Hyatt Place North Shore
Pittsburgh, PA

Senior Thesis Final Report

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The Pennsylvania State University

Architectural Engineering

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Hyatt Place North Shore

Pittsburgh, PA

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Structural Option

Building Statistics

Location: North Shore Drive Pittsburgh, PA 15212

Size: 108,743 Total SF, 67,388 SF in 178 guestrooms

No. Stories: 7 stories above grade to 70’-0”

Construction Dates: June 2009 to October 2010

Project Delivery Method: Design Bid Build

Project Team

Owner: Continental/Rockbridge North Shore Hotel

Architect: Burt Hill

CM: Continental Building Systems

Structural: Atlantic Engineering Services

MEP: HF Lenz

Site/Civil Engineer: Civil & Environmental Consultants, Inc.

Geotechnical Engineer: Michael Baker Jr., Inc.

Structural

-Structure is located on soft soil along the Allegheny River

-121 18”-140 ton auger-cast piles necessary

-Reinforced concrete masonry bearing walls resist gravity loads

-Act as shear walls to resist lateral force

-8” precast concrete planks typically spanning 30’

-Large steel transfer girder to allow open meeting space

Mechanical

-Hotel rooms equipped with 350 CFM PTACs

-3 1500 CFM (100% O.A.) AHU on roof to supply hotel corridor

-3 various AHU to supply other areas of building

1) 7970 CFM (52% O.A.) 2) 6070 CFM (35% O.A.) 3) 1500 CFM (15% O.A.)

Electrical

-3 phase 4 wire system

-1600A main distribution switchboard 480Y/277V

-800A busway up the building

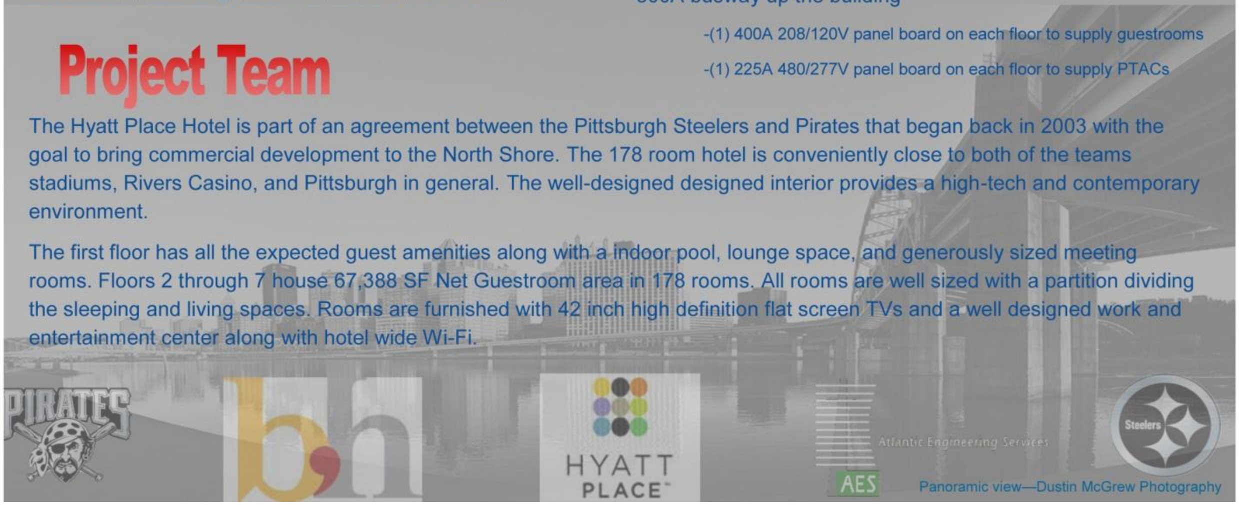
-(1) 400A 208/120V panel board on each floor to supply guestrooms

-(1) 225A 480/277V panel board on each floor to supply PTACs

Project Team

The Hyatt Place Hotel is part of an agreement between the Pittsburgh Steelers and Pirates that began back in 2003 with the goal to bring commercial development to the North Shore. The 178 room hotel is conveniently close to both of the teams stadiums, Rivers Casino, and Pittsburgh in general. The well-designed designed interior provides a high-tech and contemporary environment.

The first floor has all the expected guest amenities along with a indoor pool, lounge space, and generously sized meeting rooms. Floors 2 through 7 house 67,388 SF Net Guestroom area in 178 rooms. All rooms are well sized with a partition dividing the sleeping and living spaces. Rooms are furnished with 42 inch high definition flat screen TVs and a well designed work and entertainment center along with hotel wide Wi-Fi.



Panoramic view—Dustin McGrew Photography

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Acknowledgements

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- Tim Jones

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- Dr. Ali Memari
- Prof. Robert Holland
- Kevin Parfitt
- The entire AE faculty and staff

I would like to thank these people and the others around me that provided me with the knowledge and guidance to produce my senior thesis. Special thanks goes out to Atlantic Engineering Services for allowing me to use The Hyatt Place North Shore for my thesis study and giving me help and guidance when I needed it.

Executive Summary

The purpose of this report is to present the proposed change in location of the Hyatt Place North Shore from Pittsburgh, PA to San Diego California. After analyzing the existing structural system of the 7-story Hyatt Place North Shore it is determined that it is sufficient to carry the load and meet code standards. The 70 feet tall, 108,000 square foot structure has intermediate reinforced concrete masonry bearing walls working in combination with an 8" un-topped precast concrete plank floor structure to handle both gravity and lateral loads down into the soft soils along the Allegheny River and to bedrock approximately 70 feet below with numerous 18" diameter auger piles.

The Hyatt Place North Shore is an "L" shape that has an abundance of shear walls around its perimeter and along the double loaded corridor that runs down the middle of each leg, thus the center of rigidity is expected to be near the center of mass. But in general the "L" shape leads to the legs acting individually and creating large amounts of stress where the ends of the wings meet and at the reentrant corner. There would have to be special considerations for this building shape if the building was purposed for a location in the Western United States where seismic load is much greater. Ideally a large "L" shaped building would have a separation joint large enough to allow the two legs of the building to act independently from each other limiting the twisting action due to the orientation of shear walls. Thus the building shape leads to the thesis study for the Hyatt Place North Shore.

The proposed thesis study is to have the building relocated to California and redesigned to best meet to the seismic loads given the building layout. This will require a complete redesign of the gravity and lateral force resisting systems. The gravity structure will be steel with topped precast concrete plank floor system and the lateral system will be steel braced frames along with concrete shear walls around stairwells. These systems will be designed in RAM and ETABS and checked for validity by hand. Two lateral force resisting frames will be designed by hand in order to incorporate my MAE courses. Throughout the study there will be a focus on torsional effects and how the building reacts under seismic loads.

With the redesign of the superstructure, the cost and schedule of the building will be affected, along with the architecture. Both topics will be analyzed and used to compare the effect of location on the building as a whole. The use of the separation joint between wings of the building will also be compared. All of this information will be complied to compare the Pennsylvania location with the California location.

Building Overview: Existing

Location and Architecture

The construction of the Hyatt Place North Shore was part of an agreement between the Pittsburgh Steelers and Pirates that began back in 2003 with the goal to bring commercial development to the North Shore. The 108,000 SF, 178 room hotel is conveniently located between Heinz Field and PNCPark, with The Rivers Casino and downtown Pittsburgh nearby.

Figure 1: Areal View of the North Shore courtesy of Bing.com

The first floor has all the expected guest amenities along with an indoor pool, lounge space, and generously sized meeting rooms. The first floor has a ceiling height of 17’-4” and the upper floors are 8’-0”. Minimum floor to ceiling height is obtained with an 8 inch thick hollow core concrete plank floor system and through the use of PTACs in guestrooms. Floors 2 through 7 house 67,388 SF Net Guestroom in 178 rooms. All rooms are well sized with a partition dividing the sleeping and living spaces. Rooms are furnished with 42 inch high definition flat screen TVs and a well-designed work and entertainment center along with hotel wide Wi-Fi. Figure 2 and 3 show the layout of the ground floor and typical upper floor plan respectively.

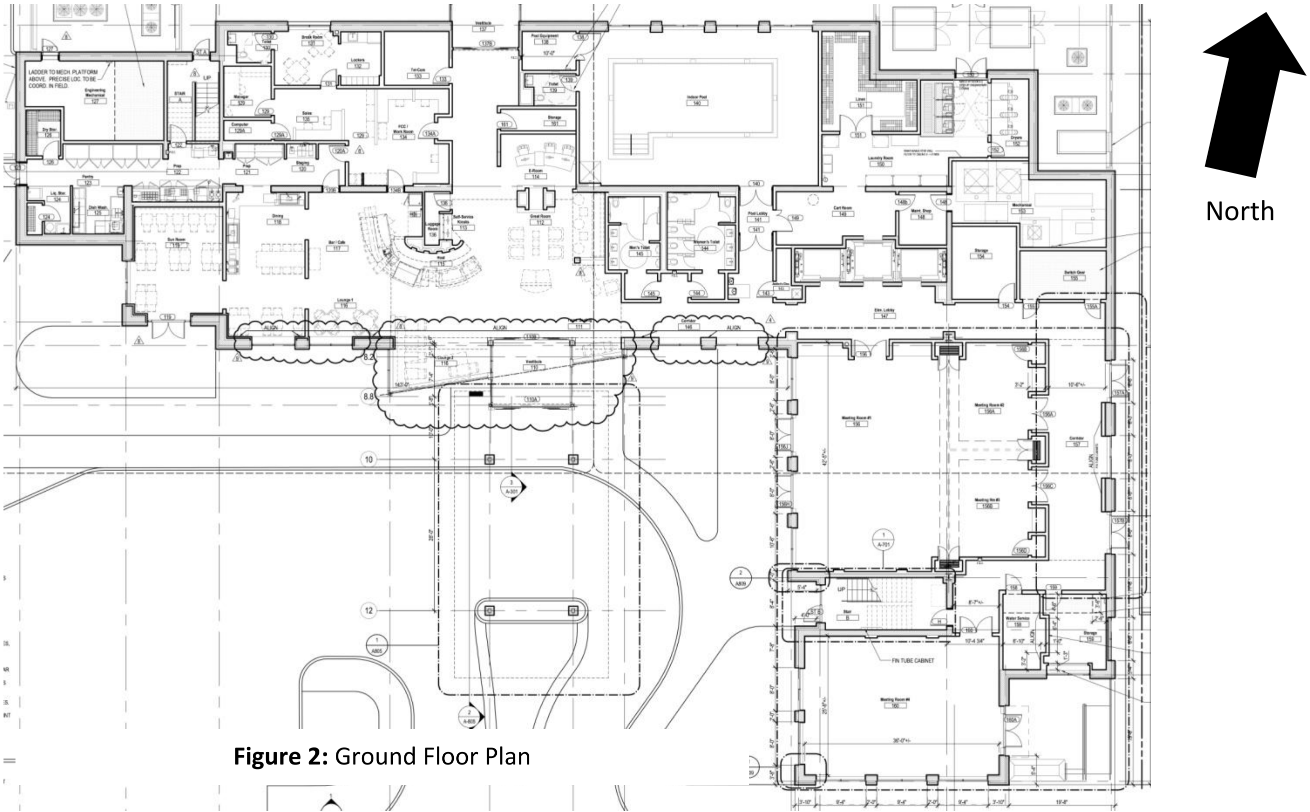


Figure 2: Ground Floor Plan

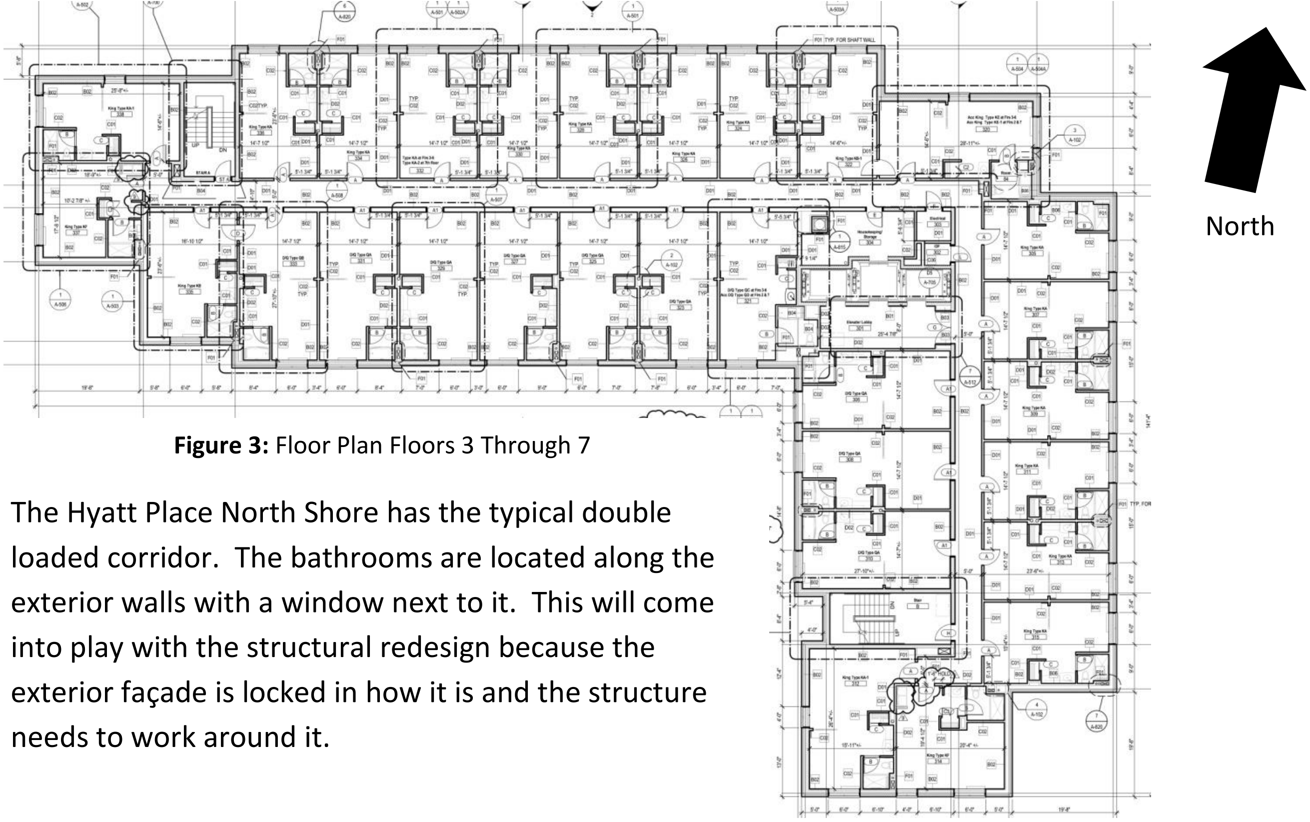


Figure 3: Floor Plan Floors 3 Through 7

The Hyatt Place North Shore has the typical double loaded corridor. The bathrooms are located along the exterior walls with a window next to it. This will come into play with the structural redesign because the exterior façade is locked in how it is and the structure needs to work around it.

Figure 4: South Elevation

Building Enclosure

Exterior elevations are mainly comprised of brick veneer cavity wall system with rigid insulation and structural CMU backup along with cast stone window headers, some strips of aluminum, metal plates, cast stone, and polished block in a way to complement the modern look of the interior. The parapet wall also varies in height from 3 feet to 9 feet creating interesting snow and wind loadings on the roof. The roof is a typical TPO membrane roof system on top of 8” precast concrete plank.

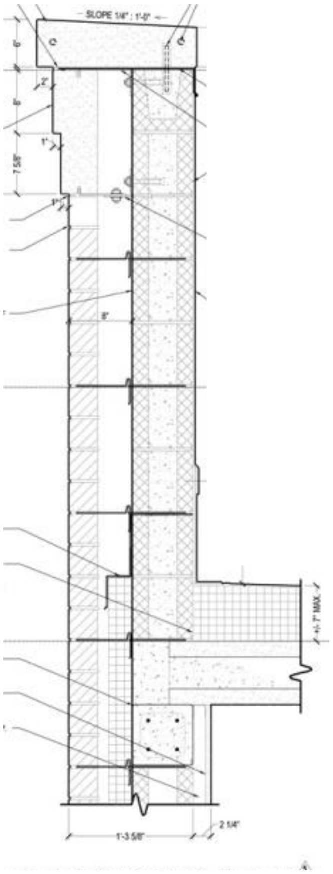


Figure 5: Typical Wall Section

Systems Overview

Construction

The Hyatt Place North Shore has a 15,500 square foot building plan, located on a 97,220 square foot site. Most of the site was originally parking spaces. There is also a large overpass for I-279, a major Pittsburgh highway, curving over the north-west corner of the site. The first and largest obstacle for the locally based general contractor, Continental Building Systems, was establishing a solid base on the soil along the Allegheny River. Construction was completed in the typical design-bid-build format in a little over a year.

Mechanical

The mechanical system can be divided into two spaces, public and private. The electrical system powers 350 cfm Packaged Terminal Air Conditioning units (PTACs) in each guestroom. This is the commonly used, simple way to provide occupants with a controllable space. The public spaces are conditioned by air handling units (AHUs) located on the roof and on the ground floor. The corridors are supplied with 100% outside air from 3 - 1500 cfm roof AHUs, the air goes down a duct decreasing in size from 26"x12" to 12"x8" on the second floor. This variable air volume system is in place throughout the public spaces. There are 3 more AHUs used to supply the remaining space on the ground floor. Also in the mechanical system are two 1,500 cfm gas boilers that heat water for domestic use, heat the pool, and are pumped to AHUs for the heating process.

Electrical and Lighting

The building is supplied using a 3 phase - 4 wire 480Y/277V system to the 1600A main distribution switchboard. It is kept at this voltage and sent up an 800A busway to a 480Y/277V panel on each floor for MEP purposes such as PTACs and also transferred down at each floor to a 208Y/120V panel to serve guestroom and general needs. In these guestrooms and public spaces, the lighting matches the modern decor and serves to create a functional space for work and relaxation.

Fire Protection

The fire protection system for the Hyatt Place North Shore was designed using the National Fire Protection Association 13 (NFPA 13) for groups designated by the International Building Code 2006 (IBC 2006). Automatic sprinkler systems were installed in accordance with NFPA 13 for group - R buildings above 4 stories. The sleeping units and corridors have 1 hour fire separation, MEP and back of house areas are sprinkled. The mass of the concrete masonry units and precast concrete planks serve the needed 2 hour fire rating. Any exposed steel members were protected as prescribed.

Vertical Transportation

There are three elevators in the building to serve the seven stories. Two of the elevators strictly service the 6 stories of guestrooms, and the third has access to the service areas such as housekeeping on each floor and laundry and MEP on the first floor.

Existing Structural Overview

The Hyatt Place North Shore is a 7 story reinforced concrete masonry unit bearing wall structure located on soft soils along the Allegheny River that utilizes precast concrete planks for ease of construction and headroom. Steel beams are used to create an open space on the ground floor for a large meeting room and in other various places where the layout makes it impossible for the concrete planks to rest on the typical masonry bearing walls. In addition, there is a large steel transfer truss on the ground floor in order to span over a meeting room . The reinforced concrete masonry bearing walls also serve as the lateral force resisting system with the aid of the precast concrete planks acting as a semi-rigid diaphragm.

Foundation:

The Hyatt Place North Shore has a 15,500 SF footprint located on soil along the Allegheny River that has a maximum allowable bearing capacity of 1,500 psf. Spread footings have been provided for the front canopy, 5'-0"x5'-0"x1'-0" concrete spread footing with a maximum load of 25 kips, and site wall foundations only. For the main structure bearing on soil doesn't provide enough resistance, here there are 121 – 18" diameter end bearing 140 ton auger-cast piles that have a minimum depth of 1'-0" into bedrock to support the building. They have a 285 kip vertical capacity and a 16 kip lateral capacity. Piles are typically expected to be 70 feet deep, but this varies per pile. As shown in *Figure 6,* pile caps are 4'-0" thick. There are 2 to 4 piles supporting each pile cap. All concrete used for shallow foundations and piers have a strength of 3000 psi and the concrete for grade beams, pile caps, and slabs on grade are 4000 psi. The first floor is a 4" concrete slab on grade with W/ 6x6-W1.4xW1.4 welded wire fabric.

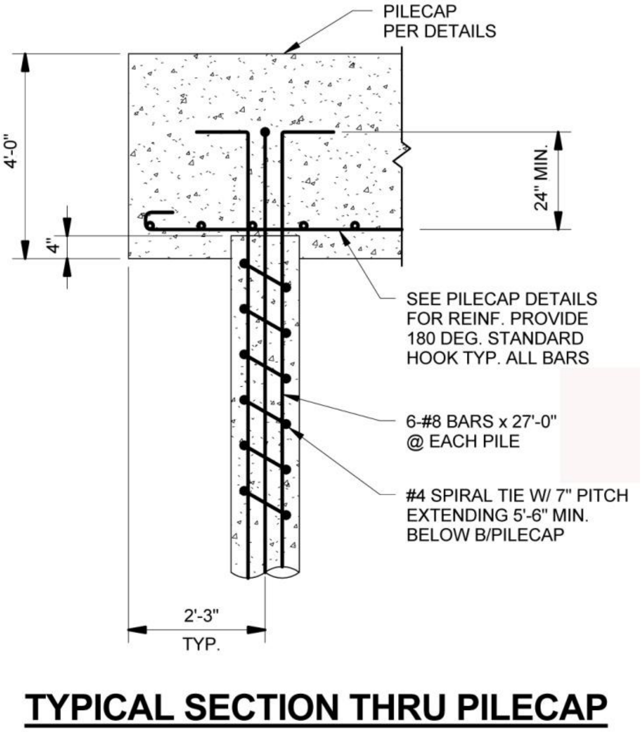
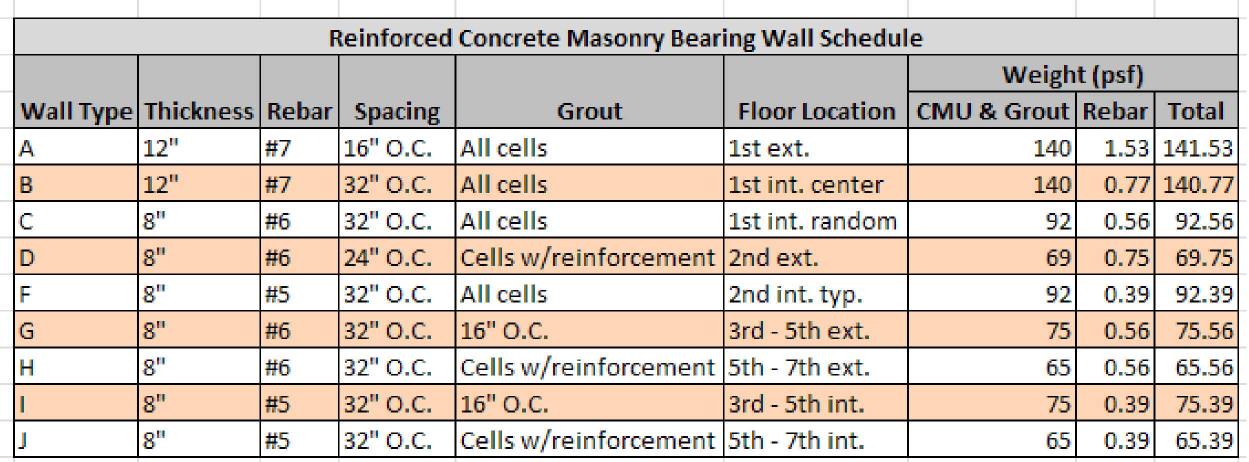


Figure 6: Section through typical pile cap

Gravity System

Walls:

Nearly all of the walls in the Hyatt Place North Shore are reinforced concrete masonry walls that resist gravity and lateral loads. The only exceptions are partition walls between the hotel rooms and other random walls not along the perimeter of the building. The walls vary in thickness and spacing of grout and reinforcing, *Table 1* shows the wall types and location. The compressive strength of the CMU units is 2800 psi and the bricks are 2500 psi, both normal weight. The grout used has a compressive strength of 3000 psi and the steel reinforcement is sized and placed as stated in *Table 1*. These walls prove more than sufficient to carry the gravity loads and also the lateral loads. Concrete lintels are placed over the window openings to span over the windows.



Reinforced Concrete Masonry Bearing Wall Schedule								
						Weight (psf)		
Wall Type	Thickness	Rebar	Spacing	Grout	Floor Location	CMU & Grout	Rebar	Total
A	12"	#7	16" O.C.	All cells	1st ext.	140	1.53	141.53
B	12"	#7	32" O.C.	All cells	1st int. center	140	0.77	140.77
C	8"	#6	32" O.C.	All cells	1st int. random	92	0.56	92.56
D	8"	#6	24" O.C.	Cells w/reinforcement	2nd ext.	69	0.75	69.75
F	8"	#5	32" O.C.	All cells	2nd int. typ.	92	0.39	92.39
G	8"	#6	32" O.C.	16" O.C.	3rd - 5th ext.	75	0.56	75.56
H	8"	#6	32" O.C.	Cells w/reinforcement	5th - 7th ext.	65	0.56	65.56
I	8"	#5	32" O.C.	16" O.C.	3rd - 5th int.	75	0.39	75.39
J	8"	#5	32" O.C.	Cells w/reinforcement	5th - 7th int.	65	0.39	65.39

Table 1: Reinforced concrete masonry bearing wall schedule

Columns:

With the masonry structure, the only 2 columns in the building are W12x136s located on the first floor and are used to transfer the load in the large transfer girder down to the foundation. The truss consists of W12x190 cords that are spaced 5 feet apart with HSS 12x8x1/2 bracing members. There are also concrete masonry piers on the first floor that support transfer beams in the lobby space and make it possible to have more open space on the first floor.

Floors:

The Hyatt Place North Shore floor system is 8” thick untopped precast concrete planks. This system simplifies design and expedites construction. The system efficiently carries the loading over relatively long spans ranging from 27’-6” to 30’-6”. The concrete compressive strength of the floors is f’c=5000 psi. Extra strength is also added by prestressing the units. *Figure 7* shows a typical connection with masonry bearing walls.

The only exception to the typical concrete plank floor is on the first floor where this is a 4 inch concrete slab on grade, which was previously discussed on page 6 in the foundations section.

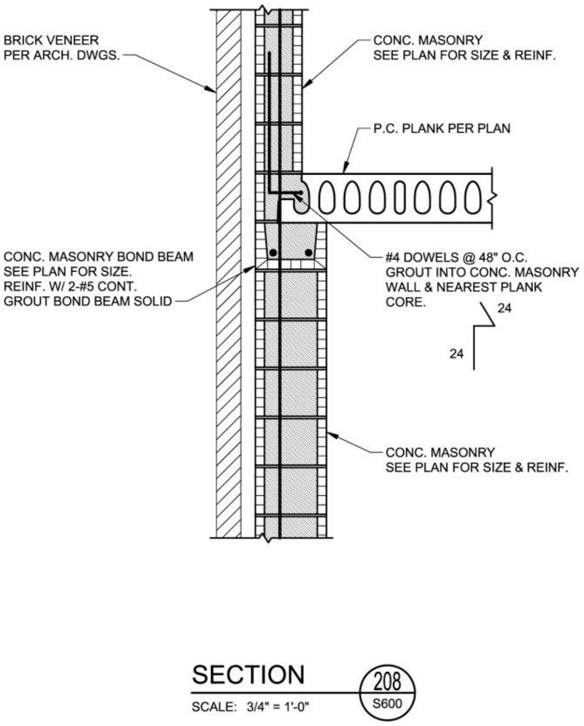


Figure 7: Typical plank and masonry wall connection

Lateral System

The lateral system for the structure is simply the gravity system. The reinforced masonry bearing walls act as shear walls and the precast concrete planks act as a semi-rigid diaphragm. The existing system has a leveling material added, for planks to be considered fully rigid there must be a 2” structural concrete topping. The load is taken from diaphragm and then into the bearing walls based upon tributary area of the shear wall. From there the load moves down to the foundation and the auger piles that are capable of resisting 16 kips of lateral force per pile. *Table 2* lists a shear check of a few walls on the ground floor of the structure. They are all adequate, and so are the others that are not listed.

$$\phi V_n = \phi A_{cv}\left[\left(\alpha_c \lambda \sqrt{f'_c}\right) + \left(\rho_t f_y\right)\right] \qquad \phi V_n \geq Vu \ \therefore Ok$$

Shear Check in 1st Story Walls															
			Vu (k)		Shear Strength Check										
Wall	Area (SF)	% Tot. Area	Hand	ETABS Rig	Lwall (in)	ear Force (k	Vert. Reinf.	Spacing (in.	Thickness (i	Acv (in2)		f'c (ksi)		ΦVn (k)	
a	0.0	0.0000	0.00	62.3	444.0	0.000	#7	16	12	5328	2	2.8	0.003125	14122	Works
b	924.0	0.0660	35.45	52.8	288.0	0.123	#7	16	12	3456	2	2.8	0.003125	9160	Works
c	2940.0	0.2100	112.80	66.10	400.0	0.282	#7	32	12	4800	2	2.8	0.001563	12384	Works
d	2880.0	0.2057	110.50	66.20	360.0	0.307	#7	32	12	4320	2	2.8	0.001563	11147	Works
e	540.0	0.0386	20.72	77.60	390.0	0.053	#7	32	12	4680	2	2.8	0.001563	12076	Works

Table 2: Sample Shear Checks in Lateral Force Resisting Walls

Proposal

Problem Statement

After analyzing the existing structural system of the 7-story Hyatt Place North Shore it was found to be sufficient to carry the gravity and lateral loads for the location in Pittsburgh, PA and meet all code requirements. The layout of the building is an "L" shape with two equal sized wings. This layout is acceptable in a region with low seismic loads, but it is not encouraged in high seismic regions. The reentrant corner provides a place for stress to concentrate leading to building envelope failures. Also "L" shaped buildings are susceptible to torsion issues due to the natural layout direction of resisting walls in the longer direction of the wing. This can lead to the right wing being loaded in plane and the left wing being loaded out of plane, depicted in *Figure 8*. The result of this is that one side deflects more than the other, and this could be amplified by torsion created due to a large difference between the center of mass and center of rigidity.

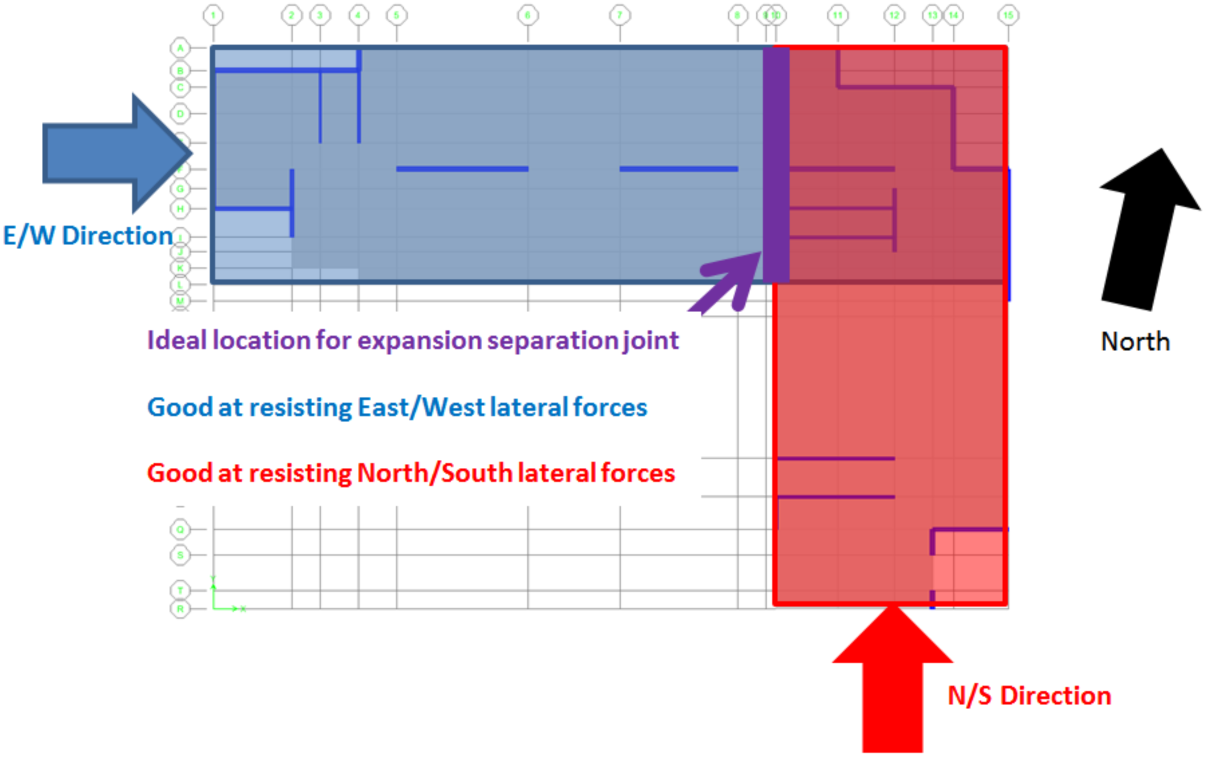


Figure 8: Existing Building Layout

Proposed Solution

For my proposal I am moving the location of my building from Pittsburgh, PA to San Diego, CA where seismic loads are much greater to over emphasize the effect of building layout on the design of the structure. This is realistic because Hyatt could decide they would like to build a similar shaped hotel structure in California. The move will lead to investigation into seismic loading and dissipation. For this investigation the structure will be redesigned in steel and as two separate wings with a focus on design of steel frames to resist earthquake loads and limit torsion. The building separation joint will allow the two wings to act independently, leading to better overall building performance in a seismic event. Steel frames have a higher ductility than masonry, which leads to a higher R-value and thus minimizing the seismic base shear. In addition steel frame structures are lighter in weight, also minimizing seismic base shear. Knowledge from AE 538 (MAE course) will be used to determine the placement of frames, load on them and design. Frames will also be placed to cause the least disturbance to the existing architecture and any changes needed will be investigated. The same precast concrete plank will be used for the floor system, but with a 2” concrete topping added to make the floor act as a rigid system, and the D-Beam from Girder-Slab Technologies will be used in order to keep a minimal floor to floor height and a flat undisturbed ceiling surface. The proposed structure’s cost and schedule will then be analyzed to compare to the existing structure in Pittsburgh, PA. The effect to existing architecture and to the existing cost and schedule will be used to compare the two building locations.

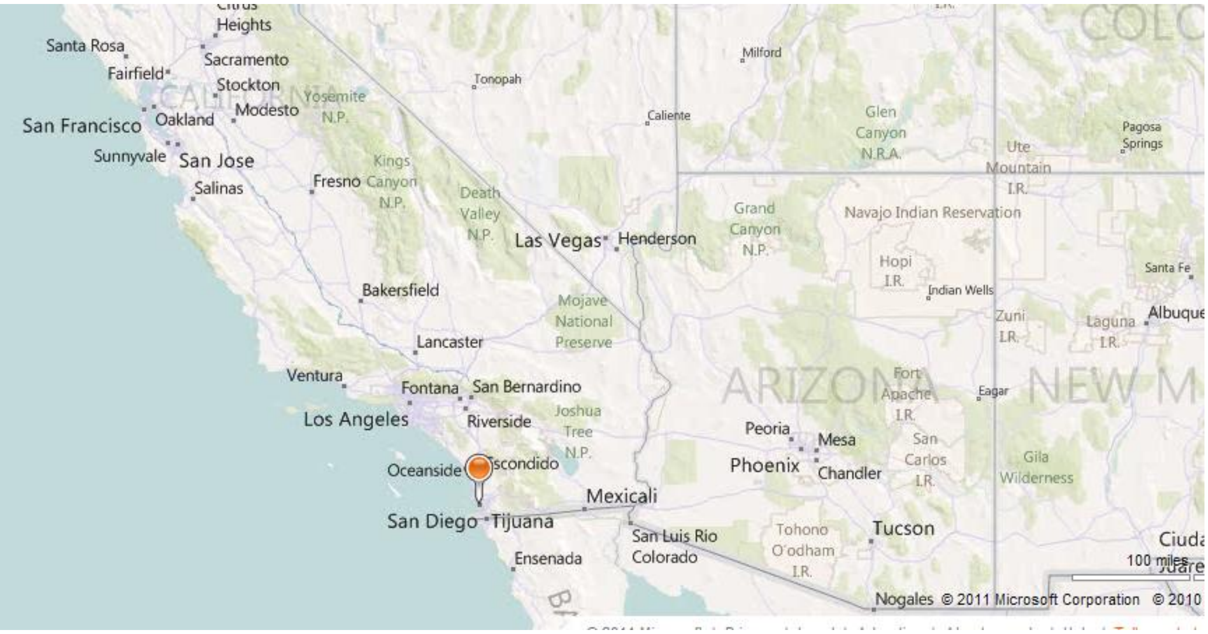


Figure 9: Map of Southwestern U.S. Courtesy of Bing.com

Proposed Structure Layout Overview

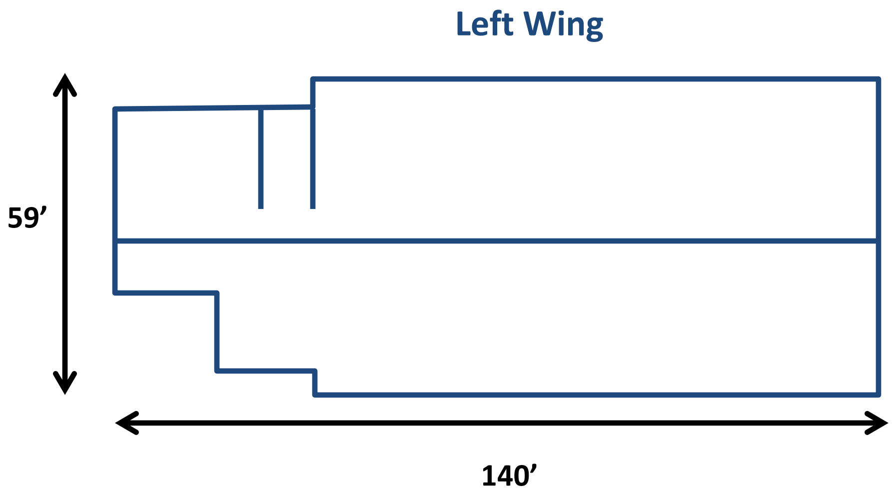


Figure 10: Simple View of Left Wing and Main Structural Lines

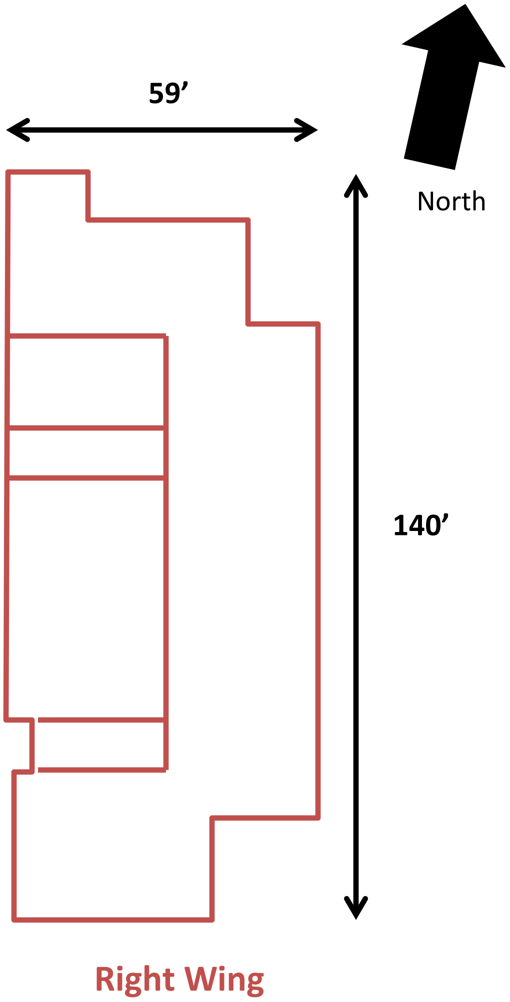


Figure 11: Simple View of Right Wing and Main Structural Lines

Figure 10 and *11* are a simplistic view of the left and right wing and their basic structural layout respectively. The lines shown depict general areas of structural elements such as steel beams and columns along the exterior of the building and along the interior corridor where concrete masonry bearing walls previously existed. Also there are lines where vertical travel elements are and special concrete shear walls will be. Special steel braced frames will be located around the perimeter and some in the perpendicular direction to balance resistance. In general left wing data will be shown with **BLUE** and right wing data will be shown with **RED**.

Materials

Concrete:	Shallow Foundations and Piers	3000 psi		
	Grade Beams and Pile Caps	4000 psi		
	Slabs on Grade	4000 psi		
	Shear Walls (Stair and Elevator Shafts)	4000 psi		
	Precast Concrete Planks	5000 psi		
Rebar:	Deformed Bars Grade 60	ASTM A615		
	Welded Wire Fabric	ASTM A185		
Structural Steel:	W Shapes	ASTM A992,	Fy = 50 ksi	Fu = 65 ksi
	Tubes (HSS Shapes)	ASTM 500 Grade B	Fy = 46 ksi	Fu = 58 ksi

Codes and Design Standards

Codes:

The following references were used by the engineer of record at Atlantic Engineering Services to carry out the structural design of the Hyatt Place North Shore

- The International Building Code 2006
- American Concrete Institute, Specifications for Masonry Structures (ACI 530.1)
- PCI MNL 120 “PCI Design Handbook – Precast and Prestressed Concrete”
- “Building Code Requirements for Reinforced Concrete, ACI 318”, American Concrete Institute
- “ACI Manual of Concrete Practice – Parts 1 Through 5”, American Concrete Institute
- “Manual of Standard Practice”, Concrete Reinforcing Steel Institute
- Specifications for Structural Steel Buildings (ANSI/AISC 360-150), American Institute of Steel Construction
- “Seismic Design Manual” American Institute of Steel Construction
- “Seismic Provisions for Structural Steel Buildings” American Institute of Steel Construction
- Minimum Design Loads for Buildings and Other Structures (ASCE 7-05), American Society of Civil Engineers – Old edition was used to be consistent with existing design
- Girder-Slab Technologies LLC, www.girder-slab.com

- Pittsburgh Flexicore P.C. Plank Specifications
- ETABS Modeling and Analysis – Computer & Structure, Inc.
- RAM Structural System
- RSMeans CostWorks – RS Means Construction Publishers and Consultants, Building Cost Data

Drift Criteria:

The following allowable drift criteria found in the International Building Code, 2006 edition.

- Allowable Building Drift: Δ_{wind} =H/400
- Allowable Story Drift: $\Delta_{seismic}$ = .02H_{sx} (all other structures)

Load Combinations:

The following load cases from ASCE 7-05 section 2.3 for factored loads using strength design; the greyed out portions don't apply in this case. These load combinations were considered in the ETABS model to determine the controlling case for the N/S and E/W directions. The existing structure is seismically controlled and the proposed location lowers the basic wind speed from 90 to 85 mph and greatly increases the seismic load, thus it is assumed that the building will be controlled by seismic load combinations.

• 1.4 (D + F)	COMBO1
• 1.2 (D + F + T) + 1.6(L + H) + .5(L_r or S or R)	COMBO2
• 1.2D + 1.6(L_r or S or R) + (L or .8W)	COMBO3
• 1.2D + 1.6W + L + .5(L_r or S or R)	COMBO4
• **1.2D + 1.0E + L** + .2S	**COMBO5 (controlling member design case)**
• .9D + 1.6W + 1.6H	COMBO6
• **.9D + 1.0E** + 1.6H	**COMBO7 (controlling case for uplift)**

Due to location, seismic loads are too great for wind to overcome even with the 1.6 multiplier in COMBO4 and COMBO6. Load combinations will be further discussed in the ETABS portion of the report. There are also load combinations for the earthquake load due to Seismic Design Category D, listed below.

- 100%X + 30%Y
- 30%X + 100%Y

Structural Study (depth)

Building Load Summary

The first step to redesigning the gravity and lateral force resisting structural systems is to determine the loads that they need to resist. The gravity loads are all similar to the existing structure and are listed below. There were changes to the wind loads because of a decrease in basic wind speed and different size wall areas that are loaded by wind. The seismic design loads change due to changes in location, and the structural systems ductility, redundancy, and overall weight. All of these changes are summarized below in preparation for design.

Gravity

Load conditions determined from ASCE 7-05

Gravity Load Summary		
Dead Loads		
Reinforced Concrete	**150**	**pcf**
Steel	**490**	**pcf**
Precast Concrete Plank	**88**	**psf**
Plank	63	psf
2" Structural Toping	25	psf
Superimposed Dead Load	**30**	psf
MEP	10	psf
Partitions	15	psf
Miscelaneous	5	psf
Snow Load	**0**	**psf**
Live Loads		
Public Areas	**100**	**psf**
Lobbies	**100**	**psf**
Public Corridors	**100**	**psf**
Room Corridors	**40**	**psf**
Hotel Rooms	**40**	**psf**
Stairs	**100**	**psf**
Mechanical	**125**	**psf**
Fitness Room	**100**	**psf**
Roof Live	**20**	**psf**

Table 3: Gravity Loads

Wind

Wind load is a pressure load applied to the exterior surface of the building. Different areas of the United States are more likely to be subject to high wind loads than others. Areas along the Gulf of Mexico and Atlantic Ocean coastlines are regions that have to be designed for higher wind loads due to the possibility of hurricanes during the summer. Once inland and away from that danger the design wind load comes from summer thunderstorm or cold fronts in the spring or fall. There are tornadoes, but they act over a very concentrated area with wind speeds too great to design for. The basic wind speed for Pittsburgh, PA and the majority of the U.S. is 90 mph, for California it is a slightly less 85 mph. Other factors such as topography and the effect of the height of the building are taken into effect by ASCE 7-05. A simplified Method 1 procedure is allowed for simple rigid buildings less than 60 ft tall. The variables for each wing needed to complete the Method 2 – Analytical Procedure are summarized below in *Table 4* and *5* since the Hyatt Place Hotel is 87.8 feet tall to the top of the penthouse. The values in *Table 3* vary with height, which is why wind pressures vary with height. Figure 12 shows how geometry affects the pressures on the building because of the area the wind hits verse the distance it must travel over the roof to get to the leeward side. With the variables from ASCE the wind pressure on the wall is determined and then the tributary area for each floor diaphragm is used to get the force acting on the diaphragm. Tables 6 and 7 are the procedures to find the forces at each level in each direction. Hand calculations are in appendix A.

Level	Height (ft)	K_z	q_z
2	19	0.89	13.99
3	28.8	0.97	15.25
4	38.6	1.03	16.19
5	48.4	1.08	16.98
6	58.2	1.12	17.61
7	68	1.16	18.24
Roof	77.8	1.2	18.87
PH Roof	87.7	1.23	19.34
Mean Ht	82.8	1.22	19.18

Table 3: Effect of Height on Pressures

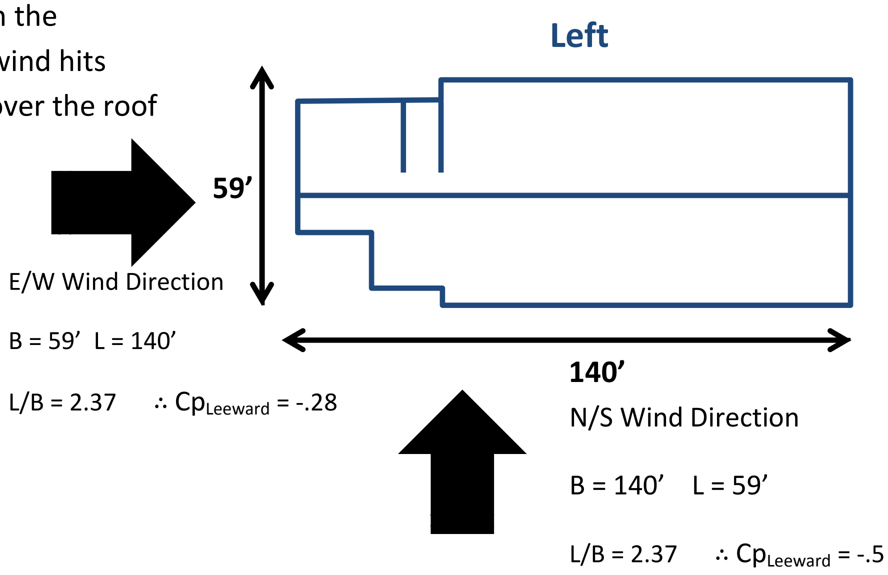


Figure 12: Effect of Building Geometry

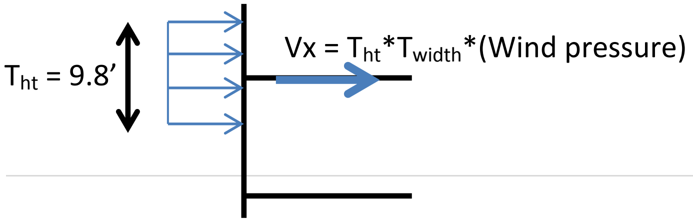


Figure 13: Load Path

Wind Design Variables Left Wing			
			ASCE Reference
Basic Wind Speed	V	85	Fig. 6-1
Wind Importance Factor	I	1.0	Table 6-1
Exposure Category		C	Sec 6.5.6.3
Directionality Factor	K_d	0.85	Table 6-4
Topographic Factor	K_{zt}	1.0	Sec 6.5.7.1
Velocity Pressure Exposure Coeficient Evaluated at Height Z	K_z	Varies (see appendix)	Table 6-3
Velocity Pressure at Height Z	q_z	Varies (see appendix)	Eq. 6-15
Velocity Pressure at Mean Roof Height	q_h	19.18	Eq. 6-15
Equivalent Height of Structure	>	52.68	Table 6-2
Intensity of Turbulence	I_z	0.185	Eq. 6-5
Integral Length Scale of Turbulence	L_z	538.91	Eq. 6-7
Background Response Factor (East/West)	Q	0.888	Eq. 6-6
Background Response Factor (North/South)	Q	0.857	Eq. 6-7
Gust Effect Factor	G	.85 (period = .8728 sec - rigid)	Eq. 6-4
Internal Pressure Coeficient	GC_{pi}	.18 (enclosed building)	Fig. 6-5
External Pressure Coeficient (Windward)	C_p	0.8	Fig. 6-6
External Pressure Coeficient (N/S Leeward)	C_p	-0.5	Fig. 6-6
External Pressure Coeficient (E/W Leeward)	C_p	-0.28	Fig. 6-6
External Pressure Coeficient (Side)	C_p	-0.7	Fig. 6-6

Table 4: Wind Design Variables for Left Wing

Wind Design Variables Right Wing			
			ASCE Reference
Basic Wind Speed	V	85	Fig. 6-1
Wind Importance Factor	I	1.0	Table 6-1
Exposure Category		C	Sec 6.5.6.3
Directionality Factor	K_d	0.85	Table 6-4
Topographic Factor	K_{zt}	1.0	Sec 6.5.7.1
Velocity Pressure Exposure Coeficient Evaluated at Height Z	K_z	Varies (see appendix)	Table 6-3
Velocity Pressure at Height Z	q_z	Varies (see appendix)	Eq. 6-15
Velocity Pressure at Mean Roof Height	q_h	19.18	Eq. 6-15
Equivalent Height of Structure	>	52.68	Table 6-2
Intensity of Turbulence	I_z	0.185	Eq. 6-5
Integral Length Scale of Turbulence	L_z	538.91	Eq. 6-7
Background Response Factor (East/West)	Q	0.857	Eq. 6-6
Background Response Factor (North/South)	Q	0.888	Eq. 6-7
Gust Effect Factor	G	.85 (period = .8766 sec - rigid)	Eq. 6-4
Internal Pressure Coeficient	GC_{pi}	.18 (enclosed building)	Fig. 6-5
External Pressure Coeficient (Windward)	C_p	0.8	Fig. 6-6
External Pressure Coeficient (N/S Leeward)	C_p	-0.5	Fig. 6-6
External Pressure Coeficient (E/W Leeward)	C_p	-0.28	Fig. 6-6
External Pressure Coeficient (Side)	C_p	-0.7	Fig. 6-6

Table 5: Wind Design Variables for Right Wing

Wind Loads Left Wing N/S & Right Wing E/W														
L = 59 ft B = 140 ft L/B = .42														
Level	Height Above Ground (z) (ft)	Story Height (ft)	K_z	q_z	q_h	Windward Pressure (psf) G = .85		Total Pressure (psf)	Force of Windward Pressure Only (k)	Force of Total Pressure (k)	Windward Shear Story (k)	Total Story Shear (k)	Windward Moment (ft-k)	Total Moment (ft-k)
					h = 82.8 ft	Windward	Leeward							
					Kz = 1.22	Cp = .8	Cp = -.5							
Penthouse Roof	88	10	1.23	19.34	19.18	13.15	-8.15	21.30	9.21	2.13	9.21	2.13	810.11	187.46
Main Roof	78	10	1.2	18.87	19.18	12.83	-8.15	20.98	17.96	16.49	27.17	18.62	1401.21	1286.43
7th Floor	68.167	9.83	1.16	18.24	19.18	12.40	-8.15	20.55	17.36	28.78	44.53	47.40	1183.68	1961.61
6th Floor	58.33	9.83	1.12	17.61	19.18	11.97	-8.15	20.13	16.76	28.18	61.30	75.58	977.89	1643.55
5th Floor	48.5	9.83	1.08	16.98	19.18	11.55	-8.15	19.70	16.16	27.58	77.46	103.15	784.00	1337.49
4th Floor	38.667	9.83	1.03	16.19	19.18	11.01	-8.15	19.16	15.41	26.82	92.88	129.98	595.97	1037.24
3rd Floor	28.83	9.83	0.97	15.25	19.18	10.37	-8.15	18.52	14.52	25.93	107.40	155.91	418.55	747.56
2nd Floor	19	19	0.89	13.99	19.18	9.51	-8.15	17.66	19.31	35.86	126.71	191.77	366.92	681.33
										Windward Base Shear =			126.71	Kips
										Total Base Shear =			191.77	Kips
										Sum of Windward Moment =			6538.34	ft-k
										Sum of Total Moment =			8882.68	ft-k

Table 6: Wind Forces Against the Long Side

Wind Loads Left Wing E/W & Right Wing N/S														
L = 140 ft B = 59 ft L/B = 2.37														
Level	Height Above Ground (z) (ft)	Story Height (ft)	K_z	q_z	q_h	Windward Pressure (psf) G = .85		Total Pressure (psf)	Force of Windward Pressure Only (k)	Force of Total Pressure (k)	Windward Shear Story (k)	Total Story Shear (k)	Windward Moment (ft-k)	Total Moment (ft-k)
					h = 82.8 ft	Windward	Leeward							
					Kz = 1.22	Cp = .8	Cp = -.28							
Penthouse Roof	88	10	1.23	19.34	19.18	13.15	-4.56	17.72	3.88	1.77	3.88	1.77	341.41	155.90
Main Roof	78	10	1.2	18.87	19.18	12.83	-4.56	17.40	7.57	6.77	11.45	8.54	590.51	527.98
7th Floor	68.167	9.83	1.16	18.24	19.18	12.40	-4.56	16.97	7.32	10.01	18.77	18.55	498.84	682.43
6th Floor	58.33	9.83	1.12	17.61	19.18	11.97	-4.56	16.54	7.07	9.76	25.83	28.31	412.11	569.21
5th Floor	48.5	9.83	1.08	16.98	19.18	11.55	-4.56	16.11	6.81	9.51	32.65	37.82	330.40	461.02
4th Floor	38.667	9.83	1.03	16.19	19.18	11.01	-4.56	15.57	6.50	9.19	39.14	47.00	251.16	355.30
3rd Floor	28.83	9.83	0.97	15.25	19.18	10.37	-4.56	14.93	6.12	8.81	45.26	55.82	176.39	254.04
2nd Floor	19	19	0.89	13.99	19.18	9.51	-4.56	14.08	8.14	12.04	53.40	67.86	154.63	228.83
										Windward Base Shear =			53.40	Kips
										Total Base Shear =			67.86	Kips
										Sum of Windward Moment =			2755.44	ft-k
										Sum of Total Moment =			3234.71	ft-k

Table 7: Wind Forces Against the Short Side

The two proposed building wings have the similar dimensions, just oriented 90 degrees different. The wind controls in the direction with the larger surface area to catch the wind. With a larger tributary area catching the wind, there is more load being applied to the diaphragm, which is then mainly loaded into the walls that are parallel to the wind direction. So the effect is compounding, but in this case pales in comparison to the expected seismic loads. The controlling wind case comes in the **North/South** direction for the **Left Wing** and the **East/West** direction for the **Right Wing**. *Figures 14* and *15* show the wind forces on the building section. This is one example where having the ability for the wings to act independently comes in handy. When the Left Wing is fully loaded with 191.77 kips of base shear, the Right Wing is loaded with 67.83 kips of base shear. The difference in force and wall orientation could lead to sizable differences in building deflection, but that is fine as long as there is a properly sized separation gap between the wings. Next step is to determine the seismic forces on the diaphragms, for most of the west coast this will be the force used for design.

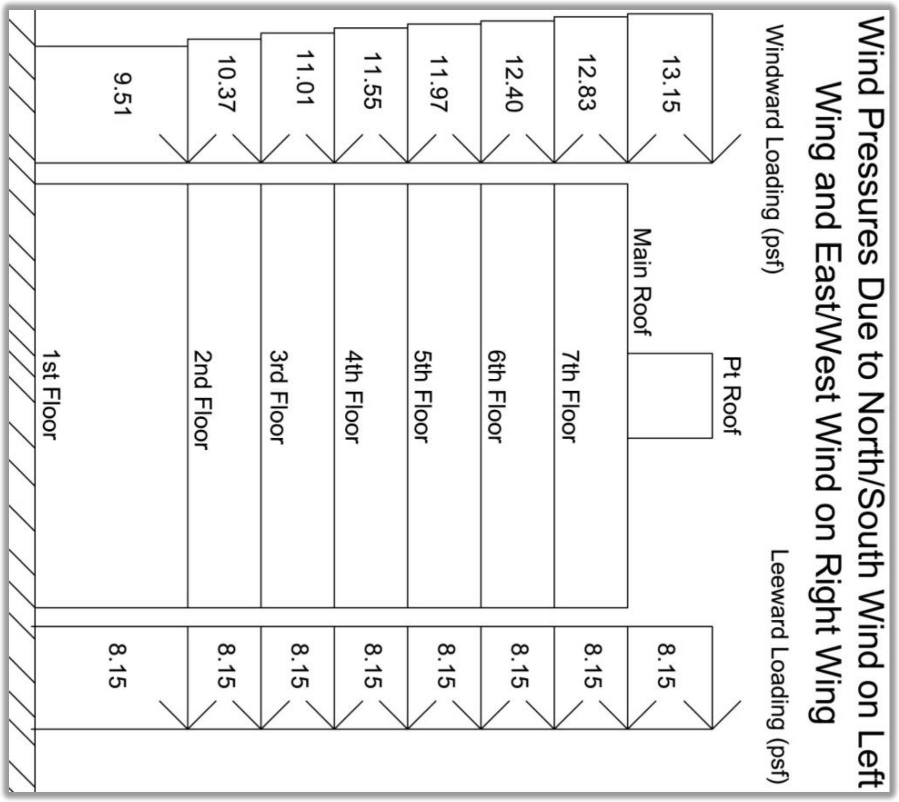


Figure 14: Wind Pressures On Building Facade

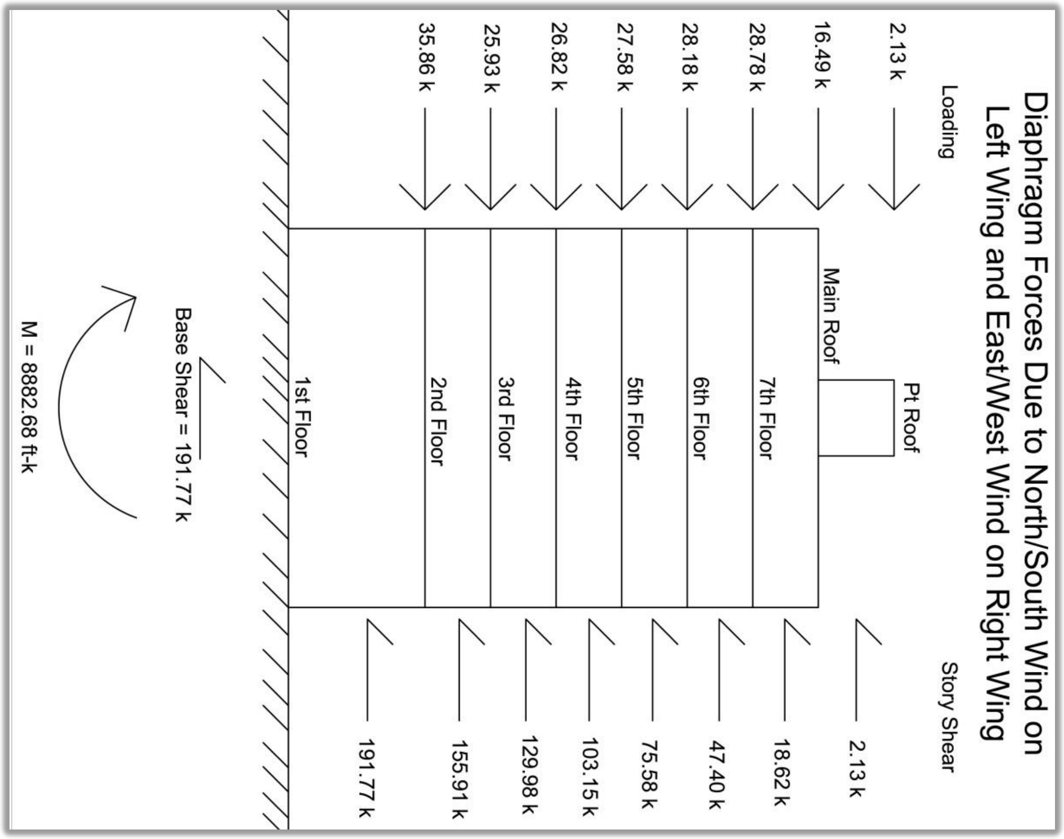


Figure 15: Wind Forces on Building Diaphragms

Seismic

The more predominate lateral load for the western half of the U.S. is seismic. Seismic loads on buildings originate in the earth's crust when two tectonic plates moving against each other build up enough stress that they suddenly break apart releasing energy through the rock and up to the surface. Earthquakes typically occur along fault lines where two plates meet; California is located along the intersection of North American Plate and the Pacific Plate, shown in *Figure 16*. This is part of the "Ring of Fire", the most active region in the world for earthquakes. There have been 3 violent earthquakes along this ring in the past year. The strength of the earthquake depends on how deep in the ground it originated and the type of rock. ASCE uses historical records and local geology to help predict the type of earthquake, its strength and likelihood of occurrence. After that ASCE also takes into effect building factors. Different buildings react differently to earth shacking. Mainly the period of a building and its ductility play a role on the load the building feels. A more ductile building has a higher R-value which leads to a lower seismic base shear; R-value depends on the seismic force resisting system. This along with building weight is the two main ways that the designer can limit the design seismic load. Each of the wings has a combination of special concentric braced frames (SCBFs) and special reinforced concrete shear walls (SRCSWs). The only SRCSWs are around the stair and elevator shafts, but the R-value for each direction is picked based on the lower R-value for frames resisting in that direction. *Figure 17* shows the controlling R-value for each direction of each wing.

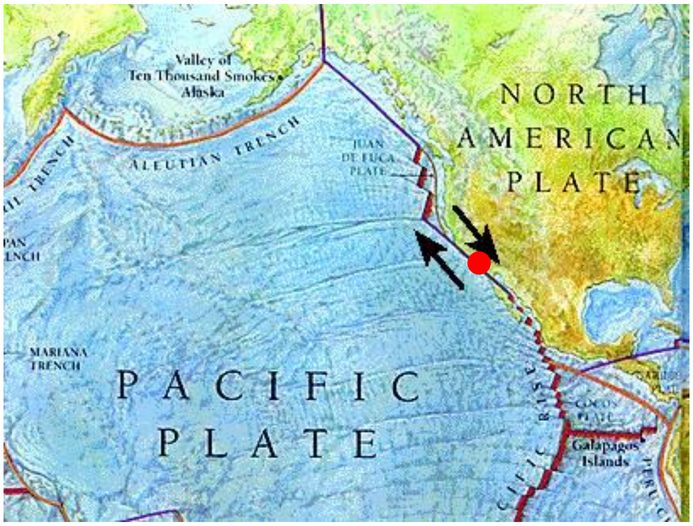


Figure 16: Tectonic Plate – from http://hisvorpal.wordpress.com/2010/07/02/north-to-alaska-2010-a-moose-odyssey/

Figure 17: Ring of Fire - from http://www.blippitt.com/west-coast-earthquake-imminent-fault-line-near-total-failure-video

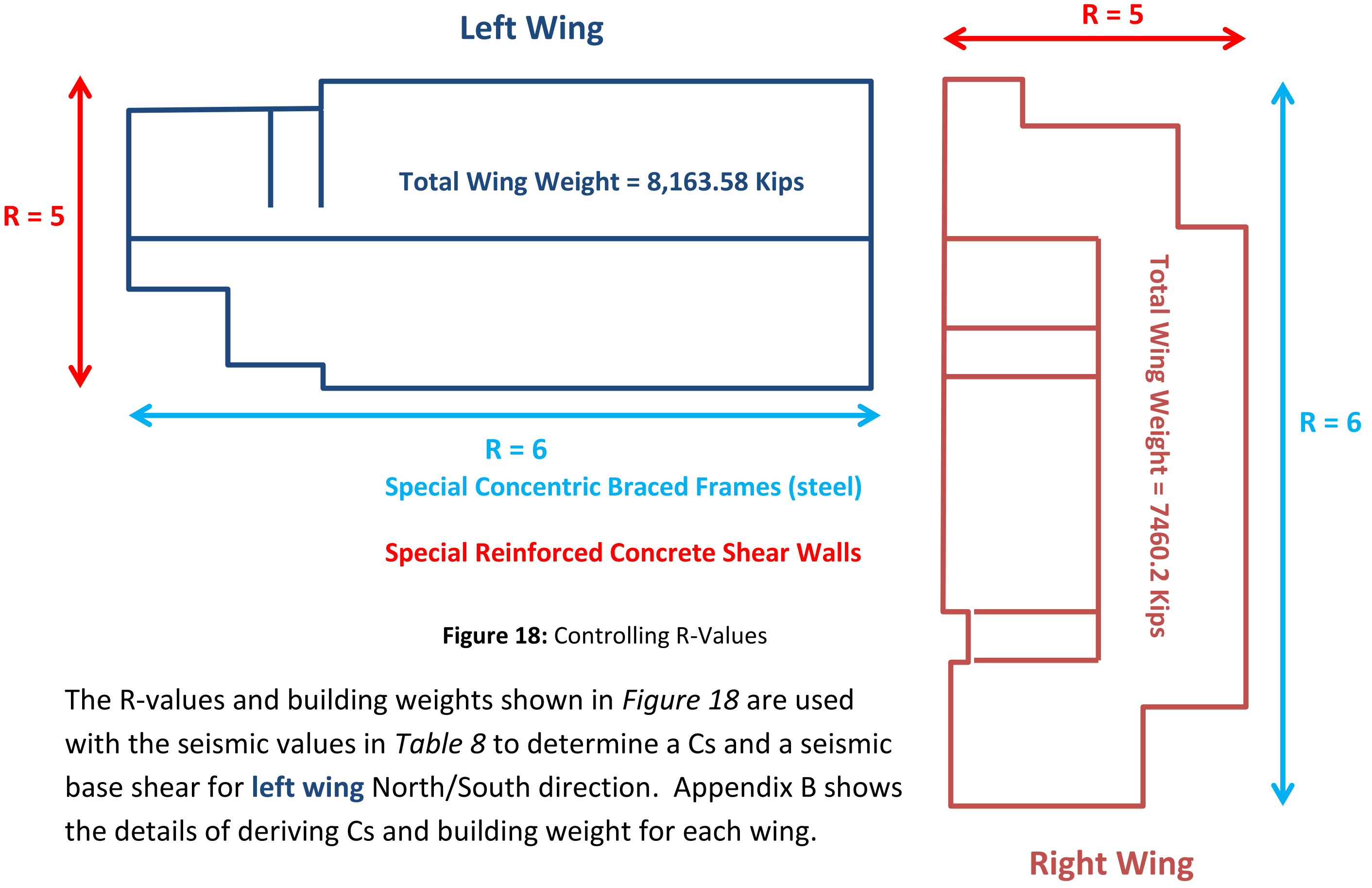


Figure 18: Controlling R-Values

The R-values and building weights shown in *Figure 18* are used with the seismic values in *Table 8* to determine a Cs and a seismic base shear for **left wing** North/South direction. Appendix B shows the details of deriving Cs and building weight for each wing.

$$V_{base} = C_s * W$$

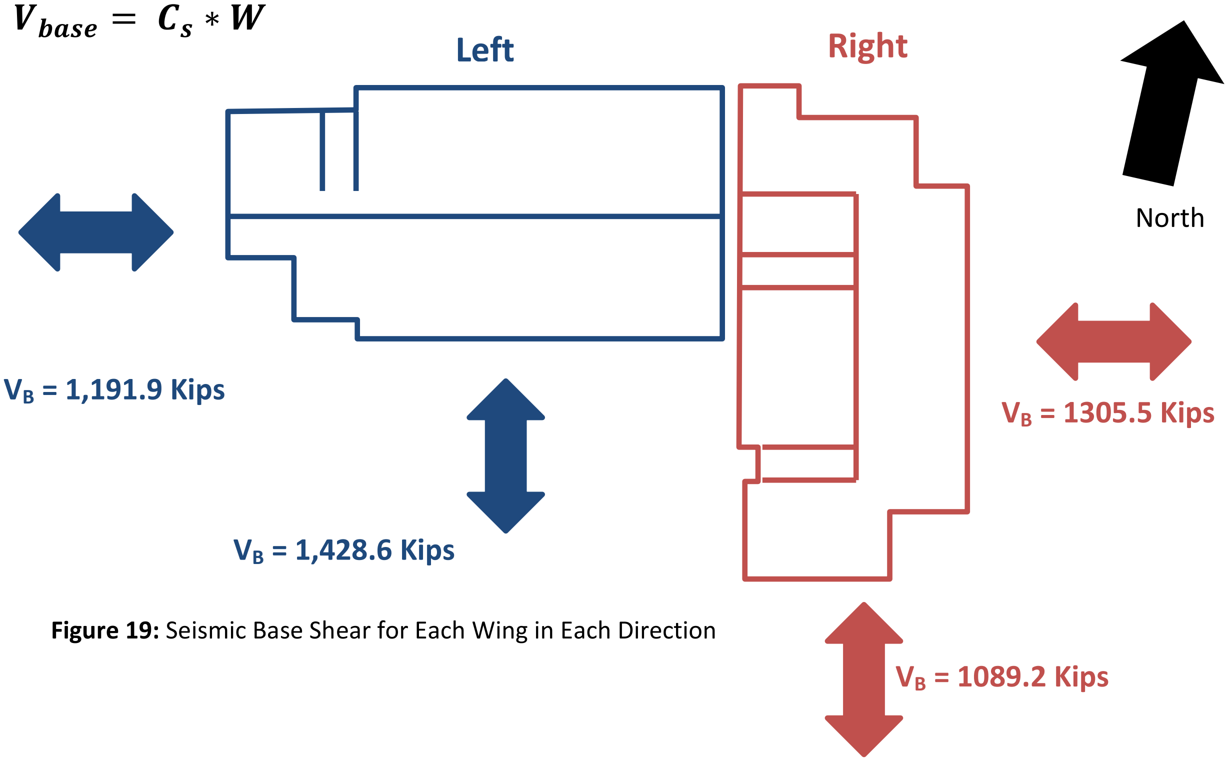


Figure 19: Seismic Base Shear for Each Wing in Each Direction

Seismic Design Variables (Left Wing N-S Direction)			
			ASCE Reference
Soil Classification		D (stiff soil)	Table 20.3-1
Occupancy Category		II	Table 1-1
Seismic Force Resisting System		Special Concentric braced frames (R = 6), **special reinforced concrete shear walls (R = 5)**	Table 12.2-1
Response Modification Factor	R	5	Table 12.2-2
Seismic Importance Factor	I	1.0	Table 11.5-1
Spectral Response Acceleration, Short	S_s	1.5	USGS Website
Spectral Response Acceleration, 1 sec.	S_1	0.5	USGS Website
Site Coeficient	F_a	1	Table 11.4-1
Site Coeficient	F_v	1.5	Table 11.4-2
MCE Spectral Response Acceleraton, Short	S_{MS}	1.5	Eq. 11.4-1
MCE Spectral Response Acceleration, 1 sec	S_{M1}	0.75	Eq. 11.4-2
Design Spectral Acceleration, Short	S_{DS}	1	Eq. 11.4-3
Design Spectral Acceleration, 1 sec.	S_{D1}	0.5	Eq. 11.4-4
Seismic Design Category	SDC	D (has some special design considerations)	11.6-1
Approximate Period Parameter	C_t	.02 (all other systems)	Table 12.8-2
Approximate Period Parameter	x	.75 (all other systems)	Table 12.8-3
Building Height	h_n	88'-0"	
Approximate Fundamental Period	T_a	0.57 sec.	Eq. 12.8-7
Long Period Transition Period	T_L	8 sec.	Fig. 22-15
Seismic Response Coeficient	C_s	0.175	Eq. 12.8-2
Structure Period Exponent	k	1.035 (2.5 sec. > T > .5 sec.)	Sec 12.8.3
Seismic Base Shear	**V**	**1428.6 kips**	**Eq. 12.8-1**

Table 8: Seismic Design Variables for the Left Wing in the North/South Direction

The S_S and S_1 values for San Diego, CA were found on the USGS website. The distinction of "Seismic Design Category D" has to be taken into account with some design considerations. Next the seismic base shear is distributed using the relative weight and height of the story when compared to the whole building.

$$C_{Vx} = \frac{w_x h_x^k}{\sum w_i h_i^k} \qquad k = 1.035 \qquad\qquad F_x = C_{Vx} V$$

These equations were used to make an excel spreadsheet to find the forces at each level in both directions in both wings. *Table 9* is the spreadsheet for the **left wing** in the North/South direction, the rest are included in appendix B.

Seismic Story Shear and Moment Calculations Left Wing (N-S)								
Level	Story Weight (K)	Height (ft)	K	$w_x h_x^k$	Vertical Distribution Factor C_{vx}	Forces (K) Fx	Story Shear (K) Vx	Moments (ft-K) Mx
Penthouse Roof	38.8	88.0	1.0	3992.6	0.0	12.7	12.7	1115.5
Main Roof	1083.5	78.0	1.0	98435.3	0.2	312.5	325.2	25366.3
7th Floor	1151.8	68.2	1.0	91021.0	0.2	289.0	614.2	41868.2
6th Floor	1151.8	58.3	1.0	77462.3	0.2	245.9	860.1	50172.2
5th Floor	1151.8	48.5	1.0	63993.4	0.1	203.2	1063.3	51571.2
4th Floor	1151.8	38.7	1.0	50616.2	0.1	160.7	1224.0	47329.6
3rd Floor	1158.4	28.8	1.0	37566.2	0.1	119.3	1343.3	38727.4
2nd Floor	1275.5	19.0	1.0	26865.2	0.1	85.3	1428.6	27143.4
Total	**8163.6**			**449952.2**				**283293.8**

Table 9: Seismic Story Shear and Moment Calculations for the Left Wing in the North/South Direction

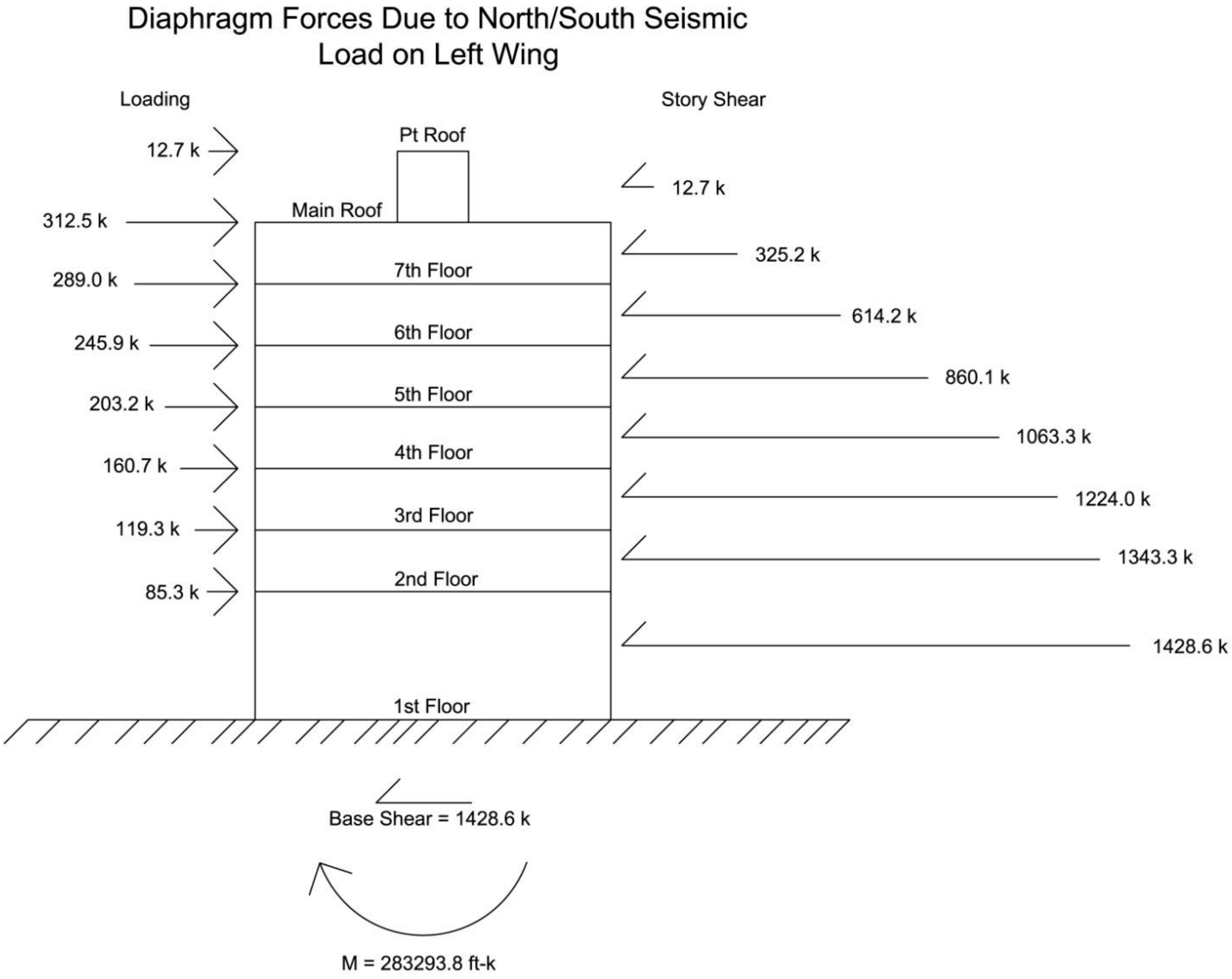


Figure 20: Diaphragm Forces Due to N/S Seismic Load on the Left Wing

Load Path

As can be seen in *Figure 20*, the story shear builds up as you go down the building, this leads into the discussion of load path. In most cases the gravity load path is fairly simple, as is the case with the Hyatt Place structure. Load starts out on the 2 way precast concrete plank floor slab and is then distributed to beams at either end in the span direction of the slab. Next the load in the beam is carried to the columns and down the columns to the foundation. This occurs on each floor and the amount of load in the columns adds up as you move down the structure. *Figure 21* shows a simple description of the typical gravity load path.

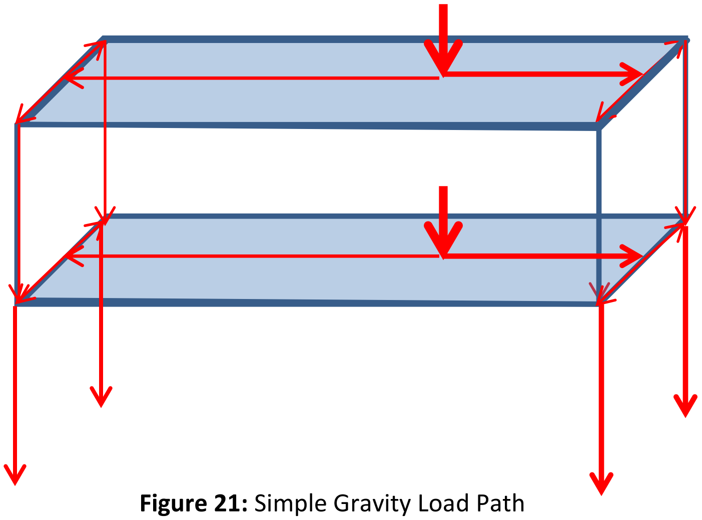


Figure 21: Simple Gravity Load Path

The load path for lateral load is similar in that it is additive as you move down the structure, with the lowest bay in a braced frame being designed for the highest load. The difference is that the load starts out as a horizontal load in the diaphragm and braced frames or shear walls channel load down to the foundation. *Figure 22* helps to explain how a Special Steel Concentric Braced frame turns **horizontal load** into **vertical load** in the columns. With seismic loading both the tension and compression braces are considered, but the tension brace is considered to take the majority of the load because the compression brace will eventually buckle due to the cyclic loading. In an X-Brace, Figure 22, the compressive brace and tensile brace loads add together to create uplift forces in the near column that counteract gravity loads and depending on the size of the gravity load can lead to issues of uplift at the base. The far column has downward force that is added to get gravity force and leads to the column design load. The connections are considered to be pinned, so the columns take mainly axial load. In a steel moment frame all of the members end up sized larger.

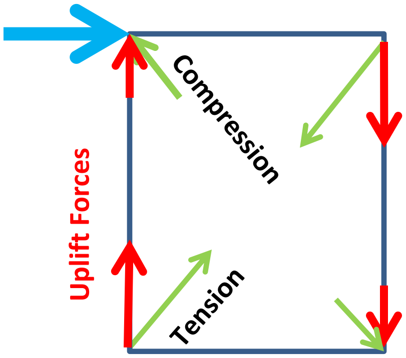


Figure 22: Load Path in Special Concentric Braced Frames

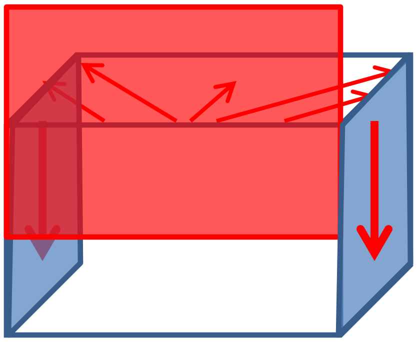


Figure 23: Wind Load Path

Wind Load originates as a pressure load on the exterior of the building. Using the concept of tributary area then the rigid floor diaphragm is loaded and this load is taken to lateral force resisting systems based on rigidity, load follows stiffness. In *Figure 23* the red depicts load.

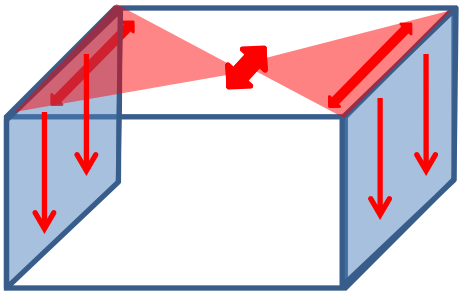


Figure 24: Seismic Load Path

Seismic load path acts in a slightly different manner. Seismic load on a building comes from the building's inertial resistance to movement. In a seismic event the ground moves back and forth and due to the fact that the building has mass, it wants to stay still; this is why heavier buildings have a higher seismic load. The amount of seismic load at a particular floor level depends on its weight and height above ground level. The force at that level acts at the center of mass. For this reason it is important to evenly layout lateral force resisting systems to try and keep the center of rigidity as close to the center of mass as possible. Any difference in these two leads to a twisting action on the building called torsion that leads to more force in lateral force resisting members.

Design Process Overview

Now that all of the loads on the structure and their paths have been determined it is time to begin the design phase. First step is to determine good locations for resistance elements. The locations of lateral force resisting elements are determined first because they have a greater potential to disrupt the architecture. The thought process behind their location will be discussed later on in the paper in the architectural study. Limitations on the span of the D-Beam were also considered when laying out lateral and gravity columns. The maximum span of the D-Beam is 15 feet, so this dictates the maximum column layout perpindicular to the span of the precast concrete planks. This spacing works nicely because it is also the width of hotel rooms. Now beams are layed out as needed to transfer load to the columns. One transfer truss is necessary on the ground floor of the right wing in order to keep open space for a large meeting room. *Figure 25* and *26* show the determined layout for columns in both wings of the building, with gravity members in red and lateral in black.

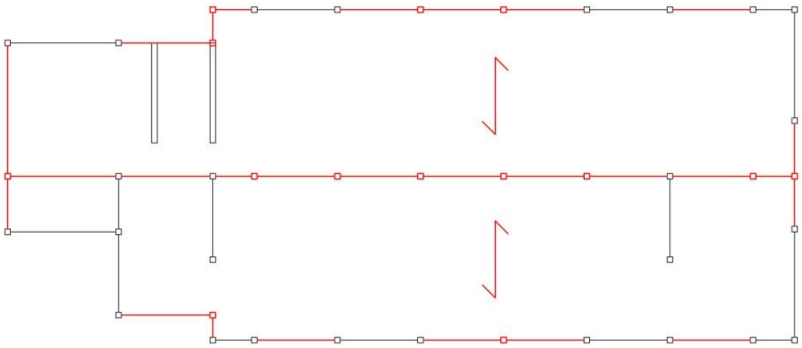


Figure 25: Left Wing Layout

With the layout determined the columns and beams were put in RAM to design for gravity loads and then spot checks were preformed by hand to confirm the design. Moving onto the lateral design the first step is to layout basic frames and determine their rigidities realative to each other in order to find the center of rigidity and design forces in each frame. Next frames were designed by hand and an ETABS model was constructed to confirm their design and the overall preformance of the structure. Lastly the ETABS model is also used to find overall building displacements and properly size the separation joint between the left and right wing.

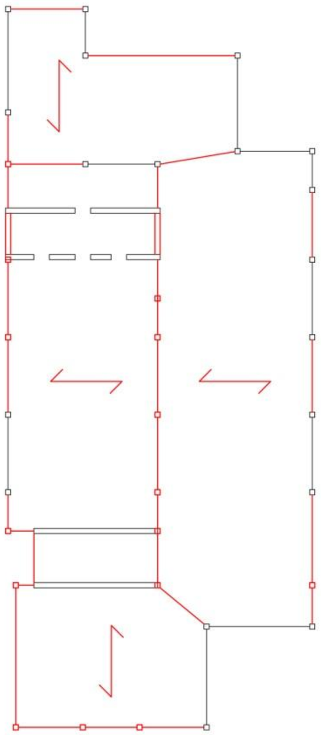


Figure 26: Right Wing Layout

Gravity Redesign

Floor System

The first portion of the gravity redesign is the floor system. The maximum span, dead load, and floor depth are integral parts to the next phases in design; beams and then columns. Basically the previously mentioned load path for gravity loads is followed for the order of member design. The floor system chosen to be used for the redesign is precast concrete planks with a 2 inch structural concrete topping and castellated D-Beams that minimize the floor to floor height at the interior spans and keep the ceiling flat. Precast concrete planks were used in the existing structure and the Girder-Slab system was investigated in technical report #2. The Hyatt Place North Shore existing floor system is 8" thick untopped precast concrete planks. This system simplifies design and expedites construction. The system efficiently carries the loading over relatively long spans ranging from 27'-6" to 30'-6". The concrete compressive strength of the floors is f'c=5000 psi. Extra strength is also added by prestressing the units. The planks used for this floor system will be the same except that they will have a 2" concrete topping that makes the floor act as a rigid diaphragm which is necessary for Seismic Design Category D.

Summary

Materials: **Concrete:** 4'-0" x 8" topped
f'c = 5000 psi

Grout: f'c = 4000 psi

Steel: DB 9x46 29000 ksi

Thickness: 10" (from concrete toping to bottom plank and girder)

Loading: Superimposed = 30 psf

Live Load = 40 psf

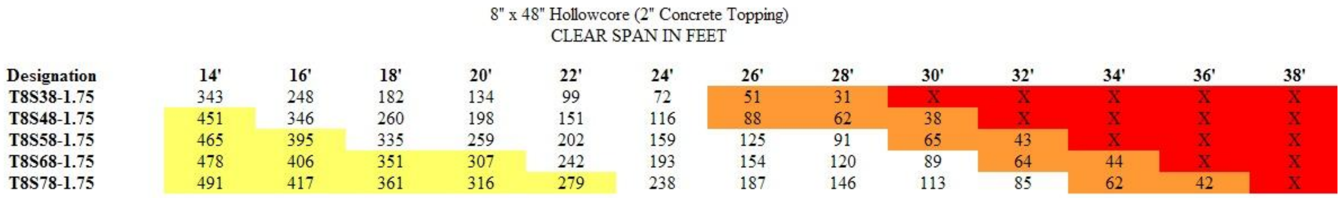
Total = 1.2*30 + 1.6*40 = 100 psf

Allowable = 106 psf (*Table 10*)

*Specify T8S78-1.75

Total System Weight:

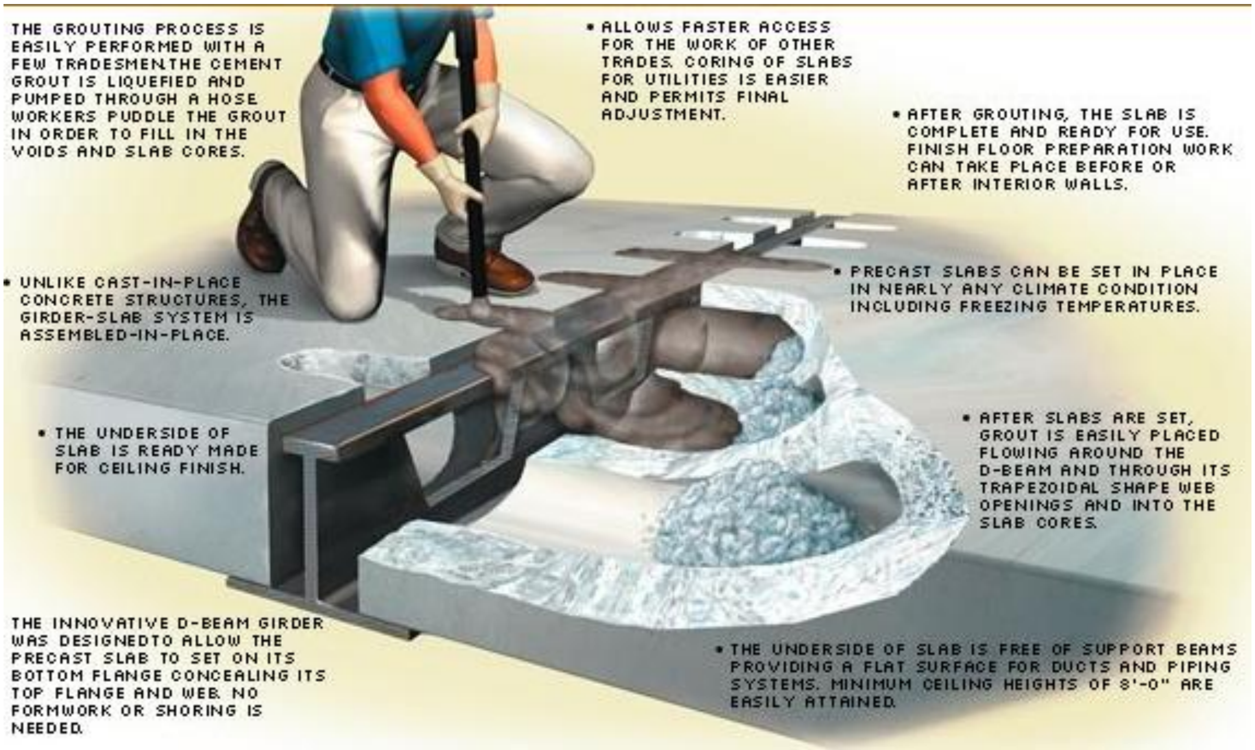
Plank Weight =	63 psf
Structural Toping =	25 psf
Total system =	**88 psf**



8" x 48" Hollowcore (2" Concrete Topping)
CLEAR SPAN IN FEET

Designation	14'	16'	18'	20'	22'	24'	26'	28'	30'	32'	34'	36'	38'
T8S38-1.75	343	248	182	134	99	72	51	31	X	X	X	X	X
T8S48-1.75	451	346	260	198	151	116	88	62	38	X	X	X	X
T8S58-1.75	465	395	335	259	202	159	125	91	65	43	X	X	X
T8S68-1.75	478	406	351	307	242	193	154	120	89	64	44	X	X
T8S78-1.75	491	417	361	316	279	238	187	146	113	85	62	42	X

Table 10: Load Capacity of Precast Concrete Plank with 2" Concrete Topping


THE GROUTING PROCESS IS EASILY PERFORMED WITH A FEW TRADESMEN.THE CEMENT GROUT IS LIQUEFIED AND PUMPED THROUGH A HOSE. WORKERS PUDDLE THE GROUT IN ORDER TO FILL IN THE VOIDS AND SLAB CORES.

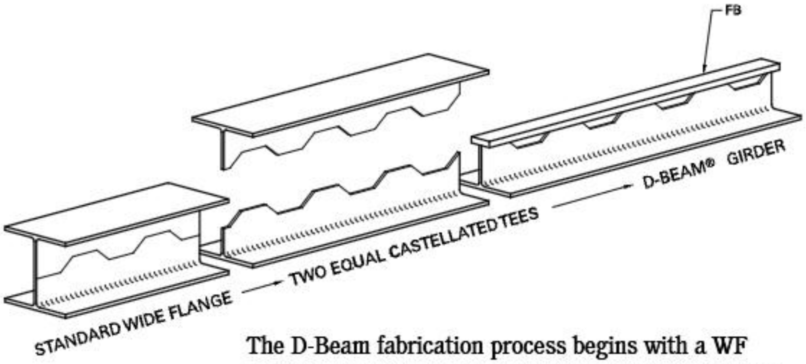
- ALLOWS FASTER ACCESS FOR THE WORK OF OTHER TRADES. CORING OF SLABS FOR UTILITIES IS EASIER AND PERMITS FINAL ADJUSTMENT.
- AFTER GROUTING, THE SLAB IS COMPLETE AND READY FOR USE. FINISH FLOOR PREPARATION WORK CAN TAKE PLACE BEFORE OR AFTER INTERIOR WALLS.
- UNLIKE CAST-IN-PLACE CONCRETE STRUCTURES, THE GIRDER-SLAB SYSTEM IS ASSEMBLED-IN-PLACE.
- PRECAST SLABS CAN BE SET IN PLACE IN NEARLY ANY CLIMATE CONDITION INCLUDING FREEZING TEMPERATURES.
- THE UNDERSIDE OF SLAB IS READY MADE FOR CEILING FINISH.
- AFTER SLABS ARE SET, GROUT IS EASILY PLACED FLOWING AROUND THE D-BEAM AND THROUGH ITS TRAPEZOIDAL SHAPE WEB OPENINGS AND INTO THE SLAB CORES.

THE INNOVATIVE D-BEAM GIRDER WAS DESIGNEDTO ALLOW THE PRECAST SLAB TO SET ON ITS BOTTOM FLANGE CONCEALING ITS TOP FLANGE AND WEB. NO FORMWORK OR SHORING IS NEEDED.

- THE UNDERSIDE OF SLAB IS FREE OF SUPPORT BEAMS PROVIDING A FLAT SURFACE FOR DUCTS AND PIPING SYSTEMS. MINIMUM CEILING HEIGHTS OF 8'-0" ARE EASILY ATTAINED.

Figure 27: Girder-Slab System Section View –

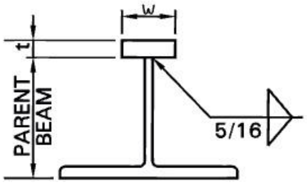
from – Girder-Slab Technologies – www.gider-slab.com/systems.asp

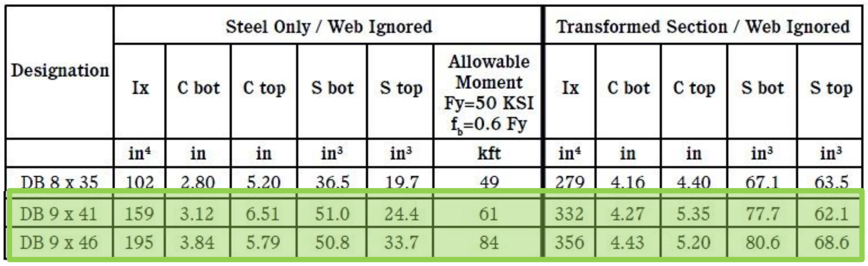


The D-Beam fabrication process begins with a WF section, uniquely cut to produce two D-Beam Girders without waste.

Figure 28: D-Beam construction - from – Girder-Slab Technologies – www.gider-slab.com/systems.asp

A system with a shallow beam is made possible by composite action between the D-Beam and the precast concrete planks. They are grouted together to make them act as a stronger unit. *Figure 27* shows overall system section and *Figure 28* shows how the D-Beam is constructed. Girder-Slab Technologies also provides design values for the D-Beam and sample calculations which are available in appendix 3. *Table 11* shows the variables





Designation	Steel Only / Web Ignored						Transformed Section / Web Ignored				
	Ix	C bot	C top	S bot	S top	Allowable Moment Fy=50 KSI f_b=0.6 Fy	Ix	C bot	C top	S bot	S top
	in⁴	in	in	in³	in³	kft	in⁴	in	in	in³	in³
DB 8 x 35	102	2.80	5.20	36.5	19.7	49	279	4.16	4.40	67.1	63.5
DB 9 x 41	159	3.12	6.51	51.0	24.4	61	332	4.27	5.35	77.7	62.1
DB 9 x 46	195	3.84	5.79	50.8	33.7	84	356	4.43	5.20	80.6	68.6

Table 12: D-Beam Properties

needed for design.

Beams

The beams necessary to be designed fall into four categories:

1. D-Beams located along interior spans
2. W-Shapes located on exterior spans, perpendicular to span direction
3. W-Shapes located on exterior spans, parallel to span direction
4. W-Shapes located in lateral frames (to be designed later)

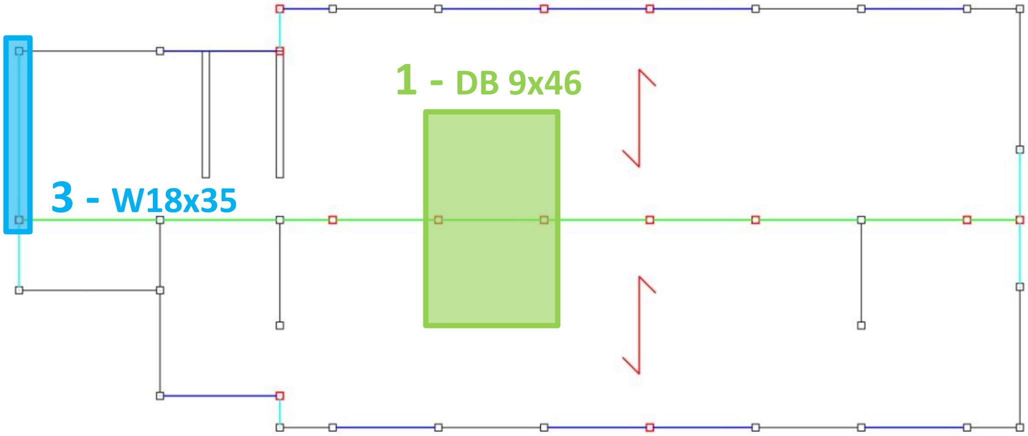


Figure 29: Location of Beams in Left Wing

Beam	LL	DL (plank)	SDL	DL (ext wall)	Trib. Width	Δ limit
1 – D-Beam	40 psf	88 psf	30 psf	None	30.5 ft	L/240
2 – W-Shape	40 psf	88 psf	30 psf	.462 klf	15 ft	L/600
3 – W-Shape	None	None	None	.462 klf	None	L/600

Table 12: Summary of Loads on Beams

These beams fall into different categories based upon the load they must carry. The data found in *Table 12* was used to design each of the 3 beams listed above by hand. Full calculations can be found in appendix C. The exterior beams were controlled by their deflection limit due to the fact that they are supporting a masonry façade and masonry is brittle and more prone to cracking and failure with deflection. *Figure 31* shows a sample cross section of the exterior wall and how the brick is supported. The assumed wall weight to be supported is 47 psf and each beam uses a steel angle to support 9.8 feet of wall.

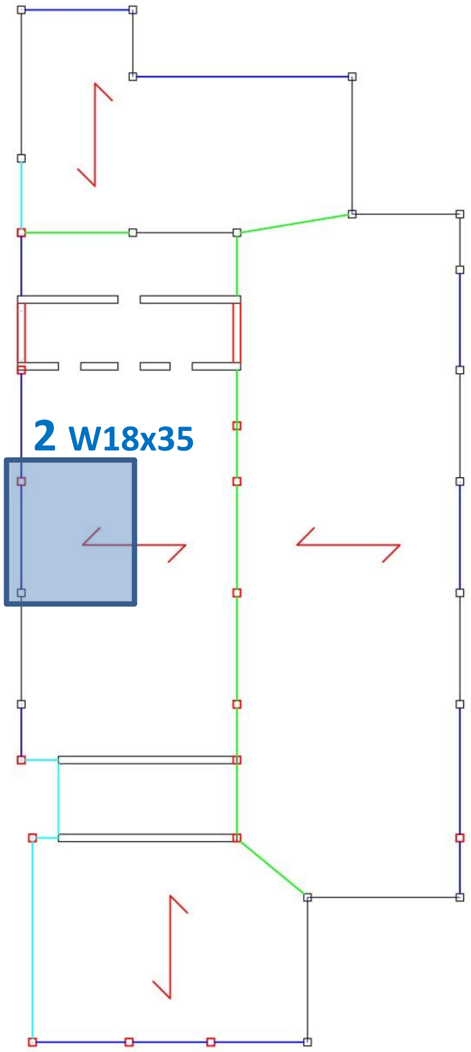


Figure 30: Location of Beams in Right Wing

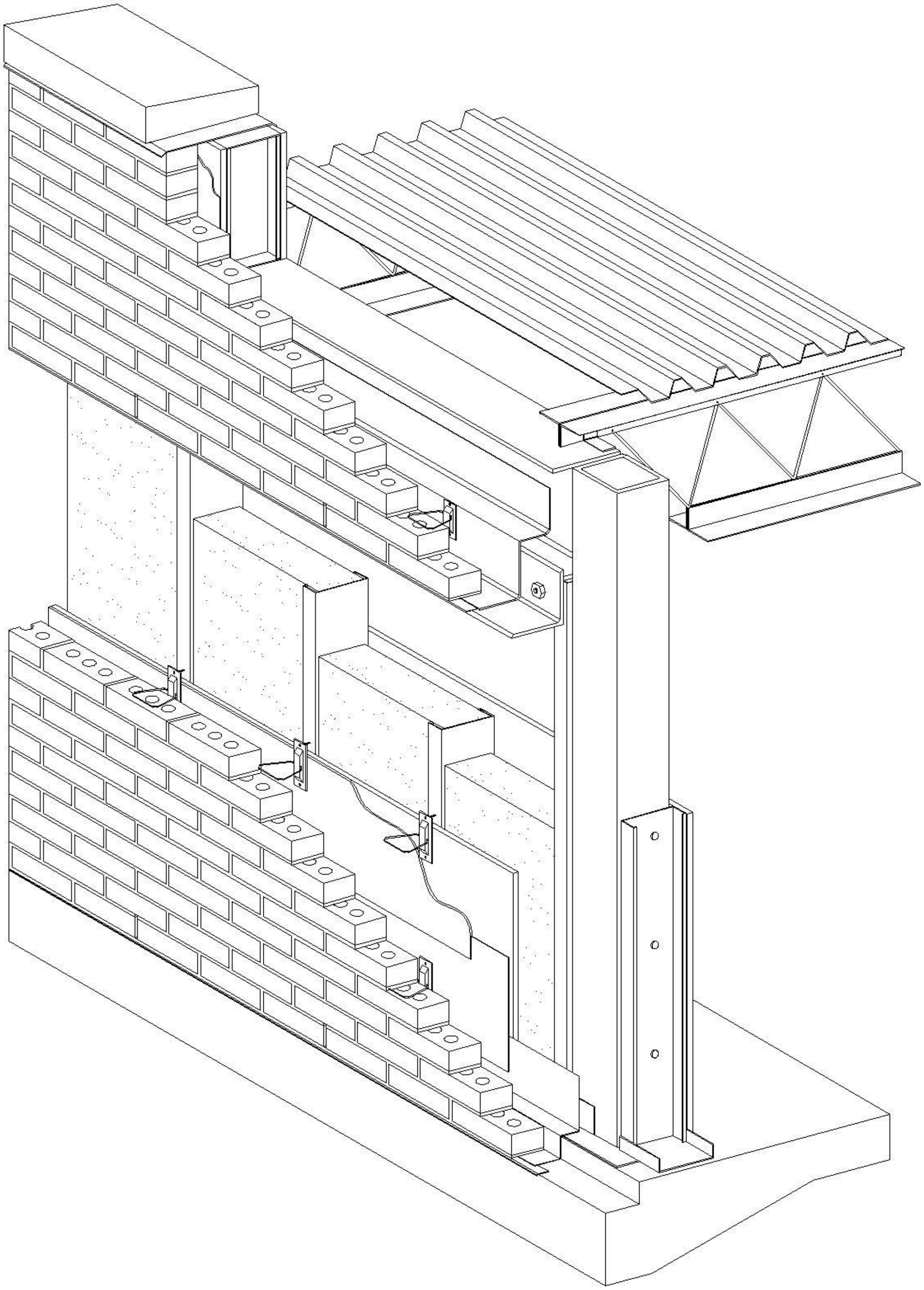


Figure 31: Sample Brick Veneer Wall Section – from http://www.masonrysystems.org/information/cavity-wall-brick-veneer-steel-stud/

Columns

Next the load moves through the beam in the form of shear, with the largest forces being at the ends where they are pin connected to the columns. All of the beam column connections are pin connected, even the braced frames, so the majority of the load in the columns are axial. There is the possibility of some moment being put into the column through the connection and some through PΔ-effects due to building drift from lateral loads, shown in *Figure 32*. PΔ-effects will be checked once a lateral model in ETABS determines story drift values. This is one good reason to allow for some extra load when looking up column sizes in AISC Table 4-1, so that there is room for combined loading in the H1-1 equations and tables shown in AISC Table 6-1.

$pP_r + b_x M_{rx} + b_y M_{ry} \leq 1$ (equation H1-1a) $\;\;\;\;\;$ ${}^{1}/_{2}\,pP_r + {}^{9}/_{8}\left(b_x M_{rx} + b_y M_{ry}\right) \leq 1$ (equation H1-1b)

If the steel superstructure was designed to have moment resisting frames then these equations would be much more crucial and member sizes would increase. Gravity columns were sized based upon their tributary area and the floors that they carry and their length, all connections are considered pin-pin (K=1) except the ground floor column that is pin-fixed (K=.7). *Figure 33* shows tributary areas for different columns, column 1 is designed in appendix C by hand for the ground floor. Gravity only columns are sized as W10s. The D-Beam limited the tributary area, if a different system was used and tributary areas were 30'x30' as opposed to 15' by 30' max, then larger columns sizes may have been needed. Lateral columns take more axial load and some are W12s, this will be discussed in the lateral redesign section.

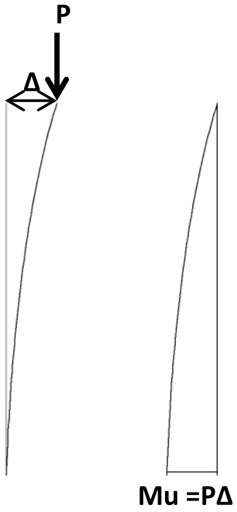


Figure 32: P-Delta Effect

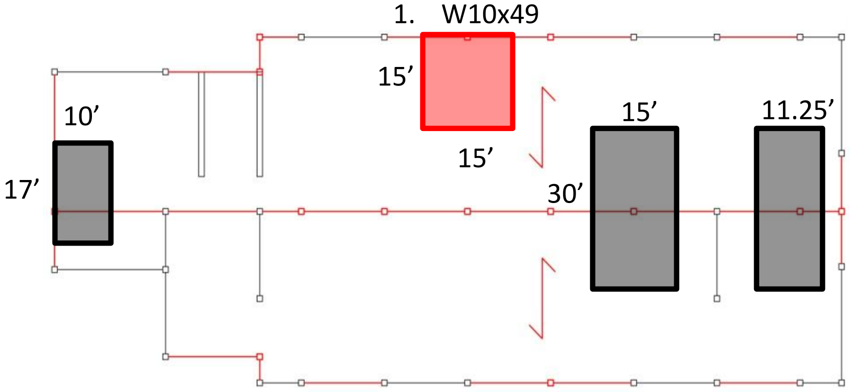


Figure 33: Left Wing Gravity Column sample Tributary Area

Tributary areas in the right wing are similar to those of the left wing, final designs will be show in RAMs results. Also in the right wing 1 large transfer truss was required to span a meeting room on the ground floor, its design and columns are also discused later. Column splices were considered to be after 3rd floor and 6th floor to try and make an efficient structure, *Figure 34*.

RAM

A RAM structural model was utilized to design all gravity columns, and exterior beams. In interior spans the D-Beam was used and sized by hand. In RAM it is possible to control the same things that are taken into account by hand. The span direction of the slab was put in so that beams and columns got the correct load. Some beams and columns do not take load from the slab because the beams run parallel to the span direction of the slab. The slab is still connected to the beam in lateral frames that run parallel to assure that lateral frames receive proper diaphragm loading.

Beams

The exterior beams were loaded with the area load from the 1-way slab and a line load of .462 klf as previously determined, the deflection limit was also set to L/600. RAM is set to output the most efficient member for the design parameters, but sometimes this ends up in taller members than desired. In this situation the individual member is looked at to see its Ix value and then go to AISC Table 3-3 to pick a member with a suitable Ix for deflection and suitable depth for architectural reasons. Gravity beams were limited to a max of W18s. Lateral beams do not have the same architectural restrictions, which is good because SCBF beams tend to be large. *Figure 35* shows the “View/Update” option in RAM. It was also determined the transfer member in the right wing needs to be designed as a transfer truss, this will be discussed in the next section. Drawings with all beam sizes are available in appendix D.

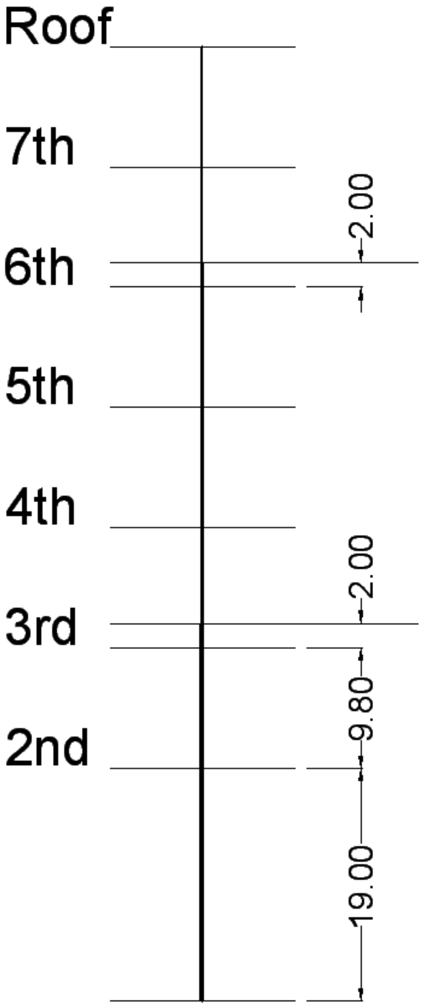
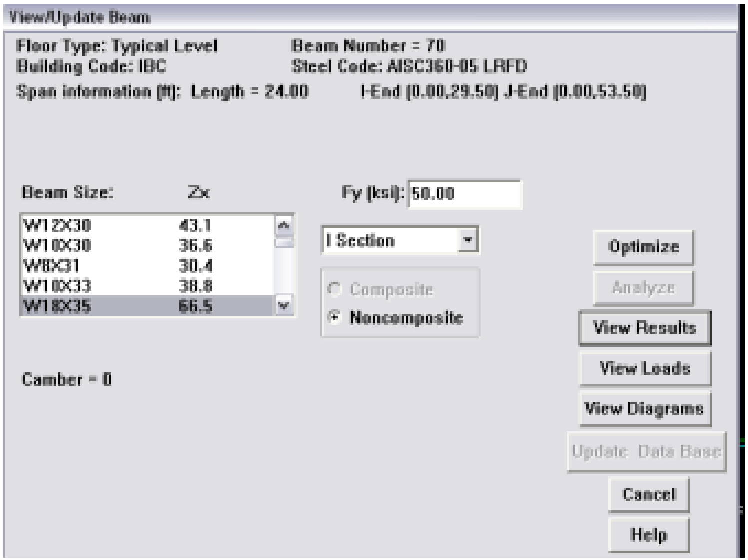


Figure 34: View of Column Splice Location



Floor Type: Typical Level
Building Code: IBC
Beam Number = 70
Steel Code: AISC360-05 LRFD
Span information (ft): Length = 24.00 I-End (0.00,29.50) J-End (0.00,53.50)

Fy (ksi): 50.00

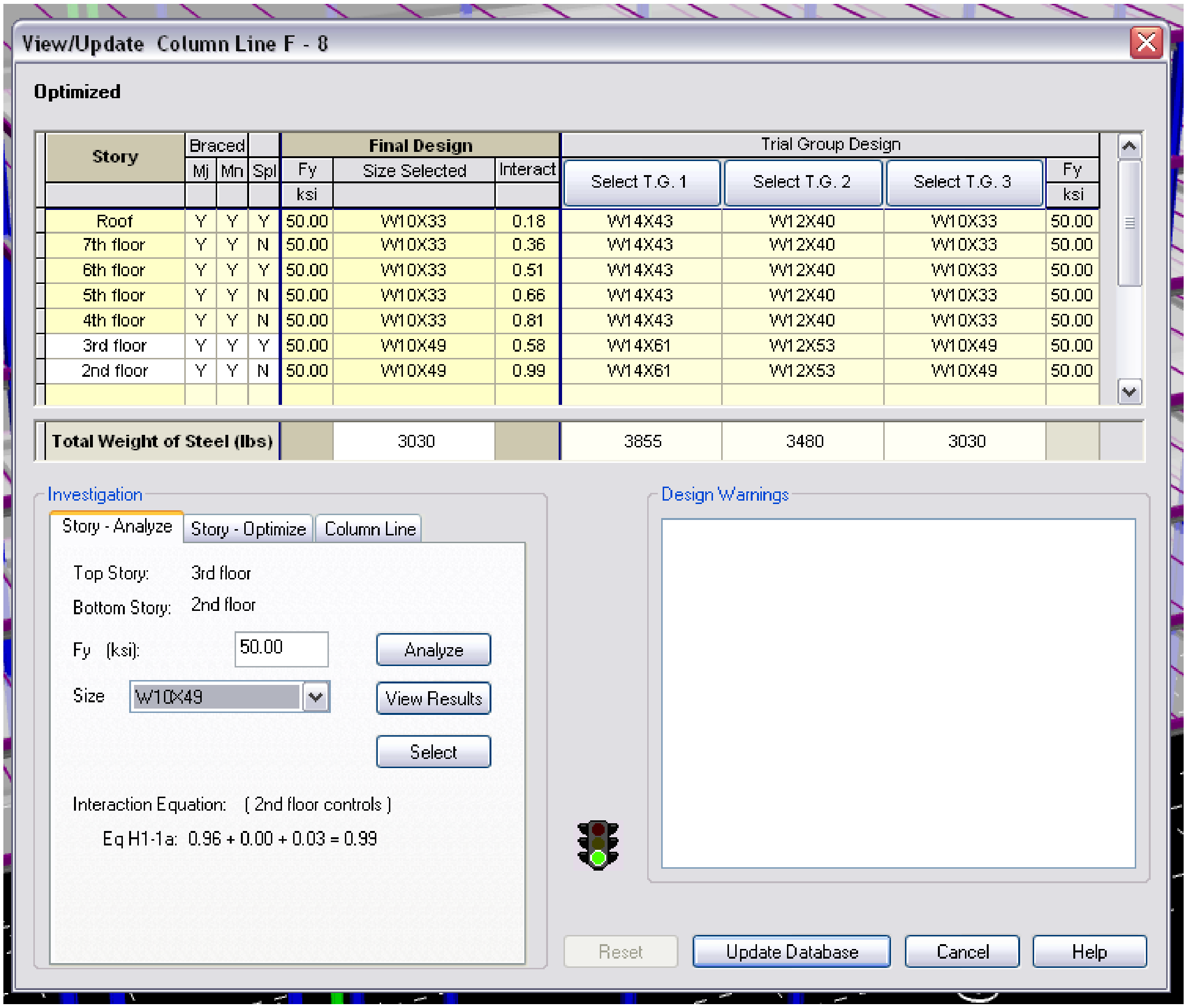
Beam Size:	Zx
W12X30	43.1
W10X30	36.6
W8X31	30.4
W10X33	38.8
W18X35	66.5

Camber = 0

Figure 35: View/Update in RAM Beam

Columns

Design of columns follows in similar fashion to design of beams. Members are put into the model and RAM sizes them to optimize weight. If any discrepancies with desired members are found they can be selected individually. It is also easy to see the values for H1-1 equations and adjust member if it is known more capacity is going to be taken. The sizes of columns in RAM were found to match up with hand calculations. *Figure 36* shows the H1-1 equation for the same column that was sized by hand earlier. Appendix D shows all gravity columns sized in RAM.



View/Update Column Line F - 8

Optimized

Story	Braced			Final Design			Trial Group Design			
	Mj	Mn	Spl	Fy	Size Selected	Interact	Select T.G. 1	Select T.G. 2	Select T.G. 3	Fy
				ksi						ksi
Roof	Y	Y	Y	50.00	W10X33	0.18	W14X43	W12X40	W10X33	50.00
7th floor	Y	Y	N	50.00	W10X33	0.36	W14X43	W12X40	W10X33	50.00
6th floor	Y	Y	Y	50.00	W10X33	0.51	W14X43	W12X40	W10X33	50.00
5th floor	Y	Y	N	50.00	W10X33	0.66	W14X43	W12X40	W10X33	50.00
4th floor	Y	Y	N	50.00	W10X33	0.81	W14X43	W12X40	W10X33	50.00
3rd floor	Y	Y	Y	50.00	W10X49	0.58	W14X61	W12X53	W10X49	50.00
2nd floor	Y	Y	N	50.00	W10X49	0.99	W14X61	W12X53	W10X49	50.00
Total Weight of Steel (lbs)					3030		3855	3480	3030	

Investigation

Top Story: 3rd floor
Bottom Story: 2nd floor
Fy (ksi): 50.00
Size: W10X49

Interaction Equation: (2nd floor controls)
Eq H1-1a: 0.96 + 0.00 + 0.03 = 0.99

Figure 36: View/Update in RAM Column

Transfer Truss

In both RAM and by hand it was determined that 45 feet was too long to span with a W-Shape. The loads on the member produce a moment in the center of 10,312 ft-kips. With this load, AISC Table 3-10 says that only a W36x800 would work, and that is a very large member. The existing Hyatt Place structure has a 5 foot deep transfer girder, this design was used as a starting point since loads should be similar or slightly less in a steel structure.

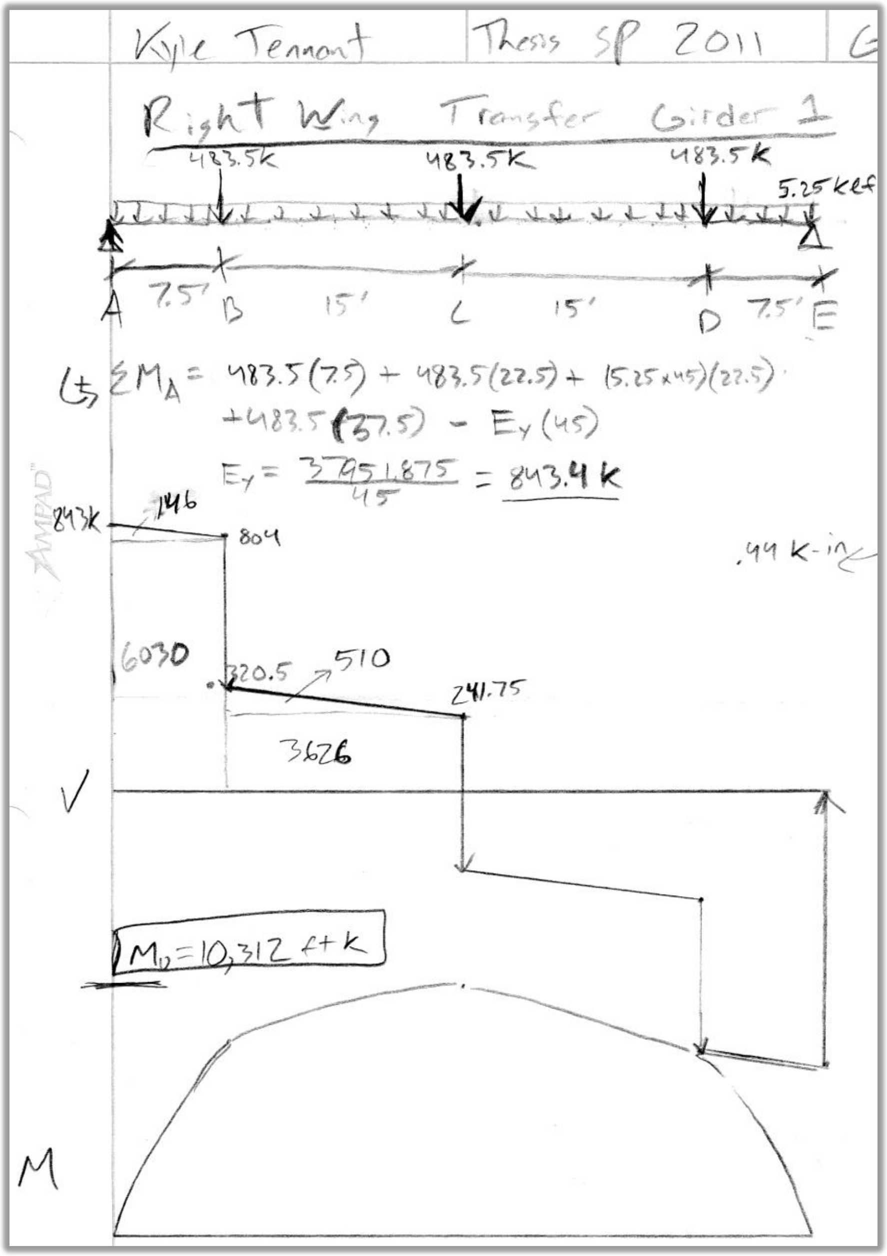
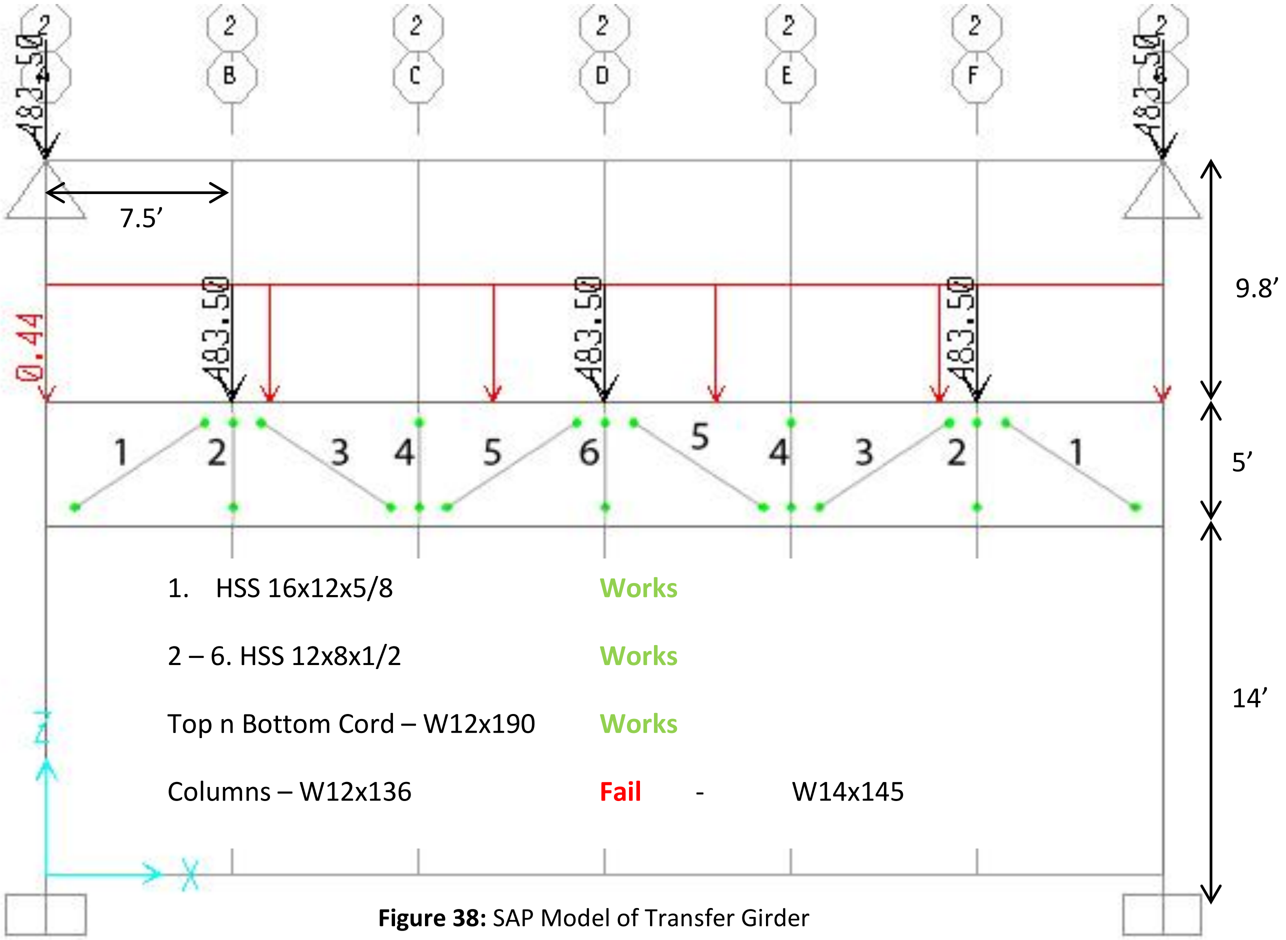


Figure 37: Shear and Moment Diagram for Transfer Span

The transfer truss was modeled and loaded in SAP to determine if the load in members was below their capacity and aid in any redesign needed. *Figure 38* shows the model loads and members. The column was modeled from the foundation to the 3rd floor level because moment will be taken by the column member and it is expected to take the load because D-Beams are not able to be moment connected to columns because they lack a substantial top flange.



1. HSS 16x12x5/8 Works

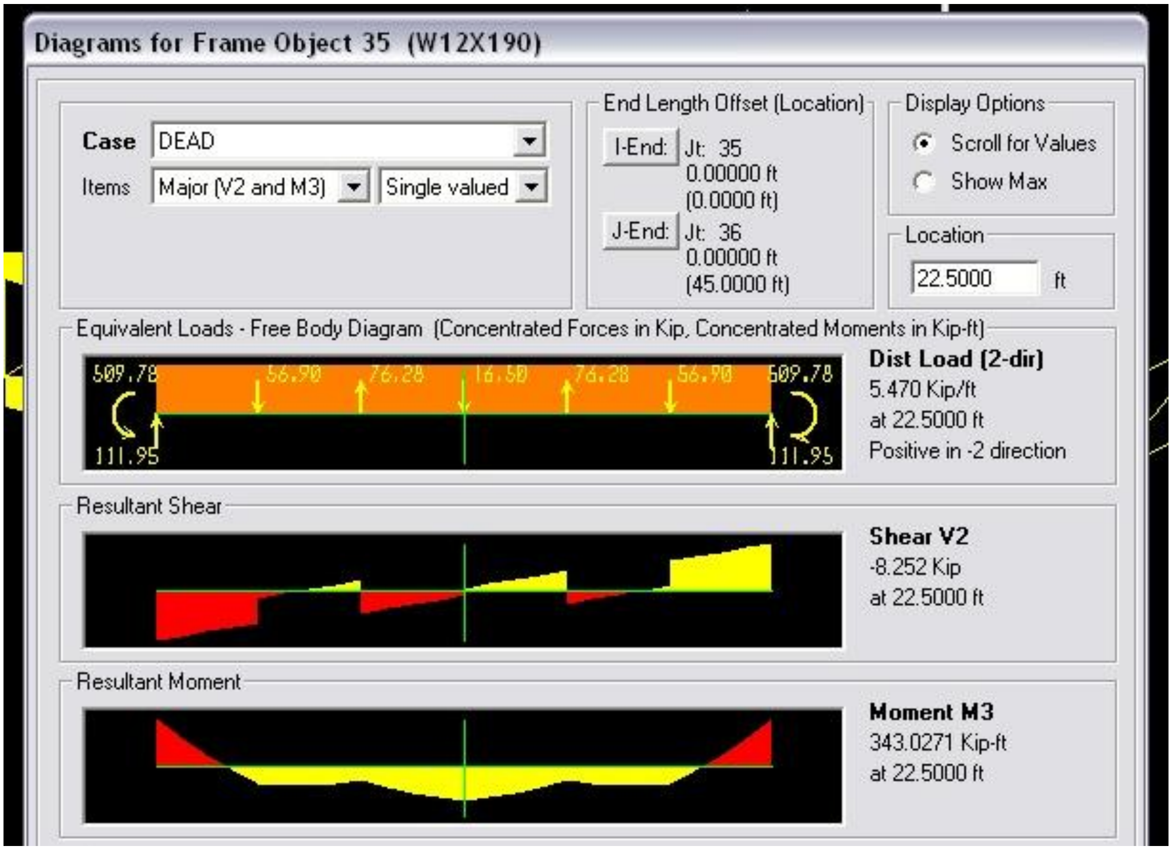
2 – 6. HSS 12x8x1/2 Works

Top n Bottom Cord – W12x190 Works

Columns – W12x136 Fail - W14x145

Figure 38: SAP Model of Transfer Girder

All the braces were pin-pin and the cords are moment connected to the columns. The forces were looked at in each member to make sure they are sufficient. *Figures 39* and *40* show the member forces in the top cord of the truss, the remainder of member force diagrams and calculations are found in appendix D. The advantage of the truss can be seen in the member forces of the top cord. The beam is mainly in compression rather than having an enormous bending moment in the center. Then the bottom cord is mainly in tension and the braces also transfer axial load. The top and bottom cord have a moment arm between them that creates moment couple. The model was also used to check deflection, *Figure 41*.



Diagrams for Frame Object 35 (W12X190)

Case DEAD
Items Major (V2 and M3) Single valued

End Length Offset (Location)
I-End: Jt: 35 0.00000 ft (0.0000 ft)
J-End: Jt: 36 0.00000 ft (45.0000 ft)

Display Options
● Scroll for Values
○ Show Max

Location 22.5000 ft

Equivalent Loads - Free Body Diagram (Concentrated Forces in Kip, Concentrated Moments in Kip-ft)

Dist Load (2-dir)
5.470 Kip/ft
at 22.5000 ft
Positive in -2 direction

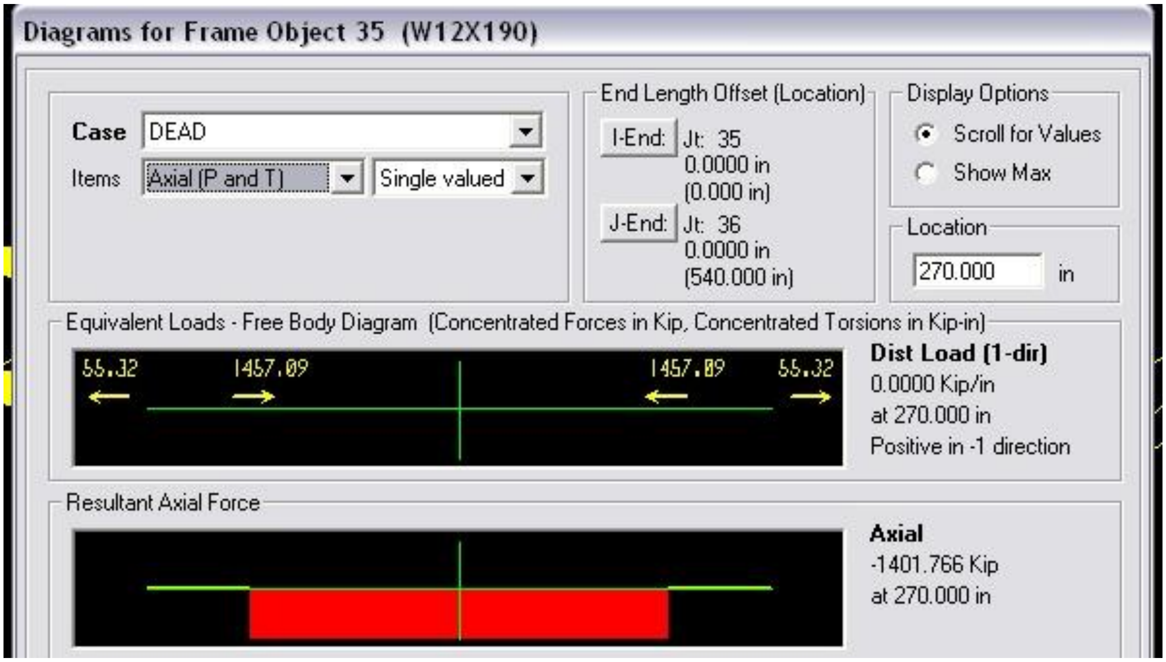
Resultant Shear

Shear V2
-8.252 Kip
at 22.5000 ft

Resultant Moment

Moment M3
343.0271 Kip-ft
at 22.5000 ft

Figure 39: SAP Model of Top Cord Shear and Moment



Diagrams for Frame Object 35 (W12X190)

Case DEAD
Items Axial (P and T) Single valued

End Length Offset (Location)
I-End: Jt: 35 0.0000 in (0.000 in)
J-End: Jt: 36 0.0000 in (540.000 in)

Display Options
● Scroll for Values
○ Show Max

Location 270.000 in

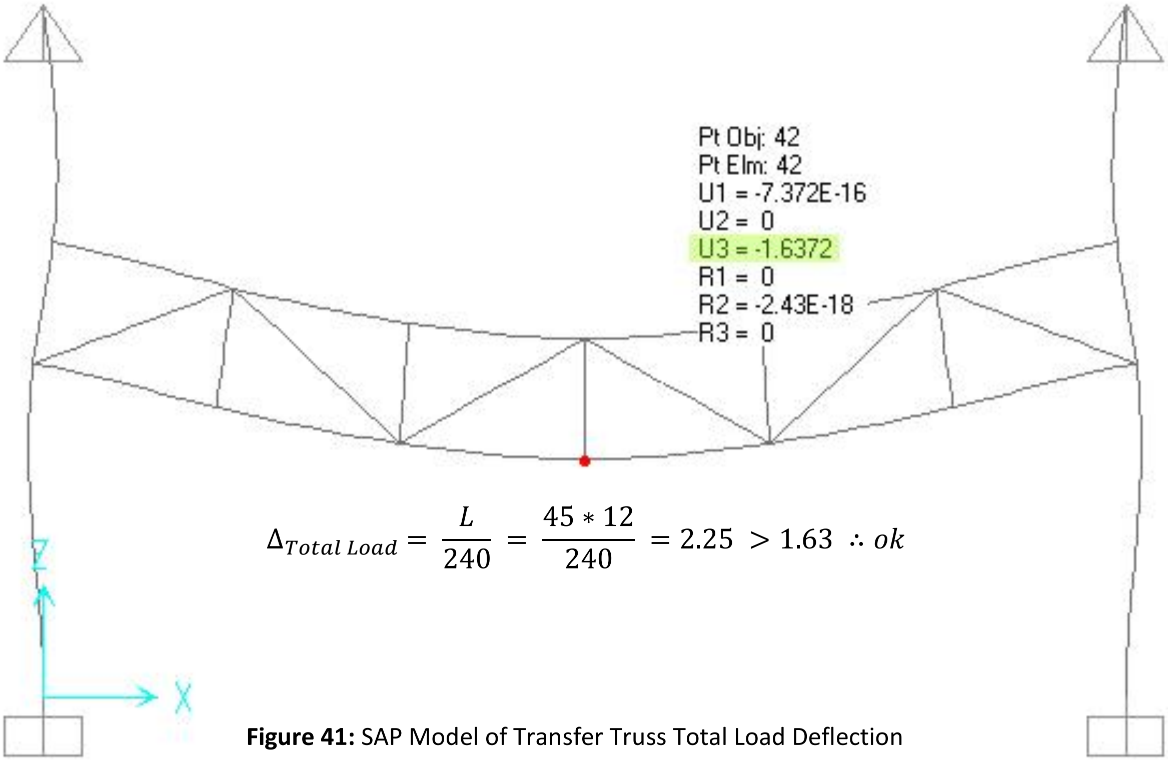
Equivalent Loads - Free Body Diagram (Concentrated Forces in Kip, Concentrated Torsions in Kip-in)

Dist Load (1-dir)
0.0000 Kip/in
at 270.000 in
Positive in -1 direction

Resultant Axial Force

Axial
-1401.766 Kip
at 270.000 in

Figure 40: SAP Model of Top Cord Axial Compression



$$\Delta_{Total\ Load} = \frac{L}{240} = \frac{45 * 12}{240} = 2.25 \ > 1.63 \ \therefore ok$$

Figure 41: SAP Model of Transfer Truss Total Load Deflection

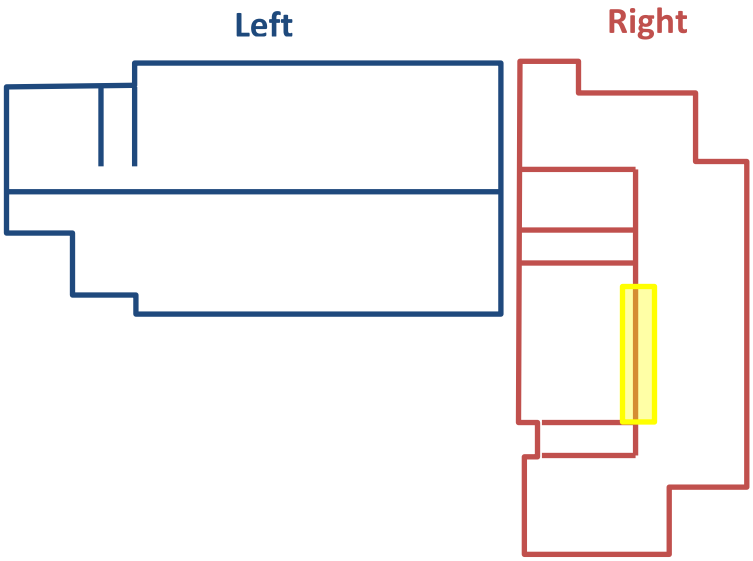


Figure 42: Location of Transfer Truss Over Ground Floor

Lateral Redesign

With the gravity loads designed for is time to move on to design members to transfer lateral loads to the foundation. For San Diego, CA the lateral loads become more influential in design than they were in Pittsburgh, PA. Previously the system that carried the gravity load also easily carried the lateral loads. The structural system has be changed to steel in order to limit the seismic base shear through the reduction of building weight and an increased R-value. To increase the R-value it has been decided to use Special Reinforced Concrete Shear Walls (R = 5) around stair and elevator shafts and use Special Concentric Braced Frames (R = 6) in exterior and some interior locations. Multiple types of braced frames were considered when weighing architectural impact, strength, ductility, and cost. Moment frames were not considered, a few types of concentric and eccentric braced frames are possible given the architectural layout. Concentric braced frames can be worked in around the architecture and they provide a simpler solution than eccentric braced frames. In the lateral redesign 2 concentric braced frames will be designed by hand, X-Braced, and Inverted-V Braced. These two of these braces are shown in *Figure 43*. The frames will be designed for strength by hand and then ETABS will be used to look at the building reaction as a whole and size the separation joint in between building wings.

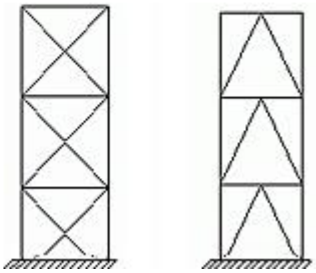


Figure 43: Types of Braced Frames

Lateral Element Location

Location of resistance is very important to the lateral force resistance of the building. Force follows stiffness and seismic load originates at the center of mass, so even placement of lateral resistance is important to building behavior and the total amount of lateral load that braced frames have to take. When load is applied away from the center of resistance it causes there to be torsion about the center of rigidity. The torsion puts additional load in lateral frames, additive in some and subtractive in others. As you can see in *Figure 44* the wall with less resistance ends up with more load being added to it due to torsion and leads to a more uneven displacement. If the difference in displacement is too great then there is a torsional amplification factor (Ax) multiplied times the torsional moment.

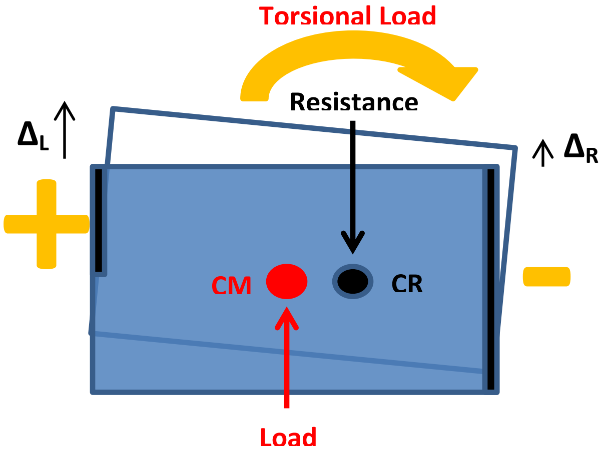


Figure 44: Effect of Eccentricity

$$1.2 * \left(\frac{\Delta_L + \Delta_R}{2}\right) \leq \Delta_L \therefore \ Torsionally\ Irregular$$

This is why the "L" was divided up into two wings; each wing tends to be naturally better at resisting force in the long direction. Splitting the building into two similar sized rectangles makes balancing forces much more reasonable. Frames are evenly placed around the exterior where the architectural façade permits, and additionally in the interior in the short direction. It is a goal to provide an approximately equal amount of resistance in the North/South and East/West directions. *Figures 45* and *46* show the location of lateral force resisting elements in the left and right wings respectively.

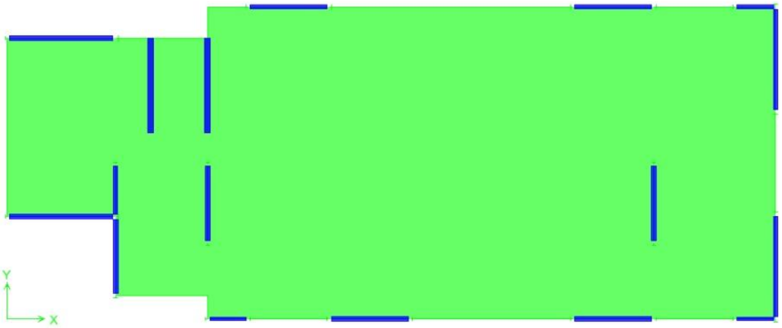


Figure 45: Location of Lateral Elements in Left Wing

Overall elements were able to be placed evenly around each wing. The thought process behind locations will be explained in the architectural study.

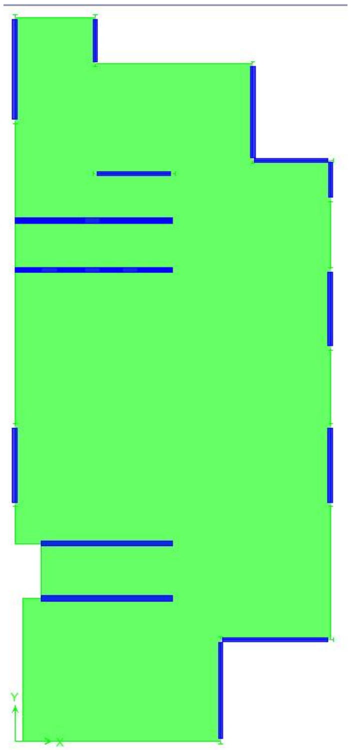


Figure 46: Location of Lateral Elements in Right Wing

Stiffness

The next step in determining forces in lateral force resisting members is to determine the stiffness of all frames relative to each other. Stiffer elements deflect less. All of the frames were modeled in ETABS with the same size members and appied a 1 kip load at the top in order to estimate relative stiffness. The deflection was taken off and used to determine a stiffness for each.

$$K = \frac{P}{\Delta}$$

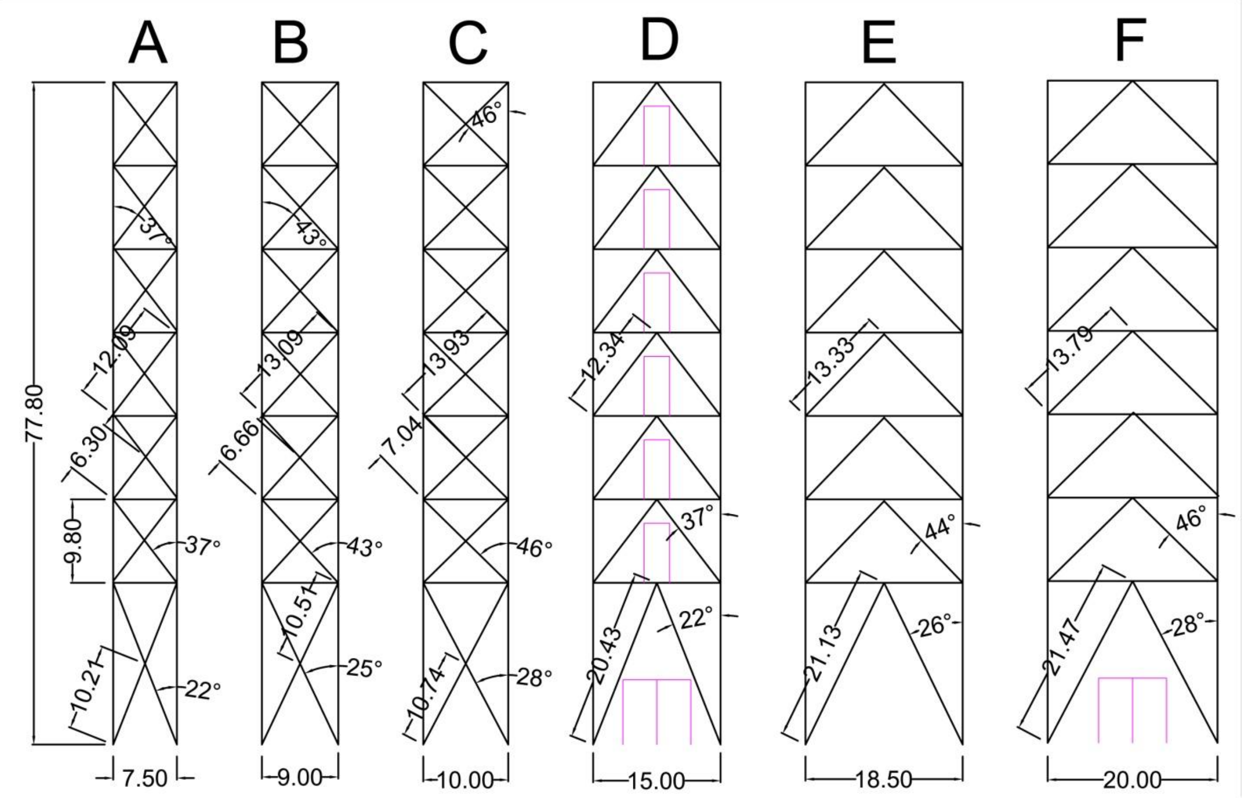


Figure 47: Types of Steel Braced Frames

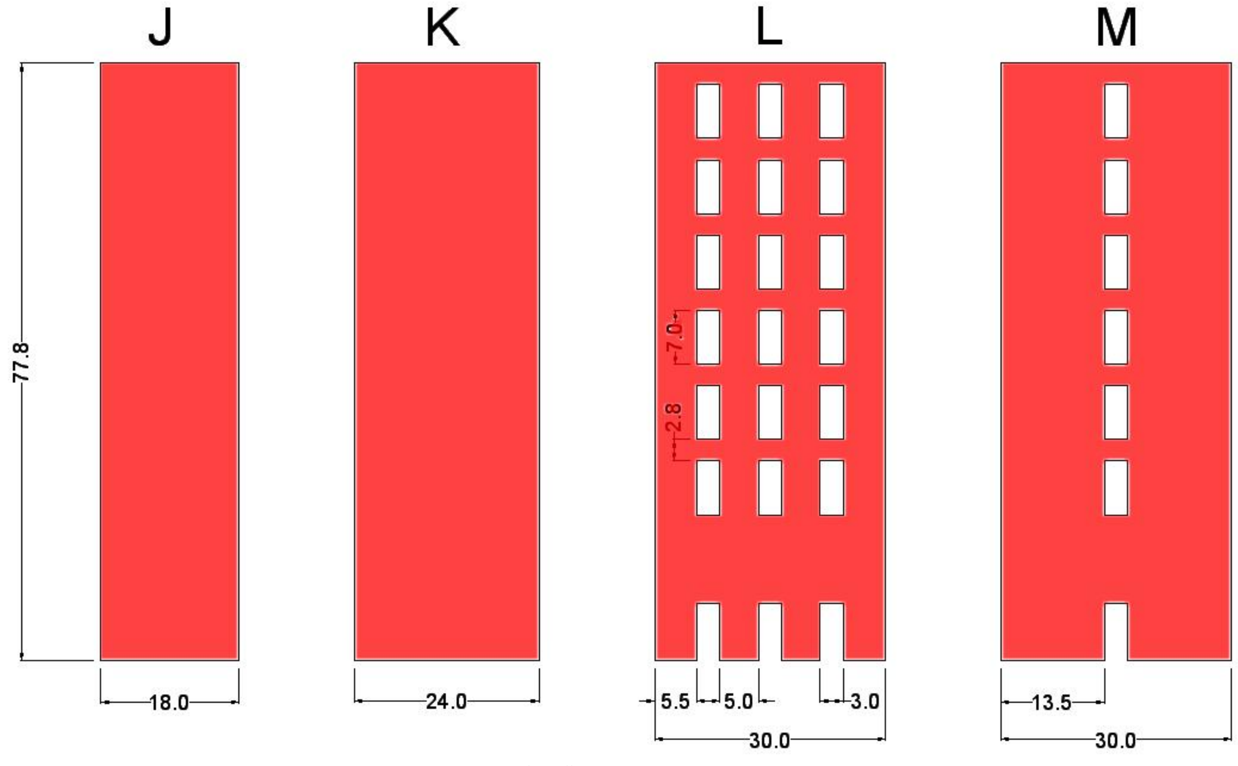


Figure 48: Types of 12" Thick Concrete Shear Walls

P = 1 kip			**Left Wing**		**Right Wing**		**Relitive Stiffness**				**Direct Force (base story)**			
							Left Wing		**Right Wing**		**Left Wing**		**Right Wing**	
Brace/Shear Wall	Δ	Stiffness	# N-S	# E-W	# N-S	# E-W	N-S	E-W	N-S	E-W	N-S	E-W	N-S	E-W
A	0.0892	11.21076	0	3	1	0		0.032	0.034			37.643	36.739	
B	0.0637	15.69859	0	0	1	0			0.047				51.446	
C	0.0527	18.97533	1	0	0	0	0.048				68.931			
D	0.026	38.46154	3	5	3	2	0.098	0.108	0.116	0.050	139.72	129.14	126.04	64.719
E	0.0164	60.97561	0	0	1	0			0.183				199.83	
F	0.0155	64.51613	2	2	2	1	0.164	0.182	0.194		234.37	216.63	211.43	
G	0.7079	1.412629	0	0	0	4				0.002				2.377
H	0.1218	8.210181	0	0	2	0			0.025				26.906	
I	0.0406	24.63054	0	0	0	2				0.032				41.446
J	0.0154	64.93506	2	0	0	0	0.165				235.89			
K	0.0066	151.5152	0	0	0	2				0.195				254.95
L (Coupling SW1)	0.0067	149.2537	0	0	0	1				0.192				251.15
M (Coupling SW2)	0.0055	181.8182	0	0	0	1				0.234				305.94
Total Stiffness =			393.3	355.0	332.3	775.5					Base Shear			
Did Not Use											1428.6	1191.9	1089	1305

Table 13: Wall Relative Stiffness per Direction and Direct Force

Table 13 shows a lot of good information. It has the stiffness of all the lateral force elements, the total amount of stiffness in each direction of each wing, relative stiffness of the walls in each wing, and the direct force in each wall due to the base shear in that direction.

$$V_{di} = \frac{R_i}{\sum R_i} V_{Base}$$

Walls G, H, and I were short concrete shear walls around the elevator, they were too small to be effective, so the wall as a whole was made to be a shear wall with holes punched in it and it acts very rigid. Having concrete shear walls throws off the balance of the rigidity in different directions. The **left wing** has very few concrete shear walls and is very balanced in the N/S vs. E/W directions. The **right wing's** E/W direction has 4 shear walls and 3 braced frames making this direction twice as stiff as the N/S direction. This is ok as long as the rigidity is still evenly distributed, which it is. Also this will lead to less possible building deflection in the direction towards the left wing, and thus allowing a smaller separation gap. Overall 3 of the 4 directions have very similar total stiffness, which leads to frames having similar loads in those 3 directions and allowing for 1 design of each type of braced frame without sacrificing efficiency. Finding the direct shear in each lateral force resisting element is the first step to finding the total design force.

$$V_{Total\ Shear} = V_{di} \pm V_{ti}$$

Center of Mass and Center of Rigidity

Finding the torsional shear in walls comes back to the idea of location. Using the location and rigidity of each lateral force resisting element the Center of Rigidity can be found and compared to the Center of Mass in order to find the eccentricity and resulting torsion.

$$X_R = \frac{\sum(R_i X_i)}{\sum R_i} \qquad Y_R = \frac{\sum(R_i Y_i)}{\sum R_i} \qquad \textit{Center of Rigidity (CR)}$$

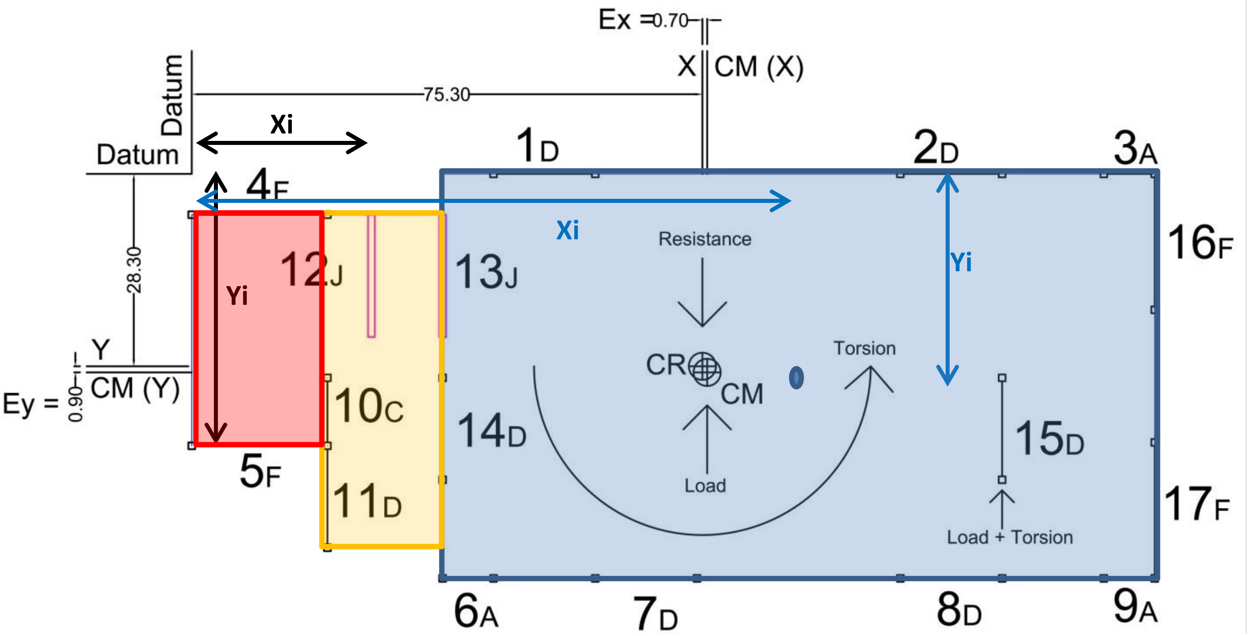


Figure 49: Determination of Center of Mass and Center of Rigidity in the Left Wing

$$X_M = \frac{\sum(A_i X_i)}{\sum A_i} \qquad Y_M = \frac{\sum(A_i Y_i)}{\sum A_i} \qquad \textit{Center of Mass (CM)}$$

Figure 49 shows a sample of how the Center of Mass and Center of Rigidity was found for the left wing. *Figure 50* shows the same for the right wing, and *Table 14* shows the excel spreadsheet that was used to calculate both of which. The calculated CR is for the Roof diaphragm because the point load used to calculate the frame stiffness was at the roof level. The CR for the left wing in ETABS was within 1 foot of the hand calculation; the small difference was probably due to slight differences in the approximated frames and the final design with larger members. In the right wings differences are slightly greater. As you move down the building the CR shifts slightly because the stiffness of frames changes differently as you move down them. The CM stays the same, this is usually the case.

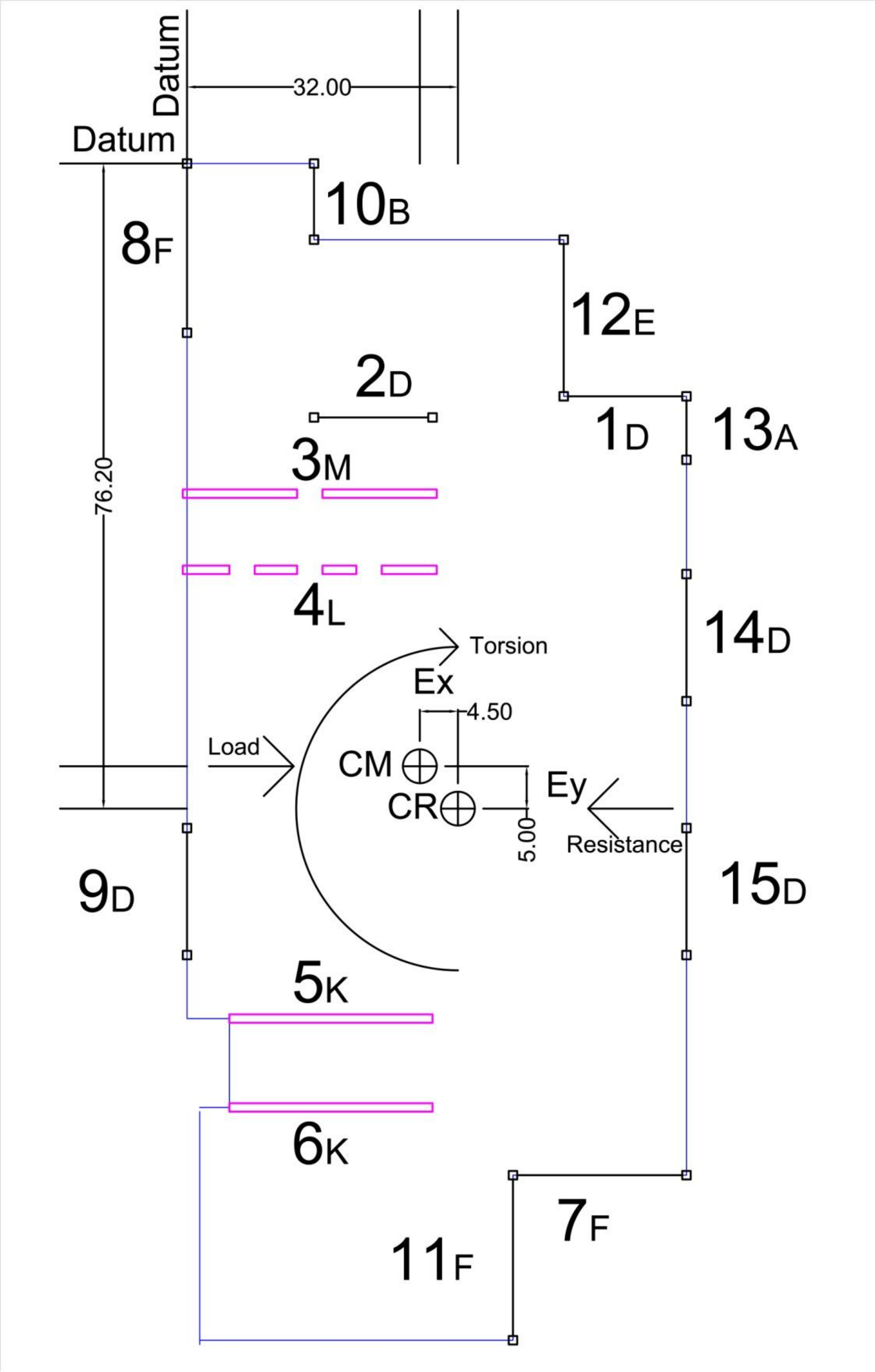


Figure 50: Determination of Center of Mass and Center of Rigidity in the Right Wing

Center of Rigidity									
Wall Type		**Left Wing**				**Right Wing**			
Name	**Ri**	**Yi (ft)**	**Ri*Yi**	**Xi (ft)**	**Ri*Xi**	**Yi (ft)**	**Ri*Yi**	**Xi (ft)**	**Ri*Xi**
1-D	38.5	0.0	0.0						
2-D	38.5	0.0	0.0						
3-A	11.2	0.0	0.0						
4-F	64.5	6.0	387.0						
5-F	64.5	40.0	2580.0						
6-A	11.2	59.5	666.4						
7-D	38.5	59.5	2290.8						
8-D	38.5	59.5	2290.8						
9-A	11.2	59.5	666.4						
10-C	19.0			20.0	380.0				
11-D	38.5			20.0	770.0				
12-J	64.9			26.5	1719.9				
13-J	64.9			37.0	2401.3				
14-D	38.5			37.0	1424.5				
15-D	38.5			119.5	4600.8				
16-F	64.5			142.0	9159.0				
17-F	64.5			142.0	9159.0				
1-D	38.5					27.5	1058.8		
2-D	38.5					30.0	5454.0		
3-M	181.8					39.0	5822.7		
4-L	149.3					48.0	7272.0		
5-K	151.5					101.0	15301.5		
6-K	151.5					111.5	16892.3		
7-F	65.5					119.5	7827.3		
8-F	64.5							0.0	0.0
9-D	38.5							0.0	0.0
10-B	15.7							15.0	235.5
11-F	65.5							38.5	2521.8
12-E	61.0							44.5	2714.5
13-A	11.2							59.0	660.8
14-D	38.5							59.0	2271.5
15-D	38.5							59.0	2271.5
		∑Ri =	**316.5**	**∑Ri =**	**393.3**	**∑Ri =**	**776.6**	**∑Ri =**	**333.4**
		∑RiXi =	**8881.3**	**∑RiYi =**	**29614.4**	**∑RiXi =**	**59628.5**	**∑RiYi =**	**10675.6**
Hand		**ȳ =**	**28.1**	**ẍ =**	**75.3**	**ȳ =**	**76.8**	**ẍ =**	**32.0**
ETABS		**ȳ =**	**29.1**	**ẍ =**	**76.5**	**ȳ =**	**70.8**	**ẍ =**	**37.1**
Center of Mass		**ȳ =**	**29.2**	**ẍ =**	**76**	**ȳ =**	**71.2**	**ẍ =**	**27.50**
Ecentricity **(+) Moment**	**(-) Moment**	**Ey =**	**-1.1**	**Ex =**	**-0.7**	**Ey =**	**5.6**	**Ex =**	**4.5**
Length Perpindicular to Load		Ly =	59.5	Lx =	142	Ly =	139	Lx =	59
% Eccentricity			1.9		0.5		4.0		7.7
Accidental Ecentricity		Ey_{acc} =	3.0	Ex_{acc} =	7.1	Ey_{acc} =	7.0	Ex_{acc} =	3.0

Table 14: Determination of Center of Mass and Center of Rigidity

Torsion

Now that the CM and CR are known for each wing it is possible to determine the building torsion for each wing and compare it to the existing structure. As previously stated this is due to a difference in CM and CR and is additive in some elements and subtractive in others. For the most part the in both wings the torsional moment is additive due to the fact that the accidental eccentricity is large enough to overcome eccentricities that would cause a negative effect. This is a good thing; it means that the eccentricity in each wing and the resulting torsion is low, especially in the left wing. In fact ETABS says that at the top diaphragm there is almost no eccentricity. The equations below are used to find the total building torsion in *Table 15* below. When the building torsion in each wing is compared to that of the existing structure it can be seen that the division of the building into wings and well thought out placement of lateral elements was a success.

$$T = V * (e \pm e_{acc}) \qquad e = CM - CR \qquad e_{acc} = .05 * Building\ Width$$

The eccentricities in the proposed wings were small enough to keep total building torsion nearly as small as the existing structure even though the forces on the new structure are 3 times as large.

Forces in Lateral Force Resisting Elements

Next the torsion force is distributed to individual frames based on their rigidity and location relative to the Center of Rigidity, this value is known as d_i.

$$V_{ti} = T * \left(\frac{d_i R_i}{J}\right) \qquad J = \sum(d_i R_i^2)$$

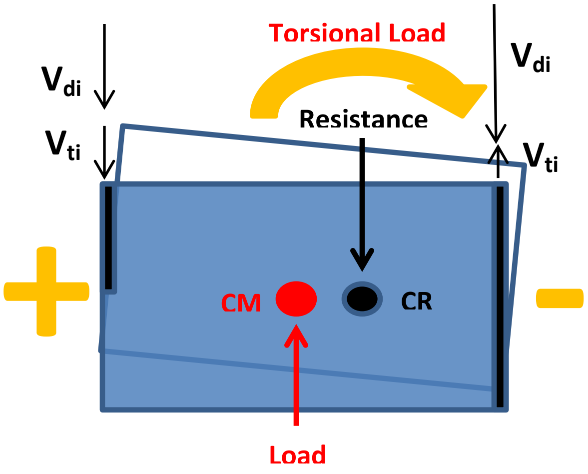


Figure 51: Addition of Forces

Sometimes the V_{ti} acts in the same direction as V_{di} in which case the force is additive, and sometimes they act in opposite ways. *Figure 51* demonstrates these cases. The arrows are not draw to scale, but are relative. The direct forces are a lot large than the torsional forces, and torsional forces are stronger as you move away from the CR. This is why most controlling frames are far away from the CR, *Figure 52*. The values in *Table 16* use the principles from above to find the total force in lateral elements and determine the controlling load case for each type of frame in order to be designed to resist them.

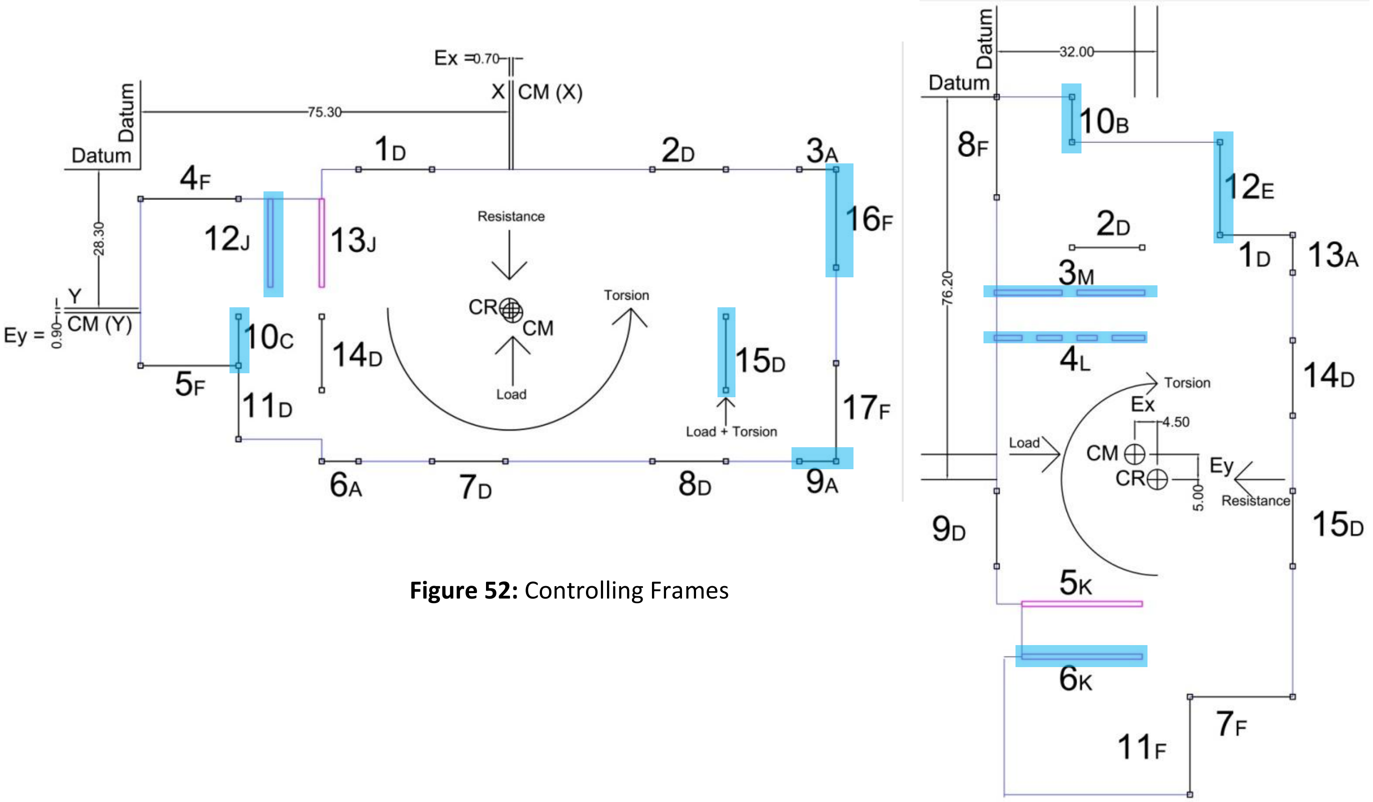


Figure 52: Controlling Frames

Shear in Lateral Force Elements										
		Wall Name	**Ri**	**di (ft)**	**Ri*di**	**Ri*di^2**	**E_{total} (ft)**	**V_{ti} (kips)***	**V_{di} (kips)***	**V_{Total} (kips)***
Left Wing	East-West	1-D	38.5	28.3	1089.6	30834.3	6.2	6.0	125.0	131.0
		2-D	38.5	28.3	1089.6	30834.3	6.2	6.0	125.0	131.0
		3-A	11.2	28.3	317.0	8970.0	6.2	1.7	38.0	39.7
		4-F	64.5	22.3	1438.4	32075.2	6.2	7.9	218.5	226.5
		5-F	64.5	11.7	754.7	8829.4	8.0	5.4	218.5	223.9
		6-A	11.2	31.2	349.4	10902.5	8.0	2.5	38.0	40.5
		7-D	38.5	31.2	1201.2	37477.4	8.0	8.6	130.3	138.8
		8-D	38.5	31.2	1201.2	37477.4	8.0	8.6	130.3	138.8
		9-A	11.2	31.2	349.4	10902.5	8.0	2.5	38.0	40.5
	North-South	10-C**	19.0	55.3	1050.7	58103.7	2.3	2.6	68.9	71.5
		11-D	38.5	55.3	2129.1	117736.5	2.3	5.2	139.7	144.9
		12-J	64.9	48.8	3167.1	154555.5	2.3	7.8	235.9	243.7
		13-J	64.9	38.3	2485.7	95201.2	2.3	6.1	235.9	242.0
		14-D	38.5	38.3	1474.6	56475.3	2.3	3.6	139.7	143.3
		15-D	38.5	44.2	1701.7	75215.1	3.7	6.7	139.7	146.4
		16-F**	64.5	66.7	4302.2	286953.4	3.7	17.0	234.4	251.3
		17-F	64.5	66.7	4302.2	286953.4	3.7	17.0	234.4	251.3
Right Wing	East-West	1-D	38.5	48.7	1875.0	91310.1	12.0	25.9	65.5	91.4
		2-D	38.5	46.2	1778.7	82175.9	12.0	24.6	65.5	90.1
		3-M	181.8	37.2	6763.0	251582.1	12.0	93.5	309.8	403.2
		4-L	149.3	28.2	4210.3	118729.3	12.0	58.2	254.3	312.5
		5-K	151.5	24.8	3757.2	93178.6	2.0	8.7	258.1	266.8
		6-K	151.5	35.3	5348.0	188782.6	2.0	12.3	258.1	270.5
		7-F	64.5	43.3	2792.9	120930.4	2.0	6.4	93.6	100.0
	North-South	8-F	64.5	32.0	2064.0	66048.0	7.5	14.9	218.8	233.6
		9-D	38.6	32.0	1235.2	39526.4	7.5	8.9	125.1	134.0
		10-B	15.7	17.0	266.9	4537.3	7.5	1.9	53.2	55.2
		11-F	64.5	6.5	419.3	2725.1	1.5	0.6	186.3	185.7
		12-E	61.0	12.5	762.5	9531.3	1.5	1.1	206.8	205.7
		13-A	11.2	27.0	302.4	8164.8	1.5	0.4	38.0	37.6
		14-D	38.5	27.0	1039.5	28066.5	1.5	1.5	130.4	128.9
		15-D	38.5	27.0	1039.5	28066.5	1.5	1.5	130.4	128.9
*V is the base shear		Left Wing	J = ∑Ri*di^2 =	**1339497.1**	**V_{NS} =**	**1428.6**	**V_{EW} =**	**1191.9**	Frame Design Load	
**MAE frame design		Right Wing	J = ∑Ri*di^2 =	**1133354.9**	**V_{NS} =**	**1089.2**	**V_{EW} =**	**1305.5**	**(+)**	**(-)**

Table 16: Total Force in Each Frame

Loading Due to Out of Plane Loading

With the 100% Y-direction + 30% X-direction loading there will be out of plane loading in lateral force resisting elements. The loading in the out of plane walls will be due to torsion, as shown in Figure 53. As seen in Table 16 most torsional loads are relatively small compared to direct forces, and the out of plane force has a .3 multiplier.

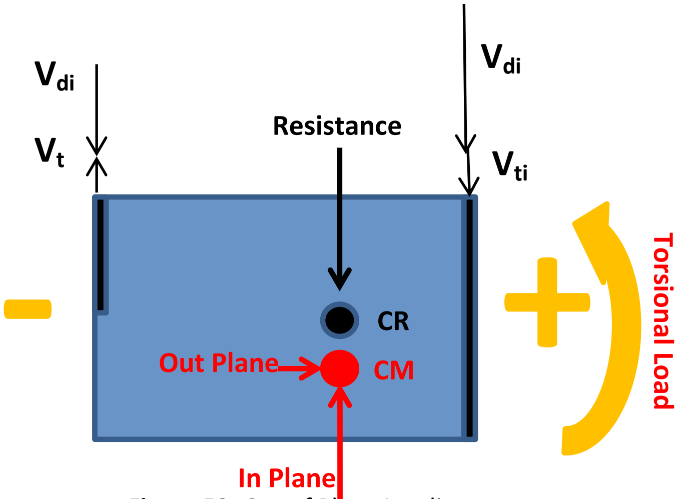


Figure 53: Out of Plane Loading

Design Special Concentric Braced Frames (MAE Coursework)

The load path has now led to the design of the lateral force resisting elements. Knowledge gained from AE 538 is used to design a Special Concentric Braced Frame. There are two types of concentric braced frames utilized in the Hyatt Place structural redesign. For bay sizes less than 15' X-braces are used, and for bays of 15' to 20' inverted-V braces are used. *Figure 47* shows all of the braces designed. The reason not all of one type or the other is used is due to geometry. The angle the brace is at effects the how it takes load and 45 degrees is the ideal angle to take load. Above or below 45 degrees and either the X or Y component is greater. This is realized when designing the bottom bay in each brace. The ground floor has a height of 19' as compared to 9.8' on all of the floors above that, thus making the bottom braces at a much more acute angle. With the brace being that steeply inclined, the horizontal shear force (Vx) is more than doubled when it is translated to an axial load in the brace. Another thing that can be drawn from *Figure 47* is similar angles between some of the braces. Because all of the bay sizes of the X-braces are half the width of an invert-V brace, Frame A & D have braces at 37 degrees, Frame B & E have braces at 43 and 44 degrees, and Frame C and F have braces at 46 degrees. This will translate horizontal forces to vertical force in a similar fashion in these corresponding frames, but the X-braces are braced in the middle and thus have a shorter un-braced length and will buckle less easily, leading to the possibility of using smaller size braces. Overall the braces all have relatively ideal geometries for steel braced frames. The X-braces will prove to be more efficient at carrying load and easier to be designed due to the fact that the inverted-V braces meet at the center of the beam and the X-braces meet at the column intersection. In the inverted-V frames the beam has to carry a very large amount of load making it a much larger member than its corresponding X-braced frame. For this reason it is the inverted-V braced frame that will be discussed thoroughly in this section, specifically Frame 16-F from the left wing, *Figure 54*.

Seismic Story Shear Loads on Braced Frame 16-F (Left Wing N-S)

Level	Story Weight (K)	Height (ft)	K	$w_x h_x^k$	Vertical Distribution Factor C_{vx}	Forces (K) Fx	Story Shear (K) Vx	Moments (ft-K) Mx
Main Roof	1083.51	78	1.035	98435.29	0.220727	59.53006	59.53006	4643.344
7th Floor	1151.84	68.167	1.035	91020.96	0.204101	55.04614	114.5762	7810.316
6th Floor	1151.84	58.33	1.035	77462.29	0.173698	46.84636	161.4226	9415.778
5th Floor	1151.84	48.5	1.035	63993.35	0.143496	38.70083	200.1234	9705.984
4th Floor	1151.84	38.667	1.035	50616.2	0.1135	30.61082	230.7342	8921.8
3rd Floor	1158.4	28.83	1.035	37566.24	0.084237	22.71869	253.4529	7307.047
2nd Floor	1275.5	19	1.035	26865.22	0.060241	16.2471	269.7	5124.3
Total	**8124.77**			**445959.6**				**52928.57**

Table 17: Seismic Forces on Frame 16-F

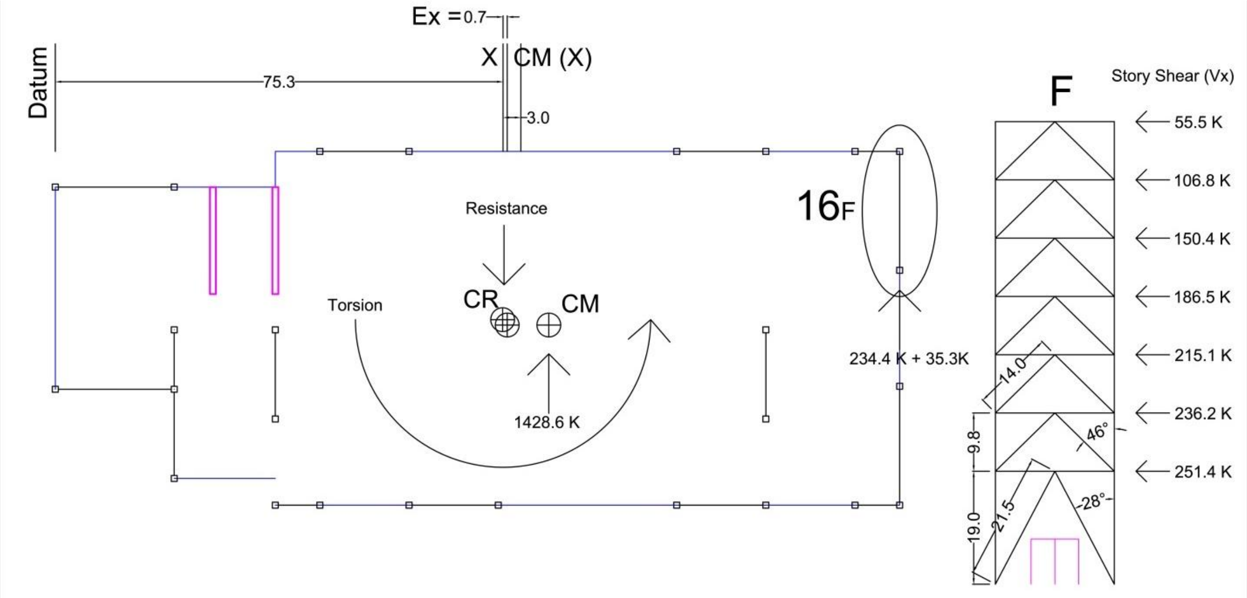


Figure 54: Seismic Forces on Frame 16-F and Location in Left Wing

Table 17 shows how the Vx at ground level was translated to forces at all the other diaphragms in the same fashion as seismic base shear of the building being assigned to different diaphragms in a building. Frame 16-F has a larger force than other F braces in the left wing because it is farthest from the center of rigidity (largest di) and the accidental eccentricity is larger in the X-direction than the Y-direction. The story shears will be used to design the brace, beam, and column at each level because all the load has to get to the foundation so it adds up as you go down.

Special Concentric Braced Frame Behavior

The main idea of the “Special” Concentric Braced Frame is to have the brace elements yield and dissipate energy but have the beams and columns remain elastic so that the structure stays stable. The bracing element is designed to plastically dissipate energy during the cyclic loading of an earthquake. “Special” frames are more ductile than “ordinary” ones, thus the higher R-value of 6. They also have a higher C_d-value than “ordinary” frames because of their ductility and ability to continue to take load after many cycles of loading and increased deformation. The tension brace is intended to yield and compression brace to buckle, having a tension and compression brace allows the frame to dissipate energy in each direction without have to displace as far as a single brace. The best brace at dissipating energy is neither too slender or short and stocky. There are limitations on slenderness and width-to-thickness ratios in order to assure that the compression brace is able to continue cycling from loaded to unloaded.

$$\frac{KL}{r} \leq \frac{1000}{\sqrt{F_y}} \qquad Slenderness \qquad\qquad \frac{b}{t} \leq \frac{110}{\sqrt{F_y}} \qquad \text{width-to-thickness}$$

Design Process

Brace - The brace is designed first. It takes the horizontal load and transforms it into axial load based on geometry. It is assumed that each brace takes half of the load even though the tension brace is more efficient at carrying load and will be able to carry load longer. There is also some gravity load transferred into the braces. A brace is picked based on compression strength, tension strength, slenderness, or buckling limitations. Compression strength is always going to control over tension, long members may not pass slenderness requirements and thin walled members might not pass buckling requirements. But even over the long 21.5 foot span of ground level braces slenderness or buckling still doesn't control. In Frame F it was a close call between buckling and compression strength. Rectangular HSS is more susceptible to buckling issues than square HSS, so square HSS were used. The brace in the frame also is responsible for the majority of the deflection, so deflection was also checked as a limit state.

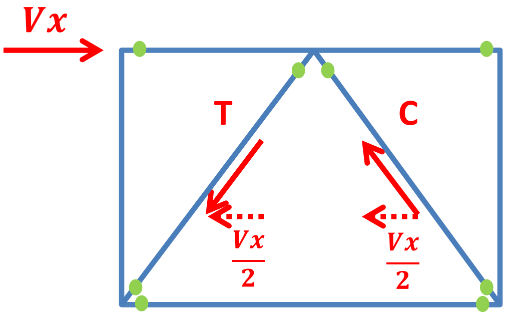


Figure 55: Seismic Forces on Braces

Beam - The beam is designed strong enough to remain elastic. Because the member is designed to remain elastic there is an Ry multiplier ($F_{ye} = R_y * F_y$). For A992 steel Ry = 1.1. The Ry is to account for the difference in expected yield stress and minimum yield stress. The beam is to be designed as if the braces are not there to help aid in supporting gravity loads and then there is an additional load due to an unbalance in tension and compression strength of the braces. Because the compression brace is going to yield first but still have ability to carry some load, there is considered to be 100% tensile capacity vertical load minus 30% of the compression capacity vertical load, *Figure 56*. The beam also takes axial load from the braces, not moment, because the connection is moment released. Both the tension and compression brace load the beam axially in the same direction, but since it is loaded in the middle the load is split in two and taken by each half of the beam. The beam is then checked to make sure it can adequately take the combined axial and bending load.

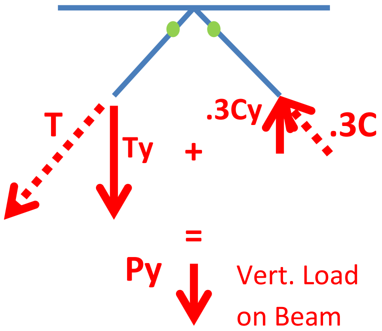


Figure 56: Seismic Forces on Beam

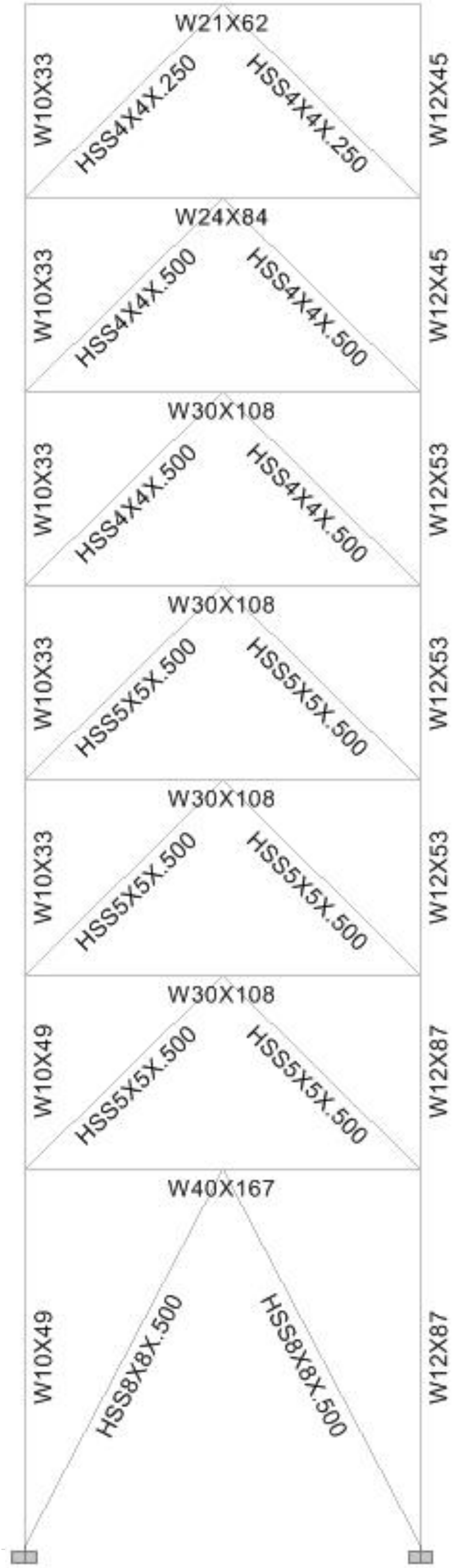


Figure 57: Designed Members16-F Frame

Columns

The shear in the beam and axial in the brace get transferred into the columns on path to the foundation. Columns are sized to take half of the vertical seismic load on the beam and all of the gravity loads on them. Some of the frames run parallel to the slab span and don't carry much gravity load. These frames will be susceptible to uplift forces on the foundations. Reactions at the base of frames will be checked in ETABS. *Figure 56* shows the members determined by hand for Frame F, full calculations can be found in appendix F. It can be seen that the right column is larger than the left. This is because Frame A also frames into this column, so it was designed considering to carry the load that Frame A would also put into the column. There are 4 other types of frame intersections as shown in *Figure 58* shows these locations. Where there are frame intersections the column strong axis was oriented in the axis that would be most beneficial given the amount of other shear walls nearby.

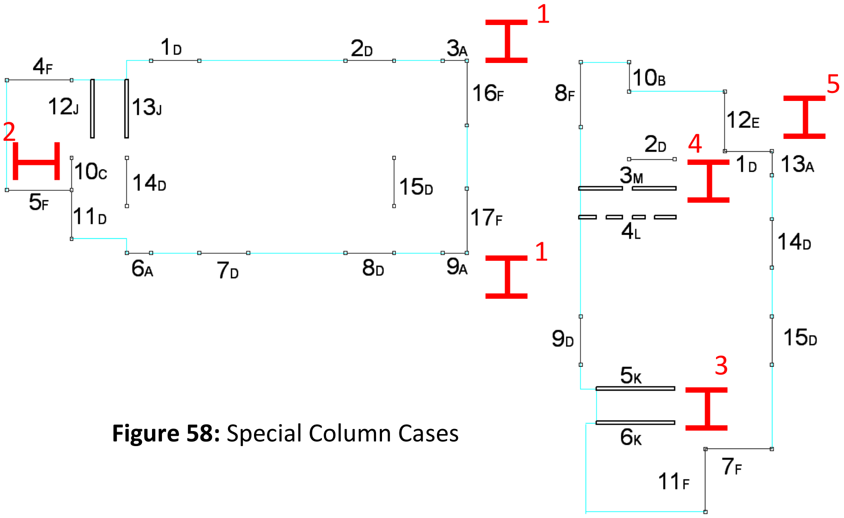


Figure 58: Special Column Cases

Table 18 and *19* summarize all of the braces designed to resist the Hyatt Place's lateral loads in San Diego, CA.

			Designed Members		
	Frame	**Level**	**Brace Chosen**	**Beam Chosen (table 3-10)**	**Column Chosen (table 4-1)**
Inverted V - Brace	**F**	**Roof**	HSS 4x4x.25	W21x62	
		7	HSS 4x4x.5	W24x84	W10x33
		6	HSS 5x5x.5	W30x108	
		5	HSS 5x5x.5	W30x108	W10x33
		4	HSS 5x5x.5	W30x108	
		3	HSS 5x5x.5	W30x108	
		2	HSS 8x8x.5	W40x167	W10x49
	E	**Roof**	HSS 4x4x.25	W21x62	
		7	HSS 4x4x.5	W24x84	W10x33
		6	HSS 4x4x.5	W24x84	
		5	HSS 5x5x.5	W30x108	W10x33
		4	HSS 5x5x.5	W30x108	
		3	HSS 5x5x.5	W30x108	
		2	HSS 7x7x.625	W36x135	W10x39
	D	**Roof**	HSS 4x4x.25	W21x62	
		7	HSS 4x4x.5	W27x84	W10x33
		6	HSS 4x4x.5	W27x84	
		5	HSS 5x5x.5	W30x108	W10x49
		4	HSS 5x5x.5	W30x108	
		3	HSS 5x5x.5	W30x108	
		2	HSS 7x7x.625	W36x135	W10x68

Table 18: Designed Inverted-V Braces

			Designed Members		
	Frame	**Level**	**Brace Chosen**	**Beam Chosen (table 3-10)**	**Column Chosen (table 4-1)**
X - Brace	**C**	**Roof**	HSS 2x2x.25	W10x33	
		7	HSS 2x2x.25	W10x33	W10x33
		6	HSS 3x3x.25	W10x33	
		5	HSS 3x3x.25	W10x33	W10x33
		4	HSS 3x3x.25	W10x33	
		3	HSS 3x3x.25	W10x33	
		2	HSS 4x4x.3125	W10x33	W10x33
	B	**Roof**	HSS 2x2x.25	W10x33	
		7	HSS 2x2x.25	W10x33	W10x33
		6	HSS 3x3x.1875	W10x33	
		5	HSS 3x3x.1875	W10x33	W10x33
		4	HSS 3x3x.1875	W10x33	
		3	HSS 3x3x.1875	W10x33	
		2	HSS 4x4x.3125	W10x33	W10x39
	A	**Roof**	HSS 2x2x.25	W10x33	
		7	HSS 2x2x.25	W10x33	W10x33
		6	HSS 3x3x.1875	W10x33	
		5	HSS 3x3x.1875	W10x33	W10x39
		4	HSS 3x3x.1875	W10x33	
		3	HSS 3x3x.1875	W10x33	
		2	HSS 4x4x.3125	W10x33	W10x60

Table 19: Designed X-Braces

ETABS

RAM was utilized to aid in design of the gravity system; ETABS is used to test how the designed braced frames and 12" shear walls react under lateral loads. The left and right wing were modeled in separate models with Special Concentric Braced Frames and Special Reinforced Concrete Shear Walls that were distributed loads from the rigid diaphragm. There were 4 earthquake load cases in each, all applied to the center of mass with a 5% accidental eccentricity. The moment was released in all beams and braces of braced frames, and the base of the model was fixed.

1. 100% North/South (Y)
2. 100% East/West (X)
3. **100% North/South (Y) + 30% East/West (X)**
4. **100% East/West (X) + 30% North/South (Y)**

Likely to control because there is more load, but it is subtractive in some cases.

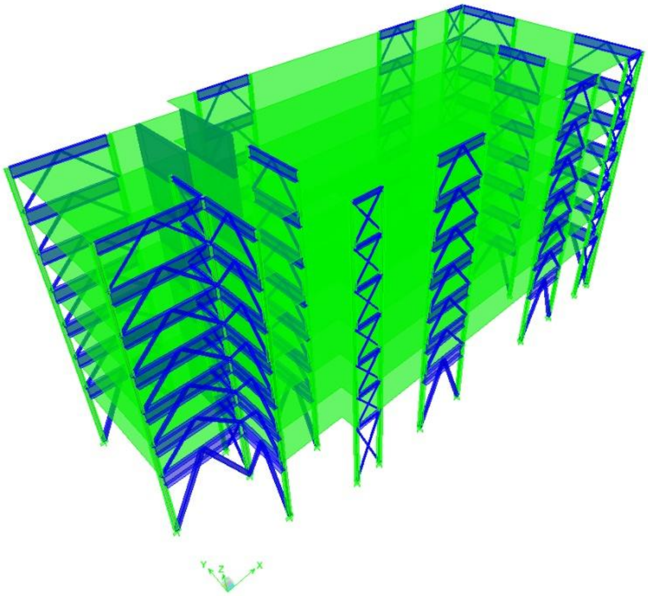


Figure 59: Left Wing ETABS Model

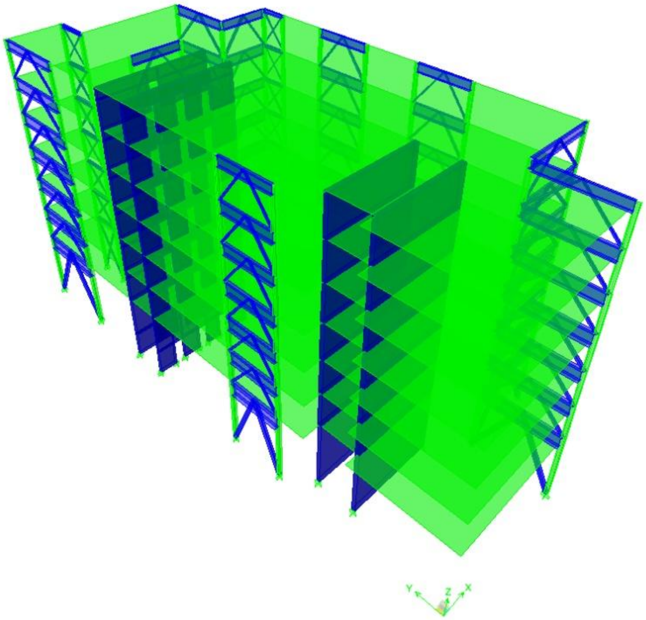


Figure 60: Right Wing ETABS Model

Results

An ETABS model was created in order to see how all of the lateral force resisting elements act when tied together by a rigid diaphragm. One measure of how the structural elements work together as a whole is the building mode shapes. By looking at the mode shapes and their periods you can tell in which directions the building is stronger and weaker and overall if its stiffness is near the expected for the type of structure and height. If this period is shorter than the CuTa, it must be used for the seismic load calculation. A top view of each building's first mode shape is shown to the right. The direction of the wings first mode don't line up, therefore good to have independent motion.

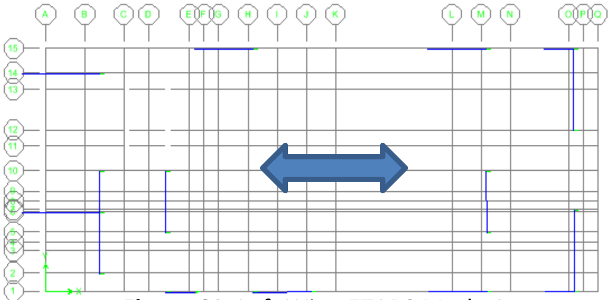


Figure 61: Left Wing ETABS Mode 1

Mode Shapes (by mode)				
Mode	**LW**	**Direction**	**RW**	**Direction**
1	1.0767	X -(E/W)	1.0303	Y - (N/S)
2	0.8952	Y - (N/S)	0.5726	Z Axis
3	0.6423	Z - Axis	0.5217	X - (E/W)

Table 21: Mode Shapes

Mode Shapes (by direction)				
Direction	**LW**	**Mode**	**RW**	**Mode**
Y - (N/S)	0.8952	2	1.0303	1
X - (E/W)	1.0767	1	0.5217	3
Z Axis	0.6423	3	0.5726	2

Table 22: Mode Shapes

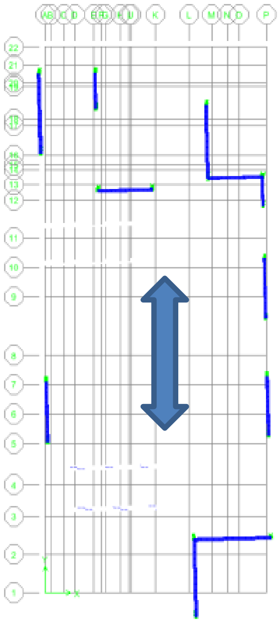


Figure 62: Right Wing ETABS Mode 1

Another important thing to look at is the displacement of the top diaphragm. A well laid out, uniformly rigid structure will have displacements that are fairly similar at oposite ends of the structure. *Figure 63* shows the locations that displacements were taken from for comparison. If the displacements differ too much then the building is considered torsionally irregular and an amplification factor of Ax times the torsional moment.

$$\text{if } \Delta_R \geq 1.2\frac{\Delta_L+\Delta_R}{2} \therefore$$

$$Torsionally\ Irregular$$

$$A_x = \left(\frac{\Delta_{max}}{1.2\Delta_{avg}}\right)^2$$

The left wing is slightly irregular in the N/S direction. This is determined to be due to the difference of rigidity at the top vs bottom of the braced frames when compared to the concrete shear walls, Figure 64.

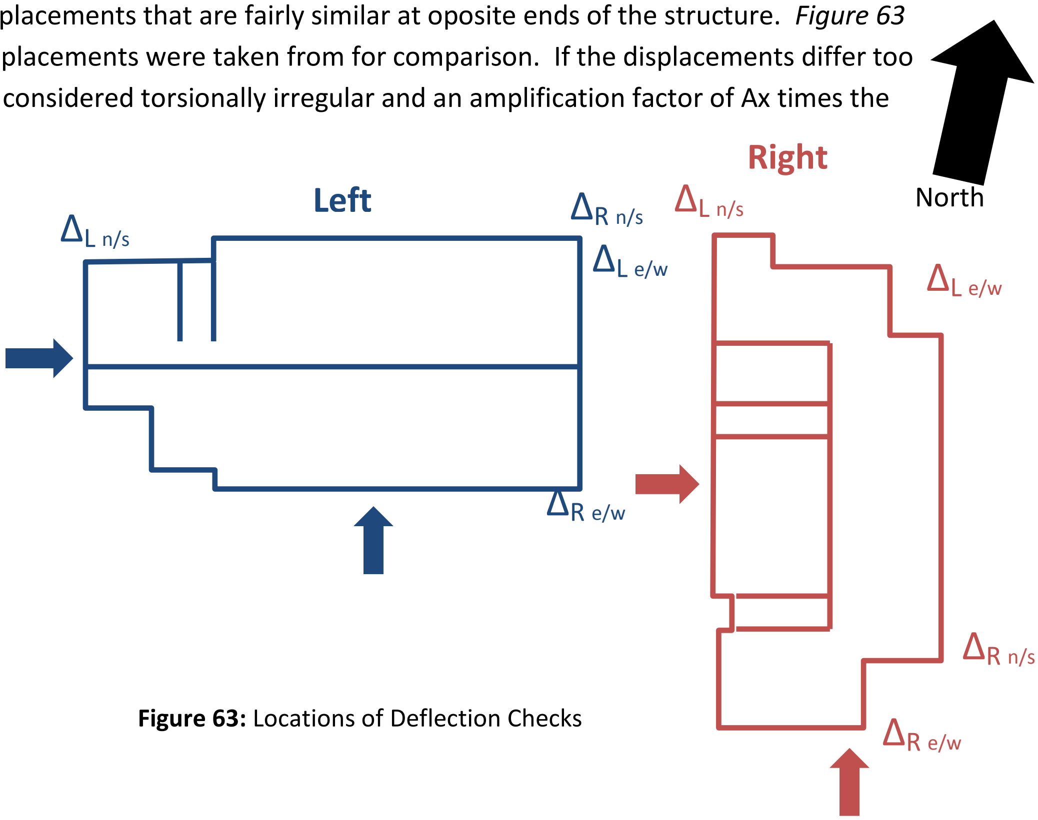


Figure 63: Locations of Deflection Checks

Torsionally Irregular Check						
Wing	Direction	Δ_L (in)	Δ_R (in)	$1.2\Delta_{avg}$ (in)	Torsionally Irregular	Ax
Left	N/S	1.81	3.02	2.90	YES	1.04
	E/W	3.012	2.88	3.54	NO	none
	N/S + .3E/W	1.87	2.98	2.91	YES	1.02
	.3N/S + E/W	2.96	2.98	3.56	NO	none
Right	N/S	2.35	2.06	2.65	NO	none
	E/W	0.84	0.56	0.84	NO	none
	N/S + .3E/W	2.79	2.39	3.11	NO	none
	.3N/S + E/W	1.1	0.78	1.13	NO	none

Table 23: Torsional Irregularity Check

The graph depicts how the center of rigidity is moving toward the concrete shear walls, thus creating more torsion as you go lower in the building. The eccentricity at the top diaphragm is good. Seems like when combining systems it is a good idea to keep them evenly laid out around the CM. The right wing has shear walls, but a set on either side of the CM, and it preforms better.

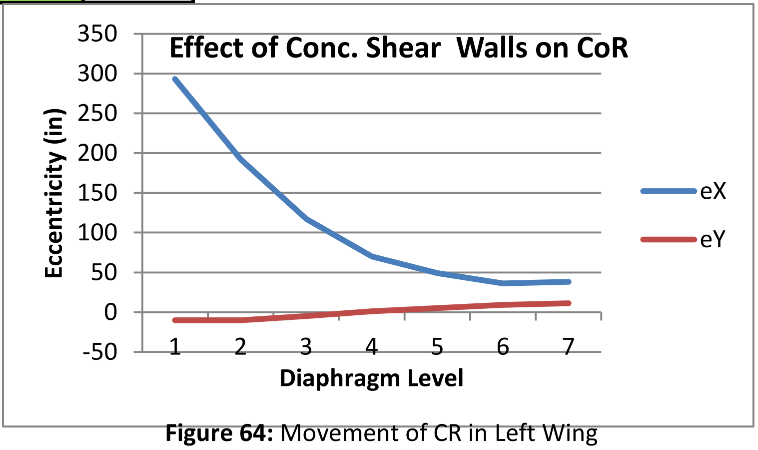


Figure 64: Movement of CR in Left Wing

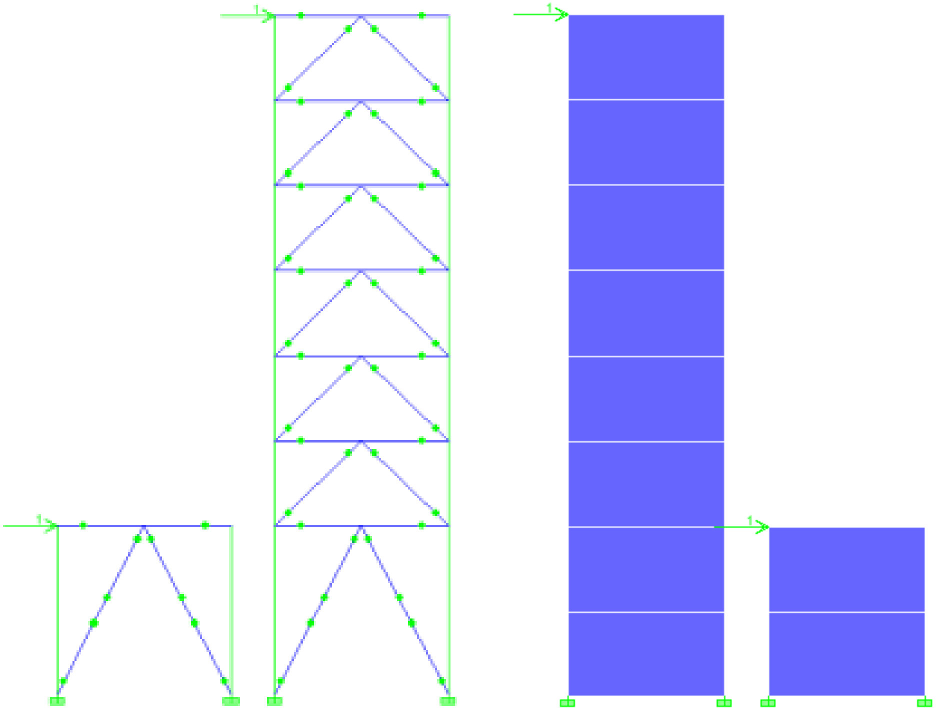


Figure 65: Frame-F and Wall-J with 1 Kip Loads in ETABS

System Rigidity Comparison							
	Steel Braced Frame F			Conc. Shear Wall J			
	Δ	Ri	%Ri	Δ	Ri	%Ri	Sum Ri
Roof	0.001571	636.54	44.29078	0.001249	800.6405	55.70922	1437.18
1st Diaphragm	0.000428	2336.45	4.67706	0.000021	47619.05	95.32294	49955.50

Table 24: Frame-F and Wall-J with 1 Kip Loads in ETABS

Testing a Frame F and Wall J in ETABS confirms the hypothesis that the movement of the CR and the result of torsion is due to how shear walls retains its stiffness at the bottom diaphragm and braced frames do not. Thus if systems are combined it is best to make sure that each system's center of rigidity line up to decrease the effect of lost stiffness at lower levels. It happened to work out this way in the right wing, and it behaves better.

For this building another important building characteristic is how much each wing deflects in the X-direction. The deflection of each wing toward each other determines the necessary size of the separation gap.

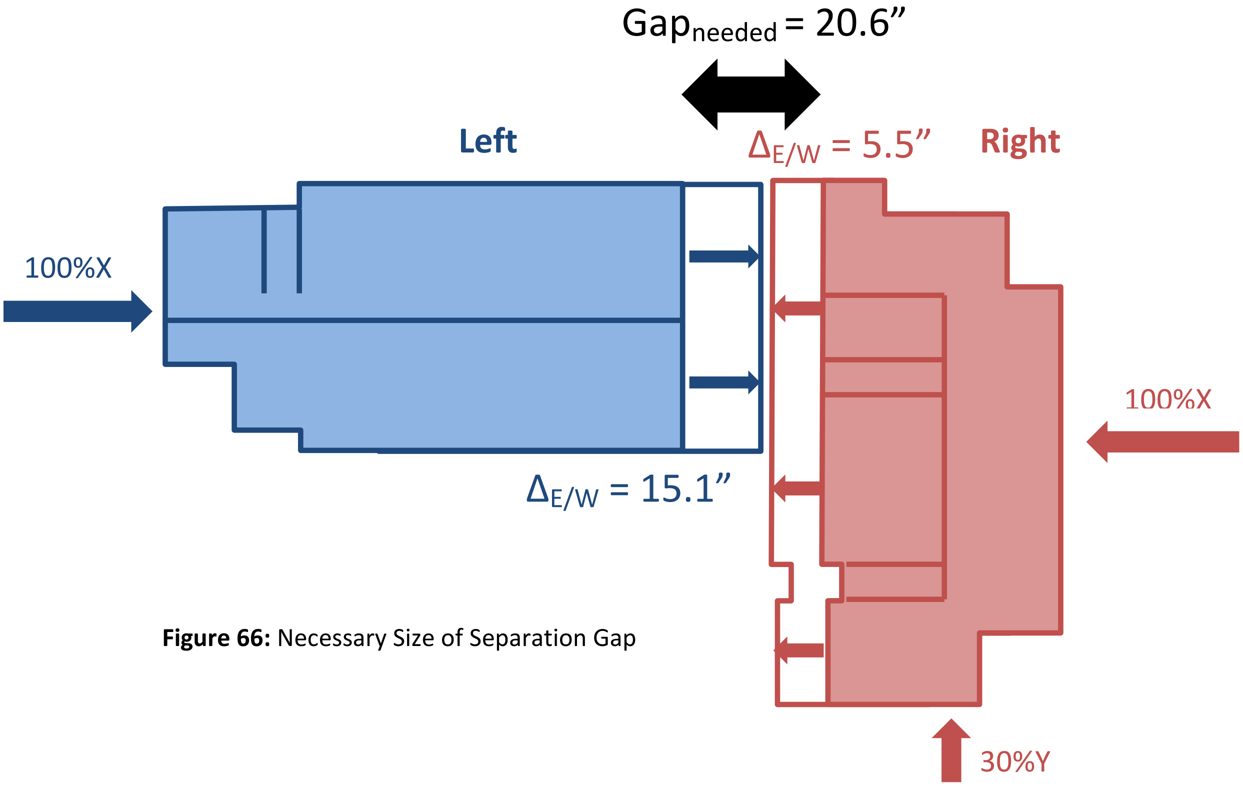


Figure 66: Necessary Size of Separation Gap

Figure 66 shows the maximum modeled deflection of each wing toward the other and the resulting amount of necessary gap. For The left wing the 100%X earthquake combo controlled and for the right wing the 100%X + 30%Y controlled. The deflection found in ETABS then needs to be multiplied times the C_d factor for the lateral force resisting system in that direction. For both SCBF and SRCSW the C_d factor is 5. The reason this is so high is because of the ductility of the system. Both systems are detailed in such a way that they are able to sustain large displacements and still carry loads. This would require a 20.6” gap between the buildings.

Conclusions

From the data already seen it appears that the buildings while behaving well could perform better. For the left wing it would be good to try and eliminate the torsional irregularity and stiffen the building in the East/West direction to lower the required size of the separation gap. The right wing overall preforms well, but some eccentricity in the East/West direction could be eliminated.

Redesign

A couple of changes were made to the amount and location of lateral force resisting elements in order to try and optimize the design. These were locations that were originally thought might not be necessary. They require slightly more coordination with the existing architecture, but can work. In the left wing there was 1 North/South braced frame added on the right side to try and pull the CR closer to the CM and there were multiple braced frames added to the East/West direction to try and reduce the displacement toward the right wing *Figure 67*.

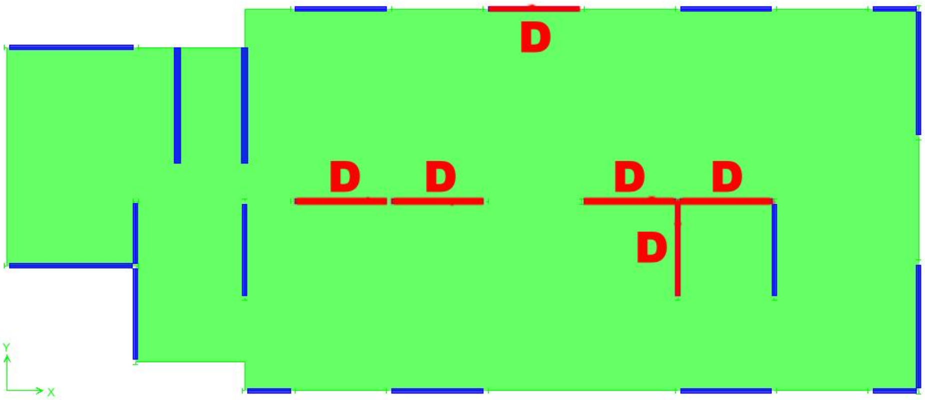


Figure 67: Left Wing Redesign

In the right wing there was one frame added to the left of the CR to try and lower the X-eccentricity, *Figure 68*.

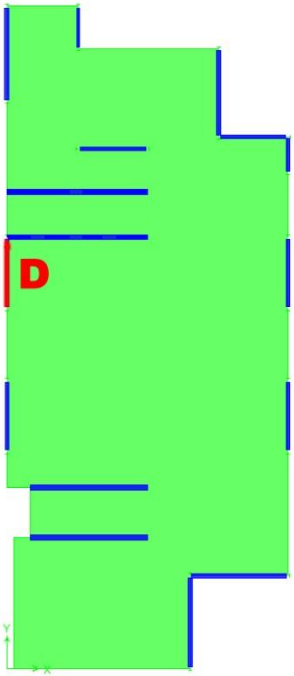


Figure 68: Right Wing Redesign

Mode Shapes Redesign (by direction)					
Direction	LW	Mode	LW (revised)	Mode	Better?
Y - (N/S)	0.8952	2	0.8506	1	Yes
X - (E/W)	1.0767	1	0.7641	2	Yes
Z Axis	0.6423	3	0.6269	3	Yes
	RW	Mode	RW (revised)	Mode	Better?
Y - (N/S)	1.0303	1	0.9375	1	Yes
X - (E/W)	0.5217	3	0.4965	3	Yes
Z Axis	0.5726	2	0.5687	2	Yes

Table 25: Comparison of Mode Shapes

Each of the building results will be check again, starting with mode shapes. The periods are better than previously, which would tend to lead to better overall results and less displacement.

Torsionally Irregular Check (redesign)						
Wing	Direction	Δ_L (in)	Δ_R (in)	$1.2\Delta_{avg}$ (in)	Torsionally Irregular	Ax
Left	N/S	1.74	2.57	2.59	NO	none
	E/W	1.47	1.47	1.76	NO	none
	N/S + .3E/W	1.76	2.6	2.62	NO	none
	.3N/S + E/W	1.45	1.56	1.81	NO	none
Right	N/S	1.89	1.8	2.21	NO	none
	E/W	0.72	0.61	0.80	NO	none
	N/S + .3E/W	2.2	2.08	2.57	NO	none
	.3N/S + E/W	0.86	0.57	0.86	NO	none

Table 26: Redesign Torsional Irregularity

The addition of 1 braced frame on the left side of the left wing helped offset the effects of the concrete shear wall rigidity enough to barely keep the wing from being torsionally irregular. The right wing is slightly more irregular than before, but is still not torsionally irregular and gives less displacement toward the left wing. Overall the addition of more braced frames creates a better preforming building overall, and this wing combination will be checked against code allowances.

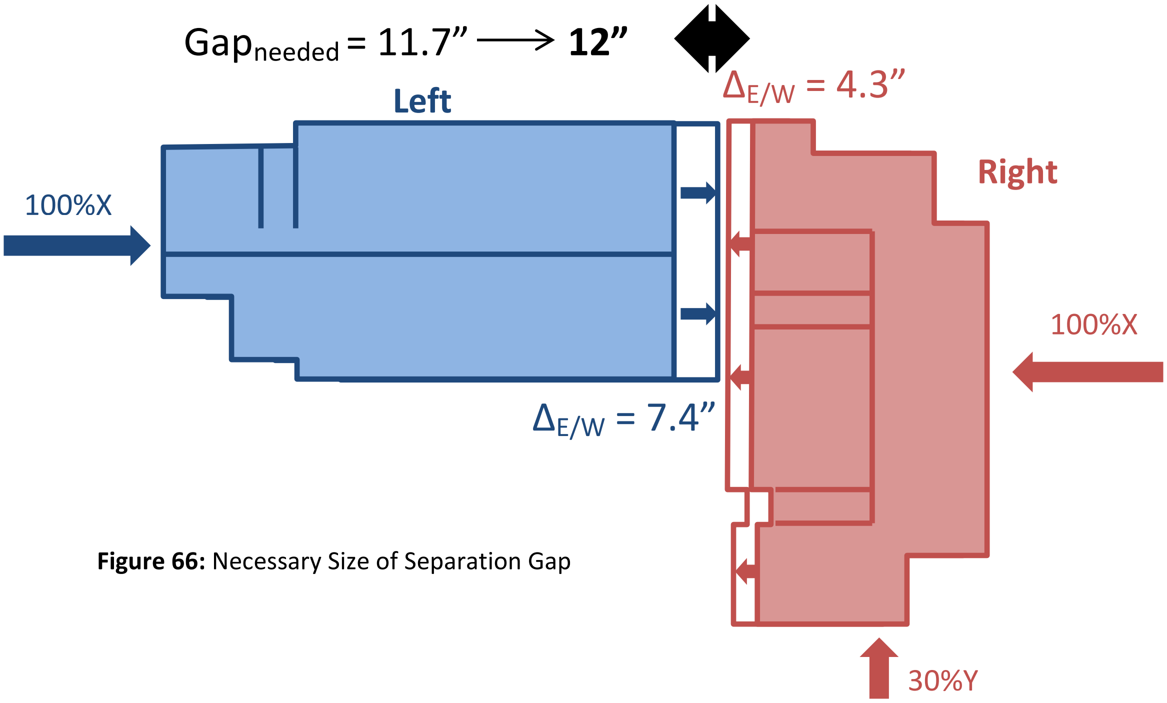


Figure 66: Necessary Size of Separation Gap

Left Wing Total Building Torsion															
Total Building Torsion in North/South Direction							Total Building Torsion in East/West Direction							100%N/s + 30%E/W	30%N/s + 100%E/W
Story	F_y (k)	L_x (ft)	e_{acc} (ft)	e_i (ft)	e_{tot} (ft)	M (k-ft)	Story	F_x (k)	L_y (ft)	e_{acc} (ft)	e_i (ft)	e_{tot} (ft)	M (k-ft)	M (k-ft)	M (k-ft)
7.0	312.5	140.0	2.6	0.7	3.3	-1026.6	7.0	260.7	59.0	-3.1	-1.1	-4.2	-1088.6	-1353.2	-1396.6
6.0	289.0	140.0	2.5	0.7	3.2	-928.9	6.0	241.1	59.0	-3.0	-1.1	-4.1	-999.2	-1228.7	-1277.9
5.0	245.9	140.0	3.7	0.7	4.4	-1073.8	5.0	205.2	59.0	-3.0	-1.1	-4.1	-843.2	-1326.7	-1165.3
4.0	203.2	140.0	5.5	0.7	6.2	-1261.6	4.0	169.5	59.0	-3.0	-1.1	-4.1	-696.6	-1470.5	-1075.1
3.0	160.7	140.0	9.6	0.7	10.3	-1648.3	3.0	134.1	59.0	-3.0	-1.1	-4.1	-548.2	-1812.8	-1042.7
2.0	119.3	140.0	15.9	0.7	16.6	-1982.1	2.0	99.5	59.0	-2.9	-1.1	-4.0	-402.1	-2102.7	-996.7
1.0	85.3	140.0	24.4	0.7	25.1	-2141.6	1.0	71.2	59.0	-2.8	-1.1	-3.9	-274.9	-2224.1	-917.4
Total Direction Torsion =						-10062.8	Total Direction Torsion =						-4852.9	-11518.6	-7871.7
Counter Clockwise				Clockwise			Counter Clockwise				Clockwise				
Right Wing Total Building Torsion															
Total Building Torsion in North/South Direction							Total Building Torsion in East/West Direction							100%N/s + 30%E/W	30%N/s + 100%E/W
Story	F_y (k)	L_x (ft)	e_{acc} (ft)	e_i (ft)	e_{tot} (ft)	M (k-ft)	Story	F_x (k)	L_y (ft)	e_{acc} (ft)	e_i (ft)	e_{tot} (ft)	M (k-ft)	M (k-ft)	M (k-ft)
7.0	312.5	59.0	-6.7	-4.5	-11.2	3506.3	7.0	283.7	140.0	-5.3	-5.6	-10.9	-3083.1	2581.4	-2031.2
6.0	289.0	59.0	-6.4	-4.5	-10.9	3142.6	6.0	264.1	140.0	-4.7	-5.6	-10.3	-2731.8	2323.1	-1789.0
5.0	245.9	59.0	-5.8	-4.5	-10.3	2538.1	5.0	224.7	140.0	-4.1	-5.6	-9.7	-2182.2	1883.4	-1420.7
4.0	203.2	59.0	-5.0	-4.5	-9.5	1923.3	4.0	185.7	140.0	-3.2	-5.6	-8.8	-1642.9	1430.4	-1065.9
3.0	160.7	59.0	-3.6	-4.5	-8.1	1309.5	3.0	146.8	140.0	-2.2	-5.6	-7.8	-1143.7	966.4	-750.8
2.0	119.3	59.0	-2.0	-4.5	-6.5	769.7	2.0	109.1	140.0	-1.2	-5.6	-6.8	-738.1	548.3	-507.2
1.0	85.3	59.0	-0.2	-4.5	-4.7	401.8	1.0	78.7	140.0	-0.6	-5.6	-6.2	-485.0	256.3	-364.5
Total Direction Torsion =						13591.3	Total Direction Torsion =						-12006.7	9989.3	-7929.3
Counter Clockwise				Clockwise			Counter Clockwise				Clockwise				

Table 26: Effect of Torsion

Table 26 shows the amount of torsion at diaphragm level and total in that wing. It also shows what the torsion is like in the special earthquake load combinations and the controlling case. This information is backed up by the ETABS model; both of those combinations almost caused the building to be torsionally irregular and lead to building's maximum displacements.

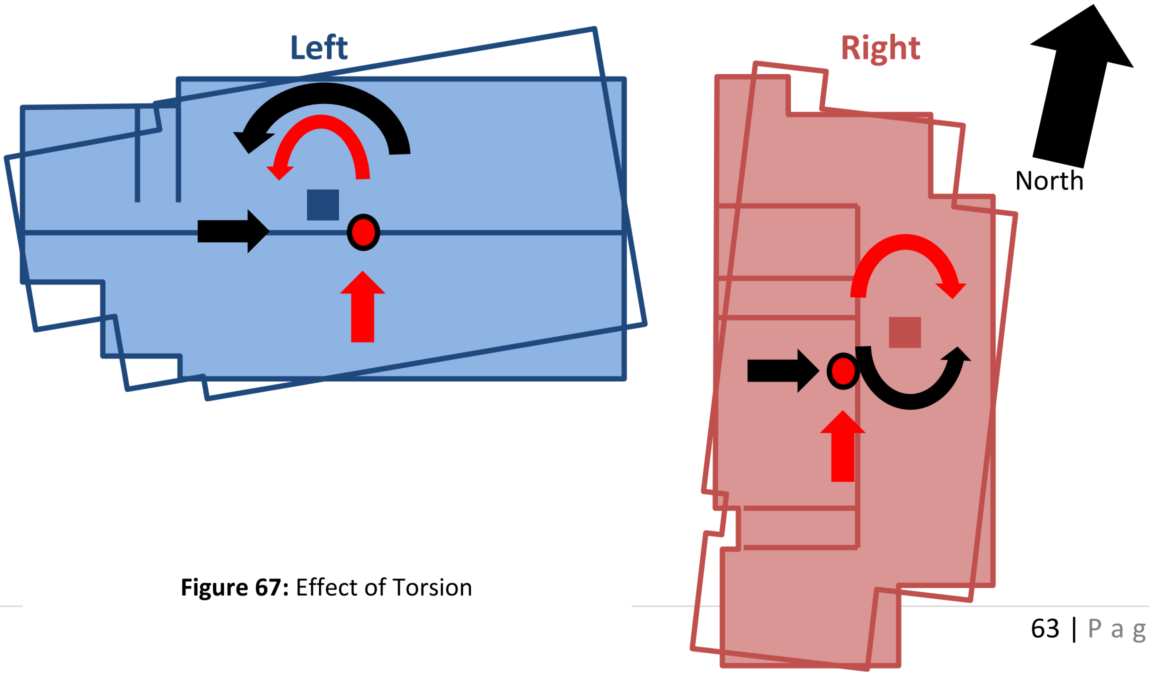


Figure 67: Effect of Torsion

Figure 67 shows how each building would act under the controlling earthquake case of 100%N/S + 30%E/W. Torsion in the left wing is additive and sums to a large number, mainly because the concrete shear walls create a large eccentricity at the ground level. The left wing fairs slightly better because torsion in one direction is counteracted by the torsion in the other direction. In the end the two wings want to rotate in opposite directions, so if they were connected into an L shape it would behave poorly as a unit. The two wings would most likely still want to rotate in opposite directions and create large forces on the center of the building and at the reentrant corner.

Code Check

The International Building Code sets certain standards that the structural system has to meet for safety or building requirements. The allowable deflection during a seismic event is governed by life safety. It is realized that seismic loading is going to be too great to try and keep not structural members from being damaged. For this reason the allowable deflection for seismic loading is less stringent than wind.

The following allowable drift criteria found in the International Building Code, 2006 edition.

- Allowable Building Drift: Δ_{wind} =H/400
- Allowable Story Drift: $\Delta_{seismic}$ = .02H_{sx} (all other structures)

Left Wing Seismic Story Drifts									
Story	Story Ht. (in)	Item	Load	Location (in.)			DriftX	C_d = 5 (SCBF)	
				X	Y	Z		Δ (in)	Δ_{Allow} (in)
5	117.6	Max Drift X	100%E/W + 30%N/S	804	0	698.4	0.002104	1.237152	2.352
5	117.6	Max Drift Y	100%N/S + 30%E/W	1704	120	698.4	0.003628	2.133264	2.352
Right Wing Seismic Story Drifts									
5	117.6	Max Drift X	100%E/W +_30%N/S	0	1668	698.4	0.001193	0.701484	2.352
5	117.6	Max Drift Y	100%N/S +_30%E/W	0	996	698.4	0.003196	1.879248	2.352

Table 27: Seismic Story Drift Check

The structure was designed for high seismic loads and a wind load 7 times smaller. The resulting deflection of the building under wind load was under an inch and the allowable was 2.34 inches at the roof level.

$$\Delta_{allowable} = \frac{78 * 12}{400} = 2.34\ in.$$

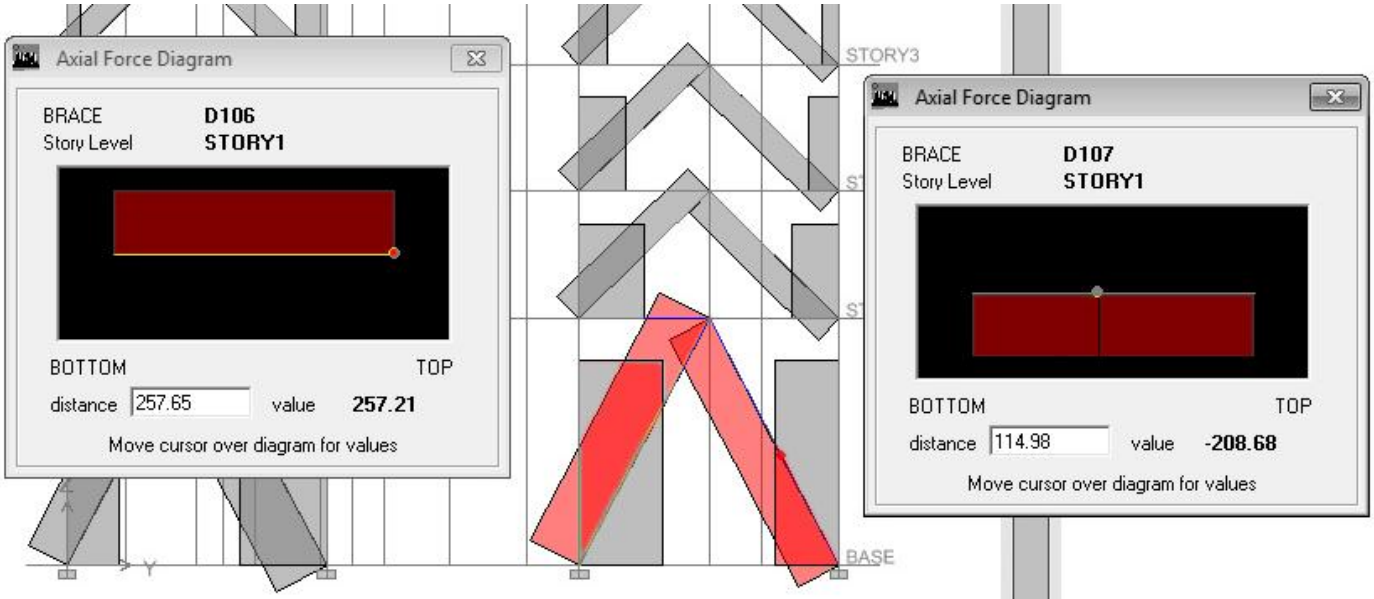
Design Checks

ETABS can also be used as a way to check hand calculations. The first checked calculation was location of the center of rigidity, *Table 28*. All of the hand calculations line up very well with that of ETABS, except for the right wing X location. Small deviations are probably due to the fact that the rigidities of the braced frames were estimated by using all of the same members in the frames, assuming that the geometry would lead to the frame's stiffness. There must have been an error somewhere in the calculation that is 32% off.

Hand Vs. ETABS								
	CRX				CRY			
	Hand	ETABS	Diff.	% Diff.	Hand	ETABS	Diff.	% Diff.
LW	904	874	29	3	337	352	-16	-5
RW	384	506	-122	-32	854	850	4	0

Table 28: Hand vs. ETABS Center of Rigidity Calculations

Check Force in Brace 16-F (Left Wing)

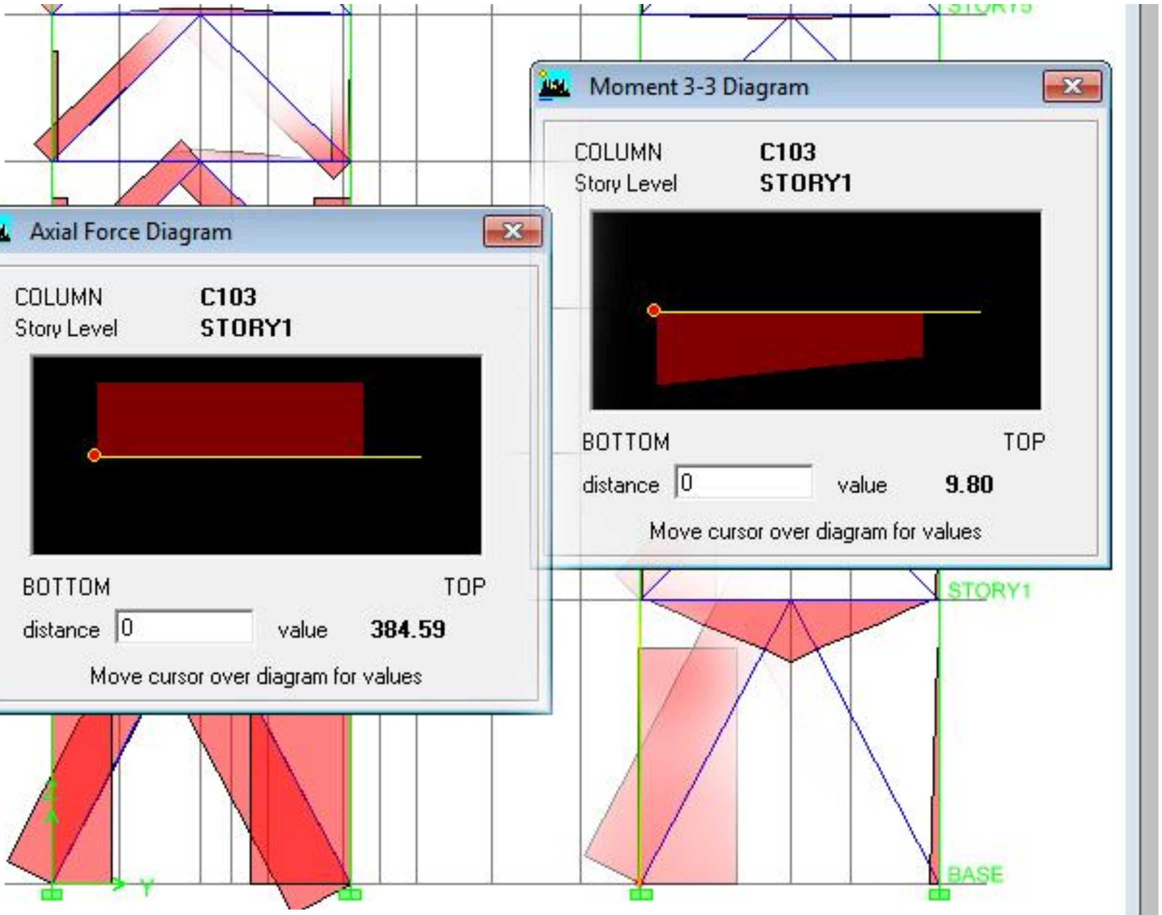


Axial Force Diagram
BRACE D106
Story Level STORY1
BOTTOM TOP
distance 257.65 value 257.21
Move cursor over diagram for values

Axial Force Diagram
BRACE D107
Story Level STORY1
BOTTOM TOP
distance 114.98 value -208.68
Move cursor over diagram for values

Figure 68: Check Axial Compression in Brace in Frame 16-F

$\boldsymbol{\emptyset Pn\ of\ HSS\ 8x8x.5\ (KL=22) =\ 336\ kips\ > 208\ kips} \quad (\boldsymbol{ok})$ (293K predicted, less because of changes addition of another frame and possibly building effects)



Axial Force Diagram

COLUMN C103
Story Level STORY1

BOTTOM TOP
distance 0 value 384.59
Move cursor over diagram for values

Moment 3-3 Diagram

COLUMN C103
Story Level STORY1

BOTTOM TOP
distance 0 value 9.80
Move cursor over diagram for values

Figure 69: Check Column in Frame 16-F

$$\boldsymbol{\emptyset Pn\ (tension\ yeild)\ of\ W10x49\ =\ 648\ kips\ > (.9-.2)*48k\ (DL) - 384.6\ \ kips} \\ \boldsymbol{= 351\ kips\ upflift\quad (ok)}$$

Lateral forces tend to put one column in tension and one in compression when loaded. Frame 16-F in the left wing has a very small gravity load on it because it runs parallel to the span direction of the 1-way precast concrete plank slab and therefore only carries self-weight (not calculated) and the dead load from wall load. This was determined to be the controlling uplift case when designing the braced frames, table found in appendix F, and in this case the modeled force is even greater than predicted. Also there's a noticeably less amount of axial force in the exterior column, this is because that column frames into Frame A and then makes the whole end like a column with Frame A ending up out of plain force on its members. This also affects rigidity because how it acts as a unit. Another crucial thing to check is that the beams and columns have more room left in the H1-1 equations for additional load than the braces do. This is because the braces are designed to yield and the beams and columns to remain elastic. For this the ETABS steel check was utilized. In the majority of frames this was found to be true.

Design of Frame D Along Interior Hallway

As is shown in *Figure 70*, Frame type D was added along the interior hallway of the left wing with the doors into hotel rooms going in the middle of it. These frames were added in order to minimize the deflection towards the right wing, which they did by 1.5 inches. These frames must be adjusted because they carry more gravity load than exterior frames. *Table 29* shows the resulting size of the members in the interior frames.

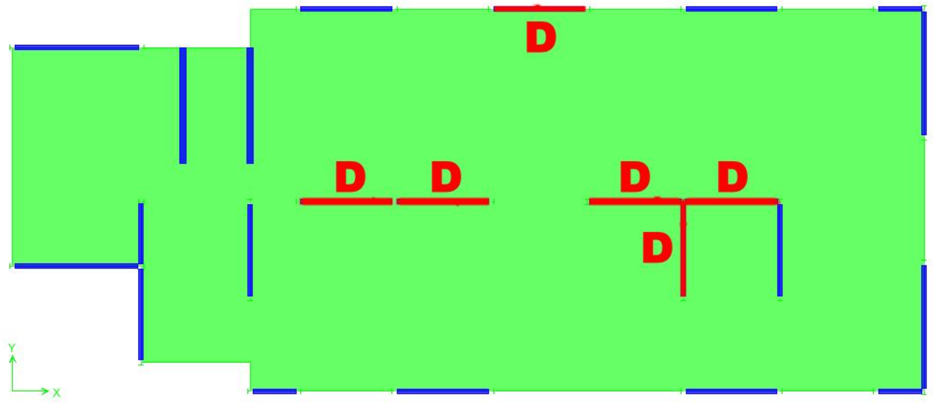


Figure 70: Design Interior Frame D

Frame	Brace Check	Beam Check		Revised Beam	Check Interaction (Table 6-1)	Revised Column Chosen (table 4-1)
$D_{interior}$	HSS 4x4x.25	W21x62	1.10	W21x73	0.92	
	HSS 4x4x.5	W27x84	1.06	W27x94	0.93	W10x45
	HSS 4x4x.5	W27x84	1.06	W27x94	0.93	
	HSS 5x5x.5	W30x108	0.99			W10x60
	HSS 5x5x.5	W30x108	0.99			
	HSS 5x5x.5	W30x108	0.99			
	HSS 7x7x.625	W36x135	1.02	W36x150	0.89	W10x100

Table 29: Design Interior Frame D

Shear Wall Size Check

A hand calculation was done to determine if the thickness of the shear wall would be adequate. The detailed design of the wall was not in scope of work. It would be necessary for the wall to have special rebar layout requirements in order to obtain the ductility that is assumed by a Special Reinforced Concrete Shear Wall. Check appendix H for a full hand calculation verification of the wall thickness.

Structural Conclusion

Now that the members with the greatest loads have been spot checked, this concludes the structural depth of the proposal. The proposed building was found to be sufficient under gravity and lateral loads and to behave normally under an extreme earthquake event. The addition of the separation joint and even placement of frames around the wing help provide a good solution to maintaining the “L” shape of the Hyatt Place if it were to be moved into a region of high seismic activity.

Architecture Study (Breadth 1)

In many cases structural needs for lateral loads can become an architectural emphasis of the building. In this architectural study the goal was nearly the opposite, it was to keep the lateral and gravity systems out of sight and keep the buildings appearance as a whole as near to the existing design as possible. Many times chain corporations such as hotels or large restaurants desire to have an iconic symbol that people will remember and thus hopefully lead to returning customers. It does not seem that this is the case with Hyatt Place hotels, but it is investigated to see if it is possible to architecturally design an ordinary looking hotel façade and building plan that can be built in diverse locations. The main focus of this study will be to find a braced frame layout that is unobtrusive to the building façade and secondary is to minimize alterations needed to the architectural plan.

Figure 71 shows how Hyatt Place Hotels do not have a distinct architectural style on their facades other than a tendency to more heavy and massive materials. One thing that can be noticed is the way the hotel rooms are laid out in 3 of 4 structures, and play a key role in the building façade. The windows or are offset in the hotel rooms so that windows are against each other. In the Hyatt Place North Shore this is done because the bathrooms are placed next to the windows so that bathrooms in adjacent hotel rooms have a common wall and thus simpler mechanical layout. This creates an interesting problem when laying out braced frames without disturbing the façade.

Figure 71: Hotel Building Facades

Figure 72: Hyatt Place Building Faced and Frame Possible Frame Location

When looking at the Hyatt Place building façade there are two possible locations for lateral frames. The red box show a typical location for a braced frame with the columns located where the interior partition walls meet with the exterior wall, leading to an easier and less intrusive column layout. The blue box shows a location for lateral resistance that doesn't involve going around the window and allows for more freedom. Both systems will require a sacrifice from one side or the other. The first key is to look at the options available with each location. One other noticeable thing about the building façade is that no matter what the ground floor window layout will need to be redone. As is typically the case the lobby level has open spaces and public areas that desire large windows. In a high seismic region such as California there are limitations on how much the rigidity of the bottom level can change. Irregularities in stiffness create regions of stress and possible failure, so this has to be avoided in this case.

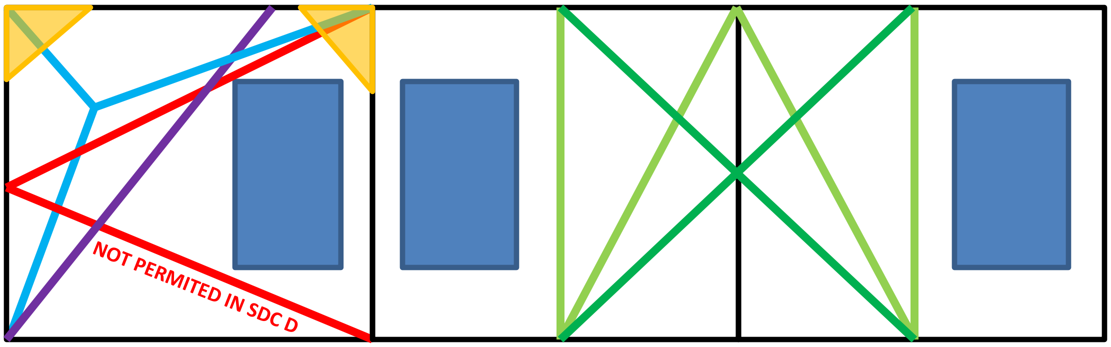


Figure 73: Lateral System Layout Around Typical Hotel Facade

There are two possible solutions, *Figure 73* shows the first one, working around the architecture and *Figure 74* and *75* show two possible ways to change the typical hotel layout/architecture in order to be more accommodating to different building locations and the loads that come along with them. In *Figure* 73 the cheapest lateral system is green colored (Inverted-V is light green), the dark green (X-Brace) being the cheapest because it provides the smallest structural members. The red frame (K-Brace) is would work for areas with low lateral loads, but is not permitted in California. The other bracing ideas need larger member sizes or more detailing. Either way it is an expensive solution, and moderate price range hotel construction is desired to be cheap and simplistic. Even with the green frames there is still architectural disturbance at the ground level. *Figure* 74 shows a proposed common hotel building façade design that will better suit more locations and structures so that building plans can be transplanted with less complication and cost.

Figure 74: Hotel Buildings Windows Stay in Vertical Shafts to Provide More Flexibility for Structural Plan

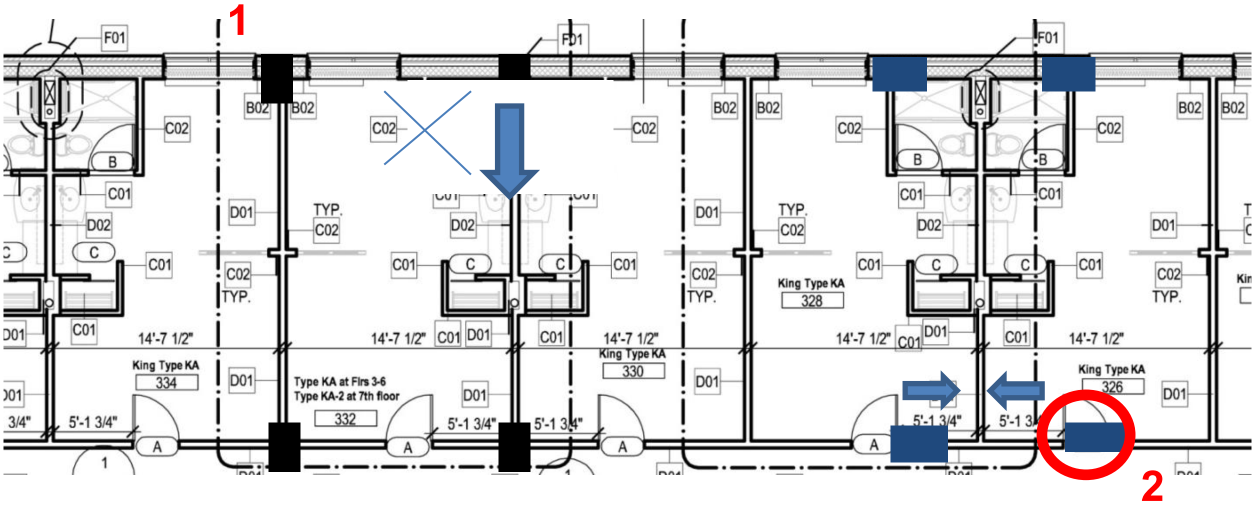


Figure 75: Hotel Room Layout in Plan

In *Figure 75* the different layouts of lateral systems are shown in plan view of the hotel rooms. Idea #1 is to move the bathroom backwards and shift the spaces around and idea two is to slimply just shift the door toward the inside wall to avoid new columns. Idea 1 seems good because it keeps the columns in the walls, but while providing structural simplicity it takes away from privacy of the room because the vanity sink is right beside the door and it makes the space longer and narrow. In option 2 the layout the of the room remains intact as the architect designed it and the columns on the exterior wall are atleast partically hidden by the intersection with the exterior wall. In option 2 there will also be an architectual feature made out of the column in the wall in order to minimize its distrubance. In the proposed structural design option 2 was used, as is discused previously throughout the report. So the idea of vertical windows lines continuing down to the ground level in Figure 75 is used and the windows on the ground level are increased in height to make up for the slight loss in width. Then there are smaller shorter windows added so that frames can be put in if needed but still not visible. The windows around the building on the bottom level are lined up with the windows on the upper level to create a more uniform look through the building and allow for more structural options. On the right wing the doors by the meeting room were realigned to allow them to fit between the columns of braced frame D (1). Overall this was the only major change to the right wing. The left wing had an overall shift of the walls from the red line over of 5 feet, and a the bathrooms were switched in order to fit the braced frames to prevent torsional irregularity and to bring the columns down without transfer girders. The windows at the pool area were realigned around the braced frame in that corner. Lastly the windows were taken into acount when sizing exterior beams, a maximum beam size of W18s were used and 1 foot was added to each story to not need changes to the windows.

Changes to Acomidate Proposed Structure

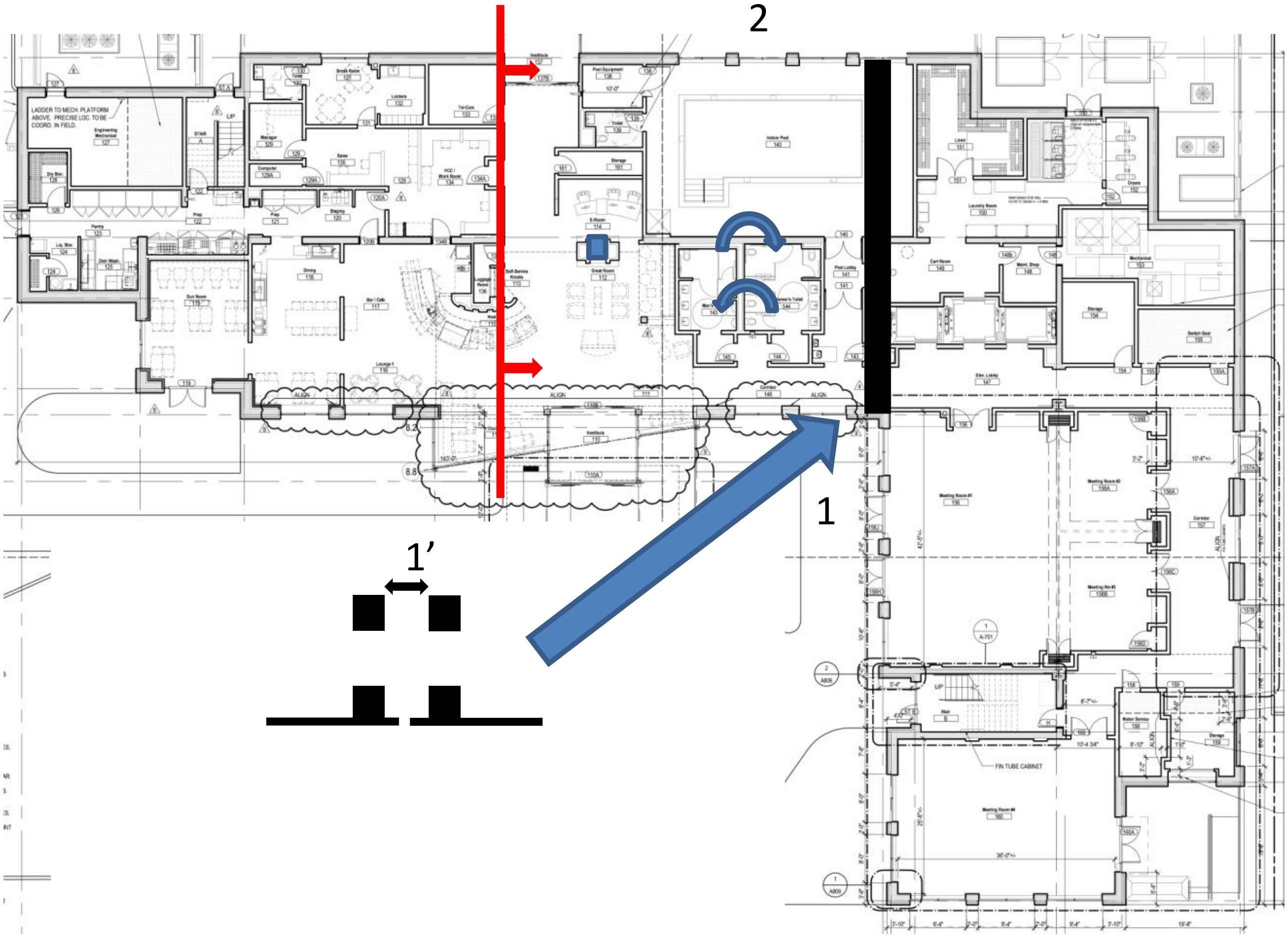


Figure 76: Location of Architectural Changes

Construction Cost and Schedule Study (Breadth 2)

Construction cost and schedule is important when comparing the feasibility of two buildings. Moving the Hyatt Place hotel to a high seismic region will require a more detailed structure to be built in order to provide the ductility to remain safe during earthquake loading. Schedule is important to a hotel owner, the faster the building is up and ready to be used, the faster he can start making a profit. For this reason the cost, schedule and planning logistics of both buildings was analyzed to determine the effect of designing for earthquake loading.

Cost

In this comparison the masonry shear walls that carry both the gravity and lateral load are compared with steel W-Shapes that support gravity loads and frame into braces that take the majority of the lateral load. The proposed building also has concrete shear walls to add complication to the mix. The existing building is very simple, but labor intensive to build. But the simplicity of the materials used allows the cost to be very low. The proposed building has mainly a steel structure, which leads to higher costs. On top of that there are a large number of braces and large beams in special concentric braced frames to take lateral load and concrete shear walls and concrete toping in order to have a rigid diaphragm. So the move to the move to California adds a few hundred thousand dollars to what would have been the necessary cost to build the same structure in Pittsburgh, PA. On top of that the precast concrete plank is more expensive in California due to localized material costs. *Table 30* shows a summary of costs.

Cost Comparison			
	Existing	Proposed	
Structure	688976	1677784	
Floor	1040835	1165113	**% Difference**
Total	**1729811**	**2994897**	**42%**

Table 30: Cost Comparison

Cost of Existing Masonry Structure													
Line Number	**Material**	**Amount**	**Unit**	**Crew**	**Daily Output**	**Days to Complete** (1 Crew)	**Labor Hours/Units**	**Labor Hours**	**Material Cost/Unit**	**Labor Cost/Unit**	**Equipment Cost/Unit**	**Total Cost/Unit**	**Total Cost with**	**Total Cost**
42210141150	8" CMU, reinforced	57650	SF	D-8	395	146	0.101	5823	2.36	3.91	0	6.27	8.51	$ 490,601.50
42210141250	12" CMU, Reinforced	15498	SF	D-9	300	52	0.16	2480	3.35	6.06	0	6.41	12.8	$ 198,374.40
34113500100	8" Hollowcore, untoped	95753	SF	C-11	3200	30	0.023	2202	7.16	1.3	0.72	9.18	10.87	$ 1,040,835.11
													Total Existing System Cost =	$ 1,729,811.01

Table 31: Detailed Existing Cost

Cost of Proposed Steel Structure														
Line Number	**Material**	**Amount**	**Unit**	**Crew**	**Daily Output**	**Days to Complete** (1 Crew)	**Labor Hours/Units**	**Labor Hours**	**Material Cost/Unit**	**Labor Cost/Unit**	**Equipment Cost/Unit**	**Total Cost/Unit**	**Total Cost with O&P**	**Total Cost**
Steel Superstructure														
51223177000	Columns - W10x68	3214	L.F.	E2	984	3	0.057	183	89.35	2.65	1.63	93.63	93.63	$ 300,926.82
51223177050	Columns - W10x45	2273	L.F.	E2	1032	2	0.054	123	59.02	2.52	1.56	63.1	70.96	$ 161,292.08
51223756900	Beams - W16x31	4830	L.F.	E2	900	5	0.062	299	40.61	2.9	1.79	45.3	51.8	$ 250,194.00
51223756300	Beams - W30x108	2710	L.F.	E5	1200	2	0.067	182	141.87	3.14	1.46	146.47	162.92	$ 441,513.20
512234004	Bracing - Extrapolated From 3x3	5712	L.F.	E3	48	119	0.058	331	7.15	20.42	2.57	28.13	44.25	$ 252,756.00
78116100400	Fireproofing	40404	S.F.	G2	1500	27	0.016	646	0.53	0.38	0.08	0.99	1.24	$ 50,100.96
													Steel Frame Total =	$ 1,406,682.10
Concrete Shear Walls														
33105350300	N.W. Concrete, 4000psi	413	C.Y.						100.43			100.43	110.18	$ 45,467.61
32110502700	Reinforcement, #7 to #11	17	Ton						44.06			44.06	48.25	$ 833.76
31113852550	Formwork	22464	SFCA	C2	395	57	0.122	2741	0.64	5.53		6.17	9.24	$ 207,567.36
33105705200	Placing, pumped	413	C.Y.	C20	110	4	0.582	240		21.6	7.26	29.22	41.76	$ 17,232.96
													Shear Walls Total =	$ 271,101.69
Precast Concrete Plank														
34113500100	8" Hollowcore, untoped	95753	SF	C-11	3200	30	0.023	2202	7.98	1.08	0.63	9.66	11.23	$ 1,075,306.19
33105350300	N.W. Concrete, 4000psi	591	C.Y.						100.43			100.43	110.18	$ 65,123.86
33105705200	Placing, pumped	591	C.Y.	C20	110	5	0.582	344		21.6	7.26	29.22	41.76	$ 24,683.00
Seismic Separation Joint													Precast Plank Total =	$ 1,165,113.05
$200 plf	760 linear feet (ext. and interior)	$152,000											Total Proposed System Cost =	$ 2,994,896.84

Table 32: Detailed Proposed Cost

Costs Mainly Due To Seismic

Schedule

Schedule Summary			
	Existing	**Proposed**	**% Change**
1st Floor	24	13	-45.8%
2nd - 7th	8	9	12.5%
Total	**72**	**71**	**-1.4%**

The owner is always anxious to get into his building, so the schedule is almost always an important factor, and definitely is when it is a hotel building. There are many ways that moving a building to a high seismic region could lead to a longer schedule and complications involving staging of tasks. The existing structure is very labor intensive but is also very simplistic and straight forward. It takes time to lay masonry, but there is no time spent waiting for concrete to setup or working on tedious steel connections. A big issue with the proposed building's schedule is a staging. Like the issue of unbalanced stiffness at the lower stories, the concrete shear walls pose a problem with the steps to building the structure. It takes time to make the formwork and it takes even more time to let the concrete setup enough to place the next level. The shear walls will need to be started ahead of time and have connection plates set and cured before the steel structure can erected. Concrete needs 7 days to be setup before the next level can be placed. With 2 days needed to step formwork for the next pour the crew will have 2 days of down time each week (pour on Mondays and form on Thursdays and Fridays). The concrete crews C-20 and C-2 will not be needed the majority of the time. If there is only 1 C-2 crew on the jobsite, then they can spend the 4 days of the week that there is now pouring to be setting up the formwork for the next pour. One crew will be working on laying plank and one on erecting steel and 4 on bracing in the frames. Bracings is very time intensive with many intricate connections, so it will be worked on continuously the whole time the building is going up. With the proposed building there can be multiple tasks going on at once in order to try and keep time down, but it will require a lot of coordination, and any set backs on shear wall construction or steel frame erecting will lead to major backups. Overall there are many complications added to the schedule of the building because of the details and different systems used to take the increased lateral loads. The masonry structure would be preferred for a more predictable and simplistic schedule. *Tables 33* and *34* show the complications with crews and coordination. But in the end it is possible to achieve the same schedule in a high seismic region.

Schedule 1st Floor (existing)								
Order	**Task**	**Amount**	**Crews**	**Daily Output**	**Days**	**Total Days**	**Crew Type**	**# on Jobsite**
Complete 1st	12" CMU	15498	3	300	17.2	**24.0**	D-8	1 to 4
	8" CMU	1638	1	395	4.1		D-9	2*
	Plank	13679	2	3200	2.1		C-11	2
Schedule 2nd Through 7th (existing) 6 floors							*Only on First Floor	
	Task	**Amount**	**Crews**	**Daily Output**	**Days**	**Total Days**		
Complete 1st	8" CMU	9335.5	4	395	5.9	**8.0**		
	Plank	13679	2	3200	2.1			
				Total Days of Building =		**72**		

Table 33: Existing Schedule Per Floor

Schedule 1st Floor (existing)													
Coordination	**Task**	**Amount**	**Crews**	**Daily Output**	**Days Shear Wall**	**Days Steel Frame**	**Days Plank**	**Days Steel Bracing**	**Fireproofing**	**Total Days of Work**	**Total Per Floor***	**Crew Type**	**# on Jobsite**
Complete 1st	Formwork	2736	1	395	6.9					**7.4**		E2	1
1 Week Allowance	Placing Concrete	51	1	110	0.5							E3	4
Complete Before Beams	Columns-W10x68	802	1	984		0.8				**2.4**	**13.0**	E5	1
	Columns-W10x45	568	1	1032		0.6						C2	1
Complete Before Bracing	Beams-W30x108	390	1	900		0.4						C11	2
	Beams-W16x31	690	1	1200		0.6						C20	1
Before Topping	Plank	13697	1	3200			4.3			**5.0**		G2	1
	Plank Toping	85	1	110			0.8					Cranes	2
As Placing Plank	Bracing	1360	4	48				7.1		**7.1**			
After Floor is Done	Fireproofing	8355	1	1500					5.6	**5.6**			
Schedule 2nd Through 7th (existing)													
Complete 1st	Formwork	1440	1	395	3.6					**3.9**			
1 Week Allowance	Placing Concrete	27	1	110	0.2								
Complete Before Beams	Columns-W10x68	401	1	984		0.4				**1.7**	**9.0**		
	Columns-W10x45	284	1	1032		0.3							
Complete Before Bracing	Beams-W30x108	390	1	900		0.4							`
	Beams-W16x31	690	1	1200		0.6							
Before Topping	Plank	13697	1	3200			4.3			**5.0**			
	Plank Toping	85	1	110			0.8						
Anytime post beam	Bracing	816	4	48				4.3		**4.3**			
After Floor is Done	Fireproofing	5484		1500					3.7	**4***			
							Total Days of Building =				**71**		
*coordinated so that some tasks can be worked on similaneously so total days per floor are less than total days of work													
**done after completion of floor so only added time is to the end of the structure													

Table 34: Proposed Schedule and Crews Per Floor

Conclusions

After redesigning the Hyatt Place for a new location in San Diego, CA many conclusions were draw about the effect of seismic load on the existing building shape, architecture and cost. The effects of building torsion were able to be limited through the use of Special Concentric Braced Frames, Special Reinforced Concrete Shear Walls, and a building separation joint. The gravity and lateral systems were able to be designed around the existing architecture and conclusions were drawn on how to better overall architecturally design buildings to fit in different locations with different types of load. It was also determined that the systems needed to resist these forces will result in a substantial increase in total building cost and will lead to a more complicate schedule that has the possibility of delays.

The structural depth consisted of a full load path determination in the vertical and horizontal directions. Gravity loads successfully transferred from the precast concrete plank to D-Beams with the use of the Girder-Slab System and to the foundation within the allowable code deflection of L/240 for total dead load in the interior spans and L/600 in the exterior spans that support brick façade. The transfer truss spanning in the right wing was redesigned to carry the new loads efficiently using its geometry to limit moment.

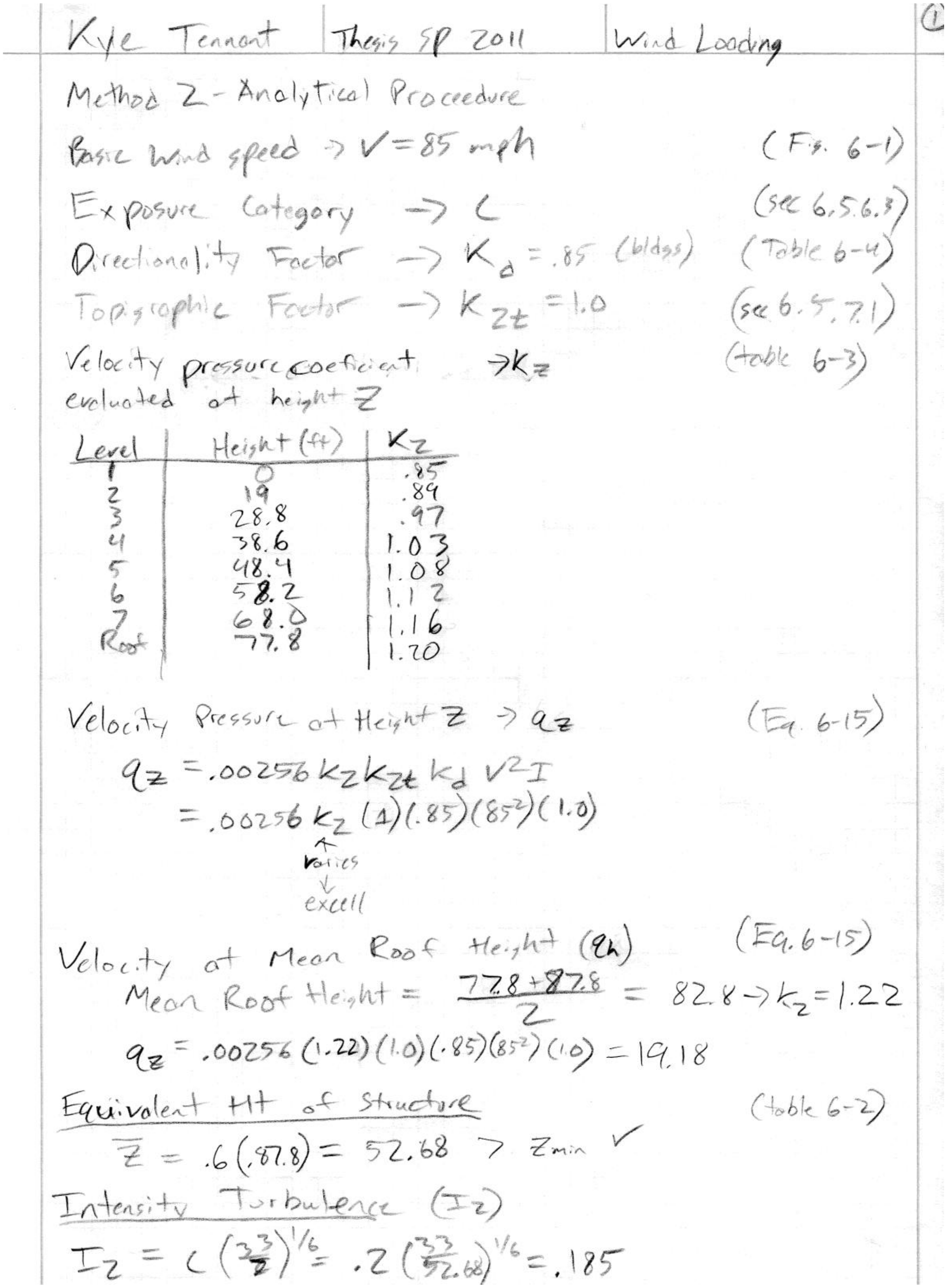
A large part of the gravity system also acted to help resist the lateral loads due to the great number of brace frames designed. Most of the brace frames were laid out along the exterior of the building in between windows to allow for Special Concentric Braced Frames as oppose to more expensive alternatives. With the frames around the exterior the columns were able to remain W10s due to the small tributary area and mainly axial loads. The beams in the Inverted-V braces had to be sized very large in order to take the forces coming out of the tension and compression braces.

It was noticed that braced frames and concrete shear walls behave very differently at different heights. The fact that concrete shear walls maintain their rigidity better led to the left wing becoming much more torsionally irregular than expected. The conclusion was drawn that when two different materials are used to resist lateral forces the center of rigidity of the two systems should line up to limit building torsion.

Once the building wings were modeled it was found that left wing had torsion acting counterclockwise and the right wing had torsion acting clockwise. The difference in behavior would likely have led to poor seismic performance if the building were to be left as an “L” shape. The necessary building separation joint was sized to be 12 inches. This separation will allow the structures to stay separate and the buildings to act independently and remain structurally safe in a seismic event.

Appendices

Appendix A: Wind Calculations



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Method 2 - Analytical Proceedure

Basic Wind speed → $V = 85$ mph (Fig. 6-1)

Exposure Category → C (sec 6.5.6.3)

Directionality Factor → $K_d = .85$ (bldgs) (Table 6-4)

Topigraphic Factor → $K_{zt} = 1.0$ (sec 6.5.7.1)

Velocity pressure coeficient evaluated at height Z → K_z (table 6-3)

Level	Height (ft)	Kz
1	0	.85
2	19	.89
3	28.8	.97
4	38.6	1.03
5	48.4	1.08
6	58.2	1.12
7	68.0	1.16
Roof	77.8	1.20

Velocity Pressure at Height Z → q_z (Eq. 6-15)

$$q_z = .00256\, k_z k_{zt} k_d V^2 I$$
$$= .00256\, \underset{\substack{\uparrow \\ \text{varies} \\ \downarrow \\ \text{excell}}}{k_z} (1)(.85)(85^2)(1.0)$$

Velocity at Mean Roof Height (q_h) (Eq. 6-15)

$$\text{Mean Roof Height} = \frac{77.8 + 87.8}{2} = 82.8 \rightarrow k_z = 1.22$$

$$q_z = .00256(1.22)(1.0)(.85)(85^2)(1.0) = 19.18$$

Equivalent Ht of Structure (table 6-2)

$$\bar{z} = .6(87.8) = 52.68 > z_{min} \checkmark$$

Intensity Turbulence (I_z)

$$I_z = c\left(\frac{33}{\bar{z}}\right)^{1/6} = .2\left(\frac{33}{52.68}\right)^{1/6} = .185$$

Kyle Tennant ②

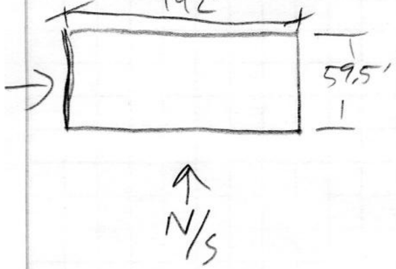
Integral Length Scale of Turbulence $(L_{\bar{z}})$ (Eq 6-7)

$$L_{\bar{z}} = \ell \left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 500\left(\frac{52.68}{33}\right)^{1/5} = \boxed{549.03}$$

Background Response Factor (Q) (Eq 6-6)

$$Q = \sqrt{\frac{1}{1+.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{.63}}} \qquad \bar{h} = 82.8'$$

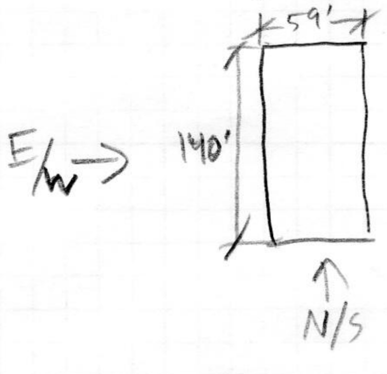
Left Wing



North/South
B = 140'
Q = .857

East/West
B = 59'
Q = .888

Right Wing



North/South
B = 59'
Q = .888

East/West
B = 140'
Q = .857

Gust Effect Factor (Eq. 6-4)

Sec. 6.2 → Rigid building → building whose fundamental frequency (Hertz) is greater than 1.

Hertz = period^{-1}

From ETABS mode, mode 1 is .8728 (LW) ∴ rigid

.8766 (RW) ∴ rigid

↓

G = .85

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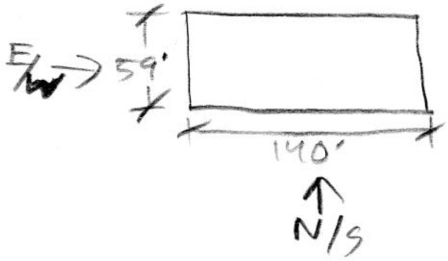
Left Wing

Cp (Fig 6-6)



North/South

L = 59'
B = 140' $\frac{L}{B} = .42 < 1$

	Cp	Use with
Windward Wall	.8	q_z
Leeward Wall	-.5	q_h
Side Wall	-.7	q_h

East/West

L = 140'
B = 59' $\frac{L}{B} = 2.37$

	Cp	Use with
Windward Wall	.8	q_z
Leeward Wall	-.28 *	q_h
Side Wall	-.7	q_h

* interpolated

Right Wing East/West North/South

Same dimensions, but building is rotated 90°

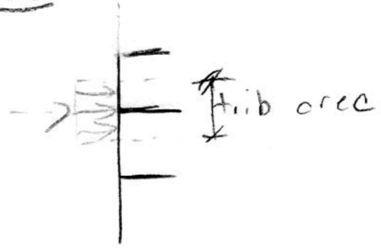
Wind Pressure

$P_z = q_z G C_p - \cancel{q_h G C_{pi}}$ (windward) $\cancel{GC_{pi} = \pm .18}$ not considered for my building type

$P_h = q_h G C_p - \cancel{q_h G C_{pi}}$ (leeward)

→ calculations are done in excel spreadsheet

Determine Forces



Deterine the psf per floor and the area of wall that each diaphragm takes to determine the force on each diaphragm.

Appendix B: Seismic Load Calculations

Left Wing (W1) N-S Direction

Location: San Diego, CA

Seismic Ground Motion Values

$R = 6 \rightarrow$ Special Concentric Braced Frames

$R = 5 \rightarrow$ special reinforced concrete shear walls

$I = 1$

$S_S = 1.5$ (USGS Website)

$S_1 = .5$

Soil Site Class = D

$S_{MS} = F_a S_S = 1(1.5) = 1.5$

$S_{M1} = F_v S_1 = 1.5(.5) = .75$

$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3} 1.5 = 1$

$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3} .75 = .5$

SDC : D

$\hookrightarrow S_{DS} > .5$

Approximate Fundamental Period $(T_a) = C_t h_n^x = .02(88)^{.75} = .57 sec$

Calculate Seismic Response Coefficient (C_s) (N-S)

$$C_s = \left| \begin{array}{l} \frac{S_{D1}}{T_a\left(\frac{R}{I}\right)} \text{ for } T_a < T_L = \frac{.5}{.57\left(\frac{5}{1}\right)} = \boxed{.175} \\ \frac{S_{DS}}{\left(\frac{R}{I}\right)} = \frac{1}{\left(\frac{5}{I}\right)} = .2 \end{array} \right.$$

(E-W) (Concent

$\frac{.5}{.57\left(\frac{6}{1}\right)} = \boxed{.146}$

$\frac{1}{\frac{6}{1}} = .167$

Seismic Base Shear (N-S)

$V = C_s W = .175(8,163.6) = \boxed{1428.6 \text{ kips}}$

(E-W)

$\boxed{1,191.88 \text{ Kips}}$

Story Shears

$F_x = C_{vx} V$

$k = 1.035$

$C_{vx} = \frac{w_x h_x^k}{\sum w_i h_i^k}$

(Done in excel)

Right Wing (W2) (all the same except weight)

N-S Direction

$R = 6$ (concentric brace frames)

$C_s = .146$

Weight = 7460.2 kips

$V_{base} = 1089.2 K$

E-W Direction

$R = 5$ (Special conc. shear walls)

$C_s = .175$

$V_{base} = 1305.5 K$

Weight of Building (Left Wing)

				/Lengt h			Weight (kips)		Model Imput Area Mass
Floor	Component	Weight (psf)	Weight (plf)		#	Area	Componen	Total Floor	
2nd	Int. Columns		77	14.5	8		8.93	1275.52	2.9539E-06
2nd	Ext. Columns		49	14.5	25		17.76		
2nd	Reinforced Concrete	150				486	72.90		
2nd	D-Beams (avg.)		46	15	8		5.52		
2nd	Beams (total)*						17.40		
2nd	Edge Beams (total)						4.40		
2nd	Ext. Wall	47				4954.5	232.86		
2nd	Precast Plank	88				7,761	682.92		
2nd	SDL	30				7,761	232.82		
3rd	Int. Columns		77	10	8		6.16	1158.40	2.68268E-06
3rd	Ext. Columns		49	10	25		12.25		
3rd	Reinforced Concrete	150				324	48.60		
3rd	D-Beams (avg.)		46	15	9		6.21		
3rd	Beams (total)*						10.50		
3rd	Edge Beams (total)						3.70		
3rd	Ext. Wall	47				3303	155.24		
3rd	Precast Plank	88				7,761	682.92		
3rd	SDL	30				7,761	232.82		
4th thru 7th	Int. Columns		45	10	8		3.60	1151.84	2.66749E-06
4th thru 7th	Ext. Columns		33	10	25		8.25		
4th thru 7th	Reinforced Concrete	150				324	48.60		
4th thru 7th	D-Beams (avg.)		46	15	9		6.21		
4th thru 7th	Beams (total)*						10.50		
4th thru 7th	Edge Beams (total)						3.70		
4th thru 7th	Ext. Wall	47				3303	155.24		
4th thru 7th	Precast Plank	88				7,761	682.92		
4th thru 7th	SDL	30				7,761	232.82		
Roof	Int. Columns		33	5	8		1.32	1083.51	2.50926E-06
Roof	Ext. Columns		33	5	25		4.13		
Roof	Penthouse Columns		33	5	4		0.66		
Roof	Reinforced Concrete	150				324	48.60		
Roof	D-Beams (avg.)		46	15	9		6.21		
Roof	Beams (total)*						10.50		
Roof	Edge Beams (total)						3.70		
Roof	Ext. Wall	47				1971.5	92.66		
Roof	Precast Plank	88				7,761	682.92		
Roof	SDL	30				7,761	232.82		
Penthouse	Columns		33	5	4		0.66	38.79	2.90475E-06
Penthouse	Beams (total)*						0.77		
Penthouse	Exterior Wall	47				320.00	15.04		
Penthouse	Precast Plank	63				240.00	15.12		
Penthouse	SDL	30				240.00	7.2		
							Total = 8163.58		

Seismic Design Variables (Left Wing E-W Direction)

			ASCE Reference
Soil Classification		D (stiff soil)	Table 20.3-1
Occupancy Category		II	Table 1-1
Seismic Force Resisting System		**Special Concentric braced frames (R = 6)**, ecentric braced frames (R = 7)	Table 12.2-1
Response Modification Factor	R	5	Table 12.2-2
Seismic Importance Factor		1.0	Table 11.5-1
Spectral Response Acceleration, Short	S_s	1.5	USGS Website
Spectral Response Acceleration, 1 sec.	S_1	0.5	USGS Website
Site Coeficient	F_a	1	Table 11.4-1
Site Coeficient	F_v	1.5	Table 11.4-2
MCE Spectral Response Acceleraton, Short	S_{MS}	1.5	Eq. 11.4-1
MCE Spectral Response Acceleration, 1 sec	S_{M1}	0.75	Eq. 11.4-2
Design Spectral Acceleration, Short	S_{DS}	1	Eq. 11.4-3
Design Spectral Acceleration, 1 sec.	S_{D1}	0.5	Eq. 11.4-4
Seismic Design Category	SDC	D (has some special design considerations)	11.6-1
Approximate Period Parameter	C_t	.02 (all other systems)	Table 12.8-2
Approximate Period Parameter	x	.75 (all other systems)	Table 12.8-3
Building Height	h_n	88'-0"	
Approximate Fundamental Period	T_a	0.57 sec.	Eq. 12.8-7
Long Period Transition Period	T_L	8 sec.	Fig. 22-15
Seismic Response Coeficient	C_s	0.146	Eq. 12.8-2
Structure Period Exponent	k	1.035 (2.5 sec. > T > .5 sec.)	Sec 12.8.3
Seismic Base Shear	**V**	**1191.9 kips**	**Eq. 12.8-1**

Seismic Story Shear and Moment Calculations Left Wing (E-W)

Level	Story Weight (K)	Height (ft)	K	$w_x h_x^k$	Vertical Distribution Factor C_{vx}	Forces (K) Fx	Story Shear (K) Vx	Moments (ft-K) Mx
Penthouse Roof	38.8	88.0	1.0	3992.6	0.0	10.6	10.6	930.7
Main Roof	1083.5	78.0	1.0	98435.3	0.2	260.7	271.3	21163.4
7th Floor	1151.8	68.2	1.0	91021.0	0.2	241.1	512.4	34931.2
6th Floor	1151.8	58.3	1.0	77462.3	0.2	205.2	717.6	41859.3
5th Floor	1151.8	48.5	1.0	63993.4	0.1	169.5	887.1	43026.5
4th Floor	1151.8	38.7	1.0	50616.2	0.1	134.1	1021.2	39487.7
3rd Floor	1158.4	28.8	1.0	37566.2	0.1	99.5	1120.7	32310.8
2nd Floor	1275.5	19.0	1.0	26865.2	0.1	71.2	1191.9	22646.1
Total	**8163.6**			**449952.2**				**236355.8**

Weight of Building (Right Wing)

Floor	Component	Weight (psf)	eight (p	Height /Length	#	Area	Weight (kips)		Model Input Area Mass
							Component	Total Floor	
2nd	Columns Total			14.5			28.94	1176.14	3.08291E-06
2nd	Reinforced Concrete	150				486	72.90		
2nd	D-Beams (avg.)		46	15	8		5.52		
2nd	Beams (total)*						17.40		
2nd	Edge Beams (total)						4.40		
2nd	Ext. Wall	47				4954.5	232.86		
2nd	Precast Plank	88				6,899	607.14		
2nd	SDL	30				6,899	206.98		
3rd	Columns Total			10			19.64	1058.01	2.77326E-06
3rd	Reinforced Concrete	150				324	48.60		
3rd	D-Beams (avg.)		46	15	9		6.21		
3rd	Beams (total)*						10.50		
3rd	Edge Beams (total)						3.70		
3rd	Ext. Wall	47				3303	155.24		
3rd	Precast Plank	88				6,899	607.14		
3rd	SDL	30				6,899	206.98		
4th thru 7th	Columns Total			10			12.85	1051.22	2.75546E-06
4th thru 7th	Reinforced Concrete	150				324	48.60		
4th thru 7th	D-Beams (avg.)		46	15	9		6.21		
4th thru 7th	Beams (total)*						10.50		
4th thru 7th	Edge Beams (total)						3.70		
4th thru 7th	Ext. Wall	47				3303	155.24		
4th thru 7th	Precast Plank	88				6,899	607.14		
4th thru 7th	SDL	30				6,899	206.98		
Roof	Int. Columns		33	5	36		5.94	982.39	2.57504E-06
Roof	Penthouse Columns		33	5	4		0.66		
Roof	Reinforced Concrete	150				324	48.60		
Roof	D-Beams (avg.)		46	15	9		6.21		
Roof	Beams (total)*						10.50		
Roof	Edge Beams (total)						3.70		
Roof	Ext. Wall	47				1971.5	92.66		
Roof	Precast Plank	88				6,899	607.14		
Roof	SDL	30				6,899	206.98		
Penthouse	Columns		33	5	4		0.66	38.79	2.92291E-06
Penthouse	Beams (total)*						0.77		
Penthouse	Exterior Wall	47				320.00	15.04		
Penthouse	Precast Plank	63				240.00	15.12		
Penthouse	SDL	30				240.00	7.2		
							Total =	7460.20	

Seismic Design Variables (Right Wing E-W Direction)			
			ASCE Reference
Soil Classification		D (stiff soil)	Table 20.3-1
Occupancy Category		II	Table 1-1
Seismic Force Resisting System		Special Concentric braced frames (R = 6),**special reinforced concrete shear walls (R = 5)**	Table 12.2-1
Response Modification Factor	R	5	Table 12.2-2
Seismic Importance Factor	I	1.0	Table 11.5-1
Spectral Response Acceleration, Short	S_s	1.5	USGS Website
Spectral Response Acceleration, 1 sec.	S_1	0.5	USGS Website
Site Coeficient	F_a	1	Table 11.4-1
Site Coeficient	F_v	1.5	Table 11.4-2
MCE Spectral Response Acceleraton, Short	S_{MS}	1.5	Eq. 11.4-1
MCE Spectral Response Acceleration, 1 sec	S_{M1}	0.75	Eq. 11.4-2
Design Spectral Acceleration, Short	S_{DS}	1	Eq. 11.4-3
Design Spectral Acceleration, 1 sec.	S_{D1}	0.5	Eq. 11.4-4
Seismic Design Category	SDC	D (has some special design considerations)	11.6-1
Approximate Period Parameter	C_t	.02 (all other systems)	Table 12.8-2
Approximate Period Parameter	x	.75 (all other systems)	Table 12.8-3
Building Height	h_n	88'-0"	
Approximate Fundamental Period	T_a	0.57 sec.	Eq. 12.8-7
Long Period Transition Period	T_L	8 sec.	Fig. 22-15
Seismic Response Coeficient	C_s	0.175	Eq. 12.8-2
Structure Period Exponent	k	1.035 (2.5 sec. > T > .5 sec.)	Sec 12.8.3
Seismic Base Shear	**V**	**1305.5 kips**	**Eq. 12.8-1**

Seismic Story Shear and Moment Calculations Right Wing (E-W)								
Level	Story Weight (K)	Height (ft)	K	$w_x h_x^k$	Vertical Distribution Factor C_{vx}	Forces (K) Fx	Story Shear (K) Vx	Moments (ft-K) Mx
Penthouse Roof	38.8	88.0	1.0	3992.6	0.0	12.7	12.7	1116.9
Main Roof	982.4	78.0	1.0	89249.6	0.2	283.7	296.4	23119.5
7th Floor	1051.2	68.2	1.0	83068.2	0.2	264.1	560.5	38205.3
6th Floor	1051.2	58.3	1.0	70694.2	0.2	224.7	785.2	45800.3
5th Floor	1051.2	48.5	1.0	58402.0	0.1	185.7	970.8	47085.9
4th Floor	1051.2	38.7	1.0	46193.7	0.1	146.8	1117.7	43217.6
3rd Floor	1058.0	28.8	1.0	34310.3	0.1	109.1	1226.8	35367.3
2nd Floor	1176.1	19.0	1.0	24771.6	0.1	78.7	1305.5	24804.5
Total	**7460.1**			**410682.2**				**258717.3**

Seismic Design Variables (Right Wing N-S Direction)			
			ASCE Reference
Soil Classification		D (stiff soil)	Table 20.3-1
Occupancy Category		II	Table 1-1
Seismic Force Resisting System		**Special Concentric braced frames (R = 6)**	Table 12.2-1
Response Modification Factor	R	5	Table 12.2-2
Seismic Importance Factor		1.0	Table 11.5-1
Spectral Response Acceleration, Short	S_s	1.5	USGS Website
Spectral Response Acceleration, 1 sec.	S_1	0.5	USGS Website
Site Coeficient	F_a	1	Table 11.4-1
Site Coeficient	F_v	1.5	Table 11.4-2
MCE Spectral Response Acceleraton, Short	S_{MS}	1.5	Eq. 11.4-1
MCE Spectral Response Acceleration, 1 sec	S_{M1}	0.75	Eq. 11.4-2
Design Spectral Acceleration, Short	S_{DS}	1	Eq. 11.4-3
Design Spectral Acceleration, 1 sec.	S_{D1}	0.5	Eq. 11.4-4
Seismic Design Category	SDC	D (has some special design considerations)	11.6-1
Approximate Period Parameter	C_t	.02 (all other systems)	Table 12.8-2
Approximate Period Parameter	x	.75 (all other systems)	Table 12.8-3
Building Height	h_n	88'-0"	
Approximate Fundamental Period	T_a	0.57 sec.	Eq. 12.8-7
Long Period Transition Period	T_L	8 sec.	Fig. 22-15
Seismic Response Coeficient	C_s	0.146	Eq. 12.8-2
Structure Period Exponent	k	1.035 (2.5 sec. > T > .5 sec.)	Sec 12.8.3
Seismic Base Shear	**V**	**1089.2 kips**	**Eq. 12.8-1**

Seismic Story Shear and Moment Calculations Right Wing (N-S)								
Level	Story Weight (K)	Height (ft)	K	$w_x h_x^k$	Vertical Distribution Factor C_{vx}	Forces (K) Fx	Story Shear (K) Vx	Moments (ft-K) Mx
Penthouse Roof	38.8	88.0	1.0	3992.6	0.0	10.6	10.6	931.8
Main Roof	982.4	78.0	1.0	89249.6	0.2	236.7	247.3	19289.0
7th Floor	1051.2	68.2	1.0	83068.2	0.2	220.3	467.6	31875.3
6th Floor	1051.2	58.3	1.0	70694.2	0.2	187.5	655.1	38211.9
5th Floor	1051.2	48.5	1.0	58402.0	0.1	154.9	810.0	39284.6
4th Floor	1051.2	38.7	1.0	46193.7	0.1	122.5	932.5	36057.2
3rd Floor	1058.0	28.8	1.0	34310.3	0.1	91.0	1023.5	29507.5
2nd Floor	1176.1	19.0	1.0	24771.6	0.1	65.7	1089.2	20694.8
Total	**7460.1**			**410682.2**				**215852.0**

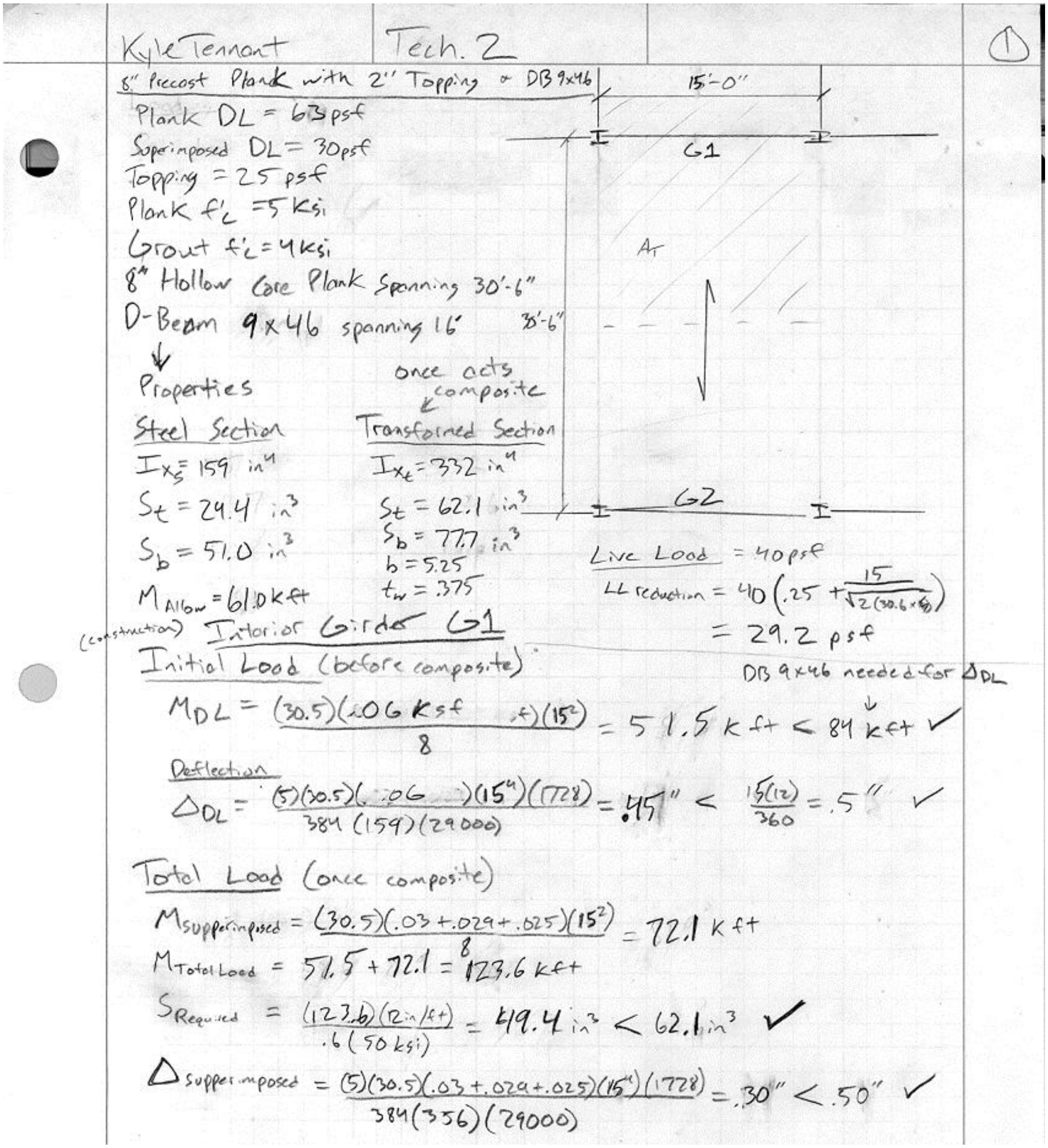
Seismic Load Combinations

100% N/S & 30% E/W				
Level	North/South (Y)		East/West (X)	
	Forces (K) Fx	Story Shear (K) Vx	Forces (K) Fx	Story Shear (K) Vx
Penthouse Roof	**10.6**	10.6	**3.8**	3.8
Main Roof	**236.7**	247.3	**85.1**	88.9
7th Floor	**220.3**	467.6	**79.2**	168.1
6th Floor	**187.5**	655.1	**67.4**	235.6
5th Floor	**154.9**	810.0	**55.7**	291.3
4th Floor	**122.5**	932.5	**44.1**	335.3
3rd Floor	**91.0**	1023.5	**32.7**	368.0
2nd Floor	**65.7**	1089.2	**23.6**	391.7

30% N/S & 100% E/W				
Level	North/South (Y)		East/West (X)	
	Forces (K) Fx	Story Shear (K) Vx	Forces (K) Fx	Story Shear (K) Vx
Penthouse Roof	**3.2**	3.2	**12.7**	12.7
Main Roof	**71.0**	74.2	**283.7**	296.4
7th Floor	**66.1**	140.3	**264.1**	560.5
6th Floor	**56.2**	196.5	**224.7**	785.2
5th Floor	**46.5**	243.0	**185.7**	970.8
4th Floor	**36.8**	279.8	**146.8**	1117.7
3rd Floor	**27.3**	307.1	**109.1**	1226.8
2nd Floor	**19.7**	326.8	**78.7**	1305.5

Appendix C: Gravity Calculations

Beams: 1. D-Beam



Kyle Tennant Tech. 2 ①

8" Precast Plank with 2" Topping + DB 9x46

Plank DL = 63 psf
Superimposed DL = 30 psf
Topping = 25 psf
Plank f'_c = 5 ksi
Grout f'_c = 4 ksi
8" Hollow Core Plank Spanning 30'-6"
D-Beam 9x46 spanning 16'

↓
Properties

once acts composite

Steel Section	Transformed Section
$I_{x_s} = 159$ in^4	$I_{x_t} = 332$ in^4
$S_t = 24.4$ in^3	$S_t = 62.1$ in^3
$S_b = 51.0$ in^3	$S_b = 77.7$ in^3
	$b = 5.25$
$M_{Allow} = 61.0$ kft	$t_w = .375$

Live Load = 40 psf

$$LL\ reduction = 40\left(.25 + \frac{15}{\sqrt{2(30.6 \times 16)}}\right)$$

$$= 29.2\ psf$$

(construction) Interior Girder G1

Initial Load (before composite)

DB 9x46 needed for Δ_{DL}

$$M_{DL} = \frac{(30.5)(.06\ ksf\ \ ft)(15^2)}{8} = 51.5\ kft < 84\ kft \checkmark$$

Deflection

$$\Delta_{DL} = \frac{(5)(30.5)(.06)(15^4)(1728)}{384\,(159)(29000)} = .45'' < \frac{15(12)}{360} = .5'' \checkmark$$

Total Load (once composite)

$$M_{supperimposed} = \frac{(30.5)(.03+.029+.025)(15^2)}{8} = 72.1\ kft$$

$$M_{Total\ Load} = 51.5 + 72.1 = 123.6\ kft$$

$$S_{Required} = \frac{(123.6)(12\ in/ft)}{.6\,(50\ ksi)} = 49.4\ in^3 < 62.1\ in^3 \checkmark$$

$$\Delta_{supperimposed} = \frac{(5)(30.5)(.03+.029+.025)(15^4)(1728)}{384(356)(29000)} = .30'' < .50'' \checkmark$$

Kyle Tennant Tech 2 ②

Composite Steel and Precast Plank continued

Check Compressive Stress on Concrete

→ transformed steel section must be converted to concrete section

$N \text{ value} = \frac{E_{steel}}{E_{conc.}} = \frac{29{,}000}{57{,}000\sqrt{4000}} = 8.04 \therefore S_{t_c} = 8.04(62.1) = 499.3 \text{ in}^3$

$f_c = \frac{72.1(12)}{499.3} = 1.73 \text{ ksi} \quad F_c = .45(4\text{ksi}) = 1.8 > 1.73$ ✓

Check Bottom Flange Tension Stress (check for total load)

$f_b = \frac{51.5(12)}{\underset{\uparrow \ DL}{51}} + \frac{72.1(12)}{\underset{\uparrow \ SupL}{77.7}} = 12.1 + 11.1 = 23.2 \text{ ksi}$

allowable → $F_b = .9(50) = 45 \text{ksi} > 23.2 \text{ksi}$ ✓

Check Shear

Load = 60 + 30 + 29.2 + 25 = 144.2 psf = .144 klf

$W = .144(30.5) = 4.4 \text{ klf}$

$R = \frac{4.4(16)}{2} = 35.2 \text{K}$

$f_v = \frac{35.2}{.375(5.75)} = 16.3 \text{ksi} < F_v = .4(50) = 20 \text{ ksi}$ ✓

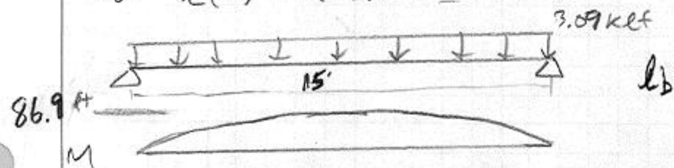
Exterior Girder G2

Loading

DL: $(63 + 30 + 25)(\frac{30.5}{2}) = 1.8 \text{ klf}$

LL: 40psf reduced → $40(.25 + \frac{15}{\sqrt{2(15 \times \frac{30.5}{2})}}) = 38.1 \text{psf}(\frac{30.5}{2}) = .581 \text{ klf}$

$w_u = 1.2(1.8) + 1.6(.581) = 3.09 \text{ klf}$



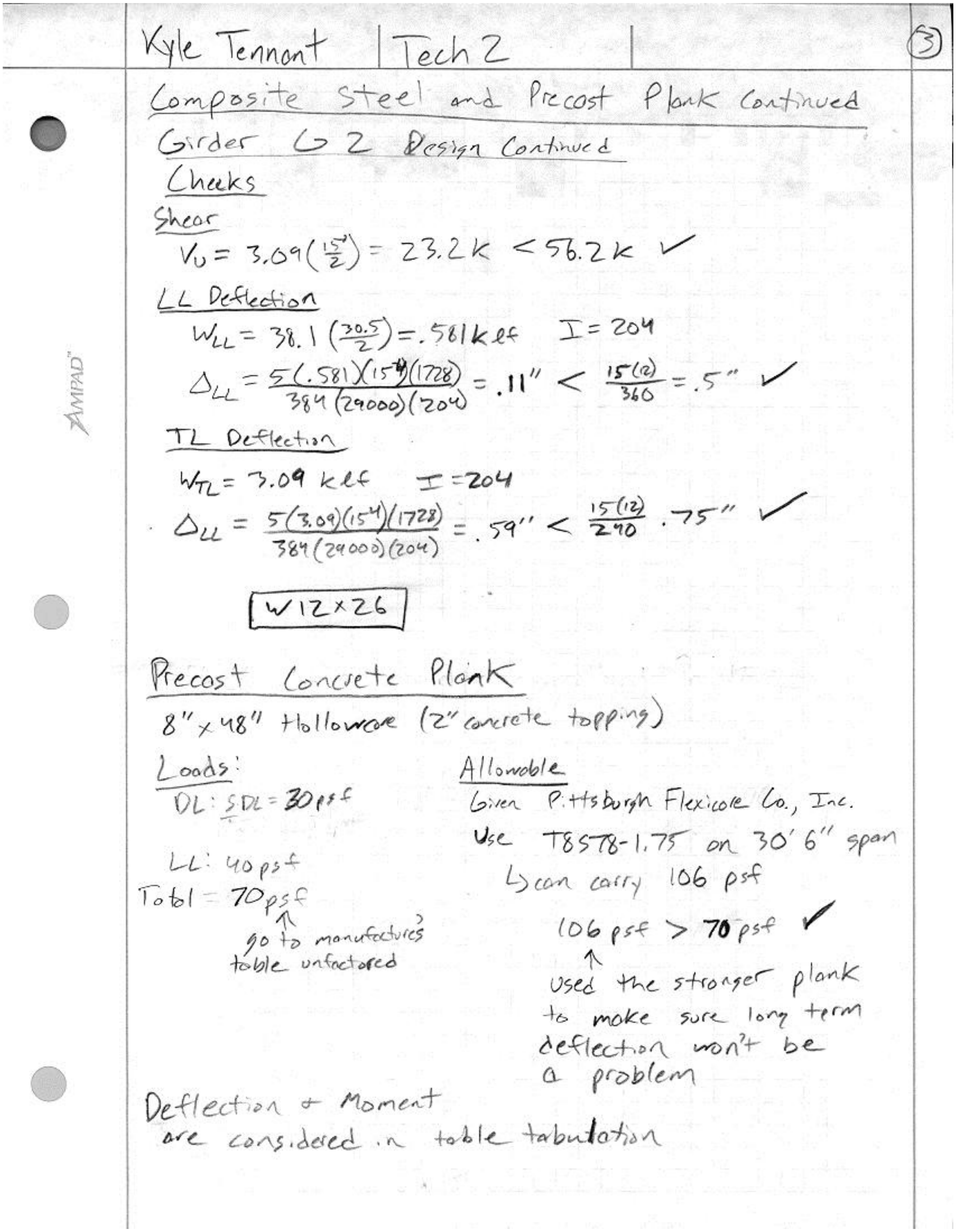
$M_u = \frac{3.09(15^2)}{8} = 86.9 \text{ k-ft}$

$l_b = 15'$ ↓ Table 3-10 15' unbraced

W12x26 → $\phi M_n = 87.7 \text{ ft-k}$ ✓

Table 3-2 $I = 204 \text{ in}^4$

→ Planks are resting on top flange, the connection is uncertain ∴ assumed to not be laterally braced to be conservative



Kyle Tennant | Tech 2 | ③

Composite Steel and Precost Plank Continued

Girder G2 Design Continued

Cheeks

Shear

$V_u = 3.09(\frac{15'}{2}) = 23.2k < 56.2k$ ✓

LL Deflection

$W_{LL} = 38.1(\frac{30.5}{2}) = .581 klf \quad I = 204$

$\Delta_{LL} = \frac{5(.581)(15^4)(1728)}{384(29000)(204)} = .11'' < \frac{15(12)}{360} = .5''$ ✓

TL Deflection

$W_{TL} = 3.09 klf \quad I = 204$

$\Delta_{LL} = \frac{5(3.09)(15^4)(1728)}{384(29000)(204)} = .59'' < \frac{15(12)}{240} .75''$ ✓

$\boxed{W12 \times 26}$

Precost Concrete Plank

8" × 48" Hollowcore (2" concrete topping)

Loads:

DL: SDL = 30psf

LL: 40 psf

Total = 70psf

↑ go to manufacturer's table unfactored

Allowable

Given Pittsburgh Flexicore Co., Inc.

Use T8S78-1.75 on 30' 6" span

↳ can carry 106 psf

106 psf > 70 psf ✓

↑ Used the stronger plank to make sure long term deflection won't be a problem

Deflection & Moment are considered in table tabulation

2. Exterior Beam

Beams (B)

Loading

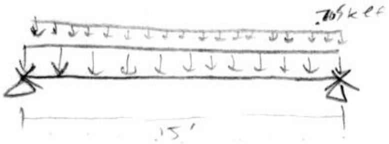
DL Plank → 63 psf
Topping → 25 psf
Superimposed → 30psf

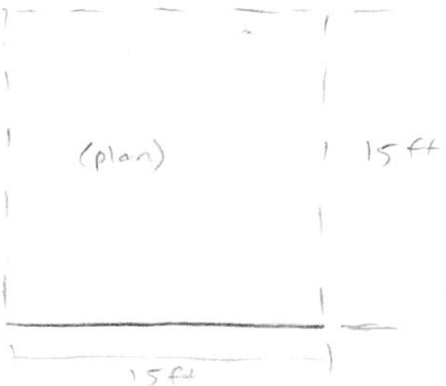
LL 40psf (reducible)

Wall load → 30psf × 9ft = .27 klf

Beam B1

laterally braced?





LLreduction

$$= 40\left(.25 + \frac{15}{\sqrt{2(15 \times 15)}}\right) = 38.3\text{ psf}$$

* Because these beams all support massonry facade they are subject to a more stringent deflection criteria of L/600

↳ This makes deflection likely to control in all exterior beam cases.

Load combo

$1.2D + 1.6L$

$$(63+25+30)1.2 + (38.3)1.6 = 202.88\text{ psf}$$

Make Linear Load

$$202.88\text{psf}(15\text{ft}) = 3.043\text{ klf}$$

$w_L = .919\text{ klf}$ $w_D = 2.12\text{ klf}$ $+.705(1.2)$

↳ try to keep beams to min height to minimally disturb existing building elevation

$$M_u = \frac{3.889(15^2)}{8} = 109.4\text{ ftk}$$ $L_b = 15'$

↓ Table 3-10

W14x30

Table 3-2

$\phi M_n = 172\text{ k-ft}$

$I = 291\text{ in}^4$

$\boxed{W18\times35}$

↳ Max desired depth of 18"

Allowable Deflection

$$\frac{L}{600} = \frac{15(12)}{600} = .3$$

LiveLoad Deflection

$$\Delta_{max_{LL}} = \frac{5\left(\frac{.919}{1.6}\right)(15^4)(12^3)}{384(29000)(291)} = .08'' < \text{Allowable } \frac{15(12)}{360} = .5'' \checkmark$$

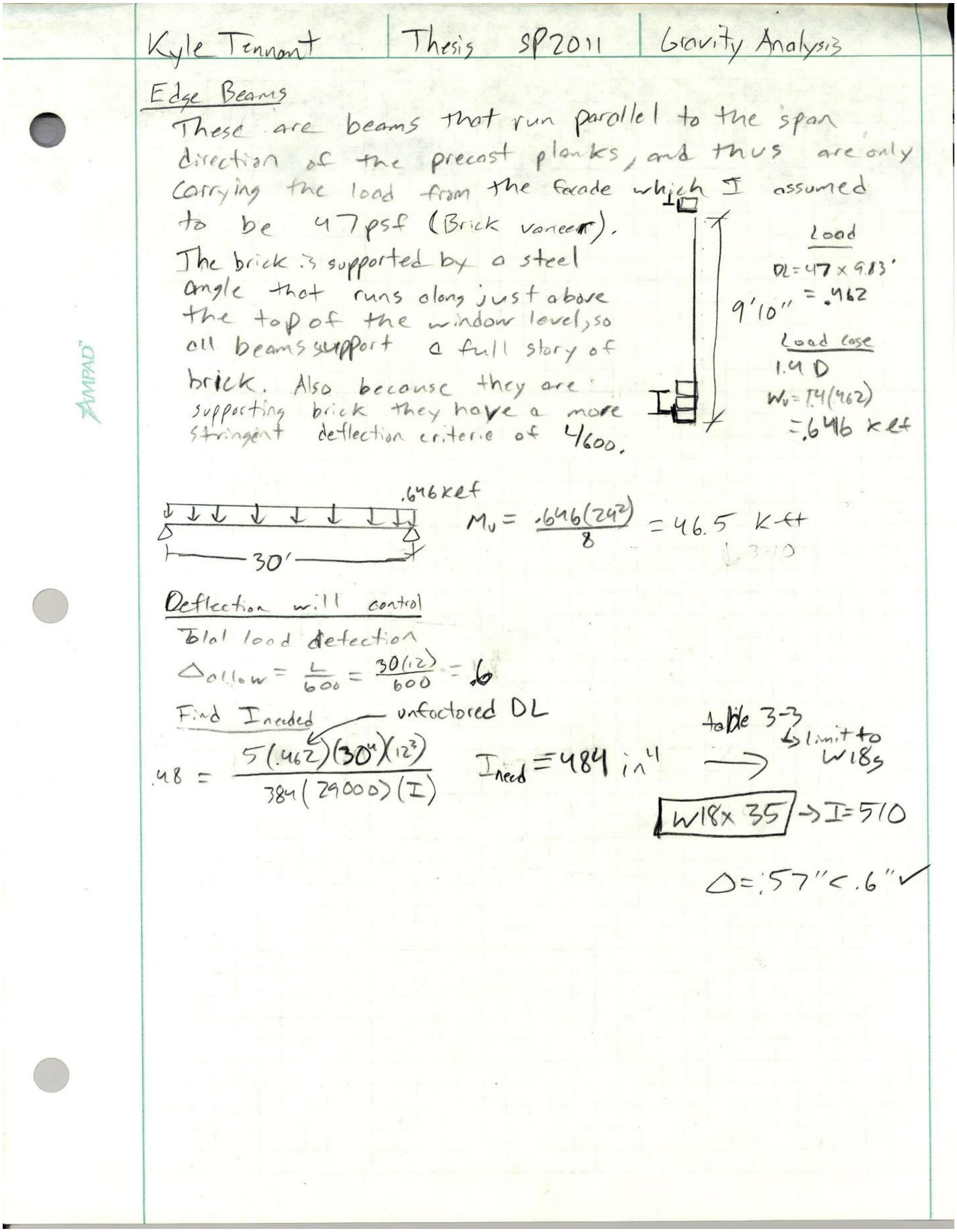
Total Load Deflection

$$\Delta_{max_{TL}} = \frac{5\left(\frac{.919}{1.6} + \frac{2.96}{1.2}\right)(15^4)(12^3)}{384(29000)(291)} = .52'' > .3''$$ $\Delta_{LL} = .23'' \checkmark$

Need $I = 399$ → W18x35 $I = 510$

Checked Dead Load deflection due to the fact that ~~the~~ majority of the load comes from dead load →

3. Edge Beam



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Edge Beams

These are beams that run parallel to the span direction of the precast planks, and thus are only carrying the load from the facade which I assumed to be 47psf (Brick vaneer).

The brick is supported by a steel angle that runs along just above the top of the window level, so all beams support a full story of brick. Also because they are supporting brick they have a more stringent deflection criteria of L/600.

9'10"

Load

DL = 47 × 9.83'
= .462

Load case

1.4 D

$W_u = 1.4(.462)$
$= .646$ klf

.646klf

30'

$M_u = \frac{.646(24^2)}{8} = 46.5$ k-ft

3-10

Deflection will control

Total load defection

$\Delta_{allow} = \frac{L}{600} = \frac{30(12)}{600} = .6$

Find Ineeded

unfactored DL

$.48 = \frac{5(.462)(30^4)(12^3)}{384(29000)(I)}$

$I_{need} = 484 \text{ in}^4$ $\longrightarrow$ table 3-3 → limit to W18s

$\boxed{W18 \times 35} \rightarrow I = 510$

$\Delta = .57'' < .6''$ ✓

Gravity Calculations – Column Design

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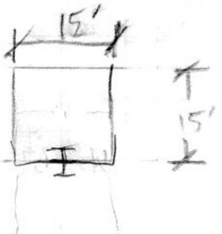
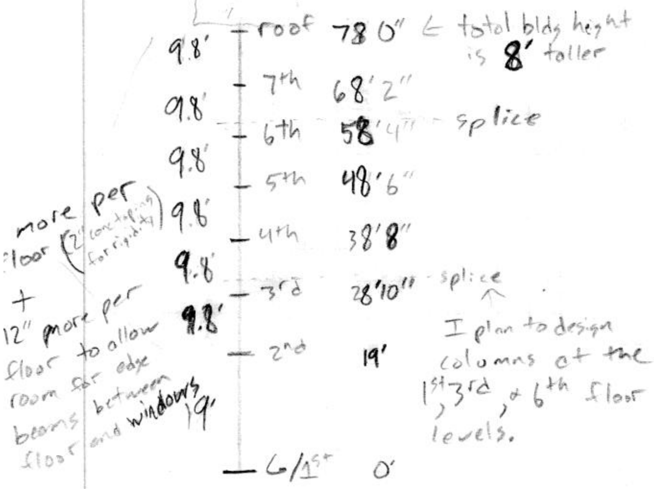
Columns → design of gravity columns

Column a → the most typical column

Tributary Area – 15 x 15 = 225 sq ft



1st Floor Level → holds 6 floors + 1 roof
3rd Floor Level → holds 4 floors + 1 roof
6th Floor Level → holds 1 floor + 1 roof



ASCE section 4.7.2

Loading

$LL_{red} = 40\left(.25 + \left(\frac{15}{\sqrt{4(6)(225)}}\right)\right) = .45 > .40 \Rightarrow$ use .45

$40(.45) = 18$ psf

Roof Live Load = 20psf ASCE 4.8.2

Trib Area = 450 ∴

$L_r = L_o R_1 R_2$

$R_1 = 1.2 - .001(450) = .75$

$R_2 = 1$ (flat roof

$LL_{red} = 20(.75)(1) = 15$ psf ≥ 12 psf ✓

Dead Load = (plank, topping, superimposed) = 118 psf + beam self weight + facade W14x30

Total Loads

$LL = 225(.018)(6) + 225(.015)(1)$

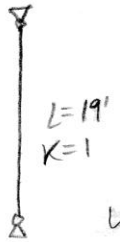
$= 27.7K$

$DL = \overset{185.9}{225(.118)(7)} + \overset{3.15k}{.03(15)(7)}$

$+ .047(15)\underset{55}{(78)} = 244$

Total Factored

$P_u = 244(1.2) + 27.7(1.6) = \underline{337.1K}$



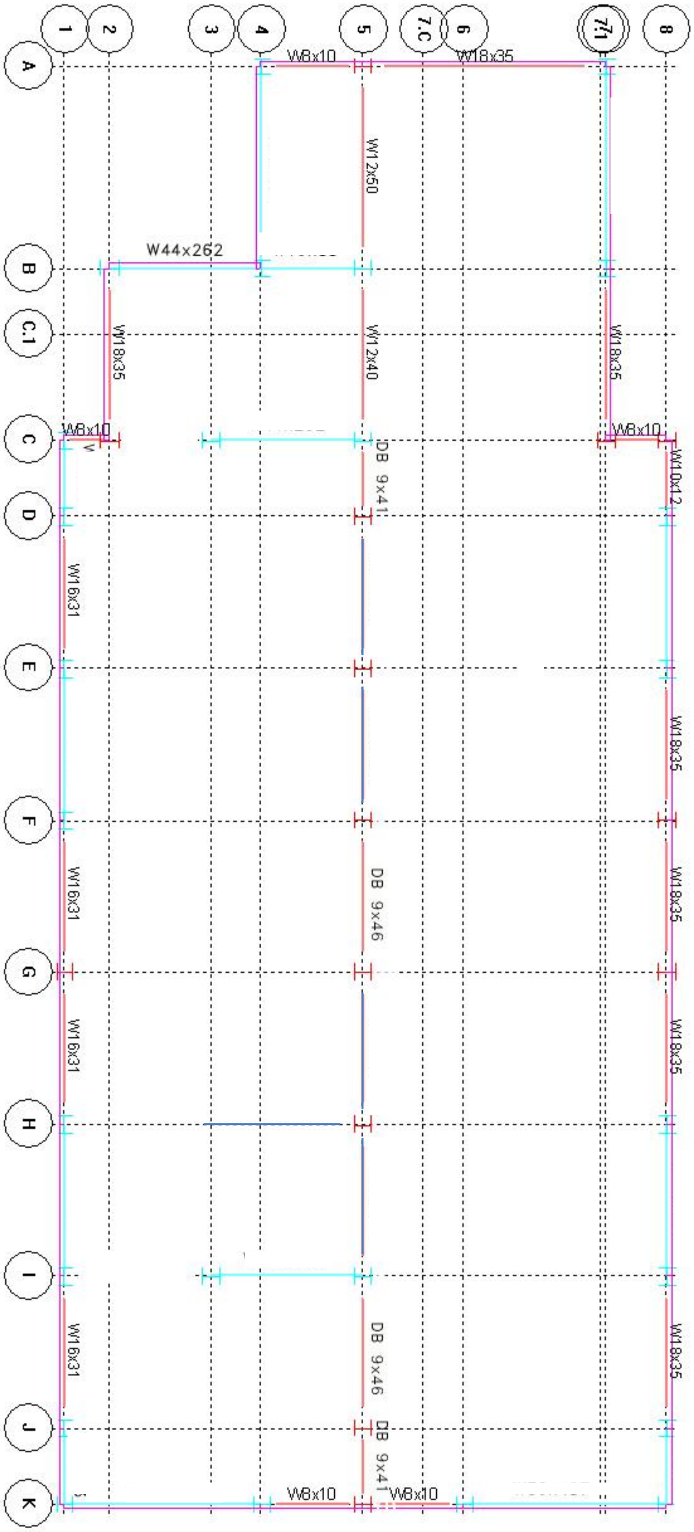
KL=19
$P_u = 337.1$ k
↓ Table 4-1
W10x49 $\phi P_n = 360k$
↑
Some room for PΔ effects

or

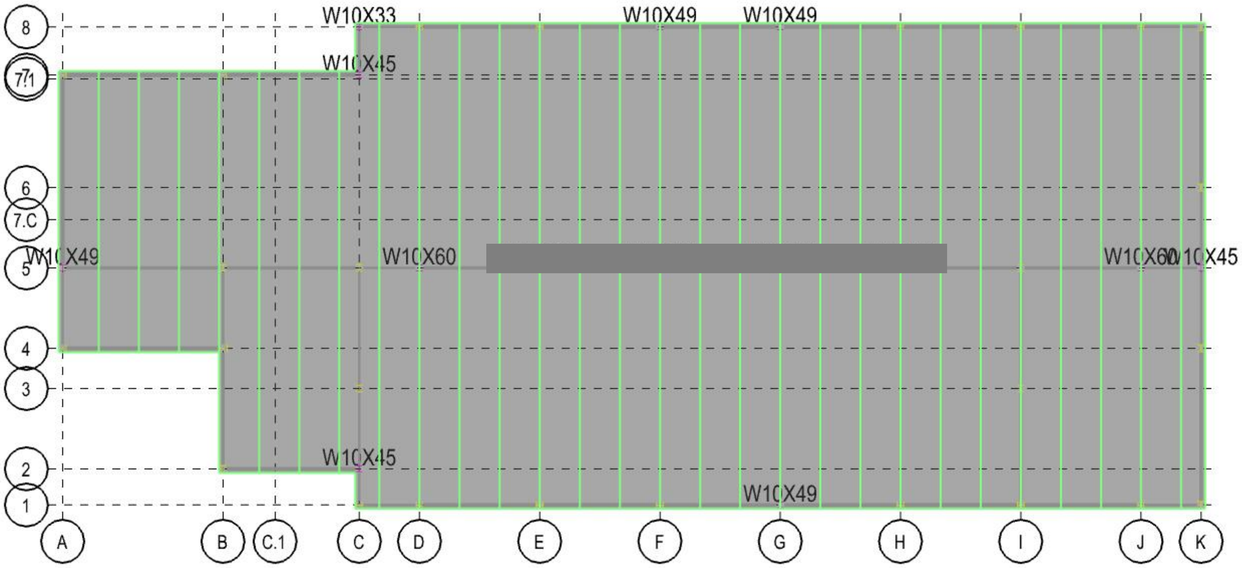
W10x54 $\phi P_n = 399$
↓
more room for PΔ effects

Appendix D: RAM Analysis

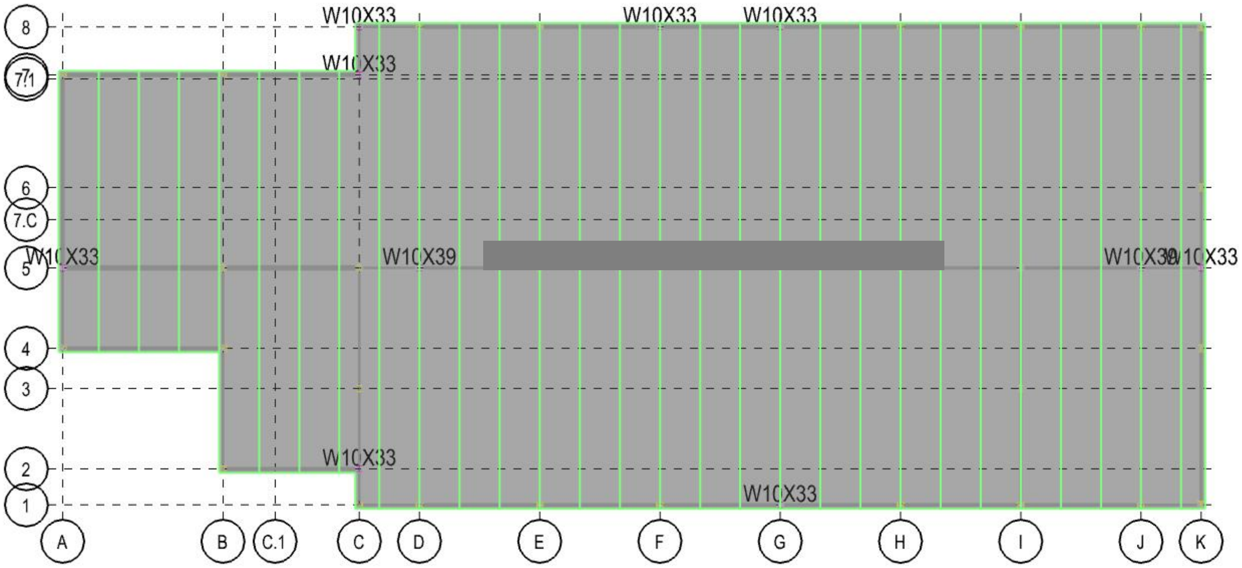
Left Wing – Beams (Typical)



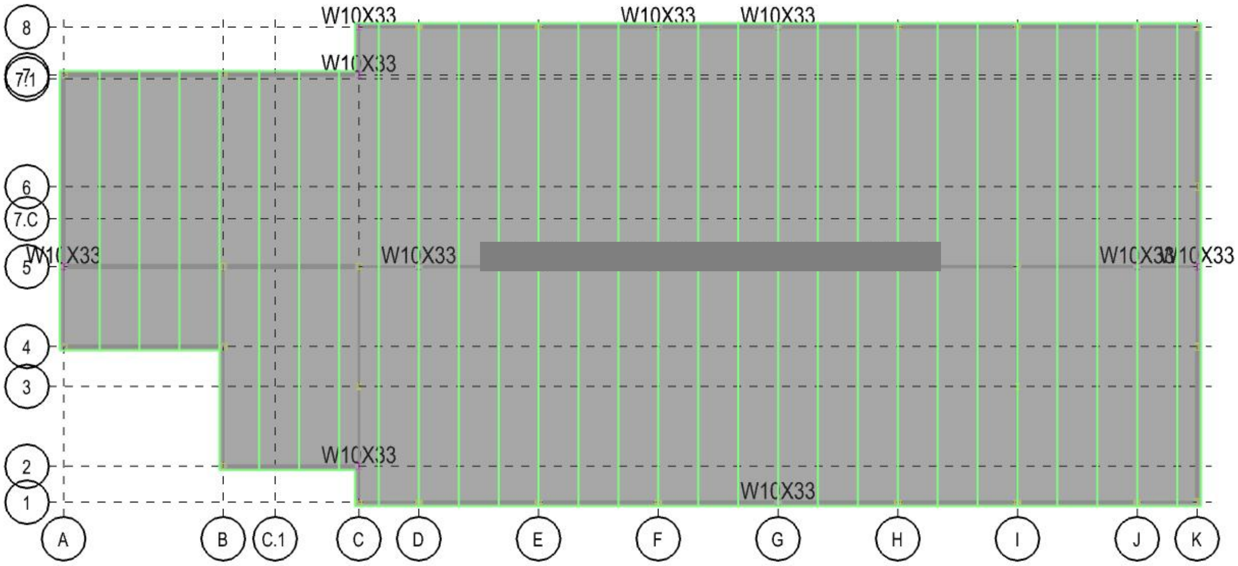
Left Wing – 1st and 2nd Floor Columns



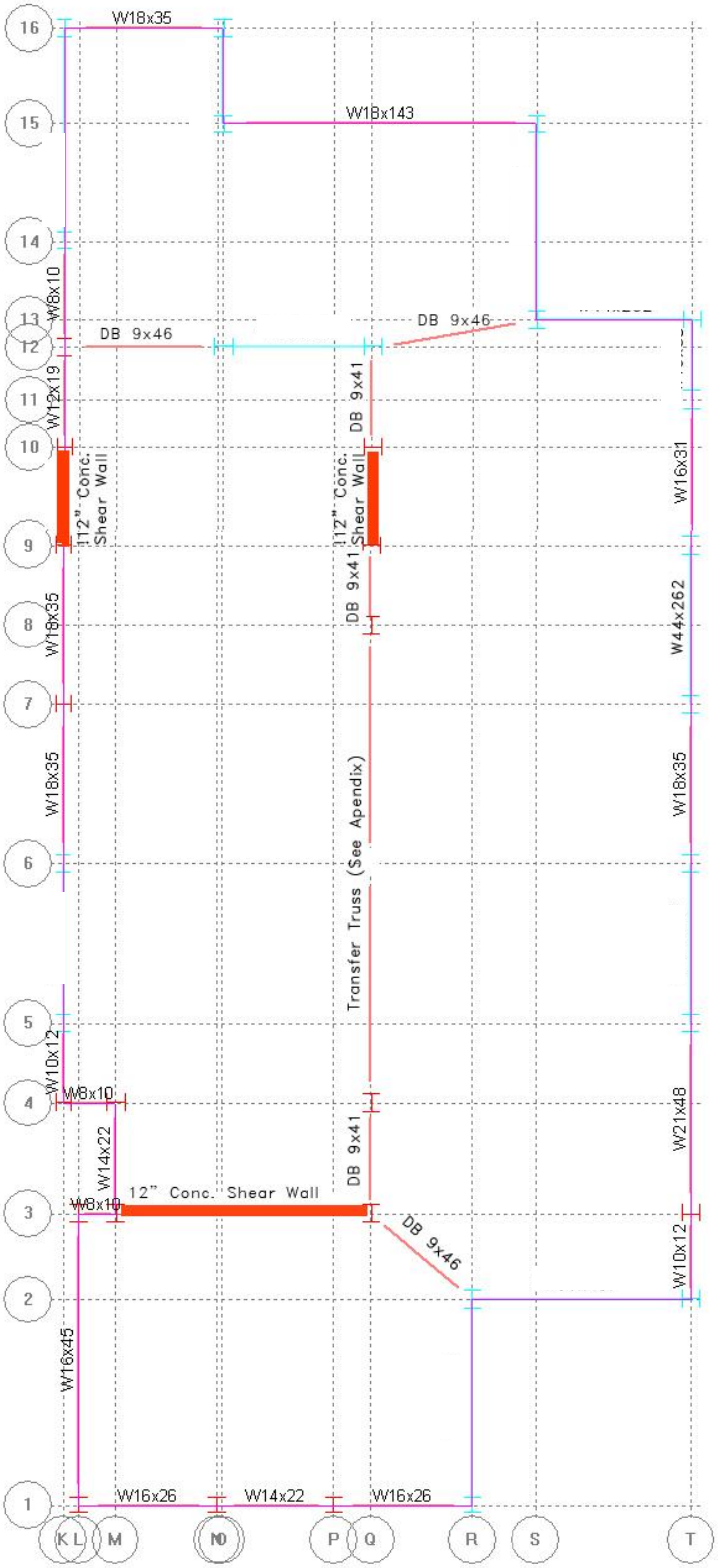
3rd – 5th Floor Columns



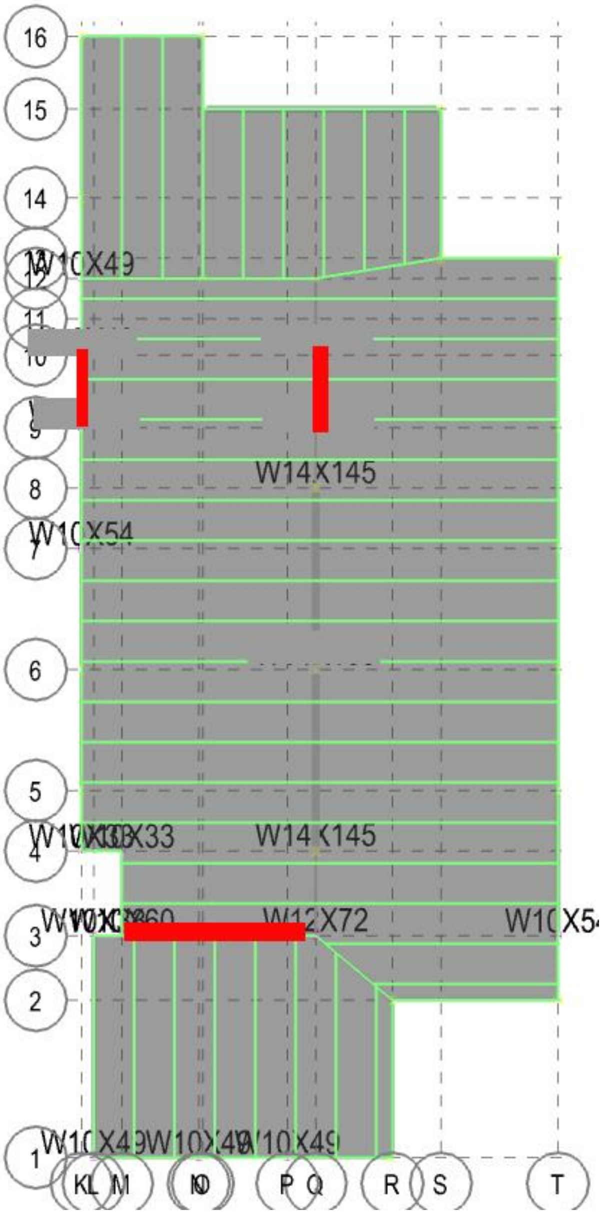
6th and 7th Floor Columns



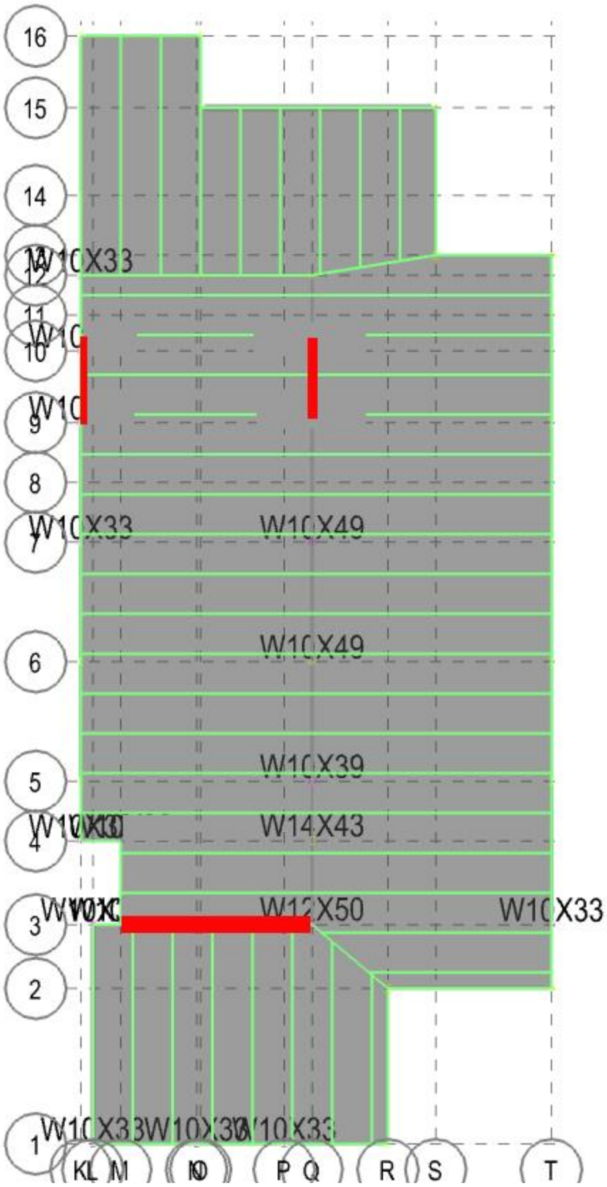
Right Wing – Beams (Typical)



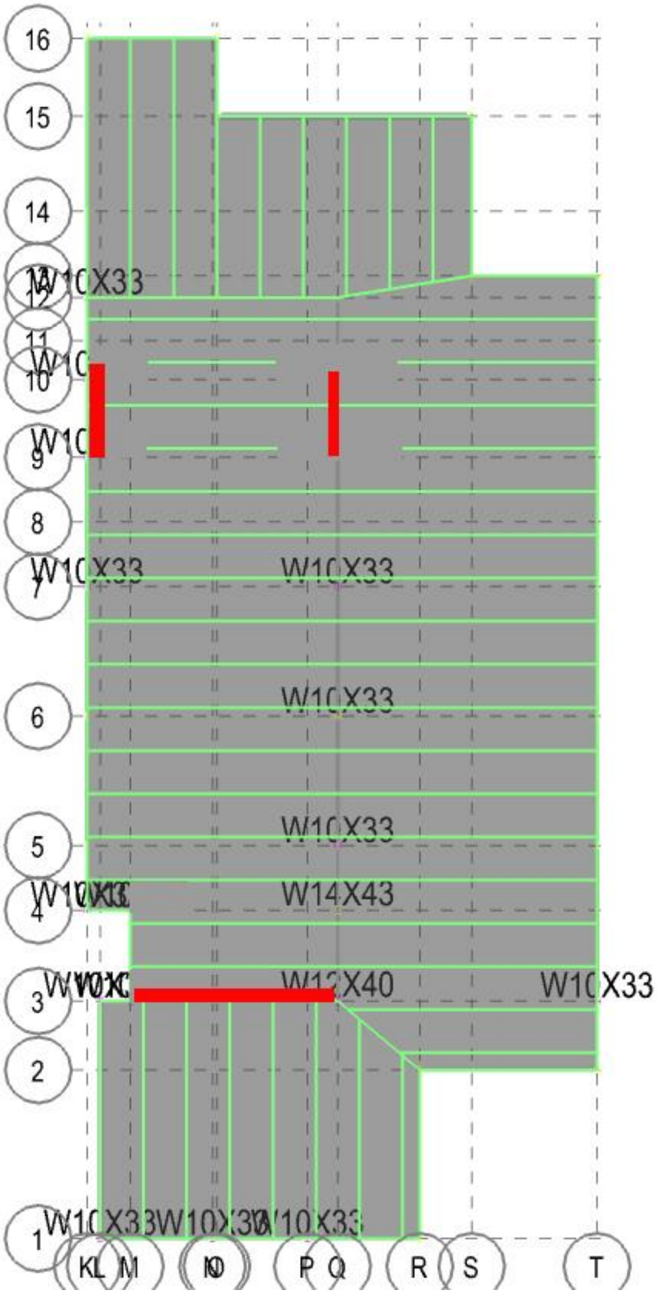
Right Wing – 1st Floor Columns



3rd – 5th Floors Columns (center line columns extend down to 2nd floor and bear on transfer truss)

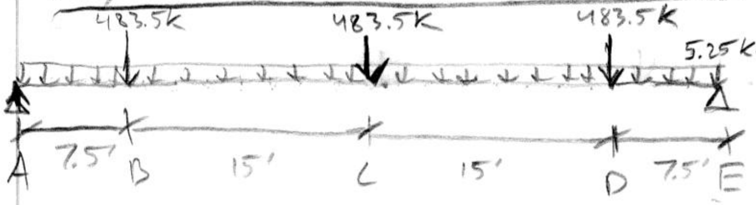


Right Wing – 6th and 7th Floor Columns

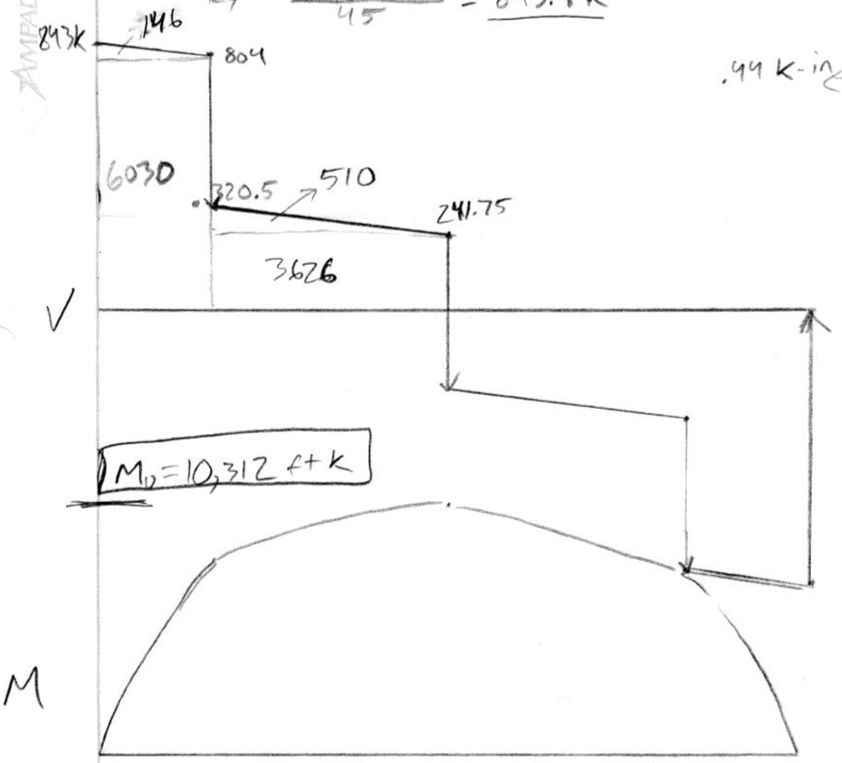


Appendix E: Transfer Truss Design

Right Wing Transfer Girder 1 (RWTG1)

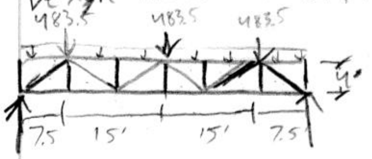


$\circlearrowleft^{+} \Sigma M_A = 483.5(7.5) + 483.5(22.5) + (5.25 \times 45)(22.5)$
$+ 483.5(37.5) - E_y(45)$
$E_y = \frac{37951.875}{45} = \underline{843.4\text{ k}}$



$\boxed{M_u = 10{,}312 \text{ ft k}}$

Design Truss in SAP



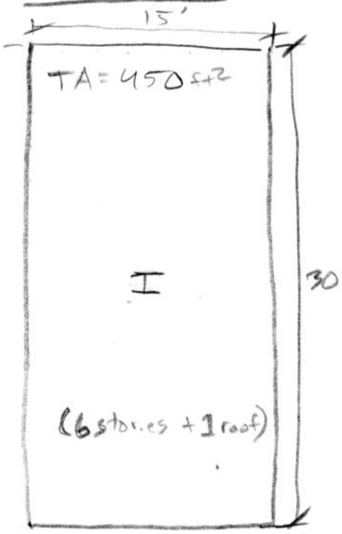
$\Delta_{allow} = \frac{45(12)}{240} = 2.25$
↑ total load

Ground Column E
↳ no upper stories
$P_u = 843.4\text{ K}$
$KL = .7(19) = 13.3 = 14$
↓ 4-1
$\boxed{W12 \times 96}$ $\phi P_n = 1020\text{ k}$

Ground Column A
$P_u = 843 + 483. = 1326\text{ k}$
$\boxed{W12 \times 136}$ → $\phi P_n = 1420\text{ k}$ ✓

Distributed Load
DL = 118 psf
LL = 40 psf
$LL_{red} = 40\left(.25 + \left(\frac{15}{\sqrt{2(30)(45)}}\right)\right)$
= 21.5 > 50% ✓
Trib width = 30 ft
$W_D = 1.2(118)(30) = 4.25\text{ klf}$
$W_L = 1.6(21.5)(30) = 1.03\text{ klf}$
.44 k-in ← $\underline{W_u = 5.28\text{ klf}}$

Point Loads



$LL_{red} = 40\left(.25 + \left(\frac{15}{\sqrt{4(450)}}\right)\right)$
= 24.1 > 40% ✓
$W_D = 1.2(118(6)(450))$
= 382.3 K
$W_L = 1.6(24.1(5)450)$
+1.6(20(1)(450))
= 101.2 K
$\underline{W_u = 483.5\text{ K}}$

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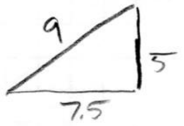
Transfer Truss Design

Brace 1 → braces considered pinned and to tak axial loads → compresion

From SAP → $P_u = 1178$ k Table 4-3 $\phi P_n = 1210$ K ✓

KL = 9'



Brace 2 $P_u = 88.6$ K HSS 12x8x½ $\phi P_n = 659$ K ✓

Brace 3 $P_u = 572.8$ K HSS 12x8x½ $\phi P_n = 659$ k ✓

Brace 4 $P_u = 77.6$ K

Brace 5 $P_u = 371.$ K

Brace 6 $P_u = 56.2$ K

Top Beam mainly in Compression

Middle Axial → 1457.09 K
Bending → 343 ft-K

End Axial → 55.
Bending → 509 ftK

table 6-1 → full ~~lateral~~ braced

W12x190 → $p = .398\times10^{-3}$ $b_x = .768\times10^{-3}$

$.398\times10^{-3}(1457) = .579$ → H1-1a → $.579 + (.768\times10^{-3})(343) = .84 < 1$ ✓

Bottom Beam mainly in tension

middle Axial → 1708 K
Bending → 370 ftK

End Axial → 923 K
Bending → 467 ftK

Tension & Flexure

$L_b = 7.5'$

5-1 W12x190 $\phi P_n = 2510$ $\phi M_n = 910$ → $\left(\frac{1708}{2510}\right) + \left(\frac{370}{910}\right) = 1.08 > 1.0$ X

W12x210 $\phi P_n = 2780$ $\phi M_n = 1040$ → $\left(\frac{1708}{2780}\right) + \left(\frac{370}{1040}\right) = .97 < 1.0$ ✓

$\boxed{\text{W12x210}}$ Bottom Cord

Kyle Tennant | Thesis SP 2011 | Gravity Analysis

Transfer Truss Design

Column Design

Moment = 476 ft·k

Axial = 843.4K (hand calc... didn't think model of frame was correct)

↓6-1 W12×136 → $p = .684 \times 10^{-3}$ $b_x = 1.13 \times 10^{-3}$

KL = 14'

$.684 \times 10^{-3}(843) = .54 > .2$ → H1-1a

↓

$.54 + (1.13 \times 10^{-3})(476) = 1.08 > 1.0$ ✗

Try W14×132 $p = .663 \times 10^{-3}$ $b_x = 1.02 \times 10^{-3}$

H1-1a = 1.04 > 1.00 ✗

Try W14×145 $p = .593 \times 10^{-3}$ $b_x = .912 \times 10^{-3}$

H1-1a = .93 < 1.0 ✓

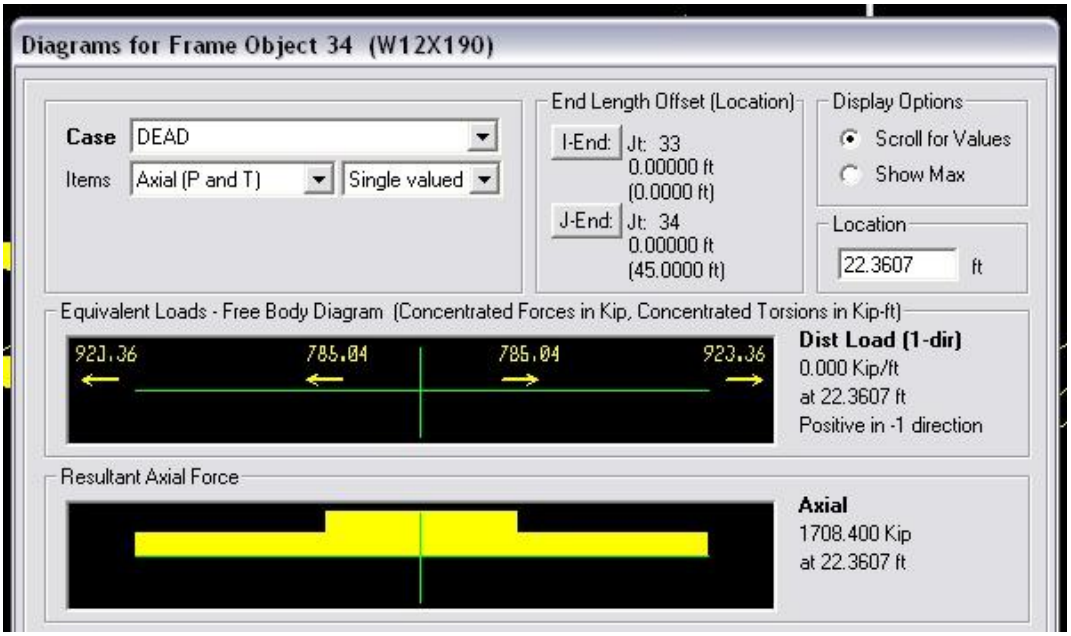
$\boxed{\text{W14×145}}$

Deflection

$\Delta_{TL\,max} = 1.64'' < \Delta_{TL\,Allow} = \frac{45(12)}{240} = 2.25''$ ✓

↑
at center

Bottom Beam



Diagrams for Frame Object 34 (W12X190)

Case: DEAD
Items: Axial (P and T) Single valued

End Length Offset (Location)
I-End: Jt: 33 0.00000 ft (0.0000 ft)
J-End: Jt: 34 0.00000 ft (45.0000 ft)

Display Options
● Scroll for Values
○ Show Max

Location: 22.3607 ft

Equivalent Loads - Free Body Diagram (Concentrated Forces in Kip, Concentrated Torsions in Kip-ft)

Dist Load (1-dir)
0.000 Kip/ft
at 22.3607 ft
Positive in -1 direction

Resultant Axial Force

Axial
1708.400 Kip
at 22.3607 ft

Diagrams for Frame Object 34 (W12X190)

Case: DEAD
Items: Major (V2 and M3) Single valued

End Length Offset (Location)
I-End: Jt: 33 0.00000 ft (0.0000 ft)
J-End: Jt: 34 0.00000 ft (45.0000 ft)

Display Options
● Scroll for Values
○ Show Max

Location: 22.5000 ft

Equivalent Loads - Free Body Diagram (Concentrated Forces in Kip, Concentrated Moments in Kip-ft)

Dist Load (2-dir)
0.190 Kip/ft
at 22.5000 ft
Positive in -2 direction

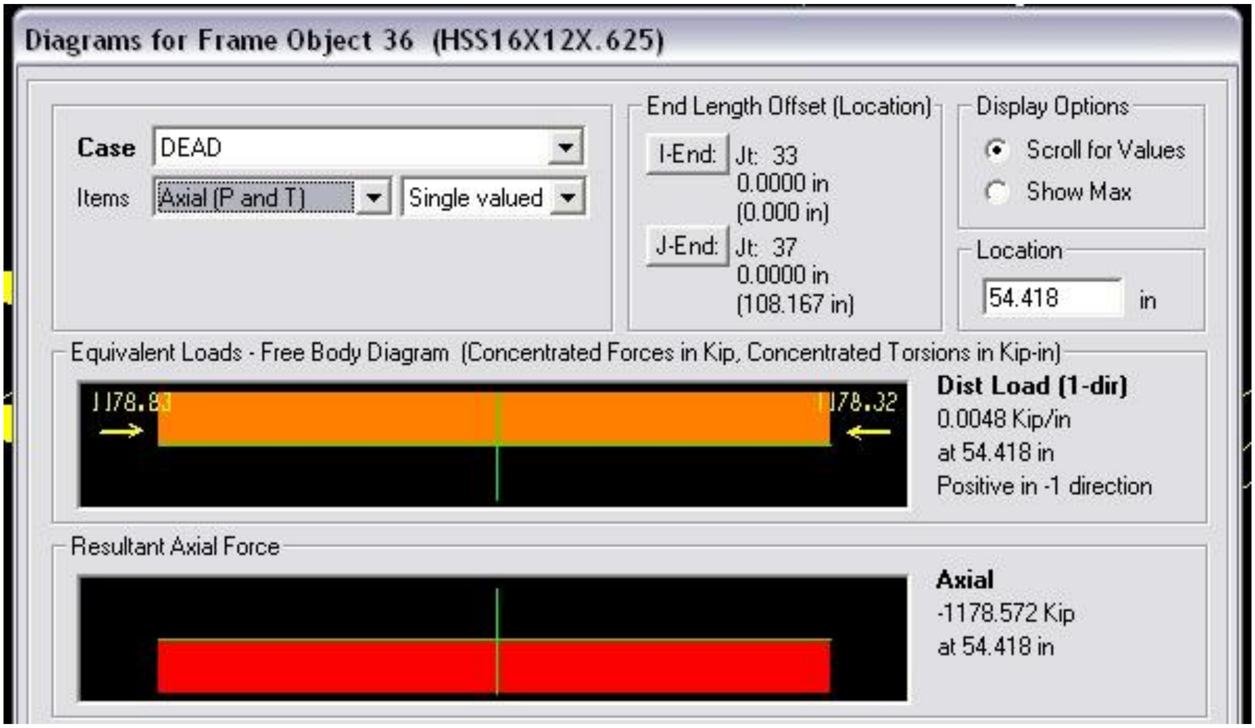
Resultant Shear

Shear V2
-28.105 Kip
at 22.5000 ft

Resultant Moment

Moment M3
370.6162 Kip-ft
at 22.5000 ft

Brace 1 (Maximum loaded HSS 16x12x.625 brace)



Diagrams for Frame Object 36 (HSS16X12X.625)

Case DEAD
Items Axial (P and T) Single valued

End Length Offset (Location)
I-End: Jt: 33 0.0000 in (0.000 in)
J-End: Jt: 37 0.0000 in (108.167 in)

Display Options
● Scroll for Values
○ Show Max

Location 54.418 in

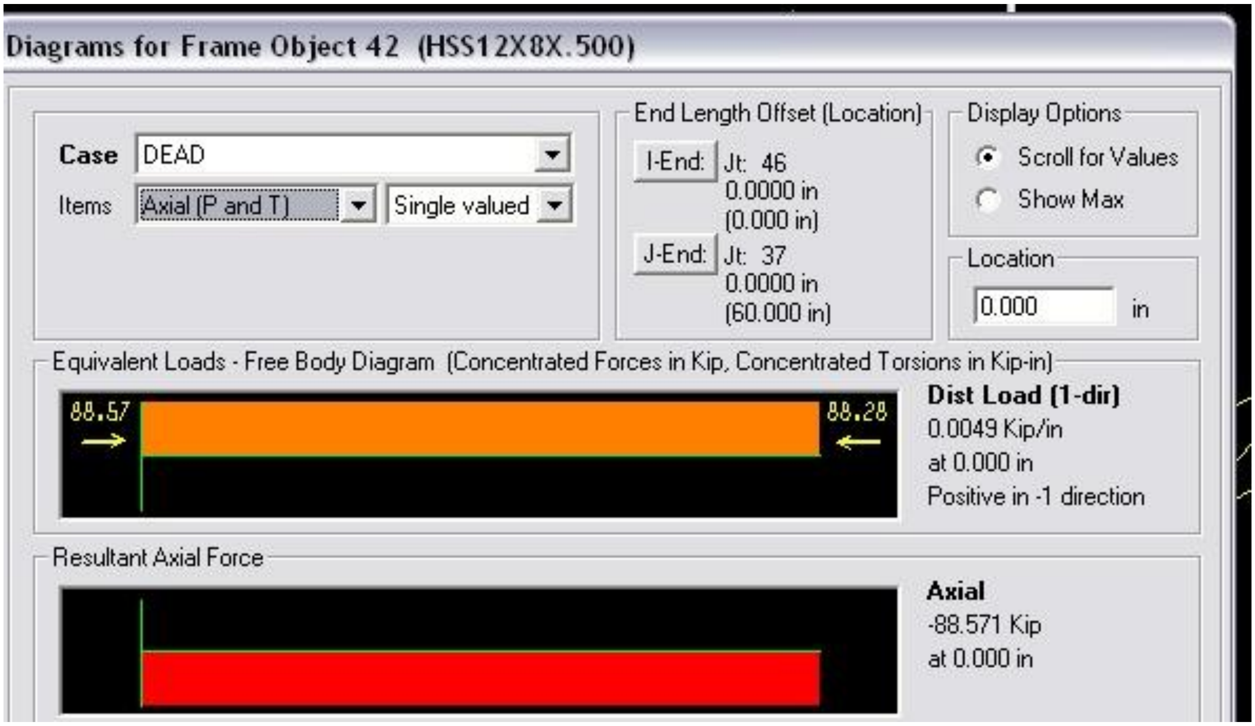
Equivalent Loads - Free Body Diagram (Concentrated Forces in Kip, Concentrated Torsions in Kip-in)

Dist Load (1-dir)
0.0048 Kip/in
at 54.418 in
Positive in -1 direction

Resultant Axial Force

Axial
-1178.572 Kip
at 54.418 in

Brace 2 (Maximum vertical loaded HSS 12x8x.5 brace)



Diagrams for Frame Object 42 (HSS12X8X.500)

Case DEAD
Items Axial (P and T) Single valued

End Length Offset (Location)
I-End: Jt: 46 0.0000 in (0.000 in)
J-End: Jt: 37 0.0000 in (60.000 in)

Display Options
● Scroll for Values
○ Show Max

Location 0.000 in

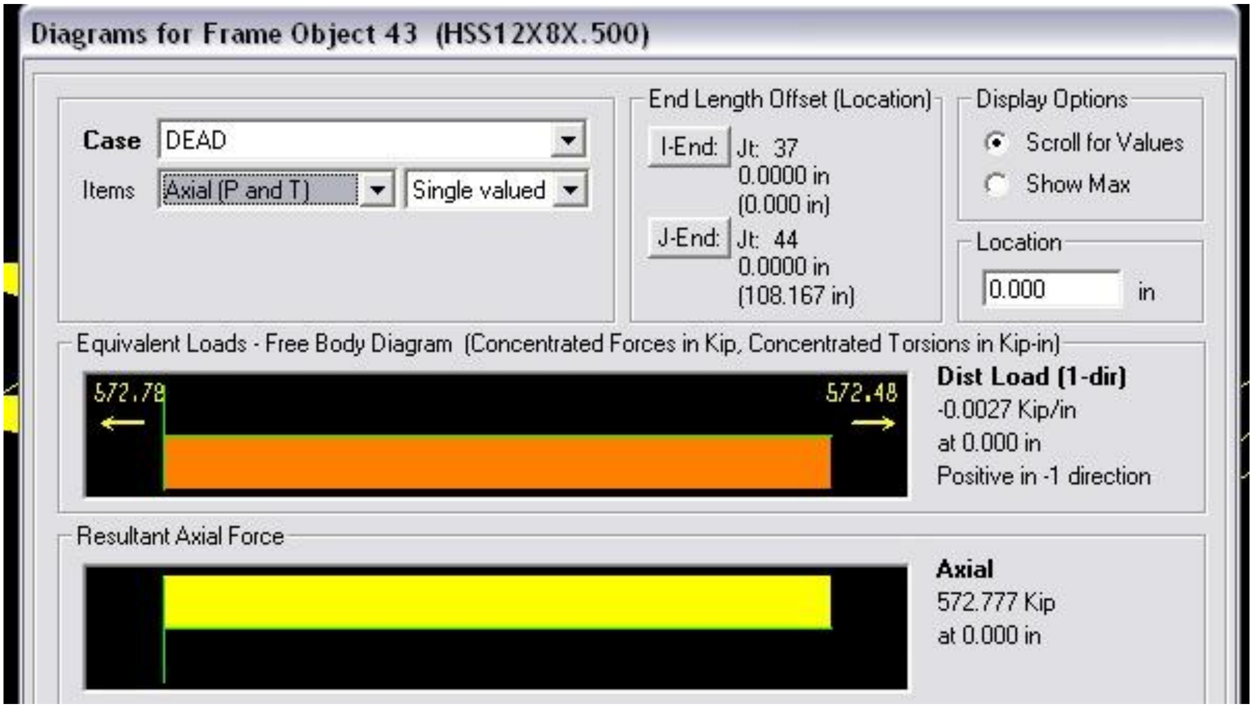
Equivalent Loads - Free Body Diagram (Concentrated Forces in Kip, Concentrated Torsions in Kip-in)

Dist Load (1-dir)
0.0049 Kip/in
at 0.000 in
Positive in -1 direction

Resultant Axial Force

Axial
-88.571 Kip
at 0.000 in

Brace 3 (Maximum loaded diagonal HSS 12x8x.5 brace)



Diagrams for Frame Object 43 (HSS12X8X.500)

Case DEAD
Items Axial (P and T) Single valued

End Length Offset (Location)
I-End: Jt: 37
0.0000 in
(0.000 in)
J-End: Jt: 44
0.0000 in
(108.167 in)

Display Options
● Scroll for Values
○ Show Max

Location
0.000 in

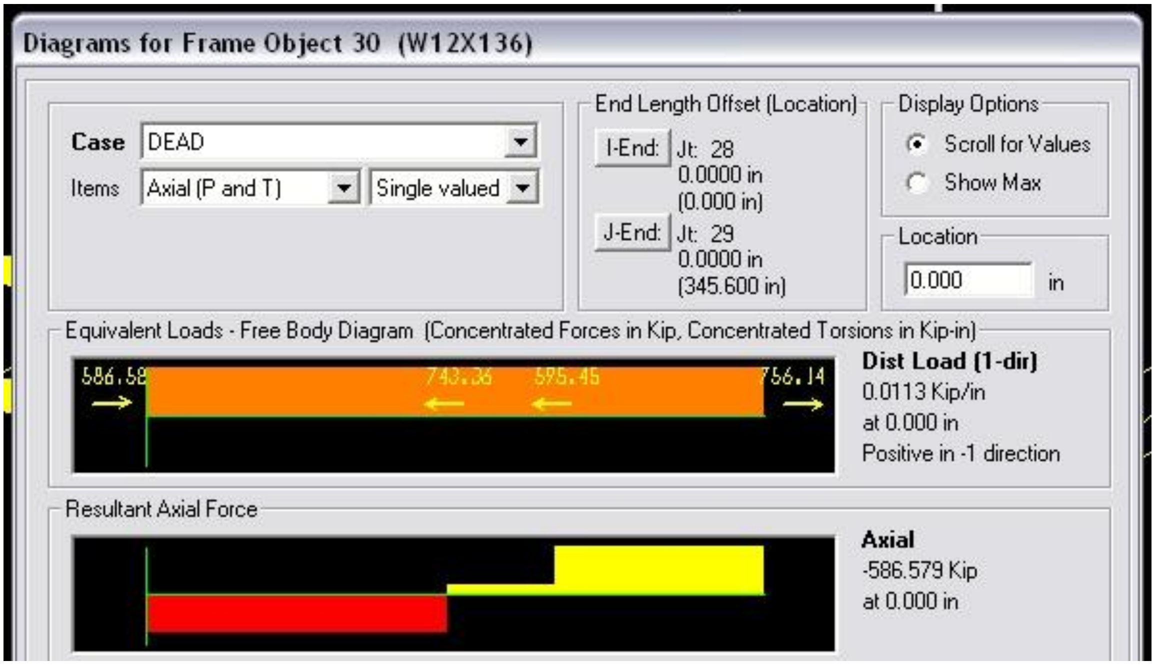
Equivalent Loads - Free Body Diagram (Concentrated Forces in Kip, Concentrated Torsions in Kip-in)

Dist Load (1-dir)
-0.0027 Kip/in
at 0.000 in
Positive in -1 direction

Resultant Axial Force

Axial
572.777 Kip
at 0.000 in

Column



Diagrams for Frame Object 30 (W12X136)

Case DEAD
Items Axial (P and T) | Single valued

End Length Offset (Location)
I-End: Jt: 28
0.0000 in
(0.000 in)
J-End: Jt: 29
0.0000 in
(345.600 in)

Display Options
● Scroll for Values
○ Show Max

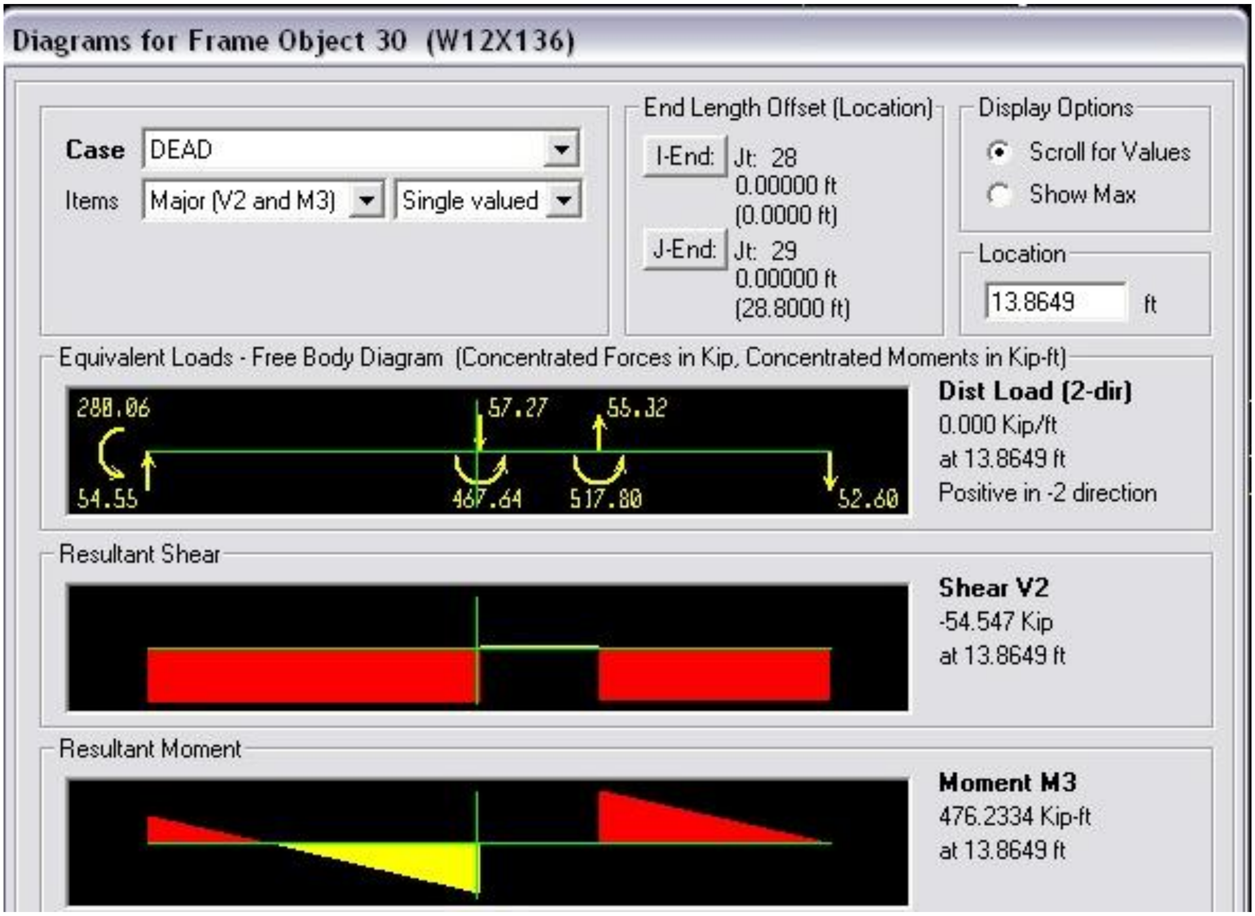
Location
0.000 in

Equivalent Loads - Free Body Diagram (Concentrated Forces in Kip, Concentrated Torsions in Kip-in)

Dist Load (1-dir)
0.0113 Kip/in
at 0.000 in
Positive in -1 direction

Resultant Axial Force

Axial
-586.579 Kip
at 0.000 in



Diagrams for Frame Object 30 (W12X136)

Case DEAD
Items Major (V2 and M3) | Single valued

End Length Offset (Location)
I-End: Jt: 28
0.00000 ft
(0.0000 ft)
J-End: Jt: 29
0.00000 ft
(28.8000 ft)

Display Options
● Scroll for Values
○ Show Max

Location
13.8649 ft

Equivalent Loads - Free Body Diagram (Concentrated Forces in Kip, Concentrated Moments in Kip-ft)

Dist Load (2-dir)
0.000 Kip/ft
at 13.8649 ft
Positive in -2 direction

Resultant Shear

Shear V2
-54.547 Kip
at 13.8649 ft

Resultant Moment

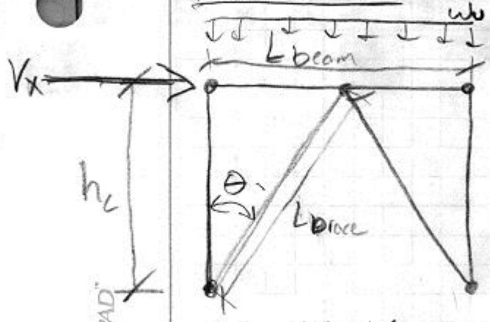
Moment M3
476.2334 Kip-ft
at 13.8649 ft

Appendix F: Braced Frame Design (MAE Coursework)

Frame F | Kyle Tennant | Thesis SP 2011 | MAE Braced Frame Design ①

→ Using strength Design → Following example in Seismic Design Manual

Frame F → Special Concentric Braced Frame



Inverted V-Type

Pin connections (Shear) →
↳ cheaper than connections for moment frames

→ want to design braces to yield and beams and columns to remain elastic
→ brace member plastically dissipates seismic energy
→ not designing connections

Brace → inelastic
→ check slenderness of $\frac{KL}{r} \leq \frac{110}{\sqrt{F_y}}$ ~~$4\sqrt{E/F_y}$~~
→ check width-to-thickness ratio of brace
↳ limit $\frac{b_f}{t_f}$ & $\frac{h}{t_w}$ to prevent buckling

Columns → remain elastic
→ T/C capacity $= 1.1 \overset{\swarrow 1.0}{R_y} F_y A_g$
→ $\frac{b}{t}$ limit for buckling

	1st Story	Upper Stories (ft)
H_c	19.0	9.8
L_{beam}	20.0	20.0
L_{brace}	21.5*	14.0
θ	28°	46°
V_x	Varies Per story	
W_u	.647 klf	.647 klf ← only suppports brick facade

* Could be braced at 10.7ft if necessary

Beams → remain elastic
→ design for entire tributary gravity load not considering braces
→ design for unbalanced brace load
↳ T > C → downward force
↳ 100% T_{yield} + 30% P_{cr}

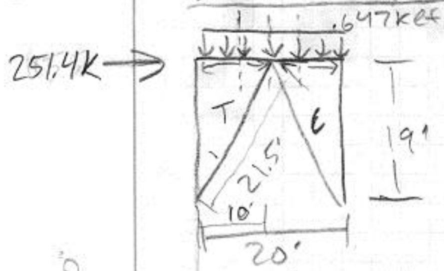
SDC: D $S_{DS} = 1$ (from seismic calc)

SCBF → $R = 6$
$\Omega = 2$
$C_d = 5$
$\rho = 1.0$ → sec. 12.3.4.2 redundancy needs met
$R_y = 1.6$ (Inverted V)

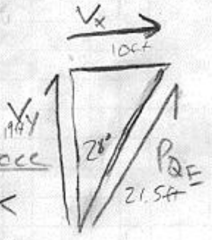
Frame F | Kyle Tennant | Thesis SP 2011 | MAE Braced Frame ②

1st Story

Brace design



$\frac{P_{Q_E}}{V_x} = \frac{21.5'}{10'}$



Find axial forces on brace

$P_{Q_E} = \left(\frac{21.5}{10}\right)\frac{269.7}{2} = 289.5K$

$P_D = \left(\frac{21.5}{19}\right)(5(.462)) = 2.6K$ ← unfactored

Find Maximum Compressive For in Brace

$P_u = (1.2 + .2S_{DS})D + \rho Q_E + L + .2S$

$P_u = 1.4(2.6) + 289.5 = 293.2K$ ← compression

L = 21.5' (unbraced length of brace)

K = 1 (pin pin)

Large loads due to the unfavorable 28° angle

Find Maximum Tension In Brace

$T_u = (.9 - .2S_{DS})D + \rho Q_E + 1.6H$

$T_u = (.7)(2.6) + (-289.5) = 287.7K$

Pick a member for Brace

→ Want a square HSS member so there is no weak axis → but want to keep 1 axis under 10 inches to easily fit inside of column flanges.

→ HSS members work well as braces

Try HSS 8x8x.5 $A_g = 13.5 \text{ in}^2$ $r = 3.04$

compression → $KL_y = 22\text{ft} \Rightarrow \phi P_n = 336$ (Table 4-4) ✓

tension → yield $\Rightarrow \phi P_n = 559K$ (Table 5-5)
rupture $\Rightarrow \phi P_n = 439K$

Check Local Buckling

$b/t \leq \frac{110}{\sqrt{F_y}}$ $14.2 \leq \frac{110}{\sqrt{46}} = 16.2$ ✓

b/t ← table 1-11

Check Slenderness ← SCBF

$\frac{KL}{r} \leq \frac{1000}{\sqrt{F_y}}$ $\frac{(1)(21.5)(12)}{3.04} = 85 < \frac{1000}{\sqrt{46}} = 147$ ✓

$r_{needed} = 1.75$

* Axial comp. strength was still limiting factor for

∴ bracing the brace would not help

$\boxed{\text{HSS } 14 \times 10 \times 5/8}$

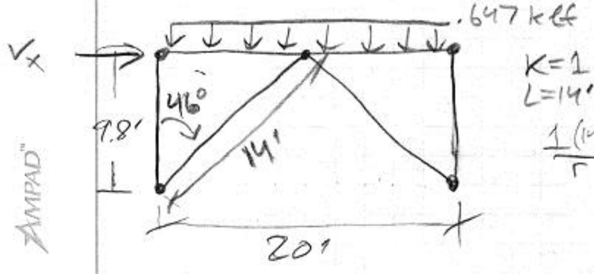
~~$r_y = r_x = 2.58$~~ ✓

~~$b/t = 9.05$~~ ✓

Frame F | Kyle Tennant | Thesis SP 2011 | MAE Braced Frame ⑤

Brace Design (upper levels)

→ find minimum size that can be used based on slenderness and then determine necessary size for different lateral loads



K=1
L=14' > find r

$\frac{1(14)(12)}{r} = \frac{1000}{\sqrt{46}}$ $r_{needed} = 1.14$

✓

Check Local Buckling

$\frac{b}{t} < \frac{110}{\sqrt{F_y}}$

< 16.2

$P_{QE} = \left(\frac{14}{10}\right) V_x$ $P_D = \left(\frac{14}{9.8}\right)(5)(.462)$
$= 3.3k$

The rest of the braces for "Inverted V Braces" (E, D, and F upper levels) were designed in an excel spreadsheet. Load and limit states were calculated and then steel manual was used to come to a decision.

Check Δ_{story} → look at brace elongation

Allowable $= .02 h_{story} = .02(19 \times 12) = 4.56$ (ground story)
$\overset{\uparrow}{\text{table}}$ 12.12.1

$.02(9.8 \times 12) = 2.35$ (upper stories)

Actual 5 for Special Concentric Braced Frames Axial tension ↓

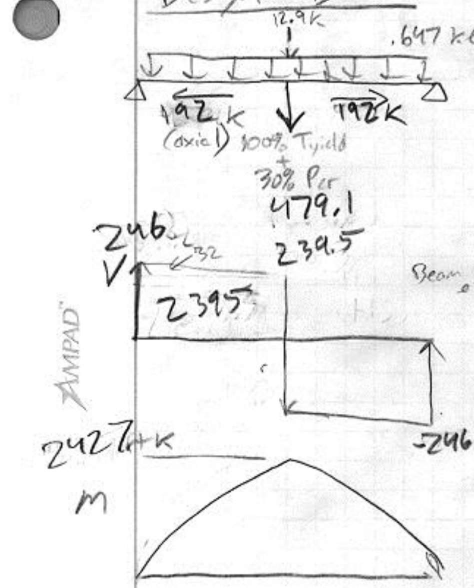
$\Delta_2 = \frac{PL}{AE} \cdot C_d$ brace → HSS 14x10x5/8 L=21.5ft A=25.7 P=577K

$= \frac{287(21.5 \times 12)}{(13.5)(29000)}(5) = .95 in \nearrow \rightarrow \left(\frac{10}{21.5}\right).95 = .44 in$ ✓ (level 2)

$\Delta_3 = \frac{175(14 \times 12)}{(7.81)(29000)}(5) = .64 in \nearrow \rightarrow \left(\frac{10}{14}\right).64 = .46 in$ ✓ (level 3)
↑
HSS5x5x.5

Frame F | Kyle Tennant | Thesis SP 2011 | MAE Braced Frame ④

Design Beam → Remains elastic



$* \frac{L}{600}$ for deflection (supports brick)
↳ not necessary for earthquake loading

Force in Tension brace

HSS 8x8x.5 $A_g = 13.5$

Level 1

$P_t = R_y F_y A_g = 1.1(46)(13.5) = 683$ K ↓

down

Beam remains elastic → 1.1 46 depends on brace

Force in Compression brace

$P_c = .3P_n$ $P_n = F_{cr} A_g$ $F_{cr} = \left[.658^{\frac{F_y}{F_e}}\right] F_y$

(because it buckles first)

$P_c = .3(34)(13.5)$
$= 141.7K \uparrow$

$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2}$
$= \frac{\pi^2 (29000)}{\left(\frac{(1(21.5)(12))}{3.04}\right)^2} = 39.8$

$F_{cr} = \left[.658^{\frac{46}{39.8}}\right] 46$
$= 34.9$

Make load vertical

$P_{ty} = \left(\frac{19}{21.5}\right) - 683 = -604.5$ K↓

$P_{cy} = \left(\frac{19}{21.5}\right)(141.7) = 125.4k\uparrow$

Unbalance

$-604.5 + 125.4 = -479.1$ K↓

Find Beam Axial load

$P_{tx} = \left(\frac{10}{21.5}\right)(683) = 318$ K ←

$P_{cy} = \left(\frac{10}{21.5}\right)(141) = 66$ ← K ← $\frac{318+66}{2}$ ← half to each half of beam

$P_x = 192.1$ K ←

Combined Loading

Table 6-1 → $p = .512 \times 10^{-3}$ $b_x = .342 \times 10^{-3}$

$.512 \times 10^{-3}(192) = .098 < .2$ → H.1-1b

$\frac{.098}{2} + \left(\frac{9}{8}\right)(.342 \times 10^{-3})(2427) = .98 \le 1.0$ ✓

3-10 → $L_b = 10'$ → W40x167

Shear

$V_u = 246 < V_n = 757$ ✓
↑
3-2

Unbraced Length

Table 3-2

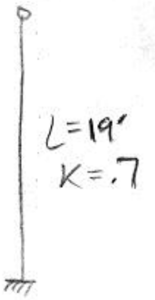
$L_b = 10' < L_r = 24.8$ ✓

$\boxed{W40x167}$

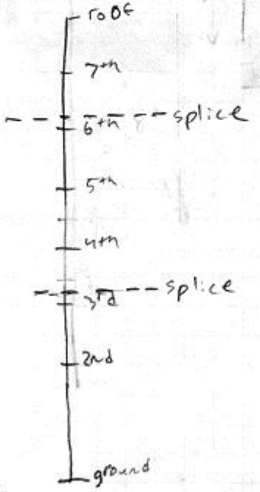
Frame F

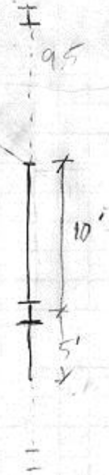
Kyle Tennant | Thesis SP 2011 | MAE Braced Frame ⑤

Column ~~Design~~ Design (Level 1)



$L=19'$
$K=.7$





Only has DL from Ext. Wall

$W_u = .462$

$P_u = .462(15) = 6.9k$ (per floor)

Load on 'GL column =

$P_D = 6.9(7) = 48.5K$

$P_L = 0$

$P_{Q_E} = \frac{\text{Vert Pt. Load on column}}{2}$

$= \frac{479.3}{2} = 239.6K$

Compressive Force

$P_u = (1.2 + .2(1))48.5 + 239 + 0 = \underline{-307.5K}$

$T_u = (.9 - .2(1))48.5 - 239 = 205k$

uplift →

↓ 4-1 $KL = .7(19) = 13.3 \to 14$

$\boxed{W12\times50} \to \phi P_n = 384k$ ∴ room for PΔ

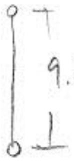
$\to \boxed{W10\times49} \to \phi P_n = 471K$

↑ W10x49 is better for slenderness (higher r_y)

Slenderness ✓

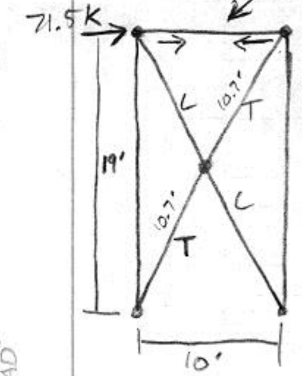
Compact ✓

Upper Stories



$K=1.0$ $KL = 9.8'$ → design in excell

Frame C | Kyle Tennant | Thesis SP 2011 | MAE Braced Frames | ⑥



no distributed load
↳ parallel to deck span direction
→ Frame B $W_D = .462\ kef$
→ Frame A $W_D = .462 kef + \overset{\text{slab + SDL}}{1.77 kef}$
$W_L = .6\ kef \leftarrow 40 psf\ LL$
→ All Frames have axial load in beams + columns due to braces

Design Brace

Compressive Force

$$P_u = \left(\frac{10.7}{5}\right)\frac{76}{2} = -81.6k \searrow$$

↓ 4-4 → KL = 11

HSS 4x4x.3125 → $\phi P_n = 100K$ ✓

Tension Force

$$T_u = \left(\frac{10.7}{5}\right)\frac{76}{2} = 81.6k \nearrow$$

5-5 → Yielding = 170k ✓ Rupture = 134k ✓

Check Local Buckling

$$b/t < \frac{110}{\sqrt{F_y}}$$

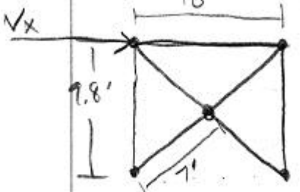
$$\underset{\text{Table 1-12}}{6.68} \leq 16.2$$

Check Slenderness

$$\frac{KL}{r} \leq \frac{1000}{\sqrt{F_y}} \rightarrow \frac{1(10.7)(12)}{r} = \frac{1000}{\sqrt{F_y}} \rightarrow \frac{128.4}{100}$$

braced at 10.7'

$r_{needed} = .87 < 1.70$ ✓



Done in excell table

∴ HSS 4.5x4.5x.5 works and axial compression controls

✱ if not braced at middle then $r_{need} = 2.58$ ∴ slenderness would control
(braced ↑ connected)

Compressive Force + Tension Force

$$P_u = \left(\frac{7}{5}\right)V_x$$

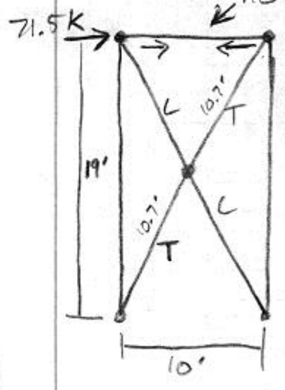
Slenderness

$r_{needed} = .57$

↑ ∴ unlikely to control

Beam → no load from lateral forces

Frame C | Kyle Tennant | Thesis SP 2011 | MAE Braced Frames | ⑥



no distributed load
↳ parallel to deck span direction

→ Frame B $w_D = .462$ kef

→ Frame A $w_D = .462$ kef + 1.77 kef ← slab + SDL
$w_L = .6$ kef ← 40 psf LL

→ All Frames have axial load in beams + columns due to braces

Design Brace

Compressive Force

$$P_u = \left(\frac{10.7}{5}\right)\frac{76}{2} = -81.6k \searrow$$

↓ 4-4 → KL = 11 ↓

HSS 4x4x.3125 → $\phi P_n = 100K$ ✓

Tension Force

$$T_u = \left(\frac{10.7}{5}\right)\frac{76}{2} = 81.6k \nearrow$$

5-5 → Yielding = 170k ✓ Rupture = 134k ✓

Check Local Buckling

$$b/t < \frac{110}{\sqrt{F_y}}$$

$$\underset{\text{Table 1-12}}{6.68} \leq 16.2$$

Check Slenderness

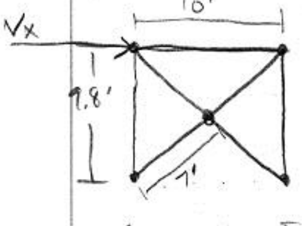
$$\frac{KL}{r} \leq \frac{1000}{\sqrt{F_y}} \rightarrow \frac{1(10.7)(12)}{r} = \frac{1000}{\sqrt{F_y}} \rightarrow \frac{128.4}{100}$$

braced at 10.7'

$$r_{needed} = .87 < 1.70 \checkmark$$

∴ HSS 4.5x4.5x.5 works and axial compression controls

* if not braced at middle then $r_{need} = 2.58$ ∴ slenderness would control
connected



Done in excell table

Compressive Force + Tension Force

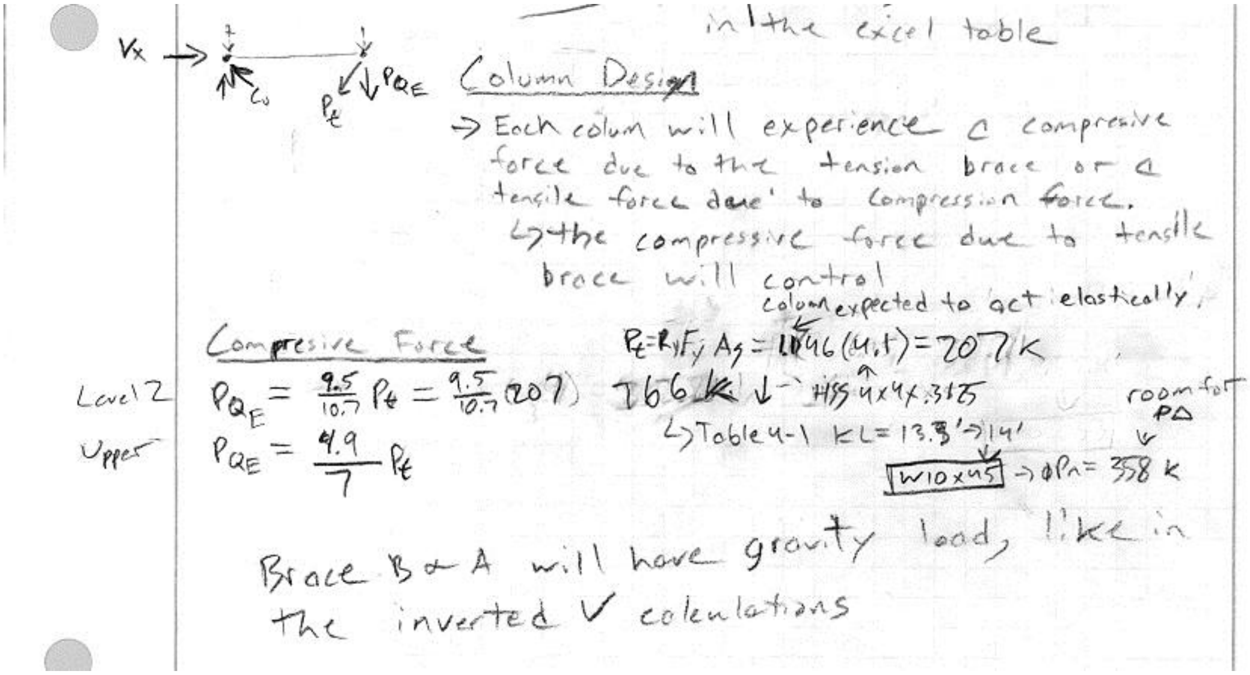
$$P_u = \left(\frac{7}{5}\right)V_x$$

Slenderness

$$r_{needed} = .57 = .84 \text{ in}$$

∴ unlikely to control

Beam → no load from lateral forces



in the excel table

Column Design

→ Each colum will experience a compressive force due to the tension brace or a tensile force due to compression force.

↳ the compressive force due to tensile brace will control

colum expected to act elastically

Compresive Force

$P_t = R_y F_y A_g = 1.1\,46(4.1) = 207\text{ K}$

Level 2 $P_{Q_E} = \frac{9.5}{10.7} P_t = \frac{9.5}{10.7}(207)$ 166 K ↓ HSS 4x4x.3125

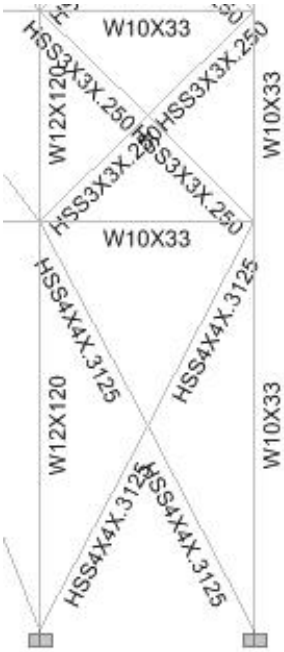
↳ Table 4-1 KL = 13.8' → 14'

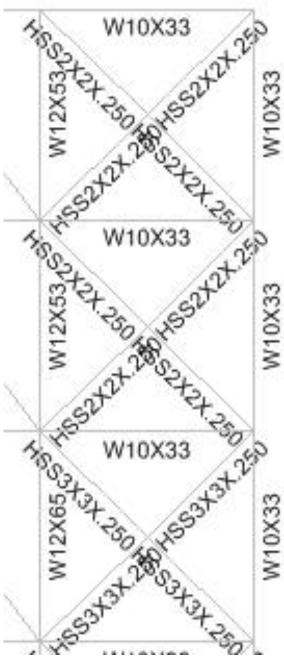
Upper $P_{Q_E} = \frac{4.9}{7} P_t$

room for PΔ

$\boxed{W10x45} \rightarrow \phi P_n = 358\text{ K}$

Brace B & A will have gravity load, like in the inverted V calculations





Excel Spreadsheets:

	Frame	Level	Frame Info			Vx (k)	Load On Beam		L_{trib} (ft)
			Column	Beam	Brace				
			Height (ft)	Width (ft)	Length (ft)		W_{Dead} (klf)	W_{Live} (klf)	
Inverted V - Brace	**F**	**Roof**	9.8	20	14.0	59.5	0.462	0	5
		7	9.8	20	14.0	114.6	0.462	0	5
		6	9.8	20	14.0	161.4	0.462	0	5
		5	9.8	20	14.0	200.1	0.462	0	5
		4	9.8	20	14.0	230.7	0.462	0	5
		3	9.8	20	14.0	253.5	0.462	0	5
		2	19	20	21.5	269.7	0.462	0	5
	E	**Roof**	9.8	18.5	13.5	45.12	0.462	0	4.6
		7	9.8	18.5	13.5	87.11	0.462	0	4.6
		6	9.8	18.5	13.5	122.85	0.462	0	4.6
		5	9.8	18.5	13.5	152.38	0.462	0	4.6
		4	9.8	18.5	13.5	175.73	0.462	0	4.6
		3	9.8	18.5	13.5	193.08	0.462	0	4.6
		2	19	18.5	21.1	205.60	0.462	0	4.6
	D	**Roof**	9.8	15	12.3	33.93	2.232	0.6	3.75
		7	9.8	15	12.3	65.30	2.232	0.6	3.75
		6	9.8	15	12.3	91.99	2.232	0.6	3.75
		5	9.8	15	12.3	114.05	2.232	0.6	3.75
		4	9.8	15	12.3	131.49	2.232	0.6	3.75
		3	9.8	15	12.3	144.44	2.232	0.6	3.75
		2	19	15	20.4	153.70	2.232	0.6	3.75
X - Brace	**C**	**Roof**	9.8	10	7.0	16.78	0	0	5
		7	9.8	10	7.0	32.29	0	0	5
		6	9.8	10	7.0	45.49	0	0	5
		5	9.8	10	7.0	56.39	0	0	5
		4	9.8	10	7.0	65.02	0	0	5
		3	9.8	10	7.0	71.42	0	0	5
		2	19	10	10.7	76.00	0	0	5
	B	**Roof**	9.8	9	6.7	12.11	0.462	0	4.5
		7	9.8	9	6.7	23.39	0.462	0	4.5
		6	9.8	9	6.7	32.98	0.462	0	4.5
		5	9.8	9	6.7	40.91	0.462	0	4.5
		4	9.8	9	6.7	47.18	0.462	0	4.5
		3	9.8	9	6.7	51.84	0.462	0	4.5
		2	19	9	10.5	55.20	0.462	0	4.5
	A	**Roof**	9.8	7.5	6.2	8.65	2.232	0.6	3.8
		7	9.8	7.5	6.2	16.65	2.232	0.6	3.8
		6	9.8	7.5	6.2	23.46	2.232	0.6	3.8
		5	9.8	7.5	6.2	29.09	2.232	0.6	3.8
		4	9.8	7.5	6.2	33.54	2.232	0.6	3.8
		3	9.8	7.5	6.2	36.84	2.232	0.6	3.8
		2	19	7.5	10.2	39.20	2.232	0.6	3.8

Strength Design Brace Member											
Forces In Brace					Local Buckling	Slenderness	Brace Chosen	Controling Case	Check Δ_{story}		Total Frame Δ (in)
									Allowable (in)	Actual (in)	
P_{Qe} (k)	P_{dead} (k)	P_{live} (k)	Pu (k) (comp.)	Tu (K) (tension)		r (in)			.02 Ht. (in)	Tension Brace	
41.7	3.3	0.0	46.3	39.4	16.2	1.14	HSS 4x4x.25	Buckling	2.35	0.24	2.48
80.2	3.3	0.0	84.8	77.9	16.2	1.14	HSS 4x4x.5	Axial Comp.	2.35	0.27	
113.0	3.3	0.0	117.6	110.7	16.2	1.14	HSS 5x5x.5	Axial Comp.	2.35	0.29	
140.1	3.3	0.0	144.7	137.8	16.2	1.14	HSS 5x5x.5	Axial Comp.	2.35	0.36	
161.5	3.3	0.0	166.2	159.2	16.2	1.14	HSS 5x5x.5	Axial Comp.	2.35	0.42	
177.4	3.3	0.0	182.1	175.1	16.2	1.14	HSS 5x5x.5	Axial Comp.	2.35	0.46	
289.5	2.6	0.0	293.2	287.7	16.2	1.75	HSS 8x8x.5	Axial Comp.	4.56	0.44	
32.9	2.9	0.0	37.0	30.8	16.2	1.10	HSS 4x4x.25	Buckling	2.35	0.18	1.94
63.5	2.9	0.0	67.5	61.4	16.2	1.10	HSS 4x4x.5	Axial Comp.	2.35	0.20	
89.5	2.9	0.0	93.6	87.4	16.2	1.10	HSS 4x4x.5	Axial Comp.	2.35	0.28	
111.0	2.9	0.0	115.1	109.0	16.2	1.10	HSS 5x5x.5	Axial Comp.	2.35	0.26	
128.0	2.9	0.0	132.1	126.0	16.2	1.10	HSS 5x5x.5	Axial Comp.	2.35	0.31	
140.6	2.9	0.0	144.7	138.6	16.2	1.10	HSS 5x5x.5	Axial Comp.	2.35	0.34	
234.9	2.4	0.0	238.2	233.2	16.2	1.72	HSS 7x7x.625	Axial Comp.	4.56	0.38	
27.9	10.5	2.8	43.3	20.5	16.2	1.00	HSS 4x4x.25	Buckling	2.35	0.09	1.25
53.7	10.5	2.8	69.1	46.3	16.2	1.00	HSS 4x4x.5	Axial Comp.	2.35	0.12	
75.7	10.5	2.8	91.0	68.3	16.2	1.00	HSS 4x4x.5	Axial Comp.	2.35	0.18	
93.8	10.5	2.8	109.2	86.5	16.2	1.00	HSS 5x5x.5	Axial Comp.	2.35	0.17	
108.2	10.5	2.8	123.5	100.8	16.2	1.00	HSS 5x5x.5	Axial Comp.	2.35	0.20	
118.8	10.5	2.8	134.2	111.5	16.2	1.00	HSS 5x5x.5	Axial Comp.	2.35	0.22	
209.3	9.0	2.4	222.5	203.0	16.2	1.66	HSS 7x7x.625	Axial Comp.	4.56	0.27	
11.7	0.0	0.0	11.7	11.7	16.2	0.57	HSS 2x2x.25	Axial Comp.	2.35	0.08	1.15
22.6	0.0	0.0	22.6	22.6	16.2	0.57	HSS 2x2x.25	Axial Comp.	2.35	0.15	
31.8	0.0	0.0	31.8	31.8	16.2	0.57	HSS 3x3x.25	Axial Comp.	2.35	0.14	
39.5	0.0	0.0	39.5	39.5	16.2	0.57	HSS 3x3x.25	Axial Comp.	2.35	0.17	
45.5	0.0	0.0	45.5	45.5	16.2	0.57	HSS 3x3x.25	Axial Comp.	2.35	0.19	
50.0	0.0	0.0	50.0	50.0	16.2	0.57	HSS 3x3x.25	Axial Comp.	2.35	0.21	
81.6	0.0	0.0	81.6	81.6	16.2	0.87	HSS 4x4x.3125	Axial Comp.	4.56	0.21	
9.0	1.4	0.0	10.9	8.0	16.2	0.54	HSS 2x2x.25	Axial Comp.	2.35	0.05	0.90
17.3	1.4	0.0	19.3	16.3	16.2	0.54	HSS 2x2x.25	Axial Comp.	2.35	0.10	
24.4	1.4	0.0	26.4	23.4	16.2	0.54	HSS 3x3x.1875	Axial Comp.	2.35	0.12	
30.2	1.4	0.0	32.2	29.3	16.2	0.54	HSS 3x3x.1875	Axial Comp.	2.35	0.14	
34.9	1.4	0.0	36.9	33.9	16.2	0.54	HSS 3x3x.1875	Axial Comp.	2.35	0.17	
38.3	1.4	0.0	40.3	37.3	16.2	0.54	HSS 3x3x.1875	Axial Comp.	2.35	0.18	
64.5	1.2	0.0	66.1	63.7	16.2	0.86	HSS 4x4x.3125	Axial Comp.	4.56	0.14	
7.1	5.3	1.4	15.2	3.4	16.2	0.50	HSS 2x2x.25	Axial Comp.	2.35	0.02	0.52
13.7	5.3	1.4	21.8	10.0	16.2	0.50	HSS 2x2x.25	Axial Comp.	2.35	0.05	
19.3	5.3	1.4	27.4	15.6	16.2	0.50	HSS 3x3x.1875	Axial Comp.	2.35	0.06	
23.9	5.3	1.4	32.0	20.2	16.2	0.50	HSS 3x3x.1875	Axial Comp.	2.35	0.08	
27.6	5.3	1.4	35.7	23.9	16.2	0.50	HSS 3x3x.1875	Axial Comp.	2.35	0.10	
30.3	5.3	1.4	38.4	26.6	16.2	0.50	HSS 3x3x.1875	Axial Comp.	2.35	0.11	
53.4	4.6	1.2	60.4	50.2	16.2	0.83	HSS 4x4x.3125	Axial Comp.	4.56	0.09	

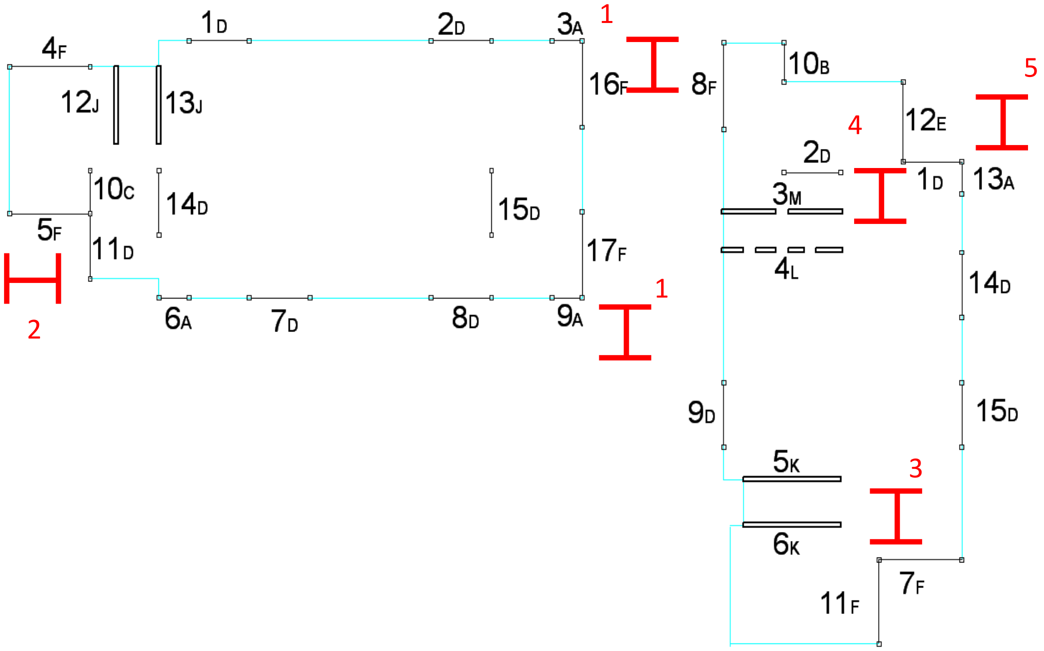
Frame	Level	Strength Beam Design																	
		Brace Info						Loads										Beam Chosen (table 3-10)	Check Interaction (Table 6-1)
								Axial In Brace		Vertical Pt Load Ctr Beam			Axial In Beam			Wu (klf)	Mu (k-ft)		
		Brace	Fy (ksi)	A_g (in^2)	r (in)	Fe	Fcr	P_c	P_t	P_{cy}	P_{ty}	Py (k)	P_{cx}	P_{tx}	Px (k)				
F	Roof	HSS 4x4x.25	46	3.37	1.52	23.40	59.50	60.2	170.5	42.1	119.4	77.3	43.0	121.8	82.4	0.647	418.6	W21x62	0.83
	7	HSS 4x4x.5	46	6.02	1.41	20.14	69.14	124.9	304.6	87.4	213.2	125.8	89.2	217.6	153.4	0.647	661.4	W24x84	0.86
	6	HSS 5x5x.5	46	7.88	1.82	33.55	41.50	98.1	398.7	68.7	279.1	210.4	70.1	284.8	177.4	0.647	1084.4	W30x108	0.90
	5	HSS 5x5x.5	46	7.88	1.82	33.55	41.50	98.1	398.7	68.7	279.1	210.4	70.1	284.8	177.4	0.647	1084.4	W30x108	0.90
	4	HSS 5x5x.5	46	7.88	1.82	33.55	41.50	98.1	398.7	68.7	279.1	210.4	70.1	284.8	177.4	0.647	1084.4	W30x108	0.90
	3	HSS 5x5x.5	46	7.88	1.82	33.55	41.50	98.1	398.7	68.7	279.1	210.4	70.1	284.8	177.4	0.647	1084.4	W30x108	0.90
	2	HSS 8x8x.5	46	13.5	3.04	39.81	34.98	141.7	683.1	125.4	604.5	479.1	66.0	318.2	192.1	0.647	2428.0	W40x167	0.98
E	Roof	HSS 4x4x.25	46	3.37	1.52	25.26	55.12	55.7	170.5	40.5	124.0	83.5	38.2	117.0	77.6	0.647	413.8	W21x62	0.82
	7	HSS 4x4x.5	46	6.02	1.41	21.74	64.05	115.7	304.6	84.1	221.5	137.4	79.4	209.1	144.2	0.647	663.1	W24x84	0.86
	6	HSS 4x4x.5	46	6.02	1.41	21.74	64.05	115.7	304.6	84.1	221.5	137.4	79.4	209.1	144.2	0.647	663.1	W24x84	0.86
	5	HSS 5x5x.5	46	7.88	1.82	36.22	38.44	90.9	398.7	66.1	290.0	223.9	62.4	273.7	168.0	0.647	1063.1	W30x108	0.88
	4	HSS 5x5x.5	46	7.88	1.82	36.22	38.44	90.9	398.7	66.1	290.0	223.9	62.4	273.7	168.0	0.647	1063.1	W30x108	0.88
	3	HSS 5x5x.5	46	7.88	1.82	36.22	38.44	90.9	398.7	66.1	290.0	223.9	62.4	273.7	168.0	0.647	1063.1	W30x108	0.88
	2	HSS 7x7x.625	46	11.6	2.58	29.60	47.04	163.7	587.0	147.2	527.7	380.6	71.7	256.9	164.3	0.647	1787.7	W36x135	0.99
D	Roof	HSS 4x4x.25	46	3.37	1.61	33.80	41.20	41.7	170.5	33.1	135.4	102.3	25.3	103.6	64.5	3.638	486.1	W21x62	0.94
	7	HSS 4x4x.5	46	6.02	1.82	43.19	32.24	58.2	304.6	46.2	241.9	195.7	35.4	185.1	110.3	3.638	836.1	W27x84	0.97
	6	HSS 4x4x.5	46	6.02	1.82	43.19	32.24	58.2	304.6	46.2	241.9	195.7	35.4	185.1	110.3	3.638	836.1	W27x84	0.97
	5	HSS 5x5x.5	46	7.88	2.17	61.40	22.68	53.6	398.7	42.6	316.6	274.1	32.6	242.3	137.5	3.638	1130.1	W30x108	0.92
	4	HSS 5x5x.5	46	7.88	2.17	61.40	22.68	53.6	398.7	42.6	316.6	274.1	32.6	242.3	137.5	3.638	1130.1	W30x108	0.92
	3	HSS 5x5x.5	46	7.88	2.17	61.40	22.68	53.6	398.7	42.6	316.6	274.1	32.6	242.3	137.5	3.638	1130.1	W30x108	0.92
	2	HSS 7x7x.625	46	11.6	3.09	45.44	30.64	106.6	587.0	99.2	546.0	446.8	39.2	215.5	127.3	3.638	1777.7	W36x135	0.97
C	Roof	HSS 2x2x.25	46	1.51	0.7	20.08	69.34	31.4	76.4							0	0.0	W10x33	
	7	HSS 2x2x.25	46	1.51	0.7	20.08	69.34	31.4	76.4							0	0.0	W10x33	
	6	HSS 3x3x.25	46	2.44	1.11	49.92	27.89	20.4	123.5							0	0.0	W10x33	
	5	HSS 3x3x.25	46	2.44	1.11	49.92	27.89	20.4	123.5							0	0.0	W10x33	
	4	HSS 3x3x.25	46	2.44	1.11	49.92	27.89	20.4	123.5							0	0.0	W10x33	
	3	HSS 3x3x.25	46	2.44	1.11	49.92	27.89	20.4	123.5							0	0.0	W10x33	
	2	HSS 4x4x.3125	46	4.1	1.41	34.25	40.65	50.0	207.5							0	0.0	W10x33	
B	Roof	HSS 2x2x.25	46	1.51	0.7	22.23	62.62	28.4	76.4							0.647	6.5	W10x33	
	7	HSS 2x2x.25	46	1.51	0.7	22.23	62.62	28.4	76.4							0.647	6.5	W10x33	
	6	HSS 3x3x.1875	46	1.89	1.14	58.30	23.88	13.5	95.6							0.647	6.5	W10x33	
	5	HSS 3x3x.1875	46	1.89	1.14	58.30	23.88	13.5	95.6							0.647	6.5	W10x33	
	4	HSS 3x3x.1875	46	1.89	1.14	58.30	23.88	13.5	95.6							0.647	6.5	W10x33	
	3	HSS 3x3x.1875	46	1.89	1.14	58.30	23.88	13.5	95.6							0.647	6.5	W10x33	
	2	HSS 4x4x.3125	46	4.1	1.41	35.72	38.97	47.9	207.5							0.647	6.5	W10x33	
A	Roof	HSS 2x2x.25	46	1.51	0.7	25.85	53.87	24.4	76.4							3.638	25.6	W10x33	
	7	HSS 2x2x.25	46	1.51	0.7	25.85	53.87	24.4	76.4							3.638	25.6	W10x33	
	6	HSS 3x3x.1875	46	1.89	1.14	67.78	20.54	11.6	95.6							3.638	25.6	W10x33	
	5	HSS 3x3x.1875	46	1.89	1.14	67.78	20.54	11.6	95.6							3.638	25.6	W10x33	
	4	HSS 3x3x.1875	46	1.89	1.14	67.78	20.54	11.6	95.6							3.638	25.6	W10x33	
	3	HSS 3x3x.1875	46	1.89	1.14	67.78	20.54	11.6	95.6							3.638	25.6	W10x33	
	2	HSS 4x4x.3125	46	4.1	1.41	37.84	36.79	45.3	207.5							3.638	25.6	W10x33	

Frame	Level	Strength Column Design									
		Load on Beam		Axial Load on Column						KL (ft)	Column Chosen (table 4-1)
		W_{Dead} (klf)	W_{Live} (klf)	L_{beam} (ft)	P_D (k)	P_L (k)	P_Q (k)	Pu (k)	Tu (k)		
F	Roof	0.462	0	15	6.9	0.0	38.6	48.3	33.8	9.8	
	7	0.462	0	15	13.9	0.0	62.9	82.3	53.2	9.8	W10x33
	6	0.462	0	15	20.8	0.0	105.2	134.3	90.6	9.8	
	5	0.462	0	15	27.7	0.0	105.2	144.0	85.8	9.8	W10x33
	4	0.462	0	15	34.7	0.0	105.2	153.7	80.9	9.8	
	3	0.462	0	15	41.6	0.0	105.2	163.4	76.1	9.8	
	2	0.462	0	15	48.5	0.0	239.6	307.5	205.6	13.3	W10x49
E	Roof	0.462	0	10	4.6	0.0	41.7	48.2	38.5	9.8	
	7	0.462	0	10	9.2	0.0	68.7	81.6	62.2	9.8	W10x33
	6	0.462	0	10	13.9	0.0	68.7	88.1	59.0	9.8	
	5	0.462	0	10	18.5	0.0	111.9	137.8	99.0	9.8	W10x33
	4	0.462	0	10	23.1	0.0	111.9	144.3	95.8	9.8	
	3	0.462	0	10	27.7	0.0	111.9	150.8	92.5	9.8	
	2	0.462	0	10	32.4	0.0	190.3	235.6	167.6	13.3	W10x39
D	Roof	2.232	0.6	15	33.5	9.0	51.2	102.5	27.7	9.8	
	7	2.232	0.6	15	67.0	18.0	97.8	200.6	51.0	9.8	W10x33
	6	2.232	0.6	15	100.4	27.0	97.8	251.9	27.5	9.8	
	5	2.232	0.6	15	133.9	36.0	137.0	342.5	43.3	9.8	W10x49
	4	2.232	0.6	15	167.4	45.0	137.0	393.9	19.9	9.8	
	3	2.232	0.6	15	200.9	54.0	137.0	445.3	-3.6	9.8	
	2	2.232	0.6	15	234.4	63.0	223.4	583.0	59.3	13.3	W10x68
C	Roof	0.000	0	5	0.0	0.0	53.5	53.5	53.5	9.8	
	7	0.000	0	5	0.0	0.0	53.5	53.5	53.5	9.8	W10x33
	6	0.000	0	5	0.0	0.0	86.4	86.4	86.4	9.8	
	5	0.000	0	5	0.0	0.0	86.4	86.4	86.4	9.8	W10x33
	4	0.000	0	5	0.0	0.0	86.4	86.4	86.4	9.8	
	3	0.000	0	5	0.0	0.0	86.4	86.4	86.4	9.8	
	2	0.000	0	5	0.0	0.0	183.6	183.6	183.6	13.3	W10x33
B	Roof	0.462	0	4.5	2.08	0.00	56.3	59.2	54.8	9.8	
	7	0.462	0	4.5	4.16	0.00	56.3	62.1	53.4	9.8	W10x33
	6	0.462	0	4.5	6.24	0.00	70.4	79.2	66.1	9.8	
	5	0.462	0	4.5	8.32	0.00	70.4	82.1	64.6	9.8	W10x33
	4	0.462	0	4.5	10.40	0.00	70.4	85.0	63.2	9.8	
	3	0.462	0	4.5	12.47	0.00	70.4	87.9	61.7	9.8	
	2	0.462	0	4.5	14.55	0.00	187.5	207.9	177.3	13.3	W10x39
A	Roof	2.232	0.6	11.25	25.1	6.8	60.7	99.2	43.1	9.8	
	7	2.232	0.6	11.25	50.2	13.5	60.7	137.7	25.5	9.8	W10x33
	6	2.232	0.6	11.25	75.3	20.3	75.9	191.5	23.2	9.8	
	5	2.232	0.6	11.25	100.4	27.0	75.9	230.1	5.6	9.8	W10x39
	4	2.232	0.6	11.25	125.6	33.8	75.9	268.6	-11.9	9.8	
	3	2.232	0.6	11.25	150.7	40.5	75.9	307.1	-29.5	9.8	
	2	2.232	0.6	11.25	175.8	47.3	193.0	462.7	69.9	13.3	W10x60
Uplift at Base (member self weights not included)											

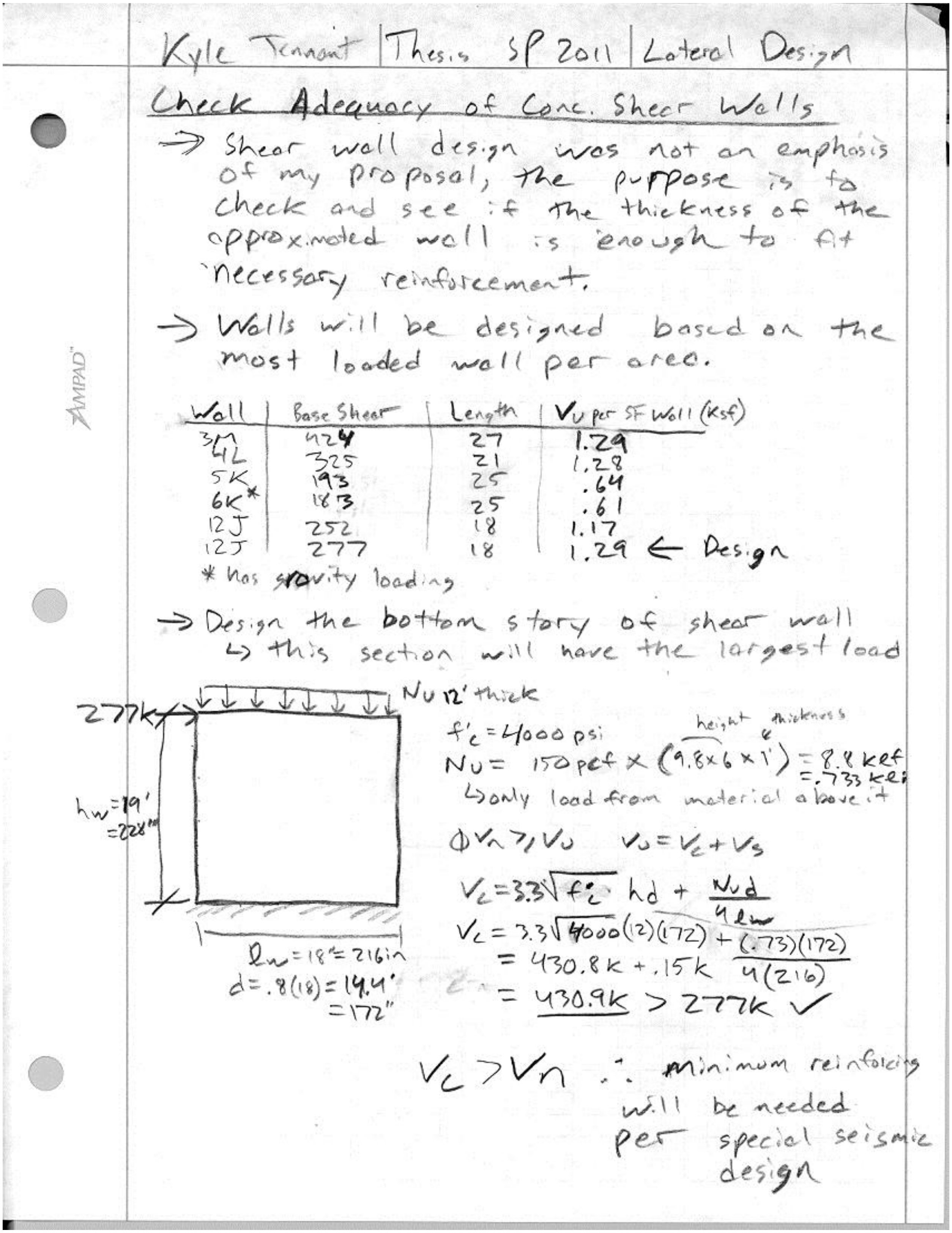
Special Case 1	Column Chosen (table 4-1)	Special Case 2	Column Chosen (table 4-1)
147.5		204.4	
220.0	W12x45	336.4	W12x53
325.9		472.7	
374.1	W12x53	573.0	W12x65
422.3		634.0	
470.6		695.1	
770.2	W12x87	1074.1	W12x120

Special Case 3	Column Chosen (table 4-1)	Special Case 4	Column Chosen (table 4-1)
96.7		150.8	
164.6	W12x40	282.2	W12x45
268.6		340.1	
288.1	W12x45	480.3	W12x65
307.5		538.2	
326.9		596.0	
615.0	W12x72	818.6	W12x87

Special Case 5	Column Chosen (table 4-1)
111.8	
158.5	W10x33
173.8	
213.0	W10x33
213.0	
213.0	
416.4	W10x54



Appendix H: Shear Wall Thickness Adequacy



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Check Adequacy of Conc. Shear Walls

→ Shear wall design was not an emphasis of my proposal, the purpose is to check and see if the thickness of the approximated wall is enough to fit necessary reinforcement.

→ Walls will be designed based on the most loaded wall per area.

Wall	Base Shear	Length	V_u per SF Wall (Ksf)
3M	424	27	1.29
4L	325	21	1.28
5K	193	25	.64
6K*	183	25	.61
12J	252	18	1.17
12J	277	18	1.29 ← Design

* has gravity loading

→ Design the bottom story of shear wall
↳ this section will have the largest load

277k

N_u 12' thick

$h_w = 19'$
$= 228''$

$l_w = 18' = 216in$
$d = .8(18) = 14.4'$
$= 172''$

$f'_c = 4000\ psi$

$N_u = 150\ pcf \times (\overset{height}{9.8 \times 6} \times \overset{thickness}{1'}) = 8.8\ kcf$
$= .733\ kci$

↳ only load from material above it

$\phi V_n \geq V_u \qquad V_u = V_c + V_s$

$V_c = 3.3\sqrt{f'_c}\,hd + \frac{N_u d}{4 l_w}$

$V_c = 3.3\sqrt{4000}(12)(172) + \frac{(.73)(172)}{4(216)}$

$= 430.8k + .15k$

$= \underline{430.9k} > 277k$ ✓

$V_c > V_n$ ∴ minimum reinforcing will be needed per special seismic design

Appendix I: Cost Data: Member Information

An estimate of members was done for the cost and schedule estimate.

Existing:

Reinforced Concrete Masonry Bearing Wall Schedule

						Weight (psf)			Load Carrying Capibility	
Wall Type	**Thickness**	**Rebar**	**Spacing**	**Grout**	**Floor Location**	**CMU & Grout**	**Rebar**	**Total**	**Gravity (plf)**	**Lateral (plf)**
A	12"	#7	16" O.C.	All cells	1st ext.	140	1.53	141.53		
B	12"	#7	32" O.C.	All cells	1st int. center	140	0.77	140.77		
C	8"	#6	32" O.C.	All cells	1st int. random	92	0.56	92.56		
D	8"	#6	24" O.C.	Cells w/reinforcement	2nd ext.	69	0.75	69.75		
E	8"	#5	32" O.C.	All cells	2nd int. typ.	92	0.39	92.39		
F	8"	#6	32" O.C.	16" O.C.	3rd - 5th ext.	75	0.56	75.56		
G	8"	#6	32" O.C.	Cells w/reinforcement	5th - 7th ext.	65	0.56	65.56		
H	8"	#5	32" O.C.	16" O.C.	3rd - 5th int.	75	0.39	75.39		
I	8"	#5	32" O.C.	Cells w/reinforcement	5th - 7th int.	65	0.39	65.39		

Masonry Wall Areas

Floor	**Component**	**Height**	**Length**	**Area**
1	Wall A	18	687	12366.00
1	Wall B	18	174	3132.00
1	Wall C	18	91	1638.00
2	Wall D	8.66	687	5949.42
2	Wall E	8.66	391	3386.06
3	Wall F	8.66	687	5949.42
3	Wall G	8.66	391	3386.06
4	Wall F	8.66	687	5949.42
4	Wall G	8.66	391	3386.06
5	Wall H	8.66	687	5949.42
5	Wall I	8.66	391	3386.06
6	Wall H	8.66	687	5949.42
6	Wall I	8.66	391	3386.06
7	Wall H	8.66	687	5949.42
7	Wall I	8.66	391	3386.06
	12" Total =	15498.00	8" Total =	57650.88

Precast Concrete Plank

Floor	Area
2	13679
3	13679
4	13679
5	13679
6	13679
7	13679
Roof	13679
Total	95753

Proposed: Steel Estimate

Columns LW					
Story			1 and 2	3 to 5	6 and 7
Length			28.8	29.4	19.6
Gravity	Interior	1	W10x60	W10x39	W10x33
	Exterior	8	W10x49	W10x33	W10x33
Lateral	DL	19	W10x60	W10x39	W10x33
	Wall Load	7	W10x49	W10x33	W10x33
	Interior	5	W10x100	W10x60	W10x49
Beams					
Average Length			15	15	15
Gravity	Interior	5	W16x31	W16x31	W16x31
	Exterior	14	W16x31	W16x31	W16x31
Lateral	X-Brace	4	W10x33	W10x33	W10x33
	V-Brace	16	W36x135	W30x108	W24x84
Braces					
Average Length			20	12	12
X-Brace		8	HSS 4x4x.3125	HSS 3x3x.1875	HSS 2x2x.25
V-Brace		40	HSS 7x7x.5	HSS 5x5x.5	HSS 4x4x.25

Columns RW					
Story			1 and 2	3 to 5	6 and 7
Length			28.8	29.4	19.6
Gravity	Interior	4	W10x60	W10x39	W10x33
	Exterior	7	W10x49	W10x33	W10x33
Lateral	DL	10	W10x60	W10x39	W10x33
	Wall Load	7	W10x49	W10x33	W10x33
	Interior	2	W10x100	W10x60	W10x49
Beams					
Average Length			15	15	15
Gravity	Interior	7	W16x31	W16x31	W16x31
	Exterior	14	W16x31	W16x31	W16x31
Lateral	X-Brace	2	W10x33	W10x33	W10x33
	V-Brace	10	W36x135	W30x108	W24x84
Braces					
Average Length			20	12	12
X-Brace		4	HSS 4x4x.3125	HSS 3x3x.1875	HSS 2x2x.25
V-Brace		20	HSS 7x7x.5	HSS 5x5x.5	HSS 4x4x.25

Columns 1st				
	#	Length (ft)	Total (ft)	Total Wt. (lbs)
W10x49	29	19	551	26999
W10x60	34	19	646	31654
W10x100	7	19	133	6517
Columns 2nd				
	#	Length (ft)	Total (ft)	Total Wt. (lbs)
W10x49	29	9.8	284.2	13926
W10x60	34	9.8	333.2	16327
W10x100	7	9.8	68.6	3361
Columns 3nd through 5th (per floor)				
	#	Length (ft)	Total (ft)	Total Wt. (lbs)
W10x33	29	9.8	284.2	13926
W10x39	34	9.8	333.2	16327
W10x60	7	9.8	68.6	3361
Columns 6th and 7th (per floor)				
	#	Length (ft)	Total (ft)	Total Wt. (lbs)
W10x33	63	9.8	617.4	30253
W10x49	7	9.8	68.6	3361

Beams 1st floor				
	#	Length (ft)	Total (ft)	Total Wt. (lbs)
W10x33	6	15	90	4410
W16x31	40	15	600	26999
W36x135	26	15	390	26999
Beams 2nd through 5th floor				
	#	Length (ft)	Total (ft)	Total Wt. (lbs)
W10x33	6	15	90	26999
W16x31	40	15	600	26999
W30x108	26	15	390	26999
Beams 2nd through 5th floor				
	#	Length (ft)	Total (ft)	Total Wt. (lbs)
W10x33	6	15	90	26999
W16x31	40	15	600	26999
W24x87	26	15	390	26999

Columns (appox per floor)				
	#	Length (ft)	Total (ft)	Total Wt. (lbs)
W10x49	29	9.8	284.2	13926
W10x60	41	9.8	401.8	19688

Columns (appox per bldg)					
	#	Length (ft)	Total (ft)	Total Wt. (lbs)	
W10x49	232	9.8	2273.6	111406	
W10x60	328	9.8	3214.4	192864	avg. wt.
		Total =	5488	304270	55.44286

Beams (Approx per floor)				
	#	Length (ft)	Total (ft)	Total Wt. (lbs)
W10x33	6	15	90	2970
W16x31	40	15	600	19800
W30x108	26	15	390	12870

Beams (aprox whole bldg)				
	#	Length (ft)	Total (ft)	Total Wt. (lbs)
W16x31	322	15	4830	48213.06
W30x108	182	15	2730	294840
		Total =	7560	343053

Braces (per Floor aproxmate)				
	#	Length (ft)	Total (ft)	Total Wt. (lbs)
HSS 5x5x.5	68	12	816	19828.8

Braces (bldg aproximate)				
	#	Length (ft)	Total (ft)	Total Wt. (lbs)
HSS 5x5x.5	476	12	5712	138801.6

Proposed: Concrete Estimate

Concrete Shear Walls

Wall	# of Walls	Length (ft)	Height (ft)	Thickness (ft)	Penetration Area (SF)	# Penetrations	Area (C.Y)	Surface Area	Aprox. Steel tons plf	Steel (lbs)
J	2	18	78	1	22.5	0	103.16667	5616	0.12	4.32
M	1	30	78	1	22.5	1	85.833333	4680	0.12	3.6
L	1	30	78	1	22.5	3	85.833333	4680	0.12	3.6
K	2	24	78	1	22.5	0	137.83333	7488	0.12	5.76
						Total C.Y. =	412.66667	22464	Total tons =	17.28