Structural Engineering Report No. ST-06-1

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

Steven M. Gartner and Jerome F. Hajjar

May 2006

Department of Civil Engineering Institute of Technology University of Minnesota Minneapolis, Minnesota 55455

### 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.

#### ACKNOWLEDGEMENTS

The research presented in this report was funded by the National Science Foundation (Grant No. CMS-0084848) under Dr. Vijaya Gopu and by the University of Minnesota. The authors would like to thank Susan La Fore for her design work and assistance with the three-story building designs, and Cenk Tort for his collaborative work and assistance with this study. Any opinions, findings, and conclusions or recommendations expressed in this material are those of the authors and do not necessarily reflect the views of the National Science Foundation or of the University of Minnesota.

# TABLE OF CONTENTS

A	bstrac	t	•••••		i
A	cknow	vledge	emei	nts	ii
Li	st of l	Figure	es		vi
Li	st of 7	Fables	s		ix
1	Intro	oducti	ion.		1
	1.1	Back	gro	und	1
	1.2	Obje	ectiv	es and Scope of Work	2
	1.3	Orga	niza	ation of the Report	
2	Lite	rature	e Rev	view	5
	2.1	Perfo	orma	ance-Based Design	5
	2.2	Engi	neer	ring Demand Parameters, EDPs	9
	2.3	RCF	ТС	olumn Damage Functions	
	2.4	Push	love	r Analysis	
3	Line	ear Sta	atic	Analysis	
	3.1	Desi	gn P	Parameters	
	3.	1.1	Site	Location	
	3.	1.2	Site	Conditions	
	3.	1.3	Bui	lding Layouts	
	3.	1.4	Nor	ninal Loads	
		3.1.4	4.1	Design Building Code	
		3.1.4	4.2	Dead Loads	
		3.1.4	4.3	Live Loads	
		3.1.4	4.4	Wind Loads	
		3.1.4	4.5	Lateral Seismic Loads	
		3.1.4	4.6	Vertical Seismic Loads	

	3.2 Design Loads	. 22
	3.2.1 Load Combinations	. 22
	3.2.2 Stability Coefficient, θ	. 23
	3.2.3 Moment Amplification Factors, B <sub>1</sub> and B <sub>2</sub>	. 24
	3.3 Member Design Strengths	. 25
	3.3.1 Girders	. 25
	3.3.2 RCFT Columns	. 26
	3.3.2.1 Effective Length Factors, K <sub>x</sub> and K <sub>y</sub>	. 27
	3.3.2.2 Cross Sectional Properties	. 27
	3.3.2.2.1 Steel HSS	. 27
	3.3.2.2.2 Concrete Core	. 28
	3.3.2.3 Elastic Design Values of E, A, and I	. 29
	3.3.3 Interstory Drift Limits	. 30
4	Nonlinear Static Pushover Analysis	. 31
	4.1 Analysis Procedure	. 31
	4.2 Analysis Results	. 32
5	Design and Analysis Results	. 34
	5.1 Linear Static Analysis Results	. 34
	5.2 Nonlinear Static Pushover Analysis Results	.43
	5.2.1 Elastic Stiffness, Capacity, and Relative Energy	. 45
	5.2.1.1 Ramberg-Osgood Equation Approximation	. 48
	5.2.2 System Overstrength Factor, $\Omega$	. 50
6	Assessment of the Final Suite of Buildings	. 52
	6.1 Method 1: Elastic Seismic Design Spectrum Comparison	. 52
	6.2 Method 2: Pushover Curve Envelope	. 54
	6.3 Method 3: System Overstrength Factor Verification	. 65

7 Summary and Conclusions	\$4
7.1 Summary of Results	\$4
7.2 Conclusions	57
7.3 Future Work	8
References	\$9
Appendix A: 9-Story Building Nominal Loads9	2
Appendix B: Building Design 9A Calculations11	0
Appendix C: Building Design 9B Calculations 12	27
Appendix D: Building Design 9C Calculations 14	4
Appendix E: 18-Story Building Nominal Loads16	51
Appendix F: Building Design 18A Calculations17	'9
Appendix G: Building Design 18B Calculations 19	17
Appendix H: Building Design 18C Calculations	5
Appendix I: Design Example	3
Appendix J: Column Interaction Calculations Macro24	-0
Appendix K: Steel Tube Institute HSS Equations	-5
Appendix L: Methods for Calculating a Preliminary EI <sub>eff</sub>	0
Appendix M: Calculating the Baseline Flexural Rigidity Ratios25	52
Appendix N: Calibration of the Ramberg-Osgood Equation	5

## LIST OF FIGURES

Figure 2.1.1	Determining Possible Performance Levels From a Pushover Analysis (from ATC, 2003)
Figure 2.1.2	ATC (2003) Performance Level Continuum (from ATC, 2003)7
Figure 2.1.3	Example Annual Probability Curve for a Maximum Story Drift EDP (from Moehle and Deierlein, 2004)
Figure 3.1.1.1	Site Location for the Buildings Designed in This Study (from LaFore and Hajjar, 2005)
Figure 3.1.3.1	Typical 9-Story Building Plan View15
Figure 3.1.3.2	Typical 18-Story Building Plan View16
Figure 3.1.3.3	Typical 9-Story Building Elevation View (Moment Frame "A2-F2") 17
Figure 3.1.3.4	Typical 18-Story Building Elevation View (Moment Frame "A3-G3") 17
Figure 3.3.2.2.1	Typical HSS Cross Section
Figure 4.1.1	Example of How Lateral Loads Are Applied With $P_1 > P_2 > P_3$
Figure 4.2.1	Building Response Parameters That Are Derived From a Pushover Analysis
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
Figure 5.2.1	Pushover Analysis Curves For All Thirteen buildings of This Study 44
Figure 5.2.2	Normalized Pushover Analysis Curves For All Thirteen buildings of This Study
Figure 5.2.1.1	Normalized Drift, $\Delta/h_r$ , Used to End the Relative Energy Calculations 47

Figure 5.2.1.1.1	9-Story and 18-Story Building Pushover Curves, Capacity Curves, and the Ramberg-Osgood Equation Approximated Curves	49
Figure 5.2.2.1	Overstrength Factor, $\Omega$ , as a Function of the Number of Stories, n	51
Figure 5.2.2.2	Overstrength Factor, $\Omega$ , as a Function of the Building Design Period, $T_a$	51
Figure 5.2.2.3	Overstrength Factor, $\Omega$ , as a Function of the Roof Height, $h_r$	51
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, $T_a$	53
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	54
Figure 6.2.1	Idealized Building System Envelope	55
Figure 6.2.2	Approximate Building Seismic Weight per Story	56
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	60
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)	62
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)	62
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	63
Figure 6.2.7	Idealized Building System Normalized Envelope	63
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 Systems Were Omitted for Clarity)	64
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	65

Figure 6.3.1	The 9 Baseline $\eta$ Values in Relation to the Envelope of Possible Design Parameters $F_{yc}$ and $f'_c$	77
Figure 6.3.2	Three Surfaces of the Baseline Values of $\boldsymbol{\eta}$	79
Figure 6.3.3	All Three Surfaces of the Baseline Values of $\boldsymbol{\eta}$ and the Two Zones Between Them.	82

# LIST OF TABLES

Table 1.2.1	Design Parameters For Each Building	3
Table 2.1.1	ATC (2003) Recommended Discrete Levels of Performance (ATC, 2003)	.7
Table 3.3.2.1	RCFT Compressive Strength Design Steps Using the 2005 AISC Specification	26
Table 3.3.2.2	RCFT Flexural Strength Design Steps Using the 2005 AISC Specification	26
Table 3.3.2.3	RCFT Interaction Value Design Steps Using the 2005 AISC Specification	26
Table 5.1.1	Linear Static Analysis Design Values for Each Building	35
Table 5.1.2	Exterior Column Sections for the 3-Story Buildings	37
Table 5.1.3	Exterior Column Sections for the 9-Story and 18-Story Buildings	37
Table 5.1.4	Interior Column Sections for the 3-Story Buildings	38
Table 5.1.5	Interior Column Sections for the 9-Story and 18-Story Buildings	38
Table 5.1.6	Girder Sections for the 3-Story Buildings	39
Table 5.1.7	Girder Sections for the 9-Story and 18-Story Buildings	39
Table 5.1.8	Linear Static Analysis Results for Each Building	41
Table 5.1.9	Approximate and Rationally Calculated Fundamental Periods of Each 9-Story and 18-Story Building	43
Table 5.2.1.1	Nonlinear Static Pushover Analysis Results for Each Building	16
Table 5.2.1.2	Capacity and Relative Energy Absorbed for Each Building	17
Table 5.2.1.1.1	Ramberg-Osgood Equation Parameters and Constants	18
Table 5.2.2.1	Anchor Points Used in Figure 5.2.2.1 Through Figure 5.2.2.3	51
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	59
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)	51

Table 6.3.1	Values of $\eta_1$ per Story in Each Building
Table 6.3.2	Values of $\eta_2$ per Building
Table 6.3.3	Values of $\eta_3$ per Story in Each Building
Table 6.3.4	Values of $\eta_4$ per Story in Each Building
Table 6.3.5	Values of $\eta_5$ per Story in Each Building71
Table 6.3.6	Reduction Factor, RF, per Material Property72
Table 6.3.7	Values of $\eta_6$ per Story in Each Building
Table 6.3.8	Values of $\eta_7$ per Story in Each Building
Table 6.3.9	Values of $\eta_8$ per Story in Each Building
Table 6.3.10	Values of $\eta_9$ per Story in Each Building
Table 6.3.11	Mean Flexural Rigidity Ratio, η, for Each Building Using Equation 6.3-1 Through Equation 6.3-976
Table 6.3.12	Mean Flexural Rigidity Ratio, η, According to the Number of Stories in the Building
Table 6.3.13	Design Parameters for Each Baseline Value of $\eta$
Table 6.3.14	Baseline Values of η78
Table 6.3.15	Mean Values of d/t

# 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 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 46 ksi or 80 ksi and design concrete compressive strength values of 46 ksi. A concrete density of 145 lb/ft<sup>3</sup> 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		3-Story Buildings						9-Story Buildings			18-Story Buildings		
Parameter	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 <i>[feet]</i>	30	30	30	30	30	20	20	30	30	30	20	20	20
Column F <sub>yc</sub> [ksi]	46	80	80	46	80	46	80	46	80	50	46	80	50
Column f′ <sub>c</sub> [ksi]	4	16	16	4	16	4	16	4	16	16	4	16	16
Girder F <sub>yg</sub> <i>[ksi]</i>	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	≤ 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 secondorder 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,  $\eta$ .

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).

$$v(DV) = \iiint G \langle DV | DM \rangle | dG \langle DM | EDP \rangle | dG \langle EDP | IM \rangle | d\lambda (IM)$$
(2.1-1)

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.

$$D^{\text{RCFT}} = \frac{d}{d_{\text{o}}}$$
(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.

$$E^{\text{RCFT}} = \frac{E}{E_{\text{total}}}$$
(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- $\Delta$  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 ( $\Omega$ ) 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
  - $\circ$  (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

Design and Evaluation of Rectangular Concrete Filled Tube (RCFT) Frames for Seismic Demand Assessment



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<sup>3</sup> 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:

	0	Columns, beams, girders, miscellaneous structural system	20 lb/ft <sup>2</sup>
	0	Flooring	1 lb/ft <sup>2</sup>
	0	Composite floor system (concrete + metal decking)	50 lb/ft <sup>2</sup>
	0	Ceiling (from story below) + fireproofing	$2 \text{ lb/ft}^2$
	0	HVAC + electrical (from story below)	7 lb/ft <sup>2</sup>
		<ul> <li>Total floor dead loads:</li> </ul>	80 lb/ft²
	0	Exterior walls (applied to surface area of the wall)	25 lb/ft <sup>2</sup>
•	Roof c	lead loads include the following:	
	0	Roofing	7 lb/ft <sup>2</sup>
	0	Composite roof system (concrete + metal decking)	50 lb/ft <sup>2</sup>
	0	(Roof) beams, girders, miscellaneous structural system	20 lb/ft <sup>2</sup>
	0	Ceiling (from story below) + fireproofing	2 lb/ft <sup>2</sup>
	0	HVAC + electrical (from story below)	7 lb/ft <sup>2</sup>
		<ul> <li>Total roof dead loads:</li> </ul>	86 lb/ft²
	0	Parapet (applied to surface area of the wall)	25 lb/ft <sup>2</sup>

• Penthouse dead loads include the following:

0	Composite roof system (concrete + metal decking)	50 lb/ft <sup>2</sup>
0	Ceiling + fireproofing	2 lb/ft <sup>2</sup>
0	Columns, beams, girders, miscellaneous structural system	20 lb/ft <sup>2</sup>
0	Mechanical equipment	40 lb/ft <sup>2</sup>
0	Flooring	1 lb/ft <sup>2</sup>
	<ul> <li>Total penthouse dead loads:</li> </ul>	113 lb/ft <sup>2</sup>
0	Exterior walls (applied to surface area of the wall)	25 lb/ft <sup>2</sup>

## 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/ft <sup>2</sup>
• Moveable partitions	20 lb/ft <sup>2</sup>
<ul> <li>Total floor live load</li> </ul>	70 lb/ft <sup>2</sup>
Building (roof)	
<ul> <li>Roof live load</li> </ul>	20 lb/ft <sup>2</sup>
<ul> <li>Total roof live load</li> </ul>	20 lb/ft <sup>2</sup>
Penthouse	
• General live load	20 lb/ft <sup>2</sup>
• Roof live load	0 lb/ft <sup>2</sup>
<ul> <li>Total penthouse live load</li> </ul>	20 lb/ft <sup>2</sup>
	Building (floors) <ul> <li>Office building occupancy</li> <li>Moveable partitions <ul> <li>Total floor live load</li> </ul> </li> <li>Building (roof) <ul> <li>Roof live load</li> <li>Total roof live load</li> </ul> </li> <li>Penthouse <ul> <li>General live load</li> <li>Roof live load</li> <li>Total penthouse live load</li> </ul> </li> </ul>

## 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.

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

*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
  - $\circ$  S<sub>S</sub> = 1.5g
  - $\circ$  S<sub>1</sub> = 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
  - $\circ F_a = 1.0$
  - $\circ$  F<sub>v</sub> = 1.5
- Seismic-Force Resisting System = "Special Composite Moment Frames"
  - $\circ$  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<sub>t</sub> and x, respectively.

$$T_a = C_t h_r^x$$
 (3.1.4.5-1)

*Where*:  $h_r$  = building roof elevation

The design base shear is a percentage of the seismic weight, D', of the building. Since the 9story 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.

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

$$C_{vx} = \frac{W_{x}h_{x}^{k}}{\sum_{i=1}^{n} W_{i}h_{i}^{k}}$$
(3.1.4.5-3)

In accordance with IBC 2003, the redundancy coefficient,  $\rho$ , 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,  $\theta$ , was used to estimate the stability of each building. More specifically, the stability coefficient can approximate if geometric nonlinearities (P- $\Delta$  effects) should be anticipated or if they can be ignored. When  $\theta$  is between 0 and 0.10, P- $\Delta$  effects can be ignored. If  $\theta$  is between 0.10 and the maximum allowed, then it can be anticipated that P- $\Delta$  effects will start to affect the response of the building when the building is beyond its elastic limit. If  $\theta$  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  $\beta$ , which is the ratio of the story shear demand,  $\Sigma V_u$ , 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,  $\Sigma P_u$ , 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  $\theta$  is  $\Sigma P_u$ . Each load combination will have different values of factored gravity loads, making  $\Sigma P_u$  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  $\Sigma P_u$ .

$$\theta_{i} = \frac{\sum P_{u}}{\sum V_{i}H_{i}} = \frac{\sum P_{u}}{K_{i}H_{i}}$$
(3.2.2-1)

 $\begin{array}{ll} \textit{Where:} & \Sigma P_u = \textit{total factored axial load on all of the columns in a story} \\ V_i = \textit{total horizontal force to story i} \\ H_i = \textit{story (column) height} \\ \Delta_i = \textit{elastic interstory drift due to } V_i \end{array}$ 

 $K_i = story stiffness$ 

In addition to estimating the stability of a building,  $\theta$  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  $\theta$  and the column effective length factors,  $K_x$  and  $K_y$ , and the "B<sub>2</sub> 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<sub>1</sub> and B<sub>2</sub>

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.

$$B_{1} = \frac{C_{m}}{\left(1 - \frac{P_{u}}{P_{e1}}\right)} \ge 1.0$$
(3.2.3-1)

$$C_{\rm m} = 0.6 - 0.4 \left( \frac{M_1}{M_2} \right)$$
 (3.2.3-2)

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

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

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

When the B<sub>2</sub> factor was required, every column in a story of a building used the same value of B<sub>2</sub>. However, each load combination will result in a different B<sub>2</sub> factor for each story since  $\Sigma P_u$  will vary depending on the load factors and applied gravity loads. B<sub>2</sub> was calculated for each story by using the stability coefficient,  $\theta$ , as shown in Equation 3.2.3-4.
$$B_2 = \frac{1}{1 - \theta}$$
(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-columnweak-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.

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

Where:

 $\Sigma M_{\rm pc} = \Sigma Z_{\rm c} (F_{\rm vc} - P_{\rm uc} / (A_{\rm c} + A_{\rm s}))$  $\Sigma M_{pg} = \Sigma (1.1 R_v F_{vg} Z_g + M_v)$  $A_c$  = 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	Po	I.2.2b	l2-15
2	C <sub>2</sub> = 0.85	I.2.2b	RCFT Section
3	El <sub>eff</sub>	I.2.2b	l2-16
4	C <sub>3</sub>	I.2.2b	l2-17
5	Pe	l.2.1b	12-4
6	α	l.2.1b	12-2
7	Λ	l.2.1b	12-8 or 12-9
8	$P_n = \Lambda P_o$	l.2.1b	12-7
9	φ <sub>c</sub> = 0.75	1.4	LRFD
10	φ <sub>c</sub> P <sub>n</sub>	1.4	

 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	$\phi_b=0.90$	1.4	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	С	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 <sub>cx</sub> & M <sub>cy</sub>	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<sub>x</sub> and K<sub>y</sub>

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,  $\theta$ . This relationship is illustrated in Equation 3.3.2.1-1. Since the two varying parameters  $P_u$  and  $\theta$  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  $\theta$  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  $\theta_y$  instead of  $\theta_x$  in Equation 3.3.2.1-1.

$$K_{x} = \sqrt{\left(\frac{\pi^{2}(EI_{eff})}{H^{2}}\right)\left(\frac{\theta_{x}}{0.85P_{u}}\right)}$$
(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  $\leq$  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  $\frac{5}{8}$  inch, and 3.0 times 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

$$A_{c} = (d - 2r)(b - 2r) + 2(r - t)(b + d - 4r) + \pi(r - t)^{2}$$
(3.3.2.2.2-1)

$$I_{c} = \frac{1}{12} (b - 2t) (d - 2r)^{3} + 2 \left[ \frac{1}{12} (b - 2r) (r - t)^{3} + (b - 2r) (r - t) \left[ \frac{d - r - t}{2} \right]^{2} \right] + \dots$$

$$\dots + 2 \left\{ (r - t)^{4} \left( \frac{\pi}{8} - \frac{8}{9\pi} \right) + \left[ \frac{\pi (r - t)^{2}}{2} \right] \left[ \frac{d}{2} - t - (r - t) \left( 1 - \frac{4}{3\pi} \right) \right]^{2} \right\}$$
(3.3.2.2.2.2)

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

 $\begin{array}{ll} \textit{Where:} & A_c = \text{area of the concrete core} \\ b = \text{outside width} \\ d = \text{outside depth} \\ I_c = \text{moment of inertia of concrete core} \\ r = \text{outside corner radii} \\ t = \text{design wall thickness} \\ Z_c = \text{plastic modulus of concrete core} \end{array}$ 

#### 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.

$$A_{e} = \frac{EA_{eff}}{E'}$$
(3.3.2.3-1)

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

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

#### 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,  $\delta_x$ , could not exceed 0.02H<sub>i</sub>. The corresponding maximum *elastic* interstory drift allowed in the elastic analysis was determined by using Equation 3.3.3-1.

$$\Delta_{\rm e} = \frac{\delta_{\rm x} I}{C_{\rm d}} \tag{3.3.3-1}$$

 $\begin{array}{ll} \textit{Where:} & \Delta_e = elastic \mbox{ interstory drift limit} \\ \delta_x = \mbox{inelastic interstory drift limit} \\ I = \mbox{ occupancy importance factor} \\ C_d = \mbox{deflection amplification factor} \end{array}$ 

# **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,  $\Omega$ , 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,  $\Delta$ . 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<sub>e</sub>, the capacity of the building, the relative energy absorbed by the building, and the overstrength factor,  $\Omega$ .

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			3-St	ory Buildi	ngs			9-St	ory Build	ings	18-S	tory Build	lings
Design Values	3A	3B	3C	3D	3E	3F	3G	9A	9B	9C	18A	18B	18C
Design Fundamental Period for Seismic Loading T <sub>a</sub> [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 <sub>s</sub> [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 <sub>s</sub> <i>[kips]</i>	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 <sub>design</sub> [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 <sub>wind</sub> [kips]	16	16	16	16	16	16	16	79	79	79	134	134	134

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,  $\delta_x$ , was not to exceed 2% of the story height (0.02H<sub>i</sub>) 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										
Otory	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 Building	s	-	18-Story Building	S	
Story	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										
Otory	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 Building	s		18-Story Building	s
Story	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 20x20x1/2 HSS 16x16x5/8	
7	HSS 20x20x5/8	HSS 20x20x5/8 HSS 16x16x5/8		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									
11001	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 Buildin	igs	18	18-Story Buildings				
FIOOI	9A	9B	9C	18A	18B	18C			
1									
2	W30×90	W30x99	W27x84	W24x76	W24x84	W24x68			
3	W30×90	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

W24x76

W24x84

W27x84

W30x90









Figure 5.1.6: Design 18C Section Sizes



Figure 5.1.3: Design 9B Section Sizes



Figure 5.1.5: Design 9C Section Sizes

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

Linear Elastic		3-Story Buildings								ngs	18-Story Buildings		
Analysis Results	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 A <sub>design</sub> <i>[inches]</i>	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 <i>[inches]</i>	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 <sub>I</sub> [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 $\Delta_{CG}$ [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 <sub>e</sub> [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.

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

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

 $\begin{array}{ll} \textit{Where:} & V_{design} = building \ total \ design \ seismic \ base \ shear} \\ C_s = design \ seismic \ response \ coefficient \\ \Delta_{CG} = elastic \ drift \ at \ the \ building \ center \ of \ gravity \\ g = acceleration \ of \ gravity \\ K_e = building \ elastic \ stiffness = V_{design} \ / \ \Delta_{CG} \end{array}$ 

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	9-S1	tory Buildi	ings	18-Story Buildings				
Period	9A	9B	9C	18A	18B	18C		
Building Design Period T <sub>a</sub> [seconds]	1.264	1.264	1.264	2.201	2.201	2.201		
Building Approximated Period T <sub>1</sub> (and T <sub>2</sub> ) <i>[seconds]</i>	2.10	2.15	2.08	3.46	3.58	3.40		
Building Period by Rational Analysis Tr [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,  $\Delta_{roof}$ . Using these two parameters, a number of system characteristics were determined for each building including the elastic stiffness, K<sub>e</sub>, the capacity of the building, the relative energy absorbed by the building, and the overstrength factor,  $\Omega$ .

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).





## 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  $\Delta/h_r$  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  $\Delta/h_r$  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  $\Delta/h_r$  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	3-Story Buildings							9-Story Buildings			18-Story Buildings		
Results	3A	3B	3C	3D	3E	3F	3G	9A	9B	9C	18A	18B	18C
2D Moment Frame Seismic (Design) Base Shear V <sub>design</sub> [kips]	225	225	225	282	282	246	246	340	340	340	316	316	316
2D Moment Frame "Elastic Limit" Base Shear V <sub>e</sub> [kips]	349	437	751	444	419	387	497	546	623	543	622	757	433
Maximum 2D Moment Frame Base Shear V <sub>max</sub> [kips]	856	872	859	1,070	1,025	888	1,154	897	986	878	700	799	687
Roof Drift at Design Base Shear <u>A<sub>design</sub></u> [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 <sub>e</sub> [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 <sub>max</sub> <i>[inches]</i>	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 <sub>CG</sub> [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 <sub>e</sub> [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 <sub>1</sub> [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 <sub>2</sub> [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,  $\Delta/h_r$ , 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		3-Story Buildings								9-Story Buildings			18-Story Buildings		
Results	3A	3B	3C	3D	3E	3F	3G	9A	9B	9C	18A	18B	18C		
$\begin{array}{c} \text{Normalized} \\ \text{Roof} \\ \text{Displacement} \\ \text{at Energy} \\ \text{Calculation} \\ \text{Termination} \\ \Delta / \ h_r \end{array}$	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 <i>[inches]</i>	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 <i>[kip-ft]</i>	441	474	497	561	552	499	553	935	971	894	948	924	846		
Capacity <i>[kips]</i>	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  $\Delta/h_r$  equaled 0.02/k on the normalized pushover curve.

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

Where:

 $\Delta_{\rm roof}$  = 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	3-Story Buildings								9-Story Buildings			18-Story Buildings		
Results	3A	3B	3C	3D	3E	3F	3G	9A	9B	9C	18A	18B	18C	
Elastic Stiffness at the Roof K <sub>roof</sub> [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 <sub>PO</sub> [kips]	825	868	857	1,022	987	868	1,105	877	957	827	700	799	687	
Ending ∆ <sub>roof</sub> [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,  $\Omega$ , represents the ratio between the maximum base shear from the pushover analysis, V<sub>PO</sub>, to the design seismic base shear, V<sub>design</sub>. 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  $\Omega$  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).

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

Three system characteristics were found to help predict  $\Omega$ . These three system characteristics are the number of stories of a building, n, the roof elevation, h<sub>r</sub>, and the fundamental design period, T<sub>a</sub>, 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.

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

$$\Omega = \left(\frac{T_{a}}{1.55}\right)^{2} - 2T_{a} + 4.7$$
(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 Ta	Roof Height h <sub>r</sub>	
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	





# **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.





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<sub>max</sub> and T<sub>min</sub>
- Maximum and minimum elastic stiffness, k<sub>max</sub> and k<sub>min</sub>
- Maximum and minimum overstrength factors,  $\Omega_{max}$  and  $\Omega_{min}$
- 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.

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

Where:

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

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

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

Weight, 
$$W = V_{base} / C_s$$
 (6.2-5)

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

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  $k_{min}$ .

$$k_{\max/\min} = m \left(\frac{2\pi}{T_{\min/\max}}\right)^2$$
(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,  $\Omega$ . 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,  $\Omega_{max}$ , 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,  $\Omega_{min}$ , 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,  $\Omega$ , 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 <sub>max</sub> [seconds]	1.18	2.95	4.87
Minimum Fundamental Period T <sub>min</sub> [seconds]	0.74	1.77	3.08
Maximum Elastic Stiffness K <sub>max</sub> [kips/in]	349	185	77
Minimum Elastic Stiffness K <sub>min</sub> [kips/in]	136	67	31
Maximum Overstrength Factor Ω <sub>max</sub>	3.77	2.84	2.31
Minimum Overstrength Factor Ω <sub>min</sub>	1.00	1.00	1.00
Maximum Base Shear V <sub>max</sub> [kips]	869	965	732
Minimum Base Shear V <sub>min</sub> [kips]	231	340	316
Upper Bound Roof Drift Transition Point $\Delta_{max}$ [inches]	2.49	5.22	9.46
Lower Bound Roof Drift Transition Point $\Delta_{min}$ [inches]	1.70	5.11	10.21

- **Table 6.2.1:** 3-Story, 9-Story, and 18-Story Upper and Lower BoundEnvelope 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.



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,  $\Omega$ , 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 <sub>max</sub> [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 <sub>min</sub> [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 <sub>max</sub> [kips/in]	670	444	295	259	233	213	198	167	151	137	124	113	103	94	85
Minimum Elastic Stiffness K <sub>min</sub> [kips/in]	134	135	124	104	91	80	73	59	52	47	42	39	37	35	33
Maximum Overstrength Factor Ω <sub>max</sub>	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 <sub>min</sub>	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 <sub>max</sub> [kips]	327	613	1,003	996	998	983	978	910	860	819	773	757	755	750	742
Minimum Base Shear V <sub>min</sub> [kips]	76	153	282	295	310	319	331	332	325	319	309	309	314	317	318
Upper Bound Roof Drift Transition Point A <sub>max</sub> [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 Amin [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<br/>(the 3-Story, 9-Story, and 18-Story Data Points are in Table 6.2.1)







**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.

$$\mathbf{k}_{\text{max/min}}' = \left(\mathbf{k}_{\text{max/min}}\right) \left(\frac{\mathbf{h}_{\text{r}}}{\mathbf{V}_{\text{design}}}\right)$$
(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  $\Delta_{roof}$ . 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,  $\eta$ , 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  $\eta$  and that of its baseline value of  $\eta$ . If the actual value of  $\eta$  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  $\eta$  is smaller than its baseline value the building system is considered under designed, and if  $\eta$  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  $\eta$  for the moment-resisting frame that is being checked.  $\eta$  can be calculated in numerous ways, nine of which are presented in this study. Equation 6.3-1 shows how  $\eta$  is first calculated for every story, i, of a building. A single value of  $\eta$  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  $\eta$  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  $\eta$  and are the basic ways to calculate  $\eta$ . The specific story-based values of  $\eta_1$  and the total building-based value of  $\eta_1$  as well as the building-based value of  $\eta_2$  are listed in Table 6.3.1 and Table 6.3.2, respectively, for each of the thirteen buildings in this study.

$$\eta_{1} = \left[\frac{\Sigma EI_{eff}}{(\Sigma EI)_{a} + (\Sigma EI)_{b}}\right]_{i}$$
(6.3-1)

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

Where:	i = story number
	$EI_{eff}$ = flexural rigidity of a composite column
	$(\Sigma EI)_a$ = total flexural rigidity of the girders <i>above</i> story i
	$(\Sigma EI)_b$ = total flexural rigidity of the girders <i>below</i> 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  $\eta$  could be developed. Overall the value of  $\eta$  did not vary significantly between these nine different methods. By providing numerous methods for calculating  $\eta$  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  $\eta$ , 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:

В	number of bays in the story
d <sub>c</sub>	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
Ec	modulus of elasticity of concrete = $w_c^{1.5} \sqrt{f'_c}$
E'c	modified modulus of elasticity of concrete
$\mathrm{EI}_{\mathrm{eff}}$	effective flexural rigidity of a composite column
f' <sub>c</sub>	minimum concrete compressive strength
$F_{yg}$	minimum girder yield strength
$F_{yc}$	minimum column yield strength
Η	story height; column length
Ig	girder moment of inertia
$I_{c\_s}$	column moment of inertia of the steel HSS portion
Κ	story stiffness
$\Sigma K_{col}$	$\Sigma(\text{EI}_{\text{eff}} / \text{H})$
L	girder length
$M_{pc}$	column plastic moment
$M_{pg}$	girder plastic moment
$\mathbf{R}_{\mathbf{y}}$	expected yield strength factor
t	nominal thickness of the HSS column wall
Wc	density of concrete = $145 \text{ lb/ft}^3$

Flexural Rigidity Ratio η <sub>1</sub>													
Story						Building	Design	Numbe	r				
Siory	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  $\eta_1$  per Story in Each Building

	Flexural Rigidity Ratio η₂												
0	Building Design Number												
Story	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  $\eta_2$  per Building

$$\eta_{3} = \left(\frac{B+1}{B}\left[\left(\frac{d_{c}}{d_{g}}\right)\left(\frac{R_{y}F_{yg}}{F_{yc}}\right)\left(\frac{0.6EI_{eff}}{E_{s}I_{c_{s}}}\right)\right]_{i}$$
(6.3-3)

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<sub>eff</sub> = average effective flexural rigidity for story i I<sub>c\_s</sub> = average column moment of inertia for story i

	Flexural Rigidity Ratio η <sub>3</sub>													
Story	Building Design Number           3A         3B         3C         3D         3E         3F         3G         9A         9B         9C         18A         18B         18C													
Story	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  $\eta_3$  per Story in Each Building

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

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

Flexural Rigidity Ratio η₄														
Story					I	Building	Design	Numbe	r					
Story	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  $\eta_4$  per Story in Each Building

$$\eta_{5} = \left(\frac{B+1}{B} \int \left[ \left(\frac{E_{c}}{E_{s}}\right) \left(\frac{d_{c}^{4}}{23I_{g}}\right) + \left(\frac{I_{c_{s}}}{2I_{g}}\right) \left(1 - \frac{E_{c}}{E_{s}}\right) \right]_{i}$$
(6.3-5)

Where:

 $E'_{c} = min[(0.6 + 8t/d_{c}) \text{ or } 0.9]E_{c}$ 

 $d_c$  = 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 η₅														
Story	Building Design Number														
Story	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  $\eta_5$  per Story in Each Building

$$\eta_{6} = \left(\frac{B+1}{B}\right) \left\{ \left(\frac{M_{pc}}{M_{pg}}\right) \left(\frac{R_{y}F_{yg}}{F_{yc}}\right) \left(\frac{1}{2E_{s}d_{g}}\right) \left(\frac{E_{c}M_{pc}}{5(1.4t)^{2}F_{yc}} + d_{c}(E_{s} - E_{c})\right) \right\}_{i} (RF)$$
(6.3-6)

Where:

 $\begin{array}{l} d_c = average \ column \ depth \ for \ story \ I \\ d_g = average \ girder \ depth \ for \ story \ i = average \ of \ \Sigma d_{g_a} + \Sigma d_{g_b} \\ E'_c = min[ \ ( \ 0.6 + 8t/d_c \ ) \ or \ 0.9 \ ]E_c \\ RF = reduction \ factor \ per \ Table \ 6.3.6 \end{array}$ 

F <sub>yc</sub>	ť۰	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 <sub>6</sub>														
Story	Building Design Number														
Story	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  $\eta_6$  per Story in Each Building

$$\eta_{7} = (B+1) \left[ d_{c}^{3} \left( E_{c}^{'} \left( \frac{d_{c}}{12} \right) + 0.6t \left( E_{s} - E_{c}^{'} \right) \right) \left( \frac{6}{KH^{2}L_{ave}} \right) \right]_{i} - \frac{1}{2} \left( \frac{H}{L_{ave}} \right)_{i}$$
(6.3-7)

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 <sub>7</sub>													
Story	Building Design Number													
Story	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  $\eta_7$  per Story in Each Building

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

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

Flexural Rigidity Ratio η₀													
Story						Building	Design	Numbe	r				
Story	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  $\eta_8$  per Story in Each Building

$$\eta_{9} = \left(\frac{B+1}{B}\right) \left\{ \left(\frac{0.6}{E_{s}}\right) \left(\frac{R_{y}F_{yg}}{F_{yc}}\right) \left(\frac{d_{c}}{d_{g}}\right) \left[\frac{E_{c}}{7} \left(\frac{d_{c}}{t}\right) + \left(E_{s} - E_{c}^{'}\right)\right] \right\}_{i}$$
(6.3-9)

Where:  $E'_{c} = min[(0.6 + 8t/d_{c}) \text{ or } 0.9]E_{c}$  $d_{c} = average column depth for story i$  $d_{g} = average girder depth for story i$ 

					Flexur	al Rigio	dity Rat	tio η <sub>9</sub>					
Story						Building	Design	Numbe	r				
Slory	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  $\eta_9$  per Story in Each Building

The mean value of  $\eta$  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  $\eta$  and the design values of  $F_{yc}$  and  $f'_c$ . The 3-story buildings have larger values of  $\eta$  compared to the 9-story and the 18-story buildings. Table 6.3.12 provides an overall value of  $\eta$  based on the number of stories in a building, and based on the material design strengths of the RCFT columns. The mean value of  $\eta$  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 St		Values of η												
u/t	F <sub>yc</sub>	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		
LOW	80	16		0.71			1.04		1.05		0.54			0.55	
	50	16										1.96			2.11
піСП	80	16			1.05										

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

Material S	stress (ksi)	Values of η							
F <sub>yc</sub>	f′c	3-Story	3-Story 9-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,  $\eta$ , According to<br/>the Number of Stories in the Building

The reason for the buildings made of high strength material having lower values of  $\eta$  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<sub>a</sub> and EI<sub>b</sub>, 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  $\eta$  for the lower strength buildings and lower values of  $\eta$  for the higher strength buildings.

After analyzing all thirteen buildings in this study and calculating a value of  $\eta$  for each building, it was determined that  $\eta$  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.  $\eta$  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  $\eta$  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  $\eta$  so that the actual value of  $\eta$  could be compared against a corresponding baseline (i.e., target) value of  $\eta$ . A second set of moment-resisting frames was used to calculate the baseline values of  $\eta$ . 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.

85

Base Line Number	F <sub>yc</sub> [ksi]	f′ <sub>c</sub> [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		







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  $\eta$  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  $\eta$  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  $\eta$  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  $\eta$  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 ≤	: 2.26 √(E	/F <sub>yc</sub> )	d/t ≈ 80			
F <sub>yc</sub> [ksi]	ť₀ [ksi]	AVE η	F <sub>yc</sub> [ksi]	f′₀ [ksi]	AVE η	F <sub>yc</sub> [ksi]	f′₀ [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  $\eta$ 

Even though the same general pattern of decreasing values of  $\eta$  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  $\eta$  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,  $\eta$  will only increase in small increments as  $f'_c$  increases since the rigidity of the girders does not change.



Figure 6.3.2 illustrates how the three baseline d/t categories of low d/t, maximum allowed d/t, and d/t  $\approx$  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  $\eta$  of buildings with d/t ratios that are at or just under the AISC d/t limit. Surface #3 (d/t  $\approx$  80) is made of  $\eta$  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:

$$\eta = a + b(f_{c}') + c(f_{c}'^{2}) + d(F_{yc}) + e(F_{yc}f_{c}') + f(F_{yc}f_{c}'^{2}) + \dots$$
  
...+ g(F\_{yc}^{2}) + h(F\_{yc}^{2}f\_{c}')\_{c} + i(F\_{yc}^{2}f\_{c}'^{2}) (6.3-10)

 $d/t \le 2.26 \ \sqrt{(E/F_y)}$ :

$$\eta = a + b(f_c') + c(F_{yc}) + d(F_{yc}f_c') + e(F_{yc}^2) + f(F_{yc}^2f_c')$$
(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

 $d/t \approx 80$ :

$$\eta = a + b(f'_{c}) + c(f'_{c}) + d(F_{yc}) + e(F_{yc}f'_{c}) + f(F_{yc}f'_{c}) + ...$$
  
...+ g(F'\_{yc}) + h(F'\_{yc}f'\_{c}) + i(F'\_{yc}f'\_{c}) (6.3-12)

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  $\eta$  do not vary significantly when  $F_{yc}$  is held constant. Therefore, by taking average values of  $\eta$  from Table 6.3.14 for each value of the design yield strength  $F_{yc}$ , an estimated value of  $\eta$  can be determined for a RCFT moment-resisting frame depending on how large of a d/t ratio is required.

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

$$a = -3.19610427886004E-05$$

$$b = 1.01948679817606E-01$$

$$d/t \le 2.26 \ \sqrt{(E/F_y)}: a = -2.9533313486192E-05 c = 4.19852340833987E+03 b = 9.88113400161356E-02 b = 9.88113400161356E-02 b = 3.13847915123324E-01 c = 4.21373956584290E+03 b = 3.13847915123324E-01 c = 4.21373956584290E+03$$

c = 1.00997659850529E+03

Low d/t ratios:

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  $\eta$  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  $\eta$  has been set to. If the building is being designed to have an overall  $\eta$  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.

LOV	V d/t	d/t ≤ 2.2	6 √(E/F <sub>yc</sub> )	d/t ≈ 80		
F <sub>yc</sub> [ksi]	AVE. d/t	F <sub>yc</sub> [ksi]	AVE. d/t	F <sub>yc</sub> [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

$\frac{d}{d} = \frac{F}{F}$	$\frac{y_c}{y_c} + \int$	$\overline{F_{yc}}$	(6.3-14)
t a	a (	b	

Low d/t ratios:	a = 1.23159224083901E+01 c = -5.45687025577683E-01	b = 9.43499630252338E+03
$d/t \le 2.26 \ \sqrt{(E/F_y)}:$	a = -2.33848326894338E+01 c = -3.92910165308971E-01	b = 1.17819836272101E+06
$d/t \approx 80$ :	a = 3.5240366710384E+00 c = -5.0629760078164E-01	b = 1.49301458159099E+05

Another way to measure the value of  $\eta$  for a particular building is to plot its building value of  $\eta$  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  $\eta$  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  $\eta$  that would fall into Zone 2 since it would result in d/t values larger than the AISC limit.



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  $\eta$  that are in Zone 2. All of the remaining 10 building designs have values of  $\eta$  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,  $\eta$ , is not smaller nor significantly larger than its corresponding baseline value. By comparing the actual values of  $\eta$  from Table 6.3.12 to their baseline values of  $\eta$  in Table 6.3.14, the low strength buildings have actual  $\eta$  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  $\eta$  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  $\eta$  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  $\eta$  near the upper baseline limits for both the low strength and high strength buildings. The 9-story and 18-story buildings have  $\eta$  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  $\eta$  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,  $\Omega$ , 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, 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,  $\eta$ , 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  $\eta$  is determined by using a database of values that was developed for this study. The actual  $\eta$  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  $\eta$  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  $\eta$  is much greater than its target value, the building is considered to be further optimized.

Nine equations were developed to calculate  $\eta$  for a particular building. 135 additional moment-resisting frames were then approximated so that representative baseline values (i.e., target values) of  $\eta$  could be calculated. By comparing the baseline values with the actual building values, all thirteen buildings of this study were shown to have  $\eta$  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*, 2<sup>nd</sup> 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*, 3<sup>rd</sup> 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, 13<sup>th</sup> World Conference on Earthquake Engineering, Vancouver, B.C..

Muvdi, B.B. and McNabb, J.W. (1991). *Engineering Mechanics of Materials*, 3<sup>rd</sup> 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.

CUSTOMER       RCFT PARAMETRIC STUDY       CKD       DATE         SUBJECT       2-D MOMENT FRAME [MF A2 - F2] ANALYSIS LOAD SUMMARY         11       BUILDING GEOMETRY         • NUMBER OF STORIES, Ns       9 STORIES         • NUMBER OF BAYS:       • ALONG THE NS FACE OF THE BUILDING, Ns, s       5 BAYS         • BUILDING LENGTH (CTR-TO-CTR OF COLUMNS):       • ALONG THE EW FACE OF THE BUILDING, Le, w       150.011         • BUILDING LENGTH (CTR-TO-CTR OF COLUMNS):       • ALONG THE EW FACE OF THE BUILDING, Le, w       150.011         • CTR-TO-CTR DISTANCE (SPACNG) OF THE FOLLOWING ITEM(S):       • 10.011       10.011         • PARAPET HEIGHT       9.511       • BEAMS       10.011         • PARAPET HEIGHT       • BEAMS       10.011       BAY LEW         • DISTANCE OF THE SOUTHWEST CORNER (OF THE PENTHOUSE):       · yp       30.011       BAY LEW         • DIRECTION THAT THE COMPOSITE FLOOR SYSTEM SPANS       E-W       THEREFORE, THE BEAMS SPANS       E-W       THEREFORE, THE BEAMS SPANS         • DIRECTION THAT THE COMPOSITE FLOOR SYSTEM SPANS       E-W       THEREFORE, THE BUILDING ALONG THE E-W FACE OF THE BUILDING ALONG THE NS FACE       GINDERS ARE PAAALLEL         • DIRECTION THAT THE COMPOSITE FLOOR SYSTEM SPANS       E-W       THEREFORE, THE BEAMS SPANS       E-W       THEREFORE, THE BUILDING ALONG THE E-W FACE OF THE BUILDING ALONG TH	
SUBJECT       2-D MOMENT FRAME [MF A2 - F2] ANALYSIS LOAD SUMMARY         (1) BUILDING GEOMETRY            • NUMBER OF STORIES, N <sub>S</sub> • ALONG THE N'S FACE OF THE BUILDING, N <sub>B,NS</sub> • SBAYS         • ALONG THE EW FACE OF THE BUILDING, N <sub>B,NS</sub> • SBAYS         • ALONG THE EW FACE OF THE BUILDING, N <sub>B,NS</sub> • SBAYS         • ALONG THE EW FACE OF THE BUILDING, L <sub>NS</sub> • SDAYS         • ALONG THE EW FACE OF THE BUILDING, L <sub>NS</sub> • DUILDING LENGTH (CTR-TO-CTR OF COLUMNS):         • ALONG THE EW FACE OF THE BUILDING, L <sub>NS</sub> • DUILDING LENGTH (CTR-TO-CTR OF COLUMNS):         • ALONG THE EW FACE OF THE BUILDING, L <sub>P,NS</sub> • DOI th         • DEFAMS         • DEFAMS         • DEFAMS         • DISTANCE (SPACING) OF THE FOLLOWING ITEM(S):         • DEFAMS         • DISTANCE OF THE SOUTHWEST CORNER (OF THE PENTHOUSE): - X <sub>P</sub> • SOO th         • LENGTH ALONG THE EW FACE OF THE BUILDING, L <sub>P,NS</sub> • SOO th         • JENGTH ALONG THE EW FACE OF THE BUILDING, L <sub>P,NS</sub> • SOO th         • LENGTH ALONG THE EW FACE OF THE BUILDING, L <sub>P,NS</sub> • SOO th         • JENGTH ALONG THE EW FACE OF THE BUILDING, L <sub>P,NS</sub> • SOO th         • JENGTH ALONG THE EW FACE OF THE BUILDING, L <sub>P,NS</sub> • SOO th         • JENGTH ALONG THE EW FACE OF THE BUILDING, L <sub>P,NS</sub> • SOO th         • JENGTH         • DISTANCE OF THE SOUTHWEST CORNER (OF THE PENTHOUSE): - X <sub>P</sub> • SOO th         • Y <sub>P</sub> • SOO th         • JENGTH         • DIRECTION THAT THE COMPOSITE FLOOR SYSTEM SPANS         • MEREFORE, THE BEAMS SPAN         • DIRECTION THAT THE COMPOSITE FLOOR SYSTEM SPANS         • MEREFORE, THE BUILDING         • ALONG EACH FACE OF THE BUILDING         • STORY HEIGHT, h <sub>1</sub> • JO IN THE NS FACE         • O INTERVE DIRECTION         • INTERVE DIRECTION         • O INTERVEDINECTION         • O INTERVENCE OF THE BUIL	OF
$ \begin{array}{ c c c c } \hline \textbf{SULDING GEOMETRY} \\ \hline \textbf{0} & \textbf{NUMBER OF STORIES, N_{S}} & \textbf{9 STORIES} \\ \hline \textbf{0} & \textbf{NUMBER OF BAYS:} & \textbf{1} ALONG THE N.S FACE OF THE BUILDING, N_{B,R,W}} & \textbf{5} BAYS \\ \hline \textbf{1} ALONG THE E-W FACE OF THE BUILDING, N_{B,E,W}} & \textbf{5} BAYS \\ \hline \textbf{1} BUILDING LENGTH (CTR-TO-CTR OF COLUMNS): \\ \hline \textbf{1} ALONG THE N.S FACE OF THE BUILDING, L_{H,S}} & \textbf{150.0 ft} \\ \hline \textbf{1} ALONG THE E-W FACE OF THE BUILDING, L_{E,W}} & \textbf{150.0 ft} \\ \hline \textbf{1} CTR-TO-CTR DISTANCE (SPACING) OF THE FOLLOWING ITEM(S): \\ \hline \textbf{1} BEAMS & \textbf{10.0 ft} \\ \hline \textbf{1} BEAMS & \textbf{10.0 ft} \\ \hline \textbf{1} DISTANCE OF THE SOUTHWEST CORNER (OF THE FULLOWING ITEM(S): \\ \hline \textbf{1} LENGTH ALONG THE E-W FACE OF THE BUILDING, L_{P,NS} & \textbf{90.0 ft} \\ \hline \textbf{1} LENGTH ALONG THE E-W FACE OF THE BUILDING, L_{P,NS} & \textbf{90.0 ft} \\ \hline \textbf{1} DISTANCE OF THE SOUTHWEST CORNER (OF THE PENTHOUSE): & x_{P} & \textbf{30.0 ft} \\ \hline \textbf{1} y_{P} & \textbf{30.0 ft} \\ \hline \textbf{1} y_{P} & \textbf{30.0 ft} \\ \hline \textbf{1} DIRECTION THAT THE COMPOSITE FLOOR SYSTEM SPANS & E-W THEREFORE, THE BEAMS SPAN \\ \hline \textbf{1} ON MERT (CTR-TO-CTR OF GIRDERS) \\ \hline \textbf{1} ON THE E-W DIRECTION THAT THE SOURCE OF THE SUBJEMENT OT THE NS FACE OF THE BUILDING, LENGTH ALONG THE E-W DIRECTION THAT THE COMPOSITE FLOOR SYSTEM SPANS & E-W THEREFORE, THE BEAMS SPAN \\ \hline \textbf{1} ON MERT (CTR-TO-CTR OF GIRDERS) \\ \hline \textbf{1} ON MERT (CTR-$	
• NUMBER OF STORIES, Ns       9 STORIES         • NUMBER OF BAYS:       • ALONG THE N-S FACE OF THE BUILDING, NB_NS       5 BAYS         • ALONG THE E-W FACE OF THE BUILDING, NB_E-W       5 BAYS         • BUILDING LENGTH (CTR-TO-CTR OF COLUMNS):       • ALONG THE N-S FACE OF THE BUILDING, LB_NS       150.0 ft         • ALONG THE E-W FACE OF THE BUILDING, LB_NS       150.0 ft       • ALONG THE E-W FACE OF THE BUILDING, LB_NS       150.0 ft         • CTR-TO-CTR DISTANCE (SPACING) OF THE FOLLOWING ITEM(S):       • BEAMS       10.0 ft       • UBARAPET HEIGHT       13.0 ft         • PARAPET HEIGHT       3.5 ft       • BEAMS       10.0 ft       • UBARAPET HEIGHT       13.0 ft         • PENTHOUSE:       • HEIGHT       13.0 ft       90.0 ft       • BAY 1_E-W,       90.0 ft         • DISTANCE OF THE SOUTHWEST CORNER (OF THE PENTHOUSE):       • xp       30.0 ft       • PLAN VIE         • DIRECTION THAT THE COMPOSITE FLOOR SYSTEM SPANS       E-W       THEREFORE, THE BEAMS SPAN         • MUMBER       § 13.0 ft       13.0 ft       (GROBERS ARE PRATILE)       ALONG EACH FACE OF THE BUILDING, LP_LSN URGCTION, GROBERS ARE PRATILE         • MUMBER       § 13.0 ft       13.0 ft       13.0 ft       THE E-W GROECOF       GROBERS ARE PRATILE	
• NUMBER OF BAYS: • ALONG THE N-S <i>FACE</i> OF THE BUILDING, N <sub>B,NS</sub> 5 BAYS • ALONG THE E-W <i>FACE</i> OF THE BUILDING, N <sub>B,EW</sub> 5 BAYS • BUILDING LENGTH (CTR-TO-CTR OF COLUMNS): • ALONG THE N-S <i>FACE</i> OF THE BUILDING, L <sub>NS</sub> 150.0 ft • ALONG THE E-W <i>FACE</i> OF THE BUILDING, L <sub>EW</sub> 150.0 ft • ALONG THE E-W <i>FACE</i> OF THE BUILDING, L <sub>EW</sub> 150.0 ft • CTR-TO-CTR DISTANCE (SPACING) OF THE FOLLOWING ITEM(S): • BEAMS 10.0 ft • DEAMS 10.0 ft • LENGTH ALONG THE N-S <i>FACE</i> OF THE BUILDING, L <sub>P,NS</sub> 90.0 ft • LENGTH ALONG THE N-S <i>FACE</i> OF THE BUILDING, L <sub>P,NS</sub> 90.0 ft • LENGTH ALONG THE E-W <i>FACE</i> OF THE BUILDING, L <sub>P,NS</sub> 90.0 ft • DISTANCE OF THE SOUTHWEST CORNER (OF THE PENTHOUSE): • x <sub>P</sub> 30.0 ft • JY <sub>P</sub> 30.0 ft • DIRECTION THAT THE COMPOSITE FLOOR SYSTEM SPANS E-W THEREFORE, THE BEAMS SPAN • DIRECTION THAT THE COMPOSITE FLOOR SYSTEM SPANS E-W THEREFORE, THE BEAMS SPAN • O STORY STORY HEIGHT, h <sub>S</sub> (GROERS ARE PARALLEL • TO THE E-W DIRECTION) (CTR-TO-CTR OF GIRDERS) • 13.0 ft • 13.0	
• BUILDING LENGTH (CTR-TO-CTR OF COLUMNS): • ALONG THE NS FACE OF THE BUILDING, $L_{h,S}$ 150.0 ft • ALONG THE E-W FACE OF THE BUILDING, $L_{e,W}$ 150.0 ft • BLAMS 10.0 ft • BEAMS 10.0 ft • BEAMS 10.0 ft • BEAMS 10.0 ft • DARAPET HEIGHT 3.5 ft • LENGTH ALONG THE NS FACE OF THE BUILDING, $L_{P,MS}$ 90.0 ft • LENGTH ALONG THE NS FACE OF THE BUILDING, $L_{P,MS}$ 90.0 ft • LENGTH ALONG THE NS FACE OF THE BUILDING, $L_{P,MS}$ 90.0 ft • LENGTH ALONG THE E-W FACE OF THE BUILDING, $L_{P,MS}$ 90.0 ft • DISTANCE OF THE SOUTHWEST CORNER (OF THE PENTHOUSE): - $x_{P}$ 30.0 ft • JENGTH ALONG THE E-W FACE OF THE BUILDING, $L_{P,EW}$ 90.0 ft • DISTANCE OF THE SOUTHWEST CORNER (OF THE PENTHOUSE): - $x_{P}$ 30.0 ft • $y_{P}$ 30.0 ft • DIRECTION THAT THE COMPOSITE FLOOR SYSTEM SPANS E-W THEREFORE, THE BEAMS SPAN • DIRECTION THAT THE COMPOSITE FLOOR SYSTEM SPANS E-W THEREFORE, THE BEAMS SPAN • DIRECTION THAT THE COMPOSITE FLOOR SYSTEM SPANS E-W THEREFORE, THE BEAMS SPAN • DIRECTION THAT THE COMPOSITE FLOOR SYSTEM SPANS E-W THEREFORE, THE BEAMS SPAN • DIRECTION THAT THE COMPOSITE FLOOR SYSTEM SPANS E-W THEREFORE, THE BEAMS SPAN • DIRECTION THE N-S FACE OF THE BUILDING ALONG THE N-S FACE OF THE BUILDING • ON THE N-S FAC	
<ul> <li>CTR-TO-CTR DISTANCE (SPACING) OF THE FOLLOWING ITEM(S):         <ul> <li>BEAMS</li> <li>BEAMS</li> <li>PARAPET HEIGHT</li> <li>S5 ft</li> <li>PENTHOUSE:</li> <li>HEIGHT</li> <li>LENGTH ALONG THE N-S FACE OF THE BUILDING, LP, NS</li> <li>BAY 1_E-W</li> </ul> </li> <li>DISTANCE OF THE SOUTHWEST CORNER (OF THE PENTHOUSE):</li> <li>YP</li> <li>DIRECTION THAT THE COMPOSITE FLOOR SYSTEM SPANS</li> <li>THEREFORE, THE BEAMS SPAN</li> <li>STORY STORY HEIGHT, hs</li> <li>MUMBER (CTR-TO-CTR OF GRIDERS)</li> <li>MISTANCE OF THE STORY HEIGHT, hs</li> <li>MISTORY STORY HEIGHT, hs</li> <li>MISTORY ALSO THE TO COMPOSITE FLOOR SYSTEM SPANS</li> </ul> <li>BAY WIDTH, wb</li> <li>ALONG THE N-S FACE OF THE BUILDING ALONG THE N-S FACE OF THE BUILDING (GINDERS ARE PARALLEL TO THE N-S DIRECTION)</li>	
• PARAPET HEIGHT • PENTHOUSE: - HEIGHT - LENGTH ALONG THE N-S FACE OF THE BUILDING, $L_{P_LNS}$ • DISTANCE OF THE SOUTHWEST CORNER (OF THE PENTHOUSE): - $x_P$ • DISTANCE OF THE SOUTHWEST CORNER (OF THE PENTHOUSE): - $x_P$ • DIRECTION THAT THE COMPOSITE FLOOR SYSTEM SPANS • DIRECTION THAT THE COMPOSITE FLOOR SYSTEM SPANS • $\frac{STORY}{NUMBER}$ • $\frac{STORY}{(CTR-TO-CTR OF GIRDERS)}$ • $\frac{13.0 \text{ ft}}{13.0 \text{ ft}}$ • $\frac{13.0 \text{ ft}}{7}$ • $\frac{13.0 \text{ ft}}{13.0 \text{ ft}}$ • $\frac{13.0 \text{ ft}}{7}$ • $\frac{13.0 \text{ ft}}{13.0 \text{ ft}}$	
<ul> <li>PENTHOUSE: - HEIGHT</li> <li>LENGTH ALONG THE N-S FACE OF THE BUILDING, L<sub>P_JNS</sub></li> <li>UENGTH ALONG THE E-W FACE OF THE BUILDING, L<sub>P_JNS</sub></li> <li>UISTANCE OF THE SOUTHWEST CORNER (OF THE PENTHOUSE): - x<sub>p</sub></li> <li>DISTANCE OF THE SOUTHWEST CORNER (OF THE PENTHOUSE): - x<sub>p</sub></li> <li>O DISTANCE OF THE SOUTHWEST CORNER (OF THE PENTHOUSE): - x<sub>p</sub></li> <li>O DIRECTION THAT THE COMPOSITE FLOOR SYSTEM SPANS</li> <li>E-W THEREFORE, THE BEAMS SPAN</li> <li>O STORY STORY HEIGHT, h<sub>s</sub></li> <li>O STORY (CTR-TO-CTR OF GIRDERS)</li> <li>O 13.0 ft</li> <li>ALONG THE N-S FACE</li> <li>(GIRDERS ARE PARALLEL TO THE E-W DIRECTION)</li> </ul>	
o       DISTANCE OF THE SOUTHWEST CORNER (OF THE PENTHOUSE): - xp       30.0 ft         - yp       10.0 ft         - yp       13.0 ft         - yp       - yp	(ASSUMED)
o     DIRECTION THAT THE COMPOSITE FLOOR SYSTEM SPANS     E-W     THEREFORE, THE BEAMS SPAN       0     STORY     STORY HEIGHT, hs     0       0     NUMBER     (CTR-TO-CTR OF GIRDERS)       9     13.0 ft       8     13.0 ft       7     13.0 ft       7     13.0 ft	<b>^</b> ^
o     DIRECTION THAT THE COMPOSITE FLOOR SYSTEM SPANS     E-W     THEREFORE, THE BEAMS SPAN       o     STORY     STORY HEIGHT, h <sub>s</sub> O     BAY WIDTH, w <sub>b</sub> NUMBER     (CTR-TO-CTR OF GIRDERS)     ALONG EACH FACE OF THE BUILDING       9     13.0 ft     ALONG THE N-S FACE       7     13.0 ft     (GIRDERS ARE PARALLEL TO THE E-W DIRECTION)     (GIRDERS ARE PARALLEL TO THE N-S DIRECTION)	<u>1</u>
o         STORY         STORY HEIGHT, hs         o         BAY WIDTH, wb           NUMBER         (CTR-TO-CTR OF GIRDERS)         ALONG EACH FACE OF THE BUILDING           9         13.0 ft         ALONG THE N-S FACE         ALONG THE E-W           7         13.0 ft         (GIRDERS ARE PARALLEL TO THE E-W DIRECTION)         TO THE N-S DIRECTION	IN THE N-S DIRECTION
NUMBER         (CTR-TO-CTR OF GIRDERS)         ALONG EACH FACE OF THE BUILDING           9         13.0 ft         ALONG THE N-S FACE         ALONG THE E-W           8         13.0 ft         (GIRDERS ARE PARALLEL TO THE E-W DIRECTION)         (GIRDERS ARE PARALLEL TO THE E-W DIRECTION)         (GIRDERS ARE PARALLEL TO THE E-W DIRECTION)	
8     13.0 ft     (GIRDERS ARE PARALLEL TO THE E-W DIRECTION)     (GIRDERS ARE PARALLEL TO THE E-W DIRECTION)     (GIRDERS ARE PARALLEL TO THE NS DIRECTION)	
7 13.0 ft TO THE E-W DIRECTION) TO THE N-S DIRECT	-ACE NLLEL
	ION)
5 13.0 ft BAY CTR-TO-CTR NILMBER CTR-	TO-CTR
4 13.0 ft OF COLUMNS OF COLUMNS OF COLUMNS OF COLUMNS	LUMNS
2 13.0 ft 2 N-S 30.0 ft 2 E-W 3	.0 ft
1 13.0 ft 3_N-S 30.0 ft 3_E-W 3	.0 ft
4_N-S 30.0 ft 4_E-W 3	.0 ft
<u>5_N-S 30.0 tt 5_E-W 3</u>	.0 ft
STORYN s	
	سلہ جہ
	<b>_</b>
<i>STORY#2</i>	hs
	↓
	<b>_</b>
5 TORY #1	
	$\perp$
BAY#1 BAY#2 BA	YN <sub>B</sub>
FI EVATION VIEW	

ශිශි පිතක්කලලන්තු	JOB NC	JOB NO. 9-STORY BUILDINGS						BY	SMG	DATE	9/16/04	SHEET NO.		
oo piriimeen miii	CUSTO	CUSTOMER RCFT PARAMETRIC STUDY						CKD		DATE		OF		
SUBJECT	2.	-D MO	MENT	FRAM	IE [MF	A2 - F	2] AN	ALYSI	S LOAD S	SUMMA	RY	•		
[2] 2-D MOMENT FRA	AME GEOME	TRY												
o TYPE C	OF FRAME:						MOM	ENT FRAM	E					
o DIREC	TION THAT T	HE MOME	NT FRAME F	RUNS PAR	ALLEL WITH	ł:		E-W	(THEREF	ORE, THIS M		WILL RESIST		
o MOME	NT FRAME N	IAME:					М	F A2 - F2	SEISMIC LOADS IN THE E-W DIRECTION )					
o DISTAN Ti	NCE FROM T	THE CLOSE	ST COLUM	N STACK IN THE BUILD	I THE MOMI ING, Yc	ENT FRAME	Ξ	120.0 ft	THIS FRAME I	S LOCATED	IN THE INTERIC	OR OF THE BUILDING		
o NUMBE	0 NUMBER OF BAYS IN THIS MOMENT FRAME:							5 BAYS		AND THIS FHAME SUPPORTS PART OF THE PENTHOUSE.				
o THE FII W	<ul> <li>THE FIRST BAY IN THIS MOMENT FRAME IS THE SAME BAY AS WHICH BAY NUMBER IN THE BUILDING LAYOUT?</li> </ul>								THIS MOM	THIS MOMENT FRAME HAS A TOTAL LENGTH OF 150ft.				
DISTANCE TO THE CLOSEST (GRAVITY/MOMENT) FRAME:     DISTANCE TO THE CLOSEST (GRAVITY/MOMENT) FRAME ON THE OTHER SIDE:     30.0 ft														
Coordinates of the MOMENT FRAME:         Condinates of the MOMENT FRAME:         End Column Location:       X         Farthest West       0.0 ft         120.0 ft       Northwest         30.0 ft       30.0 ft         Northwest       30.0 ft         120.0 ft       Northeast         120.0 ft       30.0 ft         Southeast       120.0 ft										1				
		1	2	3	4	5	6							
	NUMBER	x y	x y	x y	x y	x y	x y	х у	x y x	y x	y x y			
o ASSUN THE		HE PENTH G IS TRUE	OUSE PERI FOR THIS M	METER IS A	ALWAYS LC	DCATED OV	YER A GRAV	/ity/mome	ENT FRAME,			1		
(S	TACK) NO.	1	2	3	4	5	6							
c (S SU Wł	Column Stack) IS Pporting Hich Part Of The Nthouse?	NONE	EXTERIOR	EXTERIOR	EXTERIOR	EXTERIOR	NONE							

GG Engling	antina	JOB NO. 9	-STORY BU	IILDINGS		BY SMG			SHEET NO.
		CUSTOME	R RCFT PA	RAMETRIC	STUDY	CKD	D	ATE	OF
SUBJECT		2-D	MOMENT	FRAME [	MF A2 - F2]	ANALYSIS	LOAD SUN	IMARY	
[3] BUILD	o BUILI	.oad Ding (Floors): Ding (Roof):	- COLUMNS - EXTERIOF - FLOORING - COMPOSI - CEILING (F - HVAC + EL - PARAPET	S, BEAMS, GIRDE R WALLS TE FLOOR SYSTI ROM STORY BE LECTRICAL (FRO	RS, MISC. STRUCTUR EM (CONCRETE + ME' LOW) + FIREPROOFIN M STORY BELOW)	AL SYSTEM COMPO TAL DECKING) G	DNENTS 20 25 1    50 2    7    25	Ib/ft <sup>2</sup> (Applied to Si b/ft <sup>2</sup> b/ft <sup>2</sup> b/ft <sup>2</sup> b/ft <sup>2</sup> (Applied to Si	urface Area of the WALL
COMPOSITE ROOF SYSTEM (CONCRETE + METAL DECKING)     PENTHOUSE:     COMPOSITE ROOF SYSTEM (CONCRETE + METAL DECKING)     CELLING (FROM STORY BELOW) + FIREPROOFING     CELLING (FROM STORY BELOW)     COMPOSITE ROOF SYSTEM (CONCRETE + METAL DECKING)     O     PENTHOUSE:     COMPOSITE ROOF SYSTEM (CONCRETE + METAL DECKING)     CELLING (FROM STORY BELOW)     CELING (FROM STORY BE									
NUN	NUMBER (STORY) D.L. (SURFACE) AREA DEAD LOAD						TOTAL I	DEAD LOAD	TOTAL STORY DL
STORY	FLOOR	HEIGHT	ROOF / FLOOR	PENTHOUSE	ROOF / FLOOR	PENTHOUSE	ROOF / FLOOR	PENTHOUSE	ΣΡ <sub>i</sub>
				8,100 ft <sup>2</sup>		127.44 lb/ft <sup>2</sup>		1,032.26 kips	
9	ROOF	13.0 ft	22,500 ft <sup>2</sup>		88.33 lb/ft <sup>2</sup>		1,987.43 kips		3.020 kips
8	9	13.0 ft	22,500 ft <sup>2</sup>		88.67 lb/ft <sup>2</sup>		1,995.08 kips		5,015 kips
7	8	13.0 ft	22,500 ft <sup>2</sup>		88.67 lb/ft <sup>2</sup>		1,995.08 kips		7,010 kips
6		13.0 ft	22,500 ft <sup>2</sup>		88.67 lb/tt <sup>2</sup>		1,995.08 kips		9,005 kips
5	6	13.0 ft	22,500 ft <sup>2</sup>		88.67 lb/ft <sup>2</sup>		1,995.08 kips		11,000 kips
4	5	13.0 ft	22,500 π <sup>2</sup>		88.67 lb/tt2		1,995.08 Kips		12,995 kips
3	4	13.0 ft	22,500 ft <sup>2</sup>		88.67 lb/ft <sup>2</sup>		1,995.08 kips		14,990 kips
2	3	13.0 ft	22,500 ft <sup>2</sup>		88.67 lb/ft <sup>2</sup>		1,995.08 kips		16,985 kips
1	2 GROUND	13.0 ft	22,500 ft <sup>2</sup> N.A.		88.67 lb/ft² N.A.		1,995.08 kips N.A.		18,980 kips
	·		I	l	Building Total D	ead Load (Ground F	loor + 1st Story Deac	l Load NOT Included) =	18,980 kips

#### Design and Evaluation of Rectangular Concrete Filled Tube (RCFT) Frames for Seismic Demand Assessment

GG Englineering		JOI ନାଶ୍ର	JOB NO. 9-STORY BUILDINGS				BY SMG	DATE 9/	/16/04	SHEET NO.
			CUSTOMER RCFT PARAMETRIC STUDY				CKD	DATE	-	OF
SUBJECT       2-D MOMENT FRAME [MF A2 - F2] ANALYSIS LOAD SUMMARY										
[4] BUILDING LIVE LOAD										
• BUILDING (FLOORS): - OFFICE BUILDING OCCUPANCY PER IBC 2003, TABLE 1607.1 - (MOVEABLE) PARTITIONS PER IBC 2003, SECTION 1607.5								50 lb/ft² 20 lb/ft²		
• BUILDING (ROOF): - MINIMUM ROOF LL PER IBC 2003, SECTION 1607.11 20 lb/ft <sup>2</sup> (ROOF LIVE LOAD, Lr)										
O PENTHOUSE:     GENERAL PENTHOUSE (INTERIOR) LIVE LOAD     PENTHOUSE (ROOF) LIVE LOAD								20 lb/ft²     (TREATED AS ROOF LIVE LOAD, Lr)       0 lb/ft²     (ROOF LIVE LOAD, Lr)		
NUM	/RFR		L L. (SUBE/			ΩΔΩ	TOTAL LIVE LOAD		TOTAL STORY LL, ΣP:	
STORY	FLOOR	HEIGHT	ROOF / FLOOR	PENTHOUSE	ROOF / FLOOR	PENTHOUSE	ROOF / FLOOR	PENTHOUSE	FLOOR LL	ROOFLL
				8 100 ft2		20 lb/ft2		162.0 kips		
	ROOF		14.400 ft <sup>2</sup>		20 lb/ft <sup>2</sup>		288.0 kips			
9	9	13.0 ft	22,500 ft <sup>2</sup>		70 lb/ft <sup>2</sup>		1,575.0 kips		0 kips	450 kips
8	8	13.0 ft	22,500 ft <sup>2</sup>		70 lb/ft <sup>2</sup>		1,575.0 kips		1,575 kips	450 kips
7	7	13.0 ft	22,500 ft <sup>2</sup>		70 lb/ft <sup>2</sup>		1,575.0 kips		3,150 kips	450 kips
6	6	13.0 ft	22,500 ft <sup>2</sup>		70 lb/ft <sup>2</sup>		1,575.0 kips		4,725 kips	450 kips
5	5	13.0 ft	22,500 ft <sup>2</sup>		70 lb/ft <sup>2</sup>		1,575.0 kips		6,300 kips	450 kips
4	4	13.0 ft	22,500 ft <sup>2</sup>		70 lb/ft <sup>2</sup>		1,575.0 kips		7,875 kips	450 kips
3	3	13.0 ft	22,500 ft <sup>2</sup>		70 lb/ft <sup>2</sup>		1,575.0 kips		9,450 Kips	450 Kips
2	2	13.0 π 13.0 #	22,500 ft <sup>2</sup>		70 lb/ft <sup>2</sup>		1,575.0 kips		12,600 kips	450 kips
I	GROUND	13.0 11	N.A.		N.A.		N.A.		12,000 KIps	450 Kips
	└───┼						ļ			1
1										

Building Total Live Load and Roof Live Load =

12,600 kips

450 kips
GG Engineering	JOB N	10. 9-S	TORY E	BUILDIN	GS			BY	SMG		DATE	9/16/04	SHEET NO.
	CUST	OMER	RCFT F	PARAME	ETRIC S	TUDY		СКД	1		DATE		OF
SUBJECT		2-D M	OMEN	T FRA	ME (M	F A2 -	F2] AN	NALYS	IS LO	AD S	UMMAF	RY	
[5] MOMENT FRAM	o Su	DAD IMMARY: EREFORE: NOTI	NUM NUM DIRE LOC/ DOE: DIST. DIST. DEAI E: PAR/ PAR/	BER OF STI BER OF BA CTION THA ATION OF TI S THIS FRA ANCE TO TI ANCE TO TI O LOAD TRI O LOAD TRI APET DEAL	ORIES, N <sub>S</sub> YS, N <sub>B</sub> T THE <i>MOM</i> HE MOMEN ME SUPPO HE CLOSES HE CLOSES HE CLOSES BUTARY W D LOAD (PE D LOAD (PE	IENT FRAM IT FRAME V RT PART O ST (GRAVIT ST (GRAVIT IDTH TO TH R ft <sup>2</sup> OF RO R FOOT OF	IE RUNS PA VRT THE BU F THE PENT Y/MOMENT) Y/MOMENT) IIS MOMENT OF SURFAC PARAPET I	RALLEL W JILDING PE THOUSE GI FRAME: FRAME OF FRAME: C AREA) LENGTH)		ADS? ER SIDE: ➡	9 STORIES 5 BAYS E-W INTERIOR YES 30.0 ft 30.0 ft 2.33 lb/ft <sup>2</sup> 87.5 lb/ft	S OR/LEVEL	
(S <sup>T</sup> PF	COLUMN TACK) NO.	1	2	3	4	5	6						
	OADS TO THE OLUMNS	0.0 kips	9.6 kips	19.1 kips	19.1 kips	9.6 kips	0.0 kips						
N	FLOOR NUMBER		UN	IFACTORED	O (NOMINAL OF THE M	.) DEAD LO MOMENT FI	AD BEAM E	ND REACT	IONS TO E R/LEVEL	ACH COL	UMN		
	ROOF	15.5 kips	35.4 kips	44.9 kips	44.9 kips	35.4 kips	15.5 kips						
	8 7 6 5 4 3 2 GROUND	13.3 kips 13.3 kips 13.3 kips 13.3 kips 13.3 kips 13.3 kips 13.3 kips 13.3 kips N.A.	26.6 kips 26.6 kips 26.6 kips 26.6 kips 26.6 kips 26.6 kips 26.6 kips N.A.	26.6 kips 26.6 kips 26.6 kips 26.6 kips 26.6 kips 26.6 kips 26.6 kips N.A.	26.6 kips 26.6 kips 26.6 kips 26.6 kips 26.6 kips 26.6 kips 26.6 kips N.A.	26.6 kips 26.6 kips 26.6 kips 26.6 kips 26.6 kips 26.6 kips 26.6 kips N.A	13.3 kips 13.3 kips 13.3 kips 13.3 kips 13.3 kips 13.3 kips 13.3 kips 13.3 kips N.A.						



ශිශි පිතක්තලලන්ත	JOB N สา	i0. 9-S	TORYE	BUILDIN	GS			BY	SMG		DATE	9/16/04	SHEET NO.
oo riiginisen iiit	CUST	OMER	RCFT F	PARAME	ETRIC S	TUDY		СКД	1		DATE		OF
SUBJECT	2	2-D M(	OMEN	T FRA	ME [M	F A2 -	F2] AN	NALYS	IS LO	AD SI	JMMAF	RY	
[7] MOMENT FRAM	o SUI	MMARY:	NUMI NUMI DIRE LOCA DIST/ DIST/ LIVE	3ER OF STO 3ER OF BA CTION THA ATION OF TH S THIS FRAI ANCE TO TH ANCE TO TH LOAD TRIB	DRIES, N <sub>S</sub> YS, N <sub>B</sub> T THE <i>MOM</i> HE MOMEN HE CLOSES HE CLOSES UTARY WIE	IENT FRAM IT FRAME V RT PART O ST (GRAVIT ST (GRAVIT DTH TO THIS	E RUNS PA VRT THE BU F THE PENT V/MOMENT) V/MOMENT I	Rallel W Jilding Pe Thouse Gf Frame: Frame Of Frame:		NDS? ER SIDE:	9 STORIES 5 BAYS E-W INTERIOR YES 30.0 ft 30.0 ft 30.0 ft	; DR/LEVEL	
	COLUMN				₩	5	↓ 		₩		_ <b>↓</b>		
(S PE L	TACK) NO. INTHOUSE OADS TO THE COLUMNS	0.0 kips	2 1.5 kips	3 3.0 kips	4 3.0 kips	<b>5</b> 1.5 kips	6 0.0 kips						
Ι Γ.	FLOOR NUMBER		10	IFACTORE	D (NOMINA OF THE M	L) LIVE LOA	AD BEAM EI RAME AT EV	ND REACT	ONS TO E	ACH COLL	ЛМИ		
	9 8 7 6 5 4 3 2 3ROUND	10.5 kips 10.5 kips 10.5 kips 10.5 kips 10.5 kips 10.5 kips 10.5 kips 10.5 kips 10.5 kips 10.5 kips	21.0 kips 21.0 kips	21.0 kips 21.0 kips	21.0 kips 21.0 kips	21.0 kips 21.0 kips	10.5 kips 10.5 kips						



GG Engingering	JOB N	10. 9-S	TORY E	BUILDIN	IGS			BY	SMG		DATE	9/16/04	SHEET NO.
	CUST	OMER	RCFT F	PARAMI	ETRIC S	TUDY		CKE	)		DATE		OF
SUBJECT	2	2-D M	OMEN	T FRA	ME [M	F A2 -	F2] AN	NALYS	SIS LO	AD SI	UMMA	۲F	
[9] MOMENT FRAM	o SU	WEIGHT ( MMARY: EREFORE NOT	DEAD LOAI NUM NUM DIRE LOC/ DOE: DIST. DIST. DIST. SEIS E: PAR PAR PAR	D + PARTIT BER OF ST BER OF BA CTION THA ATION OF T S THIS FRA ANCE TO TI ANCE TO TI ANCE TO TI MIC WEIGH APET SEISI APET SEISI ITTION LIVE	ION LIVE L ORIES, N <sub>S</sub> YS, N <sub>B</sub> T THE MOMEN HE MOMEN ME SUPPO HE CLOSES HE CLOSES HE CLOSES HE CLOSES HE CLOSES	OAD) MENT FRAME V RT PART O ST (GRAVIT ST (GRAVIT RY WIDTH <sup>-</sup> T (PER ft <sup>2</sup> C T (PER FOC	YRT THE BU F THE PENT Y/MOMENT) Y/MOMENT) TO THIS MO DF ROOF SU DT OF PARA	RALLEL W IILDING PE THOUSE GI FRAME: FRAME OI MENT FRA MENT FRA RFACE AR PET LENG	ITTH: RIMETER: RAVITY LOA N THE OTHE ME: IEA) TH)	NDS? ER SIDE:	9 STORIES 5 BAYS E-W INTERIOF YES 30.0 ft 30.0 ft 2.33 lb/ft <sup>2</sup> <i>FLC</i>	S NOR/LEVEL	
	COLUMN TACK) NO.	1	2	3	4	5	6						
	OADS TO THE OLUMNS	0.0 kips	9.6 kips	19.1 kips	19.1 kips	9.6 kips	0.0 kips						
N	FLOOR UMBER		UNFA	CTORED (I	NOMINAL) S OF THE I	SEISMIC WE	EIGHT BEAN RAME AT E\	I END REA	ACTIONS TO DR/LEVEL	D EACH C	OLUMN		
	ROOF	15.5 kips	35.4 kips	44.9 kips	44.9 kips	35.4 kips	15.5 kips						
	9 8 7 6 5	16.3 kips 16.3 kips 16.3 kips 16.3 kips 16.3 kips	32.6 kips 32.6 kips 32.6 kips 32.6 kips 32.6 kips	32.6 kips 32.6 kips 32.6 kips 32.6 kips 32.6 kips	32.6 kips 32.6 kips 32.6 kips 32.6 kips 32.6 kips	32.6 kips 32.6 kips 32.6 kips 32.6 kips 32.6 kips	16.3 kips 16.3 kips 16.3 kips 16.3 kips 16.3 kips						
	4	16.3 kips	32.6 kips	32.6 kips	32.6 kips	32.6 kips	16.3 kips						
G	3 2 GROUND	16.3 kips 16.3 kips N.A.	32.6 kips 32.6 kips N.A.	32.6 kips 32.6 kips N.A.	32.6 kips 32.6 kips N.A.	32.6 kips 32.6 kips N.A.	16.3 kips 16.3 kips N.A.						



					-	
GG Englineering	JOB NO. 9-STORY BUILDINGS	3	BY SMG	DATE 9	9/16/04	SHEET NO.
	CUSTOMER RCFT PARAMETE	RIC STUDY	CKD	DATE		OF
SUBJECT	2-D MOMENT FRAME	E [MF A2 - F2] AN/	ALYSIS LOAD	SUMMAR	Y	
o PER THE EX	CEPTION OF SECTION 1614 OF THE IBC 2003,	ASCE 7 IS PERMITTED TO BE U	SED TO DETERMINE THE	E SEISMIC LOADS	Section 1614	(p. 302)
	DRE, THE FOLLOWING DESIGN PARAMETERS A	ARE TAKEN FROM ASCE 7-02.			Table 1 1	(p. 4)
SEISMIC US	E GROUP				Section 9.1.3	(p. 96)
					Table 9.1.3	(p. 96)
0 OCCUPANC	Y IMPORTANCE FACTOR, I <sub>E</sub>	1.00			Section 9.1.4 Table 9.1.4	(p. 96) (p. 97)
o THE MAPPE	D SPECTRAL ACCELERATIONS:	FOR SHORT PERIODS, FOR A 1-SECOND PERIOD	$S_{\rm S} = 1.5 \text{ g}$ , $S_{\rm 1} = 0.6 \text{ g}$		Section 9.4.1 Figure 9.4.1.1	.2 (p. 107) I(c) & (d)
o SITE CLASS			D		Section 9.4.1 Note the "Exc	.2.1 (p. 108) ception"
<ul> <li>SITE COEFF</li> </ul>	ICIENTS:		$F_{a} = 1.0$ $F_{v} = 1.5$		Table 9.4.1.2 Table 9.4.1.2	.4a (p. 129) .4b (p. 130)
o MAX. CONSI	DERED SPECTRAL RESPONSE ACCELERATIO	NS: FOR SHORT PERIODS, S FOR A 1-SECOND PERIOD,	S <sub>MS</sub> = 1.5 g S <sub>M1</sub> = 0.9 g		Equation 9.4. Equation 9.4.	1.2.4-1 (p. 129) 1.2.4-2 (p. 129)
o DESIGN SPE	ECTRAL RESPONSE ACCELERATIONS:	FOR SHORT PERIODS, 5 FOR A 1-SECOND PERIOD,	$S_{DS} = 1.0 \text{ g}$ $S_{D1} = 0.6 \text{ g}$		Equation 9.4. Equation 9.4.	1.2.5-1 (p. 129) 1.2.5-2 (p. 129)
o SEISMIC DE	SIGN CATEGORY	D			Table 9.4.2.1 Table 9.4.2.1	a (p. 131) b (p. 132)
o BASIC SEIS	MIC-FORCE RESISTING SYSTEM	"SPECIAL COMPOSITE MOM	ENT FRAMES"		Table 9.5.2.2	(p. 134)
RESPONSE	MODIFICATION COEFFICIENT, R	8			Table 9.5.2.2	(p. 134)
DEFLECTION	NAMPLIFICATION FACTOR, Cd	5.5			Table 9.5.2.2	(p. 134)
o FUNDAMEN	TAL PERIOD, T				Section 9.5.5	.3 (p. 147)
BUILDING P	ERIOD COEFFICIENTS:	$C_{T} = 0.028$ x = 0.8			Table 9.5.5.3 Table 9.5.5.3	.2 (p. 147) .2 (p. 147)
ELEVATION	OF THE BUILDING ROOF ABOVE THE BASE, $h_n$	117 ft				
COEFFICIEN	T FOR UPPER LIMIT ON CALCULATED PERIOD	, C <sub>U</sub> 1.4			Table 9.5.5.3	.1 (p. 147)
APPROXIMA	TE FUNDAMENTAL PERIOD, T <sub>a</sub>	1.264 sec			Equation 9.5.	5.3.2-1 (p. 147)
PERIOD FRO	DM RATIONAL ANALYSIS, T <sub>R</sub>	NONE CALCULATE	Đ			
MAXIMUM A	LLOWED PERIOD, C <sub>U</sub> T <sub>a</sub>	1.770 sec			Section 9.5.5	.3 (p. 147)
DESIGN PEF	RIOD, T	1.264 sec			Section 9.5.5	.3 (p. 147)
o ANALYSIS P	ROCEDURE	EQUIVALENT LATERAL FOR	CE ANALYSIS		Section 9.5.2 Table 9.5.2.5	5.1 (p. 139) .1 (p. 140)
0 SEISMIC RE	SPONSE COEFFICIENT, C <sub>S</sub>	$C_{S} = 0.125 \text{ g}$			Equation 9.5.	5.2.1-1 (p. 146)
		$C_S \leq 0.060 \text{ g}$			Equation 9.5.	5.2.1-2 (p. 146)
		$C_S \ge 0.044 \text{ g}$			Equation 9.5.	5.2.1-3 (p. 146)
		$DESIGN C_{S} = 0.060 g$				
o SEISMIC (DE	ESIGN) BASE SHEAR, V	V = 0.060 W			Equation 9.5.	5.2-1 (p. 146)

IER         RCFT           D         MOMEN           THE SEISMIC LO         0           0         BUILDI           0         DISTRI           0         DESIGI           0         EFFEC           0         DESIGI           0         EFFEC           0         DESIGI           1         ELEVATION           117.0 ft         104.0 ft           91.0 ft         78.0 ft           65.0 ft         52.0 ft           39.0 ft         39.0 ft	PARAMET IT FRAM DADS PER ASC NG FUNDAMEN BUTION EXPON N BASE SHEAR TIVE SEISMIC V N SHEAR AT TH DEAD ROOF / FLR 2,084.93 kips 1,995.08 kips 1,	RIC STUI E [MF A E 7-02 [IBC : ITAL (DESIGN IENT, k R, V WEIGHT, W HE BASE OF T LOAD PENTHOUSE  1,032.26 kips 	DY 2 - F2] A 2003 ] ) PERIOD, T HE BUILDING, V (MOVEABLE PA ROOF / FLR  0.0 kips 450.0 kips 450.0 kips 450.0 kips	CKD 1.264 1.3 0.06 22,677 ( 1,360. RTITION) L.L. PENTHOUSE  0.0 kips     	A sec 38 30 W 2.8 kips 7 kips (SEISMIC) WEIGHT w <sub>x</sub>  3,117.19 kips 2,445.08 kips 2,445.08 kips 2,445.08 kips 2,445.08 kips 2,445.08 kips	SUMMA SUMA SU	Sec Equ Equ k (kip-ft) 2.228E+06 1.235E+06 1.235E+06 9.986E+05 7.765E+05	Ction 9.5.5. uation 9.5.5 uation 9.5.5 C <sub>vx</sub> C <sub>vx</sub> 0.2792 0.1861 0.1547 0.1251 0.0973	OF 4 (p. 148 5.4-1 (p. 148 5.4-2 (p. 148 5.4-2 (p. 148 F <sub>x</sub> 5.4-2 (p. 148 5.4-2 (p. 148) 5.4-2 (p. 148 5.4-2 (p. 148) 5.4-2 (p. 148
FLOOR           0         EFFC           0         DESIG           0         DESIG           0         EFFC           0         DESIG           0         EFFC           0         DESIG           10         DESIG           117.0 ft         104.0 ft           91.0 ft         78.0 ft           65.0 ft         52.0 ft           39.0 ft         39.0 ft	IT FRAM DADS PER ASC ONG FUNDAMEN BUTION EXPON N BASE SHEAR TIVE SEISMIC V N SHEAR AT TH DEAD ROOF / FLR	E [MF A E 7-02 [IBC: ITAL (DESIGN IENT, k R, V NEIGHT, W IE BASE OF T LOAD PENTHOUSE  1,032.26 kips 	2 - F2] A 2003 ] ) PERIOD, T HE BUILDING, V (MOVEABLE PA ROOF / FLR 0.0 kips 450.0 kips 450.0 kips 450.0 kips	1.264 1.264 1.3 0.06 22,677 1,360. RTITION) L.L. PENTHOUSE  0.0 kips     	4 sec 38 50 W .8 kips .7 kips (SEISMIC) WEIGHT w <sub>x</sub>  3,117.19 kips 2,445.08 kips 2,445.08 kips 2,445.08 kips 2,445.08 kips 2,445.08 kips	SUMMA SEISMIC WT. TOTAL PER FLOOR 3,117.2 kips 5,562.3 kips 8,007.4 kips 10,452.5 kips	xxhxk Equ Equ Equ (kip-ft)  2.228E+06 1.235E+06 1.235E+06 9.986E+05 7.765E+05	Cition 9.5.5. uation 9.5.5 uation 9.5.5 C <sub>vx</sub>  0.2792 0.1861 0.1547 0.1251 0.0973	4 (p. 148 5.4-1 (p. 148 5.4-2 (p. 148 F <sub>x</sub>  379.9 kips 253.2 kips 210.5 kips 170.2 kips 132.4 kips
THE SEISMIC LO           o         BUILDI           o         DISTRI           o         DISTRI           o         DESIGI           o         EFFEC           o         DESIGI           o         O           o         DESIGI           o         O           o         O           o <th>DADS PER ASC NG FUNDAMEN BUTION EXPON N BASE SHEAR TIVE SEISMIC V N SHEAR AT TH DEAD ROOF / FLR </th> <th>E 7-02 [IBC : ITAL (DESIGN IENT, k k, V WEIGHT, W IE BASE OF T LOAD PENTHOUSE  1,032.26 kips </th> <th>2003 ] ) PERIOD, T HE BUILDING, V (MOVEABLE PA ROOF / FLR  0.0 kips 450.0 kips 450.0 kips 450.0 kips</th> <th>1.264 1.1 0.06 22,677 V 1,360. V 1,360. RTITION) L.L. PENTHOUSE  0.0 kips    </th> <th>4 sec 38 30 W 28 kips 7 kips (SEISMIC) WEIGHT w<sub>x</sub>  3,117.19 kips 2,445.08 kips 2,445.08 kips 2,445.08 kips 2,445.08 kips</th> <th>SEISMIC WT. TOTAL PER FLOOR 3,117.2 kips 5,562.3 kips 8,007.4 kips 10,452.5 kips</th> <th>Sec Equ Equ (kip-ft)  2.228E+06 1.485E+06 1.235E+06 9.986E+05 7.765E+05</th> <th>ction 9.5.5. uation 9.5.5 uation 9.5.5 C<sub>vx</sub>  0.2792 0.1861 0.1547 0.1251 0.0973</th> <th>4 (p. 148 5.4-1 (p. 148 5.4-2 (p. 148 F<sub>x</sub>  379.9 kips 253.2 kips 210.5 kips 170.2 kips 132.4 kips</th>	DADS PER ASC NG FUNDAMEN BUTION EXPON N BASE SHEAR TIVE SEISMIC V N SHEAR AT TH DEAD ROOF / FLR 	E 7-02 [IBC : ITAL (DESIGN IENT, k k, V WEIGHT, W IE BASE OF T LOAD PENTHOUSE  1,032.26 kips 	2003 ] ) PERIOD, T HE BUILDING, V (MOVEABLE PA ROOF / FLR  0.0 kips 450.0 kips 450.0 kips 450.0 kips	1.264 1.1 0.06 22,677 V 1,360. V 1,360. RTITION) L.L. PENTHOUSE  0.0 kips    	4 sec 38 30 W 28 kips 7 kips (SEISMIC) WEIGHT w <sub>x</sub>  3,117.19 kips 2,445.08 kips 2,445.08 kips 2,445.08 kips 2,445.08 kips	SEISMIC WT. TOTAL PER FLOOR 3,117.2 kips 5,562.3 kips 8,007.4 kips 10,452.5 kips	Sec Equ Equ (kip-ft)  2.228E+06 1.485E+06 1.235E+06 9.986E+05 7.765E+05	ction 9.5.5. uation 9.5.5 uation 9.5.5 C <sub>vx</sub>  0.2792 0.1861 0.1547 0.1251 0.0973	4 (p. 148 5.4-1 (p. 148 5.4-2 (p. 148 F <sub>x</sub>  379.9 kips 253.2 kips 210.5 kips 170.2 kips 132.4 kips
O BUILDI     O DISTRI     O DISTRI     O DESIGI     O EFFEC     O DESIGI     T     FLOOR     FLOOR     T     T     FLOOR     T	NG FUNDAMEN BUTION EXPON N BASE SHEAR TIVE SEISMIC V N SHEAR AT TH DEAD ROOF / FLR 2,084.93 kips 1,995.08 kips 1,995.08 kips 1,995.08 kips 1,995.08 kips 1,995.08 kips	ITAL (DESIGN IENT, k R, V NEIGHT, W IE BASE OF T LOAD PENTHOUSE  1,032.26 kips  	) PERIOD, T HE BUILDING, V (MOVEABLE PA ROOF / FLR 0.0 kips 450.0 kips 450.0 kips 450.0 kips	1.264 1.1 0.06 22,677 V 1,360. V 1,360. RTITION) L.L. PENTHOUSE  0.0 kips    	4 sec 38 50 W 7.8 kips 7 kips (SEISMIC) WEIGHT w <sub>x</sub>  3,117.19 kips 2,445.08 kips 2,445.08 kips 2,445.08 kips 2,445.08 kips	SEISMIC WT. TOTAL PER FLOOM 3,117.2 kips 5,562.3 kips 8,007.4 kips 10,452.5 kips	w <sub>x</sub> h <sub>x</sub> <sup>k</sup> (kip-ft) 2.228E+06 1.235E+06 9.986E+05 7.765E+05	C <sub>vx</sub> C <sub>vx</sub> 0.2792 0.1861 0.1547 0.1251 0.0973	F <sub>x</sub>  379.9 kips 253.2 kips 210.5 kips 170.2 kips 132.4 kips
	BUTION EXPON N BASE SHEAR TIVE SEISMIC V N SHEAR AT TH DEAD ROOF / FLR 	IENT, k k, V NEIGHT, W IE BASE OF T LOAD PENTHOUSE  1,032.26 kips  	(MOVEABLE PA ROOF / FLR  0.0 kips 450.0 kips 450.0 kips 450.0 kips	1.3 0.06 22,677 V 1,360. V 1,360. RTITION) L.L. PENTHOUSE  0.0 kips    	38 50 W 7.8 kips 7 kips (SEISMIC) WEIGHT wx  3,117.19 kips 2,445.08 kips 2,445.08 kips 2,445.08 kips 2,445.08 kips	SEISMIC WT. TOTAL PER FLOOR 3,117.2 kips 5,562.3 kips 8,007.4 kips 10,452.5 kips	w <sub>x</sub> h <sub>x</sub> <sup>k</sup> (kip-ft)  2.228E+06 1.235E+06 9.986E+05 7.765E+05	C <sub>vx</sub>  0.2792 0.1861 0.1547 0.1251 0.0973	F <sub>x</sub> 379.9 kips 253.2 kips 210.5 kips 170.2 kips 132.4 kips
<ul> <li>DESIGN</li> <li>EFFEC</li> <li>DESIGN</li> <li>DESIGN</li> <li>DESIGN</li> <li>DESIGN</li> <li>DESIGN</li> <li>TELEVATION</li> <li>h<sub>x</sub></li> <li></li> <li>117.0 ft</li> <li>104.0 ft</li> <li>91.0 ft</li> <li>78.0 ft</li> <li>65.0 ft</li> <li>52.0 ft</li> <li>39.0 ft</li> </ul>	N BASE SHEAR TIVE SEISMIC V N SHEAR AT TH DEAD ROOF / FLR 	ILOAD PENTHOUSE	HE BUILDING, V (MOVEABLE PA ROOF / FLR 0.0 kips 450.0 kips 450.0 kips 450.0 kips 450.0 kips	0.06 22,677 V 1,360. V 1,360. PENTHOUSE  0.0 kips    	0 W .8 kips .7 kips (SEISMIC) WEIGHT w <sub>x</sub>  3,117.19 kips 2,445.08 kips 2,445.08 kips 2,445.08 kips 2,445.08 kips	SEISMIC WT. TOTAL PER FLOOR  3,117.2 kips 5,562.3 kips 8,007.4 kips 10,452.5 kips	w <sub>x</sub> h <sub>x</sub> <sup>k</sup> (kip-ft) 2.228E+06 1.485E+06 1.235E+06 9.986E+05 7.765E+05	C <sub>vx</sub>  0.2792 0.1861 0.1547 0.1251 0.0973	F <sub>x</sub>  379.9 kips 253.2 kips 210.5 kips 170.2 kips 132.4 kips
<ul> <li>FLOOR</li> <li>DESIG</li> <li>DESIG</li> <li>DESIG</li> <li>DESIG</li> <li>DESIG</li> <li>T</li> <li>FLOOR</li> <li>h_x</li> <li></li> <li>117.0 ft</li> <li>104.0 ft</li> <li>91.0 ft</li> <li>78.0 ft</li> <li>65.0 ft</li> <li>52.0 ft</li> <li>39.0 ft</li> </ul>	TIVE SEISMIC V N SHEAR AT TH DEAD ROOF / FLR  2,084.93 kips 1,995.08 kips 1,995.08 kips 1,995.08 kips 1,995.08 kips 1,995.08 kips 1,995.08 kips	NEIGHT, W IE BASE OF T LOAD PENTHOUSE  1,032.26 kips   	HE BUILDING, V (MOVEABLE PA ROOF / FLR  0.0 kips 450.0 kips 450.0 kips 450.0 kips 450.0 kips	22,677 V 1,360. RTITION) L.L. PENTHOUSE  0.0 kips      	.8 kips 7 kips (SEISMIC) WEIGHT wx  3,117.19 kips 2,445.08 kips 2,445.08 kips 2,445.08 kips 2,445.08 kips 2,445.08 kips	SEISMIC WT. TOTAL PER FLOOR 3,117.2 kips 5,562.3 kips 8,007.4 kips 10,452.5 kips	w <sub>x</sub> h <sub>x</sub> <sup>k</sup> (kip-ft)  2.228E+06 1.485E+06 1.235E+06 9.986E+05 7.765E+05	C <sub>vx</sub>  0.2792 0.1861 0.1547 0.1251 0.0973	F <sub>x</sub> 379.9 kips 253.2 kips 210.5 kips 170.2 kips 132.4 kips
<ul> <li>FLOOR</li> <li>DESIG</li> <li>ELEVATION</li> <li>h<sub>x</sub></li> <li></li> <li>117.0 ft</li> <li>104.0 ft</li> <li>91.0 ft</li> <li>78.0 ft</li> <li>65.0 ft</li> <li>52.0 ft</li> <li>39.0 ft</li> </ul>	DEAD DEAD ROOF / FLR  2,084.93 kips 1,995.08 kips 1,995.08 kips 1,995.08 kips 1,995.08 kips 1,995.08 kips 1,995.08 kips	IE BASE OF T IE BASE OF T LOAD PENTHOUSE  1,032.26 kips  	HE BUILDING, V (MOVEABLE PA ROOF / FLR 0.0 kips 450.0 kips 450.0 kips 450.0 kips 450.0 kips	<ul> <li>RTITION) L.L.</li> <li>PENTHOUSE</li> <li></li> <li></li></ul>	7 kips (SEISMIC) WEIGHT w <sub>x</sub>  3,117.19 kips 2,445.08 kips 2,445.08 kips 2,445.08 kips 2,445.08 kips	SEISMIC WT. TOTAL PER FLOOR  3,117.2 kips 5,562.3 kips 8,007.4 kips 10,452.5 kips	w <sub>x</sub> h <sub>x</sub> <sup>k</sup> (kip-ft) 2.228E+06 1.485E+06 1.235E+06 9.986E+05 7.765E+05	C <sub>vx</sub>  0.2792 0.1861 0.1547 0.1251 0.0973	F <sub>x</sub>  379.9 kips 253.2 kips 210.5 kips 170.2 kips 132.4 kips
T FLOOR LEVATION h <sub>x</sub>  117.0 ft 104.0 ft 91.0 ft 78.0 ft 65.0 ft 52.0 ft 39.0 ft	DEAD ROOF / FLR  2,084.93 kips 1,995.08 kips 1,995.08 kips 1,995.08 kips 1,995.08 kips 1,995.08 kips	LOAD PENTHOUSE  1,032.26 kips   	(MOVEABLE PA ROOF / FLR 0.0 kips 450.0 kips 450.0 kips 450.0 kips 450.0 kips	RTITION) L.L. PENTHOUSE  0.0 kips   	(SEISMIC) WEIGHT w <sub>x</sub> 3,117.19 kips 2,445.08 kips 2,445.08 kips 2,445.08 kips 2,445.08 kips	SEISMIC WT. TOTAL PER FLOOR 3,117.2 kips 5,562.3 kips 8,007.4 kips 10,452.5 kips	w <sub>k</sub> h <sub>x</sub> <sup>k</sup> (kip-ft) 2.228E+06 1.485E+06 1.235E+06 9.986E+05 7.765E+05	C <sub>vx</sub>  0.2792 0.1861 0.1547 0.1251 0.0973	F <sub>x</sub>  379.9 kips 253.2 kips 210.5 kips 170.2 kips 132.4 kips
FLOOR ELEVATION h <sub>x</sub>  117.0 ft 104.0 ft 91.0 ft 78.0 ft 65.0 ft 52.0 ft 39.0 ft	DEAD ROOF / FLR  2,084.93 kips 1,995.08 kips 1,995.08 kips 1,995.08 kips 1,995.08 kips 1,995.08 kips	LOAD PENTHOUSE  1,032.26 kips   	(MOVEABLE PA ROOF / FLR 1  0.0 kips 450.0 kips 450.0 kips 450.0 kips 450.0 kips	RTITION) L.L. PENTHOUSE  0.0 kips   	(SEISMIC) WEIGHT w <sub>x</sub> 3,117.19 kips 2,445.08 kips 2,445.08 kips 2,445.08 kips 2,445.08 kips	SEISMIC WT. TOTAL PER FLOOR 3,117.2 kips 5,562.3 kips 8,007.4 kips 10,452.5 kips	w <sub>x</sub> h <sub>x</sub> <sup>k</sup> (kip-ft)  2.228E+06 1.485E+06 1.235E+06 9.986E+05 7.765E+05	C <sub>vx</sub>  0.2792 0.1861 0.1547 0.1251 0.0973	F <sub>x</sub>  379.9 kips 253.2 kips 210.5 kips 170.2 kips 132.4 kips
$\begin{array}{c c} \hline \\ T \\ \hline \\ \\ \\ \\ \\ \\ \\ \\ \\ \\ \\ \\ \\ \\ \\$	ROOF / FLR 	PENTHOUSE  1,032.26 kips   	ROOF / FLR  0.0 kips 450.0 kips 450.0 kips 450.0 kips 450.0 kips	PENTHOUSE  0.0 kips   	WEIGHT w <sub>x</sub>  3,117.19 kips 2,445.08 kips 2,445.08 kips 2,445.08 kips 2,445.08 kips	3,117.2 kips 5,562.3 kips 8,007.4 kips 10,452.5 kips	(kip-ft)  2.228E+06 1.485E+06 1.235E+06 9.986E+05 7.765E+05	0.2792 0.1861 0.1547 0.1251 0.0973	F <sub>x</sub>  379.9 kips 253.2 kips 210.5 kips 170.2 kips 132.4 kips
 117.0 ft 104.0 ft 91.0 ft 78.0 ft 65.0 ft 52.0 ft 39.0 ft	 2,084.93 kips 1,995.08 kips 1,995.08 kips 1,995.08 kips 1,995.08 kips	 1,032.26 kips    	 0.0 kips 450.0 kips 450.0 kips 450.0 kips 450.0 kips	 0.0 kips  	 3,117.19 kips 2,445.08 kips 2,445.08 kips 2,445.08 kips 2,445.08 kips	 3,117.2 kips 5,562.3 kips 8,007.4 kips 10,452.5 kips	 2.228E+06 1.485E+06 1.235E+06 9.986E+05 7.765E+05	 0.2792 0.1861 0.1547 0.1251 0.0973	 379.9 kips 253.2 kips 210.5 kips 170.2 kips 132.4 kips
117.0 ft 104.0 ft 91.0 ft 78.0 ft 65.0 ft 52.0 ft 39.0 ft	2,084.93 kips 1,995.08 kips 1,995.08 kips 1,995.08 kips 1,995.08 kips 1,995.08 kips	1,032.26 kips	0.0 kips 450.0 kips 450.0 kips 450.0 kips 450.0 kips	0.0 kips   	3,117.19 kips 2,445.08 kips 2,445.08 kips 2,445.08 kips 2,445.08 kips	3,117.2 kips 5,562.3 kips 8,007.4 kips 10,452.5 kips	2.228E+06 1.485E+06 1.235E+06 9.986E+05 7.765E+05	0.2792 0.1861 0.1547 0.1251 0.0973	379.9 kips 253.2 kips 210.5 kips 170.2 kips 132.4 kips
91.0 ft 78.0 ft 65.0 ft 52.0 ft 39.0 ft	1,995.08 kips 1,995.08 kips 1,995.08 kips 1,995.08 kips		450.0 kips 450.0 kips 450.0 kips 450.0 kips		2,445.08 kips 2,445.08 kips 2,445.08 kips 2,445.08 kips	5,562.3 kips 8,007.4 kips 10,452.5 kips	1.235E+06 9.986E+05 7.765E+05	0.1547 0.1251 0.0973	210.5 kips 210.5 kips 170.2 kips 132.4 kips
78.0 ft 65.0 ft 52.0 ft 39.0 ft	1,995.08 kips 1,995.08 kips 1,995.08 kips		450.0 kips 450.0 kips		2,445.08 kips 2,445.08 kips	8,007.4 kips 10,452.5 kips	9.986E+05 7.765E+05	0.1251 0.0973	170.2 kips 132.4 kips
65.0 ft 52.0 ft 39.0 ft	1,995.08 kips 1,995.08 kips		450.0 kips		2,445.08 kips	10,452.5 kips	7.765E+05	0.0973	132.4 kips
52.0 ft 39.0 ft	1,995.08 kips		1			• • • • • • • • • • • •			
39.0 ft	1 1		450.0 kips		2,445.08 kips	12,897.6 kips	5.707E+05	0.0715	97.3 kips
	1,995.08 kips		450.0 kips		2,445.08 kips	15,342.7 kips	3.837E+05	0.0481	65.4 kips
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
13.0 ft	1,995.08 kips		450.0 kips		2,445.08 kips	22,678.0 kips	8.424E+04	0.0106	14.4 kips
0.0 ft	N.A.		N.A.		0.0 kips		0.000E+00	0.0000	0.0 kips
	19.077	83 kips	3 600 0	) kips	22.677 83 kine		7.981F±06		1,360 7 kine
		U.U IT N.A.	0.0 ft N.A	U.U IT N.A N.A.	0.0 ft N.A N.A	U.0 ft         N.A.          N.A.          U.0 kips           Image: Control of the second secon	0.0 ft     N.A.      N.A.      0.0 kps	0.011       N.A.        0.0 kips       0.00E400           0.0 kips        0.0 kips       0.00E400             0.0 kips       0.0 kips       0.0 kips               0.0 kips       0.0 kips       0.0 kips	0.011       N.A.        0.0 kps       0.002+00       0.0002+00       0.0002                0.0 kps        0.002+00       0.0002+00<

ଅକ୍ଷ	දික හැඩික අ	രണ്ടിതര	JOB NC	). 9-ST(	ORY BU	ILDING	S			BY	SMG	DA	TE 9	/16/04	SHE	ET NO.
oo i	ZUGUNA	र जा गालि	CUSTO	MER R	CFT PA	RAMET	RIC ST	UDY		CKD		DA	TE			DF
SUBJI	ECT		2-	D MO	MENT	FRAM	E [MF	A2 - F	2] AN/	ALYSIS	S LOAD	SUM	MAR	/		
[1	3] RED	UNDANCY C	OEFFICIEN	IT, ρ, PER /	ASCE 7-02	[ IBC 2003 ]								Section	9.5.2.4	(p. 138)
			0	SEISMIC D		FGORY			D							
			0	ARE THER	E ONLY SP	ECIAL MON	IENT FRAM	IES?	YES							
			0	REDUNDA	NCY COEFI	FICIENT, p		<b>r</b> <sub>i</sub>	=2 - <u>20</u>	$\overline{A_i}$				Equation	9.5.2.4.2-1	(p. 138)
								ρ <sub>ma</sub>	<sub>x</sub> = 0.19	(FRC	M THE TABLE	BELOW)				
			o	(DESIGN)	REDUNDAN		CIENT, p	¢	= 1.00							
ST NU		AREA OF THE	COL 1						RAME FOR	EVERY ST	ORY OF THE E	BUILDING	ì	DESIGN STORY	r <sub>max_i</sub>	ρ <sub>i</sub>
	9	22,500 ft <sup>2</sup>	8 kips	20 kips	20 kips	20 kips	20 kips	8 kips						379.9 kips	0.0737	0.1909
	8	22,500 ft <sup>2</sup>	17 kips	31 kips	31 kips	31 kips	31 kips	17 kips						9/16/04         SHE           ARY         Section 9.5.2.4           ARY         Section 9.5.2.4           Equation 9.5.2.4.2-         SHEAR           379.9 kips         0.0737           633.1 kips         0.0686           843.6 kips         0.0697           1,013.8 kips         0.0690           1,243.5 kips         0.0691           1,360.7 kips         0.0657	0.0686	0.0564
	7	22,500 ft <sup>2</sup>	22 kips	42 kips	42 kips	42 kips	42 kips	22 kips						843.6 kips	0.0697	0.0870
	6 5	22,500 ft <sup>2</sup>	27 kips	50 kips	50 kips	50 kips	50 kips	27 kips 31 kips						1,013.8 kips	0.0690	0.0676
	4	22,500 ft <sup>2</sup>	32 kips	62 kips	62 kips	62 kips	62 kips	32 kips						9/16/04         SHE           IRY         Section 9.5.2.4           Rey         Equation 9.5.2.4.2-1           Stepsilon 9.5.2.4.2-1         Stepsilon 9.5.2.4.2-1<	0.0898	
	3	22,500 ft <sup>2</sup>	35 kips	65 kips	64 kips	64 kips	65 kips	35 kips							0.0676	
	2	22,500 ft <sup>2</sup>	38 kips	66 kips	65 kips	65 kips	66 kips	38 kips						1,346.3 kips	0.0681	0.0421
	1	22,500 ft <sup>2</sup>	46 kips	62 kips	62 kips	62 kips	62 kips	46 kips						1,360.7 kips	0.0657	-0.0294

ශීශී (පිතත්කික	මැත්තක	JOB NO. 9-STORY	BUILDINGS		BY SM	IG	DATE	9/16/04	SHEET NO.
ee piiĝinie	sen multi	CUSTOMER RCFT	PARAMETRIC	STUDY	CKD		DATE		OF
SUBJECT		2-D MOMEN	NT FRAME [M	IF A2 - F2] AN	ALYSIS L	OAD S	UMMAI	۲Y	
[14] WIND	LOADS PE	R THE INTERNATIONAL BUI	LDING CODE (IBC) 2003	3 [ ASCE 7 ]					
o PE	ER SECTIOI THEREFOI	N 1609.1.1 OF THE IBC 2003, RE, THE FOLLOWING DESIGI	SECTION 6 OF ASCE 7 N PARAMETERS ARE T.	SHALL BE USED TO DET. AKEN FROM ASCE 7-02.	ERMINE THE WIN	ID LOADS.		Section 16	)9 (p. 283)
o OC	CCUPANCY	CATEGORY		п				Table 1-1	(p. 4)
o IM	IPORTANCE	E FACTOR, I		1.00				Table 6-1	(p. 73)
o (3-	-SECOND G	GUST) BASIC WIND SPEED, V		85 mph				Figure 6-1	(p. 36)
o EX	XPOSURE C	CATEGORY		В				Section 6.5	.6.3 (p. 28)
o WI	IND DIRECT	TIONALITY FACTOR, K <sub>d</sub> , FOR	WWFRS OF A BUILDING	0.85				Table 6-4	(p. 76)
0 TC	DPOGRAPH	IC FACTOR, K <sub>zt</sub>		1.0				Section 6.5 Equation 6	.7.2 (p. 30) -3 (p. 30)
o EN	NCLOSURE	CLASSIFICATION	ENCLOSED - SINCE THAT WIND BORNE I	NOT IN A HURICANE REG DEBRIS WILL PENETRATE	ION AND THERE	IS A SMALL AND CLADE	CHANCE DING.	Section 6.5 Section 6.2	.9 (p. 30) (p. 23)
o BL	JILDING TYI	PE	SIMPLE DIAPHRAGM FLOOR DIAPHRAGM	I - WIND LOADS ARE TRA S TO THE MWFRS (MOME	NSFERRED THR NT FRAMES).	ough the F	Roof and	Section 6.2	(p. 24)
o AP AP	PPROXIMA1 PPROXIMA1	TE BUILDING (MAX ALLOWED TE BUILDING FREQUENCY, n	) PERIOD	1.770 sec <i>(F</i> 0.565 Hz S	FROM SEISMIC C. INCE n1 < 1.0 TH	ALCULATIOI E BUILDING	NS) IS FLEXIBLE	Section 6.2	(p. 24)
o DIF	RECTION T	HAT THE MOMENT FRAME R	UNS PARALLEL WITH:	E-W					
o Bl	JILDING WI	DTH (DIMENSION PERPENDI	CULAR TO WIND DIREC	TION), 150.0 ft A	LONG THE E-W F	FACE			
o BL	JILDING DE	PTH (DIMENSION PARALLEL	TO WIND DIRECTION),	L 150.0 ft A	LONG THE N-S F.	ACE			
o ME	EAN ROOF	HEIGHT ABOVE GRADE, h		117.0 ft					
o VE	ELOCITY PF	RESSURES		qz = 15.72 Kz lb/ft <sup>2</sup> qh = 16.19 lb/ft <sup>2</sup>				Section 6.5 Equation 6	.10 (p. 31) .15 (p. 31)
o GL	UST EFFEC	T FACTOR, G <sub>f</sub>		$\label{eq:resonance} \begin{array}{ c c c } \hline From \ Table \ 6-2 \\ \hline c = 0.30 & z_{min} = 30 \ ft \\ \hline \overline{\alpha} = 1/4 & \overline{\epsilon} = 1/3 \\ \hline \overline{b} = 0.45 & \ell = 320 \\ \alpha = 7.0 & z_g = 1200 \end{array}$	ft ) ft			Section 6.5	.8.2 (p. 30)
				$\begin{array}{l} g_{O} = g_{V} = \ 3.4 \\ g_{R} = \ 4.051 \\ \overline{z} = \ 70.2 \ ft \\ I \ \overline{z} = \ 0.265 \\ \overline{V} \ \overline{z} = \ 67.75 \ ft/se \\ L \ \overline{z} = \ 411.55 \end{array}$	с			Section 6.5 Equation 6 Section 6.5 Equation 6 Equation 6 Equation 6	.8.2 9 .8.1 -5 -14 -7
				$\label{eq:gamma_linear} \begin{split} \hline Equation \ 6-13 \\ \hline \eta_h = \ 4.488 & R_h = \ 0.19 \\ \eta_B = \ 5.754 & R_B = \ 0.15 \\ \eta_L = \ 5.754 & R_L = \ 0.15 \end{split}$	8 9 9				
			Assumed Critical Dam Resonant Respo Background	$N_1 = 3.432$ $R_n = 0.064$ aping Ratio, $\beta = 0.05$ nse Factor, R = 0.156 Response, Q = 0.822				Equation 6 Equation 6 Equation 6 Equation 6	12 .11 .10 .6
			Gust Effe	ect Factor, G <sub>f</sub> = 0.837				Equation 6	-8
			Gust Effe	ect Factor, G <sub>1</sub> = 0.837				Equation 6	-8

GG Englin	neering	JOB NC	). 9-ST(	ORY BL	IILDING	àS				BY	SMG		DAT	E 9/	/16/04	SHE	ET NO.
	neernng)	CUSTO	MER R	CFT PA	RAME	TRIC ST	UDY			CKD			DAT	E			OF
SUBJECT		2	-D MO	MENT	FRAM	NE (MF	A2 -	F2] /	ANAI	YSI	S LO	AD S	UMN	IARY	,		
[15] WI	ND LOADS CC	ONTINUED															
0	WALL EXTER	NAL PRES	SURE COE	FFICIENTS,	Cp	SURFAC	E	L/B	(	C <sub>p</sub>					Section 6.5	.11.2.1	(p. 31)
	Plus sians sian	ifv pressures	actina towards	the surface.	v	VINDWARD	WALL LS	ALL 1.00	0 -(	0.8 0.5					Figure 6-6		(p. 51)
Ν	legative signs sign	nify pressures	acting away f	rom the surfac	e.	LEEWARD V	VALL	ALL	-(	0.7							
0	ROOF EXTER	NAL PRES	SURE COE	FFICIENT, (	₽ <sub>p</sub>		FOR:	θ < 10°		]					Section 6.5	.11.2.1	(p. 31)
		h = 117.0 L = 150.0	) ft ) ft			DISTA LEAI	NCE FR	IOM GE	$C_p$						Figure 6-6		(p. 51)
	h	i/L = 0.780 i/2 = 58.5	) (Assume ft	h/L > 1.0 )		0 t	oh/2 h/2		-1.3 -0.7								
o	INTERNAL PF	RESSURE C	OEFFICIEN	T, GC <sub>pi</sub>			GC <sub>pi</sub> =	± 0.18							Section 6.5 Figure 6-5	.11.1	(p. 31) (p. 49)
	FLOOR LEVE	L WIND LOAD	S PER EQUA	TION 6-19 AN	D PARAPET	WIND LOADS	PER EQU	ATION 6-2	0		WINDWA	RD WALL	LEEWAI	RD WALL	TOTAL WI	ND SHEAR	DESIGN
FLOOR NUMBER	ELEVATION	(B				ERS	Kz	q <sub>z</sub> (psf)	K <sub>h</sub>	q <sub>h</sub> (psf)	PRESSU W	JRE (psf) ITH	PRESSI W	URE (psf) ITH	W		SHEAR
PENTHOUSE		130.0 ft	117.0 ft	13.0 ft	90.0 ft	1,170 ft <sup>2</sup>	1.07	16.82	1.03	16.19	(+GC <sub>pi</sub> ) 10.0	(-GC <sub>pi</sub> ) 14.18	(+GC <sub>pi</sub> ) -12.4	(-GC <sub>pi</sub> )	(+GC <sub>pi</sub> ) 26.2 kips	(-GC <sub>pi</sub> ) 28.3 kips	FLOOR 28.3 kips
PARAPET		120.5 ft	117.0 ft	3.5 ft	150.0 ft	525 ft <sup>2</sup>	1.04	16.35	1.03	16.19	29	.43	-15	7.99	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 ft <sup>2</sup>	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 ft <sup>2</sup>	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 ft <sup>2</sup>	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
5	65.0 π 52.0 ft	71.5 π 58.5 ft	58.5 π 45.5 ft	13.0 π 13.0 ft	150.0 π 150.0 ft	1,950 π <sup>2</sup>	0.90	13.36	1.03	16.19	10.0	12.39	-12.4	-10.0	43.7 kips	43.7 Kips 42.6 kips	43.7 kips
4	39.0 ft	45.5 ft	32.5 ft	13.0 ft	150.0 ft	1,950 ft <sup>2</sup>	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 ft <sup>2</sup>	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 ft <sup>2</sup>	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	
L																	

									-						
ශීශී විකර	alineerina	JOB NO	). 9-ST(	ORY BU	IILDING	S			BY	SMG	D	ATE 9,	/16/04	SHE	ET NO.
00 Ling	Sunnesen unde	CUSTO	MER R	CFT PA	RAMET	RIC ST	UDY		CKD		D	ATE			OF
SUBJECT	-	2.	-D MO	MENT	FRAM	IE [MF	A2 - F	2] AN	ALYSI	S LOA	DSUN	IMARY	,		
[ 16 ]	WIND LOADS C	CONTINUED													
	• BUIDLING W	IDTH (DIME)	NSION PER	PENDICULA	AR TO WINE	DIRECTIO	N), B	150.0 ft	ALONG TH	IE E-W FAC	E				
	• ECCENTRIC	ITY ALONG	THE WINDV	ARD FACE	OF THE BU	JILDING, e <sub>x</sub>	ł	± 22.5 ft					Figure 6-9		(p. 54)
	• BUIDLING D	EPTH (DIMEI	NSION PAR	ALLEL TO V		CTION), L		150.0 ft	ALONG TH	IE N-S FACI	Ē				
	• ECCENTRIC	ITY ALONG	THE SIDEW	ALL OF THE	E BUILDING	, e <sub>y</sub>	ŧ	± 22.5 ft					Figure 6-9		(p. 54)
	• ECCENTRIC	ITY FOR FLE	XIBLE STR	UCTURES (	e <sub>x</sub> AND e <sub>y</sub> )		±	39.38 ft					Equation 6-	21	(p. 33)
	<ul> <li>TO SIMPLIF CALCULATE</li> <li>TORSION LC</li> </ul>	Y THE TORS ED ONCE ALC DADS ARE S	IONAL MON ONG ONE F	MENT CALC PRINCIPAL I 60 THAT TH	ULATIONS, DIRECTION IE MAXIMU	THE BUILD AND THEN M SHEAR F	DING PLAN USED IN B PER MOMEN	IS ASSUME OTH PRINC	ED TO BE SO CIPAL DIREC = STORY S	QUARE SO CTIONS. HEAR x ( 1 /	THAT THE	WIND SHEA OMENT FRA	AR LOADS ( AMES + 0.00	CAN BE 02 x ECCEN	ITRICITY )
	<b></b>		ASCE 7	Figure 6-9	CASE #1	ASCE 7	Figure 6-9	CASE #2	ASCE 7	Figure 6-9	CASE #3	ASCE 7	Figure 6-9 (	CASE #4	1
	FLOOR		SHEAR LOA	DS DUE TO:	TOTAL	SHEAR LOA	ADS DUE TO:	TOTAL	SHEAR LOA	ADS DUE TO:	TOTAL	SHEAR LOA	DS DUE TO:	TOTAL	
	NUMBER	ELEVATION	SHEAR	TORSION	STORY	SHEAR	TORSION	STORY	SHEAR	TORSION	STORY	SHEAR	TORSION	STORY	
	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	
	GROUND	0.0 ft	39.0 Kips N.A.	± 0.0 kips N.A.	39.0 Kips N.A.	29.3 kips N.A.	± 2.3 kips N.A.	N.A.	N.A.	± 0.0 kips N.A.	N.A.	N.A.	± 1.7 kips N.A.	22.0 Kips N.A.	

314.5 kips

314.5 kips

236.0 kips

419.0 kips

ලිලි Spatpaartina	JOB NO. 9	-STORY E	BUILDING	ìS		BY	SMG	DAT	E 9/16/04	SHEET NO.
go Tirîniren Irê	CUSTOMER	RCFT F	PARAMET	FRIC STU	DY	CKD	)	DAT	Ē	OF
SUBJECT	2-D I	MOMEN	T FRAN	IE [MF A	2 - F2] /	ANALYS		) SUMN	IARY	
[17] 9-STORY BUILD o SUMMARY o DESIGN A	INGS MOMENT F (: TC NI DI LC DC DD DI DI DI DI DI DI DI DI DI DI DI DI	RAME ANALY: DTAL NUMBER UMBER OF STO UMBER OF BA IRECTION THA' DCATION OF TH OES THIS FRAI ISTANCE TO TH ISTANCE TO TH ISTANCE TO TH ESIGN SEISMIC ASIC WIND SP UILDING HAS * PER WIND DES ATERAL LOAD	SIS LOAD SU OF (IDENTIC, ORIES IN THE YS IN THE MO T THE MOMENT HE MOMENT HE CLOSEST HE CLOSEST C BASE SHEA TED TRIGID DIAPHI SIGNJ, AND AL S ARE DISTR	MMARY AL) MOMENT FRA DMENT FRAME NT FRAME RU FRAME WRT TI I PART OF THE (GRAVITY/MO) (GRAVITY/MO) IR RAGMS" (PER - L OF THE MON IBUTED EQUAL	FRAMES ALON IME, N <sub>S</sub> J, N <sub>B</sub> NS PARALLEL HE BUILDING I PENTHOUSE MENT) FRAME MENT) FRAME SEISMIC DESI MENT FRAMES LLY AMONG A	Ig the e-w d . With: Perimeter: : Gravity Loa : : On the othe GN), the buil S have the s LL of the MC	IRECTION	4 9 STORIES 5 BAYS E-W INTERIOR YES 30.0 ft 30.0 ft 0.060 W 85 mph SIDERED A "S AT EACH FLC S IN EACH PI	IMPLE DIAPHRAGM DOR LEVEL. THERL RINCIPAL DIRECTIC	I" BUILDING EFORE, ALL N OF THE BUILDING.
	NUN	MBER	STORY	FLOOR	DESIGN S (THESE ARE	SHEAR LOAD AT I THE POINT LOA	EACH FLOOR LEV DS THAT SHALL E	VEL PER MOMEN 3E USED IN THE	VT FRAME 2-D MODEL)	
	STORY	FLOOR	HEIGHT	ELEVATION h <sub>x</sub>	SEISMIC LOAD	CASE #1	WIND CASE #2	LOAD CASE #3	CASE #4	
	 9 8 7 6 5 4 3 2 1	 ROOF 9 8 7 6 5 4 3 2 GROUND	 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft	117.0 ft 104.0 ft 91.0 ft 78.0 ft 65.0 ft 52.0 ft 39.0 ft 26.0 ft 13.0 ft 0.0 ft	 95.0 kips 63.3 kips 52.6 kips 42.6 kips 33.1 kips 24.3 kips 16.4 kips 9.4 kips 3.6 kips N.A.	 19.1 kips 11.5 kips 11.3 kips 10.9 kips 10.7 kips 10.4 kips 10.0 kips 9.8 kips N.A.	 17.2 kips 11.4 kips 11.2 kips 10.8 kips 10.5 kips 10.2 kips 9.9 kips 9.6 kips N.A.	 14.3 kips 8.7 kips 8.5 kips 8.2 kips 7.8 kips 7.8 kips 7.3 kips N.A	12.9 kips 8.5 kips 8.4 kips 8.3 kips 7.9 kips 7.6 kips 7.4 kips 7.2 kips 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,  $\theta$ , 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<sub>R</sub> 3. 1.2D + 1.6L<sub>R</sub> + f<sub>1</sub>L 4. 1.2D + 1.6L<sub>R</sub> + 0.8W 5. 1.2D + 1.6W + f<sub>1</sub>L + 0.5L<sub>R</sub> 6. 1.2D + 1.0E + f<sub>1</sub>L Where:  $f_1 = 0.5$  $E = \rho Q_E + 0.2S_{DS}D'$ 

D' = seismic weight

GG Engineering	JOB NO. 9 - STOR	Y BUILDINGS	BY SMG	DATE 9/16/04	SHEET NO.
	CUSTOMER DESI	GN 9A	CKD	DATE	OF
SUBJECT		DESIGN PARAMETERS	SUMMARY		MOMENT FRAME MF A2 - F2
o DESIGN	I INPUTS:	0 TOAL NUMBER OF COLUMNS BEIN	IG ANALYZED	54	
		• YIELD STRENGTH:	H CONCRETE REINFORCEME	SS, F <sub>y</sub> = 46 ksi NT, F <sub>yr</sub> = 0 ksi	
		0 MODULUS OF ELASTICITY:	H CONCRETE REINFORCEME	SS, E <sub>s</sub> = 29,000 ksi NT, E <sub>cr</sub> = 29,000 ksi	
		0 MINIMUM CONCRETE COMPRESS	IVE STRENGTH	f' <sub>c</sub> = 4.0 ksi	
		• CONCRETE DENSITY		w = 145 lb/ft <sup>3</sup>	
		◎ CONCRETE REINFORCEMENT	ARI	EA, $A_{sr} = 0.0 \text{ in}^2$ $I_{srr} = 0.0 \text{ in}^4$ $I_{yyr} = 0.0 \text{ in}^4$ $Z_{srr} = 0.0 \text{ in}^3$ $Z_{yyr} = 0.0 \text{ in}^3$	
		0 RESISTANCE FACTORS	AXIAL COMPRESSI FLEXURAL BENDI	ON, $\phi_{c} = 0.75$ NG, $\phi_{b} = 0.90$	
		<ul> <li>SEISMIC PARAMETERS</li> <li>VE</li> </ul>	REDUNDANCY COEFFICI	ENT, $\rho = 1.00$ $0.2S_{DS} = 0.20$	
	FAC	ORTHOGONAL LOAD FACTOR ALC TOR TO ACCOUNT FOR 5% ACCIDENTAL TOR	NG Y-AXIS OF SHARED COL SION ("SIMPLIFIED APPROA	UMNS = 0.30 CH") = 0.025	

SUBJECT         DESCRIPTION         CALL         DATE         C           SUBJECT         B2 CALCULATION - FOR BENDING ALONG THE X-AXIS OF THE COLUMN         WINNERTTRANE (#4.2.7.8)           .         LOD COMBINITION: 1.20 + 1.60 × 0.001         (L.2.4)           .         LOD COMBINITION: 1.20 + 1.60 × 0.001         (L.2.4.4)           .         LOD COMBINITION: 1.20 + 1.60 × 0.001         (L.2.4.4)           .         MUNICIPI (MERCINIC)         LOD COMBINITION: 1.20 + 1.60 × 0.001           .         MUNICIPI (MERCINIC)         MUNICIPI (MERCINIC)         LOD COMBINITION: 1.20 + 1.60 × 0.001           .         MUNICIPI (MERCINIC)         MUNICIPI (MERCINIC)         DITAL (MERCINIC)         DITAL (MERCINIC)           .         MUNICIPI (MERCINIC)	GG Eng	jineeri	JOB NC	). 9 - STOR	Y BUILI	DINGS			BY	SMG	DATE	9/16/04	S⊦	IEET NO.
• Interpretention         Interpre	SUBJECT		CUSTO	MER DESI	GN 9A - <b>FOR</b>	BENI	DING A	ALONG		-AXIS C	DATE DF THE	COLUM	<u>м</u>	
<table-cell></table-cell>												0020	·	MF A2 - F2
UNIT OF ORGEN INFLUENCE         TOTAL LOVACTORED AXIAL LOAD PER STORY ON ALL COLUMNSE         TOTAL INFACTORED AXIAL DOAD PER STORY ON ALL COLUMNSE	o	LOAD	COMBINATION =	1.2D + 1.6Lr + 0.	8W	( L.C. # 4	)							
LENGIN NUMBER         LEASTIC LCAP         MEASTIC NEERING LCAP         LEASTIC NEERING LCAP         (Inp) T         LEASTIC NEERING LCAP         (Inp) T         LEASTIC NEERING LCAP         (Inp) T         LEASTIC NEERING LCAP         (Inp) T         LEASTIC NEERING LCAP         TOTAL IS         STORY VERTICAL DCAP         TOTAL IS         STORY VERTICAL DCAP         STORY P           9         101         3048         0.31         3.262         0         450         3.118         3.263         0.0         4.34.0         1.022         0.0         4.34.0         1.022           9         13.011         3048         0.351         7.010         3.160         4.50         1.030         0.00         720.0         0.00         4.34.0         1.022           13.011         1.044 kps         0.561         7.075         450         1.034         10.806         0.0         720.0         0.00         13.20.0         1.043           5         13.011         1.146 kps         0.661         12.896         7.875         450         13.320         0.00         720.0         0.00         16.314.0         1.065           2         13.011         1.364 kps         0.511         1.1689         1.2890         1.260         1.289 <td< th=""><th></th><th></th><th>DUE TO FORCE BENDING ALONG T MOMENT FRA</th><th>es that cause The <mark>X-axis</mark> of the Me columns</th><th>TOTAL U STORY OF ("LEANEI</th><th><i>NFACTORE</i> N ALL COLL R" + "NON-L</th><th>D AXIAL L JMNS OF T EANER" C</th><th>OAD PER HE STORY OLUMNS)</th><th>ΤΟΤΑ</th><th>L FACTORED A</th><th>XIAL LOAD,</th><th><math>\Sigma P_u</math>, PER STOP</th><th>łΥ</th><th></th></td<>			DUE TO FORCE BENDING ALONG T MOMENT FRA	es that cause The <mark>X-axis</mark> of the Me columns	TOTAL U STORY OF ("LEANEI	<i>NFACTORE</i> N ALL COLL R" + "NON-L	D AXIAL L JMNS OF T EANER" C	OAD PER HE STORY OLUMNS)	ΤΟΤΑ	L FACTORED A	XIAL LOAD,	$\Sigma P_u$ , PER STOP	łΥ	
Number         Holom L         SetSML U         OPIFT US         DEAD UET 3H, UET 3	07001	LENGTH	SHEAR DUE TO	ELASTIC INTERSTORY		(ki	ps)			LOAD F	ACTOR		TOTAL	STORY B <sub>2i X-AXIS</sub>
9         13.01         390 kps         0.3 in         3.02         0         450         3.18         3.824.0         0.0         720.0         0.0         4.344.0         1.022           8         13.01         633 kps         0.42 in         5.015         1.575         450         5.583         6.016.0         0.0         720.0         0.0         6.738.0         1.039           7         13.01         1.01 kps         0.51 in         7.010         3.150         450         6.068         8.412.0         0.00         720.0         0.0         9.132.0         1.037           6         13.01         1.01 kps         0.6 in         11.000         6.300         450         12.888         13.200.0         0.0         720.0         0.0         13.820.0         1.043           4         13.01         1.244 kps         0.6 in         12.985         7.675         450         10.345         15.984.0         0.0         720.0         0.0         18.760.0         1.075           2         13.01         1.346 kps         0.53 in         14.989         9.450         17.768         17.988.0         0.00         720.0         0.0         21.020         1.056         1.304         <	NUMBER	HEIGHT) L	HORIZONTAL LOAD E ΣH:	DRIFT $\Delta_{oh}$ DUE TO $\Sigma H_i$	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD	SEISMIC WEIGHT DL + P-LL	D.L.	<i>L.L.</i> 0	ROOF L.L.	SEISMIC VERTICAL 0.2S <sub>DS</sub> = 0	ΣP <sub>ui</sub> (kips)	
8         13.01         633.kps         0.42 in         5.05         1.575         450         5.585         6.018.0         0.0         72.00         0.0         9,732.0         1.037           6         13.01         1.014 kps         0.56 in         7,00         3,150         450         10.085.0         0.00         720.0         0.00         9,132.0         1.037           6         13.01         1.014 kps         0.56 in         10.00         1.280         1200.0         0.00         720.0         0.00         132.00         1.04           4         13.01         1.1244 kps         0.8 in         12.95         7,875         450         15.342         15.540         0.00         720.0         0.00         163.00         1.055           2         13.01         1.348 kps         0.51 in         1.999         9,450         450         17.78         1798.0         0.0         720.0         0.00         12.102.0         1.055           2         13.01         1.361 kps         0.51 in         1.989         1.025         450         2.2678         2.2776.0         0.0         720.0         0.0         2.102.0         1.034           1         13.61 kps	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
7       13.0 ft       944 kps       0.51 in       7.01       3.150       450       8.008       8.412.0       0.0       720.0       0.0       11,52.0       1.043         5       13.0 ft       1.014 kps       0.56 in       9.005       4.725       450       10.435       13.086.0       0.00       720.0       0.00       11,52.0       1.043         5       13.0 ft       1.146 kps       0.6 in       12.095       7.75       450       15.54.0       0.00       720.0       0.00       13.92.0       1.049         3       13.0 ft       1.346 kps       0.57 in       14.900       9.450       450       15.782       17.98.0       0.00       720.0       0.00       12.70.0       1.055         2       13.0 ft       1.346 kps       0.53 in       16.855       11.025       450       20.233       20.382.0       0.00       720.0       0.00       12.102.0       1.056         1       13.0 ft       1.346 kps       0.3 in       15.89       12.800       450       22.678       22.776.0       0.00       720.0       0.00       23.46.0       1.034         1       13.0 ft       1.45 kps       0.3 in       15.99       1.260       4	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
6       13.0 tt       1.014 Mps       0.56 in       9.005       4.725       450       10.453       10.806.0       0.0       720.0       0.0       13.201       1.464 Mps         5       13.0 tt       1.146 Mps       0.6 in       11.00       6.300       450       12.898       13.200       0.0       720.0       0.0       13.201       1.049         4       13.0 tt       1.244 Mps       0.6 in       12.995       7.875       450       17.788       0.00       720.0       0.0       16.314.0       1.053         2       13.0 tt       1.346 Mps       0.57 in       14.99       9.50       450       20.233       20.382.0       0.0       72.00       0.0       21.102.0       1.056         1       13.0 tt       1.361 Kps       0.3 in       16.985       11.025       450       22.678       22.776.0       0.0       72.0       0.0       23.486.0       1.034         1       13.0 tt       1.361 Kps       0.3 in       16.980       12.800       450       22.678       22.776.0       0.0       72.0       0.0       23.486.0       1.034         1       13.0 tt       1.361 Kps       0.3 in       16.980       14.90       14.9	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
5       13.0 tt       1.146 kips       0.6 in       12.00       5,200       10.0       720.0       0.0       13.920.0       10.92         4       13.0 tt       1.244 kips       0.6 in       12.995       7.875       450       15.94.0       0.0       720.0       0.0       15.31.0       16.33.0         3       13.0 tt       1.364 kips       0.57 in       14.990       9.450       450       17.88       7.980       0.00       720.0       0.0       18.706.0       10.655         2       3.0 tt       1.364 kips       0.53 in       16.985       11.025       450       20.23       20.00       720.0       0.0       18.706.0       1.055         1       13.0 tt       1.364 kips       0.3 in       18.980       12.600       450       22.378       22.776.0       0.0       720.0       0.0       23.496.0       1.034         1       13.0 tt       1.361 kips       0.3 in       18.980       12.600       450       22.378       22.776.0       0.0       720.0       0.0       23.496.0       1.034         1       13.0 tt       1.361 kips       0.3 in       18.980       12.600       450       2.476.0       1.0.0       1.0.0	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
4       13.0 tt       1.244 kips       0.6 in       12.995       7.875       450       15.340       0.0       720.0       0.0       16.314.0       1.063         3       13.0 tt       1.306 kips       0.57 in       14.990       9.450       450       20.233       20.382.0       0.0       720.0       0.0       18.780.0       1.065         2       13.0 tt       1.346 kips       0.3 in       16.985       11.025       450       20.233       20.382.0       0.0       720.0       0.0       21.102.0       1.066         1       13.0 tt       1.361 kips       0.3 in       18.980       12.600       450       22.678       22.776.0       0.0       720.0       0.0       23.486.0       1.034         1       13.0 tt       1.361 kips       0.3 in       18.980       12.600       450       22.678       22.776.0       0.0       720.0       0.0       23.486.0       1.034         1       13.0 tt       1.361 kips       0.3 in       16.980       12.600       450       22.678       22.776.0       0.0       720.0       0.0       23.486.0       1.034         1       1.91 kips       1.91 kips       1.91 kips       1.91 kips       1.9	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
3       13.0 tl       1,309 kps       0.57 in       14,990       9,450       450       17,788       17,888.0       0.0       720.0       0.0       18,708.0       1.065         2       13.0 tl       1,346 kps       0.53 in       16,685       11,025       450       20,233       20,882.0       0.0       720.0       0.0       21,102.0       1.065         1       13.0 tl       1,361 kps       0.3 in       18,980       12,600       450       22,578       22,776.0       0.0       720.0       0.0       23,496.0       1.034         1       13.0 tl       1,361 kps       0.3 in       18,980       12,600       450       22,678       22,776.0       0.0       720.0       0.0       23,496.0       1.034         1       13.0 tl       1,361 kps       0.3 in       18,980       12,600       450       22,678       22,776.0       0.0       720.0       0.0       23,496.0       1.034         1       13.0 tl       1,361 kps       0.3 in       18,980       14,50       14,50       14,50       14,50       14,50       14,50       14,50       14,50       14,50       14,50       14,50       14,50       14,50       14,50       14,50	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
2       13.0 ft       1.346 kips       0.53 in       16.965       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       13.980       12.600       450       22.678       22.776.0       0.0       720.0       0.0       23.496.0       1.034         1       13.0 ft       1.361 kips       0.3 in       13.980       12.600       450       22.678       22.776.0       0.0       720.0       0.0       23.496.0       1.034         1       1.30 ft       1.361 kips       0.3 in       13.980       12.600       450       22.678       22.776.0       0.0       720.0       0.0       23.496.0       1.034         1       1.4	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
1 13.0 tt 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	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

ଜ	ලි <b>පි</b> බැැ	lmeeri	ଜଣ	JOB NC	). 9 - STOR	Y BUILI	DINGS			BY	SMG	DATE	9/16/04	SH	IEET NO.
ଞ	o canĝ	unnæði i	nn Si	CUSTO	MER DESI	GN 9A				CKD		DATE			OF
รเ	JBJECT		<b>B2</b>	CALC	ULATION	- FOR	BEND	DING A	ALONG	THE Y	(-AXIS <mark>O</mark>	F THE	COLUM	N MC	MENT FRAME MF A2 - F2
	0	LOAD	COMB	INATION =	1.2D + 1.6Lr + 0.	8W									
			BEND M	ING ALONG T MOMENT FRA	HE Y-AXIS OF THE ME COLUMNS	TOTAL U STORY OF ("LEANER	NFACTORE N ALL COLL R" + "NON-L	ED AXIAL L JMNS OF T .EANER" C	OAD PER HE STORY OLUMNS)	TOTA	AL FACTORED A	XIAL LOAD,	ΣP <sub>u</sub> , PER STOF	łΥ	STORY
	STORY	LENGTH (STORY	SHEA S	AR DUE TO EISMIC	INTERSTORY	DEAD	(KI	ROOF	OFIOMIO		LOAD FA	CTOR	SEISMIC	TOTAL	B <sub>2i_Y-AXIS</sub>
	NUMBER	<i>HEIGHT)</i> L	HOF L	RIZONTAL OAD E	$\Delta_{oh}$ DUE TO $\Sigma H_i$	LOAD DL	LIVE LOAD LL	LIVE	WEIGHT DL + P-LL	D.L. 1.2	<i>L.L.</i> 0	1.6	VERTICAL $0.2S_{DS} = 0$	(kips)	
	9	13.0 ft	3	80 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	6	33 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	8	44 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	1,0	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	1,1	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	1,2	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

6	G Eng	jineeri		9 - STOR		DINGS			BY	SMG	DATE	9/16/04	SI	HEET NO. OF
S	JBJECT		CUSTO	STABILI		EFFIC	IENT	ALON		IMN X-A	XIS, θ <sub>x</sub>		<u>м</u>	OMENT FRAME MF A2 - F2
	0 0 0	LOAD C DEFLEC (SEISM MOMEN TOTAL	COMBINATION: CTION AMPLIFICA IC) IMPORTANCE IT FRAME RESIS SEISMIC SHEAR DUE TO FORCE BENDING ALONG T MOMENT FRA TOTAL STORY	ATION FACTOR: FACTOR TS WHAT % OF T TO THE BUIDLIN STHAT CAUSE HE X-AXIS OF THE ME COLUMNS	1.2D + 1.6 C <sub>d</sub> = I <sub>E</sub> = IHE 2 IG? 2 TOTAL U STORY OI ("LEANEI	5.5 1.0 25% NFACTORE N ALL COLL R" + "NON-	D AXIAL L' JMNS OF T LEANER" C	oad Per He Story Olumns)	ТОТА	L FACTORED	AXIAL LOAD, 1	ΣΡ <sub>υ</sub> , PER STO	RY	STABILITY
	STORY NUMBER	LENGTH <i>(STORY</i> <i>HEIGHT)</i> L	SHEAR DUE TO ANY HORIZONTAL LOAD	ELASTIC INTERSTORY DRIFT Δ <sub>oh</sub> DUE TO ΣHi	DEAD LOAD DL	(kij LIVE LOAD LL	SEISMIC WEIGHT DL + P-LL	D.L.	LOAD F	ROOF L.L.	SEISMIC VERTICAL	TOTAL ΣΡ <sub>ui</sub> (kips)	PER STORY θ <sub>i</sub>	
	9 8 7 6 5 4 3 2 1	13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft	2n <sub>1</sub> 380 kips 633 kips 844 kips 1,014 kips 1,244 kips 1,309 kips 1,346 kips 1,361 kips	0.3 in 0.42 in 0.51 in 0.6 in 0.6 in 0.57 in 0.53 in 0.3 in	3,020 5,015 7,010 9,005 11,000 12,995 14,990 16,985 18,980	0 1,575 3,150 4,725 6,300 7,875 9,450 11,025 12,600	Lr 450 450 450 450 450 450 450 450	3,118 5,563 8,008 10,453 12,898 15,342 17,788 20,233 22,678	3,624.0 6,018.0 8,412.0 10,806.0 13,200.0 15,594.0 17,988.0 20,382.0 22,776.0		720.0 720.0 720.0 720.0 720.0 720.0 720.0 720.0 720.0	0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0	4,344.0 6,738.0 9,132.0 11,526.0 13,920.0 16,314.0 18,708.0 21,102.0 23,496.0	0.022 0.029 0.035 0.041 0.047 0.050 0.052 0.053 0.033

GC	) Eng	ineeri	ЈОВ NC	). 9 - STOF	{Y BUILI	DINGS			BY	SMG	DATE	9/16/04	SI	HEET NO.
		,	CUSTO	MER DESI	GN 9A				СКД		DATE			OF
SUE	BJECT			STABIL	ІТҮ СС	)EFFIC	IENT	ALON	G COLU	MN Y-A	XIS, θ <sub>y</sub>		M	OMENT FRAME MF A2 - F2
Г	0 0 0	LOAD C DEFLEC (SEISM MOMEN TOTAL	COMBINATION: CTION AMPLIFIC IC) IMPORTANCI VT FRAME RESIS SEISMIC SHEAR DUE TO FORC BENDING ALONG	ATION FACTOR E FACTOR STS WHAT % OF TO THE BUIDLIN WES THAT CAUSE THE Y-AXIS OF THE	1.2D + 1.6 $C_d =$ $I_E =$ THE VG?	3Lr + 0.8W 5.5 1.0 25%		OAD PER	ТОТА	L FACTORED	AXIAL LOAD,	ΣP <sub>u</sub> , PER STC	PRY	
1	STORY NUMBER	LENGTH <i>(STORY</i> <i>HEIGHT)</i> L	MOMENT FR/ TOTAL STORY SHEAR DUE TC ANY HORIZONTAL LOAD	HAME COLUMNS ELASTIC INTERSTORY DRIFT A <sub>oh</sub>	("LEANER DEAD LOAD	RALL COLU R" + "NON-L (kip LIVE LOAD	ROOF	SEISMIC WEIGHT	D.L.	LOAD F	FACTOR ROOF L.L.	SEISMIC VERTICAL	TOTAL ΣP <sub>ui</sub> (kips)	STABILITY COEFFICIENT PER STORY θ <sub>i</sub>
	9 8 7 6 5 4 3 2 1	13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft	ΣH <sub>i</sub> 380 kips 633 kips 844 kips 1,014 kips 1,244 kips 1,309 kips 1,346 kips 1,361 kips	DUE TO ΣH <sub>i</sub> 0.3 in 0.42 in 0.51 in 0.6 in 0.6 in 0.57 in 0.53 in 0.3 in	DL 3,020 5,015 7,010 9,005 11,000 12,995 14,990 16,985 18,980	LL 0 1,575 3,150 4,725 6,300 7,875 9,450 11,025 12,600	Lor 450 450 450 450 450 450 450 450 450	DL + P-LL 3,118 5,563 8,008 10,453 12,898 15,342 17,788 20,233 22,678 22,678	1.2 3,624.0 6,018.0 8,412.0 10,806.0 13,200.0 15,594.0 17,988.0 20,382.0 22,776.0	0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0	1.6 720.0 720.0 720.0 720.0 720.0 720.0 720.0 720.0 720.0 720.0	0.2Sps = 0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0	4,344.0 6,738.0 9,132.0 11,526.0 13,920.0 16,314.0 18,708.0 21,102.0 23,496.0	0.022 0.029 0.035 0.041 0.047 0.050 0.052 0.053 0.033

CLUE Linguistics unique SUBJECT         CUSTOMER         DESIGN 9A         CKD         DATE         OF           SUBJECT         B2 CALCULATION - FOR BENDING ALONG THE X-AXIS OF THE COLUMN         Moment Para Mark - re           3         LOAD COMMINATION = 120 + 0.54 + 0.51 + 1.601         (LC 4.5)           1         IDMETOTOPICS THAT CARE STORY (STORY SERVIC) SERVICE (STORY SERVIC) (STORY SERVICE ALCOLOR AND COLUMNS)         TOTAL (PACTORED AXIAL LOAD, 5P, PER STORY (STORY SERVICE) (STORY SERVICE ALCOLOR AND COLUMNS)         TOTAL (PACTORED AXIAL LOAD, 5P, PER STORY (STORY SERVICE) (STORY SERVICE ALCOLOR AND COLUMNS)         TOTAL (PACTORED AXIAL LOAD, 5P, PER STORY (STORY SERVICE) (STORY SERVICE) (STORY SERVICE ALCOLOR AND COLUMNS)         TOTAL (PACTORED AXIAL LOAD, 5P, PER STORY (STORY SERVICE) (STORY SERVICE)	ලිශි පිතක්	ineeri	เธงส	JOB NC	). 9 - STOR	Y BUILI	DINGS			BY	SMG	DATE	9/16/04	S⊦	IEET NO.
SUBJECT         B2 CALCULATION - FOR BENDING ALONG THE X-AXIS OF THE COLUMN         MOMENT PARA MPA2 - F2           a         LOAD COMENTATION = 12D + 0.9L + 0.9L + 1.9K         (LC.#5)           a         DOIL TO FORCES THIN CAUGE INVENTION THE ALGORITHM FAUGUSE INVENTION TO ALGORITHM FAUGUS	se miĝi	1010001	nnæl	CUSTO	MER DESI	GN 9A				CKD		DATE		]_	OF
Image: 1	SUBJECT		B2	CALC	ULATION	- FOR	BENL	DING A	ALONG	THE )	(-AXIS (	OF THE	COLUM	N <sup>MC</sup>	MENT FRAME MF A2 - F2
Liewith         Liewith         Liewith         Liewith         Liewith         Story (CAUSING Product)         Story (CAUSING Pr	0	LOAD	СОМВ	UE TO FORCE	<b>1.2D + 0.5L + 0.5</b>	5Lr + 1.6W	( L.C. # 5								
LENGTH NUMBER         LEAR DUE TO L         TITERSTORY L         DEAD DEAD         DIFERSTORY L         DEAD DEAD         LOW Poils LOAD         DEMON UCAD L         DEMON L         DEMON L         DL         LLL         ROOF LL 0.280s = 0         SESSMC VERTICAL D280s = 0         SP (kpp)         Picture (kpp)         DEAD DL         LLL         LLD         LLL         LLL         ROOF LL L         SESSMC D280s = 0         SP (kpp)         Picture (kpp)         Picture (kpp)         DL         LLL         LLL <thll< thr="">         LLL         <thll< th=""> <thll<< td=""><td></td><td></td><td>BENI</td><td>DING ALONG T MOMENT FRA</td><td>HE X-AXIS OF THE ME COLUMNS</td><td>STORY OF ("LEANEI</td><td>N ALL COLL R" + "NON-L</td><td>JMNS OF T EANER" C</td><td>HE STORY</td><td>тоти</td><td>AL FACTORED</td><td>AXIAL LOAD,</td><td>ΣP<sub>u</sub>, PER STO</td><td>łY</td><td>STORY</td></thll<<></thll<></thll<>			BENI	DING ALONG T MOMENT FRA	HE X-AXIS OF THE ME COLUMNS	STORY OF ("LEANEI	N ALL COLL R" + "NON-L	JMNS OF T EANER" C	HE STORY	тоти	AL FACTORED	AXIAL LOAD,	ΣP <sub>u</sub> , PER STO	łY	STORY
9         13.0 ft         390 kps         0.3 in         3.020         0         4.50         3.116         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         5.583         6.018.0         787.5         225.0         0.0         7,030.5         1.031           7         13.0 ft         .044 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         1.14 kips         0.56 in         9.005         4.725         450         10.453         10.800.0         2.362.5         225.0         0.0         13.393.5         1.050           4         13.0 ft         1.24 kips         0.6 in         12.995         7.875         450         15.342         15.594.0         3.337.5         225.0         0.0         12.795.5         1.065           3         13.0 ft         1.306 kps         0.3 in         16.985         11.025         450         20.33         20.382.0         5.512.5         225.0         0.0         29.301.0 <th>STORY NUMBER</th> <th>LENGTH (STORY HEIGHT) L</th> <th>SHE S HO</th> <th>AR DUE TO EISMIC RIZONTAL .OAD E 5H</th> <th>INTERSTORY DRIFT Δ<sub>oh</sub> DUE TO ΣH<sub>i</sub></th> <th>DEAD LOAD DL</th> <th>LIVE LOAD LL</th> <th>ROOF LIVE LOAD</th> <th>SEISMIC WEIGHT DL + P-LL</th> <th>D.L.</th> <th>L.L. 0.5</th> <th>ROOF L.L.</th> <th>SEISMIC VERTICAL 0.2S<sub>DS</sub> = 0</th> <th>TOTAL ΣP<sub>ui</sub> (kips)</th> <th>B<sub>2i_X-AXIS</sub></th>	STORY NUMBER	LENGTH (STORY HEIGHT) L	SHE S HO	AR DUE TO EISMIC RIZONTAL .OAD E 5H	INTERSTORY DRIFT Δ <sub>oh</sub> DUE TO ΣH <sub>i</sub>	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD	SEISMIC WEIGHT DL + P-LL	D.L.	L.L. 0.5	ROOF L.L.	SEISMIC VERTICAL 0.2S <sub>DS</sub> = 0	TOTAL ΣP <sub>ui</sub> (kips)	B <sub>2i_X-AXIS</sub>
	9 8 7 6 5 4 3 2 1	13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft	3 6 8 1, 1, 1, 1, 1, 1,	80 kips 33 kips 444 kips 014 kips 146 kips 244 kips 309 kips 361 kips 361 kips	0.3 in 0.42 in 0.51 in 0.56 in 0.6 in 0.57 in 0.53 in 0.3 in	3,020 5,015 7,010 9,005 11,000 12,995 14,990 16,985 18,980	0 1,575 3,150 4,725 6,300 7,875 9,450 11,025 12,600	450 450 450 450 450 450 450 450	3,118 5,563 8,008 10,453 12,898 15,342 17,788 20,233 22,678	3,624.0 6,018.0 8,412.0 10,806.0 13,200.0 15,594.0 17,988.0 20,382.0 22,776.0	0.0 787.5 1,575.0 2,362.5 3,150.0 3,937.5 4,725.0 5,512.5 6,300.0	225.0 225.0 225.0 225.0 225.0 225.0 225.0 225.0 225.0		3,849.0 7,030.5 10,212.0 13,393.5 16,575.0 19,756.5 22,938.0 26,119.5 29,301.0	1.020 1.031 1.041 1.050 1.065 1.068 1.071 1.043

6	) G Eng	ilneerf	JC	OB NO	. 9 - STOR	Y BUILI	DINGS			BY	SMG	DATE	9/16/04	SH	IEET NO.
			C	USTO	MER DESI	GN 9A				CKD		DATE			OF
S	UBJECT		B2 C/		JLATION	- FOR	BENL	DING A	LONG	THE	(-AXIS	OF THE	COLUM	V MO	OMENT FRAME MF A2 - F2
	0	LOAD	COMBINA	TION =	1.2D + 0.5L + 0.5	iLr + 1.6W									
			DUE T BENDING MOM	O FORCE ALONG T IENT FRAI	S THAT CAUSE HE <mark>Y-AXIS</mark> OF THE ME COLUMNS	TOTAL U STORY ON ("LEANER	N <i>FACTORE</i> NALL COLL R" + "NON-L	D AXIAL LI IMNS OF TI EANER'' C	OAD PER HE STORY OLUMNS)	тотл	AL FACTORED	axial load, :	$\Sigma P_u$ , PER STOF	Y	
	STORY		SHEAR I	DUE TO	ELASTIC INTERSTORY		(ki	os)			LOAD	ACTOR		TOTAL	STORY B <sub>2i_Y-AXIS</sub>
	NUMBER	HEIGHT) L		DNTAL DE	$DRIFT_{\Delta_{oh}}$ DUE TO $\Sigma H_{i}$	DEAD LOAD DL	LIVE LOAD LL	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 <sub>DS</sub> = 0	ΣP <sub>ui</sub> (kips)	
	9	13.0 ft	380 k	tips	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 k	tips	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 k	tips	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

6	iG Eng	ineeri	JOB NC	9 - STOR	Y BUILI	DINGS			BY	SMG	DATE	9/16/04	S	HEET NO.
0			CUSTO	MER DESI	GN 9A				CKD		DATE			
5	UBJEC I			STABILI	тү сс	DEFFIC	IENT	ALON	G COLU	IMN X-A	XIS, θ <sub>x</sub>		М	MF A2 - F2
	0 0 0	LOAD C DEFLEC (SEISM MOMEN TOTAL	COMBINATION: CTION AMPLIFIC, IC) IMPORTANCE IT FRAME RESIS SEISMIC SHEAR	1. ATION FACTOR: FACTOR TS WHAT % OF 1 TO THE BUIDLIN	2D + 0.5L + C <sub>d</sub> = I <sub>E</sub> = THE IG?	0.5Lr + 1.6V 5.5 1.0 25%	w							
			DUE TO FORCE BENDING ALONG T MOMENT FRA	es that cause The <mark>X-axis</mark> of the Me columns	TOTAL U		D AXIAL L	OAD PER HE STORY	ΤΟΤΑ	L FACTORED	AXIAL LOAD,	$\Sigma P_u$ , PER STO	RY	
		LENGTH	TOTAL STORY SHEAR DUE TO	ELASTIC	("LEANE!	R" + "NON-L (kip	OLUMINS)		LOAD F	ACTOR		τοται	COEFFICIENT	
	STORY NUMBER	(STORY HEIGHT) L	ANY HORIZONTAL LOAD ΣH <sub>i</sub>	DRIFT $\Delta_{oh}$ DUE TO $\Sigma H_i$	DEAD LOAD DL	LIVE LOAD LL	SEISMIC WEIGHT DL + P-LL	D.L. 1.2	L.L. 0.5	ROOF L.L. 0.5	SEISMIC VERTICAL 0.2S <sub>DS</sub> = 0	ΣP <sub>ui</sub> (kips)	θ	
	9	13.0 ft	380 kips	0.3 in	3,020	0	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	5,563	6,018.0	787.5	225.0	0.0	7,030.5	0.030	
	7	13.0 ft 13.0 ft	844 kips 1 014 kips	0.51 in 0.56 in	7,010 9.005	3,150 4,725	8,008 10,453	8,412.0 10,806.0	1,575.0 2,362.5	225.0 225.0	0.0	10,212.0	0.040	
	5	13.0 ft	1,146 kips	0.6 in	11,000	6,300	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	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
		13.0 π	1,361 Kips	U.3 IN	18,980	12,000	450	22,0/8	22,176.0	6,300.0	225.0	0.0	29,301.0	0.041

6	G Eng	ineeri	JOB NO	D. 9 - STOF	Y BUILI	DINGS			BY	SMG	DATE	9/16/04	S⊦	IEET NO.
			CUSTO	MER DESI	GN 9A				CKD		DATE			. OF
S	JBJECT		B2 CALC	ULATION	- FOR	BENL	DING A	ALONG	THE X	-AXIS C	F THE	COLUM	V <sup>MC</sup>	MENT FRAME MF A2 - F2
	0	LOAD	COMBINATION =	1.2D + 0.5L + 1.0	θE	( L.C. # 6	)							
			DUE TO FORC BENDING ALONG MOMENT FR	ES THAT CAUSE THE <mark>X-AXIS</mark> OF THE AME COLUMNS	TOTAL U STORY OF ("LEANEI	<i>NFACTORE</i> N ALL COLL R" + "NON-L	ED AXIAL L JMNS OF T EANER" C	OAD PER HE STORY OLUMNS)	TOTA	L FACTORED A	XIAL LOAD,	$\Sigma P_u$ , PER STOF	Y	
	OTODY	LENGTH	SHEAR DUE TO	ELASTIC INTERSTORY		(ki	ps)			LOAD FA	ACTOR		TOTAL	STORY B <sub>2i X-AXIS</sub>
	NUMBER	HEIGHT) L	HORIZONTAL LOAD E ΣH:	DRIFT $\Delta_{oh}$ DUE TO $\Sigma H_i$	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD	SEISMIC WEIGHT DL + P-LL	D.L. 1.2	L.L. 0.5	ROOF L.L. 0	SEISMIC VERTICAL 0.2S <sub>DS</sub> = 0.2	ΣP <sub>ui</sub> (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	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

6	iG Eng	ineeri		). 9 - STOR	Y BUILI	DINGS			BY	SMG	DATE	9/16/04	SH	IEET NO.
			CUSTO	MER DESI	GN 9A				CKD		DATE			OF
S	UBJECT		B2 CALC	ULATION	- FOR	BENL	DING A	ALONG	THE Y	'-AXIS (	OF THE	COLUM	N <sup>MC</sup>	DMENT FRAME MF A2 - F2
	0	LOAD	COMBINATION =	1.2D + 0.5L + 1.0	Ε									
			DUE TO FORCE BENDING ALONG T MOMENT FRA	ES THAT CAUSE THE Y-AXIS OF THE ME COLUMNS	TOTAL U STORY OF ("LEANER	<i>NFACTORE</i> N ALL COLL R" + "NON-L	D AXIAL L JMNS OF T EANER" C	OAD PER HE STORY OLUMNS)	TOTA	AL FACTORED	axial load,	$\Sigma P_u$ , PER STO	ΥY	
	STORY NUMBER	LENGTH (STORY HEIGHT)	SHEAR DUE TO SEISMIC HORIZONTAL	ELASTIC INTERSTORY DRIFT $\Delta_{oh}$	DEAD LOAD	(ki LIVE LOAD	ROOF	SEISMIC	D.L.	LOAD F	ACTOR ROOF L.L.	SEISMIC VERTICAL	TOTAL ΣP <sub>ui</sub> (kips)	STORY B <sub>2i_Y-AXIS</sub>
		L	ΣH <sub>i</sub>	DUE TO $\Sigma H_{\rm i}$	DL	LL	LOAD Lr	DL + P-LL	1.2	0.5	0	$0.2S_{DS} = 0.2$	( F-7	
	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 m	9,005	4,/25	450	10,453	10,806.0	2,362.5	0.0	2,090.6	18 020 4	1.057
	5	13.0 ft	1,140 Kips	0.6 in	12 995	0,300 7 875	400	15 342	15 594 0	3 937 5	0.0	2,579.6	10,929.0 22 599 0	1.075
	3	13.0 ft	1,244 kips	0.57 in	12,995	9.450	450	17 788	17 988 0	4 725 0	0.0	3,008.4	26 270 6	1.075
	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

6	iG Eng	jineeri	JOB NC	). 9 - STOR	Y BUILI	DINGS			BY	SMG	DATE	9/16/04	SI	HEET NO.
s	UBJECT		CUSTO	MER DESI	GN 9A				СКД					
	020201			STABILI		DEFFIC	JENI	ALON	i COLU	IMN X-A	$XIS, \theta_{x}$			MF A2 - F2
	0 0 0	LOAD C DEFLEC (SEISM MOMEN TOTAL	COMBINATION: CTION AMPLIFIC, IC) IMPORTANCE IT FRAME RESIS SEISMIC SHEAR	ATION FACTOR: FACTOR TS WHAT % OF T TO THE BUIDLIN	1.2D + 0. C <sub>d</sub> = I <sub>E</sub> = I <sub>E</sub> = IG?	5L + 1.0E 5.5 1.0 25%								
			DUE TO FORCE BENDING ALONG 1 MOMENT FRA	es that cause The <mark>X-axis</mark> of the Me columns	TOTAL U STORY OI		D AXIAL L	OAD PER HE STORY	ΤΟΤΑ	L FACTORED	axial load, :	ΣP <sub>u</sub> , PER STO	RY	STABILITY
	STORY	LENGTH	TOTAL STORY SHEAR DUE TO	ELASTIC INTERSTORY	( LEANEI	k + NON-L (kij	ps)	OLUMINS)		LOAD F	ACTOR		TOTAL	COEFFICIENT PER STORY
	NUMBER	HEIGHT)		DRIFT Δ <sub>oh</sub> DUE TO ΣH <sub>i</sub>	DEAD LOAD DL	D.L.	L.L. 0.5	ROOF L.L.	SEISMIC VERTICAL 0.2Sps = 0.2	ΣΡ <sub>ui</sub> (kips)	$\theta_i$			
	9	13.0 ft	380 kips	0.3 in	3,020	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	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	10,453	10,806.0	2,362.5	0.0	2,090.6	15,259.1	0.054	
	4	13.0 ft	1,140 kips	0.6 in	12 995	15,200.0	3,150.0	0.0	3 068 4	22 599 9	0.004			
	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

രിരി ജന്തിനുകുന്നത	JOB N	IO. 9 - 3	STORY BUI	LDINGS		BY	SMG	DATE	9/16/04	SHEET NO.
	CUST	OMER	DESIGN 9A	١		СКD		DATE		OF
SUBJECT		ST	ABILITY C	OEFFICI	ENT ALON		JMN X-A	XIS, θ <sub>x</sub>		MOMENT FRAME MF A2 - F2
<ul> <li>LOAD COMBIN</li> <li>DEFLECTION</li> </ul>	INATION: I AMPLIFI	ICATION FA	1.2D + <b>ACTOR:</b> C <sub>d</sub>	0.5L + 1.0E = 5.5						
S	STORY UMBER	LENGTH (STORY HEIGHT) L	DUE TO FORCE BENDING ALONG T MOMENT FRAI TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E	S THAT CAUSE HE X-AXIS OF THE ME COLUMNS INTERSTORY DRIFT Δ <sub>oh</sub> DUE TO ΣH <sub>i</sub>	TOTAL STORY SHEAR CAPACITY (OF ALL OF THE SEISMIC RESISTING MOMENT	RATIO OF SHEAR DEMAND / SHEAR CAPACITY PER STORY β	MAXIMUM ALLOWED STABILITY COEFFICIENT PER STORY θ <sub>i_max</sub>	STABILITY COEFFICIENT PER STORY θ <sub>i</sub>	COMMENT	
	9 8 7 6 5 4 3 2 1	13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft	<u>Σ</u> H <sub>i</sub> 380 kips 633 kips 844 kips 1,014 kips 1,244 kips 1,346 kips 1,346 kips 1,361 kips	0.3 in 0.42 in 0.51 in 0.56 in 0.6 in 0.57 in 0.53 in 0.3 in	11,029 kips 11,029 kips 13,507 kips 12,221 kips 14,997 kips 14,997 kips 14,997 kips 14,997 kips 14,997 kips	0.0345 0.0574 0.0625 0.0830 0.0938 0.0829 0.0873 0.0898 0.0908	0.250 0.250 0.250 0.250 0.250 0.250 0.250 0.250	0.021 0.034 0.045 0.054 0.070 0.073 0.076 0.047	ОК ОК ОК ОК ОК ОК	

60	) Eng	jineeri	JOB NC	). 9 - STOR	Y BUILI	DINGS			BY	SMG	DATE	9/16/04	SI	HEET NO.
SUE	BJECT		CUSTO	MER DESI	GN 9A	EFFIC		ALON		IMN Y-A	DATE XIS, θ <sub>v</sub>			OMENT FRAME
	0 0 0	LOAD C DEFLE( (SEISM MOMEN TOTAL	COMBINATION: CTION AMPLIFIC, IC) IMPORTANCE IT FRAME RESIS SEISMIC SHEAR DUE TO FORCE BENDING ALONG T MOMENT FRA	ATION FACTOR: FACTOR TS WHAT % OF T TO THE BUIDLIN TO THE BUIDLIN STHAT CAUSE HE Y-AXIS OF THE ME COLUMNS	1.2D + 0. C <sub>d</sub> = I <sub>E</sub> = IG? 2 TOTAL U STORY OF ("LEANE"	5L + 1.0E 5.5 1.0 25% NALL COLU R" + "NON-L	D AXIAL LI MNS OF T EANER" C	OAD PER HE STORY OLUMNS)	ТОТА	L FACTORED	AXIAL LOAD,	ερ <sub>υ</sub> , per sto	RY	STABILITY
LENGTH STORY NUMBERSHEAR DUE TO (STORY LSHEAR DUE TO ANY HCIGHT) LELASTIC INTERSTORY DRIFT DANY HORIZONTAL LOAD DLELASTIC (kips)Image: Complex c														$\begin{array}{c} \text{COEFFICIENT} \\ \text{PER STORY} \\ \theta_i \end{array}$
	9 8 7 6 5 4 3 2 1	13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft	380 kips 633 kips 844 kips 1,014 kips 1,244 kips 1,309 kips 1,346 kips 1,361 kips	0.3 in 0.42 in 0.51 in 0.6 in 0.6 in 0.57 in 0.53 in 0.3 in	3,020 5,015 7,010 9,005 11,000 12,995 14,990 16,985 18,980	0 1,575 3,150 4,725 6,300 7,875 9,450 11,025 12,600	450 450 450 450 450 450 450 450	3,118 5,563 8,008 10,453 12,898 15,342 17,788 20,233 22,678	3,624.0 6,018.0 8,412.0 10,806.0 13,200.0 15,594.0 17,988.0 20,382.0 22,776.0	0.0 787.5 1,575.0 2,362.5 3,150.0 3,937.5 4,725.0 5,512.5 6,300.0		623.6 1,112.6 1,601.6 2,579.6 3,068.4 3,557.6 4,046.6 4,535.6	4,247.6 7,918.1 11,588.6 15,259.1 18,929.6 22,599.9 26,270.6 29,941.1 33,611.6	0.021 0.034 0.045 0.054 0.064 0.070 0.073 0.076 0.047

ශිශි පිතත්තලමාන්තන	JOBN	NO. 9 -	STORY BUI	LDINGS		BY	SMG	DATE	9/16/04	SHEET NO.		
ee cinginneerning	CUST	OMER	DESIGN 9A	١		CKD		DATE		OF		
SUBJECT	1	ST	ABILITY C	OEFFICI	ENT ALON		JMN Y-A	XIS, θ <sub>y</sub>		MOMENT FRAME MF A2 - F2		
• LOAD COME	BINATION:	:	1.2D +	0.5L + 1.0E								
o <b>DEFLECTIOI</b>	N AMPLIF	ICATION F	ACTOR: C <sub>d</sub>	= 5.5								
	DUE TO FORCES THAT CAUSE BENDING ALONG THE Y-AXIS OF THE MOMENT FRAME COLUMNS CAPACITY TOTAL STORY I CAPACITY DEMAND / ALLOWED STABILITY											
٢	STORY NUMBER	LENGTH (STORY HEIGHT) L	TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH <sub>i</sub>	$\begin{array}{c} \text{INTERSTORY} \\ \text{DRIFT} \\ \Delta_{\text{oh}} \\ \text{DUE TO } \Sigma\text{H}_{\text{i}} \end{array}$	(OF ALL OF THE SEISMIC RESISTING MOMENT FRAMES)	DEMAND / SHEAR CAPACITY PER STORY β	STABILITY COEFFICIENT PER STORY θ <sub>i_max</sub>	COEFFICIENT PER STORY θ <sub>i</sub>	COMMENT			
	9	13.0 ft	380 kips	0.3 in	11,029 kips	0.0345	0.250	0.021	ОК			
	8 7	13.0 ft	633 kips 844 kips	0.42 in 0.51 in	11,029 kips	0.0574	0.250	0.034	OK OK			
	, 6	13.0 ft	1,014 kips	0.56 in	12,221 kips	0.0830	0.250	0.054	ок			
	5	13.0 ft	1,146 kips	0.6 in	12,221 kips	0.0938	0.250	0.064	ок			
	4	13.0 ft	1,244 kips	0.6 in	14,997 kips	0.0829	0.250	0.070	ОК			
	3	13.0 ft	1,309 kips	0.57 in	14,997 kips	0.0873	0.250	0.073	OK			
	2	13.0 π 13.0 ft	1,346 kips 1,361 kips	0.53 in 0.3 in	14,997 kips 14,997 kips	0.0898	0.250	0.076	ок			

	COLUMN	MAXIMUM	CONTROLLING	BUILDING MAX.	COMMENTS FO	R: DESIGN 9A
NAME	MEMBER SIZE	INTERACTION VALUE	LOAD COMBINATION	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!< td=""><td>OK - STEEL AREA IS &gt; 1% OF TOTAL AREA</td><td>OK - STEEL HSS IS COMPACT</td></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	0.457/23031	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	HOO 22 X 22 X U.625	0.40000010	Ь		OK STEEL AREA IS > 1% OF IUTAL AREA	OK STEEL HOSIS COMPACT
F2-3	HOO 22 X 22 X U.625	0.404002813	Ь		OK STEEL AREA IS > 1% OF IUTAL AREA	OK STEEL HOSIS COMPACT
F2-4	HSS 22 X 22 X 0.625	0.405054/19	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
F2-5	HSS 22 X 22 X U.5	0.46194958/	ь		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
F2-6	HSS 22 X 22 X U.5	0.43764115	6		OK - STEEL AHEA IS > 1% OF TOTAL AHEA	OK - STEEL HSS IS COMPACT
F2-/	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	ь		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
F2-9	HOO 20 X 20 X 0.5	0.418925396	ь		UN - STEEL AMEA IS > 1% UP TUTAL AREA	UN - 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,  $\theta$ , 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:

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

	JOB NO. 9 -	STORY BUILDINGS	BY	SMG	DATE 9/16/04	SHEET NO.
go gugunganng	CUSTOMER	DESIGN 9B	СКД		DATE	OF
SUBJECT		DESIGN PARAMET	ERS SUM	MARY		MOMENT FRAME MF A2 - F2
o DESIGN	I INPUTS:	o TOAL NUMBER OF COLU	MNS BEING ANALY	ZED	54	
		• YIELD STRENGTH:	CONCRE	TE REINFORCE	HSS, F <sub>y</sub> = 80 ksi MENT, F <sub>yr</sub> = 0 ksi	
		MODULUS OF ELASTICIT	Y: CONCRE		HSS, E <sub>s</sub> = 29,000 ksi MENT, E <sub>cr</sub> = 29,000 ksi	
		MINIMUM CONCRETE CO	MPRESSIVE STREI	NGTH	f' <sub>c</sub> = 16.0 ksi	
		o CONCRETE DENSITY			w = 145 lb/ft <sup>3</sup>	
		o CONCRETE REINFORCEN	MENT	,	AREA, $A_{sr} = 0.0 \text{ in}^2$ $I_{sor} = 0.0 \text{ in}^4$ $I_{yyr} = 0.0 \text{ in}^4$ $Z_{or} = 0.0 \text{ in}^3$ $Z_{yyr} = 0.0 \text{ in}^3$	
		0 RESISTANCE FACTORS		AXIAL COMPRE	$SSION, \phi_c = 0.75$ $NDING, \phi_b = 0.90$	
		SEISMIC PARAMETERS		NDANCY COEFF EISMIC "FACTOF	icient, $ρ = 1.00$ R, " 0.2S <sub>DS</sub> = 0.20	
		OR IHOGONAL LOAD FAC FACTOR TO ACCOUNT FOR 5% ACCIDEN	TOR ALONG Y-AXI ITAL TORSION ("SII	S OF SHARED C	OLUMNS = 0.30 OACH'') = 0.025	

G	G Eng	ineeri	JOB NC	). 9 - STOR	Y BUILI	DINGS			BY	SMG	DATE	DATE 9/16/04		SHEET NO.		
			CUSTO	MER DESI	IGN 9B				CKD		DATE	DATE		OF		
รเ	JBJECT		B2 CALC	ULATION	- FOR	BEND	DING A	ALONG	THE X	-AXIS <mark>O</mark>	F THE	COLUM	V <sup>MC</sup>	MENT FRAME MF A2 - F2		
	0	LOAD	COMBINATION =	1.2D + 1.6Lr + 0.	8W	( L.C. #4	)									
			DUE TO FORCE BENDING ALONG MOMENT FRA	D FORCES THAT CAUSE ALONG THE X-AXIS OF THE ENT FRAME COLUMNS ("LEANER" + "NON-LEANER" COLUMNS)			TOTAL FACTORED AXIAL LOAD, $\Sigma P_u, PER$ STORY				ïY					
		LENGTH	SHEAR DUE TO	DUE TO INTERSTORY		DUE TO		(ki	os)	,		LOAD FA	CTOR		TOTAL	STORY B21 X-AXIS
	NUMBER	(STORY HEIGHT) L	HORIZONTAL LOAD E ΣH <sub>i</sub>	$\begin{array}{c} DRIFT \\ \Delta_{oh} \\ DUE \ TO \ \SigmaH_{i} \end{array}$	DEAD LIVE ROOF SEISMIC LOAD LOAD LOAD LOAD UAD DL + P-LL LL Lr			D.L. L.L. ROOF L.L. SEISM VERTIC 1.2 0 1.6 0.2S <sub>DS</sub> =			$SEISMIC$ $VERTICAL$ $0.2S_{DS} = 0$	ΣP <sub>ui</sub> (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	0.07 in	10,985	10,025	450	20,233	20,382.0	0.0	720.0	0.0	21,102.0	1.062		

G	G Ema	ineeri	JOB NO	). 9 - STOF	RY BUILI	DINGS			BY	SMG	DATE	9/16/04	SF	IEET NO.
•	~ —@		CUSTO	MER DESI	GN 9B				CKD		DATE			OF
รเ	JBJECT		B2 CALC	ULATION	- FOR	BENL	DING A	ALONG	THE Y	(-AXIS <mark>O</mark>	F THE	COLUM	N MC	MENT FRAME MF A2 - F2
	0	LOAD	COMBINATION =	1.2D + 1.6Lr + 0	.8W									
			DUE TO FORCI BENDING ALONG MOMENT FRA	es that cause The <mark>Y-AXIS</mark> of the Me columns	TOTAL UNFACTORED AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY				тоти	TOTAL FACTORED AXIAL LOAD, $\SigmaP_u$ , PER STORY				
		LENGTH	TOTAL STORY SHEAR DUE TO	L STORY R DUE TO		(ki	ps)	OLUMINS)		LOAD FA	CTOR		TOTAL	
	STORY NUMBER	(STORY HEIGHT) L	SEISMIC HORIZONTAL LOAD E ΣH;	$\begin{array}{c} DRIFT \\ \Delta_{oh} \\ DUE \ TO \ \SigmaH_{i} \end{array}$	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	D.L. 1.2	<i>L.L.</i> 0	ROOF L.L.	SEISMIC VERTICAL 0.2S <sub>DS</sub> = 0	ΣP <sub>ui</sub> (kips)	21_1-4415
	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 7,875	450	12,898	13,200.0	0.0	720.0	0.0	16 314 0	1.047
	4	13.0 ft	1,244 kips	0.59 in	12,995	9 450	450	15,342	17 988 0	0.0	720.0	0.0	18,314.0	1.052
	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

JINGER . LOAD ( . DEFLE		D. 9 - STOF	RY BUILI GN 9B ITY CC	DINGS DEFFIC	CIENT	ALON	BY CKD G COLU	SMG IMN X-A	DATE DATE	9/16/04	SI	
		MER DESI	GN 9B ITY CC	DEFFIC	CIENT	ALON	скр G COLU	IMN X-A	DATE			
o LOAD C	OMBINATION:	STABIL		DEFFIC	IENT	ALON	G COLU	IMN X-A	XIS. θ.		м	
o <b>LOAD (</b> o <b>DEFLE</b> )	OMBINATION:								, · · <b>x</b>			MF A2 - F2
o (SEISM O MOMEI TOTAL	CTION AMPLIFIC, IC) IMPORTANCE IT FRAME RESIS SEISMIC SHEAR	ATION FACTOR: E FACTOR TS WHAT % OF T TO THE BUIDLIN	1.2D + 1.6 C <sub>d</sub> = I <sub>E</sub> = THE VG?	SLr + 0.8W 5.5 1.0 25%								
•	DUE TO FORCE BENDING ALONG T MOMENT FRA	ES THAT CAUSE THE <mark>X-AXIS</mark> OF THE ME COLUMNS	TOTAL U STORY OF	NFACTORE N ALL COLU R" + "NON-L	D AXIAL L JMNS OF T EANER" C	OAD PER HE STORY OLUMNS)	TOTA	L FACTORED	axial load,	$\Sigma P_u$ , PER STC	PRY	STABILITY
	TOTAL STORY	ELASTIC	( -=/	(kip	ps)			LOAD F	ACTOR			COEFFICIENT
(STORY R HEIGHT) L	ANY HORIZONTAL LOAD		DEAD LOAD	LIVE LOAD	ROOF LIVE LOAD	SEISMIC WEIGHT	D.L.	L.L.	ROOF L.L.	SEISMIC VERTICAL	TOTAL ΣP <sub>ui</sub> (kips)	PER STORY θ <sub>i</sub>
_	ΣH <sub>i</sub>	00210211	DL	LL	Lr		1.2	U	1.6	0.23 <sub>DS</sub> = 0		
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
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
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
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
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
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
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
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
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
	LENGTH (STORY HEIGHT) L 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft	Due to FORCI BENDING ALONG MOMENT FRA           TOTAL STORY LENGTH HEIGHT)         SHEAR DUE TO ANY HORZONTAL LOAD ΣHi           13.0 ft         380 kips           13.0 ft         380 kips           13.0 ft         1,014 kips           13.0 ft         1,146 kips           13.0 ft         1,244 kips           13.0 ft         1,309 kips           13.0 ft         1,346 kips           13.0 ft         1,361 kips	DUE TO FORCES THAT CAUSE BENDING ALONG THE X-AXIS OF THE MOMENT FRAME COLUMNS           LENGTH (STORY L         TOTAL STORY SHEAR DUE TO ANY HORIZONTAL LOAD ΣH,         ELASTIC INTERSTORY DRIFT 4 0,00 2 H,           13.0 ft         380 kips         0.32 in           13.0 ft         380 kips         0.32 in           13.0 ft         633 kips         0.56 in           13.0 ft         1,014 kips         0.56 in           13.0 ft         1,146 kips         0.58 in           13.0 ft         1,244 kips         0.59 in           13.0 ft         1,309 kips         0.59 in           13.0 ft         1,361 kips         0.37 in	DUE TO FORCES THAT CAUSE BENDING ALONG THE XAXIS OF THE MOMENT FRAME COLUMNS         TOTAL U STORY OF (LENGTH I HEIGHT)         TOTAL STORY HARZONTAL LOAD         TOTAL STORY I INTERSTORY DRIFT         TOTAL U STORY OF (LENGTH HORIZONTAL LOAD         TOTAL U STORY OF DRIFT         TOTAL U STORY OF I Asn DUE TO 2H,         TOTAL U STORY OF I INTERSTORY DRIFT           13.0 ft         380 kips         0.32 in         3,020           13.0 ft         633 kips         0.5 in         5,015           13.0 ft         1,014 kips         0.58 in         7,010           13.0 ft         1,014 kips         0.58 in         11,000           13.0 ft         1,244 kips         0.59 in         12,995           13.0 ft         1,309 kips         0.59 in         14,990           13.0 ft         1,361 kips         0.37 in         18,980	Due to FORCES THAT CAUSE BEINDING ALLONG THE X-AKIS OF THE MOMENT FRAME COLUMNS         TOTAL UNFACTORE STORY ON ALL COLU (LENGTH)           LENGTH (STORY) L         TOTAL STORY ANY HEIGHT) L         ELASTIC INTERSTORY DALY LOAD         ELASTIC INTERSTORY DRIFT An         DEAD LOAD         LIVE LOAD           13.0 ft         380 kips         0.32 in         3,020         0           13.0 ft         633 kips         0.5 in         5,015         1,575           13.0 ft         1,014 kips         0.56 in         9,005         4,725           13.0 ft         1,244 kips         0.59 in         12,995         7,875           13.0 ft         1,309 kips         0.59 in         12,995         7,875           13.0 ft         1,346 kips         0.58 in         16,985         11,025           13.0 ft         1,361 kips         0.58 in         16,985         11,025           13.0 ft         1,361 kips         0.37 in         18,980         12,600	DUE TO FORCES THAT CAUSE BENDING ALONG THE XAXIS OF THE MOMENT FRAME COLUMNS         TOTAL UNFACTORED AXIAL L STORY ON ALL COLUMNS OF T ("LEARTH", HEIGHT") LENGTH (STORY HEIGHT) L         TOTAL STORY SHEAR DUE TO ANY HORIZONTAL LOAD 2H,         TOTAL STORY INTERSTORY DIFT DUE TO 2H,         TOTAL UNFACTORED AXIAL L STORY ON ALL COLUMNS OF T ("LEANER" + "NON-LEANER" C (kips)           13.0 ft         380 kips         0.32 in DUE TO 2H,         DE LOAD DL         LIVE LOAD LOAD         UVE LOAD           13.0 ft         633 kips         0.32 in         3,020         0         450           13.0 ft         633 kips         0.5 in         5,015         1,575         450           13.0 ft         1,014 kips         0.56 in         9,005         4,725         450           13.0 ft         1,244 kips         0.59 in         12,995         7,875         450           13.0 ft         1,306 kips         0.58 in         16,985         11,025         450           13.0 ft         1,361 kips         0.37 in         18,980         12,600         450	DUE TO FORCES THAT CAUSE BENDING ALONG THE X-MIS OF THE MOMENT FRAME COLUMNS         TOTAL XINFACTORED AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY OF THE STORY INTERSTORY I	Due to forces that cause behavior along the X-ANS or the Moment France Columns of the X and Y (STORY)         TOTAL UNFACTORED AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY (LEANER" + "NON-LEANER" COLUMNS)         TOTA           LENGTHI (STORY)         TOTAL STORY ANY LOAD 2H, UCAD 13.0 ft         Stelar DUE TO ANY DUE TO 2H, DUE TO 2H, D	Due to proces that CANSE BERNING ALONG THE XANS OF THE MOMENT FRAME COLUMNS         TOTAL LANFACTORED AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY (LEANER* + 'NON-LEANER' COLUMNS)         TOTAL FACTORED (kips)           LENGTH (STORY (JEANER* ) (JEANER* + 'NON-LEANER' COLUMNS)         LOAD F (LEANER* + 'NON-LEANER' COLUMNS)         LOAD F (LEANER* + 'NON-LEANER' COLUMNS)         LOAD F (LEANER* + 'NON-LEANER' COLUMNS)           13.0 ft         ANV LOAD         LOAD D DUE TO 2H, 2H, DUE TO 2H, DUE TO	EDLE TO FORCES TWI CAUSE MOMENT PRAVE COLUMNS         TOTAL LIVE/ACTORED AXIAL LOAD PER STORY ON ALL COLUMNS OF THE STORY (LEANETT, STORY ON ALL COLUMNS)         TOTAL FACTORED AXIAL LOAD, (LEANET, STORY ON ALL COLUMNS)           LENGT (STORY LUCAD DATE 2H 2H 310 ft         ELASTIC DATE 2H 2H 30 ft         ELASTIC (LOAD DATESTORY DRIFT	BULT DE FORCES TWI CAUSE BONDING JUNCH THE AWAS OF THE MOMENT PRAME COLLIMS INTERNATION OF THE STORY INC. STORY ON ALL COLLIMNS OF THE STORY ULB.REF * NON-LEARNET COLLIMNS (K/ps)         TOTAL LANGACORED AXIAL LOAD, PP., PER STO (LAR.PF * NON-LEARNET COLLIMNS)           LENOTT INTERNATION (STORY HEIGHT LOAD DH DIE DIA DU DIE DA DIE DIA DIA DIE DIA DIA DIA DIA DIA DIA DIA DIA DIA DIA	IDENT TORUSE THAT CAUSE MOMENT FRAME COLUMNS OF THE STORY ON ALL COLUMNS OF THE STORY

ලි (විත අ	เทออฑ์	JOB NC	). 9 - STOF	Y BUILI	DINGS			BY	SMG	DATE	9/16/04	S	HEET NO.	
9 Dug		CUSTO	MER DESI	GN 9B				CKD		DATE			OF	
BJECT			STABIL		DEFFIC	CIENT	ALON	G COLU	MN Y-A	XIS, θ <sub>y</sub>		м	OMENT FRAM MF A2 - F2	
0 0 0	LOAD C DEFLEC (SEISMI MOMEN TOTAL S	OMBINATION: CTION AMPLIFIC: C) IMPORTANCE IT FRAME RESIS SEISMIC SHEAR DUE TO FORCE BENDING ALONG T MOMENT FRA TOTAL STORY SHEAR DUE TO	ATION FACTOR: E FACTOR TS WHAT % OF TO THE BUIDLIN ES THAT CAUSE THE Y-AXIS OF THE ME COLUMNS ELASTIC LINTERSTORY	1.2D + 1.6 C <sub>d</sub> = I <sub>E</sub> = IHE IG?	5.5 1.0 25% NALCOLL R" + "NON-L (ki	ED AXIAL L JMNS OF T LEANER" C ps)	oad Per He Story Olumns)	TOTAI	- FACTORED A	XXIAL LOAD,	ΣP <sub>u</sub> , PER STC	RY	STABILITY COEFFICIEN PER STORY	
	(STORY		DRIFT	DEAD	LIVE	ROOF	SEISMIC	D.L.	L.L.	ROOF L.L.	SEISMIC	τΟΤΑL ΣΡ <sub>ui</sub>		
NUNDER	L	LOAD	Δ <sub>oh</sub> DUE TO ΣΗ		LOAD	LIVE	WEIGHT	10	0	16	VERTICAL	(kips)		
	10.04	ΣH <sub>i</sub>	0.00	2 000	~	Lr	2 1 1 0	1.2	U A A	700.0	0.20DS = U	4 0 4 4 0	0.000	
9	13.0 ft	380 Kips	0.32 in	3,020	1 575	450	3,118	3,624.0	0.0	720.0	0.0	4,344.0	0.023	
8 7	13.0 II	033 KIPS	0.5 IN	5,015	1,5/5	450	5,563	0,018.0	0.0	720.0	0.0	0,738.0	0.034	
/ 6	13.0 IT	044 KIPS	0.58 in	9.005	3,150	450	0,008	0,412.0	0.0	720.0	0.0	9,132.0	0.040	
5	12.0 ft	1,014 kips	0.50 in	9,005	4,725	450	10,455	12 200 0	0.0	720.0	0.0	12,020.0	0.041	
4	13.0 ft	1,140 kips	0.50 in	12 995	7 875	450	15 342	15,200.0	0.0	720.0	0.0	16 314 0	0.045	
3	13.0 ft	1,244 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	
-	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	
G	G Eng	ineert		). 9 - STOF		DINGS			BY	SMG	DATE	9/16/04	SH	IEET NO. OF
----	--------	------------------	---	--	---------------------------------	---------------------------------------	------------------------------------	--------------------------------	-------------	-------------	------------------	------------------------------------	----------------------------	------------------------
รเ	JBJECT			MER DESI	GN 9B	DEN			CKD			001111		DMENT FRAME
			B2 CALC	ULATION	- FUR	BENL	JING A	ALONG		-AXIS (		COLUM	V	MF A2 - F2
	0	LOAD	COMBINATION =	1.2D + 0.5L + 0.5	5Lr + 1.6W	( L.C. # 5	)							
			DUE TO FORCE BENDING ALONG T MOMENT FRA	ES THAT CAUSE THE X-AXIS OF THE ME COLUMNS	TOTAL U STORY ON ("LEANER	NFACTORE N ALL COLL R" + "NON-L	D AXIAL L JMNS OF T EANER" C	OAD PER HE STORY OLUMNS)	ΤΟΤΑ	L FACTORED	AXIAL LOAD,	$\Sigma P_u$ , PER STOP	łΥ	OTODY
	STORY	LENGTH (STORY	SHEAR DUE TO SEISMIC	INTERSTORY		(ki	ps) BOOF			LOAD F	ACTOR	SEISMIC	TOTAL	B <sub>2i_X-AXIS</sub>
	NUMBER	HEIGHT) L	HORIZONTAL LOAD E ΣΗ:	$\Delta_{oh}$ DUE TO $\Sigma H_i$	LOAD DL	LIVE LOAD LL	LIVE LOAD	WEIGHT DL + P-LL	D.L. 1.2	L.L. 0.5	ROOF L.L. 0.5	VERTICAL 0.2S <sub>DS</sub> = 0	ک۳ <sub>ui</sub> (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

6	යි පිතය	linear	നത	JOB NC	). 9 - STOR	Y BUILI	DINGS			BY	SMG	DATE	9/16/04	Sł	IEET NO.
ଞ	ia lanĝ	nnnææn i		CUSTO	MER DESI	GN 9B				CKD		DATE		1_	OF
S	JBJECT		B2 (		ULATION	- FOR	BENL	DING A	LONG	THE	Y-AXIS	OF THE	COLUM	N <sup>MO</sup>	DMENT FRAME MF A2 - F2
	0	LOAD	COMBII	IATION =	1.2D + 0.5L + 0.5	öLr + 1.6W									
			DUI BENDII M	E TO FORCE NG ALONG T OMENT FRA	es that cause The <mark>Y-AXIS</mark> of the Me columns	TOTAL U STORY ON	NFACTORE	D AXIAL LO	OAD PER HE STORY	тот	AL FACTORED	axial load,	$\Sigma P_u$ , PER STOP	łΥ	
		LENGTH	TOTA SHEAR	L STORY R DUE TO	ELASTIC		(ki	ps)	OLONING)		LOAD F	ACTOR		τοται	STORY
	STORY NUMBER	(STORY HEIGHT) L	SE HORI LC	ISMIC ZONTAL DAD E ΣH:	DRIFT $\Delta_{oh}$ DUE TO $\Sigma H_i$	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD	SEISMIC WEIGHT DL + P-LL	D.L. 1.2	L.L. 0.5	ROOF L.L. 0.5	$\begin{array}{l} SEISMIC\\ VERTICAL\\ 0.2S_{DS}=0 \end{array}$	ΣP <sub>ui</sub> (kips)	D <sub>2i_Y-AXIS</sub>
	9	13.0 ft	38	0 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	63	3 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	84	4 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	1,0	14 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	1,14	46 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	1,24	14 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	1,30	09 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	1,34	io kips	0.58 in	18,985	12,600	450	20,233	20,382.0	6 300 0	225.0	0.0	20,119.5	1.078

iG Emai	ineeri	JOB NO	). 9 - STOF	RY BUIL	DINGS			BY	SMG	DATE	9/16/04	s	HEET NO.
0.000		CUSTC	MER DESI	GN 9B				CKD		DATE		_	OF
UBJECT		-	STABIL		DEFFIC	CIENT	ALON	G COLL	JMN X-A	XIS, θ <sub>x</sub>		М	OMENT FRAME MF A2 - F2
0 0 0	LOAD C DEFLEC (SEISM MOMEN TOTAL	COMBINATION: CTION AMPLIFIC IC) IMPORTANCI NT FRAME RESIS SEISMIC SHEAR	1 ATION FACTOR: E FACTOR STS WHAT % OF TO THE BUIDLII	.2D + 0.5L + C <sub>d</sub> = I <sub>E</sub> = THE NG?	- 0.5Lr + 1.6 5.5 1.0 25%	w							
		DUE TO FORC BENDING ALONG MOMENT FR/	ES THAT CAUSE THE X-AXIS OF THE AME COLUMNS	TOTAL L STORY O ("LEANE	<i>INFACTORE</i> N ALL COLU R" + "NON-L	ED AXIAL L JMNS OF T LEANER'' C	.OAD PER HE STORY OLUMNS)	ΤΟΤΑ	L FACTORED	axial load,	$\Sigma P_u$ , PER STO	RY	STABILITY
STORY NUMBER	LENGTH <i>(STORY</i> <i>HEIGHT)</i> L	HORIZONTAL LOAD	ELASTIC INTERSTORY DRIFT Δ <sub>oh</sub> DUE TO ΣH	DEAD LOAD	(ki LIVE LOAD	PS) ROOF LIVE LOAD	SEISMIC WEIGHT	D.L.	LOAD F	ROOF L.L.	SEISMIC VERTICAL	TOTAL ΣP <sub>ui</sub> (kips)	COEFFICIENT PER STORY θ <sub>i</sub>
9 8 7 6 5 4 3 2 1	13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft	ΣH <sub>i</sub> 380 kips 633 kips 844 kips 1,014 kips 1,244 kips 1,309 kips 1,346 kips 1,361 kips	0.32 in 0.5 in 0.58 in 0.58 in 0.59 in 0.59 in 0.58 in 0.37 in	DL 3,020 5,015 7,010 9,005 11,000 12,995 14,990 16,985 18,980	LL 0 1,575 3,150 4,725 6,300 7,875 9,450 11,025 12,600	Lr 450 450 450 450 450 450 450	DL + P-LL 3,118 5,563 8,008 10,453 12,898 15,342 17,788 20,233 22,678	1.2 3,624.0 6,018.0 8,412.0 10,806.0 13,200.0 15,594.0 17,988.0 20,382.0 22,776.0	0.5 0.0 787.5 1,575.0 2,362.5 3,150.0 3,937.5 4,725.0 5,512.5 6,300.0	0.5 225.0 225.0 225.0 225.0 225.0 225.0 225.0 225.0 225.0	0.2S <sub>DS</sub> = 0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0	3,849.0 7,030.5 10,212.0 13,393.5 16,575.0 19,756.5 22,938.0 26,119.5 29,301.0	0.021 0.036 0.045 0.047 0.054 0.060 0.066 0.072 0.051

(G	(G Eng	lineerf	ma	JOB NC	9 - STOR	Y BUILI	DINGS			BY	SMG	DATE	9/16/04	SI	HEET NO.
Ý	о <u>—</u>	,	@	CUSTO	MER DESI	GN 9B				СКД		DATE			OF
SI	JBJECT				STABILI	тү сс	DEFFIC	IENT	ALON	G COLU	IMN Y-A	XIS, θ <sub>y</sub>		M	OMENT FRAME MF A2 - F2
	0 0 0	LOAD C DEFLEG (SEISM MOMEI TOTAL	CTION CTION IC) IM IT FRJ SEISI	INATION: I AMPLIFIC IPORTANCE AME RESIS MIC SHEAR ULE TO FORCE DING ALONG T MOMENT FRA	1. ATION FACTOR: FACTOR TS WHAT % OF 1 TO THE BUIDLIN S THAT CAUSE HE Y-AXIS OF THE ME COLUMNS	2D + 0.5L + C <sub>d</sub> = l <sub>E</sub> = IHE 2 IG? TOTAL U STORY OF STORY OF	0.5Lr + 1.61 5.5 1.0 25% NFACTORE N ALL COLU	W ED AXIAL LO MINS OF D	DAD PER HE STORY	ТОТА	L FACTORED	AXIAL LOAD,	ΣP <sub>u</sub> , PER STO	RY	STABILITY
		LENGTH	TOT SHE	AL STORY AR DUE TO	ELASTIC	("LEANEI	nON-L+ "NON-L+" kip)	EANER" CO ps)	OLUMNS)		LOAD F	ACTOR			COEFFICIENT
	STORY NUMBER	(STORY HEIGHT) L	HOI	ANY RIZONTAL LOAD		DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD	SEISMIC WEIGHT DL + P-LL	D.L.	L.L. 0.5	ROOF L.L. 0.5	SEISMIC VERTICAL 0.2S <sub>DS</sub> = 0	ΙΟΙΑL ΣΡ <sub>ui</sub> (kips)	θ <sub>i</sub>
	9 8 7 6 5 4 3 2 1	13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft	3 6 8 1, 1, 1, 1, 1, 1,	<u>Σ</u> H <sub>i</sub> 180 kips 183 kips 144 kips 114 ki	DUE TO ΣH <sub>i</sub> 0.32 in 0.5 in 0.58 in 0.59 in 0.59 in 0.58 in 0.37 in	DL 3,020 5,015 7,010 9,005 11,000 12,995 14,990 16,985 18,980	LL 0 1,575 3,150 4,725 6,300 7,875 9,450 11,025 12,600	Lr 450 450 450 450 450 450 450 450	DL + P-LL 3,118 5,563 8,008 10,453 12,898 15,342 17,788 20,233 22,678	1.2 3,624.0 6,018.0 8,412.0 10,806.0 13,200.0 15,594.0 17,988.0 20,382.0 22,776.0	0.5 0.0 787.5 1,575.0 2,362.5 3,150.0 3,937.5 4,725.0 5,512.5 6,300.0	0.5 225.0 225.0 225.0 225.0 225.0 225.0 225.0 225.0 225.0	0.2S <sub>DS</sub> = 0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0	3,849.0 7,030.5 10,212.0 13,393.5 16,575.0 22,938.0 26,119.5 29,301.0	0.021 0.036 0.045 0.054 0.060 0.066 0.072 0.051

G	G Eng	ineeri	JOB NC	). 9 - STOR	Y BUILI	DINGS			BY	SMG	DATE	9/16/04	S⊦	IEET NO.
			CUSTO	MER DESI	GN 9B				CKD		DATE			UF
รเ	JBJECT		B2 CALC	ULATION	- FOR	BENL	DING A	ALONG	THE X	-AXIS C	OF THE	COLUM	V <sup>MC</sup>	MENT FRAME MF A2 - F2
	0	LOAD	COMBINATION =	1.2D + 0.5L + 1.0	E	( L.C. # 6	)							
			DUE TO FORCE BENDING ALONG T MOMENT FRA	ES THAT CAUSE THE <mark>X-AXIS</mark> OF THE ME COLUMNS	TOTAL U STORY Of ("LEANER	NFACTORE NALL COLU R" + "NON-L	D AXIAL L IMNS OF TI EANER" C	OAD PER HE STORY OLUMNS)	ΤΟΤΑ	L FACTORED 4	XIAL LOAD,	$\Sigma P_u$ , PER STOF	Y	
		LENGTH	SHEAR DUE TO	ELASTIC INTERSTORY	<b>.</b>	(ki	os)	,		LOAD F	ACTOR		TOTAL	
	STORY NUMBER	(STORY HEIGHT) L	SEISMIC HORIZONTAL LOAD E ΣΗ:	$\begin{array}{c} DRIFT \\ \Delta_{oh} \\ DUE \ TO \ \SigmaH_{i} \end{array}$	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD	SEISMIC WEIGHT DL + P-LL	D.L. 1.2	L.L. 0.5	ROOF L.L. 0	SEISMIC VERTICAL 0.2S <sub>DS</sub> = 0.2	ΣP <sub>ui</sub> (kips)	21
	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

ଜ	G Eng	ineeri	JOB NC	). 9 - STOR	Y BUILI	DINGS			BY	SMG	DATE	9/16/04	S⊦	IEET NO.
-			CUSTO	MER DESI	GN 9B				CKD		DATE			OF
รเ	JBJECT		B2 CALC	ULATION	- FOR	BENL	DING A	ALONG	THE Y	-AXIS O	F THE	COLUMI	V <sup>мс</sup>	DMENT FRAME MF A2 - F2
	o	LOAD	COMBINATION =	1.2D + 0.5L + 1.0	E									
			DUE TO FORCE BENDING ALONG T MOMENT FRA	es that cause The <mark>Y-AXIS</mark> of the Me columns	TOTAL U STORY Of ("LEANER	<i>NFACTORE</i> N ALL COLL R" + "NON-L	ED AXIAL L JMNS OF T .EANER" C	OAD PER HE STORY OLUMNS)	TOTA	AL FACTORED A	XIAL LOAD,	$\Sigma P_u$ , PER STOR	Y	
		LENGTH	SHEAR DUE TO	ELASTIC INTERSTORY	,	(ki	ps)	,		LOAD FA	CTOR		τοται	STORY Br: x avec
	STORY NUMBER	(STORY HEIGHT) L	SEISMIC HORIZONTAL LOAD E		DEAD LOAD	LIVE LOAD	ROOF LIVE LOAD	SEISMIC WEIGHT	D.L.	L.L.	ROOF L.L.	SEISMIC VERTICAL	ΣP <sub>ui</sub> (kips)	-21_Y-AXIS
			ΣH <sub>i</sub>	DUE TO SH	DL	LL	Lr	DL + P-LL	1.2	0.5	0	$0.2S_{DS} = 0.2$		
	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	1.023
	8	13.0 11	844 king	0.59 in	5,015	1,5/5	450	5,563	0,U18.U	1 575 0	0.0	1,112.6	11 500 0	1.042
	6	13.0 ft	044 KIPS	0.58 in	9.005	3,150 4 725	400	0,008 10.453	0,412.0	2,362.5	0.0	2,090.6	15,259 1	1.054
	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	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 E	ngineer		D. 9 - STOF	RY BUIL	DINGS			BY	SMG	DATE	9/16/04	S	HEET NO.
	9	CUSTO	MER DESI	GN 9B				CKD		DATE			OF
SUBJE	СТ		STABIL	тү со	DEFFIC	IENT	ALON	g <mark>COL</mark> U	IMN X-A	XIS, θ <sub>x</sub>		Μ	OMENT FRAME MF A2 - F2
	<ul> <li>LOAD</li> <li>DEFLE</li> <li>(SEISM</li> </ul>	COMBINATION: CTION AMPLIFIC NIC) IMPORTANCI	ATION FACTOR: E FACTOR	1.2D + 0. C <sub>d</sub> = I <sub>E</sub> =	5L + 1.0E 5.5 1.0								
	TOTAL	DUE TO FORC BENDING ALONG MOMENT FR/	ES THAT CAUSE THE X-AXIS OF THE AME COLUMNS	TOTAL L STORY O	25% INFACTORE N ALL COLU R" + "NON-L	D AXIAL L IMNS OF T EANER" C	OAD PER HE STORY OLUMNS)	τοτα	L FACTORED	AXIAL LOAD,	$\Sigma P_u$ , PER STC	RY	STABILITY
STOI NUME	LENGTH RY <i>(STORY</i> ER <i>HEIGHT)</i> L	SHEAR DUE TO ANY HORIZONTAL LOAD		DEAD LOAD DI	LIVE LOAD	ROOF LIVE LOAD	SEISMIC WEIGHT DL + P-LL	D.L.	LUAD F	ROOF L.L.	SEISMIC VERTICAL	TOTAL ΣP <sub>ui</sub> (kips)	
9 8 7 6 5 4 3 2 1	13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft	ΣΗ <sub>i</sub> 380 kips 633 kips 844 kips 1,014 kips 1,244 kips 1,244 kips 1,309 kips 1,346 kips 1,361 kips	0.32 in 0.5 in 0.58 in 0.58 in 0.59 in 0.59 in 0.59 in 0.58 in 0.37 in	3,020 5,015 7,010 9,005 11,000 12,995 14,990 16,985 18,980	0 1,575 3,150 4,725 6,300 7,875 9,450 11,025 12,600	Lr 450 450 450 450 450 450 450	3,118 5,563 8,008 10,453 12,898 15,342 17,788 20,233 22,678	1.2 3,624.0 6,018.0 8,412.0 10,806.0 13,200.0 15,594.0 17,988.0 20,382.0 22,776.0	0.3 0.0 787.5 1,575.0 2,362.5 3,150.0 3,937.5 4,725.0 5,512.5 6,300.0	0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0	6236 1,112.6 1,601.6 2,090.6 2,579.6 3,068.4 3,557.6 4,046.6 4,535.6	4,247.6 7,918.1 11,588.6 15,259.1 18,929.6 22,599.9 26,270.6 29,941.1 33,611.6	0.023 0.040 0.051 0.054 0.061 0.069 0.076 0.083 0.059

CUSTOMER SUBJECT S • LOAD COMBINATION: • DEFLECTION AMPLIFICATION STORY NUMBER LENGT 4EIGH 2 13.01	DESIGN 9B ABILITY COEFFIC 1.2D + 0.5L + 1.0E FACTOR: C <sub>d</sub> = 5.5 DUE TO FORCES THAT CAUSE BENDING ALONG THE X-AXIS OF THE MOMENT FRAME COLUMNS TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL UNTERSTORY SEISMIC HORIZONTAL UNTERSTORY	TOTAL STORY SHEAR CAPACITY (OF ALL OF	CKD G COLU RATIO OF SHEAR	IMN X-A	DATE XIS, θ <sub>x</sub>		OF MOMENT FRAME MF A2 - F2
SUBJECT S • LOAD COMBINATION: • DEFLECTION AMPLIFICATION STORY NUMBER HEIGH 1.3.0 f	ABILITY COEFFIC         1.2D + 0.5L + 1.0E         FACTOR:       C <sub>d</sub> = 5.5         DUE TO FORCES THAT CAUSE         BENDING ALONG THE X-AXIS OF THE         MOMENT FRAME COLUMNS         TOTAL STORY         SHEAR DUE TO         SHEAR DUE TO         ORIFT         HORIZONTAL         UNTERSTORY         SEISMIC         DUE TO FORCES THAT CAUSE	TOTAL STORY SHEAR CAPACITY (OF ALL OF	G COLU	JMN X-A	XIS, θ <sub>x</sub>		MOMENT FRAME MF A2 - F2
<ul> <li>LOAD COMBINATION:</li> <li>DEFLECTION AMPLIFICATION</li> <li>STORY LENGT NUMBER HEIGH L</li> <li>13.0 f</li> </ul>	1.2D + 0.5L + 1.0E FACTOR: C <sub>d</sub> = 5.5 DUE TO FORCES THAT CAUSE BENDING ALONG THE X-AXIS OF THE MOMENT FRAME COLUMNS TOTAL STORY TOTAL STORY SEISMIC HORIZONTAL Auto HORIZONTAL Auto HORIZONTAL Auto HORIZONTAL Auto HORIZONTAL Auto HORIZONTAL Auto HORIZONTAL Auto HORIZONTAL Auto HORIZONTAL Auto HORIZONTAL Auto HORIZONTAL Auto HORIZONTAL Auto HORIZONTAL Auto HORIZONTAL HORIX HORIZONTAL HORIZ	TOTAL STORY SHEAR CAPACITY (OF ALL OF	RATIO OF SHEAR				
STORY NUMBER 9 13.0 f	TOTAL STORY SEISMIC DRIFT H SHEAR DUE TO SEISMIC DRIFT H ORIZONTAL Δ <sub>0</sub> LOAD E DIFT TO THE	SHEAR CAPACITY (OF ALL OF	SHEAR				
9 13.0 f	$\Sigma H_i$	RESISTING MOMENT FRAMES)	DEMAND / SHEAR CAPACITY PER STORY β	MAXIMUM ALLOWED STABILITY COEFFICIENT PER STORY θ <sub>i_max</sub>	$\begin{array}{c} \text{STABILITY}\\ \text{COEFFICIENT}\\ \text{PER STORY}\\ \theta_i \end{array}$	COMMENT	
8 13.01 7 13.01 6 13.01 5 13.01 4 13.01 2 13.01 1 13.01 1 13.01	380 kips         0.32 in           633 kips         0.5 in           844 kips         0.58 in           1,014 kips         0.56 in           1,146 kips         0.58 in           1,244 kips         0.59 in           1,309 kips         0.59 in           1,346 kips         0.58 in           1,361 kips         0.37 in	14,082 kips 14,082 kips 17,183 kips 17,107 kips 20,898 kips 20,898 kips 20,898 kips 20,898 kips	0.0270 0.0449 0.0593 0.0670 0.0595 0.0626 0.0644 0.0651	0.250 0.250 0.250 0.250 0.250 0.250 0.250 0.250	0.023 0.040 0.051 0.054 0.061 0.069 0.076 0.083 0.059	ок ок ок ок ок ок	

(G	iG Eng	ineeri		). 9 - STOR	≀Y BUILI	DINGS			BY	SMG	DATE	9/16/04	SI	HEET NO.
~	С ШВ	100000000	CUSTO	MER DESI	GN 9B				СКД		DATE			OF
SI	JBJECT			STABIL		)EFFIC	IENT	ALON	G COLU	IMN Y-A	XIS, θ <sub>y</sub>		M	OMENT FRAME MF A2 - F2
	0 0 0	LOAD C DEFLEC (SEISM MOMEN TOTAL	COMBINATION: CTION AMPLIFIC, IC) IMPORTANCE IT FRAME RESIS SEISMIC SHEAR DUE TO FORC BENDING ALONG ' MOMENT FRA TOTAL STORY SHEAR DUE TO ANY	ATION FACTOR: FFACTOR TO THE BUIDLIN TO THE BUIDLIN STHAT CAUSE INTE Y-AXIS OF THE WE COLUMNS ELASTIC INTERSTORY	1.2D + 0. C <sub>d</sub> = I <sub>E</sub> = <i>THE</i> 2 <i>TOTAL U</i> STORY OI ("LEANEI	5L + 1.0E 5.5 1.0 25% N ALL COLL R" + "NON-L (kij	ED AXIAL L JMNS OF T EANER" C ps)	OAD PER HE STORY OLUMNS)	ТОТА	L FACTORED	AXIAL LOAD, :	ΣP <sub>u</sub> , PER STO	RY	STABILITY COEFFICIENT PER STORY
	NUMBER	(STORY HEIGHT) L	HORIZONTAL LOAD ΣΗ:	DRIFT $\Delta_{oh}$ DUE TO $\Sigma H_i$	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD	SEISMIC WEIGHT DL + P-LL	D.L.	L.L. 0.5	ROOF L.L.	SEISMIC VERTICAL 0.2S <sub>DS</sub> = 0.2	ΣΡ <sub>ui</sub> (kips)	θί
	9 8 7 6 5 4 3 2 1	13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft	ΣH <sub>i</sub> 380 kips 633 kips 844 kips 1,014 kips 1,146 kips 1,244 kips 1,309 kips 1,346 kips 1,361 kips	0.32 in 0.5 in 0.58 in 0.58 in 0.59 in 0.59 in 0.59 in 0.37 in	3,020 5,015 7,010 9,005 11,000 12,995 14,990 16,985 18,980	LL 0 1,575 3,150 4,725 6,300 7,875 9,450 11,025 12,600	Lr 450 450 450 450 450 450 450 450	3,118 5,563 8,008 10,453 12,898 15,342 17,768 20,233 22,678	1.2 3,624.0 6,018.0 8,412.0 10,806.0 13,200.0 15,594.0 17,988.0 20,382.0 22,776.0	0.5 0.0 787.5 1,575.0 2,362.5 3,150.0 3,937.5 4,725.0 5,512.5 6,300.0		$0.2s_{06} = 0.2$ 623.6 1,112.6 1,601.6 2,090.6 2,579.6 3,068.4 3,557.6 4,046.6 4,535.6	4,247.6 7,918.1 11,588.6 15,259.1 18,929.6 22,599.9 26,270.6 29,941.1 33,611.6	0.023 0.040 0.051 0.054 0.061 0.069 0.076 0.083 0.059

GG Engineering	JOB NO.	9 - 5	STORY BUI	LDINGS		BY	SMG	DATE	9/16/04	SHEET NO.
	CUSTON	MER	DESIGN 9B			CKD		DATE		OF
SUBJECT		STA	BILITY C	OEFFICI	ENT ALO		IMN Y-A	XIS, θ <sub>y</sub>		MOMENT FRAME MF A2 - F2
<ul> <li>LOAD COMBIN</li> <li>DEFLECTION</li> </ul>		ATION FA	1.2D + CTOR: Cd DUE TO FORCE BENDING ALONG TI MOMENT FRAI TOTAL STORY SHEAR DUE TO SFISMIC	0.5L + 1.0E = 5.5 S THAT CAUSE HE YAXIS OF THE ME COLUMNS INTERSTORY DRIFT	TOTAL STORY SHEAR CAPACITY (OF ALL OF THE SEISMIC	RATIO OF SHEAR DEMAND / SHEAR CAPACITY	MAXIMUM ALLOWED STABILITY COEFFICIENT	STABILITY COEFFICIENT PER STORY	COMMENT	
N	UMBER HE	EIGHT) L	HORIZONTAL LOAD E ΣH <sub>i</sub>	$\Delta_{oh}$ DUE TO $\Sigma H_i$	RESISTING MOMENT FRAMES)	PER STORY β	$\theta_{i_{max}}$	θ <sub>i</sub>		
F	9 1:	13.0 ft	380 kips	0.32 in	14,082 kips	0.0270	0.250	0.023	ОК	
	8 1	13.0 ft	633 kips	0.5 in	14,082 kips	0.0449	0.250	0.040	ок	
	7 1:	13.0 ft	844 kips	0.58 in	17,183 kips	0.0491	0.250	0.051	ок	
	6 1	13.0 ft	1,014 kips	0.56 in	17,107 kips	0.0593	0.250	0.054	ОК	
	5 1	13.0 ft	1,146 kips	0.58 in	17,107 kips	0.0670	0.250	0.061	ОК	
	4 1. 3 1 <sup>1</sup>	13.0 ft	1,244 Kips	0.59 in 0.59 in	20,898 kips	0.0595	0.250	0.069	OK	
	2 1	13.0 ft	1,346 kips	0.58 in	20,898 kips	0.0644	0.250	0.083	ок	
	1 1:	13.0 ft	1,361 kips	0.37 in	20,898 kips	0.0651	0.250	0.059	ок	

	COLUMN	MAXIMUM	CONTROLLING	BUILDING MAX.	COMMENTS FO	R: DESIGN 9B
NAME	MEMBER SIZE	INTERACTION VALUE	LOAD COMBINATION	INTERACTION 0.4703	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!< td=""><td>OK - STEEL AREA IS &gt; 1% OF TOTAL AREA</td><td>OK - STEEL HSS IS COMPACT</td></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'<sub>c</sub> = 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,  $\theta$ , and the moment magnification factor, B<sub>2</sub>, 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:

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 Ennineerina	JOB NO. 9 -	STORY BUILI	DINGS	BY	SMG	DATE	9/16/04	SHEET NO.
00 mmBlannaan nonfel	CUSTOMER	DESIGN 9C		CKD		DATE		OF
SUBJECT		DES	IGN PARAMETERS	SUM	MARY			MOMENT FRAME MF A2 - F2
o DESIGN	I INPUTS:	o	TOAL NUMBER OF COLUMNS BEI	NG ANALYZ	ΖED	54		
		0	YIELD STRENGTH:	CONCRET	H TE REINFORCEMEI	SS, F <sub>y</sub> = 50 k NT, F <sub>yr</sub> = 0 ksi	si	
		o	MODULUS OF ELASTICITY:	CONCRET	H E REINFORCEMEN	SS, E <sub>s</sub> = 29,0 NT, E <sub>cr</sub> = 29,0	00 ksi 00 ksi	
		0	MINIMUM CONCRETE COMPRESS	IVE STREN	IGTH	f' <sub>c</sub> = 16.0	ksi	
		o	CONCRETE DENSITY			w = 145	b/ft <sup>3</sup>	
		0	CONCRETE REINFORCEMENT		ARE	EA, $A_{sr} = 0.0$ is $I_{xor} = 0.0$ is $I_{yyr} = 0.0$ is $Z_{xor} = 0.0$ is $Z_{yyr} = 0.0$ is	n² 1^4 1^4 1 <sup>3</sup>	
		0	RESISTANCE FACTORS	ļ	IXIAL COMPRESSI FLEXURAL BENDI	ON, $\phi_{c} = 0.75$ NG, $\phi_{b} = 0.90$		
		0	SEISMIC PARAMETERS	REDUN RTICAL SE	IDANCY COEFFICIE	ENT, $\rho = 1.00$ $0.2S_{DS} = 0.20$		
		FACTOR TO AC	ORTHOGONAL LOAD FACTOR ALC COUNT FOR 5% ACCIDENTAL TOR	ong y-axis ' <i>Sion</i> ("Sin	OF SHARED COL	UMNS = 0.30 CH") = 0.02	5	

G	G Eng	lineeri	JOB	NO. 9 - STOP		DINGS			BY	SMG	DATE	9/16/04	S⊦	IEET NO. OF
รเ	JBJECT		B2 CAL		- <b>FO</b> F	R BENL	DING A	ALONG	THE X	(-AXIS (	OF THE	COLUM	V <sup>MC</sup>	MENT FRAME MF A2 - F2
	o	LOAD	COMBINATIO	/ = 1.2D + 1.6Lr + 0	.8W	( L.C. # 4	)							
			DUE TO FC BENDING ALO MOMENT	RCES THAT CAUSE IG THE X-AXIS OF THE FRAME COLUMNS	TOTAL L STORY O ("LEANE	<i>INFACTORE</i> N ALL COLL R" + "NON-L	D AXIAL L JMNS OF T EANER" C	OAD PER HE STORY OLUMNS)	ΤΟΤΑ	L FACTORED	AXIAL LOAD,	$\Sigma P_u$ , PER STOP	Y	
	STORY NUMBER	LENGTH (STORY HEIGHT) L	SHEAR DUE SEISMIC HORIZONTA LOAD E ΣH <sub>i</sub>	L ELASTIC INTERSTORY DRIFT L DUE TO SH	DEAD LOAD DL	(ki LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	D.L. 1.2	LOAD F L.L. 0	ROOF L.L.	SEISMIC VERTICAL 0.2S <sub>DS</sub> = 0	TOTAL ΣP <sub>ui</sub> (kips)	STORY B <sub>2i_X-AXIS</sub>
	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	0.50 in	7,010	3 150	450	8,000	8 410 0	0.0	720.0	0.0	0 132 0	1.000
	6	13.0 ft	044 KIPS	0.6 in	9,005	4 725	450	0,008 10.453	0,412.U	0.0	720.0	0.0	3,132.U	1.042
	5	13.0 ft	1,146 kins	0.0 m	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 kins	0.6 in	12 995	7 875	450	15 342	15 594 0	0.0	720.0	0.0	16 314 0	1.050
	3	13.0 ft	1,244 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
	2	13.0 ft	1,346 kips	0.5 in 0.25 in	16,985	11,025	450	20,233 22,678	20,382.0 22,776.0	0.0	720.0	0.0	21,102.0 23,496.0	1.053

G	G Eng	lineeri	Job Mg	NO. 9 - STOI	ry Buil	DINGS			BY	SMG	DATE	9/16/04	S⊦	IEET NO.
	Ŭ		CUS	TOMER DES	IGN 9C				CKD		DATE			OF
SI	JBJECT		B2 CAL	CULATION	I - FOF	R BENL	DING A	ALONG	THE	Y-AXIS C	F THE	COLUM	N MC	DMENT FRAME MF A2 - F2
	0	LOAD	COMBINATIO	V = 1.2D + 1.6Lr + (	).8W									
			DUE TO FO BENDING ALO MOMENT	DRCES THAT CAUSE NG THE Y-AXIS OF THE FRAME COLUMNS	TOTAL L STORY O ("LEANE	<i>INFACTORE</i> N ALL COLU R" + "NON-L	ED AXIAL L JMNS OF T LEANER" C	OAD PER HE STORY OLUMNS)	тот	AL FACTORED A	XIAL LOAD,	$\Sigma P_u$ , PER STOF	łΥ	
	STORY	LENGTH (STORY	SHEAR DUE SEISMIC	TO INTERSTORY		(ki	ps)			LOAD FA	ACTOR	05/01/10	TOTAL	STORY B <sub>2i_Y-AXIS</sub>
	NUMBER	HEIGHT) L		AL DRIFT Δ <sub>oh</sub> DUE TO ΣH <sub>i</sub>	DEAD LOAD DL	LIVE LOAD LL	LIVE	SEISMIC WEIGHT DL + P-LL	D.L. 1.2	L.L. 0	ROOF L.L.	SEISMIC VERTICAL 0.2S <sub>DS</sub> = 0	ΣP <sub>ui</sub> (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	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	1,340 Kips	0.5 in	18,980	12 600	450	20,233	20,362.0	0.0	720.0	0.0	23 496 0	1.028

6	G Eng	jineeri		9 - STOR		DINGS			BY	SMG	DATE	9/16/04	SI	HEET NO.
S	JBJECT		CUSTO	STABILI		EFFIC	IENT	ALON		IMN X-A	XIS, θ <sub>x</sub>		<u>м</u>	OMENT FRAME MF A2 - F2
	0 0 0	LOAD C DEFLEC (SEISM MOMEN TOTAL	COMBINATION: CTION AMPLIFICA IC) IMPORTANCE IT FRAME RESIS SEISMIC SHEAR DUE TO FORCE BENDING ALONG T MOMENT FRA	ATION FACTOR: FACTOR TS WHAT % OF T TO THE BUIDLIN S THAT CAUSE HE X-AXIS OF THE ME COLUMNS	1.2D + 1.6 C <sub>d</sub> = I <sub>E</sub> = THE IG? TOTAL U STORY OI ("LEANE!	5.5 1.0 25% NFACTORE N ALL COLL R" + "NON-L	D AXIAL L DINS OF T EANER" C	oad Per He Story Olumns)	тота	L FACTORED	AXIAL LOAD,	ΣP <sub>u</sub> , PER STO	RY	STABILITY
	STORY NUMBER	LENGTH (STORY HEIGHT) L	HORIZONTAL LOAD	ELASTIC INTERSTORY DRIFT A <sub>oh</sub> DUE TO ΣH <sub>i</sub>	DEAD LOAD DL	(kij LIVE LOAD LL	ROOF LIVE LOAD	SEISMIC WEIGHT DL + P-LL	D.L.	LOAD F	ROOF L.L.	SEISMIC VERTICAL 0.2Sps = 0	TOTAL ΣP <sub>ui</sub> (kips)	COEFFICIENT PER STORY θ <sub>i</sub>
	9 8 7 6 5 4 3 2 1	13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft	2/1 380 kips 633 kips 844 kips 1,014 kips 1,244 kips 1,309 kips 1,346 kips 1,361 kips	0.31 in 0.47 in 0.58 in 0.6 in 0.6 in 0.58 in 0.5 in 0.25 in	3,020 5,015 7,010 9,005 11,000 12,995 14,990 16,985 18,980	0 1,575 3,150 4,725 6,300 7,875 9,450 11,025 12,600	Lr 450 450 450 450 450 450 450 450	3,118 5,563 8,008 10,453 12,898 15,342 17,788 20,233 22,678	3,624.0 6,018.0 8,412.0 10,806.0 13,200.0 15,594.0 17,988.0 20,382.0 22,776.0		720.0 720.0 720.0 720.0 720.0 720.0 720.0 720.0	0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0	4,344.0 6,738.0 9,132.0 11,526.0 13,920.0 16,314.0 18,708.0 21,102.0 23,496.0	0.023 0.032 0.040 0.044 0.047 0.050 0.053 0.050 0.028

CUSTOMER DESIGN 9C         CKD         DATE           SUBJECT         STABILITY COEFFICIENT ALONG COLUMN Y-AXIS, 0,           • LOAD COMBINATION:         1.20+1.6Lr+0.8W           • DEFLECTION AMPLIFICATION FACTOR:         C_g = 55           • (SEISMIC) IMPORTANCE FACTOR:         C_g = 56           • (SEISMIC) IMPORTANCE FACTOR:         C_g = 56           • MOMENT FRAME RESISTS WHAT % OF THE TOTAL SEISMIC SHEAR TO THE BUILING:         28%           TOTAL SEISMIC SHEAR TO THE BUILING:         28%           Image: Distribution of the source the MOMENT FRAME RESISTS WHAT % OF THE MOMENT FRAME COLUMNS:         TOTAL UNFACTORED AXIAL LOAD PER TOTAL SEISMIC SHEAR NOT THE SUBLE (LEANGTH)         TOTAL UNFACTORED AXIAL LOAD PER (LEANGTH)         TOTAL LOAD CLUMNS OF THE SUBMY           NUMBER HER HER TOTAL SEIST WHAT % OF THE MOMENT FRAME COLUMNS         TOTAL UNFACTORED COLUMNS)         LOAD DA COTOR           NUMBER HER HER DUE TO STORY ON ALL COLUMNS OF THE SUBMY         TOTAL UNFACTORED COLUMNS)         LOAD DA COTOR           NUMBER HER HER DUE TO SH         DL         LVVE         HOVE SEISMIC LOAD DA COTOR         Zei (DAD DA COTOR           NUMBER HER HER DUE TO SH         DL         DL         LVVE         VERTOR         LOAD DA COTOR           NUMBER HER HER DUE TO SH         DL         DL         LVVE <th>OF MOMENT FRAME MF A2 - F2</th> <th></th> <th></th> <th></th> <th></th> <th></th> <th></th> <th></th> <th></th> <th></th> <th>M</th> <th>ineeri</th> <th>ig Eng</th>	OF MOMENT FRAME MF A2 - F2										M	ineeri	ig Eng
SUBJECT         STABILITY COEFFICIENT ALONG COLUMN Y-AXIS, θy                LOAD COMBINATION: I.2.1 bl + 1.6Lr + 0.8W            0 LOAD COMBINATION: I.2.1 bl + 1.6Lr + 0.8W            0 DEFLECTION AMPLIFICATION FACTOR: G = 5.5            0 BEFLECTION AMPLIFICATION FACTOR: G = 5.5            0 MOMENT FRAME RESISTS WHAT % OF THE TOTAL SEISMICS WHAT % OF THE STORY ON ALL COLUMNS OF THE STORY MOMENT FRAME COLMANS            TOTAL LIARAGETORED AVIAL LOAD PER MOMENT FRAME COLMANS            TOTAL LIARAGETORY MOMENT FRAME COLMANS            TOTAL LIARAGETORY MERGETY            NOTE: TOTAL LIAR	MOMENT FRAME MF A2 - F2		DATE		СКД				GN 9C	MER DESI	CUSTO		
O       LOAD COMBINATION:       1.2D + 1.6Lr + 0.8W         9       DEFLECTION AMPLIFICATION FACTOR:       C = 5.         9. (SEISMIC) MPORTANCE FACTOR       L = 10.         9. OMMENT FRAME RESISTS WHAT % OF THE TOTAL SEISMIC SHEAR TO THE BUILDING?       25%         1       DUE TO PORCES THAT CAUSE BENDING ALCING THE FAMS OF THE MOMENT FRAME COLUMNS       TOTAL INFACTORED AXIAL LOAD PER MOMENT FRAME COLUMNS       TOTAL FACTORED AXIAL LOAD, EP., PER STOR (LENGTH VICUL STORY)         1       LENGTH VICUL STORY INFERSTORY NUMBER       HEAR DUE TO INTERSTORY LENGTH VICUL STORY NUMBER       TOTAL UNFACTORED AXIAL LOAD PER (LENGTH VICUL STORY)       TOTAL FACTORED AXIAL LOAD, EP., PER STORY (LEANER" + NON-LEANER" COLUMNS)         1       LENGTH VICUL STORY INTERSTORY NUMBER       NUMER TRAME COLUMNS (STORY)       TOTAL INFACTORED AXIAL LOAD, EP., PER STORY (LEANER" + NON-LEANER" COLUMNS)         1       LENGTH VICUL STORY INTERSTORY NUMBER       NUMER TRAME COLUMNS       TOTAL SEASON         1       13.0 ft 338 upp 0.31 in 3.00 1.014 kips 0.31 in 3.00 1.014 kips 0.51 in 1.000 6.300 1.450 3.018 8.412.0 0.0 720.0 0.0 720.0 0.0 5 1.30.0 ft 1.014 kips 0.61 in 1.020 6.300 450 12.888 13.200.0 0.0 720.0 0.0 720.0 0.0 5 1.30.0 ft 1.346 kips 0.51 in 1.8980 12.600 450 22.678 22.778.0 0.0 720.0 0.0 720.0 0.0 720.			XIS, θ <sub>y</sub>	MN Y-A	G COLU	ALON	IENT	EFFIC	түсс	STABILI	• • • • • • • • • • • • • • • • • • •		UBJECT
MOMENT FRAME COLUMNS         SIGHY ON ALL COLUMNS OF THE SIGHY TOTAL STORY NUMBER         TOTAL STORY STORY NUMBER         TOTAL STORY LENGTH         SIGHY ON ALL COLUMNS OF THE SIGHY TOTAL STORY NUMBER         CLOAD FACTOR           ULENGTH         CARAN OF DAIL         ELASTIC An DUE TO 2H, DL         ELASTIC LOAD DL         ELASTIC LOAD LOAD LOAD         ELONER' = VOLUMER' LOAD LOAD         SEISMIC LOAD LF         DL         LL         ROOF LL.         SEISMIC VERTICAL           9         13.0 ft         380 kips         0.31 in DUE TO 2H, DL         3,020         0         450         3,118         3,624.0         0.0         720.0         0.0           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         13.0 ft         1,014 kips         0.6 in         12,095         7,875         450         10,865.0         0.0         720.0         0.0           3         13.0 ft         1,146 kips         0.6 in         12,995         7,875         450         15,342         15,594.0         0.0         720.0         0.0           2         13.0 ft         1,361 kips         0.5 in         14,	v [	ΣΡ <sub>υ</sub> , PER STORY	AXIAL LOAD, 3	L FACTORED	ТОТА	OAD PER		Lr + 0.8W 5.5 1.0 25%	1.2D + 1.6 C <sub>d</sub> = I <sub>E</sub> = I <sub>E</sub> ? IG? TOTAL U	TION FACTOR: FACTOR TS WHAT % OF 1 TO THE BUIDLIN	MBINATION: TION AMPLIFICA ) IMPORTANCE F FRAME RESIS: EISMIC SHEAR DUE TO FORCE BENDING ALONG T	LOAD CO DEFLEC (SEISMIC MOMEN TOTAL S	0 0 0
P         13.0 ft         380 kips         0.31 in         3.020         0         450         3.118         3.624.0         0.0         720.0         0.0           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           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           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           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           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           2         13.0 ft         1.309 kips         0.58 in         14.990         9.450         450         17.788         17.988.0         0.0         720.0	STABILITY COEFFICIENT           TOTAL         PER STORY           ΣP <sub>ui</sub> θ <sub>i</sub>	SEISMIC VERTICAL	FACTOR	LOAD F	D.L.	SEISMIC WEIGHT	ROOF LIVE LOAD	LIVE	DEAD LOAD	ME COLUMNS ELASTIC INTERSTORY DRIFT Δ <sub>oh</sub> DUE TO ΣH <sub>i</sub>	MOMENT FRAM TOTAL STORY SHEAR DUE TO ANY HORIZONTAL LOAD	LENGTH (STORY HEIGHT) L	STORY NUMBER
	4,344.0 0.023 3,738.0 0.032 9,132.0 0.040 1,526.0 0.044 3,920.0 0.047 6,314.0 0.050 8,708.0 0.053 11,102.0 0.050 3,496.0 0.028	0.2S <sub>DS</sub> = 0 0.0 4 0.0 6 0.0 19 0.0 11 0.0 11 0.0 11 0.0 2 0.0 23 0.0 23 0.0 23 0.0 23 0.0 23 0.0 23 0.0 11 0.0 11 0.0 11 0.0 12 0.0	1.6 720.0 720.0 720.0 720.0 720.0 720.0 720.0 720.0	0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0	1.2 3,624.0 6,018.0 8,412.0 10,806.0 13,200.0 15,594.0 17,988.0 20,382.0 22,776.0	DL + P-LL 3,118 5,563 8,008 10,453 12,898 15,342 17,788 20,233 22,678	Lr 450 450 450 450 450 450 450	LL 0 1,575 3,150 4,725 6,300 7,875 9,450 11,025 12,600	DL 3,020 5,015 7,010 9,005 11,000 12,995 14,990 16,985 18,980	0.31 in 0.47 in 0.58 in 0.6 in 0.61 in 0.58 in 0.58 in 0.25 in	ΣH <sub>i</sub> 380 kips 633 kips 844 kips 1,014 kips 1,244 kips 1,309 kips 1,346 kips 1,361 kips	13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft	9 8 7 6 5 4 3 2 1

6	iG Eng	ineeri		). 9 - STOF	RY BUILI GN 9C	DINGS			BY CKD	SMG	DATE DATE	9/16/04	SH	IEET NO. OF
S	UBJECT		B2 CALC	ULATION	- FOR	BENL	DING A	ALONG	THE X	-AXIS C	F THE	COLUM	N <sup>MC</sup>	MENT FRAME MF A2 - F2
	0	LOAD	COMBINATION =	1.2D + 0.5L + 0.5	5Lr + 1.6W	( L.C. # 5	)							
			DUE TO FORCE BENDING ALONG MOMENT FRA	ES THAT CAUSE THE X-AXIS OF THE ME COLUMNS	TOTAL U STORY OF ("LEANE!	NFACTORE N ALL COLU R" + "NON-L	ED AXIAL L JMNS OF T .EANER" C	OAD PER HE STORY OLUMNS)	τοτα	L FACTORED	AXIAL LOAD,	$\Sigma P_u$ , PER STOF	ïY	
	OTODV	LENGTH	SHEAR DUE TO	ELASTIC INTERSTORY		(ki	ps)			LOAD F.	ACTOR		TOTAL	STORY B <sub>2i X-AXIS</sub>
	NUMBER	(STORT HEIGHT) L	HORIZONTAL LOAD E ΣH <sub>i</sub>	DRIFT Δ <sub>oh</sub> DUE TO ΣH <sub>i</sub>	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 <sub>DS</sub> = 0	ΣP <sub>ui</sub> (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	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

6	iG Eng	lineeri	ing -	JOB NC	). 9 - STOF	RY BUILI	DINGS			BY CKD	SMG	DATE DATE	9/16/04	S⊦	IEET NO. OF
S	JBJECT		B2 (	ALC	ULATION	- FOR	BEND	DING A	ALONG	THE Y	-AXIS O	F THE	COLUM	V <sup>MC</sup>	MENT FRAME MF A2 - F2
	0 STORY NUMBER 9 8 7 6 5 4 3 2 1	LOAD ( (STORY HEIGHT) L 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft	DUE BENDIN MM TOTAIA SHEAF SE HORI LC : : 38/ 633 84/ 1,0° 1,1/2 1,3/2 1,3/2	ATTON = TO FORCE IS ALONG T DMENT FRA STORY R DUE TO ISMIC ZONTAL AD E EH I kips 4 kips 4 kips 4 kips 4 kips 16 kips 16 kips 16 kips 16 kips 16 kips 16 kips	1.2D + 0.5L + 0.5	SLr + 1.6W TOTAL U STORY OI ("LEANEI DEAD LOAD DL 3,020 5,015 7,010 9,005 11,000 12,995 14,990 16,985 18,980	NFACTORE N ALL COLL (kij LIVE LOAD LL 0 1,575 3,150 4,725 6,300 7,875 9,450 11,025 12,600	D AXIAL L IMNS OF T EANER" C OS) ROOF LIVE LOAD LT 450 450 450 450 450 450 450 450 450 450	OAD PER HE STORY OLUMNS) JL + P-LL 3,118 5,563 8,008 10,453 12,898 15,342 17,788 20,233 22,678	DL. 1.2 3,624.0 6,018.0 8,412.0 10,806.0 13,200.0 15,594.0 17,988.0 20,382.0 22,776.0	L FACTORED A L.L. 0.5 0.0 787.5 1,575.0 2,362.5 3,150.0 3,937.5 4,725.0 5,512.5 6,300.0	XIAL LOAD, CCTOR ROOF L.L. 0.5 225.0 25	ΣP <sub>u</sub> , PER STOR VERTICAL 0.2S <sub>DS</sub> = 0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0	Y TOTAL ΣP <sub>u</sub> (kips) 3,849.0 7,030.5 10,212.0 13,393.5 16,575.0 19,756.5 22,938.0 26,119.5 29,301.0	STORY B <sub>21_Y-AXIS</sub> 1.021 1.035 1.047 1.054 1.060 1.065 1.070 1.066 1.036
			,												

6	iG Eng	ineeri	JOB NC	). 9 - STOR	Y BUILI	DINGS			BY	SMG	DATE	9/16/04	SI	HEET NO.
	<u> </u>		CUSTO	MER DESI	GN 9C				CKD		DATE			OF
S	JBJECT			STABILI	түсс	DEFFIC	IENT	ALON	G COLU	IMN X-A	XIS, θ <sub>x</sub>		М	OMENT FRAME MF A2 - F2
	0 0 0	LOAD C DEFLEC (SEISMI MOMEN TOTAL :	OMBINATION: CTION AMPLIFIC. C) IMPORTANCE IT FRAME RESIS SEISMIC SHEAR DUE TO FORCE	1. ATION FACTOR: FACTOR TS WHAT % OF T TO THE BUIDLIN ES THAT CAUSE	2D + 0.5L + C <sub>d</sub> = I <sub>E</sub> = IG?	- 0.5Lr + 1.6 5.5 1.0 25%								
			BENDING ALONG T MOMENT FRA	THE X-AXIS OF THE	STORY OF	N ALL COLU R" + "NON-L	JMNS OF T EANER" C	HE STORY OLUMNS)	ΤΟΤΑ	L FACTORED	AXIAL LOAD,	ΣP <sub>u</sub> , PER STO	RY	STABILITY
	STORY NUMBER	LENGTH <i>(STORY</i> <i>HEIGHT)</i> L	SHEAR DUE TO ANY HORIZONTAL LOAD ΣH <sub>i</sub>	ELASTIC INTERSTORY DRIFT Δ <sub>oh</sub> DUE TO ΣH <sub>i</sub>	DEAD LOAD DL	(ki LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	D.L. 1.2	LOAD F L.L. 0.5	ROOF L.L.	SEISMIC VERTICAL 0.2S <sub>DS</sub> = 0	TOTAL ΣP <sub>ui</sub> (kips)	COEFFICIENT PER STORY θ <sub>i</sub>
	9 8 7 6 5 4 3 2 1	13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft	271; 380 kips 633 kips 844 kips 1,014 kips 1,146 kips 1,244 kips 1,309 kips 1,346 kips 1,361 kips	0.31 in 0.47 in 0.58 in 0.61 in 0.61 in 0.58 in 0.58 in 0.25 in	3,020 5,015 7,010 9,005 11,000 12,995 14,990 16,985 18,980	0 1,575 3,150 4,725 6,300 7,875 9,450 11,025 12,600	Lr 450 450 450 450 450 450 450 450	3,118 5,563 8,008 10,453 12,898 15,342 17,788 20,233 22,678	3,624.0 6,018.0 8,412.0 10,806.0 13,200.0 15,594.0 17,988.0 20,382.0 22,776.0	0.0 787.5 1,575.0 2,362.5 3,150.0 3,937.5 4,725.0 5,512.5 6,300.0	225.0 225.0 225.0 225.0 225.0 225.0 225.0 225.0		3,849.0 7,030.5 10,212.0 13,393.5 16,575.0 22,938.0 26,119.5 29,301.0	0.020 0.033 0.045 0.051 0.057 0.061 0.065 0.062 0.035

SUBJECT         STABILITY COEFFICIENT ALONG COLUMN Y-AXIS, 9,         MMM           9         LOAD COMBINATION:         120+031+054:+18W	6	G Eng	ineeri	JOB NC	). 9 - STOR	Y BUILI	DINGS			BY	SMG	DATE	9/16/04	S	HEET NO.
MDEET COEFFICIENT ALONG COLUMN Y-AXIS, 6,       MINITY       MARKED       P. LOAD COMBINITION:     1.20+0.5L+0.5L+1.8W       P. CARL COMMUNICATION FACTOR:     C-5.5       P. CARL COMPARANCE FACTOR:     L-10       P. CARL COMPARANCE FACTOR:     CARL COLUMNE OF THE STORY       P. CARL COMPARANCE FACTOR:     TOTAL INFACTOR:       P. CARL COMPARANCE FACTOR:     CARL COLUMNE OF THE STORY       P. CARL COMPARANCE FACTOR:     CARL COLUMNE OF THE STORY       P. CARL COMPARANCE FACTOR:     CARL COLUMNE OF THE STORY       P. CARL COMPARANCE FACTOR:     CARL COLUMNE OF THE STORY       NUMBER     CARL COLUMNE OF THE STORY <tr< th=""><th></th><th></th><th></th><th>сиѕто</th><th>MER DESI</th><th>GN 9C</th><th></th><th></th><th></th><th>CKD</th><th></th><th>DATE</th><th></th><th></th><th>OF</th></tr<>				сиѕто	MER DESI	GN 9C				CKD		DATE			OF
<text><text><text><text></text></text></text></text>	SL	JBJECT			STABILI	TY CC	)EFFIC	CIENT	ALON	G COLU	IMN Y-A	XIS, θ <sub>y</sub>		м	OMENT FRAME MF A2 - F2
Image: Product of the construct of		0 0 0	LOAD C DEFLEC (SEISM MOMEN TOTAL	COMBINATION: CTION AMPLIFIC. IC) IMPORTANCE IT FRAME RESIS SEISMIC SHEAR	1. ATION FACTOR: E FACTOR ITS WHAT % OF 1 TO THE BUIDLIN	2D + 0.5L + C <sub>d</sub> = I <sub>E</sub> = THE IG?	0.5Lr + 1.6 5.5 1.0 25%	w							
STOR         LENDTH         SHEAR DOTAL STORY (STORY ILCAD         ELARTIC MAY         CLARTIC ILCAD         CLARTIC ILCAD         CLARTIC ILCAD         TOTAL STORY (STORY DATE         SHEAR DOTAL MAY         TOTAL STORY ANY         TOTAL STORY MIDE TO ANY         TOTAL ANY         TOTAL STORY MIDE TO ANY				DUE TO FORCE BENDING ALONG T MOMENT FRA	ES THAT CAUSE THE Y-AXIS OF THE ME COLUMNS	TOTAL U STORY OF		D AXIAL L	OAD PER HE STORY	ΤΟΤΑ	L FACTORED	AXIAL LOAD,	$\Sigma P_u$ , PER STO	RY	
STORY NUMBER         ANV L         ANV L         DRIFT LOAD DL         DEAD LOAD DL         LIVE LOAD DL         POOT LOAD LOAD DL         LLL LOAD DL         NOOT LL LOAD DL         DESMO 0.5         DL         LL         MOOT LL NOOT LL         SESMO 0.5         DEFTTOLL 0.25m         OP (Hpp)         PP (Hpp)         etch (Hpp)           9         13.0 ft         330 kps         0.31 in         3.020         0         450         3.118         3.824.0         0.0         225.0         0.0         3.840.0         0.0           8         13.0 ft         633 kps         0.47 in         5.015         1.575         450         5.063         6.018.0         787.5         225.0         0.0         1.0212.0         0.0           6         13.0 ft         1.014 kips         0.6 in         9.005         4.725         450         10.453         10.806.0         2.382.5         225.0         0.0         16.575.0         0.0           4         13.0 ft         1.304 kps         0.58 in         14.990         9.450         450         15.342         15.594.0         3.937.5         225.0         0.0         18.755.0         0.0           2         13.0 ft         1.304 kips         0.58 in         14.990         9.450<			LENGTH	TOTAL STORY SHEAR DUE TO	ELASTIC INTERSTORY	( LEANER	h + NON-L (kip	ps)	JLUIVIINS)		LOAD F	ACTOR		TOTAL	COEFFICIENT PER STORY
9         13.0 h         380 kips         0.31 in         3.020         0         450         3.118         3.624.0         0.0         225.0         0.0         3.849.0         0.0           8         13.0 h         633 kips         0.71 in         5.015         1.75         450         5.663         6.018.0         77.5         225.0         0.0         7.030.5         0.0           6         13.0 h         644 kips         0.58 in         7.010         3.150         450         8.008         8.412.0         1.575.0         225.0         0.0         10.212.0         0.0           6         13.0 h         1.146 kips         0.61 in         11.000         6.300         450         12.898         10.806.0         2.362.5         2.50         0.0         16.575.0         0.0           4         13.0 h         1.344 kips         0.6 in         12.995         7.875         450         15.342         15.984.0         3.937.5         225.0         0.0         2.938.0         0.0           2         13.0 h         1.364 kips         0.58 in         14.990         9.450         20.332         20.382.0         5.512.5         2.0         0.0         2.9381.0         0.0      <		STORY NUMBER	(STORY HEIGHT) L	ANY HORIZONTAL LOAD ΣH;	$\begin{array}{c} DRIFT \\ \Delta_{oh} \\ DUE \ TO \ \SigmaH_{i} \end{array}$	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD	SEISMIC WEIGHT DL + P-LL	D.L. 1.2	L.L. 0.5	ROOF L.L. 0.5	SEISMIC VERTICAL 0.2S <sub>DS</sub> = 0	ΣP <sub>ui</sub> (kips)	θ <sub>i</sub>
8         13.0 tt         633 kips         0.47 in         5.015         1.575         450         5.583         6.018.0         787.5         225.0         0.00         1.030 tt         10.01 kips         0.58 in         7.010         3.150         450         8.008         8.412.0         1.575.0         225.0         0.00         10.212.0         0.00           6         13.0 tt         1.14 kips         0.6 in         9.005         4.72         450         10.463         10.806.0         2.362.0         0.00         16.375.0         0.00           4         13.0 tt         1.14 kips         0.6 in         12.995         7.875         450         15.342         15.54.0         3.937.5         225.0         0.00         16.75.0         0.00           3         13.0 tt         1.30 kips         0.5 in         16.985         11.025         450         17.788         17.988.0         4.725.0         225.0         0.00         22.938.0         0.00           1         13.0 tt         1.36 kips         0.25 in         18.980         12.600         450         22.678         22.776.0         6.300.0         225.0         0.0         29.301.0         0.0           1         13.0 tt		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
7       13.0 ft       844 kips       0.58 in       7.010       3,150       450       8,008       8,1120       1.575.0       225.0       0.0       10,1212       0.0         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       13,335.5       0.0         4       13.0 ft       1,44 kips       0.6 in       11,000       6,300       450       12,898       13,200.0       3,150.0       225.0       0.00       16,375.0       0.0         4       13.0 ft       1,244 kips       0.6 in       12,995       7,875       450       15,342       15,54.0       3,937.5       225.0       0.0       12,930.0       0.0         2       13.0 ft       1,304 kips       0.5 in       16,985       11,025       450       20,233       20,382.0       5,512.5       225.0       0.0       26,119.5       0.0         1       13.0 ft       1,364 kips       0.25 in       18,980       12,600       450       22,678       22,776.0       6,300.0       225.0       0.0       29,301.0       0.0         1       13.0 ft       1,361 kips       0.25 in       18,98		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
1       1,014 kps       0.6 lin       11,00       6,200       10,435       10,436       2,305.0       2,25.0       0.0       16,576.0       0.0         4       13.01 t       1,144 kps       0.6 lin       12,095       7,675       450       15,544.0       3,937.5       225.0       0.0       16,576.0       0.0         3       13.01 t       1,244 kps       0.6 lin       12,995       7,875       450       15,342       15,594.0       3,937.5       225.0       0.0       16,576.0       0.0         3       13.01 t       1,346 kps       0.58 in       14,990       9,450       450       17,788       17,988.0       4,725.0       225.0       0.0       22,938.0       0.0         2       13.01 t       1,346 kps       0.5 in       16,895       11,025       450       20,233       20,382.0       5,512.5       225.0       0.0       25,119.5       0.0         1       13.01 t       1,361 klps       0.25 in       18,980       12,600       450       22,678       22,776.0       6,300.0       225.0       0.0       29,301.0       0.0         1       13.01 t       1,361 klps       0.25 in       18,980       12,600       450       2		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
4       13.0 tř.       1.244 kips       0.56 in       12.995       7.875       450       15.942       15.942       3.937.5       225.0       0.0       22.938.0       0.0         2       13.0 tř.       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       0.0         1       13.0 tř.       1.361 kips       0.25 in       18.980       12.600       450       22.776.0       6.300.0       225.0       0.0       28.301.0       0.0         1       13.0 tř.       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       0.0         1       13.0 tř.       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       0.0         1       13.0 tř.       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       0.0         1       13.0 tř.       1.461 kip       1.461 kip       1.461		5	13.0 ft	1,014 kips	0.61 in	9,005	4,725 6.300	450	12,898	13,200.0	2,362.5	225.0	0.0	16.575.0	0.057
3       13.0 ft       1.309 kps       0.58 in       14.990       9.450       450       17.788       17.988.0       4.725.0       225.0       0.0       25.119.5       0.0         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       0.0         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       0.0         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       0.0         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       0.0         1       1.1 <td></td> <td>4</td> <td>13.0 ft</td> <td>1,244 kips</td> <td>0.6 in</td> <td>12,995</td> <td>7,875</td> <td>450</td> <td>15,342</td> <td>15,594.0</td> <td>3,937.5</td> <td>225.0</td> <td>0.0</td> <td>19,756.5</td> <td>0.061</td>		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
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       0.0         1       13.0 ft       1,361 kips       0.25 in       18,980       12,600       450       22,678       22,778.0       6,300.0       225.0       0.0       29,301.0       0.0         1       13.0 ft       1,361 kips       0.25 in       18,980       12,600       450       22,678       22,778.0       6,300.0       225.0       0.0       29,301.0       0.0         1       13.0 ft       1,361 kips       0.25 in       18,980       12,600       450       22,678       22,778.0       6,300.0       225.0       0.0       29,301.0       0.0         1       11.0 ft       1,361 kips       0.25 in       18,980       12,600       1450		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
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 0.0		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 Eng	jineeri				DINGS			BY	SMG	DATE	9/16/04	S⊦	IEET NO. OF
SUBJECT		B2 CAL		- FOR	R BENL	DING A	ALONG	THE X	-AXIS C	OF THE	COLUM	N <sup>MC</sup>	MENT FRAME MF A2 - F2
0	LOAD	COMBINATION	'= 1.2D + 0.5L + 1.	DE	( L.C. # 6	)							
		DUE TO FO BENDING ALON MOMENT I TOTAL STOF	RCES THAT CAUSE G THE X-AXIS OF THE RAME COLUMNS	TOTAL U STORY OI ("LEANE	INFACTORE N ALL COLL R" + "NON-L	ED AXIAL L JMNS OF T LEANER" C	.OAD PER THE STORY COLUMNS)	τοτα	L FACTORED A	AXIAL LOAD,	$\Sigma P_u$ , PER STO	ΥY	STORY
STORY NUMBER	LENGTH (STORY HEIGHT) L	SHEAR DUE SEISMIC HORIZONTA LOAD E	CO INTERSTORY DRIFT Δ <sub>oh</sub> DUE TO ΣH <sub>i</sub>	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD	SEISMIC WEIGHT DL + P-LL	D.L.	LOAD F	ROOF L.L.	SEISMIC VERTICAL 0.2S <sub>DS</sub> = 0.2	TOTAL ΣP <sub>ui</sub> (kips)	B <sub>2i_X-AXIS</sub>
9 8 7 6 5 4 3 2 1	13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft	380 kips 633 kips 844 kips 1,014 kips 1,244 kips 1,309 kips 1,346 kips 1,361 kips	0.31 in 0.47 in 0.58 in 0.6 in 0.6 in 0.58 in 0.58 in 0.25 in	3,020 5,015 7,010 9,005 11,000 12,995 14,990 16,985 18,980	0 1,575 3,150 4,725 6,300 7,875 9,450 11,025 12,600	450 450 450 450 450 450 450 450	3,118 5,563 8,008 10,453 12,898 15,342 17,788 20,233 22,678	3,624.0 6,018.0 8,412.0 10,806.0 13,200.0 15,594.0 17,988.0 20,382.0 22,776.0	0.0 787.5 1,575.0 2,362.5 3,150.0 3,937.5 4,725.0 5,512.5 6,300.0	0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0	623.6 1,112.6 1,601.6 2,579.6 3,068.4 3,557.6 4,046.6 4,535.6	4,247.6 7,918.1 11,588.6 15,259.1 18,929.6 22,599.9 26,270.6 29,941.1 33,611.6	1.023 1.039 1.054 1.061 1.069 1.075 1.081 1.077 1.041

ଜ	iG Eng	jineeri	JOB NO	D. 9 - STOF	RY BUILI	DINGS			BY	SMG	DATE	9/16/04	S⊦	IEET NO.
	0	,	CUSTO	MER DESI	GN 9C				CKD		DATE			OF
SI	JBJECT		B2 CALC	ULATION	- FOR	BENL	DING A	ALONG	THE Y	-AXIS C	<b>F THE</b>	COLUMI	и мо	MENT FRAME MF A2 - F2
	0	LOAD	DUE TO FORC	1.2D + 0.5L + 1.0	DE TOTAL U	NFACTORE	D AXIAL L	OAD PER	TOTA			TP PEB STOR	Y	
			MOMENT FR.		STORY OF	N ALL COLL R" + "NON-L (kii	JMNS OF T EANER" C	HE STORY OLUMNS)	101/					STORY
	STORY NUMBER	LENGTH (STORY HEIGHT)	SHEAR DUE TO SEISMIC HORIZONTAL	INTERSTORY DRIFT	DEAD LOAD	LIVE	ROOF LIVE	SEISMIC WEIGHT	D.L.	LUAD F	ROOF L.L.	SEISMIC VERTICAL	TOTAL ΣP <sub>ui</sub> (kips)	B <sub>2i_Y-AXIS</sub>
	0	L 12.0.#	ΣH <sub>i</sub>	DUE TO ΣH <sub>i</sub>	DL	LL	LOAD Lr	DL + P-LL	1.2	0.5	0	$0.2S_{DS} = 0.2$	4 047 6	1 022
	8	13.0 ft	633 kips	0.31 m	5.015	1.575	450 450	5,563	5,624.0 6.018.0	787.5	0.0	1.112.6	4,247.0	1.023
	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

6	G Eng	jineeri	JOB NC	). 9 - STOR	Y BUILI	DINGS			BY	SMG	DATE	9/16/04	SI	HEET NO.
			CUSTO	MER DESI	GN 9C				CKD		DATE			OF
S	JBJECT			STABILI	түсс	DEFFIC	IENT	ALON	G COLU	IMN X-A	<b>ΧΙS</b> , θ <sub>x</sub>		М	OMENT FRAME MF A2 - F2
	0 0 0	LOAD C DEFLEC (SEISM MOMEN TOTAL	COMBINATION: CTION AMPLIFIC, IC) IMPORTANCE IT FRAME RESIS SEISMIC SHEAR	ATION FACTOR: FACTOR TS WHAT % OF T TO THE BUIDLIN	1.2D + 0. C <sub>d</sub> = I <sub>E</sub> =	5L + 1.0E 5.5 1.0 25%								
			DUE TO FORCE BENDING ALONG 1 MOMENT FRA	es that cause The <mark>X-axis</mark> of the Me columns	TOTAL U STORY OI		D AXIAL L	OAD PER HE STORY	ΤΟΤΑ	L FACTORED	AXIAL LOAD,	$\Sigma P_u$ , PER STO	RY	STABILITY
	STORY NUMBER	LENGTH (STORY HEIGHT) L	TOTAL STORY SHEAR DUE TO ANY HORIZONTAL LOAD	ELASTIC INTERSTORY DRIFT Aoh	DEAD	(kij LIVE LOAD	ROOF LIVE LOAD	SEISMIC WEIGHT	D.L.	LOAD F	ROOF L.L.	SEISMIC VERTICAL	TOTAL ΣP <sub>ui</sub> (kips)	COEFFICIENT PER STORY θ <sub>i</sub>
	9 8 7 6 5 4 3 2 1	13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft	ΣH <sub>i</sub> 380 kips 633 kips 844 kips 1,014 kips 1,146 kips 1,244 kips 1,309 kips 1,346 kips 1,361 kips	0.31 in 0.47 in 0.58 in 0.6 in 0.61 in 0.58 in 0.58 in 0.25 in 0.25 in	DL 3,020 5,015 7,010 9,005 11,000 12,995 14,990 16,985 18,980	LL 0 1,575 3,150 4,725 6,300 7,875 9,450 11,025 12,600	Lr 450 450 450 450 450 450 450	DL + P-LL 3,118 5,563 8,008 10,453 12,898 15,342 17,788 20,233 22,678	1.2 3,624.0 6,018.0 8,412.0 10,806.0 13,200.0 15,594.0 17,988.0 20,382.0 22,776.0	0.5 0.0 787.5 1,575.0 2,362.5 3,150.0 3,937.5 4,725.0 5,512.5 6,300.0	0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0	$0.2S_{D8} = 0.2$ 623.6 1,112.6 1,601.6 2,090.6 2,579.6 3,068.4 3,557.6 4,046.6 4,535.6	4,247.6 7,918.1 11,588.6 15,259.1 18,929.6 29,941.1 33,611.6	0.022 0.038 0.051 0.058 0.065 0.070 0.075 0.071 0.040

<b>ഭി</b> ദ്ദ Engineering	JOBI	NO. 9 -	STORY BUI	LDINGS		BY	SMG	DATE	9/16/04	SHEET NO.
66 Gm2mm8	CUST	OMER	DESIGN 90	;		CKD		DATE		OF
SUBJECT		ST	ABILITY C	OEFFICI	ENT ALO		JMN X-A	XIS, θ <sub>x</sub>		MOMENT FRAME MF A2 - F2
• LOAD COME	BINATION	:	1.2D +	0.5L + 1.0E						
o <b>DEFLECTIO</b>	N AMPLIF	CATION F	ACTOR: C <sub>d</sub>	= 5.5						
Г			DUE TO FORCE	S THAT CAUSE			r		<b>—</b>	
			BENDING ALONG T MOMENT FRA	HE X-AXIS OF THE WE COLUMNS	TOTAL STORY SHEAR	RATIO OF SHEAR	MAXIMUM			
-	STORY NUMBER	LENGTH (STORY HEIGHT) L	TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH <sub>i</sub>	$\begin{array}{c} \text{INTERSTORY} \\ \text{DRIFT} \\ & \Delta_{\text{oh}} \\ \text{DUE TO } \Sigma \text{H}_{\text{i}} \end{array}$	(OF ALL OF THE SEISMIC RESISTING MOMENT FRAMES)	DEMAND / SHEAR CAPACITY PER STORY β	ALLOWED STABILITY COEFFICIENT PER STORY θ <sub>i_max</sub>	STABILITY COEFFICIENT PER STORY θ <sub>i</sub>	COMMENT	
í T	9	13.0 ft	380 kips	0.31 in	57,421 kips	0.0066	0.250	0.022	ОК	
	8	13.0 ft	633 kips	0.47 in	57,421 kips	0.0110	0.250	0.038	ок	
	7	13.0 ft	844 kips 1 014 kips	0.58 in 0.6 in	10,145 kips	0.0832	0.250	0.051	ок ок	
	5	13.0 ft	1,146 kips	0.61 in	6,668 kips	0.1719	0.250	0.065	ок	
	4	13.0 ft	1,244 kips	0.6 in	6,192 kips	0.2009	0.250	0.070	ок	
	3	13.0 ft	1,309 kips	0.58 in	6,192 kips	0.2114	0.250	0.075	ок	
	2	13.0 ft	1,346 kips	0.5 in	6,192 kips	0.2174	0.250	0.071	ок	
	1	13.0 ft	1,361 kips	0.25 in	6,192 kips	0.2198	0.250	0.040	ок	
_										

GG	Eng	ineeri	JOB NC	9 - STOR	Y BUILI	DINGS			BY	SMG	DATE	9/16/04	SI	HEET NO.	
	0		CUSTO	MER DESI	GN 9C				CKD		DATE			OF	
SUB	JECT			STABILI	түсс	DEFFIC	IENT	ALON	g Colu	IMN Y-A	XIS, θ <sub>y</sub>		M	OMENT FRAME MF A2 - F2	
Г	0       LOAD COMBINATION:       1.2D + 0.5L + 1.0E         0       DEFLECTION AMPLIFICATION FACTOR:       Cg = 5.5         0       (SEISMIC) IMPORTANCE FACTOR       IE = 1.0         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 ("LENGTH" SHEAR DUE TO I PUTTOROUTION (Kips)       TOTAL FACTORED AXIAL LOAD, ΣP <sub>u</sub> , PER STORY (LOAD FACTOR       STAT COEFI														
S	STORY JMBER	LENGTH (STORY HEIGHT) L	MOMENT FRA TOTAL STORY SHEAR DUE TO ANY HORIZONTAL LOAD	ELASTIC INTERSTORY DRIFT 40h	("LEANER DEAD LOAD	LIVE	ROOF LIVE LOAD	SEISMIC WEIGHT	D.L.	LOAD F	ROOF L.L.	SEISMIC VERTICAL	TOTAL ΣP <sub>ui</sub> (kips)	STABILITY COEFFICIENT PER STORY θ <sub>i</sub>	
	9 8 7 6 5 4 3 2 1	13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft	ΣH <sub>i</sub> 380 kips 633 kips 844 kips 1,014 kips 1,244 kips 1,309 kips 1,346 kips 1,361 kips	0.31 in 0.47 in 0.58 in 0.6 in 0.61 in 0.58 in 0.58 in 0.55 in 0.25 in	3,020 5,015 7,010 9,005 11,000 12,995 14,990 16,985 18,980	0 1,575 3,150 4,725 6,300 7,875 9,450 11,025 12,600	Lr 450 450 450 450 450 450 450	3,118 5,563 8,008 10,453 12,898 15,342 17,788 20,233 22,678	3,624.0 6,018.0 8,412.0 10,806.0 13,200.0 15,594.0 17,988.0 20,382.0 22,776.0	0.0 787.5 1,575.0 2,362.5 3,150.0 3,937.5 4,725.0 5,512.5 6,300.0		0.29 <sub>05</sub> = 0.2 623.6 1,112.6 1,601.6 2,090.6 2,579.6 3,068.4 3,557.6 4,046.6 4,535.6	4,247.6 7,918.1 11,588.6 15,259.1 18,929.6 29,941.1 33,611.6	0.022 0.038 0.051 0.058 0.065 0.070 0.075 0.071 0.040	

ഭിട്ര Faainaariaa	JOBN	NO. 9 -	STORY BUI	LDINGS		BY	SMG	DATE	9/16/04	SHEET NO.				
ee constances and	CUST	OMER	DESIGN 90	;		CKD		DATE		OF				
SUBJECT	•	ST	ABILITY C	OEFFICI	ENT ALO		JMN Y-A	XIS, θ <sub>y</sub>		MOMENT FRAME MF A2 - F2				
• LOAD COMB	INATION	:	1.2D +	0.5L + 1.0E										
o <b>DEFLECTION</b>	o DEFLECTION AMPLIFICATION FACTOR: C <sub>d</sub> = 5.5													
Γ			DUE TO FORCE BENDING ALONG T MOMENT FRA	S THAT CAUSE HE <mark>Y-AXIS</mark> OF THE ME COLUMNS	TOTAL STORY SHEAR	RATIO OF SHEAR								
N	STORY IUMBER	LENGTH (STORY HEIGHT) L	TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH <sub>i</sub>	INTERSTORY DRIFT $\Delta_{oh}$ DUE TO $\Sigma H_i$	(OF ALL OF THE SEISMIC RESISTING MOMENT FRAMES)	DEMAND / SHEAR CAPACITY PER STORY β	ALLOWED STABILITY COEFFICIENT PER STORY θ <sub>i_max</sub>	COEFFICIENT PER STORY θ <sub>i</sub>	COMMENT					
	9	13.0 ft	380 kips	0.31 in	57,421 kips	0.0066	0.250	0.022	ок					
	8 7	13.0 ft	633 kips 844 kips	0.47 in 0.58 in	57,421 kips	0.0110	0.250	0.038	ок					
	6	13.0 ft	1,014 kips	0.6 in	6,668 kips	0.1521	0.250	0.058	ок					
	5	13.0 ft	1,146 kips	0.61 in	6,668 kips	0.1719	0.250	0.065	ок					
	4	13.0 ft	1,244 kips	0.6 in	6,192 kips	0.2009	0.250	0.070	ОК					
	3 2	13.0 ft	1,309 kips	0.58 in 0.5 in	6,192 kips 6,192 kips	0.2114	0.250	0.075	ок					
	1	13.0 ft	1,361 kips	0.25 in	6,192 kips	0.2198	0.250	0.040	ок					

COLUMN		MAXIMUM	CONTROLLING	BUILDING MAX.	COMMENTS FO	R: DESIGN 9C
NAME	MEMBER SIZE	INTERACTION VALUE	LOAD COMBINATION	INTERACTION 0.9104	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 0.3/5	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	0.403094007	6		OK STEEL AREA IS > 1% OF TOTAL AREA	
D2-4	HSS 25 x 25 x 0 2125	0.335405864	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	
D2-5	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
D2-0	HSS 23 × 23 × 0.3123	0.336370974	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	
D2-7	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 ABEA IS > 1% OF TOTAL ABEA	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 ABEA IS > 1% OF TOTAL ABEA	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!< td=""><td>OK - STEEL AREA IS &gt; 1% OF TOTAL AREA</td><td>N.G STEEL HSS IS NOT COMPACT!</td></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.



								D) (	0140	D.175	1/06/05				
GG Englineering	JOR NO. 1	18-51	υκγ Β	UILDIN	35			ВÅ	SMG	DATE	4/22/05	SHEET NO.			
6 6	CUSTOMER RCFT PARAMETRIC STUDY								CKD DATE _						
UBJECT	2-D	MON	IENT	FRAM	E (MF	A3 - G	i3] AN	ALYSI	S LOAD	SUMMA	RY				
[2] 2-D MOMENT FRA	ME GEOMETRY														
o TYPE C	F FRAME:						МОМ	ENT FRAM	E						
o DIRECT	ION THAT THE N	<i>I</i> OMENT	FRAME F	RUNS PARA		E-W	( THEREF	FORE, THIS M	OMENT FRAME	WILL RESIST					
o MOME	IT FRAME NAME	:					M	F A3 - G3	SEI	SMIC LOADS	IN THE E-W DIR	ECTION )			
o DISTAN To	ICE FROM THE C O THE SOUTHWE	CLOSES EST COF	T COLUMN	N STACK IN THE BUILDI	THE MOME	ENT FRAME	1	80.0 ft	THIS FRAME	IS LOCATED	IN THE INTERIC	R OF THE BUILDING			
o NUMBE	R OF BAYS IN T	HIS MON	MENT FRA	ME:				6 BAYS	AND THIS F	AND THIS FRAME SUPPORTS PART OF THE PENTHOUS					
o THE FIF W	RST BAY IN THIS HICH BAY NUME	MOMEN BER IN T	IT FRAME HE BUILD	IS THE SAI	ME BAY AS JT?			1_N-S	THIS MON	IENT FRAME	HAS A TOTAL L	ENGTH OF 120ft.			
o DISTAN o DISTAN	ICE TO THE CLO ICE TO THE CLO	SEST (G SEST (G	BRAVITY/M	IOMENT) FI IOMENT) FI	RAME: RAME ON T	HE OTHER	SIDE:	20.0 ft 20.0 ft							
	COORDII COLUMN IUMBER X	Co End NATES ( 1	A Column L Farthe Farthe DF THE M 2 X Y	of the MOM ocation: est West of est East 12 OMENT FR 3 × y	ENT FRAMI X Y 0.0 ft 80.0 20.0 ft 80.0 AME COLU 4 X y	E: ft ft MN STACK 5 X y	S WRT THE	southin Southin Northin Southe Souther Souther 7 X y	tion X Y rest 40.0 ft 40.0 ft rest 40.0 ft 80.0 ft rast 80.0 ft 80.0 ft rast 80.0 ft 40.0 ft /EST CORNER OF THE BUILDING						
	0	80	20 80	40 80	60 80	80 80	100 80	120 80				1			
o ASSUM THE (ST SU WF	ING THAT THE P FOLLOWING IS COLUMN (ACK) NO. COLUMN TACK) IS PPORTING IICH PART OF THE THOUSE?		USE PERI OR THIS M 2 NONE	METER IS A OMENT FR 3 EXTERIOR	ALWAYS LC AME: 4 EXTERIOR	5 EXTERIOR	6 NONE	/ITY/MOME 7 NONE	INT FRAME,						
					1	1		1		1					

66	) Engline	). Antina	JOB NO. 1	8-STORY B	UILDINGS		BY S	SMG	DATE	4/22/05	SHEET NO.
			CUSTOMER	RCFT PA	RAMETRIC	STUDY	CKD		DATE	E	OF
SUE	BJECT		2-D I	MOMENT	FRAME [	MF A3 - G3]	ANALYSIS	LOAD S	UMM	ARY	
	[3] BUILDI	NG DEAD I	OAD								
		o BUILI	Ding (Floors):	- COLUMNS - EXTERIOF - FLOORING - COMPOSI - CEILING (F - HVAC + EI	(Applied to Su	rface Area of the WALL					
		o BUILI	Ding (Roof):	<ul> <li>PARAPET</li> <li>ROOFING</li> <li>COMPOSI</li> <li>(ROOF) BE</li> <li>CEILING (F</li> <li>HVAC + EI</li> </ul>	TE ROOF SYSTE EAMS, GIRDERS, FROM STORY BE LECTRICAL (FRO	(Applied to Su	(Applied to Surface Area of the WALL)				
		o PEN	THOUSE:	- COMPOSI - CEILING + - EXTERIOF - COLUMNS - MECHANIG - FLOORING	TE ROOF SYSTE FIREPROOFING WALLS & BEAMS, GIRDE CAL EQUIPMENT	(Applied to Su	rface Area of the WALL				
	NUM	BER	(STORY)	D.L. (SURF	ACE) AREA	DEAD L	OAD	TO	TAL DEAD	D LOAD	TOTAL STORY DL
	STORY	FLOOR	HEIGHT	ROOF / FLOOR	PENTHOUSE	ROOF / FLOOR	PENTHOUSE	ROOF / FL	OOR	PENTHOUSE	ΣΡ <sub>i</sub>
					1,600 ft <sup>2</sup>		145.50 lb/ft <sup>2</sup>			232.8 kips	
	18	ROOF	13.0 ft	14,400 ft <sup>2</sup>		88.92 lb/ft <sup>2</sup>		1,280.45	kips		1 513 kins
	17	18	13.0 ft	14,400 ft <sup>2</sup>		90.83 lb/ft <sup>2</sup>		1,307.95	kips		2 821 kins
	16	17	13.0 ft	14,400 ft <sup>2</sup>		90.83 lb/ft <sup>2</sup>		1,307.95	kips		4 129 kins
	15	16	13.0 ft	14,400 ft <sup>2</sup>		90.83 lb/ft <sup>2</sup>		1,307.95	kips		5 437 kips
	14	15	13.0 ft	14,400 ft <sup>2</sup>		90.83 lb/ft <sup>2</sup>		1,307.95	kips		6 745 kips
	19	14	12.0 ft	14,400 ft <sup>2</sup>		90.83 lb/ft <sup>2</sup>		1,307.95	kips		8,052 kips
	10	13	12.0 #	14,400 ft <sup>2</sup>		90.83 lb/ft <sup>2</sup>		1,307.95	kips		0,003 kips
	12	12	12.0 #	14,400 ft <sup>2</sup>		90.83 lb/ft <sup>2</sup>		1,307.95	kips		9,301 Kips
	11	11	13.011	14,400 ft <sup>2</sup>		90.83 lb/ft <sup>2</sup>		1,307.95	kips		10,669 kips
	10	10	13.0 ft	14,400 ft <sup>2</sup>		90.83 lb/ft <sup>2</sup>		1,307.95	kips		11,977 kips
	9	9	13.0 ft	14,400 ft <sup>2</sup>		90.83 lb/ft <sup>2</sup>		1,307.95	kips		13,285 kips
	0 7	8	12.0 ft	14,400 ft <sup>2</sup>		90.83 lb/ft <sup>2</sup>		1,307.95	kips		14,593 kips
	1	7	13.011	14,400 ft <sup>2</sup>		90.83 lb/ft <sup>2</sup>		1,307.95	kips		15,901 kips
	6	6	13.0 ft	14,400 ft <sup>2</sup>		90.83 lb/ft <sup>2</sup>		1,307.95	kips		17,209 kips
	5	5	12.0 ft	14,400 ft <sup>2</sup>		90.83 lb/ft <sup>2</sup>		1,307.95	kips		10,517 kips
	4	4	12.0 ft	14,400 ft <sup>2</sup>		90.83 lb/ft <sup>2</sup>		1,307.95	kips		21,122 kips
	2	3	12.0 #	14,400 ft <sup>2</sup>		90.83 lb/ft <sup>2</sup>		1,307.95	kips		21,133 kips
	2	2	12.0 #	14,400 ft <sup>2</sup>		90.83 lb/ft <sup>2</sup>		1,307.95	kips		22,441 Kips
				N.A.		N.A.					
I								1	De 11	NOT ! !	00.710.11
						Building Total D	ead Load (Ground F	loor + 1st Story	Dead Loa	d NOT Included) =	23,748 kips

ලිලි විකැ	Ineerik	JOI	3 NO. 18-ST	ORY BUILD	DINGS		BY SMG	DATE 4,	/22/05	SHEET NO.				
		CU	STOMER R	CFT PARAM	DATE		OF							
SUBJECT			2-D MO		AME [MF A3	- G3] ANA	LYSIS LOAD	SUMMAR	1					
[4] B	[4] BUILDING LIVE LOAD													
o BUILDING (FLOORS):       - OFFICE BUILDING OCCUPANCY PER IBC 2003, TABLE 1607.1       50 lb/lt²         - (MOVEABLE) PARTITIONS PER IBC 2003, SECTION 1607.5       20 lb/lt²														
	οB	BUILDING	20 lb/ft <sup>2</sup> ( /	ROOF LIVE LOAL	), Lr)									
	O PENTHOUSE:     GENERAL PENTHOUSE (INTERIOR) LIVE LOAD     PENTHOUSE (ROOF) LIVE LOAD     O Ib/ft <sup>2</sup> (TREATED AS ROOF LIVE LOAD, Lr)     O Ib/ft <sup>2</sup> (ROOF LIVE LOAD, Lr)													
NUM	IRFR (		L L. (SUBF)		LIVEL	۵۵۵	TOTAL LIVE		TOTAL ST	ORY LL. ΣP;				
STORY	FLOOR	HEIGHT	ROOF / FLOOR	PENTHOUSE	ROOF / FLOOR	PENTHOUSE	ROOF / FLOOR	PENTHOUSE	FLOOR LL	ROOFLL				
				1,600 ft <sup>2</sup>		20 lb/ft <sup>2</sup>		32.0 kips						
	ROOF		12,800 ft <sup>2</sup>		20 lb/ft <sup>2</sup>		256.0 kips							
18	18	13.0 ft	14,400 ft <sup>2</sup>		70 lb/ft <sup>2</sup>		1,008.0 kips		0 kips	288 kips				
17	17	13.0 ft	14,400 ft <sup>2</sup>		70 lb/ft <sup>2</sup>		1,008.0 kips		1,008 kips	288 kips				
16	16	13.0 ft	14,400 ft2		70 lb/ft <sup>2</sup>		1,008.0 kips		2,016 kips	288 kips				
15	15	13.0 ft	14,400 ft2		70 lb/ft <sup>2</sup>		1,008.0 kips		3,024 kips	288 kips				
14	14	13.0 ft	14,400 ft2		70 lb/ft <sup>2</sup>		1,008.0 kips		4,032 kips	288 kips				
13	13	13.0 ft	14,400 ft2		70 lb/ft <sup>2</sup>		1,008.0 kips		5,040 kips	288 kips				
12	12	13.0 ft	14,400 ft2		70 lb/ft <sup>2</sup>		1,008.0 kips		6,048 kips	288 kips				
11	11	13.0 ft	14,400 ft2		70 lb/ft <sup>2</sup>		1,008.0 kips		7,056 kips	288 kips				
10	10	13.0 ft	14,400 ft2		70 lb/ft <sup>2</sup>		1,008.0 kips		8,064 kips	288 kips				
9	9	13.0 ft	14,400 ft <sup>2</sup>		70 lb/ft <sup>2</sup>		1,008.0 kips		9,072 kips	288 kips				
8	8	13.0 ft	14,400 ft <sup>2</sup>		70 lb/ft <sup>2</sup>		1,008.0 kips		10,080 kips	288 kips				
7	7	13.0 ft	14,400 ft <sup>2</sup>		70 lb/ft <sup>2</sup>		1,008.0 kips		11,088 kips	288 kips				

12,096 kips

13,104 kips

14,112 kips

15,120 kips

16,128 kips

17,136 kips

17,136 kips

----

----

----

----

----

288 kips

6

5

4

3

2

1

6

5

4

3

2

GROUND

13.0 ft

13.0 ft

13.0 ft

13.0 ft

13.0 ft

13.0 ft

14,400 ft<sup>2</sup>

14,400 ft<sup>2</sup>

14,400 ft<sup>2</sup>

14,400 ft<sup>2</sup>

14,400 ft<sup>2</sup>

N.A.

----

----

----

----

----

----

70 lb/ft<sup>2</sup>

70 lb/ft<sup>2</sup>

70 lb/ft<sup>2</sup>

70 lb/ft<sup>2</sup>

70 lb/ft<sup>2</sup>

N.A.

----

----

----

----

----

1,008.0 kips

1,008.0 kips

1,008.0 kips

1,008.0 kips

1,008.0 kips

N.A.

Building Total Live Load and Roof Live Load =

GG Engineerir	JOB I	NO. 18-	STORY	BUILDI	NGS	IGS			SMG	DATE	4/22/05	SHEET NO.	
	CUST	TOMER	RCFT F	PARAMI	ETRIC S	TUDY		CKE	)	DATE		<sup>OF</sup>	
SUBJECT 2-D MOMENT FRAME [MF A3 - G3] ANALYSIS LOAD SUMMARY													
[5] MOMENT FRA	o Si	OAD JMMARY: IEREFORE NOT	NUM NUM DIRE LOC/ DOE: DIST. DIST. DIST. DEAI PAR.	BER OF ST BER OF BA CTION THA ATION OF T S THIS FRA ANCE TO T ANCE TO T D LOAD TRI APET DEAL	ORIES, N <sub>S</sub> YS, N <sub>B</sub> T THE <i>MOM</i> HE MOMEN ME SUPPO HE CLOSES HE CLOSES HE CLOSES BUTARY W D LOAD (PE D LOAD (PE	IENT FRAM T FRAME V RT PART O ST (GRAVIT ST (GRAVIT ST (GRAVIT NDTH TO TH R ft° OF RO R FOOT OF	IE RUNS PA VRT THE BL F THE PEN' Y/MOMENT) Y/MOMENT) IIS MOMENT OF SURFAC PARAPET	RALLEL W IILDING PE THOUSE GI FRAME: FRAME OF FRAME: E AREA) .ENGTH)	ITH: RIMETER: RAVITY LOADS? N THE OTHER SIDE	18 STORIE 6 BAYS E-W INTERIOF YES 20.0 ft 20.0 ft 2.92 lb/ft <sup>2</sup> 87.5 lb/ft	S S DOR/LEVEL		
<u>(</u> F	COLUMN STACK) NO. PENTHOUSE	1	2	3	4	5	6	7					
	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		UN	IFACTOREI	O (NOMINAL	.) DEAD LO	AD BEAM E	ND REACT	TIONS TO EACH CO	DLUMN			
	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 9.1 kips	18.2 kips 18.2 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips 18.2 kips	9.1 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 12	9.1 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips 18.2 kips	18.2 kips	9.1 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 7	9.1 kips 9.1 kips	18.2 kips 18.2 kips	18.2 kips 18.2 kips	18.2 kips 18.2 kips	18.2 kips 18.2 kins	18.2 kips 18.2 kips	9.1 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			<b>↓↓</b>		
	2	9.1 kips 9.1 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips	18.2 kips	9.1 kips 9.1 kips					
	GROUND	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.	N.A.					
											<b>├</b> ── <b>│</b>		
											+		
L													



ශිශි පිතැබ්තලලාබ්ග	JOB । ରମ	NO. 18-STORY BUILDINGS							SMG	I	DATE	4/22/05	SHEET NO.
oo tuitimeenii	CUST	FOMER	RCFT F	PARAM	ETRIC S	TUDY		CKE	)	I	DATE		OF
SUBJECT		2-D M	OMEN	T FRA	ME [M	F A3 -	G3] AI	NALYS	SIS LOA	d su	MMAF	٩Y	
[7] MOMENT FRA	ME LIVE LC	AD											
	o Si	JMMARY: IEREFORE		1 S? SIDE:	8 STORIES 6 BAYS E-W INTERIOR YES 20.0 ft 20.0 ft <i>fLO</i>	S OR/LEVEL							
			- <del>4</del> -	생 생 생					-¥-		- <b>4</b> -		
(	COLUMN STACK) NO.	1	2	3	4	5	6	7					
F	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		U	NFACTORE	D (NOMINA OF THE M	L) LIVE LO. IOMENT FI	AD BEAM E RAME AT E'	ND REACT	IONS TO EACH DR/LEVEL	H COLUM	IN		
-	ROOF	2.0 kips	4.0 kips	5.0 kips	6.0 kips	5.0 kips	4.0 kips	2.0 kips					
	17	7.0 kips 7.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips	14.0 kips	7.0 kips					
	16 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 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 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	7.0 kips 7.0 kips	14.0 kips 14.0 kips	14.0 kips 14.0 kips	14.0 kips 14.0 kips	14.0 kips 14.0 kips	14.0 kips 14.0 kips	7.0 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					
-	3	7.0 kips 7.0 kips	14.0 kips 14.0 kips	14.0 kips 14.0 kips	14.0 kips 14.0 kips	14.0 kips 14.0 kips	14.0 kips 14.0 kips	7.0 kips 7.0 kips					
	2 GROUND	7.0 kips N.A.	14.0 kips N.A.	14.0 kips N.A.	14.0 kips N.A.	14.0 kips N.A.	14.0 kips N.A.	7.0 kips N.A.					
-													


GG Engineerir	JOB । ଲିଷ୍	NO. 18-	STORY	BUILDI	NGS			BY	SMG	DATE	4/22/05	SHEET NO.
	CUST	FOMER	RCFT F	PARAMI	ETRIC S	TUDY		CKE	)	DATE		OF
SUBJECT		2-D M(	OMEN	T FRA	ME (M	F A3 -	G3] AI	NALYS	SIS LOAD	SUMMA	RY	
[9] MOMENT FR	<b>ame seismi</b> o Si o Tr	2-D M C WEIGHT ( JIMMARY: HEREFORE NOT.	DEAD LOAN NUM NUM DIRE LOC, DOE DIST DIST CSEIS E: PAR. PAR PAR	T FRA D + PARTIT BER OF ST BER OF BA CTION THA ATION OF T S THIS FRA ANCE TO T ANCE TO T MIC WEIGH APET SEISI APET SEISI TITION LIVE	ME [M ION LIVE L ORIES, N <sub>S</sub> YS, N <sub>B</sub> T THE <i>MOM</i> HE MOMEN HE CLOSES HE CLOSES HE CLOSES IT TRIBUTAI MIC WEIGH SCLOAD	F A3 - OAD) MENT FRAME V RT PART O ST (GRAVIT ST (GRAVIT RY WIDTH <sup>+</sup> T (PER fr <sup>2</sup> C T (PER FOC	G3] AI	RALLEL W ILDING PE HOUSE G FRAME O MENT FRA RFACE AF PET LENG	AITH: RIMETER: RAVITY LOADS? N THE OTHER SID ME: REA) TTH)	18 STORIE 6 BAYS E-W INTERIOF YES 20.0 ft 20.0 ft 20.0 ft 20.0 ft 20.0 ft 20.0 ft <i>20.2 lb/ft</i> <sup>2</sup>	RY 55 3 2 DOR/LEVEL	
г	COLUMN		₩		Щ. 	_			₩ 			
	(STACK) NO.	1	2	3	4	5	6	1			<u> </u>	
	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		UNFA	CTORED (I	NOMINAL) S	EISMIC WI	EIGHT BEAI	I END RE	ACTIONS TO EACH	I COLUMN		
	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 16	11.1 kips 11.1 kips	22.2 kips 22.2 kips	22.2 kips 22.2 kips	22.2 kips 22.2 kips	22.2 kips 22.2 kips	22.2 kips 22.2 kips	11.1 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				
	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				
	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				
		11.1 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips	22.2 kips	11.1 kips				
	GHOOND	14.75	19.75	14.75	1.0.	14.5	1	N.A.				
-												



ශිශි පිතක්	അമ്പങ്ങ	JOB NO. 18-STORY BUILDINGS		BY	SMG	DATE 4	/22/05	SHE	ET NO.
ee riiĝi	meand	CUSTOMER RCFT PARAMETRIC	C STUDY	CKD		DATE		(	OF
SUBJECT		2-D MOMENT FRAME	MF A3 - G3] AN	ALYSI	S LOAD S	UMMAR	Y .		
[11] \$	EISMIC LOADS	PER THE INTERNATIONAL BUILDING CODE (IBC	C) 2003 [ ASCE 7 ]						
	PER THE EX	CEPTION OF SECTION 1614 OF THE IBC 2003, AS RE, THE FOLLOWING DESIGN PARAMETERS ARI	CE 7 IS PERMITTED TO BE U E TAKEN FROM ASCE 7-02.	SED TO D	ETERMINE THE S	EISMIC LOADS.	Section 1614		(p. 302)
	OCCUPANCY	CATEGORY	п				Table 1-1		(p. 4)
	SEISMIC USE	GROUP	Ι				Section 9.1.3 Table 9.1.3		(p. 96) (p. 96)
	DOCCUPANCY	IMPORTANCE FACTOR, I <sub>E</sub>	1.00				Section 9.1.4 Table 9.1.4		(p. 96) (p. 97)
	D THE MAPPED	9 SPECTRAL ACCELERATIONS:	FOR SHORT PERIODS	$S_{S} = 1.5 g$ $S_{1} = 0.6 g$	]		Section 9.4.1.1 Figure 9.4.1.1	2 (c) & (	(p. 107) d)
	0 SITE CLASS			D			Section 9.4.1.2 Note the "Exce	2.1 eption"	(p. 108)
	SITE COEFFI	CIENTS:		$F_a = 1.0$ $F_v = 1.5$			Table 9.4.1.2.4 Table 9.4.1.2.4	4a 4b	(p. 129) (p. 130)
	MAX. CONSIE	DERED SPECTRAL RESPONSE ACCELERATIONS	: FOR SHORT PERIODS, FOR A 1-SECOND PERIOD,	S <sub>MS</sub> = 1.5 g S <sub>M1</sub> = 0.9 g	3		Equation 9.4.1 Equation 9.4.1	1.2.4-1 1.2.4-2	(p. 129) (p. 129)
	DESIGN SPE	CTRAL RESPONSE ACCELERATIONS:	FOR SHORT PERIODS, FOR A 1-SECOND PERIOD,	$S_{DS} = 1.0 g$ $S_{D1} = 0.6 g$	3		Equation 9.4.1 Equation 9.4.1	1.2.5-1 1.2.5-2	(p. 129) (p. 129)
	SEISMIC DES	SIGN CATEGORY	D				Table 9.4.2.1a Table 9.4.2.1b	L )	(p. 131) (p. 132)
(	D BASIC SEISM	IIC-FORCE RESISTING SYSTEM	"SPECIAL COMPOSITE MOM	ENT FRAM	IES"		Table 9.5.2.2		(p. 134)
	RESPONSE M	IODIFICATION COEFFICIENT, R	8				Table 9.5.2.2		(p. 134)
	DEFLECTION	AMPLIFICATION FACTOR, $C_d$	5.5				Table 9.5.2.2		(p. 134)
c c	D FUNDAMENT	AL PERIOD, T					Section 9.5.5.	3	(p. 147)
	BUILDING PE	RIOD COEFFICIENTS:	$C_{T} = 0.028$ x = 0.8				Table 9.5.5.3.2 Table 9.5.5.3.2	2 2	(p. 147) (p. 147)
	ELEVATION C	DF THE BUILDING ROOF ABOVE THE BASE, $h_n$	234 ft						
	COEFFICIEN	FOR UPPER LIMIT ON CALCULATED PERIOD, $C_{\rm I}$	J 1.4				Table 9.5.5.3.	1	(p. 147)
	APPROXIMAT	TE FUNDAMENTAL PERIOD, T <sub>a</sub>	2.201 sec				Equation 9.5.5	5.3.2-1	(p. 147)
	PERIOD FRO	M RATIONAL ANALYSIS, T <sub>R</sub>	NONE CALCULATE	Ð					
	MAXIMUM AL	LOWED PERIOD, C <sub>U</sub> T <sub>a</sub>	3.081 sec				Section 9.5.5.	3	(p. 147)
	DESIGN PER	IOD, T	2.201 sec				Section 9.5.5.	3	(p. 147)
(	0 ANALYSIS PF	ROCEDURE	EQUIVALENT LATERAL FOR	CE ANAL	(SIS		Section 9.5.2. Table 9.5.2.5.	5.1 1	(p. 139) (p. 140)
	D SEISMIC RES	PONSE COEFFICIENT, C <sub>S</sub>	$C_{S} = 0.125 \text{ g}$				Equation 9.5.5	5.2.1-1	(p. 146)
			$C_S \leq \ 0.035 \ g$				Equation 9.5.5	5.2.1-2	(p. 146)
			$C_S \ge 0.044 \text{ g}$				Equation 9.5.5	5.2.1-3	(p. 146)
			$DESIGN C_S = 0.044 g$						
	o SEISMIC (DE	SIGN) BASE SHEAR, V	V = 0.044 W				Equation 9.5.5	5.2-1	(p. 146)

<b>ഭി</b> ശി (Enorineerino)	JOB NO. 18-STORY BUILDINGS	BY SMG	DATE 4/22/05	SHEET NO.
CC CIURIng Con multiple	CUSTOMER RCFT PARAMETRIC STUDY	СКД	DATE	OF
SUBJECT	2-D MOMENT FRAME [MF A3 - G3	] ANALYSIS LOAD S	SUMMARY	
[12] VERTICAL DIST	IBUTION OF THE SEISMIC LOADS PER ASCE 7-02 [IBC 2003]		Section Equation	9.5.5.4 (p. 148) n 9.5.5.4-1 (p. 148)
	o BUILDING FUNDAMENTAL (DESIGN) PERIOD, T	T 2.201 sec	Lynano	19.0.0.4-2 (p. 1-0)
	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 BUILDIN	NG, V 1,263.8 kips		

NUM	1BER	STORY	FLOOR	DEAD	LOAD	(MOVEABLE P	ARTITION) L.L.	(SEISMIC)	SEISMIC WT.	w,h, <sup>k</sup>	0	F
STORY	FLOOR	HEIGHT	ELEVATION, h <sub>x</sub>	ROOF / FLR	PENTHOUSE	ROOF / FLR	PENTHOUSE	WEIGHT W <sub>x</sub>	FLOOR	(kip-ft)	U <sub>vx</sub>	Fx
	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
I	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
	I		I	23,826	.4 kips	4,896	.0 kips	28,722.4 kips		2.630E+08		1,263.6 ki
				- 10-0	1	,	P	,				/

ඳිලි පිතක්ග	രെഷ്ണന	JOB NC	). 18-S <sup>−</sup>	FORY B	UILDIN	GS			BY	SMG	DAT	TE 4/2	2/05	SHE	ET NO.
oo riitiin	igen iniĝi	CUSTO	MER R	CFT PA	RAMET	RIC ST	UDY		CKD		DAT	Ē		(	OF
SUBJECT		2-	D MO	MENT	FRAM	E [MF	A3 - G	i3] AN/	ALYSI	S LOAD	SUM	MARY			
[13] RE	DUNDANCY C	OEFFICIEN	IT, ρ, PER /	ASCE 7-02	[ IBC 2003 ]	I							Section 9	9.5.2.4	(p. 138)
		0	SEISMIC D	ESIGN CAT	EGORY			D							
		0	ARE THER	E ONLY SF	PECIAL MON	IENT FRAM	IES?	YES							
		0	REDUNDA	NCY COEF	FICIENT, p		r <sub>i</sub>	=2 - 20					Equation	9.5.2.4.2-1	(p. 138)
							ρ <sub>ma</sub>	<sub>x</sub> = -0.20	(FRC	OM THE TABLE	BELOW)				
		0	(DESIGN)	REDUNDAN	ICY COEFF	ICIENT, ρ	1	) = 1.00							
STORY	AREA OF	9	EISMIC SH	IEAR IN EA	CH COLUM	N OF THE I	MOMENT F	RAME FOR	EVERY ST	ORY OF THE E	UILDING	1	DESIGN		
NUMBER	THE DIAPHRAGM	COL 1	COL 2	COL 3	COL 4	COL 5	COL 6	COL 7					STORY SHEAR	r <sub>max_i</sub>	$\rho_{i}$
18	14,400 ft <sup>2</sup>	1 kips	7 kips	10 kips	10 kips	10 kips	7 kips	1 kips				1	184.8 kips	0.0758	-0.1988
17	14,400 ft <sup>2</sup>	6 kips	14 kips	16 kips	16 kips	16 kips	14 kips	6 kips				3	351.5 kips	0.0637	-0.6164
16	14,400 ft <sup>2</sup>	10 kips	20 kips	22 kips	22 kips	22 kips	20 kips	10 kips				Ę	500.5 kips	0.0615	-0.7100
15	14,400 ft <sup>2</sup>	12 kips	26 kips	28 kips	28 kips	28 kips	26 kips	12 kips				6	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				1	749.1 Kips	0.0617	-0.7012
12	14,400 ft <sup>2</sup>	18 kips	38 kips	41 kips	41 kips	41 kips	38 kips	18 kips					938.1 kips	0.0612	-0.7233
11	14,400 ft <sup>2</sup>	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 ft <sup>2</sup>	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 ft <sup>2</sup>	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 ft <sup>2</sup>	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 ft <sup>2</sup>	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 ft <sup>2</sup>	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 ft <sup>2</sup>	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 π <sup>2</sup>	25 kips	52 kips	53 kips	53 Kips	53 Kips	52 kips	25 Kips				1,	,252.8 kips	0.0592	-0.8153
2	14,400 ft <sup>2</sup>	30 kips	51 kips	51 kips	51 kips	51 kips	51 kips	30 kips				1	,259.5 kips	0.0565	-0.9499
1	14.400 ft <sup>2</sup>	36 kips	48 kips	49 kips	49 kips	49 kips	48 kips	36 kips				1.	,263.6 kips	0.0551	-1.0248
1															
_															

ශීශී (Enolineering	JOB NO. 18-STORY	/ BUILDINGS		BY	SMG	DATE	4/22/05	SHEET NO.
	CUSTOMER RCFT	PARAMETRIC S	STUDY	CKD		DATE		OF
SUBJECT	2-D MOMEN	IT FRAME [M	F A3 - G3] A	NALYS	IS LOAD S	SUMMA	RY	
[14] WIND LOADS F	PER THE INTERNATIONAL BUIL	DING CODE (IBC) 2003	[ASCE 7]					
o PER SECTI THEREF	ON 1609.1.1 OF THE IBC 2003, S ORE, THE FOLLOWING DESIGN	SECTION 6 OF ASCE 7 S I PARAMETERS ARE TA	SHALL BE USED TO L KEN FROM ASCE 7-1	DETERMINE T 02.	HE WIND LOADS.		Section 16	)9 (p. 283)
o OCCUPANO	Y CATEGORY		п				Table 1-1	(p. 4)
o IMPORTANO	CE 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		В				Section 6.5	.6.3 (p. 28)
o WIND DIRE	CTIONALITY FACTOR, K <sub>d</sub> , FOR M	WWFRS OF A BUILDING	0.85				Table 6-4	(p. 76)
0 TOPOGRAP	HIC FACTOR, K <sub>zt</sub>		1.0				Section 6.5 Equation 6	.7.2 (p. 30) -3 (p. 30)
0 ENCLOSUR	E CLASSIFICATION	ENCLOSED - SINCE N THAT WIND BORNE D	NOT IN A HURICANE	REGION AND RATE THE WIN	THERE IS A SMALI IDOWS AND CLAD	L CHANCE DING.	Section 6.5 Section 6.2	.9 (p. 30) (p. 23)
o BUILDING T	YPE	SIMPLE DIAPHRAGM	- WIND LOADS ARE TO THE MWFRS (MO	TRANSFERRE	ED THROUGH THE ES).	ROOF AND	Section 6.2	(p. 24)
<ul> <li>APPROXIM/ APPROXIM/</li> </ul>	ATE BUILDING (MAX ALLOWED ATE BUILDING FREQUENCY, n1	) PERIOD	3.081 sec 0.325 Hz	(FROM SEI SINCE n1 <	SMIC CALCULATIO 1.0 THE BUILDING	NS) IS FLEXIBLE	Section 6.2	(p. 24)
o DIRECTION	THAT THE MOMENT FRAME RU	JNS PARALLEL WITH:	E-W					
o BUILDING V	VIDTH (DIMENSION PERPENDIC	CULAR TO WIND DIREC	TION), 120.0 ft	ALONG TH	E E-W FACE			
o BUILDING D	EPTH (DIMENSION PARALLEL	TO WIND DIRECTION), L	120.0 ft	ALONG TH	E N-S FACE			
o MEAN ROO	F HEIGHT ABOVE GRADE, h		234.0 ft					
0 VELOCITY F	PRESSURES		qz = 15.72 Kz lb qh = 19.81 lb/f	/ft²			Section 6.5 Equation 6	.10 (p. 31) -15 (p. 31)
0 GUST EFFE	CT FACTOR, G <sub>f</sub>		From Table Q				Section 6.5	.8.2 (p. 30)
			$\begin{array}{c} \mbox{From Table 6}\\ \hline c = 0.30 & z_{min} = 3\\ \hline \alpha = 1/4 & \overline{\epsilon} = \\ \hline b = 0.45 & \ell = 3\\ \alpha = 7.0 & z_g = 3 \end{array}$	2 30 ft 1/3 320 ft 1200 ft				
			$\begin{array}{c} g_Q = g_V = 3.4 \\ g_R = 3.912 \\ \overline{z} = 140.4 \\ I_{\overline{z}} = 0.236 \\ \overline{V}_{\overline{z}} = 80.57 \\ L_{\overline{z}} = 518.52 \end{array}$	ft ft/sec 2			Section 6.5 Equation 6 Section 6.5 Equation 6 Equation 6 Equation 6	.8.2 9 8.1 5 - 14 -7
			$\begin{array}{l} \eta_{B}=2.227 & R_{B}=0 \\ \eta_{L}=2.227 & R_{L}=0 \\ \end{array}$ $\begin{array}{l} N_{1}=2.092 \\ R_{n}=0.087 \end{array}$	).349 ).349			Equation 6 Equation 6	-12 -11
		Assumed Critical Dam Resonant Respon Background F	ping Ratio, $\beta = 0.05$ ise Factor, R = 0.293 Response, Q = 0.818				Equation 6 Equation 6	-10 -6
		Gust Effe	ct Factor, G <sub>f</sub> = 0.863				Equation 6	-8

രദ	monther	voorđena	JOB NC	D. 18-S	FORY B	UILDIN	GS				BY	SMG		DAT	E 4/	/22/05	SHE	ET NO.
ଏଜ ଜ	niĝin	iraunia	CUSTO	MER R	CFT PA	RAMET	RIC ST	UDY			CKD			DAT	E			OF
SUBJE	СТ		2-	D MOI	MENT	FRAM	E [MF	A3 -	G3]	ANAI	YSI	S LO	AD S	SUMN	IARY	/		
[ 15	] WI	ND LOADS CO	ONTINUED															
	0	WALL EXTER	NAL PRES	SURE COE	FFICIENTS,	C <sub>p</sub>	SURFAC	E	L/B	0	) <sub>p</sub>					Section 6.5	.11.2.1	(p. 31)
	N	Plus signs sign egative signs sigr	ify pressures anify pressures	acting towards acting away fi	the surface. rom the surfac	ce. L	INDWARD SIDE WAL EEWARD V	WALL LS VALL	ALL 1.00 ALL	0 -0 -0	.8 1.5 1.7					Figure 6-6		(p. 51)
	o	ROOF EXTER	NAL PRES	SURE COE	FFICIENT, C	C <sub>p</sub>		FOR:	<i>θ</i> < 10°							Section 6.5	.11.2.1	(p. 31)
			h = 234.0	D ft D ft			DIST/ LEAI	ANCE FR	OM GE	C <sub>p</sub>						Figure 6-6		(p. 51)
		h	/L = 1.950	) (Assume	h/L > 1.0 )		0 1	oh/2		-1.3								
		h	/2 = 11/.0	) ft			>	h/2		-0.7								
	0	INTERNAL PR	ESSURE C	OEFFICIEN	T, GC <sub>pi</sub>			GC <sub>pi</sub> =	± 0.18							Section 6.5 Figure 6-5	.11.1	(p. 31) (p. 49)
<b>—</b>		FLOOR LEVE	L WIND LOAD	S PER EQUA	TION 6-19 AN	D PARAPET I	VIND LOADS	PER EQU	ATION 6-2	0		WINDWA	RD WALL	LEEWAR	RD WALL			DESIGN
FLC	DOR	ELEVATION	(B	BUILDING) WIN	ND TRIBUTAR	Y PARAMETE	RS	к.	qz	ĸ	q <sub>h</sub>	PRESSU	JRE (psf) TH	PRESSI W	JRE (psf) ITH	TOTAL WI	ND SHEAR TH	WIND SHEAR
NUM	IBER	LEEVATION	TOP ELEV.	BOT. ELEV.	HEIGHT	WIDTH	AREA		(psf)	· · ·	(psf)	$(+GC_{pi})$	$(-GC_{pi})$	$(+GC_{pi})$	$(-GC_{pi})$	$(+GC_{pi})$	$(-GC_{pi})$	PER FLOOR
PENTH			247.0 ft	234.0 ft	13.0 ft	40.0 ft	520 ft <sup>2</sup>	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
RO	OF	234.0 ft	237.5 ft 234.0 ft	234.0 ft	6.5 ft	120.0 ft	420 ft <sup>2</sup>	1.27	19.97	1.26	19.81	10.11	17.24	-15.53	-10.0	24.3 kips 20.0 kips	24.3 kips 21.2 kips	24.3 kips 21.2 kips
1	8	221.0 ft	227.5 ft	214.5 ft	13.0 ft	120.0 ft	1,560 ft <sup>2</sup>	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
1	7	208.0 ft	214.5 ft	201.5 ft	13.0 ft	120.0 ft	1,560 ft <sup>2</sup>	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
1	6	195.0 ft	201.5 ft	188.5 ft	13.0 ft	120.0 ft	1,560 ft <sup>2</sup>	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
1	5	182.0 ft	188.5 ft	175.5 ft	13.0 ft	120.0 ft	1,560 ft <sup>2</sup>	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
1	4	169.0 ft	175.5 ft	162.5 ft	13.0 ft	120.0 ft	1,560 ft <sup>2</sup>	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
1	3	156.0 ft	162.5 ft	149.5 ft	13.0 ft	120.0 ft	1,560 ft <sup>2</sup>	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
	1	143.0 ft	136.5 ft	123.5 ft	13.0 ft	120.0 ft	1,560 ft <sup>2</sup>	1.08	16.98	1.26	19.61	10.0	15.29	-15.53	-10.0	39.8 kips	40.0 kips	40.0 kips 39.5 kips
1	0	117.0 ft	123.5 ft	110.5 ft	13.0 ft	120.0 ft	1,560 ft <sup>2</sup>	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	e e	104.0 ft	110.5 ft	97.5 ft	13.0 ft	120.0 ft	1,560 ft <sup>2</sup>	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
٤	3	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	7	78.0 ft	84.5 ft	71.5 ft	13.0 ft	120.0 ft	1,560 ft <sup>2</sup>	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
e	6	65.0 ft	71.5 ft	58.5 ft	13.0 ft	120.0 ft	1,560 ft <sup>2</sup>	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 ft <sup>2</sup>	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
	3	26.0 ft	43.5 ft	19.5 ft	13.0 ft	120.0 ft	1,560 ft <sup>2</sup>	0.79	11.32	1.20	19.81	10.0	11.38	-15.53	-10.0	39.8 kips	33.4 kips	33.4 kips
2	2	13.0 ft	19.5 ft	6.5 ft	13.0 ft	120.0 ft	1,560 ft <sup>2</sup>	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
GRC	OUND	0.0 ft	6.5 ft	0.0 ft	6.5 ft	120.0 ft	780 ft <sup>2</sup>	0.57	8.96	1.26	19.81	10.0	10.0	-15.53	-10.0	19.9 kips	15.6 kips	N.A.
				I		l		I	l	l	l	l		l	L	754 kips	727 kips	

GG Engling	aaring	JOB NO. 18-	STORY BUILDINGS		BY	SMG	DATE	4/22/05	SHEET NO.
		CUSTOMER	RCFT PARAMETRIC STU	DY	CKD		DATE		OF
SUBJECT		2-D M(	OMENT FRAME (MF A	43 - G3] AN	ALYSI	S LOAD S	UMMA	RY	
[16] WIND	LOADS CC	DNTINUED							
o Bl	UIDLING WII	DTH (DIMENSION PI	ERPENDICULAR TO WIND DIRECTION)	), B 120.0 ft	ALONG T	HE E-W FACE			
o EC	CCENTRICIT	TY ALONG THE WIN	DWARD FACE OF THE BUILDING, $e_x$	± 18.0 ft				Figure 6-9	(p. 54)
o Bl	UIDLING DE	PTH (DIMENSION P	ARALLEL TO WIND DIRECTION), L	120.0 ft	ALONG T	HE N-S FACE			
o EC	CCENTRICIT	TY ALONG THE SIDE	EWALL OF THE BUILDING, e <sub>y</sub>	± 18.0 ft				Figure 6-9	(p. 54)
o E0	CCENTRICIT	Y FOR FLEXIBLE S	TRUCTURES ( $e_x$ 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.

		ASCE 7	Figure 6-9 (	CASE #1	ASCE 7	Figure 6-9 (	CASE #2	ASCE 7	Figure 6-9 (	CASE #3	ASCE 7	Figure 6-9 (	CASE #4
FLOOR	ELEVATION	SHEAR LOA	DS DUE TO:	TOTAL STORY	SHEAR LOA	DS DUE TO:	TOTAL STORY	SHEAR LOA	DS DUE TO:	TOTAL STORY	SHEAR LOA	DS DUE TO:	TOTAL STORY
		SHEAR	TORSION	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.									
				711.5 kips			533.7 kips			533.7 kips			400.4 kips

0 TORSION LOADS ARE SIMPLIFIED SO THAT THE MAXIMUM SHEAR PER MOMENT FRAME = STORY SHEAR x (1 / NO. OF MOMENT FRAMES + 0.002 x ECCENTRICITY)

Construction         Customer         RCFT PARAMETRIC STUDY         CKD         DATE         OF           SUBJECT         2-D MOMENT FRAME [MF A3 - G3] ANALYSIS LOAD SUMMARY	ශිලි පිතැබ්තලලන්තු	JOB NO. 1	8-STORY	BUILDIN	GS		BY	SMG	DAT	E 4/22/05	SHEET NO.
SUBJECT         2-D MOMENT FRAME MALYSIS LOAD SUMMARY           (17)         16-STORY BULLIDIKS MOMENT FRAME MALYSIS LOAD SUMMARY         0 </th <th>oo riikimieei mik</th> <th>CUSTOME</th> <th>RCFT F</th> <th>PARAMET</th> <th>RIC STU</th> <th>YC</th> <th>CKE</th> <th>)</th> <th>DAT</th> <th>E</th> <th> OF</th>	oo riikimieei mik	CUSTOME	RCFT F	PARAMET	RIC STU	YC	CKE	)	DAT	E	OF
17.1         19-STORY BUILDINGS MOMENT FRAME ANALYSIS LOAD SUMMARY           9         SUMMARY:         TOTAL NUMBER OF EXPERIMENT FRAME.N,         18 STORES           NUMBER OF DEVES IN THE MOMENT FRAME.N,         18 STORES           DIRECTOR THAT THE MOMENT FRAME.N,         18 STORES           DIRECTOR THAT THE MOMENT FRAME.N,         18 STORES           DORSTONCT THAT THE MOMENT FRAME.N,         EW           DOSTING THE MOMENT FRAME.NON FRAME         WIEFEIOR           DOSTING TO THE CLOSEST (GRAVITYMOMENT) FRAME:         200 R           DOSTING TO THE CLOSEST (GRAVITYMOMENT) FRAME:         200 R           DISTINGE TO THE CLOSEST (GRAVITYMOMENT) FRAME:         200 R           DISTINGE TO THE CLOSEST (GRAVITYMOMENT) FRAME:         200 R           DESIGN ASSUMPTION:         BULDING MAG MAG THEO STORES CLOSENT, THE BULDING CONCOLUMENT FRAME DIATED AND TABLE DATABATION TO COMPARE DATABATION TO C	SUBJECT	2-D I	MOMEN	T FRAM	E [MF A	3 - G3] /	ANALYS	SIS LOAI	D SUMN	IARY	·
NMBER         FLOOR         FLOOR <th< th=""><th>[17] 18-STORY BUI o SUMMAI</th><th>LDINGS MOMENT RY: T( N) D L( D) D D D D D D D D D D D D D D D D D D</th><th>FRAME ANAL' DTAL NUMBER UMBER OF ST UMBER OF BA IRECTION THA DCATION OF TI OES THIS FRA ISTANCE TO T ISTANCE TO T ESIGN SEISMI ASIC WIND SP UILDING HAS ' PER WIND DES ATERAL LOAD</th><th>YSIS LOAD SI R OF (IDENTIC, ORIES IN THE VYS IN THE MOMENT HE MOMENT I ME SUPPORT HE CLOSEST C BASE SHEA FEED TRIGID DIAPHI SIGN), AND AL S ARE DISTRI</th><th>JIMMARY AL) MOMENT FR MOMENT FRAME NOMENT FRAME RUT FRAME RU FRAME WRT T FRAME WRT T FRAME WRT T FRAME WRT T (GRAVITY/MOM (GRAVITY/MOM (GRAVITY/MOM RAGMS" (PER 1) L OF THE MOM BUTED EQUAL</th><th>FRAMES ALON IME, N<sub>S</sub> INS PARALLEI HE BUILDING E PENTHOUSE MENT) FRAME SEISMIC DESI MENT FRAME SEISMIC DESI</th><th>Ig the e-w c . With: Perimeter: Gravity Lo/ : On the othi GN), the built S have the s LL of the MC</th><th>DIRECTION 1 ADS? ER SIDE: LDING IS CONS AME RIGIDITY DMENT FRAME</th><th>4 8 STORIES 6 BAYS E-W INTERIOR YES 20.0 ft 20.0 ft 0.044 W 85 mph SIDERED A "S AT EACH FLC S IN EACH PL</th><th>IMPLE DIAPHRAGM OOR LEVEL. THERE RINCIPAL DIRECTIC</th><th>" BUILDING FORE, ALL N OF THE BUILDING.</th></th<>	[17] 18-STORY BUI o SUMMAI	LDINGS MOMENT RY: T( N) D L( D) D D D D D D D D D D D D D D D D D D	FRAME ANAL' DTAL NUMBER UMBER OF ST UMBER OF BA IRECTION THA DCATION OF TI OES THIS FRA ISTANCE TO T ISTANCE TO T ESIGN SEISMI ASIC WIND SP UILDING HAS ' PER WIND DES ATERAL LOAD	YSIS LOAD SI R OF (IDENTIC, ORIES IN THE VYS IN THE MOMENT HE MOMENT I ME SUPPORT HE CLOSEST C BASE SHEA FEED TRIGID DIAPHI SIGN), AND AL S ARE DISTRI	JIMMARY AL) MOMENT FR MOMENT FRAME NOMENT FRAME RUT FRAME RU FRAME WRT T FRAME WRT T FRAME WRT T FRAME WRT T (GRAVITY/MOM (GRAVITY/MOM (GRAVITY/MOM RAGMS" (PER 1) L OF THE MOM BUTED EQUAL	FRAMES ALON IME, N <sub>S</sub> INS PARALLEI HE BUILDING E PENTHOUSE MENT) FRAME SEISMIC DESI MENT FRAME SEISMIC DESI	Ig the e-w c . With: Perimeter: Gravity Lo/ : On the othi GN), the built S have the s LL of the MC	DIRECTION 1 ADS? ER SIDE: LDING IS CONS AME RIGIDITY DMENT FRAME	4 8 STORIES 6 BAYS E-W INTERIOR YES 20.0 ft 20.0 ft 0.044 W 85 mph SIDERED A "S AT EACH FLC S IN EACH PL	IMPLE DIAPHRAGM OOR LEVEL. THERE RINCIPAL DIRECTIC	" BUILDING FORE, ALL N OF THE BUILDING.
$ \begin{array}{ c c c c c c c c c c c c c c c c c c c$		NUN	MBER	STORY		DESIGN : (THESE ARE	SHEAR LOAD AT THE POINT LOA	EACH FLOOR LEN DS THAT SHALL E	/EL PER MOME! 3E USED IN THE	NT FRAME 2-D MODEL)	
Image         Image <th< td=""><td></td><td>STORY</td><td>FLOOR</td><td>HEIGHT</td><td>h<sub>x</sub></td><td>SEISMIC LOAD</td><td>CASE #1</td><td>WIND CASE #2</td><td>LOAD CASE #3</td><td>CASE #4</td><td></td></th<>		STORY	FLOOR	HEIGHT	h <sub>x</sub>	SEISMIC LOAD	CASE #1	WIND CASE #2	LOAD CASE #3	CASE #4	
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         280.0 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         7.3 kips           14         14         13.0 ft         195.0 ft         32.1 kips         10.3 kips         9.6 kips         7.7 kips         7.3 kips           13         13         13.0 ft         169.0 ft         25.4 kips         10.2 kips         9.6 kips         7.7 kips         7.3 kips           12         12         13.0 ft         165.0 ft         21.9 kips         10.1 kips         9.5 kips         7.6 kips         7.0 kips           11         13.0 ft         130.0 ft         130.0 ft         16.6 kips         9.9 kips         7.3 kips         7.0 kips           10         10         13.0 ft         190.0 ft         10.3 kips         9.7 kips         7.1 kips         6.8 kips           11         11         13.0 ft         10.4 ft         10.3 kips         9.1 kips			 ROOF		 234.0 ft	 46.2 kips	 15.0 kips	 13.3 kips	 11.2 kips	 10.1 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         196.0 ft         25.4 kips         10.2 kips         9.6 kips         7.7 kips         7.3 kips           12         13.0 ft         196.0 ft         25.4 kips         10.2 kips         9.6 kips         7.6 kips         7.1 kips           12         13.0 ft         143.0 ft         146.6 ft         21.9 kips         10.1 kips         9.5 kips         7.6 kips         7.0 kips           11         11         13.0 ft         143.0 ft         143.0 ft         166.6 kips         9.9 kips         9.3 kips         7.4 kips         6.6 kips           9         9         13.0 ft         117.0 ft         12.8 kips         9.1 kips         9.3 kips         7.1 kips         6.6 kips           6         6         13.0 ft         104.0 ft		18	18	13.0 ft	221.0 ft	41.7 kips	10.6 kips	9.9 kips	7.9 kips	7.5 kips	
160       160       160       160       130.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       168.0 ft       25.4 kips       10.2 kips       9.6 kips       7.7 kips       7.3 kips         13       13       13.0 ft       166.0 ft       21.9 kips       10.1 kips       9.5 kips       7.6 kips       7.1 kips         12       12       12.0 ft       13.0 ft       166.0 kips       9.9 kips       9.4 kips       7.5 kips       7.0 kips         10       11       13.0 ft       130.0 ft       116.0 kips       9.9 kips       9.3 kips       7.4 kips       6.8 kips         9       9       13.0 ft       117.0 ft       12.3 kips       9.7 kips       9.1 kips       7.4 kips       6.8 kips         6       6       13.0 ft       110.0 ft       10.3 kips       9.1 kips       8.8 kips       7.0 kips       6.8 kips         7       13.0 ft       14.0 ft       10.3 kips       9.1 kips       8.8 kips       7.0 kips       6.8 kips <tr< td=""><td></td><td>17</td><td>17</td><td>13.0 ft</td><td>208.0 ft</td><td>37.3 kips</td><td>10.5 kips</td><td>9.9 kips</td><td>7.9 kips</td><td>7.4 kips</td><td></td></tr<>		17	17	13.0 ft	208.0 ft	37.3 kips	10.5 kips	9.9 kips	7.9 kips	7.4 kips	
16         15         1.00.11         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         13.0 ft         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         9.0 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         7.1 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         22.0 ft         2.9 k		15	16	13.0 ft	195.0 ft	33.1 kips	10.4 kips	9.8 kips	7.8 kips	7.4 kips	
14         169.0 ft         254 kps         10.2 kips         9.6 kips         7.7 kips         7.3 kps           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         130.0 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         130.0 ft         15.6 kips         9.9 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           7         7         13.0 ft         91.0 ft         8.1 kips         9.5 kips         8.9 kips         7.1 kips         6.6 kips           6         6         13.0 ft         52.0 ft         2.9 kips         8.8 kips         7.0 kips         6.5 kips           5         5         13.0 ft         2.9 kips         8.8 kips         8.1 kips         6.5 kips<		13	15	13.0 ft	182.0 ft	29.1 kips	10.3 kips	9.6 kips	7.7 kips	7.3 kips	
13       156.0 ft       21.9 kips       10.1 kips       9.5 kips       7.6 kips       7.1 kips         12       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       130.0 ft       15.6 kips       9.9 kips       9.3 kips       7.4 kips       7.0 kips         9       9       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         7       7       13.0 ft       10.0 ft       10.3 kips       9.5 kips       8.9 kips       7.1 kips       6.6 kips         7       7       13.0 ft       91.0 ft       8.1 kips       9.5 kips       8.9 kips       7.1 kips       6.6 kips         5       5       13.0 ft       52.0 ft       2.9 kips       8.9 kips       8.1 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         2       2		13	14	13.0 ft	169.0 ft	25.4 kips	10.2 kips	9.6 kips	7.7 kips	7.3 kips	
11       11       13.0 ft       12.8 kips       9.9 kips       9.3 kips       7.4 kips       7.0 kips         9       9       13.0 ft       10.4 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       7.0 kips       6.6 kips         7       7       13.0 ft       91.0 ft       8.1 kips       9.5 kips       8.9 kips       7.0 kips       6.5 kips         6       6       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       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         2       2       13.0 ft       26.0 ft       0.8 kips       7.9 kips       7.5 kips       6.0 kips       5.6 kips		12	13	13.0 ft	156.0 ft	21.9 kips	10.1 kips	9.5 kips	7.6 kips	7.1 kips	
10         11         13.0 ft         13.0 ft         13.0 ft         13.0 ft         13.0 ft         13.0 ft         12.8 kips         9.3 kips         3.3 kips         7.4 kips         7.0 kips           9         9         13.0 ft         117.0 ft         12.8 kips         9.7 kips         9.1 kips         7.3 kips         6.9 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         7         7.8 0.0 ft         6.1 kips         9.3 kips         8.8 kips         7.0 kips         6.5 kips           6         6         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         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           2         2         13.0 ft         26.0 ft         0.8 kips         7.9 kips         7.5 kips         6.0 kips         5.6 kips           1         GROUND		11	12	13.0 ft	143.0 ft	18.6 kips	10.0 kips	9.4 kips	7.5 kips	7.0 kips	
9         9         13.0 ft         10.0 ft         10.3 kips         9.6 kips         9.0 kips         7.2 kips         6.8 kips           8         8         13.0 ft         104.0 ft         10.3 kips         9.6 kips         9.0 kips         7.2 kips         6.8 kips           7         7         13.0 ft         91.0 ft         8.1 kips         9.5 kips         8.9 kips         7.1 kips         6.6 kips           6         6         13.0 ft         78.0 ft         6.1 kips         9.3 kips         8.8 kips         7.0 kips         6.5 kips           6         6         6         13.0 ft         78.0 ft         6.1 kips         9.3 kips         8.8 kips         7.0 kips         6.5 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         30.0 ft         1.7 kips         8.6 kips         8.1 kips         6.5 kips         6.1 kips           2         13.0 ft         26.0 ft         0.8 kips         7.9 kips         6.3 kips         5.9 kips           2         2         13.0 ft         0.0 ft         N.A.         N.A.         N.A.         <		10	10	13.0 ft	130.0 π 117.0 ft	12.8 kine	9.9 kips	9.3 Kips	7.4 Kips	7.0 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         78.0 ft         6.1 kips         9.3 kips         8.8 kips         7.0 kips         6.5 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         52.0 ft         2.9 kips         8.9 kips         8.4 kips         6.7 kips         6.3 kips           3         3         13.0 ft         2.9 kips         8.6 kips         8.1 kips         6.5 kips         6.1 kips           2         2         13.0 ft         26.0 ft         0.8 kips         7.9 kips         6.3 kips         5.9 kips           1         GROUND         13.0 ft         0.0 ft         N.A.         N.A.         N.A.         N.A.		9	9	13.0 ft	104.0 ft	10.3 kips	9.6 kips	9.0 kips	7.2 kips	6.8 kips	
$ \begin{array}{ c c c c c c c c c c c c c c c c c c c$		8	8	13.0 ft	91.0 ft	8.1 kips	9.5 kips	8.9 kips	7.1 kips	6.6 kips	
$ \begin{array}{ c c c c c c c c c c c c c c c c c c c$		7	7	13.0 ft	78.0 ft	6.1 kips	9.3 kips	8.8 kips	7.0 kips	6.5 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       6.0 kips       5.6 kips         1       GROUND       13.0 ft       0.0 ft       N.A.       N.A.       N.A.       N.A.		6	6	13.0 ft	65.0 ft	4.3 kips	9.1 kips	8.5 kips	6.8 kips	6.4 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         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.		5	5	13.0 ft	52.0 ft	2.9 kips	8.9 kips	8.4 kips	6.7 kips	6.3 kips	
0     3     1.0.4.1     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.		4	4	13.0 π 13.0 ft	39.0 ft	1.7 kips	8.6 kips	8.1 kips	6.5 kips	6.1 kips	
2         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.		2	3	13.0 ft	26.0 ft	0.8 kips	8.4 kips	7.9 kips	6.3 kips	5.9 kips	
		1	2	13.0 ft	13.0 ft	0.2 kips	7.9 kips	7.5 kips	6.0 kips	5.6 kips	
			GROUND		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			<u> </u>			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,  $\theta$ , 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. 1.4D2.  $1.2D + 1.6L + 0.5L_R$ 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.  $1.2D + 1.0E + f_1L$

 $\begin{array}{ll} \textit{Where:} & f_1 = 0.5 \\ & E = \rho Q_E + 0.2 S_{DS} D' \\ & D' = \text{seismic weight} \end{array}$ 

COLLAND.LINE     DESIGN 18A     CKD     DATE     OF       SUBJECT     DESIGN PARAMETERS SUMMARY     MMENT FRAME MEAN 50       • DESIGN INPUTS:     • TOLL NUMBER OF COLUMNS BEING ANLIVED     198       • VIELID STREAM     UNSUL STREAM     LINE 5,	CUSTOMER SUBJECT	DESIGN 18A DESIGN PARAMETERS	CKD SUMMARY NG ANALYZED H CONCRETE REINFORCEMEN IVE STRENGTH ARI AXIAL COMPRESSI	DATE 126 I27 I28 I29 I29 I29 I29 I29 I29 I29 I29	OF MOMENT FRAME MF A3 - G3
SUBJECT DESIGN PARAMETERS SUMMARY 104  • DESIGN INPUTS: • TOU, MUMBER OF GOLUMINS REING ANALYZED • 10 • YELD STIFENET: • YELD	o design inputs:	O TOAL NUMBER OF COLUMNS BEIN     O TOAL NUMBER OF COLUMNS BEIN     O YIELD STRENGTH:     O MODULUS OF ELASTICITY:     O MODULUS OF ELASTICITY:     O MINIMUM CONCRETE COMPRESSIN     O CONCRETE DENSITY     O CONCRETE REINFORCEMENT     O RESISTANCE FACTORS	SUMMARY NG ANALYZED H CONCRETE REINFORCEMEN IVE STRENGTH ARI AXIAL COMPRESSI	126 SS, $F_y = 46$ ksi NT, $F_{yr} = 0$ ksi SS, $E_s = 29,000$ ksi NT, $E_{cr} = 29,000$ ksi f'_c = 4.0 ksi w = 145 lb/ft <sup>3</sup> EA, $A_{sr} = 0.0$ in <sup>2</sup> $I_{xer} = 0.0$ in <sup>2</sup> $I_{xer} = 0.0$ in <sup>3</sup> $Z_{yyr} = 0.0$ in <sup>3</sup>	MOMENT FRAME MF A3 - G3
• DESIGN INPUTS:         • TONL NUMBER OF COLUMNS BEING ANALYZED         197           • YELD STRENDTH         H93, F, - 43, 50           • O'NODULUS OF ELASTOTY         H93, E, - 50,000, 50           • OMODULUS OF ELASTOTY         H93, E, - 430,000, 50           • OMODULUS OF ELASTOTY         H93, E, - 430,000, 50           • OMODULUS OF ELASTOTY         H93, E, - 430,000, 50           • OMODULUS OF ELASTOTY         H93, E, - 430,000, 50           • OMODULUS OF ELASTOTY         H93, E, - 430,000, 50           • OMODULUS OF ELASTOTY         H93, E, - 430,000, 50           • OMODULUS OF ELASTOTY         H93, E, - 430,000, 50           • OMODULUS OF ELASTOTY         H93, E, - 430,000, 50           • OMODULUS OF ELASTOTY         H93, E, - 430,000, 50           • OMODULUS OF ELASTOTY         N + 436, B1           • OMODULUS OF ELASTOTY         N + 445, B1           • OMODULUS OF ELASTOTY         N + 445, B1           • O = 000, B1         O = 000, B1           • O = 000, B1         O = 000, B1           • O = 000, B1         O = 000, B1           • O = 000, B1         O = 000, B1	o DESIGN INPUTS:	<ul> <li>TOAL NUMBER OF COLUMNS BEIN</li> <li>YIELD STRENGTH:</li> <li>MODULUS OF ELASTICITY:</li> <li>MINIMUM CONCRETE COMPRESSIN</li> <li>CONCRETE DENSITY</li> <li>CONCRETE REINFORCEMENT</li> <li>RESISTANCE FACTORS</li> </ul>	NG ANALYZED H CONCRETE REINFORCEMEI H CONCRETE REINFORCEMEI IVE STRENGTH ARI	126 SS, $F_y = 46$ ksi NT, $F_{yr} = 0$ ksi SS, $E_s = 29,000$ ksi NT, $E_{cr} = 29,000$ ksi f'_c = 4.0 ksi w = 145 lb/ft <sup>3</sup> EA, $A_{sr} = 0.0$ in <sup>2</sup> $I_{xr} = 0.0$ in <sup>2</sup> $I_{xr} = 0.0$ in <sup>3</sup> $Z_{yr} = 0.0$ in <sup>3</sup>	
• DESIGN INPUTS:       • TOL NUMBER OF COLUMNS BEING ANALYZED       18         • YELD STRENGTH:       HSS. F, - 4 44         CONDETE RENFORCEMENT, F, - 0 181       CONDETE RENFORCEMENT, F, - 0 181         • MODULUS OF ELASTICITY       HSS. F, - 2 3000 141         • OUDULUS OF ELASTICITY       HSS. F, - 2 3000 141         • CONCRETE COMPRESSIVE STRENGTH       IF, - 4 9 34         • CONCRETE DENSITY       W - 10 587         • CONCRETE CONTRESSIVE STRENGT       W - 10 587         • CONCRETE RENFORCEMENT       W - 10 587         • CONCRETE FACTORS       AXMU CONCRETECTS         • RESISTANCE FACTORS       REDUIDANCY CONCRETERING, + 0.07         • SESSIND PARAMETERS       REDUIDANCY CONCRETERING, + 0.07         • SESSIND PARAMETERS       REDUIDANCY CONCRETERING, + 0.03	o DESIGN INPUTS:	<ul> <li>TOAL NUMBER OF COLUMNS BEIN</li> <li>YIELD STRENGTH:</li> <li>MODULUS OF ELASTICITY:</li> <li>MINIMUM CONCRETE COMPRESSIVING</li> <li>CONCRETE DENSITY</li> <li>CONCRETE REINFORCEMENT</li> <li>RESISTANCE FACTORS</li> </ul>	NG ANALYZED H CONCRETE REINFORCEMEI IVE STRENGTH ARI	126 ISS, $F_y = 46$ ksi NT, $F_{yr} = 0$ ksi SS, $E_s = 29,000$ ksi NT, $E_{cr} = 29,000$ ksi f'_c = 4.0 ksi w = 145 lb/ft <sup>3</sup> EA, $A_{sr} = 0.0$ in <sup>2</sup> $I_{xer} = 0.0$ in <sup>2</sup> $I_{xer} = 0.0$ in <sup>3</sup> $Z_{yyr} = 0.0$ in <sup>3</sup>	
<ul> <li>VELO STRENGTH: USBS (F, - 40 bit) CONCRETE REINFORCEMENT, F, - 2000 bit)</li> <li>MODULUS OF ELASTICITY: RSS, E, - 2000 bit) CONCRETE REINFORCEMENT, F, - 2000 bit)</li> <li>MINIMUM CONCRETE COMPRESSIVE STRENGTH (-, 4.0 bit)</li> <li>CONCRETE REINFORCEMENT (-, 0.0 bit)</li> <li>CONCRETE REINFORCEMENT (</li></ul>		<ul> <li>VIELD STRENGTH:</li> <li>MODULUS OF ELASTICITY:</li> <li>MINIMUM CONCRETE COMPRESSION</li> <li>CONCRETE DENSITY</li> <li>CONCRETE REINFORCEMENT</li> <li>RESISTANCE FACTORS</li> </ul>	H CONCRETE REINFORCEMEN IVE STRENGTH ARI	SS, $F_y = 46$ ksi NT, $F_{yr} = 0$ ksi SS, $E_a = 29,000$ ksi NT, $E_{cr} = 29,000$ ksi $f'_c = 4.0$ ksi w = 145 lb/fl <sup>3</sup> EA, $A_{sr} = 0.0$ in <sup>2</sup> $I_{sr} = 0.0$ in <sup>2</sup> $I_{sr} = 0.0$ in <sup>4</sup> $Z_{sr} = 0.0$ in <sup>3</sup> $Z_{syr} = 0.0$ in <sup>3</sup>	
MODULUS OF ELASTICITY     HSS.E, = 20.000 isi     COMCRETE REINFORCEMENT     (0.0si     O MINIMUM CONCRETE COMPRESSIVE STRENGTH     (0.0si     O CONCRETE DENSITY     (0.0si		<ul> <li>MODULUS OF ELASTICITY:</li> <li>MINIMUM CONCRETE COMPRESSIVING</li> <li>CONCRETE DENSITY</li> <li>CONCRETE REINFORCEMENT</li> <li>RESISTANCE FACTORS</li> </ul>	H CONCRETE REINFORCEMEI IVE STRENGTH ARI AXIAL COMPRESSI	SS, $E_{a} = 29,000$ ksi NT, $E_{cr} = 29,000$ ksi f'_c = 4.0 ksi w = 145 lb/ft <sup>3</sup> EA, $A_{sr} = 0.0$ in <sup>2</sup> $I_{sr} = 0.0$ in <sup>4</sup> $I_{srr} = 0.0$ in <sup>3</sup> $Z_{syr} = 0.0$ in <sup>3</sup>	
<ul> <li>MINIMUM CONCRETE COMPRESSIVE STRENGTH</li> <li>CONCRETE DENSITY</li> <li>CONCRETE REINFORCEMENT</li> <li>AREA, A, = 0 0 PF</li> <li>L, = 0 0 PF</li> <li>L, = 0 0 PF</li> <li>Z, = 0 0 PF</li> <li>Z, = 0 0 PF</li> <li>RESISTANCE FACTORS</li> <li>AXALL COMPRESSION, a, = 0.75</li> <li>FLEURIAL BEDNING, = 0.90</li> <li>SEISMIC PARAMETERS</li> <li>REDUNDANCY OCEFFICIENT, p = 1.00</li> <li>UERTICAL SEGMIC FACTOR: 0.25 (a.20)</li> <li>ORTHOGONAL LOAD FACTOR ALONG YAUSI OF SHARED COLUMNS = 0.30</li> <li>FACTOR TO ACCOUNT FOR 5% ACCIDENTAL TORSION ("SIMPLIFED APPROACH") = 0.025</li> </ul>		<ul> <li>MINIMUM CONCRETE COMPRESSION</li> <li>CONCRETE DENSITY</li> <li>CONCRETE REINFORCEMENT</li> <li>RESISTANCE FACTORS</li> </ul>	IVE STRENGTH ARI	$f'_{c}$ = 4.0 ksi w = 145 lb/ft <sup>3</sup> EA, A <sub>sr</sub> = 0.0 in <sup>2</sup> $I_{xor}$ = 0.0 in <sup>4</sup> $I_{yyr}$ = 0.0 in <sup>4</sup> $Z_{xor}$ = 0.0 in <sup>3</sup> $Z_{yyr}$ = 0.0 in <sup>3</sup>	
<ul> <li>OCNORETE DENSITY</li> <li>CONCRETE RENFORCEMENT</li> <li>AREA A<sub>2</sub> = 0.0 inf U<sub>2</sub> = 0.0 inf U<sub>2</sub> = 0.0 inf Z<sub>2</sub> = 0.0 inf Z<sub>2</sub> = 0.0 inf Z<sub>2</sub> = 0.0 inf Concrete DENSITY</li> <li>RESISTANCE FACTORS</li> <li>AXIAL COMPESSION U<sub>2</sub> = 0.75 RESUMDING, 0 = 0.90</li> <li>SEISMIC PARAMETERS</li> <li>REDUNDANCY COEFFICIENT, p = 1.00 VENTRAL SEISMIC FACTOR U.SEISMIC FACTOR U.SEISMIC FACTOR 0.035</li> <li>ORTHOGOMAL LOD FACTOR ALMON V_MODEL 0.01</li> <li>FACTOR TO ACCOUNT FOR 5% ACCIDENTAL TORSION ("SIMPLIFED APPROACH") = 0.025</li> </ul>		<ul> <li>CONCRETE DENSITY</li> <li>CONCRETE REINFORCEMENT</li> <li>RESISTANCE FACTORS</li> </ul>	ARI AXIAL COMPRESSI	w = 145 lb/ft <sup>3</sup> EA, $A_{sr} = 0.0 \text{ in}^2$ $I_{xr} = 0.0 \text{ in}^4$ $I_{yrr} = 0.0 \text{ in}^4$ $Z_{xr} = 0.0 \text{ in}^3$ $Z_{yyr} = 0.0 \text{ in}^3$	
CONCRETE REINFORCEMENT     AREA A, = 0.0 in*     L, = 0.0 in*		<ul> <li>CONCRETE REINFORCEMENT</li> <li>RESISTANCE FACTORS</li> </ul>	ARI AXIAL COMPRESSI	EA, $A_{sr} = 0.0 \text{ in}^2$ $I_{srr} = 0.0 \text{ in}^4$ $I_{yyr} = 0.0 \text{ in}^4$ $Z_{xrr} = 0.0 \text{ in}^3$ $Z_{yyr} = 0.0 \text{ in}^3$	
0       RESISTANCE FACTORS       AXIAL COMPRESSION, q <sub>0</sub> = 0.90         1       SEISMIC PARAMETERS       REDUNDANCY COEFFICIENT, p = 1.00         1       VERTICAL SEISMIC PARAMETERS       REDUNDANCY COEFFICIENT, p = 0.00         1       ORTHOGONAL LODA FOCULY AVIS OF SHARED COLUMNS = 0.30         1       ORTHOGONAL LODA FOCULY AVIS OF SHARED COLUMNS = 0.30         1       FACTOR TO ACCOUNT FOR 5% ACCIDENTAL TORSION ("SIMPLIFIED APPROACH") = 0.025		0 RESISTANCE FACTORS	AXIAL COMPRESSI		
<ul> <li>SEISMIC PARAMETERS</li> <li>REDUNDANCY COEFFICIENT, p = 1.00</li> <li>VERTICAL SEISMIC "FACTOR," 0.28mg = 0.20</li> <li>ORTHOGONAL LOAD FACTOR ALONG Y-AXIS OF SHARED COLUMNS = 0.30</li> <li>FACTOR TO ACCOUNT FOR 5% ACCIDENTAL TORSION ("SIMPLIFIED APPROACH") = 0.025</li> </ul>			FLEXURAL BEND	$\text{ION}, \phi_{\text{c}} = 0.75$ $\text{ING}, \phi_{\text{b}} = 0.90$	
		o SEISMIC PARAMETERS VER ORTHOGONAL LOAD FACTOR ALO FACTOR TO ACCOUNT FOR 5% ACCIDENTAL TORS	REDUNDANCY COEFFICI IRTICAL SEISMIC "FACTOR," ( )NG Y-AXIS OF SHARED COL <i>ISION</i> ("SIMPLIFIED APPROAC	ENT, ρ = 1.00 0.2S <sub>DS</sub> = 0.20 UMNS = 0.30 CH") = 0.025	

GG	Eng	ineeri	JOB NC	). 18 - STO MER DESI	RY BUII GN 18A	DINGS			BY CKD	SMG	DATE DATE	4/22/05	SH	IEET NO. OF
SUBJ	ECT		B2 CALC	ULATION	- FOR	BEND	DING A	ALONG	THE X	-AXIS O	F THE	COLUM	V <sup>MC</sup>	MENT FRAME MF A3 - G3
SNU	ORY MBER 18 17 16 15 11 10 9 8 7 6 5 4 3 2 1	LOAD ( STORY HEIGHT) L 13.0 ft 13.0 ft	COMBINATION =	1.2D + 1.6Lr + 0. IS THAT CAUSE HE X-AXIS OF THE ME COLUMNS ELASTIC INTERSTORY DRIFT $\Delta_{oh}$ DUE TO $\Sigma H_i$ 0.25 in 0.45 in 0.45 in 0.55 in 0.55 in 0.55 in 0.57 in 0.57 in 0.58 in 0.59 in 0.59 in 0.59 in 0.59 in 0.59 in 0.51 in 0.51 in 0.52 in 0.52 in 0.55 in 0.55 in 0.57 in 0.57 in 0.57 in 0.58 in 0.59 in 0.59 in 0.59 in 0.59 in 0.51 in 0.52 in 0.52 in 0.52 in 0.55 in 0.55 in 0.57 in 0.55 in 0.57 in 0.55 in 0.57 in 0.57 in 0.58 in 0.59 in 0.52 in 0.52 in 0.52 in 0.55 in 0.57 in 0.55 in 0.57 in 0.52 in 0.55 in 0.57 in 0.55 in 0.55 in 0.57 in 0.52 in 0.52 in 0.55 in	8W TOTAL U STORY OF ("LEANED LOAD DL 1,513 2,821 4,129 5,437 6,745 8,053 9,361 10,669 11,977 13,285 14,593 15,901 17,209 18,517 19,825 21,133 22,441 23,749	(L.C. #4, NFACTORE VALCOLU "T + "NON-LU LOAD LL 0 1.008 2,016 3,024 4,032 5,040 6,048 7,056 8,064 9,072 10,080 11,088 12,096 13,104 14,112 15,120 16,128 17,136	) D AXIAL L MNS OF T EANER" C 288 288 288 288 288 288 288 28	OAD PER HE STORY OLUMNS) SEISMIC WEIGHT DL + P-LL 1,591 3,187 4,783 6,379 7,975 9,571 11,167 12,763 14,359 15,955 17,551 19,147 20,743 22,339 23,935 25,531 27,127 28,723	D.L. 1.2 1,815.6 3,385.2 4,954.8 6,524.4 8,094.0 9,663.6 11,233.2 12,802.8 14,372.4 15,942.0 17,511.6 19,081.2 20,650.8 22,220.4 23,790.0 25,359.6 26,929.2 28,498.8	L FACTORED A LOAD FA L.L. 0 0.0 0.0 0.0 0.0 0.0 0.0	XIAL LOAD, CTOR ROOF LL. 1.6 460.8	ΣP <sub>u</sub> , PER STOP	Y TOTAL ΣPu (kips) 2,276.4 3,846.0 5,415.6 6,985.2 8,554.8 10,124.4 11,694.0 13,263.6 14,833.2 16,402.8 17,972.4 19,542.0 21,111.6 22,681.2 24,250.8 25,820.4 27,390.0 28,959.6	STORY B <sub>21,X-AXIS</sub> 1.020 1.027 1.032 1.034 1.040 1.044 1.046 1.050 1.051 1.054 1.060 1.063 1.069 1.074 1.079 1.082 1.081 1.049

GG Eng	jineeri	JOB NC	). 18 - STO	RY BUII	LDINGS			BY	SMG	DATE	4/22/05	Sł	HEET NO.
-		CUSTO	MER DESI	GN 18A	۱ 			CKD		DATE			. OF
SUBJECT		B2 CALC	ULATION	- FOR	BEND	)ING /	ALONG	THE	Y-AXIS (	OF THE	COLUM	N MC	OMENT FRAME MF A3 - G3
o	LOAD	COMBINATION =	1.2D + 1.6Lr + 0.	.8W									
		DUE TO FORCE BENDING ALONG T MOMENT FRA	ES THAT CAUSE THE Y-AXIS OF THE ME COLUMNS	TOTAL U STORY OF ("LEANE	<i>INFACTORE</i> N ALL COLU R" + "NON-L	D AXIAL L JMNS OF 1 EANER" C	OAD PER HE STORY OLUMNS)	тот,	AL FACTORED	AXIAL LOAD,	$\Sigma P_u$ , PER STO	RY	
STORY NUMBER	LENGTH <i>(STORY</i> <i>HEIGHT)</i> L	SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH:	ELASTIC INTERSTORY DRIFT Δ <sub>oh</sub> DUE TO ΣH <sub>i</sub>	DEAD LOAD DL	(ki LIVE LOAD LL	ROOF LIVE LOAD	SEISMIC WEIGHT DL + P-LL	D.L. 1.2	LOAD F	ROOF L.L.	SEISMIC VERTICAL 0.2S <sub>DS</sub> = 0	TOTAL ΣP <sub>ui</sub> (kips)	STORY B <sub>2i_Y-AXIS</sub>
18	13.0 ft	185 kips	0.25 in	1,513	0	Lr 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	17 551	17,942.0	0.0	460.8	0.0	16,402.8	1.054
7	13.0 ft	1,100 kips	0.57 in	15 901	11 088	288	19 147	19.081.2	0.0	460.8	0.0	19 542 0	1.000
6	13.0 ft	1,200 kips	0.57 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
<b></b>													·

-													
ශිශි (දි	Mineeri	JOB N	0. 18 - STO	RY BUI	LDINGS	;		BY	SMG	DATE	4/22/05	s	HEET NO.
00 8	iQuuueeu i	CUSTO	OMER DESI	GN 18A	L.			СКД		DATE		7_	OF
SUBJEC	Т		STABIL		DEFFIC	CIENT	ALON	G COLU	MN X-A	XIS, θ <sub>x</sub>		N	OMENT FRAME MF A3 - G3
	o LOAD o	COMBINATION:		1.2D + 1.0	6Lr + 0.8W								
	o <b>DEFLE</b>	CTION AMPLIFIC	CATION FACTOR:	C <sub>d</sub> =	5.5								
	o (SEISM	IIC) IMPORTANC	E FACTOR	I <sub>E</sub> =	1.0								
	o MOMEI TOTAL	NT FRAME RESI SEISMIC SHEAI	STS WHAT % OF R TO THE BUIDLIN	THE NG?	25%								
		DUE TO FORG BENDING ALONG MOMENT FR	CES THAT CAUSE THE X-AXIS OF THE AME COLUMNS	TOTAL L STORY O ("LEANE	<i>INFACTORE</i> N ALL COLU R" + "NON-L	ED AXIAL L JMNS OF T LEANER" C	OAD PER HE STORY OLUMNS)	ΤΟΤΑ	L FACTORED	axial load,	ΣP <sub>u</sub> , PER STO	RY	STABILITY
0707	LENGTH	SHEAR DUE TO	ELASTIC INTERSTORY		(ki	ps)			LOAD F	ACTOR		TOTAL	PER STORY
NUMBI	R HEIGHT)	HORIZONTAL LOAD	DRIFT Δ <sub>oh</sub> DUE TO ΣHi	DEAD LOAD DI	LIVE LOAD	ROOF LIVE LOAD	SEISMIC WEIGHT DL + P-LL	D.L.	L.L.	ROOF L.L.	SEISMIC VERTICAL	ΣP <sub>ui</sub> (kips)	$\boldsymbol{\theta}_i$
18	13.0 ft	∑H <sub>i</sub> 185 kine	0.25 in	1 513	0	Lr 288	1 501	1 815 6	0.0	460.8	0.0	2 276 4	0.020
17	13.0 ft	352 kins	0.25 m	2 821	1.008	288	3 187	3 385 2	0.0	460.8	0.0	3 846 0	0.020
16	13.0 ft	501 kins	0.37 in	4 129	2 016	288	4 783	4 954 8	0.0	460.8	0.0	5 415 6	0.020
15	13.0 ft	633 kins	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 Eng	ineeri		). 18 - STO	RY BUII	LDINGS			BY	SMG	DATE	4/22/05	SI	HEET NO.
		CUSTO	MER DESI	GN 18A	۰.			CKD		DATE			OF
SUBJECT			STABILI	түсс	)EFFIC	IENT	ALON	G COLU	MN Y-AX	<b>(IS</b> , θ <sub>y</sub>		M	OMENT FRAME MF A3 - G3
0 0	LOAD C DEFLEC (SEISM)	OMBINATION: TION AMPLIFICA	ATION FACTOR: E FACTOR	1.2D + 1.6 C <sub>d</sub> = I <sub>E</sub> =	3Lr + 0.8W 5.5 1.0								
0	MOMEN TOTAL :	IT FRAME RESIS SEISMIC SHEAR	TS WHAT % OF 1 TO THE BUIDLIN	THE IG?	25%								
		DUE TO FORCE BENDING ALONG T MOMENT FRA	S THAT CAUSE THE Y-AXIS OF THE ME COLUMNS	TOTAL U STORY OF ("LEANE!	<i>'NFACTORE</i> N ALL COLL R" + "NON-L	D AXIAL LO JMNS OF T EANER" C	OAD PER HE STORY OLUMNS)	TOTA	L FACTORED A	XIAL LOAD,	ΣP <sub>u</sub> , PER STOP	YY	STABILITY
STORY NUMBER	LENGTH <i>(STORY</i> <i>HEIGHT)</i> L	SHEAR DUE TO ANY HORIZONTAL LOAD	ELASTIC INTERSTORY DRIFT Δ <sub>oh</sub> DUE TO ΣH <sub>i</sub>	DEAD LOAD DL	(kir LIVE LOAD LL	ROOF LIVE LOAD	SEISMIC WEIGHT DL + P-LL	D.L. 1.2	LOAD FA	ROOF L.L.	SEISMIC VERTICAL 0.2S <sub>DS</sub> = 0	TOTAL ΣP <sub>ui</sub> (kips)	COEFFICIENT PER STORY θ <sub>i</sub>
18 17 16 15 14 13 12 11 10 9 8 7 6 5 4 3 2 1	13.0 ft 13.0 ft	2.h 185 kips 352 kips 501 kips 633 kips 749 kips 851 kips 1,013 kips 1,013 kips 1,075 kips 1,126 kips 1,200 kips 1,224 kips 1,224 kips 1,253 kips 1,260 kips 1,264 kips	0.25 in 0.37 in 0.45 in 0.45 in 0.52 in 0.55 in 0.55 in 0.55 in 0.57 in 0.57 in 0.58 in 0.59 in 0.59 in 0.59 in 0.58 in 0.59 in 0.54 in 0.32 in	1,513 2,821 4,129 5,437 6,745 8,053 9,361 10,669 11,977 13,285 14,593 15,901 17,209 18,517 19,825 21,133 22,441 23,749	0 1,008 2,016 3,024 4,032 5,040 6,048 7,056 8,064 9,072 10,080 11,088 12,096 13,104 14,112 15,120 16,128 17,136	Lr 288 288 288 288 288 288 288 288 288 28	1,591 3,187 4,783 6,379 7,975 9,571 11,167 12,763 14,359 15,955 17,551 19,147 20,743 22,339 23,935 25,531 27,127 28,723	1,815.6 3,385.2 4,954.8 6,524.4 8,094.0 9,663.6 11,233.2 12,802.8 14,372.4 15,942.0 17,511.6 19,081.2 20,650.8 22,220.4 23,790.0 25,359.6 26,929.2 28,498.8	0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0	460.8 460.8 460.8 460.8 460.8 460.8 460.8 460.8 460.8 460.8 460.8 460.8 460.8 460.8 460.8 460.8 460.8 460.8 460.8 460.8	0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0	2,276.4 3,846.0 5,415.6 6,985.2 8,554.8 10,124.4 11,694.0 13,263.6 14,833.2 16,402.8 17,972.4 19,542.0 21,111.6 22,681.2 24,250.8 25,820.4 27,390.0 28,959.6	0.020 0.026 0.031 0.033 0.038 0.042 0.044 0.048 0.049 0.051 0.056 0.060 0.064 0.069 0.073 0.075 0.047

6	iG Eng	jineeri	JOB NC DOB NC CUSTO	). 18 - STO MER DESI	RY BUII GN 18A	DINGS			BY CKD	SMG	DATE DATE	4/22/05	S⊦	IEET NO. OF
SI	UBJECT		B2 CALC	ULATION	- FOR	BEND	DING A	ALONG	THE X	-AXIS O	F THE	COLUM	V <sup>MC</sup>	MENT FRAME MF A3 - G3
	o	LOAD	DUE TO FORCE BENDING ALONG T	<b>1.2D + 0.5L + 0.5</b> IS THAT CAUSE HE X-AXIS OF THE COLUMN	iLr + 1.6W TOTAL U	(L.C. #5)	) D axial L Imns of T	OAD PER HE STORY	ТОТА	L FACTORED A	XIAL LOAD,	$\Sigma P_u$ , PER STOR	Y	
	STORY NUMBER	LENGTH (STORY HEIGHT)	TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL	ELASTIC INTERSTORY DRIFT	("LEANER	LIVE	EANER" C ps) ROOF LIVE	SEISMIC	D.L.	LOAD FA	ACTOR ROOF L.L.	SEISMIC VERTICAL	TOTAL ΣP <sub>ui</sub>	STORY B <sub>2i_X-AXIS</sub>
	18	L 13.0 ft	LOAD E ΣΗ <sub>i</sub> 185 kips	DUE TO ∑H <sub>i</sub>	LOAD DL 1.513		LOAD Lr 288	UL + P-LL	1.2	0.5	0.5 144.0	$0.2S_{DS} = 0$ 0.0	(KIPS)	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 15	13.0 ft 13.0 ft	501 kips 633 kips	0.45 in 0.47 in	4,129 5,437	2,016 3,024	288 288	4,783 6,379	4,954.8 6,524.4	1,008.0 1,512.0	144.0 144.0	0.0	6,106.8 8,180.4	1.036 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 12	13.0 ft	851 kips 938 kips	0.55 in 0.55 in	8,053 9,361	5,040 6.048	288 288	9,571 11 167	9,663.6 11,233,2	2,520.0 3 024 0	144.0 144.0	0.0	12,327.6 14 401 2	1.054
	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	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 7	13.0 ft	1,168 kips	0.57 in 0.57 in	14,593	11,080	288	19,147	17,511.6	5,040.0	144.0	0.0	22,695.6	1.076
	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 0.54 in	21,133	15,120 16,128	288	25,531	25,359.6	7,560.0 8 064 0	144.0 144.0	0.0	33,063.6 35 137 2	1.108
	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 Ena	ineen	JOB NC	). 18 - STO	RY BUII	DINGS			BY	SMG	DATE	4/22/05	Sł	IEET NO.
	Juneen	CUSTO	MER DESI	GN 18A				CKD		DATE			OF
SUBJECT		B2 CALC	ULATION	- FOR	BEND	DING /	ALONG	THE	Y-AXIS	OF THE	COLUM	N <sup>MO</sup>	OMENT FRAME MF A3 - G3
o	LOAD	COMBINATION =	1.2D + 0.5L + 0.	5Lr + 1.6W									
		DUE TO FORCE BENDING ALONG T MOMENT FRA	ES THAT CAUSE THE Y-AXIS OF THE ME COLUMNS	TOTAL U STORY OF	NFACTORE N ALL COLU R" + "NON-L	D AXIAL L IMNS OF 1 EANER" (	OAD PER THE STORY COLUMNS)	тот	AL FACTORED	AXIAL LOAD,	ΣP <sub>u</sub> , PER STO	RY	
STORY NUMBER	LENGTH <i>(STORY</i> <i>HEIGHT)</i> L	SHEAR DUE TO SEISMIC HORIZONTAL LOAD E 2H:	ELASTIC INTERSTORY DRIFT $\Delta_{oh}$ DUE TO $\Sigma H_i$	DEAD LOAD DL	(kij LIVE LOAD LL	ROOF LIVE LOAD	SEISMIC WEIGHT DL + P-LL	D.L. 1.2	LOAD F	FACTOR ROOF L.L. 0.5	SEISMIC VERTICAL 0.2S <sub>DS</sub> = 0	TOTAL ΣP <sub>ui</sub> (kips)	STORY B <sub>2i_Y-AXIS</sub>
18 17	13.0 ft	185 kips 352 kips	0.25 in 0.37 in	1,513	0	288	1,591 3 187	1,815.6	0.0	144.0 144.0	0.0	1,959.6	1.017
16	13.0 ft	502 kips	0.45 in	4,129	2,016	288	4,783	4,954.8	1,008.0	144.0	0.0	6,106.8	1.026
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
0 7	13.0 ft	1,100 kips	0.57 in	14,595	11,088	200	19 147	10,081.2	5,040.0	144.0	0.0	22,095.0	1.070
6	13.0 ft	1,200 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 Eng	ineeri		). 18 - STO MER DESI	RY BUI	LDINGS	i		BY CKD	SMG	DATE DATE	4/22/05	S	HEET NO.
SUBJECT			STABIL	түсс	DEFFIC	IENT	ALON	G COLU	IMN X-A	XIS, θ <sub>x</sub>		м	OMENT FRAME MF A3 - G3
0 0 0	LOAD ( DEFLEC (SEISM MOMEN TOTAL	COMBINATION: CTION AMPLIFIC IC) IMPORTANCE IT FRAME RESIS SEISMIC SHEAR DUE TO FORCI	1. ATION FACTOR: E FACTOR TS WHAT % OF T TO THE BUIDLIN	2D + 0.5L + C <sub>d</sub> = I <sub>E</sub> = IG?	- 0.5Lr + 1.6 5.5 1.0 25%	W	OAD DED						
		BENDING ALONG	THE X-AXIS OF THE ME COLUMNS	STORY O	N ALL COLL R" + "NON-L	JMNS OF T EANER" C	HE STORY	TOTA	L FACTORED	AXIAL LOAD,	$\Sigma P_u$ , PER STO	RY	STABILITY
	LENGTH	SHEAR DUE TO	ELASTIC INTERSTORY	(	(ki	ps)	,		LOAD F	ACTOR		TOTAL	COEFFICIENT PER STORY
STORY NUMBER	$\begin{array}{c c c c c c c c c c c c c c c c c c c $												$\Theta_{i}$
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
10	13.0 ft	1,013 kips	0.57 in	11,009	8 064	200	12,703	12,002.0	3,526.0 4 032 0	144.0	0.0	18 548 4	0.059
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

ଜ	යි <b>සි</b> බැග්	ineeri	JOB NC	). 18 - STO	ry Buii	DINGS			BY	SMG	DATE	4/22/05	SI	HEET NO.
<u> </u>			CUSTO	MER DESI	GN 18A				СКД		DATE			OF
รเ	JBJECT			STABIL	түсс	DEFFIC	CIENT	ALON	g Colu	IMN Y-A	XIS, θ <sub>y</sub>		M	OMENT FRAME MF A3 - G3
	0 0 0	LOAD C DEFLEC (SEISMI MOMEN TOTAL :	COMBINATION: CTION AMPLIFIC, IC) IMPORTANCE IT FRAME RESIS SEISMIC SHEAR	1 ATION FACTOR: E FACTOR TS WHAT % OF TO THE BUIDLIN	2D + 0.5L + C <sub>d</sub> = I <sub>E</sub> = IHE IG?	• 0.5Lr + 1.6 5.5 1.0 25%	w							
			DUE TO FORCE	ES THAT CAUSE	TOTAL U		ED AXIAL L	OAD PER	ΤΟΤΑ	L FACTORED	AXIAL LOAD,	$\Sigma P_u$ , PER STO	ιRY	
				IME COLUMINS	("LEANE	R" + "NON-L	EANER" C	OLUMNS)						STABILITY
1		LENGTH	SHEAR DUE TO	ELASTIC INTERSTORY		(ki	ps)			LOAD F	ACTOR		τοται	PER STORY
	STORY NUMBER	(STORY HEIGHT)	ANY HORIZONTAL	DRIFT	DEAD	LIVE	SEISMIC	D.L.	L.L.	ROOF L.L.	SEISMIC VERTICAL	ΣP <sub>ui</sub>	θ <sub>i</sub>	
		L	LOAD ΣH <sub>i</sub>	DUE TO SH	DL	LUAD	DL + P-LL	1.2	0.5	0.5	0.2S <sub>DS</sub> = 0	(kips)		
	18	13.0 ft	185 kips	0.25 in	1,513	0	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	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	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	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

6	G Eng	ineeri		. 18 - STO	RY BUI	LDINGS			BY	SMG	DATE	4/22/05	S⊦	IEET NO. OF
SI	JBJECT		B2 CALC				DING	ALONG		-AXIS C		COLUM	V <sup>MC</sup>	MENT FRAME
					101	DENE		Lond				0020		MF A3 - G3
	0	LOAD	COMBINATION =	1.2D + 0.5L + 1.0	E	(L.C. #6	)							
			DUE TO FORCE BENDING ALONG T MOMENT FRA	S THAT CAUSE HE X-AXIS OF THE ME COLUMNS	TOTAL U STORY OF ("LEANEI	<i>INFACTORE</i> N ALL COLU R" + "NON-L	D AXIAL L IMNS OF T EANER" C	OAD PER HE STORY OLUMNS)	ΤΟΤΑ	L FACTORED /	AXIAL LOAD,	$\Sigma P_u$ , PER STOR	Y	07007
	STORY	LENGTH (STORY	SHEAR DUE TO	ELASTIC		(kij	DS)			LOAD F.	ACTOR	05/01//0	TOTAL	STORY B <sub>2i_X-AXIS</sub>
	NUMBER	HEIGHT) L	HORIZONTAL LOAD E	DRIFT Δ <sub>oh</sub> DUE TO ΣH <sub>i</sub>	DEAD LOAD DL	LIVE LOAD LL	LIVE	SEISMIC WEIGHT DL + P-LL	D.L.	L.L. 0.5	ROOF L.L.	VERTICAL 0.2S <sub>DS</sub> = 0.2	ΣP <sub>ui</sub> (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	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 7.056.0	0.0	4,467.8	33,240.2	1.113
	4	13.0 ft	1,200 kips	0.59 in	21 133	15,120	200	25,535	25,750.0	7,050.0	0.0	5 106 2	38 025 8	1.121
	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 Eng	ineer	JOB NC	). 18 - STO	RY BUII	DINGS			BY	SMG	DATE	4/22/05	SH	IEET NO.
	JUUUGGUL	CUSTO	MER DESI	GN 18A				CKD		DATE			OF
SUBJECT		B2 CALC	ULATION	- FOR	BEND	DING /	ALONG	THE	Y-AXIS C	OF THE	COLUM	N <sup>MO</sup>	DMENT FRAME MF A3 - G3
o	LOAD	COMBINATION =	1.2D + 0.5L + 1.0	DE									
		DUE TO FORCE BENDING ALONG T MOMENT FRA	ES THAT CAUSE THE <mark>Y-AXIS</mark> OF THE ME COLUMNS	TOTAL U STORY OF	NFACTORE N ALL COLU R" + "NON-L	D AXIAL L IMNS OF T EANER" C	OAD PER HE STORY OLUMNS)	тот	AL FACTORED	AXIAL LOAD,	ΣP <sub>u</sub> , PER STO	RY	
STORY NUMBER	LENGTH (STORY HEIGHT) L	SHEAR DUE TO SEISMIC HORIZONTAL LOAD E	ELASTIC INTERSTORY DRIFT $\Delta_{oh}$ DUE TO $\Sigma H_i$	DEAD LOAD DL	(kij LIVE LOAD LL	ROOF LIVE LOAD	SEISMIC WEIGHT DL + P-LL	D.L. 1.2	LOAD F.	ACTOR ROOF L.L.	SEISMIC VERTICAL 0.2S <sub>DS</sub> = 0.2	TOTAL ΣP <sub>ui</sub> (kips)	STORY B <sub>2i_Y-AXIS</sub>
18	13.0 ft	185 kips	0.25 in	1.513	0	Lr 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
	13.011	1,204 Kips	0.32 III	23,749	17,156	200	20,723	20,490.0	6,306.0	0.0	5,744.0	42,011.4	1.0/5

										-		-	
GG Eng	ineeri	JOB NO	). 18 - STO	RY BUI	LDINGS	;		ВҮ	SMG	DATE	4/22/05	S	HEET NO.
	,	CUSTO	MER DESI	GN 18A	L .			СКД		DATE			OF
SUBJECT		_	STABIL	түсс	DEFFIC	CIENT	ALON	G COLU	IMN X-A	XIS, θ <sub>x</sub>		м	OMENT FRAME MF A3 - G3
0 0 0	LOAD ( DEFLEG (SEISM MOMEI TOTAL	COMBINATION: CTION AMPLIFIC. IC) IMPORTANCE IT FRAME RESIS SEISMIC SHEAR	ATION FACTOR: E FACTOR TTS WHAT % OF TO THE BUIDLIN	1.2D + 0. C <sub>d</sub> = I <sub>E</sub> = THE IG?	.5L + 1.0E 5.5 1.0 25%								
		DUE TO FORCE BENDING ALONG MOMENT FRA	ES THAT CAUSE THE <mark>X-AXIS</mark> OF THE ME COLUMNS	TOTAL L STORY O	INFACTORE N ALL COLL B" + "NON-I	ED AXIAL L JMNS OF T EANEB" C	OAD PER HE STORY OLUMNS)	ΤΟΤΑ	L FACTORED	AXIAL LOAD,	$\Sigma P_u$ , PER STO	RY	STABILITY
		TOTAL STORY	ELASTIC	( LEANE	(ki	DS)	OLUNINS)		I OAD F	ACTOR			COEFFICIENT
STORY NUMBER	$\begin{array}{c c c c c c c c c c c c c c c c c c c $												PER STORY θ <sub>i</sub>
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	93122	0.044
14	12.0.#	740 kips	0.47 in	6 745	4.022	200	7.075	9 004 0	2.016.0	0.0	1,275.0	11 705 0	0.052
14	13.0 1	749 kips	0.52 m	6,745	4,032	200	7,975	0,094.0	2,016.0	0.0	1,595.0	11,705.0	0.052
13	13.0 π	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 BL	JILDINGS		BY	SMG	DATE	4/22/05	SHEET NO.
	CUST	OMER	DESIGN 18	A		CKD		DATE		OF
SUBJECT		ST	ABILITY C	OEFFICI	ENT ALO		IMN X-A	XIS, θ <sub>x</sub>		MOMENT FRAME MF A3 - G3
• LOAD COME	BINATION	:	1.2D +	0.5L + 1.0E						
o DEFLECTIO	N AMPLIF	CATION F	ACTOR: C <sub>d</sub>	= 5.5						
Г			DUE TO FORCE	S THAT CAUSE	TOTAL STORY					
			BENDING ALONG T MOMENT FRA	HE X-AXIS OF THE ME COLUMNS	SHEAR	RATIO OF SHEAB	MAXIMUM			
			TOTAL STORY	INTERCTORY	(OF ALL OF	DEMAND /	ALLOWED STABILITY	STABILITY COEFFICIENT	001005117	
	STORY	(STORY	SHEAR DUE TO SEISMIC	DRIFT	THE SEISMIC RESISTING	CAPACITY	COEFFICIENT PER STORY	PER STORY	COMMENT	
1	NUMBER	HEIGHT)	HORIZONTAL LOAD E	Δ <sub>oh</sub> DUE TO ΣΗ	MOMENT	PER STORY β	$\theta_{i\_max}$	21		
		1	ΣH <sub>i</sub>	BOE TO ZH	FRAMES)	F				
	18	13.0 ft	185 kips	0.25 in	13,494 kips	0.0137	0.250	0.018	ок	
	17	13.0 ft	352 kips	0.37 in	13,494 kips	0.0261	0.250	0.031	ок	
	16	13.0 ft	501 kips	0.45 in	13,494 kips	0.0371	0.250	0.040	ОК	
	15	13.0 ft	633 kips	0.47 in	11,476 kips	0.0552	0.250	0.044	ок	
	14	13.0 ft	749 kips	0.52 in	11,476 kips	0.0653	0.250	0.052	ок	
	13	13.0 ft	851 Kips	0.55 in	11,476 Kips	0.0742	0.250	0.058	OK	
	12	13.0 ft	936 Kips 1 013 kips	0.55 m	14,019 kips	0.0669	0.250	0.062	OK	
	10	13.0 ft	1.075 kips	0.55 in	12.867 kips	0.0835	0.250	0.070	ок	
	9	13.0 ft	1,126 kips	0.55 in	12,867 kips	0.0875	0.250	0.074	ок	
	8	13.0 ft	1,168 kips	0.57 in	12,867 kips	0.0908	0.250	0.082	ок	
	7	13.0 ft	1,200 kips	0.57 in	12,867 kips	0.0933	0.250	0.087	ок	
	6	13.0 ft	1,224 kips	0.58 in	12,867 kips	0.0951	0.250	0.094	ок	
	5	13.0 ft	1,241 kips	0.59 in	12,867 kips	0.0964	0.250	0.101	ок	
	4	13.0 ft	1,253 kips	0.59 in	12,867 kips	0.0974	0.250	0.108	ок	
	3	13.0 ft	1,260 kips	0.58 in	12,867 kips	0.0979	0.250	0.112	ОК	
	2	13.0 ft	1,263 kips	0.54 in	12,867 kips	0.0982	0.250	0.111	ОК	
	1	13.0 ft	1,264 kips	0.32 in	12,867 kips	0.0982	0.250	0.069	ОК	
L										
1										

								-		-			
GG Eng	ineeri		D. 18 - STO	RY BUI	LDINGS	;		BY	SMG	DATE	4/22/05	s	HEET NO.
	,	CUSTO	OMER DESI	GN 18A	l.			СКД		DATE			OF
SUBJECT			STABIL		DEFFIC	CIENT	ALON	G COLU	IMN Y-A	XIS, θ <sub>y</sub>		м	OMENT FRAME MF A3 - G3
0 0 0	LOAD ( DEFLEC (SEISM MOMEI TOTAL	COMBINATION: CTION AMPLIFIC IC) IMPORTANC IT FRAME RESI SEISMIC SHEAF	CATION FACTOR: E FACTOR STS WHAT % OF R TO THE BUIDLIN	1.2D + 0. C <sub>d</sub> = I <sub>E</sub> = IG?	5L + 1.0E 5.5 1.0 25%								
		DUE TO FORC BENDING ALONG MOMENT FR	ES THAT CAUSE THE Y-AXIS OF THE AME COLUMNS	TOTAL L STORY O	INFACTORE	ED AXIAL L JMNS OF T	OAD PER HE STORY	ΤΟΤΑ	L FACTORED	AXIAL LOAD,	$\Sigma P_u$ , PER STO	RY	
		TOTAL STORY	ELASTIC	("LEANE	R" + "NON-L (ki	_EANER" C ps)	OLUMNS)		LOAD F	ACTOR			COEFFICIENT
STORY NUMBER	$\left( \begin{array}{c} \text{Length} \\ (STORY \\ R \\ HEIGHT \\ L \\ L \\ 13.0 \text{ ft} \end{array} \right) \text{SPLAR DUE TO HAVE } \left( \begin{array}{c} \text{INTERSTORY} \\ DRIFT \\ DRIFT \\ DL \\ D$												PER STORY θ <sub>i</sub>
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
	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
5	13.0 π 12.0 #	1,224 kips	0.58 in	19,517	12,096	288	20,743	20,650.8	6,048.0	0.0	4,148.0	30,847.4	0.094
5	13.0 ft	1,241 kips	0.59 m	10,517	14 112	200	22,339	22,220.4	7.056.0	0.0	4,407.0	35,240.2	0.101
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 N	NO. 18	- STORY BL	JILDINGS		BY	BY SMG DATE 4/22/05			SHEET NO.
	CUST	OMER	DESIGN 18	A		CKD		DATE		OF
SUBJECT		ST	ABILITY C	OEFFICI	ENT ALOI	MOMENT FRAME MF A3 - G3				
○ LOAD COME ○ DEFLECTIO	BINATION: N AMPLIF	: ICATION F	1.2D + ACTOR: C <sub>o</sub> DUE TO FORCE BENDING ALONG T MOMENT FRA	0.5L + 1.0E = 5.5 IS THAT CAUSE HE Y-AXIS OF THE ME COLUMNS	TOTAL STORY SHEAR	RATIO OF SHEAR	MAXIMUM	STADII ITY		
٩	STORY NUMBER	LENGTH (STORY HEIGHT) L	TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH <sub>i</sub>	INTERSTORY DRIFT Δ <sub>oh</sub> DUE TO ΣH <sub>i</sub>	(OF ALL OF THE SEISMIC RESISTING MOMENT FRAMES)	DEMAND / SHEAR CAPACITY PER STORY β	STABILITY COEFFICIENT PER STORY θ <sub>i_max</sub>	COEFFICIENT PER STORY θ <sub>i</sub>	COMMENT	
	18	13.0 ft	185 kips	0.25 in	13,494 kips	0.0137	0.250	0.018	ок	
	17	13.0 ft	352 kips	0.37 in	13,494 kips	0.0261	0.250	0.031	ок	
	16	13.0 ft	501 kips	0.45 in	13,494 kips	0.0371	0.250	0.040	ок	
	15	13.0 ft	633 kips	0.47 in	11,476 kips	0.0552	0.250	0.044	ОК	
	14	13.0 ft	749 kips	0.52 in	11,476 kips	0.0653	0.250	0.052	ок	
	13	13.0 ft	851 kips	0.55 in	11,476 kips	0.0742	0.250	0.058	ОК	
	12	13.0 ft	938 Kips	0.55 in	14,019 kips	0.0669	0.250	0.062	OK	
	10	13.0 ft	1,013 kips	0.57 in	12 867 kips	0.0723	0.250	0.000	OK	
	9	13.0 ft	1,126 kips	0.55 in	12,867 kips	0.0875	0.250	0.074	ок	
	8	13.0 ft	1,168 kips	0.57 in	12,867 kips	0.0908	0.250	0.082	ок	
	7	13.0 ft	1,200 kips	0.57 in	12,867 kips	0.0933	0.250	0.087	ок	
	6	13.0 ft	1,224 kips	0.58 in	12,867 kips	0.0951	0.250	0.094	ок	
	5	13.0 ft	1,241 kips	0.59 in	12,867 kips	0.0964	0.250	0.101	ок	
	4	13.0 ft	1,253 kips	0.59 in	12,867 kips	0.0974	0.250	0.108	ок	
	3	13.0 ft	1,260 kips	0.58 in	12,867 kips	0.0979	0.250	0.112	ок	
	2	13.0 ft	1,263 kips	0.54 in	12,867 kips	0.0982	0.250	0.111	ок	
	1	13.0 ft	1,264 kips	0.32 in	12,867 kips	0.0982	0.250	0.069	οκ	

	COLUMN		CONTROLLING	BUILDING MAX.	COMMENTS FO	R: DESIGN 18A
NAME	MEMBER SIZE	INTERACTION VALUE	LOAD COMBINATION	INTERACTION 0.9533	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	0.2409/658/	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
A3-0 A3-9	HSS 20 x 20 x 0.5	0.227481653	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 B3-4	HSS 20 X 20 X 0.5	0.549216082	6		OK - STEEL AREA IS > 1% OF TOTAL AREA 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	0.1884/6/1/	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
B3-16 B2 17	HSS 16 x 16 x 0.75	0.133627385	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
C2 12	HSS 18 X 18 X 0.625	0.431545974	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
D3-0	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 ABEA IS > 1% OF TOTAL ABEA	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		UK - STEEL AREA IS > 1% OF TOTAL AREA	
D3-18		0.113080092	e d	CONTROL SI	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	CONTROLLING	BUILDING MAX.	COMMENTS FO	R: DESIGN 18A
NAME	MEMBER SIZE	INTERACTION VALUE	LOAD COMBINATION	INTERACTION 0.9533	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,  $\theta$ , 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.

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

Contraction       Carton design 188       Critical       Date       OF         SUBJECT       DESIGN PARAMETERS SUMMARY       Instance       Instance       Instance         SUBJECT       DESIGN PARAMETERS SUMMARY       Instance       Instance       Instance         SUBJECT       DESIGN NPUTS:       Instance       Instance       Instance       Instance         Instance       Instance       Instance       Instance       Instance       Instance       Instance         Instance       Instance       Instance       Instance       Instance       Instance       Instance       Instance       Instance       Instance	ദ്രି <u>ള</u> ്ള പ്രത്താം പ	JOB NO. 18	- STORY BUILDINGS	BY SMG	DATE 4/22/05	SHEET NO.
SUBJECT DESIGN PARAMETERS SUMMARY 128 13 • DESIGN INPUTS: • TOLI MUMBER OF COLLANS BEING ANALIZED 13 • YELD STRENCT: HSS F, = 00 W CONCRETE REINFORCEMENT, F, = 00 W CONCRETE DENSITY w. 145 BH • OCONCRETE COMPRESSIVE STRENCTI F, = 100 W L, = 00 W L, =	co milimeermili	CUSTOMER	DESIGN 18B	СКД	DATE	OF
• DESIGN INPUTS:       • TOL NUMBER OF COLUMNS BEING ANALYZED       124         • YELD STRENGTH:       HSS. F., 10 SM         • ONDULUS OF FLASTIONY:       HSS. F., 20 SM         • ONDERFE DENSITY       HSS. F., 20 SM         • ONDERFE TENFORCEMENT       AFEA A., 0 SM         • ONDERFE TENFORCEMENT       HSS. F., 20 SM         • ONDERFE TENF	SUBJECT		DESIGN PARAMETERS	SUMMARY		MOMENT FRAME MF A3 - G3
• DESIGN INPUTS:         • TOL INJURGED OF COLLINS BEING ANALYZED         13           • YELD STRENGTH:         HSS, F, = 0.54         CONCRETE REINFORCEMENT, F, = 0.54           • MCOLLUS OF FLATTOTY:         HSS, F, = 0.54         CONCRETE REINFORCEMENT, F, = 0.54           • MCOLLUS OF FLATTOTY:         HSS, F, = 0.54         CONCRETE REINFORCEMENT, F, = 0.54           • MCOLLUS OF FLATTOTY:         HSS, F, = 0.54         CONCRETE REINFORCEMENT         F, = 1.05           • MONDUM CONCRETE COMPRESSIVE STRENGT         F, = 1.05         F, = 0.54         F, = 0.54           • OCHORETE REINFORCEMENT         AFREA, A, = 0.07         F, = 0.074         F, = 0.074         F, = 0.074           • OCHORETE REINFORCEMENT         AFREA, A, = 0.07         F, = 0.074         F, = 0.074         F, = 0.074           • OCHORETE REINFORCEMENT         AFREA, A, = 0.07         F, = 0.074         F, = 0.074         F, = 0.074           • CONCRETE REINFORCEMENT         AFREA, A, = 0.07         F, = 0.074         F, = 0.074 <th></th> <th></th> <th></th> <th></th> <th></th> <th></th>						
<ul> <li>NILLD STREINGTH: 1958, F, = 804</li> <li>MODULUS OF ELASTICITY INS. F, = 28000 bsi CONCRETE REINFORCEMENT, F, = 0.804</li> <li>MODULUS OF ELASTICITY INS. F, = 28000 bsi CONCRETE REINFORCEMENT F, = 18000 bsi CONCRETE REINFORCEMENT F, = 18000 bsi CONCRETE REINFORCEMENT F, = 18000 bsi US CONCRETE REINFORCEMENT F, = 1800 bsi US CONCRETE REINFORCEMENT AREA, A, = 0.014 U, = 0.014</li></ul>	o DESIGN	I INPUTS:	o TOAL NUMBER OF COLUMNS BEI	NG ANALYZED	126	
<ul> <li>MODULUS OF ELASTICITY: INS. E., 2 30,00 ksi CONCRETE COMPRESSIVE STRENOTH</li> <li>MINIMUM CONCRETE COMPRESSIVE STRENOTH</li> <li>CONCRETE DENSITY</li> <li>CONCRETE DENSITY</li> <li>CONCRETE REINFORCEMENT</li> <li>ADEMA J., 0 004 J., 0 0</li></ul>			o YIELD STRENGTH:	CONCRETE REINFORCEM	HSS, F <sub>y</sub> = 80 ksi ENT, F <sub>yr</sub> = 0 ksi	
MINIMUM CONCRETE DENSITY     w.145 MIT     w.145 MIT     OCNORETE DENSITY     w.145 MIT     OCNORETE REINFORCEMENT     AREA, A,= 0.0 FF     L,= 0.0 FF			• MODULUS OF ELASTICITY:	CONCRETE REINFORCEM	HSS, E <sub>s</sub> = 29,000 ksi ENT, E <sub>cr</sub> = 29,000 ksi	
OCACRETE DENSITY     w 148 lbt*      CONCRETE RENFORCEMENT     AFEA A, - 0.0 in*     Up - 0.0 in*4     Up - 0.0 in*4     Z, - 0.0 in*     Concrete DENSITY     SESSITIONE FACTORS     AXIAL COMPRESSION, - 0.75     ELEVIPAL BENINNE, -0.90     OFFICIENT, - 1.00     VERTICAL SEISMIC FACTOR: 0.25     OFFICIENT, - 1.00     VERTICAL SEISMIC FACTOR: 0.25     OFFICIENT, - 0.005     FACTOR TO ACCOUNT FOR 5% ACCIDENTAL TORSION (SMPLIFIED APPROACH) - 0.005			O MINIMUM CONCRETE COMPRESS	SIVE STRENGTH	f' <sub>c</sub> = 16.0 ksi	
OCHCRETE REINFORCEMENT     AREA, A., E. 0.0 In <sup>4</sup> L, C. 0.0 In <sup>4</sup> L, C. 0.0 In <sup>4</sup> Z, C. 0			o CONCRETE DENSITY		w = 145 lb/ft <sup>3</sup>	
<ul> <li>■ RESISTANCE FACTORS</li> <li>AXIAL COMPRESSION, 6,= 0.90</li> <li>■ SEISMIC PARAMETERS</li> <li>■ REDUNDANCY COEFFICIENT, p = 1.00</li> <li>■ VERTICAL SEISMIC ACTOR: 0.25,= 0.20</li> <li>■ ORTHOGONAL LOAD FACTOR: 0.25,= 0.20</li> <li>■ ORTHOGONAL LOAD FACTOR: 0.40,5 0F SHARED COLUMNIS = 0.30</li> <li>■ FACTOR TO ACCOUNT FOR 5% ACCIDENTAL TORSION ("SIMPLIFIED APPROACH") = 0.025</li> </ul>			<ul> <li>CONCRETE REINFORCEMENT</li> </ul>	A	REA, $A_{sr} = 0.0 \text{ in}^2$ $I_{cor} = 0.0 \text{ in}^4$ $I_{yyr} = 0.0 \text{ in}^4$ $Z_{cor} = 0.0 \text{ in}^3$ $Z_{yyr} = 0.0 \text{ in}^3$	
• SEISMIC PARAMETERS REDUNDANCY COEFFICIENT, p = 1.00 VERTICAL SEISMIC "FACTOR," 0.25mg = 0.20 ORTHOGONAL LOAD FACTOR ALONG Y-AXIS OF SHARED COLUMNS = 0.30 FACTOR TO ACCOUNT FOR 5% ACCIDENTAL TORSION ("SIMPLIFIED APPROACH") = 0.025			0 RESISTANCE FACTORS	AXIAL COMPRES FLEXURAL BEN	SION, $\phi_c = 0.75$ DING, $\phi_b = 0.90$	
			o SEISMIC PARAMETERS VI ORTHOGONAL LOAD FACTOR AL FACTOR TO ACCOUNT FOR 5% ACCIDENTAL TOP	REDUNDANCY COEFFI ERTICAL SEISMIC "FACTOR ONG Y-AXIS OF SHARED CC <i>SION</i> ("SIMPLIFIED APPRO	DIENT, p = 1.00 "0.2S <sub>DS</sub> = 0.20 DLUMNS = 0.30 ACH") = 0.025	

6	iG Eng	jineeri		NO. 18 - STO	RY BUII GN 18B	LDINGS			BY CKD	SMG	DATE DATE	4/22/05	SHEET NO.		
S	JBJECT		B2 CAL	CULATION	- FOR	R BEND	DING /	ALONG		(-AXIS O	F THE	COLUMI	<b>и</b> мс	MENT FRAME <i>MF A3 - G3</i>	
	0 STORY NUMBER 18 17 16 15 14 13 12 11 10 9 8 7 6 5 4 3 2 1	LOAD ( (STORY HEIGHT) L 13.0 ft 13.0 ft	COMBINATION DUE TO FO BENDING ALON MOMENT TOTAL STOF SHEAR DUE SEISMIC HORIZONTA LOAD E 2H, 185 kips 352 kips 352 kips 501 kips 938 kips 1,013 kips 1,075 kips 1,126 kips 1,224 kips 1,224 kips 1,260 kips 1,264 kips	I = 1.2D + 1.6Lr + 0           RCES THAT CAUSE GATE X-AXIS OF THE RAME COLUMNS           INTERSTORY DRIFT Adh DUE TO 2H, U = 0.23 in 0.53 in 0.53 in 0.53 in 0.57 in 0.57 in 0.59 in 0.57 in 0.59 in 0.61 in 0.62 in 0.61 in 0.62 in 0.61 in 0.61 in	8W TOTAL <i>U</i> STORY OI ("LEANE! DEAD LOAD DL 1,513 2,821 4,129 5,437 6,745 8,053 9,361 10,669 11,977 13,285 14,593 15,901 17,209 18,517 19,825 21,133 22,441 23,749	(L.C. # 4)	) D AXIAL L JMNS OF T EANER" C ps) ROOF LIVE LOAD LT 288 288 288 288 288 288 288 28	OAD PER HE STORY OLUMNS) SEISMIC WEIGHT DL + P-LL 1,591 3,187 4,783 6,379 7,975 9,571 11,167 12,763 14,359 15,955 17,551 19,147 20,743 22,339 23,935 25,531 27,127 28,723	DL. 1.2 1,815.6 3,385.2 4,954.8 6,524.4 8,094.0 9,663.6 11,233.2 12,802.8 14,372.4 15,942.0 17,511.6 19,081.2 20,650.8 22,220.4 23,790.0 25,359.6 26,929.2 28,498.8	AL FACTORED A LOAD FA L.L. 0 0.0 0.0 0.0 0.0 0.0 0.0	XIAL LOAD, ICTOR RCOF LL. 1.6 460.8	ΣP <sub>u</sub> , PER STOR <i>SEISMIC</i> <i>VERTICAL</i> 0.2S <sub>DS</sub> = 0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0	Y TOTAL ΣP <sub>ui</sub> (kips) 2,276.4 3,846.0 5,415.6 6,985.2 8,554.8 10,124.4 11,694.0 13,263.6 14,833.2 16,402.8 17,972.4 19,542.0 21,111.6 22,681.2 24,250.8 25,820.4 27,390.0 28,959.6	STORY B <sub>21,X-AXI5</sub> 1.018 1.029 1.038 1.037 1.040 1.043 1.043 1.043 1.048 1.052 1.053 1.057 1.062 1.068 1.072 1.078 1.083 1.087 1.091 1.061	

GG Eng	ineeri	JOB NC	). 18 - STO	RY BUII	DINGS			BY	SMG	DATE	4/22/05	Sł	IEET NO.	
		CUSTO	MER DESI	GN 18B				CKD		DATE			OF	
SUBJECT		B2 CALC	ULATION	- FOR	BEND	DING /	ALONG	THE	Y-AXIS	OF THE	COLUM	N <sup>M</sup>	DMENT 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)         TOTAL FACTORED AXIAL LOAD, \$\$P_u\$, PER STORY           I FNGTH         TOTAL STORY         ELASTIC (kips)         LOAD FACTOR													
STORY NUMBER	LENGTH (STORY HEIGHT) L	SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣΗ:	ELASTIC INTERSTORY DRIFT Δ <sub>oh</sub> DUE TO ΣH <sub>i</sub>	DEAD LOAD DL	(kij LIVE LOAD LL	ROOF LIVE LOAD	SEISMIC WEIGHT DL + P-LL	D.L. 1.2	LOAD	FACTOR ROOF L.L. 1.6	SEISMIC VERTICAL 0.2S <sub>DS</sub> = 0	TOTAL ΣP <sub>ui</sub> (kips)	B <sub>2i_Y-AXIS</sub>	
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 851 kips	0.53 in	6,745 8.053	4,032	288	7,975	8,094.0	0.0	460.8	0.0	8,554.8	1.040	
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	
	13.0 ft	1,264 Kips	0.39 m	23,749	17,136	286	28,723	20,498.8		400.8	0.0	28,999.6	1.001	

ີ ທີ	) Sing	ineeri	JOB NO	D. 18 - STO	ry Bui	LDINGS			BY	SMG	DATE	4/22/05	SHEET NO.		
2		JUUUGGUU	CUSTO	DMER DESI	GN 18B	3			СКД		DATE		]_	OF	
B	BJECT			STABIL		DEFFIC	CIENT	ALON	G COLU	MN X-A	XIS, θ <sub>x</sub>		N	OMENT FRAME MF A3 - G3	
	0	LOAD C	COMBINATION:		1.2D + 1.0	6Lr + 0.8W									
	0	DEFLE	CTION AMPLIFIC	CATION FACTOR:	C <sub>d</sub> =	5.5									
	0	(SEISM	IC) IMPORTANC	E FACTOR	I <sub>E</sub> =	1.0									
	0	MOMEN TOTAL	NT FRAME RESIS SEISMIC SHEAF	STS WHAT % OF R TO THE BUIDLIN	THE NG?	25%									
			DUE TO FORC BENDING ALONG MOMENT FR.	ES THAT CAUSE THE X-AXIS OF THE AME COLUMNS	TOTAL L STORY O ("LEANE	<i>Infactore</i> N All Colu R" + "Non-I	ED AXIAL L JMNS OF T LEANER" C	OAD PER HE STORY OLUMNS)	ΤΟΤΑ	L FACTORED	AXIAL LOAD,	$\Sigma P_u$ , PER STO	RY	STABILITY	
		LENGTH	SHEAR DUE TO	INTERSTORY		(ki	ps)			LOAD F	ACTOR		TOTAL	PER STORY	
N	IUMBER	(STORY         ANY         DRIFT         DEAD         LIVE         ROOF         SE           HEIGHT         HORIZONTAL         Δ <sub>oh</sub> LOAD         LOAD         LIVE         LIVE         LIVE         LIVE         LIVE         LIVE         UNAD         SE         UNAD         UNAD							D.L.	L.L.	ROOF L.L.	SEISMIC VERTICAL	ΣP <sub>ui</sub> (kips)	$\boldsymbol{\theta}_i$	
			ΣH <sub>i</sub>	DOE TO ZH	DL	LL	Lr	DL + P-LL	1.2	0	1.6	$0.2S_{DS} = 0$			
	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 π 10.0 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 π 10.0 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	
	14	13.0 ft	533 Kips	0.5 in	5,437	3,024	288	5,379	6,524.4 8.004.0	0.0	460.8	0.0	0,985.2	0.035	
	14	13.0 ft	749 Kips 851 kips	0.53 m	8 053	4,032	200	9,571	0,094.0	0.0	460.8	0.0	0,554.0	0.039	
	12	13.0 ft	038 kips	0.54 m	0,000	6.048	200	9,571 11 167	9,003.0 11 233 2	0.0	460.8	0.0	11 694 0	0.041	
	11	13.0 ft	1 013 kins	0.57 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         SHEE												HEET NO.		
SUBJECT		CUSTO	MER DESI	GN 18B	EFFIC		ALON		MN Y-A					
0	LOAD C DEFLEC	OMBINATION:	ATION FACTOR:	1.2D + 1.6 C <sub>d</sub> =	6Lr + 0.8W 5.5		ALOIN			<b>XIO</b> , U <sub>y</sub>			MF A3 - G3	
0 0	(SEISM MOMEN TOTAL	C) IMPORTANCE IT FRAME RESIS SEISMIC SHEAR	FACTOR TS WHAT % OF 1 TO THE BUIDLIN	IE =	25%									
	DUE TO FORCES THAT CAUSE BENDING ALONG THE Y-AXIS OF THE MOMENT FRAME COLUMNS TOTAL STORY ON ALL COLUMNS OF THE STORY TOTAL STORY ON ALL COLUMNS OF THE STORY ("LEANER" + "NON-LEANER" COLUMNS) ("LEANER" + "NON-LEANER" COLUMNS) (LEANER" + "NON-LEANER" COLUMNS)													
STORY NUMBER	LENGTH (STORY HEIGHT) L	SHEAR DUE TO ANY HORIZONTAL LOAD ΣH <sub>i</sub>	INTERSTORY DRIFT Δ <sub>oh</sub> DUE TO ΣH <sub>i</sub>	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	D.L. 1.2	<i>L.L.</i> 0	ROOF L.L. 1.6	SEISMIC VERTICAL 0.2S <sub>DS</sub> = 0	TOTAL ΣP <sub>ui</sub> (kips)	PER STORY $\theta_i$	
18 17 16 15 14 13 12 11 10 9 8 7 6 5 4 3 2 1	13.0 ft 13.0 ft	185 kips 352 kips 501 kips 633 kips 749 kips 851 kips 938 kips 1,013 kips 1,075 kips 1,126 kips 1,264 kips 1,260 kips 1,263 kips 1,264 kips 1,264 kips	0.23 in 0.4 in 0.53 in 0.53 in 0.54 in 0.57 in 0.59 in 0.57 in 0.59 in 0.59 in 0.61 in 0.62 in 0.62 in 0.61 in 0.62 in 0.63 in	1,513 2,821 4,129 5,437 6,745 8,053 9,361 10,669 11,977 13,285 14,593 15,901 17,209 18,517 19,825 21,133 22,441 23,749	0 1,008 2,016 3,024 4,032 5,040 6,048 7,056 8,064 9,072 10,080 11,088 12,096 13,104 14,112 15,120 16,128 17,136	288 288 288 288 288 288 288 288 288 288	1,591 3,187 4,783 6,379 7,975 9,571 11,167 12,763 14,359 15,955 17,551 19,147 20,743 22,339 23,935 25,531 27,127 28,723	1,815.6 3,385.2 4,954.8 6,524.4 8,094.0 9,663.6 11,233.2 12,802.8 14,372.4 15,942.0 17,511.6 19,081.2 20,650.8 22,220.4 23,790.0 25,359.6 26,929.2 28,498.8	0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0	460.8 460.8 460.8 460.8 460.8 460.8 460.8 460.8 460.8 460.8 460.8 460.8 460.8 460.8 460.8 460.8 460.8	0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0	2,276.4 3,846.0 5,415.6 6,985.2 8,554.8 10,124.4 11,694.0 13,263.6 14,833.2 16,402.8 17,972.4 19,542.0 21,111.6 22,681.2 24,250.8 25,820.4 27,390.0 28,959.6	0.018 0.028 0.037 0.035 0.039 0.041 0.046 0.050 0.050 0.050 0.054 0.058 0.064 0.067 0.073 0.077 0.080 0.083 0.057	

(	ic Eng	jineeri	JOB NC	). 18 - STO MER DESI	RY BUII GN 18B	DINGS			BY CKD	SMG	DATE DATE	4/22/05	SH	SHEET NO.	
s	JBJECT		B2 CALC	ULATION	- FOR	BEND	DING A	ALONG	THE X	-AXIS O	F THE	COLUM	V <sup>MC</sup>	MENT FRAME MF A3 - G3	
	0	LOAD	DUE TO FORCE	1.2D + 0.5L + 0.5	5 <mark>Lr + 1.6W</mark> TOTAL <i>U</i>	(L.C. # 5)	) D AXIAL L	OAD PER	ΤΟΤΑ	L FACTORED A	XIAL LOAD.	ΣP., PER STOP	Y		
			MOMENT FRA	ELASTIC	STORY ON ("LEANER	N ALL COLU R" + "NON-L (kij	MNS OF T EANER" C ps)	HE STORY OLUMNS)		LOAD FA		21 ,, 1 21 0101		STORY	
	STORY NUMBER	(STORY HEIGHT) L	SEISMIC HORIZONTAL LOAD E ΣΗ <sub>i</sub>	INTERSTORY DRIFT Δ <sub>oh</sub> DUE TO ΣH <sub>i</sub>	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 <sub>DS</sub> = 0	TOTAL ΣP <sub>ui</sub> (kips)	B <sub>2i_X-AXIS</sub>	
	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 7.075	6,524.4	1,512.0	144.0	0.0	8,180.4	1.043	
	14	13.0 ft	851 kips	0.53 m	6,745 8.053	4,032	200	9.571	0,094.0 9.663.6	2,016.0	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	
	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	

ශිශි පිතය	പ്പത്തി	JOB N តាតា	0. 18 - STC	RY BUI	LDINGS			BY	SMG	DATE	4/22/05	SH	IEET NO.	
99 ENG	Inngan	CUST	OMER DESI	GN 18B	}			CKD		DATE		T	OF	
SUBJECT		B2 CALC	ULATION	- FOR	R BENL	DING /	ALONG	THE	Y-AXIS (	OF THE	COLUM	N M	DMENT 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)         TOTAL FACTORED AXIAL LOAD, ΣP <sub>u</sub> , PER STORY           TOTAL STORY         TOTAL STORY         ("LEANER" + "NON-LEANER" COLUMNS)         LOAD FACTOR													
STORY NUMBER	LENGTH <i>(STORY</i> <i>HEIGHT)</i> L	SHEAR DUE T SEISMIC HORIZONTAL LOAD E ΣΗ:	C ELASTIC INTERSTORY DRIFT Δ <sub>oh</sub> DUE TO ΣH <sub>i</sub>	DEAD LOAD DL	(ki LIVE LOAD LL	ROOF LIVE LOAD	SEISMIC WEIGHT DL + P-LL	D.L. 1.2	LOAD F	ACTOR ROOF L.L. 0.5	SEISMIC VERTICAL 0.2S <sub>DS</sub> = 0	TOTAL ΣP <sub>ui</sub> (kips)	STORY B <sub>2i_Y-AXIS</sub>	
18 17	13.0 ft 13.0 ft	185 kips 352 kips	0.23 in 0.4 in	1,513 2,821	0 1,008	288 288	1,591 3,187	1,815.6 3,385.2	0.0 504.0	144.0 144.0	0.0	1,959.6 4,033.2	1.016 1.030	
16 15 14 13 12	13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft	501 kips 633 kips 749 kips 851 kips 938 kips 1.013 kips	0.53 in 0.5 in 0.53 in 0.54 in 0.57 in	4,129 5,437 6,745 8,053 9,361	2,016 3,024 4,032 5,040 6,048 7,056	288 288 288 288 288 288	4,783 6,379 7,975 9,571 11,167 12,763	4,954.8 6,524.4 8,094.0 9,663.6 11,233.2	1,008.0 1,512.0 2,016.0 2,520.0 3,024.0 3,528.0	144.0 144.0 144.0 144.0 144.0 144.0	0.0 0.0 0.0 0.0 0.0	6,106.8 8,180.4 10,254.0 12,327.6 14,401.2	1.043 1.043 1.049 1.053 1.059	
10 9 8 7 6	13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft	1,075 kips 1,126 kips 1,168 kips 1,200 kips 1,224 kips	0.57 in 0.58 in 0.59 in 0.61 in 0.61 in	11,977 13,285 14,593 15,901 17,209	8,064 9,072 10,080 11,088 12,096	288 288 288 288 288 288	14,359 15,955 17,551 19,147 20,743	14,372.4 15,942.0 17,511.6 19,081.2 20,650.8	4,032.0 4,536.0 5,040.0 5,544.0 6,048.0	144.0 144.0 144.0 144.0 144.0	0.0 0.0 0.0 0.0 0.0 0.0	18,548.4 20,622.0 22,695.6 24,769.2 26,842.8	1.067 1.073 1.079 1.088 1.094	
5 4 3 2 1	13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft	1,241 kips 1,253 kips 1,260 kips 1,263 kips 1,264 kips	0.62 in 0.62 in 0.61 in 0.6 in 0.39 in	18,517 19,825 21,133 22,441 23,749	13,104 14,112 15,120 16,128 17,136	288 288 288 288 288 288	22,339 23,935 25,531 27,127 28,723	22,220.4 23,790.0 25,359.6 26,929.2 28,498.8	6,552.0 7,056.0 7,560.0 8,064.0 8,568.0	144.0 144.0 144.0 144.0 144.0	0.0 0.0 0.0 0.0 0.0	28,916.4 30,990.0 33,063.6 35,137.2 37,210.8	1.102 1.109 1.114 1.120 1.079	
GG Eng	ineeri	JOB NC	). 18 - STO	RY BUII	LDINGS			BY	SMG	DATE	4/22/05	s	HEET NO.	
-----------------	--	--	--	---	--	-------------------------------------	--------------------------------	----------	-------------	---------------------	---	-------------------------------------	-----------------------------	
66 9		CUSTO	MER DESI	GN 18B				СКД		DATE			OF	
SUBJECT			STABIL	түсс	DEFFIC	CIENT	ALON	g Colu	MN X-A	XIS, θ <sub>x</sub>		М	OMENT FRAME MF A3 - G3	
0 0 0	LOAD C DEFLEC (SEISM MOMEN TOTAL	COMBINATION: CTION AMPLIFIC, IC) IMPORTANCE IT FRAME RESIS SEISMIC SHEAR	1. ATION FACTOR: E FACTOR TTS WHAT % OF T TO THE BUIDLIN	2D + 0.5L + C <sub>d</sub> = I <sub>E</sub> = THE IG?	- 0.5Lr + 1.6 5.5 1.0 25%	w								
		DUE TO FORCE BENDING ALONG 1 MOMENT FRA	ES THAT CAUSE THE <mark>X-AXIS</mark> OF THE ME COLUMNS	TOTAL U STORY OI	INFACTORE N ALL COLL B" + "NON-I	ED AXIAL L JMNS OF T EANER" C	OAD PER HE STORY OLUMNS)	ΤΟΤΑ	L FACTORED	AXIAL LOAD,	$\Sigma P_u$ , PER STO	RY	STABILITY	
		TOTAL STORY	ELASTIC		(ki	ps)	3201VII VO)		LOAD F	ACTOR			COEFFICIENT	
STORY NUMBER	(STORY HEIGHT) L	ANY HORIZONTAL LOAD ΣH <sub>i</sub>	INTERSTORY DRIFT $\Delta_{oh}$ DUE TO $\Sigma$ H <sub>i</sub>	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	D.L.	L.L. 0.5	ROOF L.L.	SEISMIC VERTICAL 0.2S <sub>DS</sub> = 0	TOTAL ΣP <sub>ui</sub> (kips)	PER STORY θ <sub>i</sub>	
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 kins	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	
10	12.0 ft	029 kips	0.54 in	0,000	6.049	200	11 167	11 000.0	2,024.0	144.0	0.0	14 401 2	0.056	
12	13.0 1	938 kips	0.57 m	9,301	0,040	200	10,700	11,233.2	3,024.0	144.0	0.0	14,401.2	0.056	
11	13.0 π	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	

_ @	G Eng	ineeri	JOB NC	). 18 - STO MER DESI	RY BUII GN 18B	LDINGS			BY CKD	SMG	DATE DATE	4/22/05	S	HEET NO.
SI	JBJECT			STABIL	түсс	DEFFIC	IENT	ALON	G COLU	IMN Y-A	XIS, θ <sub>y</sub>		м	OMENT FRAME MF A3 - G3
	0 0 0	LOAD C DEFLEC (SEISM MOMEN TOTAL	COMBINATION: CTION AMPLIFIC, IC) IMPORTANCE IT FRAME RESIS SEISMIC SHEAR	1 ATION FACTOR: E FACTOR TS WHAT % OF TO THE BUIDLIN	2D + 0.5L + C <sub>d</sub> = I <sub>E</sub> = THE IG?	0.5Lr + 1.6 5.5 1.0 25%	w							
			DUE TO FORCE BENDING ALONG T	ES THAT CAUSE	TOTAL U		ED AXIAL LO	OAD PER	TOTA	L FACTORED	AXIAL LOAD,	$\Sigma P_u$ , PER STO	RY	
			TOTAL STORY	ELASTIC	("LEANE!	R" + "NON-L (ki	EANER" C	OLUMNS)			ACTOR			STABILITY COEFFICIENT
	STORY NUMBER	(STORY HEIGHT) L	ANY HORIZONTAL LOAD		DEAD LOAD	LIVE	ROOF LIVE LOAD	SEISMIC WEIGHT	D.L.	L.L.	ROOF L.L.	SEISMIC VERTICAL	TOTAL ΣP <sub>ui</sub> (kips)	PER STORY θ <sub>i</sub>
		10.0 %	∑H <sub>i</sub>	DUE TO SH	DL	LL	Lr	DL + P-LL	1.2	0.5	0.5	0.2S <sub>DS</sub> = 0	1 050 0	0.010
18         13.0 π         185 кips         0.23 in         1,513         0         288         1,591         1,815.6         0.0         144.0           17         13.0 ft         352 kips         0.4 in         2,821         1,008         288         3,187         3,385.2         504.0         144.0           16         13.0 ft         501 kips         0.53 in         4,129         2,016         288         4,783         4,954.8         1,008.0         144.0													1,959.6	0.016
	17         13.0 ft         501 kips         0.4 in         2,821         1,008         288         3,187         3,385.2         504.0         144.0         0.0           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           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													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           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           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													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           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           13         13.0 ft         851 kips         0.54 in         8,053         5,040         288         9,571         9,652.6         2,520.0         144.0         0.0													10,254.0	0.047
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           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         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
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         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
	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           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           10         140.4         0.59 in         10,669         7,056         288         12,763         12,802.8         3,528.0         144.0         0.0         16													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 π 13.0 ft	1,168 kips	0.59 in	14,593	11,080	288	17,551	19.081.2	5,040.0	144.0	0.0	22,695.6	0.073
	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 π	1,264 Kips	0.39 in	23,749	17,136	288	28,723	28,498.8	8,558.0	144.0	0.0	37,210.8	0.074

(	ig Eng	jineeri	JOB NC DIG CUSTO	). 18 - STO MER DESI	RY BUII GN 18B	DINGS			BY CKD	SMG	DATE DATE	4/22/05	S⊦	
S	UBJECT		B2 CALC	ULATION	- FOR	BEND	DING A	ALONG	THE X	-AXIS O	F THE	COLUMI	V <sup>MC</sup>	MENT FRAME MF A3 - G3
	o STORY NUMBER 18	LOAD ( STORY HEIGHT) L 13.0 ft	COMBINATION = DUE TO FORCE BENDING ALONG T MOMENT FRA TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH <sub>i</sub> 185 kips	1.2D + 0.5L + 1.0 S THAT CAUSE THE X-AXIS OF THE ME COLUMNS ELASTIC INTERSTORY DRIFT $\Delta_{oh}$ DUE TO $\Sigma H_i$ 0.23 in	TOTAL U STORY OF ("LEANER LOAD DL 1,513	(L.C. #6) NFACTORE NALL COLU a" + "NON-L (kig LIVE LOAD LL 0	) MNS OF T EANER" C 205) ROOF LIVE LOAD Lr 288	OAD PER HE STORY OLUMNS) SEISMIC WEIGHT DL + P-LL 1,591	D.L. 1.2 1,815.6	L FACTORED A LOAD FA L.L. 0.5 0.0	AXIAL LOAD, ACTOR ROOF L.L. 0 0.0	ΣP <sub>u</sub> , PER STOR SEISMIC VERTICAL 0.2S <sub>DS</sub> = 0.2 318.2	Y TOTAL ΣP <sub>ui</sub> (kips) 2,133.8	STORY B <sub>2LX-AXIS</sub> 1.017
	17 16 15 14 13 12 11 10 9 8 7 6 5 4 3 2 1	13.0 ft 13.0 ft	352 kips 501 kips 633 kips 749 kips 851 kips 938 kips 1,013 kips 1,075 kips 1,126 kips 1,200 kips 1,224 kips 1,253 kips 1,260 kips 1,264 kips 1,264 kips	0.4 in 0.53 in 0.51 in 0.54 in 0.57 in 0.59 in 0.59 in 0.59 in 0.61 in 0.62 in 0.61 in 0.62 in 0.61 in 0.63 in	2,821 4,129 5,437 6,745 8,053 9,361 10,669 11,977 13,285 14,593 15,901 17,209 18,517 19,825 21,133 22,441 23,749	1,008 2,016 3,024 4,032 5,040 6,048 7,056 8,064 9,072 10,080 11,088 12,096 13,104 14,112 15,120 16,128 17,136	288 288 288 288 288 288 288 288 288 288	3,187 4,783 6,379 7,975 9,571 11,167 12,763 14,359 15,955 17,551 19,147 20,743 22,339 23,935 25,531 27,127 28,723	3,385.2 4,954.8 6,524.4 8,094.0 9,663.6 11,233.2 12,802.8 14,372.4 15,942.0 17,511.6 19,081.2 20,650.8 22,220.4 23,790.0 25,359.6 26,929.2 28,498.8	504.0 1,008.0 1,512.0 2,016.0 2,520.0 3,024.0 3,528.0 4,032.0 4,536.0 5,040.0 5,544.0 6,048.0 6,552.0 7,056.0 7,566.0 8,064.0 8,568.0	0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0	637.4 956.6 1,275.8 1,595.0 1,914.2 2,233.4 2,552.6 2,871.8 3,191.0 3,510.2 3,829.4 4,148.6 4,467.8 4,787.0 5,106.2 5,425.4 5,744.6	4,526.6 6,919.4 9,312.2 11,705.0 14,097.8 16,490.6 18,883.4 21,276.2 23,669.0 26,061.8 28,454.6 30,847.4 33,240.2 35,633.0 38,025.8 40,418.6 42,811.4	1.034 1.049 1.049 1.056 1.061 1.069 1.076 1.078 1.085 1.092 1.102 1.109 1.119 1.127 1.134 1.140 1.093

										1			
ලිලි පිතුන	linaari	JOB NC	). 18 - STO	RY BUI	DINGS			BY	SMG	DATE	4/22/05	SH	IEET NO.
ලල පැලී	nnaau	СИЗТО	MER DESI	GN 18B				СКД		DATE		1_	OF
SUBJECT		B2 CALC	ULATION	- FOR	BEND	DING A	ALONG	THE )	-AXIS C	OF THE	COLUM	N <sup>MC</sup>	)MENT FRAME MF A3 - G3
0	LOAD	COMBINATION =	1.2D + 0.5L + 1.0	Ε									
		DUE TO FORCE BENDING ALONG T MOMENT FRA	es that cause The <mark>Y-axis</mark> of the Me columns	TOTAL U STORY OI		D AXIAL L	OAD PER HE STORY	тот/	AL FACTORED /	AXIAL LOAD,	ΣP <sub>u</sub> , PER STO	٦Y	
	LENGTH	TOTAL STORY SHEAR DUE TO	ELASTIC	("LEANE	K" + "NON-L (kij	EANER" C os)	OLUMINS)		LOAD F.	ACTOR			STORY
STORY NUMBER	(STORY HEIGHT) L	SEISMIC HORIZONTAL LOAD E	INTERSTORY DRIFT Δ <sub>oh</sub> DUE TO ΣH <sub>i</sub>	DEAD LOAD DL	LIVE LOAD LL	ROOF LIVE LOAD	SEISMIC WEIGHT DL + P-LL	D.L. 1.2	L.L. 0.5	ROOF L.L.	SEISMIC VERTICAL 0.2S <sub>DS</sub> = 0.2	TOTAL ΣP <sub>ui</sub> (kips)	B <sub>2i_Y-AXIS</sub>
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	0.0 IN	22,441	17,126	288	27,127	26,929.2	8,064.0	0.0	5,425.4	40,418.6	1.140
		,		20,110		200	20,720	20,100.0	0,000.0		6,1110	,	

LOAD COMI DEFLECTIO (SEISMIC) II MOMENT FI TOTAL SEIS TOTAL SEIS TOTAL SHI STORY L	JOB NO CUSTOI CUSTOI BINATION: DN AMPLIFICA MPORTANCE RAME RESIST SMIC SHEAR DUE TO FORCE: VDING ALONG TH MOMENT FRAM	NER DESI MER DESI STABILI STABILI ITION FACTOR: FACTOR IS WHAT % OF THE S THAT CAUSE #E X-AXIS OF THE	RY BUII GN 188 ITY CC 1.2D + 0. C <sub>d</sub> = I <sub>E</sub> = I <sub>E</sub> =	DINGS DEFFIC 5.5 1.0 25%	XENT	ALONG	BY CKD G COLU	SMG	DATE DATE XIS, θ <sub>x</sub>	4/22/05	S	OF OF OMENT FRAME MF A3 - G3
LOAD COMI DEFLECTIO (SEISMIC) II MOMENT FI TOTAL SEIS TORY L	CUSTOI BINATION: DN AMPLIFICA MPORTANCE RAME RESIST SMIC SHEAR DUE TO FORCES VDING ALONG TH MOMENT FRAM	MER DESI STABILI ITION FACTOR: FACTOR IS WHAT % OF T TO THE BUIDLIN S THAT CAUSE #E X-AXIS OF THE	GN 18B ITY CC 1.2D + 0. C <sub>d</sub> = I <sub>E</sub> =	5L + 1.0E 5.5 1.0	JENT	ALONG	скр G COLU	MN X-A	date XIS, θ <sub>x</sub>		 	OF OMENT FRAME MF A3 - G3
LOAD COMI DEFLECTIO (SEISMIC) II MOMENT FI TOTAL SEIS TOTAL SEIS TORY L L	BINATION: IN AMPLIFICA MPORTANCE RAME RESIST SMIC SHEAR DUE TO FORCES VDING ALONG TH MOMENT FRAM	STABILI ITION FACTOR: FACTOR IS WHAT % OF T TO THE BUIDLIN	1.2D + 0. C <sub>d</sub> = I <sub>E</sub> = THE 2	5L + 1.0E 5.5 1.0	CIENT	ALON	G COLU	MN X-A	XIS, θ <sub>x</sub>		м 	OMENT FRAME MF A3 - G3
LOAD COMI DEFLECTIO (SEISMIC) II MOMENT FI TOTAL SEIS TOTAL SEIS TO TOTAL SEIS TO TOTAL SEIS TO TOTAL SEIGHT L	BINATION: DN AMPLIFICA MPORTANCE RAME RESIST SMIC SHEAR DUE TO FORCES VDING ALONG TH MOMENT FRAM MOMENT FRAM	TION FACTOR: FACTOR IS WHAT % OF T TO THE BUIDLIN S THAT CAUSE IE X-AXIS OF THE	1.2D + 0. C <sub>d</sub> = I <sub>E</sub> = THE IG?	5L + 1.0E 5.5 1.0								
DEFLECTIO (SEISMIC) II MOMENT FI TOTAL SEIS BEN TOTAL SEIS TOTAL SHI STORY L	DN AMPLIFICA MPORTANCE RAME RESIST SMIC SHEAR DUE TO FORCES VDING ALONG TH MOMENT FRAM	TION FACTOR: FACTOR IS WHAT % OF 1 TO THE BUIDLIN S THAT CAUSE IE X-AXIS OF THE	C <sub>d</sub> = I <sub>E</sub> = IG?	5.5 1.0								
(SEISMIC) II MOMENT FI TOTAL SEIS ENGTH STORY EIGHT) L	IN AMPLIFICA MPORTANCE RAME RESISI SMIC SHEAR DUE TO FORCE: UDING ALONG TH MOMENT FRAN TAL STORY I	FACTOR FACTOR TS WHAT % OF 1 TO THE BUIDLIN S THAT CAUSE HE X-AXIS OF THE	ι <sub>E</sub> = ΓΗΕ ΙG?	1.0								
(SEISMIC) II MOMENT FI TOTAL SEIS BER TO ENGTH SHE STORY L H( L	MPORTANCE RAME RESIST SMIC SHEAR DUE TO FORCES NDING ALONG TH MOMENT FRAM	FACTOR TS WHAT % OF 1 TO THE BUIDLIN S THAT CAUSE HE X-AXIS OF THE	ι <sub>E</sub> = ΓΗΕ 2 ΙG?	1.0								
MOMENT FI TOTAL SEIS BEP TO ENGTH STORY L L H C	DUE TO FORCES NDING ALONG TH MOMENT FRAN	TS WHAT % OF T TO THE BUIDLIN S THAT CAUSE HE X-AXIS OF THE	THE IG?	25%								
BEN TO STORY L BEIGHT) HC	DUE TO FORCE NDING ALONG TH MOMENT FRAN	S THAT CAUSE		/0								
TO ENGTH SHE STORY EIGHT) HO	MOMENT FRAM		TOTAL U	INFACTORE	D AXIAL L	OAD PER	τοτα				BV	
TO ENGTH SHE STORY EIGHT) HO L	TAL STORY	VE COLUMNS	STORY OF	N ALL COLU B" + "NON-I	JMNS OF T	HE STORY						STABILITY
STORY EIGHT) HO L	EAR DUE TO	ELASTIC	( 22, 112	(kir	ps)	0201110)		LOAD F	ACTOR		TOTAL	COEFFICIENT
L	ANY ORIZONTAL	DRIFT	DEAD	LIVE	ROOF	SEISMIC	D.L.	L.L.	ROOF L.L.	SEISMIC	ΣP <sub>ui</sub>	θ <sub>i</sub>
	LOAD	Δ <sub>oh</sub> DUE TO ΣΗ	LOAD	LOAD	LIVE		10	0.5	0	VERTICAL	(kips)	
0.0.4	ΣH <sub>i</sub>	0.00	1.540		Lr	DE +1-LL	1.2	0.5	0	0.23 <sub>DS</sub> = 0.2	0.400.0	0.017
3.0 π	185 Kips	0.23 in	1,513	1.000	288	1,591	1,815.6	0.0	0.0	318.2	2,133.8	0.017
3.0 π	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
3.0 ft	SUT 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
3.0 ft	749 kips	0.5 m	5,437	3,024	200	7 975	0,524.4 8.094.0	2,016,0	0.0	1,275.0	9,312.2	0.047
3.0 ft	851 kips	0.55 in	8 053	5 040	288	9.571	9 663 6	2,520.0	0.0	1,000.0	14 097 8	0.057
3.0 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
3.0 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
3.0 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
3.0 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
3.0 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
3.0 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
3.0 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
3.0 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
3.0 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
3.0 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
3.0 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
3.0 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
				ſ								
				ſ								
				ſ								
				ľ								
				ſ								
				ľ								
				ľ								
				ľ								
				ľ								
				ľ								
13. 13. 13. 13. 13. 13. 13. 13. 13. 13.	oft oft oft oft oft oft oft oft oft oft	0 ft 749 kips 0 ft 749 kips 0 ft 851 kips 0 ft 938 kips 0 ft 1,013 kips 0 ft 1,075 kips 0 ft 1,126 kips 0 ft 1,168 kips 0 ft 1,200 kips 0 ft 1,224 kips 0 ft 1,241 kips 0 ft 1,263 kips 0 ft 1,263 kips 0 ft 1,264 kips	0 ft         749 kips         0.53 in           0 ft         749 kips         0.53 in           0 ft         851 kips         0.54 in           0 ft         938 kips         0.57 in           0 ft         1,013 kips         0.59 in           0 ft         1,013 kips         0.59 in           0 ft         1,013 kips         0.59 in           0 ft         1,264 kips         0.58 in           0 ft         1,126 kips         0.59 in           0 ft         1,200 kips         0.61 in           0 ft         1,200 kips         0.61 in           0 ft         1,224 kips         0.62 in           0 ft         1,263 kips         0.62 in           0 ft         1,263 kips         0.61 in           0 ft         1,263 kips         0.61 in           0 ft         1,264 kips         0.39 in	0 ft         749 kips         0.53 in         5,457           0 ft         749 kips         0.53 in         6,745           0 ft         851 kips         0.54 in         8,053           0 ft         938 kips         0.57 in         9,361           0 ft         1,013 kips         0.59 in         10,669           0 ft         1,075 kips         0.57 in         11,977           0 ft         1,075 kips         0.57 in         13,285           0 ft         1,126 kips         0.58 in         13,285           0 ft         1,126 kips         0.59 in         14,593           0 ft         1,200 kips         0.61 in         15,901           0 ft         1,224 kips         0.61 in         17,209           0 ft         1,241 kips         0.62 in         18,517           0 ft         1,263 kips         0.61 in         21,133           0 ft         1,263 kips         0.61 in         23,749	0 ft         749 kips         0.53 in         6,745         4,032           0 ft         749 kips         0.53 in         6,745         4,032           0 ft         938 kips         0.57 in         9,361         6,048           0 ft         1,013 kips         0.59 in         10,669         7,056           0 ft         1,075 kips         0.57 in         11,977         8,064           0 ft         1,075 kips         0.57 in         11,977         8,064           0 ft         1,126 kips         0.58 in         13,285         9,072           0 ft         1,126 kips         0.59 in         14,593         10,080           0 ft         1,200 kips         0.61 in         15,901         11,088           0 ft         1,224 kips         0.61 in         17,209         12,096           0 ft         1,241 kips         0.62 in         18,517         13,104           0 ft         1,260 kips         0.61 in         21,133         15,120           0 ft         1,263 kips         0.66 in         22,441         16,128           0 ft         1,264 kips         0.39 in         23,749         17,136	0.11       0.05 kips       0.5 kips       0.5 kin       5,407       5,624       205         0 ft       749 kips       0.53 in       6,745       4,032       288         0 ft       938 kips       0.57 in       9,361       6,048       288         0 ft       1,013 kips       0.59 in       10,669       7,056       288         0 ft       1,075 kips       0.57 in       11,977       8,064       288         0 ft       1,126 kips       0.58 in       13,285       9,072       288         0 ft       1,168 kips       0.59 in       14,593       10,080       288         0 ft       1,200 kips       0.61 in       15,901       11,088       288         0 ft       1,224 kips       0.61 in       17,209       12,096       288         0 ft       1,241 kips       0.62 in       19,825       14,112       288         0 ft       1,260 kips       0.61 in       21,133       15,120       288         0 ft       1,263 kips       0.61 in       22,749       17,136       288         0 ft       1,264 kips       0.39 in       23,749       17,136       288	0.11         0.03 kips         0.51 kip         0.61 kips         0.61 kips         0.63 kips         0.63 kips         0.63 kips         0.63 kips         0.63 kips         0.63 kips         0.65 kips         0.64 kips         0.63 kips         0.67 kips         0.61 kips         0.62 kips         10.080         288         17.55 kips         0.62 kips         10.080         288         19.147           0 ft         1.200 kips         0.61 kip         17.209         12.096         288         20.743         20 kips         2.61 kips         2.733         2.733           0 ft         1.226 kips         0.62 kip         18.517         13.104         288         22.339         2.61 kip         2.83 kip         2.5331         2.61 kip         2.3749	0.11         0.03 hip         0.3 hi         0.407         0.407         2.00         0.013         0.424.4           0 ft         749 kips         0.53 in         6.745         4.032         288         7,975         8,094.0           0 ft         851 kips         0.54 in         8,053         5,040         288         9,571         9,663.6           0 ft         938 kips         0.57 in         9,361         6,048         288         11,167         11,233.2           0 ft         1,013 kips         0.59 in         10,669         7,056         288         12,763         12,802.8           0 ft         1,075 kips         0.57 in         11,977         8,064         288         14,359         14,372.4           0 ft         1,126 kips         0.58 in         13,285         9,072         288         15,955         15,942.0           0 ft         1,200 kips         0.61 in         15,901         11,088         288         19,147         19,081.2           0 ft         1,224 kips         0.61 in         17,209         12,096         288         20,743         20,650.8           0 ft         1,264 kips         0.61 in         21,133         15,120         288	Off         Osfin         O	0.1       0.5.1 m       0.457       0.407       0.407       1.512.0       0.53         0 tt       749 kips       0.53 in       6,745       4,032       288       7,975       8,094.0       2,016.0       0.0         0 tt       851 kips       0.57 in       9,361       6,048       288       9,571       9,663.6       2,520.0       0.0         0 tt       1,013 kips       0.57 in       10,669       7,056       288       12,763       12,802.8       3,528.0       0.0         0 tt       1,107 kips       0.57 in       11,977       8,064       288       14,359       14,372.4       4,032.0       0.0         0 tt       1,126 kips       0.58 in       13,285       9,072       288       15,955       15,942.0       4,536.0       0.0         0 tt       1,168 kips       0.59 in       14,593       10,080       288       17,551       17,511.6       5,040.0       0.0         0 tt       1,200 kips       0.61 in       17,209       12,096       288       20,743       20,650.8       6,048.0       0.0         0 tt       1,224 kips       0.61 in       21,133       15,120       288       23,935       23,790.0       7,056.0	Ort         Octomps         Octom         Octom <th< td=""><td>0.1         0.5.1         0.5.1         0.7.2         10.7.5         0.7.7         0.7.6         0.7.6         0.7.7.7         0.7.7         0.7.7         <t< td=""></t<></td></th<>	0.1         0.5.1         0.5.1         0.7.2         10.7.5         0.7.7         0.7.6         0.7.6         0.7.7.7         0.7.7         0.7.7 <t< td=""></t<>

								-					
හැළ සිත	ineeri	JOB NO	Э. 18 - STO	RY BUI	LDINGS	;		BY	SMG	DATE	4/22/05	SI	HEET NO.
ee Bug	JUUUGGUU	CUSTO	MER DESI	GN 18B	3			СКД		DATE		]	OF
SUBJECT			STABIL	ІТҮ СС	DEFFIC	CIENT	ALON	G COLU	IMN Y-A	XIS, θ <sub>y</sub>		M	OMENT FRAME MF A3 - G3
0 0 0	LOAD C DEFLEC (SEISM MOMEN TOTAL	COMBINATION: CTION AMPLIFIC IC) IMPORTANC IT FRAME RESIS SEISMIC SHEAF	ATION FACTOR: E FACTOR STS WHAT % OF R TO THE BUIDLII	1.2D + 0 C <sub>d</sub> = I <sub>E</sub> = <b>THE</b> <b>VG?</b>	.5L + 1.0E 5.5 1.0 25%								
		DUE TO FORC BENDING ALONG MOMENT FR	ES THAT CAUSE THE Y-AXIS OF THE AME COLUMNS	TOTAL L STORY O ("LEANE	<i>INFACTORE</i> N ALL COLU R" + "NON-I	ED AXIAL L JMNS OF T LEANER" C	OAD PER HE STORY OLUMNS)	ΤΟΤΑ	L FACTORED	AXIAL LOAD,	$\Sigma P_u$ , PER STO	RY	STABILITY
STORY NUMBER	LENGTH (STORY HEIGHT) L	SHEAR DUE TO ANY HORIZONTAL LOAD ΣH <sub>i</sub>	) ELASTIC INTERSTORY DRIFT Δ <sub>oh</sub> DUE TO ΣH <sub>i</sub>	DEAD LOAD DL	(ki LIVE LOAD LL	ps) ROOF LIVE LOAD Lr	SEISMIC WEIGHT DL + P-LL	D.L. 1.2	LOAD F L.L. 0.5	ROOF L.L.	SEISMIC VERTICAL 0.2S <sub>DS</sub> = 0.2	TOTAL ΣP <sub>ui</sub> (kips)	COEFFICIENT PER STORY θ <sub>i</sub>
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 π 10.0 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	0.0 in	22,441	17 126	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
												l	
												l	
												l	
												l	
												l	
												1	
												I	

Construction         Customer         Design 188         CKD         Date         OF           SUBJECT         STABILITY COEFFICIENT ALONG COLUMN Y-AXIS, 6y         Moment Frame           SUBJECT         STABILITY COEFFICIENT ALONG COLUMN Y-AXIS, 6y         Moment Frame           o         LOAD COMBINATION:         120+0.5L+1.0E           o         Deflection Amplification Factors:         C_ = 5.5           TOTAL STORY         Strain Provide Strain Coulds         Open and the VASG of the Total STORY (FF Batter)           STORY         LENGTH         Strain Provide Strain Coulds         Open and the VASG of the Total STORY (FF Batter)           NUMBER         NUMBER         DUE TO TOTAL STORY         Strain Provide Strain Provi	ශිශි පිතක්කලෙන්තන	JOB N	NO. 18 ·	- STORY BL	JILDINGS		BY	SMG	DATE	4/22/05	SHEET NO.
SUBJECT         STABILITY COEFFICIENT ALONG COLUMN Y-AXIS, $\theta_y$ MOMENT PRAME MF A3- 63           0         LOAD COMENATION:         120 + 0.5L + 1.0E           O CONTROLOGIES INFORMED           ODE TO FIGURES INFORMED           STORY (LENOT)         MAXADE TO FIGURES INFORMED           STORY (LENOT)         TO FIGURES INFORMED           STORY (LENOT)         STORY (LENOT)         TOTAL STORY (CALL OF SHEAR)         STORY (COLSPAN)         COLSPAN (COLSPAN)           STORY (LENOT)         STORY (COLSPAN)         STORY (COLSPAN)         COLSPAN (COLSPAN)           STORY (COLSPAN)         STORY (COLSPAN)         STORY (COLSPAN)         COLSPAN (COLSPAN)           STORY (COLSPAN)         STORY (COLSPAN)         STORY (COLSPAN)         COLSPAN (COLSPAN)           18         TORY (COLSPAN)         STORY (COLSPAN)         COLSPAN (COLSPAN)           INTERSTORY (COLSPAN)         COLSPAN (COLSPAN)           10         STORY (COLSPAN)         COLSPAN (COLSPAN)	ee Englinneer und	CUST	OMER	DESIGN 18	B		CKD		DATE		OF
•       LOAD COMBINATION: $1.20 + 0.5L + 1.0E$ •       DEFLECTION AMPLIFICATION FACTOR: $C_q = 5.5$ •       Dist to for the concest that CAUSE: MOMENT FRAME COLUMNS       TOTAL STORY STREAT       NAMENA AND CONSTRAINED       MANDAL STREAT       MANDAL STREAT       STREAT         STORY       LENGTH       SHEAR       DUE TO FORCES THAT CAUSE: MOMENT FRAME COLUMNS       TOTAL STORY UNITERSTORY       TOTAL STORY (STORY)       STREAT       STREAT       STREAT       STREAT       STREAT       STREAT       COMMENT         STORY       LENGTH       SHEAR       DUE TO       INTERSTORY DUE TO 1N       STREAT       STREAT </th <th>SUBJECT</th> <th></th> <th>ST</th> <th>ABILITY C</th> <th>OEFFICI</th> <th>ENT ALOI</th> <th></th> <th>JMN Y-A</th> <th>XIS, θ<sub>y</sub></th> <th></th> <th>MOMENT FRAME MF A3 - G3</th>	SUBJECT		ST	ABILITY C	OEFFICI	ENT ALOI		JMN Y-A	XIS, θ <sub>y</sub>		MOMENT FRAME MF A3 - G3
	SUBJECT • LOAD COMB • DEFLECTION	INATION: AMPLIE STORY IUMBER 18 17 16 15 14 13 12 11 10 9 8 7 6 5 4 3 2 1	ST/ ICATION F/ ICATION	ABILITY C           1.2D +           ACTOR:         C           DUE TO FORCE           BENDING ALONG T           MOMENT FRA           TOTAL STORY           SHEAR DUE TO           SHEAR DUE TO           OBEISMIC           HORIZONTAL           LOAD E $\Sigma H_i$ 185 kips           352 kips           501 kips           633 kips           749 kips           851 kips           938 kips           1,013 kips           1,075 kips           1,126 kips           1,200 kips           1,224 kips           1,263 kips           1,263 kips           1,264 kips	$\begin{array}{l} \textbf{OEFFICI}\\ \textbf{0.5L} + 1.0E\\ = 5.5\\ \textbf{S} THAT CAUSE\\ \textbf{HE Y-AXIS OF THE}\\ \textbf{WE COLUMNS}\\ \textbf{INTERSTORY}\\ \textbf{DRIFT}\\ \Delta_{oh}\\ \textbf{DUE TO } \Sigma \textbf{H}_i\\ \textbf{0.23 in}\\ \textbf{0.4 in}\\ \textbf{0.53 in}\\ \textbf{0.51 in}\\ \textbf{0.53 in}\\ \textbf{0.57 in}\\ \textbf{0.59 in}\\ \textbf{0.57 in}\\ \textbf{0.59 in}\\ \textbf{0.57 in}\\ \textbf{0.59 in}\\ \textbf{0.51 in}\\ \textbf{0.59 in}\\ \textbf{0.61 in}\\ \textbf{0.61 in}\\ \textbf{0.61 in}\\ \textbf{0.61 in}\\ \textbf{0.61 in}\\ \textbf{0.61 in}\\ \textbf{0.39 in}\\ \end{array}$	TOTAL STORY SHEAR CAPACITY (OF ALL OF THE SEISMIC RESISTING MOMENT FRAMES)           16,718 kips           20,093 kips           16,430 kips           16,430 kips           20,047 kips	RATIO OF SHEAR DEMAND / SHEAR CAPACITY PER STORY β 0.0111 0.0211 0.0300 0.0315 0.0373 0.0518 0.0571 0.0617 0.0536 0.0562 0.0583 0.0599 0.0611 0.0625 0.0629 0.0630 0.0631	MAXIMUM ALLOWED STABILITY COEFFICIENT PER STORY θ <sub>L</sub> max 0.250	XIS, θ <sub>y</sub> STABILITY COEFFICIENT PER STORY θ <sub>i</sub> 0.017 0.033 0.047 0.033 0.047 0.053 0.057 0.064 0.071 0.072 0.078 0.084 0.093 0.099 0.106 0.113 0.118 0.123 0.085	СОММЕНТ ОК ОК ОК ОК ОК ОК ОК ОК ОК ОК	MOMENT FRAME MF A3 - G3

	COLUMN	MAXIMUM	CONTROLLING	BUILDING MAX.	COMMENTS FO	R: DESIGN 18B
NAME	MEMBER SIZE	INTERACTION	LOAD	INTERACTION 0.4688	COLUMN STEEL AREA CHECK	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 A3-6	HSS 16 x 16 x 0.625	0.225823288	c c		OK - STEEL AREA IS > 1% OF TOTAL AREA 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	0.104/9312	5		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
A3-17 A3-18	HSS 12 x 12 x 0.75	0.108161754	5		OK - STEEL AREA IS > 1% OF TOTAL AREA OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
R3-1	HSS 16 x 16 x 0.625	0.357266114	6		OK - STEEL ANEA IS > 1% OF TOTAL ANEA	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	0.234/44351	6		OK - STEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
B3-13 B2-14	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 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
C2 0	HSS 16 x 16 x 0.625	0.39112462	6		OK STEEL AREA IS > 1% OF TOTAL AREA	OK STEEL HSS IS COMPACT
03-10	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		UK - SIEEL 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
D3-6	HSS 16 x 16 x 0.625	0.31630084	6		OK - STEEL ABEA IS > 1% OF TOTAL ABEA	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		UK - SIEEL AREA IS > 1% OF TOTAL AREA	OK - STEEL HSS IS COMPACT
D2 10	HOO 12 X 12 X 0.75	0.152201457	6		OK STEEL AREA IS > 1% OF IUTAL AREA	
F3-1	HSS 16 x 16 x 0.625	0.09001832	e d	CONTROL SI	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	CONTROLLING	BUILDING MAX.	COMMENTS FO	R: DESIGN 18B
NAME	MEMBER SIZE	INTERACTION VALUE	LOAD COMBINATION	INTERACTION 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,  $\theta$ , 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<sub>R</sub> 3. 1.2D + 1.6L<sub>R</sub> +  $f_1L$ 4. 1.2D + 1.6L<sub>R</sub> + 0.8W 5. 1.2D + 1.6W +  $f_1L$  + 0.5L<sub>R</sub> 6. 1.2D + 1.0E +  $f_1L$ *Where*:  $f_1 = 0.5$ 

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

<b>ദ്രേ ട്രഹ്ത്താലത്താ</b>	JOB NO. 18	- STORY BUILDINGS	BY	SMG	DATE 4/22/05	SHEET NO.
CC Entfluences antfl	CUSTOMER	DESIGN 18C	CKD		DATE	OF
SUBJECT		DESIGN PARAMETERS	SUM	IARY		MOMENT FRAME MF A3 - G3
o DESIGI	INPUTS:	0 TOAL NUMBER OF COLUMNS BEI	NG ANALYZ	ED	126	
		o YIELD STRENGTH:	CONCRET	H E REINFORCEME	SS, F <sub>y</sub> = 50 ksi NT, F <sub>yr</sub> = 0 ksi	
		• MODULUS OF ELASTICITY:	CONCRET	H E REINFORCEMEI	SS, E <sub>s</sub> = 29,000 ksi NT, E <sub>cr</sub> = 29,000 ksi	
		• MINIMUM CONCRETE COMPRESS	SIVE STREN	GTH	f' <sub>c</sub> = 16.0 ksi	
		o CONCRETE DENSITY			w = 145 lb/ft <sup>3</sup>	
		o CONCRETE REINFORCEMENT		ARI	EA, $A_{sr} = 0.0 \text{ in}^2$ $I_{xor} = 0.0 \text{ in}^4$ $I_{yyr} = 0.0 \text{ in}^4$ $Z_{xor} = 0.0 \text{ in}^3$ $Z_{yyr} = 0.0 \text{ in}^3$	
		o RESISTANCE FACTORS	Α	XIAL COMPRESSI FLEXURAL BENDI	ON, $\phi_c = 0.75$ NG, $\phi_b = 0.90$	
		O SEISMIC PARAMETERS VE ORTHOGONAL LOAD FACTOR ALC FACTOR TO ACCOUNT FOR 5% ACCIDENTAL TOP	REDUN ERTICAL SE DNG Y-AXIS <i>ISION (</i> "SIM	DANCY COEFFICI ISMIC "FACTOR," I OF SHARED COL PLIFIED APPROA	ENT, $\rho = 1.00$ $0.2S_{DS} = 0.20$ UMNS = 0.30 CH") = 0.025	

G(	d Eng	jineeri		). 18 - STO MER DESI	RY BUII GN 18C	LDINGS			BY CKD	SMG	DATE DATE	4/22/05	SH	IEET NO. OF
SU	BJECT		B2 CALC	ULATION	- FOR	BEND	DING A	ALONG	THE X	-AXIS O	F THE	COLUM	V <sup>MC</sup>	MENT FRAME MF A3 - G3
SU	BJECT o STORY NUMBER 18 17 16 15 14 13 12 11 10 9 8 7 6 5 4 3 2 1	LOAD ( LENGTH (STORY HEIGHT) L 13.0 ft 13.0 ft 13.	B2 CALC           DUE TO FORCI BENDING ALONG 1 MOMENT FR/ TOTAL STORY           TOTAL STORY           SEISMIC HORIZONTAL LOAD E ΣH;           185 kips           352 kips           501 kips           633 kips           749 kips           851 kips           938 kips           1,013 kips           1,075 kips           1,260 kips           1,261 kips           1,264 kips	ULATION 1.2D + 1.6Lr + 0. ES THAT CAUSE THE X-AXIS OF THE ME COLUMNS ELASTIC INTERSTORY DUE TO $\Sigma$ H <sub>1</sub> 0.25 in 0.53 in 0.53 in 0.53 in 0.55 in 0.58 in 0.58 in 0.58 in 0.58 in 0.58 in 0.56 in 0.56 in 0.56 in 0.56 in 0.56 in 0.56 in 0.56 in 0.53 in 0.51 in 0.52 in 0.52 in 0.53 in 0.53 in 0.55 in 0.55 in 0.56 in 0.56 in 0.56 in 0.56 in 0.56 in 0.56 in 0.56 in 0.56 in 0.53 in 0.55 in 0.55 in 0.56 in 0.56 in 0.56 in 0.56 in 0.56 in 0.52 in 0.55 in 0.56 in 0.55 in 0.56 in 0.56 in 0.56 in 0.56 in 0.56 in 0.56 in 0.55 in 0.56 in 0.56 in 0.56 in 0.55 in 0.56 in 0.56 in 0.56 in 0.56 in 0.56 in 0.56 in 0.56 in 0.57 in 0.58 in 0.56 in 0.56 in 0.56 in 0.56 in 0.56 in 0.56 in 0.57 in 0.58 in 0.56 in 0.56 in 0.56 in 0.52 in 0.55 in 0.56 in 0.56 in 0.56 in 0.56 in 0.52 in 0.56 in 0.56 in 0.52 in 0.52 in 0.56 in 0.56 in 0.52 in 0.52 in 0.56 in 0.56 in 0.52 in 0.52 in 0.52 in 0.55 in 0.56 in 0.56 in 0.52 in 0.52 in 0.55 in 0.56 in 0.56 in 0.52 in 0.52 in 0.55 in 0.56 in 0.52 in 0.52 in 0.52 in 0.55 in 0.56 in 0.55 in 0.56 in 0.52 in 0.52 in 0.55 in 0.55 in 0.56 in 0.55 in 0.56 in 0.55 in 0.56 in 0.55 in 0.56 in 0.55 in 0.56 in 0.56 in 0.57 in 0.58 in 0.56 in 0.57 in 0.58 in 0.56 in 0.57 in 0.58 in 0.56 in 0.56 in 0.57 in 0.58 in	- FOR 8W TOTAL U STORY OU ("LEANEI 1,513 2,821 4,129 5,437 6,745 8,053 9,361 10,669 11,977 13,285 14,593 15,901 17,209 18,517 19,825 21,133 22,441 23,749	(L.C. #4 (L.C. #4 NFACTOREL NALL COLL R" + "NON-L (kii) UVE LOAD LL 0 1,008 2,016 3,024 4,032 5,040 6,048 7,056 8,064 9,072 10,080 11,088 12,096 13,104 14,112 15,120 16,128 17,136	DING A DAXIAL L MNS OF T EANER" C 38) ROOF LIVE LOAF LVE LOAF 288 288 288 288 288 288 288 288 288 28	ALONG OAD PER HE STORY OLUMNS) SEISMIC WEIGHT DL + P-LL 1,591 3,187 4,783 6,379 7,975 9,571 11,167 12,763 14,359 15,955 17,551 19,147 20,743 22,339 23,935 25,531 27,127 28,723	D.L. 1.2 1.815.6 3,385.2 4,954.8 6,524.4 8,094.0 9,663.6 11,233.2 12,802.8 14,372.4 15,942.0 17,511.6 19,081.2 20,650.8 22,220.4 23,790.0 25,359.6 26,929.2 28,498.8	-AXIS O L FACTORED A L.L. 0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0	F THE XIAL LOAD, CTOR ROOF LL. 1.6 460.8 460.8 460.8 460.8 460.8 460.8 460.8 460.8 460.8 460.8 460.8 460.8 460.8 460.8 460.8 460.8 460.8 460.8 460.8	SEISMIC           VERTICAL           0.2Sps = 0           0.0	Y TOTAL ΣP <sub>u</sub> (kips) 2,276.4 3,846.0 5,415.6 6,985.2 8,554.8 10,124.4 11,694.0 13,263.6 14,833.2 16,402.8 17,972.4 19,542.0 21,111.6 22,681.2 24,250.8 25,820.4 27,390.0 28,959.6	MENT FRAME MF A3 - G3 STORY B <sub>21,X-A005</sub> 1.020 1.028 1.028 1.043 1.042 1.042 1.042 1.042 1.046 1.051 1.057 1.061 1.065 1.075 1.075 1.075 1.075 1.071 1.037

		JOB NO	D. 18 - STO	RY BUII	DINGS			BY	SMG	DATE	4/22/05	SI	HEET NO
GG Eng	ineeri		MER DESI	GN 18C				СКД		DATE			OF
SUBJECT		B2 CALC	ULATION	- FOR	BEND	DING /	ALONG	THE	Y-AXIS	OF THE	COLUM	N <sup>M</sup>	DMENT FRAME MF A3 - G3
o	LOAD	COMBINATION =	1.2D + 1.6Lr + 0.	8W									
		DUE TO FORC BENDING ALONG MOMENT FR.	ES THAT CAUSE THE <mark>Y-AXIS</mark> OF THE AME COLUMNS	TOTAL U STORY OF		D AXIAL L	LOAD PER THE STORY	ТОТ	TAL FACTORE	) AXIAL LOAD,	$\Sigma P_u$ , PER STO	RY	
STORY NUMBER	LENGTH (STORY HEIGHT) L	TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E	ELASTIC INTERSTORY DRIFT Δ <sub>oh</sub>	DEAD	LIVE LOAD	ROOF	SEISMIC	D.L.	LOAD	FACTOR	SEISMIC VERTICAL	TOTAL ΣP <sub>ui</sub> (kips)	STORY B <sub>2i_Y-AXIS</sub>
		ΣΗ	DUE TO $\Sigma H_i$	DL	LL	LUAD	DL + P-LL	1.2	0	1.6	$0.2S_{\text{DS}}=~0$		
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 8,052	4,032	288	7,975	8,094.0	0.0	460.8	0.0	8,554.8	1.042
12	13.0 ft	938 kips	0.55 in	9,053 9,361	5,040 6.048	200 288	9,571	9,003.0	0.0	460.8	0.0	11 694 0	1.042
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
		I	<u></u>										

දීයි පිසක්	ineeri	JOB NO	). 18 - STO	RY BUI	LDINGS	;		BY	SMG	DATE	4/22/05	s	HEET NO.
se mißi	00099900	CUSTO	MER DESI	GN 18C	;			СКД		DATE		7_	OF
UBJECT		•	STABIL	ІТҮ СС	DEFFIC	CIENT	ALON	G COLU	IMN X-A	XIS, θ <sub>x</sub>		M	OMENT FRAME MF A3 - G3
0	LOAD	COMBINATION:		1.2D + 1.0	6Lr + 0.8W								
				0									
0	DEFLEC	CTION AMPLIFIC	ATION FACTOR:	U <sub>d</sub> =	5.5								
0	(SEISM	IC) IMPORTANCE	E FACTOR	I <sub>E</sub> =	1.0								
0	MOMEN TOTAL	IT FRAME RESIS SEISMIC SHEAR	TS WHAT % OF TO THE BUIDLIN	THE VG?	25%								
		DUE TO FORCE BENDING ALONG 1 MOMENT FRA	ES THAT CAUSE THE <mark>X-AXIS</mark> OF THE AME COLUMNS	TOTAL U STORY O	INFACTORE	ED AXIAL L	OAD PER HE STORY	ΤΟΤΑ	L FACTORED	AXIAL LOAD,	$\Sigma P_u$ , PER STC	RY	STABILITY
	LENGTH	TOTAL STORY SHEAR DUE TO	ELASTIC	( LEANE	k + NON-L (ki	ps)	OLUMINS)		LOAD	FACTOR			COEFFICIENT
STORY	(STORY		DRIFT	DEAD	LIVE	ROOF	SEISMIC	D.L.	11.	BOOFLL	SEISMIC	TOTAL ΣP <sub>ui</sub>	
NUMBER	HEIGHT) L	LOAD		LOAD	LOAD	LIVE LOAD	WEIGHT	D.E.	L.L.	NOOT E.E.	VERTICAL	(kips)	
		ΣH <sub>i</sub>	DOE TO ZHi	DL		Lr	DL + P-LL	1.2	0	1.6	$0.2S_{DS} = 0$		
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 π 12.0 ft	749 Kips	0.55 in	6,745 8,052	4,032	288	7,975	8,094.0	0.0	460.8	0.0	8,554.8	0.040
10	13.0 ft	038 kine	0.55 in	0,000	6.048	200	11 167	11 233 2	0.0	400.8	0.0	11 694 0	0.040
11	13.0 ft	1 013 kins	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

				יוו וס עס				DV	SMC	DATE	4/00/05		
GG Eng	ineeri	MG JOB NC	0. 18-510	RYBUI	LDINGS			ВТ	SIVIG	DATE	4/22/05	S	HEET NO.
6		CUSTO	MER DESI	GN 18C	;			СКД		DATE		_	OF
SUBJECT			STABIL	түсс	DEFFIC	CIENT	ALON	GCOLU	MN Y-A	XIS, θ <sub>y</sub>		М	OMENT FRAME MF A3 - G3
0	LOAD C	OMBINATION:		1.2D + 1.6	6Lr + 0.8W								
0	DEFLEO	CTION AMPLIFIC	ATION FACTOR:	C <sub>d</sub> =	5.5								
				- u									
0	(SEISM	IC) IMPORTANCE	FACTOR	I <sub>E</sub> =	1.0								
0	MOMEN TOTAL	IT FRAME RESIS SEISMIC SHEAR	TS WHAT % OF TO THE BUIDLIN	THE IG?	25%								
		DUE TO FORCE BENDING ALONG T MOMENT FRA	es that cause The <mark>Y-axis</mark> of the Me columns	TOTAL U STORY O	INFACTORE	ED AXIAL L JMNS OF T	OAD PER HE STORY	ΤΟΤΑ	L FACTORED	AXIAL LOAD,	ΣP <sub>u</sub> , PER STO	RY	STABILITY
	LENGTH	TOTAL STORY SHEAR DUE TO	ELASTIC		(ki	ps)	SLOWING)		LOAD F	ACTOR		TOTAL	COEFFICIENT
STORY	(STORY		DRIFT	DEAD	LIVE	ROOF	SEISMIC	D.L.	L.L.	ROOF L.L.	SEISMIC	τΟΤΑL ΣΡ <sub>ui</sub>	
NOMBER	L L	LOAD	Δ <sub>oh</sub> DUF TO ΣΗ	LOAD	LOAD	LIVE	WEIGHT	10		10	VERTICAL	(kips)	
10	10.0.4	∑H <sub>i</sub>		DL		Lr		1.2	0	1.6	0.23 <sub>DS</sub> = 0	0.070.4	0.000
18	13.0 π 12.0 ft	185 kips	0.25 In	1,513	1 009	288	1,591	1,815.0	0.0	460.8	0.0	2,276.4	0.020
16	12.0 ft	501 kips	0.59 in	2,021	2,016	200	3,107	3,305.2	0.0	400.0	0.0	5,040.0	0.027
15	13.0 ft	633 kips	0.53 in	5 /37	3.024	200	6 379	6 524 4	0.0	460.8	0.0	6 985 2	0.037
14	13.0 ft	749 kins	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

6	iG Eng	jineeri	ng -	JOB NC	). 18 - STO MER DESI	RY BUII	LDINGS			BY CKD	SMG	DATE DATE	4/22/05	S⊦	IEET NO. OF
S	JBJECT		B2 (		ULATION	- FOR	BEND	DING A	ALONG	THE X	(-AXIS O	F THE	COLUM	V <sup>MC</sup>	MENT FRAME MF A3 - G3
2	0 STORY NUMBER 18 17 16 15 14 13 12	LOAD ( (STORY HEIGHT) L 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft	DUR EENDID MM TOTAIA SEE HORI 18 35 50 63 74 85 93	LATION = TO FORCE SALONG T DMENT FRA L STORY A DUE TO ISMIC ZONTAL AD E EH, 5 kips 2 kips 3 kips 9 kips 1 kips 8 kips	Li2D + 0.5L + 0.5           1.2D + 0.5L + 0.5           SS THAT CAUSE           HE X-AXIS OF THE ME COLUMNS           ELASTIC           INTERSTORY           DRIFT           Adh           DUE TO ΣH,           0.25 in           0.53 in           0.55 in           0.55 in	- FOR 5Lr + 1.6W TOTAL U STORY OF ("LEANER DEAD LOAD DL 1,513 2,821 4,129 5,437 6,745 8,053 9,361	(L.C. #5 NFACTORE NALCOLL NALCOLL AT + "NON-L (Kij LIVE LOAD LL 0 1,008 2,016 3,024 4,032 5,040 6,048	DING A DAXIAL L MINS OF T EANER" C ISS ROOF LIVE LOAD Lr 288 288 288 288 288 288 288 288 288 28	OAD PER HE STORY OLUMNS) SEISMIC WEIGHT DL + P-LL 1,591 3,187 4,783 6,379 7,975 9,571 11,167	THE X TOTA D.L. 1.2 1,815.6 3,385.2 4,954.8 6,524.4 8,094.0 9,663.6 11,233.2	C-AXIS O AL FACTORED A: L.L. 0.5 0.0 504.0 1,008.0 1,512.0 2,016.0 2,520.0 3,024.0	F THE XIAL LOAD, CTOR ROOF LL. 0.5 144.0 144.0 144.0 144.0 144.0 144.0 144.0	Seismic vertical           0.2S <sub>DS</sub> = 0           0.0           0.0           0.0           0.0           0.0           0.0           0.0           0.0           0.0           0.0           0.0	Y TOTAL ΣP <sub>u</sub> (kips) 1,959.6 4,033.2 6,106.8 8,180.4 10,254.0 12,327.6 14,401.2	MENT PHAME <i>MF A3 - G3</i> STORY B <sub>21,X-AXIS</sub> 1.017 1.029 1.043 1.050 1.051 1.052 1.057
	11 10 9 8 7 6 5 4 3 2 1	13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft 13.0 ft	1,0' 1,0' 1,1' 1,1' 1,2' 1,2' 1,2' 1,2' 1,2' 1,2	13 kips 75 kips 26 kips 28 kips 20 kips 24 kips 33 kips 33 kips 34 kips 34 kips	0.58 in 0.61 in 0.52 in 0.56 in 0.56 in 0.56 in 0.53 in 0.48 in 0.24 in	10,669 11,977 13,285 14,593 15,901 17,209 18,517 19,825 21,133 22,441 23,749	7,056 8,064 9,072 10,080 11,088 12,096 13,104 14,112 15,120 16,128 17,136	288 288 288 288 288 288 288 288 288 288	12,763 14,359 15,955 17,551 19,147 20,743 22,339 23,935 25,531 27,127 28,723	12,802.8 14,372.4 15,942.0 17,511.6 19,081.2 20,650.8 22,220.4 23,790.0 25,359.6 26,929.2 28,498.8	3,528.0 4,032.0 4,536.0 5,544.0 6,048.0 6,552.0 7,056.0 7,560.0 8,064.0 8,568.0	144.0 144.0 144.0 144.0 144.0 144.0 144.0 144.0 144.0	0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0	16,474.8 18,548.4 20,622.0 22,695.6 24,769.2 26,842.8 28,916.4 30,990.0 33,063.6 35,137.2 37,210.8	1.064 1.072 1.079 1.078 1.080 1.084 1.091 1.097 1.098 1.094 1.047

GG Ena	lineerf	JOB NC	). 18 - STO	RY BUI	LDINGS			BY	SMG	DATE	4/22/05	SH	IEET NO.
		CUSTO	MER DESI	GN 18C	;			СКД		DATE			OF
SUBJECT		B2 CALC	ULATION	- FOR	BEND	DING /	ALONG	THE	Y-AXIS (	OF THE	COLUM	N <sup>MO</sup>	MENT FRAME MF A3 - G3
o	LOAD	COMBINATION =	1.2D + 0.5L + 0.5	5Lr + 1.6W									
		DUE TO FORCE BENDING ALONG T MOMENT FRA	ES THAT CAUSE THE Y-AXIS OF THE ME COLUMNS	TOTAL U STORY OI ("LEANE	<i>INFACTORE</i> N ALL COLU R" + "NON-L	D AXIAL L IMNS OF 1 EANER" C	.OAD PER 'HE STORY 'OLUMNS)	тот	AL FACTORED	AXIAL LOAD,	$\Sigma P_u$ , PER STO	RΥ	
STORY NUMBER	LENGTH (STORY HEIGHT) L	SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣΗ;	ELASTIC INTERSTORY DRIFT Δ <sub>oh</sub> DUE TO ΣH <sub>i</sub>	DEAD LOAD DL	(ki LIVE LOAD LL	ROOF LIVE LOAD	SEISMIC WEIGHT DL + P-LL	D.L. 1.2	LOAD F	ROOF L.L.	SEISMIC VERTICAL 0.2S <sub>DS</sub> = 0	TOTAL ΣP <sub>ui</sub> (kips)	STORY B <sub>2i_Y-AXIS</sub>
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 8 180 4	1.043
13	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	1.050
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	10,517	14 112	200	22,339	22,220.4	7,056,0	144.0	0.0	20,910.4	1.091
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

( <b>යි</b> යි Ema	ineeri	JOB N	0. 18 - STO	RY BUI	LDINGS	5		BY	SMG	DATE	4/22/05	s	HEET NO.
	JUUU () () () () () () () () () () () () ()	CUST	OMER DESI	GN 18C	;			СКД		DATE			OF
SUBJECT			STABIL		DEFFIC	CIENT	ALON	G COLU	IMN X-A	XIS, θ <sub>x</sub>		М	OMENT FRAME MF A3 - G3
0 0 0	LOAD C DEFLEC (SEISM MOMEN TOTAL	COMBINATION: CTION AMPLIFI IC) IMPORTANC IT FRAME RES SEISMIC SHEA	1 CATION FACTOR: SE FACTOR STS WHAT % OF R TO THE BUIDLIN	2D + 0.5L + C <sub>d</sub> = I <sub>E</sub> = I <u>HE</u> IG?	- 0.5Lr + 1.6 5.5 1.0 25%	w							
		DUE TO FOR BENDING ALONG MOMENT FR	THE X-AXIS OF THE	TOTAL L		ED AXIAL L	OAD PER HE STORY	ΤΟΤΑ	L FACTORED	AXIAL LOAD,	$\Sigma P_u$ , PER STO	RY	STABILITY
		TOTAL STORY	ELASTIC	( LEANE	ki (ki	ps)	OLUWING)		LOAD F	ACTOR			COEFFICIENT
STORY NUMBER	(STORY HEIGHT) L	ANY HORIZONTAL LOAD ΣH <sub>i</sub>	INTERSTORY DRIFT Δ <sub>oh</sub> DUE TO ΣH <sub>i</sub>	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 <sub>DS</sub> = 0	TOTAL ΣP <sub>ui</sub> (kips)	PER STORY θ <sub>i</sub>
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
6	13.0 π 12.0 #	1,200 kips	0.56 in	17 200	12,006	288	19,147	19,081.2	5,544.0	144.0	0.0	24,769.2	0.074
5	13.0 ft	1,224 Kips	0.55 in	18 517	13 104	200	20,743	20,030.8	6 552 0	144.0	0.0	20,042.0	0.077
4	13.0 ft	1,241 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

	fineer	JOB NC	). 18 - STO	RY BUII	DINGS			BY	SMG	DATE	4/22/05	S	HEET NO.
৯৫ দোটি	nnsen	СИЗТО	MER DESI	GN 18C	;			СКД		DATE		1_	OF
SUBJECT			STABIL	түсс	DEFFIC	CIENT	ALON	G COLU	IMN Y-A	XIS, θ <sub>y</sub>		M	OMENT FRAME MF A3 - G3
0 0 0	LOAD C DEFLEC (SEISM MOMEN TOTAL	COMBINATION: CTION AMPLIFICA IC) IMPORTANCE IT FRAME RESIS SEISMIC SHEAR	1 ATION FACTOR: FACTOR TS WHAT % OF TO THE BUIDLIN	.2D + 0.5L + C <sub>d</sub> = I <sub>E</sub> = I <b>HE</b> 2	0.5Lr + 1.6 5.5 1.0 25%	w							
		DUE TO FORCE BENDING ALONG T MOMENT FRA	es that cause The <mark>Y-AXIS</mark> of the Me columns	TOTAL U STORY OI		ED AXIAL L	OAD PER HE STORY	ΤΟΤΑ	L FACTORED	axial load,	ΣP <sub>u</sub> , PER STC	ιRY	STABILITY
STORY	LENGTH	TOTAL STORY SHEAR DUE TO	ELASTIC INTERSTORY	("LEANEI	R" + "NON-L (kij	ps)	OLUMINS)		LOAD F	ACTOR		TOTAL	COEFFICIENT PER STORY
NUMBER	HEIGHT)	HORIZONTAL		DEAD LOAD	LIVE LOAD	LIVE LOAD	SEISMIC WEIGHT	D.L.	L.L.	ROOF L.L.	SEISMIC VERTICAL	ΣP <sub>ui</sub> (kips)	θί
18	13.0.#	∑H <sub>i</sub> 185 kips	DUE TO SH	DL		Lr	DL + P-LL	1.2	0.5	0.5	$0.2S_{DS} = 0$	1 959 6	0.017
17	13.0 ft	352 kips	0.25 in 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
10	13.0 ft	1,013 kips	0.56 m	11,977	8.064	200 288	12,763	12,002.0	4.032.0	144.0	0.0	18,548,4	0.060
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	21,133	14,112	200 288	25,935	25,359.6	7,056.0	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

(	ig Eng	jineeri	JOB NC CUSTO	). 18 - STO MER DESI	RY BUII GN 18C	DINGS			BY CKD	SMG	DATE DATE	4/22/05	S⊦	IEET NO. OF
S	UBJECT		B2 CALC	ULATION	- FOR	BEND	DING A	ALONG	THE X	-AXIS <mark>O</mark>	F THE	COLUM	V <sup>MC</sup>	MENT FRAME MF A3 - G3
	0	LOAD	COMBINATION = DUE TO FORCE BENDING ALONG T MOMENT FRA	1.2D + 0.5L + 1.0 ES THAT CAUSE THE X-AXIS OF THE ME COLUMNS	TOTAL U STORY ON	(L.C. #6) NFACTORE NALL COLU	) D AXIAL L IMNS OF T	OAD PER HE STORY	τοτα	L FACTORED A	XIAL LOAD,	$\Sigma P_u$ , PER STOR	Y	
	STORY NUMBER	LENGTH (STORY HEIGHT) L	TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E	ELASTIC INTERSTORY DRIFT A <sub>oh</sub>	("LEANEF DEAD LOAD	R" + "NON-L (kip LIVE LOAD	EANER" C ps) ROOF LIVE	OLUMNS) SEISMIC WEIGHT	D.L.	LOAD FA	ROOF L.L.	SEISMIC VERTICAL	TOTAL ΣP <sub>ui</sub> (kips)	STORY B <sub>2i_X-AXIS</sub>
	18	13.0 ft	ΣH <sub>i</sub> 185 kips	DUE IO ΣH <sub>i</sub> 0.25 in	DL 1,513	0	Lr 288	DL + P-LL 1,591	1.2 1,815.6	0.5 0.0	0.0	0.2S <sub>DS</sub> = 0.2 318.2	2,133.8	1.019
	17 16 15 14 13 12 11 10 9 8 7 6 5 4 3 2 1	13.0 ft 13.0 ft	352 kips 501 kips 633 kips 749 kips 851 kips 1,013 kips 1,075 kips 1,126 kips 1,200 kips 1,224 kips 1,224 kips 1,260 kips 1,263 kips 1,264 kips	0.39 in 0.53 in 0.55 in 0.55 in 0.55 in 0.58 in 0.61 in 0.56 in 0.56 in 0.56 in 0.56 in 0.56 in 0.53 in 0.48 in 0.24 in	2,821 4,129 5,437 6,745 8,053 9,361 10,669 11,977 13,285 14,593 15,901 17,209 18,517 19,825 21,133 22,441 23,749	1,008 2,016 3,024 4,032 5,040 6,048 7,056 8,064 9,072 10,080 11,088 12,096 13,104 14,112 15,120 16,128 17,136	288288288288288288288288288288288288288288	3,187 4,783 6,379 7,975 9,571 11,167 12,763 14,359 15,955 17,551 19,147 20,743 22,339 23,935 25,531 27,127 28,723	3,385.2 4,954.8 6,524.4 8,094.0 9,663.6 11,233.2 12,802.8 14,372.4 15,942.0 17,511.6 19,081.2 20,650.8 22,220.4 23,790.0 25,359.6 26,929.2 28,498.8	504.0 1,008.0 1,512.0 2,520.0 3,024.0 3,528.0 4,032.0 4,032.0 4,536.0 5,040.0 5,544.0 6,048.0 6,552.0 7,056.0 7,056.0 8,064.0 8,568.0	0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0	637.4 956.6 1,275.8 1,595.0 1,914.2 2,233.4 2,552.6 2,871.8 3,191.0 3,510.2 3,829.4 4,148.6 4,467.8 4,787.0 5,106.2 5,425.4 5,744.6	4,526.6 6,919.4 9,312.2 11,705.0 14,097.8 16,490.6 18,883.4 21,276.2 23,669.0 26,061.8 28,454.6 30,847.4 33,240.2 35,633.0 38,025.8 40,418.6 42,811.4	1.033 1.049 1.058 1.058 1.060 1.066 1.074 1.084 1.091 1.090 1.093 1.098 1.106 1.114 1.114 1.119 1.055

6	(G Ema	lineerí	JOB	NO. 18 - STC	RY BUI	LDINGS			BY	SMG	DATE	4/22/05	Sŀ	IEET NO.
<u> </u>	С ШВ	JULLOOL	CUS	TOMER DES	GN 18C	)			CKD		DATE		]	OF
S	JBJECT		B2 CAL	CULATION	- <b>FO</b> F	R BENE	NNG A	ALONG	THE Y	'-AXIS C	F THE	COLUMI	<u>м</u> с	MENT FRAME MF A3 - G3
	0	LOAD	COMBINATION	/ = 1.2D + 0.5L + 1.	0E									
		1	DUE TO FO BENDING ALOR MOMENT	RCES THAT CAUSE NG THE <mark>Y-AXIS</mark> OF THE FRAME COLUMNS	TOTAL L STORY O ("LEANE	<i>INFACTORE</i> N ALL COLL R" + "NON-L	D AXIAL L JMNS OF T EANER" C	OAD PER HE STORY OLUMNS)	TOTA	AL FACTORED A	XIAL LOAD,	$\Sigma P_u$ , PER STOR	Y	
	STORY NUMBER	LENGTH (STORY HEIGHT)	SHEAR DUE SEISMIC HORIZONTA	L ELASTIC INTERSTORY L DRIFT	DEAD LOAD	(kij LIVE LOAD	ROOF	SEISMIC WEIGHT	D.L.	LOAD FA	ROOF L.L.	SEISMIC VERTICAL	TOTAL ΣP <sub>ui</sub> (kips)	STORY B <sub>2i_Y-AXIS</sub>
	18	L 13.0 ft	ΣH <sub>i</sub> 185 kips	DUE TO ΣH <sub>i</sub> 0.25 in	DL 1,513	LL	LOAD Lr 288	DL + P-LL 1,591	1.2 1,815.6	0.5	0.0	0.2S <sub>DS</sub> = 0.2 318.2	2,133.8	1.019
	17 16	13.0 ft 13.0 ft	352 kips 501 kips	0.39 in 0.53 in	2,821 4,129	1,008 2,016	288 288	3,187 4,783	3,385.2 4,954.8	504.0 1,008.0	0.0 0.0	637.4 956.6	4,526.6 6,919.4	1.033 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	13.0 ft 13.0 ft	749 kips 851 kips	0.55 in 0.53 in	6,745 8 053	4,032 5,040	288 288	7,975 9.571	8,094.0 9 663 6	2,016.0 2 520 0	0.0	1,595.0 1 914 2	11,705.0 14 097 8	1.058
	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	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 0.56 in	14,593	10,080	288	17,551 19 147	17,511.6	5,040.0	0.0	3,510.2 3,829.4	26,061.8	1.090
	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

LOAD COI DEFLECTI (SEISMIC) MOMENT I TOTAL SE ENGTH SI (STORY +EIGHT) L	CUSTO CUSTO CUSTO CUSTO IMPORTANCE FRAME RESIS EISMIC SHEAR DUE TO FORCE EISMIC SHEAR DUE TO FOR	MER DESI STABILI STABILI ATION FACTOR: FACTOR TS WHAT % OF T TO THE BUIDLIN S THAT CAUSE HE X-AXIS OF THE ME COLUMNS	GN 18C ITY CC 1.2D + 0. $C_d = l_E =$ THE KG?	5L + 1.0E 5.5 1.0 25%	JENT	ALON	скр	IMN X-A	DATE XIS, θ <sub>x</sub>		M	OF OF OMENT FRAME MF A3 - G3
LOAD COI DEFLECTI (SEISMIC) MOMENT I TOTAL SE STORY HEIGHT) L	CUSTO MBINATION: TON AMPLIFICA ) IMPORTANCE FRAME RESIS: EISMIC SHEAR DUE TO FORCE EISMIC SHEAR DUE TO FORCE EISMIC SHEAR TOTAL ESTORY HEAR DUE TO	MER DESI STABIL ATION FACTOR: FACTOR TS WHAT % OF T TO THE BUIDLIN S THAT CAUSE HE X-AXIS OF THE ME COLUMNS	GN 18C ITY CC 1.2D + 0. $C_d = l_E = l_E = 1$ <i>HE</i>	<b>DEFFIC</b> .5L + 1.0E 5.5 1.0 25%	CIENT	ALON	CKD	MN X-A	DATE XIS, θ <sub>x</sub>		<u>м</u>	OF OMENT FRAME MF A3 - G3
LOAD COI DEFLECTI (SEISMIC) MOMENT I TOTAL SE STORY HEIGHT) L	MBINATION: ION AMPLIFICA IMPORTANCE FRAME RESIST SISMIC SHEAR DUE TO FORCE SENDING ALONG T MOMENT FRA TOTAL STORY HEAR DUE TO	STABIL ATION FACTOR: FACTOR TS WHAT % OF T TO THE BUIDLIN S THAT CAUSE HE X-AXIS OF THE ME COLUMNS	1.2D + 0. $C_d = l_E = l_E = l_R$	5.5 1.0 25%	CIENT	ALON	G COLU	IMN X-A	XIS, θ <sub>x</sub>		м	OMENT FRAME MF A3 - G3
LOAD COI DEFLECTI (SEISMIC) MOMENT , TOTAL SE BI ENGTH SI (STORY +EIGHT) + L	MBINATION: ION AMPLIFICA IMPORTANCE FRAME RESIS EISMIC SHEAR DUE TO FORCE IENDING ALONG T MOMENT FRAI TOTAL STORY HEAR DUE TO	ATION FACTOR: FACTOR TS WHAT % OF T TO THE BUIDLIN S THAT CAUSE HE X-AXIS OF THE ME COLUMNS	1.2D + 0. C <sub>d</sub> = I <sub>E</sub> = IG?	.5L + 1.0E 5.5 1.0 25%								
LENGTH SI (STORY + LEIGHT) +	TION AMPLIFIC! ) IMPORTANCE FRAME RESIS EISMIC SHEAR DUE TO FORCE SENDING ALONG T MOMENT FRAN TOTAL STORY HEAR DUE TO	ATION FACTOR: FACTOR TS WHAT % OF : TO THE BUIDLIN S THAT CAUSE HE X-AXIS OF THE ME COLUMNS	C <sub>d</sub> = I <sub>E</sub> = IHE IG?	5.5 1.0 25%								
(SEISMIC) MOMENT ( TOTAL SE BI LENGTH SI (STORY   LEIGHT)	DUE TO FORCE ERAME RESIS EISMIC SHEAR DUE TO FORCE ENDING ALONG TA MOMENT FRAI TOTAL STORY HEAR DUE TO	FACTOR TS WHAT % OF T TO THE BUIDLIN STHAT CAUSE HE X-AXIS OF THE ME COLUMNS	U <sub>E</sub> =	1.0 25%								
(SEISMIC) MOMENT I TOTAL SE BI LENGTH SF (STORY HEIGHT) F L	DUE TO FORCE SENSIC SHEAR DUE TO FORCE SENDING ALONG T MOMENT FRAI FOTAL STORY HEAR DUE TO	TS WHAT % OF T TO THE BUIDLIN S THAT CAUSE HE X-AXIS OF THE ME COLUMNS	IE =	1.0 25%								
MOMENT TOTAL SE B LENGTH SI (STORY HEIGHT) L	DUE TO FORCE SENDING ALONG T MOMENT FRAI FOTAL STORY HEAR DUE TO	TS WHAT % OF TO THE BUIDLIN S THAT CAUSE HE X-AXIS OF THE ME COLUMNS		25%								
ENGTH SIORY (STORY HEIGHT) L	DUE TO FORCE 3ENDING ALONG T MOMENT FRAI TOTAL STORY IHEAR DUE TO	S THAT CAUSE HE X-AXIS OF THE ME COLUMNS	TOTAL (									
LENGTH SI (STORY HEIGHT) F L	TOTAL STORY HEAR DUE TO		STORY O	INFACTORE N ALL COLL	ED AXIAL L	OAD PER HE STORY	ΤΟΤΑ	L FACTORED	AXIAL LOAD,	$\Sigma P_u$ , PER STO	RY	
(STORY HEIGHT)		ELASTIC	("LEANE	R" + "NON-L (ki	.EANER" C ps)	OLUMNS)		LOAD F	ACTOR			COEFFICIENT
L		INTERSTORY DRIFT	DEAD	LIVE	ROOF	SEISMIC	D.L.	L.L.	ROOF L.L.	SEISMIC	TOTAL ΣP <sub>ui</sub>	PER STORY θ <sub>i</sub>
-	LOAD	$\Delta_{oh}$ DUE TO $\Sigma H_i$	LOAD DL	LOAD	LIVE	WEIGHT	12	0.5	0	VERTICAL $0.2S_{po} = 0.2$	(kips)	
13.0 ft	∑H <sub>i</sub> 185 kips	0.25 in	1.513	0	Lr 288	1.591	1.815.6	0.0	0.0	318.2	2.133.8	0.018
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
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
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
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
13.0 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
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
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
13.0 ft	1,075 kips	0.61 in	11,977	8,064	288	14,359	14,3/2.4	4,032.0	0.0	2,8/1.8	21,276.2	0.077
13.0 ft	1,120 Kips	0.62 in	13,200	9,072	200	17 551	17,511.6	4,536.0	0.0	3,191.0	25,009.0	0.083
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
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
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
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
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
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
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
				1								
				1								
				1								
		L										<u> </u>
	I3.0 ft I3.0 ft	13.0 ft       851 kips         13.0 ft       938 kips         13.0 ft       1,013 kips         13.0 ft       1,075 kips         13.0 ft       1,176 kips         13.0 ft       1,168 kips         13.0 ft       1,200 kips         13.0 ft       1,224 kips         13.0 ft       1,253 kips         13.0 ft       1,260 kips         13.0 ft       1,263 kips         13.0 ft       1,264 kips	13.0 ft       851 kips       0.53 in         13.0 ft       938 kips       0.55 in         13.0 ft       1,013 kips       0.58 in         13.0 ft       1,075 kips       0.61 in         13.0 ft       1,126 kips       0.62 in         13.0 ft       1,126 kips       0.55 in         13.0 ft       1,126 kips       0.56 in         13.0 ft       1,200 kips       0.56 in         13.0 ft       1,224 kips       0.55 in         13.0 ft       1,253 kips       0.56 in         13.0 ft       1,260 kips       0.53 in         13.0 ft       1,263 kips       0.48 in         13.0 ft       1,264 kips       0.24 in	13.0 ft       851 kips       0.53 in       8,053         13.0 ft       938 kips       0.55 in       9,361         13.0 ft       1,013 kips       0.58 in       10,669         13.0 ft       1,075 kips       0.61 in       11,977         13.0 ft       1,126 kips       0.62 in       13,285         13.0 ft       1,126 kips       0.56 in       15,901         13.0 ft       1,200 kips       0.56 in       15,901         13.0 ft       1,224 kips       0.55 in       17,209         13.0 ft       1,241 kips       0.56 in       18,517         13.0 ft       1,253 kips       0.56 in       19,825         13.0 ft       1,260 kips       0.53 in       21,133         13.0 ft       1,264 kips       0.24 in       23,749	13.0 ft       851 kips       0.53 in       8,053       5,040         13.0 ft       938 kips       0.55 in       9,361       6,048         13.0 ft       1,013 kips       0.58 in       10,669       7,056         13.0 ft       1,075 kips       0.61 in       11,977       8,064         13.0 ft       1,126 kips       0.62 in       13,285       9,072         13.0 ft       1,126 kips       0.55 in       14,593       10,080         13.0 ft       1,200 kips       0.56 in       15,901       11,088         13.0 ft       1,224 kips       0.55 in       17,209       12,096         13.0 ft       1,241 kips       0.56 in       18,517       13,104         13.0 ft       1,253 kips       0.56 in       19,825       14,112         13.0 ft       1,260 kips       0.53 in       21,133       15,120         13.0 ft       1,263 kips       0.48 in       22,441       16,128         13.0 ft       1,264 kips       0.24 in       23,749       17,136	13.0 ft       851 kips       0.53 in       8,053       5,040       288         13.0 ft       938 kips       0.55 in       9,361       6,048       288         13.0 ft       1,013 kips       0.58 in       10,669       7,056       288         13.0 ft       1,075 kips       0.61 in       11,977       8,064       288         13.0 ft       1,126 kips       0.62 in       13,285       9,072       288         13.0 ft       1,168 kips       0.58 in       14,593       10,080       288         13.0 ft       1,200 kips       0.56 in       15,901       11,088       288         13.0 ft       1,224 kips       0.55 in       17,209       12,096       288         13.0 ft       1,241 kips       0.56 in       18,517       13,104       288         13.0 ft       1,253 kips       0.56 in       19,825       14,112       288         13.0 ft       1,260 kips       0.53 in       21,133       15,120       288         13.0 ft       1,263 kips       0.48 in       22,441       16,128       288         13.0 ft       1,264 kips       0.24 in       23,749       17,136       288	13.0 ft       851 kips       0.53 in       8,053       5,040       288       9,571         13.0 ft       938 kips       0.55 in       9,361       6,048       288       11,167         13.0 ft       1,013 kips       0.58 in       10,669       7,056       288       12,763         13.0 ft       1,075 kips       0.61 in       11,977       8,064       288       14,359         13.0 ft       1,126 kips       0.62 in       13,285       9,072       288       15,955         13.0 ft       1,168 kips       0.58 in       14,593       10,080       288       17,551         13.0 ft       1,200 kips       0.56 in       15,901       11,088       288       19,147         13.0 ft       1,224 kips       0.55 in       17,209       12,096       288       20,743         13.0 ft       1,241 kips       0.56 in       19,825       14,112       288       23,935         13.0 ft       1,260 kips       0.53 in       21,133       15,120       288       25,531         13.0 ft       1,264 kips       0.24 in       23,749       17,136       288       28,723         13.0 ft       1,264 kips       0.24 in       23,749       <	13.0 ft       851 kips       0.53 in       8.053       5.040       288       9.571       9.663.6         13.0 ft       938 kips       0.55 in       9.361       6.048       288       11,167       11,233.2         13.0 ft       1.013 kips       0.58 in       10.669       7.056       288       12,763       12,802.8         13.0 ft       1.075 kips       0.61 in       11,977       8.064       288       14,359       14,372.4         13.0 ft       1.126 kips       0.62 in       13.285       9.072       288       15,955       15,942.0         13.0 ft       1.168 kips       0.58 in       14,593       10.080       288       17,551       17,511.6         13.0 ft       1.200 kips       0.56 in       15,901       11.088       288       19,147       19,081.2         13.0 ft       1.224 kips       0.55 in       17,209       12,096       288       20,743       20,650.8         13.0 ft       1.241 kips       0.56 in       18,517       13,104       288       23,395       23,790.0         13.0 ft       1.260 kips       0.53 in       21,133       15,120       288       25,531       25,559.6         13.0 ft       1.264	13.0 tt       851 kips       0.53 in       8.053       5.040       288       9.571       9.663.6       2.520.0         13.0 tt       938 kips       0.55 in       9.361       6.048       288       11.167       11.233.2       3.024.0         13.0 tt       1.013 kips       0.58 in       10.669       7.056       288       12.763       12.802.8       3.528.0         13.0 tt       1.075 kips       0.61 in       11.977       8.064       288       14.359       14.372.4       4.032.0         13.0 tt       1.168 kips       0.62 in       13.285       9.072       288       15.955       15.942.0       4.536.0         13.0 tt       1.200 kips       0.56 in       15.901       11.088       288       19.147       19.081.2       5.544.0         13.0 tt       1.224 kips       0.55 in       17.209       12.096       288       22.339       22.20.4       6.552.0         13.0 tt       1.241 kips       0.56 in       19.825       14.112       288       23.935       23.790.0       7.056.0         13.0 tt       1.260 kips       0.53 in       21.133       15.120       288       27.127       26.929.2       8.064.0         13.0 tt <td< td=""><td>13.0 tt       851 kips       0.53 in       8,053       5,040       288       9,571       9,663.6       2,520.0       0.0         13.0 tt       938 kips       0.55 in       9,361       6,048       288       11,167       11,233.2       3,024.0       0.0         13.0 tt       1,013 kips       0.58 in       10,669       7,056       288       12,763       12,802.8       3,528.0       0.0         13.0 tt       1,075 kips       0.61 in       11,977       8,064       288       14,359       14,372.4       4,032.0       0.0         13.0 tt       1,126 kips       0.62 in       13,285       9,072       288       15,955       15,942.0       4,536.0       0.0         13.0 tt       1,200 kips       0.56 in       15,901       11,088       288       19,147       19,081.2       5,544.0       0.0         13.0 tt       1,224 kips       0.56 in       18,517       13,104       288       22,393       22,204       6,552.0       0.0         13.0 tt       1,260 kips       0.53 in       21,133       15,120       288       25,531       25,359.6       7,560.0       0.0         13.0 tt       1,260 kips       0.24 in       23,749</td><td>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         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         13.0 ft       1.013 kips       0.56 in       10.669       7.056       288       12,763       12,802.8       3,528.0       0.0       2,552.6         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         13.0 ft       1.126 kips       0.62 in       13,285       9.072       288       15,955       15,942.0       4,536.0       0.0       3,151.0         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         13.0 ft       1.204 kips       0.56 in       18,517       13,104       288       22,339       22,220.4       6,552.0       0.0       4,467.8         13.0 ft       1.260 kips       0.53 in       12,133       15,120       288       23,351       25,369.6</td><td>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         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         13.0 ft       1,013 kips       0.58 in       10,669       7,056       288       12,802.8       3,528.0       0.0       2,552.6       18,883.4         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         13.0 ft       1,260 kips       0.56 in       15,501       11,088       288       17,551       15,942.0       4,586.0       0.0       3,191.0       23,669.0         13.0 ft       1,220 kips       0.55 in       17,509       12,096       288       20,743       20,650.8       6,048.0       0.0       4,148.6       30,847.4         13.0 ft       1,224 kips       0.56 in       18,517       13,104       288       22,393       23,790.0       7,056.0       0.0       4,467.8       33,240.2         13.0 ft       1,2</td></td<>	13.0 tt       851 kips       0.53 in       8,053       5,040       288       9,571       9,663.6       2,520.0       0.0         13.0 tt       938 kips       0.55 in       9,361       6,048       288       11,167       11,233.2       3,024.0       0.0         13.0 tt       1,013 kips       0.58 in       10,669       7,056       288       12,763       12,802.8       3,528.0       0.0         13.0 tt       1,075 kips       0.61 in       11,977       8,064       288       14,359       14,372.4       4,032.0       0.0         13.0 tt       1,126 kips       0.62 in       13,285       9,072       288       15,955       15,942.0       4,536.0       0.0         13.0 tt       1,200 kips       0.56 in       15,901       11,088       288       19,147       19,081.2       5,544.0       0.0         13.0 tt       1,224 kips       0.56 in       18,517       13,104       288       22,393       22,204       6,552.0       0.0         13.0 tt       1,260 kips       0.53 in       21,133       15,120       288       25,531       25,359.6       7,560.0       0.0         13.0 tt       1,260 kips       0.24 in       23,749	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         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         13.0 ft       1.013 kips       0.56 in       10.669       7.056       288       12,763       12,802.8       3,528.0       0.0       2,552.6         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         13.0 ft       1.126 kips       0.62 in       13,285       9.072       288       15,955       15,942.0       4,536.0       0.0       3,151.0         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         13.0 ft       1.204 kips       0.56 in       18,517       13,104       288       22,339       22,220.4       6,552.0       0.0       4,467.8         13.0 ft       1.260 kips       0.53 in       12,133       15,120       288       23,351       25,369.6	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         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         13.0 ft       1,013 kips       0.58 in       10,669       7,056       288       12,802.8       3,528.0       0.0       2,552.6       18,883.4         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         13.0 ft       1,260 kips       0.56 in       15,501       11,088       288       17,551       15,942.0       4,586.0       0.0       3,191.0       23,669.0         13.0 ft       1,220 kips       0.55 in       17,509       12,096       288       20,743       20,650.8       6,048.0       0.0       4,148.6       30,847.4         13.0 ft       1,224 kips       0.56 in       18,517       13,104       288       22,393       23,790.0       7,056.0       0.0       4,467.8       33,240.2         13.0 ft       1,2

ତିଓ ସିଲିଆଁଲ <b>ିକ୍କ</b> ମ୍ବାଲିଶ	JOBN	NO. 18	- STORY BL	JILDINGS		BY	SMG	DATE	4/22/05	SHEET NO.
®®	CUST	OMER	DESIGN 18	C		СКД		DATE		OF
SUBJECT		ST	ABILITY C	OEFFICI	ENT ALO		JMN X-A	XIS, θ <sub>x</sub>		MOMENT FRAME MF A3 - G3
• LOAD COME	BINATION	:	1.2D +	0.5L + 1.0E						
o DEFLECTIO	N AMPLIF	CATION F	ACTOR: C <sub>d</sub>	= 5.5						
Γ			DUE TO FORCE BENDING ALONG T	S THAT CAUSE HE X-AXIS OF THE	TOTAL STORY	BATIO OF				
-			MOMENT FRA	ME COLUMNS	SHEAR CAPACITY	SHEAR	MAXIMUM ALLOWED	STABILITY		
	STORY	LENGTH	SHEAR DUE TO	INTERSTORY	(OF ALL OF THE SEISMIC	SHEAR	COEFFICIENT	PER STORY	COMMENT	
١	NUMBER	HEIGHT)	HORIZONTAL		RESISTING MOMENT	PER STORY	PER STORY θ <sub>i_max</sub>	θί		
		L	LOAD E ΣH <sub>i</sub>	DUE TO SH	FRAMES)	р				
Ι	18	13.0 ft	185 kips	0.25 in	43,402 kips	0.0043	0.250	0.018	ок	
	17	13.0 ft	352 kips	0.39 in	43,402 kips	0.0081	0.250	0.032	ок	
	16 15	13.0 ft	501 kips 633 kips	0.53 in 0.58 in	43,402 kips 65,313 kips	0.0115	0.250	0.047	ок	
	14	13.0 ft	749 kips	0.55 in	65,313 kips	0.0115	0.250	0.055	ок	
	13	13.0 ft	851 kips	0.53 in	65,313 kips	0.0130	0.250	0.056	ок	
	12	13.0 ft	938 kips	0.55 in	66,992 kips	0.0140	0.250	0.062	ок	
	11	13.0 ft	1,013 kips	0.58 in	66,992 kips	0.0151	0.250	0.069	ок	
	10	13.0 ft	1,075 kips	0.61 in	66,992 kips	0.0160	0.250	0.077	ок	
	9 8	13.0 ft	1,168 kips	0.58 in	8,090 kips	0.1444	0.250	0.084	ок	
	7	13.0 ft	1,200 kips	0.56 in	8,090 kips	0.1483	0.250	0.085	ок	
	6	13.0 ft	1,224 kips	0.55 in	8,090 kips	0.1513	0.250	0.089	ок	
	5	13.0 ft	1,241 kips	0.56 in	8,090 kips	0.1534	0.250	0.096	ок	
	4	13.0 ft	1,253 kips	0.56 in	8,090 kips	0.1549	0.250	0.102	ок	
	3	13.0 π 13.0 ft	1,260 kips	0.53 in 0.48 in	8,090 kips 8,090 kips	0.1557	0.250	0.103	ОК	
	- 1	13.0 ft	1,264 kips	0.24 in	8,090 kips	0.1562	0.250	0.052	ок	

GG Eng	ineeri		). 18 - STO	RY BUI	LDINGS			BY	SMG	DATE	4/22/05	S	HEET NO.
		CUSTO	MER DESI	GN 18C	;			СКД		DATE			OF
SUBJECT			STABILI		)EFFIC	CIENT	ALON	G COLU	IMN Y-A	XIS, θ <sub>y</sub>		м	OMENT FRAME MF A3 - G3
0 0 0	LOAD C DEFLEC (SEISM MOMEN TOTAL	COMBINATION: CTION AMPLIFIC IC) IMPORTANC IT FRAME RESIS SEISMIC SHEAF	ATION FACTOR: E FACTOR STS WHAT % OF 1 R TO THE BUIDLIN	1.2D + 0. C <sub>d</sub> = I <sub>E</sub> = IHE IG?	5L + 1.0E 5.5 1.0 25%								
		DUE TO FORC BENDING ALONG MOMENT FR	ES THAT CAUSE THE <mark>Y-AXIS</mark> OF THE AME COLUMNS	TOTAL U STORY O	INFACTORE N ALL COLL B" + "NON-I	ED AXIAL L JMNS OF T EANEB" C	OAD PER HE STORY OLUMNS)	ΤΟΤΑ	L FACTORED	axial load,	$\Sigma P_u$ , PER STO	RY	STABILITY
		TOTAL STORY	ELASTIC		(ki	ps)	OLOWING)		LOAD F	ACTOR			COEFFICIENT
STORY NUMBER	(STORY HEIGHT) L	ANY HORIZONTAL LOAD ΣH <sub>i</sub>	INTERSTORY DRIFT Δ <sub>oh</sub> DUE TO ΣH <sub>i</sub>	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	$\begin{array}{l} SEISMIC\\ VERTICAL\\ 0.2S_{DS}=\ 0.2 \end{array}$	TOTAL ΣP <sub>ui</sub> (kips)	PER STORY $\theta_i$
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	0.39 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	0.50 III	21 133	14,112	200	25,935	25,790.0	7,056.0	0.0	4,707.0	38,025,8	0.102
2	13.0 ft	1,200 kips	0.35 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	1,200 kips	0.40 in	23 749	17 136	288	28 723	28 498 8	8 568 0	0.0	5 744 6	42 811 4	0.052

ලිශි <b>සිත</b> ෝ <b>හලුදා</b> න්තය	JOB N	JOB NO. 18 - STORY BUILDINGS BY SMG					SMG	DATE	4/22/05	SHEET NO.
CC Findinger mid	CUST	OMER	DESIGN 18	SC		CKD		DATE		OF
SUBJECT	•	ST		OEFFICI	ENT ALOI		JMN Y-A	XIS, θ <sub>y</sub>		MOMENT FRAME MF A3 - G3
• LOAD COMB	o <b>LOAD COMBINATION:</b> 1.2D + 0.5L + 1.0E									
o <b>DEFLECTION</b>	o DEFLECTION AMPLIFICATION FACTOR: $C_d = 5.5$									
Γ			DUE TO FORCE BENDING ALONG T MOMENT FRA	S THAT CAUSE HE <mark>Y-AXIS</mark> OF THE ME COLUMNS	TOTAL STORY SHEAR	RATIO OF SHEAR	MAXIMUM			
N	STORY IUMBER	LENGTH (STORY HEIGHT) L	TOTAL STORY SHEAR DUE TO SEISMIC HORIZONTAL LOAD E ΣH <sub>i</sub>	INTERSTORY DRIFT Δ <sub>oh</sub> DUE TO ΣH <sub>i</sub>	CAPACITY (OF ALL OF THE SEISMIC RESISTING MOMENT FRAMES)	DEMAND / SHEAR CAPACITY PER STORY β	ALLOWED STABILITY COEFFICIENT PER STORY θ <sub>i_max</sub>	STABILITY COEFFICIENT PER STORY θ <sub>i</sub>	COMMENT	
Γ	18	13.0 ft	185 kips	0.25 in	43,402 kips	0.0043	0.250	0.018	ОК	
	17	13.0 ft	352 kips	0.39 in	43,402 kips	0.0081	0.250	0.032	ок	
	16	13.0 ft	501 kips	0.53 in	43,402 kips	0.0115	0.250	0.047	ОК	
	15 14	13.0 ft	633 Kips 749 kips	0.58 in	65,313 kips	0.0097	0.250	0.055	OK	
	13	13.0 ft	851 kips	0.53 in	65.313 kips	0.0130	0.250	0.056	ок	
	12	13.0 ft	938 kips	0.55 in	66,992 kips	0.0140	0.250	0.062	ОК	
	11	13.0 ft	1,013 kips	0.58 in	66,992 kips	0.0151	0.250	0.069	ок	
	10	13.0 ft	1,075 kips	0.61 in	66,992 kips	0.0160	0.250	0.077	ок	
	9	13.0 ft	1,126 kips	0.62 in	66,992 kips	0.0168	0.250	0.084	ок	
	8	13.0 ft	1,168 kips	0.58 in	8,090 kips	0.1444	0.250	0.083	ок	
	7	13.0 ft	1,200 kips	0.56 in	8,090 kips	0.1483	0.250	0.085	ОК	
	6	13.0 ft	1,224 kips	0.55 in	8,090 kips	0.1513	0.250	0.089	ок	
	5	13.0 ft	1,241 kips	0.56 in	8,090 kips	0.1534	0.250	0.096	ОК	
	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	
	1	13.0 ft	1,264 kips	0.24 in	8.090 kips	0.1562	0.250	0.052	OK	
		10.0 11	1,2011,000	0.2111	0,000 1000	0.1002	0.200	0.002	U.N.	
							ı I		I	

COLUMN		MAXIMUM	CONTROLLING	BUILDING MAX.	COMMENTS FOR: DESIGN 18C		
NAME	MEMBER SIZE	INTERACTION VALUE	LOAD	INTERACTION 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 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 A3-10	HSS 22 X 22 X 0.3125	0.287723224	6		OK - STEEL AREA IS > 1% OF TOTAL AREA 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 A3-17	HSS 18 x 18 x 0.25	0.239118206	6		OK - STEEL AREA IS > 1% OF TOTAL AREA 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 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!	
B3-0 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 B3-14	HSS 21 x 21 x 0.3125	0.309106373	6		OK - STEEL AREA IS > 1% OF TOTAL AREA 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 OK STEEL AREA IS > 1% OF TOTAL AREA	N.G STEEL HSS IS NOT COMPACT!	
C3-2	HSS 24 x 24 x 0.3125	0.551673571	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!	
C3-10	HSS 22 x 22 x 0.3125	0.545520578	6		OK - STEEL AREA IS > 1% OF TOTAL AREA 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	0.400934053	6		OK - STEEL AREA IS > 1% OF TOTAL AREA 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 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 D3-13	HSS 22 X 22 X 0.3125 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	CONTROL SI	UK - 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	MAXIMUM CONTROLLING		COMMENTS FO	R: DESIGN 18C
NAME	MEMBER SIZE	INTERACTION VALUE	LOAD COMBINATION	INTERACTION 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 ABEA IS > 1% OF TOTAL ABEA	N.G STEEL HSS IS NOT COMPACT!
F3-6	HSS 24 x 24 x 0 3125	0.398779724	6		OK - STEEL ABEA IS > 1% OF TOTAL ABEA	N.G STEEL HSS IS NOT COMPACT!
F3-7	HSS 24 x 24 x 0 3125	0.396293824	6		OK - STEEL ABEA IS > 1% OF TOTAL ABEA	N.G STEEL HSS IS NOT COMPACT
F3-8	HSS 24 x 24 x 0 3125	0.415664487	6		OK - STEEL ABEA IS > 1% OF TOTAL ABEA	N.G STEEL HSS IS NOT COMPACT
F3-9	HSS 22 x 22 x 0 3125	0.435779437	6		OK - STEEL ABEA IS > 1% OF TOTAL ABEA	N.G STEEL HSS IS NOT COMPACT
F3-10	HSS 22 x 22 x 0.3125	0.431239855	6		OK - STEEL ABEA 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
E2 12	HSS 21 x 21 x 0 2125	0.279550274	6			
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
E2 15	HSS 21 x 21 x 0.3125	0.377337130	6			
E2 16	LISS 21 x 21 x 0.3123	0.209443207	6			
F9 17	HCC 18 x 18 x 0.25	0.413333700	6			
F0-17	HSS 10 X 10 X 0.25	0.333606397	6		OK STEEL AREA IS > 1% OF TOTAL AREA	N.G. STEEL HSS IS NOT COMPACT
F3-10	HSS 16 X 16 X 0.25	0.203374133	6		OK STEEL AREA IS > 1% OF TOTAL AREA	N.G. STEEL HSS IS NOT COMPACT
00-1	HSS 24 X 24 X 0.3125	0.762230369	6		OK OTEL AREA IS > 1% OF TOTAL AREA	N.G STEEL HSS IS NOT COMPACT
03-2	H00 04 x 24 x 0.0125	0.515001462	5		OK OTEL 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		UK - 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		UK - 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!
•		1		1		

# **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<sub>c</sub>,
  - 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<sub>i</sub>, for the LFRS with which the column is located.
  - For this example the following parameters were used:
    - $F_{yc} = 46 \text{ ksi}$
    - $f'_c = 4 \text{ ksi}$
    - $E_s = 29,000 \text{ ksi}$
    - $E_c = 3,492 \text{ ksi}$
    - 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,*  $\Delta_e$ *:*

$$\Delta_{\rm e} = \frac{\delta_{\rm x} I}{C_{\rm d}} = \frac{(0.02*13'*12)(1.0)}{5.5} = 0.567''$$
(I-1)

- 5. Calculate a target value of the flexural rigidity ratio,  $\eta$ , 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  $\eta = 0.8$ , 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<sub>eff</sub> 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:

$$EI_{eff} = \left[ \eta + \left(\frac{H}{2L}\right) \right]_{I} \left[ \frac{VLH^{2}}{6\Delta_{e}} \right]_{I}$$
(I-2a)

$$\mathrm{EI}_{\mathrm{eff}} = \left[1.0 + \left(\frac{13'}{(2)(20')}\right)\right]_{1} \left[\frac{(316\mathrm{kips})(20'*12)(13'*12)^{2}}{6(0.567'')}\right]_{1} \tag{I-2b}$$

$$EI_{eff} = 102,690,685 \text{ k} - \text{in}^2$$

- 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) <sup>1</sup>/<sub>2</sub> inch. A depth of 20 inches will result in an approximate  $EI_{eff}$  value of 100,143,906 k-in<sup>2</sup>. Therefore, try an HSS 20x20x1/2 which has a calculated  $EI_{eff} = 99,099,314$  k-in<sup>2</sup>. 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.

$$Z_s = 1.4d^2t = 1.4(20'')^2(0.5'') = 280 \text{ in}^3$$
 (I-3)

9. Use Equation M-1 from Appendix M to estimate the plastic modulus of each girder.

$$Z_{g} < \frac{Z_{c}F_{yc}}{1.1R_{y}F_{yg}} = \frac{(280in^{3})(46ksi)}{1.1(1.1)(50ksi)} = 212 in^{3}$$
(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  $Z_x = 177$  in<sup>3</sup>.
- 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,  $A_e = 81$  in<sup>2</sup> and the modified (elastic) moment of inertia,  $I_e = 3,417$  in<sup>4</sup> 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	E MOMENT FRA ANALYZED	ME BEING	FROM THE PERPENDICULAR MOMENT FRAME (COLUMN IS SHARED)				
	AXIAL	X-AXIS MOMENT (kip-in)		AXIAL (kips)	Y-AXIS MOMENT (kip-in)			
	(Ripo)	@ TOP	@ BOT.	(hipo)	@ 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,  $\theta$ , 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 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 <sub>1</sub> FOR EACH LOAD COMBINATION						
LOAD B <sub>1x</sub> B <sub>1y</sub>						
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 <sub>2</sub> FOR EACH LOAD COMBINATION					
LOAD B <sub>2x</sub> B <sub>2y</sub>					
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: 
$$f_1 = 0.5$$
  
 $E = \rho Q_E + 0.2 S_{DS} D'$   
 $\rho = 1.00$   
 $D' = seismic weight$ 

FACTORED DESIGN FORCES ON COLUMN 'E3-1'							
	P <sub>u</sub> (kinc)	MOMENT (kip-in)					
COMPANY	(KIPS)	M <sub>ux</sub>	M <sub>uy</sub>				
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<sub>x</sub> and K<sub>y</sub>, 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<sub>eff</sub> 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 K <sub>x</sub> K <sub>y</sub>						
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  $\phi P_{n}$ , 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
    - $Z_x = Z_y = 274.55 \text{ in}^3$ •  $A_s = 37.9 \text{ in}^2$ •  $I_s = I_x = I_y = 2,367 \text{ in}^4$ •  $A_c = 360 \text{ in}^2$ •  $I_c = 11,037 \text{ in}^4$ •  $E_c = 3,492 \text{ ksi}$ •  $K_x = K_y = 1.657$ Reference table above

Since this is a square HSS: 
$$\phi_b M_{nx} = \phi_b M_{ny} = \phi_b Z_s F_{yc} = 11,366 \text{ k} - \text{in}$$
 (I-5)

$$P_o = A_s F_v + C_2 A_c f'_c = 2,969 \text{ kips}$$
 (I-6)

Where: 
$$C_2 = 0.85$$

$$(EI_{eff})_{x} = (EI_{eff})_{y} = E_{s}I_{s} + C_{3}E_{c}I_{c} = 99,099,314 \text{ k} - \text{in}^{2}$$
(I-7)

Where: 
$$C_3 = \min(0.6 + 2\left(\frac{A_s}{A_s + A_c}\right), 0.9) = 0.79$$

Since this is a square HSS:  $P_e = P_{ex} = P_{ey} = \pi^2 (EI_{eff}) / (KL)^2 = 14,638 \text{ kips}$  (I-8)

$$\alpha_{\rm x} = \alpha_{\rm y} = \sqrt{\left({\rm P_o}/{\rm P_e}\right)} = 0.45 \tag{I-9}$$

Since 
$$\alpha < 1.5$$
:  $\Lambda = 0.658^{\alpha^2} = 0.9187$  (I-10)

$$P_n = \Lambda P_o = 2,727 \text{ kips} \tag{I-11}$$

Since this is a square HSS: 
$$\phi_c P_{nx} = \phi_c P_{ny} = (0.75)P_n = 2,046$$
 kips (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				

$$C_{\lambda} = \Lambda C = (0.9187)(A_c 0.85f_c) = 1,125 \text{ kips}$$
 (I-13)

$$C_d = \phi_c C_\lambda = (0.75)(1,125 \text{ kips}) = 844 \text{ kips}$$
 (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
  - $M_{uv} = 1,744$  kip-in

Since 
$$P_u > C_d$$
:  $\frac{P_u - C_d}{\phi_c P_n - C_d} + \frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} = 0.953$  (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 Engineening	JOB NO. 18 -	STORY BUILDINGS		BY SMG	DATE	10/28/04	SHEET NO.		
ee milinneermili	CUSTOMER	DESIGN 18A		СКД	DATE		OF		
SUBJECT	COLUMN 'E	3-1' MEMBER STRENG	THS FC	R LOAD COM	BINATI	ON #6	MOMENT FRAME MF A3 - G3		
o LOAI	COMBINATION	1.2D + 0.5L	+ 1.0E	(L.C. # 6)					
o LATERAL UNBRACED LENGTH OF THE COLUMN: $L_{xx} = 13.0 \text{ ft}$ $L_{yy} = 13.0 \text{ ft}$									
o YIELD STRENGTH: HSS, $F_{yc}$ = 46 ksi CONCRETE REINFORCEMENT, $F_{yr}$ = 0 ksi									
o MOD	o MODULUS OF ELASTICITY: HSS, $\rm E_s$ = 29,000 ksi CONCRETE REINFORCEMENT, $\rm E_{cr}$ = 29,000 ksi CONCRETE, $\rm E_c$ = 3,492 ksi								
o MINI	MUM CONCRETE CO	MPRESSIVE STRENGTH f' <sub>c</sub> = 4.0 k	si						
o CON	CRETE DENSITY	w = 145 l	b/ft <sup>3</sup>						
o HSS	(COLUMN) SECTION:	HSS 20 x 2	0 x 0.5	192 lb/ft WITH CONCRETE	INCLUDED				
		STEEL (SHELL):	c	ONCRETE (CORE):					
		(MIN) WIDTH, $b = 20.0$ in	(MIN) W	/IDTH, b = 19.0 in					
		(MAX) WIDTH, $d = 20.0$ m WALL THICKNESS, $t = 0.5$ in $0.5$ in	(MAX) W	REA. $A_c = 360.142 \text{ in}^2$					
		AREA, $A_s = 37.927 \text{ in}^2$		$I_{xx} = 11,037 \text{ in}^4$					
		I <sub>xx</sub> = 2,367 in^4		I <sub>yy</sub> = 11,037 in^4					
		l <sub>yy</sub> = 2,367 in^4		$r_{xx} = 5.54$ in					
		$r_{xx} = 7.9$ in		$r_{yy} = 5.54 \text{ in}$					
		$r_{yy} = 7.9 \text{ In}$ $7 = 275 \text{ in}^3$		$Z_{xx} = 1,707 \text{ In}^3$ $Z_{xx} = 1,707 \text{ in}^3$					
		$Z_{xx} = 275 \text{ in}^3$ $Z_{yy} = 275 \text{ in}^3$		2 <sub>yy</sub> = 1,707 m <sup>2</sup>					
		Column Properties t	for Elastic An	alveis					
			/ E' = 81 in <sup>2</sup>						
		IVISUAL ANALYSIS = (EI) <sub>eff_x</sub>	/ E' = 3,417	in^4					
		When: E <sub>VISUAL ANALYSIS</sub>	= E' = 29,000	ksi					
			TT 0005 410						
0 000	POSITE COLUMIN LIM	MINIMUM (ALLOWED) $A_e = 3.98^{\circ}$	1 in <sup>2</sup>	SPECIFICATION)					
	% OF TOTA	L AREA THAT THE HSS MAKES UP = 9.53	%	OK - STEEL AREA IS > 1%	OF TOTAL A	IREA			
		(ACTUAL) d / t = 40.00	00	OK - STEEL HSS IS COMP	ACT				
		(MAXIMUM ALLOWED) d / t = 56.74	45						
- COM									
0.001		$C_2 = 0.85$	A150 51 L011	ICATION)		FOR RCFT SEC	TIONS		
		C <sub>3</sub> = 0.79				EQUATION (I2-1	7)		
		P <sub>o</sub> = 2,965	9 kips			EQUATION (I2-1	5)		
	XX-AXIS (EI)	) <sub>eff</sub> = 99,099,314 kip-in <sup>2</sup>	YY-AXIS	(EI) <sub>eff</sub> = 99,099,314 kip-ir	1 <sup>2</sup>	EQUATION (I2-1	6)		
		K <sub>x</sub> = 1.657 P = 14.638 kins		$K_y = 1.657$ P = 14.638 kins		EOUATION (12-4	1		
	Column Slenderness,	$\alpha = 0.450$	olumn Slende	mess, $\alpha = 0.450$		EQUATION (I2-2)	)		
	I	P <sub>n</sub> = 2,727 kips		$P_n = 2,727 \text{ kips}$		EQUATIONS (12-	7, I2-8, I2-9)		
		$\phi_c = 0.75$ CONTROLL	ING	$\phi_c = 0.75$	,	SECTION 14, p. l	-16		
	¢	P <sub>n</sub> = 2,046 kips 2,046 kips	S	$\phi P_n = 2,046 \text{ kips}$	]				
o FLEX	UBAL STRENGTH (P	ER SECTION 14 OF the DRAFT 2005 AISC SI	PECIFICATIO	N AND EQUATION C-14-1 C	)F THE 1994	LRFD)			
<b>XX-AXIS</b> $c_r = 0.0$ in <b>YY-AXIS</b> $c_r = 0.0$ in									
WIDTH, h <sub>2</sub> = 20.0 in WIDTH, h <sub>2</sub> = 20.0 in									
$M_n = 12,629 \text{ kip-in}$ $M_n = 12,629 \text{ kip-in}$									
$\phi_{\rm b} = 0.90$ $\phi_{\rm b} = 0.90$ $\phi_{\rm M_{-}} = 11.366  {\rm kip-in}$ $\phi_{\rm M_{-}} = 11.366  {\rm kip-in}$									
• AXIAL LOAD AT ANCHOR POINT 'C' (PER SECTION 4 & TABLE C-11.1 OF the DRAFT 2005 AISC COMMENTARY)									
0.000		C = 1,224	4 kips	C	= Pc FROM	TABLE C-I1.1 FC	R RECTANGULAR HSS		
	Column Slenderness,	$\alpha = 0.450$ Co	olumn Slende	mess, $\alpha = 0.450$ $\Delta C = C_1 = 1.125$ kins					
	10 -	$\phi_c = 0.75$ CONTROLL	ING	$\phi_{c} = 0.75$	-				
$C_d = \phi_c C_k = 844 \text{ kips} \qquad \qquad$									

## 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 \  \, \star
                         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
                                        'Mcx = fbMnx per LRFD
Dim Mcx As Double
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

```
End Sub
```

'Only 6 load combinations are analyzed

```
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("AD" & 6 + J) = FactoredLoadsMatrix(J, 16)
Range("AD" & 6 + J) = FactoredLoadsMatrix(J, 17)
Range("AE" & 6 + J) = FactoredLoadsMatrix(J, 18)
Next J
Next J
```

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)
        \label{eq:columnStrengthsMatrix(J, 1 + (I - 1) * 6) = Range("AJ51") \\ ColumnStrengthsMatrix(J, 2 + (I - 1) * 6) = Range("Z51") \\ \end{array}
                                                                            'Min_Pa
                                                                            '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)
        'Mux
                                                             'Muy
        'Per Eqn's (C-I4-la) and (C-I4-lb) 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)
Range("AK" & 6 + J) = ColumnCompactnessCheckMatrix(I)
                                                                 'Column Steel Area Check result
                                                                 '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).

$$A_{s} = 2t(b+d-4r) + \pi(2r-t)$$
 (K-1)

$$I_{x} = (b-2r)\frac{t^{3}}{6} + t(b-2r)\frac{(d-t)^{2}}{2} + t\frac{(d-2r)^{3}}{6} + 4\left[r^{4} - (r-t)^{4}\left[\frac{\pi}{16} - \frac{4}{9\pi}\right] + \dots$$

$$\dots + \pi r^{2}\left[\frac{(d-2r)}{2} + \frac{4r}{3\pi}\right]^{2} - \pi (r-t)^{2}\left[\frac{(d-2r)}{2} + \frac{4(r-t)}{3\pi}\right]^{2}$$
(K-2)

$$Z_{x} = t(b-2r)(d-t) + \frac{t}{2}(d-2r)^{2} + \pi r^{2} \left[\frac{d}{2} - r + \frac{4r}{3\pi}\right] - \dots$$
  
...-  $\pi (r-t)^{2} \left[\frac{d}{2} - r + \frac{4(r-t)}{3\pi}\right]$  (K-3)

Section No.	HSS NAME	t <sub>NOM</sub> (in)	DEPTH d (in)	WIDTH b (in)	t <sub>DES</sub> (in)	Outside Corner Radius r (in)	As (in²)	lx (in <sup>4</sup> )	Zx (in <sup>3</sup> )
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

						Outside			
Section		t <sub>NOM</sub>	DEPTH	WIDTH	t <sub>DES</sub>	Corner	As	Ix	Zx
No.	HSS NAME	(in)	d (in)	D (in)	(in)	Radius r	(in²)	(in <sup>4</sup> )	(in <sup>3</sup> )
			()	(,		(in)			
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 <sub>NOM</sub> (in)	DEPTH d (in)	WIDTH b (in)	t <sub>DES</sub> (in)	Outside Corner Radius r (in)	As (in²)	lx (in <sup>4</sup> )	Zx (in <sup>3</sup> )
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 Eleff

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,  $\eta$ , 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:

$$Z_{s} \approx 1.4d^{2}t \tag{L-1}$$

$$I_{s} \approx \frac{Z_{s}d}{2.4} \cong 0.6 \text{td}^{3}$$
 (L-2)

• For the concrete portion of the RCFT:

$$I_c \approx \frac{d^4}{12} - I_s \tag{L-3}$$

### □ Approximating El<sub>eff</sub>

$$EI_{eff} \approx E'_{c} \frac{d^{4}}{12} + 0.6d^{3}t(E_{s} - E'_{c})$$
 (L-4)

Where: 
$$E'_{c} = \min\left(0.6 + \frac{8t}{d}, 0.9\right) E_{c}$$
 (L-5)

### **Estimating the Required Eleff**

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,  $\eta$ , for the building. Reference Table 6.3.14 for average values of  $\eta$  based on the design values of  $F_{yc}$  and  $f'_{c}$ . Once the total required  $EI_{eff}$  for a story,  $\Sigma EI_{eff}$ , is estimated a required value of  $EI_{eff}$  for a column can be determined by taking the average of  $\Sigma EI_{eff}$  for the story.

$$\left(\Sigma EI_{eff}\right)_{i} = \left[\eta + \left(\frac{H}{2L}\right)\right]_{i} \left[\frac{VLH^{2}}{6\Delta}\right]_{i}$$
(L-6)

$$\left(\mathrm{EI}_{\mathrm{eff}}\right)_{\mathrm{ave}} = \frac{\left(\Sigma \mathrm{EI}_{\mathrm{eff}}\right)_{\mathrm{i}}}{\mathrm{B} + 1} \tag{L-7}$$

$$EI_{eff} = (EI_{eff})_{ave}$$
(L-8)

Where: $\eta = (desired)$  flexural rigidity ratio (Table 6.3.14 or Equation 6.3-13)H = story heightL = average girder length in the storyV = design story shear

- $\Delta$  = 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,  $\eta$ , of the building to a set of baseline values of  $\eta$ . 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  $\eta$  is determined for a particular pair of  $F_{yc}$  and  $f'_c$ , the actual value of  $\eta$  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  $\eta$  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  $F_{vc}$  and  $f'_{c}$  for each of the three different d/t categories) five buildings were designed using the same value of  $F_{vc}$  and  $f'_{c}$ . Once their column and girder sizes were chosen their individual  $\eta$  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 18story building layout. The mean  $\eta$  value from these five building  $\eta$  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,  $\Delta_i$ , 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.

$$Z_{g} < \frac{Z_{c}F_{yc}}{1.1R_{y}F_{yg}}$$
(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.

$$K_{col} = \frac{EI_{eff}}{H}$$
(M-2)

- 7. Calculate the story total flexural stiffness,  $\Sigma K_{col}$ .
- 8. Calculate the minimum required (elastic) story stiffness,  $K_{i}$ .

$$\mathbf{K}_{i} = \frac{\mathbf{V}_{i}}{\Delta_{i}} \tag{M-3}$$

- 9. Calculate  $\eta$  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  $\approx$  80 category).
- 11. Assign the  $\eta$  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  $\eta$ .
- 12. Calculate the maximum allowed girder depth,  $d_g$ , using a modified version of Equation 6.3-3 as shown in Equation M-4.

$$d_{g} = \left(\frac{B+1}{B} \int \left[ \left(\frac{d_{c}}{\eta}\right) \left(\frac{R_{y}F_{yg}}{F_{yc}}\right) \left(\frac{0.6EI_{eff}}{E_{s}I_{c_{s}}}\right) \right]_{i}$$
(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  $\eta$  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           Building         Final Section Sizes and Properties         Flexural Rigidity Ratio, η           Building         Building         Maximum         Flexural Rigidity Ratio, η											
			Building		Final Sectio	n Sizes and Prope	rties	Flexural Rig	idity Ratio, η			
d/t	F <sub>ye</sub> (ksi)	f'c (ksi)	Geometry and Loading Based on Design No.	Story	Suggested Column	Maximum Allowed Z <sub>0</sub> (in <sup>3</sup> )	Suggested Girder	Story Value	Building Mean Value			
			ЗA	3 2 1	HSS 15 x 15 x 0.75 HSS 18 x 18 x 0.75 HSS 19 x 19 x 0.75	90 259 288	W18x40 W24x94 W30X90	0.64 0.74 0.74	0.71			
			3D	3 2 1	HSS 15 x 15 x 0.75 HSS 20 x 20 x 0.75 HSS 21 x 21 x 0.75	90 319 352	W18x40 W30X99 W30X108	0.60 0.90	0.78			
			3F	3	HSS 11 x 11 x 0.75 HSS 15 x 15 x 0.75	49 180	W14x30 W24X68	0.44 0.73	0.63			
				9	HSS 16 x 16 x 0.75 HSS 12 x 12 x 0.75 HSS 15 x 15 x 0.75	204 58 180	W24X76 W14x34 W24X68	0.73				
				7	HSS 17 x 17 x 0.75	231	W24X84	0.66				
			94	6	HSS 19 x 19 x 0.75	288	W30X90	0.83	0.75			
				4	HSS 20 x 20 x 0.75	319	W30X99	0.80				
				3	HSS 21 x 21 x 0.75 HSS 21 x 21 x 0.75	352	W30X108 W30X108	0.91				
LOW	46	4		1	HSS 21 x 21 x 0.75	352	W30X108	0.87				
				18 17	HSS 9 x 9 x 0.625 HSS 11 x 11 x 0.75	27 97	W12x16 W21x44	0.45				
				16	HSS 12 x 12 x 0.75	115	W18X55	0.51				
				15 14	HSS 14 x 14 x 0.625 HSS 14 x 14 x 0.75	130 156	W21x57 W24X62	0.62				
				13	HSS 15 x 15 x 0.75	180	W24X68	0.68				
				12	HSS 16 x 16 x 0.75 HSS 16 x 16 x 0.75	204	W24X76	0.80				
			18A	10	HSS 17 x 17 x 0.75	231	W24X84	0.87	0.77			
				9 8	HSS 17 x 17 x 0.75 HSS 17 x 17 × 0.75	231	W24X84	0.82	57			
				7	HSS 18 x 18 x 0.75	259	W24x94	0.97				
				6	HSS 18 x 18 x 0.75 HSS 18 x 18 × 0.75	259	W24x94	0.95				
				4	HSS 18 x 18 x 0.75	259	W24x94	0.93				
				3	HSS 18 x 18 x 0.75	259	W24x94	0.91				
				1	HSS 18 x 18 x 0.75	259	W24x94 W24x94	0.91				
			9B	3	HSS 13 x 13 x 0.625	98	W21x44	0.33	0.39			
				1	HSS 16 x 16 x 0.75	355	W30X108	0.40	0.00			
			ЗE	3 2	HSS 13 x 13 x 0.75 HSS 16 x 16 x 0.75	118 355	W18X55 W30X108	0.36	0.44			
				1	HSS 18 x 18 x 0.75	450	W30X116	0.55				
			3G	2	HSS 10 x 10 x 0.75 HSS 13 x 13 x 0.75	235	W18x35 W24X84	0.29	0.39			
					1	HSS 14 x 14 x 0.75	272	W18x119 W18x35	0.46			
				8	HSS 13 x 13 x 0.75	235	W24X84	0.33				
					7	HSS 15 x 15 x 0.625 HSS 16 x 16 x 0.75	260	W24x94 W30X108	0.39			
			9B	5	HSS 16 x 16 x 0.75	16 x 16 x 0.75 355 W30X108	0.40	0.41				
				4	HSS 17 x 17 x 0.75	401	W30X116	0.49				
				2	HSS 17 x 17 x 0.75	401	W30X116	0.43				
$\leq$ 2.26 $\sqrt{(E/Fy)}$	80	16		1	HSS 17 x 17 x 0.75	401	W30X116 W14x22	0.43				
				17	HSS 10 x 10 x 0.625	116	W18X55	0.31				
				16 15	HSS 11 x 11 x 0.75 HSS 12 x 12 x 0.75	169 200	W21X68 W24X76	0.37				
					14	HSS 13 x 13 x 0.625	195	W24X68	0.41			
				13 12	HSS 13 x 13 x 0.75 HSS 14 x 14 x 0.625	235 227	W24X84 W24X84	0.39 0.43				
				11	HSS 14 x 14 x 0.75	272	W18x119	0.45				
			18B	10 9	HSS 14 x 14 x 0.75 HSS 15 x 15 x 0.625	272 260	W18x119 W24x94	0.41	0.43			
				8	HSS 15 x 15 x 0.75	313	W30X99	0.53				
				7	HSS 15 x 15 x 0.75 HSS 15 x 15 x 0.75	313 313	W30X99 W30X99	0.50				
				5	HSS 15 x 15 x 0.75	313	W30X99	0.48				
				4	HSS 15 x 15 x 0.75 HSS 15 x 15 x 0.75	313 313	W30X99 W30X99	0.47				
				2	HSS 15 x 15 x 0.75 HSS 15 x 15 x 0.75	313 313	W30X99 W30X99	0.46				
				3	HSS 19 x 19 x 0.25	84	W18x40	1.07				
	80	16	3C	2	HSS 24 x 24 x 0.3125 HSS 24 x 24 x 0.3125	333 333	W30X99 W30X99	1.78	1.43			
		1		9	HSS 18 x 18 x 0.25	47	W14x26	1.37				
				8 7	HSS 24 x 24 x 0.3125 HSS 24 x 24 x 0.3125	208 208	W24X76 W24X76	2.70 1.98				
				6	HSS 27 x 27 x 0.375	316	W30X99	2.78				
			aC	5	HSS 28 x 28 x 0.375 HSS 28 x 28 x 0.375	340 340	W30X99 W30X99	2.81 2.57	2.37			
				3	HSS 28 x 28 x 0.375	340	W30X99	2.43				
				2	HSS 28 x 28 x 0.375 HSS 28 x 28 x 0.375	340 340	W30X99 W30X99	2.36 2.33				
				18	HSS 12 x 12 x 0.25	21	W12x16	1.03				
- 00				1/	HSS 18 x 18 x 0.25	74 94	W16x40 W18x40	1.64				
~ 00	50	16		15	HSS 19 x 19 x 0.25	104	W18X50	1.69				
	50	16		14	HSS 21 x 21 x 0.3125	144	W21X62 W24X62	2.03				
				12	HSS 22 x 22 x 0.3125	175	W21X68	2.20				
			180	11	HSS 24 x 24 x 0.3125	191 208	W24X68 W24X76	2.42	0.40			
			100	9	HSS 24 x 24 x 0.3125	208	W24X76	2.55	2.13			
				8 7	HSS 24 x 24 x 0.3125	208	W24X76	2.45				
				6	HSS 24 x 24 x 0.3125	208	W24X76	2.32				
				5 4	HSS 24 x 24 x 0.3125	208	W24X/6 W24X76	2.28				
				3	HSS 24 x 24 x 0.3125	208	W24X76	2.24				
				2	HSS 24 x 24 x 0.3125	208	W24X/6 W24X76	2.24				

		Member	Sizes that Wer	e Used to Calibra	te ¶ at Base L	Line Design Values 1, 2, and	d 3 (Per Figure 6.	3.1) for Low d/t ratio	s			
				0.11		Final Section	n Sizes and Prope	arties	Flexural Rig	idity Ratio, η		
dit	η Base Line Number	F <sub>yc</sub> (ksi)	f'c (ksi)	Building Geometry and Loading Based on Design No.	Story	Suggested Column	Maximum Allowed Z <sub>g</sub> (in <sup>3</sup> )	Suggested Girder	Story Value	Building Mean Value		
				ЗA	3 2 1	HSS 15 x 15 x 0.75 HSS 18 x 18 x 0.75 HSS 19 x 19 x 0.75	90 259 288	W18x40 W24x94 W30X90	0.64 0.74 0.74	0.71		
				3D	3	HSS 15 x 15 x 0.75 HSS 20 x 20 x 0.75	90 319	W18x40 W30X99	0.60	0.78		
				3F	3	HSS 21 x 21 x 0.75 HSS 11 x 11 x 0.75 HSS 15 x 15 x 0.75	49 180	W14x30 W24X68	0.84	0.63		
					1	HSS 16 x 16 x 0.75 HSS 12 x 12 x 0.75	204 58	W24X76 W14x34	0.73			
					8	HSS 15 x 15 x 0.75	180	W24X68 W24X84	0.56			
					6	HSS 19 x 19 x 0.75	288	W30X90	0.83			
				9A	5 4	HSS 20 x 20 x 0.75 HSS 20 x 20 x 0.75	319 319	W30X99 W30X99	0.88	0.75		
					3	HSS 21 x 21 x 0.75	352	W30X108	0.91			
		40			1	HSS 21 x 21 x 0.75 HSS 21 x 21 x 0.75	352	W30X108 W30X108	0.88			
		40			18	HSS 9 x 9 x 0.625	27	W12x16	0.45			
					16	HSS 12 x 12 x 0.75	115	W18X55	0.51			
					15 14	HSS 14 x 14 x 0.625 HSS 14 x 14 x 0.75	130 156	W21x57 W24X62	0.62			
					13	HSS 15 x 15 x 0.75	180	W24X68	0.68			
					12	HSS 16 x 16 x 0.75 HSS 16 x 16 x 0.75	204 204	W24X76 W24X76	0.80			
				18A	10	HSS 17 x 17 x 0.75	231	W24X84	0.87	0.77		
					9	HSS 17 x 17 x 0.75 HSS 17 x 17 x 0.75	231 231	W24X84 W24X84	0.82			
					7	HSS 18 x 18 x 0.75	259	W24x94	0.97			
					5	HSS 18 x 18 x 0.75 HSS 18 x 18 x 0.75	259 259	W24x94 W24x94	0.95			
					4	HSS 18 x 18 x 0.75	259	W24x94	0.92			
					3	HSS 18 x 18 x 0.75 HSS 18 x 18 x 0.75	259 259	W24x94 W24x94	0.91			
					1	HSS 18 x 18 x 0.75	259	W24x94	0.91			
				3B	2	HSS 18 x 18 x 0.75	259	W24x94	0.85	0.81		
				1	HSS 19 x 19 x 0.75 HSS 15 x 15 x 0.75	288	W30X90 W18x40	0.86				
			3E	2	HSS 19 x 19 x 0.75	288	W30X90	0.84	0.84			
						1	HSS 21 x 21 x 0.75 HSS 11 x 11 x 0.75	352	W30X108 W14x30	0.99		
				3G	2	HSS 15 x 15 x 0.75	180	W24X68	0.83	0.72		
					9	HSS 16 x 16 x 0.75 HSS 12 x 12 x 0.75	204	W24X/6 W14x34	0.83			
							8	HSS 15 x 15 x 0.75	180	W24X68	0.63	
					6	HSS 18 x 18 x 0.75	75 231 75 259	W24x94	0.76	0.81		
					10		9B 5 HSS 19 x 19 x 0.75	288 319	W30X90 W30X99	0.83	0.81	
LOW			10	10			3	HSS 20 x 20 x 0.75	319	W30X99	0.87	
							2	HSS 21 x 21 x 0.75 HSS 21 x 21 x 0.75	352 352	W30X108 W30X108	1.03	
	2	46					18	HSS 8 x 8 x 0.75	26	W12x16	0.31	
					17	HSS 11 x 11 x 0.625 HSS 12 x 12 x 0.75	115	W18X40 W18X55	0.51			
					15	HSS 13 x 13 x 0.75	135	W24X55	0.59			
					13	HSS 15 x 15 x 0.75	180	W24X68	0.78			
					12	HSS 16 x 16 x 0.75 HSS 16 x 16 x 0.75	204	W24X76 W24X76	0.92			
				18B	10	HSS 17 x 17 x 0.75	231	W24X84	1.01	0.83		
					9	HSS 17 x 17 x 0.75 HSS 17 x 17 x 0.75	231 231	W24X84 W24X84	0.95			
					7	HSS 17 x 17 x 0.75	231	W24X84	0.87			
					5	HSS 18 x 18 x 0.75	259	W24x94	1.07			
1					4	HSS 18 x 18 x 0.75 HSS 18 x 18 x 0.75	259 259	W24x94 W24x94	1.06			
					2	HSS 18 x 18 x 0.75	259	W24x94	1.05			
					3	HSS 18 x 18 x 0.75 HSS 15 x 15 x 0.75	259 90	W24x94 W18x40	1.05			
1				3A	2	HSS 18 x 18 x 0.75 HSS 19 x 19 x 0.75	259 288	W24x94 W30X90	0.93 0.95	0.89		
				3D	3 2 1	HSS 15 x 15 x 0.75 HSS 19 x 19 x 0.75 HSS 21 x 21 x 0.75	90 288 352	W18x40 W30X90 W30X108	0.73 0.93 1.09	0.92		
				3F	3 2 1	HSS 11 x 11 x 0.75 HSS 15 x 15 x 0.75 HSS 16 x 16 x 0.75	49 180 204	W14x30 W24X68 W24X76	0.52 0.91 0.91	0.78		
					9	HSS 12 x 12 x 0.75 HSS 15 x 15 x 0.75	58 180	W14x34 W24X68	0.48			
					7	HSS 17 x 17 x 0.75 HSS 18 x 18 x 0.75	231	W24X84 W24x94	0.82			
	3 46 1		9A	5	HSS 19 x 19 x 0.75	288	W30X90	0.91	0.89			
				3	HSS 20 x 20 x 0.75	319	W30X99	0.96				
		16		1	HSS 21 x 21 x 0.75	352	W30X108	1.12				
		10		17	HSS 11 x 11 x 0.625	80	W18x40	0.56				
				15	HSS 13 x 13 x 0.75	135	W24X55	0.64				
				14	HSS 14 x 14 x 0.75 HSS 15 x 15 x 0.75	156	W24X62 W24X68	0.72				
1				12	HSS 16 x 16 x 0.75 HSS 16 x 16 x 0.75	204 204	W24X76 W24X76	1.00				
			18A	10 9	HSS 16 x 16 x 0.75 HSS 17 x 17 x 0.75	204 231	W24X76 W24X84	0.83	0.87			
1				8	HSS 17 x 17 x 0.75 HSS 17 x 17 x 0.75	231 231	W24X84 W24X84	0.99				
				6	HSS 17 x 17 x 0.75	231	W24X84	0.93				
1					4	HSS 17 x 17 x 0.75	231	W24X84	0.91			
					3	HSS 18 x 18 x 0.75 HSS 18 x 18 x 0.75	259 259	W24x94 W24x94	1.16 1.15			
			l I	1	1	HSS 18 x 18 x 0.75	259	W24x94	1.15	1		

	Member Sizes that Were Used to Calibrate <b>1</b> at Base Line Design Values 4, 5, and 6 (Per Figure 6.3.1) for Low dit ratios																		
				Building		Final Section	n Sizes and Prope	erties	Flexural Rig	idity Ratio, η									
d/t	η Base Line Number	F <sub>yc</sub> (ksi)	f'c (ksi)	Geometry and Loading Based on Design No.	Story	Suggested Column	Maximum Allowed Z <sub>g</sub> (in <sup>3</sup> )	Suggested Girder	Story Value	Building Mean Value									
				3B	3 2 1	HSS 14 x 14 x 0.75 HSS 17 x 17 x 0.75 HSS 18 x 18 x 0.75	107 316 355	W18X50 W30X99 W30X108	0.47 0.58 0.59	0.54									
				3E	3 2 1	HSS 14 x 14 x 0.75 HSS 18 x 18 x 0.75 HSS 19 x 19 x 0.75	107 355 395	W18X50 W30X108 W30X116	0.43 0.57 0.55	0.52									
				3G	3 2 1	HSS 11 x 11 x 0.625 HSS 14 x 14 x 0.75 HSS 15 x 15 x 0.75	55 214 246	W14x34 W12x136 W27X84	0.34 0.52 0.53	0.46									
					9	HSS 12 x 12 x 0.625 HSS 14 x 14 x 0.75	66 214	W14x34 W12x136	0.33										
					7	HSS 16 x 16 x 0.75	280	W18x119	0.50										
				9B	5	HSS 17 x 17 x 0.75 HSS 18 x 18 x 0.75	315	W30X99 W30X108	0.51	0.50									
					4	HSS 18 x 18 x 0.75 HSS 19 x 19 x 0.75	355 395	W30X108 W30X116	0.50										
					2	HSS 19 x 19 x 0.75	395	W30X116	0.57										
	4	63	4		1	HSS 19 x 19 x 0.75 HSS 9 x 9 x 0.5	395	W30X116 W12x22	0.56										
					17	HSS 10 x 10 x 0.75	109	W12x72 W21x57	0.34										
					15	HSS 13 x 13 x 0.625	154	W24X62	0.42										
					14	HSS 13 x 13 x 0.75 HSS 14 x 14 x 0.75	185 214	W24X68 W12x136	0.39										
					12	HSS 15 x 15 x 0.75	246	W27X84	0.59										
				18B	10	HSS 15 x 15 x 0.75	246	W27X84	0.52	0.49									
				100	9	HSS 16 x 16 x 0.75 HSS 16 x 16 x 0.75	280 280	W18x119 W18x119	0.61	0.45									
					7	HSS 16 x 16 x 0.75	280	W18x119	0.56										
					5	HSS 16 x 16 x 0.75 HSS 16 x 16 x 0.75	280	W18x119 W18x119	0.54										
					4	HSS 16 x 16 x 0.75	280	W18x119 W18x119	0.52										
					2	HSS 16 x 16 x 0.75	280	W18x119	0.51										
					1	HSS 16 x 16 x 0.75 HSS 13 x 13 x 0.75	280 93	W18x119 W18x40	0.51										
				ЗA	2	HSS 16 x 16 x 0.75 HSS 17 x 17 x 0.75	280 316	W18x119 W30X99	0.50 0.52	0.46									
				3D	3	HSS 14 x 14 x 0.75 HSS 18 x 18 x 0.75	107	W18X50 W30X108	0.49	0.60									
					1	HSS 19 x 19 x 0.75	395	W30X116	0.64										
				3F	3	HSS 11 x 11 x 0.625 HSS 14 x 14 x 0.75	55 214	W14x34 W12x136	0.39 0.59	0.53									
					1	HSS 15 x 15 x 0.75	246	W27X84 W18x35	0.61										
					8	HSS 14 x 14 x 0.75	214	W12x136 W18x119 W30X99	0.45	0.50									
					7	HSS 16 x 16 x 0.75 HSS 17 x 17 x 0.75	280 316		0.57 0.59										
			9A 5 4 3 2	HSS 18 x 18 x 0.75	355	W30X108 W30X108	0.65	0.56											
LOW					10	10	10							3	HSS 18 x 18 x 0.75	355	W30X108	0.54	
	-	c0							2	HSS 19 x 19 x 0.75 HSS 19 x 19 x 0.75	395 395	W30X116 W30X116	0.67						
	5	63	10		18	HSS 8 x 8 x 0.75 HSS 10 x 10 x 0.75	35	W14x22 W12y72	0.31										
					16	HSS 11 x 11 x 0.75	133	W18x65	0.34										
					15	HSS 13 x 13 x 0.625 HSS 13 x 13 x 0.75	154 185	W24X62 W24X68	0.49										
					13	HSS 14 x 14 x 0.75	214	W12x136	0.55										
					11	HSS 15 x 15 x 0.75	246	W27X84	0.60										
				18A	10	HSS 15 x 15 x 0.75 HSS 16 x 16 x 0.75	246 280	W27X84 W18x119	0.55	0.54									
					8	HSS 16 x 16 x 0.75 HSS 16 x 16 x 0.75	280 280	W18x119 W18x119	0.67										
					6	HSS 16 x 16 x 0.75	280	W18x119	0.63										
1					4	HSS 16 x 16 x 0.75 HSS 16 x 16 x 0.75	280	W18x119 W18x119	0.61										
1					3 2	HSS 16 x 16 x 0.75 HSS 16 x 16 x 0.75	280 280	W18x119 W18x119	0.60										
1					1	HSS 16 x 16 x 0.75	280	W18x119 W18x40	0.60										
				3B	2	HSS 16 x 16 x 0.75 HSS 17 x 17 x 0.75	280 316	W18x119 W30X99	0.55	0.51									
1				3E	3	HSS 14 x 14 x 0.625 HSS 17 x 17 x 0.75	90 316	W18x40 W30X99	0.46 0.56	0.58									
1					1 3	HSS 19 x 19 x 0.75 HSS 11 x 11 x 0.625	395 55	W30X116 W14x34	0.71										
1				3G	2	HSS 14 x 14 x 0.625 HSS 15 x 15 x 0.75	179 246	W24X68 W27X84	0.55	0.55									
1					9 8	HSS 11 x 11 x 0.75 HSS 14 x 14 x 0.75	67 214	W18x35 W12x136	0.31 0.49										
1					7	HSS 15 x 15 x 0.75 HSS 17 x 17 x 0.75	246 316	W27X84 W30X99	0.46 0.65										
1	6 63			9B	5 4	HSS 17 x 17 x 0.75 HSS 18 x 18 x 0.75	316 355	W30X99 W30X108	0.55 0.64	0.57									
1					3 2	HSS 18 x 18 x 0.75 HSS 19 x 19 x 0.75	355 395	W30X108 W30X116	0.60 0.74										
1		16		1	HSS 19 x 19 x 0.75 HSS 8 x 8 x 0.625	395 29	W30X116 W12x16	0.73											
1				17 16	HSS 10 x 10 x 0.75 HSS 11 x 11 x 0.75	109 133	W12x72 W18x65	0.39 0.37											
1				15 14	HSS 12 x 12 x 0.75 HSS 13 x 13 x 0.75	158 185	W24X62 W24X68	0.41 0.49											
1				13 12	HSS 14 x 14 x 0.75 HSS 14 x 14 x 0.75	214 214	W12x136 W12x136	0.60 0.51											
1				11 10	HSS 15 x 15 x 0.75 HSS 15 x 15 x 0.75	246 246	W27X84 W27X84	0.66											
1			18B	9 8	HSS 15 x 15 x 0.75 HSS 16 x 16 x 0.75	246 280	W27X84 W18x119	0.56	0.57										
1				7	HSS 16 x 16 x 0.75 HSS 16 x 16 x 0.75	280	W18x119 W18x119	0.71											
1				5	HSS 16 x 16 x 0.75	280	W18x119	0.68											
1				3	HSS 16 x 16 x 0.75	280	W18x119	0.66											
					2	HSS 16 x 16 x 0.75 HSS 16 x 16 x 0.75	280 280	W18x119 W18x119	0.66										

		Members	Sizes that Wer	e Used to Calibra	te ¶ at Base I	Line Design Values 7, 8, and	d 9 (Per Figure 6.	3.1) for Low d/t ratio	s					
				Building		Final Section	on Sizes and Prope	arties	Flexural Rig	idity Ratio, η				
d/t	η Base Line Number	F <sub>yc</sub> (ksi)	f'c (ksi)	Geometry and Loading Based on Design No.	Story	Suggested Column	Maximum Allowed Z <sub>9</sub> (in <sup>3</sup> )	Suggested Girder	Story Value	Building Mean Value				
				зA	3 2 1	HSS 13 x 13 x 0.75 HSS 16 x 16 x 0.75 HSS 17 x 17 x 0.75	118 355 401	W18X55 W30X108 W30X116	0.32 0.43 0.45	0.40				
				3D	3 2	HSS 13 x 13 x 0.75 HSS 17 x 17 x 0.75 HSS 18 x 19 x 0.75	118 401 450	W18X55 W30X116 W20X116	0.30 0.44	0.39				
				3F	3	HSS 10 x 10 x 0.75 HSS 11 x 11 x 0.625 HSS 13 x 13 x 0.75	70 235	W18x35 W24X84	0.34	0.34				
					1 9	HSS 14 x 14 x 0.75 HSS 11 x 11 x 0.75	272 85	W18x119 W18x40	0.36					
					8	HSS 13 x 13 x 0.75	235	W24X84	0.27					
					6	HSS 16 x 16 x 0.75	355	W30X99 W30X108	0.38					
				9A	5	HSS 17 x 17 x 0.75	401	W30X116 W30X116	0.43	0.38				
					3	HSS 18 x 18 x 0.75	450	W30X116	0.46					
	_				2	HSS 18 x 18 x 0.75 HSS 18 x 18 x 0.75	450 450	W30X116 W30X116	0.44					
		80	4		18	HSS 9 x 9 x 0.5	38	W12x26	0.33					
					17	HSS 10 x 10 x 0.625 HSS 11 x 11 x 0.75	116	W18X55 W21X68	0.25					
					15	HSS 12 x 12 x 0.75	200	W24X76	0.33					
					13	HSS 13 x 13 x 0.75	235	W24X84	0.39					
					12	HSS 14 x 14 x 0.75	272	W18x119 W18x119	0.40					
				18A	10	HSS 15 x 15 x 0.75	313	W30X99	0.47	0.40				
					9	HSS 15 x 15 x 0.75 HSS 15 x 15 x 0.75	313	W30X99 W30X99	0.44					
					7	HSS 15 x 15 x 0.75	313	W30X99	0.39					
					6	HSS 15 x 15 x 0.75 HSS 16 x 16 x 0.625	313 296	W30X99 W30X90	0.37					
					4	HSS 16 x 16 x 0.75	355	W30X108	0.52					
					3	HSS 16 x 16 x 0.75 HSS 16 x 16 x 0.75	355 355	W30X108 W30X108	0.51					
					1	HSS 16 x 16 x 0.75	355	W30X108	0.51					
			3B	3	HSS 13 x 13 x 0.75 HSS 15 x 15 x 0.75	313	W18X55 W30X99	0.37	0.37					
					1	HSS 16 x 16 x 0.75	355	W30X108	0.38					
				3E	2	HSS 17 x 17 x 0.75	401	W30X116	0.54	0.45				
					1	HSS 18 x 18 x 0.75	450	W30X116	0.50					
				3G	2	HSS 13 x 13 x 0.75	235	W24X84	0.39	0.36				
					1	HSS 14 x 14 x 0.75 HSS 11 x 11 x 0.75	272	W18x119 W18x40	0.41					
					8	HSS 13 x 13 x 0.75	235	W24X84	0.31					
						7	HSS 15 x 15 x 0.75 HSS 16 x 16 x 0.75	313 355	W30X99 W30X108	0.42				
				9B	5	HSS 16 x 16 x 0.75	355 V 355 V	W30X108	0.36	0.41				
LOW					4	HSS 17 x 17 x 0.75 HSS 17 x 17 x 0.75	401 401	01 W30X116 01 W30X116	0.44					
					2	HSS 18 x 18 x 0.75	450	W30X116	0.52					
	8	80	10	10	10	10	10		18	HSS 18 x 18 x 0.75 HSS 8 x 8 x 0.625	450	W30X116 W14x22	0.51	
					17	HSS 10 x 10 x 0.625	116	W18X55	0.29					
					15	HSS 12 x 12 x 0.75	200	W24X76	0.38					
					14 13	HSS 13 x 13 x 0.625 HSS 13 x 13 x 0.75	195 235	W24X68 W24X84	0.36					
					12	HSS 14 x 14 x 0.75	272	W18x119	0.47					
				105	11	HSS 14 x 14 x 0.75 HSS 15 x 15 x 0.625	272 260	W18x119 W24x94	0.41 0.45					
				100	9	HSS 15 x 15 x 0.75	313	W30X99	0.51	0.40				
					7	HSS 15 x 15 x 0.75	313	W30X99 W30X99	0.48					
1					6	HSS 15 x 15 x 0.75 HSS 15 x 15 x 0.75	313 313	W30X99 W30X99	0.44					
					4	HSS 15 x 15 x 0.75	313	W30X99	0.42					
1					3	HSS 15 x 15 x 0.75 HSS 15 x 15 x 0.75	313 313	W30X99 W30X99	0.42					
1					1	HSS 15 x 15 x 0.75	313	W30X99	0.42					
				ЗA	2	HSS 15 x 13 x 0.625	313	W30X99	0.33	0.39				
				٩D	3	HSS 16 x 16 x 0.75 HSS 13 x 13 x 0.75 HSS 16 x 16 - 0.75	355 118 955	W30X108 W18X55	0.42	0.44				
					1 3	HSS 10 x 10 x 0.75 HSS 18 x 18 x 0.75 HSS 10 x 10 x 0.75	450 69	W30X105 W30X116 W18x35	0.55	0.44				
				3F	2	HSS 13 x 13 x 0.75 HSS 14 x 14 x 0.75	235 272	W24X84 W18x119	0.43 0.46	0.39				
					9	HSS 11 x 11 x 0.625 HSS 13 x 13 x 0.75	70 235	W18x35 W24X84	0.25 0.33					
					7	HSS 15 x 15 x 0.625 HSS 16 x 16 x 0.75	260 355	W24x94 W30X108	0.39					
				9A	5	HSS 16 x 16 x 0.75 HSS 17 x 17 x 0.75	355 401	W30X108 W30X116	0.40	0.41				
	9 80			3	HSS 17 x 17 x 0.75	401	W30X116	0.45						
		10		1	HSS 17 x 17 x 0.75	401	W30X116	0.43						
		10		17	HSS 10 x 10 x 0.625	116	W18X55	0.31						
				15	HSS 12 x 12 x 0.75	200	W21X68 W24X76	0.37						
1				14 13	HSS 13 x 13 x 0.625 HSS 13 x 13 x 0.75	195 235	W24X68 W24X84	0.41 0.39						
					12 11	HSS 14 x 14 x 0.625 HSS 14 x 14 x 0.75	227 272	W24X84 W18x119	0.43 0.45					
1				18A	10 9	HSS 14 x 14 x 0.75 HSS 15 x 15 x 0.625	272 260	W18x119 W24x94	0.41 0.48	0.43				
					8	HSS 15 x 15 x 0.75	313	W30X99	0.53	0.43				
1					6	HSS 15 x 15 x 0.75	313	W30X99	0.49					
					5	HSS 15 x 15 x 0.75 HSS 15 x 15 x 0.75	313 313	W30X99 W30X99	0.48 0.47					
					3	HSS 15 x 15 x 0.75 HSS 15 x 15 x 0.75	313	W30X99 W30X99	0.46					
1				1	1	HSS 15 x 15 x 0.75	313	W30X99	0.46	I				

	Member Sizes that Were Used to Calibrate η at Base Line Design Values 1, 2, and 3 (Per Figure 6.3.1) for dit \$ 2.26 ∜(E/Fy) Final Section Sizes and Properties Flexural Rigidity Ratio, η												
				D. 11.		Final Sectio	n Sizes and Prope	rties	Flexural Rig	idity Ratio, η			
d/t	η Base Line Number	F <sub>ye</sub> (ksi)	f'c (ksi)	Building Geometry and Loading Based on Design No.	Story	Suggested Column	Maximum Allowed Z <sub>0</sub> (in <sup>3</sup> )	Suggested Girder	Story Value	Building Mean Value			
				ЗA	3 2 1	HSS 21 x 21 x 0.375 HSS 25 x 25 x 0.5 HSS 27 x 27 x 0.5	88 333 388	W18x40 W30X99 W30X116	1.37 1.89 2.07	1.78			
				3D	3 2 1	HSS 21 x 21 x 0.375 HSS 27 x 27 x 0.5 HSS 28 x 28 x 0.5	88 388 417	W18x40 W30X116 W30X116	1.29 2.03 1.84	1.72			
				3F	3 2 1	HSS 17 x 17 x 0.3125 HSS 21 x 21 x 0.375 HSS 23 x 23 x 0.5	48 176 282	W14x30 W21X68 W18y119	1.36 1.63 2.25	1.75			
					9 8 7	HSS 23 x 23 x 0.3 HSS 18 x 18 x 0.375 HSS 21 x 21 x 0.375 HSS 23 x 23 x 0.5	65 176 282	W14x34 W21X68 W18x119	1.21 1.21 1.54				
				9A	6 5 4 3	HSS 26 x 26 x 0.5 HSS 27 x 27 x 0.5 HSS 28 x 28 x 0.5 HSS 28 x 28 x 0.5	360 388 417 417	W30X108 W30X116 W30X116 W30X116	1.98 1.99 2.08 1.97	1.75			
	1	46	4		2 1 18	HSS 28 x 28 x 0.5 HSS 28 x 28 x 0.5 HSS 13 x 13 x 0.3125	417 417 28	W30X116 W30X116 W12x16	1.91 1.88 1.21				
					17 16 15	HSS 17 x 17 x 0.3125 HSS 19 x 19 x 0.375 HSS 21 x 21 x 0.375	96 144 176	W21x44 W21X62 W21X68	1.65 1.94 2.18				
					14 13 12	HSS 21 x 21 x 0.375 HSS 22 x 22 x 0.5 HSS 23 x 23 x 0.5	176 257 282	W21X68 W24x94 W18x119	1.79 2.31 2.44				
				18A	11 10	HSS 23 x 23 x 0.5 HSS 24 x 24 x 0.5 HSS 25 x 25 x 0.5	282 307 333	W18x119 W21x122 W30X99	2.24 2.45 2.71	2.38			
					8	HSS 25 x 25 x 0.5 HSS 25 x 25 x 0.5 HSS 25 x 25 x 0.5	333 333	W30X99 W30X99	2.61				
					5 4	HSS 26 x 26 x 0.5 HSS 26 x 26 x 0.5 HSS 26 x 26 x 0.5	360 360 360	W30X108 W30X108 W30X108	2.86 2.82 2.79				
					3 2 1	HSS 26 x 26 x 0.5 HSS 26 x 26 x 0.5 HSS 26 x 26 x 0.5	360 360 360	W30X108 W30X108 W30X108	2.77 2.76 2.76				
				3B	3 2 1	HSS 21 x 21 x 0.375 HSS 25 x 25 x 0.5 HSS 27 x 27 x 0.5	88 333 388	W18x40 W30X99 W30X116	1.72 2.32 2.56	2.20			
				3E	3 2 1	HSS 21 x 21 x 0.375 HSS 27 x 27 x 0.5 HSS 28 x 28 x 0.5	88 388 417	W18x40 W30X116 W30X116	1.62 2.51 2.29	2.14			
				3G	3 2 1	HSS 17 x 17 x 0.3125 HSS 21 x 21 x 0.375 HSS 23 x 23 x 0.5	48 176 282	W14x30 W21X68 W18x119	1.72 2.06 2.75	2.18			
					9 8 7	HSS 18 x 18 x 0.375 HSS 21 x 21 x 0.375 HSS 23 x 23 x 0.5	65 176 282	W14x34 W21X68 W18x119	1.49 1.53 1.88				
				9B	6	HSS 25 x 25 x 0.5 HSS 27 x 27 x 0.5	333 388 417	W30X99 W30X116	2.11 2.47 2.59	2.14			
$\leq$ 2.26 $\sqrt{(E/Fy)}$					3 2	HSS 28 x 28 x 0.5 HSS 28 x 28 x 0.5 HSS 28 x 28 x 0.5	417 417 417	W30X116 W30X116	2.45				
	2	46	10		18	HSS 13 x 13 x 0.3125 HSS 17 x 17 x 0.3125	28 96	W12x16 W21x44	2.35 1.49 2.07				
					16 15 14	HSS 19 x 19 x 0.375 HSS 21 x 21 x 0.375 HSS 21 x 21 x 0.375	144 176 176	W21X62 W21X68 W21X68	2.40 2.72 2.25				
					13 12 11	HSS 21 x 21 x 0.375 HSS 22 x 22 x 0.5 HSS 23 x 23 x 0.5	176 257 282	W21X68 W24x94 W18x119	1.94 2.51 2.73				
				18B	10 9 8	HSS 24 x 24 x 0.5 HSS 24 x 24 x 0.5 HSS 25 x 25 x 0.5	307 307 333	W21x122 W21x122 W30X99	3.01 2.86 3.21	2.81			
					7	HSS 25 x 25 x 0.5 HSS 25 x 25 x 0.5	333 333	W30X99 W30X99	3.11 3.04				
					5 4 3	HSS 26 x 26 x 0.5 HSS 26 x 26 x 0.5 HSS 26 x 26 x 0.5	360 360 360	W30X108 W30X108 W30X108	3.48 3.44 3.42				
				~	2 1 3	HSS 26 x 26 x 0.5 HSS 26 x 26 x 0.5 HSS 21 x 21 x 0.375	360 360 88	W30X108 W30X108 W18x40	3.41 3.41 1.97	0.51			
				3A 3D	2 1 3 2	HSS 25 x 25 x 0.5 HSS 27 x 27 x 0.5 HSS 21 x 21 x 0.375 HSS 27 x 27 x 0.5	333 388 88 388	W30X99 W30X116 W18x40 W30X116	2.64 2.91 1.86 2.86	2.51			
				3F	1 3 2	HSS 28 x 28 x 0.5 HSS 17 x 17 x 0.3125 HSS 21 x 21 x 0.375	417 48 176	W30X116 W14x30 W21X68	2.62 1.98 2.37	2.49			
					1 9 8 7	HSS 23 x 23 x 0.5 HSS 18 x 18 x 0.375 HSS 21 x 21 x 0.375 HSS 23 x 23 x 0.5	282 65 176 282	W18x119 W14x34 W21X68 W18x119	3.11 1.69 1.75 2.13				
				9A	6 5 4	HSS 25 x 25 x 0.5 HSS 27 x 27 x 0.5 HSS 28 x 28 x 0.5	333 388 417	W30X99 W30X116 W30X116	2.39 2.81 2.95	2.43			
	3	46	16		3 2 1 18	HSS 28 x 28 x 0.5 HSS 28 x 28 x 0.5 HSS 28 x 28 x 0.5 HSS 13 x 13 x 0.3125	417 417 417 28	W30X116 W30X116 W30X116 W12x16	2.79 2.71 2.68 1.69				
					17 16 15	HSS 17 x 17 x 0.3125 HSS 19 x 19 x 0.375 HSS 21 x 21 x 0.375 HSS 21 x 21 x 0.375	96 144 176	W21x44 W21X62 W21X68 W21X68	2.38 2.74 3.12				
					13 12 11	HSS 21 x 21 x 0.375 HSS 21 x 21 x 0.375 HSS 22 x 22 x 0.5 HSS 23 x 23 x 0.5	176 257 282	W21X68 W24x94 W18x119	2.59 2.24 2.83 3.09				
				18A	10 9 8	HSS 24 x 24 x 0.5 HSS 24 x 24 x 0.5 HSS 25 x 25 x 0.5	307 307 333	W21x122 W21x122 W30X99	3.41 3.24 3.64	3.19			
					7 6 5	HSS 25 x 25 x 0.5 HSS 25 x 25 x 0.5 HSS 26 x 26 x 0.5	333 333 360	W30X99 W30X99 W30X108	3.53 3.46 3.95				
					4 3 2 1	HSS 26 x 26 x 0.5 HSS 26 x 26 x 0.5	360 360 360 360	W30X108 W30X108 W30X108 W30X108	3.91 3.89 3.88 3.88				

		Member Size	s that Were U	sed to Calibrate <b>η</b>	at Base Line	Design Values 4, 5, and 6 (	Per Figure 6.3.1)	for d/t ≤ 2.26 √(E/F	y)					
				Puilding		Final Sectio	n Sizes and Prope	rties	Flexural Rig	dity Ratio, η				
d/t	η Base Line Number	F <sub>ye</sub> (ksi)	f'c (ksi)	Geometry and Loading Based on Design No.	Story	Suggested Column	Maximum Allowed Z <sub>g</sub> (in <sup>3</sup> )	Suggested Girder	Story Value	Building Mean Value				
				3B	3 2 1	HSS 18 x 18 x 0.375 HSS 22 x 22 x 0.5 HSS 24 x 24 x 0.5	89 353 420	W18x40 W30X108 W30X116	0.73 1.16 1.33	1.07				
				3E	3 2	HSS 18 x 18 x 0.375 HSS 24 x 24 x 0.5	89 420	W18x40 W30X116	0.68	1.00				
				3G	3	HSS 15 x 15 x 0.3125 HSS 19 x 19 x 0.5	52 263	W14x30 W18x119	0.78	1.25				
					9	HSS 21 x 21 x 0.5 HSS 16 x 16 x 0.375	321	W30X99 W18x35	1.58					
					8	HSS 18 x 18 x 0.375 HSS 21 x 21 x 0.5	178 321	W24X68 W30X99	0.64					
				9B	6 5	HSS 22 x 22 x 0.5 HSS 24 x 24 x 0.5	353 420	W30X108 W30X116	1.05	1.01				
					4 3	HSS 24 x 24 x 0.5 HSS 24 x 24 x 0.5	420 420	W30X116 W30X116	1.16 1.09					
					2	HSS 24 x 24 x 0.5 HSS 24 x 24 x 0.5	420 420	W30X116 W30X116	1.05					
	4	63	4		18	HSS 12 x 12 x 0.3125	33	W12x22	0.85					
					16	HSS 17 x 17 x 0.375	158	W24X62	1.24					
					15 14	HSS 18 x 18 x 0.375 HSS 18 x 18 x 0.375	178 178	W24X68 W24X68	1.17 0.94					
					13 12	HSS 19 x 19 x 0.5 HSS 20 x 20 x 0.5	263 291	W18x119 W30X90	1.30					
					11	HSS 21 x 21 x 0.5	321	W30X99	1.57					
				18B	9	HSS 22 x 22 x 0.5	353	W30X99 W30X108	1.66	1.46				
					8	HSS 22 x 22 x 0.5 HSS 22 x 22 x 0.5	353 353	W30X108 W30X108	1.59 1.54					
					6	HSS 23 x 23 x 0.5	386	W30X116	1.79					
					4	HSS 23 x 23 x 0.5	386	W30X116	1.74					
					3	HSS 23 x 23 x 0.5 HSS 23 x 23 x 0.5	386 386	W30X116 W30X116	1.73 1.73					
					1	HSS 23 x 23 x 0.5 HSS 18 x 18 x 0.375	386 89	W30X116 W18x40	1.73					
				ЗA	2	HSS 22 x 22 x 0.5	353	W30X108	1.42	1.33				
					3	HSS 18 x 18 x 0.375	420	W18x40	0.86					
				3D	2 1	HSS 24 x 24 x 0.5 HSS 24 x 24 x 0.5	420 420	W30X116 W30X116	1.60 1.26	1.24				
				3F	3	HSS 15 x 15 x 0.3125 HSS 18 x 18 x 0.375	52 178	W14x30 W24X68	1.00	1.22				
			10		1	HSS 20 x 20 x 0.5	291	W30X90	1.58					
					9	HSS 16 x 16 x 0.375 HSS 18 x 18 x 0.375	70 178	W18x35 W24X68	0.92					
					7	HSS 20 x 20 x 0.5 HSS 22 x 22 x 0.5	291 353	W30X90 W30X108	1.09					
				9A	5	HSS 24 x 24 x 0.5	420	W30X116	1.57	1.23				
$\leq$ 2.26 $\sqrt{(E/Fy)}$				10	10	10	10	10		3	HSS 24 x 24 x 0.5	420	W30X116	1.35
	5	63							10	10	10		2	HSS 24 x 24 x 0.5 HSS 24 x 24 x 0.5
	0				18 17	HSS 11 x 11 x 0.3125 HSS 15 x 15 x 0.3125	28 103	W12x16 W18X50	0.71					
					16	HSS 17 x 17 x 0.375	158	W24X62	1.54					
					14	HSS 18 x 18 x 0.375	178	W24X68	1.19					
					13 12	HSS 19 x 19 x 0.5 HSS 20 x 20 x 0.5	263 291	W18x119 W30X90	1.57					
					11 10	HSS 20 x 20 x 0.5 HSS 21 x 21 x 0.5	291 321	W30X90 W30X99	1.57					
				18A	9	HSS 21 x 21 x 0.5	321	W30X99	1.69	1.70				
					7	HSS 22 x 22 x 0.5	353	W30X108	1.89					
					6 5	HSS 22 x 22 x 0.5 HSS 22 x 22 x 0.5	353 353	W30X108 W30X108	1.85					
					4 3	HSS 23 x 23 x 0.5 HSS 23 x 23 x 0.5	386 386	W30X116 W30X116	2.15 2.13					
					2	HSS 23 x 23 x 0.5	386	W30X116	2.13					
	<u> </u>			3B	3	HSS 18 x 18 x 0.375 HSS 22 x 22 x 0.5	89 353	W18x40 W30X108	1.06	1.41				
					1	HSS 23 x 23 x 0.5 HSS 18 x 18 x 0.375	386	W30X116 W18x40	1.57					
				3E	2 1 3	HSS 24 x 24 x 0.5 HSS 24 x 24 x 0.5 HSS 15 x 15 x 0.3125	420 420 52	W30X116 W30X116 W14x30	1.82 1.43 1.16	1.41				
				3G	2 1	HSS 18 x 18 x 0.375 HSS 20 x 20 x 0.5	178 291	W24X68 W30X90	1.25 1.79	1.40				
					9 8	HSS 16 x 16 x 0.375 HSS 18 x 18 x 0.375	70 178	W18x35 W24X68	1.05 0.93					
					7	HSS 20 x 20 x 0.5 HSS 22 x 22 x 0.5	291 353	W30X90 W30X108	1.23 1.45					
				98	5	HSS 23 x 23 x 0.5 HSS 24 x 24 x 0.5	386 420	W30X116 W30X116	1.51	1.3/				
					2	HSS 24 x 24 x 0.5 HSS 24 x 24 x 0.5	420	W30X116 W30X116	1.5%					
	6	63	16		18 17	HSS 11 x 11 x 0.3125 HSS 15 x 15 x 0.3125	28	W12x16 W18X50	0.81					
					16 15	HSS 17 x 17 x 0.375 HSS 18 x 18 x 0.375	158	W24X62 W24X68	1.75					
					14 13	HSS 18 x 18 x 0.375 HSS 19 x 19 x 0.5	178 263	W24X68 W18x119	1.37 1.77					
					12 11	HSS 20 x 20 x 0.5 HSS 20 x 20 x 0.5	291 291	W30X90 W30X90	1.95 1.78					
				18B	10 9	HSS 21 x 21 x 0.5 HSS 21 x 21 x 0.5	321 321	W30X99 W30X99	2.02 1.92	1.84				
					8 7	HSS 22 x 22 x 0.5 HSS 22 x 22 x 0.5	353 353	W30X108 W30X108	2.21 2.15					
					6	HSS 22 x 22 x 0.5 HSS 22 x 22 x 0.5	353 353	W30X108 W30X108	2.10					
					4	HSS 22 x 22 x 0.5	353	W30X108	2.04					
					2	HSS 22 x 22 x 0.5 HSS 22 x 22 x 0.5	353	W30X108 W30X108	2.03					

	1	Member Size	s that Were U	sed to Calibrate 1	at Base Line	Design Values 7, 8, and 9 (	Per Figure 6.3.1)	for d/t ≤ 2.26 √(E/F	y)			
				Building		Final Sectio	n Sizes and Prope	rties	Flexural Rig	idity Ratio, η		
dit	η Base Line Number	F <sub>yc</sub> (ksi)	f'c (ksi)	Geometry and Loading Based on Design No.	Story	Suggested Column	Maximum Allowed Z <sub>g</sub> (in <sup>3</sup> )	Suggested Girder	Story Value	Building Mean Value		
				ЗA	3 2	HSS 16 x 16 x 0.375 HSS 20 x 20 x 0.5	89 370	W18x40 W30X108	0.43 0.79 0.77	0.66		
				3D	3	HSS 16 x 16 x 0.5 HSS 21 x 21 x 0.5	119 408	W18X55 W30X116	0.53 0.76	0.72		
				3E	1 3 2	HSS 22 x 22 x 0.625 HSS 14 x 14 x 0.375 HSS 17 x 17 x 0.5	560 68 268	W30X116 W18x35 W18x119	0.86	0.86		
					1 9	HSS 19 x 19 x 0.5 HSS 15 x 15 x 0.375	334 79	W30X99 W18x40	1.05			
					8 7	HSS 17 x 17 x 0.5 HSS 19 x 19 x 0.5	268 334	W18x119 W30X99	0.65			
				9A	5	HSS 21 x 21 x 0.5 HSS 21 x 21 x 0.5	408	W30X116 W30X116 W30X116	0.74 0.67	0.67		
					3	HSS 21 x 21 x 0.5 HSS 21 x 21 x 0.5	408	W30X116 W30X116	0.62			
	7	80	4		18	HSS 11 x 11 x 0.3125	35	W14x22	0.56			
					17 16	HSS 13 x 13 x 0.3125 HSS 16 x 16 x 0.375	97 178	W21x44 W24X68	0.48			
					15	HSS 16 x 16 x 0.375	178	W24X68	0.69			
					14 13	HSS 17 x 17 x 0.5 HSS 18 x 18 x 0.5	268 300	W18x119 W30X90	0.95			
					12	HSS 18 x 18 x 0.5	300	W30X90	0.91			
					11 10	HSS 19 x 19 x 0.5 HSS 19 x 19 x 0.5	334 334	W30X99 W30X99	1.04 0.96			
				18A	9	HSS 20 x 20 x 0.5	370	W30X108	1.13	1.00		
					8	HSS 20 x 20 x 0.5 HSS 20 x 20 x 0.5	370 370	W30X108 W30X108	1.07			
					6	HSS 21 x 21 x 0.5	408	W30X116	1.24			
					5	HSS 21 x 21 x 0.5 HSS 21 x 21 x 0.5	408 408	W30X116 W30X116	1.22			
					3	HSS 21 x 21 x 0.5	408	W30X116	1.20			
					2	HSS 21 x 21 x 0.5 HSS 21 x 21 x 0.5	408	W30X116 W30X116	1.19			
					3	HSS 16 x 16 x 0.375	89	W18x40	0.54			
				3B	2	HSS 20 x 20 x 0.5	370	W30X108	0.97	0.82		
					3	HSS 16 x 16 x 0.375	89	W18x40	0.50			
				3E	2	HSS 21 x 21 x 0.5 HSS 22 x 22 x 0.625	408 560	W30X116 W30X116	0.93	0.82		
					3	HSS 14 x 14 x 0.375	68	W18x35	0.84			
				3G	2	HSS 17 x 17 x 0.5 HSS 19 x 19 x 0.5	268 334	W18x119 W30X99	1.05	1.06		
					9	HSS 15 x 15 x 0.375	79	W18x40	0.70			
			10		8	HSS 16 x 16 x 0.5 HSS 19 x 19 x 0.5	238 334	W24X84 W30X99	0.60			
					6	HSS 20 x 20 x 0.5	370	W30X108	0.87			
					9B 5 HSS 21 x 21 x 0.5 408 4 HSS 21 x 21 x 0.5 408 3 HSS 21 x 21 x 0.5 408		98	5	HSS 21 x 21 x 0.5 HSS 21 x 21 x 0.5	408	W30X116 W30X116	0.92
$\leq$ 2.26 $\sqrt{(E/Fy)}$						W30X116	0.77					
					2	HSS 21 x 21 x 0.5 HSS 21 x 21 x 0.5	408	W30X116 W30X116	0.75			
	8	80			18	HSS 11 x 11 x 0.3125	35	W14x22	0.71			
					17	HSS 13 x 13 x 0.3125 HSS 15 x 15 x 0.375	97	W21x44 W24X62	0.63			
					15	HSS 16 x 16 x 0.375	178	W24X68	0.87			
					14	HSS 16 x 16 x 0.375 HSS 17 x 17 x 0.5	1/8 268	W24X68 W18x119	0.69			
					12	HSS 18 x 18 x 0.5	300	W30X90	1.11			
					11	HSS 19 x 19 x 0.5 HSS 19 x 19 x 0.5	334 334	W30X99 W30X99	1.27			
1				188	9	HSS 20 x 20 x 0.5	370	W30X108	1.38	1.15		
1					8	HSS 20 x 20 x 0.5	370	W30X108 W30X108	1.32			
					6	HSS 20 x 20 x 0.5	370	W30X108	1.25			
					4	HSS 21 x 21 x 0.5	408	W30X108 W30X116	1.49			
					3	HSS 21 x 21 x 0.5	408	W30X116	1.48			
					- 1	HSS 21 x 21 x 0.5	408	W30X116	1.47			
				зА	3 2	HSS 16 x 16 x 0.375 HSS 20 x 20 x 0.5	89 370	W18x40 W30X108	0.63	0.94		
					1 3	HSS 21 x 21 x 0.5 HSS 16 x 16 x 0.375	408 89	W30X116 W18x40	1.09 0.59			
				3D	2 1 3	HSS 21 x 21 x 0.5 HSS 21 x 21 x 0.5 HSS 14 x 14 x 0.375	408 408 68	W30X116 W30X116 W18x35	1.06 0.82 0.96	0.82		
				3F	2	HSS 17 x 17 x 0.5 HSS 18 x 18 x 0.5 HSS 14 x 14 :: 0.075	268 300	W18x119 W30X90	1.18	1.10		
					8	HSS 16 x 16 x 0.375	178	W24X68	0.58			
				Ι.	6	HSS 18 x 18 x 0.5 HSS 20 x 20 x 0.5	300 370	w30X90 W30X108	0.79 0.99			
	9 80			9A	5 4	HSS 21 x 21 x 0.5 HSS 21 x 21 x 0.5	408 408	W30X116 W30X116	1.04 0.94	0.83		
				3 2	HSS 21 x 21 x 0.5 HSS 21 x 21 x 0.5	408 408	W30X116 W30X116	0.89 0.85				
		16		1	HSS 21 x 21 x 0.5 HSS 10 x 10 x 0.3125	408 29	W30X116 W12x16	0.84				
				17	HSS 13 x 13 x 0.3125 HSS 15 x 15 x 0.375	97 157	W21x44 W24X62	0.73				
				15	HSS 16 x 16 x 0.375 HSS 16 x 16 x 0.375	178	W24X68 W24X68	1.01				
				13	HSS 17 x 17 x 0.5	268	W18x119	1.11				
				12	HSS 18 x 18 x 0.5 HSS 18 x 18 x 0.5	300	W30X90 W30X90	1.26				
				18A	10 9	HSS 19 x 19 x 0.5 HSS 19 x 19 x 0.5	334 334	W30X99 W30X99	1.34	1.14 1.34 1.26 1.19		
1					8	HSS 20 x 20 x 0.5	370	W30X108	1.50			
1				6	HSS 20 x 20 x 0.5	370	W30X108 W30X108	1.45				
1				5	HSS 20 x 20 x 0.5 HSS 20 x 20 x 0.5	370 370	W30X108 W30X108	1.39				
1					3	HSS 20 x 20 x 0.5	370	W30X108	1.37			
1			1	1	1	HSS 20 x 20 x 0.5	370	W30X108 W30X108	1.36			

		Member	Sizes that Wer	e Used to Calibra	te ¶ at Base I	ine Design Values 1, 2, and	1 3 (Per Figure 6.	3.1) for d/t ratios = 1	30			
						Final Sectio	n Sizes and Prope	erties	Flexural Rig	idity Ratio, η		
d/t	η Base Line Number	F <sub>yc</sub> (ksi)	f'c (ksi)	Building Geometry and Loading Based on Design No.	Story	Suggested Column	Maximum Allowed Z <sub>9</sub> (in <sup>3</sup> )	Suggested Girder	Story Value	Building Mean Value		
				за	3 2 1	HSS 24 x 24 x 0.3125 HSS 29 x 29 x 0.375 HSS 29 x 29 x 0.375	96 336 336	W21x44 W30X99 W30X99	2.02 2.70 2.23	2.32		
				3D	3	HSS 24 x 24 x 0.3125 HSS 29 x 29 x 0.375	96 336	W21x44 W30X99	1.91 2.19	2.16		
				3F	3 2	HSS 19 x 19 x 0.3125 HSS 24 x 24 x 0.3125	60 191	W14x34 W24X68	2.30 2.12 2.43	2.64		
					1 9	HSS 27 x 27 x 0.375 HSS 20 x 20 x 0.3125	291 67	W30X90 W18x35	3.36 1.59			
					8	HSS 24 x 24 x 0.3125	191	W24X68	1.80			
					6	HSS 29 x 29 x 0.375	336	W30X90 W30X99	2.30			
				9A	5	HSS 29 x 29 x 0.375	336	W30X99	2.15	1.96		
					3	HSS 29 x 29 x 0.375	336	W30X99	1.85			
					2	HSS 29 x 29 x 0.375 HSS 29 x 29 x 0.375	336 336	W30X99 W30X99	1.79			
	1	46	4		18	HSS 13 x 13 x 0.3125	28	W12x16	1.21			
					17	HSS 18 x 18 x 0.3125 HSS 21 x 21 x 0.3125	107 146	W18X50 W21X62	2.06 2.50			
					15	HSS 23 x 23 x 0.3125	176	W21X68	2.72			
					14	HSS 24 x 24 x 0.3125 HSS 25 x 25 x 0.375	250	W24 X68 W27 X84	3.03			
					12	HSS 26 x 26 x 0.375 HSS 27 x 27 x 0.375	269	W18x119 W30X90	3.15			
				18A	10	HSS 28 x 28 x 0.375	313	W30X99	3.58	3 10		
					9	HSS 28 x 28 x 0.375 HSS 29 x 29 x 0.375	313 336	W30X99 W30X99	3.40 3.73			
					7	HSS 29 x 29 x 0.375	336	W30X99	3.62			
					6 5	HSS 29 x 29 x 0.375 HSS 29 x 29 x 0.375	336 336	W30X99 W30X99	3.54 3.49			
					4	HSS 29 x 29 x 0.375	336	W30X99	3.46			
					2	HSS 29 x 29 x 0.375 HSS 29 x 29 x 0.375	336	W30X99 W30X99	3.44 3.43			
					1	HSS 29 x 29 x 0.375	336	W30X99	3.42			
			3B	2	HSS 28 x 28 x 0.375	313	W30X99	3.04	2.84			
				1	HSS 29 x 29 x 0.375 HSS 24 x 24 x 0.3125	336 96	W30X99 W21x44	2.87				
				3E	2	HSS 29 x 29 x 0.375	336	W30X99	2.82	2.75		
					3	HSS 30 x 30 x 0.5 HSS 19 x 19 x 0.3125	4/9 60	W30X116 W14x34	2.97			
				3G	2	HSS 24 x 24 x 0.3125	191	W24X68	3.16	3.38		
			1         FISS 20: 22: 03:125         67           9         HSS 20: 20: 03:125         67           8         HSS 20: 42: 03:125         191           7         HSS 20: 42: 03:125         191           6         HSS 20: 42: 03:125         193           6         HSS 20: 42: 03:125         336           9B         5         HSS 20: 42: 03:15         336           4         HSS 20: 42: 03:15         336           3         HSS 20: 42: 03:15         336           2         HSS 20: 42: 03:15         336           1         HSS 20: 42: 03:15         336				9	HSS 20 x 20 x 0.3125	67	W18x35	2.01	
					8	HSS 24 x 24 x 0.3125 HSS 27 x 27 x 0.375	191 291	W24X68 W30X90	2.32			
				336	W30X99	3.16						
		46		10	9B	5	HSS 29 x 29 x 0.375 HSS 29 x 29 x 0.375	336 336	W30X99 W30X99	2.77 2.53	2.53	
= 80						3	HSS 29 x 29 x 0.375	336	W30X99	2.40		
					10	2 HSS 29 x 29 x 0.375 336 1 HSS 29 x 29 x 0.375 336	336 336	W30X99 W30X99	2.32			
	2	40	10		18	HSS 13 x 13 x 0.3125	28	W12x16	1.49			
					16	HSS 21 x 21 x 0.3125	146	W21X62	3.19			
					15 14	HSS 23 x 23 x 0.3125 HSS 24 x 24 x 0.3125	176 191	W21X68 W24X68	3.50 3.43			
					13	HSS 24 x 24 x 0.3125	191	W24X68	2.98			
					12	HSS 26 x 26 x 0.375 HSS 27 x 27 x 0.375	269 291	W18x119 W30X90	4.01 4.26			
				18B	10	HSS 27 x 27 x 0.375	291	W30X90	4.00	3.83		
					8	HSS 28 x 28 x 0.375	313	W30X99	4.20			
1					7 6	HSS 29 x 29 x 0.375 HSS 29 x 29 x 0.375	336 336	W30X99 W30X99	4.66 4.56			
1					5	HSS 29 x 29 x 0.375	336	W30X99	4.49			
1					4	HSS 29 x 29 x 0.375	336	W30X99 W30X99	4.45			
1					2	HSS 29 x 29 x 0.375 HSS 29 x 29 x 0.375	336 336	W30X99 W30X99	4.41 4.41			
				34	3	HSS 24 x 24 x 0.3125 HSS 28 x 28 x 0.375	96 313	W21x44 W30X00	3.03	3.30		
1					1	HSS 29 x 29 x 0.375	336	W30X99 W21-44	3.33	0.00		
				3D	2	HSS 29 x 29 x 0.375 HSS 30 x 30 x 0.5 HSS 19 x 10 x 0.2105	336 479 60	W30X99 W30X116	3.27 3.40	3.18		
				3F	2	HSS 24 x 24 x 0.3125 HSS 24 x 24 x 0.3125	191 191	W24X68 W24X68	3.68	3.22		
					9 8	HSS 20 x 20 x 0.3125 HSS 24 x 24 x 0.3125	67 191	W18x35 W24X68	2.32 2.70			
1					7 6	HSS 26 x 26 x 0.375 HSS 29 x 29 x 0.375	269 336	W18x119 W30X99	2.94 3.66			
1	3 46 .		9A	5 4	HSS 29 x 29 x 0.375 HSS 29 x 29 x 0.375	336 336	W30X99 W30X99	3.21 2.95	2.88			
1				3 2	HSS 29 x 29 x 0.375 HSS 29 x 29 x 0.375	336 336	W30X99 W30X99	2.79 2.70				
1		16		1	HSS 29 x 29 x 0.375 HSS 13 x 13 x 0.3125	336 28	W30X99 W12x16	2.67 1.69				
1				17 16	HSS 17 x 17 x 0.3125 HSS 20 x 20 x 0.3125	96 133	W21x44 W18x65	2.38 3.04				
1				15 14	HSS 23 x 23 x 0.3125 HSS 24 x 24 x 0.3125	176 191	W21X68 W24X68	4.06 3.99				
1				13 12	HSS 24 x 24 x 0.3125 HSS 26 x 26 x 0.375	191 269	W24X68 W18x119	3.48 4.63				
1				11	HSS 26 x 26 x 0.375 HSS 27 x 27 x 0.375	269 291	W18x119 W30X90	4.27	4.36			
1			18A	9	HSS 28 x 28 x 0.375	313	W30X99	5.06				
1				7	HSS 29 x 29 x 0.375	336	W30X99	*.07 5.41				
1				6 5	HSS 29 x 29 x 0.375 HSS 29 x 29 x 0.375	336 336	W30X99 W30X99	5.29 5.22				
1					4	HSS 29 x 29 x 0.375 HSS 29 x 29 x 0.375	336 336	W30X99 W30X99	5.16 5.13			
					2	HSS 29 x 29 x 0.375 HSS 29 x 29 x 0.375	336	W30X99 W30X99	5.12			

	Member Sizes that Were Used to Calibrate <b>q</b> at Base Line Design Values 4, 5, and 6 (Per Figure 6.3.1) for dt = 80												
				Duilding		Final Section	n Sizes and Prope	arties	Flexural Rig	idity Ratio, η			
dit	η Base Line Number	F <sub>yc</sub> (ksi)	f'c (ksi)	Geometry and Loading Based on Design No.	Story	Suggested Column	Maximum Allowed Z <sub>g</sub> (in <sup>3</sup> )	Suggested Girder	Story Value	Building Mean Value			
				3B	3 2 1	HSS 21 x 21 x 0.3125 HSS 25 x 25 x 0.375 HSS 27 x 27 x 0.375	100 342 398	W21x44 W30X99 W30X116	1.20 1.54 1.70	1.48			
				3E	3 2 1	HSS 22 x 22 x 0.3125 HSS 27 x 27 x 0.375 HSS 29 x 29 x 0.375	110 398 460	W21X50 W30X116 W30X116	1.36 1.66 1.73	1.59			
				3G	3 2 1	HSS 17 x 17 x 0.3125 HSS 23 x 23 x 0.3125 HSS 24 x 24 x 0.3125	66 241 262	W14x34 W24X84 W18x119	1.36 2.06 1.89	1.77			
					9	HSS 18 x 18 x 0.3125	74	W16x40	1.05				
					7	HSS 24 x 24 x 0.3125	262	W18x119	1.30				
				9B	6	HSS 26 x 26 x 0.375 HSS 27 x 27 x 0.375	369	W30X108 W30X116	1.62	1.56			
				00	4	HSS 28 x 28 x 0.375	429	W30X116	1.71	1.00			
					3	HSS 29 x 29 x 0.375 HSS 29 x 29 x 0.375	460 460	W30X116 W30X116	1.85				
	4	63	4		1	HSS 29 x 29 x 0.375	460	W30X116	1.77				
					18 17	HSS 12 x 12 x 0.3125 HSS 16 x 16 x 0.3125	33 116	W12x22 W18X55	0.85				
					16	HSS 18 x 18 x 0.3125	147	W21X62	1.35				
					15	HSS 20 x 20 x 0.3125 HSS 22 x 22 x 0.3125	182 220	W24X68 W12x136	1.57				
					13	HSS 23 x 23 x 0.3125	241	W24X84	1.94				
					11	HSS 24 x 24 x 0.3125	262	W18x119	1.88				
				18B	10 9	HSS 24 x 24 x 0.3125 HSS 25 x 25 x 0.375	262 342	W18x119 W30X99	1.75	1.92			
					8	HSS 25 x 25 x 0.375	342	W30X99	2.12				
					7	HSS 25 x 25 x 0.375 HSS 26 x 26 x 0.375	342 369	W30X99 W30X108	2.05 2.34				
					5	HSS 26 x 26 x 0.375	369	W30X108	2.30				
					4	HSS 26 x 26 x 0.375 HSS 26 x 26 x 0.375	369 369	W30X108 W30X108	2.28 2.26				
					2	HSS 26 x 26 x 0.375	369	W30X108	2.26				
					3	HSS 26 X 26 X 0.375 HSS 21 X 21 X 0.3125	100	W30X108 W21x44	1.55				
				ЗA	2	HSS 25 x 25 x 0.375 HSS 27 x 27 x 0.375	342	W30X99 W30X116	1.97	1.90			
					3	HSS 22 x 22 x 0.3125	110	W21X50	1.75				
			3D	2	HSS 27 x 27 x 0.375 HSS 29 x 29 x 0.375	398 460	W30X116 W30X116	2.14 2.24	2.05				
				05	3	HSS 16 x 16 x 0.3125	58	W14x34	1.33				
							3F	2	HSS 23 x 23 x 0.3125 HSS 24 x 24 x 0.3125	241 262	W24X84 W18x119	2.67 2.47	2.16
					9	HSS 17 x 17 x 0.3125	66	W14x34	1.05				
					7	HSS 22 x 22 x 0.3125 HSS 24 x 24 x 0.3125	220	W12x136 W18x119	1.65				
				94	6	HSS 25 x 25 x 0.375 HSS 27 x 27 x 0.375	342	W30X99 W30X116	1.78	1.94			
						4	HSS 28 x 28 x 0.375	429	W30X116	2.10	1.04		
= 80										3	HSS 29 x 29 x 0.375 HSS 29 x 29 x 0.375	460 460	W30X116 W30X116
	5	63	10		1	HSS 29 x 29 x 0.375	460	W30X116	2.30				
					18	HSS 11 x 11 x 0.3125 HSS 15 x 15 x 0.3125	28 103	W12x16 W18X50	0.71				
					16	HSS 18 x 18 x 0.3125	147	W21X62	1.73				
					14	HSS 22 x 22 x 0.3125	220	W12x136	2.02				
					13	HSS 23 x 23 x 0.3125 HSS 24 x 24 x 0.3125	241 262	W24X84 W18x119	2.52				
					11	HSS 24 x 24 x 0.3125	262	W18x119	2.45				
				18A	10 9	HSS 24 x 24 x 0.3125 HSS 24 x 24 x 0.3125	262 262	W18x119 W18x119	2.29 2.17	2.37			
					8	HSS 25 x 25 x 0.375	342	W30X99	2.71				
					6	HSS 25 x 25 x 0.375 HSS 25 x 25 x 0.375	342 342	W30X99 W30X99	2.63				
					5	HSS 26 x 26 x 0.375 HSS 26 x 26 x 0.375	369	W30X108 W30X108	2.95				
					3	HSS 26 x 26 x 0.375	369	W30X108	2.90				
					2	HSS 26 x 26 x 0.375 HSS 26 x 26 x 0.375	369 369	W30X108 W30X108	2.90 2.89				
1				3B	3 2	HSS 21 x 21 x 0.3125 HSS 24 x 24 x 0.3125	100 262	W21x44 W18x119	1.80 1.78	2.04			
1					1	HSS 27 x 27 x 0.375 HSS 22 x 22 x 0.3125	398 110	W30X116 W21X50	2.53				
1				3E	2	HSS 27 x 27 x 0.375 HSS 29 x 29 x 0.375	398 460	W30X116 W30X116	2.48	2.38			
				3G	3	HSS 16 x 16 x 0.3125 HSS 22 x 22 x 0.3125	58	W14x34 W12x136	1.54	2 34			
1					1	HSS 24 x 24 x 0.3125	262	W18x119	2.89	2.04			
1					8 7	HSS 22 x 22 x 0.3125	220	W12x136	1.92				
					6	HSS 25 x 25 x 0.375	342	W30X99	2.06	0.00			
	6 63 1		96	4	HSS 27 x 27 x 0.375 HSS 28 x 28 x 0.375	429	W30X116 W30X116	2.44	2.22				
1				2	HSS 28 x 28 x 0.375 HSS 29 x 29 x 0.375	429	W30X116 W30X116	2.43					
1		16		1 18	HSS 29 x 29 x 0.375 HSS 11 x 11 x 0.3125	460 28	W30X116 W12x16	2.67 0.81					
1				17 16	HSS 15 x 15 x 0.3125 HSS 18 x 18 x 0.3125	103 147	W18X50 W21X62	1.42 2.00					
1				15 14	HSS 20 x 20 x 0.3125 HSS 22 x 22 x 0.3125	182 220	W24X68 W12x136	2.34 2.84					
1				13 12	HSS 23 x 23 x 0.3125 HSS 24 x 24 x 0.3125	241 262	W24X84 W18x119	2.94 3.12					
1				11 10	HSS 24 x 24 x 0.3125 HSS 24 x 24 x 0.3125	262 262	W18x119 W18x119	2.87 2.68					
1			18B	9	HSS 24 x 24 x 0.3125	262	W18x119	2.55	2.70				
1				7	HSS 25 x 25 x 0.375	342	W30X99	3.05	2.70				
1				6 5	HSS 25 x 25 x 0.375 HSS 25 x 25 x 0.375	342 342	W30X99 W30X99	2.98 2.93					
1				4	HSS 25 x 25 x 0.375 HSS 26 x 26 x 0.375	342 369	W30X99 W30X108	2.90 3.37					
1					2	HSS 26 x 26 x 0.375	369	W30X108	3.36				
I	1	1	1	1	1	HSS 26 x 26 x 0.3/5	369	W30X108	3.36				

Member Sizes that Were Used to Calibrate <b>q</b> at Base Line Design Values 7, 8, and 9 (Per Figure 6.3.1) for dit = 80										
dit	η Base Line Number	F <sub>yc</sub> (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 <sub>9</sub> (in <sup>3</sup> )	Suggested Girder	Story Value	Building Mean Value
				ЗA	3 2 1	HSS 20 x 20 x 0.3125 HSS 24 x 24 x 0.3125 HSS 25 x 25 x 0.375	116 333 435	W18X55 W30X99 W30X116	0.99 1.16 1.26	1.13
				3D	3	HSS 20 x 20 x 0.3125 HSS 25 x 25 x 0.375	116 435	W18X55 W30X116	0.92	1.15
				3F	3	HSS 27 x 27 x 0.375 HSS 15 x 15 x 0.3125 HSS 21 x 21 x 0.3125	65 255	W14x34 W24x94	0.78	1.27
					1	HSS 23 x 23 x 0.3125 HSS 16 x 16 x 0.3125	306 74	W30X90 W16x40	1.59	
				9A	8	HSS 20 x 20 x 0.3125	231	W24X84	0.87	
			4		6	HSS 23 x 23 x 0.3125 HSS 24 x 24 x 0.3125	306 333	W30X90 W30X99	1.09	1.07
					5	HSS 25 x 25 x 0.375	435	W30X116	1.21	
					3	HSS 26 x 26 x 0.375	435	W30X116	1.10	
					2	HSS 26 x 26 x 0.375 HSS 27 x 27 x 0.375	469 506	W30X116 W30X116	1.17	
	7	80			18	HSS 11 x 11 x 0.3125	35	W14x22	0.56	
				104	17	HSS 14 x 14 x 0.3125 HSS 17 x 17 x 0.3125	113 167	W18X55 W21X68	0.71	1.36
					15	HSS 19 x 19 x 0.3125	208	W24X76	1.27	
					14	HSS 20 x 20 x 0.3125 HSS 21 x 21 x 0.3125	231 255	W24 X84 W24 x94	1.27	
					12	HSS 22 x 22 x 0.3125	280	W18x119	1.44	
					10	HSS 24 x 24 x 0.3125	333	W30X90 W30X99	1.58	
				10/1	9	HSS 24 x 24 x 0.3125 HSS 24 x 24 x 0.3125	333	W30X99 W30X99	1.66	
					7	HSS 24 x 24 x 0.3125	333	W30X99	1.53	
					6	HSS 24 x 24 x 0.3125 HSS 24 x 24 x 0.3125	333 333	W30X99 W30X99	1.50 1.47	
					4	HSS 24 x 24 x 0.3125	333	W30X99	1.46	
					3	HSS 24 x 24 x 0.3125 HSS 24 x 24 x 0.3125	333 333	W30X99 W30X99	1.45	
					1	HSS 24 x 24 x 0.3125	333	W30X99	1.44	
				3B	2	HSS 19 x 19 x 0.3125 HSS 24 x 24 x 0.3125	104 333	W18X50 W30X99	1.03	1.26
		80	10		1	HSS 24 x 24 x 0.3125	333	W30X99	1.24	
				3E	2	HSS 24 x 24 x 0.3125	333	W30X99	1.19	1.37
					1	HSS 27 x 27 x 0.375 HSS 15 x 15 x 0.3125	506 65	W30X116 W14x34	1.69	
				3G	2	HSS 21 x 21 x 0.3125	255	W24x94	1.85	1.64
					1 9	HSS 23 x 23 x 0.3125 HSS 16 x 16 x 0.3125	306 74	W30X90 W16x40	2.08	
				98	8	HSS 20 x 20 x 0.3125	231	W24X84	1.12	1.32
					6	HSS 23 x 23 x 0.3125 HSS 24 x 24 x 0.3125	306	W30X90 W30X99	1.42	
= 80					5	HSS 24 x 24 x 0.3125	333	W30X99	1.19	
					3	HSS 26 x 26 x 0.375	469	W30X116	1.56	
	8				2	HSS 26 x 26 x 0.375 HSS 26 x 26 x 0.375	469 469	W30X116 W30X116	1.51 1.49	
				188	18	HSS 11 x 11 x 0.3125	35	W14x22	0.71	1.72
					17	HSS 14 x 14 x 0.3125 HSS 16 x 16 x 0.3125	113	W18X55 W21X62	0.90	
					15	HSS 18 x 18 x 0.3125	187	W24X68	1.30	
					14	HSS 21 x 21 x 0.3125	255	W24x94	1.65	
					12	HSS 22 x 22 x 0.3125 HSS 23 x 23 x 0.3125	280 306	W18x119 W30X90	1.88	
					10	HSS 23 x 23 x 0.3125	306	W30X90	1.93	
					9	HSS 24 x 24 x 0.3125 HSS 24 x 24 x 0.3125	333	W30X99 W30X99	2.17 2.09	
					7	HSS 24 x 24 x 0.3125	333	W30X99	2.02	
					5	HSS 24 x 24 x 0.3125	333	W30X99	1.94	
					4	HSS 24 x 24 x 0.3125 HSS 24 x 24 x 0.3125	333 333	W30X99 W30X99	1.92	
					2	HSS 24 x 24 x 0.3125	333	W30X99	1.90	
					3	HSS 19 x 19 x 0.3125	333 104	W18X50	1.90	1.00
				JA	2	HSS 24 x 24 x 0.3125 HSS 24 x 24 x 0.3125	333	W30X99 W30X99	1.78	1.48
				3D	3 2 1	HSS 20 x 20 x 0.3125 HSS 24 x 24 x 0.3125 HSS 27 x 27 x 0.375	116 333 506	W18X55 W30X99 W30X116	1.39 1.42 1.97	1.60
				3F	3 2 1	HSS 15 x 15 x 0.3125 HSS 20 x 20 x 0.3125 HSS 23 x 23 x 0.3125	65 231 306	W14x34 W24X84 W30X90	1.16 1.76 2.43	1.79
				9A	9 8	HSS 16 x 16 x 0.3125 HSS 20 x 20 x 0.3125	74 231	W16x40 W24X84	0.94 1.31	2.00
					7	HSS 23 x 23 x 0.3125 HSS 24 x 24 x 0.3125	306 333	W30X90 W30X99	1.67 1.61	
					5 4	HSS 24 x 24 x 0.3125 HSS 25 x 25 x 0.375	333 435	W30X99 W30X116	1.40 1.64	
			16		3 2	HSS 26 x 26 x 0.375 HSS 26 x 26 x 0.375	469 469	W30X116 W30X116	1.81 1.76	
	9	80			1	HSS 26 x 26 x 0.375 HSS 10 x 10 x 0.3125	469 29	W30X116 W12x16	1.74 0.49	
				18A	17	HSS 14 x 14 x 0.3125 HSS 16 x 16 x 0.3125	113 148	W18X55 W21X62	1.05	
					15	HSS 18 x 18 x 0.3125 HSS 20 x 20 x 0.3125	187 231	W24X68 W24X84	1.51	
					13	HSS 21 x 21 x 0.3125 HSS 22 x 22 x 0.3125	255 280	W24x94 W18x119	2.03	
					11	HSS 23 x 23 x 0.3125	306	W30X90	2.42	
					9	HSS 24 x 24 x 0.3125	333	W30X90	2.25	
					8	HSS 24 x 24 x 0.3125 HSS 24 x 24 x 0.3125	333 333	W30X99 W30X99	2.45 2.37	
					6 5	HSS 24 x 24 x 0.3125 HSS 24 x 24 x 0.3125	333 333	W30X99 W30X99	2.32 2.28	
					4	HSS 24 x 24 x 0.3125	333	W30X99	2.26	
					2	HSS 24 x 24 x 0.3125	333	W30X99	2.24	
L				1	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<sub>roof</sub>, 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.

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

$$d = \Delta_{\text{roof}} - \frac{V_{\text{PO}}}{K_{\text{roof}}} = G \left(\frac{V_{\text{PO}}}{K_{\text{roof}}}\right)^{s}$$
(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.

$$\log(d) = \log(G) + s \log\left(\frac{V_{PO}}{K_{roof}}\right)$$
(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.