FDA OC/ORA Office Building

Silver Spring, MD

Final Report

Adam Love

Structural Option

AE Consultant: Dr. Hanagan

April 7th, 2010

FDA OC/ ORA Office Building

Silver Spring, MD

Adam Love
Structural Option

www.engr.psu.edu/ae/thesis/portfolios/2010/ael5029/index

<u>Architecture</u>:

- Mixed use office building that forms the final phase of the consolidation efforts of the FDA.
- The 500,000 S.F. Office Building was laid out to morrior the existing buildings on the site. The building is broken up into 4 wings, A through D.

<u>Structural</u>:

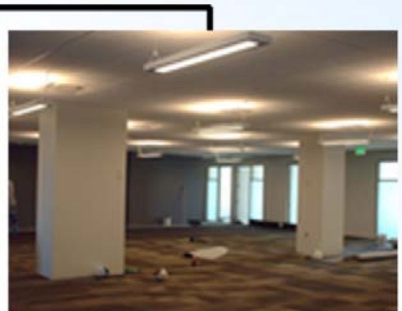
- Spread Footings were used for the building's foundation. Where 95% compaction could not be met; Geopiers were used under the footings.
- The structure of the building was designed to prevent progressive collapse. The exterior beams of each floor are the primary elements in the progressive collapse design.

<u>Building Statistics</u>:
Size: 500,000 S.F.
Construction Dates: 6/07 to 12/10
Final Contract Cost: $110 Million
Delivery Method: Lump Sum Project

<u>Mechanical Systems</u>:

- 4 AHU's are provied on the roof with VAV boxes throughout the building.
- The AHU range rom 20,000 to 30,000 L/s.

<u>Project Team</u>:
Owner:
GSA
Occupant:
FDA
General Contractor:
Tompkins Builders
Construction Manager:
Heery-Tishman
Architectural Engineer:
KlingStubbins in association with RTKL
Geotechnical Engineer:
Schnabel Engineering Association



<u>Lighting / Electrical Systems</u>:

- Daylighting controls are provided with the florescent lighting system.
- 13.8 kV is supplied from the Central Utility Plant.
- A Unit Substation in Building 31 supplies the swtchgear and then supplies 208Y/120 V to each wing.

Table of Contents

Executive Summary

A building design is very integrated; every change to a building has some effect on other functions of the building. A change to the structure can affect cost, schedule, serviceability, MEP Coordination, and Architecture. It is the purpose of this thesis to investigate some of the affects of a change to the building, while exploring more advanced structure design topics. It is proposed to change the superstructure of the FDA OC/ORA Office Building Wing B from concrete to steel.

Using the existing grid and column lay out with a few minor changes, the steel framing was determined, and the gravity system was designed. Braced frames around the core were designed to take the primary lateral load and to limit the deflection of the wing. Exterior moment frames were designed to aid in the resistance of progressive collapse and reduce the eccentric effects of the wing.

To show the constructability of the new structure typical connections were designed for both the gravity and lateral system. A typical beam to girder connection and a girder to column connection were designed for the gravity system. Typical lateral frame connections were designed as a flange bolted moment connection, and a heavy braced connection.

As the threat for unforeseeable events increase, the design of structures to resist progressive collapse is becoming more important. The original structure was designed to resist progressive collapse, and part of this thesis was devoted to the research of the design of structures to resist progressive collapse. Two methods exist for the design to resist collapse, and both methods are presented in this thesis. To further understand the design progressive collapse a method will be chosen, and the design of removal of a column will be implemented.

Impacts on the cost and schedule were studied and compared to the original design and the new structure design. Both cost and schedules were created using RS Means Building Construction Data 2007.

The mechanical system was studied in respects to the ceiling to floor space where the mechanical ductwork passed through, the new structure design limited the space for the mechanical ducts and at a critical location, and two of the ducts will be resized. The ducts were resized using the existing airflow through the system, and were sized to fit in the new adjusted space.

Introduction

Starting the fifth phase of the consolidation efforts by the FDA, the OC/ ORA Office building plans to move the Office of Commissioner (OC), Office of Regulatory Affairs (ORA) Office building to the White Oak Campus. On the site of the former US Navy facility at the Federal Research Center- Naval Ordnance Laboratory, the OC/ ORA Office Building sits on the southern end, and forms its shape around the existing buildings.

Forming an S shaped building, the 500,000 S.F. office building was laid out and designed to mirror the existing buildings on the site and to form a unique face of the campus from the main drive off of New Hampshire Ave. Broken up into two buildings with four wings, Building 31 is comprised of Wing A, and Building 32 is comprised of wings B through D (Figure 1).

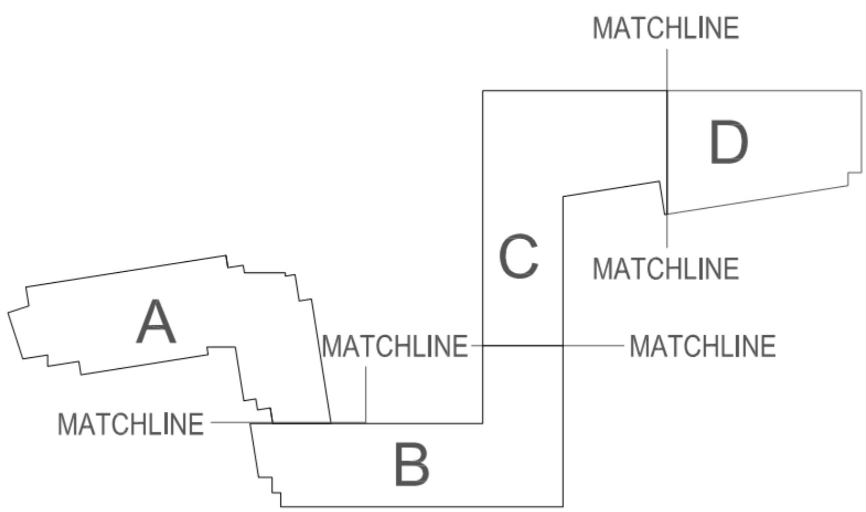


Figure 1: Key Plan

Structural System

Foundation:

The foundation of the building is separated into two categories. Spread footings that bear on undisturbed soil or spread footings that sit on a number of Geopiers. Schnabel Engineering conducted soil test to determine the bearing capacities of the soils. Where 95% compaction could not be met the use of Geopiers or vibropiers was recommended.

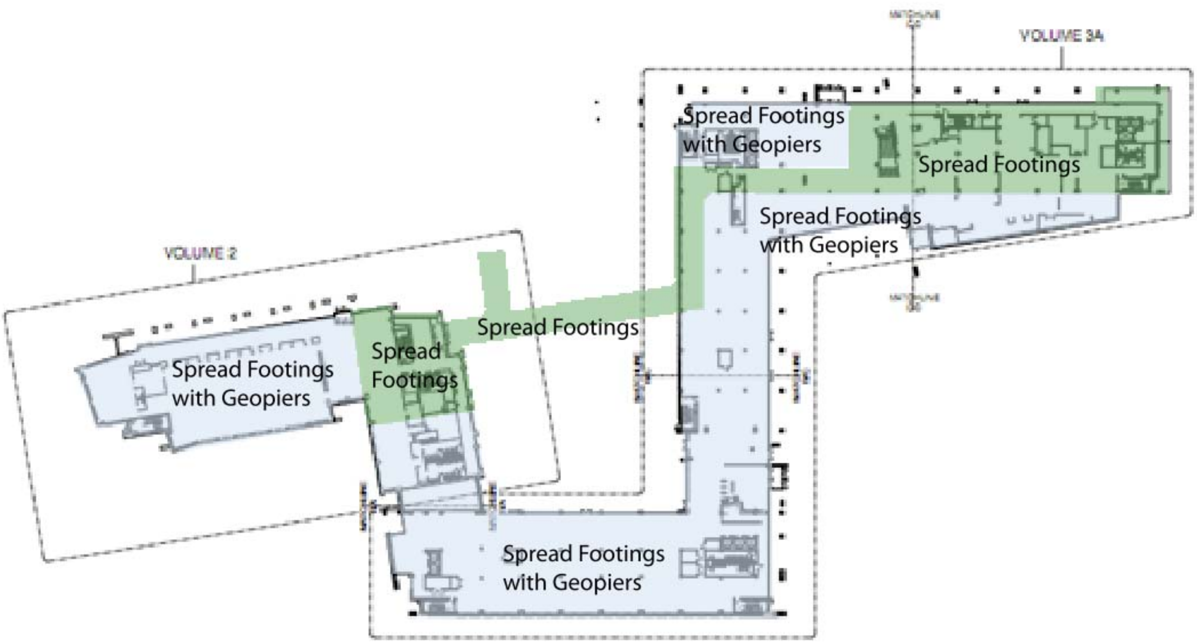


Figure 2: Foundation Key

For non-basement areas of Building 31 (Wing A), the western and central wings (Wings B and C) of Building 32, and the non-basement areas of Wing D, deep existing fill is expected within the majority of the buildings footprint. Geopiers are to be used in these areas to provide adequate bearing capacity (Figure 2). Geopiers use the concept of over consolidation to increase the soils bearing capacity. The 30 inch diameter Geopeirs should reach a depth of at least 10 feet. A detail of the typical spread footing with Geopiers is shown in Figure 3.

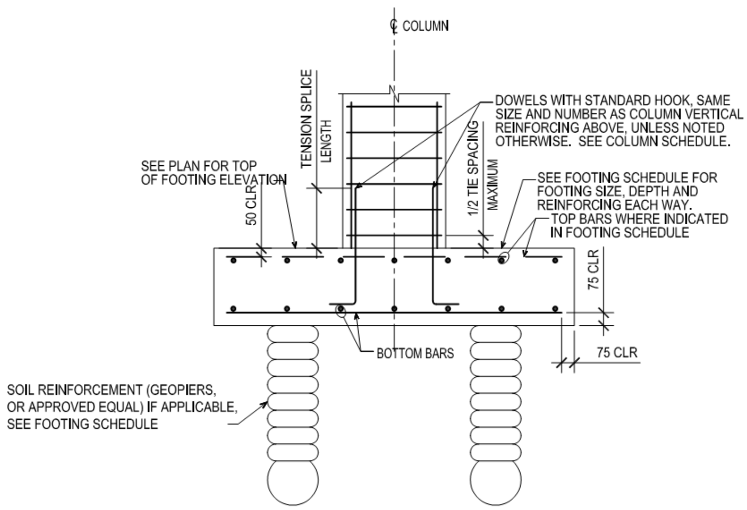


Figure 3: Typical Geopier Foundation Detail

For the basement level of Building 31 (Wing A), the basement level of Wing D of Building 32, and the underground tunnels, the foundations reach a sufficient depth where the bearing capacities on the spread footings are adequate (Figure 2).

Normal weight concrete was designed to be used with all the spread footings of the foundations. With a unit weight of 2350 kg/m^3 (147 pcf), the concrete has a 28 day strength of 28 MPa (4061 psi) concrete. A water to cement ratio of .48 is specified along with only 1% maximum chloride content.

Schnabel Engineering recommended the use minimum safe bearing capacities at the different locations of the foundation system. Where spread footings bear on undisturbed soil a bearing capacity of 192 kPa (4010 psf) was estimated. Beneath the spread footings of Wing A, where Geopiers were used, the estimated bearing capacity is 192 kPa (4010 psf). In the sections of Building 32 where Geopiers were used, a bearing capacity of 287 kPa (5994 psf) was estimated.

Floor System:

Building 31:

Building 31 utilizes a one way slab floor system for the majority of the buildings layout. The typical one way slab construction is an 8.07 inch thick slab with 5.91 inch drop panels, unless noted differently on the drawings. On the first three floors of Wing A there is a large open assembly space, and prevents any typical bay spacing. However, on the fourth floor the typical bay spacing is 21.85' x 26.74' to 19.685' x 19.685'.

Resistance to progressive collapse was designed into the exterior reinforced beams of building 31. Typical progressive collapse beam sizes range from 23.62" x 42.32" to 18.11" x 35.43". The interior beams on Building 31 are reinforced concrete beams with typical sizes of 18.11" x 35.43" to 18.11" x 23.62".

A large assembly pace on the first floor of Wing A is open up through the third floor. On the fourth floor framing level, post tension transfer girders were designed to support the column loads above the fourth floor and transfer the load to the foundation (Figure 4). The post tension transfer girders are 35.43" x 70.89" and have a post tension strand force of 4540 kN.

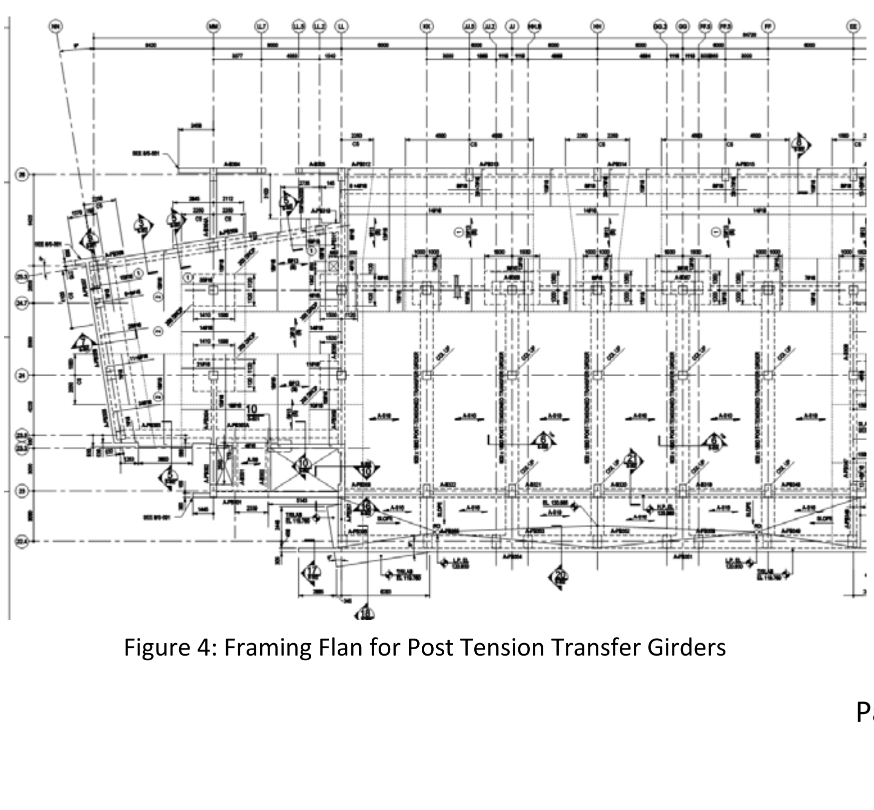


Figure 4: Framing Flan for Post Tension Transfer Girders

An atrium is provided between Wing A and Wing B that is primarily a steel superstructure with lightweight concrete on metal deck (Figure 5). The walkways over the atrium connecting the two wings are cast in place lightweight concrete on steel metal deck. The rib height on the metal deck is 50 mm with an additional 83 mm of concrete above. Supporting the walkway is W360 x 32.9 steel beams that frame into W360 x 32.9 girders with a shear connection. On the Wing A side of the atrium the girders site on an L152x152x9.5 that is attached to the concrete beam in Wing A. On the Wing B side on the atrium, an expansion joint is place, so the girders rest on a sliding connection that is connected to a beam in Wing B (Figure 6 and 7).

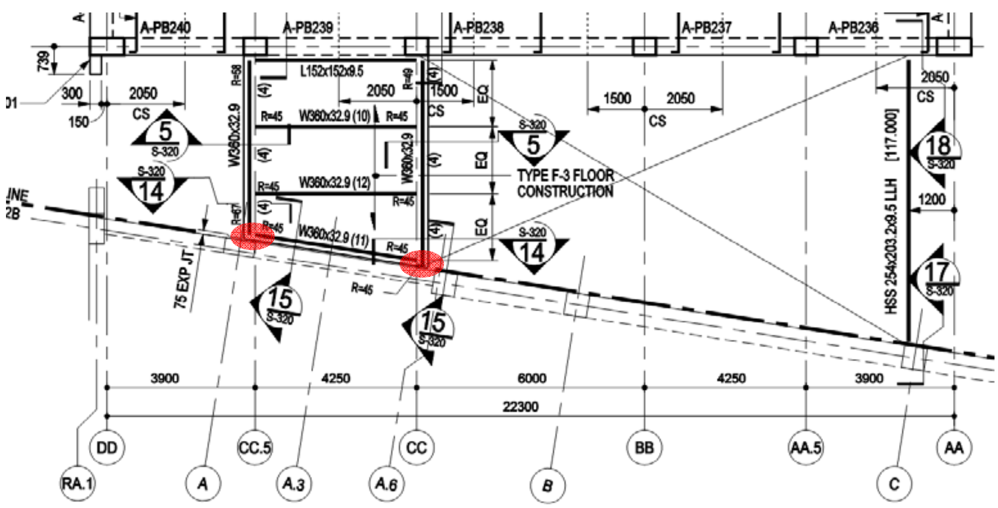


Figure 5: Wing A Atrium

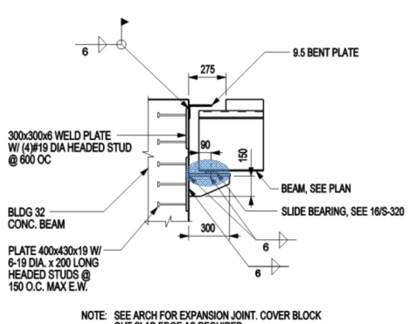


Figure 6: Expansion Joint Detail (Red)

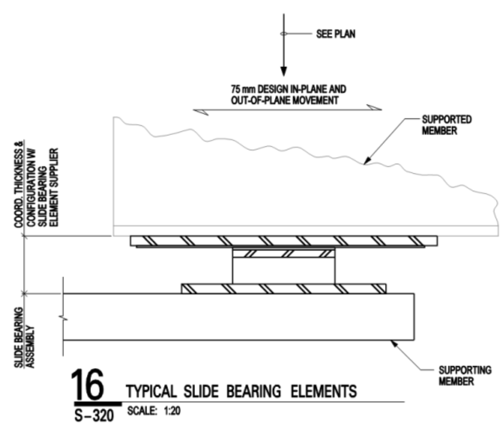


Figure 7: Expansion Joint Detail (Red)

Building 32:

Building 32 utilizes a two way flat slab system for the majority of the building's floor system. A 5.91 inch thick slab on grade is provided for the ground level and the basement levels of the building. The two-way flat slab is typically 9.449" thick with a 7.09" thick drop panel, unless noted differently on the structural drawings. The typical interior bay spacing for Building 32 is 29.528' x 19.685', and the typical exterior bay spacing of 27.559' x 29.528', figure 8 shows the typical layout of the bays.

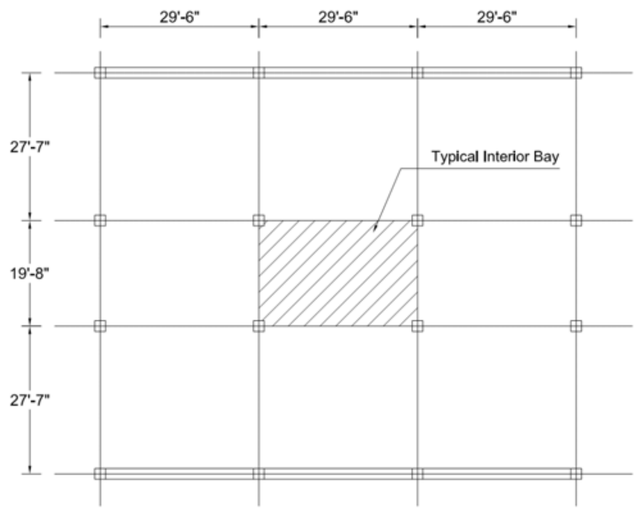


Figure 8: Building 32 Wing B Typical Bay Layout

Resistance to progressive collapse was designed into the exterior reinforced concrete beams of building 32. Typical progressive collapse beam sizes ranging from 23.62" x 40.95" to 15.75" x 40.95".

Atriums are provided between Wings B and C, and between wings C and D. The floor system for the atriums is a cast in place lightweight concrete on metal deck. The rib height on the metal deck is 1.97" with an additional 2.52" of concrete above. Supporting the walkways are W150 x 30 steel beams that frame into W610 x 217 girders with a shear connections. Expansion joints at the Intersections of the wings are provided and sliding connections are required at the atrium walkways.

Columns

Typical reinforced concrete columns were designed for the FDA OC/ ORA Office Building. Designed as the primary gravity system, the typical sizes of the columns are 600mm x 600mm, 900mm x 600mm, and 600 mm diameter. Various types of columns are provided ranging from square columns, rectangular columns and circular columns (Figure 9). The concrete for the columns is a normal weight concrete with 28 day strength of 28 MPa (4061 psi). The slab and the beams are monolithic with the columns forming a continuous system.

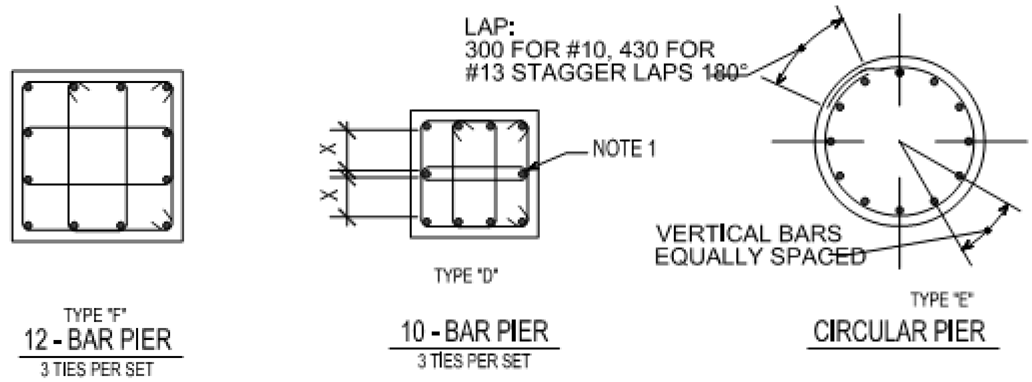


Figure 9: Typical Column Details

Lateral System

Ordinary reinforced concrete shear walls were design for the primary lateral resisting system. The typical shear wall has #16 at 300mm (#5 at 11.82 inches) for both vertical and horizontal reinforcement with 13 #16 (13 #5) for the end zone reinforcement and #13 ties at 300mm (#5 ties at 11.81 inches) for the vertical reinforcement (Figure 10 and 11).

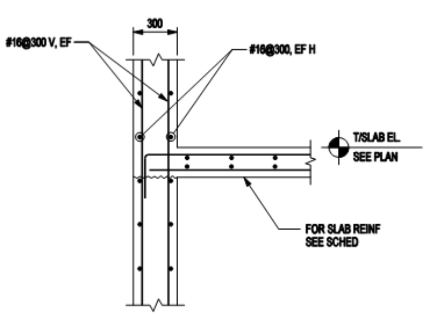


Figure 10: Shear Wall Detail

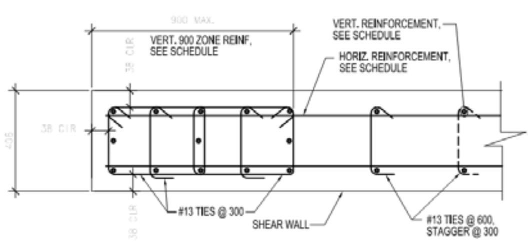


Figure 11: Shear Wall End Zone

Shear walls are provided around each elevator core and the stair shaft of Wing A. Wings B through D provide shear walls around each elevator core; Figures 16 through 19 shows the location of the shears walls in each wing, shown in red. At the intersection of each wing, in the atriums, slide bearing connections are provided at the expansion joints, shown in blue. These connections allow each wing's lateral systems to act independently of the other wing.

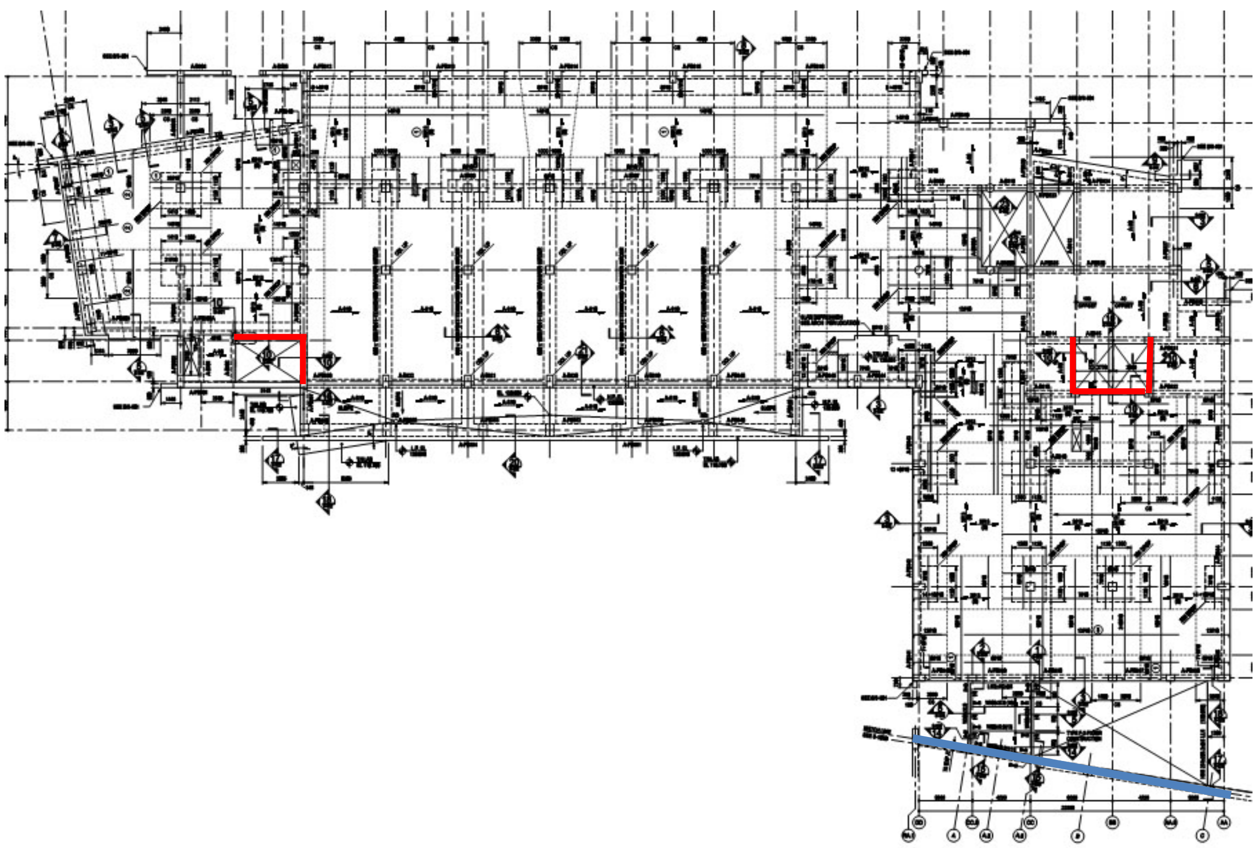


Figure 12: Shears Walls of Wing A

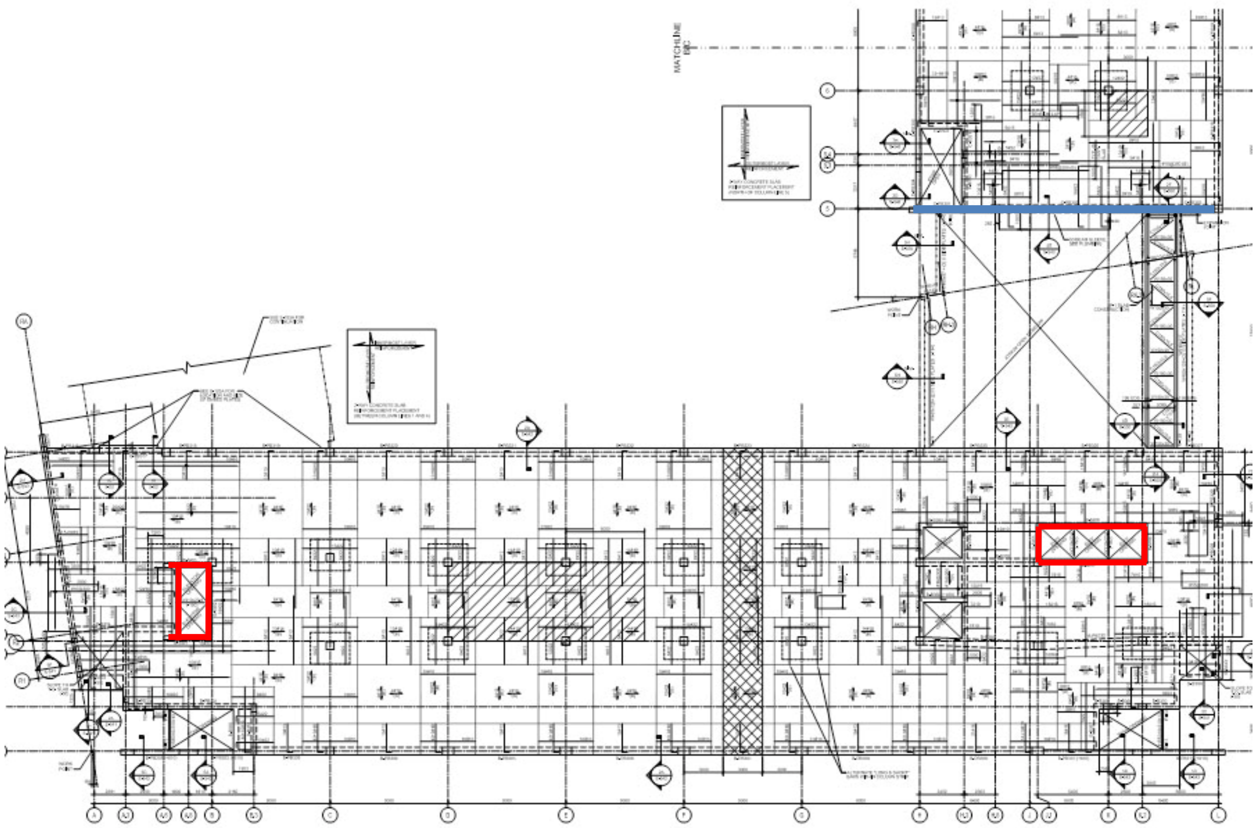


Figure 13: Shear Walls of Wing B

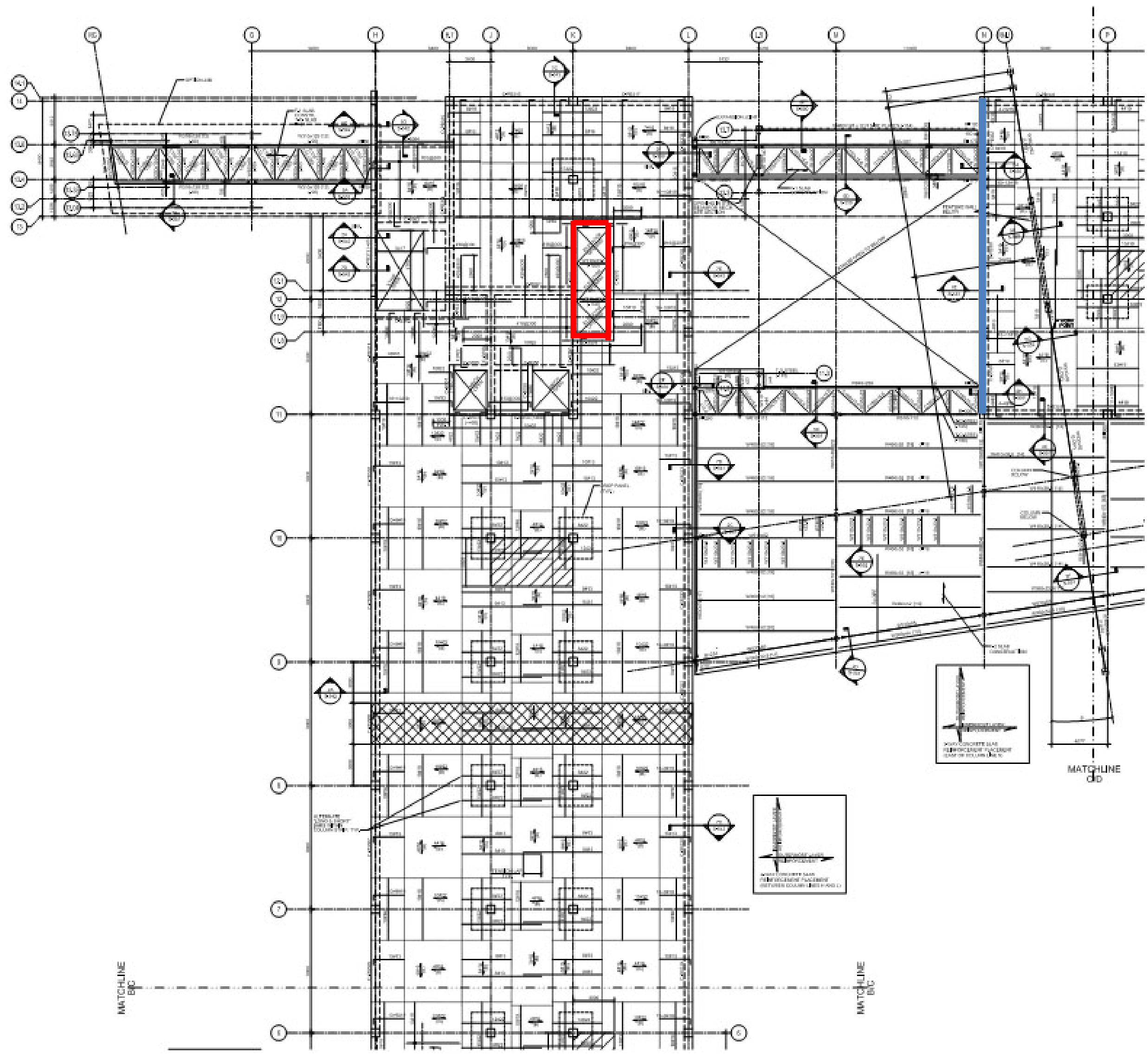


Figure 14: Shear Walls of Wing C

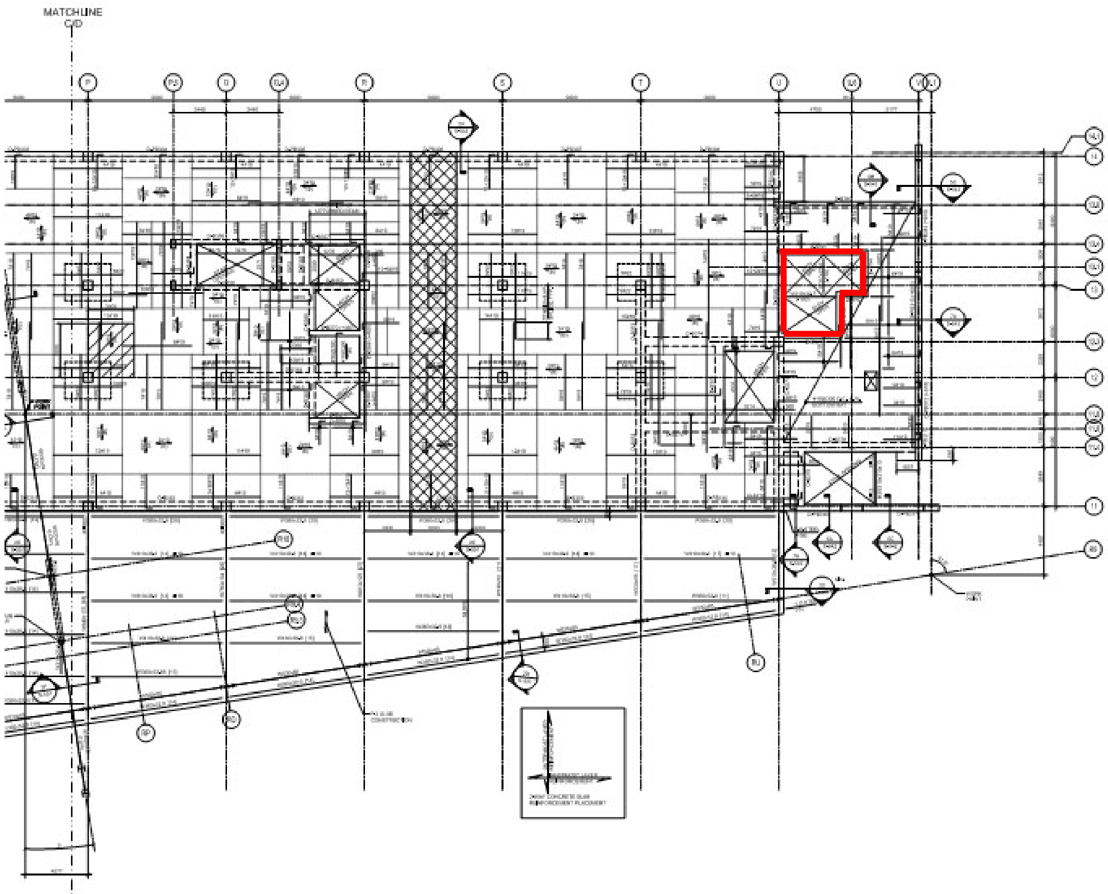


Figure 15: Shear Walls of Wing D

Load Paths

Gravity Load Resisting System:

Reinforced Concrete columns make up the primary gravity load resisting system. The live load, self weight and superimposed dead load that sits on the floor system is transferred to the reinforced concrete beams. Reinforced concrete columns pick up the loads from the beams and the load is transferred to the buildings foundations. In Wing A reinforced concrete columns bear on a post tension transfer girder. Figure 16, shows a diagram of the post tension transfer girder that transfers the gravity load to the exterior columns. Surrounding columns that the transfer girders bear on transfer the load from the girders into the columns. Columns then transfer the load into the foundation of the building.

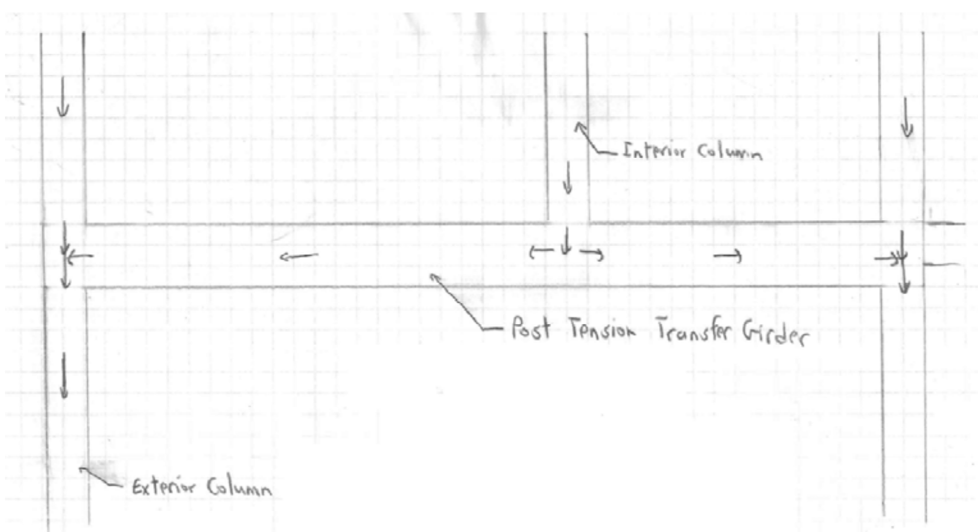


Figure 16: Transfer of Gravity Load

Resistance to progressive collapse has been designed for the office building. Design considerations that are involved with this design are removing an exterior column, and the floor system above and the adjacent columns are designed to carry the additional load.

Lateral Load Resisting System:

Reinforced concrete shear walls are the primary lateral load resisting system. Lateral force due to wind is transmitted against the curtain wall of the building. Rigid floor system picks up each story shear at each level and transmits the lateral force to the shear walls located around each elevator core. Shear walls are design to resist the moment from the lateral load. The resisting moment forces are transmitted through the shear walls onto large spread footings.

Each wing acts independently with respect to the others wings. This is primarily due to the large expansion joints provided between each wing, along with the slide bearing connections design at the atriums connections.

This report specifically looks at the lateral system in Wing B. There are eight shear walls that are provided around the two elevator cores that are provided in Wing B. In shear walls 4 and 8 coupling beams are provided between the elevator doors and shear wall piers. Figure 17 provides the layout and location of the shear walls in Wing B. Appendix C provided dimension and details of the shear walls that are provided in Wing B.

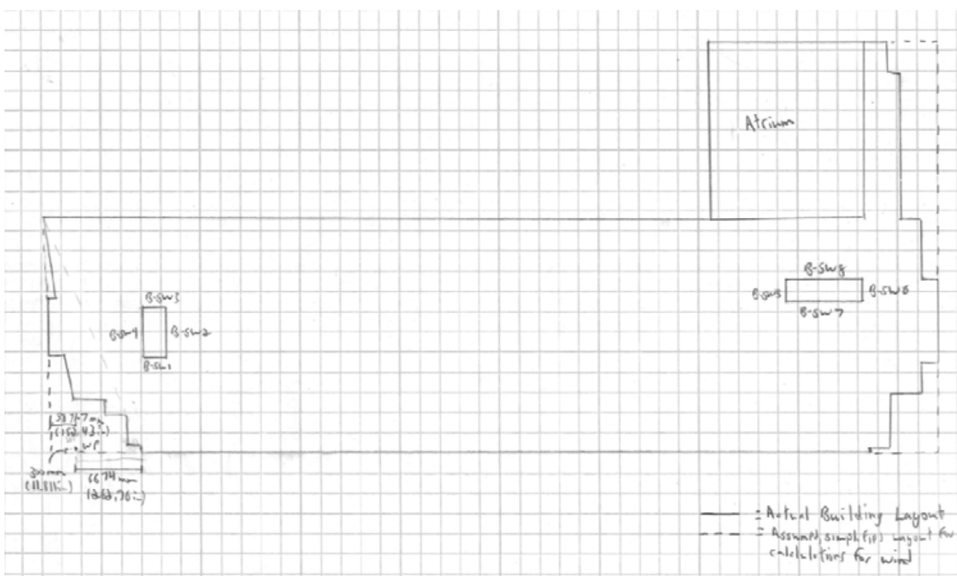


Figure 17: Wing B Shear Wall Layout

Codes and References

Design Codes:

National Model Code:

GSA Facilities Standards for the Public Building Service
International Building Code 2003

Structural Standards:

GSA Facilities Standards for the Public Building Service
ASCE 7-02, Minimum Design Loads for Buildings and other Structures

Design Codes:

AISC-ASD, Specifications for Structural Steel Buildings – Allowable Stress Design
ACE 318-02, Building code Requirements for Structural Concrete

Design Codes (Used for this Thesis)

National Model Code:

GSA Facilities Standards for the Public Building Service – 2005
2006 International Building Code

Structural Standards

GSA Facilities Standards for the Public Building Service – 2005
ASCE 7-05, Minimum Design Loads for Buildings and other Structures
ASCE 41-06, Seismic Rehabilitation of Existing Buildings

Design Standards:

Steel Construction Manual 13th edition, American Institute of Steel Construction
ACI 318-05, Building Code Requirements for Structural Concrete, ACI
Design of Buildings to Resist Progressive Collapse 2005, Unified Facilities Criteria
Progressive Collapse Analysis and Design Guidelines June 2003, GSA

Gravity Loads

The primary design guide lines for the FDA OC/ORA Office Building are the GSA Facilities Standards for the Public Service-2005, and the ASCE 7-02. The GSA outlines general requirements for the required live load for office interiors and the telecom room. The GSA Facilities Standards for the Public Building Service requires the designer to implement progressive collapse design into the structural design.

The latest version of design codes is being used for the analysis of the buildings gravity and lateral systems. When comparing to the designed loads and the ASCE 7-05 required loads, only one major difference appeared. ASCE 7-05 requires a load of 100 psf for special purpose roofs, specifically green roofs. Comparing to the designed load of 31.33 psf, one possible reason for the significant difference is the dead load; the structural engineer added a green roof dead load.

Live Loads					
	Design		GSA 05	ASCE 7-05	
Location	kPa	psf	psf	psf	
Office	3.8	79.36	80	50	(Partitions)
Typical Roof	1.5	31.33		20	
Public Lobbies	4.8	100.25		100	
Mech Room	7.3	152.46		150	(Assumed)
Telecom Room	12	250.63	250	150	
Pedestrian Bridge	4.8	100.25		60	
Balconies	4.8	100.25		100	
High Density Filing	12	250.63		250	(Assumed)
Green Roof	1.5	31.33		100	

Figure 18: Live Loads

Dead Loads		
	psf	
Superimposed Dead Load (MEP, Ceiling)	15	(Assumed)
Roofing System	40	(Assumed)
Mechanical Unit	150	(Assumed)
Exteior Curtain Wall	30	(Assumed)
Atrium Cutrain Wall	20	(Assumed)
Mechanical Pentouse Walls	20	(Assumed)

Figure 19: Dead Loads

SNOW LOADS (S)					ASCE 7-05 Ref.
Ground Snow Load	p_g =	25	psf		Figure 7-1
Exposure Factor	C_e =	1		Terrain Category B	Table 7-2
Thermal Factor	C_t =	1			Table 7-3
Importance Factor	I =	1		Occupance Category II	Table 7-3
	p_f =	17.5	psf	p_f = .7*Ce*Ct*I*pg	Eq. 7-1
	p_{fmin} =	20	psf	$p_{fmin} = p_g$*I	Section 7.3
	p_f =	**20**	**psf**		
Snow Drift					
Snow Density	γ =	30	pcf		Eq. 7-3
	h =	14.66	ft		
	h_d =	0.67	ft		
	h_c =	13.99	ft		
Snow Surcharge	**S_d =**	**52.5**	**psf**		Section 7.7.1

Figure 20: Snow Loads

Lateral Loads

To simplify the lateral analysis of the office building, lateral loads were determined for only Wing B. This was allowed because the wings have different lateral systems that do not interact with the other wings. The structural engineer also provided large expansion joints in the atriums that connect each wing, along with slide bearing connections. The slide bearing connections allow the wings to move and react independently from the lateral forces.

Wind Loads

The wind loads were determined using Method 2 of the ASCE 7-05 Chapter 6. The first assumption under the wind analysis was that the 5 story braced frames and exterior moment frames structure would act rigidly under lateral loads. Appendix A contains a summary of the results from the Wind Calculations. Detailed information on the calculation of the wind design variables can be provided upon request.

In the North to South direction the Base Shear was larger than the East to West direction; this is due to the large façade area in this direction. The wind forces are shown in Figures 21 and 22.

Design Wind Loads in N-S Direction				
	External Windward Load (kips)	External Leeward Loads (kips)	Base Shear (kips)	
			1.0W	1.6W
Level 1	0	0		
Level 2	31.771	16.907	**48.678**	**77.884**
Level 3	33.480	15.398	**48.878**	**78.205**
Level 4	36.700	15.398	**52.098**	**83.356**
level 5	39.327	15.398	**54.725**	**87.560**
Roof	25.274	9.578	**34.851**	**55.762**
Parapet	5.010	1.879	**6.889**	**11.022**
Base Shear			**246.119**	**393.790**

Figure 21: N-S Wind Loads

Design Wind Loads in E-W Direction				
	External Windward Load (kips)	External Leeward Loads (kips)	Base Shear (kips)	
			1.0W	1.6W
Level 1	0	0		
Level 2	14.675	7.809	**22.484**	**35.974**
Level 3	15.464	7.112	**22.576**	**36.122**
Level 4	16.951	7.112	**24.064**	**38.502**
level 5	18.165	7.112	**25.277**	**40.443**
Roof	11.674	4.424	**16.098**	**25.756**
Parapet	2.314	0.868	**3.182**	**5.091**
Base Shear			**113.680**	**181.888**

Figure 22: E-W Wind Loads

Seismic Loads

Seismic Loads for the FDA OC/ ORA Office Building were calculated using ASCE 7-05 Chapter 11 and 12. Initially the self weight of each floor needed to be estimated for the seismic calculations. This was done by assuming the framing systems for each floor were close enough to be approximated as the equal. The slab, beams and columns were all measured and their self weights were added up is Microsoft Excel. The exterior wall weight was assumed to be 30 psf because of the cmu backup behind the brick veneer curtain wall.

The Seismic Design Category was calculated using Table 11.6-1 and 11.6-2 in the ASCE 7-05. A SDC of A was determined for the Wing B of the office building; Appendix B contains the summary of the results from the seismic calculations, more detailed calculations can be provided upon request. The SDC calculated is different than the SDC of B that was designed by the structural engineer. A possible reason for this difference is the use of the USGS Ground Motion Parameter gave a much lower mapped acceleration. The story lateral forces and story shear forces were calculated with the equivalent lateral force procedure, using excel in Figure 35. Figure 23 shows a table of the story forces along with the calculated base shear of 95 k.

Seismic Loads					
Level	Story Weight w_x (kips)	Height h_x (ft)	Lateral Force F_x (Kips)	Base Shear (kips)	
2	1711.82	15.82	17.12		
3	1696.03	28.31	16.96		
4	1696.03	41.2	16.96		
5	1696.03	54.09	16.96		
Roof	2680.3	66.98	26.80		
			$\Sigma F_x = V_x =$	**95**	**kips**

Figure 23: Seismic Loads

Load Combinations

Load Combinations provided by ASCE 7-05 for strength design are listed below.

- 1.4(D + F)
- 1.2(D + F + T) + 1.6(L + H) + .5(L_r or S or R)
- 1.2D + 1.6(L_r or S or R) + (L or .8W)
- 1.2D + 1.6W + L + .5(L_r or S or R)
- 1.2D + 1.0E + L +.2S
- .9D + 1.6W + 1.6H
- .9D + 1.0E +1.6H

Lateral load analysis was performed for this report and the load combinations that did not include lateral load forces were disregarded. It was also noted that the load combination including a factor of .9D are used to calculate the uplift forces for the lateral loads. For strength design the load combinations including 1.6W and 1.0E were the controlling load combinations. The combinations that were considered for this report are;

- 1.2D + 1.6W + L + .5(L_r or S or R)
- 1.2D + 1.0E + L +.2S
- .9D + 1.6W + 1.6H
- .9D + 1.0E +1.6H

Proposed Thesis

Using the existing grid for Wing B, a steel framing layout for each floor will be determined and modeled using RAM Structural System. Typical steel on metal deck system will be designed for the gravity system, and later the impact on MEP will be considered. After the initial design is accomplished, the lateral loads will be determined and the lateral resisting system will be designed.

In the previous semester's work it was determined that the existing lateral system provided an eccentricity problem and deflection issues. It is planned to design braced frames around the two elevator cores of Wing B, and design exterior moment resisting frames. It is expected that the moment frames will help reduce the eccentricity problems observed in the previous semesters work.

Advanced computer modeling techniques will be included in the lateral design to optimize the lateral system design. A RAM Structural System Model will be created to initiate the design process and determine the required sizes for the lateral members. Using an ETABS model, torsional properties and distribution of lateral forces will be analyzed. An optimized layout will be determined to minimize eccentricity effects of the loading conditions

The design of structures to resist progressive collapses was designed in the original structure. It is intended to research the procedures required to design a steel structure to resist progressive collapse. Currently there are two primary methods for progressive collapse design. The first method is standards published by The General Service Administration, and the second is released by the Department of Defenses.

Steel Connection Design

The connections for steel buildings have to be designed to not only meet the structural requirements, but also meet constructability requirements. For the master's integration, the typical connections for the steel building will be designed. The typical beam connection will include shear connections for the floor systems, and moment connections for the moment resisting frames. Also the braced frames will incorporate the design of a bracing connection.

In-Depth Cost and Schedule Comparison

The first breadth study was chosen with its connection to the structural depth. The proposed changes to the lateral system with post tension design will have an impact on the scheduling on construction. The scheduling changes that would involve the additional construction time for the jacking of the post tension strands. A cost comparison of the existing structural system to the proposed changes will be made to the lateral system. Once the scheduling impact and the cost changes are considered, the feasibility of redesigning the progressive collapse beams as post tension beams will be evaluated.

MEP Coordination Study

After the gravity system is design and the depth of the structural beams a MEP Coordination Study will be performed. Using the current ceiling height and re-adjusting the mechanical, electrical and plumbing that passes through a section of the building, the change in the allowable space without having interferences. The proposal is to re-design the MEP coordination to allow for the increased depth of the structural floor without impacting the architectural of the space.

MAE Integrated Work

It was the purpose of this thesis to integrate the course work learned in the graduate course into the thesis. There are multiple courses that were used to aid in the integration of the graduate course work, and one topic which was not learned in any course was implemented into the thesis. Advance Computer Modeling: AE 597A, Steel Connection Design: AE 534 and Progressive Collapse Design.

Advance computer modeling techniques were implanted in the creation of two 3D Models to aid in the design of the steel structure. RAM Structural Analysis was initially used for the gravity system design and the lateral system design. Using lateral design concepts and mode and period analysis the design of the lateral system was chosen to minimize torisonal effects on the structure. An advanced 3D ETABS model was created to determine the lateral distribution of forces. Only the lateral system was modeled, and a 1000 kip load was applied to the structure, and the results were analyzed. This model was used to determine the validity of the RAM Model in the design checks of the lateral system.

Typical Steel Connections were designed for both the gravity and lateral system, using the information learned in the connection design course. Simple shear connections were design for the beam to girder connection and girder to column connections. The lateral connections were more advance and required more design work. A moment connection for the moment frame was designed, using a shear tab and bolted welded flange plates. A heavy braced connection was also designed using the uniform force method. The angle to gusset, gusset to column and beam to column connections were all design and checked for all the limit states.

The design of structures to resist progressive collapse was also looked at for the new structural design. Progressive Collapse Design is not covered in any course, and its advanced research in nature, put it in a higher level of work for this thesis. There are two methods of progressive collapse, both methods were studied and the procedures for each method will be reported on. Time permitting the application of this research will be implemented to the structure.

Thesis Research

Course Materials:

AE 597A "Advance Computer Modeling"
AE 534 "Steel Connection Design"
AE 403 "Advanced Steel Design"

Geschwinder, Louis F. Unified Design of Steel Structures. John Wiley & Sons Inc., 2008.

Structural Standards:

ASCE 7-05, Minimum Design Loads for Buildings and other Structures
ASCE 41-06, Seismic Rehabilitation of Existing Buildings

Design Standards:

Steel Construction Manual 13th edition, American Institute of Steel Construction, 2005.

Unified Facilities Criteria. Design of Buildings to Resist Progressive Collapse. Depart of Defense, July 2009.

General Services Administration. *Progress Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects*. U.S. General Services Administration, June 2003.

Structural Steel Framing

The existing grid for Wing B was used to determine the column layout for the new structural system, Figure 24. It was determined the column sizes would be smaller in steel, and impact on the architectural space would be positive and not be considered. A few locations the columns were aligned with the grids, where previous design was offset. The impact on the space was assumed minimal. The framing was separated into three sections; left, middle, and right. The middle framing layout was the most typical. Near the ends the framing is not as typical, and a general layout was used. Columns were designed to be two story columns and spliced every other floor.

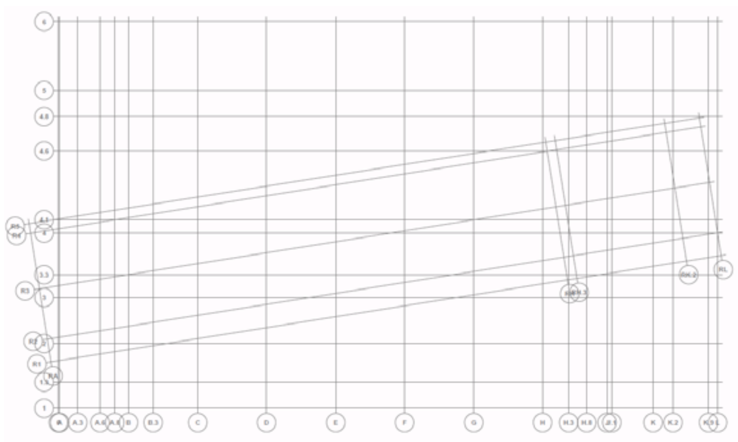


Figure 24: RAM Grid Layout

Gravity System Design

The gravity system was designed using RAM Structural Systems using the gravity loads determined to act on the structure. The girders were designed to span across the short direction between columns in the vertical direction. Intermediate beams were laid in the opposite direction with intermediate beams intersecting the mid span of the girder in the interior bays, and intersecting the third points for the exterior bays. A 2" LOK Metal Steel Deck was pick from United Steel Decking to support the superimposed loads, and the designed beam spacing is less than the allowable unsupported width. The floor system is designed to be 4 ½" concrete which was more than adequate to support the loads. Live load reduction was used to design the beams and columns for the gravity system, and composite action was assumed and designed.

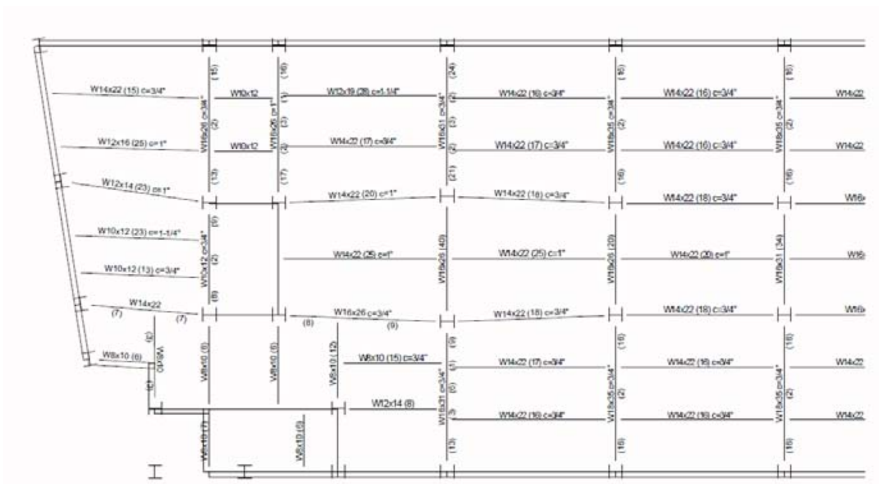


Figure 25: Framing Plan Part 1

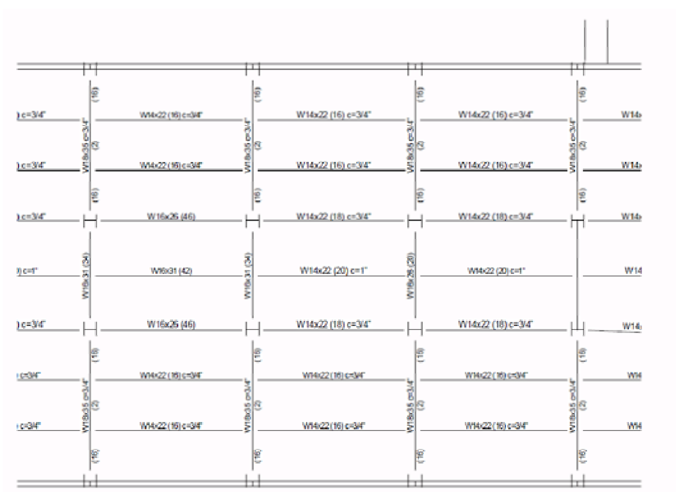


Figure 26: Framing Plan Part

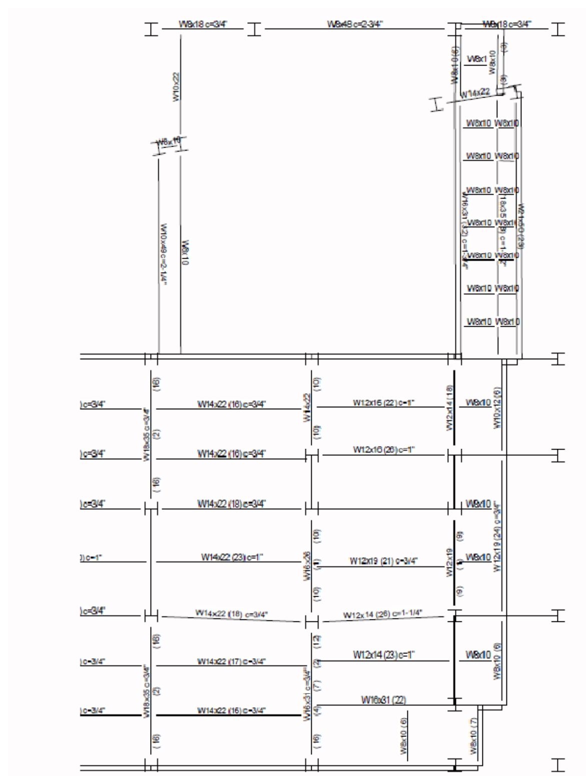


Figure 27: Framing Plan Part 3

Typical framing plans were created from RAM and can be found in Appendix C. Figures 25 through 27 show the second floor framing plan with the beam design in detail. The typical interior beam was designed as a W14 x 22 with a 1" camber. The beams frame into the girders that were designed as W16 x 31 for the Interior Bay and W18 x 35 for the exterior bays. The gravity columns were also designed, the design summary can be found in Appendix C, and Figure 28 shows an excerpt from the design summary. The interior columns along Grid D were designed as W10 x 54 for stories 2 and 3, W10 x 39 for stories 4 and 5, and a W10 x 33 for the column below the roof story. The gravity check for the column was designed as a W12 x 65; however a W10 x 77 would have been sufficient. The difference in the designs is attributed to live load reduction which was included in the RAM design, and not in the hand checks.

Column Line D-2

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	65.7	1.8	10.2	6	0.64 Eq (II1-1a)	0.0	50	W10X33
Floor 5	134.8	1.6	4.6	3	0.72 Eq (H1-1a)	0.0	50	W10X39
Floor 4	191.1	1.4	5.5	6	0.99 Eq (H1-1a)	0.0	50	W10X39
Floor 3	258.4	1.5	5.2	3	0.78 Eq (H1-1a)	0.0	50	W10X54
Floor 2	318.5	0.0	5.7	6	1.00 Eq (H1-1a)	0.0	50	W10X54

Column Line D-3

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	65.7	1.8	10.2	11	0.64 Eq (H1-1a)	0.0	50	W10X33
Floor 5	134.8	1.6	4.6	4	0.72 Eq (H1-1a)	0.0	50	W10X39
Floor 4	191.1	1.4	5.5	11	0.99 Eq (H1-1a)	0.0	50	W10X39
Floor 3	258.4	1.5	5.2	4	0.78 Eq (H1-1a)	0.0	50	W10X54
Floor 2	318.5	0.0	5.7	10	1.00 Eq (H1-1a)	0.0	50	W10X54

Figure 28: Column Design Summary

Hand calculations were performed for random gravity members to determine the validity of the RAM Model. Using the Thirteenth Edition of the AISC Steel Manual composite beams and girders were sized and checked against other limit states; refer to Appendix E for the calculations. It was determined that the gravity members sized in RAM matched the sizes determined by hand calculations.

There are two columns that were not designed with RAM due to the slenderness limits. The two columns are on the south side of the building, and are primary architectural features and take very little load. The columns were designed as composite columns with a W10x33 and 18" by 22" Concrete column surrounding the W shape. The effective length of the column and the slenderness effects reduced the load carrying capacity of the column. However, the column received very little load, and the column was still adequate to carry the loads.

Foundation

The structural redesign to structural steel reduced the required load on the foundations. Spread footings were designed using the allowable bearing capacity determined from the soils report; 6 kips per square foot was used for the soil under Wing B. The spread footings were designed for the gravity load from the column, the column sits on a base plate that attached to a concrete pier that will transfer load to the footing.

The new spread footing was designed to be a square footing with a length and width of 10 feet. After the size of the footing was chosen, the depth and reinforcement was designed. A height of 24 inches was sufficient for the punching shear and flexure forces. Using (11) #7 reinforcing bars in both directions for the design of the reinforcement for the tensile flexure forces. The new design was compared to the original footing design, and the new footing decreased the height of the footing by 12 inches. Since the building got lighter with the new structural steel design, it was expected to see a smaller spread footing. The design of the footing can be found in Appendix G.

An overall stability check was performed on the building to determine if overturning would be a problem. Using the Wind in the North South direction and the dead load for the building, the resistance to overturning was check. It was determined that the uplift force was counteracted by the gravity load. However in a few instances the base plates to the foundation piers will have to be design to take individual uplift forces from the lateral resisting system.

Lateral System Design

It was determined that braced frames around the core would be designed as the primary lateral system to control the drift of the building, and moment frames around the exterior would decrease the eccentricity of the lateral system. In the design of structures to resist progressive collapse, moment frames are the preferred method which also contributed in the reduction of the eccentric effects. Initially the gravity sizes of the beams and columns were used to size the lateral members. It was later determined that the beams were undersized and needed to be revaluated, Figure 29 shows the 3D model that was created in RAM.

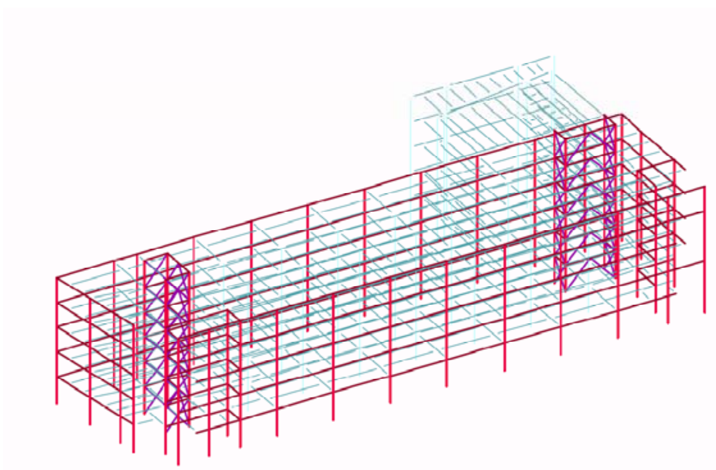


Figure 29: RAM Lateral Model

Using the AISC Thirteenth Edition of the Steel Manual, and equations for max moment for a fix ended beam, assuming gravity loads would control the design of the moment frame beams. A preliminary size for the moment frame beams were designed, assuming mid span bracing at the bottom flange of the beam and full lateral bracing on the top of the beam. The sizes were inputted into the RAM Model, and the analysis of the lateral system began.

Using RAM Analysis the periods of vibrations and Center of Rigidities were calculated. It was observed that the braced frames at the elevator cores and moment frames at the exterior did decrease the eccentricity from the shear wall design, but still produced an eccentric loading. A new design was created, and compared to the first braced frame design. A braced frame was added between the two elevator cores along grid H, shown in Figure 30.

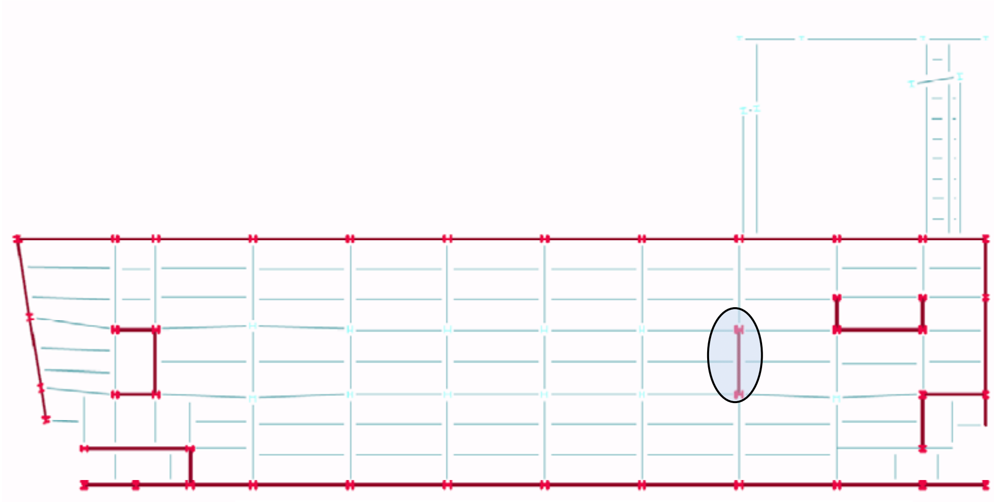


Figure 30: Lateral Framing

Lateral analysis of both models was performed and the center of rigidities were calculated and compared, Figure 31. It was determined that the second model that includes the added brace did decrease the eccentric effects of the lateral system. The new design is more efficient than the original braced frame design, and the final design steps were carried out. A 3D ETABS model was created to perform distribution analysis of the lateral system, the Modal periods of vibrations were calculated and compare, Figure 32. It was observed that the Mode 1 for all three models was the torisional mode, and the additional brace did decrease the period.

Center of Rigidity Compariosn for RAM Models						
	RAM Model 1		RAM Model 2		COM of Both Models	
	X (ft)	Y (ft)	X (ft)	Y (ft)	X (ft)	Y(ft)
Roof	85.26	41.25	128.47	41.35	154.33	49.98
Floor 5	91.23	41.02	131.95	41.09	142.63	41.12
Floor 4	104.65	41.025	140.67	41.13	142.64	41.12
Floor 3	126.14	40386	153.38	40.86	142.65	41.11
Floor 2	158.01	40.42	171.24	40.32	138.75	40.98
* RAM Model 2 adds Brace at Grid H						

Figure 31: Center of Rigidity Comparisons

3D Model Period Comaprisons										
RAM Model 1				RAM Model 2				ETABS Model		
	Direction	Period (s)			Direction	Period (s)			Direction	Period (s)
Mode 1	2.145	Z		Mode 1	1.4798	Z		Mode 1	1.1971	Z
Mode 2	1.2959	X		Mode 2	1.3521	X		Mode 2	1.0513	X
Mode 3	1.1244	Z		Mode 3	1.1173	Z		Mode 3	0.8627	Z
*Braced Frames at Core				*Additional Brace				*Same as RAM Model 2		
*Exterior Moment Frames										

Figure 32: Modal Period Comparisons

The lateral system was analyzed using RAM Structural Analysis in the RAM Frame Module. Lateral members were sized and analyzed for different load combinations in RAM, the typical brace member was a double angle ranging from 2L 5”x 5” x ¾” to 2L 4” x 4” x ½”. Two brace configurations were chosen for the lateral system. The first is a concentric chevron bracing used on the braced frames with a width of twenty feet or more, Figure 33. The second brace is concentric x bracing for the braced frames with a width of fifteen feet or less, Figure 34. These configurations will maximize the axial force in the braces to best optimize the braced frame system. Moment frames were designed around the exterior of the building to include lateral resisting elements and also aid in the design against progressive collapse. Figure 35 and 36 shows the layout used in the moment frame design along Grid 1 and Grid 4 respectfully. A full detail of each frame design can be found in appendix D.

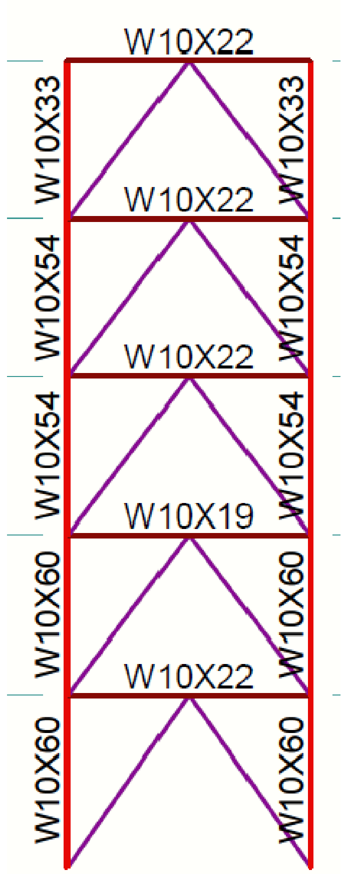


Figure 33: Chevron Brace Frame

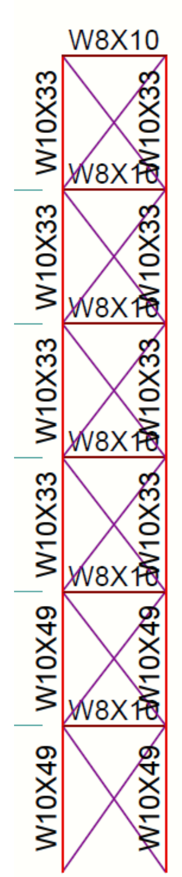


Figure 34: X Braced Frame

Figure 35: Moment Frame along Grid 1

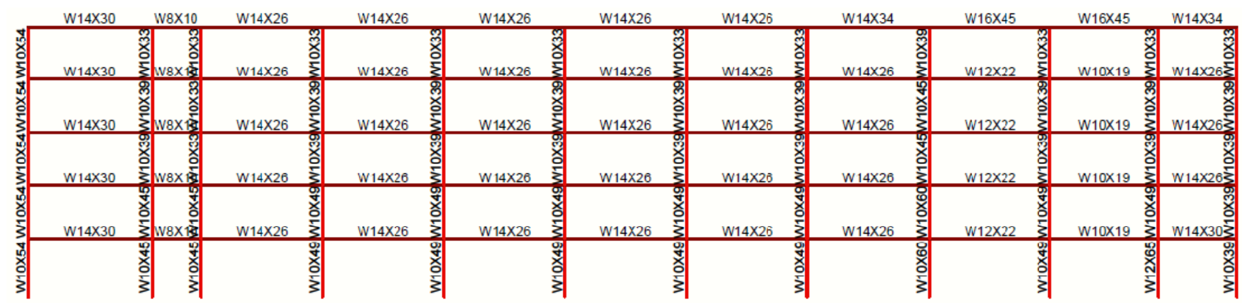


Figure 36: Moment Frame along Grid 4

After the lateral system was designed for strength requirements, the drift was analyzed using RAM Structural Systems. A drift limit of L/400 for story drift and story displacements was used to determine if the lateral system met the drift requirements. It was determined that the lateral system met the requirements and did not pose any drift problems. Figure 37 is the drift report summarized from the RAM Output.

Drift Study from RAM									
		Displacements				Drift			
	hx	Displ X	Allow.	Displ Y	Allow.	Drift X	Allow.	Drift Y	Allow.
Roof	12.89	0.3611	1.9716	1.1775	1.9716	0.0543	0.3867	0.2794	0.059148
5th	12.89	0.3068	1.5849	0.8981	1.5849	0.0747	0.3867	0.29	0.047547
4th	12.89	0.2321	1.1982	0.6081	1.1982	0.0878	0.3867	0.271	0.035946
3rd	12.89	0.1443	0.8115	0.3371	0.8115	0.1443	0.3867	0.3371	0.024345
2nd	14.16	0.0596	0.4248	0.135	0.4248	0.0596	0.4248	0.135	0.012744

Figure 37: Drift Calculations

Computer Modeling

Advanced computer modeling techniques were used to generate a more accurate design along with variations in the design for different load conditions. RAM Structural Systems was used as the primary modeling system to design both the gravity systems and lateral systems. Assumptions for the RAM Structural Model are:

1. P- Delta Effects Included.
2. Center line modeling.
3. Panel Zone Shear is not included.
4. Braces assumed to be pinned at both ends.
5. Moment Beams assumed to be fixed at both ends.
6. The structure is assumed to be pinned at the base due to the spread footings.

Along with a RAM Structural Model, a 3D ETABS model was created to model the lateral system only. The design that was determined in RAM for the lateral system will be modeled in ETABS using the same assumptions. The ETABS model will be used to perform an analysis of the lateral systems to assist in the design process of lateral resisting elements. Figure 38 shows the summary of the analysis.

ETABS Lateral Distribtion for Floor 2		
X Direction Loading		
	V (kips)	%
Moment Frame 1	-31.408	0.031
Moment Frame 1.3	-5.0398	0.005
Moment Frame 2	-2.694	0.003
Moment Frame 4	-48.5955	0.049
Brace Frame 2	-210.508	0.210
Brace Frame 3.1	-282.217	0.282
Brace Frame 3.2	-421.105	0.420
Total	**-1001.57**	
Y Direction Loading		
Moment Frame RA	0.594	-0.001
Moment Frame B.3	-1.1589	0.001
Moment Frame K.2	-3.47	0.003
Moment Frame L	-2.874	0.003
Brace Frame B	-366.71	0.367
Brace Frame H	-358.85	0.359
Brace Frame J.1	-117.28	0.117
Brace Frame K.2	-149.767	0.150
Total	**-1000.11**	

Figure 38: ETABS Lateral Study

Connection Design

Typical gravity and lateral connections were designed using the requirements and design guides prescribed by the Thirteenth Edition of the Steel Construction Manual. A typical beam to girder connection was designed as a simple shear tab connection shown in Figure 39. A shear tab connection was chosen for the constructability issues in lowering the beam down between the girders with little interferences. Limit states for bolt shear, bearing, tear out were and block shear were considered in the design of the shear tab connection using the Thirteenth Edition of the Steel Construction Manual.

A typical Girder to column web connection was designed as a seated connection, shown in Figure 40. The girder flange width is less than the width of the opening between flanges, so a simple seated connection was chosen for the connection. Limit states considered for the seated connection were beam web yielding and crippling, along with angle flexure and yielding. A stabilizing angle is welded to the top of the beam after erection to prevent the beam from rolling over, but it is not design to take any load.

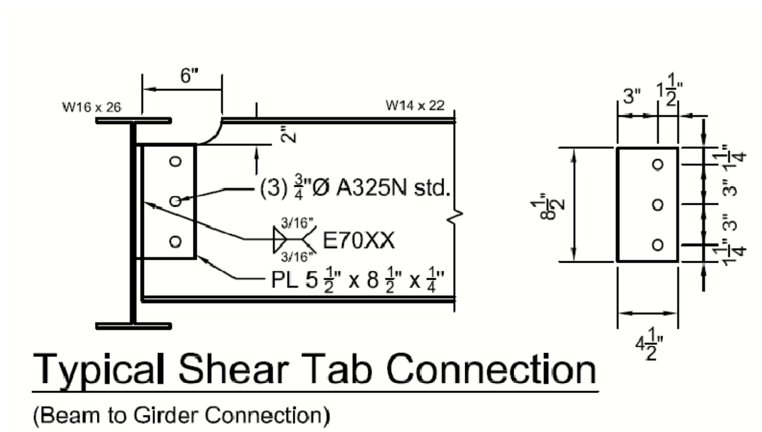


Figure 39: Shear Tab Connection

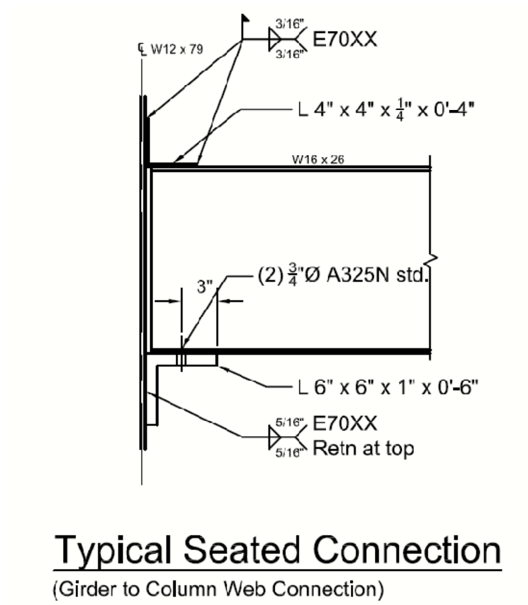


Figure 40: Seated Connection

Typical lateral connections were designed for the building, a moment connection and a heavy braced connection. The moment connection was designed in two parts, a shear tab connection to take the shear load, and a bolted flange plate welded to the column for the moment load. The plates bolted to the beam flange were checked for both the compressive and tensile loads from the moment, and the controlling design was used for all sides of the moment connection. It was determined the 2 rows of (3) ¾" bolts would be used to support the moment capacity. Figure 41 shows the detail of the moment connection.

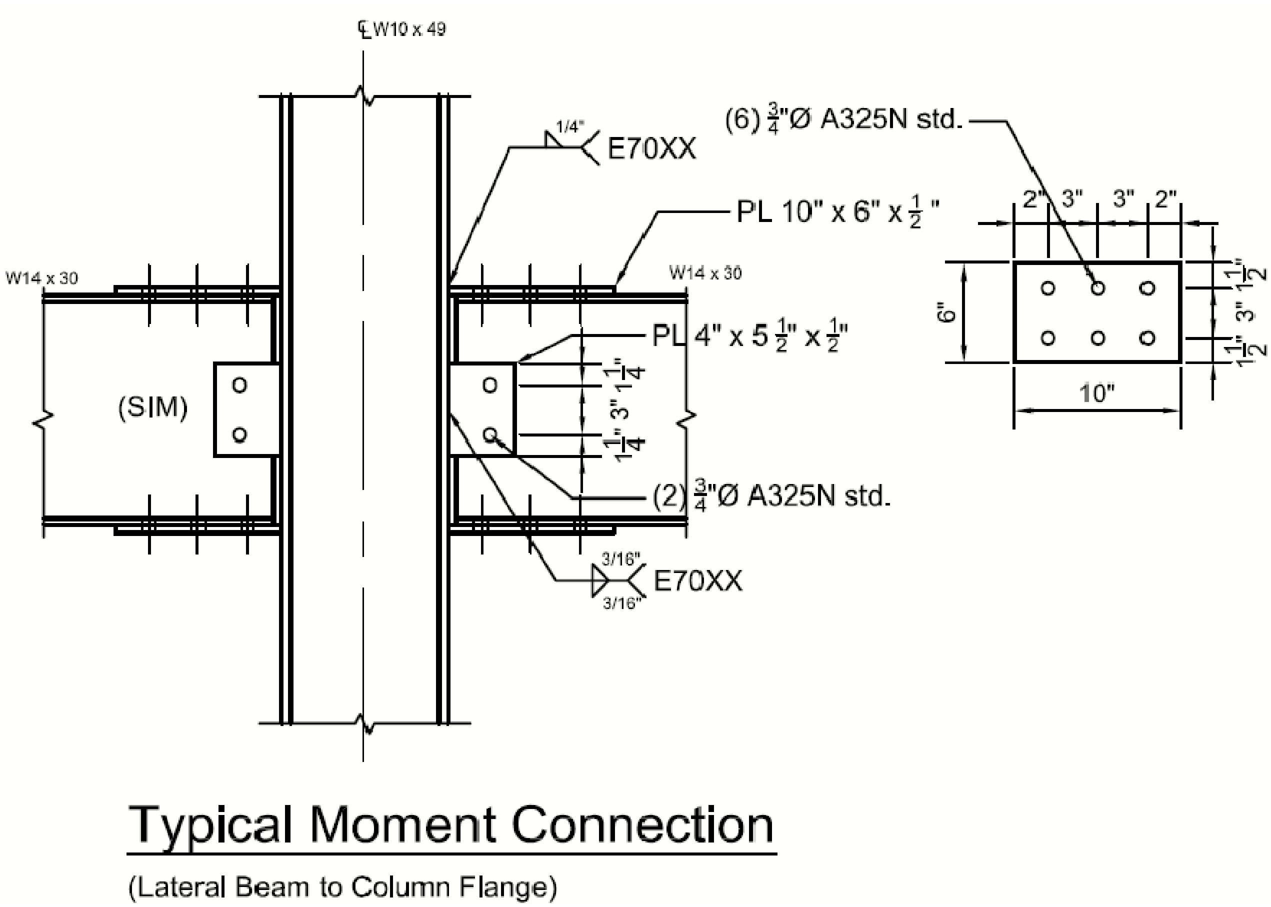


Figure 41: Moment Connection

A heavy braced connection was designed for the braced frame to adequately take lateral loads acting on the braced frames. A double angle was designed for the brace and it was designed as a bolted connection to the gusset plate. Using the Uniform Force method to size the gusset plate, dimension were chosen to reduce the moments at the connection interfaces. A bolted welded double angle was designed for the gusset to column flange connection. Prying of the angle was checked, and did not control the design of the connection. A simple shear tab connection was used for the beam to column connection. Compressive buckling of the plate did not control the design, the design of the shear resistance was sufficient. Figure 42 shows a detail of the braced connection.

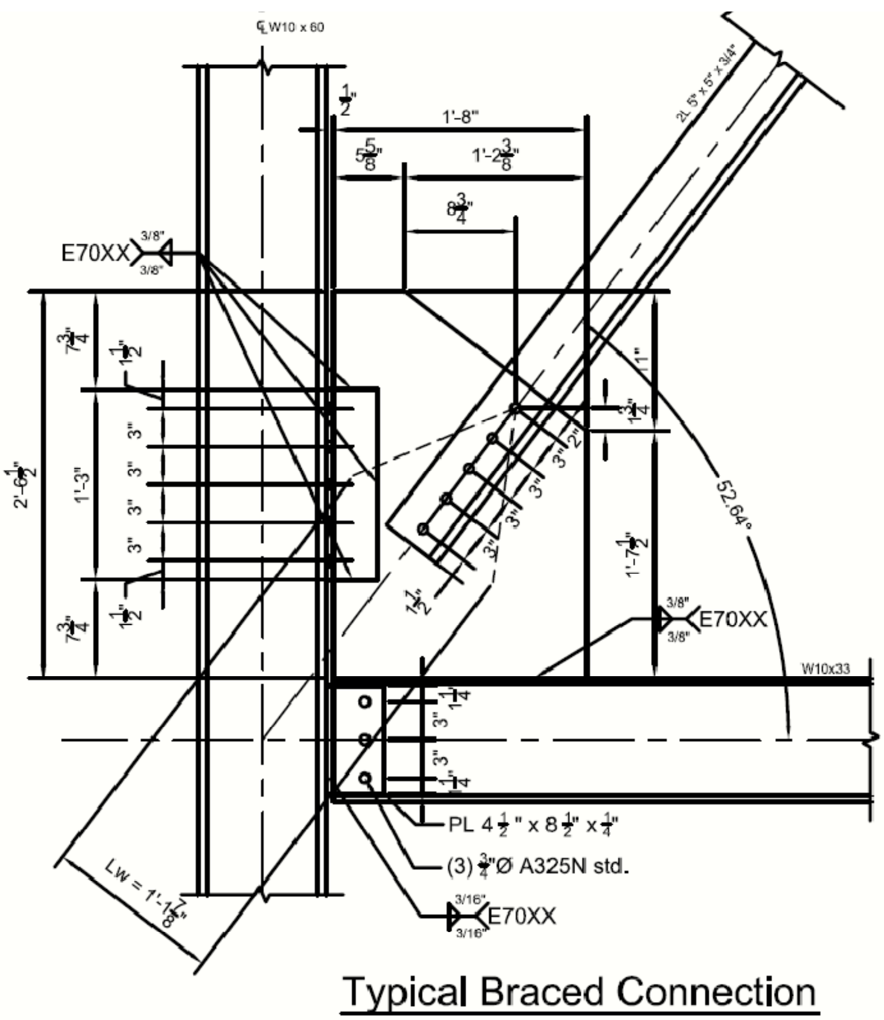


Figure 42: Heavy braced Connection

Progressive Collapse

Progressive collapse or disproportionate collapse in respect to the original cause of the damage, is when a local element is damaged leading to a collapse of adjoining members. The concepts behind the design of structures to meet progressive collapse requirements are to design redundancy in the gravity system so that if a member is damaged the collapse will remain localized. There are currently two methods of designing structures to resist progressive collapse, the first method is proposed by the General Service Administration, and the second is set by the Department of Defense.

Both methods recommend that the structure implement passive defenses to mitigate the effects on the structure. These methods include but are not limited to stand of distances and blast resistance design. Design the exterior of the structure to be more robust to resist the damaging effects from the unforeseen event.

For the purpose of this thesis the Alternate Path Method is studied. This method assumes the loss of a primary structure element and the adjoining structure elements must be designed to support the additional load. Each beam will theoretically be designed to support double span conditions. Figure 43 shows the concepts behind the removal of the column and the start of the Alternate Path Method.

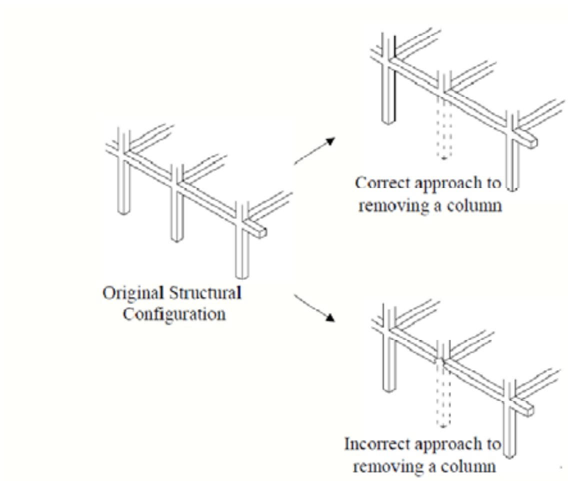


Figure 43: Progressive Collapse Concepts

GSA Standards

Design Guidance

This method places the imports on the floor girders and beams be designed to span two full spans, which requires beam to beam continuity over the missing column. The first requirement is to design connections that are capable of producing discrete beam-to-beam continuity. This states that the connection be able to provided a beam-to-beam continuity link that is able to redistribute gravity loads for a multiple span condition.

Connections should be designed to meet three criteria; resilience, redundancy, rotational capacity. Providing connection resilience is in the designing of the configuration of the weld geometry, to provide a ductile connection under instantaneous loss of a primary structural element. Connection redundancy is implemented so that there are multiple load paths to be used to distribute the gravity loads. Connection rotational capacity is provided to allow the connection to deform and rotate and form the formation of the plastic hinges in the beam of girder, while maintaining sufficient strength after the loss of the column. Figure 44, shows the response expected from a typical steel connection.

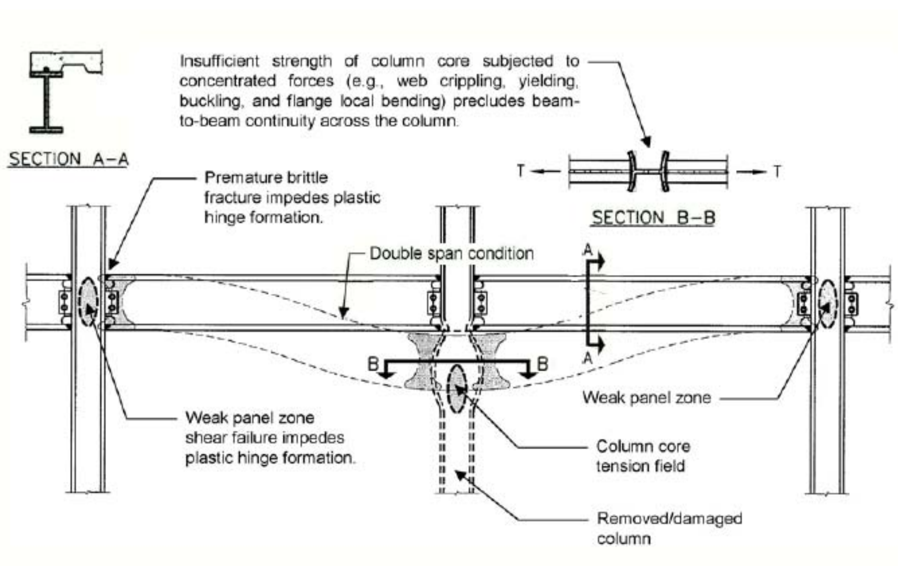


Figure 44: Response of the structure

Analysis Procedure

Analysis procedures for this method use linear elastic, static analysis approach to look at the potential for progressive collapse. It is recommended that the use of 3-dimensional models be used to accurately account for the effects from secondary elements. 2-dinemensional models are accepted but are more conservative. A structure can be analyzed for progressive collapse by looking at components and connections for the loss in primary vertical support.

For typical structure configurations the analysis considerations should include all unique structural difference that would require a different analysis for the design to resist progressive collapse. Such difference are, beam to column connection change for simple to moment, or large change in span sizes on either side of the column, and a change in beam size.

Exterior considerations for the loss of a primary structural element include the removal of a column at critical locations. If any other location is determined to be critical in the design of progressive collapse it is required to consider that condition as well. Figure 45 shows a diagram that illustrates the key locations of column removal. The standard procedures are the analysis of the loss of a column for one floor above grade at the corner of the building, middle of the long side of the building, and the middle of the short side of the building. Interior considerations for progressive collapse shown in Figure 46, explain for interior columns unprotected by underground parking or uncontrolled public ground floor.

When determining the potential of a structure for progressive collapse the following vertical load shall be applied to the structure; Load = 2(DL + .25LL). For exterior column removals the maximum extent of collapse to be considered is the minimum of; the structural bays directly adjacent to the removed column, and 1,800 square feet at the floor level directly above the removed column. For interior considerations the maximum extent of collapse is to be the structural bays adjacent to the removed column up to 3,600 square feet at the floor level directly above the removed column.

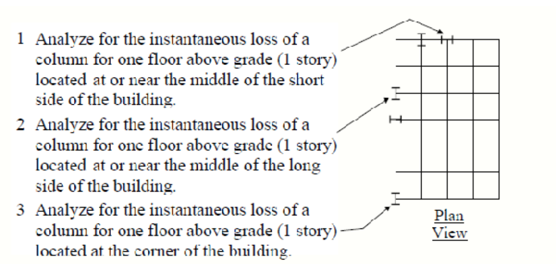


Figure 45: Exterior Considerations

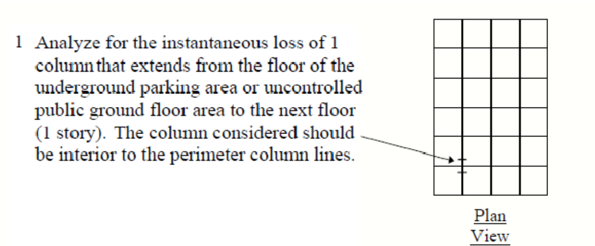


Figure 46: Interior Considerations

Acceptance Criteria

Using linear elastic analysis results the forces and rotations are compared to the acceptance criteria to determine if the structure is adequate to resist progressive collapse. Once the primary structural element is removed the affected; beams, girders, columns, and connections are compared to maximum allowable demands. They are checked by Demand-Capacity Ratios (DCR). Members that exceed these limits will generate a plastic hinge and the redistribution of forces until the structure is adequate, Figure 47 shows the formation of the plastic hinge. Some members will have to be redesigned so they will meet these criteria, and a collapse mechanism is not initiated.

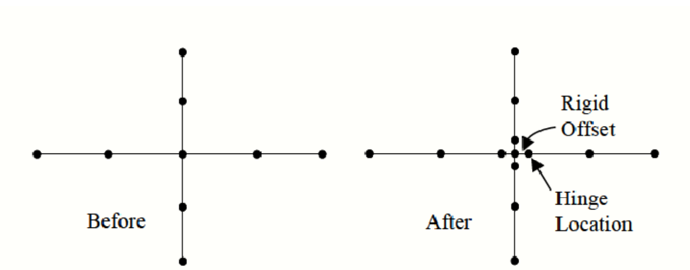


Figure 47: Plastic Hinge Formation

Primary and secondary components must be checked for the acceptance criteria from Equation 1. Where Q_{UD} is the acting force demand determined by analysis, and Q_{CE} is the expected ultimate, un-factored capacity of the component under analysis. The DCR is then compared to the allowable DCR requirements. For beams under flexure the DCR limit is either 2 or 3 depending on the beam dimensions.

$$DCR = {Q_{UD}}/{Q_{CE}} \qquad \text{Equation 1}$$

Non Linear Analysis

Using the method set forth by the General Service Administration, progressive collapse analysis and redesign was implemented. Using the moment frame along Grid 1, a typical column along the middle of the frame was removed and the structure was analyzed to determine if it was adequate to resist progressive collapse. A non linear method was used to model the progressive collapse which is assumed to be a more conservative method. Using virtual work to determine the plastic hinge formation to create a collapse mechanism, the plastic moment required was determined.

Assuming that each level must support the summation of the gravity load above that floor, the beam must span a double span condition was well as the full load transfer from the column above. Using the load combination prescribed by the GSA the floor loads were determined and transferred to the beam under investigation. Using virtual work, the required plastic moment for the beam at level 2, supporting a 1434 kip load at the location of the removed column, is 7059.6 ft-kips. Using a DCR of 3, a W33 x 169 was adequate to take the applied load, to resist the formation of a collapse mechanism.

The applied moment was transferred to the column with half the gravity load to design the column for combined axial and flexure loads. The column used a DCR of 2, and the interaction diagram for the column was studied. A W14 x 500 was selected to support the large moment with the relatively small axial load. This column is considerable large, and assumed to be highly conservative.

This method of progressive collapse design does not account for the total structures resistance to progressive collapse instead it applies the load to the bay directly being analyzed and requires the beam to support the load, and prevent the formation of a plastic hinge. Realistic the structure will act as a truss, bridging over the removed column. To obtain this type of result the structure must be model in 3D modeling software. This method will accurately show how the structure responds under the loss of the column and how the structure works as one to resist the formation of a plastic hinge.

Department of Defense Standards

The Department of Defense standards for progressive collapse design use three methods for design and analysis which are; The Tie Force Method, Alternate Path Method, and Enhanced Local Resistance. The Tie Force Method approach assumes the building is tied together as a continuous structure, and the loading conditions will be used to check the structural members for acceptance. The Alternate Path assumes the loss of a primary structural element and the structural is checked for double span conditions. The Enhanced Local Resistance checks the flexural and shear resistance of the perimeter columns to check for the design of progressive collapse. For the purpose of this thesis and the comparison to the GSA Standards, only the Alternate Path Method will be researched and studied.

Design Guidance

For the Alternate Path Method, there are three analysis procedures that can be used to design for progressive collapse. The three methods are; Linear Static, Nonlinear Static, and Nonlinear Dynamic. Each method uses ASCE 7 and ASCE 41 for the load combinations and material strength estimates.

The structure is separated into two elements; primary and secondary components. Primary components are the structural members that will directly resist the potential collapse, or the members that are going to be designed in progressive collapse. Secondary components are any other structural elements that do not meet the requirements as primary components.

The designs are categorized into force controlled actions and deformation controlled actions. For moment frames the shear and axial loads are considered as force controlled actions and the moment is considered a deformation action. Progressive Collapse should be analyzed for the following stories; first story about grade, story directly below roof, story at mid height, and story above the location of a change in wall size. The removal of columns should be considered for the corner column, column at mid span along the long side, and the column at mid span along the short side, Figure 48.

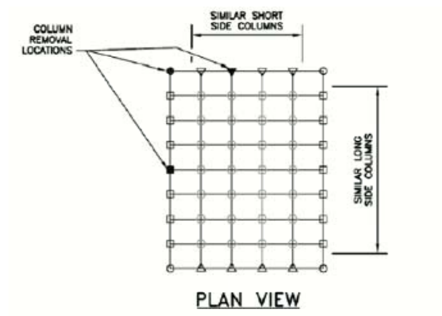


Figure 48: Column Removal

Linear Static Procedure

The use of the linear static procedure is limited to certain structures that are considered to be structurally regular. If the structure is considered to be irregular the linear static procedure may be used if the DCR's are less than 2.0; Equation 1. Where Q_{UD} is the acting force demand determined by analysis, and Q_{CE} is the expected ultimate, un-factored capacity of the component under analysis. The DCR is then compared to the allowable DCR requirements. For beams under flexure the DCR limit is either 2 or 3 depending on the beam dimensions.

Two loading conditions shall be considered one for deformation controlled actions and another for force controlled actions. The increased load shall be applied to the bays immediately adjacent to the removed element and at all floors above the removed element, Equations 2 and 3. For the rest of the bays not adjacent to the removed column a load that is not modified for the force and deformation controlled actions, Equation 4. Ω_{Ld} and Ω_{LF} are load increased factors that are calculated from the table 3-4 provide in Appendix J. For lateral loads that will be applied to the structure, Equation 5 shall be applied in each side of the building in combination with each gravity combination.

$$G_{LD} = \Omega_{LD}[(.9 \text{ or } 1.2)D + (0.5L \text{ or } 0.2S)] \quad Equation\ 2$$

$$G_{LF} = \Omega_{LF}[(.9 \text{ or } 1.2)D + (0.5L \text{ or } 0.2S)] \quad Equation\ 3$$

$$G = [(.9 \text{ or } 1.2)D + (0.5L \text{ or } 0.2S)] \quad Equation\ 4$$

$$L_{LAT} = .002\sum P, P = gravity\ load\ acting\ on\ a\ floor \quad Equation\ 5$$

For element acceptance criteria the components are analyzed using the linear procedures. For deformation controlled actions the deformation controlled action is compared to the expected strength of the component modified by a m-factor and strength reduction factor. The strength reduction factors are determined by the force that is being analyzed and the requirements set by the steel design standards. M-factors are determined using Table 5-1 supplied in Appendix J.

$$Deformation\ Controlled\ Actions:\ \Phi m Q_{CE} \geq\ Q_{UD}$$

$$Force\ Controlled\ Actions:\ \Phi m Q_{CL} \geq\ Q_{UF}$$

Nonlinear Static Procedure

The use of nonlinear static procedure is used for any structure; there are no DCR or geometric irregularity limitations. A 3-dimensional model must be used to model the primary and/or secondary structural elements. If secondary elements are modeled there stiffness's must be ignored.

Increase gravity loads shall be applied to the bays directly adjacent to the removed column, and every floor above the removed column, Equation 6. For the rest of the bays not adjacent to the removed column a load that is not modified for the force and deformation controlled actions, Equation 7. Ω_N is the load increased factor that are calculated from the table 3-5 provide in Appendix J. For lateral loads that will be applied to the structure, Equation 8 shall be applied in each side of the building in combination with each gravity combination.

$$G_N =\ \Omega_N[(.9\ or\ 1.2)D + (0.5L\ or\ 0.2S)] \quad Equation\ 6$$

$$G =\ [(.9\ or\ 1.2)D + (0.5L\ or\ 0.2S)] \quad Equation\ 7$$

$$L_{LAT} =\ .002 \sum P\ , P = gravity\ load\ acting\ on\ a\ floor \quad Equation\ 8$$

For element acceptance criteria the components are analyzed using the nonlinear procedures. Primary and secondary elements shall have deformation capacities greater the maximum calculated deformation demands. The strength reduction factors are determined by the force that is being analyzed and the requirements set by the steel design standards.

$$Force\ Controlled\ Actions:\ \Phi Q_{CL} \geq\ Q_{UF}$$

Nonlinear Dynamic Procedure

The use of nonlinear dynamic procedure is used for any structure; there are no DCR or geometric irregularity limitations. A 3-dimensional model must be used to model the primary and/or secondary structural elements. If secondary elements are modeled there stiffness's must be ignored.

A gravity load combination is applied to the entire structure, Equation 9. For lateral loads that will be applied to the structure, Equation 10 shall be applied in each side of the building in combination with each gravity combination. For lateral loads that will be applied to the structure, Equation 10 all be applied in each side of the building in combination with each gravity combination. The loading procedure for the dynamic analysis is to start at zero load and proportionally increase the gravity and lateral loads to the entire model, the column has not been removed yet. Once the structure reaches equilibrium the column is removed instantaneously, the duration of the removal should be less than one tenth of the period. The analysis is continued until a maximum deflection is reached or one cycle of vertical motion occurs.

$$G_{ND} = \Omega_{ND}[(.9 \; or \; 1.2)D + (0.5L \; or \; 0.2S)] \quad Equation \; 9$$

$$L_{LAT} = .002 \sum P, P = gravity \; load \; acting \; on \; a \; floor \quad Equation \; 10$$

For element acceptance criteria the components are analyzed using the nonlinear procedures. Primary and secondary elements shall have deformation capacities greater the maximum calculated deformation demands. The strength reduction factors are determined by the force that is being analyzed and the requirements set by the steel design standards.

$$Force \; Controlled \; Actions: \; \Phi Q_{CL} \geq Q_{UF}$$

Cost and Schedule Comparison

A cost and schedule comparison of the original concrete design to the new design of structural steel, was created using RS Means 2007. The final contract cost for the project is $110 Million; however the final structural cost is unknown. Takeoffs for both structural systems were performed to compare the change in cost and the change in schedule. Construction began in 2007, which is why RS Means 2007 was chosen to perform the cost and schedule for this thesis.

Goals

1. Reduce the cost of the structure
2. Reduce the schedule of the structure
3. Compare results to actual market in DC

Cost Analysis

Detailed structural takeoffs were performed for the entire building for both designs. Concrete takeoffs were taken from the construction documents, and the steel takeoff was taken from the 3D RAM Model. More detail takeoffs of the structure can be found in Appendix H. RS Means 2007 was used for the cost takeoff to accurately portray the cost difference RS Means shows for the two systems.

Cost Summary		
	Cost	Cost per SF
Steel Design	$2,848,700.43	$22.93
Concrete Design	$3,472,186.16	$27.95

Figure 49: Cost Summary

A summary of the cost analysis is provided in Figure 49. IT was observed that the new steel design was cheaper than the concrete cost takeoff that I performed. For the new steel design it was determined the structural system would cost approximately $23 per square foot This value matched with the RS Means Assemblies value reported in previous thesis work, when the comparison of different floor systems. The Concrete design obtained a $28 per square foot cost. This value was increasingly higher than the value obtain from RS Means Assemblies, the difference can be attributed to the higher amount of formwork for the larger beams, because this cost report for an interior bay only and not an exterior bay.

Schedule Analysis

From the quantity takeoff performed for the cost study, the schedule of tasks was created. Using the recommended crews and the crew output data from RS Means, a detailed schedule breakdown was created using Excel. This schedule assumes a linear progression of takes, display that the next task will start directly at the completion of the previous task. Appendix H is provided for a detailed takeoff of the schedule breakdown. Microsoft Project was used to create a more accurate schedule to show how tasks can over lap during the construction process. A summary of the schedule comparison is provided in Figure 50.

Schedule Summary		
	# Days *	# Days **
Steel Design	300	133
Concrete Design	1071	456
* Assumes Linear Progression of Work		
** Assumse early start times on some taks		

Figure 50: Schedule Summary

It was observed that the new steel design could be constructed in a shorter duration than the original concrete design. In both schedules created the steel outperformed the concrete, but in the schedule using Microsoft Project it was observed the steel design could be complete in 133 days, and the concrete design took 456 days. The difference in schedule days is approximately 5 months and for a project that does not have a time constraint is not an issue.

It is important to note that local markets would vary from what RS Means shows as the cost and schedule breakdown. In the DC market the preferred method of construction is a two way flay slab. In this area the contractors are more efficient in erecting two way flat slab buildings. The cost and schedule for these systems will decrease with the increase experienced in the area. This effect can actually cause the concrete system to be more effective than a typical steel system. Also when there is a height restriction on the building, a two way flat slab system provides low floor to floor heights.

Mechanical Coordination

The impact of the new floor system is considered on the mechanical system, and new design of the mechanical duct is proposed. Originally the floor system depth at the critical duct location is 9 ½ inches. The new steel design places a W18 x 35 across the corridor, with a new floor system depth of 22 1/4". Figure 51 shows the location of the critical on the framing plans.

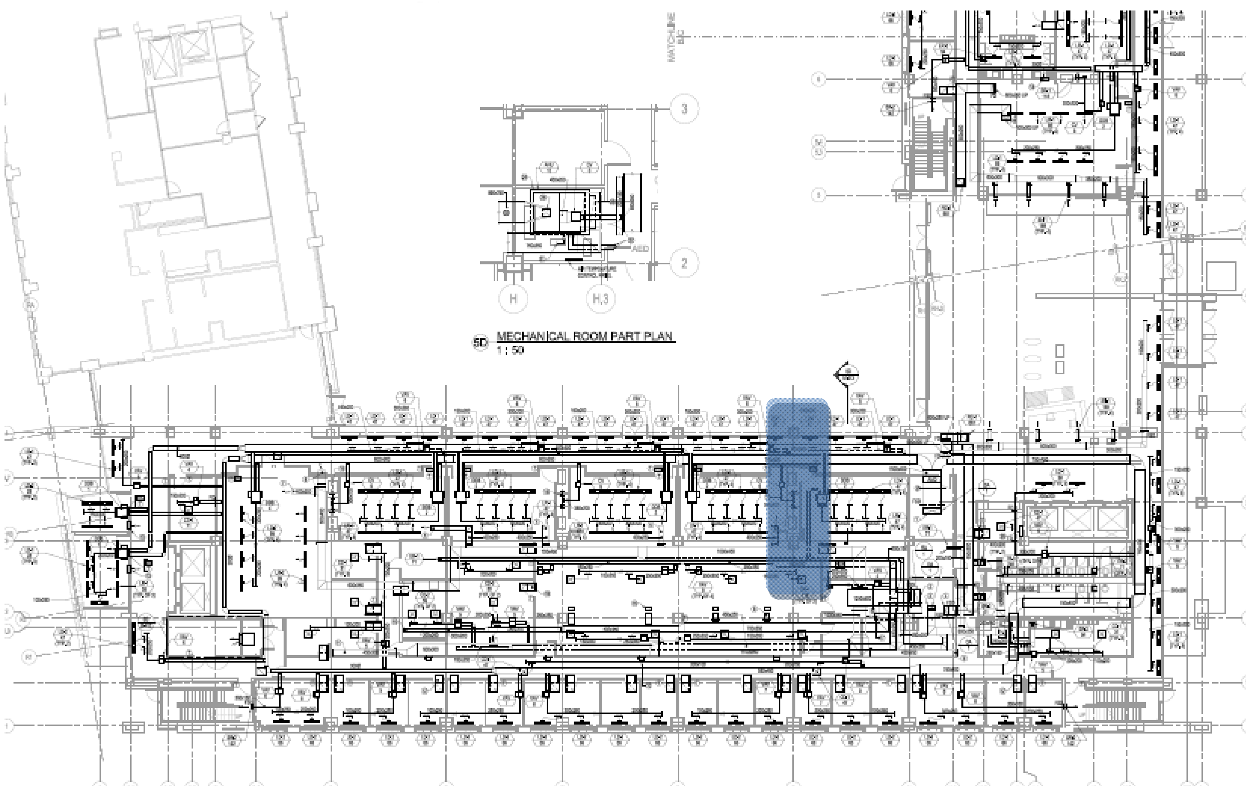


Figure 51: Critical Mechanical Location

Goals

1. Evaluate the existing mechanical system
2. Determine new design for mechanical ducts
3. Determine if new design is adequate

It was determined that the original mechanical duct design would not be able to fit under the new structure, and the mechanical ducts would need to be redesigned. Using the equivalent velocity method, the area of each duct was determined. Using the existing airflow passing through the two supply ducts, the total airflow was approximated. Assume a duct velocity of 20 feet per second, which is standard for a main supply duct, the required area for the duct was calculated. Figure 52 shows a summary of the mechanical design.

Mechanical Design			
	cfm	v (ft/s)	A (ft^2)
Duct 1	3385.97	20	2.82
Duct 2	2101.93	20	1.75

Figure 52: Design Summary

After looking at the allowable space between the bottom of the structural steel, to the top of the lighting system, the mechanical ducts were sized. Using the area found from the equal velocity method, the allowable height for each duct was determined, and the width was sized to match the allowable area, Figure 53 shows a summer of the design. Figure 54 is providing to show the cross section of the new design. Although the duct could be redesigned to fit in the new space, the friction changes for the new duct size were not taken into account. A deeper mechanical study including friction loss could be performed. Details of the redesign can be found in Appendix I.

Mechanical Design Summary			
		Width	Heigth
Original Design	Duct 1	29.53"	15.75"
	Duct 2	25.59"	11.81"
New Design	Duct 1	35.5"	12"
	Duct 2	24"	11"

Figure 53: Mechanical Design Table

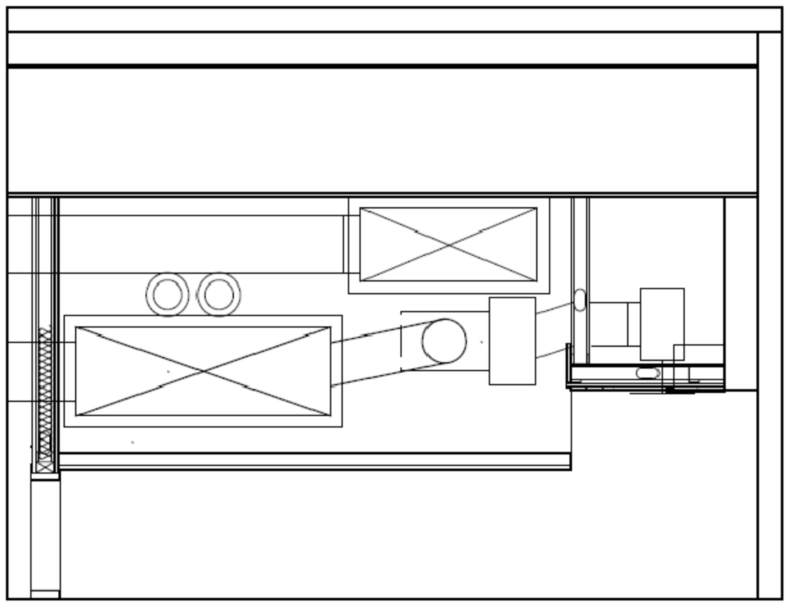


Figure 54: New Mechanical Design

Conclusion

The goal of this thesis is to study the effects of an entire structural redesign from impacts on the structure to cost and schedule. The structural system was redesigned using typical steel on metal deck floor system. The gravity system was designed using the ASCE 7-05 and AISC Steel Manual 13th Edition. IT was determined that the steel was more effective at spanning longer distances; however the depth of the floor system did conflict with the mechanical system.

The new lateral system design was more involved and required an iterative approach. Once the initial design was chosen it was analyzed and a better optimized. Braced frames around the core were designed to take the primary lateral load and to limit the deflection of the wing. Exterior moment frames were designed to aid in the resistance of progressive collapse and reduce the eccentric effects of the wing. Through the lateral analysis, a braced frame along Grid H was added to reduce the eccentric effects on the wing.

In structural steel design, the connections are very important in the transfer of the loads. Typical gravity connections were designed as a simple shear tab connection and an un-stiffened seated connection. Lateral connections were design as a moment connection for the moment frame, and a heavy braced connection for the braced frame.

As the threat for unforeseeable events increase, the design of structures to resist progressive collapse is becoming more important. The original structure was designed to resist progressive collapse, and part of this thesis was devoted to the research of the design of structures to resist progressive collapse. Two methods exist for the design to resist collapse, and both methods are presented in this thesis. The GSA method for progressive collapse was implemented for the moment frame along Grid 1.

The design procedure used was non-linear static because the instantaneous loss of column is non linear in nature. Using virtual work, and plastic hinge formations to form the collapse mechanism, the beams were sized. For the beam at level 2, the beam required to meet the acceptance criteria is W33 x 169. This size was exceedingly larger than the original design. It is apparent that the GSA Method and non linear method form a highly conservative method for progressive collapse.

Impacts on the cost and schedule were studied and compared to the original design and the new structure design. Both cost and schedules were created using RS Means Building Construction Data 2007. It was determined that the structural steel cost was $23 per square foot, and the original cost was $28 per square foot. It is important to note that the local market in DC favors two way flat slab constructions and the cost and schedule will be favored towards the original design.

The mechanical system was looked in respects to the ceiling to floor space where the mechanical ductwork passed through, the new structure design limited the space for the mechanical ducts and at a critical location, two of the ducts need to be resized. The first floor corridor was the most critical location for redesign because it had two ducts passing through the W18x35. The ducts were resized using the existing airflow through the system, and were sized to fit in the new adjusted space.

Acknowledgements

The author of this report wishes to recognize the following individuals for their assistance with this thesis study.

Turner Construction
Mike S. Stigliano

Tompkins Builders
Jeremy Wong

The Pennsylvania State University

Dr. Linda Hanagan
Dr. Louis Geschwindner
Dr. Ali Memari
M. Kevin Parfitt

A special thanks to family, friends and classmates for their support during the course of this year.

Appendix A: Wind Loads

Basic Wind Information				(ASCE Ref)
Basic Wind Speed	V =	90	mph	ASCE 7-05 Figure 6-1
Directionality Factor	k_d =	0.85		ASCE 7-05 Table 6-4
Importance Factor	I =	1.0		ASCE 7-05 Table 6-1
Exposure Category		B		ASCE 7-05 6.5.6
Topgraphic Factor	k_{zt} =	1.0		ASCE 7-05 6.5.7
	z_g =	1200	ft	
	α =	7		
Velocity Presure Expusure Coefficient evaluated at Height z	K_z =	Varies		
Velocity Presure Expusure Coefficient evaluated at Mean Roof Height	K_h =	0.8930		
Velocity Pressure at Height z	q_z =	Varies		
Velocity Pressure at Mean Roof Height	q_h =	15.7		
Equivalent height of Structure	h =	70.1		
Intensity of turbulance	I_z =	0.3		
Integral Length Scale of Turbulence	L_z =	347.0		
Background Response Factor (N/S)	Q =	0.778		
Background Reponse Factor (E/W)	Q =	0.829		
Gust Effect Factor (N/S)	G =	0.850		
Gust Effect Factor (E/W)	G =	0.850		
Internal Pressure Coefficients	G_{cpi} =	± 0.18		
External Pressure Coefficient (Windward)	C_p =	0.8		
External Pressure Coefficient (N/S Leeward)	C_p =	-0.3		
External Pressure Coefficient (E/W Leeward)	C_p =	-0.5		
External Pressure Coefficient (Sidewall)	C_p =	-0.7		
External Pressure Coefficient (Roof Section 1)	C_p =	-0.9	(From Windward Edget to 70.14 ft.)	
External Pressure Coefficient (Roof Section 2)	C_p =	-0.5	(From 70.14 to 140.28 ft.)	
External Pressure Coefficient (Roof Section 3)	C_p =	-0.3	(From 140.28 to 297.53 ft.)	
Basic Building Information				
Mean Building Height	h =	21379	mm	
		70.14	ft	
N-S	L =	137.44	ft	
	B =	297.55	ft	
E-W	L =	297.55	ft	
	B =	137.44	ft	

Design Wind Pressures p in N-S Direction							
Location	Story Height		Level Height		K_z	q_z (psf)	External Pressure qGCp (psf)
	(mm)	(ft)	(mm)	(ft)			
Windward	0	0	0	0			
	4700	15.420	4700	15.4199	0.5793	10.210	6.943
	3930	12.894	8630	28.3136	0.6891	12.146	8.259
	3930	12.894	12560	41.2073	0.7671	13.521	9.194
	3930	12.894	16490	54.1010	0.8291	14.614	9.938
	3930	12.894	20420	66.9948	0.8814	15.535	10.564
	959	3.146	21379	70.1411	0.8930	15.740	10.703
Leeward				All	0.8930	15.740	-4.014
Side				All	0.8930	15.740	-9.365
Roof	(From Windward Edget to 70.14 ft.)			70.1411	0.8930	15.740	-12.041
	(From 70.14 to 140.28 ft.)			70.1411	0.8930	15.740	-6.689

Design Wind Pressures p in E-W Direction							
Location	Story Height		Level Height		K_z	q_z (psf)	External Pressure qGCp (psf)
	(mm)	(ft)	(mm)	(ft)			
Windward	0	0	0	0			
	4700	15.420	4700	15.4199	0.5793	10.210	6.943
	3930	12.894	8630	28.3136	0.6891	12.146	8.259
	3930	12.894	12560	41.2073	0.7671	13.521	9.194
	3930	12.894	16490	54.1010	0.8291	14.614	9.938
	3930	12.894	20420	66.9948	0.8814	15.535	10.564
	959	3.146	21379	70.1411	0.8930	15.740	10.703
Leeward				All	0.8930	15.740	-4.014
Side				All	0.8930	15.740	-9.365
Roof	(From Windward Edget to 70.14 ft.)			70.1411	0.8930	15.740	-12.041
	(From 70.14 to 140.28 ft.)			70.1411	0.8930	15.740	-6.689
	(From 140.28 to 297.53 ft.)			70.1411	0.8930	15.740	-4.014

Design Wind Loads in N-S Direction				
	External Windward Load (kips)	External Leeward Loads (kips)	Base Shear (kips)	
			1.0W	1.6W
Level 1	0	0		
Level 2	31.771	16.907	**48.678**	**77.884**
Level 3	33.480	15.398	**48.878**	**78.205**
Level 4	36.700	15.398	**52.098**	**83.356**
level 5	39.327	15.398	**54.725**	**87.560**
Roof	25.274	9.578	**34.851**	**55.762**
Parapet	5.010	1.879	**6.889**	**11.022**
Base Shear			**246.119**	**393.790**

Design Wind Loads in E-W Direction				
	External Windward Load (kips)	External Leeward Loads (kips)	Base Shear (kips)	
			1.0W	1.6W
Level 1	0	0		
Level 2	14.675	7.809	**22.484**	**35.974**
Level 3	15.464	7.112	**22.576**	**36.122**
Level 4	16.951	7.112	**24.064**	**38.502**
level 5	18.165	7.112	**25.277**	**40.443**
Roof	11.674	4.424	**16.098**	**25.756**
Parapet	2.314	0.868	**3.182**	**5.091**
Base Shear			**113.680**	**181.888**

Wind Loads

1. Determine basic wind speed V from Fig 6-1 ASCE 7-05

   From Fig 6-1 ⇒ $V = 40$ m/s (90 mph)

2. Determine wind directionality factor K_d from Table 6-4

   By Table 6-4
   Buildings
   MWFRS $K_d = .85$
   C-C $K_d = .85$

3. Determine importance factor I from Table 6-1

   By Table 6-I, Occupancy II : $I = 1.00$

4. Determine exposure category (6.5.6)

   By ASCE 7-5 5.56.2 Surface Roughness Categories
   Surface Roughness B

   By ASCE 7-5 6.56.3 Exposure Categories
   Exposure B applies

5. Topographic Factor : ASCE 7-5 6.5.7

   Not all 5 conditions are met $K_{zt} = 1.0$

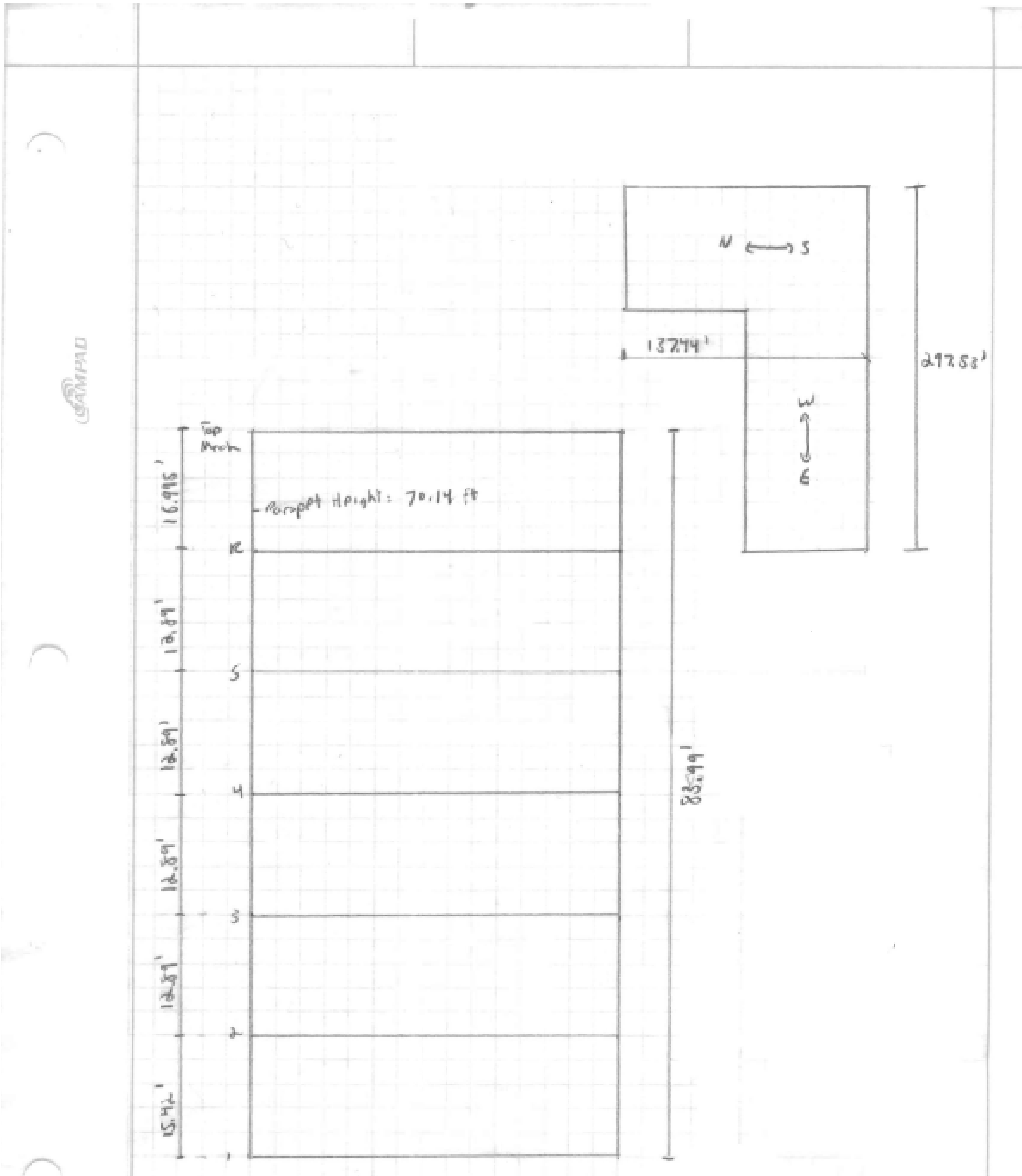
6. Determine velocity pressure exposure coefficents K_z and K_h
   From Table 6-2 w/ Exposure B $\alpha = 7$, $z_g = 1200$ ft

   From Table 6-3

   $$z_2 = 2.01\left(\frac{15.4199}{1200}\right)^{2/7} = .5793$$

   $$q_z = .00256(.5793)(.85)(1)(90)^2(1) = 10.21$$

   see excel for complete table


N ⟷ S
137.44'
247.58'
W
E
Top Mech
Parapet Height = 70.14 ft
16.995'
R
12.89'
5
12.89'
4
12.89'
3
12.89'
2
15.42'
1
88.99'

Method 2: Gust Effect Factors G and G_f

$B = 137.436'$
$L = 297.55'$
$h = 70.14'$

* Rigid is assume to be rigid, design as concentric braced frames

By 6.5.8.1
$G = .85$ is more conservative if $h/L < 4$ $\frac{70.14}{137.44} = .51 < 4$

$G = .85$

Determine Pressure coefficient C_p for the walls and roof from fig 6-6

For wind in the E-W direction

Windward wall: $C_p = .8$ for use w/ q_z
Leeward wall: $C_p = -.5$ for use w/ q_h
Sidewall: $C_p = -.7$ for use with q_h

For wind in the N-S direction

Windward wall: $C_p = .8$ for use w/ q_z
Leeward wall: $\frac{297.53}{137.436} = 2.16$ $C_p = -.2$ for use w/ q_h
Sidewall: $C_p = -.7$ for use with q_h

For Roof E-W, N-S

E-S $h/L = \frac{70.14}{137.44} = .51 \approx .5$ N-S $\frac{h}{L} = \frac{70.14}{297.53} = .24 < .25$

$C_p = -.9, \pm .18$ for windward edge to 70.14'
$C_p = -.5, \pm .18$ from 70.14 to 140.28'
$C_p = -.3, \pm .18$ from 140.28 to 297.55'

- Determine design wind press p_z and p_h

$p_z = q_z G C_p$ - $G C_{pi}$

Appendix B: Seismic Loads

Seismic Design Variables			(ASCE 7-05 Ref.)
Soil Classification		C	
Occupancy		II	(Table 1-1)
Structural System		Building Frame System: Ordinary reinforce concrete shear walls	(Table 12.2-1)
Spectral Response Acceleration, short	S_s	0.155	(USGS)
Spectral Response Acceleration, 1 s	S_1	0.05	(USGS)
Site Coefficient	Fa	1.2	(Table 11.4-1)
Site Coefficient	Fv	1.7	(Table 11.4-2)
Soil Modified Accelerationd, short	S_{ms}	0.186	(Eq. 11.4-1)
Soil Modified Accelerationd, 1 s	S_{m1}	0.085	(Eq. 11.4-2)
Design Spectral Acceleration, short	S_{DS}	0.124	(Eq. 11.4-3)
Design Spectral Acceleration, 1 s	S_{D1}	0.057	(Eq. 11.4-4)
Approximate Period Parameter	C_t	0.002	(Table 12.8-2)
Approximate Period Parameter	x	0.750	(Table 12.8-2)
Building height (above grade)	h_n	70.14 ft	
Approximate Fundamental Period	T_a	0.485	(Eq. 12.8-7)
Fundamental Period	T_s	0.460	
80% of Fundamental Period	$.8T_s$	0.368	
Seismic Design Category	S_{DC}	A	(Table 11.6-1)
Seismic Response Coefficient	C_s	0.012	(Eq 12.8-3)
Structure Period Exponet	k	1.250	(Sec. 12.8.3)
Seismic Base Shear	V	270.3 kips	(Eq. 12.8-1)

Seismic Loads					
Level	Story Weight w_x (kips)	Height h_x (ft)	Lateral Force F_x (Kips)	Base Shear (kips)	
2	1711.82	15.82	17.12		
3	1696.03	28.31	16.96		
4	1696.03	41.2	16.96		
5	1696.03	54.09	16.96		
Roof	2680.3	66.98	26.80		
			$\Sigma F_x = V_x =$	**95**	**kips**

Seismic Loads

Design Data

Location: Silver Spring, Md
Soil Classification: Site Class C
Occupancy: Office where less than 300 people congregate
Material: Structural A992 steel
Structural System: Concentric Braced Frames, Moment Frames

Seismic Ground Motion Values

1. Determine the mapped acceleration S_s and S_1
From the USGS Ground Motion Parameter Calculator

$S_s = .155$ $S_1 = .05$

2. Determine soil - modified acceleration S_{ms} and S_{m1}

$F_a = 1.2$ By Table 11.4-1
$F_v = 1.7$ By Table 11.4-2

$S_{ms} = 1.2(.155) = .186$
$S_{m1} = 1.7(.05) = .085$

3. Determine design acceleration

$S_{DS} = 2/3(.186) = .124$

$S_{D1} = 2/3(.085) = .057$

Determine the SDC

- $S_s = .124 \le .15$ $S_1 = .057 > .04$ ∴ cannot be automaticaly assigned into SDC A

- Occupancy Category: II

- Conditions in 11.6 are not met
↳ Determine SDC by Table 11.6-1, 11.6-2
- From Table 11.6-1 with $S_{DS} = .124$, Occupancy II ⇒ SDC = A

- From Table 11.6-2 with $S_{D1} = .057$, Occupancy II ⇒ SDC = A

Seismic Cont

11.7 Design Requirements for Seismic Design Category A

by 11.7.2 Lateral forces

$F_x = .01 w_x$ F_x: design lateral force applied at story x
w_x = portion of the total Dead Load of the structure at level x

Calculation of typical floor self weight

Floor Area = 20824 SF
Floor Perimeter = 809.674 ft

Floor Heights: 2nd - 14.16 ft
3rd-5th - 12.89 ft

Atrium Walk Area: 602 SF
Atrium Roof Area: 3711 SF

Weight of 2nd floor

Equiv weight of steel on metal deck floor system ≈ 50 psf
Includes slab, decking, Beams, Colums, Girders

DL: 50 psf
SDL: 15 psf

2nd Floor weight = 20824(65)/1000 = 1353.56 kips
Wall Perimeter = $809.674\left(\frac{14.16}{2} + \frac{12.86}{2}\right)$ 30 psf /1000 = 328.16 kips

Atrium Weight: 602(50)/1000 = 30.1 k

W_2 = 1711.82 kips

F_2 = 17.12 kips

Weight of 3rd-5th Floors Typical

Floor weight = 20824(65)/1000 = 1353.56 kips
Wall Perimeter = 809.674(12.86)30/1000 = 312.37 kips
Atrium weight = 602(50)/1000 30.1 k

W_{3-5} = 1696.03 kips

F_{3-5} = 16.96 kips

Weight of Roof

Roof weight = 20824(65 + 20 (Snow))/1000 = 1770.04 kips
Wall Perimeter = $809.674\left(\frac{12.86}{2} + 3.146\right)$ 30/1000 = 232.60 kips

Atrium Roof weight = 3711(65+20)/1000 = 315.44 kips

W_R = 2318.08 kips

Seismic Cont.

Weight Cont:

- Mechanical Penthouse 1

W Roof : $\frac{457.5(40)}{1000}$ = 18.3 Kips

walls : 88.582(14.659)(20) = 25.97 kips

Colmns / Beams = 4.99 kips

W_{P1} = 48.76 kips

- Mechanical Penthouse 2

W_{roof} : 356.96(40) = 14.278 kips

walls = 82.02(15.687)(20) = 25.63 kips

Columns / Beams = 4.725 Kips

W_{P2} = 44.63 kips

- Mechanical Penthouse 3

W_{roof} = 392.6(40) = 15.704 kips

walls = 79.4 (15.28)(20) = 24.265 kips

columns / Beams = 3.887 kips

W_{P3} = 43.86 kips

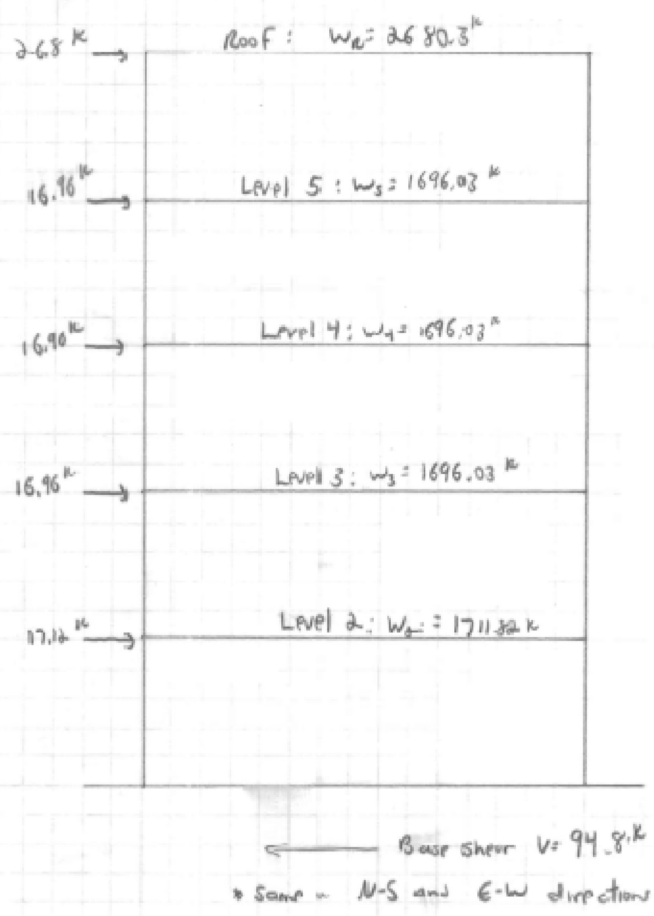
- Mechanical Unit

W = 1500(150) /1000 = 225 kips

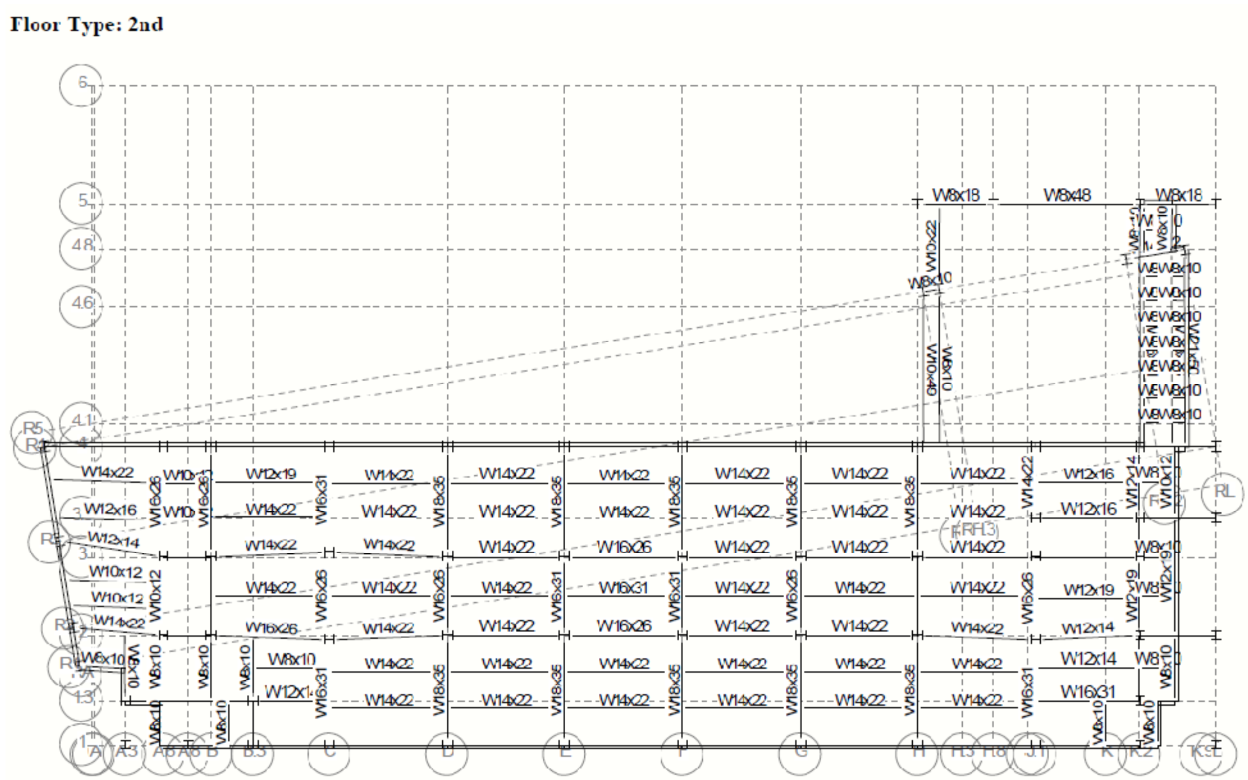
Total Roof = 2680.3 kips

F_R = 26.80 kips

Final Report

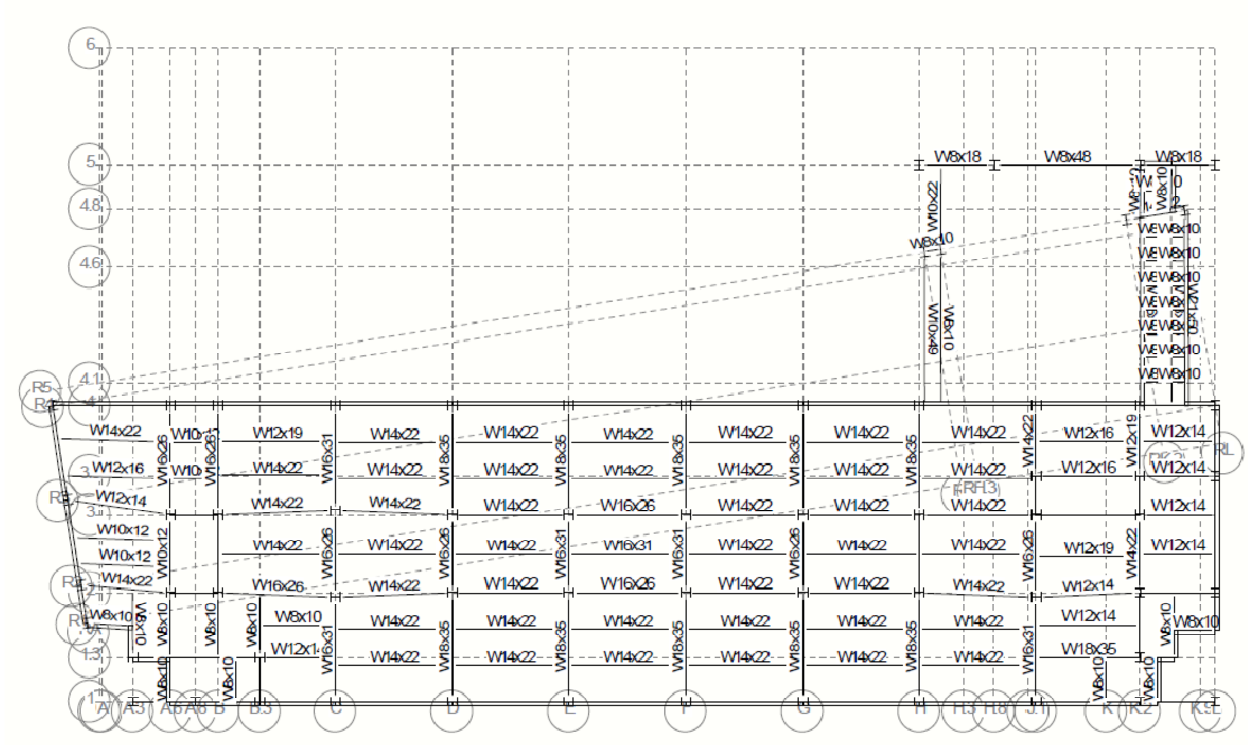


Appendix C: Steel Framing

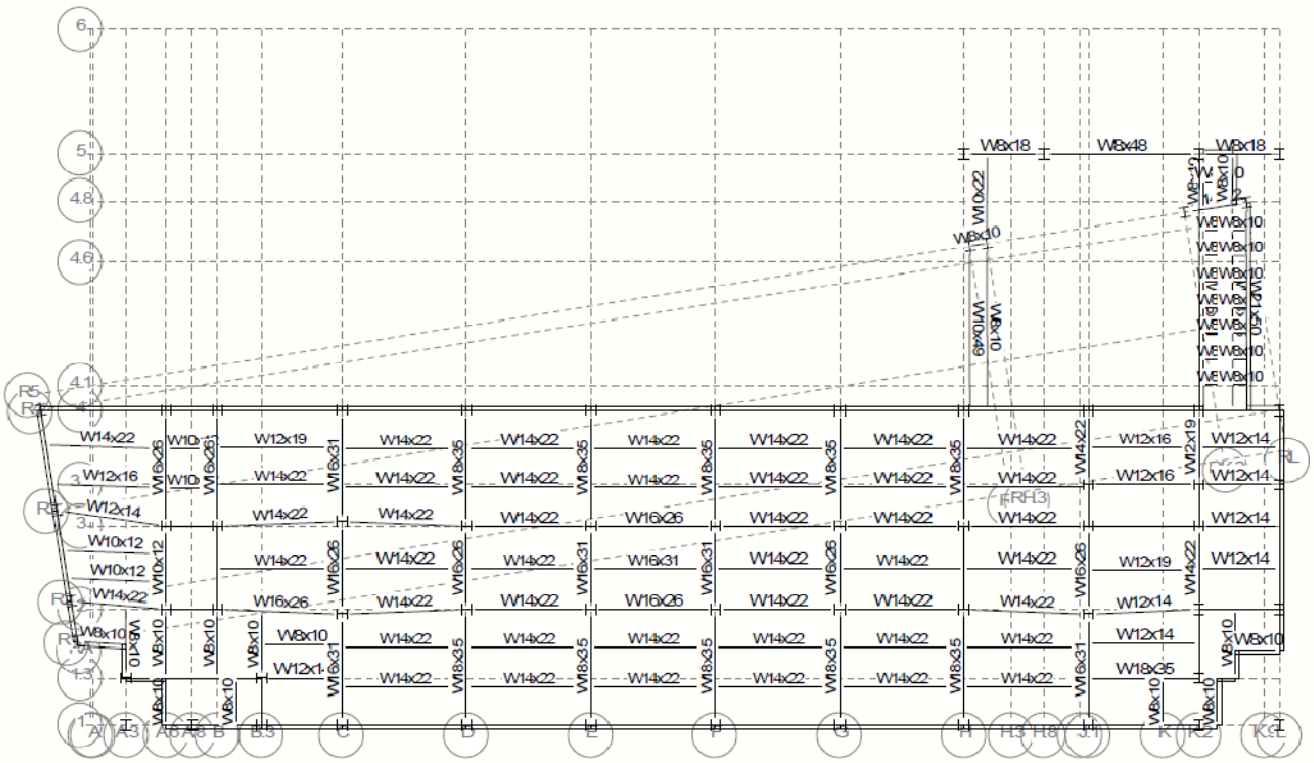


Final Report

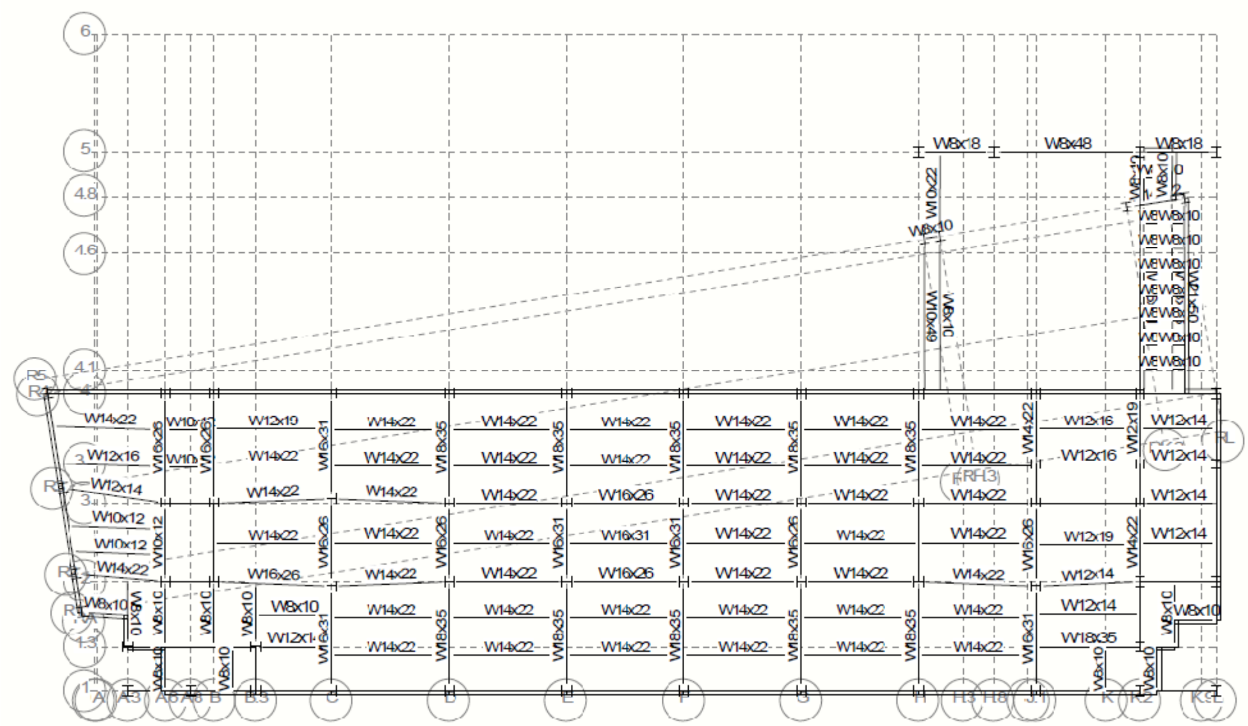
Floor Type: 3rd



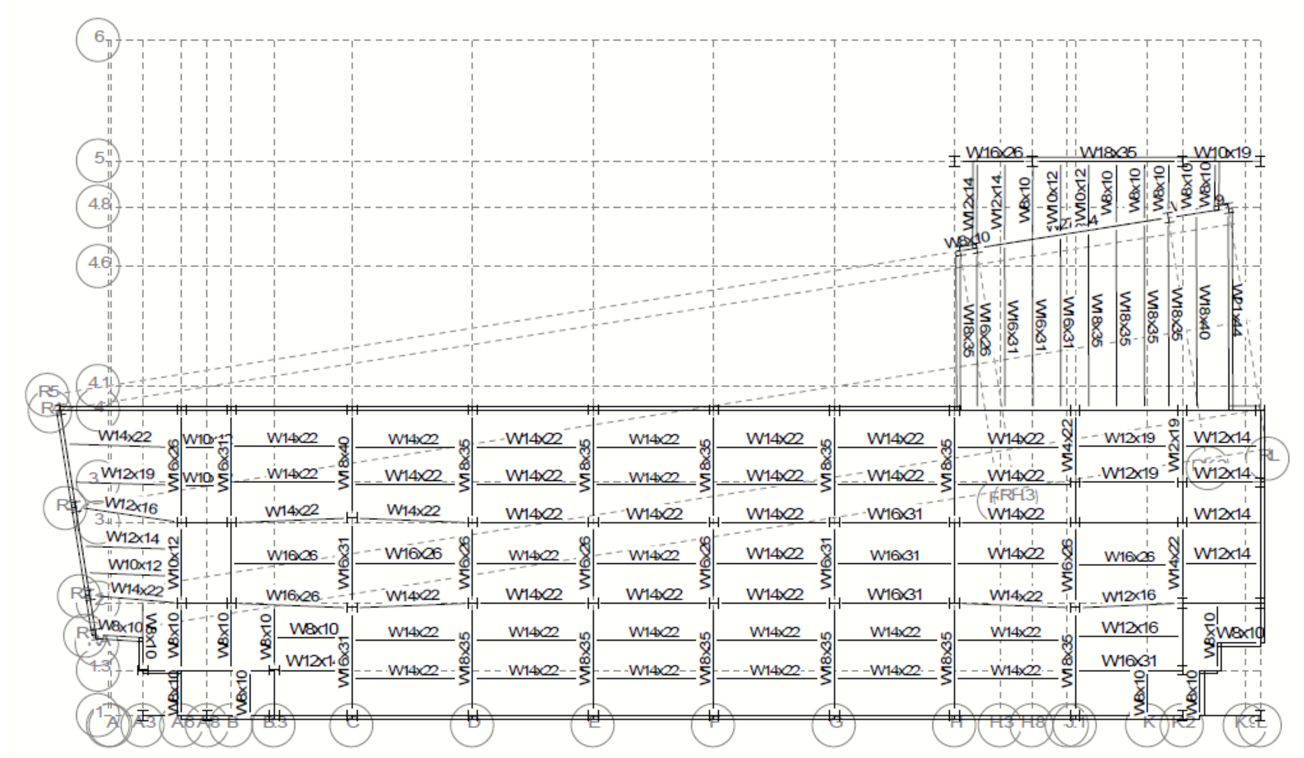
Floor Type: 4th



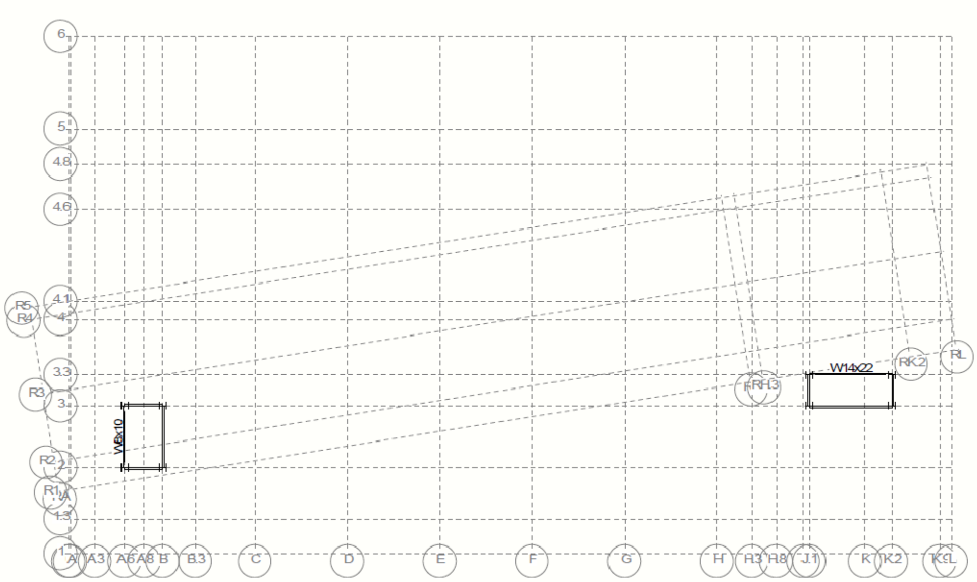
Floor Type: 5th



Floor Type: Roof



Floor Type: Penthouse



Gravity Column Design Summary

Column Line 59.66ft-26.41ft

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	63.8	2.3	7.6	6	0.57 Eq (H1-1a)	0.0	50	W10X33
Floor 5	131.9	2.0	3.4	3	0.69 Eq (H1-1a)	0.0	50	W10X39
Floor 4	186.5	1.9	4.6	6	0.96 Eq (H1-1a)	0.0	50	W10X39
Floor 3	252.3	1.9	3.8	3	0.75 Eq (H1-1a)	0.0	50	W10X54
Floor 2	310.6	0.4	4.1	6	0.96 Eq (H1-1a)	0.0	50	W10X54

Column Line 59.66ft-48.39ft

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	66.8	2.0	9.9	11	0.63 Eq (H1-1a)	0.0	50	W10X33
Floor 5	137.4	1.7	4.5	4	0.62 Eq (H1-1a)	0.0	50	W10X45
Floor 4	199.8	1.6	4.3	4	0.86 Eq (H1-1a)	0.0	50	W10X45
Floor 3	263.7	1.7	5.1	4	0.71 Eq (H1-1a)	0.0	50	W10X60
Floor 2	324.8	0.2	5.1	10	0.90 Eq (H1-1a)	0.0	50	W10X60

Column Line D-2

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	65.7	1.8	10.2	6	0.64 Eq (H1-1a)	0.0	50	W10X33
Floor 5	134.8	1.6	4.6	3	0.72 Eq (H1-1a)	0.0	50	W10X39
Floor 4	191.1	1.4	5.5	6	0.99 Eq (H1-1a)	0.0	50	W10X39
Floor 3	258.4	1.5	5.2	3	0.78 Eq (H1-1a)	0.0	50	W10X54
Floor 2	318.5	0.0	5.7	6	1.00 Eq (H1-1a)	0.0	50	W10X54

Column Line D-3

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	65.7	1.8	10.2	11	0.64 Eq (H1-1a)	0.0	50	W10X33
Floor 5	134.8	1.6	4.6	4	0.72 Eq (H1-1a)	0.0	50	W10X39
Floor 4	191.1	1.4	5.5	11	0.99 Eq (H1-1a)	0.0	50	W10X39
Floor 3	258.4	1.5	5.2	4	0.78 Eq (H1-1a)	0.0	50	W10X54
Floor 2	318.5	0.0	5.7	10	1.00 Eq (H1-1a)	0.0	50	W10X54

Column Line E-2

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	65.7	2.6	10.4	7	0.65 Eq (H1-1a)	0.0	50	W10X33
Floor 5	138.5	2.2	5.0	2	0.64 Eq (H1-1a)	0.0	50	W10X45
Floor 4	202.7	2.0	4.8	2	0.88 Eq (H1-1a)	0.0	50	W10X45
Floor 3	268.0	2.0	5.7	2	0.73 Eq (H1-1a)	0.0	50	W10X60
Floor 2	328.8	0.5	5.7	6	0.92 Eq (H1-1a)	0.0	50	W10X60

Column Line E-3

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	65.7	2.6	10.4	10	0.65 Eq (H1-1a)	0.0	50	W10X33
Floor 5	138.5	2.2	5.0	5	0.64 Eq (H1-1a)	0.0	50	W10X45

Floor 4	202.7	2.0	4.8	5	0.88 Eq (H1-1a)	0.0	50	W10X45
Floor 3	268.0	2.0	5.7	5	0.73 Eq (H1-1a)	0.0	50	W10X60
Floor 2	328.8	0.5	5.7	10	0.92 Eq (H1-1a)	0.0	50	W10X60

Column Line F-2

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	65.7	2.6	10.4	6	0.65 Eq (H1-1a)	0.0	50	W10X33
Floor 5	138.5	2.2	5.0	3	0.64 Eq (H1-1a)	0.0	50	W10X45
Floor 4	202.7	2.0	4.8	3	0.88 Eq (H1-1a)	0.0	50	W10X45
Floor 3	268.0	2.0	5.7	3	0.73 Eq (H1-1a)	0.0	50	W10X60
Floor 2	328.8	0.5	5.7	6	0.92 Eq (H1-1a)	0.0	50	W10X60

Column Line F-3

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	65.7	2.6	10.4	11	0.65 Eq (H1-1a)	0.0	50	W10X33
Floor 5	138.5	2.2	5.0	4	0.64 Eq (H1-1a)	0.0	50	W10X45
Floor 4	202.7	2.0	4.8	4	0.88 Eq (H1-1a)	0.0	50	W10X45
Floor 3	268.0	2.0	5.7	4	0.73 Eq (H1-1a)	0.0	50	W10X60
Floor 2	328.8	0.5	5.7	10	0.92 Eq (H1-1a)	0.0	50	W10X60

Column Line G-2

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	80.7	8.0	5.4	8	0.65 Eq (H1-1a)	0.0	50	W10X33
Floor 5	150.4	1.6	4.8	2	0.68 Eq (H1-1a)	0.0	50	W10X45
Floor 4	211.9	1.5	4.5	2	0.91 Eq (H1-1a)	0.0	50	W10X45
Floor 3	273.5	1.5	5.4	2	0.74 Eq (H1-1a)	0.0	50	W10X60
Floor 2	333.7	0.0	5.5	6	0.93 Eq (H1-1a)	0.0	50	W10X60

Column Line G-3

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	80.7	8.0	5.4	9	0.65 Eq (H1-1a)	0.0	50	W10X33
Floor 5	150.4	1.6	4.8	5	0.68 Eq (H1-1a)	0.0	50	W10X45
Floor 4	211.9	1.5	4.5	5	0.91 Eq (H1-1a)	0.0	50	W10X45
Floor 3	273.5	1.5	5.4	5	0.74 Eq (H1-1a)	0.0	50	W10X60
Floor 2	333.7	0.0	5.5	10	0.93 Eq (H1-1a)	0.0	50	W10X60

Column Line RH-R5

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	18.6	10.4	0.7	1	0.18 Eq (H1-1b)	99.0	50	W10X33
Floor 5	29.2	2.8	0.2	1	0.16 Eq (H1-1b)	99.0	50	W10X33
Floor 4	39.7	2.8	0.2	1	0.25 Eq (H1-1a)	99.0	50	W10X33
Floor 3	50.3	3.0	0.3	1	0.31 Eq (H1-1a)	99.0	50	W10X33
Floor 2	60.9	2.7	0.2	1	0.40 Eq (H1-1a)	99.0	50	W10X33

Column Line RH.3-R5

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	86.0	1.4	34.4	6	0.95 Eq (H1-1a)	99.0	50	W10X45
Floor 5	92.4	1.4	0.2	1	0.52 Eq (H1-1a)	99.0	50	W10X33
Floor 4	98.7	1.4	0.2	1	0.56 Eq (H1-1a)	99.0	50	W10X33
Floor 3	104.9	1.5	0.3	1	0.59 Eq (H1-1a)	99.0	50	W10X33
Floor 2	111.3	1.3	0.3	1	0.69 Eq (H1-1a)	99.0	50	W10X33

Column Line H.8-5

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	43.1	0.0	13.1	6	0.69 Eq (H1-1a)	90.0	50	W14X43
Floor 5	61.7	0.0	0.9	1	0.62 Eq (H1-1a)	90.0	50	W14X43
Floor 4	76.2	0.0	0.9	1	0.76 Eq (H1-1a)	90.0	50	W14X43
Floor 3	90.8	0.0	1.0	1	0.79 Eq (H1-1a)	90.0	50	W14X48
Floor 2	105.4	0.0	0.9	1	0.92 Eq (H1-1a)	90.0	50	W14X48

Column Line 236.82ft-26.41ft

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	61.1	2.4	9.7	6	0.60 Eq (H1-1a)	0.0	50	W10X33
Floor 5	125.2	2.1	4.4	3	0.68 Eq (H1-1a)	0.0	50	W10X39
Floor 4	181.8	2.0	4.1	3	0.93 Eq (H1-1a)	0.0	50	W10X39
Floor 3	238.7	2.0	4.9	3	0.73 Eq (H1-1a)	0.0	50	W10X54
Floor 2	294.2	0.7	5.0	6	0.92 Eq (H1-1a)	0.0	50	W10X54

Column Line RK.2-R5

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	93.3	9.5	35.4	10	0.91 Eq (H1-1a)	99.0	50	W14X68
Floor 5	111.4	13.0	4.4	1	0.81 Eq (H1-1a)	99.0	50	W14X61
Floor 4	125.0	15.8	4.3	1	0.91 Eq (H1-1a)	99.0	50	W14X61
Floor 3	142.0	16.4	4.6	1	0.91 Eq (H1-1a)	99.0	50	W14X68
Floor 2	155.8	20.6	4.2	1	1.00 Eq (H1-1a)	99.0	50	W14X68

Column Line K.2-5

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	36.1	0.0	11.6	10	0.59 Eq (H1-1a)	90.0	50	W14X43
Floor 5	57.7	0.0	0.5	4	0.57 Eq (H1-1a)	90.0	50	W14X43

Floor 4	76.1	0.0	0.5	4	0.75 Eq (H1-1a)	90.0	50	W14X43
Floor 3	94.4	0.0	0.5	4	0.82 Eq (H1-1a)	90.0	50	W14X48
Floor 2	112.8	0.0	0.3	1	0.97 Eq (H1-1a)	90.0	50	W14X48

Column Line RL-R5

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	44.7	16.0	9.8	1	0.64 Eq (H1-1a)	99.0	50	W10X33
Floor 5	82.8	5.6	6.1	1	0.66 Eq (H1-1a)	99.0	50	W10X33
Floor 4	120.0	5.5	5.8	1	0.86 Eq (H1-1a)	99.0	50	W10X33
Floor 3	156.9	5.8	6.0	1	0.75 Eq (H1-1a)	99.0	50	W10X45
Floor 2	193.5	5.3	7.7	1	0.99 Eq (H1-1a)	99.0	50	W10X45

Steel Beam Design Summary

Floor Type: Penthouse

Bm #	Length	+Mu	-Mu	Mn	Fy	Beam Size	Studs
	ft	kip-ft	kip-ft	kip-ft	ksi		
1	19.69	31.4	0.0	68.3	50.0	W8X10	6
7	26.25	145.0	0.0	210.9	50.0	W14X22	14

Floor Type: Roof

Bm #	Length	+Mu	-Mu	Mn	Fy	Beam Size	Studs
	ft	kip-ft	kip-ft	kip-ft	ksi		
3	28.53	186.6	0.0	220.2	50.0	W14X22	18
4	27.05	160.3	0.0	191.1	50.0	W12X19	21
6	26.36	143.6	0.0	173.3	50.0	W12X16	23
7	24.58	116.8	0.0	137.6	50.0	W12X14	16
8	23.55	92.7	0.0	113.9	50.0	W10X12	17
10	22.74	154.6	0.0	211.9	50.0	W14X22	7, 7
11	11.59	28.9	0.0	61.8	50.0	W8X10	6
12	17.22	54.3	0.0	68.3	50.0	W8X10	3, 3
15	11.15	3.4	0.0	68.0	50.0	W8X10	7
16	16.40	0.5	0.0	68.7	50.0	W8X10	6
17	19.69	116.2	0.0	137.8	50.0	W10X12	9, 2, 10
19	27.56	317.9	0.0	378.7	50.0	W16X26	19, 2, 21
21	12.14	36.1	0.0	52.5	50.0	W10X12	
22	12.14	36.2	0.0	52.5	50.0	W10X12	
25	16.40	0.5	0.0	68.7	50.0	W8X10	6
27	29.55	253.5	0.0	317.6	50.0	W16X26	11, 19
28	29.53	222.6	0.0	371.5	50.0	W16X26	44
29	27.56	339.1	0.0	408.0	50.0	W16X31	15, 2, 3, 1, 13
30	29.55	213.4	0.0	254.7	50.0	W14X22	28
31	29.53	199.5	0.0	239.5	50.0	W14X22	23
32	29.53	194.6	0.0	233.8	50.0	W14X22	20
34	11.15	0.2	0.0	68.2	50.0	W8X10	6
37	16.00	64.5	0.0	100.8	50.0	W8X10	16
38	19.09	93.1	0.0	114.5	50.0	W12X14	8
39	19.09	75.9	0.0	89.5	50.0	W8X10	18
40	26.41	365.8	0.0	433.0	50.0	W16X31	18, 4, 8, 2, 13
42	29.53	201.2	0.0	239.5	50.0	W14X22	22
43	29.53	196.2	0.0	233.8	50.0	W14X22	21
44	21.98	338.4	0.0	403.5	50.0	W16X31	34
45	29.55	206.9	0.0	244.7	50.0	W14X22	24
46	29.53	222.6	0.0	371.5	50.0	W16X26	44
47	26.41	417.4	0.0	563.1	50.0	W18X40	10, 1, 10, 1, 20
48	29.55	206.9	0.0	244.7	50.0	W14X22	24

Bm #	Length	+Mu	-Mu	Mn	Fy	Beam Size	Studs
49	29.53	196.2	0.0	233.8	50.0	W14X22	21
50	29.53	201.2	0.0	239.5	50.0	W14X22	22
52	27.56	442.9	0.0	525.0	50.0	W18X35	25, 2, 24
54	29.53	201.2	0.0	239.5	50.0	W14X22	22
55	29.53	201.2	0.0	239.5	50.0	W14X22	22
56	19.69	289.3	0.0	341.2	50.0	W16X26	30
57	29.53	206.7	0.0	244.7	50.0	W14X22	24
58	29.53	212.3	0.0	254.7	50.0	W14X22	28
59	27.56	442.9	0.0	525.0	50.0	W18X35	24, 2, 25
60	29.53	206.7	0.0	244.7	50.0	W14X22	24
61	29.53	201.2	0.0	239.5	50.0	W14X22	22
62	29.53	201.2	0.0	239.5	50.0	W14X22	22
64	27.56	444.5	0.0	529.4	50.0	W18X35	25, 2, 25
66	29.53	201.2	0.0	239.5	50.0	W14X22	22
67	29.53	201.2	0.0	239.5	50.0	W14X22	22
68	19.69	284.6	0.0	336.0	50.0	W16X26	28
69	29.53	206.7	0.0	244.7	50.0	W14X22	24
70	29.53	212.3	0.0	254.7	50.0	W14X22	28
71	27.56	444.5	0.0	529.4	50.0	W18X35	25, 2, 25
72	29.53	206.7	0.0	244.7	50.0	W14X22	24
73	29.53	201.2	0.0	239.5	50.0	W14X22	22
74	29.53	201.2	0.0	239.5	50.0	W14X22	22
76	27.56	444.5	0.0	529.4	50.0	W18X35	25, 2, 25
78	29.53	201.2	0.0	239.5	50.0	W14X22	22
79	29.53	201.2	0.0	239.5	50.0	W14X22	22
80	19.69	284.6	0.0	336.0	50.0	W16X26	28
81	29.53	206.7	0.0	244.7	50.0	W14X22	24
82	29.53	212.3	0.0	254.7	50.0	W14X22	28
83	27.56	444.5	0.0	529.4	50.0	W18X35	25, 2, 25
84	29.53	206.7	0.0	244.7	50.0	W14X22	24
85	29.53	201.2	0.0	239.5	50.0	W14X22	22
86	29.53	201.2	0.0	239.5	50.0	W14X22	22
88	27.56	444.5	0.0	529.4	50.0	W18X35	25, 2, 25
90	29.53	201.2	0.0	239.5	50.0	W14X22	22
91	29.53	201.2	0.0	239.5	50.0	W14X22	22
92	19.69	333.4	0.0	396.0	50.0	W16X31	32
93	29.53	241.7	0.0	376.5	50.0	W16X31	30
94	29.53	292.1	0.0	447.1	50.0	W16X31	50
95	27.56	444.5	0.0	529.4	50.0	W18X35	25, 2, 25
96	29.53	241.7	0.0	376.5	50.0	W16X31	30
97	29.53	201.2	0.0	239.5	50.0	W14X22	22
98	29.53	201.2	0.0	239.5	50.0	W14X22	22
100	27.56	442.9	0.0	525.0	50.0	W18X35	25, 2, 24
102	29.53	201.2	0.0	239.5	50.0	W14X22	22
103	29.53	196.2	0.0	233.8	50.0	W14X22	21

Bm #	Length	+Mu	-Mu	Mn	Fy	Beam Size	Studs
105	29.55	206.9	0.0	244.7	50.0	W14X22	24
106	29.53	217.2	0.0	254.7	50.0	W14X22	29
107	27.56	444.5	0.0	529.4	50.0	W18X35	25, 2, 25
108	29.53	206.7	0.0	244.7	50.0	W14X22	24
109	29.53	201.2	0.0	239.5	50.0	W14X22	22
110	29.53	201.2	0.0	239.5	50.0	W14X22	22
112	18.91	120.2	0.0	184.2	50.0	W16X26	
113	38.95	225.8	0.0	392.2	50.0	W18X35	14
114	4.12	1.6	0.0	58.6	50.0	W8X10	5
115	39.59	228.5	0.0	292.6	50.0	W16X26	20
116	21.28	82.4	0.0	114.3	50.0	W12X14	7
117	47.54	1355.0	0.0	1600.0	50.0	W27X84	90
118	40.68	286.7	0.0	377.5	50.0	W16X31	32
119	20.19	74.5	0.0	114.6	50.0	W12X14	8
120	36.86	381.4	0.0	485.1	50.0	W18X35	40
121	41.76	300.5	0.0	387.1	50.0	W16X31	36
122	19.11	66.5	0.0	79.5	50.0	W8X10	12
123	42.84	314.7	0.0	445.5	50.0	W16X31	56
124	18.03	59.2	0.0	89.7	50.0	W10X12	8
125	26.41	390.0	0.0	531.6	50.0	W18X35	22, 5, 13, 2, 12
127	26.25	124.5	-186.5	368.5	50.0	W16X31	31
	8.67	0.0	-186.5				
128	26.25	144.7	0.0	173.3	50.0	W12X16	23
129	20.83	288.8	0.0	342.3	50.0	W16X26	15, 1, 15
130	26.27	156.2	0.0	183.0	50.0	W12X16	26
131	26.25	176.2	0.0	276.8	50.0	W16X26	17
134	17.72	224.4	0.0	269.1	50.0	W14X22	14, 14
135	26.25	164.8	0.0	196.1	50.0	W12X19	22
136	26.25	158.1	0.0	191.0	50.0	W12X19	20
138	43.92	330.3	0.0	437.0	50.0	W18X35	28
139	16.95	52.3	0.0	89.6	50.0	W10X12	8
140	45.01	345.1	0.0	450.9	50.0	W18X35	32
141	15.86	45.8	0.0	68.6	50.0	W8X10	8
142	46.09	341.9	0.0	466.6	50.0	W18X35	38
143	14.78	37.2	0.0	62.1	50.0	W8X10	6
144	11.15	0.2	0.0	68.2	50.0	W8X10	6
145	47.03	354.5	0.0	471.1	50.0	W18X35	40
146	13.84	32.5	0.0	62.1	50.0	W8X10	6
147	14.87	179.2	0.0	214.9	50.0	W12X19	13, 7, 6
150	19.69	226.5	0.0	271.0	50.0	W14X22	14, 1, 13
152	19.03	95.0	0.0	114.5	50.0	W12X14	8
154	19.03	95.0	0.0	114.5	50.0	W12X14	8
155	17.72	190.8	0.0	225.8	50.0	W12X19	28
156	19.03	90.3	0.0	114.5	50.0	W12X14	8

Bm #	Length	+Mu	-Mu	Mn	Fy	Beam Size	Studs
157	19.03	85.6	0.0	114.5	50.0	W12X14	8
159	19.03	63.9	0.0	90.0	50.0	W10X19	
160	48.11	413.6	0.0	563.2	50.0	W18X40	52
161	12.76	24.6	0.0	62.0	50.0	W8X10	6
162	11.15	3.4	0.0	68.0	50.0	W8X10	7
163	12.04	21.8	0.0	68.0	50.0	W8X10	6
164	16.40	45.3	0.0	68.2	50.0	W8X10	3, 3
165	10.35	23.4	0.0	61.8	50.0	W8X10	6
166	49.36	487.9	0.0	680.6	50.0	W21X44	46

Floor Type: 5th

Bm #	Length ft	+Mu kip-ft	-Mu kip-ft	Mn kip-ft	Fy ksi	Beam Size	Studs
3	28.53	172.0	0.0	212.4	50.0	W14X22	15
4	27.05	147.7	0.0	178.4	50.0	W12X16	25
6	26.36	132.6	0.0	159.1	50.0	W12X14	23
7	24.58	107.8	0.0	128.8	50.0	W10X12	23
8	23.55	85.8	0.0	103.3	50.0	W10X12	13
10	22.74	143.8	0.0	211.9	50.0	W14X22	7, 7
11	11.59	27.3	0.0	61.8	50.0	W8X10	6
12	17.22	51.9	0.0	68.3	50.0	W8X10	3, 3
15	11.15	3.2	0.0	68.0	50.0	W8X10	7
16	16.40	0.5	0.0	68.7	50.0	W8X10	6
17	19.69	107.4	0.0	132.1	50.0	W10X12	8, 2, 9
19	27.56	291.1	0.0	346.8	50.0	W16X26	13, 2, 15
21	12.14	33.4	0.0	52.5	50.0	W10X12	
22	12.14	33.5	0.0	52.5	50.0	W10X12	
25	16.40	0.5	0.0	68.7	50.0	W8X10	6
27	29.55	232.3	0.0	277.1	50.0	W16X26	8, 9
28	29.53	203.9	0.0	244.7	50.0	W14X22	25
29	27.56	309.9	0.0	368.7	50.0	W16X26	17, 2, 3, 1, 16
30	29.55	196.1	0.0	233.8	50.0	W14X22	20
31	29.53	183.7	0.0	220.2	50.0	W14X22	17
32	29.53	178.8	0.0	211.7	50.0	W12X19	28
34	11.15	0.2	0.0	68.2	50.0	W8X10	6
37	16.00	59.7	0.0	88.9	50.0	W8X10	12
38	19.09	86.1	0.0	114.5	50.0	W12X14	8
39	19.09	70.2	0.0	84.6	50.0	W8X10	15
40	26.41	334.1	0.0	401.6	50.0	W16X31	13, 3, 6, 1, 9
42	29.53	185.1	0.0	220.2	50.0	W14X22	16
43	29.53	180.7	0.0	220.2	50.0	W14X22	17
44	21.98	309.7	0.0	367.5	50.0	W16X26	40
45	29.55	190.3	0.0	227.4	50.0	W14X22	18
46	29.53	203.9	0.0	244.7	50.0	W14X22	25
47	26.41	379.0	0.0	449.7	50.0	W16X31	21, 2, 3, 2, 24

Bm #	Length	+Mu	-Mu	Mn	Fy	Beam Size	Studs
48	29.55	190.3	0.0	227.4	50.0	W14X22	18
49	29.53	180.7	0.0	220.2	50.0	W14X22	17
50	29.53	185.1	0.0	220.2	50.0	W14X22	16
52	27.56	403.2	0.0	478.2	50.0	W18X35	16, 2, 16
54	29.53	185.1	0.0	220.2	50.0	W14X22	16
55	29.53	185.1	0.0	220.2	50.0	W14X22	16
56	19.69	265.6	0.0	312.9	50.0	W16X26	20
57	29.53	190.2	0.0	227.4	50.0	W14X22	18
58	29.53	195.2	0.0	233.8	50.0	W14X22	20
59	27.56	403.2	0.0	478.2	50.0	W18X35	16, 2, 16
60	29.53	190.2	0.0	227.4	50.0	W14X22	18
61	29.53	185.1	0.0	220.2	50.0	W14X22	16
62	29.53	185.1	0.0	220.2	50.0	W14X22	16
64	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
66	29.53	185.1	0.0	220.2	50.0	W14X22	16
67	29.53	185.1	0.0	220.2	50.0	W14X22	16
68	19.69	339.7	0.0	400.9	50.0	W16X31	34
69	29.53	249.2	0.0	384.0	50.0	W16X26	46
70	29.53	311.7	0.0	421.9	50.0	W16X31	42
71	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
72	29.53	249.2	0.0	384.0	50.0	W16X26	46
73	29.53	185.1	0.0	220.2	50.0	W14X22	16
74	29.53	185.1	0.0	220.2	50.0	W14X22	16
76	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
78	29.53	185.1	0.0	220.2	50.0	W14X22	16
79	29.53	185.1	0.0	220.2	50.0	W14X22	16
80	19.69	339.7	0.0	400.9	50.0	W16X31	34
81	29.53	190.2	0.0	227.4	50.0	W14X22	18
82	29.53	195.2	0.0	233.8	50.0	W14X22	20
83	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
84	29.53	190.2	0.0	227.4	50.0	W14X22	18
85	29.53	185.1	0.0	220.2	50.0	W14X22	16
86	29.53	185.1	0.0	220.2	50.0	W14X22	16
88	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
90	29.53	185.1	0.0	220.2	50.0	W14X22	16
91	29.53	185.1	0.0	220.2	50.0	W14X22	16
92	19.69	261.7	0.0	312.9	50.0	W16X26	20
93	29.53	190.2	0.0	227.4	50.0	W14X22	18
94	29.53	195.2	0.0	233.8	50.0	W14X22	20
95	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
96	29.53	190.2	0.0	227.4	50.0	W14X22	18
97	29.53	185.1	0.0	220.2	50.0	W14X22	16
98	29.53	185.1	0.0	220.2	50.0	W14X22	16
100	27.56	403.2	0.0	478.2	50.0	W18X35	16, 2, 16
102	29.53	185.1	0.0	220.2	50.0	W14X22	16

Bm #	Length	+Mu	-Mu	Mn	Fy	Beam Size	Studs
103	29.53	180.7	0.0	220.2	50.0	W14X22	17
105	29.55	190.3	0.0	227.4	50.0	W14X22	18
106	29.53	199.5	0.0	239.5	50.0	W14X22	23
107	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
108	29.53	190.2	0.0	227.4	50.0	W14X22	18
109	29.53	185.1	0.0	220.2	50.0	W14X22	16
110	29.53	185.1	0.0	220.2	50.0	W14X22	16
112	18.91	43.3	0.0	70.8	50.0	W8X18 2	
113	38.95	125.8	0.0	251.7	50.0	W10X49	
114	4.12	1.3	0.0	37.0	50.0	W8X10 2	
115	39.59	2.8	0.0	37.0	50.0	W8X10	
116	21.28	35.4	0.0	108.3	50.0	W10X22 2	
117	36.86	112.5	0.0	204.2	50.0	W8X48 2	
118	26.41	353.3	0.0	423.2	50.0	W16X31	16, 4, 6, 2, 12
120	26.25	111.8	-177.1	402.5	50.0	W18X35	24
	8.67	0.0	-177.1				
121	26.25	133.6	0.0	159.1	50.0	W12X14	23
122	20.83	265.2	0.0	313.4	50.0	W16X26	10, 1, 10
123	26.27	144.0	0.0	168.7	50.0	W12X14	26
124	26.25	161.5	0.0	191.0	50.0	W12X19	21
127	17.72	206.7	0.0	247.6	50.0	W14X22	10, 10
128	26.25	151.6	0.0	183.0	50.0	W12X16	26
129	26.25	145.5	0.0	173.3	50.0	W12X16	22
131	11.15	0.2	0.0	68.2	50.0	W8X10	6
132	14.87	118.8	0.0	138.3	50.0	W14X22	
135	19.69	208.7	0.0	248.5	50.0	W14X22	10, 1, 10
137	19.03	87.9	0.0	114.5	50.0	W12X14	8
139	19.03	87.9	0.0	114.5	50.0	W12X14	8
140	17.72	176.3	0.0	210.3	50.0	W12X19	22
141	19.03	83.6	0.0	114.5	50.0	W12X14	8
142	19.03	79.2	0.0	114.5	50.0	W12X14	8
144	19.03	56.8	0.0	70.8	50.0	W8X18 2	
145	47.74	259.1	0.0	388.6	50.0	W16X31	32
146	13.13	18.6	0.0	68.1	50.0	W8X10	6
147	6.89	6.8	0.0	37.0	50.0	W8X10	
148	6.89	7.2	0.0	37.0	50.0	W8X10	
149	6.89	7.2	0.0	37.0	50.0	W8X10	
150	6.89	7.2	0.0	37.0	50.0	W8X10	
151	6.88	7.3	0.0	37.0	50.0	W8X10	
152	6.89	7.2	0.0	37.0	50.0	W8X10	
153	6.89	7.2	0.0	37.0	50.0	W8X10	
154	6.89	7.1	0.0	37.0	50.0	W8X10	
155	11.15	3.2	0.0	68.0	50.0	W8X10	7
156	48.83	307.0	0.0	461.8	50.0	W18X35	28
157	12.04	25.4	0.0	68.0	50.0	W8X10	3, 3

Bm #	Length	+Mu	-Mu	Mn	Fy	Beam Size	Studs
158	3.35	1.7	0.0	37.0	50.0	W8X10	
159	3.35	1.7	0.0	37.0	50.0	W8X10	
160	3.35	1.7	0.0	37.0	50.0	W8X10	
161	3.35	1.7	0.0	37.0	50.0	W8X10	
162	3.35	1.7	0.0	37.0	50.0	W8X10	
163	3.35	1.7	0.0	37.0	50.0	W8X10	
164	3.35	1.7	0.0	37.0	50.0	W8X10	
165	16.40	43.3	0.0	68.2	50.0	W8X10	3, 3
166	10.35	22.1	0.0	61.8	50.0	W8X10	6
167	49.36	332.6	0.0	646.9	50.0	W21X50	23

Floor Type: 4th

Bm #	Length ft	+Mu kip-ft	-Mu kip-ft	Mn kip-ft	Fy ksi	Beam Size	Studs
3	28.53	172.0	0.0	212.4	50.0	W14X22	15
4	27.05	147.7	0.0	178.4	50.0	W12X16	25
6	26.36	132.6	0.0	159.1	50.0	W12X14	23
7	24.58	107.8	0.0	128.8	50.0	W10X12	23
8	23.55	85.8	0.0	103.3	50.0	W10X12	13
10	22.74	143.8	0.0	211.9	50.0	W14X22	7, 7
11	11.59	27.3	0.0	61.8	50.0	W8X10	6
12	17.22	51.9	0.0	68.3	50.0	W8X10	3, 3
14	11.15	3.2	0.0	68.0	50.0	W8X10	7
16	16.40	0.5	0.0	68.7	50.0	W8X10	6
17	19.69	107.4	0.0	132.1	50.0	W10X12	8, 2, 9
19	27.56	291.1	0.0	346.8	50.0	W16X26	13, 2, 15
21	12.14	33.4	0.0	52.5	50.0	W10X12	
22	12.14	33.5	0.0	52.5	50.0	W10X12	
24	16.40	0.5	0.0	68.7	50.0	W8X10	6
26	29.55	232.3	0.0	277.1	50.0	W16X26	8, 9
27	29.53	203.9	0.0	244.7	50.0	W14X22	25
28	27.56	309.9	0.0	368.7	50.0	W16X26	17, 2, 3, 1, 16
29	29.55	196.1	0.0	233.8	50.0	W14X22	20
30	29.53	183.7	0.0	220.2	50.0	W14X22	17
31	29.53	178.8	0.0	211.7	50.0	W12X19	28
33	11.15	0.2	0.0	68.2	50.0	W8X10	6
36	16.00	59.7	0.0	88.9	50.0	W8X10	12
37	19.09	86.1	0.0	114.5	50.0	W12X14	8
38	19.09	70.2	0.0	84.6	50.0	W8X10	15
39	26.41	334.1	0.0	401.6	50.0	W16X31	13, 3, 6, 1, 9
41	29.53	185.1	0.0	220.2	50.0	W14X22	16
42	29.53	180.7	0.0	220.2	50.0	W14X22	17
43	21.98	309.7	0.0	367.5	50.0	W16X26	40
44	29.55	190.3	0.0	227.4	50.0	W14X22	18
45	29.53	203.9	0.0	244.7	50.0	W14X22	25

Bm #	Length	+Mu	-Mu	Mn	Fy	Beam Size	Studs
46	26.41	379.0	0.0	449.7	50.0	W16X31	21, 2, 3, 2, 24
47	29.55	190.3	0.0	227.4	50.0	W14X22	18
48	29.53	180.7	0.0	220.2	50.0	W14X22	17
49	29.53	185.1	0.0	220.2	50.0	W14X22	16
51	27.56	403.2	0.0	478.2	50.0	W18X35	16, 2, 16
53	29.53	185.1	0.0	220.2	50.0	W14X22	16
54	29.53	185.1	0.0	220.2	50.0	W14X22	16
55	19.69	265.6	0.0	312.9	50.0	W16X26	20
56	29.53	190.2	0.0	227.4	50.0	W14X22	18
57	29.53	195.2	0.0	233.8	50.0	W14X22	20
58	27.56	403.2	0.0	478.2	50.0	W18X35	16, 2, 16
59	29.53	190.2	0.0	227.4	50.0	W14X22	18
60	29.53	185.1	0.0	220.2	50.0	W14X22	16
61	29.53	185.1	0.0	220.2	50.0	W14X22	16
63	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
65	29.53	185.1	0.0	220.2	50.0	W14X22	16
66	29.53	185.1	0.0	220.2	50.0	W14X22	16
67	19.69	339.7	0.0	400.9	50.0	W16X31	34
68	29.53	249.2	0.0	384.0	50.0	W16X26	46
69	29.53	311.7	0.0	421.9	50.0	W16X31	42
70	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
71	29.53	249.2	0.0	384.0	50.0	W16X26	46
72	29.53	185.1	0.0	220.2	50.0	W14X22	16
73	29.53	185.1	0.0	220.2	50.0	W14X22	16
75	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
77	29.53	185.1	0.0	220.2	50.0	W14X22	16
78	29.53	185.1	0.0	220.2	50.0	W14X22	16
79	19.69	339.7	0.0	400.9	50.0	W16X31	34
80	29.53	190.2	0.0	227.4	50.0	W14X22	18
81	29.53	195.2	0.0	233.8	50.0	W14X22	20
82	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
83	29.53	190.2	0.0	227.4	50.0	W14X22	18
84	29.53	185.1	0.0	220.2	50.0	W14X22	16
85	29.53	185.1	0.0	220.2	50.0	W14X22	16
87	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
89	29.53	185.1	0.0	220.2	50.0	W14X22	16
90	29.53	185.1	0.0	220.2	50.0	W14X22	16
91	19.69	261.7	0.0	312.9	50.0	W16X26	20
92	29.53	190.2	0.0	227.4	50.0	W14X22	18
93	29.53	195.2	0.0	233.8	50.0	W14X22	20
94	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
95	29.53	190.2	0.0	227.4	50.0	W14X22	18
96	29.53	185.1	0.0	220.2	50.0	W14X22	16
97	29.53	185.1	0.0	220.2	50.0	W14X22	16
99	27.56	403.2	0.0	478.2	50.0	W18X35	16, 2, 16

Bm #	Length	+Mu	-Mu	Mn	Fy	Beam Size	Studs
101	29.53	185.1	0.0	220.2	50.0	W14X22	16
102	29.53	180.7	0.0	220.2	50.0	W14X22	17
104	29.55	190.3	0.0	227.4	50.0	W14X22	18
105	29.53	199.5	0.0	239.5	50.0	W14X22	23
106	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
107	29.53	190.2	0.0	227.4	50.0	W14X22	18
108	29.53	185.1	0.0	220.2	50.0	W14X22	16
109	29.53	185.1	0.0	220.2	50.0	W14X22	16
111	18.91	43.3	0.0	70.8	50.0	W8X18 2	
112	38.95	125.8	0.0	251.7	50.0	W10X49	
113	4.12	1.3	0.0	37.0	50.0	W8X10 2	
114	39.59	2.8	0.0	37.0	50.0	W8X10	
115	21.28	35.4	0.0	108.3	50.0	W10X22 2	
116	36.86	112.5	0.0	204.2	50.0	W8X48 2	
117	26.41	353.3	0.0	423.2	50.0	W16X31	16, 4, 6, 2, 12
119	26.25	111.8	-177.1	402.5	50.0	W18X35	24
	8.67	0.0	-177.1				
120	26.25	133.6	0.0	159.1	50.0	W12X14	23
121	20.83	265.2	0.0	313.4	50.0	W16X26	10, 1, 10
122	26.27	144.0	0.0	168.7	50.0	W12X14	26
123	26.25	161.5	0.0	191.0	50.0	W12X19	21
126	17.72	206.7	0.0	247.6	50.0	W14X22	10, 10
127	26.25	151.6	0.0	183.0	50.0	W12X16	26
128	26.25	145.5	0.0	173.3	50.0	W12X16	22
130	11.15	0.2	0.0	68.2	50.0	W8X10	6
131	14.87	118.8	0.0	138.3	50.0	W14X22	
133	19.69	208.7	0.0	248.5	50.0	W14X22	10, 1, 10
135	19.03	87.9	0.0	114.5	50.0	W12X14	8
137	19.03	87.9	0.0	114.5	50.0	W12X14	8
138	17.72	176.3	0.0	210.3	50.0	W12X19	22
139	19.03	83.6	0.0	114.5	50.0	W12X14	8
140	19.03	79.2	0.0	114.5	50.0	W12X14	8
142	19.03	56.8	0.0	70.8	50.0	W8X18 2	
143	47.74	259.1	0.0	388.6	50.0	W16X31	32
144	13.13	18.6	0.0	68.1	50.0	W8X10	6
145	6.89	6.8	0.0	37.0	50.0	W8X10	
146	6.89	7.2	0.0	37.0	50.0	W8X10	
147	6.89	7.2	0.0	37.0	50.0	W8X10	
148	6.89	7.2	0.0	37.0	50.0	W8X10	
149	6.88	7.3	0.0	37.0	50.0	W8X10	
150	6.89	7.2	0.0	37.0	50.0	W8X10	
151	6.89	7.2	0.0	37.0	50.0	W8X10	
152	6.89	7.1	0.0	37.0	50.0	W8X10	
153	11.15	3.2	0.0	68.0	50.0	W8X10	7
154	48.83	307.0	0.0	461.8	50.0	W18X35	28

Bm #	Length	+Mu	-Mu	Mn	Fy	Beam Size	Studs
46	29.53	203.9	0.0	244.7	50.0	W14X22	25
47	26.41	379.0	0.0	449.7	50.0	W16X31	21, 2, 3, 2, 24
48	29.55	190.3	0.0	227.4	50.0	W14X22	18
49	29.53	180.7	0.0	220.2	50.0	W14X22	17
50	29.53	185.1	0.0	220.2	50.0	W14X22	16
52	27.56	403.2	0.0	478.2	50.0	W18X35	16, 2, 16
54	29.53	185.1	0.0	220.2	50.0	W14X22	16
55	29.53	185.1	0.0	220.2	50.0	W14X22	16
56	19.69	265.6	0.0	312.9	50.0	W16X26	20
57	29.53	190.2	0.0	227.4	50.0	W14X22	18
58	29.53	195.2	0.0	233.8	50.0	W14X22	20
59	27.56	403.2	0.0	478.2	50.0	W18X35	16, 2, 16
60	29.53	190.2	0.0	227.4	50.0	W14X22	18
61	29.53	185.1	0.0	220.2	50.0	W14X22	16
62	29.53	185.1	0.0	220.2	50.0	W14X22	16
64	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
66	29.53	185.1	0.0	220.2	50.0	W14X22	16
67	29.53	185.1	0.0	220.2	50.0	W14X22	16
68	19.69	339.7	0.0	400.9	50.0	W16X31	34
69	29.53	249.2	0.0	384.0	50.0	W16X26	46
70	29.53	311.7	0.0	421.9	50.0	W16X31	42
71	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
72	29.53	249.2	0.0	384.0	50.0	W16X26	46
73	29.53	185.1	0.0	220.2	50.0	W14X22	16
74	29.53	185.1	0.0	220.2	50.0	W14X22	16
76	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
78	29.53	185.1	0.0	220.2	50.0	W14X22	16
79	29.53	185.1	0.0	220.2	50.0	W14X22	16
80	19.69	339.7	0.0	400.9	50.0	W16X31	34
81	29.53	190.2	0.0	227.4	50.0	W14X22	18
82	29.53	195.2	0.0	233.8	50.0	W14X22	20
83	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
84	29.53	190.2	0.0	227.4	50.0	W14X22	18
85	29.53	185.1	0.0	220.2	50.0	W14X22	16
86	29.53	185.1	0.0	220.2	50.0	W14X22	16
88	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
90	29.53	185.1	0.0	220.2	50.0	W14X22	16
91	29.53	185.1	0.0	220.2	50.0	W14X22	16
92	19.69	261.7	0.0	312.9	50.0	W16X26	20
93	29.53	190.2	0.0	227.4	50.0	W14X22	18
94	29.53	195.2	0.0	233.8	50.0	W14X22	20
95	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
96	29.53	190.2	0.0	227.4	50.0	W14X22	18
97	29.53	185.1	0.0	220.2	50.0	W14X22	16
98	29.53	185.1	0.0	220.2	50.0	W14X22	16

Bm #	Length	+Mu	-Mu	Mn	Fy	Beam Size	Studs
100	27.56	403.2	0.0	478.2	50.0	W18X35	16, 2, 16
102	29.53	185.1	0.0	220.2	50.0	W14X22	16
103	29.53	180.7	0.0	220.2	50.0	W14X22	17
105	29.55	190.3	0.0	227.4	50.0	W14X22	18
106	29.53	199.5	0.0	239.5	50.0	W14X22	23
107	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
108	29.53	190.2	0.0	227.4	50.0	W14X22	18
109	29.53	185.1	0.0	220.2	50.0	W14X22	16
110	29.53	185.1	0.0	220.2	50.0	W14X22	16
112	18.91	43.3	0.0	70.8	50.0	W8X18 2	
113	38.95	125.8	0.0	251.7	50.0	W10X49	
114	4.12	1.3	0.0	37.0	50.0	W8X10 2	
115	39.59	2.8	0.0	37.0	50.0	W8X10	
116	21.28	35.4	0.0	108.3	50.0	W10X22 2	
117	36.86	112.5	0.0	204.2	50.0	W8X48 2	
118	26.41	353.3	0.0	423.2	50.0	W16X31	16, 4, 6, 2, 12
120	26.25	111.8	-177.1	402.5	50.0	W18X35	24
	8.67	0.0	-177.1				
121	26.25	133.6	0.0	159.1	50.0	W12X14	23
122	20.83	265.2	0.0	313.4	50.0	W16X26	10, 1, 10
123	26.27	144.0	0.0	168.7	50.0	W12X14	26
124	26.25	161.5	0.0	191.0	50.0	W12X19	21
127	17.72	206.7	0.0	247.6	50.0	W14X22	10, 10
128	26.25	151.6	0.0	183.0	50.0	W12X16	26
129	26.25	145.5	0.0	173.3	50.0	W12X16	22
131	11.15	0.2	0.0	68.2	50.0	W8X10	6
132	14.87	118.8	0.0	138.3	50.0	W14X22	
135	19.69	208.7	0.0	248.5	50.0	W14X22	10, 1, 10
137	19.03	87.9	0.0	114.5	50.0	W12X14	8
139	19.03	87.9	0.0	114.5	50.0	W12X14	8
140	17.72	176.3	0.0	210.3	50.0	W12X19	22
141	19.03	83.6	0.0	114.5	50.0	W12X14	8
142	19.03	79.2	0.0	114.5	50.0	W12X14	8
144	19.03	56.8	0.0	70.8	50.0	W8X18 2	
145	47.74	259.1	0.0	388.6	50.0	W16X31	32
146	13.13	18.6	0.0	68.1	50.0	W8X10	6
147	6.89	6.8	0.0	37.0	50.0	W8X10	
148	6.89	7.2	0.0	37.0	50.0	W8X10	
149	6.89	7.2	0.0	37.0	50.0	W8X10	
150	6.89	7.2	0.0	37.0	50.0	W8X10	
151	6.88	7.3	0.0	37.0	50.0	W8X10	
152	6.89	7.2	0.0	37.0	50.0	W8X10	
153	6.89	7.2	0.0	37.0	50.0	W8X10	
154	6.89	7.1	0.0	37.0	50.0	W8X10	
155	11.15	3.2	0.0	68.0	50.0	W8X10	7

Bm #	Length	+Mu	-Mu	Mn	Fy	Beam Size	Studs
156	48.83	307.0	0.0	461.8	50.0	W18X35	28
157	12.04	25.4	0.0	68.0	50.0	W8X10	3, 3
158	3.35	1.7	0.0	37.0	50.0	W8X10	
159	3.35	1.7	0.0	37.0	50.0	W8X10	
160	3.35	1.7	0.0	37.0	50.0	W8X10	
161	3.35	1.7	0.0	37.0	50.0	W8X10	
162	3.35	1.7	0.0	37.0	50.0	W8X10	
163	3.35	1.7	0.0	37.0	50.0	W8X10	
164	3.35	1.7	0.0	37.0	50.0	W8X10	
165	16.40	43.3	0.0	68.2	50.0	W8X10	3, 3
166	10.35	22.1	0.0	61.8	50.0	W8X10	6
167	49.36	332.6	0.0	646.9	50.0	W21X50	23

Floor Type: 2nd

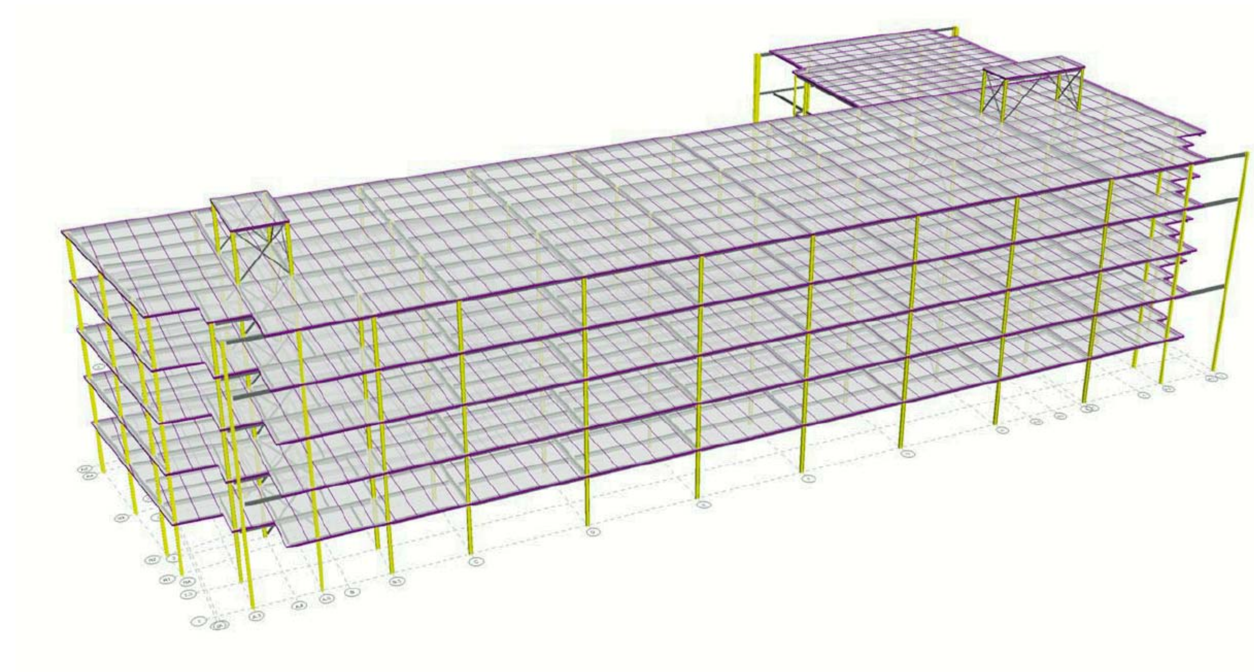
Bm #	Length ft	+Mu kip-ft	-Mu kip-ft	Mn kip-ft	Fy ksi	Beam Size	Studs
3	28.53	172.0	0.0	212.4	50.0	W14X22	15
4	27.05	147.7	0.0	178.4	50.0	W12X16	25
6	26.36	132.6	0.0	159.1	50.0	W12X14	23
7	24.58	107.8	0.0	128.8	50.0	W10X12	23
8	23.55	85.8	0.0	103.3	50.0	W10X12	13
10	22.74	143.8	0.0	211.9	50.0	W14X22	7, 7
11	11.59	27.3	0.0	61.8	50.0	W8X10	6
12	17.22	51.9	0.0	68.3	50.0	W8X10	3, 3
14	11.15	3.2	0.0	68.0	50.0	W8X10	7
16	16.40	0.5	0.0	68.7	50.0	W8X10	6
17	19.69	107.4	0.0	132.1	50.0	W10X12	8, 2, 9
19	27.56	291.1	0.0	346.8	50.0	W16X26	13, 2, 15
21	12.14	33.4	0.0	52.5	50.0	W10X12	
22	12.14	33.5	0.0	52.5	50.0	W10X12	
24	16.40	0.5	0.0	68.7	50.0	W8X10	6
26	29.55	232.3	0.0	277.1	50.0	W16X26	8, 9
27	29.53	203.9	0.0	244.7	50.0	W14X22	25
28	27.56	309.9	0.0	368.7	50.0	W16X26	17, 2, 3, 1, 16
29	29.55	196.1	0.0	233.8	50.0	W14X22	20
30	29.53	183.7	0.0	220.2	50.0	W14X22	17
31	29.53	178.8	0.0	211.7	50.0	W12X19	28
33	11.15	0.2	0.0	68.2	50.0	W8X10	6
36	16.00	59.7	0.0	88.9	50.0	W8X10	12
37	19.09	86.1	0.0	114.5	50.0	W12X14	8
38	19.09	70.2	0.0	84.6	50.0	W8X10	15
39	26.41	334.1	0.0	401.6	50.0	W16X31	13, 3, 6, 1, 9
41	29.53	185.1	0.0	220.2	50.0	W14X22	16
42	29.53	180.7	0.0	220.2	50.0	W14X22	17
43	21.98	309.7	0.0	367.5	50.0	W16X26	40

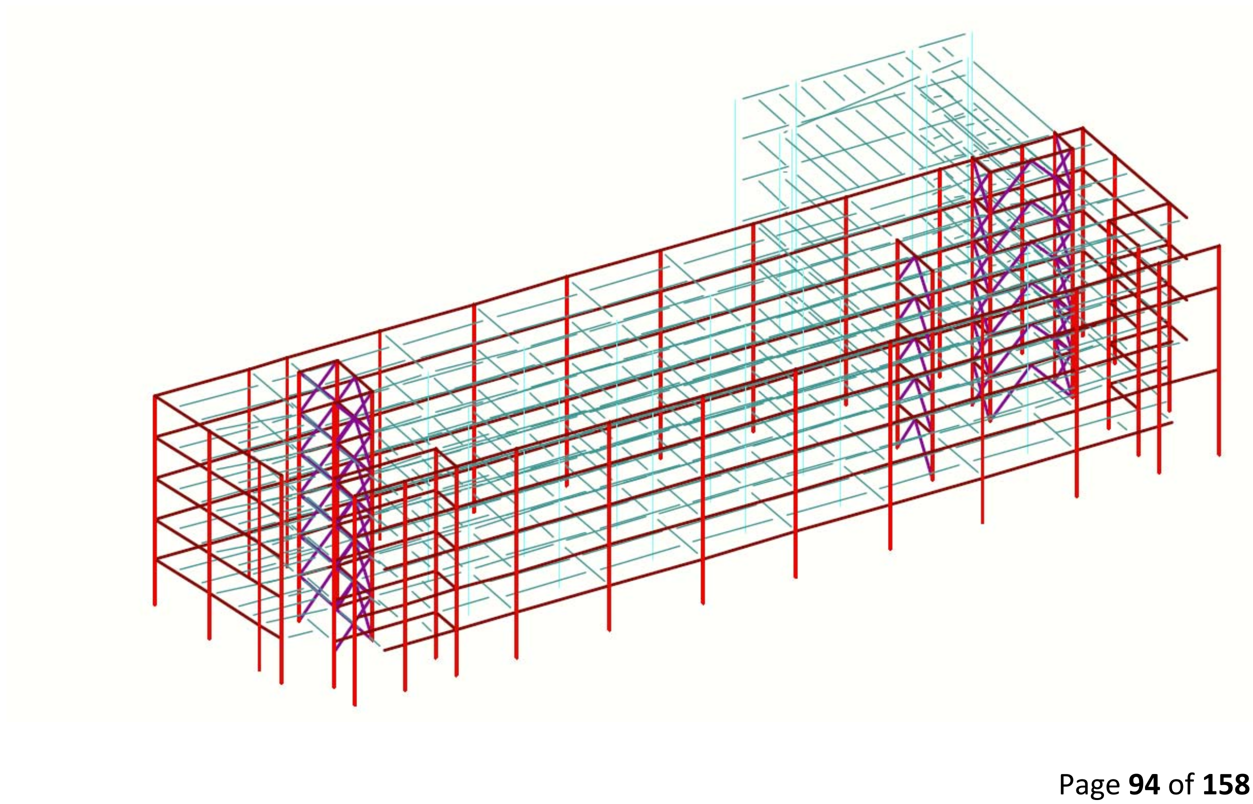
Bm #	Length	+Mu	-Mu	Mn	Fy	Beam Size	Studs
44	29.55	190.3	0.0	227.4	50.0	W14X22	18
45	29.53	203.9	0.0	244.7	50.0	W14X22	25
46	26.41	379.0	0.0	449.7	50.0	W16X31	21, 2, 3, 2, 24
47	29.55	190.3	0.0	227.4	50.0	W14X22	18
48	29.53	180.7	0.0	220.2	50.0	W14X22	17
49	29.53	185.1	0.0	220.2	50.0	W14X22	16
51	27.56	403.2	0.0	478.2	50.0	W18X35	16, 2, 16
53	29.53	185.1	0.0	220.2	50.0	W14X22	16
54	29.53	185.1	0.0	220.2	50.0	W14X22	16
55	19.69	265.6	0.0	312.9	50.0	W16X26	20
56	29.53	190.2	0.0	227.4	50.0	W14X22	18
57	29.53	195.2	0.0	233.8	50.0	W14X22	20
58	27.56	403.2	0.0	478.2	50.0	W18X35	16, 2, 16
59	29.53	190.2	0.0	227.4	50.0	W14X22	18
60	29.53	185.1	0.0	220.2	50.0	W14X22	16
61	29.53	185.1	0.0	220.2	50.0	W14X22	16
63	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
65	29.53	185.1	0.0	220.2	50.0	W14X22	16
66	29.53	185.1	0.0	220.2	50.0	W14X22	16
67	19.69	339.7	0.0	400.9	50.0	W16X31	34
68	29.53	249.2	0.0	384.0	50.0	W16X26	46
69	29.53	311.7	0.0	421.9	50.0	W16X31	42
70	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
71	29.53	249.2	0.0	384.0	50.0	W16X26	46
72	29.53	185.1	0.0	220.2	50.0	W14X22	16
73	29.53	185.1	0.0	220.2	50.0	W14X22	16
75	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
77	29.53	185.1	0.0	220.2	50.0	W14X22	16
78	29.53	185.1	0.0	220.2	50.0	W14X22	16
79	19.69	339.7	0.0	400.9	50.0	W16X31	34
80	29.53	190.2	0.0	227.4	50.0	W14X22	18
81	29.53	195.2	0.0	233.8	50.0	W14X22	20
82	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
83	29.53	190.2	0.0	227.4	50.0	W14X22	18
84	29.53	185.1	0.0	220.2	50.0	W14X22	16
85	29.53	185.1	0.0	220.2	50.0	W14X22	16
87	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
89	29.53	185.1	0.0	220.2	50.0	W14X22	16
90	29.53	185.1	0.0	220.2	50.0	W14X22	16
91	19.69	261.7	0.0	312.9	50.0	W16X26	20
92	29.53	190.2	0.0	227.4	50.0	W14X22	18
93	29.53	195.2	0.0	233.8	50.0	W14X22	20
94	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
95	29.53	190.2	0.0	227.4	50.0	W14X22	18
96	29.53	185.1	0.0	220.2	50.0	W14X22	16

Bm #	Length	+Mu	Mu	Mn	Fy	Beam Size	Studs
97	29.53	185.1	0.0	220.2	50.0	W14X22	16
99	27.56	403.2	0.0	478.2	50.0	W18X35	16, 2, 16
101	29.53	185.1	0.0	220.2	50.0	W14X22	16
102	29.53	180.7	0.0	220.2	50.0	W14X22	17
104	29.55	190.3	0.0	227.4	50.0	W14X22	18
105	29.53	199.5	0.0	239.5	50.0	W14X22	23
106	27.56	404.6	0.0	484.1	50.0	W18X35	16, 2, 16
107	29.53	190.2	0.0	227.4	50.0	W14X22	18
108	29.53	185.1	0.0	220.2	50.0	W14X22	16
109	29.53	185.1	0.0	220.2	50.0	W14X22	16
111	18.91	43.3	0.0	70.8	50.0	W8X18 2	
112	38.95	125.8	0.0	251.7	50.0	W10X49	
113	4.12	1.3	0.0	37.0	50.0	W8X10 2	
114	39.59	2.8	0.0	37.0	50.0	W8X10	
115	21.28	35.4	0.0	108.3	50.0	W10X22 2	
116	36.86	112.5	0.0	204.2	50.0	W8X48 2	
117	26.41	355.3	0.0	423.2	50.0	W16X31	16, 4, 7, 2, 12
119	26.25	115.4	-151.8	329.9	50.0	W16X31	22
	8.67	0.0	-151.8				
120	26.25	133.6	0.0	159.1	50.0	W12X14	23
121	20.83	265.2	0.0	313.4	50.0	W16X26	10, 1, 10
122	26.27	144.0	0.0	168.7	50.0	W12X14	26
123	26.25	161.5	0.0	191.0	50.0	W12X19	21
126	17.72	206.7	0.0	247.6	50.0	W14X22	10, 10
127	26.25	151.6	0.0	183.0	50.0	W12X16	26
128	26.25	145.5	0.0	173.3	50.0	W12X16	22
130	11.15	0.2	0.0	68.2	50.0	W8X10	6
131	14.87	118.8	0.0	138.3	50.0	W14X22	
133	8.67	15.2	0.0	37.0	50.0	W8X10	
134	19.69	167.8	0.0	199.4	50.0	W12X19	9, 1, 9
136	8.67	18.2	0.0	37.0	50.0	W8X10	
138	8.67	18.2	0.0	37.0	50.0	W8X10	
139	17.72	136.0	0.0	161.4	50.0	W12X14	18
141	8.67	16.4	0.0	37.0	50.0	W8X10	
143	19.03	56.8	0.0	70.8	50.0	W8X18 2	
144	47.74	259.1	0.0	388.6	50.0	W16X31	32
145	13.13	18.6	0.0	68.1	50.0	W8X10	6
146	6.89	6.8	0.0	37.0	50.0	W8X10	
147	6.89	7.2	0.0	37.0	50.0	W8X10	
148	6.89	7.2	0.0	37.0	50.0	W8X10	
149	6.89	7.2	0.0	37.0	50.0	W8X10	
150	6.88	7.3	0.0	37.0	50.0	W8X10	
151	6.89	7.2	0.0	37.0	50.0	W8X10	
152	6.89	7.2	0.0	37.0	50.0	W8X10	
153	6.89	7.1	0.0	37.0	50.0	W8X10	

Bm #	Length	+Mu	-Mu	Mn	Fy	Beam Size	Studs
154	11.15	3.2	0.0	68.0	50.0	W8X10	7
155	48.83	307.0	0.0	461.8	50.0	W18X35	28
156	12.04	25.4	0.0	68.0	50.0	W8X10	3, 3
157	3.35	1.7	0.0	37.0	50.0	W8X10	
158	3.35	1.7	0.0	37.0	50.0	W8X10	
159	3.35	1.7	0.0	37.0	50.0	W8X10	
160	3.35	1.7	0.0	37.0	50.0	W8X10	
161	3.35	1.7	0.0	37.0	50.0	W8X10	
162	3.35	1.7	0.0	37.0	50.0	W8X10	
163	3.35	1.7	0.0	37.0	50.0	W8X10	
164	16.40	52.8	0.0	68.3	50.0	W8X10	6
165	29.53	161.7	0.0	216.4	50.0	W12X19	24
166	17.72	61.6	0.0	89.3	50.0	W10X12	6
167	49.36	332.6	0.0	646.9	50.0	W21X50	23

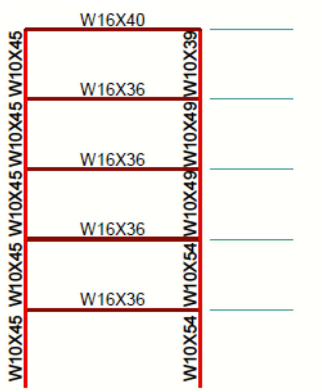
Appendix D: Lateral Framing



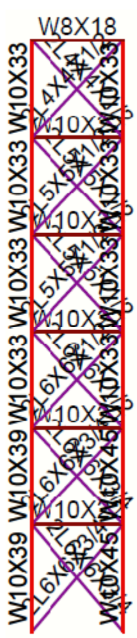


Moment Frame along Grid 1

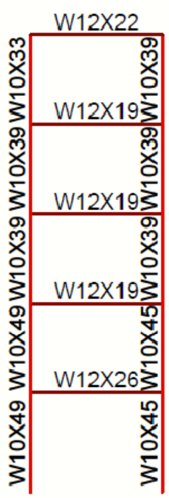
Moment Frame along Grid 1.3



Braced Frame along Grid 2



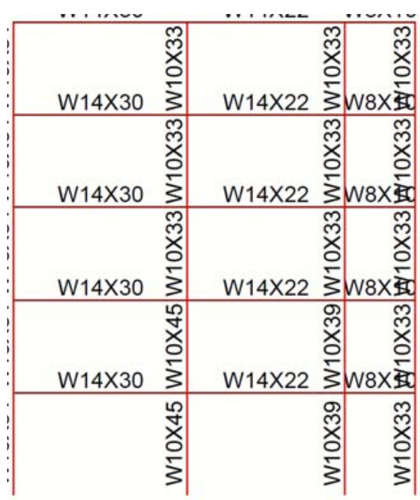
Moment Frame along Grid 2



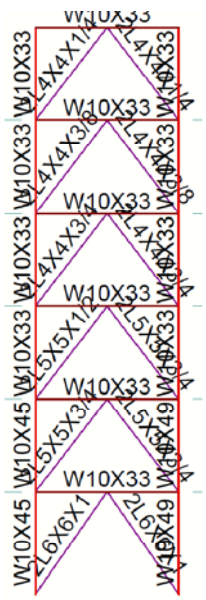
Braced Frames along Grid 3

Moment Frame along Grid 4

Moment Frame along Grid RA

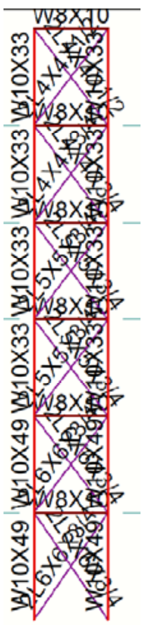


Braced Frame along Grid B

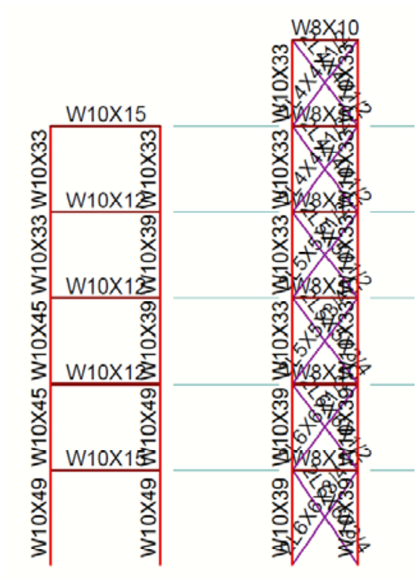


Brace Frame along Grid H

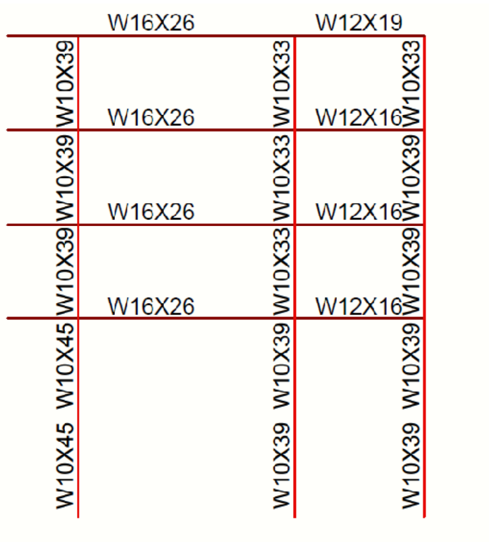
Brace Frame along Grid J



Moment Frame and Braced Frame along Grid K



Moment Frame along Grid L



Period of Vibration from RAM

FREQUENCIES AND PERIODS:

Mode	Period sec	Frequency Hz	Frequency rad/sec
1	1.4798	0.6757	4.2458
2	1.3521	0.7396	4.6470
3	1.1173	0.8950	5.6234
4	0.6283	1.5915	10.0000
5	0.6283	1.5915	10.0000
6	0.3930	2.5444	15.9869
7	0.3816	2.6207	16.4666

Center of Rigidity from RAM

		Centers of Rigidity		Centers of Mass	
Level	**Diaph. #**	**Xr** ft	**Yr** ft	**Xm** ft	**Ym** ft
Penthouse	1	38.62	37.64	24.12	37.41
Penthouse	2	238.65	44.44	249.95	52.17
Roof	1	122.97	40.40	154.37	50.00
Floor 5	1	126.11	40.10	142.64	41.10
Floor 4	1	134.75	40.00	142.66	41.10
Floor 3	1	146.66	39.83	142.65	41.10
Floor 2	1	166.28	40.03	138.90	41.01

Appendix E: Design Checks

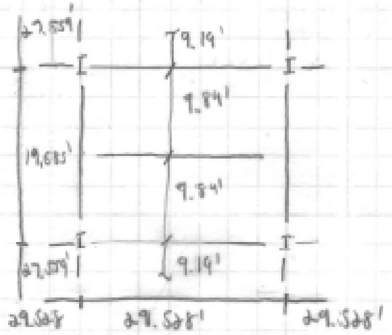
Gravity Loads

Live Loads:
- Office Live Load (Partions) : 80 psf
- Typical Roof : 20 psf
- Mechanical Room : 150 psf
- Pedestrian Bridge : 60 psf
- High Density Filing : 250 psf

Dead Loads:
- Super imposed Dead Load : 15 psf
- Roofing System : 40 psf
- Exterior Curtain Wall : 30 psf
- Atrium Curtain Wall : 20 psf
- Mechanical Unit : 150 psf

Snow Loads : 20 psf

Gravity System Design: Composite steel on metal deck



Given: LL = 80 psf
SDL: 15 psf
DL = Self weight

Assume
Normal weight concrete
use: United Steel Deck to determine deck size, and slab thickness

- By United Steel Deck determine deck size and slab thickness

Deck Properties

use 18 Gage Deck with the following Properties 2" Lok-Floor

$t = .0474$ in	$I = .560$ in^4	R_b : 1680 lb per 12" width
w : 2.4 psf	S_p : .523 in^3	ϕV_n : 3180 lb per 12" width
A_s : .710 in^2	S_n : .529 in^3	Studs : .57 # studs per ft

Composite Properties - For 18 gage deck, with 4 1/2" slab depth
Slab depth: 4.5 in
ϕM_n : 52.08 in-k
w : 42 psf

Max unshored spans
2 span : 10.48 ft

Uniform Live Loads (Including self weight): 225 psf

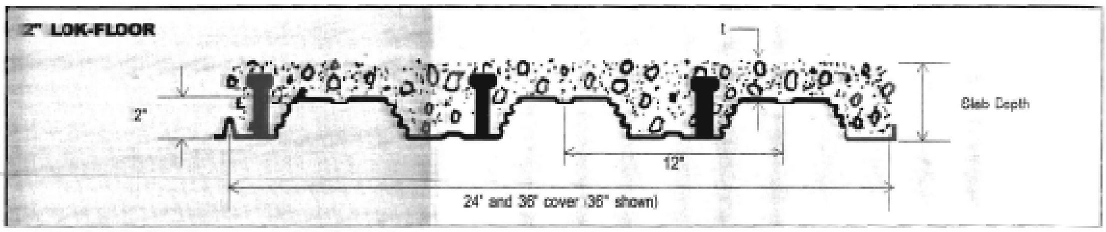
* Exceeds required loading ∴ ok

- Use: 18 gage 2" Lok-Floor Deck
with 4.5 in Normal Weight concrete
Slab depth

2 x 12" DECK F_y = 33ksi f'_c = 3 ksi   145 pcf concrete

USD United Steel Deck, Inc.

LRFD



The **Deck Section Properties** are per foot of width. The I value is for postive bending (in.^{4}); t is the gage thickness in inches; **w** is the weight in pounds per square foot; S_p and S_n are the section moduli for positive and negative bending (in.^{3}); R_b and ϕV_n are the interior reaction and the shear in pounds (per foot of width); studs is the number of studs required per foot in order to obtain the full resisting moment, ϕM_{nf}.

Gage	t	w	As	I	S_p	S_n	R_b	ϕV_n	studs
22	0.0295	1.5	0.440	0.338	0.284	0.302	714	1990	0.36
20	0.0358	1.8	0.540	0.420	0.367	0.387	1010	2410	0.43
19	0.0418	2.1	0.630	0.490	0.445	0.458	1330	2810	0.51
18	0.0474	2.4	0.710	0.560	0.523	0.529	1680	3190	0.57
16	0.0598	3.1	0.900	0.700	0.654	0.654	2470	3890	0.72

The **Composite Properties** are a list of values for the composite slab. The **slab depth** is the distance from the bottom of the steel deck to the top of the slab in inches as shown on the sketch. U.L. ratings generally refer to the cover over the top of the deck so it is important to be aware of the difference in names. ϕM_{nf} is the factored resisting moment provided by the composite slab when the "full" number of studs as shown in the upper table are in place; inch kips (per foot of width). A_c is the area of concrete available to resist shear, in.^{2} per foot of width. **Vol.** is the volume of concrete in ft.^{3} per ft.^{2} needed to make up the slab; no allowance for frame or deck deflection is included. **W** is the concrete weight in pounds per ft.^{2}. S_c is the section modulus of the "cracked" concrete composite slab; in.^{3} per foot of width. I_{av} is the average of the "cracked" and "uncracked" moments of inertia of the transformed composite slab; in.^{4} per foot of width. The I_{av} transformed section analysis is based on steel; therefore, to calculate deflections the appropriate modulus of elasticity to use is 29.5 x 10^6 psi. ϕM_{no} is the factored resisting moment of the composite slab if there are no studs on the beams (the deck is attached to the beams or walls on which it is resting) inch kips (per foot of width). ϕV_{nt} is the factored vertical shear resistance of the composite system; it is the sum of the shear resistances of the steel deck and the concrete but is not allowed to exceed $\phi 4(f'_c)^{1/2} A_c$; pounds (per foot of width). The next three columns list the **maximum unshored spans** in feet; these values are obtained by using the construction loading requirements of the SDI; combined bending and shear, deflection, and interior reactions are considered in calculating these values. A_{wwf} is the minimum area of welded wire fabric recommended for temperature reinforcing in the composite slab; square inches per foot.

COMPOSITE PROPERTIES

Gage	Slab Depth	ϕM_{nf} in.k	A_c in^2	Vol. ft^3/ft^2	W psf	S_c in^3	I_{av} in^4	ϕM_{no} in.k	ϕV_{nt} lbs.	Max. unshored spans, ft. 1span	2span	3span	A_{wwf}
22 gage	4.50	40.27	32.6	0.292	42	1.05	5.9	29.40	5030	5.82	7.83	7.92	0.023
	5.00	46.44	37.5	0.333	48	1.23	8.0	34.53	5480	5.54	7.47	7.56	0.027
	5.25	49.53	40.0	0.354	51	1.32	9.2	37.16	5720	5.41	7.31	7.39	0.029
	5.50	52.61	42.6	0.375	54	1.42	10.5	39.81	5960	5.30	7.16	7.24	0.032
	6.00	58.78	48.0	0.417	60	1.61	13.5	45.21	6460	5.09	6.89	6.97	0.036
	6.25	61.87	50.8	0.438	63	1.71	15.3	47.95	6720	5.03	6.76	6.84	0.038
	6.50	64.95	53.6	0.458	66	1.81	17.1	50.70	6980	4.97	6.65	6.72	0.041
	7.00	71.12	59.5	0.500	73	2.01	21.2	56.26	7530	4.85	6.43	6.51	0.045
	7.25	74.21	61.9	0.521	76	2.11	23.5	59.07	7750	4.79	6.32	6.41	0.047
	7.50	77.29	64.3	0.542	79	2.21	26.0	61.88	7970	4.74	6.22	6.31	0.050
20 gage	4.50	48.60	32.6	0.292	42	1.26	6.3	35.43	5450	6.81	8.97	9.27	0.023
	5.00	56.18	37.5	0.333	48	1.48	8.6	41.65	5900	6.47	8.55	8.83	0.027
	5.25	59.66	40.0	0.354	51	1.60	9.8	44.84	6140	6.33	8.36	8.63	0.029
	5.50	63.75	42.6	0.375	54	1.71	11.3	48.07	6380	6.18	8.18	8.45	0.032
	6.00	71.32	48.0	0.417	60	1.95	14.5	54.63	6880	5.94	7.85	8.11	0.036
	6.25	75.11	50.8	0.438	63	2.07	16.3	57.96	7140	5.86	7.70	7.95	0.038
	6.50	78.90	53.6	0.458	66	2.19	18.2	61.31	7400	5.79	7.56	7.80	0.041
	7.00	86.47	59.5	0.500	73	2.43	22.6	68.09	7950	5.65	7.29	7.53	0.045
	7.25	90.26	61.9	0.521	76	2.55	25.0	71.50	8170	5.58	7.17	7.41	0.047
	7.50	94.05	64.3	0.542	79	2.67	27.6	74.93	8390	5.52	7.05	7.28	0.050
19 gage	4.50	55.85	32.6	0.292	42	1.45	6.7	40.69	5850	7.65	9.76	10.08	0.023
	5.00	64.88	37.5	0.333	48	1.71	9.0	47.87	6300	7.26	9.30	9.61	0.027
	5.25	69.10	40.3	0.354	51	1.84	10.4	51.56	6540	7.09	9.09	9.39	0.029
	5.50	73.52	42.5	0.375	54	1.97	11.9	55.30	6780	6.93	8.90	9.19	0.032
	6.00	82.35	48.3	0.417	60	2.24	15.2	62.90	7280	6.65	8.54	8.83	0.036
	6.25	86.77	50.3	0.438	63	2.38	17.1	66.70	7540	6.56	8.38	8.66	0.038
	6.50	91.19	53.5	0.458	66	2.52	19.2	70.65	7800	6.48	8.23	8.50	0.041
	7.00	100.03	59.5	0.500	73	2.80	23.8	78.50	8350	6.32	7.94	8.20	0.045
	7.25	104.44	61.3	0.521	76	2.94	26.3	82.46	8570	6.24	7.81	8.07	0.047
	7.50	108.86	64.3	0.542	79	3.08	29.0	86.45	8790	6.17	7.68	7.94	0.050
18 gage	4.50	62.38	32.5	0.292	42	1.62	7.0	45.34	6080	8.42	10.48	10.83	0.023
	5.00	72.04	37.5	0.333	48	1.90	9.5	53.36	6670	7.98	9.99	10.32	0.027
	5.25	77.02	40.0	0.354	51	2.05	10.9	57.48	6910	7.79	9.77	10.10	0.029
	5.50	82.00	42.6	0.375	54	2.20	12.4	61.66	7150	7.61	9.56	9.88	0.032
	6.00	91.15	48.0	0.417	60	2.50	15.9	70.18	7650	7.30	9.18	9.48	0.036
	6.25	96.43	50.4	0.438	63	2.66	17.9	74.50	7910	7.20	9.01	9.31	0.038
	6.50	101.91	53.6	0.458	66	2.81	20.0	78.85	8170	7.11	8.85	9.14	0.041
	7.00	111.87	59.5	0.500	73	3.13	24.8	87.66	8720	6.93	8.54	8.82	0.045
	7.25	116.85	61.9	0.521	76	3.28	27.4	92.10	8940	6.85	8.40	8.68	0.047
	7.50	121.83	64.3	0.542	79	3.44	30.2	96.57	9160	6.77	8.26	8.54	0.050
16 gage	4.50	62.88	32.6	0.292	42	1.99	7.7	45.34	6080	9.58	11.63	12.02	0.023
	5.00	72.84	37.3	0.333	48	2.35	10.4	53.36	6960	9.06	11.10	11.47	0.027
	5.25	77.82	40.8	0.354	51	2.53	11.9	57.48	7450	8.85	10.85	11.22	0.029
	5.50	82.80	42.6	0.375	54	2.72	13.6	61.66	7940	8.65	10.63	10.98	0.032
	6.00	91.96	48.8	0.417	60	3.10	17.4	70.16	8400	8.29	10.21	10.55	0.036
	6.25	96.93	50.8	0.438	63	3.29	19.5	74.50	8720	8.17	10.02	10.35	0.038
	6.50	101.91	53.6	0.458	66	3.48	21.8	78.85	8980	8.07	9.84	10.17	0.041
	7.00	111.37	59.5	0.500	73	3.88	27.8	87.66	9530	7.86	9.50	9.83	0.045
	7.25	116.35	61.9	0.521	76	4.08	29.8	92.10	9750	7.77	9.35	9.66	0.047
	7.50	121.33	64.3	0.542	79	4.28	32.8	96.57	9970	7.67	9.20	9.50	0.050

2" LOK-FLOOR

2 x 12" DECK $F_y = 33ksi$ $f'_c = 3$ ksi 145 pcf concrete

LRFD

L_s Uniform Live Service Loads, psf *

Gage	Slab Depth	φMn in.k	6.00	6.50	7.00	7.50	8.00	8.50	9.00	9.50	10.00	10.50	11.00	11.50	12.00
22 gage	4.50	40.27	400	365	310	265	230	200	175	155	135	120	105	95	85
	5.00	46.44	400	400	360	305	265	230	200	175	155	140	125	110	95
	5.50	52.61	400	400	400	350	300	260	230	200	175	155	140	125	110
	6.00	58.78	400	400	400	390	335	295	260	220	200	175	155	140	125
	6.50	64.95	400	400	400	400	370	325	285	250	220	195	175	155	135
	7.00	71.12	400	400	400	400	400	355	310	275	240	215	190	170	150
	7.25	74.21	400	400	400	400	400	370	325	285	250	225	200	175	155
	7.50	77.29	400	400	400	400	400	385	340	295	260	230	205	185	165
20 gage	4.50	48.60	400	400	380	325	285	245	215	190	170	150	135	120	110
	5.00	56.18	400	400	400	380	330	285	250	220	195	175	155	140	125
	5.50	63.75	400	400	400	400	375	325	285	250	225	200	175	160	140
	6.00	71.32	400	400	400	400	400	365	320	285	250	225	200	180	160
	6.50	78.90	400	400	400	400	400	400	355	315	280	245	220	195	175
	7.00	86.47	400	400	400	400	400	400	390	345	305	270	240	215	195
	7.25	90.26	400	400	400	400	400	400	400	360	320	285	255	225	205
	7.50	94.05	400	400	400	400	400	400	400	375	330	295	265	235	210
19 gage	4.50	55.60	400	400	400	380	330	270	230	220	200	180	160	140	125
	5.00	64.68	400	400	400	400	385	335	295	260	230	205	185	165	150
	5.50	73.52	400	400	400	400	400	380	335	295	265	235	210	190	170
	6.00	82.35	400	400	400	400	400	400	375	335	295	265	235	215	190
	6.50	91.19	400	400	400	400	400	400	400	370	330	295	265	235	210
	7.00	100.03	400	400	400	400	400	400	400	400	360	320	290	260	235
	7.25	104.44	400	400	400	400	400	400	400	400	375	335	300	270	245
	7.50	108.86	400	400	400	400	400	400	400	400	395	350	315	280	255
18 gage	4.50	62.08	400	400	400	400	370	325	285	255	225	200	180	160	145
	5.00	72.04	400	400	400	400	400	375	335	295	260	235	210	190	170
	5.50	82.00	400	400	400	400	400	400	380	335	300	265	240	215	195
	6.00	91.95	400	400	400	400	400	400	400	375	335	300	270	245	220
	6.50	101.91	400	400	400	400	400	400	400	400	375	335	300	270	245
	7.00	111.87	400	400	400	400	400	400	400	400	400	365	330	295	270
	7.25	116.85	400	400	400	400	400	400	400	400	400	385	345	310	280
	7.50	121.83	400	400	400	400	400	400	400	400	400	400	360	325	290
16 gage	4.50	62.08	400	400	400	400	370	325	285	255	225	200	180	160	145
	5.00	72.04	400	400	400	400	400	375	335	295	260	235	210	190	170
	5.50	82.00	400	400	400	400	400	400	380	335	300	265	240	215	195
	6.00	91.95	400	400	400	400	400	400	400	375	335	300	270	245	220
	6.50	101.91	400	400	400	400	400	400	400	400	375	335	300	270	245
	7.00	111.87	400	400	400	400	400	400	400	400	400	365	330	295	270
	7.25	116.85	400	400	400	400	400	400	400	400	400	385	345	310	280
	7.50	121.83	400	400	400	400	400	400	400	400	400	400	360	325	290
22 gage	4.50	29.40	305	255	215	185	160	135	120	105	90	80	70	60	50
	5.00	34.53	360	300	250	215	185	160	140	120	105	90	80	70	60
	5.50	39.81	400	350	295	255	215	190	165	140	125	110	95	85	75
	6.00	45.21	400	400	340	290	250	215	185	160	140	125	110	95	85
	6.50	50.70	400	400	390	335	290	245	210	185	160	140	125	110	95
	7.00	56.26	400	400	400	380	330	275	235	205	180	155	140	120	105
	7.25	59.07	400	400	400	390	325	285	245	215	190	165	145	130	115
	7.50	61.88	400	400	400	400	345	295	260	225	200	175	155	135	120
20 gage	4.50	35.43	375	315	270	230	200	170	150	130	115	100	90	80	70
	5.00	41.65	400	375	315	270	235	205	175	155	135	120	105	95	85
	5.50	48.07	400	400	365	315	275	235	205	180	160	140	125	110	95
	6.00	54.63	400	400	400	360	310	270	235	205	180	160	140	125	110
	6.50	61.31	400	400	400	400	350	305	265	230	205	180	160	140	125
	7.00	68.09	400	400	400	400	390	335	295	260	230	205	180	160	140
	7.25	71.50	400	400	400	400	400	355	310	275	240	210	190	165	150
	7.50	74.93	400	400	400	400	400	375	325	285	250	225	200	175	155
19 gage	4.50	40.69	400	370	315	270	230	200	175	155	135	120	105	95	85
	5.00	47.87	400	400	370	315	275	240	210	185	160	145	125	115	100
	5.50	55.30	400	400	400	365	320	275	240	215	190	165	150	130	120
	6.00	62.90	400	400	400	400	360	315	275	245	215	190	170	150	135
	6.50	70.65	400	400	400	400	400	350	310	275	245	215	190	170	155
	7.00	78.50	400	400	400	400	400	395	350	305	270	240	215	190	170
	7.25	82.46	400	400	400	400	400	400	365	320	285	255	225	200	180
	7.50	86.45	400	400	400	400	400	400	385	340	300	265	235	210	190
18 gage	4.50	45.34	400	400	350	300	260	230	200	175	155	140	125	110	100
	5.00	53.36	400	400	400	355	310	270	235	210	185	165	145	130	115
	5.50	61.66	400	400	400	400	360	315	275	240	215	190	170	150	135
	6.00	70.18	400	400	400	400	400	360	315	275	245	220	195	175	155
	6.50	78.85	400	400	400	400	400	400	355	310	275	245	220	195	175
	7.00	87.66	400	400	400	400	400	400	395	350	310	275	245	220	195
	7.25	92.10	400	400	400	400	400	400	400	365	325	290	260	230	210
	7.50	96.57	400	400	400	400	400	400	400	385	340	305	270	245	220
16 gage	4.50	45.34	400	400	350	300	260	230	200	175	155	140	125	110	100
	5.00	53.36	400	400	400	335	310	270	235	210	185	165	145	130	115
	5.50	61.66	400	400	400	400	360	315	275	240	215	190	170	150	135
	6.00	70.18	400	400	400	400	400	360	315	275	245	220	195	175	155
	6.50	78.85	400	400	400	400	400	400	355	310	275	245	220	195	175
	7.00	87.66	400	400	400	400	400	400	395	350	310	275	245	220	195
	7.25	92.10	400	400	400	400	400	400	400	365	325	290	260	230	210
	7.50	96.57	400	400	400	400	400	400	400	385	340	305	270	245	220

☐ 1 STUD/FT.

☐ NO STUDS

* The Uniform Live Loads are based on the LRFD equation $\phi M_n = (1.6L + 1.2D)l^2/8$. Although there are other load combinations that may require investigation, this will control most of the time. The equation assumes there is no negative bending reinforcement over the beams and therefore each composite slab is a single span. Two sets of values are shown; ϕM_n is used to calculate the uniform load when the full required number of studs is present; ϕM_{no} is used to calculate the load when no studs are present. A straight line interpolation can be done if the average number of studs is between zero and the required number needed to develop the "full" factored moment. The tabulated loads are checked for shear controlling (it seldom does), and also limited to a live load deflection of 1/360 of the span.

An upper limit of 400 psf has been applied to the tabulated loads. This has been done to guard against equating large concentrated to uniform loads. Concentrated loads may require special analysis and design to take care of serviceability requirements not covered by simply using a uniform load value. On the other hand, for any load combination the values provided by the composite properties can be used in the calculations.

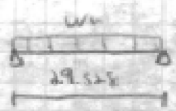
Welded wire fabric in the required amount is assumed for the table values. If welded wire fabric is not present, deduct 10% from the listed loads.

Refer to the example problems for the use of the tables.

2" LOK-FLOOR

Gravity System Design

Typical Beam Design



Tributary width: 9.84'

Loads: DL: 44.4 psf (not including beam, increase by 10%)

DL: 48.84 psf
SDL: 15 psf
LL: 80 psf (Includes Partitions)

w_u: 1.2(48.84 + 15) + 1.6(80) = 204.61 psf (1.4D DNC)
w_u: 204.61 (9.84)/1000 = 2.01 k/ft

$M_u = \frac{wl^2}{8} = \frac{2.01(29.528)^2}{8} = 219.43$ ft-k

$V_u = \frac{wl}{2} = \frac{2.01(29.528)}{2} = 29.7^k$

Assume a = 1.5 in ∴ y_2 = 4.5 − 1.5/2 = 3.75 in

Try W14x22 → ϕM_p = 125 ft-k
ϕM_n = 230 ft-k   use y_2 = 3.5 to be conservative
y_1 = 3"
ΣQ_n = 241 k
studs = $\frac{\Sigma Q_n}{Q_n} = \frac{241}{17.2}$ = 14.01 = 15 studs
across entire beam = 30 studs

Extra weight: 29.528(22) + 30(10 lb/ft) = 949.6 lb = .95 k

$M_u = \frac{(.95/29.528)(29.528)^2}{8}$ = 3.5 ft-k

M_u = 219.43 + 3.5 = 222.9 ft-k < 230 ft-k ok

Check assumption
b_{eff} = 9.84 or $\frac{29.528}{4}$ = 7.382

b_{eff} = 7.382(12) = 88.58 $a = \frac{\Sigma Q_n}{.85 f'_c b_{eff}} = \frac{241}{.85(3)(88.58)}$ = 1.07 in

a = 1.07 < 1.5 in conservative

Gravity System Design

Beam 1 cont

- check Load during construction

$CNP \quad DL = 48.84 \quad 48.84(9.84)/1000 = .437\ k/ft$

$SW = .95/29.508 = .032\ k/ft$

$M_c = \frac{wl^2}{8} = \frac{(.437 + .032)\ 29.508^2}{8} = 51.2^{'k} < \phi M_p = 125^{'k}$ ok

- check deflection

$D_{cons} = \frac{5wl^4}{384EI} \qquad w_D = .044(9.84) = .433\ k/ft \qquad w_{sw} = .032\ k/ft$

$D_{cons} = \frac{5(.433 + .032)(29.508)^4\ 1728}{384(29000)(199)} = 1.38\ in$

$\text{limit}\ l/240 = \frac{29.508(12)}{240} = 1.48\ in$

$D_{cons} = 1.38 < 1.48$ ok

$D_{LL} = \frac{5wl^4}{384EI} = \frac{5(.08)(9.84)(29.508)^4\ 1728}{384(29000)(496)} = .935\ in$

$\text{limit}\ \frac{l}{360} = \frac{29.508(12)}{360} = .98$

$D_{LL} = .936 < .98$ ok

- Check shear

$V_u = 29.7^k$

$\phi V_n = 94.8$

$V_u = 29.7^k < \phi V_n = 94.8^k$ ok

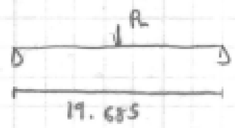
USE W14 x 22 (30) for int. Beams

$d = 13.7\ in$

Gravity System Design

Girder Design

Reaction from Beam on one side
$R = V_u = 29.7^k$
$P_u = 2(29.7) = 59.4^k$



$M_u = \frac{Pl}{4} = \frac{59.4(19.685)}{4} = 292.3$ ft-k

$V_u = \frac{P}{2} = \frac{59.4}{2} = 29.7^k$

Assume $a = 2$ in $\therefore y_2 = 4.5 - 2/2 = 3.5$ in

Try W16x26 $\phi M_p = 166^{ft\text{-}k}$
$\phi M_n = 301^{ft\text{-}k}$
$\Sigma Q_n = 289^k$
$Y_1 = 3$ in

By Table 3-21 Perpendicular Deck 3/4" stud $Q_n = 17.1^k$

studs $= \frac{\Sigma Q_n}{Q_n} = \frac{289}{17.7} = 16.9 = 17$

Total # studs = 34 studs
Beam weight = 19.685(26) + 34(10) = 851.8 lb = .852k

Check assumptions

$b_{eff} = 19.685/4 = 4.92 = 59.1$ in
$a = \frac{\Sigma Q_n}{.85\, f'_c b_e} = \frac{289}{.85(3)(59.1)} = 1.92 < 2$ in ok

$P_D = [.059(9.84)(29.528) + .95]/2 = 9.05^k$

$P_L = .08(9.84)(29.528)/2 = 11.62$

Total $P_D = 2(9.05) = 18.09^k$
$P_L = 2(11.62) = 23.24^k$

Gravity System Design

Girder Design cont.

Check deflection
construct Loads

$$D_{const} = \frac{P\ell^3}{48EI} = \frac{18.08(19.685)^3(1728)}{48(29000)(301)} = .569 \text{ in}$$

$$\text{Limit} = \ell/360 = 19.685(12)/360 = .656 \text{ in}$$

$$D_{const} = .569 < .656 \text{ ok}$$

Live Load Deflection

$$D_{LL} = \frac{P\ell^3}{48EI} = \frac{22.24(19.685)^3 1728}{48(29,000)(1720)} = .303$$

$$D_{ll} = \ell/360 = .656$$

$$D_{LL} = .303 < .656 \text{ ok}$$

use W16×26 (34) for Girders
d = 15.7 in

Approx equiv weight of floor system

slab on metal Deck
42 psf
2.4 psf
total = 44.4 psf

Beams - 3 Beams

$$\frac{22 \text{ plf }(29.528)}{29.528(19.685)} = 1.12 \text{ psf}$$

Girders - 2 Girders

$$\frac{26 \text{ plf }(19.685)}{29.528(19.685)} = .88 \text{ psf}$$

Columns

$$\frac{100 \text{ plf }(14.16)}{29.528(19.685)} = 2.44 \text{ psf}$$

total = 44.4 + 1.12 + .88 + 2.44
= 48.84 psf

Gravity System Design

Column Design (Below 2nd Floor)
(Int colum)

Typical Floor - 4
Roof - 1

$A_T = 697.51$ sf

Loads: SDL = 15 psf
DL = 50 psf
RDL = 40 psf
LL = 80 psf
RLL = 20 psf
SL = 20 psf

Load Combination
1.4D
* 1.2D + 1.6L + .5(Lr or S) - D.W.C.
1.2D + 1.6(Lr or S) + (L) - check
1.2D + L + .5(Lr or S) - D.W.C.

Axial Load acting on column below 2nd floor

$P_1 = 1.2(15+50) + 1.6(80) = 206$ psf

$P_R = 1.2(40+15+50) + .5(20) = 131$ psf

$P_u = [4(206)(697.51) + 131(697.51)]/1000 = 666.1^k$

Assume k = 1 $\quad L_{eff} = 14.16'$
By Table 4-1
W12x65 $\quad \phi P_n = 685 \quad 14'$

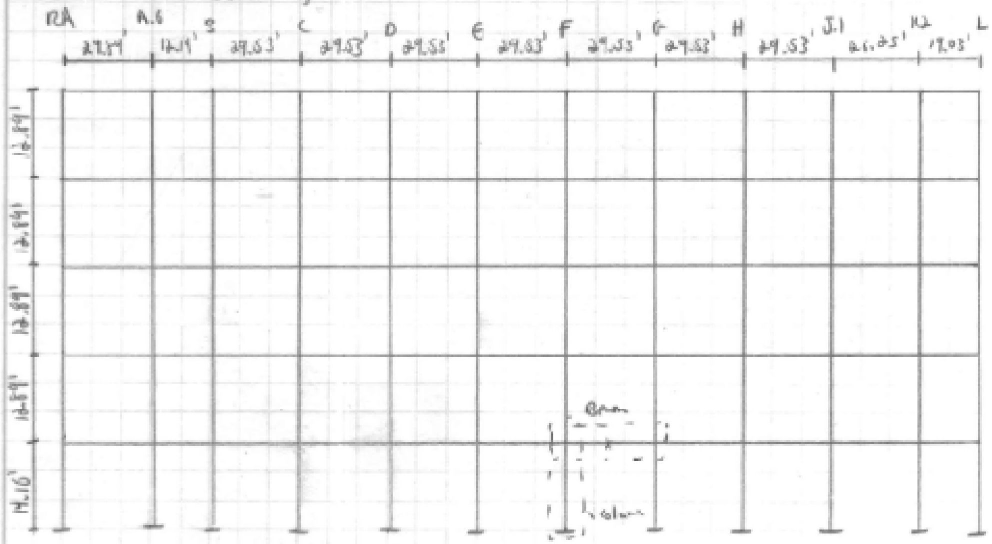
$\phi P_n = 662 \quad 15'$ $\quad$ for 14.16' = 681.3 k

$\phi P_n = 681.3^k > 666^k$ ok

use W12x65

Lateral System Design / check

- Moment Frame along Grid 4
  - Design Controlled by 1.2D + 1.6L, not lateral load controlled, verified by RAM



FLL = 80 psf
SDL = 15 psf
DL = 50 psf
RLL = 20 psf
RDL = 40 psf
WDL = 425 plf

Trib width of beam = 4.593 ft
Trib area of column = 406.91 SF / floor

Design Beam

FLL = 80(4.593)/1000 = .3674 k/ft
FDL = 65(4.593)/1000 = .2985 k/ft
wDL = .425 k/ft

$w_u = 1.2(.2985 + .425) + 1.6(.3674) = 1.456$ k/ft

Fixed - Fixed $M_u = \frac{wl^2}{12} = \frac{1.456(29.53)^2}{12} = 105.8$ ft kips

Bracing at midspan - unbraced length = 14.77 ft

W14x40 $\phi M_p = 177^{\text{ik}}$ $L_p = 5.26^{\text{ft}}$
$\phi M_r = 110^{\text{ik}}$ $L_r = 14.9^{\text{ft}}$
$L_r = 14.9^{\text{ft}} > 14.77^{\text{ft}}$ $\therefore$ $\phi M_r = 110^{\text{ik}}$ is conservative for ϕM_n
$V_u = 21.5^k < \phi V_n = 112^k$

Use W14x30 for Lateral Beam in moment frame

Lateral System Design

Lateral Column
- Moments balance due to equal bays, load is primary axial

$\left[\left[1.2(65)+1.6(80)\right]4+\left[1.2(40+65)+.5(20)\right]\right]406.9/1000 = 361.37^k$

$1.2(.425)29.53(5) = 75.3^k$

$P_u = 436.67^k$

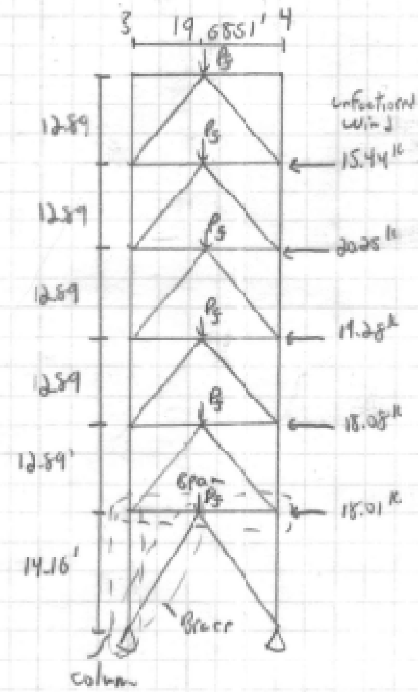
use W10x49 $\phi P_n = 450^k$ $L_e = 15'$

matches Ram Design

use W10x49 for lateral column

Lateral System Design

Braced Frame at Grid B



FLL: 80 psf
FDL: 50 psf
SDL: 15 psf
RLL: 20 psf
RDL = 40 + FDL psf = 90 psf
SL: 20 psf

- P_f is reaction from floor beam
- Trib width of beam: $\left(\frac{5.9384}{2} + \frac{29.5275}{2}\right) = 17.73$ ft
- Trib Area of column: $(17.733)\cdot\left(\frac{19.6851}{2} + \frac{27.559}{2}\right) = 418.89$ sf
- From Etabs Model
  Brace takes 37% of wind Load in N-S Direction

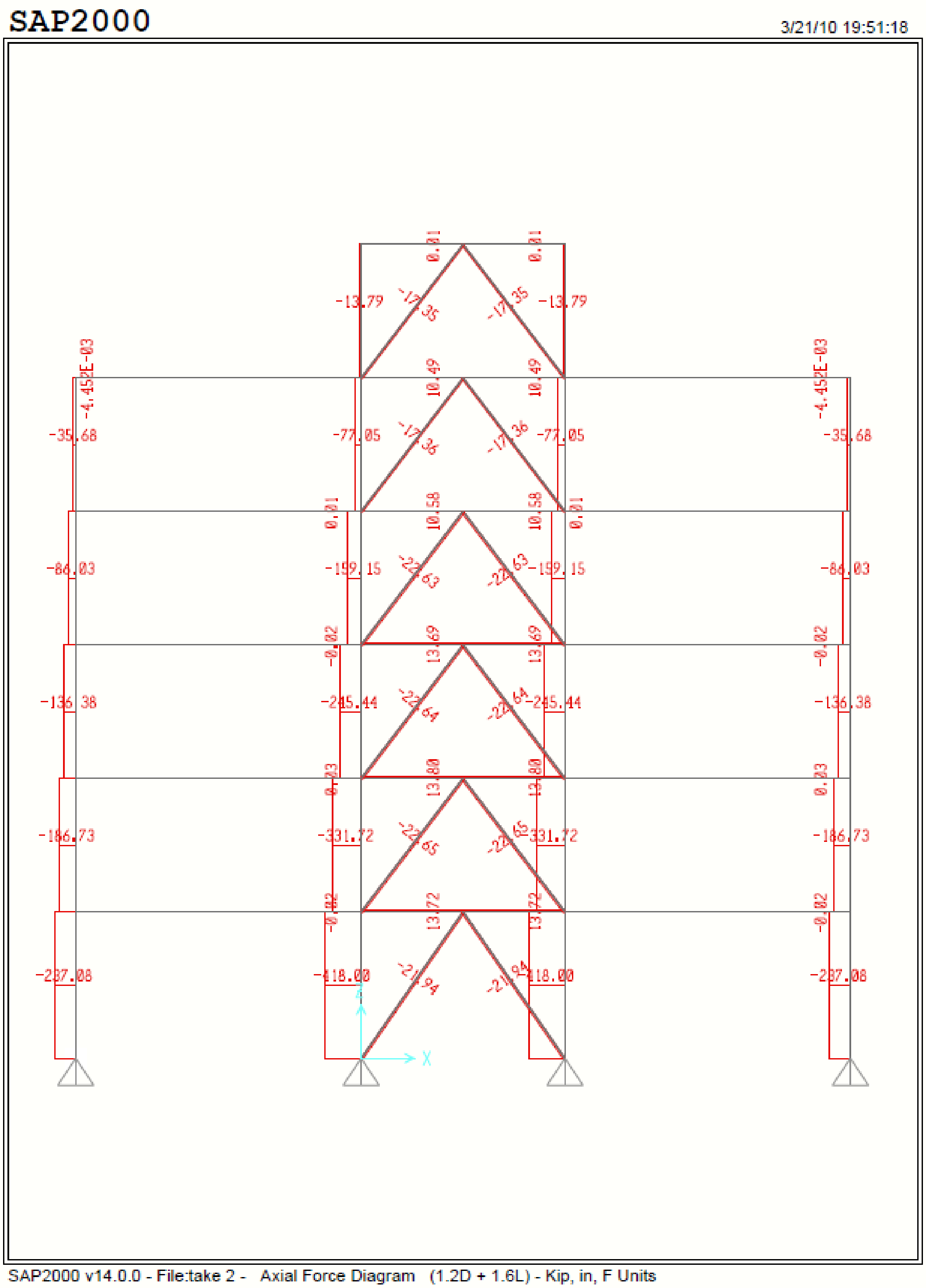
- Design Beam, Column, and Braces.
  - SAP 2000 2D Modeling was used to generate the axial forces in the braced frame.

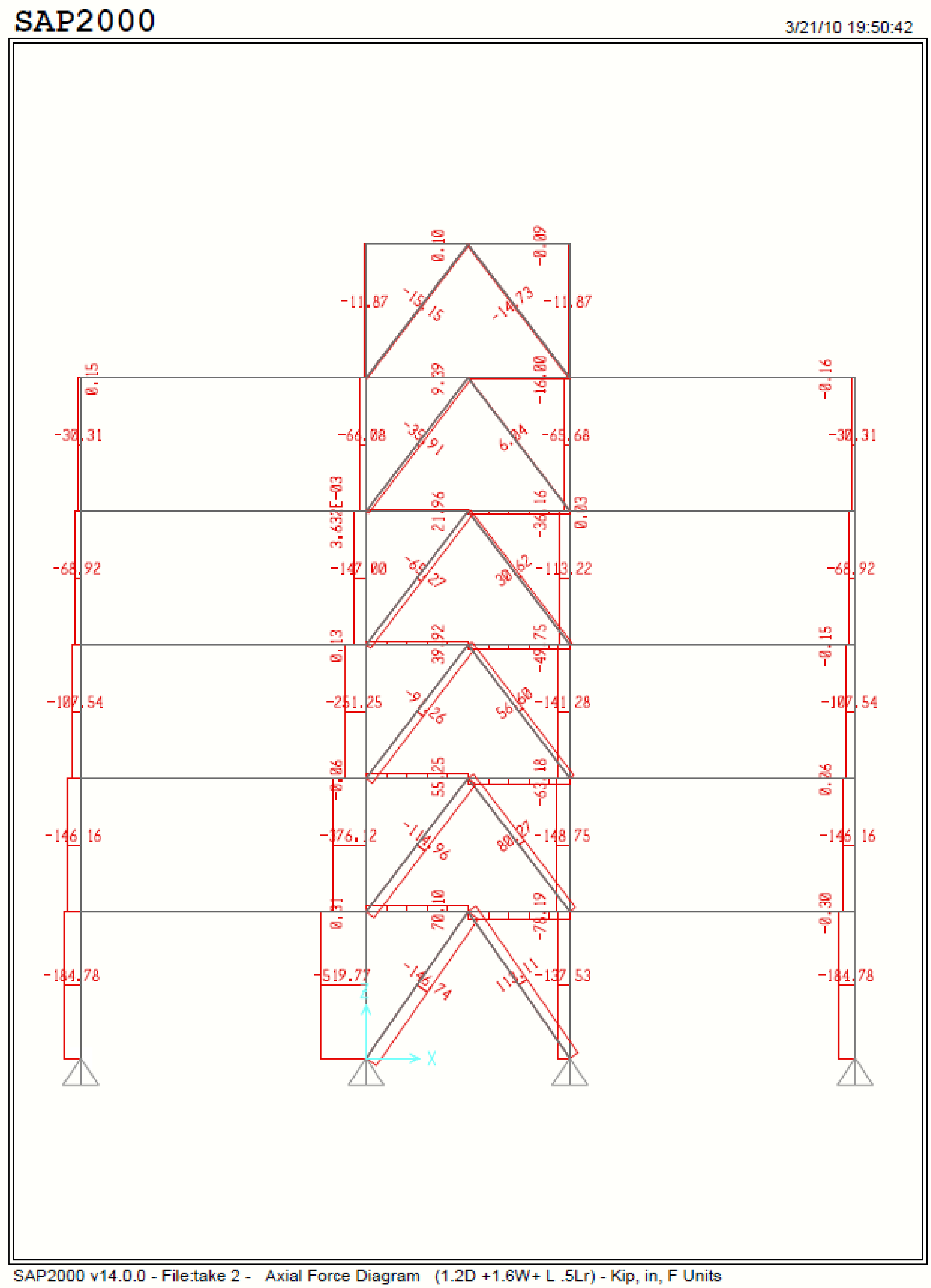
    3 Load Combinations were looked at

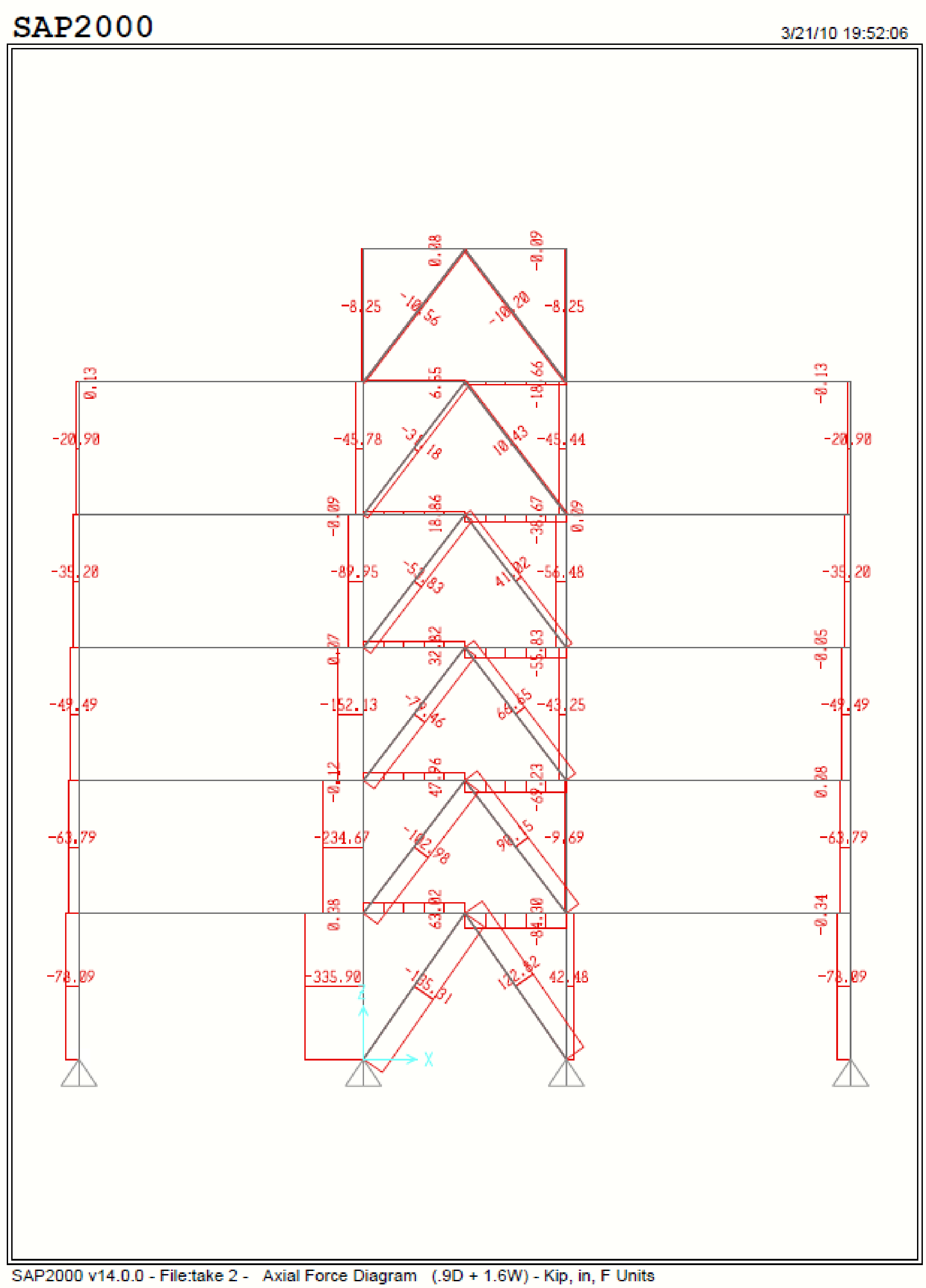
    $1.2D + 1.6L + .5L_r$

    $1.2D + 1.6W + L + .5L_r$

    $.9D + 1.6W$


SAP2000
3/21/10 19:51:18
SAP2000 v14.0.0 - File:take 2 - Axial Force Diagram (1.2D + 1.6L) - Kip, in, F Units


SAP2000
3/21/10 19:50:42
SAP2000 v14.0.0 - File:take 2 - Axial Force Diagram (1.2D +1.6W+ L .5Lr) - Kip, in, F Units


SAP2000
3/21/10 19:52:06
SAP2000 v14.0.0 - File:take 2 - Axial Force Diagram (.9D + 1.6W) - Kip, in, F Units

Lateral System Design

Braced Frame at Grid B

Use AISC 13th Table 4-1 - Axial Compression

- Beam at 2nd floor

$P = 78.19^k$

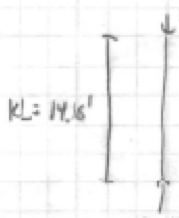
* The smallest W10 that meets slender ratio for compression is a W10x33 $\phi P_n = 143^k > P_u = 78.19^k$

Both $KL_x = KL_y$, so KL_y will control L_p.

Use W10x33

- Column below floor 2

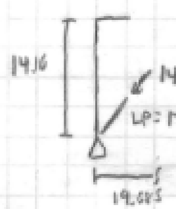
$P_u = 519.77^k$



Both $KL_x = KL_y$, so KL_y will control

Use W10x60 $\phi P_n = 555^k$ $L_p = 15' > 14.16'$ ok

- Brace below Floor 2



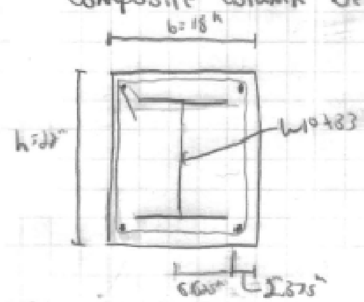
Both $KL_y = KL_x$ so KL_y will control design $KL = KL_y = 12.24'$

2L5x5x3/4" $\phi P_n = 152^k$ $L = 18'$ ok

* RAM Design use 2L6x6x3/4" Difference accounted for eccentric loading cases used in RAM that was not used in simple 2D model.

Gravity stystem Design

Composite Column Design



Column Effective length: 65.72k
Steel Shape: F_y = 50 ksi
Reinforcing #9 bars Gr 60, F_y=60
Concrete: f'_c = 5 ksi
E_c = 29,00 ksi

- Determine area

A_s = 9.71 in²
A_{sr} = 4(1) = 4 in²
A_c = 18(22) − 9.71 − 4 = 382.29 in²

- Check min steel

$\rho_s = \frac{A_s}{A_g} = \frac{9.71}{22(18)} = .0245 > .001$

$\rho_{sr} = \frac{4}{22(18)} = .0101 > .004$

- Determine P_o and P_e

C_2 = .85

C_4 = .5

$C_3 = .1 + 2\left(\frac{9.71}{382.29+9.71}\right) = .1495 < .3$

$I_s = I_y$ = 36.6 in⁴
$I_{sr} = 4(1.0)(6.25)^2 = 176$ in⁴

$I_c = \frac{22(18)^3}{12} - 36.6 - 176 = 10,479$ in⁴

P_o = 9.71(50) + 4(60) + .85(5)(382.29) = 1113.6k

EI_{eff} = 29,000(36.6) + .5(29,000)(176) + .15(3900)(10,479) = 9.74 x10⁶ in-kips

Gravity System Design

- Composite Column Design cont

$$P_e = \frac{\pi^2 EI_{eff}}{(KL)^2} = \frac{\pi^2(9.74 \times 10^6)}{(65.72 \times 12)^2} = 154.62 \text{ kips}$$

$$\frac{P_e}{P_o} = \frac{154.62}{1113.6} = .139 < .44$$

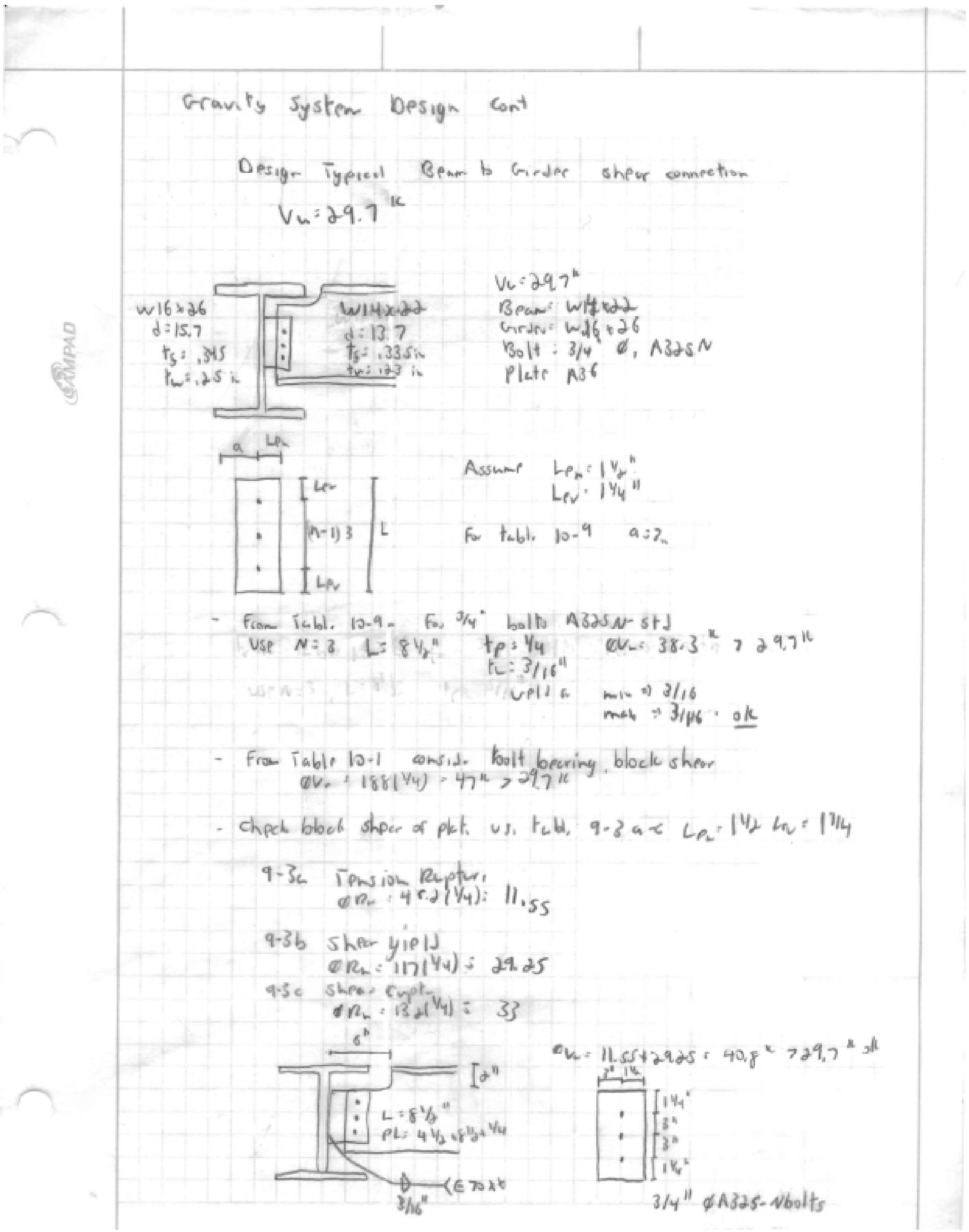
$$P_n = .877 P_e = .877(154.62) = 135.6 \text{ kips}$$

$$\phi P_n = .75(135.6) = 101.7 \text{ k}$$

$$P_u = 1.4(65.72 \times 33) = 8036.3 \text{ lbs}$$
$$+ 1.4(15' \times 26 \times 6) = ?$$
$$P_u = 6.3 \text{ kips} \quad \text{ok}$$

Appendix F: Connection Design

Typical Beam to Girder Connection



Gravity System Design cont

Design Typical Beam to Girder shear connection

$V_u = 29.7^k$

$V_u = 29.7^k$
Beam: W14x22
Girder: W16x26
Bolt: 3/4" Ø, A325N
Plate A36

Assume $L_{eh} = 1\frac{1}{2}$"
$L_{ev} = 1\frac{1}{4}$"

For table 10-9 a = 2 in

- From Table 10-9 - For 3/4" bolts A325N - std
  Use N = 3 L = $8\frac{1}{2}$" $t_p = 1/4$ $\phi V_n = 38.3^k > 29.7^k$
  $t_w = 3/16$"
  weld t_w min ⇒ 3/16
  max ⇒ 3/16 ∴ ok

- From Table 10-1 consider bolt bearing, block shear
  $\phi V_n = 188(1/4) = 47^k > 29.7^k$

- Check block shear of plt. vs. table 9-3 a-c $L_{eh} = 1\frac{1}{2}$ $L_{ev} = 1\frac{1}{4}$

9-3a Tension Rupture
$\phi R_n = 46.2(1/4) = 11.55$

9-3b Shear yield
$\phi R_n = 117(1/4) = 29.25$

9-3c Shear rupture
$\phi R_n = 132(1/4) = 33$

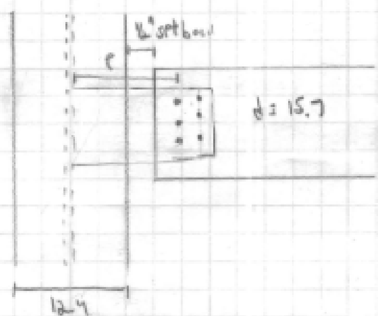
$\phi V_n = 11.55 + 29.25 = 40.8^k > 29.7^k$ ok

Typical Girder to Column Connection: Extended Shear Tab

Gravity System Design

Design Typical Girder to Column Connection (Shear Extended Shear Tab Connection

$V_u = 30.2^k$ (include self weight)



Given
$V_u = 30.2^k$
Beam: W16x26
Column: W12x79
Bolts: ¾" ⌀ A325N
Plate: A36
$e = \frac{(12.4 - .47)}{2} + \frac{1}{2} + 1\frac{1}{2} + \frac{3}{2} = 9.47''$
use $e = 9.5''$

- Required strength $= V_u = 30.2^k$
- Bolt A325 ¾" ⌀ bolt
  $\phi R_n = C(\phi r_n)$ $30.2 = C(15.9)$
  $C = 1.9$

From Table 7-8 pg. 7-38
for $s = 3''$ $e_x = 8''$, 2 rows of 4 bolts $C = 2.54 > 1.9$ ok
$\phi R_n = 2.54(15.9) = 40.4^k > 30.2^k$ ok

bearing on web
$\phi r_n = .75(2.4)(65)(.25)(.875) = 25.59^k > 15.9^k$ Bolt shear controls

bearing on plate (Assume Plate thickness of 3/8")
$\phi r_n = .75(2.4)(58)(.375)(.875) = 34.3^k > 15.9^k$
$\phi V_n = 46.6^k > 30.2^k$ ok

- Maximum plate thickness
  $1.25 F_v = 1.25(48) = 60$ ksi
  $A_b = .601$
  $C' = 26.0$ pg 7-38
  $M_n = 1.25 F_{nv} A_b C' = 60(.601)(26) = 937.56$
  $t_{max} = \frac{6 M_n}{F_y d^2} = \frac{6(937.6)}{36(11.5)^2} = 1.18'' > .375''$ ok
- Shear yielding of Plate
  $\phi R_n = (1.0)(.6F_y) A_g = 1.0(.6)(36)(11.5)(.375) = 93.15 > 30.2^k$ ok
- Shear rupture of
  $\phi R_n = .75(.6F_u) A_{nv} = .75(.6(58))(.375)[11.5 - 4(3/4 + 1/8)] = 78.3^k$
- Block shear by Table 9-3 a-c $L_{eh} = 1\frac{1}{2}$ $L_{ev} = 1\frac{1}{4}$ $n = 4$
  TR = 46.2
  SY = 166
  SR 188
  $\phi R_n = (46.2 + 166)(.375) = 79.6^k > 30.2^k$

Gravity System Design

Extended shear tab connection cont.

- check flexure using von mises shear reduction

$M_u = 30.2(9.5) = 286.9$ in-k

$S_v = \dfrac{(30.2/.9)}{9.5(.375)} = 9.42$ ksi

$F_{cr} = \sqrt{36^2 - 3(9.42)^2} = 32.09$ ksi

$\phi M_n = .9(32.1)\left(\dfrac{.375(11.5)^2}{4}\right) = 358.2 > 286.9$ in-kip

- check plate flexure rupture

$Z_{net} = Z_p - Z_{holes}$

$Z_p = \dfrac{.375(11.5)^2}{4} = 12.4$ in^3

$Z_{holes} = .375[3/4 + 1/8][2(1\tfrac{1}{2}) + 2(3+1\tfrac{1}{2})] = 3.94$ in-k

$\phi M_n = .75(58)(12.4 - 3.94) = 368.$ in-k > 286.9 in-k ok

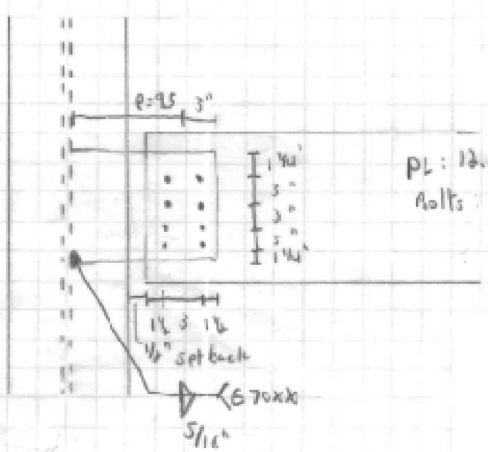
- check plate buckling

$F_{cr} = F_y Q$

$\lambda = \dfrac{h_o \sqrt{F_y}}{10 t_w \sqrt{475 + 280(h_o/c)^2}} = \dfrac{11.5\sqrt{36}}{10(.47)\sqrt{475 + 280\left(\frac{11.5}{8}\right)^2}} = .45$

$\lambda \le .7 \simeq Q = 1$

$F_{cr} = F_y$ plate buckling O.K.



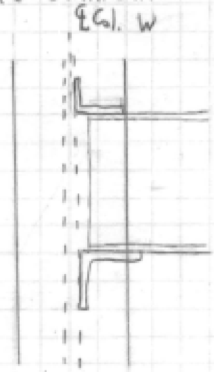
PL: 12.5" x 11.5" x 3/8"
Bolts: 2 Rows of 4 3/4"⌀ A325 N

Typical Girder to Column Web Connection: Seated

Gravity System Design

Design Typical Girder to Column web Connection

Seated Connection



Given:
$V_u = 30.2^k$
Beam: W16x26
Column: W8x79 [sic]
Bolts: 3/4" ⌀ A325N

- Required strength: $V_u = 30.2^k$
- Bolts A325 3/4" ⌀ bolts
- Constructability

T of Column: T: 9 1/8"
B_f of Beam: B_f: 5.5" Beam fits inside column web.

- Try L6x6x1" x 0'-5 1/2" A36

$R_n e = .9 F_y L t_a^2 / 4 \qquad e = \frac{N_{min}}{2} + 3/4 - t_a - 3/8 = \frac{N_{min}}{2} - 5/8$

$R_n = \frac{.9 F_y L t_a^2}{\left(\frac{N_{min}}{2} - 5/8\right)4} = \frac{.9(36)(5.5)(1)^2}{\left(\frac{N_m}{2} - 5/8\right)4} = \frac{44.55}{\left(\frac{N_m}{2} - 5/8\right)}$

① $N_{min} = \frac{R_u}{1 F_y t_w} - 2.5 k_d = \frac{R}{1(50)(.25)} - 2.5(.747)$

$N_{min} = \frac{44.55/\left(\frac{N}{2} - 5/8\right)}{1(50).25} - 2.5(.747) = \underline{2.78 \text{ in}}$

② when $N/d \le .2$

$N_{min} = \frac{d}{3}\left(\frac{t_f}{t_w}\right)^{1.5}\left[\frac{R_u}{.75(.4)t_w^2}\sqrt{\frac{t_w}{EF_y t_f}} - 1\right] \qquad t_w = .25 \quad d = 15.7 \text{ in}, \quad t_f = .345$

$\frac{15.7}{3}\left(\frac{.345}{.25}\right)^{1.5}\left[\frac{R_u}{.75(.4)(.25)^2}\sqrt{\frac{.25}{29{,}000(50)(.345)}} - 1\right] = 8.48[53.33 R_u(.000707) - 1]$

$N_{min} = .333\left[\frac{44.55}{N/2 - 5/8}\right] - 8.48 \qquad N_{min} = 3.69 \text{ in} \qquad N/d = \frac{3.69}{15.7} = .2357 > .2$

(2) cont w/ $N/d > .2$

$$N_{min} = \frac{d}{4}\left[\left(\frac{t_f}{t_w}\right)^{1.5}\left[\frac{R_u}{.75(.4)(t_w^2)}\sqrt{\frac{t_w}{E F_y t_f}} - 1\right] + .2\right]$$

$$= \frac{15.7}{4}\left[\left(\frac{.345}{.25}\right)^{1.5}\left[\frac{R_u}{.75(.4)(.25)^2}\sqrt{\frac{.25}{29{,}000(50)(.345)}} - 1\right] + .2\right]$$

$$N_{min} = 3.925\left[1.62\left(.0377 R_u - 1\right) + .2\right]$$

$$N_{min} = 3.925\left[.06107 R_u - 1.62 + .2\right]$$

$$N_{min} = .0397 R_u - 5.574 = .2397\left[\frac{44.55}{N_{min}/d - 5/8}\right] - 5.574$$

$$\underline{N_{min} = 3.58 \text{ in}}$$

(3) $N_{min} = k_{des} = 1\frac{1}{10}$

$$N_{min}\ 3.58 \qquad N/d = \frac{3.58}{15.7} = .228 > .2$$

Limit states

Beam Local web yield

$$\phi R_n = \phi(2.5k_{des} + N) F_{y_w} t_w$$
$$= 1\,(2.5(.747) + 3.58)(50)(.25) = 68.09^K$$

Beam Local web Crippling

$$N/d = \frac{3.58}{15.7} = .228 > .2$$

$$R_n = .4 t_w^2\left[1 + \left(\frac{4N}{d} - .2\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{E F_{y_w} t_f}{t_w}}$$

$$= .4(.25)^2\left[1 + \left(\frac{4(3.58)}{15.7} - .2\right)\left(\frac{.25}{.345}\right)^{1.5}\right]\sqrt{\frac{29{,}000(50)(.345)}{.25}} = 50.898^K$$

$$\phi R_n = .75(50.9) = \underline{38.17^K}$$

Seat Angle flexure

$$e = \frac{N_{rc}}{2} + \frac{3}{4} - t_a - 3/8 = \frac{3.58}{2} + \frac{3}{4} - 1 - \frac{3}{8} = 1.165 \text{ in}$$

$$\phi R_n = \frac{.9 F_y L_a t_a^2}{4e} = \frac{.9(36)(5.5)(1)^2}{4(1.165)} = 38.24^K$$

Angle shear yielding

$$\phi R_n = 1.0(.6 F_y) L_a t_a = 1(.6)(36)(5.5)(1) = 118.8^K$$

Weld rupture use tab. 8-4

Angle = 0°

$e = 3/4 + N/2 = 3/4 + \frac{3.58}{2} = 2.54"$

$a = \frac{e}{L} = \frac{2.54}{5.5} = .462 \Rightarrow$ $a = .4$ $c = 2.60$; $a = .5$ $c = 2.29$; $a = .462$ $c = 2.44$

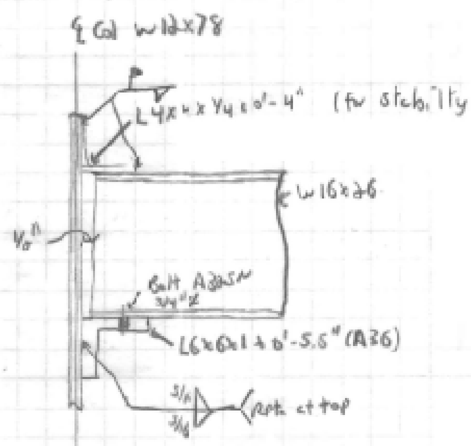
$\phi R_n = \phi C C_1 D l = .75(2.44)(1)(5)(6) = \underline{54.9^k}$ (5/16)

Base metal strength

$\phi R_n = .75(.6F_y A_{nv})$ $A_{nv} = t_{w,min}$

$= .75(.6)(50)(.47)(6) = 63.45^k$

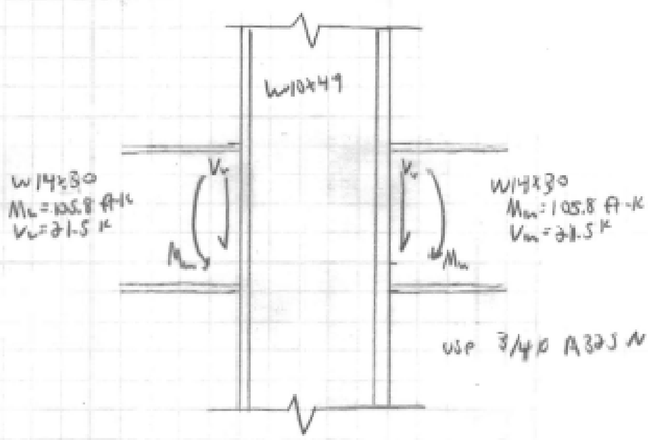
$\phi V_n = 38.17^k$
web crippling controls



Typical Moment Connection: Beam to Column Flange

Lateral System Design

– Design Beam to Column Flange Moment connection



use 3/4ø A325 N STD

Beam: W14x30 F_y = 50 ksi F_u = 65 ksi
d = 13.8 in b_f = 6.73 in t_w = .27 in
A = 8.85 in² t_f = .385 in S_x = 42 in³

Column: W10x49 F_y = 50 ksi F_u = 65 ksi
d = 10 in b_f = 10 in t_w = .34 in
A = 14.4 in² t_f = .56 in S_x = 54.6 in³

– Check the beam available flexural strength
Assume two rows of bolts STD

$A_{fg} = b_f t_f = 6.73(.385) = 2.59 \text{ in}^2$

$A_{fn} = A_{fg} - 2(d_b + 1/8)\, t_f = 2.59 - 2(3/4 + 1/8).385 = 1.92 \text{ in}^2$

$\frac{F_y}{F_u} = \frac{50}{65} = .769 < .8 \therefore Y_t = 1.0$

$F_u A_{fn} = 65(1.92) = 124.56^k$

$Y_t F_y A_{fg} = 1.0(50)(2.59) = 129.5^k > 124.56^k$

$M_n = \frac{F_u A_{fn} S_x}{A_{fg}} = \frac{65(1.92)(42)}{2.59} = 2019.81 \text{ k-in} = 168.32 \text{ k-ft}$

$\phi M_n = .9(168.32) = 151.5$ k-ft

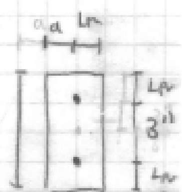
$\phi V_n = 112^k$

Lateral System Design

· Moment Connection Continued

– Design Single Plate Connection

$V_u = 21.5^k$



Assume $L_{eh} = 1\frac{1}{2}$"
$L_{ev} = 1\frac{1}{4}$"
For table 10-9 a = 3.5 anything < 3" is conservative

– From Table 10-9a F 3/4" bolt A325 N-STD
use N = 2   L = 5 1/2"   tp = 1/4" $\phi V_n = 24.5^k > 21.5^k$
tmax = 3/16"
weld min = 3/16
min = 3/16 ok

– From Table 10-1 consider bolt bearing, block shear

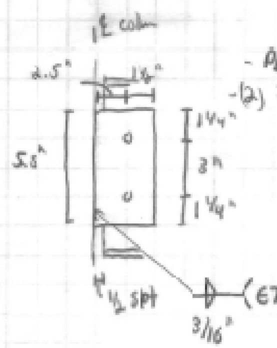
$\phi V_n = 126(1/4) = 31.5^k > 21.5^k$

– Check block shear of plate w/ table 9-3 a-c $L_{eh} = 1\frac{1}{2}$ $L_{ev} = 1\frac{1}{4}$
*controls over thicker web of beam

9-3a Tension Rupture
$\phi R_n = 46.2(1/4) = 11.55$

9-3b Shear yield
$\phi R_n = 68.8(1/4) = 17.2^k$

9-3c Shear rupture
$\phi R_n = 76.7(.25) = 19.175^k$ $\phi R_n = 11.55 + 17.2 = 28.75^k > 21.5^k$ ok



– PL : 4" x 5 1/2" x 1/4"
–(2) 3/4" A325N STD

Lateral System Design

- Moment Connection Cont.

  - Design tension flange and connection
    - Design Bolt

      $P_{uf} = \frac{M_u}{d} = \frac{105.8(12)}{13.8} = 92^k$

      what thickness would the plate have to be for tension yield

      $\phi R = \phi A_g F_y \quad b_f = 6.73 \quad w_g = 6"$

      $92 = 6\, t_p (36) = .426 \text{ in} \quad$ use $t_p = \frac{1}{2}"$

      Try PL $6" \times 1/2"$ $\quad t_f = .56 \text{ in}$ A992
      $t_p = .5$ A36 → plate controls

      shear $\phi r_n = 15.9\ ^k/\text{bolt}$

      bearing on flange $\phi r_n = 78.3\ ^k/\text{bolt}$ from table 7-5

      tearout $\phi r_n = \phi 1.2 F_u l_c t = .75(1.2)(58)(1.5 - \frac{1}{2}(3/4 + 1/16))\frac{1}{2}$
      $= 28.55\ ^k/\text{bolt}$

      $n = \frac{92}{15.9} = 5.79$ use 2 rows of 3 bolts
      A325 $3/4"\ \phi$

  - Check plate tension yielding

    $P_n = F_y A_g = 36(6)(.5) = 108^k$

    $P_{uf} = \frac{M_u}{d + t_p} = \frac{105.8(12)}{13.8 + \frac{1}{2}} = 88.78^k \quad \phi P_n = .9(108) = 97.2^k > 88.78^k$ ok

  - check plate tension rupture

    $A_n = [B_g - 2(d_b + 1/8)]\, t_p = [6 - 2(3/4 + 1/8)]\frac{1}{2} = 2.125 \text{ in}^2$

    $P_n = F_u A_n = 58(2.125) = 123.25 \quad \phi P_n = .75(123.25) = 92.44^k > 88.78^k$

  - Check flange block shear of plate $\quad L_{ev} = 1\frac{1}{2}" \quad L_{eh} = 1\frac{1}{2}"$

    9-3a $\quad TR = 54.4^k(\frac{1}{2})(2) = 54.4^k$

    $SY = 121(\frac{1}{2})(2) = 121^k$

    $SR = 139(\frac{1}{2})(2) = 139^k \quad \phi R_n = 54.4 + 121 = 175.4^k > 88.78^k$

  - Determine the req. of the fillet weld to support column flange

    applied the req. is $\perp (90°)$

    $D_{min} = \frac{88.78}{2(1.5)(1.392)(6)} = 3.54 \quad$ use $1/4"$ weld

  - check base metal strength

    $\phi R_n = .75(.6)(65)(.56)(6) = 98.28^k > 88.78^k$ ok

Lateral System Design

- Moment Connection Cont.

- Design compression flange plate

Use PL 6" x 1/2"

Assume $K = .65$ $L = 2\frac{1}{4} = 1\frac{1}{2} + \frac{1}{2}$ set

local buckling

$\frac{b_{fp}}{t_p} \le \frac{253}{\sqrt{36}}$ $\frac{6.73}{.5} \le \frac{253}{\sqrt{36}}$ $13.46 < 46.17$ ok

Flexural buckling $\frac{KL}{r} = \frac{.65(2)}{\sqrt{\frac{.5^2}{12}}} = 9.01 < 25$ $F_{cr} = F_y$

$P_n = \phi F_{cr} A_g$

$.9(36)(6)(\frac{1}{2}) = 97.2^k > 88.78^k$ ok

Comp flange plate to be design to match tension side for load reversal

- Column Side Limit states

Local Flange bending

$T_u \le \phi R$ $\phi = .9$

$R_n = 6.25\, t_f^2\, F_y = 6.25(.56)^2(50) = 98^k$

$T_u = 88.78 < 98^k$ No stiffener req.

Local web yielding

$T_u = C_u \le \phi R$

$R_n = \phi F_{yc}(5k_c + N)\, t_{wc} = 50[5(1.06 + .5)].56 = 160.4 > T_u = C_u$ ok

Local web crippling

$C_u \le \phi R$ $\phi = .75$

$R_n = .8\, t_w^2 \left[1 + 3(N/d)\left(\frac{t_{wc}}{t_f}\right)^{1.5}\right] \sqrt{\frac{E F_{yc} t_{fc}}{t_{wc}}}$

$.8(.34)^2\left[1 + 3(\frac{.5}{10})(\frac{.34}{.56})^{1.5}\right] \sqrt{29{,}000(50)(\frac{.56}{.34})} = 167.3^k$

$\phi R_n = .75(167.3) = 125.5^k > T_u = C_u$ ok

web buckling

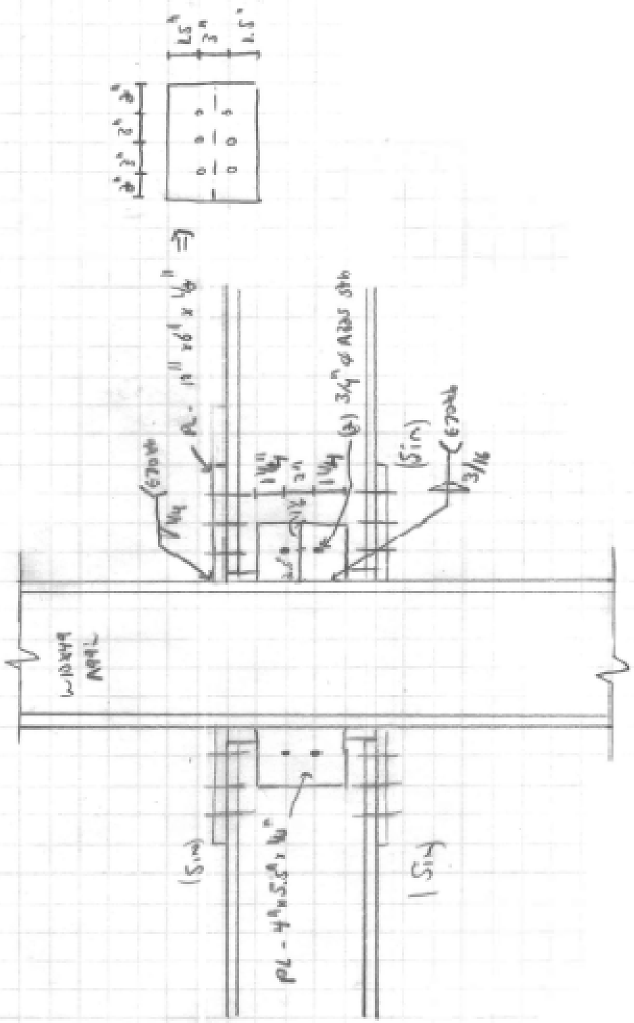
$C_u \le \phi R$ $\phi = .9$

$R_n = 24\sqrt{E F_{yc}}\left(\frac{t_{wc}^3}{h}\right) = 24\sqrt{29{,}000(50)}\left(\frac{.34^3}{7.5}\right) = 151.5$

$\phi R_n = .9(151.5) = 136.2^k > C_u$ ok

Lateral system Design cont.

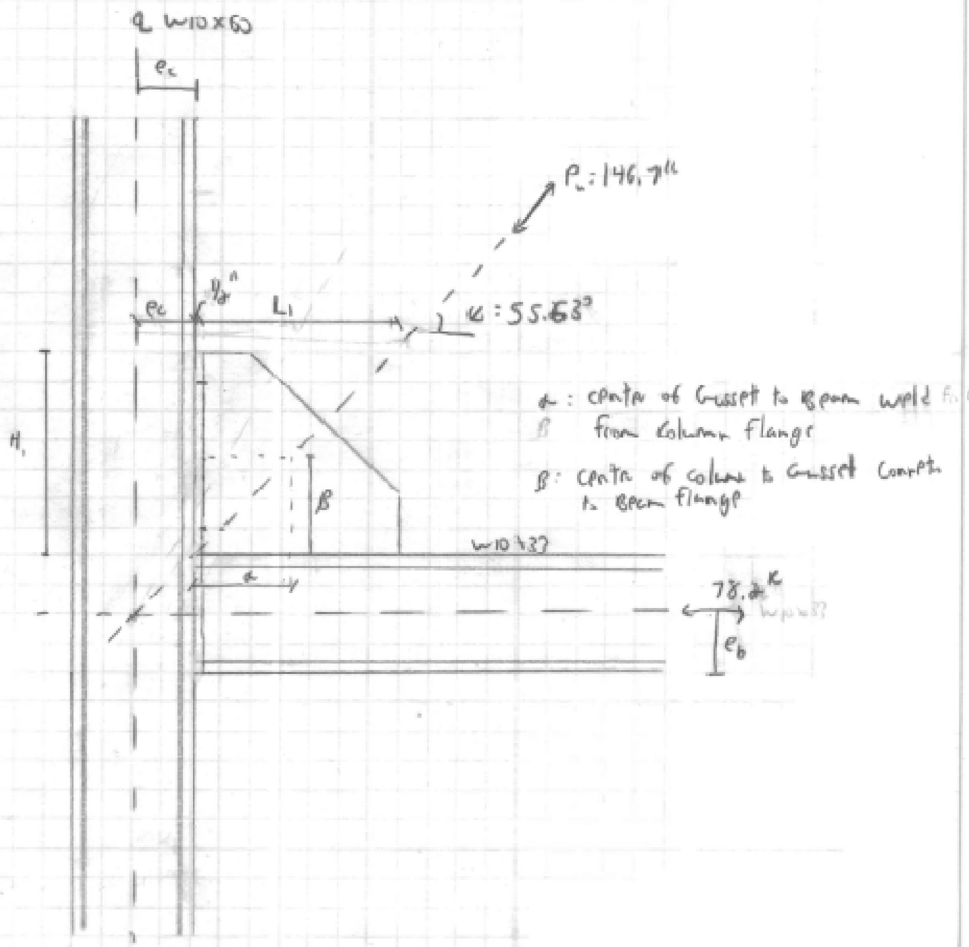
Moment Connection Design



Typical Heavy Braced Connection

Lateral System Design

Design a Braced Connection using General Uniform force



Column: W10x60

A: 17.6 in²	t_w: .42 in	t_f: .77 in
d: 10.2 in	b_f: 10.1 in	T: 7.5 in
e_c: 5.1 in		

Beam: W8x37

A: 9.71 in²	t_w: .29 in	t_f: .485 in
d: 9.73 in	b_f: 8 in	T: 7.5 in
e_b: 4.865 in		

Lateral System Design

- Limit States for Brace - 2L5"x5"x3/4"

Note $A_e = A_n U$ $\quad U = 1 - \frac{\bar{x}}{L}$ $\quad \bar{x} = 1.52 \text{ in}$

L = Length of connection

*each angle needs to looked at separately

1. Tension Yield

Assume 1 row of bolts 3/4"φ A325N std in Double Shear

$\phi R_n = \phi F_y A_g = .9(36)(14) = 453.6^k > 146.7^k$ ok

* Note Compression controlled size of angle.

2. Tension Rupture

$\phi R_n = \phi F_u A_e$

$A_e = A_n U$ $\quad U = 1 - \frac{1.52}{15} = .899$

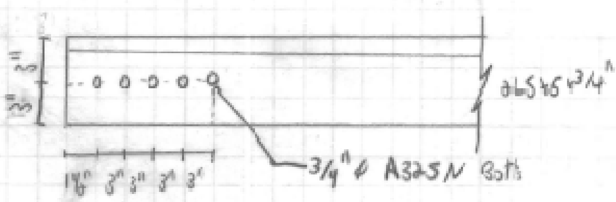
assumes (5) 3/4"φ Bolts in double shear

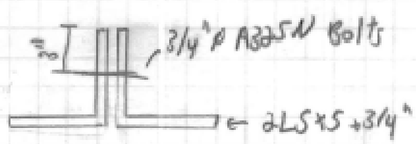
$A_g = 6.94 \text{ in}^2$

$A_n = 6.94 - 1(3/4 + 1/8)3/4 = 6.28 \text{ in}^2$

$A_e = .899(6.28) = 5.65 \text{ in}^2 \leq .85 A_g = 5.899 \text{ in}^2$

$\phi R_n = .75(58)(5.899)(2) = 513.2^k > 146.7^k$ ok





Lateral System Design

- Limit states for Brace Cont.

3. Block Shear

Using Table 9-3 $L_{eh} = 3''$ $L_{ev} = 1\frac{1}{2}''$

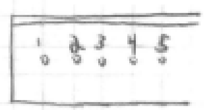
9-3a $T_R = 111\ ^k/_{in}$

9-3b $S_Y = 219\ ^k/_{in}$

9-3c $S_R = 250\ ^k/_{in}$

$\phi R_n = \overset{2\ \text{Angles}}{2}\left[\underset{\text{non uniform load}}{.5}(111).75 + .5(219)\ 3/4\right] = 247.5^k > 146.7^k$

- Limit states for Bolts



- Bolt shear = $15.9 \times 2 = 31.8^k$

- Bearing on Angle $= \phi 2.4 F_u t d_b$
$= .75(2.4)(58)(2)(.75)(.75) = 117.45^k$

- Bearing on Plate (assume ½" thick)
$= .75(2.4)(58)(.5)(.75) = 39.15^k$

- tearout Angle edge (Bolt 1)
$= \phi 1.2 F_u l_c t$
$= .75(1.2)(58)[1.5 - ½(3/4 + 1/16)](2)(3/4) = 85.64^k$

- tearout Angle middle (Bolt 2-5)
$= .75(1.2)(58)[3 - 1(3/4 + 1/16)]2(3/4) = 171.28^k$

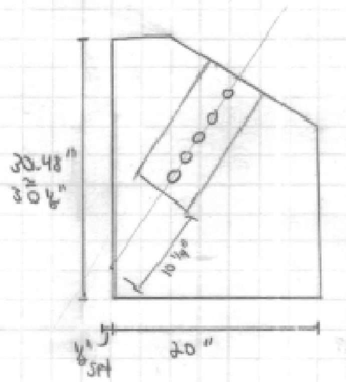
- tearout Plate edge (Bolt 5)
$= .75(1.2)(58)[1.5 - .5(3/4 + 1/16)].75 = 28.55^k$

- tearout Plate middle
$= .75(1.2)(58)[3 - 1(3/4 + 1/16)].75 = 57.09^k$

$\phi R_n = 4(31.8) + 28.55 = 155.75^k > 146.9^k$ ok

Lateral System Design

- Limits State for Gusset



* Gusset Dimmensions were designed using geometry of connection and the Uniform force method.

PL : $30\frac{1}{2}" \times 20" \times \frac{3}{4}"$

- Block shear
  * TR Does not occur

  Table 9-3b SY : $219(\frac{3}{4}) = 164^k$

  Table 9-3c SR : $250(\frac{3}{4}) = 187.5^k$

  $\phi R = 164^k > 146.5^k$ ok

- Gusset Yielding

  Whitmore section $\quad 12 \tan(30) = 6.93"$

$A_w = 2(6.93)(.75) = 10.39\ in^2$

$R_n = A_w F_y = 36(10.39) = 374.12\ k$

$\phi R_n = .9(374.12) = 336.71^k > 146.7^k$ ok

- Gusset Buckling

$r = \frac{t}{\sqrt{12}} = \frac{.75}{\sqrt{12}} = .217$

$l = 10.25\,in \qquad \frac{kl}{r} = \frac{1.2(10.25)}{.217} = 56.68 < 134$ ok

$F_e = \frac{\pi^2 E}{(\frac{kL}{r})^2} = \frac{\pi^2(29{,}000)}{(56.68)^2} = 89.1\ ksi$

$F_{cr} = \left[.658^{F_y/F_e}\right] F_y = (.658^{36/89.1})36 = 30.4\ ksi$

$P_n = F_{cr} A_w = 30.4(10.39) = 315.84^k$

$\phi P_n = .9(315.84) = 284.3^k > 146.7^k$ ok

Lateral System Design

Uniform force method

$\alpha = ?$ $e_b = 4.865''$ $\phi = 90 - 52.64 = 32.36°$

$\beta = 30.48/2 =$ $e_c = 5.1$ in

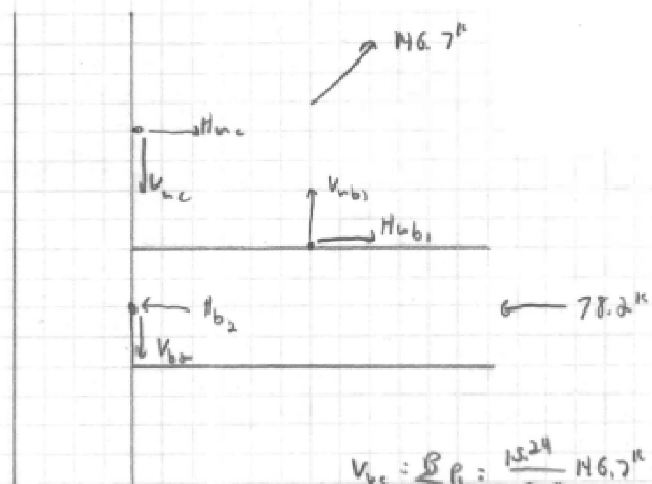
$\alpha = 4.865 \tan(32.36) - 5.1 + 15.24 \tan(32.36)$

$\alpha = 10.25$

$L_1 = 2\alpha - 1/2 = 2(10.25) - 1/2 = 20''$ good

$r = \sqrt{(10.25 + 5.1)^2 + (30.48/2 + 4.865)^2} = 25.29$ in

Gusset to



$V_{uc} = \frac{\beta}{r} P_u = \frac{15.24}{25.29} 146.7^k = \underline{88.39^k}$

$H_{uc} = \frac{e_c}{r} P = \frac{5.1}{25.29} 146.7 = \underline{29.58^k}$

$H_{ub1} = \frac{10.25}{25.29} 146.7^k = \underline{59.45^k}$

$V_{ub1} = \frac{4.865}{25.29} 146.7^k = \underline{27.17^k}$

$H_{b2} = 78.2 - 59.45 = \underline{18.75^k}$

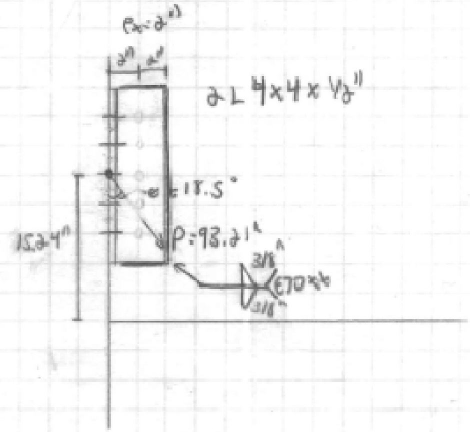
$V_{b2} = \underline{-27.17^k}$

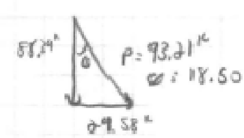
Lateral System Design

Design Double Angle Gusset to Column Connection

$V_{uc} = 88.34^k$
$H_{uc} = 29.58^k$ (Rigid)

Assume 10 3/4" ∅ A325 Bolts, 2 rows of 5 on column side of a double angle connection





- check weld strength

$l = 15"$
$kl = 3.5" = 4 - 1/2 = 3.5"$ $\quad k = .233$
$al = \dfrac{0(15) + 2(3.5)(3.5/2)}{15 + 3.5 + 3.5} = .557$ $\quad a = .037$

Table 8-8 θ = 15°

	k = .2	k = .233	k = .3
a = 0	3.01		3.56
a = .037		$\boxed{}$	
a = .1	2.87		3.40

θ = 30°

	k = .2	k = .233	k = .3
a = 0	3.00		3.41
a = .037		$\boxed{}$	
a = .1	3.13		3.62

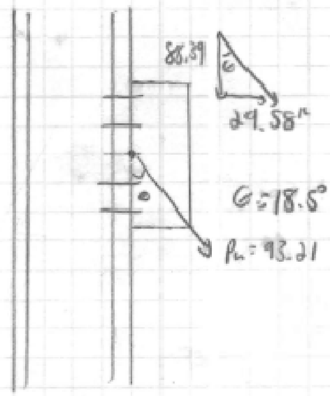
C = 3.149

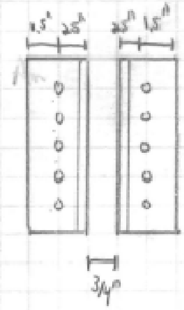
3/8

$\phi R_n = .75(3.149)(1.0)(6)(15) = 212.56^k > 93.21^k$ OK

Lateral System Design

Cont. Design Double





Angle
L4x4x5/8
Bolts
3/4" ⌀ A325N
$L_{eh} = L_{ev} = 1.5"$
S=3"

1. Calculate shear stress in bolts

$$f_v = \frac{V_u}{\# A_b} = \frac{88.39}{10(.442)} = 19.998" = 20\text{ ksi}$$

2. calculate availbl tensile per bolt

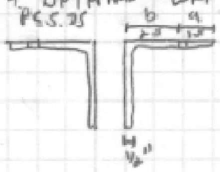
$$F'_t = \sqrt{90^2 - 6.25(20)^2} = 74.84\text{ ksi}$$

$$\phi r_{nt} = .75(74.84)(.442) = 24.808\text{ K}$$

3. Calculate r_{nt}

$$r_{nt} = \frac{29.58}{10} = 2.958\text{ k}$$

4. Determine whether prying force will occur



P_n | 15/5 = 3"
5.75

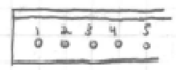
$$r_{ut} > \frac{\phi F_u P t_m^2}{4b'} = \frac{.9(58)(3)(.5)^2}{4(1.875)} = 5.22\text{ K}$$

a = 1.5"
a' = 1.5 + .25/2 = 1.88"
b = 2.5 - (1/2)/2 = 2.25"
b' = 2.25 - (3/4)/2 = 1.875"

$r_{ut} \leq 5.22$" Pyield
no prying

Lateral System Design

- Gusset to Column Double Angle
  - no prying,
  - check shear load $V_u = 88.39^k$
  - Bolt shear, bearing, tearout
    - Bolt shear $= \underline{15.9}^k$



    - Bolt bearing on Angle
      $= .75(2.4)(58)(1/2)(3/4) = 39.15^k$
    - Bearing on column flange
      $= .75(2.4)(65)(.42)(.75) = 36.9^k$
    - tearout Angle edge (Bolt 1)
      $= .75(1.2)(58)[1.5 - 1/2(3/4 + 1/16)].5 = 28.55^k$
    - tearout Angle middle (Bolt 2-5)
      $= .75(1.2)(58)[3 - (3/4 + 1/16)].5 = 57.09^k$
    - tearout column middle
      $= .75(1.2)(65)[3 - (3/4 + 1/16)].42 = 53.75^k$

$\phi R_n = 5(15.9)2 = \underline{159^k} > 88.39^k$

  - Block shear

    Using table 9-3 $L_{ev} = 1\frac{1}{2}"$ $L_{eh} = 1\frac{1}{2}"$
    9-3a $= 46.2(.5) = 23.1$ "
    9-3b $= 219(.5) = 109.5$ "
    9-3c $= 250(.5) = 125$ "

$\phi R_n = 2(23.1 + 109.5) = \underline{165.8}^k$

  - Shear yield of Angles
    $A_g = 7.49 \text{ in}^2$
    $\phi .6F_yA_{gv} = 1.0(.6F_yA_{gv}) = .6(36)(7.49)2 = 262.7^k$

  - shear rupture of Angles
    $A_n = 7.49 - 2(3/4 + 1/8).5 = 6.62 \text{ in}^2$

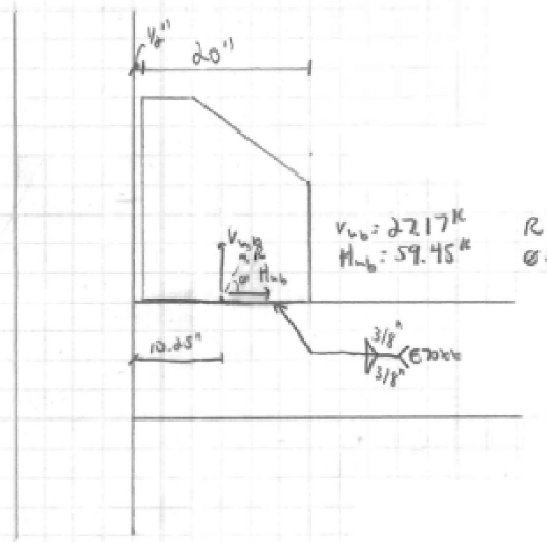
    $\phi .6F_uA_{nv} = .75(.6)(58)(6.62) = \underline{172.78}^k$

Connection over design.
<u>ok</u>

8

Lateral System Design

Braced Connection Design Continued



- Determine strength of weld group
  use Table 8-4

$al = 0"$ $\quad kl = .75"$
$l = 20"$ $\quad k = .0375"$

$\theta = 15°$

	$k = .0$	$k = .0375$	$k = .1$
$a = 0$	3.84		3.84

$\theta = 30°$

$a = 0$	4.18		4.18

$\theta = 24.56$ $\quad C = 4.057$

$\phi R_n = .75(4.057)(1.0)(6)(20) = \underline{365.13^k} > 65.36^k$ ok

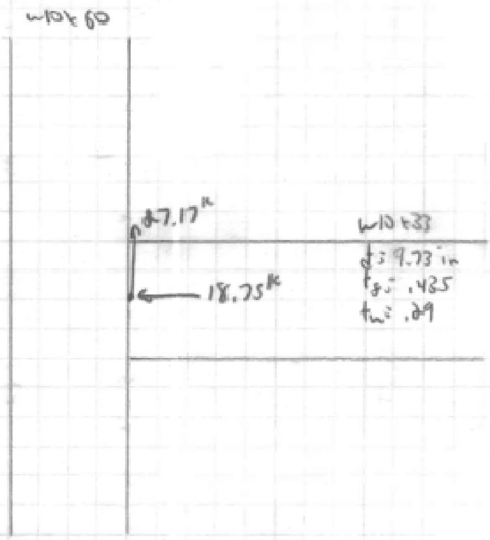
- check Base metal strength

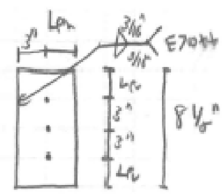
$\phi R_n = .75(.6)F_y A_{nw} = .75(.6)(50)(.435)(20)(2) = 391.5^k$ ok (t_f beam)

Lateral System Design

Braced Connection Design Cont.

- Design Beam to Column Connection





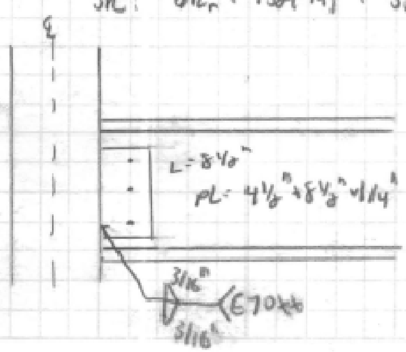
$L_{eh} = 1\frac{1}{2}"$
$L_{ev} = 1\frac{1}{4}"$
For table 10-9 a = 3

- From Table 10-9 f. 3/4" bolts A325 N-std
  Use N = 3 L = 8 1/2 $t_p = 1/4"$ $\phi V_n = 38.8^k > 27.17^k$
  $t_w = 3/16"$
  $t_{w\,min} = 3/16"$ ok

- From Table 10-1 a = 50 bolt bearing, block shear
  $\phi V_n = 188(1/4) = 47^k > 27.17^k$

- Check block shear of plate use 9-3

9-3c TR: $\phi R_n = 46.2(1/4) = 11.55^k$
SY: $\phi R_n = 117(1/4) = 29.25^k$
SR: $\phi R_n = 132(1/4) = 33^k$ $\phi R_n = 40.8^k > 27.17^k$



Appendix G: Foundation Design

Gravity System Design

Foundation Design

$P_D = [(15+50)4(697.51) + (40+15+50)(697.51)]/1000 \; 254.59^k$

$P_L = [80(4)697.51]/1000 = 223.2^k$

$P_{LR} = 20(697.51)/1000 = 13.95^k$

Allowable Bearing Capacity = 5994 psf

-Steel column to 24"x24" Concrete Pier to Spread Footing
Design and Detail spread Footing

$P = P_D + P_L = 254.59 + 223.2 + 13.95 = 491.74^k$

$q_a \geq \frac{P}{A} = \quad 5.99\ ksf \geq \frac{491.75}{B^2}$

$B \geq 9.06'$

use $B = 10$ ft

$P_u = 1.2(254.59) + 1.6(223.2) + .5(13.95) = 669.6^k$

$q = \frac{P_u}{A} = \frac{669.6}{10^2} = 6.696\ ksf$

$= 46.5\ psi$

$v_c = \phi 4\sqrt{f'_c} = .75(4)\sqrt{3000} = 164\ psi$

The equation for 2-way way shear controls by inspection

$d^2[v_c + q/4] + d[v_c + q/2]w = [q/4(B^2 - w^2)]$

$d^2[164 + \frac{46.5}{4}] + d[164 + \frac{46.5}{2}]24 = \frac{46.5}{4}(120^2 - 24^2)$

$175.63\,d^2 + 4494\,d = 16{,}0704 \quad d = 20.05''$

$h = 20.05 + 3 + .75 = 23.7996''$ use $h = 24''$ $d = 20.25$

Gravity System Design Cont.

Use $h = 24''$

$d = 24 - 3 - .75 = 20.25''$

$l = \frac{10' - 2'}{2} = 4'$

$M_u = \frac{q l^2}{2} = \frac{6.696(4)^2}{2} = 53.568 \text{ ft}\cdot\text{k}$

$a = \frac{A_s f_y}{.85 f'_c b} = 1.96 A_s$

$M_u = \phi M_n = \phi A_s f_y (d - a/2)$

$53.568(12) = .9 A_s (60)\left[20.25 - \frac{1.96 A_s}{2}\right)$ $\quad A_s = .606 \text{ in}^2$

use #7 @ 11'' o.c $\quad < 18''$ ok

$A_s = .655 \text{ in}^2$

$\rho = \frac{A_s}{bh} = \frac{.655}{(12)24} = .0022 \geq .0018$

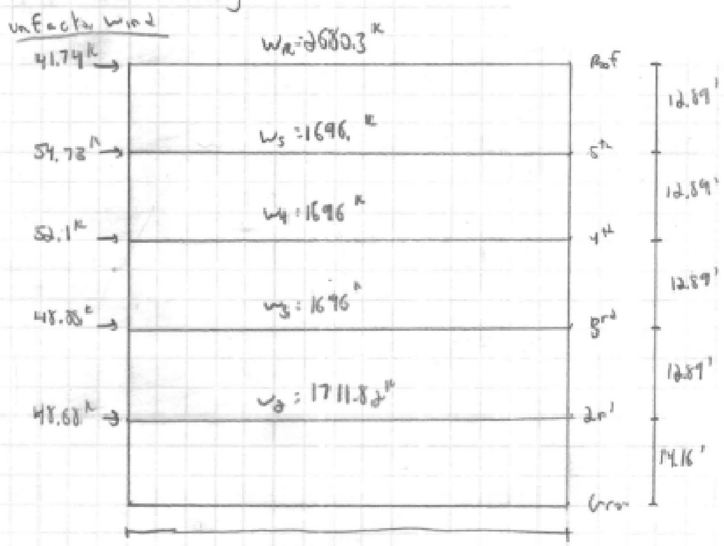
$a = 1.96 A_s = 1.96(.655) = 1.284''$ $\quad c = \frac{1.284}{.85} = 1.51 \text{ in}$

$\varepsilon_s = \frac{.003}{1.51}(20.25 - 1.51) = .037 \text{ in/in} > .005 \text{ in/in} \quad \therefore \phi = .9$

<u>use (11) #7 both ways</u>

Fondation check

-overturning



check overturning - general stability of structure

overturning moment

$$1.6\left[41.74(65.72)+54.73(52.83)+52.1(39.94)+48.88(27.05)+48.68(14.16)\right] = 15{,}563.1 \text{ ft-k}$$

$$\text{uplift} \quad T = \frac{15563.1}{74.8} = 208.1 \text{ k}$$

Gravity Load to resist uplift => .9D

$$.9(2680.3+1696+1696+1696+1711.82) = 8532.1 \;\Rightarrow\; 208.1 \text{ k}$$

* Building passes general stability check
* lateral baseplates should be designed to take tension

Appendix H: Cost and Schedule Takeoffs

Orignal Design Takeoff

Floor	Slab				Beam			Column		
	Concrete		Reinforcing	Finishing	Concrete		Reinforcing	Concrete		Reinforcing
	CY	SFCA	Ton	SF	CY	SFCA	Ton	CY	SFCA	Ton
Ground	385.63	0	120.42	20824						
2nd	586.15	17407.28	116.23	20098.55	196.75	7100.31	65.02	168.32	4574.06	33.38
3rd	607.30	18132.73	120.42	20824	196.75	7100.31	65.02	153.30	4165.95	30.40
4th	607.30	18132.73	120.42	20824	196.75	7100.31	65.02	153.30	4165.95	30.40
5th	607.30	18132.73	120.42	20824	196.75	7100.31	65.02	153.30	4165.95	30.40
Roof	607.30	18132.73	120.42	20824	196.75	7100.31	65.02	107.60	2931.85	21.34

Original Concrete Design Cost and Schedule Breakdown

Roof	Item #	Decreption	Unit	Quantity	Total Cost per Unit	Cost		# of Crews	Daily output	# of Days
	03 11 13.20 1050	Beam Formwork	S.F.C.A	7100.31	7.92	56234.46		3	275	9
	03 11 13.25 6550	Column Formwork	S.F.C.A	2931.85	6.56	19232.96		2	216	7
	03 11 13.35 2050	Slab Formwork	S.F.C.A	18132.73	6.08	110247.01		3	509	12
	03 21 10.60 0100	Beam Reinf.	Ton	65.02	1720	111838.48		3	1.6	14
	03 31 10.60 0200	Column Reinf.	Ton	21.34	1775	37869.91		3	1.5	5
	03 21 10.60 0400	Slab Reinf.	Ton	120.42	1405	169191.19		3	2.9	14
	03 31 05.35 0300	4000 psi conc.	CY	911.65	108	98458.26		N/A		0
	03 31 05.70 0200	Placing Beams	CY	196.75	30.35	5971.40		1	90	3
	03 31 05.70 0800	Placing Columns	CY	107.60	29.65	3190.25		1	92	2
	03 31 05.70 1600	Placing Slab	CY	607.30	15.22	9243.14		1	180	4
	03 35 29.30 0150	Finishing	S.F.	20824	0.45	9370.80	630847.86	3	630	12

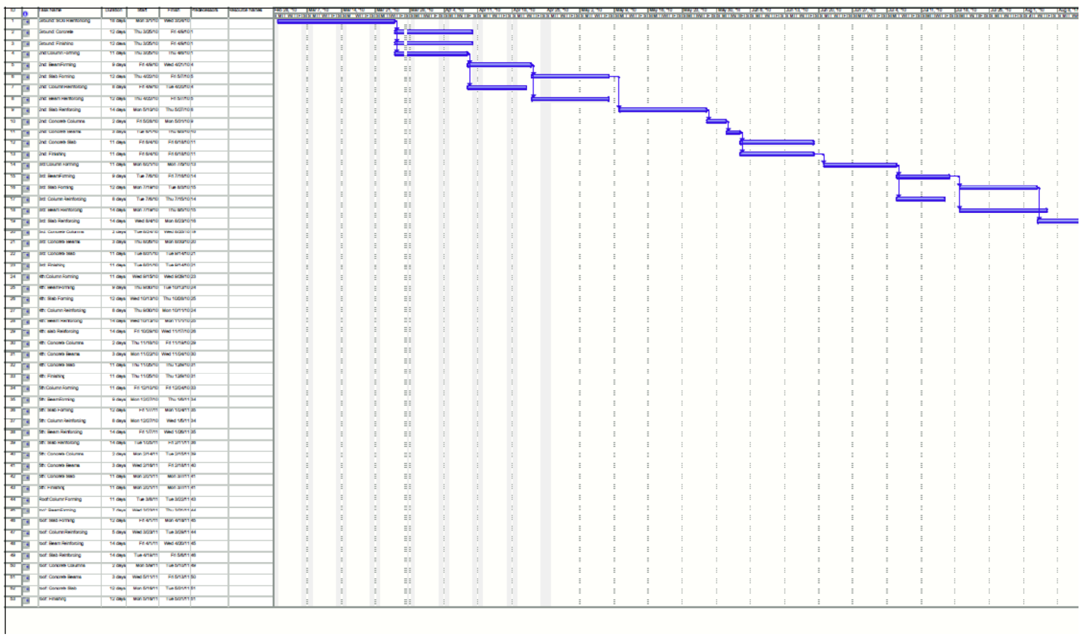
5th	Item #	Decreption	Unit	Quantity	Total Cost per Unit	Cost		# of Crews	Daily output	# of Days
	03 11 13.20 1050	Beam Formwork	S.F.C.A	7100.31	7.92	56234.46		3	275	9
	03 11 13.25 6550	Column Formwork	S.F.C.A	4165.95	6.56	27328.60		2	216	10
	03 11 13.35 2050	Slab Formwork	S.F.C.A	18132.73	6.08	110247.01		3	509	12
	03 21 10.60 0100	Beam Reinf.	Ton	65.02	1720	111838.48		3	1.6	14
	03 31 10.60 0200	Column Reinf.	Ton	30.40	1775	53957.02		3	1.5	7
	03 21 10.60 0400	Slab Reinf.	Ton	120.42	1405	169191.19		3	2.9	14
	03 31 05.35 0300	4000 psi conc.	CY	957.36	108	103394.62		N/A		0
	03 31 05.70 0200	Placing Beams	CY	196.75	30.35	5971.40		1	90	3
	03 31 05.70 0800	Placing Columns	CY	153.30	29.65	4545.46		1	92	2
	03 31 05.70 1600	Placing Slab	CY	607.30	15.22	9243.14		1	180	4
	03 35 29.30 0150	Finishing	S.F.	20824	0.45	9370.80	661322.18	3	630	12

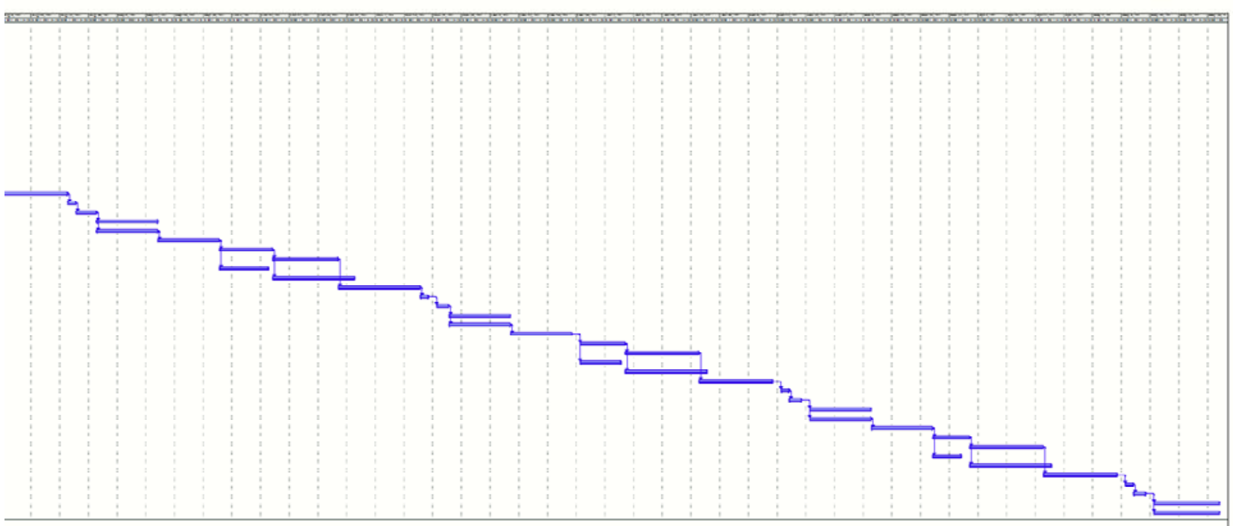
4th	Item #	Decreption	Unit	Quantity	Total Cost per Unit	Cost		# of Crews	Daily output	# of Days
	03 11 13.20 1050	Beam Formwork	S.F.C.A	7100.31	7.92	56234.46		3	275	9
	03 11 13.25 6550	Column Formwork	S.F.C.A	4165.95	6.56	27328.60		2	216	10
	03 11 13.35 2050	Slab Formwork	S.F.C.A	18132.73	6.08	110247.01		3	509	12
	03 21 10.60 0100	Beam Reinf.	Ton	65.02	1720	111838.48		3	1.6	14
	03 31 10.60 0200	Column Reinf.	Ton	30.40	1775	53957.02		3	1.5	7
	03 21 10.60 0400	Slab Reinf.	Ton	120.42	1405	169191.19		3	2.9	14
	03 31 05.35 0300	4000 psi conc.	CY	957.36	108	103394.62		N/A		0
	03 31 05.70 0200	Placing Beams	CY	196.75	30.35	5971.40		1	90	3
	03 31 05.70 0800	Placing Columns	CY	153.30	29.65	4545.46		1	92	2
	03 31 05.70 1600	Placing Slab	CY	607.30	15.22	9243.14		1	180	4
	03 35 29.30 0150	Finishing	S.F.	20824	0.45	9370.80	661322.18	3	630	12

3rd	Item #	Decreption	Unit	Quantity	Total Cost per Unit	Cost		# of Crews	Daily output	# of Days
	03 11 13.20 1050	Beam Formwork	S.F.C.A	7100.31	7.92	56234.46		3	275	9
	03 11 13.25 6550	Column Formwork	S.F.C.A	4165.95	6.56	27328.60		2	216	10
	03 11 13.35 2050	Slab Formwork	S.F.C.A	18132.73	6.08	110247.01		3	509	12
	03 21 10.60 0100	Beam Reinf.	Ton	65.02	1720	111838.48		3	1.6	14
	03 31 10.60 0200	Column Reinf.	Ton	30.40	1775	53957.02		3	1.5	7
	03 21 10.60 0400	Slab Reinf.	Ton	120.42	1405	169191.19		3	2.9	14
	03 31 05.35 0300	4000 psi conc.	CY	957.36	108	103394.62		N/A		0
	03 31 05.70 0200	Placing Beams	CY	196.75	30.35	5971.40		1	90	3
	03 31 05.70 0800	Placing Columns	CY	153.30	29.65	4545.46		1	92	2
	03 31 05.70 1600	Placing Slab	CY	586.15	15.22	8921.14		1	180	4
	03 35 29.30 0150	Finishing	S.F.	20824	0.45	9370.80	**661000.18**	3	630	12

2nd	Item #	Decreption	Unit	Quantity	Total Cost per Unit	Cost		# of Crews	Daily output	# of Days
	03 11 13.20 1050	Beam Formwork	S.F.C.A	7100.31	7.92	56234.46		3	275	9
	03 11 13.25 6550	Column Formwork	S.F.C.A	4574.06	6.56	30005.83		2	216	11
	03 11 13.35 2050	Slab Formwork	S.F.C.A	17407.28	6.08	105836.27		3	509	12
	03 21 10.60 0100	Beam Reinf.	Ton	65.02	1720	111838.48		3	1.6	14
	03 31 10.60 0200	Column Reinf.	Ton	33.38	1775	59242.89		3	1.5	8
	03 21 10.60 0400	Slab Reinf.	Ton	116.23	1405	163297.02		3	2.9	14
	03 31 05.35 0300	4000 psi conc.	CY	951.22	108	102731.66		N/A		0
	03 31 05.70 0200	Placing Beams	CY	196.75	30.35	5971.40		1	90	3
	03 31 05.70 0800	Placing Columns	CY	168.32	29.65	4990.75		1	92	2
	03 31 05.70 1600	Placing Slab	CY	586.15	15.22	8921.14		1	180	4
	03 35 29.30 0150	Finishing	S.F.	20099	0.45	9044.35	**658114.25**	3	630	11

Ground	Item #	Decreption	Unit	Quantity	Total Cost per Unit	Cost		# of Crews	Daily output	# of Days
	03 21 10.60 0600	SOG Reinf	Ton	120.42	1425	171599.61		3	2.3	18
	03 31 05.35 0300	4000 psi conc.	CY	385.63	108	41648.00		N/A		0
	03 31 05.70 4650	Slab on Grade	CY	385.63	14.81	5711.17		1	185	3
	03 35 29.30 0150	Finishing	S.F.	20824	0.45	9370.80	**228329.58**	3	630	12
							$3,500,936.23			



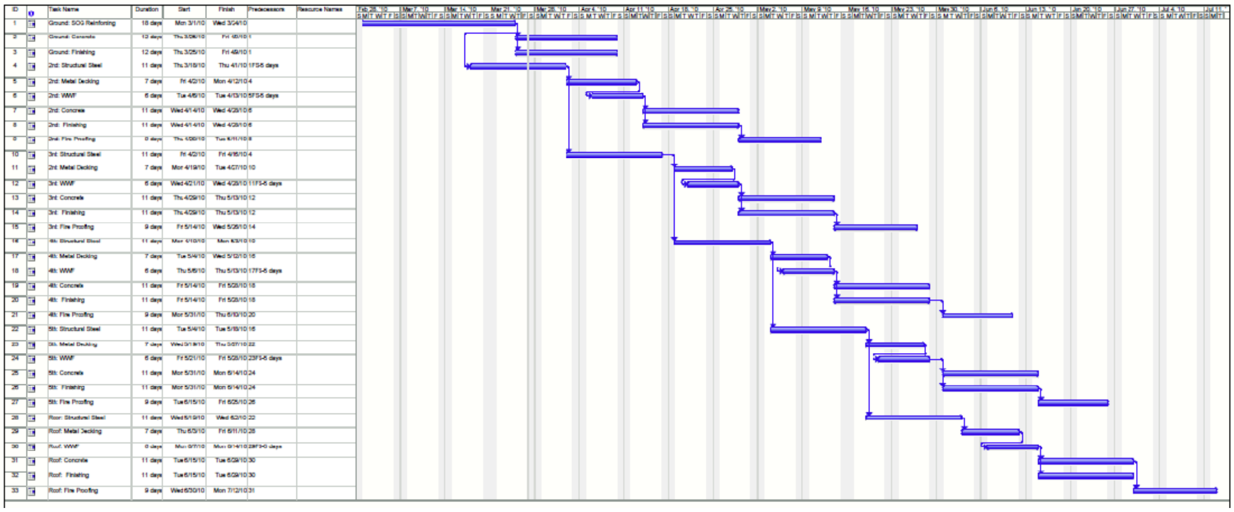


Steel Redesign Takeoff

	Beam ton	columns ton	frame ton	decking sf	concrete cy	wwf sf	fireproofing sf
Roof	85.44	7.51	55.78	20824	224.95	20824	20824
5	70.97	7.51	54.99	20824	224.95	20824	20824
4	70.97	9.78	61.60	20824	224.95	20824	20824
3	70.97	9.78	64.25	20824	224.95	20824	20824
2	70.63	12.26	73.77	20099	217.11	20098.55	20098.55

Steel Redesign Cost and Schedule Breakdown

Roof	Item #	Decreption	Unit	Quantity	Total Cost per Unit	Cost		# of Crews	Daily output per crew	# of Days	
	05 12 23.77 0800	Structural Steel	Ton	148.74	2591	385374.64		1	14.4	11	
	05 31 13.35 3400	18 Gage Decking	SF	20824	3.65	76007.60		1	3200	7	
	03 31 05.35 0300	4000 psi Concrete	CY	224.95	108	24294.67		1	N/A	0	
	03 22 04.40 0100	WWF	CSF	208.24	31.65	6590.80		1	35	6	
	03 31 05.70 1400	Elevated Slab	CY	224.95	19.55	4397.78		1	140	2	
	03 35 29.30 0150	Broom Finish	SF	20824	0.45	9370.80		3	630	12	
	07 81 16.10 0500	Fireproofing	SF	20824	1.35	28112.40	**534148.69**	2	1250	9	**47.00**
5th	**Item #**	**Decreption**	**Unit**	**Quantity**	**Total Cost per Unit**	**Cost**		**# of Crews**	**Daily output per crew**	**# of Days**	
	05 12 23.77 0800	Structural Steel	Ton	133.48	2591	345846.34		1	14.4	10	
	05 31 13.35 3400	18 Gage Decking	SF	20824	3.65	76007.60		1	3200	7	
	03 31 05.35 0300	4000 psi Concrete	CY	224.95	108	24294.67		1	N/A	0	
	03 22 04.40 0100	WWF	CSF	208.24	31.65	6590.80		1	35	6	
	03 31 05.70 1400	Elevated Slab	CY	224.95	19.55	4397.78		1	140	2	
	03 35 29.30 0150	Broom Finish	SF	20824	0.45	9370.80		3	630	12	
	07 81 16.10 0500	Fireproofing	SF	20824	1.35	28112.40	**494620.39**	2	1250	9	**46.00**
4th	**Item #**	**Decreption**	**Unit**	**Quantity**	**Total Cost per Unit**	**Cost**		**# of Crews**	**Daily output per crew**	**# of Days**	
	05 12 23.77 0800	Structural Steel	Ton	142.35	2591	368837.94		1	14.4	10	
	05 31 13.35 3400	18 Gage Decking	SF	20824	3.65	76007.60		1	3200	7	
	03 31 05.35 0300	4000 psi Concrete	CY	224.95	108	24294.67		1	N/A	0	
	03 22 04.40 0100	WWF	CSF	208.24	31.65	6590.80		1	35	6	
	03 31 05.70 1400	Elevated Slab	CY	224.95	19.55	4397.78		1	140	2	
	03 35 29.30 0150	Broom Finish	SF	20824	0.45	9370.80		3	630	12	
	07 81 16.10 0500	Fireproofing	SF	20824	1.35	28112.40	**517611.99**	2	1250	9	**46.00**
3rd	**Item #**	**Decreption**	**Unit**	**Quantity**	**Total Cost per Unit**	**Cost**		**# of Crews**	**Daily output per crew**	**# of Days**	
	05 12 23.77 0800	Structural Steel	Ton	145.01	2591	375714.46		1	14.4	11	
	05 31 13.35 3400	18 Gage Decking	SF	20824	3.65	76007.60		1	3200	7	
	03 31 05.35 0300	4000 psi Concrete	CY	224.95	108	24294.67		1	N/A	0	
	03 22 04.40 0100	WWF	CSF	208.24	31.65	6590.80		1	35	6	
	03 31 05.70 1400	Elevated Slab	CY	224.95	19.55	4397.78		1	140	2	
	03 35 29.30 0150	Broom Finish	SF	20824	0.45	9370.80		3	630	12	
	07 81 16.10 0500	Fireproofing	SF	20824	1.35	28112.40	**524488.51**	2	1250	9	**47.00**
2nd	**Item #**	**Decreption**	**Unit**	**Quantity**	**Total Cost per Unit**	**Cost**		**# of Crews**	**Daily output per crew**	**# of Days**	
	05 12 23.77 0800	Structural Steel	Ton	156.66	2591	405910.10		1	14.4	11	
	05 31 13.35 3400	18 Gage Decking	SF	20099	3.65	73359.71		1	3200	7	
	03 31 05.35 0300	4000 psi Concrete	CY	217.11	108	23448.31		1	N/A	0	
	03 22 04.40 0100	WWF	CSF	200.9855	31.65	6361.19		1	35	6	
	03 31 05.70 1400	Elevated Slab	CY	217.11	19.55	4244.58		1	140	2	
	03 35 29.30 0150	Broom Finish	SF	20099	0.45	9044.35		3	630	11	
	07 81 16.10 0500	Fireproofing	SF	20098.55	1.35	27133.04	**549501.28**	2	1250	9	**46.00**
Ground	**Item #**	**Decreption**	**Unit**	**Quantity**	**Total Cost per I**	**Cost**		**# of Crews**	**Daily output per crew**	**# of Days**	
	03 21 10.60 0600	SOG Reinf	Ton	120.4208	1425.00	171599.6077		3.00	2.3	18	
	03 31 05.35 0300	4000 psi conc.	CY	385.6296	108.00	41648			N/A	0	
	03 31 05.70 4650	Slab on Grade	CY	385.6296	14.81	5711.174815		1	185	3	
	03 35 29.30 0150	Finishing	S.F.	20824	0	9370.8	**228329.58**	3.00	630	12	**33.00**
						Total Cost	**$2,848,700.43**			**Days**	**265**


ID
Task Name
Duration
Start
Finish
Predecessors
Resource Names
1 Ground: SOG Reinforcing 18 days Mon 3/1/10 Wed 3/24/10
3 Ground: Finishing 12 days Thu 3/25/10 Fri 4/9/10 1
4 2nd: Structural Steel 11 days Thu 3/18/10 Thu 4/1/10 1FS-5 days
5 2nd: Metal Decking 7 days Fri 4/2/10 Mon 4/12/10 4
6 2nd: WWF 6 days Tue 4/6/10 Tue 4/13/10 5FS-5 days
7 2nd: Concrete 11 days Wed 4/14/10 Wed 4/28/10 6
8 2nd: Finishing 11 days Wed 4/14/10 Wed 4/28/10 6
10 3rd: Structural Steel 11 days Fri 4/2/10 Fri 4/16/10 4
11 2nd: Metal Decking 7 days Mon 4/19/10 Tue 4/27/10 10
12 3rd: WWF 6 days Wed 4/21/10 Wed 4/28/10 11FS-5 days
13 3rd: Concrete 11 days Thu 4/29/10 Thu 5/13/10 12
14 3rd: Finishing 11 days Thu 4/29/10 Thu 5/13/10 12
15 3rd: Fire Proofing 9 days Fri 5/14/10 Wed 5/26/10 14
17 4th: Metal Decking 7 days Tue 5/4/10 Wed 5/12/10 16
18 4th: WWF 6 days Thu 5/6/10 Thu 5/13/10 17FS-5 days
19 4th: Concrete 11 days Fri 5/14/10 Fri 5/28/10 18
20 4th: Finishing 11 days Fri 5/14/10 Fri 5/28/10 18
21 4th: Fire Proofing 9 days Mon 5/31/10 Thu 6/10/10 20
22 5th: Structural Steel 11 days Tue 5/4/10 Tue 5/18/10 16
24 5th: WWF 6 days Fri 5/21/10 Fri 5/28/10 23FS-5 days
25 5th: Concrete 11 days Mon 5/31/10 Mon 6/14/10 24
26 5th: Finishing 11 days Mon 5/31/10 Mon 6/14/10 24
27 5th: Fire Proofing 9 days Tue 6/15/10 Fri 6/25/10 26
28 Roof: Structural Steel 11 days Wed 5/19/10 Wed 6/2/10 22
29 Roof: Metal Decking 7 days Thu 6/3/10 Fri 6/11/10 28
31 Roof: Concrete 11 days Tue 6/15/10 Tue 6/29/10 30
32 Roof: Finishing 11 days Tue 6/15/10 Tue 6/29/10 30
33 Roof: Fire Proofing 9 days Wed 6/30/10 Mon 7/12/10 31

Appendix I: Mechanical Coordination

- Duct 1:

$$
\begin{array}{l}
20(47\ L/s) \\
3(6\ L/s) \\
2(1\ L/s) \\
2(147\ L/s) \\
2(52\ L/s) \\
3(28\ L/s) \\
\underline{4(99\ L/s)} \\
1598\ L/s \rightarrow 3385.97\ cfm
\end{array}
$$

- Duct 2:

$$
\begin{array}{l}
5(4)(47\ L/s) \\
5(2\ L/s) \\
(4\ L/s) \\
\underline{(38\ L/s)} \\
992\ L/s \rightarrow 2101.93\ cfm
\end{array}
$$

- Velocity choice

$4\ m/s: 4\ m/s\ (3.2808\ ft/m)\left(\frac{60\ s}{1\ min}\right) = 787.39\ ft/s$

$5\ m/s: 5\ m/s\ (3.2808\ ft/m)(60) = 984.2\ ft/s$

$6\ m/s: 6\ m/s\ (3.2808)(60) = 1181.1\ ft/s$

- req'd Duct Area

Duct 1: $A = q/v = 3385.97/984.2 = 3.44\ ft^2 \rightarrow 5\ m/s$

$= 3385.97/1181.1 = 2.867\ ft^2 \rightarrow 6\ m/s$

Duct 2: $A = q/v = 2101.93/984.2 = 2.14\ ft^2 \rightarrow 5\ m/s$

$= 2101.93/1181.1 = 1.78\ ft^2 \rightarrow 6\ m/s$

Mechanical Cordination Breadth

Current Sizes

Duct 1 - (750mm x 400mm), (29.53" x 15.75")

Duct 2 - (650mm x 300mm), (25.59" x 11.81")

air flow takeoff

Duct 1 - 3385.97 cfm

Duct 2 2101.93 cfm

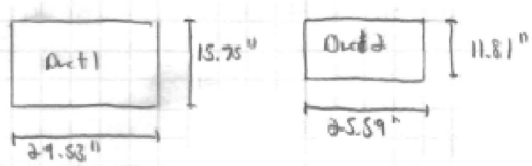
required area

Duct 1 : 2.867 ft^2

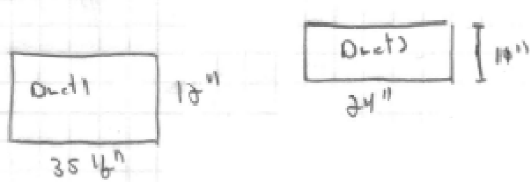
Duct 2 : 1.78 ft^2

required height by new W18x35 d=17.7" floor system

- Original Design



- redesign



Appendix J: Progressive Collapse

Table 3-4. Load Increase Factors for Linear Static Analysis

Material	Structure Type	Ω_{LD}, Deformation-controlled	Ω_{LF}, Force-controlled
Steel	Framed	$0.9\ m_{LIF} + 1.1$	2.0
Reinforced Concrete	Framed[A]	$1.2\ m_{LIF} + 0.80$	2.0
	Load-bearing Wall	$2.0\ m_{LIF}$	2.0
Masonry	Load-bearing Wall	$2.0\ m_{LIF}$	2.0
Wood	Load-bearing Wall	$2.0\ m_{LIF}$	2.0
Cold-formed Steel	Load-bearing Wall	$2.0\ m_{LIF}$	2.0

[A] Note that, per ASCE 41, reinforced concrete beam-column joints are treated as force-controlled; however, the hinges that form in the beam near the column are deformation-controlled and the appropriate m-factor from Chapter 4 of this UFC shall be applied to the calculation of the deformation-controlled load increase factor Ω_{LD}.

Table 3-5. Dynamic Increase Factors for Nonlinear Static Analysis

Material	Structure Type	Ω_N
Steel	Framed	$1.08 + 0.76/(\theta_{pra}/\theta_y + 0.83)$
Reinforced Concrete	Framed	$1.04 + 0.45/(\theta_{pra}/\theta_y + 0.48)$
	Load-Bearing Wall	2
Masonry	Load-bearing Wall	2
Wood	Load-bearing Wall	2
Cold-formed Steel	Load-bearing Wall	2

Connection Type	Linear Acceptance Criteria *m*-factors Primary[1]	Linear Acceptance Criteria *m*-factors Secondary[1]
Fully Restrained Moment Connections		
Improved WUF with Bolted Web	2.3 – 0.021d	4.9 – 0.048d
Reduced Beam Section (RBS)	4.9 – 0.025d	6.5 – 0.025d
WUF	4.3 – 0.083d	4.3 -0.048d
SidePlate®	6.7 – 0.039d[2]	11.1 – 0.062d
Partially Restrained Moment Connections (Relatively Stiff)		
Double Split Tee		
a. Shear in Bolt	4	6
b. Tension in Bolt	1.5	4
c. Tension in Tee	1.5	4
d. Flexure in Tee	5	7
Partially Restrained Simple Connections (Flexible)		
Double Angles		
a. Shear in Bolt	$5.8 - 0.107d_{bg}$[3]	$8.7 - 0.161d_{bg}$
b. Tension in Bolt	1.5	4
c. Flexure in Angles	$8.9 - 0.193d_{bg}$	$13.0 - 0.290d_{bg}$
Simple Shear Tab	$5.8 - 0.107d_{bg}$	$8.7 - 0.161d_{bg}$

(1) Refer to Section 3-2.4 for determination of Primary and Secondary classification
(2) d = depth of beam, inch
(3) d_{bg} = depth of bolt group, inch

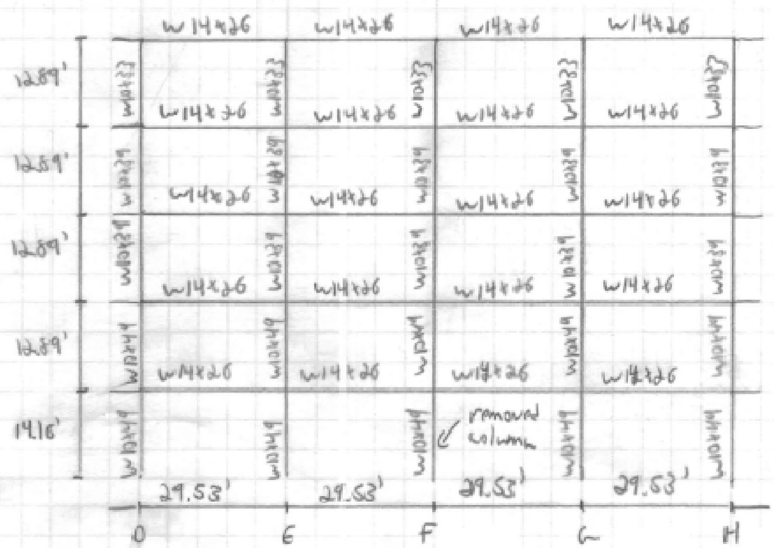
Table 5-2. Modeling Parameters and Acceptance Criteria for Nonlinear Modeling of Steel Frame Connections

Connection Type	Nonlinear Modeling Parameters[1]			Nonlinear Acceptance Criteria	
	Plastic Rotation Angle, radians		Residual Strength Ratio	Plastic Rotation Angle, radians	
	a	*b*	*c*	Primary[2]	Secondary[2]
Fully Restrained Moment Connections					
Improved WUF with Bolted Web	0.021 - 0.0003d	0.050 - 0.0006d	0.2	0.021 - 0.0003d	0.050 - 0.0006d
Reduced Beam Section (RBS)	0.050 - 0.0003d	0.070 - 0.0003d	0.2	0.050 - 0.0003d	0.070 - 0.0003d
WUF	0.0284 - 0.0004d	0.043 - 0.0006d	0.2	0.0284 - 0.0004d	0.043 - 0.0006d
SidePlate®	0.089 - 0.0005d[3]	0.169 - 0.0001d	0.6	0.089 - 0.0005d	0.169 - 0.0001d
Partially Restrained Moment Connections (Relatively Stiff)					
Double Split Tee					
a. Shear in Bolt	0.036	0.048	0.2	0.03	0.040
b. Tension in Bolt	0.016	0.024	0.8	0.013	0.020
c. Tension in Tee	0.012	0.018	0.8	0.010	0.015
d. Flexure in Tee	0.042	0.084	0.2	0.035	0.070
Partially Restrained Simple Connections (Flexible)					
Double Angles					
a. Shear in Bolt	$0.0502 - 0.0015d_{bg}$[4]	$0.072 - 0.0022d_{bg}$	0.2	$0.0502 - 0.0015d_{bg}$	$0.0503 - 0.0011d_{bg}$
b. Tension in Bolt	$0.0502 - 0.0015d_{bg}$	$0.072 - 0.0022d_{bg}$	0.2	$0.0502 - 0.0015d_{bg}$	$0.0503 - 0.0011d_{bg}$
c. Flexure in Angles	$0.1125 - 0.0027d_{bg}$	$0.150 - 0.0036d_{bg}$	0.4	$0.1125 - 0.0027d_{bg}$	$0.150 - 0.0036d_{bg}$
Simple Shear Tab	$0.0502 - 0.0015d_{bg}$	$0.072 - 0.0022d_{bg}$	0.2	$0.0502 - 0.0015d_{bg}$	$0.1125 - 0.0027d_{bg}$

(1) Refer to Figure 3-6 for definition of nonlinear modeling parameters *a*, *b*, and *c*
(2) Refer to Section 3-2.4 for determination of Primary and Secondary classification
(3) d = depth of beam, inch
(4) d_{bg} = depth of bolt group, inch

Appendix I: Progressive Collapse Design

Progressive Collaps Design
GSA - AP Method



Load Comb: 2(1.0DL + .25LL)

DD Floor: 65 psf
Roof: 95 psf
LL Floor: 80 psf
Roof: 20 psf

Extend of collapse

2(29.53)27.559 = 1627.6 ft² < 1800 ft use 1627 ≈ 1680 ft²

Loads:

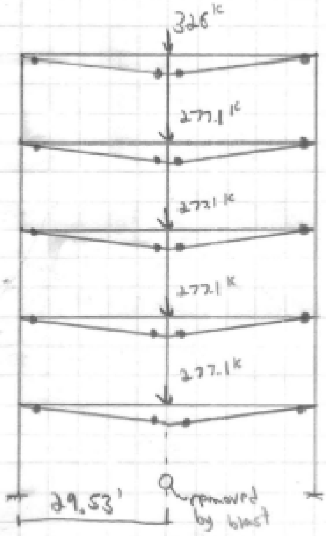
Typical Floor

$2[.065 + .25(.08)] = .17 \text{ ksf } (1680) = \underline{277.1}^{k}$

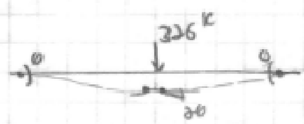
Roof

$2[.095 + .25(.02)] = .2 \text{ ksf } (1680) = \underline{326}^{k}$

Progressive Collapse Design



Level Roof - Plastic Analysis (Virtual Work)



$M_p\theta + 2\theta M_p + 2M_p\theta + M_p\theta = P(29.53(\theta))$

$6M_p\theta = 29.53(326)\theta$

$M_p = 1604.5$ 'k

Acceptance Criteria $DCR = \frac{Q_{UD}}{Q_{CE}} = \frac{\text{Acting}}{\text{Expe}} = 3$

$Q_{CE} = \frac{1604.5}{3} = 534.8$ ft k $\Rightarrow$ W21X62 $\phi M_p = 540$ 'k $L_p = 6.25'$

W21x62 $b_f = 8.24"$, $t_f = .615$, $h/t_w = 46.9$

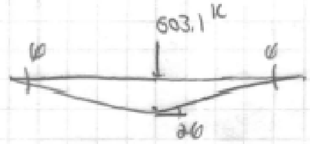
Flange slender $\frac{8.24}{2(.615)} \leq \frac{52}{\sqrt{50}}$ $6.7 \leq 7.0$ ok

Web slender $46.9 \leq \frac{418}{\sqrt{55}} = 56.36$ ok

W21x62 adequate

2

Level 5- Plastic Analysis



6MpΘ = P(29.53 Θ)

6MpΘ = (603.1)(29.53 Θ)

Mp = 2968.3 IK

Acceptance Criteria DCR = 3

$Q_{CG} = \frac{2968.3}{3} = 989.4$ IK ⇒ W30x90 ØMp = 1060 IK Lp = 7.38'

W30x90 $b_f = 10.4$ $t_f = .61$ $h/t_w = 57.5$

Flange slenderness: $\frac{10.24}{2(.61)} = 8.39 > 7.01$ no good ∴ DCR = 2

$Q_{CG} = \frac{2968.3}{2} = 1484.1$ IK ⇒ W33x118 ØMp = 1560 IK Lp = 8.19'

W33x118 $t_f = .74$ $b_f = 11.5$ $h/t_w = 54.5$

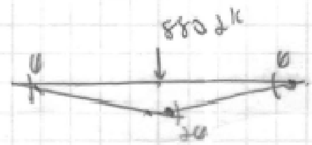
Flange slenderness: $\frac{11.5}{2(.74)} = 7.85 < 8.76$ no good

web slender. $54.5 < \frac{640}{\sqrt{55}} = 86.3$ no good

* interpolate for DCR, DCR = 2 conservative

W33x118 adequate

Level 4: Plastic Anaylsis



$6M_p\theta = 880.2(29.53)(\theta)$

$\phi M_p = 4298.3$ 'k

$\phi c_G = \frac{4298.3}{3} = 1432.8$ 'k → W24x146 → $\phi M_p = 1570$'k $L_p = 10.6$'

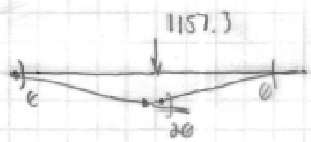
W24x146 $b_f = 12.9$ $t_f = 1.09$ $h/t_w = 33.2$

flange $\frac{12.9}{2(1.09)} = 5.92 < \frac{52}{\sqrt{55}} = 7.01$ ok

web $33.2 < \frac{418}{\sqrt{55}} = 56.4$ ok

W24x146 adequate

Level 3: Plastic Analysis



$6M_p\theta = 1157.3(29.53)\theta$

$M_p = 5695.8$'k

$\phi c_G = \frac{5695.8}{3} = 1898.6$'k → W33x141 $\phi M_p = 1930$ $L_p = 8.58$'

$b_f = 11.5$ $t_f = .96$ $h/t_w = 49.6$

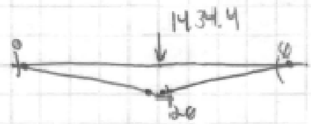
flange $\frac{b_f}{2t_f} = \frac{11.5}{2(.96)} = 5.99 \leq \frac{52}{\sqrt{55}} = 7.01$ ok

web $\frac{h}{t_w} = 49.6 < \frac{418}{\sqrt{55}} = 56.4$ ok

W33x141 adequate

Progressive Collapse Design

Level 2: Plastic Analysis



$6 M_p \theta = 1434.4 (24.53\, \theta)$

$M_p = 7059.6$ 'k

$Q_{CE} = \frac{7059.6}{3} = 2353.2$ 'k → W33x169 $\phi M_p = 2360$ 'k $L_p = 8.83$

$b_f = 11.5$ $t_f = 1.22$ $h/t_w = 44.7$

Flange $\frac{b_f}{2t_f} = \frac{11.5}{2(1.22)} = 4.71 < 7.31$ ok

Web $\frac{h}{t_w} = 44.7 < 56.4$ ok

W33x169 adequate

For moment connection @ Level 2

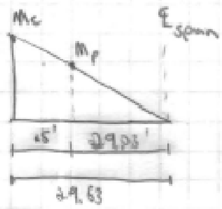
DCR = 2

$M_p = 7059.6$ $Q_{CE} = \frac{7059.6}{2} = 3529.8$ 'k

Progressive Collapse Design - GSA

Column Design / check - 2nd Flr

- Moment transferd from Beam



$$\frac{M_p}{29.03} = \frac{M_c}{29.53}$$

$$M_c = \frac{29.53}{29.03}(7059.6) = 7181.2^{1K}$$

Assum DCR = 1 , P/P_{cu} ? .5

Axial Load in column ≈ 1434.4/2 = 717.2^k from colaps loading

DCR = 2

W14x500 $p \times 10^3 = .171$ $b_x \times 10^3 = .226$

$\frac{.171(717.2)}{1000} = .123 < .2$ H1-a

$.5(.123) + 9/8(.226/1000)(35906) = .974 \leq 1.0$ (7181.2/2)

$b_f = 17$ $t_f = 3.50$ h/t_w 5.21

Flange: $\frac{b_f}{2t_f}, \frac{17}{2(3.50)} = 2.43 < \frac{52}{\sqrt{55}} = 7.01$ ok

$\frac{h}{t_w} = 5.21 < \frac{300}{\sqrt{F_y}} = \frac{300}{\sqrt{55}} = 40.5'$ ok

DCR of 2 ok

use W14x500