EVALUATION AND COMPARISON OF A NON-SEISMIC DESIGN AND SEISMIC DESIGN FOR A LOW RISE OFFICE BUILDING

by

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(ABSTRACT)

This project was conducted to examine seismic design requirements in areas of low seismic activity. To accomplish this study two designs of a hypothetical four story office building were completed. The first design was completed for gravity and wind loads. The second design was completed for gravity and seismic forces.

Comparisons of both designs were made to examine differences in member sizes, support conditions and the design of fully restrained connections.

ACKNOWLEDGEMENTS

I would like to thank my advisor, Dr. Thomas M. Murray, for his assistance in the completion of this project. I am particularly grateful for the use of his computer software throughout the course of this project.

Finally, I would like to thank Professors W.S. Easterling and D.A. Garst for serving on my project committee and for their assistance in all of my graduate studies.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

CHAPTER I

INTRODUCTION

1.1 Overview

The primary objective of this project is to compare non-seismic and seismic designs of a hypothetical four story unsymmetrical office building. The non-seismic design is for gravity and wind loads, and the seismic design is for gravity and seismic forces.

Comparisons of the two designs are made including differences in member selection, support conditions, and design of fully restrained connections.

The non-seismic design is done in accordance with the American Institute of Steel Construction (AISC), Load and Resistance Factor Design (LRFD) specification [AISC 1986], and the BOCA National Building Code [BOCA 1990]. The seismic design is done in accordance with the AISC Seismic Provisions for Structural Steel Buildings [AISC 1992].

A secondary objective is to gain a greater insight into the considerations and decision making process involved in the design of building structures.

1.2 Scope of Study

Completion of this project required the determination of a building layout, design loads, evaluation of lateral forces resisting systems, selection of members, and design of connections. Decisions such as building dimensions, stairway and elevator locations, roofing materials, and siding material while normally the responsibility of the project architects were made as part of the design process.

This project did not involve comparisons of other structural systems using either reinforced concrete or prestressed concrete. Comparison of structural systems would normally be in the conceptual design stage and would include construction cost estimates and considerations of future building use to aide in the decision making process.

There was no attempt at a detailed look at the interface of structural systems with architectural, mechanical , fire protection and electrical systems.

1.3 Method of Evaluation

Evaluations of the non seismic and seismic designs will be based on differences in steel weight, labor costs are not considered. Differences in steel weight provide a simple method of comparing the two designs. Comparisons of estimated labor costs would undoubtly give a better indication of the overall cost of the two designs, but for the reasons stated below were not used.

Unlike the determination of material cost the estimation of labor costs involves more variables. For example if the cost of connections are being estimated variables such as percentage of shop or field work, geographic locations, and difficulties in determining production rates, must be considered.

The percentage of shop and field work impacts overall project cost. Generally both the hourly labor costs and insurance rates are higher for field work. Both of these factors can be attributed to working conditions and the hazards involved in field work as well as differences in geographic locations. Estimation of production rates would require detailed knowledge of fabrication methods used, which could vary from fabricator to fabricator.

CHAPTER II

BUILDING DESCRIPTION

2.1 Building Function

The building under design is a speculative four story office building with approximately 108,000 square feet of rentable office space. This building will not have retail stores or areas of public assembly. The following assumptions were made for the speculative design:

1. Office partitions locations are not fixed.
2. Corridor locations are not fixed.
3. Bracing systems are limited to stairway/elevator shafts.
4. Ceiling heights are at 10 ft. above the floor slab on the second, third, and fourth floors.
5. Ceiling height may be up to 12 ft. above the floor slab on the first floor.

The above assumptions are required for the design of floor, roof and lateral force resisting systems.

2.2 Geographical Location

To accomplish this study a location had to be selected, for determination of applicable snow, wind and seismic loads. The building under design is assumed to be located in southern Maine.

This location was chosen because it is in a seismic zone 2 region, and has relatively high wind and snow loads. The design wind velocity and ground acceleration are typical of many of the larger cities along the Atlantic Coast of the United States. Seismic forces will be increased, since the weight of the full snow load is counted as part of the building weight in horizontal base shear calculations.

2.3 Building Architectural Details

The layout of the building, in plan, was chosen not to be symmetric in order to include the effects of building torsion in both the non-seismic and seismic designs. An L shaped building with dimensions of 260 ft. along the long leg of the building and 180 ft. along the short leg of the building. The width of the building perpendicular to the legs, is 80 ft. Refer to Figure 2.1 for building dimensions in plan.

Openings are provided at four locations in the floor slabs. Two 12 ft. x 40 ft. openings used for a combination stair and elevator shaft are provided in each leg of the building. Two 12 ft. x 18 ft. 9 in. openings for a stairway only are located at the corners of the 180 ft. section of the building. Refer to Figure 2.1 for opening locations.

Five roof openings are provided. Three 12 ft. x 18 ft. 9 in. openings for a stairway are located approximately in the corners and middle of the 260 ft. section of building.

The building has an overall height of 58 ft. The first story is 16 ft. in height and the remaining stories are all 14 ft. The first story was chosen to be higher to give additional clearance for an atrium or entry way.

As was stated in Chapter I, decisions regarding exterior siding or cladding were made as part of the structural design. For this project a brick veneer was chosen. Calculation of the brick veneer weights are given in Appendix A.

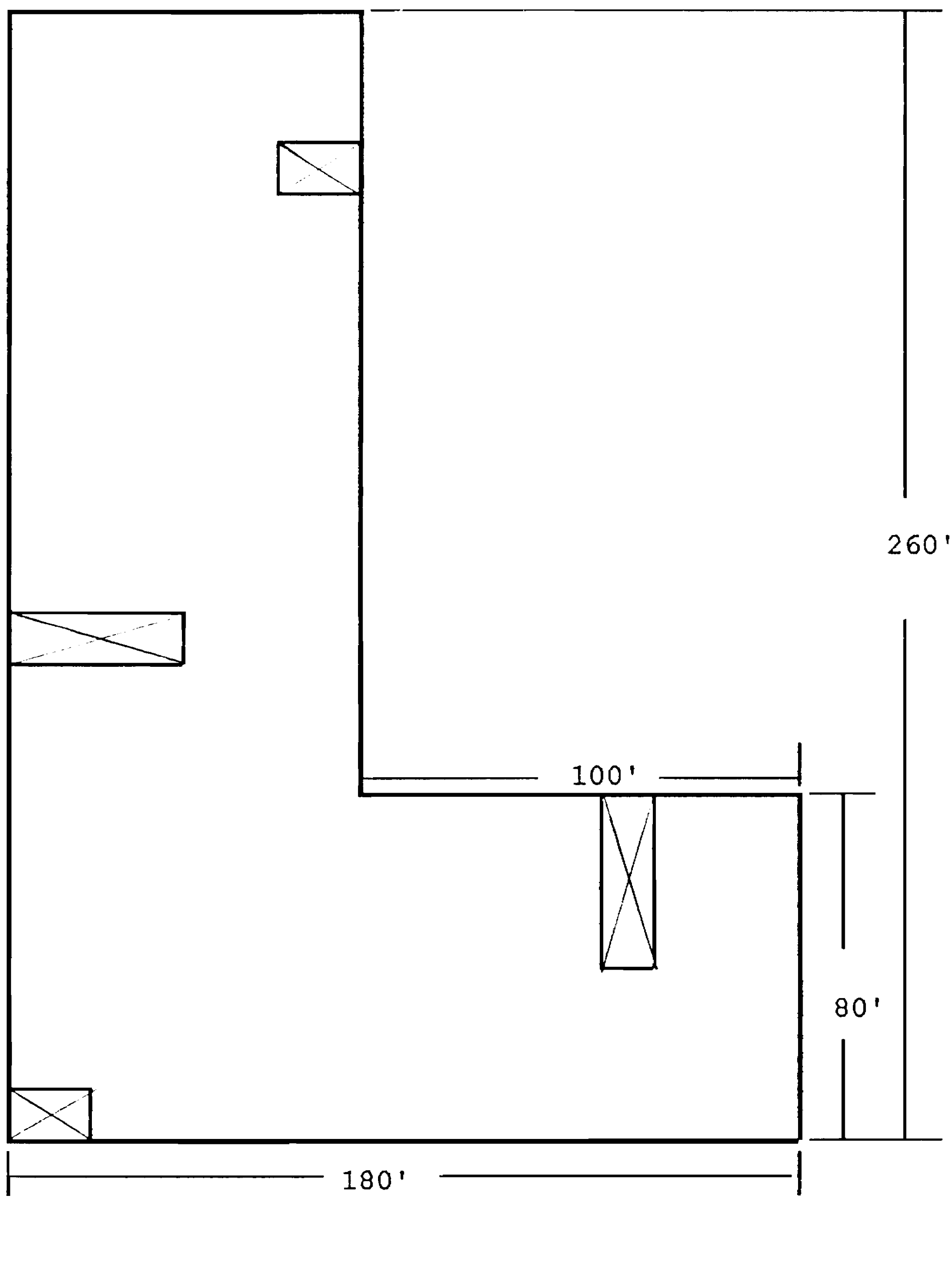

Figure 2.1

Building Plan

CHAPTER III

NON-SEISMIC DESIGN

The next phase of this project was the non-seismic structural design, for gravity loads and wind loads. The non seismic design portion of this project is divided into the following tasks.

1. Determination of framing configurations.
2. Design of floor systems.
3. Design of roof systems.
4. Design of lateral force resisting systems.
5. Pinned Column design.
6. Check of building torsion.
7. Design of partially restrained connections.
8. Design of fully restrained connections.

Design loads used in this project are calculated in Appendix A. The following sections describe the design considerations, assumptions and decisions made in each of the above phases.

3.1 Framing Configurations

After the establishment of overall building dimensions, and opening locations, bay sizes and column locations were determined. Basic bay sizes of 30 ft. x 40 ft., 40 ft. x 40 ft., and 33 ft 4 in. x 40 ft. will be used. Refer to figure 3.1 for framing configurations.

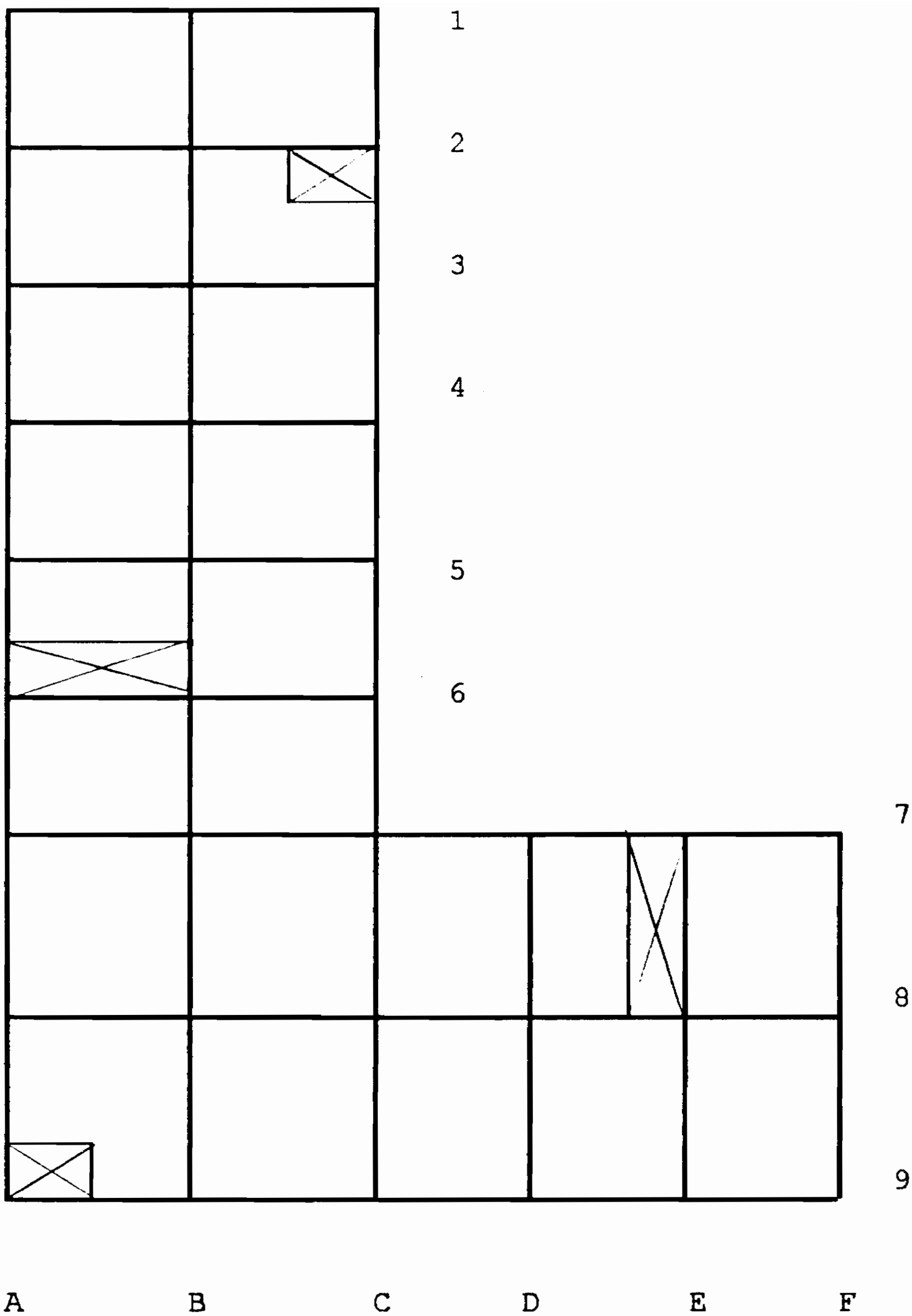

Figure 3.1

Framing Layout

The above bay sizes are based on typical configurations used in actual buildings. Spacing and orientation of beams and girders are discussed in Section 3.2.

3.2 Floor System Design

Design of the floor system, once bay sizes were selected, required a determination of an economical arrangement of beams and girders. Factors that effect the overall economy include material and labor costs for the beams and girders, fabrication cost, and the cost of providing shear connectors, if composite construction is used. Normally this process would require numerous trial designs and cost estimates.

On this project the Steel Deck Institute, SDI, PC-based, Floor Design Module [SDI 1991] was used to determine an economical design. Prior to designing the floor system the following decisions were made.

1. Selection of grade of structural steel.
2. Selection of type of concrete, either normal weight or lightweight.
3. Determination of maximum and minimum beam and girder depths.
4. Determination of unit material and labor costs.
5. Establishment of deflection criteria.
6. Selection of a minimum percentage of composite action.

For this design, A572 Grade 50 steel was selected for all floor beams and girders. Grade 50 steel was chosen because material costs are comparable to A36 steel and lighter sections are possible. Finally Grade 50 steel is becoming more prominent and will most likely be the only steel available in the future.

Two types of concrete were considered, normal weight, 150 pounds per cubic foot, and lightweight concrete, 115 pounds per cubic foot. As part of the optimization of the bay designs, both concretes were used in preliminary designs. The optimization of the bays will be discussed in more detail in this section.

Selection of minimum and maximum beam and girder depth was based on two factors. A minimum depth of 16 in. was used for all beams, sections shallower than 16 in. were not considered due to concerns about floor serviceability, vibrations and deflections.

Interior beams and girders were selected not to have a depth exceeding 24 in. A depth of 24 in. allows 2 ft. between the underside of the beam and suspended ceiling, which should be sufficient for the installation of heating ventilation and air conditioning systems, fire protection systems, and electrical systems. Restricting the maximum beam depth will also reduce the installation cost of exterior cladding. A maximum depth of 27 in. was used for spandrel beams, since there are no concerns about building utilities.

To determine the most cost effective interior bay design, unit material and labor costs are required. The following unit costs were used, and are based on current construction costs.

1. $ 0.22/lb - structural Steel
2. $ 1.75 ea. - shear connector installation
3. $ 200.00 - fabrication cost.

A maximum deflection of L/360 was used for the design of floor beams and girders. This deflection limit is for live loads, dead loads, and initial loads during construction. Deflections were calculated using service loads.

A minimum percentage of composite action of 25 percent was chosen. Although the AISC LRFD specification does not specify a minimum percentage, 25 percent is considered the practical lower limit.

Once decisions regarding steel grade, type of concrete, construction costs, and percentage of composite action were made the optimization and design of interior bays was accomplished. A total of six designs were completed for the three main bay sizes. The first three were done using a normal weight concrete, while the final three were made using lightweight concrete. Designs were done using the bay design module of the SDI program. Spacing of beams were varied between 96 in. and 120 in., with a maximum increment of 12 in. Light-weight concrete using composite construction was found to be the most economical. Refer to Table 3.1 for a summary of designs and structural steel cost.

Once it was decided to use light-weight concrete and composite construction, remaining beams and girders were designed. This included the design of framing around stairway and elevator openings, and spandrel beams. Framing around elevators was designed to support, in addition to all other loads, a 12 in. normal weight concrete masonry wall. A masonry wall was considered adequate for a 2 hour rated fire wall around the elevator shaft. Use of a 12 in. concrete masonry wall results in uniformly distributed line load of 1.2 kips per linear foot.

Two types of spandrel beams were designed. The first type is for cases parallel to the floor beams, and was designed using the SDI beam design module. The second was for the case of floor beams framing into the girder, and was designed using the SDI girder design module.

Table 3.1

Summary of Bay Designs

Bay Size	Conc. Weight	Girder Length	Girder Size	Beam Length	Beam Size	Cost
1	150 pcf	30'0"	W18x86 [0]	40'0" {2}	W21x50 [0]	$2680.
2	150 pcf	40'0"	W24x131 [0]	40"0" {3}	W21x50 [0]	$3913.
3	150 pcf	40'0"	W24x103 [0]	33'4" {3}	W18x40 [0]	$3080.
1	115 pcf	40'0"	W24x76 [26]	30'0" {3}	W16x26 [11]	$2478
2	115 pcf	40'0"	W24x103 [36]	40'0" {3}	W21x44 [16]	$3630
3	115 pcf	40'0"	W24x84 [12]	33'4" {3}	W18x35 [12]	$2902

[ ] - No. of Shear Connectors { } - No. of Beams

Bay Designation:

1. 30'0" x 40'0"
2. 40'0" x 40'0"
3. 33'4" x 40'0"

3.3 Roof System Design

Design of the roof system essentially followed the same procedure used in the design of the floor system. The non composite option of the SDI floor design program was used, with the depth of the concrete set equal to zero. Reduction of gravity loads due to wind uplift was not accounted for when proportioning roof framing. This decision was made since it is felt to be more conservative to design the roof framing for the maximum effect of snow loads and roof dead loads.

Roof level spandrel beams are designed to support roof dead loads and snow loads only, because these spandrel beams do not carry the weight of the exterior siding.

3.4 Lateral Force Resisting System Evaluation

Structural systems were designed to transfer the lateral loads, wind in the case of the non seismic design, through the building and into the buildings foundations. These systems also provide story stability or stabilizing of the pinned columns. Two methods are used in this building: concentrically braced frames and moment resiting frames. Other systems such as tubular frames, and eccentrically braced frames, although highly efficient, were not considered for a building of this size and height.

Each system has advantages and disadvantages that had to be considered when selecting methods to resist lateral loading. Bracing systems will be discussed first then moment

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Treatment of these secondary moments requires a more detailed analysis; procedures used in this project will be examined later in this section. Finally the relative stiffness must be considered when locating the frame in an unsymmetrical building due to building torsion.

Based on architectural considerations and building torsion, lateral loads and column stabilizing will be accomplished through a combination of three braced frames and two moment frames.

Braced frames based on their high relative stiffness, will be used in the short direction of the building. Specific locations are between columns column lines B2 and C2, A5 and B5, and E7 and E8. Moment frames will be used in the long direction of the building. As is explained in the discussions of building torsion Section 3.10., frames were designed in an effort to equalize the stiffness of all frames. Specific locations are between column lines A7 and F7, and B1 and B9. Refer to Figure 3.2 for frame locations.

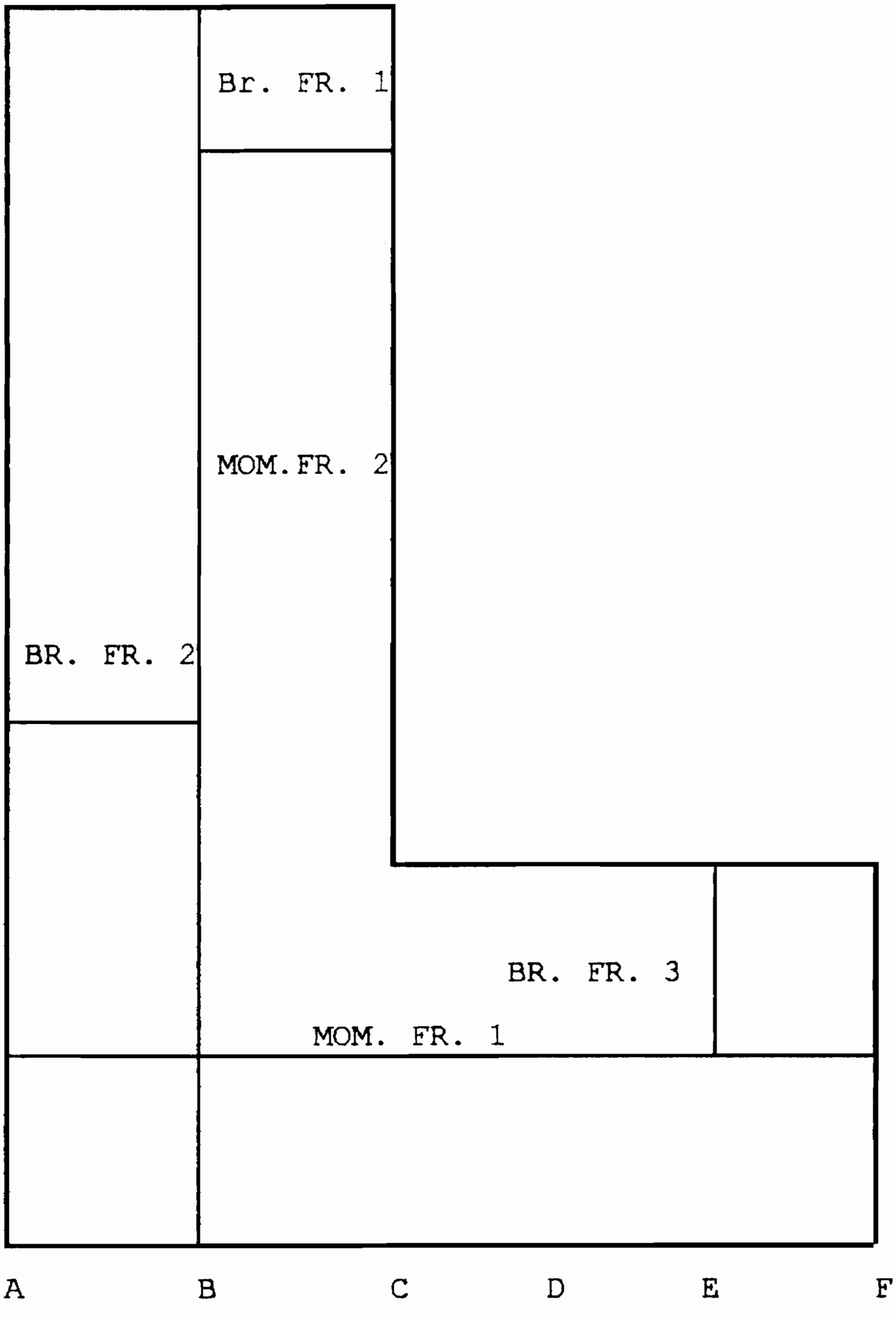


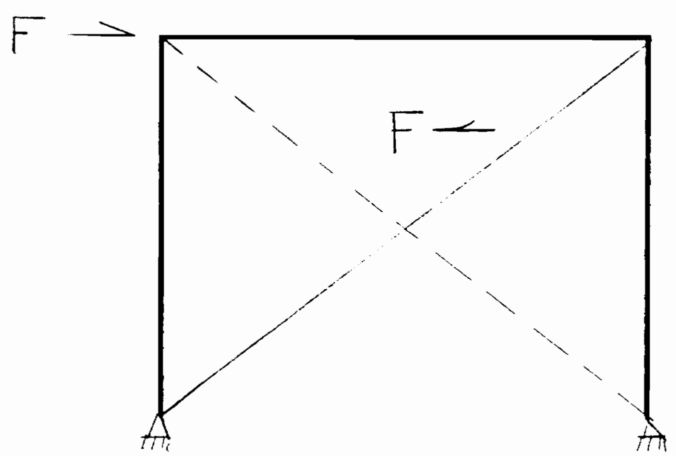
Figure 3.2

Frame Layout

3.5 Braced Frame Design

The first consideration for this stage of the design, was to select a bracing configuration. For this design cross bracing was selected. Cross bracing was chosen due to the simplicity of design, since members resist axial loads only. Other bracing systems such as V bracing or K bracing, require an interaction between braces and girders.

When using cross bracing, members can be designed using two basic assumptions. The first assumption is to design members to resist tensile loads only, therefore allowing the opposite member to buckle elastically. If this assumption is made one member must carry the entire lateral load, in tension. Figure 3.3 illustrates this assumption.



- - - Compression Member

Figure 3.3

Tension System

The second assumption is to design bracing members to resist both compression and tension. Using this assumption each member is assumed to carry one half of the lateral load. Members would usually be designed for the compressive force and would be checked for tensile capacity. Figure 3.4 illustrates this assumption.

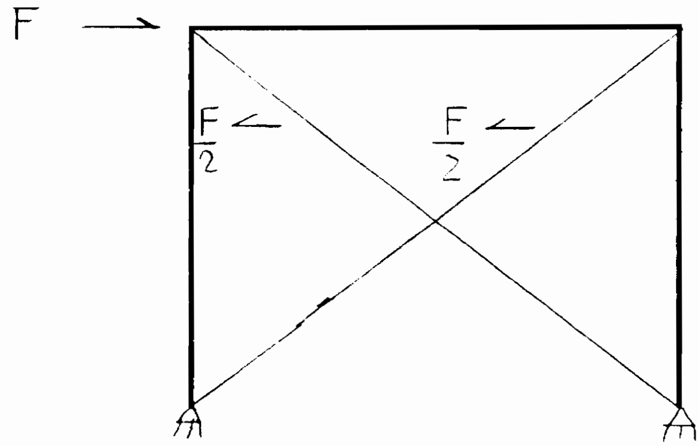


Figure 3.4

Compression/Tension System

For this project a tension system was designed for all three braced frames. Use of a combined compression and tension system would result in a system with a much higher stiffness than moment frames, which would increase building torsion. Torsion considerations will be examined in more detail in Section 3.10. Figure 3.5 illustrates the configuration used for Braced Frame 1, the remaining frames are similar.

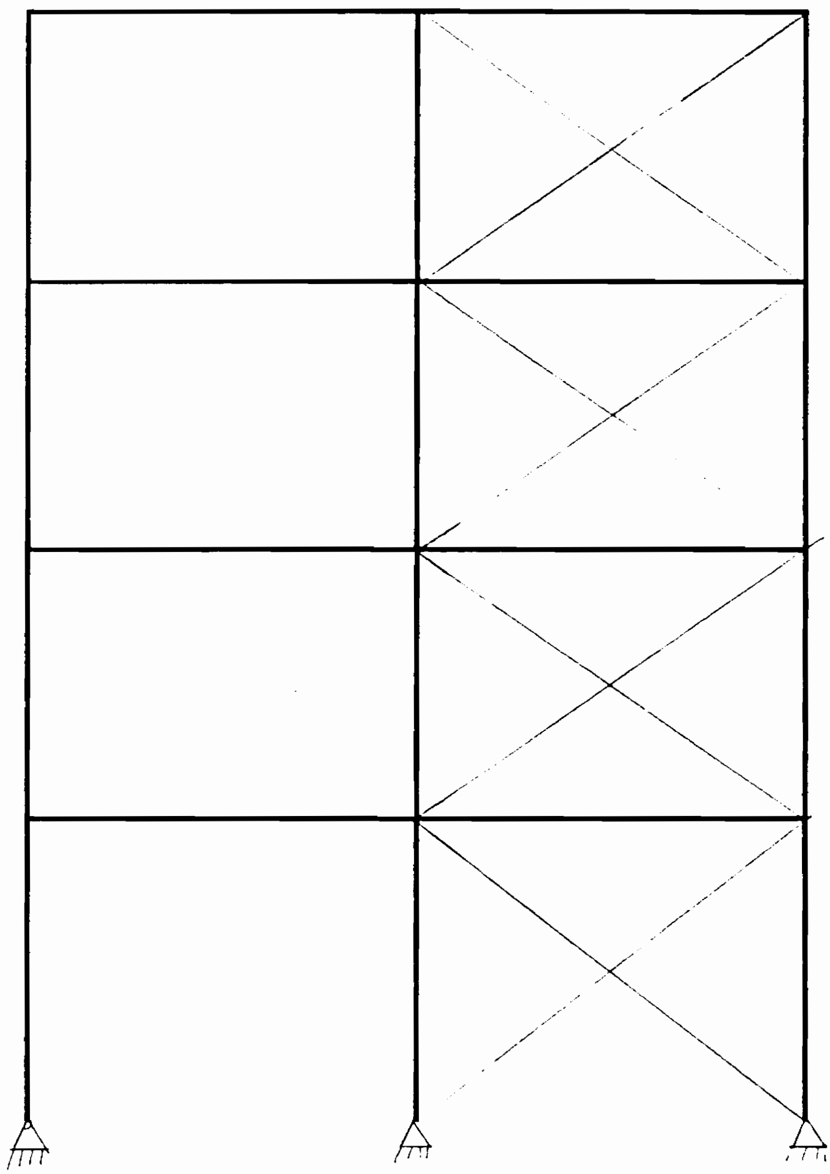


Figure 3.5

Braced Frame 1

Design of bracing members involves four steps as outlined below.

1. Calculation of applied wind loads
2. Calculation of the summation of column loads
3. Calculation of member forces due to wind and stabilizing forces.
4. Calculation of stiffness to resist sway buckling.
5. Proportioning of members to satisfy items 3 and 4 above.

Determination of wind loads was accomplished by applying the design wind pressure to the tributary area. A tributary spacing of 90 ft. was used for braced frames 1 and 2, and a 100 ft. was used for braced frame 3, tributary areas are highlighted on Figure 3.6, for east/west loading and on Figure 3.7 for north/south loading. The wind loads were assumed to act as a concentrated loads applied at frame joints, loads were applied at both the windward and leeward sides of the frame. Wind loads are transferred to either the braced frames or moment frames through diaphragm action. Diaphragms consist of concrete floor slabs and metal roof decking. Refer to Appendix A for magnitudes and locations of applied loads.

The summation of columns loads are required to design members to provide resistance to sway buckling. Summation of column loads were calculated for two LRFD load combinations.

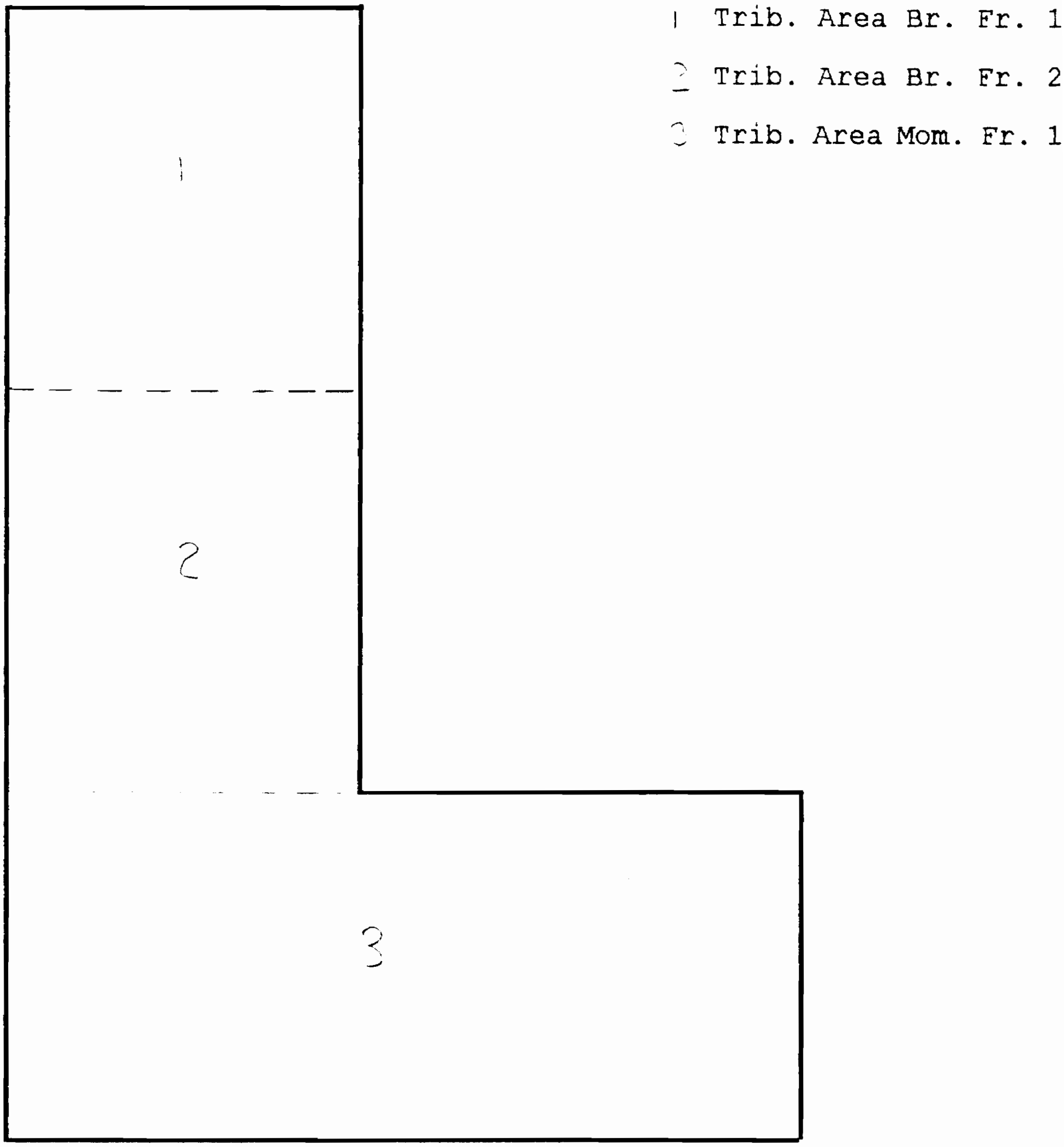


Figure 3.6

Tributary Areas East/West Loading

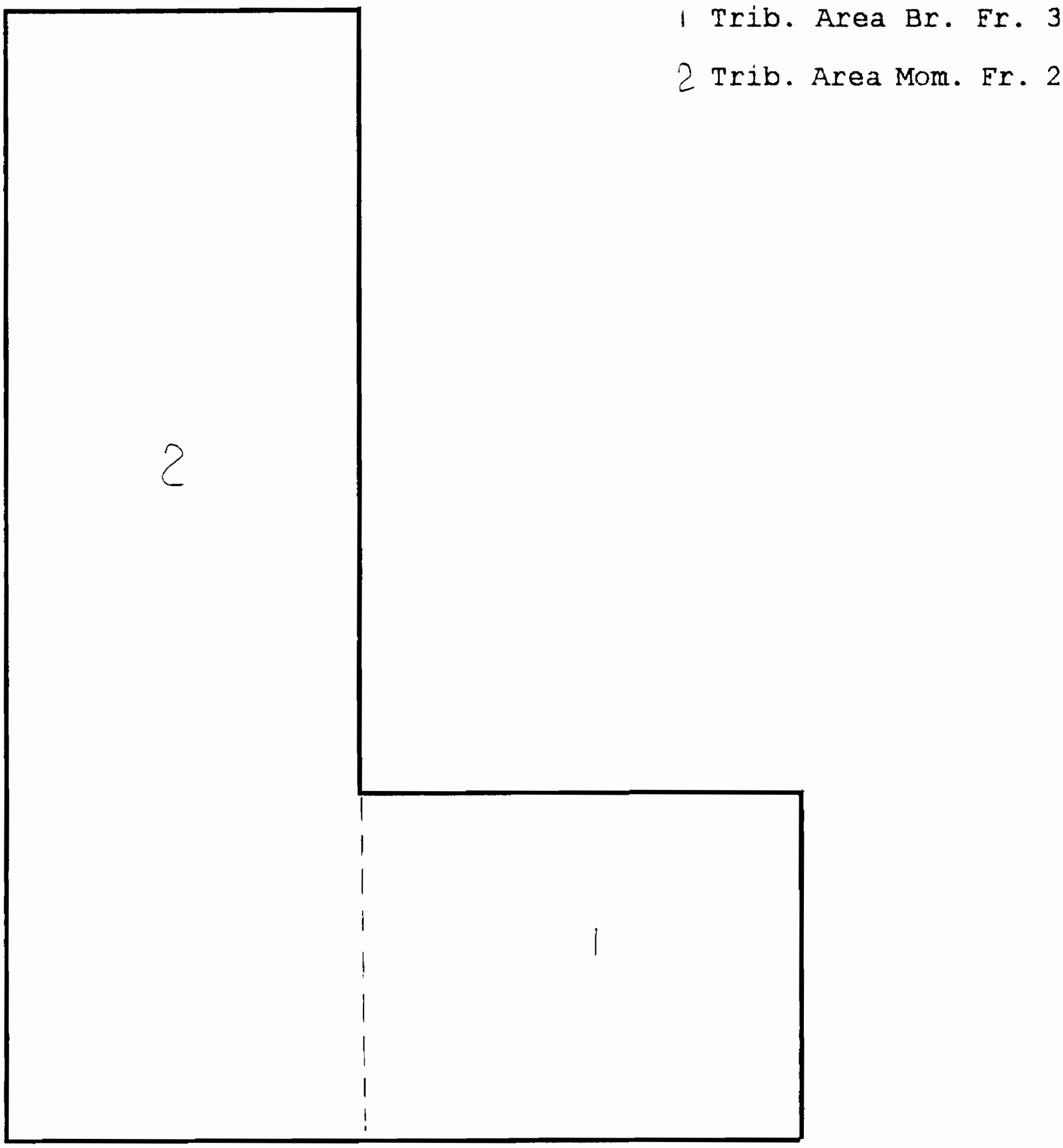


Figure 3.7

Tributary Areas North/South Loading

$$1.2D + 0.5L + 0.5S + 1.3W \qquad \text{(AISC LRFD A4-4)} \qquad (3.1)$$

$$1.2D + 1.6L + 0.5S \qquad \text{(AISC LRFD A4-2)} \qquad (3.2)$$

Once the wind loads and summation of column loads were calculated, member forces were calculated. Member forces are comprised of two components, a lateral or wind load component and a stabilizing component.

The lateral load component is calculated as follows.

$$F_w = \Sigma F_h / \cos \theta \qquad (3.3)$$

where

F_w = Member force due to wind load

ΣF_h = Summation of horizontal wind loads, windward and leeward above the member under consideration.

θ = Angle measured clockwise from the bracing member to the horizontal.

Calculation of the stabilizing component uses methods recommended by Galambos [1968] and is given by.

$$Fs = 0.004 \times \Sigma Pu \qquad (3.4)$$

where

Fs = Stabilizing component

ΣPu = Summation of column loads

The summation of column loads was calculated using load combination number A4-4.

Calculation of the required stiffness is based on summation of the column loads using load combination A4-2, as recommended by Galambos [1968]. The required stiffness, β_s is given by.

$$\beta_s = 2 \times \Sigma Pu / h \qquad (3.5)$$

where

Σ Pu = Summation of column loads

h = Story Height

Members were proportioned to resist the forces calculated above, using load combination A4-4, and checked for the minimum stiffness calculated using load combination A4-2.

During the design of bracing members it was assumed that connections would be made with one row of 3/4 inch A325N bolts. Therefore a reduction coefficient of 0.85 as specified by paragraph B3 of the AISC LRFD specification will be used. All designs were made using A36 structural steel. Therefore the tensile capacity of the member is the minimum of the following.

1. $T_n = 0.90 \times A_g \times F_y$ (LRFD D1-1) (3.6)
2. $T_n = 0.75 \times A_e \times F_u$ (LRFD D1-2) (3.7)

where

T_n = Tensile Capacity

A_g = member gross area

A_e = Effective area, 0.85 Anet

F_y = Yield stress of member

F_u = Rupture strength of member

The minimum area required to satisfy stiffness requirements is calculated as.

$$A_{br} = \frac{\beta_s \times L_{br}}{\phi \times E \times Cos\ \theta^2} \qquad (3.8)$$

where

A_{br} = Minimum gross area required for stiffness

β_s = Calculated stiffness

L_{br} = Length of bracing member

ϕ = Resistance factor

E = Modulas of Elasticity, 29,000 KSI

θ = Bracing Angle

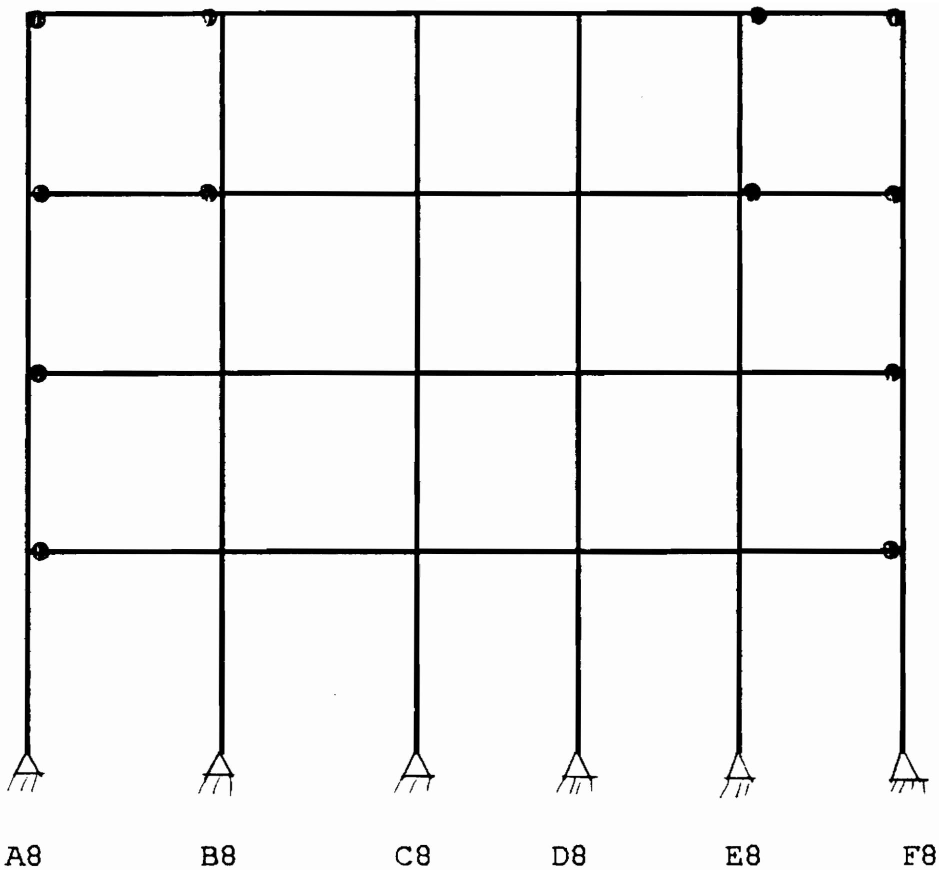
The gross area of the member proportioned to resist tensile and stabilizing forces must be greater than or equal to the area calculated using the above procedure.

3.6 Moment Frame Design

The design of the two moment frames, with the exception of connections, was accomplished in the following manner.

1. Determine initial frame configuration
2. Determine loadings, dead, live, and wind
3. Selection of trial sizes
4. Frame analysis
5. Check member capacity
6. Check story stability
7. Check building drift.
8. Revise member sizes, if required

When selecting suitable frame configurations one of the primary concerns was avoiding biaxial bending of columns. If both frames, which are perpendicular to each other, were rigidly connected, biaxial bending would occur. Members can be proportioned to resist biaxial bending, but member sizes would be substantially increased. In addition, the design and subsequent fabrication and construction of connections would be more complicated. Based on the above concerns, initial configurations were conceived for each frame, and as explained in Section 3.10 modifications to the layout of moment frame 1 were made. Refer to Figure 3.8 for Moment Frame 1 configuration and Figure 3.9 for the configuration of Moment Frame 2.



● - Pinned Joint

Figure 3.8

Moment Frame 1

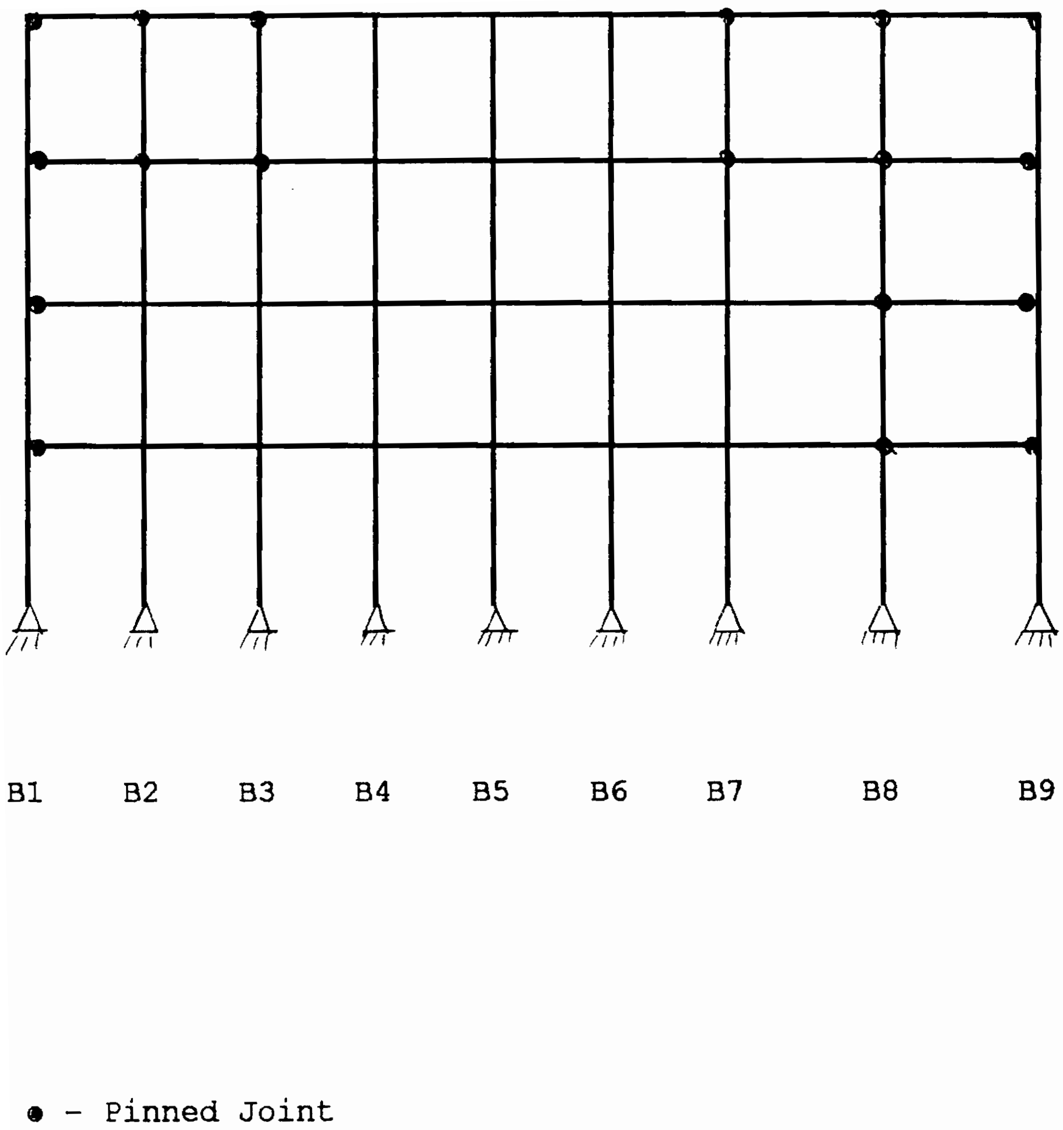


Figure 3.9

Moment Frame 2

The initial frame configurations did not use fully restrained connections at all locations. Pinned joints were used for the connections of beams to exterior columns and the connection of Moment Frame 2 with Moment Frame 1. In addition pinned joints were used at the third and fourth stories to reduce the number of bents resisting lateral loads to three. The number of bents was reduced since the total lateral load at the top stories is less than the lower stories.

Both frames are designed to resist gravity and lateral loads. Gravity loads include floor and roof dead loads, floor live loads, and snow loads. Magnitudes of gravity loads are found in Appendix A. Beams in Moment Frame 1, support a 10 foot wide tributary spacing along length of the frame. Frame columns support girder reactions, perpendicular to the frame, in addition to beam reactions. Lateral loads, for this design, consist of wind only. Design wind pressures are applied to the tributary area for each frame and concentrated loads are applied at the frame joints. Each frame has a tributary spacing of 80 ft., and therefore the wind loads are the same as for Braced Frames 1 and 2, as detailed in Appendix A.

Prior to completing an initial frame analysis, trial sections were picked for the columns and beams.

Initial column sections were picked by designing a member to carry an axial load equal to twice the actual axial load that would be applied. This is an approximation to take into account the end moments at the beam to column, connection. Members were initially designed assuming an effective length factor for the x axis, Kx ,of 1.75, and a value of 1.0 for the effective length factor for the y axis, Ky.

The selection of initial beam sizes followed much the same procedure. Slightly larger sections than picked in the design of the floor system were chosen. Section sizes were increased as an approximation of the effect of increased bending moments under lateral loading. For floor beams, a W 27 x 94 section was picked, and a W 27 x 84 section was picked for the roof beams. Deeper sections than previously selected during the floor design, were chosen to increase the moment of inertia, thereby reducing the effective length factor in the direction of bending, which also helps control building drift.

To take into account the P Delta effects, the AISC LRFD specification allows three methods of analysis. Either a second order elastic analysis using factored loads or first order elastic analysis using moment amplifiers can be used or a plastic analysis can be used. For this project a first order elastic analysis using moment amplifiers was used. Determination of the ultimate moment is done using the methods outlined below.

$$Mu = B_1 \times M_{nt} + B_2 \times M_{lt} \qquad (3.9)$$

where

B_1 = Moment amplification factor assuming no lateral translation of the frame.

B_2 = Moment amplification factor for frame free to translate

M_{nt} = Required flexural strength assuming no lateral translation of the frame.

M_{lt} = Required flexural strength as a result of lateral translation of the frame.

Values of the amplification factors depend on the magnitudes of axial loads, Euler buckling load, member end moments, and axial capacity of members resisting sway. Calculation of the amplification factors is detailed below.

$$B1 = \frac{Cm}{(1 - P_u/P_e)} \geq 1.0 \qquad \text{(AISC LRFD H1-3)} \qquad (3.10)$$

where

P_u = Required axial strength

P_e = Euler buckling load, $K < 1.0$

C_m = Moment averaging factor

Values of C_m are given in paragraph H1.2.a of the LRFD specification, and depend on the end moments and support conditions.

And

$$B_2 = \frac{1}{1 - \frac{\Sigma P_u}{\Sigma P_e}} \quad \text{(AISC LRFD H1-6)} \qquad (3.11)$$

where ΣP_u = Summation of required axial strength of columns resisting sway.

ΣP_e = Summation of Euler buckling loads in the axis of bending

Effective length factors, in the axis of bending, were calculated for two conditions. First, for joints fixed against translation and second for joints free to translate. The effective length factor was taken as 1.0 for the first case. Alignment charts given in Chapter 2 of the AISC LRFD manual were used to calculate effective length factors in the axis of bending for the unbraced condition. Stiffness reduction factors were not used in calculation of the effective length factors. Neglecting the reduction of column stiffness results in a more conservative design. Adjustment factors were applied to the calculation of beam stiffness. For the case of a far end not rigidly connected a 3/6 reduction factor was applied to the stiffness of the beam. Determination of M_{nt} and M_{lt} required two separate analyses to be conducted. An analysis with the frame restrained from translation was performed to determine magnitudes of M_{nt}. To determine M_{lt} an analysis with the frame free to translate was performed. Using the principle of superposition of elastic structures, reactions at the artificial restraints are applied to the unrestrained frame in a direction opposite of the directions calculated in the restrained analysis.

Because column end moments have been amplified, adjustments to beam end moments, with joints free to translate, are required. The total additional beam moment is calculated as follows.

$$\Delta Mom_{tot} = \sum M_{lt} \times (B2-1) \quad (3.12)$$

The additional moment is distributed to the beams in proportion to the stiffness of the beams:

$$\Delta Mom_{bm} = \frac{(I/L)}{\Sigma(I/L)}$$

where I/L = Stiffness of beam to which additional moment is distributed.

$\Sigma I/L$ = Summation of stiffness of beams framing into joint.

Verification of member capacity is done using the interaction formulas given in section H1 of the LRFD specification. Use of either equation H-1a or H-1b requires the calculation of the nominal compression capacity, and nominal flexural capacity for the member under consideration.

If $Pu/\phi Pn \geq 0.2$

$$P_u/\phi P_n + 8/9\ (M_u/\phi M_n) \leq 1.0 \quad \text{(AISC LRFD H-1a)} \quad (3.13)$$

If $P_u/\phi P_n < 0.2$

$$Pu/2\phi Pn + Mu/\phi Mn \leq 1.0 \quad \text{(AISC LRFD H-1b)} \quad (3.14)$$

where

ϕ = Resistance factor

M_n = Flexural Capacity

P_n = Compression Capacity

The above interaction formulas were checked using electronic spreadsheets. The spreadsheets that were developed calculated axial capacities, Euler buckling loads, and amplification factors once member properties were input. Once amplification factors were calculated and axial loads and moments input, the interaction equations were checked. Copies of these spreadsheets are found in Appendix B.

Just as in the case of braced frames both moment frames in addition to supporting gravity and lateral loads provide stability against story sway buckling. Story stability is achieved if the summation of column loads in a story is less than or equal to the summation of the compression capacities of columns resisting sway. Story stability was a controlling limit state in the initial design of both moment frames. This was based on gravity loading only, using LRFD Load Combination A4-2.

$$1.2D + 1.6L + 0.5S \quad \text{(AISC LRFD A4-2)} \qquad (3.14)$$

Maximum building drift, from wind loads, is not specified by the BOCA National Building Code. For this building a maximum drift of 0.4 percent of the building height was chosen. This value is typical for a speculative office building. Actual drift and drift indexes were calculated for both moment frames and are tabulated in Tables 3.2 and 3.3.

A review of Moment Frame 2 results indicates that approximately half of the drift occurs in the first story of both frames. The drift in the first story is acceptable for the following reason. Drift was calculated based on supports free to rotate. Actual anchorage of columns to the footings will provide some degree of rotational restraint. calculations.

Table 3.2

Moment Frame 1 Drift

Story Level	Height	Story Drift	Drift Index
4	14'0"	0.10"	0.0006
3	14'0"	0.23"	0.0014
2	14'0"	0.27"	0.0016
1	16'0"	0.34"	0.0020

Total Drift = 0.94"

Drift Index = 0.0013

Table 3.3

Moment Frame 2 Drift

Story Level	Height	Story Drift	Drift Index
4	14'0"	0.13"	0.0008
3	14'0"	0.30"	0.0018
2	14'0"	0.22"	0.0013
1	16'0"	1.07"	0.0056

Total Drift = 1.72"

Drift Index = 0.0025

3.7 Column Design

Columns that are not part of the moment resisting frames described in Section 3.6 were designed as pinned columns with the effective length factors for both the x an y axes taken as 1.0. Bracing of columns is achieved through moment frames and braced frames. Live load reductions were used in accordance with BOCA National Building Code provisions.

Column load tables given by AISC LRFD specification were uses for member selection. All columns use A572 grade 50 structural steel. Three sections were chosen for each column, a W10, W12, and W14. After all members were selected the total weight of all columns in each section depth was calculated. The final depth of column, was picked based on the overall least weight. It is considered desirable in a building of this size, although not required, to use the same nominal depth of column through out the building. This will result in a simplified column splice which would be more economical. For this design W10 sections had the least overall weight.

Column splices are located at a height of 33 feet, three feet above the second floor. Locating the splice about three feet above a floor slab has two advantages. First, a splice located at the slab level would complicate the connection of floor beams/girders to supporting columns. Second, splicing the columns about three feet above the a floor gives a working platform, the metal decking, for the ironworkers. This results in reduced labor costs and a safer operation.

Tentative members selected and final selections are found in Appendix B.

3.8 Type PR Connections

Partially restrained (PR) connections are used for all connections not located in Moment Frames 1 and 2. Connections are designed to resist only beam end shears, and are assumed not to resist moments, that is are assumed capable of unrestrained rotation.

Connections were designed using the CONXPRT connection Module, LRFD version, [AISC 1992]. Prior to design of connections it was necessary to select the type of connections to be designed. Possible choices fall into three categories, as follows.

1. Framing Angles
    a. Bolted both sides
    b. Bolted at beam side
    c. Welded both sides
    d. Welded at beam side only
2. Single Plates (Shear Tabs)
3. Shear End Plates

A qualitative evaluation was made for the connection of beams to girders, beams to columns, and girders to columns. Two primary factors were used in selecting connection types; ease of erection and ease of fabrication. Single shear plates are used for the connection of floor and roof beams to girders. Shear plates are shop welded to the girders and the beams are field bolted to the shear plates. When beams are back to back, shear plates can reduce erection costs and reduce risks for the ironworkers making the connection. Once a beam is bolted to a shear plate, unbolting is not required when making the opposite beam connection, which reduces both cost and hazards.

If framing angles, welded to the beam or bolted to the beam and girder web, were used, unbolting would be required when making the opposite beam connection.

Framing angles, beam welded and column bolted, were used for the girder to column connections. Connections are to the column flange or to the column web, depending on the location in the building. This connection was chosen primarily for ease of fabrication. Because shear plates are being welded for the beam to girder connection, it was decided to weld the framing angles to the girder. Therefore only welding is done on the member as opposed to welding and punching bolt holes. Unbolting of opposite connections, when connected to column webs, will be required but not nearly as much as if framing angles were used for the beam to girder connection.

Framing angles, beam welded and column bolted were used for the connection of spandrel beams to columns, either flange or web. Single shear plates were considered but due to tight clearances at column webs were rejected.

Shear end plates were not considered for any connections. End plates could only have been used if both beams and girders were always fabricated to their exact lengths.

Both the single plate and framing angle connections were designed using A36 structural steel and A325N bolts. The AISC Manual method, which is conservative, was used for the design of the single plate connections. Part of the connection design was the check of strength limit states for the beam, connection plate or framing angles, and the girder or column. Specific checks are given in Appendix C.

3.9 Type FR Connections

Fully restrained connections (FR) were used for the beam to column joints in Moment Frames 1 and 2. Connections were designed to transmit both moments and shears. The critical load combination for both frames was either,

$$1.2D + 0.5L + 0.5S + 1.3W \quad \text{(AISC LRFD A4-4)} \qquad (3.15)$$

or

$$1.2D + 1.6L + 0.5S \quad \text{(AISC LRFD A4-2)} \qquad (3.16)$$

Lateral loading generally controlled in the lower stories and full gravity loading controlled in upper stories. Wind load was applied in both principle directions of each moment frame to obtain critical moments.

Connections were designed using the CONXPT moment connection module. The moment design module uses American

Institute of Steel Construction, Allowable Stress Design specification [AISC 1989]. Adjustments in factored moments are required. For gravity loading, factored moments are divided by 1.5 and for wind loading factored moments are divided by 1.14.

Critical factored moments and adjusted moments are tabulated in Appendix C.

Flange welded connections were used for this project. Beam moments are resolved into flange forces, and these forces are transferred to the column flange through use of full penetration groove welds. End reactions are transferred to the column flange through the use of shear plates welded to the column and field bolted to the beam.

Strength limit states associated with the beam, connecting elements, and the column were checked during the design of connections. Requirements for column stiffeners were identified during the design of each connection, when stiffeners were required the CONXPRT module was also used to design stiffeners. Column stiffeners are required when one or more of the following limit states are exceeded.

1. Local Flange Bending (AISC ASD K1-1)
2. Local Web Yielding (AISC ASD K1-2)
3. Web Crippling (AISC ASD K1-4)
4. Web Compression Buckling (AISC ASD K1-8)

Connections were checked using the Load Resistance Factor Design specification. Electronic spreadsheets were developed to verify the capacity of connections, and to check strength limit states.

A summary of designs, capacity verification, and check of strength limit states is tabulated in Appendix C.

Requirements for doubler plates had to be checked during the design of fully restrained connections. Doubler plates were required if the panel shear capacity of the column was less than the calculated panel zone shear. Beam moments are resolved into forces acting at the beam flange and added to the story shear, above the beam to column joint, to calculate the panel zone shear. Moments were calculated using the following load combination.

$$1.2D + 0.5L + 0.5S + 1.3W \quad \text{(AISC LRFD A4-4)} \qquad (3.17)$$

with the wind load applied in both directions of each moment frame. Panel zone shears were calculated for each direction of loading.

Panel shear capacity is calculated as follows.

$$\phi Vn = 0.6\ Fyw\ Aw \quad \text{(AISC LRFD F2-1)} \qquad (3.18)$$

where

Fyw = yield stress of column web

Aw = area of column web

Doubler plates are required if the maximum panel zone shear calculated above exceeds the panel shear capacity. The total additional plate thickness is calculated as detailed below.

$$tpl = \frac{PZS - \phi Vn}{\phi \; 0.6 \; d} \qquad (3.19)$$

where

PZS = Panel Zone Shear

ϕVn = Panel Zone Shear Capacity

ϕ = Resistance Factor

d = Column Depth

Panel zone shears, shear capacity, and required additional plate thickness was calculated using electronic spreadsheets copies of which are included in Appendix C.

3.10 Building Torsion

Since the building under design is not symmetric, torsion will occur under wind loading. The magnitude of torsion is dependent on the eccentricity of the centroid of the applied load and the center of stiffness of the structural systems used to resist lateral loading. Torsion will cause additional forces to be applied to the lateral force resisting system, and cause additional displacements.

Treatment of torsion for this project required that the eccentricity between center of stiffness and the centroid of the applied load be reduced as much as possible. Ideally it would be desirable to eliminate eccentricities, but is not possible in this building.

The following, outlines the methods used for the treatment of building torsion.

1. Calculation of centroid of applied loads, along both axes of the building.
2. Calculation of frame stiffness.
3. Calculation of eccentrities, along both axes of the building.
4. Frame revisions.
5. Calculation and distribution of additional forces.
6. Recheck member capacities.

To assist in the calculation of centroids and eccentricities, a building coordinate system was established as shown in Figure 3.8.

The centroid of the wind load when applied in the X direction is at a location of 130 ft. above the X axis. For wind applied in the Y direction the centroid is located a distance of 90 ft. to the right of the Y axis.

For each frame a unit load of 1 kip was applied to each frame and the lateral deflections were calculated. The relative stiffness for each frame is taken as the reciprocal of the lateral deflection. These values are tabulated in Appendix D.

Based on the preliminary frame designs, the center of stiffness was located at coordinates of (92.97ft,149.12ft), which results in eccentricities of 2.97 ft and 19.12 ft. in the X and Y directions. Calculation of the center of stiffness is shown in Appendix C.

To reduce the eccentricity in the X direction the stiffness of Moment Frame 1 was increased by having all members of the frame rigidly connected. This moved the center of stiffness to coordinates of (92,97ft.,129.63ft.), which results in eccentricities of 2.97 ft. and -0.37 ft. in the X and Y directions. The final center of stiffness is shown on Figure 3.10.

Additional forces caused by torsion are distributed to each frame in proportion to the frame stiffness and distance from the center of stiffness. These additional forces are calculated with the wind applied in the positive and negative X and Y directions. Torsional forces are calculated at each story level. Calculation of additional forces is done using the equation below, as recommended by Ambrose and Vergun [1987].

$$V_t = \frac{(W)(e)(d)(R)}{\Sigma R\ d^2} \qquad (3.20)$$

where

V_t = additional torsional force

W = total wind load, applied at centroid

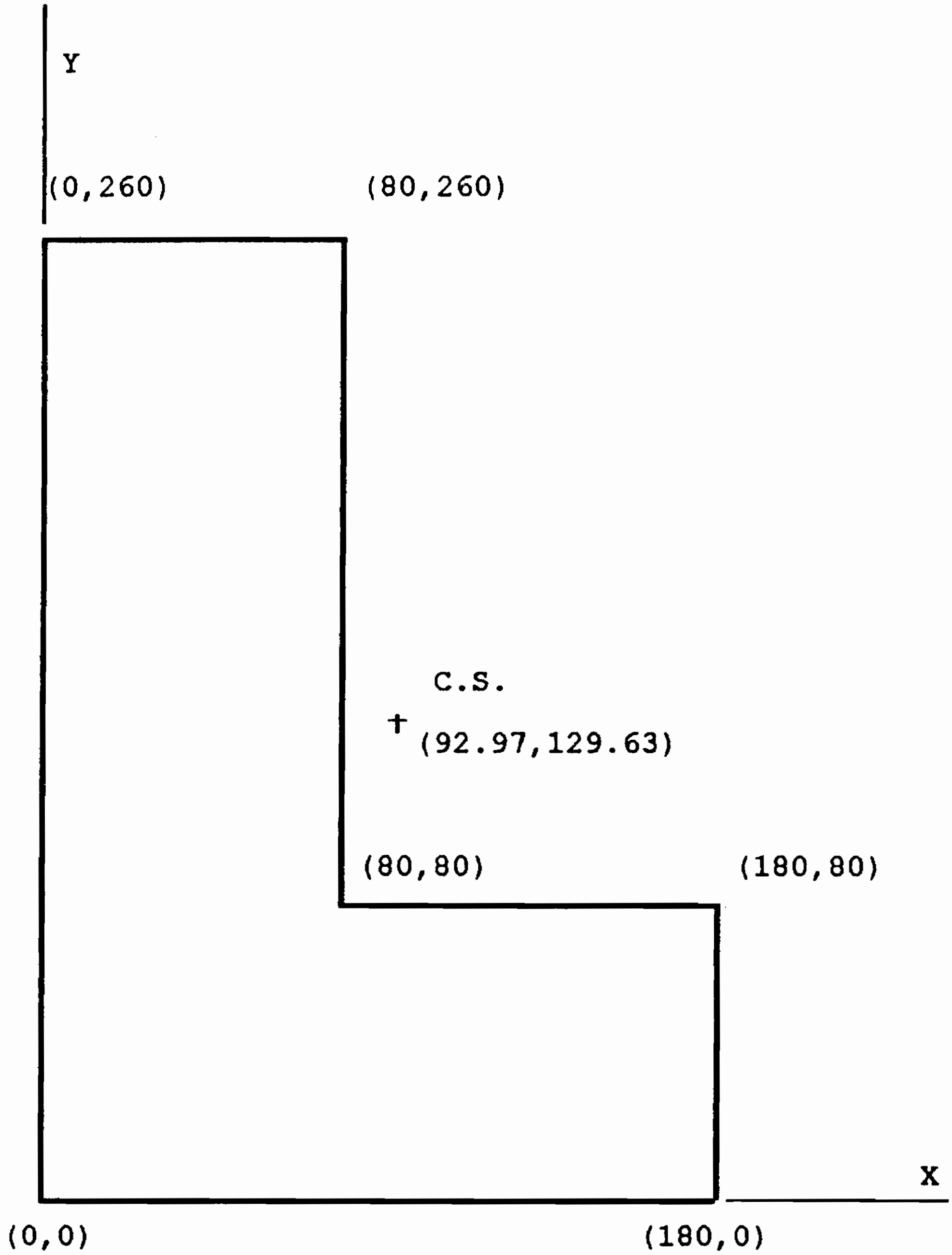
e = eccentricity

d = distance from frame to center of stiffness

R = relative stiffness of frame under consideration

ΣRd^2 = Summation of relative stiffness times distance to center of stiffness squared for all frames in building.

All frames were reanalyzed after the calculation of torsional forces and member capacities verified. The additional forces did not cause any member capacity to be exceeded.



C.S. - Center of Stiffness

Figure 3.10

Center of Stiffness

CHAPTER IV

SEISMIC DESIGN

The next phase of this project was the seismic structural design for lateral seismic forces. Design of floor and roof systems are not effected by the application of seismic forces and therefore no further designs will be completed. Lateral force resisting systems will be of the same type and be located in the same locations as the non- seismic design. The seismic design is divided into the following tasks.

1. Seismic analysis.
2. Review of AISC LRFD seismic design specification.
3. Braced frame design.
4. Moment frame design.
5. Design of fully restrained connections.

4.1 Seismic Analysis

The first phase of the seismic analysis is the selection of the method of analysis. Selection of a method is dependent on building code provisions and building geometry.

The BOCA code prescribes two methods of analysis, a static lateral force procedure and a dynamic analysis.

The static lateral force method involves calculation of a horizontal base shear. Magnitude of the base shear is dependent on ground acceleration, method of resisting lateral loads, building period, building use, soil conditions, and building weights. The base shear is distributed to each story of the structure in proportion to the story height and weight. Use of the static lateral force method is limited to regular structures, per limitations of the BOCA code.

For the case of irregular structures Paragraph 1113.5.3 of the BOCA code states that "the distribution of the lateral forces in structures that have highly irregular shapes, large differences in lateral resistance or stiffness between adjacent stories, or other unusual structural features shall be determined considering dynamic characteristics of the structure". The above definition gives a qualitative definition of irregular structures but does not give a quantitative definition. Thus, it was necessary to determine a quantitative definition of an irregular structure.

For the purposes of this structure, irregularities as defined by the Structural Engineers Association of California (SEAOC) and stated in the Seismic Design Handbook [Naeim 1989] will be used. Only those definitions which apply to this project will be discussed.

The building under design is irregular due to plan irregularities. The projection of the building past reentrant corners is greater than 15% to 20% of the plan dimension. Figure 4.1. illustrates this definition. The plan dimension is 260 ft. and therefore the ratio of A/L is equal to

100ft/180ft, or 0.56, which exceeds the maximum SEAOC value of 0.20. Based on this definition the static lateral force method was not used for the determination of seismic forces.

Therefore, a dynamic analysis was performed using three dimensional finite element analysis. A commercial finite element program, SAP 90, was used. The following steps were used in conducting this analysis.

1. Calculation of building mass.
2. Calculation of center of mass.
3. Calculation of mass moment of inertia.
4. Construction of analysis model.
5. Determination of response spectrum.
6. Analysis of structure.

Building mass was calculated at each story level by multiplying the total building dead load by the acceleration due to gravity, which is equal to 32 ft./sec^2. At the roof level the total snow load was included as part of the building weight.

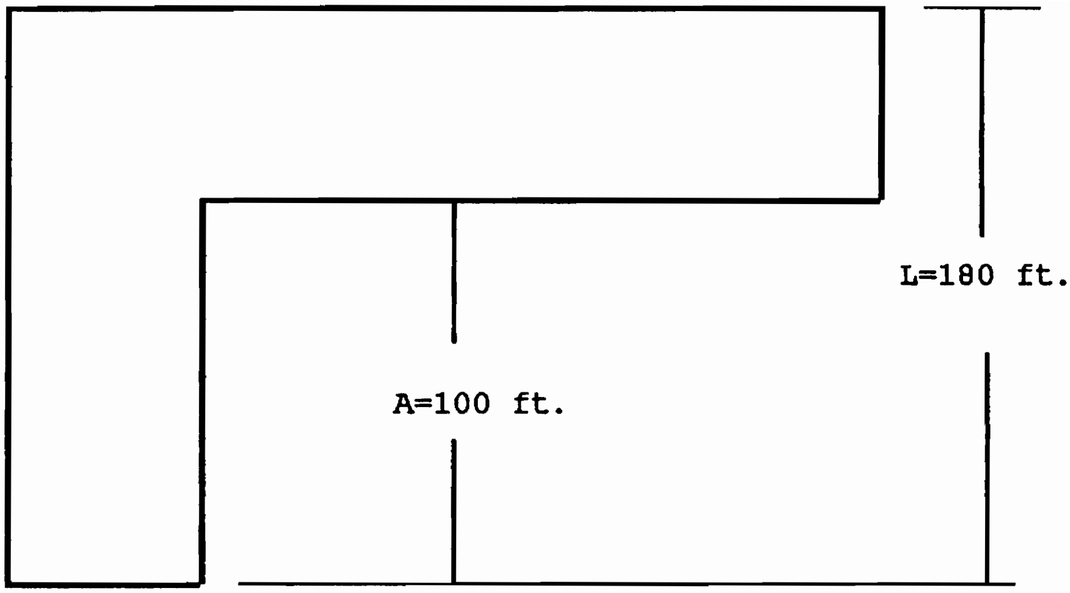


Figure 4.1

Plan Irregularity

The total building dead load was assumed to be uniformally distributed for the purposes of these calculations. This was believed to be a reasonable approach since the BOCA code requires that the effects of an accidental 5 percent eccentricity in the center of mass be investigated. Calculation of the center of mass and mass moment of inertia are found in Appendix E.

Construction of the analysis model required modeling of each lateral force resisting element and the diaphragms that transfer the lateral loads. Braced frames were modeled using frame elements, initial member sizes were assumed and revisions made during design. Columns were modeled to transfer axial loads only with no moments about the principle axes of the column. Diagonal members were also modeled for the transfer of axial load only. Floor and roof beams were modeled as simply supported beams, capable of transferring end shear reactions only.

Frame elements were also used for the moment frames; initial member sizes were the members selected during the non seismic design. Columns were modeled to transfer both axial loads, shears, and moments around the strong axis of the member. Girders were modeled to be rigidly connected to the columns and therefore capable of transferring end shear and moment reactions.

The final step in the construction of the analysis model was the diaphragms. The modeling of diaphragms using SAP 90 is a relatively straight forward process. Master joints were created at each story level and the mass at each story level lumped at this joint. This location coincides with the center of mass of each story. Frame elements were tied to the master joints when the model for each frame was created. Columns and bracing are connected to the master joints of the story above and the story below. Beams are connected to the master joint of that story.

The structural model must be subjected to a ground acceleration in order to determine the magnitude and distribution of seismic forces. The magnitude of the forces is dependent on ground acceleration and dynamic characteristics of the structure. Distribution of forces is dependent on the methods use to resist lateral loading.

To model the ground acceleration, a response spectrum was input into the analysis model. For this project the Newmark-Hall spectrum was used. Construction of the response spectrum required a determination of ground acceleration, percentage of critical damping, ground velocity, ground displacements and spectral displacements.

Ground acceleration was obtained from maps of peak accelerations provided by the BOCA code. For a southern Maine location this value is equal to 0.1g, where g is the acceleration due to gravity, 386 in/sec^2.

Percentage of critical damping was based on recommendations taken from The Seismic Design Handbook, [Naeim 1989] which is based on the prior work N.M. Newmark and W.J. Hall.

Based on the above recommendations a value of 5 percent was selected. A 5 percent value corresponds to a working stress level less than or equal to half of the yield point.

Calculation of ground velocity and ground displacements is accomplished by using the basic Newmark-Hall spectrum that is normalized for 1.0 g acceleration and 5 percent damping. For an acceleration of 1.0g the maximum ground velocity is equal to 48 in./sec and the maximum ground displacement is equal to 36 in. Therefore the velocity and displacement for this project are direct proportion to the ratio of 0.1g/1.0g, or 0.1.

After ground acceleration, velocity, and displacements were determined, amplification factors were applied. Amplification factors based on records of past earthquakes have been developed to reflect the actual response of structures. The magnitude is dependent on the percentage of critical damping. Calculation of ground acceleration, velocity, and displacement is tabulated in Appendix E.

Spectral displacements were determined by interpolation of the basic Newmark-Hall design spectrum, using amplified values for ground acceleration, velocity and displacements. Spectral displacements were determined for six different building periods, ranging from 0.4 sec to 1.0 sec.

The above values were input into the analysis model. These values are in effect the external loading acting on the structure. Actual values used are found in Appendix E.

Analysis was performed once the above tasks were completed. An analysis was done for each of the following conditions.

1. Ground acceleration applied in X direction.

2. Ground acceleration applied in Y direction.

3. Accidental eccentricity of mass of 5 percent.

The X direction is along the 180 ft. leg dimension of the building, and the Y direction is along the 260 ft. leg dimension of the building. Items 1 and 2 were repeated during the design process as members were revised. The design process used and verification of capacities is discussed in Sections 4.4 and 4.5.

4.2 AISC LRFD Seismic Design Specification

This section examines AISC LRFD Seismic Provisions for Structural Steel Buildings [AISC 1992] as they apply to this project. Particular topics include the following.

1. Seismic Performance Categories.

2. Load Combinations.

3. Requirements for Concentrically Braced Frames.

4. Requirements for Ordinary Moment Frames.

Structures fall into five seismic performance categories ranging from A to E, with category E having the most stringent requirements. Structures classification depends on the seismic hazard exposure group and the value of effective peak velocity related acceleration. The seismic hazard exposure group is related to the use of the structure. The building designed in this project falls into an exposure Group I. A Group I structure can be best described as non essential facility having no disaster recovery functions. Definitions of exposure groups are in given in Table 2-1 of the Seismic Provisions, and are reprinted as part of Table 4-1.

The final determination of seismic performance category is dependent on the magnitude of the effective peak velocity related acceleration. A building in group I having an acceleration of 0.10 g, falls into seismic performance category C. Seismic performance categories are in Table 2-2 of the seismic provision, and are reprinted as part of Table 4-2.

Table 4-1

Seismic Hazard Exposure Groups

Group III	Buildings having essential facilities that are necessary for post earthquake recovery and requiring special requirements for access and functionality.
Group II	Buildings that constitute a substantial public hazard because of occupancy or use.
Group I	All buildings not classified in Groups II and III.

Table 4-2

Seismic Performance Categories

	Seismic Hazard Exposure Group		
Value of A_v	I	II	III
$0.20 \le A_v$	D	D	E
$0.15 \le A_v < 0.20$	C	D	D
$0.10 \le A_v < 0.15$	C	C	C
$0.05 \le A_v < 0.10$	B	B	C
$A_v < 0.05$	A	A	A

where A_v = Effective peak velocity related acceleration.

The seismic provisions prescribe load combinations and load actors to be used when designing structures for seismic forces:

$1.4D$ (AISC LRFD 3-1)
$1.2D + 1.6L + 0.5(Lr \text{ or } S \text{ or } R')$ (AISC LRFD 3-2)
$1.2D + 1.6(Lr \text{ or } S \text{ or } R')$ (AISC LRFD 3-3)
$1.2D + 1.3W + 0.5(Lr \text{ or } S \text{ or } R')$ (AISC LRFD 3-4)
$1.2D \pm 1.0E + 0.5L + 0.2S$ (AISC LRFD 3-5)
$0.9D \mp (1.0E \text{ or } 1.3W)$ (AISC LRFD 3-6)

where

D = Dead Load
L = Floor Live Load
S = Snow Load
Lr = Roof Live Load
R'= Load to initial rainwater
W = Wind Load
E = Earthquake Load

This portion of the project only addressed the combinations where earthquake loads are included. It is also assumed that adequate roof drainage is present to prevent loads due to initial rainwater

Requirements for Concentrically Braced Frames are given relative to member slenderness, compressive design strength, width-thickness ratios, and lateral force distribution. Bracing members are required to meet the following specifications.

1. Slenderness -

$$\frac{L}{r} \leq \sqrt{\frac{720}{F_y}} \qquad (4.1)$$

where

L = bracing member effective length.

r = radius of gyration.

F_y = yield stress of member.

2. Compressive design strength -

$$P_u < 0.8\ \phi\ P_n \qquad (4.2)$$

where

P_u = required compressive strength.

ϕ = resistance factor.

P_n = nominal compressive strength.

3. Width thickness ratios -

Members are to comply with section B5 of AISC LRFD specification. Slender members can not be used.

4. Lateral force distribution -

Members are required to resist at least 30 percent of the total lateral force by tension braces. Tension braces are not to resist more than 70 percent of the total lateral force.

Requirements for Ordinary Moment Frames are given relative to the design of beam to column joints. Ordinary Moment Frames are required to meet the following specifications:

1. Flexural strength - Beam to column joints shall have a flexural strength equal to the plastic moment of the beam framing into the joint.

2. Shear strength - The required shear strength is determined by adding the shear caused by the plastic moment and the shear from the load combination of 1.2D + 0.5L + 0.2S.

The seismic provisions defines criteria to be met to achieve the flexural strength of the beam framing into the column. These requirements are:

1. Beam flanges are to be welded to the column using full penetration groove welds.

2. The design shear strength of the beam web joint exceeds the required shear strength, defined above. Furthermore the shear connection must meet either of the

following.

a. If $b_f t_f (d-t_f) F_{yf} > 0.7M_p$, the connection of the beam web is to made using either welding or slip critical bolts.

b. If $b_f t_f (d-t_f) F_{yf} < 0.7M_p$, the connection is to be by welding or shear tabs. Welding is to have a design strength is to have a design strength equal to a least 20 percent of the beams nominal shear strength.

Further considerations of the design of fully restrained connections will be discussed in Section 4.5.

4.3 Braced Frame Design

Initially the same bracing system used in the non-seismic design considered for the seismic design. Based on the requirements of the seismic provisions this alternative was rejected.

Members were designed for the calculated compressive force and checked for tensile capacity. Compression members were designed using the AISC LRFD seismic specification.

In all cases, compression design proved to be the most critical. Based on the fact that gross areas of selected members exceed the non seismic design, story stability was not a critical design issue.

Design of members for both compression and tension does increase the stiffness of each braced frame. As a result of this increase in stiffness each braced frame carries a larger portion of the total seismic load. Final member designs are given Appendix E.

4.4 Moment Frame Design

The seismic design of the moment frames required the verification of member capacities and examination of lateral drift under seismic loading. Design of fully restrained connections is addressed in Section 4.5.

As was previously stated the same members selected in the non seismic design of both frames were used for the initial finite element analysis. Members in both frames were found to be adequate when subjected to all seismic loadings. The calculated moments under loading to both axes of the structure as well as accidental eccentricity were lower in magnitude then under wind loading.

Control of lateral drift at the first story proved to be the most critical aspect of the frame design. Unlike the case for wind loading, the BOCA code does limit lateral drift under seismic loading. Drift is limited to 0.005 times the story height, which for the first story is equal to 0.96 in. Initial analysis, in both directions, was performed with the bases pinned. Under these support conditions drift was in the range of 1.7 in. to 2.0 in. for moment frame 1, and 2.5 in. to 3.5

in. for moment frame 2. To reduce the drift it was decided to revise the supports from pinned to fixed. This revision reduced drift to less than the allowable of 0.96 in. Revising support conditions did cause a slight change in member moments, but these moments were still significantly below the wind load moments.

Reduction of members was not considered. Any reductions would have most likely resulted in the sway buckling capacity being less than the summation of column loads under gravity loading. Also reductions would have reduced frames stiffness, which would result in increased forces being redistributed to the braced frames.

4.5 Fully Restrained Connection Design

The design of fully restrained connections to resist seismic forces was done through use of the CONXPRT module, ASD version. Connections were not designed to resist the calculated forces but were designed to comply with the LRFD Seismic provisions. Determination of connection forces required the calculation of the beam plastic moment and the shear strength required by Paragraph 8.2b of the LRFD Seismic provisions.

The plastic moment M_p given by.

$$M_p = Z_x F_y \tag{4.3}$$

where Z_x = Plastic Section Modulas, in^3.

F_y = Yield Strength, ksi

Once M_p was determined the shear due to M_p was calculated. The plastic moment is developed at the joint and at opposite end of the beam framing into the joint and the shear, V_s, is given by.

$$V_s = \frac{2M_p}{L} \tag{4.4}$$

where L = Member length framing into joint.

As stated in Section 4.2 the required shear strength is composed of two components, shear due to gravity loading and shear due to development of the plastic moment of the beam. The required shear strength is given by.

$$V_u = V_s + V_g \tag{4.5}$$

where V_s = Seismic shear, as defined above, Kips

V_g = Shear due to the Load Combination of

1.2D + 0.5L + 0.2S

The method of making the beam web to column joint was determined based on the flexural strength of the beam flanges only. The flexural strength of the beam flange is given by.

$$M_n = b_f t_f (d - t_f) F_{yf} \tag{4.6}$$

where b_f = Flange width, in.

t_f = Flange thickness, in.

d = Beam depth, in.

F_{yf} = Flange yield stress, ksi.

When M_n is equal to or greater than $0.7M_p$ the beam web joint is made using either welding or slip critical bolts. If M_n is less than $0.7M_p$ the connection is made by welding or shear tabs, with the weld having a design strength at least equal to 20 percent of the beams nominal shear strength.

Because the connections were designed using ASD specification adjustments to the values of M_p and V_u were required.

The elastic strength of the beam section was used in place of the plastic strength. The elastic strength is given by.

$$M_x = S_x F_y \tag{4.7}$$

where S_x = Elastic section modulas, in^3.

F_y = Yield stress, ksi.

An adjustment factor of 1.14 was used to convert the value of V_u to a working stress level.

The adjusted values were used as the connection forces in place of the calculated forces, beam end moments and shears. Flange welded connections, as in the case of the non-seismic design, were used for all connections. Column stiffeners are required at all joints and is discussed in Section 5.2. Column stiffeners were designed using the CONXPRT module and capacity was verified using the LRFD specification.

Connections were checked using the LRFD specification. Electronic spreadsheets were used to verify capacity of connections, and to check strength limit states.

A summary of designs, capacity verifications, and check of strength limit states are tabulated in Appendix F.

CHAPTER V

COMPARISONS OF SEISMIC AND NON-SEISMIC DESIGNS

This chapter examines differences in the non-seismic and seismic designs. Comparisons will be made of the designs of fully restrained connections and the selection of members in braced frames.

5.1 Member Differences

Differences in the amount of structural steel used in the non-seismic and seismic designs were also made. Calculation of member weights for the braced frames are tabulated in Appendix G. The total weights and additional weights are shown in Table 5-1.

Table 5.1

Bracing Member Weights

Frame	Non-Seismic	Seismic	Additional Weight
Braced Frame 1	4,682 Lbs.	23,376 Lbs	18,694 Lbs.
Braced Frame 2	5,164 Lbs.	30,621 Lbs.	25,457 Lbs.
Braced Frame 3	5,474 Lbs.	31,639 Lbs.	26,615 Lbs.

5.2 Fully Restrained Connections

Differences in overall connection weight will be compared between the non-seismic and seismic designs. Included in the weight calculations are weights of connection plates, and stiffener plates. Comparisons of the number of high strength bolts will also be made.

Calculation of connection weights are tabulated in Appendix G. The total weight of connections for moment frames 1 and 2 are shown in Table 5-2.

Table 5.2

Connection Weights

	Non-Seismic Design	Seismic Design	Additional Weight
Moment Frame 1	131 Lbs.	1227 Lbs.	1096 Lbs.
Moment Frame 2	227 Lbs.	1177 Lbs.	950 Lbs.

The additional weight of connections in the seismic design case gives a partial picture of the overall project cost. Additional weight in the seismic case is primarily the result of the requirement for column stiffeners. The installation of column stiffeners is a labor intensive activity. Although no cost estimates have been prepared it is felt that the additional labor costs would exceed additional material costs.

Differences in the number of high strength bolts are tabulated in Appendix G. The total number of bolts for moment frames 1 and 2 are shown in Table 5-3.

Table 5.3

Number of High Strength Bolts

	Non-Seismic Design	Seismic Design	Additional Bolts
Moment Frame 1	180	266	86
Moment Frame 2	185	235	50

The material cost of the additional bolts are insignificant but increase in bolts will increase labor costs. Increases in shop and field labor cost will occur in the seismic design.

CHAPTER VI

CONCLUSIONS AND RECOMMENDATIONS

The conclusion of this study is that seismic forces in areas of low seismic activity will control the design of fully restrained connections, Type FR, and bracing members for non symmetric structures. Additional material is required for Type FR connections to maintain ductility of the joints. Sizes of bracing members are also increased to maintain member ductility.

Furthermore this study showed that seismic forces in areas of low seismic activity cannot be neglected. In the case of the design of moment frames, moments due to seismic forces are less than wind forces, but the critical design of type FR connections occurs under seismic loading.

The magnitude of the moments under seismic loading depend on the distribution of forces throughout the structure. Eccentricities between the center of mass and center of stiffness and the methods used to resist lateral loads determine how the forces are distributed. Therefore a symmetric structure will have a different distribution of forces.

To gain insight into the magnitudes of moments under seismic loading for symmetric structures, comparisons between applied factored lateral loads under seismic loading and wind loading were made. Seismic loads were calculated using methods prescribed by the BOCA code. Calculation of the horizontal base shear is given by.

$$V = 2.5\ A_v IKCSW$$

where A_v = Effective peak velocity-related acceleration

I = Importance Factor

K = Horizontal Force Factor

C = Coefficient related to building period

S = Soil Profile Coefficient

W = Building Weight including full snow load

The base shear is distributed to each story in proportion to the story weight and height and is calculated as follows.

$$F_x = (V-F_t)w_x h_x \div \sum w_i h_i$$

where V = Total Lateral Force

$F_t = 0.07TV \leq 0.25V$

$= 0.00$ if $T \leq 0.7$ seconds

w_x = Weight of structure at story level x

h_x = Height of structure at story level x

$\sum w_i h_i$ = Summation of the product of the weight and height of each story

Applied loads were calculated for a one foot tributary spacing for two moment frames. The first is a four story frame which is 80 ft. wide. The second is a four story frame which is 160 ft. wide. Comparisons of the two frames are tabulated in Tables 6.1 and 6.2.

Table 6.1
Lateral Forces, 80 Ft. Frame

Story Level	F_w (Kips)	F_x (Kips)
4	0.273	0.267
3	0.523	0.154
2	0.459	0.105
1	0.443	0.056

Table 6.2

Lateral Forces, 160 Ft. Frame

Story Level	F_w (Kips)	F_x (Kips)
4	0.273	0.534
3	0.523	0.308
2	0.459	0.210
1	0.443	0.112

Based on the above comparisons it appears that wind loading would control member selection. Design of type FR connections would appear to be controlled by the seismic provisions. An analysis would have to be performed to verify the above assumptions.

Based on this study, possible design strategies for lateral force resisting systems are presented for similar buildings with similar lateral loads.

1. Design moment frames to resist wind loading.
2. Design fully restrained connection, type FR, to comply with AISC LRFD seismic design provisions.
3. Design braced frames to resist seismic forces and to comply with AISC LRFD seismic design provisions.

Prior to extending the above recommendations to buildings of different geometry, methods of resiting lateral loads, and geographic locations further studies are required. The following are possible areas for further investigation.

1. Conduct non-seismic and seismic designs for a building that is not irregular in plan. A symmetric building would use the same method of resisting lateral loads in each direction of loading. Comparisons in connections, and member differences could be made.

2. Similar studies could be performed for buildings located in areas where the design wind velocities are less but the ground accelerations are the same. This will result in wind loads being reduced and would be closer in magnitude to the seismic forces. These studies could be performed for both symmetric and non-symmetric buildings.

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Appendix A

DESIGN LOADS CALCULATIONS

APPENDIX A

Determination of building design loads are based on an estimation of building material weights, for dead load, and requirements of the BOCA National Building Code, 1990 edition, for live, snow, wind loads. Determination of seismic forces is discussed in Chapter IV.

A. Gravity Loads

1. Floor Dead Loads:

a. Uniformly distributed loads-

metal deck	3.00 psf
concrete slab	40.70 psf
ceiling	3.00 psf
mechanical	5.00 psf
	51.70 psf

b. Spandral beams -

6 in. brick wall 40 psf (Table 6, pg. 21-10 Structural Engineering Handbook)

wall height 14 ft.

W_{wall} = 40 psf (14 ft.) = 560 plf

c. Perimeter Elevator Walls -

12 in. CMU wall 80 psf (Table 6, pg. 21-10 Structural Engineering Handbook)

wall height 16 ft.

W_{wall} = 80 psf (16 ft.) = 1200 plf (1.2 klf)

2. Roof Dead Load:

a. Uniformly distributed loads -

metal deck	3.00 psf
ceiling	3.00 psf
mechanical	5.00 psf
insulation	3.00 psf
membrane	1.00 psf
	15.00 psf

3. Floor Live Load:

a. Uniformly distributed loads-

office space	50 psf	(Table 1106.1 BOCA 1990)
lobbies	100 psf	(Table 1106.1 BOCA 1990)
corridors	80 psf	(Table 1106.1 BOCA 1990)

b. Partitions 20 psf (Para. 1104.3 BOCA 1990)

4. Roof Live Loads:

Tributary Area		
0 to 200 FT^2	20 psf	(Table 1110.2 BOCA 1990)
201 to 600 FT^2	16 psf	(Table 1110.2 BOCA 1990)
Over 600 FT^2	12 psf	(Table 1110.2 BOCA 1990)

5. Snow Load:

$P_f = C_e\ I\ P_g$

$C_e = 0.7$ (Table 1111.4a BOCA 1990)

$I = 1.0$ (Table 1111.4b BOCA 1990)

$P_g = 70$ psf (Fig. 1111.2a BOCA 1990)

$P_f = (0.7)(1.0)(70\text{ psf}) = 50$ psf

B. Wind Loads

1. Wind Pressure:

$P_d = P_e I^2 C_p$

$I = 1.05$

	C_p
Walls-	
Windward	0.8
Leeward	-0.5
Roof-	-0.7

		P_d psf	
Height	P_e	Windward	Leeward
0 to 20 ft.	15 psf	13.25	-8.27
20 ft. to 40 ft.	17 psf	15.00	-9.37
40 ft. to 60 ft.	21 psf	18.52	-11.58

2. Wind Loads (tabulated per foot):

Story Level	Windward	Leeward
4	0.133 kips	0.077 kips
3	0.252 kips	0.150 kips
2	0.212 kips	0.141 kips
1	0.225 kips	0.116 kips

C. Load Combinations

	LRFD Ref
$1.4D$	(A4-1)
$1.2D + 1.6L + 0.5(L_r \text{ or } S)$	(A4-2)
$1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (0.5L \text{ or } 0.8W)$	(A4-3)
$1.2D + 1.3W + 0.5L + 0.5(Lr \text{ or } S)$	(A4-4)
$0.9D - 1.3W$	(A4-6)

where

D = dead load

L = floor live load

L_r = roof live load

S = Snow Load

W = Wind Load

D. Live Load Reduction

$L = L_o(.25 + 15/\sqrt{A_i})$ (Para. 1115.5.1 BOCA 1990)

where:

L = reduced live load

L_o = unreduced live load

A_i = influence area (square feet), four times the tributary area for a column, and two times the tributary area for beams.

APPENDIX B

NON-SEISMIC MEMBER DESIGN

Appendix B

Bracing Design

Braced Frame 1:

Reference Figure 3.5 for frame configuration

Wind Load Summary

Story Level	Windward (Kips)	Leeward (Kips)	1.3 T_u (Kips)	Cum T_u (Kips)
4	11.97	6.93	24.57	24.57
3	22.65	13.54	47.05	71.62
2	19.11	12.65	41.29	112.91
1	20.25	10.41	39.86	152.77

Summation of Column Loads

Dead Load

Story Level	P_d (Kips)	1.2 P_d (Kips)	Cum. P_d (Kips)
4	146.00	176.	176.
3	584.53	702.	878.
2	584.53	702.	1580.
1	584.53	702.	2282.

Live Load

Story Level	P_l (Kips)	0.5 P_l (Kips)	Cum. P_l (kips)	1.6 P_l (Kips)	Cum. P_l (kips)
4	11.80	6.	6.	18.	18.
3	289.8	145.	151.	464.	482.
2	289.8	145.	296.	464.	946.
1	289.8	145.	441.	464.	1410.

Snow Load

Story Level	P_s (Kips)	0.5 P_s (Kips)	Cum. P_s (Kips)	1.6 P_s (Kips)	Cum. P_s (Kips)
4	360.	180.	180.	576.	576.

Diagonal Member Design

Story Level 4-

T_{max} = (24.57K + 0.004(362.))/cos19.29 deg = 27.48 K

Agreq = 27.48K/(0.9)(36 ksi) = 0.85 in^2

Stiffness

βhreq = 2(176K+6K+576K)/14ft. = 108.29 K/ft.

Abrreq = (108.29)(42.38ft.)/(0.75)(29,000 ksi)(cos^{2}19.29 deg)

= 0.25 in^2 < 0.80 in^2 OK

Slenderness

r_{min} = (42.38ft)(12in./ft.)/300 = 1.70

Check L 5x5x5/16

A_g = 3.03 in^2 > 0.85 in^2 OK

A_{net} = 3.03 in^2 - (7/8in.)(5/16in.) = 2.76 in^2

A_e = (0.85)(2.76in^2) = 2.34 in^2

ϕTn = min of

(0.90)(3.03 in^2)(36 ksi) = 98.17 K Controls

or

(0.75)(2.34 in^2)(58 ksi) = 101.92 K

98.17 K > 27.48K OK

Use L 5x5x5/16

Story Level 3-

T_{max} = (71.72K + 0.004(1209.)/cos19.29 deg = 80.83 K

Agreq = 80.83K/(0.9)(36 ksi) = 2.49 in^2

Stiffness

βhreq = 2(878K+180K+484K)/14ft. = 220.00 K/ft.

Abrreq = (220.00)(42.38ft.)/(0.75)(29,000 ksi)($cos^2$19.29 deg)

= 0.53 in^2 < 2.49 in^2 OK

Slenderness

r_{min} = (42.38ft)(12in./ft.)/300 = 1.70

Check L 5x5x5/16

A_g = 3.03 in^2 > 2.49 in^2 OK

A_{net} = 3.03 in^2 - (7/8in.)(5/16in.) = 2.76 in^2

A_e = (0.85)(2.76in^2) = 2.34 in^2

ϕTn = min of

(0.90)(3.03 in²)(36 ksi) = 98.17 K Controls

or

(0.75)(2.34 in²)(58 ksi) = 101.92 K

98.17 K > 80.83 K OK

Use L 5x5x5/16

<u>Story Level 2-</u>

T_{max} = (112.91K + 0.004(2056.)/cos19.29 deg = 127.85 K

Agreq = 127.85K/(0.9)(36 ksi) = 3.95 in²

Stiffness

βhreq = 2(1580+946K+180K)/14ft. = 386.57 K/ft.

Abrreq = (386.57)(42.38ft.)/(0.75)(29,000 ksi)(cos²19.29 deg)

= 0.91 in² < 3.95 in² OK

Slenderness

rmin = (42.38ft)(12in./ft.)/300 = 1.70

Check L 5x5x7/16

A_g = 4.18 in² > 3.95 in² OK

A_{net} = 4.18 in² - (7/8in.)(7/16in.) = 3.80 in²

A_e = (0.85)(3.80in²) = 3.23 in²

ϕTn = min of

(0.90)(3.95 in²)(36 ksi) = 127.98 K

or

(0.75)(3.23 in²)(58 ksi) = 127.85 K Controls

127.85 K = 127.85 K OK

Use L 5x5x7/16

Story Level 1-

T_{max} = (152.77K + 0.004(2903.)/cos21.80 deg = 174.10 K

Agreq = 174.10K/(0.9)(36 ksi) = 5.38 in^2

Stiffness

βhreq = 2(2282K+1410K+180K)/16ft. = 484.00 K/ft.

Abrreq = (484.00)(43.08)/(0.75)(29,000 ksi)($cos^2$21.80 deg)

= 1.19 $in^2 < 5.38$ in^2 OK

Slenderness

rmin = (43.08ft)(12in./ft.)/300 = 1.72

Check L 5x5x5/8

A_g = 5.86 in^2 > 5.38 in^2 OK

A_{net} = 5.86 in^2 - (7/8in.)(5/8in.) = 5.31 in^2

A_e = (0.85)(5.31in^2) = 4.52 in^2

ϕTn = min of

(0.90)(5.86 in^2)(36 ksi) = 189.86 K Controls

or

(0.75)(4.52 in^2)(58 ksi) = 196.62 K

189.86 K > 174.10 K OK

Use L 5 x 5 x 5/8

Braced Frame 2:

Reference Figure 3.5 for frame configuration

Wind Load Summary

Story Level	Windward (Kips)	Leeward (Kips)	1.3 T_u (Kips)	Cum T_u (Kips)
4	11.97	6.93	24.57	24.57
3	22.65	13.54	47.05	71.62
2	19.11	12.65	41.29	112.91
1	20.25	10.41	39.86	152.77

Summation of Column Loads

Dead Load

Story Level	P_d (Kips)	1.2 P_d (Kips)	Cum. P_d (Kips)
4	146.00	176.	176.
3	598.92	719.	895.
2	598.92	719.	1614.
1	598.92	719.	2333.

Live Load

Story Level	P_l (Kips)	0.5 P_l (Kips)	Cum. P_l (kips)	1.6 P_l (Kips)	Cum. P_l (kips)
4	11.80	6.	6.	18.	18.
3	279.6	140.	146.	448.	482.
2	279.6	140.	286.	448.	914.
1	279.6	140.	426.	448.	1362.

Snow Load

Story Level	P_s (Kips)	0.5 P_s (Kips)	Cum. P_s (Kips)	1.6 P_s (Kips)	Cum. P_s (Kips)
4	360.	180.	180.	576.	576.

Diagonal Member Design

Story Level 4-

T_{max} = (24.57K + 0.004(362.))/cos19.29 deg = 27.48 K

Agreq = 27.48K/(0.9)(36 ksi) = 0.85 in^2

Stiffness

βhreq = 2(176K+6K+576K)/14ft. = 108.29 K/ft.

Abrreq = (108.29)(42.38ft.)/(0.75)(29,000 ksi)($cos^2$19.29 deg)

= 0.25 in^2 < 0.80 in^2 OK

Slenderness

r_{min} = (42.38ft)(12in./ft.)/300 = 1.70

Check L 5x5x5/16

A_g = 3.03 in^2 > 0.85 in^2 OK

A_{net} = 3.03 in^2 - (7/8in.)(5/16in.) = 2.76 in^2

A_e = (0.85)(2.76in^2) = 2.34 in^2

ϕTn = min of

(0.90)(3.03 in^2)(36 ksi) = 98.17 K Controls

or

(0.75)(2.34 in^2)(58 ksi) = 101.92 K

98.17 K > 27.48K OK

Use L 5x5x5/16

<u>Story Level 3-</u>

T_{max} = (71.72K + 0.004(1209.)/cos19.29 deg = 80.83 K

Agreq = 80.83K/(0.9)(36 ksi) = 2.49 in^2

Stiffness

βhreq = 2(878K+180K+484K)/14ft. = 220.00 K/ft.

Abrreq = (220.00)(42.38ft.)/(0.75)(29,000 ksi)($cos^2$19.29 deg)

= 0.53 in^2 < 2.49 in^2 OK

Slenderness

r_{min} = (42.38ft)(12in./ft.)/300 = 1.70

Check L 5x5x5/16

A_g = 3.03 in^2 > 2.49 in^2 OK

A_{net} = 3.03 in^2 - (7/8in.)(5/16in.) = 2.76 in^2

A_e = (0.85)(2.76in^2) = 2.34 in^2

ϕTn = min of

$(0.90)(3.03\ in^2)(36\ ksi) = 98.17$ K Controls

or

$(0.75)(2.34\ in^2)(58\ ksi) = 101.92$ K

98.17 K > 80.83 K OK

Use L 5x5x5/16

Story Level 2-

$T_{max} = (112.91K + 0.004(2080.)/\cos 19.29\ deg = 128.44$ K

Agreq = $128.44K/(0.9)(36\ ksi) = 3.96\ in^2$

Stiffness

βhreq = 2(1614+914K+180K)/14ft. = 386.86 K/ft.

Abrreq = $(386.86)(42.38ft.)/(0.75)(29{,}000\ ksi)(\cos^2 19.29\ deg)$

$= 0.85\ in^2 < 3.96\ in^2$ OK

Slenderness

rmin = (42.38ft)(12in./ft.)/300 = 1.70

Check L 5x5x7/16

$A_g = 4.18\ in^2 > 3.96\ in^2$ OK

$A_{net} = 4.18\ in^2 - (7/8in.)(7/16in.) = 3.80\ in^2$

$A_e = (0.85)(3.80in^2) = 3.23\ in^2$

ϕTn = min of

$(0.90)(3.95\ in^2)(36\ ksi) = 127.98$ K

or

$(0.75)(3.23\ in^2)(58\ ksi) = 127.85$ K Controls

127.85 < 128.44 NG

Use L 5x5x5/8, by inspection OK

Story Level 1-

Tmax = (152.77K + 0.004(2939.)/cos21.80 deg = 177.20 K

Agreq = 177.20K/(0.9)(36 ksi) = 5.47 in^2

Stiffness

Bhreq = 2(2333K+1362K+180K)/16ft. = 484.38 K/ft.

Abrreq = (484.38)(43.08)/(0.75)(29,000 ksi)($cos^2$21.80 deg)

= 1.11 in^2 < 5.47 in^2 OK

Slenderness

rmin = (43.08ft)(12in./ft.)/300 = 1.72

Check L 5x5x5/8

A_g = 5.86 in^2 > 5.47 in^2 OK

A_{net} = 5.86 in^2 - (7/8in.)(5/8in.) = 5.31 in^2

A_e = (0.85)(5.31in^2) = 4.52 in^2

ϕTn = min of

(0.90)(5.86 in^2)(36 ksi) = 189.86 K Controls

or

(0.75)(4.52 in^2)(58 ksi) = 196.62 K

189.86 K > 177.20 K OK

Use L 5x5x5/8

Braced Frame 3:

Reference Figure 3.5 for frame configuration

Wind Load Summary

Story Level	Windward (Kips)	Leeward (Kips)	1.3 T_u (Kips)	Cum T_u (Kips)
4	13.30	7.70	27.30	27.30
3	25.17	15.04	52.27	79.57
2	21.23	14.06	45.88	125.45
1	22.50	11.57	44.29	169.74

Summation of Column Loads

Dead Load

Story Level	P_d (Kips)	1.2 P_d (Kips)	Cum. P_d (Kips)
4	161.98	195.	195.
3	704.52	846.	1041.
2	704.52	846.	1887.
1	704.52	846.	2733.

Live Load

Story Level	P_l (Kips)	0.5 P_l (Kips)	Cum. P_l (kips)	1.6 P_l (Kips)	Cum. P_l (kips)
4	11.80	6.	6.	18.	18.
3	311.6	156.	162.	499.	517.
2	311.6	156.	318.	499.	1016.
1	311.6	156.	474.	499.	1115.

Snow Load

Story Level	P_s (Kips)	0.5 P_s (Kips)	Cum. P_s (Kips)	1.6 P_s (Kips)	Cum. P_s (Kips)
4	400.	200.	200.	640.	640.

Diagonal Member Design

Story Level 4-

T_{max} = (27.30K + 0.004(401.))/cos19.29 deg = 30.62 K

Agreq = 30.62K/(0.9)(36 ksi) = 0.95 in^2

Stiffness

βhreq = 2(195K+6K+640K)/14ft. = 120.14 K/ft.

Abrreq = (120.14)(42.38ft.)/(0.75)(29,000 ksi)($cos^2$19.29 deg)

= 0.26 in^2 < 0.95 in^2 OK

Slenderness

rmin = (42.38ft)(12in./ft.)/300 = 1.70

Check L 5x5x5/16

A_g = 3.03 in^2 > 0.95 in^2 OK

A_{net} = 3.03 in^2 - (7/8in.)(5/16in.) = 2.76 in^2

A_e = (0.85)(2.76in^2) = 2.34 in^2

ϕTn = min of

(0.90)(3.03 in^2)(36 ksi) = 98.17 K Controls

or

(0.75)(2.34 in^2)(58 ksi) = 101.92 K

98.17 K > 30.62K OK

Use L 5x5x5/16

Story Level 3-

T_{max} = (79.57K + 0.004(1403.)/cos19.29 deg = 90.25 K

Agreq = 90.25K/(0.9)(36 ksi) = 2.79 in^2

Stiffness

βhreq = 2(1041K+162K+640K)/14ft. = 263.29 K/ft.

Abrreq = (263.29)(42.38ft.)/(0.75)(29,000 ksi)($cos^2$19.29 deg)

= 0.58 in^2 < 2.79 in^2 OK

Slenderness

rmin = (42.38ft)(12in./ft.)/300 = 1.70

Check L 5x5x5/16

A_g = 3.03 in^2 > 2.49 in^2 OK

A_{net} = 3.03 in^2 - (7/8in.)(5/16in.) = 2.76 in^2

A_e = (0.85)(2.76in^2) = 2.34 in^2

ϕTn = min of

(0.90)(3.03 in^2)(36 ksi) = 98.17 K Controls

or

(0.75)(2.34 in^2)(58 ksi) = 101.92 K

98.17 K > 90.25 K OK

Use L 5x5x5/16

<u>Story Level 2-</u>

T_{max} = (124.45K + 0.004(2405.)/cos19.29 deg = 142.04 K

Agreq = 142.04K/(0.9)(36 ksi) = 4.38 in^2

Stiffness

βhreq = 2(1887K+1016K+200K)/14ft. = 443.29 K/ft.

Abrreq = (443.29)(42.38ft.)/(0.75)(29,000 ksi)(cos^{2}19.29 deg)

= 0.97 in^2 < 4.38 in^2 OK

Slenderness

rmin = (42.38ft)(12in./ft.)/300 = 1.70

Check L 5x5x5/8

A_g = 5.86 in^2 > 4.38 in^2 OK

A_{net} = 5.86 in^2 - (7/8in.)(7/16in.) = 5.31 in^2

A_e = (0.85)(5.31in^2) = 4.52 in^2

ϕTn = min of

(0.90)(5.86 in^2)(36 ksi) = 189.86 K Controls

or

(0.75)(4.52 in^2)(58 ksi) = 196.62 K

189.86 > 142.04 OK

Use L 5x5x5/8

Story Level 1-

T_{max} = (169.74K + 0.004(3407.)/cos21.80 deg = 197.49 K

Agreq = 197.49K/(0.9)(36 ksi) = 6.06 in^2

Stiffness

βhreq = 2(2,733K+1515K+200K)/16ft. = 556.00 K/ft.

Abrreq = (556.00)(43.08)/(0.75)(29,000 ksi)($cos^2$21.80 deg)

= 1.28 in^2 < 5.47 in^2 OK

Slenderness

rmin = (43.08ft)(12in./ft.)/300 = 1.72

Check L 5x5x3/4

A_g = 6.94 in^2 > 6.06 in^2 OK

A_{net} = 6.94 in^2 - (7/8in.)(3/4in.) = 6.28 in^2

A_e = (0.85)(6.28in^2) = 5.34 in^2

ϕTn = min of

(0.90)(6.94 in^2)(36 ksi) = 224.86 K Controls

or

(0.75)(5.34 in^2)(58 ksi) = 232.34 K

224.86 K > 197.49 K OK

Use L 5x5x3/4

Summary of Bracing Design

Braced Frame 1:

Story Level 4 - L 5 x 5 x 5/16

Story Level 3 - L 5 x 5 x 5/16

Story Level 2 - L 5 x 5 x 7/16

Story Level 1 - L 5 x 5 x 5/8

Braced Frame 2:

Story Level 4 - L 5 x 5 x 5/16

Story Level 3 - L 5 x 5 x 5/16

Story Level 2 - L 5 x 5 x 5/8

Story Level 1 - L 5 x 5 x 5/8

Braced Frame 3:

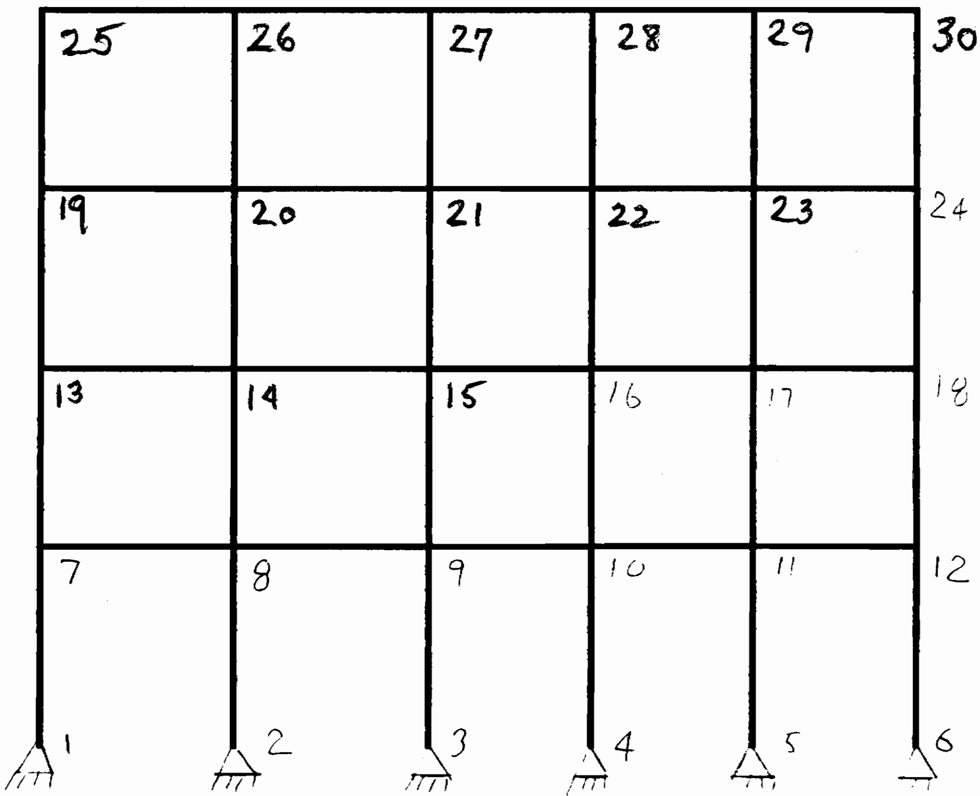
Story Level 4 - L 5 x 5 x 5/16

Story Level 3 - L 5 x 5 x 5/16

Story Level 2 - L 5 x 5 x 5/8

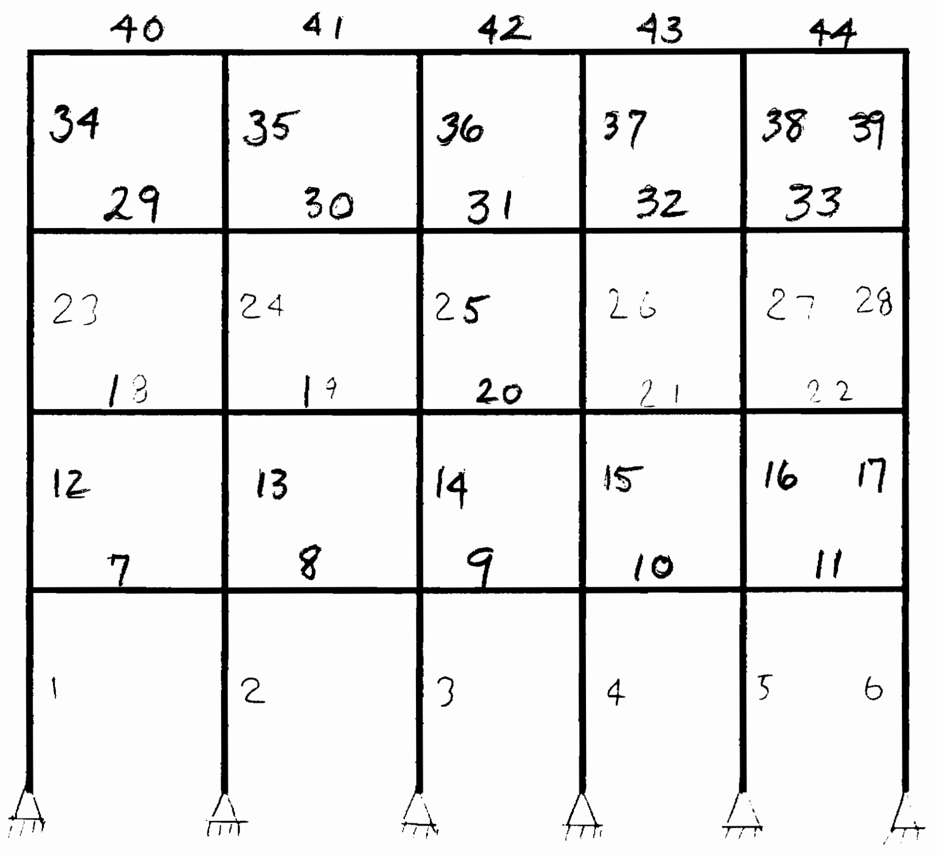
Story Level 1 - L 5 x 5 x 3/4

All members use A36 structural steel



Moment Frame 1

Joint Numbers



Moment Frame 1

Element Numbers

Moment Frame 1 Column Design

Design Strength

Mem No.	Sect.	Area	rx	ry	Kx	Ky	L	ϕPn	ϕMn
1	W14x82	24.1	6.05	2.48	1.92	1.00	16	777	521
2	W14x159	46.7	6.38	4.00	2.02	1.00	16	1782	1076
3	W14x159	46.7	6.38	4.00	1.95	1.00	16	1815	1076
4	W14x159	46.7	6.38	4.00	1.90	1.00	16	1839	1076
5	W14x159	46.7	6.38	4.00	1.90	1.00	16	1839	1076
6	W14x82	24.1	6.05	2.48	1.85	1.00	16	777	521
12	W14x82	24.1	6.05	2.48	1.48	1.00	14	862	521
13	W14x159	46.7	6.38	4.00	1.45	1.00	14	2052	1076
14	W14x159	46.7	6.38	4.00	1.38	1.00	14	2052	1076
15	W14x159	46.7	6.38	4.00	1.35	1.00	14	2052	1076
16	W14x159	46.7	6.38	4.00	1.35	1.00	14	2052	1076
17	W14x82	24.1	6.05	2.48	1.40	1.00	14	862	521
23	W14x82	24.1	6.05	2.48	1.35	1.00	14	862	521
24	W14x82	24.1	6.05	2.48	1.32	1.00	14	862	521
25	W14x82	24.1	6.05	2.48	1.28	1.00	14	862	521
26	W14x82	24.1	6.05	2.48	1.27	1.00	14	862	521
27	W14x82	24.1	6.05	2.48	1.27	1.00	14	862	521
28	W14x82	24.1	6.05	2.48	1.32	1.00	14	862	521
34	W14x82	24.1	6.05	2.48	1.00	1.00	14	862	521
35	W14x82	24.1	6.05	2.48	1.25	1.00	14	862	521
36	W14x82	24.1	6.05	2.48	1.18	1.00	14	862	521
37	W14x82	24.1	6.05	2.48	1.15	1.00	14	862	521
38	W14x82	24.1	6.05	2.48	1.22	1.00	14	862	521
39	W14x82	24.1	6.05	2.48	1.00	1.00	14	862	521

Moment Amplifier B1:

Mem No.	Section	P_e	C_m	P_u	B1
1	W14x82	6,848.94	1.00	248.45	1.04
2	W14x159	14,758.88	1.00	516.07	1.04
3	W14x159	14,758.88	1.00	471.84	1.03
4	W14x159	14,758.88	1.00	445.60	1.03
5	W14x159	14,758.88	1.00	431.92	1.03
6	W14x82	6,848.94	0.38	227.71	1.00
12	W14x82	8,945.55	0.42	254.62	1.00
13	W14x159	19,276.91	0.29	370.46	1.00
14	W14x159	19,276.91	0.30	335.42	1.00
15	W14x159	19,276.91	0.46	318.92	1.00
16	W14x159	19,276.91	0.24	307.93	1.00
17	W14x82	8,945.55	0.28	165.67	1.00
23	W14x82	8,945.55	0.25	107.30	1.00
24	W14x82	8,945.55	0.58	223.21	1.00
25	W14x82	8,945.55	0.32	201.53	1.00
26	W14x82	8,945.55	0.54	191.62	1.00
27	W14x82	8,945.55	0.54	183.30	1.00
28	W14x82	8,945.55	0.48	104.25	1.00
34	W14x82	8,945.55	0.51	42.83	1.00
35	W14x82	8,945.55	0.32	76.19	1.00
36	W14x82	8,945.55	0.29	69.01	1.00
37	W14x82	8,945.55	0.37	65.06	1.00
38	W14x82	8,945.55	0.30	58.23	1.00
39	W14x82	8,945.55	0.31	42.83	1.00

Moment Amplifier B2:

Mem No.	Section	ΣP_u	P_e	m	ΣP_e	B2
1	W14x82	4,684.40	1,857.89	6	11,147.36	1.72
2	W14x159	4,684.40	3,617.02	6	21,702.11	1.28
3	W14x159	4,684.40	3,881.36	6	23,288.18	1.25
4	W14x159	4,684.40	4,088.33	6	24,530.00	1.24
5	W14x159	4,684.40	4,088.33	6	24,530.00	1.24
6	W14x82	4,684.40	2,001.15	6	12,006.90	1.64
12	W14x82	3,518.20	4,083.98	6	24,503.88	1.17
13	W14x159	3,518.20	9,168.57	6	55,011.40	1.07
14	W14x159	3,518.20	10,122.30	6	60,733.81	1.06
15	W14x159	3,518.20	10,577.18	6	63,463.08	1.06
16	W14x159	3,518.20	10,577.18	6	63,463.08	1.06
17	W14x82	3,518.20	4,564.06	6	27,383.33	1.22
23	W14x82	1,752.00	4,908.39	6	29,450.37	1.06
24	W14x82	1,752.00	5,134.04	6	30,804.23	1.06
25	W14x82	1,752.00	5,459.93	6	32,759.58	1.06
26	W14x82	1,752.00	5,546.25	6	33,277.51	1.06
27	W14x82	1,752.00	5,546.25	6	33,277.51	1.06
28	W14x82	1,752.00	5,134.04	6	30,804.23	1.06
34	W14x82	645.80	8,945.55	6	35.782.19	1.02
35	W14x82	645.80	5,725.15	6	22,900.80	1.03
36	W14x82	645.80	6,424.55	6	25,698.21	1.03
37	W14x82	645.80	6,764.12	6	27,056.48	1.02
38	W14x82	645.80	6,010.18	6	24,040.71	1.03
39	W14x82	645.80	8,945.55	6	35,783.19	1.02

Column Evaluation:

Mem No.	Section	M_{nt}	M_{lt}	M_u	H1-1	Status
1	W14x82	27.17	207.50	386.10	0.98	OK
2	W14x159	2.79	432.70	554.69	0.75	OK
3	W14x159	10.25	444.36	566.84	0.73	OK
4	W14x159	2.12	454.40	563.84	0.71	OK
5	W14x159	0.21	452.44	559.45	0.70	OK
6	W14x82	18.40	216.57	373.51	0.93	OK
12	W14x82	82.66	80.43	176.57	0.73	OK
13	W14x159	3.88	177.17	193.15	0.27	OK
14	W14x159	16.15	185.03	212.56	0.28	OK
15	W14x159	5.36	194.12	210.87	0.27	OK
16	W14x159	15.35	192.49	219.14	0.28	OK
17	W14x82	41.89	84.42	144.47	0.37	OK
23	W14x82	64.91	66.62	135.74	0.32	OK
24	W14x82	7.55	84.49	97.13	0.42	OK
25	W14x82	10.01	86.01	100.88	0.41	OK
26	W14x82	1.46	89.03	95.44	0.39	OK
27	W14x82	8.36	90.66	104.06	0.39	OK
28	W14x82	34.63	73.20	112.25	0.28	OK
34	W14x82	28.52	34.58	63.79	0.27	OK
35	W14x82	23.55	32.23	56.72	0.15	OK
36	W14x82	13.95	45.03	60.14	0.16	OK
37	W14x82	3.99	47.18	52.32	0.13	OK
38	W14x82	15.12	36.32	52.44	0.13	OK
39	W14x82	16.24	32.57	49.46	0.14	OK

Moment Frame 1 Beam Design

Design Strength:

Mem No.	Section	Area	r_x	K_x	L	ϕP_n	ϕM_n
7	W27x94	27.7	10.9	1.00	40.00	1,022	1,040
8	W27x94	27.7	10.9	1.00	40.00	1,022	1,040
9	W27x94	27.7	10.9	1.00	33.33	1,067	1,040
10	W27x94	27.7	10.9	1.00	33.33	1,067	1,040
11	W27x94	27.7	10.9	1.00	33.33	1,067	1,040
18	W27x94	27.7	10.9	1.00	40.00	1,022	1,040
19	W27x94	27.7	10.9	1.00	40.00	1,022	1,040
20	W27x94	27.7	10.9	1.00	33.33	1,067	1,040
21	W27x94	27.7	10.9	1.00	33.33	1,067	1,040
22	W27x94	27.7	10.9	1.00	33.33	1,067	1,040
29	W27x94	27.7	10.9	1.00	40.00	1,022	1,040
30	W27x94	27.7	10.9	1.00	40.00	1,022	1,040
31	W27x94	27.7	10.9	1.00	33.33	1,067	1,040
32	W27x94	27.7	10.9	1.00	33.33	1,067	1,040
33	W27x94	27.7	10.9	1.00	33.33	1,067	1,040
40	W27x84	24.8	10.7	1.00	40.00	910	915
41	W27x84	24.8	10.7	1.00	40.00	910	915
42	W27x84	24.8	10.7	1.00	33.33	952	915
43	W27x84	24.8	10.7	1.00	33.33	952	915
44	W27x84	24.8	10.7	1.00	33.33	952	915

Moment Amplifier B1:

Mem No.	Section	P_e	C_m	P_u	B1
7	W27x94	4,088.52	1.00	23.40	1.01
8	W27x94	4,088.52	1.00	7.18	1.00
9	W27x94	5,347.76	1.00	6.32	1.00
10	W27x94	5,347.76	1.00	8.10	1.00
11	W27x94	5,347.76	1.00	26.45	1.01
18	W27x94	4,088.52	1.00	22.10	1.01
19	W27x94	4,088.52	1.00	19.09	1.01
20	W27x94	5,347.76	1.00	8.67	1.00
21	W27x94	5,347.76	1.00	4.49	1.00
22	W27x94	5,347.76	1.00	6.29	1.00
29	W27x94	4,088.52	1.00	25.22	1.01
30	W27x94	4,088.52	1.00	15.71	1.01
31	W27x94	5,347.76	1.00	6.02	1.00
32	W27x94	5,347.76	1.00	3.71	1.00
33	W27x94	5,347.76	1.00	14.38	1.00
40	W27x84	3,470.17	1.00	13.91	1.01
41	W27x84	3,470.77	1.00	10.43	1.01
42	W27x84	4,991.75	1.00	3.42	1.00
43	W27x84	4,991.75	1.00	8.92	1.00
44	W27x84	4,991.75	1.00	10.87	1.00

Beam Evaluation:

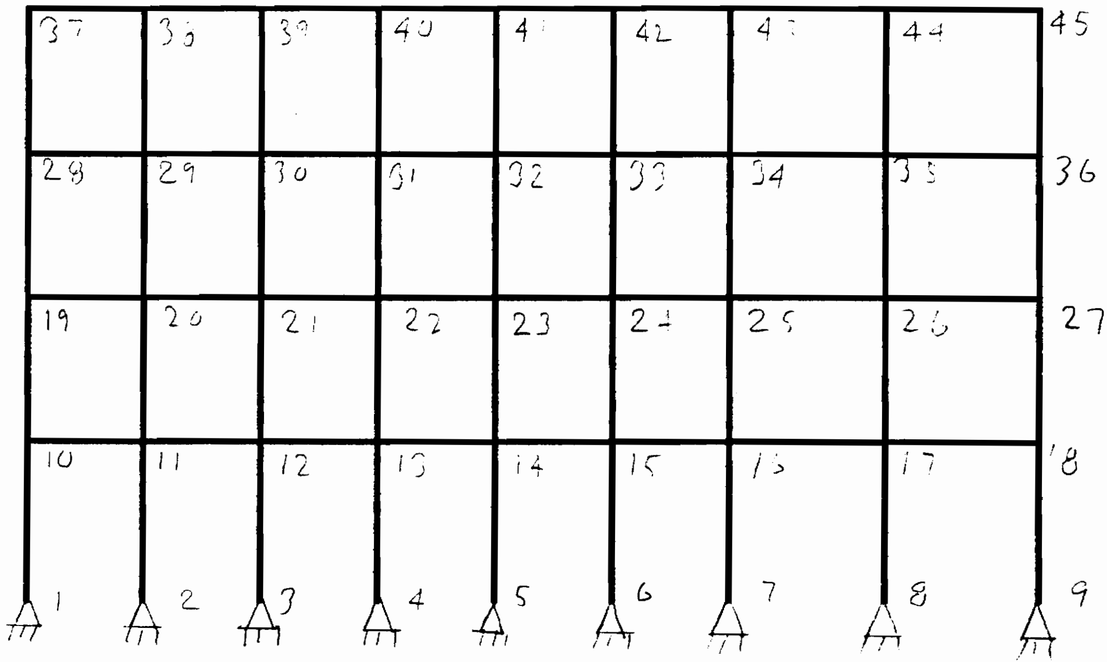
Mem No.	Section	M_{nt}	M_u	H1-1	Status
7	W27x94	514.82	519.97	0.50	OK
8	W27x94	457.86	457.86	0.44	OK
9	W27x94	460.32	460.32	0.44	OK
10	W27x94	445.32	445.32	0.43	OK
11	W27x94	479.02	479.02	0.46	OK
18	W27x94	451.58	456.40	0.44	OK
19	W27x94	252.72	255.25	0.25	OK
20	W27x94	270.37	273.07	0.26	OK
21	W27x94	248.25	248.25	0.24	OK
22	W27x94	236.36	236.36	0.23	OK
29	W27x94	246.82	246.82	0.24	OK
30	W27x94	169.65	171.35	0.16	OK
31	W27x94	254.24	254.24	0.25	OK
32	W27x94	245.00	245.00	0.27	OK
33	W27x94	176.43	176.43	0.19	OK
40	W27x84	178.35	178.35	0.19	OK
41	W27x84	192.11	192.11	0.21	OK
42	W27x84	59.53	59.53	0.07	OK
43	W27x84	78.65	78.65	0.09	OK
44	W27x84	88.42	88.42	0.10	OK

Final Member Selections Moment Frame 1

Element Number	Member Selection
1	W14 x 82
2	W14 x 159
3	W14 x 159
4	W14 x 159
5	W14 x 159
6	W14 x 159
7	W27 x 94
8	W27 x 94
9	W27 x 94
10	W27 x 94
11	W27 x 94
12	W14 x 82
13	W14 x 159
14	W14 x 159
15	W14 x 159
16	W14 x 150
17	W14 x 82
18	W27 x 94
19	W27 x 94
20	W27 x 94
21	W27 x 94
22	W27 x 94
23	W14 x 82
24	W14 x 82
25	W14 x 82
26	W14 x 82

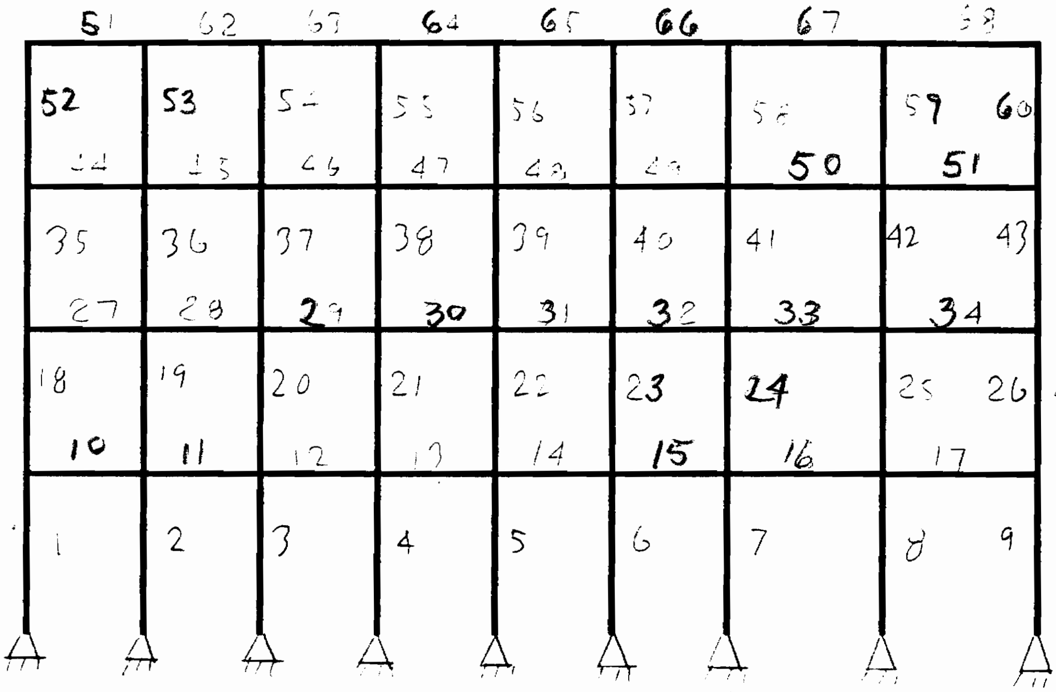
Element Number	Member Selection
27	W14 x 82
28	W14 x 82
29	W27 x 94
30	W27 x 94
31	W27 x 94
32	W27 x 94
33	W27 x 94
34	W14 x 82
35	W14 x 82
36	W14 x 82
37	W14 x 82
38	W14 x 82
39	W14 x 82
40	W27 x 84
41	W27 x 84
42	W27 x 84
43	W27 x 84
44	W27 x 84

All members use A572 Gr 50 structural steel



Moment Frame 2

Joint Numbers



Moment Frame 2

Element Numbers

Moment Frame 2 Column Design

Design Strength:

Mem No.	Sect.	Area	rx	ry	Kx	Ky	L	ϕPn	ϕMn
2	W14x159	46.7	6.38	4.00	1.90	1.00	16	1839	1076
3	W14x159	46.7	6.38	4.00	1.77	1.00	16	1898	1076
4	W14x159	46.7	6.38	4.00	1.77	1.00	16	1898	1076
5	W14x159	46.7	6.38	4.00	1.80	1.00	16	1884	1076
6	W14x159	46.7	6.38	4.00	1.80	1.00	16	1884	1076
7	W14x159	46.7	6.38	4.00	1.90	1.00	16	1839	1076
19	W14x159	46.7	6.38	4.00	1.45	1.00	14	2052	1076
20	W14x159	46.7	6.38	4.00	1.35	1.00	14	2052	1076
21	W14x159	46.7	6.38	4.00	1.35	1.00	14	2052	1076
22	W14x159	46.7	6.38	4.00	1.35	1.00	14	2052	1076
23	W14x159	46.7	6.38	4.00	1.35	1.00	14	2052	1076
24	W14x159	46.7	6.38	4.00	1.35	1.00	14	2052	1076
37	W14x82	24.1	6.05	2.48	1.45	1.00	14	862	521
38	W14x82	24.1	6.05	2.48	1.32	1.00	14	862	521
39	W14x82	24.1	6.05	2.48	1.30	1.00	14	862	521
40	W14x82	24.1	6.05	2.48	1.32	1.00	14	862	521
54	W14x82	24.1	6.05	2.48	1.32	1.00	14	862	521
55	W14x82	24.1	6.05	2.48	1.32	1.00	14	862	521
56	W14x82	24.1	6.05	2.48	1.12	1.00	14	862	521
57	W14x82	24.1	6.05	2.48	1.12	1.00	14	862	521

Moment Amplifier B1:

Mem No.	Section	P_e	C_m	P_u	B1
2	W14x159	14,758.88	1.00	428.28	1.03
3	W14x159	14,758.88	1.00	429.65	1.03
4	W14x159	14,758.88	1.00	410.37	1.03
5	W14x159	14,758.88	1.00	430.01	1.03
6	W14x159	14,758.88	1.00	381.46	1.03
7	W14x159	14,758.88	1.00	495.41	1.03
19	W14x159	19,276.91	0.25	331.87	1.00
20	W14x159	19,276.91	0.27	330.40	1.00
21	W14x159	19,276.91	0.24	291.64	1.00
22	W14x159	19,276.91	0.39	306.20	1.00
23	W14x159	19,276.91	0.23	272.03	1.00
24	W14x159	19,276.91	0.25	342.21	1.00
37	W14x82	8,945.55	0.38	210.54	1.00
38	W14x82	8,945.55	0.29	172.96	1.00
39	W14x82	8,945.55	0.23	181.48	1.00
40	W14x82	8,945.55	0.35	165.45	1.00
54	W14x82	8,945.55	0.30	80.82	1.00
55	W14x82	8,945.55	0.24	52.29	1.00
56	W14x82	8,945.55	0.28	52.41	1.00
57	W14x82	8,945.55	0.29	50.81	1.00

Moment Amplifier B2:

Mem No.	Section	ΣP_u	P_e	m	ΣP_e	B2
2	W14x159	7,437.00	4,088.34	6	24,530.03	1.44
3	W14x159	7,437.00	4,710.93	6	28,265.60	1.36
4	W14x159	7,437.00	4,710.93	6	28,265.60	1.36
5	W14x159	7,437.00	4,555.21	6	27,331.27	1.37
6	W14x159	7,437.00	4,555.21	6	27,331.27	1.37
7	W14x159	7,437.00	4,088.33	6	24,530.00	1.44
19	W14x159	5,270.00	9,168.57	6	55,011.40	1.11
20	W14x159	5,270.00	10,577.18	6	63,463.08	1.09
21	W14x159	5,270.00	10,577.18	6	63,463.08	1.09
22	W14x159	5,270.00	10,577.18	6	63,463.08	1.09
23	W14x159	5,270.00	10,577.18	6	63,463.08	1.09
24	W14x159	5,270.00	9,168.57	6	55,011.40	1.11
37	W14x82	3,103.00	5,134.04	4	20,536.15	1.18
38	W14x82	3,103.00	5,293.22	4	21,172.90	.17
39	W14x82	3,103.00	5,293.22	4	21,172.90	1.17
40	W14x82	3,103.00	5,134.04	4	20,536.15	1.18
54	W14x82	936.00	5,134.04	4	20,536.15	1.05
55	W14x82	936.00	5,134.04	4	20,536.15	1.05
56	W14x82	936.00	7,131.34	4	28,525.35	1.03
57	W14x82	936.00	7,131.34	4	28,525.35	1.03

Column Evaluation-

Mem No.	Section	M_{nt}	M_{lt}	M_u	H1-1	Status
2	W14x159	8.63	631.66	915.37	0.99	OK
3	W14x159	3.25	685.62	933.77	1.00	OK
4	W14x159	3.43	674.85	919.34	0.98	OK
5	W14x159	6.75	673.36	932.03	1.00	OK
6	W14x159	2.80	683.52	941.90	0.98	OK
7	W14x159	59.98	611.06	941.71	0.88	OK
19	W14x159	33.89	267.35	329.57	0.39	OK
20	W14x159	7.57	259.45	290.52	0.35	OK
21	W14x159	10.64	260.38	294.60	0.34	OK
22	W14x159	13.60	260.50	297.69	0.35	OK
23	W14x159	57.66	261.57	342.92	0.38	OK
24	W14x159	63.65	253.46	593.57	0.63	OK
37	W14x82	22.63	181.95	236.96	0.65	OK
38	W14x82	0.94	221.26	260.19	0.64	OK
39	W14x82	8.80	221.12	267.89	0.67	OK
40	W14x82	21.03	181.48	234.81	0.55	OK
54	W14x82	18.88	63.65	85.57	0.21	OK
55	W14x82	4.00	101.27	110.10	0.24	OK
56	W14x82	3.87	101.73	109.05	0.24	OK
57	W14x82	22.10	64.51	88.79	0.20	OK

Moment Frame 2 Beam Design

Design Strength:

Mem No.	Section	Area	r_x	K_x	L	ϕP_n	ϕM_n
10	W27x94	27.7	10.9	1.00	30.00	1,087	1,040
11	W27x94	27.7	10.9	1.00	30.00	1,087	1,040
12	W27x94	27.7	10.9	1.00	30.00	1,087	1,040
13	W27x94	27.7	10.9	1.00	30.00	1,087	1,040
14	W27x94	27.7	10.9	1.00	30.00	1,087	1,040
15	W27x94	27.7	10.9	1.00	30.00	1,087	1,040
16	W27x94	27.7	10.9	1.00	40.00	1,022	1,040
27	W27x94	27.7	10.9	1.00	30.00	1,087	1,040
28	W27x94	27.7	10.9	1.00	30.00	1,087	1,040
29	W27x94	27.7	10.9	1.00	30.00	1,087	1,040
30	W27x94	27.7	10.9	1.00	30.00	1,087	1,040
31	W27x94	27.7	10.9	1.00	30.00	1,087	1,040
32	W27x94	27.7	10.9	1.00	30.00	1,087	1,040
33	W27x94	27.7	10.9	1.00	40.00	1,022	1,040
46	W27x94	27.7	10.9	1.00	30.00	1,087	1,040
47	W27x94	27.7	10.9	1.00	30.00	1,087	1,040
48	W27x94	27.7	10.9	1.00	30.00	1,087	1,040
49	W27x94	27.7	10.9	1.00	30.00	1,087	1,040
63	W27x84	24.8	10.7	1.00	30.00	970	915
64	W27x84	24.8	10.7	1.00	30.00	970	915
65	W27x84	24.8	10.7	1.00	30.00	970	915
66	W27x84	24.8	10.7	1.00	30.00	970	915

Moment Amplifier B1:

Mem No.	Section	P_e	C_m	P_u	B1
10	W27x94	7,267.65	1.00	63.40	1.01
11	W27X94	7,267.65	1.00	57.32	1.01
12	W27x94	7,267.65	1.00	37.82	1.01
13	W27x94	7,267.65	1.00	22.71	1.01
14	W27x94	7,267.65	1.00	37.96	1.01
15	W27x94	7,267.65	1.00	53.98	1.01
16	W27x94	4,087.93	1.00	73.73	1.02
27	W27x94	7,267.65	1.00	62.45	1.01
28	W27x94	7,267.65	1.00	17.63	1.01
29	W27x94	7,267.65	1.00	18.89	1.01
30	W27x94	7,267.65	1.00	18.27	1.01
31	W27x94	7,267.65	1.00	21.85	1.01
32	W27x94	7,267.65	1.00	36.14	1.01
33	W27x94	4,087.93	1.00	13.05	1.01
46	W27x94	7,267.65	1.00	3.31	1.00
47	W27x94	7,267.65	1.00	5.56	1.00
48	W27x94	7,267.65	1.00	15.47	1.00
49	W27x94	7,267.65	1.00	6.95	1.00
63	W27x84	6,175.06	1.00	5.62	1.00
64	W27x84	6,175.06	1.00	18.48	1.00
65	W27x84	6,175.06	1.00	26.23	1.00
66	W27x84	6,175.06	1.00	25.24	1.00

Beam Evaluation:

Mem No.	Section	M_{nt}	M_u	H1-1	Status
10	W27x94	452.44	456.96	0.45	OK
11	W27x94	756.89	764.46	0.75	OK
12	W27x94	633.61	639.95	0.63	OK
13	W27x94	643.20	649.63	0.63	OK
14	W27x94	636.50	642.87	0.63	OK
15	W27x94	787.73	795.61	0.78	OK
16	W27x94	751.62	766.65	0.75	OK
27	W27x94	303.06	306.09	0.30	OK
28	W27x94	340.37	343.77	0.33	OK
29	W27x94	341.70	345.12	0.34	OK
30	W27x94	343.38	346.81	0.34	OK
31	W27x94	439.14	443.53	0.44	OK
32	W27x94	291.67	294.59	0.29	OK
33	W27x94	248.95	251.44	0.25	OK
46	W27x94	286.70	289.57	0.28	OK
47	W27x94	258.45	261.03	0.25	OK
48	W27x94	263.45	266.07	0.26	OK
49	W27x94	292.52	295.45	0.28	OK
63	W27x84	85.63	85.63	0.10	OK
64	W27x84	94.86	94.86	0.10	OK
65	W27x84	92.73	92.73	0.10	OK
66	W27x84	88.49	88.49	0.10	OK

Final Member Selections Moment Frame 2

Element Number	Member Selection
1	W14 x 82
2	W14 x 159
3	W14 x 159
4	W14 x 159
5	W14 x 159
6	W14 x 159
7	W14 x 159
8	W14 x 159
9	W10 x 49
10	W27 x 94
11	W27 x 94
12	W27 x 94
13	W27 x 94
14	W27 x 94
15	W27 x 94
16	W27 x 94
17	W24 x 103
18	W10 x 49
19	W14 x 159
20	W14 x 159
21	W14 x 159
22	W14 x 159
23	W14 x 159
24	W14 x 159
25	W14 x 159
26	W10 x 49

Element Number	Member Selection
27	W27 x 94
28	W27 x 94
29	W27 x 94
30	W27 x 94
31	W27 x 94
32	W27 x 94
33	W27 x 94
34	W24 x 103
35	W10 x 33
36	W14 x 43
37	W14 x 82
38	W14 x 82
39	W14 x 82
40	W14 x 82
41	W14 x 43
42	W14 x 82
43	W10x 33
44	W16 x 26
45	W16 x 26
46	W27 x 94
47	W27 x 94
48	W27 x 94
49	W27 x 94
50	W24 x 103
51	W24 x 103
52	W10 x 33
53	W14 x 43
54	W14 x 82

Element Number	Member Selection
55	W14 x 82
56	W14 x 82
57	W14 x 43
58	W14 x 82
59	W14 x 82
60	W10 x 33
61	W16 x 26
62	W16 x 26
63	W27 x 84
64	W27 x 84
65	W27 x 84
66	W27 x 84
67	W24 x 103
68	W24 x 103

All members use A572 Gr 50 structural steel

Pinned Columns

Tentative Selections:

Column Number	W10 Sections	W12 Sections	W14 Sections
A1 u	W10 x 33	W12 x 40	W14 x 43
A1 l	W10 x 45	W12 x 45	W14 x 48
A2 u	W10 x 33	W12 x 40	W14 x 43
A2 l	W10 x 49	W12 x 53	W14 x 61
A3 u	W10 x 33	W12 x 40	W14 x 43
A3 l	W10 x 49	W12 x 53	W14 x 61
A4 u	W10 x 33	W12 x 40	W14 x 43
A4 l	W10 x 49	W12 x 53	W14 x 61
A5 u	W10 x 33	W12 x 40	W14 x 43
A5 l	W10 x 60	W12 x 58	W14 x 61
A6 u	W10 x 33	W12 x 40	W14 x 43
A6 l	W10 x 49	W12 x 53	W14 x 61
A7 u	W10 x 33	W12 x 40	W14 x 43
A7 l	W10 x 49	W12 x 53	W14 x 61
A8 u	W10 x 33	W12 x 40	W14 x 43
A8 l	W10 x 49	W12 x 53	W14 x 61
A9 u	W10 x 33	W12 x 40	W14 x 43
A9 l	W10 x 45	W12 x 45	W14 x 48
C1 u	W10 x 33	W12 x 40	W14 x 43
C1 l	W10 x 45	W12 x 45	W14 x 48
C2 u	W10 x 33	W12 x 40	W14 x 43
C2 l	W10 x 49	W12 x 53	W14 x 61
C3 u	W10 x 33	W12 x 40	W14 x 43
C3 l	W10 x 49	W12 x 53	W14 x 61
C4 u	W10 x 33	W12 x 40	W14 x 43

Column Number	W10 Sections	W12 Sections	W12 Sections
C4 l	W10 x 49	W12 x 53	W14 x 61
C5 u	W10 x 33	W12 x 40	W14 x 43
C5 l	W10 x 49	W12 x 53	W14 x 61
C6 u	W10 x 33	W12 x 40	W14 x 43
C6 l	W10 x 49	W12 x 53	W14 x 61
C7 u	W10 x 33	W12 x 40	W14 x 43
C7 l	W10 x 77	W12 x 65	W14 x 74
C9 u	W10 x 33	W12 x 40	W14 x 43
C9 l	W10 x 49	W12 x 53	W14 x 61
D7 u	W10 x 33	W12 x 40	W14 x 43
D7 l	W10 x 49	W12 x 53	W14 x 61
D9 u	W10 x 33	W12 x 40	W14 x 43
D9 l	W10 x 49	W12 x 53	W14 x 61
E7 u	W10 x 33	W12 x 40	W14 x 43
E7 l	W10 x 54	W12 x 53	W14 x 61
E9 u	W10 x 33	W12 x 40	W14 x 43
E9 l	W10 x 49	W12 x 53	W14 x 61
F7 u	W10 x 33	W12 x 40	W14 x 43
F7 l	W10 x 49	W12 x 53	W14 x 53
F9 u	W10 x 33	W12 x 40	W14 x 43
F9 l	W10 x 49	W12 x 53	W14 x 53
B1 u	W10 x 33	W12 x 40	W14 x 43
B1 l	W10 x 49	W12 x 53	W14 x 61
B2 u	N/A	N/A	W14 x 43
B7 u	N/A	N/A	W14 x 43

u = upper story l = lower story

Total Member Weights:

W10's - 32.37 tons

W12's - 34.00 tons

W14's - 37.82 tons

All members use A572 Gr 50 structural steel

APPENDIX C

FULLY RESTRAINED CONNECTIONS

Appendix C

Beam Connection Forces Moment Frame 1

Joint No.	Moment (K-FT)	Shear (KIPS)	Joint No.	Moment (K-FT)	Shear (KIPS)
7L	0.00	0.00	7R	451.60	21.45
8L	-416.30	24.21	8R	401.63	23.18
9L	-387.06	22.47	9R	403.79	30.62
10L	-386.00	30.16	10R	381.32	29.82
11L	-390.63	30.34	11R	396.12	31.24
12L	-420.19	28.23	12R	0.00	0.00
13L	0.00	0.00	13R	167.94	20.87
14L	-221.68	25.14	14R	237.17	23.19
15L	-223.68	22.46	15R	217.76	19.53
16L	-205.06	18.51	16R	207.33	18.85
17L	-206.97	19.20	17R	216.51	21.61
18L	-148.82	16.43	18R	0.00	0.00
19L	0.00	22.68	19R	112.31	20.34
20L	-169.49	25.32	20R	163.33	23.40
21L	-140.39	22.25	21R	117.62	19.57
22L	-99.40	18.47	22R	98.99	18.43
23L	-118.90	16.92	23R	128.07	21.72
24L	-98.44	16.32	24R	0.00	0.00
25L	0.00	6.88	25R	48.56	11.09
26L	-78.85	11.09	26R	52.22	9.94
27L	-81.63	12.25	27R	58.74	9.62
28L	-52.43	8.86	28R	58.95	10.37
29L	-34.77	8.12	29R	0.00	9.24

L- Left Joint   R - Right Joint

Fully Restrained Connections

Strength Limit States:

Shear Connection-

Bolt Shear Capacity	
Beam Gross Shear	(AISC LRFD F2-1)
Beam Net Shear	(AISC LRFD J4-1)
Beam Web Bearing Tearout	(AISC LRFD J3-1a)
Plate Gross Shear	(AISC LRFD J5-1)
Plate Net Shear	(AISC LRFD J5-2)
Plate Web Bearing Tearout	(AISC LRFD J3-1a)
Plate Block Shear	
Weld Strength	(AISC LRFD J2)
Column Flange Yielding	(AISC LRFD F2-1)

Moment Connection

Beam Moment Capacity	(AISC LRFD A-F1-1)
Column Local Flange Bending	(AISC LRFD K1-1)
Column Local Web Yielding	(AISC LRFD K1-2)
Column Web Crippling	(AISC LRFD K1-4)
Column Web Buckling	(AISC LRFD K1-8)

Connection Design Summary Moment Frame 1

Conn. Id	Joint No.	Plate Size	Bolts	Stiffners Required
1	7R	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	Yes
2	8R	3/16" x3 1/2" x 14 1/2"	(5) 3/4" A325N	No
3	8L	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No
4	9R	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No
5	9L	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No
6	10R	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No
7	10L	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No
8	11R	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No
9	11L	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No
10	12L	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No
11	13R	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No
12	14R	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No
13	14L	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No
14	15R	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No
15	15L	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No
16	16R	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No

Conn. Id	Joint	Plate Size	Bolts	Stiffners Required
17	16L	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No
18	17R	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No
19	17L	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No
20	18L	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No
21	19R	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No
22	20R	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No
23	20L	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No
24	21R	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No
25	21L	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No
26	22R	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No
27	22L	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No
28	23R	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No
29	23L	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No
30	24L	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No
31	26R	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No
32	27R	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No
33	27R	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No

Conn. Id.	Joint No.	Plate Size	Bolts	Stiffners Required
34	27L	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No
35	28R	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No
36	28L	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No

L - Left Side of Joint

R - Right Side of Joint

Doubler Plate Design Moment Frame 1

Joint No.	Flange Force	Panel Capacity	Plates Required	Column Depth	Cal. Plate t
7	239.98	197.05	YES	14.31	0.15
8	315.01	301.32	YES	14.98	0.05
9	333.10	301.32	YES	14.98	0.11
10	332.20	301.32	YES	14.98	0.11
11	330.00	301.32	YES	14.98	0.10
12	224.59	197.05	YES	14.31	0.10
13	89.91	197.05	NO	14.31	N/A
14	144.24	301.32	NO	14.98	N/A
15	143.60	301.32	NO	14.98	N/A
16	142.52	301.32	NO	14.98	N/A
17	144.44	301.32	NO	14.98	N/A
18	85.32	197.05	NO	14.31	N/A
19	57.28	197.05	NO	14.31	N/A
20	58.82	197.05	NO	14.31	N/A
21	72.79	197.05	NO	14.31	N/A
22	66.54	197.05	NO	14.31	N/A
23	82.94	197.05	NO	14.31	N/A
24	50.21	197.05	NO	14.31	N/A
25	15.32	197.05	NO	14.31	N/A
26	26.75	197.05	NO	14.31	N/A
27	26.91	197.05	NO	14.31	N/A
28	25.04	197.05	NO	14.31	N/A
29	23.43	197.05	NO	14.31	N/A

Design Summary

Use 2 PL 1/8" x 11 1/4" A36 structural steel all locations

Column Stiffner Design Moment Frame 1

Joint 7-

Use 2Pl 1/2" x 4 1/2" x 11 13/16" A36 structural steel

with 3/8" fillet weld to flange and 1/4" fillet weld to web

Check of LRFD Specification

Local Buckling-

b/t = 4.5"/1 1/8" = 4.00 < λ_r = 95/ $\sqrt{36}$ OK

Flange Force-

F = (952.50 K-Ft)(12"/')/26.92" = 236.02 K

Col. Local Flange Bending-

$\phi R_n = \phi 6.25 t_f^2 F_{yf}$

$= (0.90)(6.25)(0.86)^2(50)$

= 205.60 K < 236.02 K

Col. Local Web Yielding-

$\phi R_n = \phi(5k+N) F_{yw} t_w$

= (1.0)(5(1 7/8)+.745)(50)(0.745)

= 226.19 K < 236.02 K

Col. Web Crippling-

$\phi R_n = \phi 135 t_w^2 [1 + 3(N/d)(t_w/t_f)^{1.5}] \sqrt{F_{yw} t_f / t_w}$

$= (.75)(135)(.51)^2[1+3(.745/.431)(.510/.745)^{1.5}]$

$\sqrt{50(.745/.510)}$

= 258.46 K > 236.02 K

Col. Web Buckling-

$$\phi R_n = \frac{\phi 4{,}100 t_w^3 \sqrt{F_{yw}}}{d_c}$$

$$= \frac{(0.90)(4100)(0.510)^3\sqrt{50}}{11.06}$$

= 319.94 K > 236.02 K

Compression Design Force-

= 258.46 K - 236.02 K

= 22.44 K

Compression Yield Capacity-

= (2)(4.5"-0.75")(50 ksi)

= 375.00 K > 22.44 K OK

Flange Weld Capacity-

ϕR_n = (4.5"-0.75")(4)(1.392k/")(6)

= 125.28 > 22.40 K OK

Web Weld Capacity-

ϕR_n = (11 13/16"-0.75")(4)(1.393K/")(4)

= 246.38 K > 22.40 K OK

Beam Connection Forces Moment Frame 2

Joint No.	Moment (K-FT)	Shear (KIPS)	Joint No.	Moment (K-FT)	Shear (KIPS)
11L	-396.87	25.04	11R	663.94	42.48
12L	-489.01	36.42	12R	555.80	29.73
13L	-476.07	34.89	13R	564.21	30.12
14L	-482.35	35.34	14R	558.33	29.88
15L	-467.96	34.61	15R	538.55	27.99
16L	-525.15	72.82	16R	634.42	87.70
20L	-152.50	25.08	20R	265.84	20.87
21L	-218.43	19.13	21R	298.57	21.30
22L	-243.25	20.20	22R	299.74	21.35
23L	-243.12	20.18	23R	301.21	21.69
24L	-225.84	19.81	24R	211.98	13.93
25L	-292.76	74.80	25R	599.63	86.83
30L	0.00	20.00	30R	255.85	18.55
31L	-219.01	22.01	31R	218.38	20.39
32L	-180.28	19.61	2R	251.49	21.79
33L	-207.63	18.21	33R	0.00	20.00
39L	0.00	9.04	34R	75.11	7.93
40L	-83.21	10.15	39R	78.32	9.15
41L	-69.18	8.93	40R	81.37	9.94
42L	-77.62	8.14	41R	0.00	9.04

Connection Design Summary Moment Frame 2

Conn. Id	Joint No.	Plate Size	Bolts	Stiffners Required
1	11R	1/4" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	Yes
2	11L	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No
3	12R	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No
4	12L	1/4" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No
5	13R	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	Yes
6	13L	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	Yes
7	14R	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	Yes
8	14L	1/4" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No
9	15R	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	Yes
10	15L	1/4" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No
11	16R	3/8" x 3 1/2" x 21"	(7) 7/8" A325N	Yes
12	16L	1/4" x 3 1/2" x 23 1/2"	(8) 3/4" A325N	Yes
13	20R	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No
14	20L	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No
15	21R	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No
16	21L	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No

17	2R	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No
18	22L	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No
19	23R	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No
20	23L	3/16" x 31/2" x 14 1/2"	(5) 3/4" A325N	No
21	24R	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No
22	24L	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No
23	25R	3/8" x 3 1/2" x 21"	(7) 7/8" A325N	Yes
24	25L	3/8" x 3 1/2" x 18"	(6) 7/8" A325N	No
25	30R	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No
26	31R	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No
27	31L	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No
28	32R	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No
29	32L	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No
30	33L	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No
31	39R	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No
32	40R	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No
33	40L	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No
34	41R	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No

35	41L	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No
36	42L	3/16" x 3 1/2" x 14 1/2"	(5) 3/4" A325N	No

L - Left Side of Joint

R - Right Side of Joint

Doubler Plate Design Moment Frame 2

Joint No.	Flange Force	Panel Capacity	Plates Required	Column Depth	Cal. Plate t
11	483.96	301.31	YES	14.98	0.63
12	540.67	301.31	YES	14.98	0.82
13	528.67	301.31	YES	14.98	0.78
14	528.05	301.31	YES	14.98	0.78
15	517.85	301.31	YES	14.98	0.74
16	647.50	301.31	YES	14.98	1.19
20	117.61	301.31	NO	14.98	N/A
21	250.09	301.31	NO	14.98	N/A
22	259.75	301.31	NO	14.98	N/A
23	261.19	301.31	NO	14.98	N/A
24	216.78	301.31	NO	14.98	N/A
25	266.02	301.31	NO	14.98	N/A
30	137.33	197.05	NO	14.31	N/A
31	166.63	197.05	NO	14.31	N/A
32	167.63	197.05	NO	14.31	N/A
33	113.66	197.05	NO	14.31	N/A
39	45.27	197.05	NO	14.31	N/A
40	45.79	197.05	NO	14.31	N/A
41	45.52	197.05	NO	14.31	N/A
42	39.76	197.05	NO	14.31	N/A

Design Summary

Joint 1-

Use 2PL 3/8" x 11 1/4"

Joints 12,13,14, and 15

Use 2PL 1/2" x 11 1/4"

Joint 16

Use 2PL 1 1/8" x 11 1/4"

Use A36 structural steel all locations

Column Stiffner Design Moment Frame 2

Joints 11,13,14,

Use 2Pl 3/8" x 3" x 11 13/16" A36 structural steel

with 5/16" fillet weld to flange and 1/4" fillet weld to web

Check of LRFD Specification

Local Buckling-

b/t = 3.0"/1 1/8" = 2.67 < $\lambda_r = 95/\sqrt{36}$ OK

Flange Force-*

F = (643.20 K-ft -(-468.33 K-Ft))(12"/')/26.92" = 528.67 K

Col. Local Flange Bending-

$\phi R_n = \phi 6.25 t_f^2 F_{yf}$

$= (0.90)(6.25)(1.19)^2(50)$

= 398.28 K < 528.67 K

Col. Local Web Yielding-

$\phi R_n = \phi(5k+N) F_{yw} t_w$

= (1.0)(5(1 7/8)+1.19)(50)(1.19)

= 376.97 K < 528.67 K

Col. Web Crippling-

$\phi R_n = \phi 135 t_w^2 [1 + 3(N/d)(t_w/t_f)^{1.5}] \sqrt{F_{yw} t_f / t_w}$

$= (.75)(135)(.75)^2[1+3(0.75/14.98)(.75/1.19)^{1.5}]$

$\sqrt{50(1.19/.75)}$

= 539.33 K > 528.67 K

Col. Web Buckling-

$$\phi R_n = \frac{\phi 4,100 t_w^3 \sqrt{F_{yw}}}{d_c}$$

$$= \frac{(0.90)(4100)(0.750)\sqrt[3]{50}}{11.23}$$

= 960.73 K > 236.02 K

Compression Design Force-

= 528.67 K - 376.97 K

= 151.70 K

Compression Yield Capacity-

= (2)(3.0"-0.75")(50 ksi)

= 225.00 K > 151.70 K OK

Web Weld Capacity-

ϕR_n = (11 13/16"-0.75")(4)(1.393K/")(4)

= 246.38 K > 151.70 K OK

<u>Joint 16</u>

Use 2Pl 3/8" x 5 1/2" x 11 13/16" A36 structural steel with 5/16" fillet weld to flange and 1/4" fillet weld to web

Check of LRFD Specification

Local Buckling-

b/t = 5 1/2"/1 1/8" = 4.89 < λ_r = 95/ $\sqrt{36}$ OK

Flange Force-*

F = (735.95 K-ft -(-455.43 K-Ft))(12"/')/26.92" = 531.08 K

Col. Local Flange Bending-

$\phi R_n = \phi 6.25 t_f^2 F_{yf}$

= $(0.90)(6.25)(1.19)^2(50)$

= 398.28 K < 531.08 K

Col. Local Web Yielding-

$\phi R_n = \phi(5k+N)F_{yw}t_w$

$= (1.0)(5(1\ 7/8)+1.19)(50)(1.19)$

$= 376.97\ K < 531.08\ K$

Col. Web Crippling-

$\phi R_n = \phi 135 t_w^2 [1 + 3(N/d)(t_w/t_f)^{1.5}] \sqrt{F_{yw} t_f / t_w}$

$= (.75)(135)(.75)^2[1+3(0.75/14.98)(.75/1.19)^{1.5}]$

$\sqrt{50(1.19/.75)}$

$= 539.33\ K > 531.08\ K$

Col. Web Buckling-

$$\phi R_n = \frac{\phi 4,100 t_w^3 \sqrt{F_{yw}}}{d_c}$$

$$= \frac{(0.90)(4100)(0.750)^3 \sqrt{50}}{11.23}$$

$= 960.73\ K > 531.08\ K$

Compression Design Force-

$= 531.08\ K - 376.97\ K$

$= 154.11\ K$

Compression Yield Capacity-

$= (2)(5\ 1/2" - 0.75")(50\ ksi)$

$= 412.50\ K > 154.11\ K$ OK

Web Weld Capacity-

$\phi R_n = (11\ 13/16" - 0.75")(4)(1.393K/")(4)$

$= 246.38\ K > 154.11\ K$ OK

* Used the same stiffner for locations where moments are aproximately the same.

APPENDIX D

TORSION ANALYSIS

Appendix D

Initial Relative Stiffness

Braced Frame 1:

Deflection = 0.260"

Relative Stiffness = 1K/0.260" = 3.846 K/"

Brace Frame 2:

Deflection = 0.274"

Relative Stiffness = 1K/0.274" = 3.662 K/"

Brace Frame 3:

Deflection = 0.266"

Relative Stiffness = 1k/0.266" = 3.757 K/"

Moment Frame 1:

Deflection = 0.385"

Relative Stiffness = 1K/0.385" = 2.595 K/"

Moment Fralme 2:

Deflection = 0.263"

Relative Stiffnes = 1K/0.263" = 3.757 K/"

Initial Shear Center

Calculation of X:

$$X = \frac{(3.808K/")(40')+ 3.757K/")(146'8")}{3.808K/"+3.757K/"}$$

X = 92.97'

Calculation of Y:

$$Y = \frac{(2.595K/")(40')+(3.662K/")(110')+(3.846K/")(230')}{2.595K/"+3.662K/"+3.846K/"}$$

Y = 149.22'

Initial Eccentricity

e_x = 92.97"-90" = 2.97'

e_y = 149.12'-130.00" = 19.12'

Revised Relative Stiffness Moment Frame 1

Deflection = 0.285"

Relative Stiffness = 1K/0.285" = 3.505 K/"

Revised Shear Center

Recalculation of Y:

$$Y = \frac{(3.505K/")(40')+(3.662K/")(110')+(3.846K/")(230')}{3.505K/"+3.663K/"+3.846K/"}$$

Y = 129.63'

Revised Eccentricity

e_x = 2.97' (no change)

e_y = 129.63'- 130.00' = -0.37'

Torsional Force Calculation

Wind loads are applied along the princple axes of the building. North is assumed to act in the positive Y direction.

$$V_t = \frac{(W)(e)(d)(R)}{\Sigma R d^2}$$

West to East Loading-

Story Level 4

Frame No.	Relative Stiffness	e	d	Rd^2	V_w	V_t
BR1	3.846	0.37	100.37	38,745.13	88.14	0.14
BR2	3.662	0.37	-19.63	1,411.10	88.14	-0.03
BR3	3.757	2.97	53.70	10,834.02	88.14	0.59
MF1	3.505	0.37	-89.63	28,157.55	88.14	-0.11
MF2	3.808	2.97	-52.97	10,684.57	88.14	-0.59

Σ 89,832.37

Story Level 3

Frame No.	Relative Stiffness	e	d	Rd^2	V_w	V_t
BR1	3.846	0.37	100.37	38,745.13	218.96	0.35
BR2	3.662	0.37	-19.63	1,411.10	218.96	-0.06
BR3	3.757	2.97	53.70	10,834.02	218.96	1.46
MF1	3.505	0.37	-89.63	28,157.55	218.96	-0.28
MF2	3.808	2.97	-52.97	10,684.57	218.96	-1.46

Σ 89,832.37

Story Level 2

Frame No.	Relative Stiffness	e	d	Rd^2	V_w	V_t
BR1	3.846	0.37	100.37	38,745.13	342.23	0.54
BR2	3.662	0.37	-19.63	1,411.10	342.23	-0.10
BR3	3.757	2.97	53.70	10,834.02	342.23	2.28
MF1	3.505	0.37	-89.63	28,157.55	342.23	-0.44
MF2	3.808	2.97	-52.97	10,684.57	342.23	-2.28

Σ 89,832.37

Story Level 1

Frame No.	Relative Stiffness	e	d	Rd^2	V_w	V_t
BR1	3.846	0.37	100.37	38,745.13	443.91	0.71
BR2	3.662	0.37	-19.63	1,411.10	443.91	-0.13
BR3	3.757	2.97	53.70	10,834.02	443.91	2.96
MF1	3.505	0.37	-89.63	28,157.55	443.91	-0.57
MF2	3.808	2.97	-52.97	10,684.57	443.91	-2.96

Σ 89,832.37

East to West Loading-

Story Level 4

Frame No.	Relative Stiffness	e	d	Rd^2	V_w	V_t
BR1	3.846	0.37	100.37	38,745.13	-88.14	-0.14
BR2	3.662	0.37	-19.63	1,411.10	-88.14	0.03
BR3	3.757	2.97	53.70	10,834.02	-88.14	-0.59
MF1	3.505	0.37	-89.63	28,157.55	-88.14	0.11
MF2	3.808	2.97	-52.97	10,684.57	-88.14	0.59

Σ 89,832.37

Story Level 3

Frame No.	Relative Stiffness	e	d	Rd^2	V_w	V_t
BR1	3.846	0.37	100.37	38,745.13	-218.96	0.35
BR2	3.662	0.37	-19.63	1,411.10	-218.96	-0.06
BR3	3.757	2.97	53.70	10,834.02	-218.96	1.46
MF1	3.505	0.37	-89.63	28,157.55	-218.96	-0.28
MF2	3.808	2.97	-52.97	10,684.57	-218.96	-1.46

Σ 89,832.37

Story Level 2

Frame No.	Relative Stiffness	e	d	Rd^2	V_w	V_t
BR1	3.846	0.37	100.37	38,745.13	-342.23	-0.54
BR2	3.662	0.37	-19.63	1,411.10	-342.23	0.10
BR3	3.757	2.97	53.70	10,834.02	-342.23	-2.28
MF1	3.505	0.37	-89.63	28,157.55	-342.23	0.44
MF2	3.808	2.97	-52.97	10,684.57	-342.23	2.28

Σ 89,832.37

Story Level 1

Frame No.	Relative Stiffness	e	d	Rd^2	V_w	V_t
BR1	3.846	0.37	100.37	38,745.13	-443.91	-0.71
BR2	3.662	0.37	-19.63	1,411.10	-443.91	0.13
BR3	3.757	2.97	53.70	10,834.02	-443.91	-2.96
MF1	3.505	0.37	-89.63	28,157.55	-443.91	0.57
MF2	3.808	2.97	-52.97	10,684.57	-443.91	2.96

Σ 89,832.37

North to South Loading-

Story Level 4

Frame No.	Relative Stiffness	e	d	Rd2	Vw	Vt
BR1	3.846	0.37	100.37	38,745.13	42.90	0.07
BR2	3.662	0.37	-19.63	1,411.10	42.90	-0.01
BR3	3.757	2.97	53.70	10,834.02	42.90	0.29
MF1	3.505	0.37	-89.63	28,157.55	42.90	-0.06
MF2	3.808	2.97	-52.97	10,684.57	42.90	-0.29

Σ 89,832.37

Story Level 3

Frame No.	Relative Stiffness	e	d	Rd2	Vw	Vt
BR1	3.846	0.37	100.37	38,745.13	131.90	0.21
BR2	3.662	0.37	-19.63	1,411.10	131.90	-0.04
BR3	3.757	2.97	53.70	10,834.02	131.90	0.88
MF1	3.505	0.37	-89.63	28,157.55	131.90	-0.17
MF2	3.808	2.97	-52.97	10,684.57	131.90	-0.88

Σ 89,832.37

Story Level 2

Frame No.	Relative Stiffness	e	d	Rd2	Vw	Vt
BR1	3.846	0.37	100.37	38,745.13	218.47	0.35
BR2	3.662	0.37	-19.63	1,411.10	218.47	-0.06
BR3	3.757	2.97	53.70	10,834.02	218.47	1.46
MF1	3.505	0.37	-89.63	28,157.55	218.47	-0.28
MF2	3.808	2.97	-52.97	10,684.57	218.47	-1.46

Σ 89,832.37

Story Level 1

Frame No.	Relative Stiffness	e	d	Rd2	Vw	Vt
BR1	3.846	0.37	100.37	38,745.13	284.73	0.45
BR2	3.662	0.37	-19.63	1,411.10	284.73	-0.08
BR3	3.757	2.97	53.70	10,834.02	284.73	1.90
MF1	3.505	0.37	-89.63	28,157.55	284.73	-0.37
MF2	3.808	2.97	-52.97	10,684.57	284.73	-1.90

Σ 89,832.37

South to North Loading

Story Level 4

Frame No.	Relative Stiffness	e	d	Rd^2	V_w	V_t
BR1	3.846	0.37	100.37	38,745.13	-42.90	-0.07
BR2	3.662	0.37	-19.63	1,411.10	-42.90	0.01
BR3	3.757	2.97	53.70	10,834.02	-42.90	-0.29
MF1	3.505	0.37	-89.63	28,157.55	-42.90	0.06
MF2	3.808	2.97	-52.97	10,684.57	-42.90	0.29

Σ 89,832.37

Story Level 3

Frame No.	Relative Stiffness	e	d	Rd^2	V_w	V_t
BR1	3.846	0.37	100.37	38,745.13	-131.90	-0.21
BR2	3.662	0.37	-19.63	1,411.10	-131.90	0.04
BR3	3.757	2.97	53.70	10,834.02	-131.90	-0.88
MF1	3.505	0.37	-89.63	28,157.55	-131.90	0.17
MF2	3.808	2.97	-52.97	10,684.57	-131.90	0.88

Σ 89,832.37

Story Level 2

Frame No.	Relative Stiffness	e	d	Rd^2	V_w	V_t
BR1	3.846	0.37	100.37	38,745.13	-218.47	-0.35
BR2	3.662	0.37	-19.63	1,411.10	-218.47	0.06
BR3	3.757	2.97	53.70	10,834.02	-218.47	-1.46
MF1	3.505	0.37	-89.63	28,157.55	-218.47	0.28
MF2	3.808	2.97	-52.97	10,684.57	-218.47	1.46

Σ 89,832.37

Story Level 1

Frame No.	Relative Stiffness	e	d	Rd^2	V_w	V_t
BR1	3.846	0.37	100.37	38,745.13	-284.73	-0.45
BR2	3.662	0.37	-19.63	1,411.10	-284.73	0.08
BR3	3.757	2.97	53.70	10,834.02	-284.73	-1.90
MF1	3.505	0.37	-89.63	28,157.55	-284.73	0.37
MF2	3.808	2.97	-52.97	10,684.57	-284.73	1.90

Σ 89,832.37

APPENDIX E

SEISMIC ANALYSIS

Appendix E

Page

Center of Mass Calculation

Story Level 4:

Part No.	Weight (Kips)	Mass	X^ (ft.)	Y^ (ft.)	MX^	MY^
1	332.00	0.86	40.00	130.00	34.37	111.70
2	162.00	0.42	130.00	40.00	54.50	16.77

X = 69.51 ft. Y = 100.49 ft.

Story Level 3:

Part No.	Weight (Kips)	Mass	X^ (ft.)	Y^ (ft.)	MX^	MY^
1	1474.00	3.81	40.00	130.00	152.59	495.91
2	705.00	1.82	130.00	40.00	237.19	72.98

X = 69.12 ft. Y = 100.88 ft.

Part No.	Weigtht (kips)	Mass	X^ (ft.)	Y^ (ft.)	MX^	MY^
1	1474.00	3.81	40.00	130.00	152.59	495.91
2	705.00	1.82	130.00	40.00	237.19	72.98

X = 69.12 ft. Y = 100.88 ft.

Story Level 1:

Part No.	Weight (Kips)	Mass	X^	Y^	MX^	MY^
1	1474.00	3.81	40.000	130.00	152.59	495.91
2	705.00	1.82	130.00	40.00	237.19	72.98

X = 69.12 ft. Y = 100.88 ft.

Mass Moment of Inertia Calculation

Story Level 4:

Part No.	Mass	a (ft.)	b (ft.)	Io	d^2	Md^2
1	0.86	260.00	80.00	5298.48	1742.17	1496.90
2	0.42	100.00	80.00	572.98	7317.07	3067.72

Iz = 1,502,795.08 K-sec²-in.

Story Level 3:

Part No.	Mass	a (ft.)	b (ft.)	Io	d^2	Md^2
1	3.81	260.00	80.00	23523.98	1695.82	6469.03
2	1.82	100.00	80.00	7413.03	7413.03	13525.3

Iz = 6.625,707.99 K-sec²-in.

Story Level 2:

Part No.	Mass	a (ft.)	b (ft.)	Io	d^2	Md^2
1	3.81	260.00	80.00	23523.98	1695.82	6469.03
2	1.82	100.00	80.00	7413.03	7413.03	13525.3

Iz = 6.625,707.99 K-sec²-in.

Story Level 1:

Part No.	Mass	a (ft.)	b (ft.)	Io	d^2	Md^2
1	3.81	260.00	80.00	23523.98	1695.82	6469.03
2	1.82	100.00	80.00	7413.03	7413.03	13525.3

Iz = 6.625,707.99 K-sec²-in.

Response Spectrum

Ground Acceleration:

A_v = 0.10 g (Figure 1113.1 BOCA 1990)

Critical Dampling:

D = 5% (Table 3-2 Seismic Design Handbook)

Ground Velocity:

V_g = 0.10 (48.0 in./sec.) = 4.80 in./sec.

Ground Displacement:

X = 0.10 (36 in.) = 3.60 in.

Amplified Response Parameters:

(Table 3-2 Seismic Design Handbook)

Av = (.10 g)(2.6) = 0.26 g

V_g = (4.80 in./sec.)(1.9) = 9.12 in./sec.

X = (3.60 in.)(1.4) = 5.04 in./sec.

Spectral Displacements and Building Periods:

Tp (sec.)	Sdx (in.)	Sdy (in.)
0.4	0.220	0.220
0.5	0.245	0.245
0.6	0.255	0.255
0.7	0.255	0.255
0.8	0.215	0.215
0.9	0.210	0.210
1.0	0.140	0.140

Final Braced Frame Design

Braced Frame 1:

A. Diagonal Bracing Members -

Effective Length Factors (all frames)

k_x = 0.50 therefore K_xl = (42.38')(0.50) = 21.19'

k_y = 0.67 therefore K_yl = (42,38')(0.67) = 28.39'

(Ref. pg. 255, Seismic Design Handbook)

Story Level 4 -

P_{max} = 95.30 K

2L 8x8x3/4

Section Properties

A_g = 22.9 in^2

r_x = 2.47 in

r_y = 3.49 in

Local Buckling

b/t = 8/0.75 = 10.67 < $76/\sqrt{36}$ = 12.67 OK

Slenderness

kl/r_x = (21.19 in.)(12in/ft)/2.47in.

= 102.95 < 720/ 36 = 120 OK

kl/r_y = (28.39 in.)(12 in./ft)/3.39 in.

= 97.62 < 720/ 36 = 120 OK

Compressive Capacity

ϕFcr = 17.70 ksi

$0.8\phi Pc = (0.8)(17.70\ ksi)(22.9\ in^2)$

$= 324.26\ K > 95.30\ K$ OK

Tension Capacity

Assume 3/4" A325 Bolts will be used

$A_{net} = 22.9\ in^2 - (7/8\ in.)(3/4\ in.) = 21.59\ in^2$

$A_e = (0.85)(21.59\ in^2) = 18.38\ in^2$

ϕTn = min of

$(0.90)(22.9\ in^2)(36\ ksi) = 741.96\ K$ Controls

or

$(0.75)(18.38\ in^2)(58\ ksi) = 798.20\ K$

$741.96\ K > 95.30\ K$ OK

Use 2L 8x8x3/4

Story Level 3 -

$P_{max} = 219.92\ K$

2 L 8x8x3/4

Section Properties

$A_g = 22.9\ in^2$

$r_x = 2.47$ in

$r_y = 3.49$ in

Local Buckling

$b/t = 8/0.75 = 10.67 < 76/\sqrt{36} = 12.67$ OK

Slenderness

$kl/r_x = (21.19\ in.)(12in/ft)/2.47in.$

$= 102.95 < 720/\sqrt{36} = 120$ OK

$kl/r_y = (28.39\ in.)(12\ in./ft)/3.39\ in.$

= 97.62 < 720/ 36 = 120 OK

<u>Compressive Capacity</u>

ϕFcr = 17.70 ksi

0.8ϕPc = (0.8)(17.70 ksi)(22.9 in^2)

= 324.26 K > 219.92 K OK

<u>Tension Capacity</u>

Assume 3/4" A325 Bolts will be used

A_{net} = 22.9 in^2 - (7/8 in.)(3/4 in.) = 21.59 in^2

A_e = (0.85)(21.59 in^2) = 18.38 in^2

ϕTn = min of

(0.90)(22.9 in^2)(36 ksi) = 741.96 K Controls

or

(0.75)(18.38 in^2)(58 ksi) = 798.20 K

741.96 K > 219.92 K OK

Use 2 L 8x8x3/4

<u>Story Level 2 -</u>

P_{max} = 237.69 K

2L 8x8x3/4

<u>Section Properties</u>

A_g = 22.9 in^2

r_x = 2.47 in

r_y = 3.49 in

<u>Local Buckling</u>

b/t = 8/0.75 = 10.67 < 76/ 36 = 12.67 OK

Slenderness

kl/r_x = (21.19 in.)(12in/ft)/2.47in.

= 102.95 < 720/ 36 = 120 OK

kl/r_y = (28.39 in.)(12 in./ft)/3.39 in.

= 97.62 < 720/ 36 = 120 OK

Compressive Capacity

ϕFcr = 17.70 ksi

0.8ϕPc = (0.8)(17.70 ksi)(22.9 in^2)

= 324.26 K > 237.69 K OK

Tension Capacity

Assume 3/4" A325 Bolts will be used

A_{net}= 22.9 in^2 - (7/8 in.)(3/4 in.) = 21.59 in^2

A_e = (0.85)(21.59 in^2) = 18.38 in^2

ϕTn = min of

(0.90)(22.9 in^2)(36 ksi) = 741.96 K Controls

or

(0.75)(18.38 in^2)(58 ksi) = 798.20 K

741.96 K > 237.69 K OK

Use 2L 8x8x3/4

Story Level 1 -

P_{max} = 292.73 K

2L 8x8x3/4

Section Properties

A_g = 22.9 in^2

r_x = 2.47 in

r_y = 3.49 in

<u>Local Buckling</u>

b/t = 8/0.75 = 10.67 < 76/ 36 = 12.67 OK

<u>Slenderness</u>

kl/r_x = (21.19 in.)(12in/ft)/2.47in.

= 102.95 < 720/ 36 = 120 OK

kl/r_y = (28.39 in.)(12 in./ft)/3.39 in.

= 97.62 < 720/ 36 = 120 OK

<u>Compressive Capacity</u>

ϕFcr = 17.70 ksi

0.8ϕPc = (0.8)(17.70 ksi)(22.9 in^2)

= 324.26 K > 237.69 K OK

<u>Tension Capacity</u>

Assume 3/4" A325 Bolts will be used

A_{net}= 22.9 in^2 - (7/8 in.)(3/4 in.) = 21.59 in^2

A_e = (0.85)(21.59 in^2) = 18.38 in^2

ϕTn = min of

(0.90)(22.9 in^2)(36 ksi) = 741.96 K Controls

or

(0.75)(18.38 in^2)(58 ksi) = 798.20 K

741.96 K > 237.69 K OK

Use 2L 8x8x3/4

B. Columns

<u>Upper Stories-</u>

W10 x 33, ϕPn = 239 K

$P_g = 98.35$ K

$P_e = 125.68$ K

$P_u = 98.35$ K + 125.68 K

= 224 K < 239 K OK

Use W10 x 33

Lower Stories-

W10 x 88. $\phi Pn = 746$ K

$P_g = 254.85$ K

$P_e = 436.36$ K

$P_u = 254.85$ K + 436.36 K

= 691.21 K < 746 K OK

Use W10 x 88

Braced Frame 2:

A. Diagonal Bracing Members

Story Level 4 -

P_{max} = 117.82 K

2L 8x8x3/4

Section Properties

A_g = 22.9 in^2

r_x = 2.47 in

r_y = 3.49 in

Local Buckling

b/t = 8/0.75 = 10.67 < 76/ 36 = 12.67 OK

Slenderness

kl/r_x = (21.19 in.)(12in/ft)/2.47in.

= 102.95 < 720/ 36 = 120 OK

kl/r_y = (28.39 in.)(12 in./ft)/3.39 in.

= 97.62 < 720/ 36 = 120 OK

Compressive Capacity

ϕFcr = 17.70 ksi

0.8ϕPc = (0.8)(17.70 ksi)(22.9 in^2)

= 324.26 K > 117.82 K OK

Tension Capacity

Assume 3/4" A325 Bolts will be used

A_{net}= 22.9 in^2 - (7/8 in.)(3/4 in.) = 21.59 in^2

A_e = (0.85)(21.59 in^2) = 18.38 in^2

ϕTn = min of

(0.90)(22.9 in^2)(36 ksi) = 741.96 K Controls

or

(0.75)(18.38 in^2)(58 ksi) = 798.20 K

741.96 K > 117.82 K OK

Use 2L 8x8x3/4

Story Level 3 -

P_{max} = 153.23 K

2L 8x8x3/4

Section Properties

A_g = 22.9 in^2

r_x = 2.47 in

r_y = 3.49 in

Local Buckling

b/t = 8/0.75 = 10.67 < 76/ 36 = 12.67 OK

Slenderness

kl/r_x = (21.19 in.)(12in/ft)/2.47in.

= 102.95 < 720/ 36 = 120 OK

kl/r_y = (28.39 in.)(12 in./ft)/3.39 in.

= 97.62 < 720/ 36 = 120 OK

Compressive Capacity

ϕFcr = 17.70 ksi

$0.8\phi Pc$ = (0.8)(17.70 ksi)(22.9 in^2)

= 324.26 K > 153.23 K OK

Tension Capacity

Assume 3/4" A325 Bolts will be used

A_{net}= 22.9 in^2 - (7/8 in.)(3/4 in.) = 21.59 in^2

A_e = (0.85)(21.59 in^2) = 18.38 in^2

ϕTn = min of

(0.90)(22.9 in^2)(36 ksi) = 741.96 K Controls

or

(0.75)(18.38 in^2)(58 ksi) = 798.20 K

741.96 K > 153.23 K OK

Use 2L 8x8x3/4

Story Level 2 -

P_{max} = 345.51 K

2L 8x8x7/8

Section Properties

A_g = 26.5 in^2

r_x = 2.45 in

r_y = 3.51 in

Local Buckling

b/t = 8/0.88 = 9.14 < 76/ 36 = 12.67 OK

Slenderness

kl/r_x = (21.19 in.)(12in/ft)/2.45in.

= 103.79 < 720/ 36 = 120 OK

kl/r_y = (28.39 in.)(12 in./ft)/3.51 in.

= 97.06 < 720/ 36 = 120 OK

Compressive Capacity

$\phi Fcr = 17.36$ ksi

$0.8\phi Pc = (0.8)(17.36 \text{ ksi})(26.5 \text{ in}^2)$

$= 368.03 \text{ K} > 237.69 \text{ K}$ OK

Tension Capacity

Assume 3/4" A325 Bolts will be used

$A_{net} = 26.5 \text{ in}^2 - (7/8 \text{ in.})(7/8 \text{ in.}) = 25.73 \text{ in}^2$

$A_e = (0.85)(25.73 \text{ in}^2) = 21.87 \text{ in}^2$

ϕTn = min of

$(0.90)(26.5 \text{ in}^2)(36 \text{ ksi}) = 858.60$ K Controls

or

$(0.75)(21.87 \text{ in}^2)(58 \text{ ksi}) = 951.53$ K

$858.60 \text{ K} > 345.51 \text{ K}$ OK

Use 2L 8x8x7/8

Story Level 1 -

$P_{max} = 460.18$ K

2L 8x8x1 1/8

Section Properties

$A_g = 33.5 \text{ in}^2$

$r_x = 2.42$ in

$r_y = 3.55$ in

Local Buckling

b/t = 8/1 1/8 = 7.11 < 76/ 36 = 12.67 OK

Slenderness

kl/r_x = (21.19 in.)(12in/ft)/2.42in.

= 105.07 < 720/ 36 = 120 OK

kl/r_y = (28.39 in.)(12 in./ft)/3.55 in.

= 95.97 < 720/ 36 = 120 OK

Compressive Capacity

ϕFcr = 17.31 ksi

0.8ϕPc = (0.8)(17.31 ksi)(33.5 in^2)

= 463.91 K > 460.18 K OK

Tension Capacity

Assume 3/4" A325 Bolts will be used

A_{net}= 33.5 in^2 - (7/8 in.)(1 1/8 in.) = 32.52 in^2

A_e = (0.85)(32.52 in^2) = 27.64 in^2

ϕTn = min of

(0.90)(33.5 in^2)(36 ksi) = 1085.40 K Controls

or

(0.75)(27.64 in^2)(58 ksi) = 1202.27 K

1085.40 K > 460.18 K OK

Use 2L 8x8x1 1/8

B. Columns

Upper Stories -

W10 x 33, ϕPn = 239 K

P_g = 78.00 K

P_e = 115.60 K

P_u = 78.00 K + 115.60 K

= 193.60 K < 239 K OK

Use W10 x 33

<u>Lower Stories -</u>

W10 x 88, ϕPn = 746 K

P_g = 188.00 K

P_e = 539.99 K

P_u = 188.00 K + 540.00 K

= 728.00 K < 746 K OK

Use W10 x 88

Braced Frame 3:

A. Diagonal Bracing Members

Story Level 4 -

P_{max} = 265.41 K

2L 8x8x3/4

Section Properties

A_g = 22.9 in^2

r_x = 2.47 in

r_y = 3.49 in

Local Buckling

b/t = 8/ 3/4 = 10.67 < 76/ 36 = 12.67 OK

Slenderness

kl/r_x = (21.19 in.)(12in/ft)/2.47in.

= 102.95 < 720/ 36 = 120 OK

kl/r_y = (28.39 in.)(12 in./ft)/3.49 in.

= 97.62 < 720/ 36 = 120 OK

Compressive Capacity

ϕFcr = 17.70 ksi

0.8ϕPc = (0.8)(17.70 ksi)(21.29 in^2)

= 324.26 K > 265.41 K OK

Tension Capacity

Assume 3/4" A325 Bolts will be used

A_{net} = 22.9 in^2 - (7/8 in.)(3/4 in.) = 21.59 in^2

A_e = (0.85)(21.59 in^2) = 18.38 in^2

ϕTn = min of

(0.90)(22.9 in^2)(36 ksi) = 741.96 K Controls

or

(0.75)(18.38 in^2)(58 ksi) = 798.20 K

741.96 K > 265.41 K OK

Use 2L 8x8x3/4

Story Level 3-

P_{max} = 293.73 K

2L 8x8x3/4

Section Properties

A_g= 22.9 in^2

r_x = 2.47 in

r_y = 3.49 in

Local Buckling

b/t = 8/0.75 = 10.67 < 76/ 36 = 12.67 OK

Slenderness

kl/r_x = (21.19 in.)(12in/ft)/2.47in.

= 102.95 < 720/ 36 = 120 OK

kl/r_y = (28.39 in.)(12 in./ft)/3.39 in.

= 97.62 < 720/ 36 = 120 OK

Compressive Capacity

ϕFcr = 17.70 ksi

$0.8\phi Pc$ = (0.8)(17.70 ksi)(22.9 in^2)

= 324.26 K > 293.73 K OK

Tension Capacity

Assume 3/4" A325 Bolts will be used

A_{net}= 22.9 in² - (7/8 in.)(3/4 in.) = 21.59 in²

A_e = (0.85)(21.59 in²) = 18.38 in²

ϕTn = min of

(0.90)(22.9 in²)(36 ksi) = 741.96 K Controls

or

(0.75)(18.38 in²)(36 ksi) = 798.20 K

741.96 K > 293.73 K OK

Use 2L 8x8x3/4

Story Level 2 -

P_{max} = 371.77 K

2L 8x8x1

Section Properties

A_g = 30.0 in²

r_x = 2.44 in

r_y = 3.53 in

Local Buckling

b/t = 8/1 = 10.67 < 76/ 36 = 12.67 OK

Slenderness

kl/r_x = (21.19 in.)(12in/ft)/2.44in.

= 104.21 < 720/ 36 = 120 OK

kl/r_y = (28.39 in.)(12 in./ft)/3.53 in.

= 96.51 < 720/ 36 = 120 OK

Compressive Capacity

ϕFcr = 17.28 ksi

$0.8\phi Pc$ = (0.8)(17.28 ksi)(30.0 in^2)

= 414.72 K > 371.77 K OK

Tension Capacity

Assume 3/4" A325 Bolts will be used

A_{net}= 30.0 in^2 - (7/8 in.)(1 in.) = 29.13 in^2

A_e = (0.85)(29.13 in^2) = 24.76 in^2

ϕTn = min of

(0.90)(30.0 in^2)(36 ksi) = 972.00 K Controls

or

(0.75)(24.76 in^2)(58 ksi) = 1076.90 K

972.00 K > 371.77 K OK

Use 2L 8x8x1

Story Level 1-

P_{max} = 457.77 K

2L 8x8x1 1/8

Section Properties

A_g = 33.5 in^2

r_x = 2.42 in

r_y = 3.55 in

Local Buckling

b/t = 8/1 1/8 = 7.11 < 76/ 36 = 12.67 OK

<u>Slenderness</u>

kl/r_x = (21.19 in.)(12in/ft)/2.42in.

= 105.07 < 720/ 36 = 120 OK

kl/r_y = (28.39 in.)(12 in./ft)/3.55 in.

= 95.97 < 720/ 36 = 120 OK

<u>Compressive Capacity</u>

ϕFcr = 17.31 ksi

0.8ϕPc = (0.8)(17.31 ksi)(33.5 in^2)

= 463.91 K > 457.77 K OK

<u>Tension Capacity</u>

Assume 3/4" A325 Bolts will be used

A_{net}= 33.5 in^2 - (7/8 in.)(1 1/8) = 32.52 in^2

A_e = (0.85)(32.52 in^2) = 27.64 in^2

ϕTn = min of

(0.90)(33.5 in^2)(36 ksi) = 1085.40 K Controls

or

(0.75)(27.64 in^2)(58 ksi) = 1202.34 K

741.96 K > 237.69 K OK

Use 2L 8x8x1 1/8

B. Columns

<u>Upper Stories-</u>

W10 x 33, ϕPn = 239 K

P_g = 104.84 K

P_e = 87.75 K

P_u = 104.84 K + 87.75 K

= 192.59 K < 239 K OK

Use W10 x 33

Lower Stories-

W10 x 100, ϕPn = 851 K

P_g = 263.98 K

P_e = 510.22 K

P_u = 263.98 K + 510.22 K

= 774.20 K < 851 K OK

Use W10 x 100

Moment Frame 1 Column Design

Design Strength:

Mem No.	Sect.	Area	rx	ry	Kx	Ky	L	ϕPn	ϕMn
1	W14x82	24.1	6.05	2.48	1.92	1.00	16	777	521
2	W14x159	46.7	6.38	4.00	2.02	1.00	16	1782	1076
3	W14x159	46.7	6.38	4.00	1.95	1.00	16	1815	1076
4	W14x159	46.7	6.38	4.00	1.90	1.00	16	1839	1076
5	W14x159	46.7	6.38	4.00	1.90	1.00	16	1839	1076
6	W14x82	24.1	6.05	2.48	1.85	1.00	16	777	521
12	W14x82	24.1	6.05	2.48	1.48	1.00	14	862	521
13	W14x159	46.7	6.38	4.00	1.45	1.00	14	2052	1076
14	W14x159	46.7	6.38	4.00	1.38	1.00	14	2052	1076
15	W14x159	46.7	6.38	4.00	1.35	1.00	14	2052	1076
16	W14x159	46.7	6.38	4.00	1.35	1.00	14	2052	1076
17	W14x82	24.1	6.05	2.48	1.40	1.00	14	862	521
23	W14x82	24.1	6.05	2.48	1.35	1.00	14	862	521
24	W14x82	24.1	6.05	2.48	1.32	1.00	14	862	521
25	W14x82	24.1	6.05	2.48	1.28	1.00	14	862	521
26	W14x82	24.1	6.05	2.48	1.27	1.00	14	862	521
27	W14x82	24.1	6.05	2.48	1.27	1.00	14	862	521
28	W14x82	24.1	6.05	2.48	1.32	1.00	14	862	521
34	W14x82	24.1	6.05	2.48	1.00	1.00	14	862	521
35	W14x82	24.1	6.05	2.48	1.25	1.00	14	862	521
36	W14x82	24.1	6.05	2.48	1.18	1.00	14	862	521
37	W14x82	24.1	6.05	2.48	1.15	1.00	14	862	521
38	W14x82	24.1	6.05	2.48	1.22	1.00	14	862	521
39	W14x82	24.1	6.05	2.48	1.00	1.00	14	862	521

Moment Amplifier B1:

Mem No.	Section	P_e	C_m	P_u	B1
1	W14x82	6,848.94	0.32	226.30	1.00
2	W14x159	14,758.88	0.33	501.51	1.00
3	W14x159	14,758.88	0.32	445.72	1.00
4	W14x159	14,758.88	0.31	415.56	1.00
5	W14x159	14,758.88	0.31	771.29	1.00
6	W14x82	6,848.94	0.38	258.13	1.00
12	W14x82	8,945.55	0.24	311.10	1.00
13	W14x159	19,276.91	0.26	350.73	1.00
14	W14x159	19,276.91	0.26	316.70	1.00
15	W14x159	19,276.91	0.25	301.89	1.00
16	W14x159	19,276.91	0.24	454.81	1.00
17	W14x82	8,945.55	0.24	178.36	1.00
23	W14x82	8,945.55	0.23	169.44	1.00
24	W14x82	8,945.55	0.26	201.66	1.00
25	W14x82	8,945.55	0.26	181.02	1.00
26	W14x82	8,945.55	0.24	173.36	1.00
27	W14x82	8,945.55	0.24	185.90	1.00
28	W14x82	8,945.55	0.25	102.47	1.00
34	W14x82	8,945.55	0.26	30.66	1.00
35	W14x82	8,945.55	0.23	53.30	1.00
36	W14x82	8,945.55	0.23	47.13	1.00
37	W14x82	8,945.55	0.24	47.86	1.00
38	W14x82	8,945.55	0.23	59.95	1.00
39	W14x82	8,945.55	0.24	30.18	1.00

Moment Amplifier B2:

Mem No.	Section	ΣP_u	P_e	m	ΣP_e	B2
1	W14x82	4,684.40	1,857.89	6	11,147.36	1.72
2	W14x159	4,684.40	3,617.02	6	21,702.11	1.28
3	W14x159	4,684.40	3,881.36	6	23,288.18	1.25
4	W14x159	4,684.40	4,088.33	6	24,530.00	1.24
5	W14x159	4,684.40	4,088.33	6	24,530.00	1.24
6	W14x82	4,684.40	2,001.15	6	12,006.90	1.64
12	W14x82	3,518.20	4,083.98	6	24,503.88	1.17
13	W14x159	3,518.20	9,168.57	6	55,011.40	1.07
14	W14x159	3,518.20	10,122.30	6	60,733.81	1.06
15	W14x159	3,518.20	10,577.18	6	63,463.08	1.06
16	W14x159	3,518.20	10,577.18	6	63,463.08	1.06
17	W14x82	3,518.20	4,564.06	6	27,383.33	1.22
23	W14x82	1,752.00	4,908.39	6	29,450.37	1.06
24	W14x82	1,752.00	5,134.04	6	30,804.23	1.06
25	W14x82	1,752.00	5,459.93	6	32,759.58	1.06
26	W14x82	1,752.00	5,546.25	6	33,277.51	1.06
27	W14x82	1,752.00	5,546.25	6	33,277.51	1.06
28	W14x82	1,752.00	5,134.04	6	30,804.23	1.06
34	W14x82	645.80	8,945.55	6	35.782.19	1.02
35	W14x82	645.80	5,725.15	6	22,900.80	1.03
36	W14x82	645.80	6,424.55	6	25,698.21	1.03
37	W14x82	645.80	6,764.12	6	27,056.48	1.02
38	W14x82	645.80	6,010.18	6	24,040.71	1.03
39	W14x82	645.80	8,945.55	6	35,783.19	1.02

Column Evaluation:

Mem No.	Section	M_{nt}	M_{lt}	M_u	H1-1	Status
1	W14x82	33.57	218.48	410.41	0.99	OK
2	W14x159	6.57	452.60	583.76	0.76	OK
3	W14x159	9.66	458.35	583.42	0.73	OK
4	W14x159	0.34	462.60	572.13	0.70	OK
5	W14x159	3.80	461.90	574.74	0.89	OK
6	W14x82	22.97	223.50	389.45	1.00	OK
12	W14x82	50.18	141.33	215.20	0.73	OK
13	W14x159	12.17	305.99	339.07	0.40	OK
14	W14x159	15.36	320.23	355.28	0.41	OK
15	W14x159	2.76	330.77	352.94	0.40	OK
16	W14x159	9.74	331.93	361.15	0.52	OK
17	W14x82	33.77	154.76	221.82	0.59	OK
23	W14x82	47.43	84.03	136.78	0.36	OK
24	W14x82	8.56	124.33	140.38	0.47	OK
25	W14x82	10.49	125.41	142.98	0.45	OK
26	W14x82	1.07	127.26	135.41	0.43	OK
27	W14x82	4.29	133.44	145.15	0.46	OK
28	W14x82	34.28	95.31	135.34	0.32	OK
34	W14x82	37.39	91.55	130.62	0.27	OK
35	W14x82	6.89	135.97	146.81	0.31	OK
36	W14x82	7.79	135.00	146.27	0.31	OK
37	W14x82	1.98	135.08	140.36	0.30	OK
38	W14x82	4.03	142.36	150.31	0.32	OK
39	W14x82	27.00	102.57	131.46	0.27	OK

Moment Frame 1 Beam Design

Design Strength:

Mem No.	Section	Area	r_x	K_x	L	ϕP_n	ϕM_n
7	W27x94	27.7	10.9	1.00	40.00	1,022	1,040
8	W27x94	27.7	10.9	1.00	40.00	1,022	1,040
9	W27x94	27.7	10.9	1.00	33.33	1,067	1,040
10	W27x94	27.7	10.9	1.00	33.33	1,067	1,040
11	W27x94	27.7	10.9	1.00	33.33	1,067	1,040
18	W27x94	27.7	10.9	1.00	40.00	1,022	1,040
19	W27x94	27.7	10.9	1.00	40.00	1,022	1,040
20	W27x94	27.7	10.9	1.00	33.33	1,067	1,040
21	W27x94	27.7	10.9	1.00	33.33	1,067	1,040
22	W27x94	27.7	10.9	1.00	33.33	1,067	1,040
29	W27x94	27.7	10.9	1.00	40.00	1,022	1,040
30	W27x94	27.7	10.9	1.00	40.00	1,022	1,040
31	W27x94	27.7	10.9	1.00	33.33	1,067	1,040
32	W27x94	27.7	10.9	1.00	33.33	1,067	1,040
33	W27x94	27.7	10.9	1.00	33.33	1,067	1,040
40	W27x84	24.8	10.7	1.00	40.00	910	915
41	W27x84	24.8	10.7	1.00	40.00	910	915
42	W27x84	24.8	10.7	1.00	33.33	952	915
43	W27x84	24.8	10.7	1.00	33.33	952	915
44	W27x84	24.8	10.7	1.00	33.33	952	915

Moment Amplifier B1:

Mem No.	Section	P_e	C_m	P_u	B1
7	W27x94	4,088.52	1.00	13.40	1.01
8	W27x94	4,088.52	1.00	5.18	1.00
9	W27x94	5,347.76	1.00	4.36	1.00
10	W27x94	5,347.76	1.00	5.45	1.00
11	W27x94	5,347.76	1.00	16.45	1.01
18	W27x94	4,088.52	1.00	12.10	1.01
19	W27x94	4,088.52	1.00	9.19	1.01
20	W27x94	5,347.76	1.00	4.89	1.00
21	W27x94	5,347.76	1.00	5.67	1.00
22	W27x94	5,347.76	1.00	5.78	1.00
29	W27x94	4,088.52	1.00	15.45	1.01
30	W27x94	4,088.52	1.00	5.68	1.01
31	W27x94	5,347.76	1.00	4.08	1.00
32	W27x94	5,347.76	1.00	4.47	1.00
33	W27x94	5,347.76	1.00	4.89	1.00
40	W27x84	3,470.17	1.00	3.91	1.01
41	W27x84	3,470.77	1.00	7.43	1.01
42	W27x84	4,991.75	1.00	3.42	1.00
43	W27x84	4,991.75	1.00	6.82	1.00
44	W27x84	4,991.75	1.00	7.87	1.00

Beam Evaluation:

Mem No.	Section	M_{nt}	M_u	H1-1	Status
7	W27x94	427.22	431.49	0.42	OK
8	W27x94	404.50	408.55	0.39	OK
9	W27x94	398.24	394.30	0.38	OK
10	W27x94	351.40	354.91	0.34	OK
11	W27x94	356.97	360.54	0.35	OK
18	W27x94	343.42	346.85	0.33	OK
19	W27x94	215.63	217.79	0.21	OK
20	W27x94	270.37	270.37	0.26	OK
21	W27x94	295.19	295.19	0.28	OK
22	W27x94	196.36	196.36	0.19	OK
29	W27x94	246.82	246.82	0.24	OK
30	W27x94	178.72	178.72	0.17	OK
31	W27x94	225.32	225.32	0.22	OK
32	W27x94	209.29	209.29	0.20	OK
33	W27x94	205.21	205.21	0.20	OK
40	W27x84	120.98	120.98	0.13	OK
41	W27x84	106.98	106.98	0.12	OK
42	W27x84	89.47	89.47	0.08	OK
43	W27x84	88.99	88.99	0.10	OK
44	W27x84	61.47	61.47	0.07	OK

Final Member Selections Moment Frame 1

Element Number	Member Selection
1	W14 x 82
2	W14 x 159
3	W14 x 159
4	W14 x 159
5	W14 x 159
6	W14 x 159
7	W27 x 94
8	W27 x 94
9	W27 x 94
10	W27 x 94
11	W27 x 94
12	W14 x 82
13	W14 x 159
14	W14 x 159
15	W14 x 159
16	W14 x 150
17	W14 x 82
18	W27 x 94
19	W27 x 94
20	W27 x 94
21	W27 x 94
22	W27 x 94
23	W14 x 82
24	W14 x 82
25	W14 x 82
26	W14 x 82

Element Number	Member Selection
27	W14 x 82
28	W14 x 82
29	W27 x 94
30	W27 x 94
31	W27 x 94
32	W27 x 94
33	W27 x 94
34	W14 x 82
35	W14 x 82
36	W14 x 82
37	W14 x 82
38	W14 x 82
39	W14 x 82
40	W27 x 84
41	W27 x 84
42	W27 x 84
43	W27 x 84
44	W27 x 84

All members use A572 Gr 50 structural steel

Moment Frame 2 Column Design

Design Strength:

Mem No.	Sect.	Area	rx	ry	Kx	Ky	L	ϕPn	ϕMn
2	W14x159	46.7	6.38	4.00	1.90	1.00	16	1839	1076
3	W14x159	46.7	6.38	4.00	1.77	1.00	16	1898	1076
4	W14x159	46.7	6.38	4.00	1.77	1.00	16	1898	1076
5	W14x159	46.7	6.38	4.00	1.80	1.00	16	1884	1076
6	W14x159	46.7	6.38	4.00	1.80	1.00	16	1884	1076
7	W14x159	46.7	6.38	4.00	1.90	1.00	16	1839	1076
19	W14x159	46.7	6.38	4.00	1.45	1.00	14	2052	1076
20	W14x159	46.7	6.38	4.00	1.35	1.00	14	2052	1076
21	W14x159	46.7	6.38	4.00	1.35	1.00	14	2052	1076
22	W14x159	46.7	6.38	4.00	1.35	1.00	14	2052	1076
23	W14x159	46.7	6.38	4.00	1.35	1.00	14	2052	1076
24	W14x159	46.7	6.38	4.00	1.35	1.00	14	2052	1076
37	W14x82	24.1	6.05	2.48	1.45	1.00	14	862	521
38	W14x82	24.1	6.05	2.48	1.32	1.00	14	862	521
39	W14x82	24.1	6.05	2.48	1.30	1.00	14	862	521
40	W14x82	24.1	6.05	2.48	1.32	1.00	14	862	521
54	W14x82	24.1	6.05	2.48	1.32	1.00	14	862	521
55	W14x82	24.1	6.05	2.48	1.32	1.00	14	862	521
56	W14x82	24.1	6.05	2.48	1.12	1.00	14	862	521
57	W14x82	24.1	6.05	2.48	1.12	1.00	14	862	521

Moment Amplifier B1:

Mem No.	Section	P_e	C_m	P_u	B1
2	W14x159	14,758.88	0.33	723.15	1.00
3	W14x159	14,758.88	0.30	430.34	1.00
4	W14x159	14,758.88	0.31	402.91	1.00
5	W14x159	14,758.88	0.31	421.88	1.00
6	W14x159	14,758.88	0.30	741.52	1.00
7	W14x159	14,758.88	0.34	500.93	1.00
19	W14x159	19,276.91	0.25	478.70	1.00
20	W14x159	19,276.91	0.23	313.64	1.00
21	W14x159	19,276.91	0.23	284.32	1.00
22	W14x159	19,276.91	0.23	296.55	1.00
23	W14x159	19,276.91	0.23	449.38	1.00
24	W14x159	19,276.91	0.29	335.59	1.00
37	W14x82	8,945.55	0.22	197.10	1.00
38	W14x82	8,945.55	0.24	166.07	1.00
39	W14x82	8,945.55	0.23	170.71	1.00
40	W14x82	8,945.55	0.22	247.93	1.00
54	W14x82	8,945.55	0.30	70.90	1.00
55	W14x82	8,945.55	0.24	42.18	1.00
56	W14x82	8,945.55	0.24	42.49	1.00
57	W14x82	8,945.55	0.31	74.15	1.00

Moment Amplifier B2:

Mem No.	Section	Σ P_u	P_e	m	Σ P_e	B2
2	W14x159	7,437.00	4,088.34	6	24,530.03	1.44
3	W14x159	7,437.00	4,710.93	6	28,265.60	1.36
4	W14x159	7,437.00	4,710.93	6	28,265.60	1.36
5	W14x159	7,437.00	4,555.21	6	27,331.27	1.37
6	W14x159	7,437.00	4,555.21	6	27,331.27	1.37
7	W14x159	7,437.00	4,088.33	6	24,530.00	1.44
19	W14x159	5,270.00	9,168.57	6	55,011.40	1.11
20	W14x159	5,270.00	10,577.18	6	63,463.08	1.09
21	W14x159	5,270.00	10,577.18	6	63,463.08	1.09
22	W14x159	5,270.00	10,577.18	6	63,463.08	1.09
23	W14x159	5,270.00	10,577.18	6	63,463.08	1.09
24	W14x159	5,270.00	9,168.57	6	55,011.40	1.11
37	W14x82	3,103.00	5,134.04	4	20,536.15	1.18
38	W14x82	3,103.00	5,293.22	4	21,172.90	.17
39	W14x82	3,103.00	5,293.22	4	21,172.90	1.17
40	W14x82	3,103.00	5,134.04	4	20,536.15	1.18
54	W14x82	936.00	5,134.04	4	20,536.15	1.05
55	W14x82	936.00	5,134.04	4	20,536.15	1.05
56	W14x82	936.00	7,131.34	4	28,525.35	1.03
57	W14x82	936.00	7,131.34	4	28,525.35	1.03

Column Evaluation:

Mem No.	Section	M_{nt}	M_{lt}	M_u	H1-1	Status
2	W14x159	12.81	499.53	729.69	1.00	OK
3	W14x159	1.78	520.71	708.42	0.81	OK
4	W14x159	2.23	518.23	705.50	0.80	OK
5	W14x159	6.10	512.23	710.80	0.81	OK
6	W14x159	3.73	481.00	664.54	0.94	OK
7	W14x159	59.98	485.79	545.77	0.72	OK
19	W14x159	32.69	321.97	388.77	0.55	OK
20	W14x159	6.43	371.12	411.16	0.46	OK
21	W14x159	8.92	352.72	393.38	0.38	OK
22	W14x159	11.72	346.75	389.68	0.39	OK
23	W14x159	54.42	393.94	483.81	0.39	OK
24	W14x159	62.08	355.07	460.31	0.61	OK
37	W14x82	23.50	186.78	243.90	0.51	OK
38	W14x82	2.37	224.35	264.86	0.54	OK
39	W14x82	9.41	143.54	177.35	0.43	OK
40	W14x82	21.47	117.35	159.94	0.36	OK
54	W14x82	23.90	65.80	92.99	0.29	OK
55	W14x82	4.77	96.48	106.07	0.33	OK
56	W14x82	4.25	93.82	100.88	0.32	OK
57	W14x82	26.77	62.62	91.27	0.26	OK

Moment Frame 2 Beam Design

Design Strength:

Mem No.	Section	Area	r_x	K_x	L	ϕP_n	ϕM_n
10	W27x94	27.7	10.9	1.00	30.00	1,087	1,040
11	W27x94	27.7	10.9	1.00	30.00	1,087	1,040
12	W27x94	27.7	10.9	1.00	30.00	1,087	1,040
13	W27x94	27.7	10.9	1.00	30.00	1,087	1,040
14	W27x94	27.7	10.9	1.00	30.00	1,087	1,040
15	W27x94	27.7	10.9	1.00	30.00	1,087	1,040
16	W27x94	27.7	10.9	1.00	40.00	1,022	1,040
27	W27x94	27.7	10.9	1.00	30.00	1,087	1,040
28	W27x94	27.7	10.9	1.00	30.00	1,087	1,040
29	W27x94	27.7	10.9	1.00	30.00	1,087	1,040
30	W27x94	27.7	10.9	1.00	30.00	1,087	1,040
31	W27x94	27.7	10.9	1.00	30.00	1,087	1,040
32	W27x94	27.7	10.9	1.00	30.00	1,087	1,040
33	W27x94	27.7	10.9	1.00	40.00	1,022	1,040
46	W27x94	27.7	10.9	1.00	30.00	1,087	1,040
47	W27x94	27.7	10.9	1.00	30.00	1,087	1,040
48	W27x94	27.7	10.9	1.00	30.00	1,087	1,040
49	W27x94	27.7	10.9	1.00	30.00	1,087	1,040
63	W27x84	24.8	10.7	1.00	30.00	970	915
64	W27x84	24.8	10.7	1.00	30.00	970	915
65	W27x84	24.8	10.7	1.00	30.00	970	915
66	W27x84	24.8	10.7	1.00	30.00	970	915

Moment Amplifier B1:

Mem No.	Section	P_e	C_m	P_u	B1
10	W27x94	7,267.65	1.00	43.40	1.01
11	W27X94	7,267.65	1.00	47.32	1.01
12	W27x94	7,267.65	1.00	37.82	1.01
13	W27x94	7,267.65	1.00	18.71	1.01
14	W27x94	7,267.65	1.00	27.96	1.01
15	W27x94	7,267.65	1.00	43.98	1.01
16	W27x94	4,087.93	1.00	53.73	1.02
27	W27x94	7,267.65	1.00	42.45	1.01
28	W27x94	7,267.65	1.00	17.63	1.01
29	W27x94	7,267.65	1.00	18.89	1.01
30	W27x94	7,267.65	1.00	18.27	1.01
31	W27x94	7,267.65	1.00	11.85	1.00
32	W27x94	7,267.65	1.00	26.14	1.01
33	W27x94	4,087.93	1.00	10.05	1.00
46	W27x94	7,267.65	1.00	3.31	1.00
47	W27x94	7,267.65	1.00	5.56	1.00
48	W27x94	7,267.65	1.00	10.47	1.00
49	W27x94	7,267.65	1.00	5.95	1.00
63	W27x84	6,175.06	1.00	5.62	1.00
64	W27x84	6,175.06	1.00	8.48	1.00
65	W27x84	6,175.06	1.00	16.87	1.00
66	W27x84	6,175.06	1.00	15.24	1.00

Beam Evaluation:

Mem No.	Section	M_{nt}	M_u	H1-1	Status
10	W27x94	440.16	444.56	0.44	OK
11	W27x94	456.89	461.46	0.44	OK
12	W27x94	433.61	437.95	0.42	OK
13	W27x94	343.20	346.66	0.33	OK
14	W27x94	336.50	339.87	0.33	OK
15	W27x94	487.73	497.48	0.48	OK
16	W27x94	551.62	557.14	0.54	OK
27	W27x94	253.06	306.09	0.30	OK
28	W27x94	195.37	343.77	0.33	OK
29	W27x94	241.70	345.12	0.34	OK
30	W27x94	312.38	346.81	0.34	OK
31	W27x94	339.14	342.53	0.33	OK
32	W27x94	241.21	243.62	0.23	OK
33	W27x94	288.75	288.75	0.28	OK
46	W27x94	236.70	236.70	0.24	OK
47	W27x94	238.45	238.45	0.23	OK
48	W27x94	223.45	223.45	0.21	OK
49	W27x94	192.52	192.52	0.19	OK
63	W27x84	86.63	86.63	0.09	OK
64	W27x84	84.86	84.86	0.10	OK
65	W27x84	102.73	102.73	0.11	OK
66	W27x84	98.49	98.49	0.11	OK

Final Member Selections Moment Frame 2

Element Number	Member Selection
1	W14 x 82
2	W14 x 159
3	W14 x 159
4	W14 x 159
5	W14 x 159
6	W14 x 159
7	W14 x 159
8	W14 x 159
9	W10 x 49
10	W27 x 94
11	W27 x 94
12	W27 x 94
13	W27 x 94
14	W27 x 94
15	W27 x 94
16	W27 x 94
17	W24 x 103
18	W10 x 49
19	W14 x 159
20	W14 x 159
21	W14 x 159
22	W14 x 159
23	W14 x 159
24	W14 x 159
25	W14 x 159
26	W10 x 49

Element Number	Member Selection
27	W27 x 94
28	W27 x 94
29	W27 x 94
30	W27 x 94
31	W27 x 94
32	W27 x 94
33	W27 x 94
34	W24 x 103
35	W10 x 33
36	W14 x 43
37	W14 x 82
38	W14 x 82
39	W14 x 82
40	W14 x 82
41	W14 x 43
42	W14 x 82
43	W10x 33
44	W16 x 26
45	W16 x 26
46	W27 x 94
47	W27 x 94
48	W27 x 94
49	W27 x 94
50	W24 x 103
51	W24 x 103
52	W10 x 33
53	W14 x 43
54	W14 x 82

Element Number	Member Selection
55	W14 x 82
56	W14 x 82
57	W14 x 43
58	W14 x 82
59	W14 x 82
60	W10 x 33
61	W16 x 26
62	W16 x 26
63	W27 x 84
64	W27 x 84
65	W27 x 84
66	W27 x 84
67	W24 x 103
68	W24 x 103

All members use A572 Gr 50 structural steel

APPENDIX F

FULLY RESTRAINED CONNECTIONS (SEISMIC)

Appendix F

Connection Design Summary Moment Frame 1

Conn. Id.	Joint No.	Plates	Bolts	Stiffners Required
1	7R	5/16" x 3 1/2" x 21"	(7) 7/8" A325-SC	Yes
2	8R	5/16" x 3 1/2" x 21"	(7) 7/8" A325-SC	Yes
3	8L	5/16" x 3 1/2" x 21"	(7) 7/8" A325-SC	Yes
4	9R	5/16" x 3 1/2" x 24"	(8) 7/8" A325-SC	Yes
5	9L	5/16" x 3 1/2" x 21"	(7) 7/8" A325-SC	Yes
6	10R	5/16" x 3 1/2" x 24"	(8) 7/8" A325-SC	Yes
7	10L	5/16" x 3 1/2" x 24"	(8) 7/8" A325-SC	Yes
8	11R	5/16" x 3 1/2" x 24"	(8) 7/8" A325-SC	Yes
9	11L	5/16" x 3 1/2" x 24"	(8) 7/8" A325-Sc	Yes
10	12L	5/16" x 3 1/2" x 24"	(8) 7/8" A325-Sc	Yes
11	13L	5/16" x 3 1/2" x 21"	(7) 7/8" A325-SC	Yes
12	14R	5/16" x 3 1/2" x 21"	(7) 7/8" A325-SC	Yes
13	14L	5/16" x 3 1/2" x 21"	(7) 7/8" A325-SC	Yes
14	15R	5/16" x 3 1/2" x 24"	(8) 7/8" A325-SC	Yes
15	15L	5/16" x 3 1/2" x 21"	(7) 7/8" A325-SC	Yes
16	16R	5/16" x 3 1/2" x 24"	(7) 7/8" A325-SC	Yes

Conn. Id.	Joint No.	Plates	Bolts	Stiffners Required
17	16L	5/16" x 3 1/2" x 24"	(8) 7/8" A325-SC	Yes
18	17R	5/16" x 3 1/2" x 24"	(8) 7/8" A325-SC	Yes
19	17L	5/16" x 3 1/2" x 24"	(8) 7/8" A325-SC	Yes
20	18L	5/16" x 3 1/2" x 24"	(8) 7/8" A325-SC	Yes
21	19R	5/16" x 3 1/2" x 21"	(7) 7/8" A325-SC	Yes
22	20R	5/16" x 3 1/2" x 21"	(7) 7/8" A325-SC	Yes
23	20L	5/16" x 3 1/2" x 21"	(7) 7/8" A325-SC	Yes
24	21R	5/16" x 3 1/2" x 24"	(8) 7/8" A325-SC	Yes
25	21L	5/16" x 3 1/2" x 24"	(8) 7/8" A325-SC	Yes
26	22R	5/16" x 3 1/2" x 24"	(8) 7/8" A325-SC	Yes
27	22L	5/16" x 3 1/2" x 24"	(8) 7/8" A325-SC	Yes
28	23R	5/16" x 3 1/2" x 23"	(8) 7/8" A325-SC	Yes
29	23L	5/16" x 3 1/2" x 24"	(8) 7/8" A325-SC	Yes
30	24L	5/16" x 3 1/2" x 24"	(8) 7/8" A325-SC	Yes
31	25R	5/16" x 3 1/2" x 17 1/2"	(6) 3/4" A325-SC	Yes
32	26R	1/4" x 3 1/2" x 17 1/2"	(6) 3/4" A325N	Yes
33	26L	1/4" x 3 1/2" x 17 1/2"	(6) 3/4" A325N	Yes

Conn. Id.	Joint No.	Plates	Bolts	Stiffners Rquired
34	27R	1/4" x 3 1/2" x 20 1/2"	(7) 3/4" A325N	Yes
35	27L	1/4" x 3 1/2" x 17 1/2"	(6) 3/4" A325N	Yes
36	28R	1/4" x 3 1/2" x 20 1/2"	(7) 3/4" A325N	Yes

L - Left Joint

R - Right Joint

Column Stiffner Design Moment Frame 1

Design 1-

W27 x 94's to W14 x 159's Framed right and left side.

Use 2PL 7/16" x 5 1/2" x 12 5/8", top and bottom, A36 structural steel, 3/8" fillet weld to flange and 1/4" fillet weld to web.

Check of LRFD Specification

Local Buckling-

b/t = 5 1/2"/1 1/8" = 4.40 < $\lambda_r = 95/\sqrt{36}$ OK

Flange Force-

F = (1158.33 K-ft)(12"/')/26.92" = 516.35 K

Col. Local Flange Bending-

$\phi R_n = \phi 6.25 t_f^2 F_{yf}$

$= (0.90)(6.25)(1.19)^2(50)$

= 398.28 K < 516.35 K

Col. Local Web Yielding-

$\phi R_n = \phi(5k+N) F_{yw} t_w$

$= (1.0)(5(1\ 7/8)+1.19)(50)(1.19)$

= 376.97 K < 516.35 K

Col. Web Crippling-

$\phi R_n = \phi 135 t_w^2 [1 + 3(N/d)(t_w/t_f)^{1.5}] \sqrt{F_{yw} t_f / t_w}$

$= (.75)(135)(.75)^2[1+3(0.75/14.98)(.75/1.19)^{1.5}]$

$\sqrt{50(1.19/.75)}$

= 539.33 K > 516.35 K

Col. Web Buckling-

$$\phi R_n = \frac{\phi 4,100 t_w^3 \sqrt{F_{yw}}}{d_c}$$

$$= \frac{(0.90)(4100)(0.750)^3 \sqrt{50}}{11.23}$$

$= 960.73$ K > 516.35 K

Compression Design Force-

= 516.35 K - 376.97 K

= 139.38 K

Compression Yield Capacity-

= (2)(5.5"-0.75")(7/16")(36 ksi)

= 149.63 K > 139.38 K OK

Flange Weld Capacity-

ϕR_n = (5 1/2"-0.75")(4)(1.392K/")(6)

= 158.69 K > 139.38 K OK

Web Weld Capacity-

ϕR_n = (11 13/16"-0.75")(4)(1.393K/")(4)

= 246.38 K > 139.38 K OK

<u>Design 2-</u>

W27 x 94's to W14 x 82's Framed right and left side.

Use 2PL 1 1/4" x 4 1/2" x 12 5/8", top and bottom, A36 structural steel, 13/16" fillet weld to flange and 5/16" fillet weld to web.

Check of LRFD Specification

Local Buckling-

b/t = 4 1/2"/1 1/8" = 4.00 < $\lambda_r = 95/\sqrt{36}$ OK

Flange Force-

F = (1158.33 K-ft)(12"/')/26.92" = 516.35 K

Col. Local Flange Bending-

$\phi R_n = \phi 6.25 t_f^2 F_{yf}$

$= (0.90)(6.25)(0.855)^2(50)$

= 205.61 K < 516.35 K

Col. Local Web Yielding-

$\phi R_n = \phi(5k+N)F_{yw}t_w$

= (1.0)(5(1 5/8)+0/745)(50)(0.510)

= 226.19 K < 516.35 K

Col. Web Crippling-

$\phi R_n = \phi 135 t_w^2 [1 + 3(N/d)(t_w/t_f)^{1.5}] \sqrt{F_{yw} t_f / t_w}$

$= (.75)(135)(.51)^2[1+3(0.75/14.31)(.51/0.745)^{1.5}]$

$\sqrt{50(.745/.510)}$

= 258.46 K < 516.35 K

Col. Web Buckling-

$$\phi R_n = \frac{\phi 4,100 t_w^3 \sqrt{F_{yw}}}{d_c}$$

$$= \frac{(0.90)(4100)(0.510)^3\sqrt{50}}{11.06}$$

= 319.94K < 516.35 K

Compression Design Force-

$= 516.35\ K - 226.19\ K$

$= 290.16\ K$

Compression Yield Capacity-

$= (2)(4.5"-0.75")(36\ ksi)(1\ 1/4")$

$= 303.75\ K > 290.16\ K$ OK

Axial Capacity-

$A = 1.25"(4.5")(2) + 25(0.510)^2$

$= 17.75\ in^2$

$$I = \frac{(4.5" + 4.5" + 0.510)^3(1.25")}{12}$$

$= 89.73\ in^4$

$r = \sqrt{89.73/17.75}$

$= 2.25"$

$kl/r = 0.75(14.31"-2(.855"))/2.25'$

$= 4.20$

$\phi F_{cr} = 30.56\ ksi$

$\phi Pn = (17.75)(30.56)$

$= 542.44\ K > 290.16\ K$ OK

Tension Capacity (Plate)-

$\phi R_n = (0.90)(36)(4.5-0.75)(1.25)(2)$

$= 303.75\ K$

Tension Force-

$= 516.35\ K - 205.61\ K$

$= 310.74\ K > 303.75\ K$ NG

revise to 2PL 1 3/8" x 4 1/2" x 12 5/8"

Web Weld Capacity-

ϕR_n = (12 5/8"-0.75")(4)(1.393K/")(5)

= 330.60 K > 90.16 K OK

<u>Design 3-</u>

W27 x 84's to W14 x 82's Framed right and left side.

Use 2PL 1 1/8" x 4 1/2" x 12 5/8", top and bottom, A36 structural steel, 13/16" fillet weld to flange and 5/16" fillet weld to web.

Check of LRFD Specification

Local Buckling-

b/t = 4 1/2"/1 " = 4.50 < λ_r = 95/ $\sqrt{36}$ OK

Flange Force-

F = (1017.17 K-ft)(12"/')/26.71" = 456.76 K

Col. Local Flange Bending-

$\phi R_n = \phi 6.25 t_f^2 F_{yf}$

= (0.90)(6.25)(0.855)^2(50)

= 205.61 K < 456.76 K

Col. Local Web Yielding-

$\phi R_n = \phi (5k+N) F_{yw} t_w$

= (1.0)(5(1 5/8)+0/745)(50)(0.510)

= 226.19 K < 456.76 K

Col. Web Crippling-

$\phi R_n = \phi 135 t_w^2 [1 + 3(N/d)(t_w/t_f)^{1.5}] \sqrt{F_{yw} t_f / t_w}$

=(.75)(135)(.51)^2[1+3(0.75/14.31)(.51/0.745)$^{1.5}$]

$\sqrt{50(.745/.510)}$

$= 258.46 \text{ K} < 456.76 \text{ K}$

Col. Web Buckling-

$$\phi R_n = \frac{\phi 4{,}100 t_w^3 \sqrt{F_{yw}}}{d_c}$$

$$= \frac{(0.90)(4100)(0.510)^3 \sqrt{50}}{11.06}$$

$= 319.94\text{K} < 456.76 \text{ K}$

Compression Design Force-

$= 456.76 \text{ K} - 226.19 \text{ K}$

$= 230.57 \text{ K}$

Compression Yield Capacity-

$= (2)(4.5"-0.75")(36 \text{ ksi})(1\ 1/8")$

$= 273.38 \text{ K} > 230.57 \text{ K}$ OK

Axial Capacity-

$A = 1.125"(4.5")(2) + 25(0.510)^2$

$= 16.63 \text{ in}^2$

$$I = \frac{(4.5" + 4.5" + 0.510)^3 (1.125")}{12}$$

$= 80.77 \text{ in}^4$

$r = \sqrt{80.77/16.63} = 2.20$

$kl/r = 0.75(14.31"-2(.855"))/2.20"$

$= 4.30$

$\phi F_{cr} = 30.56 \text{ ksi}$

$\phi Pn = (16.63)(30.56)$

$= 508.21 \text{ K} > 230.57 \text{ K}$ OK

Tension Capacity (Plate)-

ϕR_n = (0.90)(36)(4.5-0.75)(1.125)(2)

= 273.38 K

Tension Force-

= 456.76 K - 205.61 K

= 251.16 K < 273.38 OK

<u>Design 4-</u>

W27 x 94's to W14 x 82's Framed one side only.

Use 2PL 1 3/8" x 4 1/2" x 11 13/16", top and bottom, A36 structural steel, 13/16" fillet weld to flange and 5/16" fillet weld to web.

Check of LRFD Specification

Local Buckling-

b/t = 4 1/2"/1 1/8" = 4.00 < λ_r = 95/ $\sqrt{36}$ OK

Flange Force-

F = (1158.33 K-ft)(12"/')/26.92" = 516.35 K

Col. Local Flange Bending-

$\phi R_n = \phi 6.25 t_f^2 F_{yf}$

= $(0.90)(6.25)(0.855)^2(50)$

= 205.61 K < 516.35 K

Col. Local Web Yielding-

$\phi R_n = \phi(5k+N)F_{yw}t_w$

= (1.0)(5(1 5/8)+0/745)(50)(0.510)

= 226.19 K < 516.35 K

Col. Web Crippling-

$\phi R_n = \phi 135 t_w^2 [1 + 3(N/d)(t_w/t_f)^{1.5}] \sqrt{F_{yw} t_f / t_w}$

$= (.75)(135)(.51)^2 [1+3(0.75/14.31)(.51/0.745)^{1.5}]$

$\sqrt{50(.745/.510)}$

$= 258.46 \text{ K} < 516.35 \text{ K}$

Col. Web Buckling-

$$\phi R_n = \frac{\phi 4,100 t_w^3 \sqrt{F_{yw}}}{d_c}$$

$$= \frac{(0.90)(4100)(0.510)^3 \sqrt{50}}{11.06}$$

$= 319.94\text{K} < 516.35 \text{ K}$

Compression Design Force-

$= 516.35 \text{ K} - 226.19 \text{ K}$

$= 290.16 \text{ K}$

Compression Yield Capacity-

$= (2)(4.5"-0.75")(36 \text{ ksi})(1\ 3/8")$

$= 334.13 \text{ K} > 290.16 \text{ K}$ OK

Axial Capacity-

$A = 1.38"(4.5")(2) + 25(0.510)^2$

$= 18.92 \text{ in}^2$

$$I = \frac{(4.5" + 4.5" + 0.510)^3 (1.38")}{12}$$

$= 98.70 \text{ in}^4$

$r = \sqrt{98.70/18.92}$

$= 2.28$

$kl/r = 0.75(14.31"-2(.855"))/2.28"$

= 4.14

ϕF_{cr} = 30.56 ksi

ϕPn = (18.92)(30.56)

= 578.20 K > 290.16 K OK

Tension Capacity (Plate)-

ϕR_n = (0.90)(36)(4.5-0.75)(1.38)(2)

= 334.13.75 K

Tension Force-

= 516.35 K - 205.61 K

= 310.74 K < 334.13K OK

Web Weld Capacity-

ϕR_n = (11 13/16"-0.75")(4)(1.393K/")(5)

= 307.98 K > 290.16 K OK

<u>Design 4-</u>

W27 x 94's to W14 x 82's Framed one side only.

Use 2PL 1 3/8" x 4 1/2" x 11 13/16", top and bottom, A36 structural steel, 13/16" fillet weld to flange and 5/16" fillet weld to web.

Check of LRFD Specification

Local Buckling-

b/t = 4 1/2"/1 1/8" = 3.60 < λ_r = 95/ $\sqrt{36}$ OK

Flange Force-

F = (1017.17 K-ft)(12"/')/26.92" = 456.76 K

Col. Local Flange Bending-

$\phi R_n = \phi 6.25 t_f^2 F_{yf}$

$= (0.90)(6.25)(0.855)^2(50)$

$= 205.61$ K < 456.76 K

Col. Local Web Yielding-

$\phi R_n = \phi(5k+N)F_{yw}t_w$

$= (1.0)(5(1\ 5/8)+0/745)(50)(0.510)$

$= 226.19$ K < 456.76 K

Col. Web Crippling-

$\phi R_n = \phi 135 t_w^2[1 + 3(N/d)(t_w/t_f)^{1.5}]\sqrt{F_{yw}t_f/t_w}$

$= (.75)(135)(.51)^2[1+3(0.75/14.31)(.51/0.745)^{1.5}]$

$\sqrt{50(.745/.510)}$

$= 258.46$ K < 456.76 K

Col. Web Buckling-

$$\phi R_n = \frac{\phi 4,100 t_w^3 \sqrt{F_{yw}}}{d_c}$$

$$= \frac{(0.90)(4100)(0.510)^3\sqrt{50}}{11.06}$$

$= 319.94$K < 456.76 K

Compression Design Force-

$= 456.76$ K $- 226.19$ K

$= 230.57$ K

Compression Yield Capacity-

$= (2)(4.5"-0.75")(36$ ksi$)(1\ 1/8")$

$= 303.75$ K > 230.57 K OK

Axial Capacity-

$A = 1.125''(4.5'')(2) + 25(0.510)^2$

$= 16.63\ in^2$

$I = \dfrac{(4.5'' + 4.5'' + 0.510)^3(1.125'')}{12}$

$= 80.46\ in^4$

$r = \sqrt{80.46/16.63}$

$= 2.20$

$kl/r = 0.75(14.31''-2(.855''))/2.20''$

$= 4.30$

$\phi F_{cr} = 30.56$ ksi

$\phi Pn = (16.63)(30.56)$

$= 508.21\ K > 230.57\ K$ OK

Tension Capacity (Plate)-

$\phi R_n = (0.90)(36)(4.5-0.75)(1.125)(2)$

$= 273.38\ K$

Tension Force-

$= 456.76K - 205.61\ K$

$= 251.15\ K < 273.38K$ OK

Web Weld Capacity-

$\phi R_n = (11\ 13/16''-0.75'')(4)(1.393K/'')(5)$

$= 307.60\ K > 251.15\ K$ OK

Connection Design Summary Moment Frame 2

Conn. Id.	Joint No.	Plates	Bolts	Stiffners Required
1	11R	3/8" x 4" x 21 1/2"	(7) 1" A325-SC	Yes
2	11L	3/8" x 4" x 21 1/2"	(7) 1" A325-SC	Yes
3	12R	1/2" x 4 1/2" x 19"	(6) 1 1/8" A325-SC	Yes
4	12L	1/2" x 4 1/2" x 19"	(6) 1 1/8" A325-SC	Yes
5	13R	1/2" x 4 1/2" x 19"	(6) 1 1/8" A325-SC	Yes
6	13L	1/2" x 4 1/2" x 19"	(6) 1 1/8" A325-SC	Yes
7	14R	1/2" x 4 1/2" x 19"	(6) 1 1/8" A325-SC	Yes
8	14L	1/2" x 4 1/2" x 19"	(6) 1 1/8" A325-SC	Yes
9	15R	3/8" x 4" x 21 1/2"	(7) 1" A325-SC	Yes
10	15L	1/2" x 4 1/2" x 19"	(6) 1 1/8" A325-SC	Yes
11	16R	3/4" x 5 1/2" x 21 1/2"	(5) 1 1/2" A325-SC	Yes
12	16L	7/16" x 4 1/2" x 22"	(7) 1 1/8" A325-SC	Yes
13	20R	3/8" x 4" x 21 1/2"	(7) 1" A325-SC	Yes
14	20L	3/8" x 4" x 21 1/2"	(7) 1" A325-SC	Yes
15	21R	3/8" x 4" x 21 1/2"	(7) 1" A325-SC	Yes
16	21L	3/8" x 4" x 21 1/2"	(7) 1" A325-SC	Yes

Conn. Id	Joint No.	Plates	Bolts	Stiffners Required
17	22R	1/2" x 4 1/2" x 19"	(6) 1 1/8" A325-SC	Yes
18	22L	1/2" x 4 1/2" x 19"	(6) 1 1/8" A325-SC	Yes
19	23R	1/2" x 4 1/2" x 19"	(6) 1 1/8" A325-SC	Yes
20	23L	1/2" x 4 1/2" x 19"	(6) 1 1/8" A325-SC	Yes
21	24R	3/8" x 4" x 21 1/2"	(7) 1" A325-SC	Yes
22	24L	1/2" x 4 1/2" x 19"	(6) 1 1/8" A325-SC	Yes
23	25R	5/8" x 5 1/2" x 21 1/2"	(5) 1 1/2" A325-SC	Yes
24	25L	7/16" x 4 1/2" x 22"	(7) 1 1/8" A325-SC	Yes
25	30R	7/16" x 4" x 21 1/2"	(7) 1" A325-SC	Yes
26	31R	3/8" x 4" x 21 1/2"	(7) 1" A325-SC	Yes
27	31L	3/8" x 4" x 21 1/2"	(7) 1" A325-SC	Yes
28	32R	3/8" x 4" x 21 1/2"	(7) 1" A325-SC	Yes
29	32L	3/8" x 4" x 21 1/2"	(7) 1" A325-SC	Yes
30	33L	3/8" x 4" x 21 1/2"	(7) 1" A325-SC	Yes
31	39R	1/4" x 3 1/2" x 23 1/3"	(8) 3/4" A325-N	Yes
32	40R	3/8" x 3 1/2" x 18"	(6) 7/8" A325N	Yes

Conn. Id.	Joint No.	Plates	Bolts	Stiffners Required
33	40L	3/8" x 3 1/2" x 18"	(6) 7/8" A325N	Yes
34	41R	5/16" x 3 1/2" x 21"	(7) 7/8" A325N	Yes
35	41L	5/16" x 3 1/2" x 21"	(7) 7/8" A325N	Yes
36	42L	5/16" x 3 1/2" x 21"	(7) 7/8" A325N	Yes

L - Left Joint

R- Right Joint

Column Stiffner Design Moment Frame 2

Since the same sections as in Moment Frame 1 are used for Moment Frame 2 the same stiffners will be used as summarized below.

Design 1-

W27 x 94's to W14 x 159's Framed right and left side.

Use 2PL 7/16" x 5 1/2" x 12 5/8", top and bottom, A36 structural steel, 3/8" fillet weld to flange and 1/4" fillet weld to web.

Design 2-

W27 x 94's to W14 x 82's Framed right and left side.

Use 2PL 1 1/4" x 4 1/2" x 12 5/8", top and bottom, A36 structural steel, 13/16" fillet weld to flange and 5/16" fillet weld to web.

Design 3-

W27 x 84's to W14 x 82's Framed right and left side.

Use 2PL 1 1/8" x 4 1/2" x 12 5/8", top and bottom, A36 structural steel, 13/16" fillet weld to flange and 5/16" fillet weld to web.

Design 4-

W27 x 94's to W14 x 82's Framed one side only.

Use 2PL 1 3/8" x 4 1/2" x 11 13/16", top and bottom, A36 structural steel, 13/16" fillet weld to flange and 5/16" fillet weld to web.

Design 4-

W27 x 94's to W14 x 82's Framed one side only.

Use 2PL 1 3/8" x 4 1/2" x 11 13/16", top and bottom, A36 structural steel, 13/16" fillet weld to flange and 5/16" fillet weld to web.

APPENDIX G

NON-SEISMIC AND SEISMIC DESIGN COMPARISONS

Appendix G

Page

Connection Weights Moment Frame 1

Conn. Id.	Joint No.	Non-Seismic (Lbs)	Seismic (lbs)
1	7R	36.7	40.4
2	8R	2.7	23.7
3	8L	2.7	23.7
4	9R	2.7	24.7
5	9L	2.7	23.7
6	10R	2.7	24.7
7	10L	2.7	24.7
8	11R	2.7	24.7
9	11L	2.7	24.7
10	12L	2.7	24.7
11	13R	2.7	24.7
12	14R	2.7	40.4
13	14L	2.7	23.7
14	15R	2.7	24.7
15	15L	2.7	23.7
16	16R	2.7	24.7
17	16L	2.7	24.7
18	17R	2.7	24.7
19	17L	2.7	24.7
20	18L	2.7	43.7
21	19R	2.7	40.4
22	20R	2.7	42.8
23	20L	2.7	42.8
24	21R	2.7	43.8
25	21L	2.7	43.8
26	22R	2.7	43.8

Conn. Id.	Joint No.	Non Seismic (Lbs)	Seismic (Lbs)
27	22L	2.7	43.8
28	23R	2.7	43.8
29	23L	2.7	43.8
30	24L	2.7	43.8
31	26R	2.7	30.0
32	26L	2.7	47.5
33	27R	2.7	47.5
34	27L	2.7	48.5
35	28R	2.7	47.5
36	28L	2.7	31.5

Total Connection Weights -

Non-Seismic: 131. Lbs.

Seismic: 1223. Lbs.

Connection Weights Moment Frame 2

Conn. Id.	Joint No.	Non-Seismic (Lbs)	Seismic (Lbs)
1	11R	20.8	26.4
2	11L	2.7	26.4
3	12R	2.7	32.1
4	12L	2.7	32.1
5	13R	19.9	32.1
6	13L	19.9	32.1
7	14R	19.9	32.1
8	14L	2.7	32.1
9	15R	19.9	32.3
10	15L	2.7	32.1
11	16R	25.0	25.2
12	16L	26.0	29.4
13	20R	2.7	26.4
14	20L	2.7	26.4
15	21R	2.7	26.4
16	21L	2.7	26.4
17	22R	2.7	29.4
18	22L	2.7	29.4
19	23R	2.7	29.4
20	23L	2.7	29.4
21	24R	2.7	26.4
22	24L	2.7	29.4
23	25R	2.7	38.2
24	25L	2.7	29.5
25	30R	2.7	27.9
26	31R	2.7	26.4

Conn. Id.	Joint No.	Non Seismic (Lbs)	Seismic (Lbs)
27	31L	2.7	26.4
28	32R	2.7	26.4
29	32L	2.7	53.4
30	33L	2.7	53.4
31	39R	2.7	50.1
32	40R	2.7	51.0
33	40L	2.7	51.0
34	41R	2.7	50.8
35	41L	2.7	50.8
36	42L	2.7	50.8

Total Connection Weights -

Non-Seismic: 227. Lbs.

Seismic: 1177. Lbs.

Number of A325 Bolts Moment Frame 1

Conn. Id	Joint No.	Non-Seismic	Seismic
1	7R	5	7
2	8R	5	7
3	8L	5	7
4	9R	5	8
5	9L	5	7
6	10R	5	8
7	10L	5	8
8	11R	5	8
9	11L	5	8
10	12L	5	8
11	13R	5	7
12	14R	5	7
13	14L	5	7
14	15R	5	8
15	15L	5	7
16	16R	5	7
17	16L	5	8
18	17R	5	8
19	17L	5	8
20	18L	5	8
21	19R	5	7
22	20R	5	7
23	20L	5	7
24	21R	5	8
25	21L	5	8
26	22R	5	8

Conn. Id	Joint No.	Non Seismic	Seismic
27	22L	5	8
28	23R	5	8
29	23L	5	8
30	24L	5	8
31	26R	5	6
32	26L	5	6
33	27R	5	6
34	27L	5	7
35	28R	5	6
36	28L	5	7

Total Number of Bolts -

Non-Seismic: 180

Seismic: 266

Number of A325 Bolts Moment Frame 2

Conn. Id.	Joint No.	Non-Seismic	Seismic
1	11R	5	7
2	11L	5	7
3	12R	5	6
4	12L	5	6
5	13R	5	6
6	13L	5	6
7	14R	5	6
8	14L	5	6
9	15R	5	7
10	15L	5	6
11	16R	7	5
12	16L	5	7
13	20R	5	7
14	20L	5	7
15	21R	5	7
16	21L	5	7
17	22R	5	6
18	22L	5	6
19	23R	5	6
20	23L	5	6
21	24L	5	7
22	25L	5	6
23	30R	7	5
24	31R	6	7
25	31L	5	7
26	31R	5	7

Conn. Id.	Joint No.	Non Seismic	Seismic
27	31L	5	7
28	32R	5	7
29	32L	5	7
30	39R	5	7
31	40R	5	8
32	40L	5	6
33	41R	5	6
34	41R	5	7
35	41L	5	7
36	42L	5	7

Total Number of Bolts -

Non-Seismic: 185

Seismic: 235

Member Weights

Braced Frame 1:

Story Level	Non-Seismic	Seismic
4	873 Lbs.	6594 Lbs.
3	873 Lbs.	6594 Lbs.
2	1213 Lbs.	6594 Lbs.
1	1723 Lbs.	6594 Lbs.

Total Member Weights-

Non-Seismic: 4,682 Lbs.

Seismic: 23,376 Lbs

Braced Frame 2:

Story Level	Non-Seismic	Seismic
4	873 Lbs.	6594 Lbs.
3	873 Lbs.	6594 Lbs.
2	1695 Lbs.	7628 Lbs.
1	1723 Lbs.	9805 Lbs.

Total Member Weights-

Non-Seismic: 5,164 Lbs.

Seismic: 30,621 Lbs.

Braced Frame 3:

Story Level	Non-Seismic	Seismic
4	873 Lbs.	6594 Lbs.
3	873 Lbs.	6594 Lbs.
2	1695 Lbs.	8646 Lbs.
1	2033 Lbs.	9805 Lbs.

Total Member Weights-

Non-Seismic: 5,474 Lbs.

Seismic: 31,639 Lbs.

VITA

David N. Martin was born on April 20, 1962 in Biddeford Maine. He received his Bachelor of Science degree in Civil Engineering from the University of Maine in May of 1984. After graduation he was employed by the New York District, U.S. Army Corps of Engineers until June of 1992. While with the Corps of Engineers he served as a Project Engineer and Resident Engineer for Military construction projects in Maine, Connecticut, and Massachusetts. He is a registered Professional Engineer in the State of Maine. In August of 1992 he enrolled in the Structures Division at Virginia Tech.

David N Martin