	**0.103**
2	13.0 ft	1,263 kips	0.48 in	22,441	16,128	288	27,127	26,929.2	8,064.0	0.0	5,425.4	40,418.6	**0.098**
1	13.0 ft	1,264 kips	0.24 in	23,749	17,136	288	28,723	28,498.8	8,568.0	0.0	5,744.6	42,811.4	**0.052**

GG Engineering	JOB NO. 18 - STORY BUILDINGS	BY SMG	DATE 4/22/05	SHEET NO. ___ OF ___
	CUSTOMER DESIGN 18C	CKD	DATE	

SUBJECT

STABILITY COEFFICIENT ALONG COLUMN X-AXIS, θ_x

MOMENT FRAME
MF A3 - G3

- o ***LOAD COMBINATION:*** 1.2D + 0.5L + 1.0E
- o ***DEFLECTION AMPLIFICATION FACTOR:*** C_d = 5.5

		DUE TO FORCES THAT CAUSE BENDING ALONG THE X-AXIS OF THE MOMENT FRAME COLUMNS						
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH_i	INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	TOTAL STORY SHEAR CAPACITY (OF ALL OF THE SEISMIC RESISTING MOMENT FRAMES)	RATIO OF SHEAR DEMAND / SHEAR CAPACITY PER STORY β	MAXIMUM ALLOWED STABILITY COEFFICIENT PER STORY θ_{i_max}	STABILITY COEFFICIENT PER STORY θ_i	***COMMENT***
18	13.0 ft	185 kips	0.25 in	43,402 kips	0.0043	0.250	0.018	***OK***
17	13.0 ft	352 kips	0.39 in	43,402 kips	0.0081	0.250	0.032	***OK***
16	13.0 ft	501 kips	0.53 in	43,402 kips	0.0115	0.250	0.047	***OK***
15	13.0 ft	633 kips	0.58 in	65,313 kips	0.0097	0.250	0.055	***OK***
14	13.0 ft	749 kips	0.55 in	65,313 kips	0.0115	0.250	0.055	***OK***
13	13.0 ft	851 kips	0.53 in	65,313 kips	0.0130	0.250	0.056	***OK***
12	13.0 ft	938 kips	0.55 in	66,992 kips	0.0140	0.250	0.062	***OK***
11	13.0 ft	1,013 kips	0.58 in	66,992 kips	0.0151	0.250	0.069	***OK***
10	13.0 ft	1,075 kips	0.61 in	66,992 kips	0.0160	0.250	0.077	***OK***
9	13.0 ft	1,126 kips	0.62 in	66,992 kips	0.0168	0.250	0.084	***OK***
8	13.0 ft	1,168 kips	0.58 in	8,090 kips	0.1444	0.250	0.083	***OK***
7	13.0 ft	1,200 kips	0.56 in	8,090 kips	0.1483	0.250	0.085	***OK***
6	13.0 ft	1,224 kips	0.55 in	8,090 kips	0.1513	0.250	0.089	***OK***
5	13.0 ft	1,241 kips	0.56 in	8,090 kips	0.1534	0.250	0.096	***OK***
4	13.0 ft	1,253 kips	0.56 in	8,090 kips	0.1549	0.250	0.102	***OK***
3	13.0 ft	1,260 kips	0.53 in	8,090 kips	0.1557	0.250	0.103	***OK***
2	13.0 ft	1,263 kips	0.48 in	8,090 kips	0.1561	0.250	0.098	***OK***
1	13.0 ft	1,264 kips	0.24 in	8,090 kips	0.1562	0.250	0.052	***OK***

GG Engineering	JOB NO. 18 - STORY BUILDINGS	BY SMG	DATE 4/22/05	SHEET NO. ___ OF ___
	CUSTOMER DESIGN 18C	CKD	DATE	

SUBJECT

STABILITY COEFFICIENT ALONG COLUMN Y-AXIS, θ_y

MOMENT FRAME
MF A3 - G3

- ***LOAD COMBINATION:*** 1.2D + 0.5L + 1.0E
- ***DEFLECTION AMPLIFICATION FACTOR:*** C_d = 5.5
- ***(SEISMIC) IMPORTANCE FACTOR*** I_E = 1.0
- ***MOMENT FRAME RESISTS WHAT % OF THE TOTAL SEISMIC SHEAR TO THE BUIDLING?*** 25%

		DUE TO FORCES THAT CAUSE BENDING ALONG THE Y-AXIS OF THE MOMENT FRAME COLUMNS		TOTAL *UNFACTORED* AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) (kips)				TOTAL FACTORED AXIAL LOAD, ΣP_u, PER STORY					STABILITY COEFFICIENT PER STORY θ_i
								LOAD FACTOR					
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO ANY HORIZONTAL LOAD ΣH_i	ELASTIC INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	*D.L.* 1.2	*L.L.* 0.5	*ROOF L.L.* 0	*SEISMIC VERTICAL* $0.2S_{DS}$ = 0.2	TOTAL ΣP_{ui} (kips)	
18	13.0 ft	185 kips	0.25 in	1,513	0	288	1,591	1,815.6	0.0	0.0	318.2	2,133.8	**0.018**
17	13.0 ft	352 kips	0.39 in	2,821	1,008	288	3,187	3,385.2	504.0	0.0	637.4	4,526.6	**0.032**
16	13.0 ft	501 kips	0.53 in	4,129	2,016	288	4,783	4,954.8	1,008.0	0.0	956.6	6,919.4	**0.047**
15	13.0 ft	633 kips	0.58 in	5,437	3,024	288	6,379	6,524.4	1,512.0	0.0	1,275.8	9,312.2	**0.055**
14	13.0 ft	749 kips	0.55 in	6,745	4,032	288	7,975	8,094.0	2,016.0	0.0	1,595.0	11,705.0	**0.055**
13	13.0 ft	851 kips	0.53 in	8,053	5,040	288	9,571	9,663.6	2,520.0	0.0	1,914.2	14,097.8	**0.056**
12	13.0 ft	938 kips	0.55 in	9,361	6,048	288	11,167	11,233.2	3,024.0	0.0	2,233.4	16,490.6	**0.062**
11	13.0 ft	1,013 kips	0.58 in	10,669	7,056	288	12,763	12,802.8	3,528.0	0.0	2,552.6	18,883.4	**0.069**
10	13.0 ft	1,075 kips	0.61 in	11,977	8,064	288	14,359	14,372.4	4,032.0	0.0	2,871.8	21,276.2	**0.077**
9	13.0 ft	1,126 kips	0.62 in	13,285	9,072	288	15,955	15,942.0	4,536.0	0.0	3,191.0	23,669.0	**0.084**
8	13.0 ft	1,168 kips	0.58 in	14,593	10,080	288	17,551	17,511.6	5,040.0	0.0	3,510.2	26,061.8	**0.083**
7	13.0 ft	1,200 kips	0.56 in	15,901	11,088	288	19,147	19,081.2	5,544.0	0.0	3,829.4	28,454.6	**0.085**
6	13.0 ft	1,224 kips	0.55 in	17,209	12,096	288	20,743	20,650.8	6,048.0	0.0	4,148.6	30,847.4	**0.089**
5	13.0 ft	1,241 kips	0.56 in	18,517	13,104	288	22,339	22,220.4	6,552.0	0.0	4,467.8	33,240.2	**0.096**
4	13.0 ft	1,253 kips	0.56 in	19,825	14,112	288	23,935	23,790.0	7,056.0	0.0	4,787.0	35,633.0	**0.102**
3	13.0 ft	1,260 kips	0.53 in	21,133	15,120	288	25,531	25,359.6	7,560.0	0.0	5,106.2	38,025.8	**0.103**
2	13.0 ft	1,263 kips	0.48 in	22,441	16,128	288	27,127	26,929.2	8,064.0	0.0	5,425.4	40,418.6	**0.098**
1	13.0 ft	1,264 kips	0.24 in	23,749	17,136	288	28,723	28,498.8	8,568.0	0.0	5,744.6	42,811.4	**0.052**

GG Engineering	JOB NO. 18 - STORY BUILDINGS	BY SMG	DATE 4/22/05	SHEET NO. ___ OF ___
	CUSTOMER DESIGN 18C	CKD	DATE	

SUBJECT

STABILITY COEFFICIENT ALONG COLUMN Y-AXIS, θ_y

MOMENT FRAME
MF A3 - G3

- ***LOAD COMBINATION:*** 1.2D + 0.5L + 1.0E
- ***DEFLECTION AMPLIFICATION FACTOR:*** C_d = 5.5

		DUE TO FORCES THAT CAUSE BENDING ALONG THE Y-AXIS OF THE MOMENT FRAME COLUMNS						
STORY NUMBER	LENGTH *(STORY HEIGHT)* L	TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH_i	INTERSTORY DRIFT Δ_{oh} DUE TO ΣH_i	TOTAL STORY SHEAR CAPACITY (OF ALL OF THE SEISMIC RESISTING MOMENT FRAMES)	RATIO OF SHEAR DEMAND / SHEAR CAPACITY PER STORY β	MAXIMUM ALLOWED STABILITY COEFFICIENT PER STORY θ_{i_max}	STABILITY COEFFICIENT PER STORY θ_i	***COMMENT***
18	13.0 ft	185 kips	0.25 in	43,402 kips	0.0043	0.250	0.018	***OK***
17	13.0 ft	352 kips	0.39 in	43,402 kips	0.0081	0.250	0.032	***OK***
16	13.0 ft	501 kips	0.53 in	43,402 kips	0.0115	0.250	0.047	***OK***
15	13.0 ft	633 kips	0.58 in	65,313 kips	0.0097	0.250	0.055	***OK***
14	13.0 ft	749 kips	0.55 in	65,313 kips	0.0115	0.250	0.055	***OK***
13	13.0 ft	851 kips	0.53 in	65,313 kips	0.0130	0.250	0.056	***OK***
12	13.0 ft	938 kips	0.55 in	66,992 kips	0.0140	0.250	0.062	***OK***
11	13.0 ft	1,013 kips	0.58 in	66,992 kips	0.0151	0.250	0.069	***OK***
10	13.0 ft	1,075 kips	0.61 in	66,992 kips	0.0160	0.250	0.077	***OK***
9	13.0 ft	1,126 kips	0.62 in	66,992 kips	0.0168	0.250	0.084	***OK***
8	13.0 ft	1,168 kips	0.58 in	8,090 kips	0.1444	0.250	0.083	***OK***
7	13.0 ft	1,200 kips	0.56 in	8,090 kips	0.1483	0.250	0.085	***OK***
6	13.0 ft	1,224 kips	0.55 in	8,090 kips	0.1513	0.250	0.089	***OK***
5	13.0 ft	1,241 kips	0.56 in	8,090 kips	0.1534	0.250	0.096	***OK***
4	13.0 ft	1,253 kips	0.56 in	8,090 kips	0.1549	0.250	0.102	***OK***
3	13.0 ft	1,260 kips	0.53 in	8,090 kips	0.1557	0.250	0.103	***OK***
2	13.0 ft	1,263 kips	0.48 in	8,090 kips	0.1561	0.250	0.098	***OK***
1	13.0 ft	1,264 kips	0.24 in	8,090 kips	0.1562	0.250	0.052	***OK***

COLUMN		MAXIMUM INTERACTION VALUE	CONTROLLING LOAD COMBINATION	BUILDING MAX. INTERACTION	COMMENTS FOR: DESIGN 18C	
NAME	MEMBER SIZE			0.8061	COLUMN STEEL AREA CHECK	COLUMN COMPACTNESS CHECK
A3-1	HSS 24 x 24 x 0.3125	0.739105694	5		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
A3-2	HSS 24 x 24 x 0.3125	0.428418936	5		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
A3-3	HSS 24 x 24 x 0.3125	0.327372805	5		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
A3-4	HSS 24 x 24 x 0.3125	0.310403241	5		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
A3-5	HSS 24 x 24 x 0.3125	0.292356546	5		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
A3-6	HSS 24 x 24 x 0.3125	0.268046161	5		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
A3-7	HSS 24 x 24 x 0.3125	0.24652186	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
A3-8	HSS 24 x 24 x 0.3125	0.269021293	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
A3-9	HSS 22 x 22 x 0.3125	0.287723224	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
A3-10	HSS 22 x 22 x 0.3125	0.266550795	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
A3-11	HSS 22 x 22 x 0.3125	0.249523256	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
A3-12	HSS 22 x 22 x 0.3125	0.215422527	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
A3-13	HSS 21 x 21 x 0.3125	0.185073781	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
A3-14	HSS 21 x 21 x 0.3125	0.22153592	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
A3-15	HSS 21 x 21 x 0.3125	0.141467505	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
A3-16	HSS 18 x 18 x 0.25	0.239118206	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
A3-17	HSS 18 x 18 x 0.25	0.203495227	5		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
A3-18	HSS 18 x 18 x 0.25	0.240830076	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
B3-1	HSS 24 x 24 x 0.3125	0.607321688	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
B3-2	HSS 24 x 24 x 0.3125	0.473101027	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
B3-3	HSS 24 x 24 x 0.3125	0.413958326	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
B3-4	HSS 24 x 24 x 0.3125	0.391348622	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
B3-5	HSS 24 x 24 x 0.3125	0.376024546	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
B3-6	HSS 24 x 24 x 0.3125	0.363413376	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
B3-7	HSS 24 x 24 x 0.3125	0.358470984	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
B3-8	HSS 24 x 24 x 0.3125	0.37507842	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
B3-9	HSS 22 x 22 x 0.3125	0.386770596	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
B3-10	HSS 22 x 22 x 0.3125	0.380115986	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
B3-11	HSS 22 x 22 x 0.3125	0.355773156	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
B3-12	HSS 22 x 22 x 0.3125	0.324941602	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
B3-13	HSS 21 x 21 x 0.3125	0.309106373	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
B3-14	HSS 21 x 21 x 0.3125	0.323023282	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
B3-15	HSS 21 x 21 x 0.3125	0.248031684	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
B3-16	HSS 18 x 18 x 0.25	0.351486409	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
B3-17	HSS 18 x 18 x 0.25	0.245212104	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
B3-18	HSS 18 x 18 x 0.25	0.152407328	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
C3-1	HSS 24 x 24 x 0.3125	0.802098399	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
C3-2	HSS 24 x 24 x 0.3125	0.611388504	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
C3-3	HSS 24 x 24 x 0.3125	0.551673571	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
C3-4	HSS 24 x 24 x 0.3125	0.529573081	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
C3-5	HSS 24 x 24 x 0.3125	0.516653732	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
C3-6	HSS 24 x 24 x 0.3125	0.508987394	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
C3-7	HSS 24 x 24 x 0.3125	0.505109405	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
C3-8	HSS 24 x 24 x 0.3125	0.529187955	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
C3-9	HSS 22 x 22 x 0.3125	0.55397568	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
C3-10	HSS 22 x 22 x 0.3125	0.545520578	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
C3-11	HSS 22 x 22 x 0.3125	0.519732835	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
C3-12	HSS 22 x 22 x 0.3125	0.484613078	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
C3-13	HSS 21 x 21 x 0.3125	0.47030532	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
C3-14	HSS 21 x 21 x 0.3125	0.476636666	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
C3-15	HSS 21 x 21 x 0.3125	0.366139213	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
C3-16	HSS 18 x 18 x 0.25	0.521293534	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
C3-17	HSS 18 x 18 x 0.25	0.400934053	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
C3-18	HSS 18 x 18 x 0.25	0.269623961	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
D3-1	HSS 24 x 24 x 0.3125	0.618596464	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
D3-2	HSS 24 x 24 x 0.3125	0.469429565	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
D3-3	HSS 24 x 24 x 0.3125	0.424485356	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
D3-4	HSS 24 x 24 x 0.3125	0.407717148	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
D3-5	HSS 24 x 24 x 0.3125	0.397698947	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
D3-6	HSS 24 x 24 x 0.3125	0.392085501	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
D3-7	HSS 24 x 24 x 0.3125	0.388876066	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
D3-8	HSS 24 x 24 x 0.3125	0.40730987	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
D3-9	HSS 22 x 22 x 0.3125	0.427308735	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
D3-10	HSS 22 x 22 x 0.3125	0.420103669	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
D3-11	HSS 22 x 22 x 0.3125	0.400575365	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
D3-12	HSS 22 x 22 x 0.3125	0.373713447	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
D3-13	HSS 21 x 21 x 0.3125	0.362650685	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
D3-14	HSS 21 x 21 x 0.3125	0.366282704	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
D3-15	HSS 21 x 21 x 0.3125	0.281782968	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
D3-16	HSS 18 x 18 x 0.25	0.401805736	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
D3-17	HSS 18 x 18 x 0.25	0.309698822	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
D3-18	HSS 18 x 18 x 0.25	0.206741872	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
E3-1	HSS 24 x 24 x 0.3125	0.806085746	6	<---CONTROLS!	OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
E3-2	HSS 24 x 24 x 0.3125	0.610860687	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
E3-3	HSS 24 x 24 x 0.3125	0.553470556	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!

COLUMN		MAXIMUM INTERACTION VALUE	CONTROLLING LOAD COMBINATION	BUILDING MAX. INTERACTION	COMMENTS FOR: DESIGN 18C	
NAME	MEMBER SIZE			0.8061	COLUMN STEEL AREA CHECK	COLUMN COMPACTNESS CHECK
E3-4	HSS 24 x 24 x 0.3125	0.53255414	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
E3-5	HSS 24 x 24 x 0.3125	0.520725462	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
E3-6	HSS 24 x 24 x 0.3125	0.514241033	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
E3-7	HSS 24 x 24 x 0.3125	0.510031731	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
E3-8	HSS 24 x 24 x 0.3125	0.534585733	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
E3-9	HSS 22 x 22 x 0.3125	0.56239769	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
E3-10	HSS 22 x 22 x 0.3125	0.553371929	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
E3-11	HSS 22 x 22 x 0.3125	0.528846275	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
E3-12	HSS 22 x 22 x 0.3125	0.494646034	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
E3-13	HSS 21 x 21 x 0.3125	0.481220094	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
E3-14	HSS 21 x 21 x 0.3125	0.483904508	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
E3-15	HSS 21 x 21 x 0.3125	0.371578542	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
E3-16	HSS 18 x 18 x 0.25	0.538908611	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
E3-17	HSS 18 x 18 x 0.25	0.397168686	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
E3-18	HSS 18 x 18 x 0.25	0.427831883	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
F3-1	HSS 24 x 24 x 0.3125	0.617030273	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
F3-2	HSS 24 x 24 x 0.3125	0.475590692	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
F3-3	HSS 24 x 24 x 0.3125	0.429222755	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
F3-4	HSS 24 x 24 x 0.3125	0.412804392	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
F3-5	HSS 24 x 24 x 0.3125	0.40417452	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
F3-6	HSS 24 x 24 x 0.3125	0.398779724	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
F3-7	HSS 24 x 24 x 0.3125	0.396293824	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
F3-8	HSS 24 x 24 x 0.3125	0.415664487	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
F3-9	HSS 22 x 22 x 0.3125	0.435779437	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
F3-10	HSS 22 x 22 x 0.3125	0.431239855	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
F3-11	HSS 22 x 22 x 0.3125	0.411691157	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
F3-12	HSS 22 x 22 x 0.3125	0.385509733	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
F3-13	HSS 21 x 21 x 0.3125	0.378559374	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
F3-14	HSS 21 x 21 x 0.3125	0.377537156	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
F3-15	HSS 21 x 21 x 0.3125	0.289443267	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
F3-16	HSS 18 x 18 x 0.25	0.415953706	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
F3-17	HSS 18 x 18 x 0.25	0.333806397	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
F3-18	HSS 18 x 18 x 0.25	0.205374135	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
G3-1	HSS 24 x 24 x 0.3125	0.762250369	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
G3-2	HSS 24 x 24 x 0.3125	0.515001462	5		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
G3-3	HSS 24 x 24 x 0.3125	0.414836214	5		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
G3-4	HSS 24 x 24 x 0.3125	0.402446385	5		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
G3-5	HSS 24 x 24 x 0.3125	0.389418635	5		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
G3-6	HSS 24 x 24 x 0.3125	0.369687625	5		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
G3-7	HSS 24 x 24 x 0.3125	0.365042004	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
G3-8	HSS 24 x 24 x 0.3125	0.391198407	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
G3-9	HSS 22 x 22 x 0.3125	0.422562686	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
G3-10	HSS 22 x 22 x 0.3125	0.413509191	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
G3-11	HSS 22 x 22 x 0.3125	0.400334194	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
G3-12	HSS 22 x 22 x 0.3125	0.374817251	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
G3-13	HSS 21 x 21 x 0.3125	0.356603731	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
G3-14	HSS 21 x 21 x 0.3125	0.379187355	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
G3-15	HSS 21 x 21 x 0.3125	0.305362723	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
G3-16	HSS 18 x 18 x 0.25	0.479479217	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
G3-17	HSS 18 x 18 x 0.25	0.405458161	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!
G3-18	HSS 18 x 18 x 0.25	0.346718143	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	N.G. - STEEL HSS IS NOT COMPACT!

Appendix I

Design Example

The following example illustrates how the 2005 AISC specification (with the equations presented here in somewhat different format from the specification) was used to design a typical RCFT column. In particular this example demonstrates how column 'E3-1', in Building Design 18A, was designed. This column is located in the first story where column lines 'E' and '3' intersect (reference Figure 3.1.3.2). The remaining columns in this building were designed using the same design steps as illustrated in this example.

1. *Calculate the building geometries (story height, H, girder lengths, L, number of bays, B, etc).* Reference Appendix E for this example.

2. *Calculate the nominal gravity (dead and live) and environmental (wind and seismic) loads*. Reference Appendix F for this example.

3. *Determine the following design parameters for the column being designed*:
   a. *Yield strength of the column HSS, F_{yc},*
   b. *Concrete compressive strength of the column, f'_c,*
   c. *Modulus of elasticity of the column HSS and girders, E_s,*
   d. *Modulus of elasticity of the concrete, E_c,*
   e. *Height, H, of the story where the column will be located,*
   f. *(Average) length of the girders that are in the same story as the column,*
   g. *Story design seismic shear, V_i, for the LFRS with which the column is located.*

   - For this example the following parameters were used:
     - F_{yc} = 46 ksi
     - f'_c = 4 ksi
     - E_s = 29,000 ksi
     - E_c = 3,492 ksi
     - First story height, H = 13 feet
     - First story girder length, L = 20 feet
     - First story seismic shear, V_1 = 316 kips

4. *Reference Equation 3.3.3-1 to determine the maximum elastic interstory drift, Δ_e*:

$$\Delta_e = \frac{\delta_x I}{C_d} = \frac{(0.02*13'*12)(1.0)}{5.5} = 0.567'' \qquad \text{(I-1)}$$

5. *Calculate a target value of the flexural rigidity ratio, η, by referencing Equation 6.3-13 and using the known value of F_{yc}.* For this example assume a d/t of around 35. Therefore, from Equation 6.3-13 η = 0.8, but since the assumed d/t is larger than 22 (per Table 6.3.15) start with a value of 1.0.

6. *Use Equation L-6 through Equation L-8 from Appendix L to estimate an EI_{eff} for all of the columns in the story being analyzed.* In this example estimate the average value for all of the columns in the first story as shown below:

$$\mathrm{EI_{eff}} = \left. \left[\eta + \left(\frac{\mathrm{H}}{2\mathrm{L}}\right)\right]_1 \left[\frac{\mathrm{VLH^2}}{6\Delta_e}\right]_1 \middle/ (\mathrm{B}+1) \right. \tag{I-2a}$$

$$\mathrm{EI_{eff}} = \left. \left[1.0 + \left(\frac{13'}{(2)(20')}\right)\right]_1 \left[\frac{(316\mathrm{kips})(20'*12)(13'*12)^2}{6(0.567")}\right]_1 \middle/ (6+1) \right. \tag{I-2b}$$

$$\mathrm{EI_{eff}} = 102{,}690{,}685\ \mathrm{k-in^2}$$

7. *Estimate the required depth, d, of the RCFT column by iterating through Equation L-4 and Equation L-5 from Appendix L. Keep iterating until the value of EI_{eff} becomes close to the required value from Equation I-2.* For this example assume a d/t value of 35 and an HSS nominal wall thickness of (around) ½ inch. A depth of 20 inches will result in an approximate EI_{eff} value of 100,143,906 k-in². Therefore, try an HSS 20x20x1/2 which has a calculated EI_{eff} = 99,099,314 k-in². Note that column 'E3-1' is shared between two perpendicular moment frames so it needs to be a square HSS.

8. *Use Equation L-1 from Appendix L to estimate the plastic modulus of the column.*

$$\mathrm{Z_s} = 1.4\mathrm{d^2t} = 1.4(20")^2(0.5") = 280\ \mathrm{in^3} \tag{I-3}$$

9. *Use Equation M-1 from Appendix M to estimate the plastic modulus of each girder.*

$$\mathrm{Z_g} < \frac{\mathrm{Z_cF_{yc}}}{1.1\mathrm{R_yF_{yg}}} = \frac{(280\mathrm{in^3})(46\mathrm{ksi})}{1.1(1.1)(50\mathrm{ksi})} = 212\ \mathrm{in^3} \tag{I-4}$$

10. *Choose a preliminary girder size for the story based on the results of Step 9.* For the first story girders start out with a W24x68 which has a Z_x = 177 in³.

11. *Follow Steps 6 through 10 for the remaining stories in the building so that preliminary column and girder section sizes can be chosen for each story.*

12. *Once the preliminary column and girder sizes are chosen for each story, calculate the modified cross sectional area and modified moment of inertia for each RCFT column in the building by using Equations 3.3.2.3-1 and 3.3.2.3-2*. For this example by assuming a value of E′ = 29,000 ksi, the modified (elastic) area, $A_e = 81$ in² and the modified (elastic) moment of inertia, $I_e = 3{,}417$ in^4 for the HSS 20x20x1/2 column.

13. *Set up a computational model of a 2-D moment frame made up of the preliminary column and girder sizes and nominal loads. Perform an elastic analysis on each individual basic load case (dead load, live load, wind load, seismic load, etc.) so that the displacements and member forces can be calculated for each case separately.* Since this is an elastic analysis these member forces will be combined and factored later in the design process for each column and girder. The unfactored (nominal) forces at the end of column 'E3-1' are shown below:

NOMINAL (UNFACTORED) END FORCES ON COLUMN 'E3-1' FROM ELASTIC ANALYSIS						
BASIC LOAD CASE	FROM THE MOMENT FRAME BEING ANALYZED			FROM THE PERPENDICULAR MOMENT FRAME (COLUMN IS SHARED)		
	AXIAL (kips)	X-AXIS MOMENT (kip-in)		AXIAL (kips)	Y-AXIS MOMENT (kip-in)	
		@ TOP	@ BOT.		@ TOP	@ BOT.
DEAD LOAD	-662	15	-14	0	15	-14
LIVE LOAD	-472	10	-9	0	10	-9
ROOF LIVE LOAD	-10	0	0	0	0	0
SEISMIC WEIGHT	-797	18	-17	0	18	-17
WIND LOAD	-2	1,016	-2,150	-2	1,016	-2,150
SEISMIC LOAD	-6	2,466	-5,330	-2	740	-1,599

14. *Calculate the stability coefficients, θ, for each column by using Equation 3.2.2-1*. The values for column 'E3-1' have been calculated in Appendix F and are shown below for reference.

COLUMN 'E3-1' STABILITY COEFFICIENTS, θ FOR EACH LOAD COMBINATION		
LOAD COMBINATION	θx	θy
1	0.054	0.054
2	0.091	0.091
3	0.061	0.061
4	0.047	0.047
5	0.060	0.060
6	0.069	0.069

15. *Calculate the moment magnifiers, B_1 and B_2, by using Equations 3.2.3-1 and Equation 3.2.3-4*. The design value of each parameter for column 'E3-1' has been calculated in Appendix F and is shown below for reference.

COLUMN 'E3-1' MOMENT AMPLIFICATION FACTOR B_1 FOR EACH LOAD COMBINATION		
LOAD COMBINATION	B_{1x}	B_{1y}
1	1.000	1.000
2	1.000	1.000
3	1.000	1.000
4	1.021	1.021
5	1.027	1.027
6	1.031	1.031

COLUMN 'E3-1' MOMENT AMPLIFICATION FACTOR B_2 FOR EACH LOAD COMBINATION		
LOAD COMBINATION	B_{2x}	B_{2y}
4	1.049	1.049
5	1.064	1.064
6	1.075	1.075

16. *Combine the unfactored design forces from Step #13 using the moment magnifiers from Step #15 and the following load combinations to get the factored design forces:*

    1. $1.4D$
    2. $1.2D + 1.6L + 0.5L_R$
    3. $1.2D + 1.6L_R + f_1L$
    4. $1.2D + 1.6L_R + 0.8W$
    5. $1.2D + 1.6W + f_1L + 0.5\,L_R$
    6. $1.2D + 1.0E + f_1L$

    Where: $f_1 = 0.5$
    $E = \rho Q_E + 0.2S_{DS}D'$
    $\rho = 1.00$
    D' = seismic weight

FACTORED DESIGN FORCES ON COLUMN 'E3-1'			
LOAD COMBINATION	P_u (kips)	MOMENT (kip-in)	
		M_{ux}	M_{uy}
1	926	21	21
2	1,554	34	34
3	1,046	24	24
4	812	1,822	1,822
5	1,040	3,683	3,683
6	1,197	5,754	1,744

17. *Calculate the effective length factors, K_x and K_y, for each column in the building by using Equation 3.3.2.1-1.* For column 'E3-1' use the stability coefficients from Step #14, the factored axial load from Step #16, the value of EI_{eff} from Step #7, and a column length equal to the story height from Step #3. The effective length factors for each load combination are as follows:

COLUMN 'E3-1' EFFECTIVE LENGTH FACTORS K_x AND K_y FOR EACH LOAD COMBINATION		
LOAD COMBINATION	K_x	K_y
1	1.660	1.660
2	1.664	1.664
3	1.659	1.659
4	1.654	1.654
5	1.657	1.657
6	1.657	1.657

18. *Follow the steps listed in Table 3.3.2.2 to calculate the flexural strengths $\phi_b M_{nx}$ and $\phi_b M_{ny}$, and for each load combination follow Table 3.3.2.1 to calculate ϕP_n, for every column in the building*. Column 'E3-1' member strengths for the controlling load combination (combination #6) have been summarized below while the full calculations are at the end of this appendix.

- HSS size = HSS 20x20x1/2
  - $Z_x = Z_y = 274.55$ in³
  - $A_s = 37.9$ in²
  - $I_s = I_x = I_y = 2{,}367$ in^4
  - $A_c = 360$ in² *Reference Equation 3.3.2.2.2-1*
  - $I_c = 11{,}037$ in^4 *Reference Equation 3.3.2.2.2-2*
  - $E_s = 29{,}000$ ksi
  - $E_c = 3{,}492$ ksi
  - $K_x = K_y = 1.657$ *Reference table above*

$$\text{Since this is a square HSS: } \phi_b M_{nx} = \phi_b M_{ny} = \phi_b Z_s F_{yc} = 11{,}366 \text{ k} - \text{in} \tag{I-5}$$

$$P_o = A_s F_y + C_2 A_c f_c^{'} = 2{,}969 \text{ kips} \tag{I-6}$$

$$\text{Where: } C_2 = 0.85$$

$$\left(EI_{eff}\right)_x = \left(EI_{eff}\right)_y = E_s I_s + C_3 E_c I_c = 99{,}099{,}314 \text{ k} - \text{in}^2 \tag{I-7}$$

$$\text{Where: } C_3 = \min(0.6 + 2\left(\frac{A_s}{A_s + A_c}\right), 0.9) = 0.79$$

$$\text{Since this is a square HSS: } P_e = P_{ex} = P_{ey} = \pi^2 \left(EI_{eff}\right) / \left(KL\right)^2 = 14{,}638 \text{ kips} \tag{I-8}$$

$$\alpha_x = \alpha_y = \sqrt{\left(P_o / P_e\right)} = 0.45 \tag{I-9}$$

$$\text{Since } \alpha < 1.5: \quad \Lambda = 0.658^{\alpha^2} = 0.9187 \tag{I-10}$$

$$P_n = \Lambda P_o = 2{,}727 \text{ kips} \tag{I-11}$$

$$\text{Since this is a square HSS:}\quad \phi_c P_{nx} = \phi_c P_{ny} = (0.75)P_n = 2{,}046\text{ kips} \tag{I-12}$$

19. *Calculate the interaction value of each column for every load combination using the steps listed in Table 3.3.2.3*. The interaction values for column 'E3-1' for each load combination are listed below. The calculations for determining the interaction value of load combination #6 are illustrated after the summary table.

COLUMN 'E3-1' INTERACTION EQUATION RESULTS	
LOAD COMBINATION	INTERACTION VALUE
1	0.073
2	0.598
3	0.173
4	0.321
5	0.811
6	0.953

$$C_\lambda = \Lambda C = (0.9187)(A_c 0.85 f_c^{'}) = 1{,}125\text{ kips} \tag{I-13}$$

$$C_d = \phi_c C_\lambda = (0.75)(1{,}125\text{ kips}) = 844\text{ kips} \tag{I-14}$$

- Factored loads on column 'E3-1' for load combination #6 (from Step #16) are as follows:
  - P_u = 1,197 kips
  - M_{ux} = 5,754 k-in
  - M_{uy} = 1,744 kip-in

$$\text{Since } P_u > C_d:\quad \frac{P_u - C_d}{\phi_c P_n - C_d} + \frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} = 0.953 \tag{I-15}$$

20. *Once all of the columns and girders have been sized and their interaction values have been checked, a final interstory drift check needs to be performed for every story in the building*. In accordance with Step #4 and Equation I-1 the maximum elastic interstory drift for Building Design 18A is 0.567 inches. Appendix F lists the calculated elastic interstory drifts. From this list the calculated drift in the first story is 0.32 inches.

21. *Conclusion*: Since both the interaction values and the interstory drift in the first story are less than the allowed limits, the HSS 20x20x1/2 is okay to use as the RCFT column size in the first story of Building Design 18A.

GG Engineering	JOB NO. 18 - STORY BUILDINGS	BY SMG	DATE 10/28/04	SHEET NO.
	CUSTOMER DESIGN 18A	CKD	DATE	OF

SUBJECT

COLUMN 'E3-1' MEMBER STRENGTHS FOR LOAD COMBINATION #6

MOMENT FRAME MF A3 - G3

- LOAD COMBINATION: 1.2D + 0.5L + 1.0E *(L.C. # 6)*
- LATERAL UNBRACED LENGTH OF THE COLUMN: L_{xx} = 13.0 ft; L_{yy} = 13.0 ft
- YIELD STRENGTH: HSS, F_{yc} = 46 ksi; CONCRETE REINFORCEMENT, F_{yr} = 0 ksi
- MODULUS OF ELASTICITY: HSS, E_s = 29,000 ksi; CONCRETE REINFORCEMENT, E_{cr} = 29,000 ksi; CONCRETE, E_c = 3,492 ksi
- MINIMUM CONCRETE COMPRESSIVE STRENGTH: f'_c = 4.0 ksi
- CONCRETE DENSITY: w = 145 lb/ft³
- HSS (COLUMN) SECTION: HSS 20 x 20 x 0.5 *492 lb/ft WITH CONCRETE INCLUDED*

STEEL (SHELL):	CONCRETE (CORE):
(MIN) WIDTH, b = 20.0 in	(MIN) WIDTH, b = 19.0 in
(MAX) WIDTH, d = 20.0 in	(MAX) WIDTH, d = 19.0 in
WALL THICKNESS, t = 0.5 in *0.5 in*	AREA, A_c = 360.142 in²
AREA, A_s = 37.927 in²	I_{xx} = 11,037 in^4
I_{xx} = 2,367 in^4	I_{yy} = 11,037 in^4
I_{yy} = 2,367 in^4	r_{xx} = 5.54 in
r_{xx} = 7.9 in	r_{yy} = 5.54 in
r_{yy} = 7.9 in	Z_{xx} = 1,707 in³
Z_{xx} = 275 in³	Z_{yy} = 1,707 in³
Z_{yy} = 275 in³	

Column Properties for Elastic Analysis
$A_{VISUAL\ ANALYSIS} = (EA)_{eff} / E'$ = 81 in²
$I_{VISUAL\ ANALYSIS} = (EI)_{eff\_x} / E'$ = 3,417 in^4
When: $E_{VISUAL\ ANALYSIS} = E'$ = 29,000 ksi

- COMPOSITE COLUMN LIMITATIONS *(PER SECTION I2.2a OF the DRAFT 2005 AISC SPECIFICATION)*
  - MINIMUM (ALLOWED) A_s = 3.981 in²
  - % OF TOTAL AREA THAT THE HSS MAKES UP = 9.53% — *OK - STEEL AREA IS > 1% OF TOTAL AREA*
  - (ACTUAL) d / t = 40.000
  - (MAXIMUM ALLOWED) d / t = 56.745 — *OK - STEEL HSS IS COMPACT*

- COMPRESSIVE STRENGTH *(PER SECTION I2.2b OF the DRAFT 2005 AISC SPECIFICATION)*

C_2 = 0.85 — *FOR RCFT SECTIONS*
C_3 = 0.79 — *EQUATION (I2-17)*
P_o = 2,969 kips — *EQUATION (I2-15)*

XX-AXIS		CONTROLLING	YY-AXIS		
$(EI)_{eff}$ =	99,099,314 kip-in²		$(EI)_{eff}$ =	99,099,314 kip-in²	*EQUATION (I2-16)*
K_x =	1.657		K_y =	1.657	
P_e =	14,638 kips		P_e =	14,638 kips	*EQUATION (I2-4)*
Column Slenderness, α =	0.450		Column Slenderness, α =	0.450	*EQUATION (I2-2)*
P_n =	2,727 kips		P_n =	2,727 kips	*EQUATIONS (I2-7, I2-8, I2-9)*
ϕ_c =	0.75		ϕ_c =	0.75	*SECTION I4, p. I-16*
ϕP_n =	**2,046 kips**	*2,046 kips*	**ϕP_n =**	**2,046 kips**	

- FLEXURAL STRENGTH *(PER SECTION I4 OF the DRAFT 2005 AISC SPECIFICATION AND EQUATION C-I4-1 OF THE 1994 LRFD)*

XX-AXIS		YY-AXIS	
c_r =	0.0 in	c_r =	0.0 in
WIDTH, h_2 =	20.0 in	WIDTH, h_2 =	20.0 in
M_n =	12,629 kip-in	M_n =	12,629 kip-in
ϕ_b =	0.90	ϕ_b =	0.90
ϕM_n =	**11,366 kip-in**	**ϕM_n =**	**11,366 kip-in**

- AXIAL LOAD AT ANCHOR POINT 'C' *(PER SECTION I4 & TABLE C-I1.1 OF the DRAFT 2005 AISC COMMENTARY)*

C = 1,224 kips — *C = Pc FROM TABLE C-I1.1 FOR RECTANGULAR HSS*

XX-AXIS		CONTROLLING	YY-AXIS	
Column Slenderness, α =	0.450		Column Slenderness, α =	0.450
$\lambda C = C_\lambda$ =	1,125 kips		$\lambda C = C_\lambda$ =	1,125 kips
ϕ_c =	0.75		ϕ_c =	0.75
$C_d = \phi_c C_\lambda$ =	**844 kips**	*844 kips*	**$C_d = \phi_c C_\lambda$ =**	**844 kips**

Appendix J

Column Interaction Calculations Macro

This appendix consists of a Microsoft Excel macro that was written for this study so that the RCFT columns could be analyzed consistently for all of the building designs as well as in an expeditious way. This macro allows for numerous iterations to be made for an entire moment-resisting frame so that optimum column sizes could be chosen for each 9-story and 18-story building.

```
'                       ******************************************************
'                       *                                                    *
'                       *  Macro/VBA written per the proposed 2005 AISC Ch.I  *
'                       *  to assist in the design of composite rectangular   *
'                       *  HSS that are to be used as beam-columns in mid to  *
'                       *  low rise buildings.                                *
'                       *                                                    *
'                       *      Written in August 2004 by Steve Gartner        *
'                       *                                                    *
'                       ******************************************************

'
Dim I As Double
Dim J As Double
Dim K As Double

Dim Mrx As Double                         'Mrx = Mux per LRFD
Dim Mry As Double                         'Mry = Muy per LRFD
Dim Mcx As Double                         'Mcx = fbMnx per LRFD
Dim Mcy As Double                         'Mcy = fbMny per LRFD

Dim Pr As Double                          'Required Axial comp. strength, Pu, per LRFD
Dim Min_Pa As Double                      'Minimum design axial comp. strength from X & Y axis, per LRFD
Dim Min_Pc As Double                      'Minimum design axial comp. strength at Point C from the X & Y
                                          'axis, per LRFD
                                          'NOTE: Pc = Cd

Dim NumCol As Double                      'Number of columns in the moment frame that are being analyzed
Dim NumLC As Double                       'Number of load cases that are in the analysis

Dim ColumnNameMatrix() As String          'List of the name of each column in the moment frame
Dim ColumnLengthMatrix() As Double        'List of the length of each column in the moment frame
Dim ColumnMemberSizeMatrix() As String    'List of the HSS for each column in the moment frame
Dim FactoredLoadsMatrix() As Double       'Matrix of the factored loads from each load case for each column

Dim ColumnMemberSizeNumberMatrix() As Double
Dim ColumnStrengthsMatrix() As Double
Dim ColumnKxMatrix() As Double            'K factor along the x-axis of each column
Dim ColumnKyMatrix() As Double            'K factor along the y-axis of each column

Dim InteractionMatrix() As Double
Dim ColumnSteelAreaCheckMatrix() As String
Dim ColumnCompactnessCheckMatrix() As String
'____________________________________________________________________________________________________________

Sub CalculateColumnInteractionValues()

ClearCells
DeclareConstants
RedimMatrices
GetColumnData
CalculateColumnStrengths
CalculateInteractionValues

Sheets("FINAL SUMMARY").Select
Range("AH4").Select

End Sub
'____________________________________________________________________________________________________________

Sub ClearCells()

'Clears out all of the cells in the "Final Summary" Worksheet
Sheets("FINAL SUMMARY").Select
Range("B7:C500,I7:AF500,AJ7:CD500").Select
    Selection.ClearContents

End Sub
'____________________________________________________________________________________________________________

Sub DeclareConstants()

'Determine total number of columns in the moment frame
'that are being analyzed
Sheets("NOMINAL LDS").Select
Range("A1").Select
NumCol = Range("A6")

NumLC = 6                                               'Only 6 load combinations are analyzed

End Sub
```

```
'_______________________________________________________________________________________________

Sub RedimMatrices()

'Redimensions all of the matrices
ReDim ColumnNameMatrix(NumCol)
ReDim ColumnLengthMatrix(NumCol)
ReDim ColumnMemberSizeMatrix(NumCol)
ReDim FactoredLoadsMatrix(NumCol, 3 * NumLC)         'Matrix col. headings: Pu, Mux, Muy for each load case

ReDim ColumnMemberSizeNumberMatrix(NumCol)
ReDim ColumnStrengthsMatrix(NumCol, 6 * NumLC)       'Matrix col. headings: Min_Pa, fPnx, fMnx, fPny, fMny,
Min_Pc
ReDim ColumnKxMatrix(NumCol, NumLC)
ReDim ColumnKyMatrix(NumCol, NumLC)

ReDim InteractionMatrix(NumCol, NumLC)
ReDim ColumnSteelAreaCheckMatrix(NumCol)
ReDim ColumnCompactnessCheckMatrix(NumCol)

End Sub
'_______________________________________________________________________________________________

Sub GetColumnData()

'Copy and paste the column names and member sizes into the Final Summary worksheet
Sheets("NOMINAL LDS").Select
Range("A1").Select
For I = 1 To NumCol
    ColumnNameMatrix(I) = Range("B" & 7 + (I - 1) * 2)
    ColumnLengthMatrix(I) = Range("C" & 7 + (I - 1) * 2)
    ColumnMemberSizeNumberMatrix(I) = Range("D" & 7 + (I - 1) * 2)
    ColumnMemberSizeMatrix(I) = Range("E" & 7 + (I - 1) * 2)
Next I

Sheets("FINAL SUMMARY").Select
Range("A1").Select
For I = 1 To NumCol
    Range("B" & 6 + I) = ColumnNameMatrix(I)
    Range("C" & 6 + I) = ColumnMemberSizeMatrix(I)
Next I

'Copy and paste in the factored axial load and moments on each column of the moment frame
'for every load combination

For I = 1 To NumLC

    Sheets("B2 X-AXIS").Select
    Range("Z12").Select
    Range("Z12") = I                          'Apply load factors

    Sheets("FACTORED LOADS").Select

    For J = 1 To NumCol
        FactoredLoadsMatrix(J, 3 * (I - 1) + 1) = Range("CZ" & 6 + J)
        FactoredLoadsMatrix(J, 3 * (I - 1) + 2) = Range("DA" & 6 + J)
        FactoredLoadsMatrix(J, 3 * (I - 1) + 3) = Range("DB" & 6 + J)

        'Copy the calculated Kx and Ky values into a matrix for use later
        ColumnKxMatrix(J, I) = Range("DF" & 6 + J)
        ColumnKyMatrix(J, I) = Range("DG" & 6 + J)
    Next J

Next I

Sheets("FINAL SUMMARY").Select
Range("A1").Select
For I = 1 To NumLC
    For J = 1 To NumCol
        Range("I" & 6 + J) = FactoredLoadsMatrix(J, 1)
        Range("J" & 6 + J) = FactoredLoadsMatrix(J, 2)
        Range("K" & 6 + J) = FactoredLoadsMatrix(J, 3)
        Range("M" & 6 + J) = FactoredLoadsMatrix(J, 4)
        Range("N" & 6 + J) = FactoredLoadsMatrix(J, 5)
        Range("O" & 6 + J) = FactoredLoadsMatrix(J, 6)
        Range("Q" & 6 + J) = FactoredLoadsMatrix(J, 7)
        Range("R" & 6 + J) = FactoredLoadsMatrix(J, 8)
        Range("S" & 6 + J) = FactoredLoadsMatrix(J, 9)
        Range("U" & 6 + J) = FactoredLoadsMatrix(J, 10)
        Range("V" & 6 + J) = FactoredLoadsMatrix(J, 11)
        Range("W" & 6 + J) = FactoredLoadsMatrix(J, 12)
```

```
        Range("Y" & 6 + J) = FactoredLoadsMatrix(J, 13)
        Range("Z" & 6 + J) = FactoredLoadsMatrix(J, 14)
        Range("AA" & 6 + J) = FactoredLoadsMatrix(J, 15)
        Range("AC" & 6 + J) = FactoredLoadsMatrix(J, 16)
        Range("AD" & 6 + J) = FactoredLoadsMatrix(J, 17)
        Range("AE" & 6 + J) = FactoredLoadsMatrix(J, 18)
    Next J
Next I

End Sub
'________________________________________________________________________________________________________

Sub CalculateColumnStrengths()

Sheets("COLUMN STRENGTHS").Select
Range("AL58").Select
For I = 1 To NumLC
    For J = 1 To NumCol
        Range("AG2") = ColumnLengthMatrix(J)
        Range("AR16") = ColumnMemberSizeNumberMatrix(J)
        Range("Z46") = ColumnKxMatrix(J, I)
        Range("AV46") = ColumnKyMatrix(J, I)
        ColumnStrengthsMatrix(J, 1 + (I - 1) * 6) = Range("AJ51")       'Min_Pa
        ColumnStrengthsMatrix(J, 2 + (I - 1) * 6) = Range("Z51")        'fPnx
        ColumnStrengthsMatrix(J, 3 + (I - 1) * 6) = Range("Z58")        'fMnx
        ColumnStrengthsMatrix(J, 4 + (I - 1) * 6) = Range("AV51")       'fPny
        ColumnStrengthsMatrix(J, 5 + (I - 1) * 6) = Range("AV58")       'fMny
        ColumnStrengthsMatrix(J, 6 + (I - 1) * 6) = Range("AJ67")       'Min_Pc
        ColumnSteelAreaCheckMatrix(J) = Range("AM36")       'Checks column Steel Area
        ColumnCompactnessCheckMatrix(J) = Range("AM38")     'Checks column compactness
    Next J
Next I
Sheets("FINAL SUMMARY").Select
Range("A1").Select
For I = 1 To NumCol
    Range("BA" & 6 + I) = ColumnStrengthsMatrix(I, 6)
    Range("BB" & 6 + I) = ColumnStrengthsMatrix(I, 12)
    Range("BC" & 6 + I) = ColumnStrengthsMatrix(I, 18)
    Range("BD" & 6 + I) = ColumnStrengthsMatrix(I, 24)
    Range("BE" & 6 + I) = ColumnStrengthsMatrix(I, 30)
    Range("BF" & 6 + I) = ColumnStrengthsMatrix(I, 36)

    Range("BG" & 6 + I) = ColumnStrengthsMatrix(I, 2)
    Range("BH" & 6 + I) = ColumnStrengthsMatrix(I, 8)
    Range("BI" & 6 + I) = ColumnStrengthsMatrix(I, 14)
    Range("BJ" & 6 + I) = ColumnStrengthsMatrix(I, 20)
    Range("BK" & 6 + I) = ColumnStrengthsMatrix(I, 26)
    Range("BL" & 6 + I) = ColumnStrengthsMatrix(I, 32)

    Range("BM" & 6 + I) = ColumnStrengthsMatrix(I, 3)
    Range("BN" & 6 + I) = ColumnStrengthsMatrix(I, 9)
    Range("BO" & 6 + I) = ColumnStrengthsMatrix(I, 15)
    Range("BP" & 6 + I) = ColumnStrengthsMatrix(I, 21)
    Range("BQ" & 6 + I) = ColumnStrengthsMatrix(I, 27)
    Range("BR" & 6 + I) = ColumnStrengthsMatrix(I, 33)

    Range("BS" & 6 + I) = ColumnStrengthsMatrix(I, 4)
    Range("BT" & 6 + I) = ColumnStrengthsMatrix(I, 10)
    Range("BU" & 6 + I) = ColumnStrengthsMatrix(I, 16)
    Range("BV" & 6 + I) = ColumnStrengthsMatrix(I, 22)
    Range("BW" & 6 + I) = ColumnStrengthsMatrix(I, 28)
    Range("BX" & 6 + I) = ColumnStrengthsMatrix(I, 34)

    Range("BY" & 6 + I) = ColumnStrengthsMatrix(I, 5)
    Range("BZ" & 6 + I) = ColumnStrengthsMatrix(I, 11)
    Range("CA" & 6 + I) = ColumnStrengthsMatrix(I, 17)
    Range("CB" & 6 + I) = ColumnStrengthsMatrix(I, 23)
    Range("CC" & 6 + I) = ColumnStrengthsMatrix(I, 29)
    Range("CD" & 6 + I) = ColumnStrengthsMatrix(I, 35)

    Range("AO" & 6 + I) = ColumnKxMatrix(I, 1)
    Range("AP" & 6 + I) = ColumnKxMatrix(I, 2)
    Range("AQ" & 6 + I) = ColumnKxMatrix(I, 3)
    Range("AR" & 6 + I) = ColumnKxMatrix(I, 4)
    Range("AS" & 6 + I) = ColumnKxMatrix(I, 5)
    Range("AT" & 6 + I) = ColumnKxMatrix(I, 6)

    Range("AU" & 6 + I) = ColumnKyMatrix(I, 1)
    Range("AV" & 6 + I) = ColumnKyMatrix(I, 2)
    Range("AW" & 6 + I) = ColumnKyMatrix(I, 3)
    Range("AX" & 6 + I) = ColumnKyMatrix(I, 4)
```

```
    Range("AY" & 6 + I) = ColumnKyMatrix(I, 5)
    Range("AZ" & 6 + I) = ColumnKyMatrix(I, 6)
Next I
End Sub
'_____________________________________________________________________________________

Sub CalculateInteractionValues()

Sheets("FINAL SUMMARY").Select
For I = 1 To NumLC
    For J = 1 To NumCol
        Pr = FactoredLoadsMatrix(J, 3 * (I - 1) + 1)        'Pu
        Min_Pa = ColumnStrengthsMatrix(J, 1 + (I - 1) * 6)
        Min_Pc = ColumnStrengthsMatrix(J, 6 + (I - 1) * 6)
        Mrx = FactoredLoadsMatrix(J, 3 * (I - 1) + 2)       'Mux
        Mry = FactoredLoadsMatrix(J, 3 * (I - 1) + 3)       'Muy
        Mcx = ColumnStrengthsMatrix(J, 3 + (I - 1) * 6)
        Mcy = ColumnStrengthsMatrix(J, 5 + (I - 1) * 6)

        'Per Eqn's (C-I4-1a) and (C-I4-1b) of the AISC 2005 Commentary
        'for composite beam columns in compression
        If Pr < Min_Pc Then
            InteractionMatrix(J, I) = Mrx / Mcx + Mry / Mcy
        Else
            InteractionMatrix(J, I) = (Pr - Min_Pc) / (Min_Pa - Min_Pc) + Mrx / Mcx + Mry / Mcy
        End If

    Next J
Next I

Range("A1").Select
    For J = 1 To NumCol
        Range("L" & 6 + J) = InteractionMatrix(J, 1)            'Load Combination 1
        Range("P" & 6 + J) = InteractionMatrix(J, 2)            'Load Combination 2
        Range("T" & 6 + J) = InteractionMatrix(J, 3)            'Load Combination 3
        Range("X" & 6 + J) = InteractionMatrix(J, 4)            'Load Combination 4
        Range("AB" & 6 + J) = InteractionMatrix(J, 5)           'Load Combination 5
        Range("AF" & 6 + J) = InteractionMatrix(J, 6)           'Load Combination 6

        Range("AJ" & 6 + J) = ColumnSteelAreaCheckMatrix(J)     'Column Steel Area Check result
        Range("AK" & 6 + J) = ColumnCompactnessCheckMatrix(I)   'Column compactness Check result
    Next J

End Sub
'_____________________________________________________________________________________
```

Appendix K

Steel Tube Institute HSS Equations

The AISC design manual only lists cross sectional properties of available rectangular HSS sections that are made of ASTM A500 Grade B material. However, since ASTM A500 sections are limited to be no more than 64 inches in periphery, cross sectional property equations were required for the HSSs that have perimeters greater than 64 inches so that their cross sectional properties could be calculated. This study used the same Steel Tube Institute (STI) cross sectional property equations for rectangular HSS as the AISC uses in their design tables. Reference STI (1996) for any equations and design information that is not listed in this Appendix.

Section 3.3.2.2.1 of this study provides a summary of the manufacturing processes of HSS and how the outside corner radius, r, and *design* wall thickness, t, are determined for a particular HSS. Once r and t are calculated for a cross section the equations shown in this appendix were used to calculate the area, A_s, the moment of inertia, I_x, and the plastic modulus, Z_x, for each steel section. Following these three equations are tables that list these three cross sectional properties for square HSSs between 8 inches and 48 inches in depth.

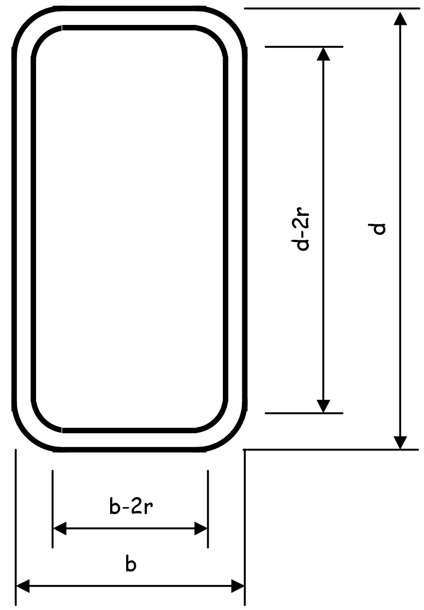

Figure K.1: Typical HSS cross section
(*same as Figure 3.3.2.2.2.1*).

$$A_s = 2t(b + d - 4r) + \pi t(2r - t) \tag{K-1}$$

$$\begin{aligned} I_x &= (b-2r)\frac{t^3}{6} + t(b-2r)\frac{(d-t)^2}{2} + t\frac{(d-2r)^3}{6} + 4\left[r^4 - (r-t)^4\right]\left[\frac{\pi}{16} - \frac{4}{9\pi}\right] + \ldots \\ &\ldots + \pi r^2\left[\frac{(d-2r)}{2} + \frac{4r}{3\pi}\right]^2 - \pi(r-t)^2\left[\frac{(d-2r)}{2} + \frac{4(r-t)}{3\pi}\right]^2 \end{aligned} \tag{K-2}$$

$$\begin{aligned} Z_x &= t(b-2r)(d-t) + \frac{t}{2}(d-2r)^2 + \pi r^2\left[\frac{d}{2} - r + \frac{4r}{3\pi}\right] - \ldots \\ &\ldots - \pi(r-t)^2\left[\frac{d}{2} - r + \frac{4(r-t)}{3\pi}\right] \end{aligned} \tag{K-3}$$

Section No.	HSS NAME	t_{NOM} (in)	DEPTH d (in)	WIDTH b (in)	t_{DES} (in)	Outside Corner Radius r (in)	As (in^2)	Ix (in^4)	Zx (in^3)
1	HSS 48 x 48 x 1	1	48	48	1.000	3.600	182.68	66,174	3,186
2	HSS 48 x 48 x 0.75	3/4	48	48	0.750	2.700	138.76	51,030	2,440
3	HSS 48 x 48 x 0.625	5/8	48	48	0.625	2.250	116.36	43,111	2,054
4	HSS 48 x 48 x 0.5	1/2	48	48	0.500	1.500	93.93	35,111	1,667
5	HSS 48 x 48 x 0.375	3/8	48	48	0.375	1.125	70.83	26,660	1,261
6	HSS 46 x 46 x 1	1	46	46	1.000	3.600	174.68	57,958	2,915
7	HSS 46 x 46 x 0.75	3/4	46	46	0.750	2.700	132.76	44,752	2,235
8	HSS 46 x 46 x 0.625	5/8	46	46	0.625	2.250	111.36	37,832	1,882
9	HSS 46 x 46 x 0.5	1/2	46	46	0.500	1.500	89.93	30,836	1,528
10	HSS 46 x 46 x 0.375	3/8	46	46	0.375	1.125	67.83	23,427	1,157
11	HSS 44 x 44 x 1	1	44	44	1.000	3.600	166.68	50,451	2,657
12	HSS 44 x 44 x 0.75	3/4	44	44	0.750	2.700	126.76	39,011	2,039
13	HSS 44 x 44 x 0.625	5/8	44	44	0.625	2.250	106.36	33,001	1,718
14	HSS 44 x 44 x 0.5	1/2	44	44	0.500	1.500	85.93	26,923	1,396
15	HSS 44 x 44 x 0.375	3/8	44	44	0.375	1.125	64.83	20,466	1,057
16	HSS 42 x 42 x 1	1	42	42	1.000	3.600	158.68	43,622	2,410
17	HSS 42 x 42 x 0.75	3/4	42	42	0.750	2.700	120.76	33,783	1,852
18	HSS 42 x 42 x 0.625	5/8	42	42	0.625	2.250	101.36	28,600	1,561
19	HSS 42 x 42 x 0.5	1/2	42	42	0.500	1.500	81.93	23,356	1,269
20	HSS 42 x 42 x 0.375	3/8	42	42	0.375	1.125	61.83	17,766	962
21	HSS 40 x 40 x 1	1	40	40	1.000	3.600	150.68	37,438	2,175
22	HSS 40 x 40 x 0.75	3/4	40	40	0.750	2.700	114.76	29,044	1,673
23	HSS 40 x 40 x 0.625	5/8	40	40	0.625	2.250	96.36	24,609	1,412
24	HSS 40 x 40 x 0.5	1/2	40	40	0.500	1.500	77.93	20,119	1,149
25	HSS 40 x 40 x 0.375	3/8	40	40	0.375	1.125	58.83	15,315	871
26	HSS 38 x 38 x 1	1	38	38	1.000	3.600	142.68	31,867	1,953
27	HSS 38 x 38 x 0.75	3/4	38	38	0.750	2.700	108.76	24,771	1,504
28	HSS 38 x 38 x 0.625	5/8	38	38	0.625	2.250	91.36	21,008	1,270
29	HSS 38 x 38 x 0.5	1/2	38	38	0.500	1.500	73.93	17,195	1,034
30	HSS 38 x 38 x 0.375	3/8	38	38	0.375	1.125	55.83	13,100	785
31	HSS 36 x 36 x 1	1	36	36	1.000	3.600	134.68	26,878	1,742
32	HSS 36 x 36 x 0.75	3/4	36	36	0.750	2.700	102.76	20,938	1,344
33	HSS 36 x 36 x 0.625	5/8	36	36	0.625	2.250	86.36	17,776	1,136
34	HSS 36 x 36 x 0.5	1/2	36	36	0.500	1.500	69.93	14,569	926
35	HSS 36 x 36 x 0.375	3/8	36	36	0.375	1.125	52.83	11,109	703
36	HSS 34 x 34 x 1	1	34	34	1.000	3.600	126.68	22,438	1,543
37	HSS 34 x 34 x 0.75	3/4	34	34	0.750	2.700	96.76	17,522	1,193
38	HSS 34 x 34 x 0.625	5/8	34	34	0.625	2.250	81.36	14,893	1,009
39	HSS 34 x 34 x 0.5	1/2	34	34	0.500	1.500	65.93	12,225	824
40	HSS 34 x 34 x 0.375	3/8	34	34	0.375	1.125	49.83	9,332	626
41	HSS 32 x 32 x 1	1	32	32	1.000	3.600	118.68	18,515	1,357
42	HSS 32 x 32 x 0.75	3/4	32	32	0.750	2.700	90.76	14,499	1,051
43	HSS 32 x 32 x 0.625	5/8	32	32	0.625	2.250	76.36	12,341	890
44	HSS 32 x 32 x 0.5	1/2	32	32	0.500	1.500	61.93	10,148	727
45	HSS 32 x 32 x 0.375	3/8	32	32	0.375	1.125	46.83	7,754	553
46	HSS 30 x 30 x 1	1	30	30	1.000	3.600	110.68	15,078	1,182
47	HSS 30 x 30 x 0.75	3/4	30	30	0.750	2.700	84.76	11,846	918
48	HSS 30 x 30 x 0.625	5/8	30	30	0.625	2.250	71.36	10,097	778
49	HSS 30 x 30 x 0.5	1/2	30	30	0.500	1.500	57.93	8,319	637
50	HSS 30 x 30 x 0.375	3/8	30	30	0.375	1.125	43.83	6,366	485

Section No.	HSS NAME	t_{NOM} (in)	DEPTH d (in)	WIDTH b (in)	t_{DES} (in)	Outside Corner Radius r (in)	As (in^2)	Ix (in^4)	Zx (in^3)
51	HSS 28 x 28 x 1	1	28	28	1.000	3.600	102.68	12,094	1,019
52	HSS 28 x 28 x 0.75	3/4	28	28	0.750	2.700	78.76	9,537	794
53	HSS 28 x 28 x 0.625	5/8	28	28	0.625	2.250	66.36	8,144	674
54	HSS 28 x 28 x 0.5	1/2	28	28	0.500	1.500	53.93	6,725	552
55	HSS 28 x 28 x 0.375	3/8	28	28	0.375	1.125	40.83	5,153	421
56	HSS 26 x 26 x 1	1	26	26	1.000	3.600	94.68	9,532	869
57	HSS 26 x 26 x 0.75	3/4	26	26	0.750	2.700	72.76	7,549	678
58	HSS 26 x 26 x 0.625	5/8	26	26	0.625	2.250	61.36	6,460	577
59	HSS 26 x 26 x 0.5	1/2	26	26	0.500	1.500	49.93	5,349	474
60	HSS 26 x 26 x 0.375	3/8	26	26	0.375	1.125	37.83	4,106	362
61	HSS 26 x 26 x 0.3125	5/16	26	26	0.313	0.938	31.69	3,461	304
62	HSS 26 x 26 x 0.25	1/4	26	26	0.250	0.750	25.48	2,801	245
63	HSS 26 x 26 x 0.1875	3/16	26	26	0.188	0.563	19.21	2,124	185
64	HSS 24 x 24 x 1	1	24	24	1.000	3.600	86.68	7,358	730
65	HSS 24 x 24 x 0.75	3/4	24	24	0.750	2.700	66.76	5,858	572
66	HSS 24 x 24 x 0.625	5/8	24	24	0.625	2.250	56.36	5,025	487
67	HSS 24 x 24 x 0.5	1/2	24	24	0.500	1.500	45.93	4,174	401
68	HSS 24 x 24 x 0.375	3/8	24	24	0.375	1.125	34.83	3,211	307
69	HSS 24 x 24 x 0.3125	5/16	24	24	0.313	0.938	29.19	2,709	258
70	HSS 24 x 24 x 0.25	1/4	24	24	0.250	0.750	23.48	2,194	208
71	HSS 24 x 24 x 0.1875	3/16	24	24	0.188	0.563	17.71	1,666	158
72	HSS 22 x 22 x 1	1	22	22	1.000	3.600	78.68	5,543	603
73	HSS 22 x 22 x 0.75	3/4	22	22	0.750	2.700	60.76	4,441	475
74	HSS 22 x 22 x 0.625	5/8	22	22	0.625	2.250	51.36	3,820	406
75	HSS 22 x 22 x 0.5	1/2	22	22	0.500	1.500	41.93	3,185	335
76	HSS 22 x 22 x 0.375	3/8	22	22	0.375	1.125	31.83	2,456	256
77	HSS 22 x 22 x 0.3125	5/16	22	22	0.313	0.938	26.69	2,075	216
78	HSS 22 x 22 x 0.25	1/4	22	22	0.250	0.750	21.48	1,683	174
79	HSS 22 x 22 x 0.1875	3/16	22	22	0.188	0.563	16.21	1,279	132
80	HSS 20 x 20 x 1	1	20	20	1.000	3.600	70.68	4,052	489
81	HSS 20 x 20 x 0.75	3/4	20	20	0.750	2.700	54.76	3,272	387
82	HSS 20 x 20 x 0.625	5/8	20	20	0.625	2.250	46.36	2,825	331
83	HSS 20 x 20 x 0.5	1/2	20	20	0.500	1.500	37.93	2,367	275
84	HSS 20 x 20 x 0.375	3/8	20	20	0.375	1.125	28.83	1,830	211
85	HSS 20 x 20 x 0.3125	5/16	20	20	0.313	0.938	24.19	1,548	178
86	HSS 20 x 20 x 0.25	1/4	20	20	0.250	0.750	19.48	1,257	144
87	HSS 20 x 20 x 0.1875	3/16	20	20	0.188	0.563	14.71	957	109
88	HSS 18 x 18 x 1	1	18	18	1.000	3.600	62.68	2,855	386
89	HSS 18 x 18 x 0.75	3/4	18	18	0.750	2.700	48.76	2,328	308
90	HSS 18 x 18 x 0.625	5/8	18	18	0.625	2.250	41.36	2,020	264
91	HSS 18 x 18 x 0.5	1/2	18	18	0.500	1.500	33.93	1,702	220
92	HSS 18 x 18 x 0.375	3/8	18	18	0.375	1.125	25.83	1,321	169
93	HSS 18 x 18 x 0.3125	5/16	18	18	0.313	0.938	21.69	1,119	143
94	HSS 18 x 18 x 0.25	1/4	18	18	0.250	0.750	17.48	911	116
95	HSS 18 x 18 x 0.1875	3/16	18	18	0.188	0.563	13.21	694	88
96	HSS 16 x 16 x 1	1	16	16	0.930	1.860	53.83	1,994	300
97	HSS 16 x 16 x 0.75	3/4	16	16	0.698	1.395	41.44	1,592	235
98	HSS 16 x 16 x 0.625	5/8	16	16	0.581	1.163	34.98	1,368	201
99	HSS 16 x 16 x 0.5	1/2	16	16	0.465	0.930	28.34	1,128	164
100	HSS 16 x 16 x 0.375	3/8	16	16	0.349	0.698	21.52	872	126

Section No.	HSS NAME	t_{NOM} (in)	DEPTH d (in)	WIDTH b (in)	t_{DES} (in)	Outside Corner Radius r (in)	As (in^2)	Ix (in^4)	Zx (in^3)
101	HSS 16 x 16 x 0.3125	5/16	16	16	0.291	0.581	18.04	738	106
102	HSS 16 x 16 x 0.25	1/4	16	16	0.233	0.465	14.52	599	86
103	HSS 16 x 16 x 0.125	1/8	16	16	0.116	0.233	7.35	308	44
104	HSS 14 x 14 x 1	1	14	14	0.930	1.860	46.39	1,288	224
105	HSS 14 x 14 x 0.75	3/4	14	14	0.698	1.395	35.86	1,039	177
106	HSS 14 x 14 x 0.625	5/8	14	14	0.581	1.163	30.33	897	151
107	HSS 14 x 14 x 0.5	1/2	14	14	0.465	0.930	24.62	743	124
108	HSS 14 x 14 x 0.375	3/8	14	14	0.349	0.698	18.73	577	95
109	HSS 14 x 14 x 0.3125	5/16	14	14	0.291	0.581	15.72	489	80
110	HSS 14 x 14 x 0.25	1/4	14	14	0.233	0.465	12.66	398	65
111	HSS 14 x 14 x 0.125	1/8	14	14	0.116	0.233	6.42	206	33
112	HSS 12 x 12 x 1	1	12	12	0.930	1.860	38.95	772	158
113	HSS 12 x 12 x 0.75	3/4	12	12	0.698	1.395	30.28	631	126
114	HSS 12 x 12 x 0.625	5/8	12	12	0.581	1.163	25.68	548	109
115	HSS 12 x 12 x 0.5	1/2	12	12	0.465	0.930	20.90	457	90
116	HSS 12 x 12 x 0.375	3/8	12	12	0.349	0.698	15.94	357	69
117	HSS 12 x 12 x 0.3125	5/16	12	12	0.291	0.581	13.39	304	58
118	HSS 12 x 12 x 0.25	1/4	12	12	0.233	0.465	10.80	248	47
119	HSS 12 x 12 x 0.125	1/8	12	12	0.116	0.233	5.49	129	24
120	HSS 10 x 10 x 1	1	10	10	0.930	1.860	31.51	416	104
121	HSS 10 x 10 x 0.75	3/4	10	10	0.698	1.395	24.70	347	85
122	HSS 10 x 10 x 0.625	5/8	10	10	0.581	1.163	21.03	304	73
123	HSS 10 x 10 x 0.5	1/2	10	10	0.465	0.930	17.18	256	61
124	HSS 10 x 10 x 0.375	3/8	10	10	0.349	0.698	13.15	202	47
125	HSS 10 x 10 x 0.3125	5/16	10	10	0.291	0.581	11.07	172	40
126	HSS 10 x 10 x 0.25	1/4	10	10	0.233	0.465	8.94	141	33
127	HSS 10 x 10 x 0.125	1/8	10	10	0.116	0.233	4.56	74	17
128	HSS 8 x 8 x 1	1	8	8	0.930	1.860	24.07	190	62
129	HSS 8 x 8 x 0.75	3/4	8	8	0.698	1.395	19.12	164	51
130	HSS 8 x 8 x 0.625	5/8	8	8	0.581	1.163	16.38	146	45
131	HSS 8 x 8 x 0.5	1/2	8	8	0.465	0.930	13.46	125	37
132	HSS 8 x 8 x 0.375	3/8	8	8	0.349	0.698	10.36	100	29
133	HSS 8 x 8 x 0.3125	5/16	8	8	0.291	0.581	8.74	86	25
134	HSS 8 x 8 x 0.25	1/4	8	8	0.233	0.465	7.08	71	20
135	HSS 8 x 8 x 0.125	1/8	8	8	0.116	0.233	3.63	37	11

Appendix L

Methods for Calculating a Preliminary EI_{eff}

To assist in sizing RCFT columns, equations were developed for this study that allowed for the major cross sectional properties to be quickly estimated. By using these equations an approximate EI_{eff} can be calculated using only the modulus of elasticity of the concrete and the steel plus the depth and thickness of the HSS. An estimate of the required EI_{eff} for a column can also be calculated if the geometry of the building, story drift, design story shear, and the value of the desired flexural rigidity ratio, η, for the building are known. This appendix summarizes these approximating equations in addition to some supporting equations that can be used when estimating RCFT column sizes.

The following approximations were used in the initial sizing of the RCFT column sections.

Note that these equations are only good for <u>*square*</u> *RCFT sections.*

- For the steel portion of the RCFT:

$$Z_s \approx 1.4d^2t \qquad \text{(L-1)}$$

$$I_s \approx \frac{Z_s d}{2.4} \cong 0.6td^3 \qquad \text{(L-2)}$$

- For the concrete portion of the RCFT:

$$I_c \approx \frac{d^4}{12} - I_s \qquad \text{(L-3)}$$

- ***Approximating EI$_{eff}$***

$$EI_{eff} \approx E_c' \frac{d^4}{12} + 0.6d^3t\left(E_s - E_c'\right) \tag{L-4}$$

$$\text{Where:} \quad E_c' = \min\left(0.6 + \frac{8t}{d}, 0.9\right)E_c \tag{L-5}$$

- ***Estimating the Required EI$_{eff}$***

If the maximum allowed elastic story drift, the design story shear, story height, the average length of the girders in the story, and the number of bays are known then the average required EI_{eff} for a story can be determined. This estimation is based on using a known (or desired) flexural rigidity ratio, η, for the building. Reference Table 6.3.14 for average values of η based on the design values of F_{yc} and f'_c. Once the total required EI_{eff} for a story, ΣEI_{eff}, is estimated a required value of EI_{eff} for a column can be determined by taking the average of ΣEI_{eff} for the story.

$$\left(\Sigma EI_{eff}\right)_i = \left[\eta + \left(\frac{H}{2L}\right)\right]_i \left[\frac{VLH^2}{6\Delta}\right]_i \tag{L-6}$$

$$\left(EI_{eff}\right)_{ave} = \frac{\left(\Sigma EI_{eff}\right)_i}{B+1} \tag{L-7}$$

$$EI_{eff} = \left(EI_{eff}\right)_{ave} \tag{L-8}$$

Where: η = (desired) flexural rigidity ratio (Table 6.3.14 or Equation 6.3-13)
H = story height
L = average girder length in the story
V = design story shear
Δ = maximum allowed elastic story drift
B = number of bays in the story

Appendix M

Calculating the Baseline Flexural Rigidity Ratios

The flexural rigidity ratio method provides a way to measure the amount of overstrength of a building that is made of RCFT columns. Overstrength can be measured by relating the actual flexural rigidity ratio, η, of the building to a set of baseline values of η. Each baseline value is calibrated from a set of buildings that were designed as liberally as possible, but still within the applicable building code limits (interstory drift, seismic loading and distribution, etc.). These baseline buildings were designed as close to the allowed limits as possible so that they would result in having the smallest possible overstrength factor. Once the baseline value of η is determined for a particular pair of F_{yc} and f'_c, the actual value of η for a building can be compared to this baseline value to determine if the building is close to or significantly stronger than its comparable optimized building.

The calibration of the baseline values of η required 135 moment frames to be designed so that the full range of possible d/t and column material strengths would be covered. The baseline values were divided into three d/t categories where each category used the same d/t limits as were used to design the original thirteen buildings of this study. The first category was for relatively low d/t ratio values (between 20 and 40). The second category was for d/t ratios at or just under the AISC limit of less than or equal to $2.26\sqrt{(E/Fy)}$. The third category was for d/t ratios at or just under 80. Nine baseline values were then established for each of these three d/t categories to represent the center point and the eight outer edge limits of the envelope of possible F_{yc} and f'_c design values (Reference Figure 6.3.1). At each of these 27 different design points (nine different combinations of F_{yc} and f'_c for each of the three different d/t categories) five buildings were designed using the same value of F_{yc} and f'_c. Once their column and girder sizes were chosen their individual η value was calculated. Three of these five buildings used the 3-story building design loads and geometries (an office building, a warehouse with 30 foot bay spacing, and a warehouse with 20 foot bay spacing), while the fourth building used the 9-story building layout and the fifth building used the 18-story building layout. The mean η value from these five building η values was used as the baseline value for a particular F_{yc} and f'_c data point and d/t ratio limit category.

The strength of each column and girder was approximated rather than actually designed according to the member strength requirements of AISC. This was done since it was found during the design of the original thirteen buildings that the buildings were all controlled by the building code interstory drift limits rather than the AISC strength requirements. Therefore these 135 moment frames were not designed by comparing the calculated forces in each member to the allowable strengths of the AISC specification. Instead they were designed through an eleven-step process that allowed for the columns and girders to be sized based only on staying within the drift limits of the building code and only approximating the member strengths.

The eleven design steps that were used to size the column and girder sizes assumes that the story height, H, the girder lengths, L, the bay spacing, B, the story seismic shear, V_i, and the elastic interstory drift limit, Δ_i, are all known. Using these five known parameters the most optimum column and girder sections sizes can be determined for a particular pair of F_{yc} and f'_c design values and for the required d/t ratio.

1. *Limit the allowable HSS wall thickness to 0.375 inches, 0.5 inches, 0.625 inches, 0.75 inches, and 1 inch based on known availability of plates for HSS of this size.*

2. *Choose a column depth, assuming a square column section.*

3. *Calculate an approximate HSS plastic modulus for each wall thickness using Equation L-1.*

4. *Calculate the maximum allowed girder plastic modulus, Z_g, for each column wall thickness from Step 1.* This step is based on using the strong column/weak beam requirement (Reference Equation 3.3.1-1) assuming that the two girders that connect to either side of a column have approximately the same plastic modulus, and the two column sections above and below a floor level also have approximately the same plastic modulus. The result is shown in Equation M-1.

$$Z_g < \frac{Z_c F_{yc}}{1.1 R_y F_{yg}} \tag{M-1}$$

5. *Calculate the flexural rigidity, EI_{eff}, for each column wall thickness using Equation L-4.*

6. *Calculate the flexural stiffness, K_{col}, for each column wall thickness.*

$$K_{col} = \frac{EI_{eff}}{H} \tag{M-2}$$

7. *Calculate the story total flexural stiffness, ΣK_{col}.*

8. *Calculate the minimum required (elastic) story stiffness, K_i.*

$$K_i = \frac{V_i}{\Delta_i} \tag{M-3}$$

9. *Calculate η for each column wall thickness using Equation 6.3-4.*

10. *Iterate through different column depths until either the lightest weight column section is chosen (for d/t ratios less than or equal to the AISC limit), or until the d/t ratio is at or just under a value of 80 (for the d/t ≈ 80 category).*

11. *Assign the η value that is associated with the chosen column size (assuming that all columns on each story are the same section size) as the story level value of η.*

12. *Calculate the maximum allowed girder depth, d_g, using a modified version of Equation 6.3-3 as shown in Equation M-4.*

$$d_g = \left(\frac{B+1}{B}\right)\left[\left(\frac{d_c}{\eta}\right)\left(\frac{R_y F_{yg}}{F_{yc}}\right)\left(\frac{0.6EI_{eff}}{E_s I_{c\_s}}\right)\right]_i \tag{M-4}$$

Note: It was assumed in this study that the moment frames with 30-foot bays had girders that only ranged between 18 inches and 30 inches in depth, while the moment frames with 20-foot bays had girders that ranged between 12 inches and 24 inches in depth.

The column and girder section sizes for each optimized moment frame that was designed using this method are shown in the tables at the end of this appendix. The first table is a summary of the column and girder sizes for the original thirteen buildings when this method of design was used. This table is only meant for comparing the original thirteen building member sizes with these more optimal member sizes from this method to see how the section sizes might actually change. The remaining nine tables in this appendix show all of the column and girder sizes for each of the 135 baseline moment frames. These tables are categorized based on the column d/t ratios, F_{yc}, f'_c, and which building layout and loading was used in their design. The final baseline values of η that resulted from using this approximation method and these 135 moment frames are shown in Table 6.3.14.

"Fully Optimized" Member Sizes for Each of the Original 13 Buildings

d/t	F_{yc} (ksi)	f'_c (ksi)	Building Geometry and Loading Based on Design No.	Story	Final Section Sizes and Properties: *Suggested* Column	Final Section Sizes and Properties: Maximum Allowed Z_g (in^3)	Final Section Sizes and Properties: *Suggested* Girder	Flexural Rigidity Ratio, η: Story Value	Flexural Rigidity Ratio, η: Building Mean Value
LOW	46	4	3A	3	HSS 15 x 15 x 0.75	90	W18x40	0.64	0.71
				2	HSS 18 x 18 x 0.75	259	W24x94	0.74	
				1	HSS 19 x 19 x 0.75	288	W30X90	0.74	
			3D	3	HSS 15 x 15 x 0.75	90	W18x40	0.60	0.78
				2	HSS 20 x 20 x 0.75	319	W30X99	0.90	
				1	HSS 21 x 21 x 0.75	352	W30X108	0.84	
			3F	3	HSS 11 x 11 x 0.75	49	W14x30	0.44	0.63
				2	HSS 15 x 15 x 0.75	180	W24X68	0.73	
				1	HSS 16 x 16 x 0.75	204	W24X76	0.73	
			9A	9	HSS 12 x 12 x 0.75	58	W14x34	0.41	0.75
				8	HSS 15 x 15 x 0.75	180	W24X68	0.56	
				7	HSS 17 x 17 x 0.75	231	W24X84	0.66	
				6	HSS 19 x 19 x 0.75	288	W30X90	0.83	
				5	HSS 20 x 20 x 0.75	319	W30X99	0.88	
				4	HSS 20 x 20 x 0.75	319	W30X99	0.80	
				3	HSS 21 x 21 x 0.75	352	W30X108	0.91	
				2	HSS 21 x 21 x 0.75	352	W30X108	0.88	
				1	HSS 21 x 21 x 0.75	352	W30X108	0.87	
			18A	18	HSS 9 x 9 x 0.625	27	W12x16	0.45	0.77
				17	HSS 11 x 11 x 0.75	97	W21x44	0.57	
				16	HSS 12 x 12 x 0.75	115	W18X55	0.51	
				15	HSS 14 x 14 x 0.625	130	W21x57	0.62	
				14	HSS 14 x 14 x 0.75	156	W24X62	0.59	
				13	HSS 15 x 15 x 0.75	180	W24X68	0.68	
				12	HSS 16 x 16 x 0.75	204	W24X76	0.80	
				11	HSS 16 x 16 x 0.75	204	W24X76	0.72	
				10	HSS 17 x 17 x 0.75	231	W24X84	0.87	
				9	HSS 17 x 17 x 0.75	231	W24X84	0.82	
				8	HSS 17 x 17 x 0.75	231	W24X84	0.78	
				7	HSS 18 x 18 x 0.75	259	W24x94	0.97	
				6	HSS 18 x 18 x 0.75	259	W24x94	0.95	
				5	HSS 18 x 18 x 0.75	259	W24x94	0.93	
				4	HSS 18 x 18 x 0.75	259	W24x94	0.92	
				3	HSS 18 x 18 x 0.75	259	W24x94	0.91	
				2	HSS 18 x 18 x 0.75	259	W24x94	0.91	
				1	HSS 18 x 18 x 0.75	259	W24x94	0.91	
≤ 2.26 √(E/Fy)	80	16	3B	3	HSS 13 x 13 x 0.625	98	W21x44	0.33	0.39
				2	HSS 15 x 15 x 0.75	313	W30X99	0.40	
				1	HSS 16 x 16 x 0.75	355	W30X108	0.42	
			3E	3	HSS 13 x 13 x 0.75	118	W18X55	0.36	0.44
				2	HSS 16 x 16 x 0.75	355	W30X108	0.41	
				1	HSS 18 x 18 x 0.75	450	W30X116	0.55	
			3G	3	HSS 10 x 10 x 0.75	69	W18x35	0.29	0.39
				2	HSS 13 x 13 x 0.75	235	W24X84	0.43	
				1	HSS 14 x 14 x 0.75	272	W18x119	0.46	
			9B	9	HSS 11 x 11 x 0.625	70	W18x35	0.25	0.41
				8	HSS 13 x 13 x 0.75	235	W24X84	0.33	
				7	HSS 15 x 15 x 0.625	260	W24x94	0.39	
				6	HSS 16 x 16 x 0.75	355	W30X108	0.48	
				5	HSS 16 x 16 x 0.75	355	W30X108	0.40	
				4	HSS 17 x 17 x 0.75	401	W30X116	0.49	
				3	HSS 17 x 17 x 0.75	401	W30X116	0.45	
				2	HSS 17 x 17 x 0.75	401	W30X116	0.43	
				1	HSS 17 x 17 x 0.75	401	W30X116	0.43	
			18B	18	HSS 8 x 8 x 0.625	37	W14x22	0.25	0.43
				17	HSS 10 x 10 x 0.625	116	W18X55	0.31	
				16	HSS 11 x 11 x 0.75	169	W21X68	0.37	
				15	HSS 12 x 12 x 0.75	200	W24X76	0.41	
				14	HSS 13 x 13 x 0.625	195	W24X68	0.41	
				13	HSS 13 x 13 x 0.75	235	W24X84	0.39	
				12	HSS 14 x 14 x 0.625	227	W24X84	0.43	
				11	HSS 14 x 14 x 0.75	272	W18x119	0.45	
				10	HSS 14 x 14 x 0.75	272	W18x119	0.41	
				9	HSS 15 x 15 x 0.625	260	W24x94	0.48	
				8	HSS 15 x 15 x 0.75	313	W30X99	0.53	
				7	HSS 15 x 15 x 0.75	313	W30X99	0.50	
				6	HSS 15 x 15 x 0.75	313	W30X99	0.49	
				5	HSS 15 x 15 x 0.75	313	W30X99	0.48	
				4	HSS 15 x 15 x 0.75	313	W30X99	0.47	
				3	HSS 15 x 15 x 0.75	313	W30X99	0.46	
				2	HSS 15 x 15 x 0.75	313	W30X99	0.46	
				1	HSS 15 x 15 x 0.75	313	W30X99	0.46	
≈ 80	80	16	3C	3	HSS 19 x 19 x 0.25	84	W18x40	1.07	1.43
				2	HSS 24 x 24 x 0.3125	333	W30X99	1.78	
				1	HSS 24 x 24 x 0.3125	333	W30X99	1.45	
	50	16	9C	9	HSS 18 x 18 x 0.25	47	W14x26	1.37	2.37
				8	HSS 24 x 24 x 0.3125	208	W24X76	2.70	
				7	HSS 24 x 24 x 0.3125	208	W24X76	1.98	
				6	HSS 27 x 27 x 0.375	316	W30X99	2.78	
				5	HSS 28 x 28 x 0.375	340	W30X99	2.81	
				4	HSS 28 x 28 x 0.375	340	W30X99	2.57	
				3	HSS 28 x 28 x 0.375	340	W30X99	2.43	
				2	HSS 28 x 28 x 0.375	340	W30X99	2.36	
				1	HSS 28 x 28 x 0.375	340	W30X99	2.33	
			18C	18	HSS 12 x 12 x 0.25	21	W12x16	1.03	2.13
				17	HSS 16 x 16 x 0.25	74	W16x40	1.64	
				16	HSS 18 x 18 x 0.25	94	W18x40	1.78	
				15	HSS 19 x 19 x 0.25	104	W18X50	1.69	
				14	HSS 20 x 20 x 0.3125	144	W21X62	1.93	
				13	HSS 21 x 21 x 0.3125	159	W24X62	2.03	
				12	HSS 22 x 22 x 0.3125	175	W21X68	2.20	
				11	HSS 23 x 23 x 0.3125	191	W24X68	2.42	
				10	HSS 24 x 24 x 0.3125	208	W24X76	2.68	
				9	HSS 24 x 24 x 0.3125	208	W24X76	2.55	
				8	HSS 24 x 24 x 0.3125	208	W24X76	2.45	
				7	HSS 24 x 24 x 0.3125	208	W24X76	2.37	
				6	HSS 24 x 24 x 0.3125	208	W24X76	2.32	
				5	HSS 24 x 24 x 0.3125	208	W24X76	2.28	
				4	HSS 24 x 24 x 0.3125	208	W24X76	2.26	
				3	HSS 24 x 24 x 0.3125	208	W24X76	2.24	
				2	HSS 24 x 24 x 0.3125	208	W24X76	2.24	
				1	HSS 24 x 24 x 0.3125	208	W24X76	2.24	

Member Sizes that Were Used to Calibrate η at Base Line Design Values 1, 2, and 3 (Per Figure 6.3.1) for Low d/t ratios

d/t	η Base Line Number	F_{yc} (ksi)	f'_c (ksi)	Building Geometry and Loading Based on Design No.	Story	Final Section Sizes and Properties: *Suggested* Column	Final Section Sizes and Properties: Maximum Allowed Z_g (in^3)	Final Section Sizes and Properties: *Suggested* Girder	Flexural Rigidity Ratio, η: Story Value	Flexural Rigidity Ratio, η: Building Mean Value
LOW	1	46	4	3A	3	HSS 15 x 15 x 0.75	90	W18x40	0.64	0.71
					2	HSS 18 x 18 x 0.75	259	W24x94	0.74	
					1	HSS 19 x 19 x 0.75	288	W30X90	0.74	
				3D	3	HSS 15 x 15 x 0.75	90	W18x40	0.60	0.78
					2	HSS 20 x 20 x 0.75	319	W30X99	0.90	
					1	HSS 21 x 21 x 0.75	352	W30X108	0.84	
				3F	3	HSS 11 x 11 x 0.75	49	W14x30	0.44	0.63
					2	HSS 15 x 15 x 0.75	180	W24X68	0.73	
					1	HSS 16 x 16 x 0.75	204	W24X76	0.73	
				9A	9	HSS 12 x 12 x 0.75	58	W14x34	0.41	0.75
					8	HSS 15 x 15 x 0.75	180	W24X68	0.56	
					7	HSS 17 x 17 x 0.75	231	W24X84	0.66	
					6	HSS 19 x 19 x 0.75	288	W30X90	0.83	
					5	HSS 20 x 20 x 0.75	319	W30X99	0.88	
					4	HSS 20 x 20 x 0.75	319	W30X99	0.80	
					3	HSS 21 x 21 x 0.75	352	W30X108	0.91	
					2	HSS 21 x 21 x 0.75	352	W30X108	0.88	
					1	HSS 21 x 21 x 0.75	352	W30X108	0.87	
				18A	18	HSS 9 x 9 x 0.625	27	W12x16	0.45	0.77
					17	HSS 11 x 11 x 0.75	97	W21x44	0.57	
					16	HSS 12 x 12 x 0.75	115	W18X55	0.51	
					15	HSS 14 x 14 x 0.625	130	W21x57	0.62	
					14	HSS 14 x 14 x 0.75	156	W24X62	0.59	
					13	HSS 15 x 15 x 0.75	180	W24X68	0.68	
					12	HSS 16 x 16 x 0.75	204	W24X76	0.80	
					11	HSS 16 x 16 x 0.75	204	W24X76	0.72	
					10	HSS 17 x 17 x 0.75	231	W24X84	0.87	
					9	HSS 17 x 17 x 0.75	231	W24X84	0.82	
					8	HSS 17 x 17 x 0.75	231	W24X84	0.78	
					7	HSS 18 x 18 x 0.75	259	W24x94	0.97	
					6	HSS 18 x 18 x 0.75	259	W24x94	0.95	
					5	HSS 18 x 18 x 0.75	259	W24x94	0.93	
					4	HSS 18 x 18 x 0.75	259	W24x94	0.92	
					3	HSS 18 x 18 x 0.75	259	W24x94	0.91	
					2	HSS 18 x 18 x 0.75	259	W24x94	0.91	
					1	HSS 18 x 18 x 0.75	259	W24x94	0.91	
	2	46	10	3B	3	HSS 15 x 15 x 0.75	90	W18x40	0.72	0.81
					2	HSS 18 x 18 x 0.75	259	W24x94	0.85	
					1	HSS 19 x 19 x 0.75	288	W30X90	0.86	
				3E	3	HSS 15 x 15 x 0.75	90	W18x40	0.67	0.84
					2	HSS 19 x 19 x 0.75	288	W30X90	0.84	
					1	HSS 21 x 21 x 0.75	352	W30X108	0.99	
				3G	3	HSS 11 x 11 x 0.75	49	W14x30	0.48	0.72
					2	HSS 15 x 15 x 0.75	180	W24X68	0.83	
					1	HSS 16 x 16 x 0.75	204	W24X76	0.83	
				9B	9	HSS 12 x 12 x 0.75	58	W14x34	0.45	0.81
					8	HSS 15 x 15 x 0.75	180	W24X68	0.63	
					7	HSS 17 x 17 x 0.75	231	W24X84	0.75	
					6	HSS 18 x 18 x 0.75	259	W24x94	0.76	
					5	HSS 19 x 19 x 0.75	288	W30X90	0.83	
					4	HSS 20 x 20 x 0.75	319	W30X99	0.93	
					3	HSS 20 x 20 x 0.75	319	W30X99	0.87	
					2	HSS 21 x 21 x 0.75	352	W30X108	1.03	
					1	HSS 21 x 21 x 0.75	352	W30X108	1.01	
				18B	18	HSS 8 x 8 x 0.75	26	W12x16	0.31	0.83
					17	HSS 11 x 11 x 0.625	80	W18x40	0.51	
					16	HSS 12 x 12 x 0.75	115	W18X55	0.56	
					15	HSS 13 x 13 x 0.75	135	W24X55	0.59	
					14	HSS 14 x 14 x 0.75	156	W24X62	0.67	
					13	HSS 15 x 15 x 0.75	180	W24X68	0.78	
					12	HSS 16 x 16 x 0.75	204	W24X76	0.92	
					11	HSS 16 x 16 x 0.75	204	W24X76	0.83	
					10	HSS 17 x 17 x 0.75	231	W24X84	1.01	
					9	HSS 17 x 17 x 0.75	231	W24X84	0.95	
					8	HSS 17 x 17 x 0.75	231	W24X84	0.90	
					7	HSS 17 x 17 x 0.75	231	W24X84	0.87	
					6	HSS 17 x 17 x 0.75	231	W24X84	0.84	
					5	HSS 18 x 18 x 0.75	259	W24x94	1.07	
					4	HSS 18 x 18 x 0.75	259	W24x94	1.06	
					3	HSS 18 x 18 x 0.75	259	W24x94	1.05	
					2	HSS 18 x 18 x 0.75	259	W24x94	1.05	
					1	HSS 18 x 18 x 0.75	259	W24x94	1.05	
	3	46	16	3A	3	HSS 15 x 15 x 0.75	90	W18x40	0.78	0.89
					2	HSS 18 x 18 x 0.75	259	W24x94	0.93	
					1	HSS 19 x 19 x 0.75	288	W30X90	0.95	
				3D	3	HSS 15 x 15 x 0.75	90	W18x40	0.73	0.92
					2	HSS 19 x 19 x 0.75	288	W30X90	0.93	
					1	HSS 21 x 21 x 0.75	352	W30X108	1.09	
				3F	3	HSS 11 x 11 x 0.75	49	W14x30	0.52	0.78
					2	HSS 15 x 15 x 0.75	180	W24X68	0.91	
					1	HSS 16 x 16 x 0.75	204	W24X76	0.91	
				9A	9	HSS 12 x 12 x 0.75	58	W14x34	0.48	0.89
					8	HSS 15 x 15 x 0.75	180	W24X68	0.68	
					7	HSS 17 x 17 x 0.75	231	W24X84	0.82	
					6	HSS 18 x 18 x 0.75	259	W24x94	0.84	
					5	HSS 19 x 19 x 0.75	288	W30X90	0.91	
					4	HSS 20 x 20 x 0.75	319	W30X99	1.02	
					3	HSS 20 x 20 x 0.75	319	W30X99	0.96	
					2	HSS 21 x 21 x 0.75	352	W30X108	1.14	
					1	HSS 21 x 21 x 0.75	352	W30X108	1.12	
				18A	18	HSS 8 x 8 x 0.75	26	W12x16	0.33	0.87
					17	HSS 11 x 11 x 0.625	80	W18x40	0.56	
					16	HSS 12 x 12 x 0.75	115	W18X55	0.60	
					15	HSS 13 x 13 x 0.75	135	W24X55	0.64	
					14	HSS 14 x 14 x 0.75	156	W24X62	0.72	
					13	HSS 15 x 15 x 0.75	180	W24X68	0.84	
					12	HSS 16 x 16 x 0.75	204	W24X76	1.00	
					11	HSS 16 x 16 x 0.75	204	W24X76	0.90	
					10	HSS 16 x 16 x 0.75	204	W24X76	0.83	
					9	HSS 17 x 17 x 0.75	231	W24X84	1.04	
					8	HSS 17 x 17 x 0.75	231	W24X84	0.99	
					7	HSS 17 x 17 x 0.75	231	W24X84	0.95	
					6	HSS 17 x 17 x 0.75	231	W24X84	0.93	
					5	HSS 17 x 17 x 0.75	231	W24X84	0.91	
					4	HSS 17 x 17 x 0.75	231	W24X84	0.90	
					3	HSS 18 x 18 x 0.75	259	W24x94	1.16	
					2	HSS 18 x 18 x 0.75	259	W24x94	1.15	
					1	HSS 18 x 18 x 0.75	259	W24x94	1.15	

Member Sizes that Were Used to Calibrate η at Base Line Design Values 4, 5, and 6 (Per Figure 6.3.1) for Low d/t ratios

d/t	η Base Line Number	F_{yc} (ksi)	f'_c (ksi)	Building Geometry and Loading Based on Design No.	Story	Final Section Sizes and Properties: *Suggested* Column	Final Section Sizes and Properties: Maximum Allowed Z_g (in³)	Final Section Sizes and Properties: *Suggested* Girder	Flexural Rigidity Ratio, η: Story Value	Flexural Rigidity Ratio, η: Building Mean Value
LOW	4	63	4	3B	3	HSS 14 x 14 x 0.75	107	W18X50	0.47	0.54
					2	HSS 17 x 17 x 0.75	316	W30X99	0.58	
					1	HSS 18 x 18 x 0.75	355	W30X108	0.59	
				3E	3	HSS 14 x 14 x 0.75	107	W18X50	0.43	0.52
					2	HSS 18 x 18 x 0.75	355	W30X108	0.57	
					1	HSS 19 x 19 x 0.75	395	W30X116	0.55	
				3G	3	HSS 11 x 11 x 0.625	55	W14x34	0.34	0.46
					2	HSS 14 x 14 x 0.75	214	W12x136	0.52	
					1	HSS 15 x 15 x 0.75	246	W27X84	0.53	
				9B	9	HSS 12 x 12 x 0.625	66	W14x34	0.33	0.50
					8	HSS 14 x 14 x 0.75	214	W12x136	0.40	
					7	HSS 16 x 16 x 0.75	280	W18x119	0.50	
					6	HSS 17 x 17 x 0.75	316	W30X99	0.51	
					5	HSS 18 x 18 x 0.75	355	W30X108	0.56	
					4	HSS 18 x 18 x 0.75	355	W30X108	0.50	
					3	HSS 19 x 19 x 0.75	395	W30X116	0.59	
					2	HSS 19 x 19 x 0.75	395	W30X116	0.57	
					1	HSS 19 x 19 x 0.75	395	W30X116	0.56	
				18B	18	HSS 9 x 9 x 0.5	30	W12x22	0.33	0.49
					17	HSS 10 x 10 x 0.75	109	W12x72	0.34	
					16	HSS 12 x 12 x 0.625	131	W21x57	0.40	
					15	HSS 13 x 13 x 0.625	154	W24X62	0.42	
					14	HSS 13 x 13 x 0.75	185	W24X68	0.39	
					13	HSS 14 x 14 x 0.75	214	W12x136	0.48	
					12	HSS 15 x 15 x 0.75	246	W27X84	0.59	
					11	HSS 15 x 15 x 0.75	246	W27X84	0.52	
					10	HSS 15 x 15 x 0.75	246	W27X84	0.47	
					9	HSS 16 x 16 x 0.75	280	W18x119	0.61	
					8	HSS 16 x 16 x 0.75	280	W18x119	0.58	
					7	HSS 16 x 16 x 0.75	280	W18x119	0.56	
					6	HSS 16 x 16 x 0.75	280	W18x119	0.54	
					5	HSS 16 x 16 x 0.75	280	W18x119	0.53	
					4	HSS 16 x 16 x 0.75	280	W18x119	0.52	
					3	HSS 16 x 16 x 0.75	280	W18x119	0.51	
					2	HSS 16 x 16 x 0.75	280	W18x119	0.51	
					1	HSS 16 x 16 x 0.75	280	W18x119	0.51	
	5	63	10	3A	3	HSS 13 x 13 x 0.75	93	W18x40	0.37	0.46
					2	HSS 16 x 16 x 0.75	280	W18x119	0.50	
					1	HSS 17 x 17 x 0.75	316	W30X99	0.52	
				3D	3	HSS 14 x 14 x 0.75	107	W18X50	0.49	0.60
					2	HSS 18 x 18 x 0.75	355	W30X108	0.66	
					1	HSS 19 x 19 x 0.75	395	W30X116	0.64	
				3F	3	HSS 11 x 11 x 0.625	55	W14x34	0.39	0.53
					2	HSS 14 x 14 x 0.75	214	W12x136	0.59	
					1	HSS 15 x 15 x 0.75	246	W27X84	0.61	
				9A	9	HSS 11 x 11 x 0.75	67	W18x35	0.29	0.56
					8	HSS 14 x 14 x 0.75	214	W12x136	0.45	
					7	HSS 16 x 16 x 0.75	280	W18x119	0.57	
					6	HSS 17 x 17 x 0.75	316	W30X99	0.59	
					5	HSS 18 x 18 x 0.75	355	W30X108	0.65	
					4	HSS 18 x 18 x 0.75	355	W30X108	0.58	
					3	HSS 18 x 18 x 0.75	355	W30X108	0.54	
					2	HSS 19 x 19 x 0.75	395	W30X116	0.67	
					1	HSS 19 x 19 x 0.75	395	W30X116	0.66	
				18A	18	HSS 8 x 8 x 0.75	35	W14x22	0.31	0.54
					17	HSS 10 x 10 x 0.75	109	W12x72	0.37	
					16	HSS 11 x 11 x 0.75	133	W18x65	0.34	
					15	HSS 13 x 13 x 0.625	154	W24X62	0.49	
					14	HSS 13 x 13 x 0.75	185	W24X68	0.45	
					13	HSS 14 x 14 x 0.75	214	W12x136	0.55	
					12	HSS 14 x 14 x 0.75	214	W12x136	0.47	
					11	HSS 15 x 15 x 0.75	246	W27X84	0.60	
					10	HSS 15 x 15 x 0.75	246	W27X84	0.55	
					9	HSS 16 x 16 x 0.75	280	W18x119	0.71	
					8	HSS 16 x 16 x 0.75	280	W18x119	0.67	
					7	HSS 16 x 16 x 0.75	280	W18x119	0.65	
					6	HSS 16 x 16 x 0.75	280	W18x119	0.63	
					5	HSS 16 x 16 x 0.75	280	W18x119	0.61	
					4	HSS 16 x 16 x 0.75	280	W18x119	0.60	
					3	HSS 16 x 16 x 0.75	280	W18x119	0.60	
					2	HSS 16 x 16 x 0.75	280	W18x119	0.60	
					1	HSS 16 x 16 x 0.75	280	W18x119	0.60	
	6	63	16	3B	3	HSS 13 x 13 x 0.75	93	W18x40	0.40	0.51
					2	HSS 16 x 16 x 0.75	280	W18x119	0.55	
					1	HSS 17 x 17 x 0.75	316	W30X99	0.57	
				3E	3	HSS 14 x 14 x 0.625	90	W18x40	0.46	0.58
					2	HSS 17 x 17 x 0.75	316	W30X99	0.56	
					1	HSS 19 x 19 x 0.75	395	W30X116	0.71	
				3G	3	HSS 11 x 11 x 0.625	55	W14x34	0.43	0.55
					2	HSS 14 x 14 x 0.625	179	W24X68	0.55	
					1	HSS 15 x 15 x 0.75	246	W27X84	0.66	
				9B	9	HSS 11 x 11 x 0.75	67	W18x35	0.31	0.57
					8	HSS 14 x 14 x 0.75	214	W12x136	0.49	
					7	HSS 15 x 15 x 0.75	246	W27X84	0.46	
					6	HSS 17 x 17 x 0.75	316	W30X99	0.65	
					5	HSS 17 x 17 x 0.75	316	W30X99	0.55	
					4	HSS 18 x 18 x 0.75	355	W30X108	0.64	
					3	HSS 18 x 18 x 0.75	355	W30X108	0.60	
					2	HSS 19 x 19 x 0.75	395	W30X116	0.74	
					1	HSS 19 x 19 x 0.75	395	W30X116	0.73	
				18B	18	HSS 8 x 8 x 0.625	29	W12x16	0.25	0.57
					17	HSS 10 x 10 x 0.75	109	W12x72	0.39	
					16	HSS 11 x 11 x 0.75	133	W18x65	0.37	
					15	HSS 12 x 12 x 0.75	158	W24X62	0.41	
					14	HSS 13 x 13 x 0.75	185	W24X68	0.49	
					13	HSS 14 x 14 x 0.75	214	W12x136	0.60	
					12	HSS 14 x 14 x 0.75	214	W12x136	0.51	
					11	HSS 15 x 15 x 0.75	246	W27X84	0.66	
					10	HSS 15 x 15 x 0.75	246	W27X84	0.60	
					9	HSS 15 x 15 x 0.75	246	W27X84	0.56	
					8	HSS 16 x 16 x 0.75	280	W18x119	0.74	
					7	HSS 16 x 16 x 0.75	280	W18x119	0.71	
					6	HSS 16 x 16 x 0.75	280	W18x119	0.69	
					5	HSS 16 x 16 x 0.75	280	W18x119	0.68	
					4	HSS 16 x 16 x 0.75	280	W18x119	0.67	
					3	HSS 16 x 16 x 0.75	280	W18x119	0.66	
					2	HSS 16 x 16 x 0.75	280	W18x119	0.66	
					1	HSS 16 x 16 x 0.75	280	W18x119	0.66	

Member Sizes that Were Used to Calibrate η at Base Line Design Values 7, 8, and 9 (Per Figure 6.3.1) for Low d/t ratios

d/t	η Base Line Number	F_{yc} (ksi)	f'_c (ksi)	Building Geometry and Loading Based on Design No.	Story	Final Section Sizes and Properties: *Suggested* Column	Maximum Allowed Z_g (in^3)	*Suggested* Girder	Flexural Rigidity Ratio, η: Story Value	Building Mean Value
LOW	7	80	4	3A	3	HSS 13 x 13 x 0.75	118	W18X55	0.32	0.40
					2	HSS 16 x 16 x 0.75	355	W30X108	0.43	
					1	HSS 17 x 17 x 0.75	401	W30X116	0.45	
				3D	3	HSS 13 x 13 x 0.75	118	W18X55	0.30	0.39
					2	HSS 17 x 17 x 0.75	401	W30X116	0.44	
					1	HSS 18 x 18 x 0.75	450	W30X116	0.42	
				3F	3	HSS 11 x 11 x 0.625	70	W18x35	0.34	0.34
					2	HSS 13 x 13 x 0.75	235	W24X84	0.34	
					1	HSS 14 x 14 x 0.75	272	W18x119	0.36	
				9A	9	HSS 11 x 11 x 0.75	85	W18x40	0.26	0.38
					8	HSS 13 x 13 x 0.75	235	W24X84	0.27	
					7	HSS 15 x 15 x 0.75	313	W30X99	0.36	
					6	HSS 16 x 16 x 0.75	355	W30X108	0.38	
					5	HSS 17 x 17 x 0.75	401	W30X116	0.43	
					4	HSS 17 x 17 x 0.75	401	W30X116	0.38	
					3	HSS 18 x 18 x 0.75	450	W30X116	0.46	
					2	HSS 18 x 18 x 0.75	450	W30X116	0.44	
					1	HSS 18 x 18 x 0.75	450	W30X116	0.44	
				18A	18	HSS 9 x 9 x 0.5	38	W12x26	0.33	0.40
					17	HSS 10 x 10 x 0.625	116	W18X55	0.25	
					16	HSS 11 x 11 x 0.75	169	W21X68	0.30	
					15	HSS 12 x 12 x 0.75	200	W24X76	0.33	
					14	HSS 13 x 13 x 0.75	235	W24X84	0.39	
					13	HSS 13 x 13 x 0.75	235	W24X84	0.31	
					12	HSS 14 x 14 x 0.75	272	W18x119	0.40	
					11	HSS 14 x 14 x 0.75	272	W18x119	0.35	
					10	HSS 15 x 15 x 0.75	313	W30X99	0.47	
					9	HSS 15 x 15 x 0.75	313	W30X99	0.44	
					8	HSS 15 x 15 x 0.75	313	W30X99	0.41	
					7	HSS 15 x 15 x 0.75	313	W30X99	0.39	
					6	HSS 15 x 15 x 0.75	313	W30X99	0.37	
					5	HSS 16 x 16 x 0.625	296	W30X90	0.42	
					4	HSS 16 x 16 x 0.75	355	W30X108	0.52	
					3	HSS 16 x 16 x 0.75	355	W30X108	0.51	
					2	HSS 16 x 16 x 0.75	355	W30X108	0.51	
					1	HSS 16 x 16 x 0.75	355	W30X108	0.51	
	8	80	10	3B	3	HSS 13 x 13 x 0.75	118	W18X55	0.37	0.37
					2	HSS 15 x 15 x 0.75	313	W30X99	0.36	
					1	HSS 16 x 16 x 0.75	355	W30X108	0.38	
				3E	3	HSS 13 x 13 x 0.75	118	W18X55	0.34	0.45
					2	HSS 17 x 17 x 0.75	401	W30X116	0.51	
					1	HSS 18 x 18 x 0.75	450	W30X116	0.50	
				3G	3	HSS 10 x 10 x 0.75	69	W18x35	0.27	0.36
					2	HSS 13 x 13 x 0.75	235	W24X84	0.39	
					1	HSS 14 x 14 x 0.75	272	W18x119	0.41	
				9B	9	HSS 11 x 11 x 0.75	85	W18x40	0.29	0.41
					8	HSS 13 x 13 x 0.75	235	W24X84	0.31	
					7	HSS 15 x 15 x 0.75	313	W30X99	0.42	
					6	HSS 16 x 16 x 0.75	355	W30X108	0.44	
					5	HSS 16 x 16 x 0.75	355	W30X108	0.36	
					4	HSS 17 x 17 x 0.75	401	W30X116	0.44	
					3	HSS 17 x 17 x 0.75	401	W30X116	0.41	
					2	HSS 18 x 18 x 0.75	450	W30X116	0.52	
					1	HSS 18 x 18 x 0.75	450	W30X116	0.51	
				18B	18	HSS 8 x 8 x 0.625	37	W14x22	0.23	0.40
					17	HSS 10 x 10 x 0.625	116	W18X55	0.29	
					16	HSS 11 x 11 x 0.75	169	W21X68	0.34	
					15	HSS 12 x 12 x 0.75	200	W24X76	0.38	
					14	HSS 13 x 13 x 0.625	195	W24X68	0.36	
					13	HSS 13 x 13 x 0.75	235	W24X84	0.36	
					12	HSS 14 x 14 x 0.75	272	W18x119	0.47	
					11	HSS 14 x 14 x 0.75	272	W18x119	0.41	
					10	HSS 15 x 15 x 0.625	260	W24x94	0.45	
					9	HSS 15 x 15 x 0.75	313	W30X99	0.51	
					8	HSS 15 x 15 x 0.75	313	W30X99	0.48	
					7	HSS 15 x 15 x 0.75	313	W30X99	0.46	
					6	HSS 15 x 15 x 0.75	313	W30X99	0.44	
					5	HSS 15 x 15 x 0.75	313	W30X99	0.43	
					4	HSS 15 x 15 x 0.75	313	W30X99	0.42	
					3	HSS 15 x 15 x 0.75	313	W30X99	0.42	
					2	HSS 15 x 15 x 0.75	313	W30X99	0.42	
					1	HSS 15 x 15 x 0.75	313	W30X99	0.42	
	9	80	16	3A	3	HSS 13 x 13 x 0.625	98	W21x44	0.33	0.39
					2	HSS 15 x 15 x 0.75	313	W30X99	0.40	
					1	HSS 16 x 16 x 0.75	355	W30X108	0.42	
				3D	3	HSS 13 x 13 x 0.75	118	W18X55	0.36	0.44
					2	HSS 16 x 16 x 0.75	355	W30X108	0.41	
					1	HSS 18 x 18 x 0.75	450	W30X116	0.55	
				3F	3	HSS 10 x 10 x 0.75	69	W18x35	0.29	0.39
					2	HSS 13 x 13 x 0.75	235	W24X84	0.43	
					1	HSS 14 x 14 x 0.75	272	W18x119	0.46	
				9A	9	HSS 11 x 11 x 0.625	70	W18x35	0.25	0.41
					8	HSS 13 x 13 x 0.75	235	W24X84	0.33	
					7	HSS 15 x 15 x 0.625	260	W24x94	0.39	
					6	HSS 16 x 16 x 0.75	355	W30X108	0.48	
					5	HSS 16 x 16 x 0.75	355	W30X108	0.40	
					4	HSS 17 x 17 x 0.75	401	W30X116	0.49	
					3	HSS 17 x 17 x 0.75	401	W30X116	0.45	
					2	HSS 17 x 17 x 0.75	401	W30X116	0.43	
					1	HSS 17 x 17 x 0.75	401	W30X116	0.43	
				18A	18	HSS 8 x 8 x 0.625	37	W14x22	0.25	0.43
					17	HSS 10 x 10 x 0.625	116	W18X55	0.31	
					16	HSS 11 x 11 x 0.75	169	W21X68	0.37	
					15	HSS 12 x 12 x 0.75	200	W24X76	0.41	
					14	HSS 13 x 13 x 0.625	195	W24X68	0.41	
					13	HSS 13 x 13 x 0.75	235	W24X84	0.39	
					12	HSS 14 x 14 x 0.625	227	W24X84	0.43	
					11	HSS 14 x 14 x 0.75	272	W18x119	0.45	
					10	HSS 14 x 14 x 0.75	272	W18x119	0.41	
					9	HSS 15 x 15 x 0.625	260	W24x94	0.48	
					8	HSS 15 x 15 x 0.75	313	W30X99	0.53	
					7	HSS 15 x 15 x 0.75	313	W30X99	0.50	
					6	HSS 15 x 15 x 0.75	313	W30X99	0.49	
					5	HSS 15 x 15 x 0.75	313	W30X99	0.48	
					4	HSS 15 x 15 x 0.75	313	W30X99	0.47	
					3	HSS 15 x 15 x 0.75	313	W30X99	0.46	
					2	HSS 15 x 15 x 0.75	313	W30X99	0.46	
					1	HSS 15 x 15 x 0.75	313	W30X99	0.46	

Member Sizes that Were Used to Calibrate η at Base Line Design Values 1, 2, and 3 (Per Figure 6.3.1) for d/t ≤ 2.26 √(E/Fy)

d/t	η Base Line Number	F_{yc} (ksi)	f'_c (ksi)	Building Geometry and Loading Based on Design No.	Story	Final Section Sizes and Properties: *Suggested* Column	Final Section Sizes and Properties: Maximum Allowed Z_g (in³)	Final Section Sizes and Properties: *Suggested* Girder	Flexural Rigidity Ratio, η: Story Value	Flexural Rigidity Ratio, η: Building Mean Value
≤ 2.26 √(E/Fy)	1	46	4	3A	3	HSS 21 x 21 x 0.375	88	W18x40	1.37	1.78
					2	HSS 25 x 25 x 0.5	333	W30X99	1.89	
					1	HSS 27 x 27 x 0.5	388	W30X116	2.07	
				3D	3	HSS 21 x 21 x 0.375	88	W18x40	1.29	1.72
					2	HSS 27 x 27 x 0.5	388	W30X116	2.03	
					1	HSS 28 x 28 x 0.5	417	W30X116	1.84	
				3F	3	HSS 17 x 17 x 0.3125	48	W14x30	1.36	1.75
					2	HSS 21 x 21 x 0.375	176	W21X68	1.63	
					1	HSS 23 x 23 x 0.5	282	W18x119	2.25	
				9A	9	HSS 18 x 18 x 0.375	65	W14x34	1.21	1.75
					8	HSS 21 x 21 x 0.375	176	W21X68	1.21	
					7	HSS 23 x 23 x 0.5	282	W18x119	1.54	
					6	HSS 26 x 26 x 0.5	360	W30X108	1.98	
					5	HSS 27 x 27 x 0.5	388	W30X116	1.99	
					4	HSS 28 x 28 x 0.5	417	W30X116	2.08	
					3	HSS 28 x 28 x 0.5	417	W30X116	1.97	
					2	HSS 28 x 28 x 0.5	417	W30X116	1.91	
					1	HSS 28 x 28 x 0.5	417	W30X116	1.88	
				18A	18	HSS 13 x 13 x 0.3125	28	W12x16	1.21	2.38
					17	HSS 17 x 17 x 0.3125	96	W21x44	1.65	
					16	HSS 19 x 19 x 0.375	144	W21X62	1.94	
					15	HSS 21 x 21 x 0.375	176	W21X68	2.18	
					14	HSS 21 x 21 x 0.375	176	W21X68	1.79	
					13	HSS 22 x 22 x 0.5	257	W24x94	2.31	
					12	HSS 23 x 23 x 0.5	282	W18x119	2.44	
					11	HSS 23 x 23 x 0.5	282	W18x119	2.24	
					10	HSS 24 x 24 x 0.5	307	W21x122	2.45	
					9	HSS 25 x 25 x 0.5	333	W30X99	2.71	
					8	HSS 25 x 25 x 0.5	333	W30X99	2.61	
					7	HSS 25 x 25 x 0.5	333	W30X99	2.53	
					6	HSS 26 x 26 x 0.5	360	W30X108	2.86	
					5	HSS 26 x 26 x 0.5	360	W30X108	2.82	
					4	HSS 26 x 26 x 0.5	360	W30X108	2.79	
					3	HSS 26 x 26 x 0.5	360	W30X108	2.77	
					2	HSS 26 x 26 x 0.5	360	W30X108	2.76	
					1	HSS 26 x 26 x 0.5	360	W30X108	2.76	
	2	46	10	3B	3	HSS 21 x 21 x 0.375	88	W18x40	1.72	2.20
					2	HSS 25 x 25 x 0.5	333	W30X99	2.32	
					1	HSS 27 x 27 x 0.5	388	W30X116	2.56	
				3E	3	HSS 21 x 21 x 0.375	88	W18x40	1.62	2.14
					2	HSS 27 x 27 x 0.5	388	W30X116	2.51	
					1	HSS 28 x 28 x 0.5	417	W30X116	2.29	
				3G	3	HSS 17 x 17 x 0.3125	48	W14x30	1.72	2.18
					2	HSS 21 x 21 x 0.375	176	W21X68	2.06	
					1	HSS 23 x 23 x 0.5	282	W18x119	2.75	
				9B	9	HSS 18 x 18 x 0.375	65	W14x34	1.49	2.14
					8	HSS 21 x 21 x 0.375	176	W21X68	1.53	
					7	HSS 23 x 23 x 0.5	282	W18x119	1.88	
					6	HSS 25 x 25 x 0.5	333	W30X99	2.11	
					5	HSS 27 x 27 x 0.5	388	W30X116	2.47	
					4	HSS 28 x 28 x 0.5	417	W30X116	2.59	
					3	HSS 28 x 28 x 0.5	417	W30X116	2.45	
					2	HSS 28 x 28 x 0.5	417	W30X116	2.37	
					1	HSS 28 x 28 x 0.5	417	W30X116	2.35	
				18B	18	HSS 13 x 13 x 0.3125	28	W12x16	1.49	2.81
					17	HSS 17 x 17 x 0.3125	96	W21x44	2.07	
					16	HSS 19 x 19 x 0.375	144	W21X62	2.40	
					15	HSS 21 x 21 x 0.375	176	W21X68	2.72	
					14	HSS 21 x 21 x 0.375	176	W21X68	2.25	
					13	HSS 21 x 21 x 0.375	176	W21X68	1.94	
					12	HSS 22 x 22 x 0.5	257	W24x94	2.51	
					11	HSS 23 x 23 x 0.5	282	W18x119	2.73	
					10	HSS 24 x 24 x 0.5	307	W21x122	3.01	
					9	HSS 24 x 24 x 0.5	307	W21x122	2.86	
					8	HSS 25 x 25 x 0.5	333	W30X99	3.21	
					7	HSS 25 x 25 x 0.5	333	W30X99	3.11	
					6	HSS 25 x 25 x 0.5	333	W30X99	3.04	
					5	HSS 26 x 26 x 0.5	360	W30X108	3.48	
					4	HSS 26 x 26 x 0.5	360	W30X108	3.44	
					3	HSS 26 x 26 x 0.5	360	W30X108	3.42	
					2	HSS 26 x 26 x 0.5	360	W30X108	3.41	
					1	HSS 26 x 26 x 0.5	360	W30X108	3.41	
	3	46	16	3A	3	HSS 21 x 21 x 0.375	88	W18x40	1.97	2.51
					2	HSS 25 x 25 x 0.5	333	W30X99	2.64	
					1	HSS 27 x 27 x 0.5	388	W30X116	2.91	
				3D	3	HSS 21 x 21 x 0.375	88	W18x40	1.86	2.44
					2	HSS 27 x 27 x 0.5	388	W30X116	2.86	
					1	HSS 28 x 28 x 0.5	417	W30X116	2.62	
				3F	3	HSS 17 x 17 x 0.3125	48	W14x30	1.98	2.49
					2	HSS 21 x 21 x 0.375	176	W21X68	2.37	
					1	HSS 23 x 23 x 0.5	282	W18x119	3.11	
				9A	9	HSS 18 x 18 x 0.375	65	W14x34	1.69	2.43
					8	HSS 21 x 21 x 0.375	176	W21X68	1.75	
					7	HSS 23 x 23 x 0.5	282	W18x119	2.13	
					6	HSS 25 x 25 x 0.5	333	W30X99	2.39	
					5	HSS 27 x 27 x 0.5	388	W30X116	2.81	
					4	HSS 28 x 28 x 0.5	417	W30X116	2.95	
					3	HSS 28 x 28 x 0.5	417	W30X116	2.79	
					2	HSS 28 x 28 x 0.5	417	W30X116	2.71	
					1	HSS 28 x 28 x 0.5	417	W30X116	2.68	
				18A	18	HSS 13 x 13 x 0.3125	28	W12x16	1.69	3.19
					17	HSS 17 x 17 x 0.3125	96	W21x44	2.38	
					16	HSS 19 x 19 x 0.375	144	W21X62	2.74	
					15	HSS 21 x 21 x 0.375	176	W21X68	3.12	
					14	HSS 21 x 21 x 0.375	176	W21X68	2.59	
					13	HSS 21 x 21 x 0.375	176	W21X68	2.24	
					12	HSS 22 x 22 x 0.5	257	W24x94	2.83	
					11	HSS 23 x 23 x 0.5	282	W18x119	3.09	
					10	HSS 24 x 24 x 0.5	307	W21x122	3.41	
					9	HSS 24 x 24 x 0.5	307	W21x122	3.24	
					8	HSS 25 x 25 x 0.5	333	W30X99	3.64	
					7	HSS 25 x 25 x 0.5	333	W30X99	3.53	
					6	HSS 25 x 25 x 0.5	333	W30X99	3.46	
					5	HSS 26 x 26 x 0.5	360	W30X108	3.95	
					4	HSS 26 x 26 x 0.5	360	W30X108	3.91	
					3	HSS 26 x 26 x 0.5	360	W30X108	3.89	
					2	HSS 26 x 26 x 0.5	360	W30X108	3.88	
					1	HSS 26 x 26 x 0.5	360	W30X108	3.88	

Member Sizes that Were Used to Calibrate η at Base Line Design Values 4, 5, and 6 (Per Figure 6.3.1) for d/t ≤ 2.26 √(E/Fy)

d/t	η Base Line Number	F_{yc} (ksi)	f'_c (ksi)	Building Geometry and Loading Based on Design No.	Story	Final Section Sizes and Properties: *Suggested* Column	Final Section Sizes and Properties: Maximum Allowed Z_g (in^3)	Final Section Sizes and Properties: *Suggested* Girder	Flexural Rigidity Ratio, η: Story Value	Flexural Rigidity Ratio, η: Building Mean Value
≤ 2.26 √(E/Fy)	4	63	4	3B	3	HSS 18 x 18 x 0.375	89	W18x40	0.73	1.07
					2	HSS 22 x 22 x 0.5	353	W30X108	1.16	
					1	HSS 24 x 24 x 0.5	420	W30X116	1.33	
				3E	3	HSS 18 x 18 x 0.375	89	W18x40	0.68	1.00
					2	HSS 24 x 24 x 0.5	420	W30X116	1.30	
					1	HSS 24 x 24 x 0.5	420	W30X116	1.01	
				3G	3	HSS 15 x 15 x 0.3125	52	W14x30	0.78	1.25
					2	HSS 19 x 19 x 0.5	263	W18x119	1.38	
					1	HSS 21 x 21 x 0.5	321	W30X99	1.58	
				9B	9	HSS 16 x 16 x 0.375	70	W18x35	0.75	1.01
					8	HSS 18 x 18 x 0.375	178	W24X68	0.64	
					7	HSS 21 x 21 x 0.5	321	W30X99	1.08	
					6	HSS 22 x 22 x 0.5	353	W30X108	1.05	
					5	HSS 24 x 24 x 0.5	420	W30X116	1.27	
					4	HSS 24 x 24 x 0.5	420	W30X116	1.16	
					3	HSS 24 x 24 x 0.5	420	W30X116	1.09	
					2	HSS 24 x 24 x 0.5	420	W30X116	1.05	
					1	HSS 24 x 24 x 0.5	420	W30X116	1.04	
				18B	18	HSS 12 x 12 x 0.3125	33	W12x22	0.85	1.46
					17	HSS 15 x 15 x 0.3125	103	W18X50	0.97	
					16	HSS 17 x 17 x 0.375	158	W24X62	1.24	
					15	HSS 18 x 18 x 0.375	178	W24X68	1.17	
					14	HSS 18 x 18 x 0.375	178	W24X68	0.94	
					13	HSS 19 x 19 x 0.5	263	W18x119	1.30	
					12	HSS 20 x 20 x 0.5	291	W30X90	1.42	
					11	HSS 21 x 21 x 0.5	321	W30X99	1.57	
					10	HSS 21 x 21 x 0.5	321	W30X99	1.46	
					9	HSS 22 x 22 x 0.5	353	W30X108	1.66	
					8	HSS 22 x 22 x 0.5	353	W30X108	1.59	
					7	HSS 22 x 22 x 0.5	353	W30X108	1.54	
					6	HSS 23 x 23 x 0.5	386	W30X116	1.79	
					5	HSS 23 x 23 x 0.5	386	W30X116	1.76	
					4	HSS 23 x 23 x 0.5	386	W30X116	1.74	
					3	HSS 23 x 23 x 0.5	386	W30X116	1.73	
					2	HSS 23 x 23 x 0.5	386	W30X116	1.73	
					1	HSS 23 x 23 x 0.5	386	W30X116	1.73	
	5	63	10	3A	3	HSS 18 x 18 x 0.375	89	W18x40	0.92	1.33
					2	HSS 22 x 22 x 0.5	353	W30X108	1.42	
					1	HSS 24 x 24 x 0.5	420	W30X116	1.63	
				3D	3	HSS 18 x 18 x 0.375	89	W18x40	0.86	1.24
					2	HSS 24 x 24 x 0.5	420	W30X116	1.60	
					1	HSS 24 x 24 x 0.5	420	W30X116	1.26	
				3F	3	HSS 15 x 15 x 0.3125	52	W14x30	1.00	1.22
					2	HSS 18 x 18 x 0.375	178	W24X68	1.08	
					1	HSS 20 x 20 x 0.5	291	W30X90	1.58	
				9A	9	HSS 16 x 16 x 0.375	70	W18x35	0.92	1.23
					8	HSS 18 x 18 x 0.375	178	W24X68	0.81	
					7	HSS 20 x 20 x 0.5	291	W30X90	1.09	
					6	HSS 22 x 22 x 0.5	353	W30X108	1.28	
					5	HSS 24 x 24 x 0.5	420	W30X116	1.57	
					4	HSS 24 x 24 x 0.5	420	W30X116	1.43	
					3	HSS 24 x 24 x 0.5	420	W30X116	1.35	
					2	HSS 24 x 24 x 0.5	420	W30X116	1.30	
					1	HSS 24 x 24 x 0.5	420	W30X116	1.29	
				18A	18	HSS 11 x 11 x 0.3125	28	W12x16	0.71	1.70
					17	HSS 15 x 15 x 0.3125	103	W18X50	1.23	
					16	HSS 17 x 17 x 0.375	158	W24X62	1.54	
					15	HSS 18 x 18 x 0.375	178	W24X68	1.47	
					14	HSS 18 x 18 x 0.375	178	W24X68	1.19	
					13	HSS 19 x 19 x 0.5	263	W18x119	1.57	
					12	HSS 20 x 20 x 0.5	291	W30X90	1.72	
					11	HSS 20 x 20 x 0.5	291	W30X90	1.57	
					10	HSS 21 x 21 x 0.5	321	W30X99	1.79	
					9	HSS 21 x 21 x 0.5	321	W30X99	1.69	
					8	HSS 22 x 22 x 0.5	353	W30X108	1.95	
					7	HSS 22 x 22 x 0.5	353	W30X108	1.89	
					6	HSS 22 x 22 x 0.5	353	W30X108	1.85	
					5	HSS 22 x 22 x 0.5	353	W30X108	1.82	
					4	HSS 23 x 23 x 0.5	386	W30X116	2.15	
					3	HSS 23 x 23 x 0.5	386	W30X116	2.13	
					2	HSS 23 x 23 x 0.5	386	W30X116	2.13	
					1	HSS 23 x 23 x 0.5	386	W30X116	2.13	
	6	63	16	3B	3	HSS 18 x 18 x 0.375	89	W18x40	1.06	1.41
					2	HSS 22 x 22 x 0.5	353	W30X108	1.61	
					1	HSS 23 x 23 x 0.5	386	W30X116	1.57	
				3E	3	HSS 18 x 18 x 0.375	89	W18x40	0.99	1.41
					2	HSS 24 x 24 x 0.5	420	W30X116	1.82	
					1	HSS 24 x 24 x 0.5	420	W30X116	1.43	
				3G	3	HSS 15 x 15 x 0.3125	52	W14x30	1.16	1.40
					2	HSS 18 x 18 x 0.375	178	W24X68	1.25	
					1	HSS 20 x 20 x 0.5	291	W30X90	1.79	
				9B	9	HSS 16 x 16 x 0.375	70	W18x35	1.05	1.37
					8	HSS 18 x 18 x 0.375	178	W24X68	0.93	
					7	HSS 20 x 20 x 0.5	291	W30X90	1.23	
					6	HSS 22 x 22 x 0.5	353	W30X108	1.45	
					5	HSS 23 x 23 x 0.5	386	W30X116	1.51	
					4	HSS 24 x 24 x 0.5	420	W30X116	1.63	
					3	HSS 24 x 24 x 0.5	420	W30X116	1.54	
					2	HSS 24 x 24 x 0.5	420	W30X116	1.49	
					1	HSS 24 x 24 x 0.5	420	W30X116	1.47	
				18B	18	HSS 11 x 11 x 0.3125	28	W12x16	0.81	1.84
					17	HSS 15 x 15 x 0.3125	103	W18X50	1.42	
					16	HSS 17 x 17 x 0.375	158	W24X62	1.75	
					15	HSS 18 x 18 x 0.375	178	W24X68	1.68	
					14	HSS 18 x 18 x 0.375	178	W24X68	1.37	
					13	HSS 19 x 19 x 0.5	263	W18x119	1.77	
					12	HSS 20 x 20 x 0.5	291	W30X90	1.95	
					11	HSS 20 x 20 x 0.5	291	W30X90	1.78	
					10	HSS 21 x 21 x 0.5	321	W30X99	2.02	
					9	HSS 21 x 21 x 0.5	321	W30X99	1.92	
					8	HSS 22 x 22 x 0.5	353	W30X108	2.21	
					7	HSS 22 x 22 x 0.5	353	W30X108	2.15	
					6	HSS 22 x 22 x 0.5	353	W30X108	2.10	
					5	HSS 22 x 22 x 0.5	353	W30X108	2.06	
					4	HSS 22 x 22 x 0.5	353	W30X108	2.04	
					3	HSS 22 x 22 x 0.5	353	W30X108	2.03	
					2	HSS 22 x 22 x 0.5	353	W30X108	2.02	
					1	HSS 22 x 22 x 0.5	353	W30X108	2.02	

Member Sizes that Were Used to Calibrate η at Base Line Design Values 7, 8, and 9 (Per Figure 6.3.1) for d/t ≤ 2.26 √(E/Fy)

d/t	η Base Line Number	F_{yc} (ksi)	f'_c (ksi)	Building Geometry and Loading Based on Design No.	Story	Final Section Sizes and Properties: *Suggested* Column	Maximum Allowed Z_g (in^3)	*Suggested* Girder	Flexural Rigidity Ratio, η: Story Value	Building Mean Value
≤ 2.26 √(E/Fy)	7	80	4	3A	3	HSS 16 x 16 x 0.375	89	W18x40	0.43	0.66
					2	HSS 20 x 20 x 0.5	370	W30X108	0.79	
					1	HSS 21 x 21 x 0.5	408	W30X116	0.77	
				3D	3	HSS 16 x 16 x 0.5	119	W18X55	0.53	0.72
					2	HSS 21 x 21 x 0.5	408	W30X116	0.76	
					1	HSS 22 x 22 x 0.625	560	W30X116	0.86	
				3F	3	HSS 14 x 14 x 0.375	68	W18x35	0.67	0.86
					2	HSS 17 x 17 x 0.5	268	W18x119	0.86	
					1	HSS 19 x 19 x 0.5	334	W30X99	1.05	
				9A	9	HSS 15 x 15 x 0.375	79	W18x40	0.56	0.67
					8	HSS 17 x 17 x 0.5	268	W18x119	0.65	
					7	HSS 19 x 19 x 0.5	334	W30X99	0.72	
					6	HSS 21 x 21 x 0.5	408	W30X116	0.87	
					5	HSS 21 x 21 x 0.5	408	W30X116	0.74	
					4	HSS 21 x 21 x 0.5	408	W30X116	0.67	
					3	HSS 21 x 21 x 0.5	408	W30X116	0.62	
					2	HSS 21 x 21 x 0.5	408	W30X116	0.60	
					1	HSS 21 x 21 x 0.5	408	W30X116	0.59	
				18A	18	HSS 11 x 11 x 0.3125	35	W14x22	0.56	1.00
					17	HSS 13 x 13 x 0.3125	97	W21x44	0.48	
					16	HSS 16 x 16 x 0.375	178	W24X68	0.95	
					15	HSS 16 x 16 x 0.375	178	W24X68	0.69	
					14	HSS 17 x 17 x 0.5	268	W18x119	0.95	
					13	HSS 18 x 18 x 0.5	300	W30X90	1.03	
					12	HSS 18 x 18 x 0.5	300	W30X90	0.91	
					11	HSS 19 x 19 x 0.5	334	W30X99	1.04	
					10	HSS 19 x 19 x 0.5	334	W30X99	0.96	
					9	HSS 20 x 20 x 0.5	370	W30X108	1.13	
					8	HSS 20 x 20 x 0.5	370	W30X108	1.07	
					7	HSS 20 x 20 x 0.5	370	W30X108	1.04	
					6	HSS 21 x 21 x 0.5	408	W30X116	1.24	
					5	HSS 21 x 21 x 0.5	408	W30X116	1.22	
					4	HSS 21 x 21 x 0.5	408	W30X116	1.21	
					3	HSS 21 x 21 x 0.5	408	W30X116	1.20	
					2	HSS 21 x 21 x 0.5	408	W30X116	1.19	
					1	HSS 21 x 21 x 0.5	408	W30X116	1.19	
	8	80	10	3B	3	HSS 16 x 16 x 0.375	89	W18x40	0.54	0.82
					2	HSS 20 x 20 x 0.5	370	W30X108	0.97	
					1	HSS 21 x 21 x 0.5	408	W30X116	0.96	
				3E	3	HSS 16 x 16 x 0.375	89	W18x40	0.50	0.82
					2	HSS 21 x 21 x 0.5	408	W30X116	0.93	
					1	HSS 22 x 22 x 0.625	560	W30X116	1.04	
				3G	3	HSS 14 x 14 x 0.375	68	W18x35	0.84	1.06
					2	HSS 17 x 17 x 0.5	268	W18x119	1.05	
					1	HSS 19 x 19 x 0.5	334	W30X99	1.28	
				9B	9	HSS 15 x 15 x 0.375	79	W18x40	0.70	0.78
					8	HSS 16 x 16 x 0.5	238	W24X84	0.60	
					7	HSS 19 x 19 x 0.5	334	W30X99	0.88	
					6	HSS 20 x 20 x 0.5	370	W30X108	0.87	
					5	HSS 21 x 21 x 0.5	408	W30X116	0.92	
					4	HSS 21 x 21 x 0.5	408	W30X116	0.83	
					3	HSS 21 x 21 x 0.5	408	W30X116	0.77	
					2	HSS 21 x 21 x 0.5	408	W30X116	0.75	
					1	HSS 21 x 21 x 0.5	408	W30X116	0.74	
				18B	18	HSS 11 x 11 x 0.3125	35	W14x22	0.71	1.15
					17	HSS 13 x 13 x 0.3125	97	W21x44	0.63	
					16	HSS 15 x 15 x 0.375	157	W24X62	0.89	
					15	HSS 16 x 16 x 0.375	178	W24X68	0.87	
					14	HSS 16 x 16 x 0.375	178	W24X68	0.69	
					13	HSS 17 x 17 x 0.5	268	W18x119	0.98	
					12	HSS 18 x 18 x 0.5	300	W30X90	1.11	
					11	HSS 19 x 19 x 0.5	334	W30X99	1.27	
					10	HSS 19 x 19 x 0.5	334	W30X99	1.18	
					9	HSS 20 x 20 x 0.5	370	W30X108	1.38	
					8	HSS 20 x 20 x 0.5	370	W30X108	1.32	
					7	HSS 20 x 20 x 0.5	370	W30X108	1.28	
					6	HSS 20 x 20 x 0.5	370	W30X108	1.25	
					5	HSS 20 x 20 x 0.5	370	W30X108	1.22	
					4	HSS 21 x 21 x 0.5	408	W30X116	1.49	
					3	HSS 21 x 21 x 0.5	408	W30X116	1.48	
					2	HSS 21 x 21 x 0.5	408	W30X116	1.47	
					1	HSS 21 x 21 x 0.5	408	W30X116	1.47	
	9	80	16	3A	3	HSS 16 x 16 x 0.375	89	W18x40	0.63	0.94
					2	HSS 20 x 20 x 0.5	370	W30X108	1.10	
					1	HSS 21 x 21 x 0.5	408	W30X116	1.09	
				3D	3	HSS 16 x 16 x 0.375	89	W18x40	0.59	0.82
					2	HSS 21 x 21 x 0.5	408	W30X116	1.06	
					1	HSS 21 x 21 x 0.5	408	W30X116	0.82	
				3F	3	HSS 14 x 14 x 0.375	68	W18x35	0.96	1.10
					2	HSS 17 x 17 x 0.5	268	W18x119	1.18	
					1	HSS 18 x 18 x 0.5	300	W30X90	1.15	
				9A	9	HSS 14 x 14 x 0.375	68	W18x35	0.58	0.83
					8	HSS 16 x 16 x 0.375	178	W24X68	0.54	
					7	HSS 18 x 18 x 0.5	300	W30X90	0.79	
					6	HSS 20 x 20 x 0.5	370	W30X108	0.99	
					5	HSS 21 x 21 x 0.5	408	W30X116	1.04	
					4	HSS 21 x 21 x 0.5	408	W30X116	0.94	
					3	HSS 21 x 21 x 0.5	408	W30X116	0.89	
					2	HSS 21 x 21 x 0.5	408	W30X116	0.85	
					1	HSS 21 x 21 x 0.5	408	W30X116	0.84	
				18A	18	HSS 10 x 10 x 0.3125	29	W12x16	0.49	1.19
					17	HSS 13 x 13 x 0.3125	97	W21x44	0.73	
					16	HSS 15 x 15 x 0.375	157	W24X62	1.02	
					15	HSS 16 x 16 x 0.375	178	W24X68	1.01	
					14	HSS 16 x 16 x 0.375	178	W24X68	0.80	
					13	HSS 17 x 17 x 0.5	268	W18x119	1.11	
					12	HSS 18 x 18 x 0.5	300	W30X90	1.26	
					11	HSS 18 x 18 x 0.5	300	W30X90	1.14	
					10	HSS 19 x 19 x 0.5	334	W30X99	1.34	
					9	HSS 19 x 19 x 0.5	334	W30X99	1.26	
					8	HSS 20 x 20 x 0.5	370	W30X108	1.50	
					7	HSS 20 x 20 x 0.5	370	W30X108	1.45	
					6	HSS 20 x 20 x 0.5	370	W30X108	1.42	
					5	HSS 20 x 20 x 0.5	370	W30X108	1.39	
					4	HSS 20 x 20 x 0.5	370	W30X108	1.38	
					3	HSS 20 x 20 x 0.5	370	W30X108	1.37	
					2	HSS 20 x 20 x 0.5	370	W30X108	1.36	
					1	HSS 20 x 20 x 0.5	370	W30X108	1.36	

Member Sizes that Were Used to Calibrate η at Base Line Design Values 1, 2, and 3 (Per Figure 6.3.1) for d/t ratios = 80

d/t	η Base Line Number	F_{yc} (ksi)	f'_c (ksi)	Building Geometry and Loading Based on Design No.	Story	Final Section Sizes and Properties			Flexural Rigidity Ratio, η	
						Suggested Column	Maximum Allowed Z_g (in^3)	*Suggested* Girder	Story Value	Building Mean Value
= 80	1	46	4	3A	3	HSS 24 x 24 x 0.3125	96	W21x44	2.02	2.32
					2	HSS 29 x 29 x 0.375	336	W30X99	2.70	
					1	HSS 29 x 29 x 0.375	336	W30X99	2.23	
				3D	3	HSS 24 x 24 x 0.3125	96	W21x44	1.91	2.16
					2	HSS 29 x 29 x 0.375	336	W30X99	2.19	
					1	HSS 30 x 30 x 0.5	479	W30X116	2.38	
				3F	3	HSS 19 x 19 x 0.3125	60	W14x34	2.12	2.64
					2	HSS 24 x 24 x 0.3125	191	W24X68	2.43	
					1	HSS 27 x 27 x 0.375	291	W30X90	3.36	
				9A	9	HSS 20 x 20 x 0.3125	67	W18x35	1.59	1.96
					8	HSS 24 x 24 x 0.3125	191	W24X68	1.80	
					7	HSS 27 x 27 x 0.375	291	W30X90	2.30	
					6	HSS 29 x 29 x 0.375	336	W30X99	2.45	
					5	HSS 29 x 29 x 0.375	336	W30X99	2.15	
					4	HSS 29 x 29 x 0.375	336	W30X99	1.96	
					3	HSS 29 x 29 x 0.375	336	W30X99	1.85	
					2	HSS 29 x 29 x 0.375	336	W30X99	1.79	
					1	HSS 29 x 29 x 0.375	336	W30X99	1.77	
				18A	18	HSS 13 x 13 x 0.3125	28	W12x16	1.21	3.10
					17	HSS 18 x 18 x 0.3125	107	W18X50	2.06	
					16	HSS 21 x 21 x 0.3125	146	W21X62	2.50	
					15	HSS 23 x 23 x 0.3125	176	W21X68	2.72	
					14	HSS 24 x 24 x 0.3125	191	W24X68	2.65	
					13	HSS 25 x 25 x 0.375	250	W27X84	3.03	
					12	HSS 26 x 26 x 0.375	269	W18x119	3.15	
					11	HSS 27 x 27 x 0.375	291	W30X90	3.34	
					10	HSS 28 x 28 x 0.375	313	W30X99	3.58	
					9	HSS 28 x 28 x 0.375	313	W30X99	3.40	
					8	HSS 29 x 29 x 0.375	336	W30X99	3.73	
					7	HSS 29 x 29 x 0.375	336	W30X99	3.62	
					6	HSS 29 x 29 x 0.375	336	W30X99	3.54	
					5	HSS 29 x 29 x 0.375	336	W30X99	3.49	
					4	HSS 29 x 29 x 0.375	336	W30X99	3.46	
					3	HSS 29 x 29 x 0.375	336	W30X99	3.44	
					2	HSS 29 x 29 x 0.375	336	W30X99	3.43	
					1	HSS 29 x 29 x 0.375	336	W30X99	3.42	
	2	46	10	3B	3	HSS 24 x 24 x 0.3125	96	W21x44	2.61	2.84
					2	HSS 28 x 28 x 0.375	313	W30X99	3.04	
					1	HSS 29 x 29 x 0.375	336	W30X99	2.87	
				3E	3	HSS 24 x 24 x 0.3125	96	W21x44	2.46	2.75
					2	HSS 29 x 29 x 0.375	336	W30X99	2.82	
					1	HSS 30 x 30 x 0.5	479	W30X116	2.97	
				3G	3	HSS 19 x 19 x 0.3125	60	W14x34	2.68	3.38
					2	HSS 24 x 24 x 0.3125	191	W24X68	3.16	
					1	HSS 27 x 27 x 0.375	291	W30X90	4.29	
				9B	9	HSS 20 x 20 x 0.3125	67	W18x35	2.01	2.53
					8	HSS 24 x 24 x 0.3125	191	W24X68	2.32	
					7	HSS 27 x 27 x 0.375	291	W30X90	2.93	
					6	HSS 29 x 29 x 0.375	336	W30X99	3.16	
					5	HSS 29 x 29 x 0.375	336	W30X99	2.77	
					4	HSS 29 x 29 x 0.375	336	W30X99	2.53	
					3	HSS 29 x 29 x 0.375	336	W30X99	2.40	
					2	HSS 29 x 29 x 0.375	336	W30X99	2.32	
					1	HSS 29 x 29 x 0.375	336	W30X99	2.30	
				18B	18	HSS 13 x 13 x 0.3125	28	W12x16	1.49	3.83
					17	HSS 17 x 17 x 0.3125	96	W21x44	2.07	
					16	HSS 21 x 21 x 0.3125	146	W21X62	3.19	
					15	HSS 23 x 23 x 0.3125	176	W21X68	3.50	
					14	HSS 24 x 24 x 0.3125	191	W24X68	3.43	
					13	HSS 24 x 24 x 0.3125	191	W24X68	2.98	
					12	HSS 26 x 26 x 0.375	269	W18x119	4.01	
					11	HSS 27 x 27 x 0.375	291	W30X90	4.26	
					10	HSS 27 x 27 x 0.375	291	W30X90	4.00	
					9	HSS 28 x 28 x 0.375	313	W30X99	4.37	
					8	HSS 28 x 28 x 0.375	313	W30X99	4.20	
					7	HSS 29 x 29 x 0.375	336	W30X99	4.66	
					6	HSS 29 x 29 x 0.375	336	W30X99	4.56	
					5	HSS 29 x 29 x 0.375	336	W30X99	4.49	
					4	HSS 29 x 29 x 0.375	336	W30X99	4.45	
					3	HSS 29 x 29 x 0.375	336	W30X99	4.42	
					2	HSS 29 x 29 x 0.375	336	W30X99	4.41	
					1	HSS 29 x 29 x 0.375	336	W30X99	4.41	
	3	46	16	3A	3	HSS 24 x 24 x 0.3125	96	W21x44	3.03	3.30
					2	HSS 28 x 28 x 0.375	313	W30X99	3.52	
					1	HSS 29 x 29 x 0.375	336	W30X99	3.33	
				3D	3	HSS 24 x 24 x 0.3125	96	W21x44	2.86	3.18
					2	HSS 29 x 29 x 0.375	336	W30X99	3.27	
					1	HSS 30 x 30 x 0.5	479	W30X116	3.40	
				3F	3	HSS 19 x 19 x 0.3125	60	W14x34	3.09	3.22
					2	HSS 24 x 24 x 0.3125	191	W24X68	3.68	
					1	HSS 24 x 24 x 0.3125	191	W24X68	2.89	
				9A	9	HSS 20 x 20 x 0.3125	67	W18x35	2.32	2.88
					8	HSS 24 x 24 x 0.3125	191	W24X68	2.70	
					7	HSS 26 x 26 x 0.375	269	W18x119	2.94	
					6	HSS 29 x 29 x 0.375	336	W30X99	3.66	
					5	HSS 29 x 29 x 0.375	336	W30X99	3.21	
					4	HSS 29 x 29 x 0.375	336	W30X99	2.95	
					3	HSS 29 x 29 x 0.375	336	W30X99	2.79	
					2	HSS 29 x 29 x 0.375	336	W30X99	2.70	
					1	HSS 29 x 29 x 0.375	336	W30X99	2.67	
				18A	18	HSS 13 x 13 x 0.3125	28	W12x16	1.69	4.36
					17	HSS 17 x 17 x 0.3125	96	W21x44	2.38	
					16	HSS 20 x 20 x 0.3125	133	W18x65	3.04	
					15	HSS 23 x 23 x 0.3125	176	W21X68	4.06	
					14	HSS 24 x 24 x 0.3125	191	W24X68	3.99	
					13	HSS 24 x 24 x 0.3125	191	W24X68	3.48	
					12	HSS 26 x 26 x 0.375	269	W18x119	4.63	
					11	HSS 26 x 26 x 0.375	269	W18x119	4.27	
					10	HSS 27 x 27 x 0.375	291	W30X90	4.63	
					9	HSS 28 x 28 x 0.375	313	W30X99	5.06	
					8	HSS 28 x 28 x 0.375	313	W30X99	4.87	
					7	HSS 29 x 29 x 0.375	336	W30X99	5.41	
					6	HSS 29 x 29 x 0.375	336	W30X99	5.29	
					5	HSS 29 x 29 x 0.375	336	W30X99	5.22	
					4	HSS 29 x 29 x 0.375	336	W30X99	5.16	
					3	HSS 29 x 29 x 0.375	336	W30X99	5.13	
					2	HSS 29 x 29 x 0.375	336	W30X99	5.12	
					1	HSS 29 x 29 x 0.375	336	W30X99	5.12	

Member Sizes that Were Used to Calibrate η at Base Line Design Values 4, 5, and 6 (Per Figure 6.3.1) for d/t ≈ 80

d/t	η Base Line Number	F_{yc} (ksi)	f'_c (ksi)	Building Geometry and Loading Based on Design No.	Story	Final Section Sizes and Properties: *Suggested* Column	Maximum Allowed Z_g (in^3)	*Suggested* Girder	Flexural Rigidity Ratio, η: Story Value	Building Mean Value
≈ 80	4	63	4	3B	3	HSS 21 x 21 x 0.3125	100	W21x44	1.20	1.48
					2	HSS 25 x 25 x 0.375	342	W30X99	1.54	
					1	HSS 27 x 27 x 0.375	398	W30X116	1.70	
				3E	3	HSS 22 x 22 x 0.3125	110	W21X50	1.36	1.59
					2	HSS 27 x 27 x 0.375	398	W30X116	1.66	
					1	HSS 29 x 29 x 0.375	460	W30X116	1.73	
				3G	3	HSS 17 x 17 x 0.3125	66	W14x34	1.36	1.77
					2	HSS 23 x 23 x 0.3125	241	W24X84	2.06	
					1	HSS 24 x 24 x 0.3125	262	W18x119	1.89	
				9B	9	HSS 18 x 18 x 0.3125	74	W16x40	1.05	1.56
					8	HSS 22 x 22 x 0.3125	220	W12x136	1.28	
					7	HSS 24 x 24 x 0.3125	262	W18x119	1.30	
					6	HSS 26 x 26 x 0.375	369	W30X108	1.62	
					5	HSS 27 x 27 x 0.375	398	W30X116	1.63	
					4	HSS 28 x 28 x 0.375	429	W30X116	1.71	
					3	HSS 29 x 29 x 0.375	460	W30X116	1.85	
					2	HSS 29 x 29 x 0.375	460	W30X116	1.79	
					1	HSS 29 x 29 x 0.375	460	W30X116	1.77	
				18B	18	HSS 12 x 12 x 0.3125	33	W12x22	0.85	1.92
					17	HSS 16 x 16 x 0.3125	116	W18X55	1.28	
					16	HSS 18 x 18 x 0.3125	147	W21X62	1.35	
					15	HSS 20 x 20 x 0.3125	182	W24X68	1.57	
					14	HSS 22 x 22 x 0.3125	220	W12x136	1.89	
					13	HSS 23 x 23 x 0.3125	241	W24X84	1.94	
					12	HSS 24 x 24 x 0.3125	262	W18x119	2.05	
					11	HSS 24 x 24 x 0.3125	262	W18x119	1.88	
					10	HSS 24 x 24 x 0.3125	262	W18x119	1.75	
					9	HSS 25 x 25 x 0.375	342	W30X99	2.21	
					8	HSS 25 x 25 x 0.375	342	W30X99	2.12	
					7	HSS 25 x 25 x 0.375	342	W30X99	2.05	
					6	HSS 26 x 26 x 0.375	369	W30X108	2.34	
					5	HSS 26 x 26 x 0.375	369	W30X108	2.30	
					4	HSS 26 x 26 x 0.375	369	W30X108	2.28	
					3	HSS 26 x 26 x 0.375	369	W30X108	2.26	
					2	HSS 26 x 26 x 0.375	369	W30X108	2.26	
					1	HSS 26 x 26 x 0.375	369	W30X108	2.26	
	5	63	10	3A	3	HSS 21 x 21 x 0.3125	100	W21x44	1.55	1.90
					2	HSS 25 x 25 x 0.375	342	W30X99	1.97	
					1	HSS 27 x 27 x 0.375	398	W30X116	2.18	
				3D	3	HSS 22 x 22 x 0.3125	110	W21X50	1.75	2.05
					2	HSS 27 x 27 x 0.375	398	W30X116	2.14	
					1	HSS 29 x 29 x 0.375	460	W30X116	2.24	
				3F	3	HSS 16 x 16 x 0.3125	58	W14x34	1.33	2.16
					2	HSS 23 x 23 x 0.3125	241	W24X84	2.67	
					1	HSS 24 x 24 x 0.3125	262	W18x119	2.47	
				9A	9	HSS 17 x 17 x 0.3125	66	W14x34	1.05	1.94
					8	HSS 22 x 22 x 0.3125	220	W12x136	1.65	
					7	HSS 24 x 24 x 0.3125	262	W18x119	1.69	
					6	HSS 25 x 25 x 0.375	342	W30X99	1.78	
					5	HSS 27 x 27 x 0.375	398	W30X116	2.10	
					4	HSS 28 x 28 x 0.375	429	W30X116	2.21	
					3	HSS 29 x 29 x 0.375	460	W30X116	2.40	
					2	HSS 29 x 29 x 0.375	460	W30X116	2.32	
					1	HSS 29 x 29 x 0.375	460	W30X116	2.30	
				18A	18	HSS 11 x 11 x 0.3125	28	W12x16	0.71	2.37
					17	HSS 15 x 15 x 0.3125	103	W18X50	1.23	
					16	HSS 18 x 18 x 0.3125	147	W21X62	1.73	
					15	HSS 20 x 20 x 0.3125	182	W24X68	2.02	
					14	HSS 22 x 22 x 0.3125	220	W12x136	2.44	
					13	HSS 23 x 23 x 0.3125	241	W24X84	2.52	
					12	HSS 24 x 24 x 0.3125	262	W18x119	2.67	
					11	HSS 24 x 24 x 0.3125	262	W18x119	2.45	
					10	HSS 24 x 24 x 0.3125	262	W18x119	2.29	
					9	HSS 24 x 24 x 0.3125	262	W18x119	2.17	
					8	HSS 25 x 25 x 0.375	342	W30X99	2.71	
					7	HSS 25 x 25 x 0.375	342	W30X99	2.63	
					6	HSS 25 x 25 x 0.375	342	W30X99	2.57	
					5	HSS 26 x 26 x 0.375	369	W30X108	2.95	
					4	HSS 26 x 26 x 0.375	369	W30X108	2.92	
					3	HSS 26 x 26 x 0.375	369	W30X108	2.90	
					2	HSS 26 x 26 x 0.375	369	W30X108	2.90	
					1	HSS 26 x 26 x 0.375	369	W30X108	2.89	
	6	63	16	3B	3	HSS 21 x 21 x 0.3125	100	W21x44	1.80	2.04
					2	HSS 24 x 24 x 0.3125	262	W18x119	1.78	
					1	HSS 27 x 27 x 0.375	398	W30X116	2.53	
				3E	3	HSS 22 x 22 x 0.3125	110	W21X50	2.04	2.38
					2	HSS 27 x 27 x 0.375	398	W30X116	2.48	
					1	HSS 29 x 29 x 0.375	460	W30X116	2.61	
				3G	3	HSS 16 x 16 x 0.3125	58	W14x34	1.54	2.34
					2	HSS 22 x 22 x 0.3125	220	W12x136	2.61	
					1	HSS 24 x 24 x 0.3125	262	W18x119	2.89	
				9B	9	HSS 17 x 17 x 0.3125	66	W14x34	1.21	2.22
					8	HSS 22 x 22 x 0.3125	220	W12x136	1.92	
					7	HSS 24 x 24 x 0.3125	262	W18x119	1.98	
					6	HSS 25 x 25 x 0.375	342	W30X99	2.06	
					5	HSS 27 x 27 x 0.375	398	W30X116	2.44	
					4	HSS 28 x 28 x 0.375	429	W30X116	2.57	
					3	HSS 28 x 28 x 0.375	429	W30X116	2.43	
					2	HSS 29 x 29 x 0.375	460	W30X116	2.70	
					1	HSS 29 x 29 x 0.375	460	W30X116	2.67	
				18B	18	HSS 11 x 11 x 0.3125	28	W12x16	0.81	2.70
					17	HSS 15 x 15 x 0.3125	103	W18X50	1.42	
					16	HSS 18 x 18 x 0.3125	147	W21X62	2.00	
					15	HSS 20 x 20 x 0.3125	182	W24X68	2.34	
					14	HSS 22 x 22 x 0.3125	220	W12x136	2.84	
					13	HSS 23 x 23 x 0.3125	241	W24X84	2.94	
					12	HSS 24 x 24 x 0.3125	262	W18x119	3.12	
					11	HSS 24 x 24 x 0.3125	262	W18x119	2.87	
					10	HSS 24 x 24 x 0.3125	262	W18x119	2.68	
					9	HSS 24 x 24 x 0.3125	262	W18x119	2.55	
					8	HSS 25 x 25 x 0.375	342	W30X99	3.14	
					7	HSS 25 x 25 x 0.375	342	W30X99	3.05	
					6	HSS 25 x 25 x 0.375	342	W30X99	2.98	
					5	HSS 25 x 25 x 0.375	342	W30X99	2.93	
					4	HSS 25 x 25 x 0.375	342	W30X99	2.90	
					3	HSS 26 x 26 x 0.375	369	W30X108	3.37	
					2	HSS 26 x 26 x 0.375	369	W30X108	3.36	
					1	HSS 26 x 26 x 0.375	369	W30X108	3.36	

Member Sizes that Were Used to Calibrate η at Base Line Design Values 7, 8, and 9 (Per Figure 6.3.1) for d/t ≈ 80

d/t	η Base Line Number	F_{yc} (ksi)	f'_c (ksi)	Building Geometry and Loading Based on Design No.	Story	Final Section Sizes and Properties			Flexural Rigidity Ratio, η	
						Suggested Column	Maximum Allowed Z_g (in^3)	*Suggested* Girder	Story Value	Building Mean Value
≈ 80	7	80	4	3A	3	HSS 20 x 20 x 0.3125	116	W18X55	0.99	1.13
					2	HSS 24 x 24 x 0.3125	333	W30X99	1.16	
					1	HSS 25 x 25 x 0.375	435	W30X116	1.26	
				3D	3	HSS 20 x 20 x 0.3125	116	W18X55	0.92	1.15
					2	HSS 25 x 25 x 0.375	435	W30X116	1.23	
					1	HSS 27 x 27 x 0.375	506	W30X116	1.31	
				3F	3	HSS 15 x 15 x 0.3125	65	W14x34	0.78	1.27
					2	HSS 21 x 21 x 0.3125	255	W24x94	1.42	
					1	HSS 23 x 23 x 0.3125	306	W30X90	1.59	
				9A	9	HSS 16 x 16 x 0.3125	74	W16x40	0.63	1.07
					8	HSS 20 x 20 x 0.3125	231	W24X84	0.87	
					7	HSS 23 x 23 x 0.3125	306	W30X90	1.09	
					6	HSS 24 x 24 x 0.3125	333	W30X99	1.04	
					5	HSS 25 x 25 x 0.375	435	W30X116	1.21	
					4	HSS 25 x 25 x 0.375	435	W30X116	1.10	
					3	HSS 26 x 26 x 0.375	469	W30X116	1.21	
					2	HSS 26 x 26 x 0.375	469	W30X116	1.17	
					1	HSS 27 x 27 x 0.375	506	W30X116	1.34	
				18A	18	HSS 11 x 11 x 0.3125	35	W14x22	0.56	1.36
					17	HSS 14 x 14 x 0.3125	113	W18X55	0.71	
					16	HSS 17 x 17 x 0.3125	167	W21X68	1.06	
					15	HSS 19 x 19 x 0.3125	208	W24X76	1.27	
					14	HSS 20 x 20 x 0.3125	231	W24X84	1.27	
					13	HSS 21 x 21 x 0.3125	255	W24x94	1.34	
					12	HSS 22 x 22 x 0.3125	280	W18x119	1.44	
					11	HSS 23 x 23 x 0.3125	306	W30X90	1.58	
					10	HSS 24 x 24 x 0.3125	333	W30X99	1.75	
					9	HSS 24 x 24 x 0.3125	333	W30X99	1.66	
					8	HSS 24 x 24 x 0.3125	333	W30X99	1.59	
					7	HSS 24 x 24 x 0.3125	333	W30X99	1.53	
					6	HSS 24 x 24 x 0.3125	333	W30X99	1.50	
					5	HSS 24 x 24 x 0.3125	333	W30X99	1.47	
					4	HSS 24 x 24 x 0.3125	333	W30X99	1.46	
					3	HSS 24 x 24 x 0.3125	333	W30X99	1.45	
					2	HSS 24 x 24 x 0.3125	333	W30X99	1.44	
					1	HSS 24 x 24 x 0.3125	333	W30X99	1.44	
	8	80	10	3B	3	HSS 19 x 19 x 0.3125	104	W18X50	1.03	1.26
					2	HSS 24 x 24 x 0.3125	333	W30X99	1.52	
					1	HSS 24 x 24 x 0.3125	333	W30X99	1.24	
				3E	3	HSS 20 x 20 x 0.3125	116	W18X55	1.19	1.37
					2	HSS 24 x 24 x 0.3125	333	W30X99	1.21	
					1	HSS 27 x 27 x 0.375	506	W30X116	1.69	
				3G	3	HSS 15 x 15 x 0.3125	65	W14x34	1.00	1.64
					2	HSS 21 x 21 x 0.3125	255	W24x94	1.85	
					1	HSS 23 x 23 x 0.3125	306	W30X90	2.08	
				9B	9	HSS 16 x 16 x 0.3125	74	W16x40	0.81	1.32
					8	HSS 20 x 20 x 0.3125	231	W24X84	1.12	
					7	HSS 23 x 23 x 0.3125	306	W30X90	1.42	
					6	HSS 24 x 24 x 0.3125	333	W30X99	1.37	
					5	HSS 24 x 24 x 0.3125	333	W30X99	1.19	
					4	HSS 25 x 25 x 0.375	435	W30X116	1.41	
					3	HSS 26 x 26 x 0.375	469	W30X116	1.56	
					2	HSS 26 x 26 x 0.375	469	W30X116	1.51	
					1	HSS 26 x 26 x 0.375	469	W30X116	1.49	
				18B	18	HSS 11 x 11 x 0.3125	35	W14x22	0.71	1.72
					17	HSS 14 x 14 x 0.3125	113	W18X55	0.90	
					16	HSS 16 x 16 x 0.3125	148	W21X62	1.04	
					15	HSS 18 x 18 x 0.3125	187	W24X68	1.30	
					14	HSS 20 x 20 x 0.3125	231	W24X84	1.65	
					13	HSS 21 x 21 x 0.3125	255	W24x94	1.74	
					12	HSS 22 x 22 x 0.3125	280	W18x119	1.88	
					11	HSS 23 x 23 x 0.3125	306	W30X90	2.07	
					10	HSS 23 x 23 x 0.3125	306	W30X90	1.93	
					9	HSS 24 x 24 x 0.3125	333	W30X99	2.17	
					8	HSS 24 x 24 x 0.3125	333	W30X99	2.09	
					7	HSS 24 x 24 x 0.3125	333	W30X99	2.02	
					6	HSS 24 x 24 x 0.3125	333	W30X99	1.97	
					5	HSS 24 x 24 x 0.3125	333	W30X99	1.94	
					4	HSS 24 x 24 x 0.3125	333	W30X99	1.92	
					3	HSS 24 x 24 x 0.3125	333	W30X99	1.91	
					2	HSS 24 x 24 x 0.3125	333	W30X99	1.90	
					1	HSS 24 x 24 x 0.3125	333	W30X99	1.90	
	9	80	16	3A	3	HSS 19 x 19 x 0.3125	104	W18X50	1.20	1.48
					2	HSS 24 x 24 x 0.3125	333	W30X99	1.78	
					1	HSS 24 x 24 x 0.3125	333	W30X99	1.45	
				3D	3	HSS 20 x 20 x 0.3125	116	W18X55	1.39	1.60
					2	HSS 24 x 24 x 0.3125	333	W30X99	1.42	
					1	HSS 27 x 27 x 0.375	506	W30X116	1.97	
				3F	3	HSS 15 x 15 x 0.3125	65	W14x34	1.16	1.79
					2	HSS 20 x 20 x 0.3125	231	W24X84	1.76	
					1	HSS 23 x 23 x 0.3125	306	W30X90	2.43	
				9A	9	HSS 16 x 16 x 0.3125	74	W16x40	0.94	1.54
					8	HSS 20 x 20 x 0.3125	231	W24X84	1.31	
					7	HSS 23 x 23 x 0.3125	306	W30X90	1.67	
					6	HSS 24 x 24 x 0.3125	333	W30X99	1.61	
					5	HSS 24 x 24 x 0.3125	333	W30X99	1.40	
					4	HSS 25 x 25 x 0.375	435	W30X116	1.64	
					3	HSS 26 x 26 x 0.375	469	W30X116	1.81	
					2	HSS 26 x 26 x 0.375	469	W30X116	1.76	
					1	HSS 26 x 26 x 0.375	469	W30X116	1.74	
				18A	18	HSS 10 x 10 x 0.3125	29	W12x16	0.49	2.00
					17	HSS 14 x 14 x 0.3125	113	W18X55	1.05	
					16	HSS 16 x 16 x 0.3125	148	W21X62	1.21	
					15	HSS 18 x 18 x 0.3125	187	W24X68	1.51	
					14	HSS 20 x 20 x 0.3125	231	W24X84	1.93	
					13	HSS 21 x 21 x 0.3125	255	W24x94	2.03	
					12	HSS 22 x 22 x 0.3125	280	W18x119	2.20	
					11	HSS 23 x 23 x 0.3125	306	W30X90	2.42	
					10	HSS 23 x 23 x 0.3125	306	W30X90	2.26	
					9	HSS 24 x 24 x 0.3125	333	W30X99	2.55	
					8	HSS 24 x 24 x 0.3125	333	W30X99	2.45	
					7	HSS 24 x 24 x 0.3125	333	W30X99	2.37	
					6	HSS 24 x 24 x 0.3125	333	W30X99	2.32	
					5	HSS 24 x 24 x 0.3125	333	W30X99	2.28	
					4	HSS 24 x 24 x 0.3125	333	W30X99	2.26	
					3	HSS 24 x 24 x 0.3125	333	W30X99	2.24	
					2	HSS 24 x 24 x 0.3125	333	W30X99	2.24	
					1	HSS 24 x 24 x 0.3125	333	W30X99	2.24	

Appendix N

Calibration of the Ramberg-Osgood Equation

The Ramberg-Osgood Equation was originally developed so that the compressive and tensile test curves of aluminum-alloy, stainless-steel, and carbon-steel could be described using only three variables – Young's Modulus of Elasticity, and two material specific constants (Ramberg & Osgood, 1943). Section 5.2.1.1 of this report describes how the Ramberg-Osgood Equation model was used in this study so that each of the thirteen pushover curves could be described by a specific equation. The three variables that were used in this study were the elastic stiffness, K_{roof}, of the 2D moment frame (instead of Young's Modulus of Elasticity) and two *building* specific constants, G and s (instead of the two *material* specific constants). This appendix describes how the elastic stiffness was calculated and how the two building specific constants were calibrated for each building. The Ramberg-Osgood Equation (Equation 5.2.1.1-1) has been reprinted below for reference.

$$\Delta_{roof} = \frac{V_{PO}}{K_{roof}} + G\left(\frac{V_{PO}}{K_{roof}}\right)^{s} \tag{5.2.1.1-1}$$

The elastic stiffness, K_{roof}, of each moment frame was the first parameter that was calculated for each of the thirteen pushover curves. The elastic stiffness value, or more specifically the slope of the linear portion of the nonlinear pushover curve, was determined by dividing the linear design base shear value that was used in the static analysis of each moment frame by the corresponding roof displacement from the nonlinear pushover analysis.

The building specific constants G and s were calculated for each pushover curve using a second curve called the stress-deviation curve (Ramberg & Osgood, 1943). The stress-deviation curve describes the plot of the base shear versus the difference between the roof displacement along a perfectly elastic line with a slope of K_{roof} and the actual roof displacement from the nonlinear pushover curve. Equation N-1 shows how the deviation, d, is calculated for each pushover curve.

$$d = \Delta_{roof} - \frac{V_{PO}}{K_{roof}} = G\left(\frac{V_{PO}}{K_{roof}}\right)^{s} \tag{N-1}$$

When a log-log plot of Equation N-1 is made, a straight line is formed. This new relationship is shown in Equation N-2. By taking the slope and the y-intercept of this straight line the building specific constants s and G, respectively, can be calculated.

$$\log(d) = \log(G) + s\log\left(\frac{V_{PO}}{K_{roof}}\right) \qquad \text{(N-2)}$$

In some cases the value of G was modified from what was initially calculated to allow for a better fitting curve and to allow for a coefficient of multiple determination, R^2, to be at or near 1.0. The specific values of G and s for each of the thirteen pushover curves used in this study are listed in Table 5.2.1.1.1.