THESIS FINAL REPORT

JORDAN RUTHERFORD | STRUCTURAL OPTION

HOTEL | NORTHEAST U.S.

ADVISOR | DR. THOMAS BOOTHBY

SUBMITTED | APRIL 3, 2013

|Hotel|Northeastern U.S.|

|Jordan Rutherford|
|Structural Option|

|building statistics|

Occupancy	Residential, Aseembly
Size	75,209 sqft.
Floors	5
Height	60'8"
Rooms	113
Cost	$9.2 million
Construction	Oct. 2011 - Nov. 2012
Method	Design-Bid-Build

|project team|

Owner
Withheld

Architect
Meyer and Associates

Developer
Continental Building Systems

MEP & Fire Protection
Prater Engineering Associates

Civil/Landscape
Civil and Environmental Consultants, Inc.

Structural
Atlantic Engineering Services

|architecture|

- Slender design for natural light in all rooms and view of the river
- Pool, Fitness Room, Meeting Room, Breakfast Area
- Facade consisting of Brick, Gypsum Sheathing, Exterior Insulation and Finish System
- Canopy at entrance for vehicular access
- Decorative cornice around entire roof

|building systems|

Structural

- Foundation consists of column spread footings and continuous wall footings.
- Structural steel is used on the first floor with masonry bearing walls on all other floors.
- Hollowcore concrete precast plank makes up the floor and roof system.
- Lateral resistance is provided by masonry shear walls.

Mechanical

- Two single zone VAV rootop units with 100% outdoor air
- Varible Refridgerant Flow (VRF) outdoor units provide 218,000 BTU/hr of cooling and 143,000 BTU/hr of heating
- Rooms have Packaged Terminal Air Conditioning Units (PTAC) with an average of 8,000 BTU/hr cooling, 7,000 BTU/hr for heat pump and 10,000 BTU/hr for electric heat.

Electrical/Lighting

- Standby Generator with 160 KW and 200 KVA is 120V and 60 Hertz.
- 13.2KV, 277V 3 phase transformer with 2500A breaker leading to main switchboard and rooftop units
- Panels are 208/120V and located on first, second, and fourth floor
- Fluorescent and Incandescent dimmers used on first floor
- Facade is illuminated by 150W PMH floodlights
- Guest rooms uses 13W Quad Pin and Guest bathrooms use 14W LED

Contents

Acknowledgements

I would like to thank Atlantic Engineering Services, specifically John Schneider and Cam Baker, for allowing me to use this building for my thesis study.

Thank you to the Penn State Architectural Engineering Department and professors.

Thank you to my advisor, Dr. Thomas Boothby, for all your help, advice, and kindness. It was a pleasure to receive your assistance and to learn from you.

Thanks to my friend Seth Moyer who grinded through the past 4 years with me. Your help and friendship was awesome.

A special thank you to my family, friends, girlfriend, and to God. I would not be here without your love and support.

Executive Summary

The Hotel in this report is located in the Northeast United States (referred to as the Hotel N.E.U.S.) along a river. Standing 60'-8" tall at its highest, the hotel contains 113 rooms and 75,209 square feet. Construction began in October of 2011 and was completed in November 2012. The project cost around $10 million.

This thesis focuses on redesigning the framing using steel construction. The existing framing consists of masonry bearing/shear walls with precast planks making up the floor system. Composite steel and concrete on metal deck will be used to replace the planks and steel beams and columns will be used in place of the masonry walls. Efficient column place will not interrupt any room spaces and keep the floor plan identical to the existing design, minimizing conflicts in architecture.

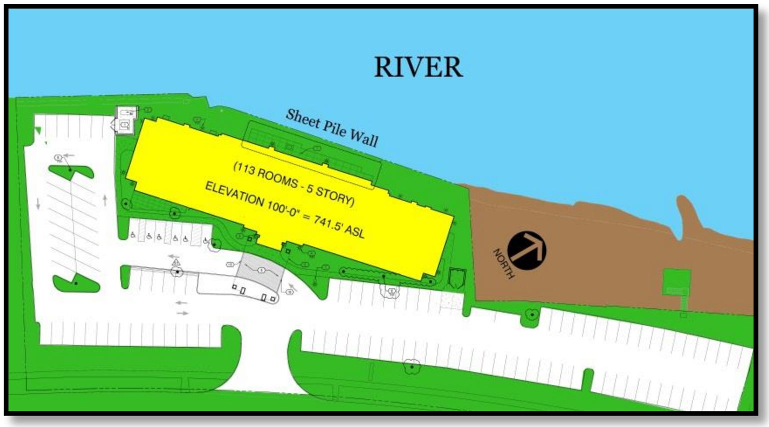
The existing shear walls can be redesigned using braced frames in the short direction and moment frames in the long direction. Utilizing braced frames in the short direction will keep them concealed in partition walls (where the shear walls currently exist). Moment frames in the long direction can allow window and door placement to remain unaltered.

By changing the material to steel, the overall building weight will significantly decrease which lowers seismic loads. The construction timeline could also be decreased. A large benefit to steel construction is that the lateral system can achieve a balanced layout. Masonry suffers with placement because it must run continuous from foundation to roof. The ability to resist lateral loads and limit drift is well met with shear walls. This report will serve as a learning tool to decipher the difference between masonry and steel construction in low rise buildings.

To break away from the characteristic hotel style in today's construction industry, the architecture of the Hotel N.E.U.S. will be revaluated. A study of old and new buildings will forge a new design for the hotel. A computer model will be created to convey the fresh architectural style. Along with this, the enclosure of the building will be investigated. After an analysis of the existing enclosure, the new façade selected for the architecture will be inspected and compared to the old system based on certain criteria. These breadths work in conjunction with each other to look at the building's shell for performance and for aesthetics.

Introduction

Located along a river in the Northeast United States (henceforth referred to as Hotel N.E.U.S.), this five story, 113 room hotel is constructed with masonry bearing walls and a precast concrete floor system. It stands in place of an old steel mill and was constructed as part of the area's development in the 1990's.



At its tallest, the building is 60'-8" tall with a long slender shape that allows for windows in every room. Its façade consists of arching exterior insulation finishing system (EIFS) and a brick veneer. The warm colors of beige and brown provide a sense of comfort and soothing that communicate the architecture's purpose, a place to rest.

All of the amenities of a hotel are included, such as a pool, fitness area, meeting room, ADA accessible rooms, and sunlight for all rooms. There is an overhang at the entrance allowing for drop off and pick up with protection from the elements. The Hotel N.E.U.S. provides 75,209 ft^2 of floor area to a location lacking such facilities. Construction started in October of 2011 and is slated to finish in November of 2012 and cost $9.2 million dollars.

Note: The overhang at the entrance is not considered in the analysis or evaluation of this building at any point.

All photos/plans/documents provided by Atlantic Engineering Services/Meyer Associates

Structural Overview

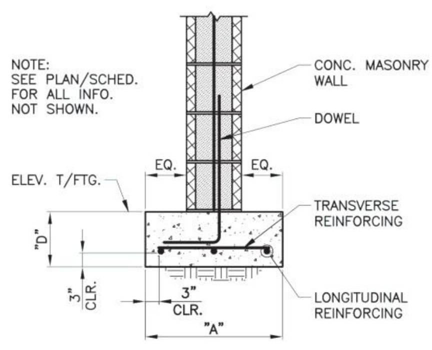
Foundations

Michael Baker Jr., Inc. provided the Geotechnical report in July of 2011. They included a history of the site that impacts the features below grade for this project. Pre-1986 the site of the Hotel N.E.U.S. was occupied by a steel mill. Cooling towers were located at the footprint of the current building while a gantry crane and tracks were to the Southwest. The sheet pile retaining wall was constructed in 1979. In 1990's a development of the area began and the mill was removed. Foundations and other below grade structures were usually removed to about to about one foot below grade. In 2001 a Damon's Restaurant and parking lot were constructed in the area that the hotel is to be located. Fill was added to the site during this time.

Geotechnical Consultants, Inc. drilled seven borings in April of 2001 to support Damon's Restaurant and those reports were included and mostly consisted of Slag and Concrete with little Silt. Terra Testing excavated four test pits and drilled thirteen test borings in April of 2011. They totaled 10 linear feet of rock and 282 linear feet of soil (see Figure 3 for location of all borings). The major finding in these tests was that there were buried concrete obstructions. They were determined to be the concrete pad that supported the cooling towers in the past.

The fill was considered to be suitable for a shallow spread foundation system. The bearing pressure was controlled by a limiting settlement of one inch and the capacity of the soil. The allowable bearing capacity of the soil increases with the size of the footing. Larger footings cause much higher stresses however, so the bearing pressure decreases with larger sizes (see Figure 1 for tables providing various sizes). A minimum of a 3' x 3' reinforced footing was suggested and no less than 16.7' center-to-center distance between wall footings. Footings bearing on the concrete pad were allowed a reduction of 1.5'.

Continuous wall footings range from 2'-0" wide to 9'-0" wide with typically #5 or #7 for longitudinal and transverse reinforcement. Column footings ranged from 6'x6'x1'-6" to 8'x8'x1'-8" (see Figure 1 for footing schedule). Typical piers are 24"x24" with 4-#6 vertical with #3 at 12" ties.



TYPICAL CONTINUOUS MASONRY WALL FOOTING DETAIL

CONTINUOUS WALL FOOTING SCHEDULE

MARK	WIDTH "A"	DEPTH "D"	LONGITUDINAL REINFORCING	TRANSVERSE REINFORCING	MARK
WF1	2'–0"	1'–0"	2–#5 CONT.	#5x1'–6" @ 24" O.C.	WF1
WF2	3'–0"	1'–0"	3–#5 CONT.	#5x2'–6" @ 24" O.C.	WF2
WF3	9'–0"	1'–6"	6–#7 CONT.	#7x8'–6" @ 12" O.C.	WF3
WF4	5'–0"	1'–0"	3–#7 CONT.	#7x4'–6" @ 18" O.C.	WF4
WF5	6'–0"	1'–3"	6–#5 CONT.	#5x5'–6" @ 18" O.C.	WF5

Figure 1: Continuous Masonry Wall Footing detail and schedule

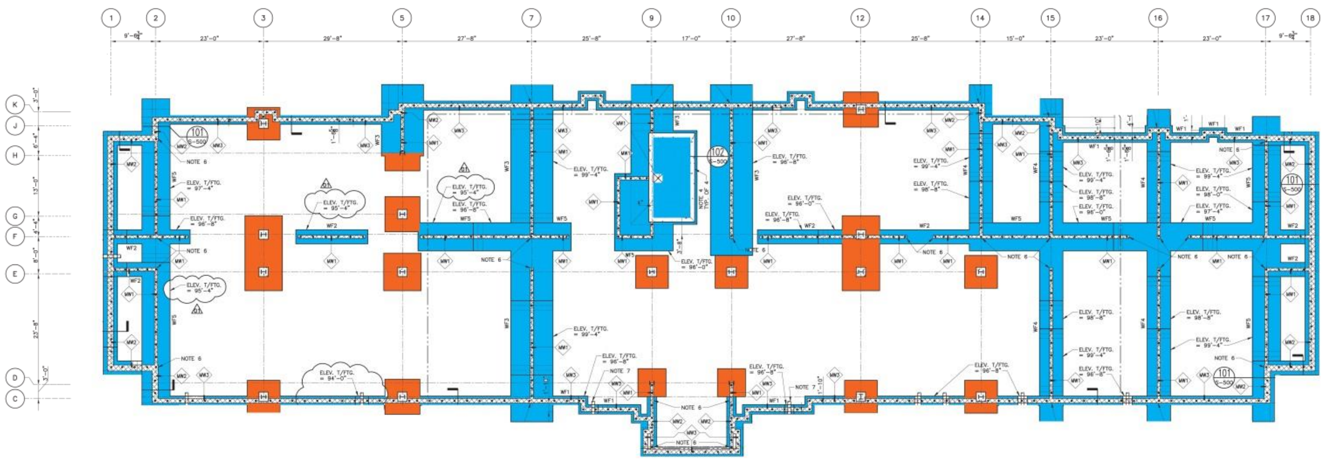


Figure 2: Foundation Plan.
Blue- wall footings
Orange- Column Footings

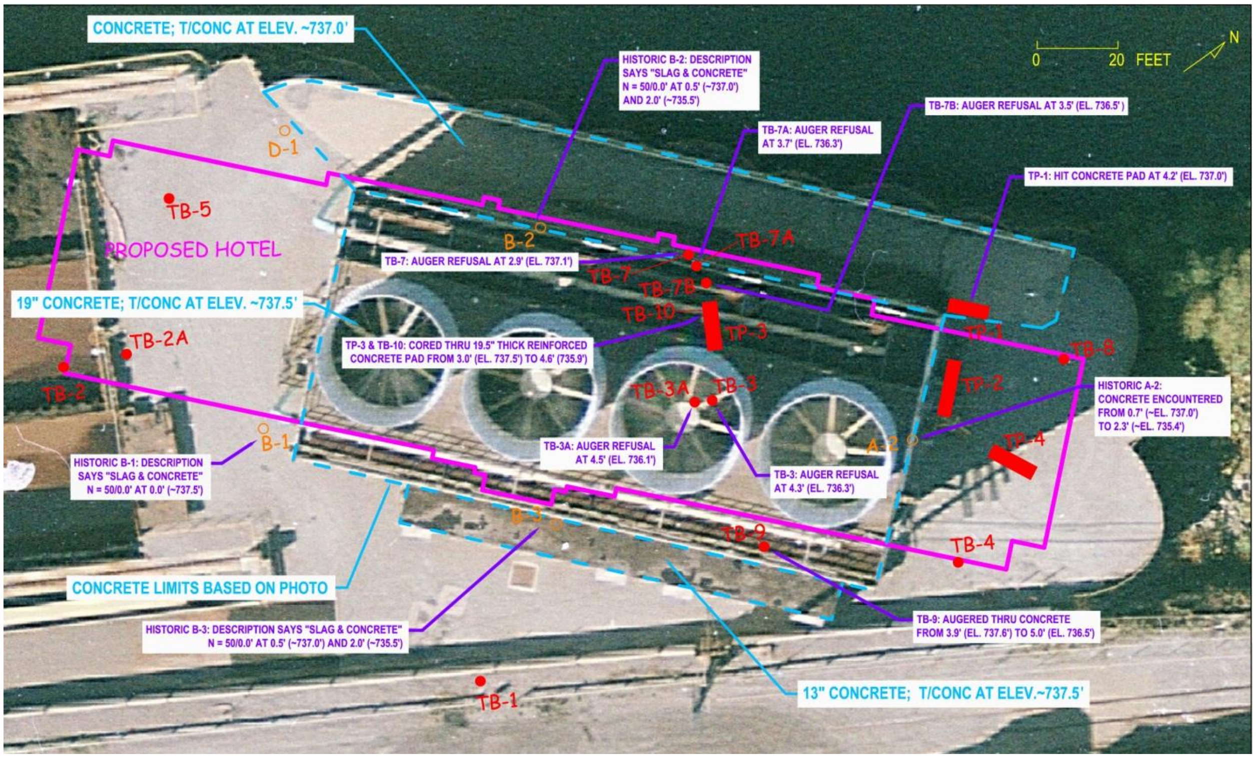


Figure 3: Site map showing test borings, existing mat foundation, hotel footprint, and location of former cooling towers.

Floor System

The floor system is composed of 8" Hollowcore precast concrete plank. There is a 3/4" topping to level off the floor since the planks have camber when they come out of production. The plank allows for long spans between the bearing walls. The smallest span is 15'-0" while the largest is 29'-8". Due to the large open spaces on the first floor, large transfer beams are used to carry the walls on the second floor up to the roof. These wide flange beams are approximately 30" in depth and weigh anywhere from 90 to 191 pounds per foot. Smaller beams span the corridor between walls and are much smaller, ranging from W6x25 to W24x68.

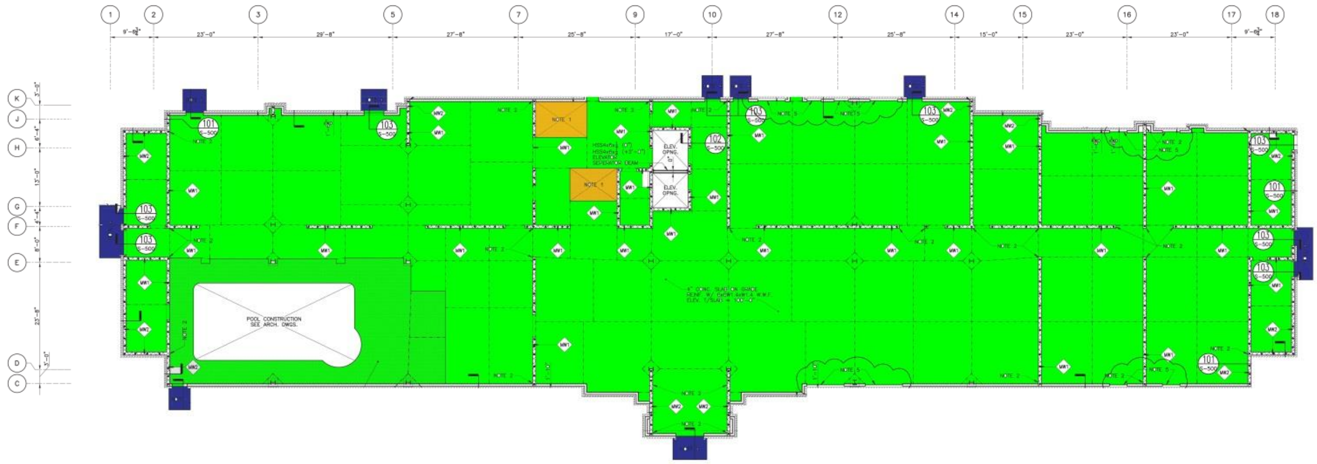


Figure 4: Slab on grade. Light green- 4" Conc. Slab on grade w/ 6x6W1.4xW1.4 W.W.F.

Orange- 3'-0" thick Conc. Slab w/ #5@12" O.C. Top and B.E.W. Isolated from adjacent slab.

Blue- Exterior 4" Conc. Slab on grade w/ 6x6W1.4xW1.4 W.W.F sloped away from building.

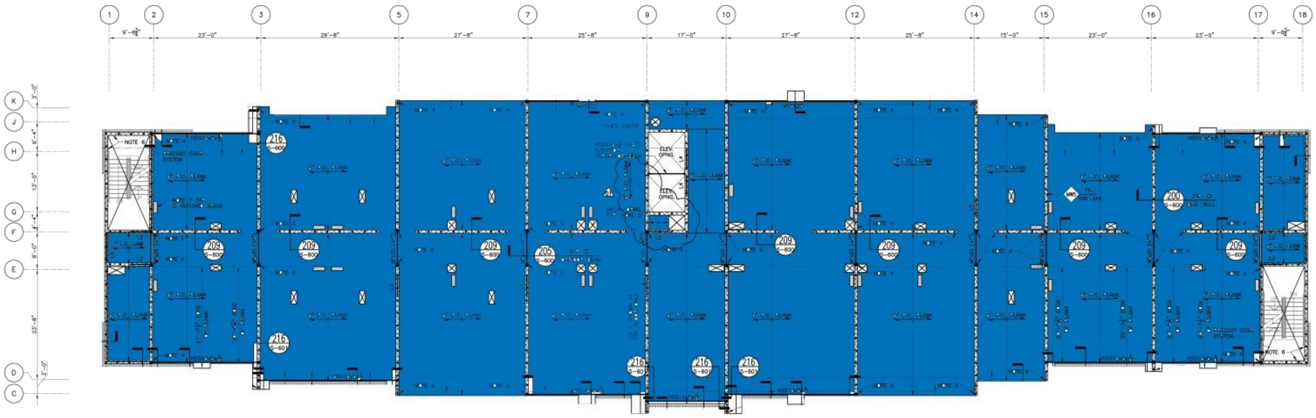


Figure 5: Typical Floor plank layout

Framing System

The framing system for the Hotel N.E.U.S consists of steel columns on the first floor mixed with masonry bearing walls. Due to the gathering areas and general openness of the first floor, steel columns are used. These columns only exist on this floor, save for column C12 and E12 that span the first two floors (see Figure 7) Everywhere else in the building, masonry walls are used to support the floor system. The exterior is supported by cold-formed steel (see Figure 7 for sections) Bays are typical except for on the second floor where an opening exists for an open ceiling breakfast region. The longest bearing wall is about 28' long, located on column line 9 near the center of the building where it is widest.

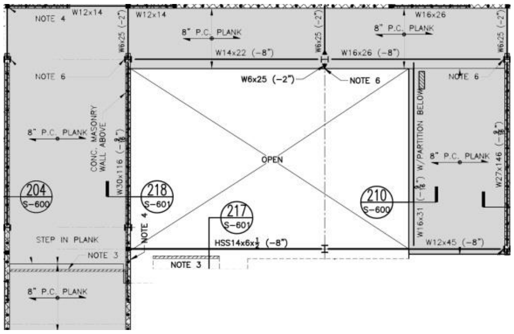
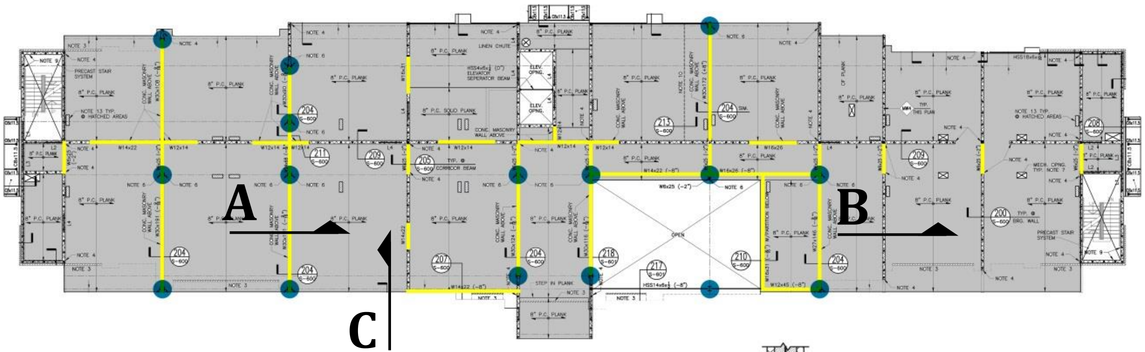
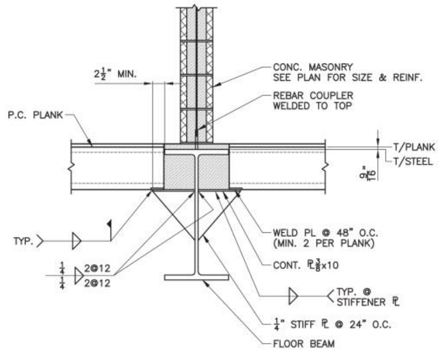
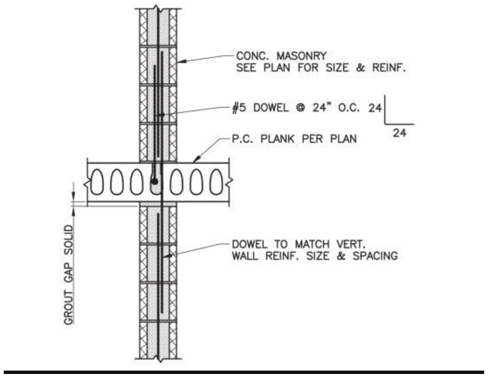


Figure 6: Open section on second floor

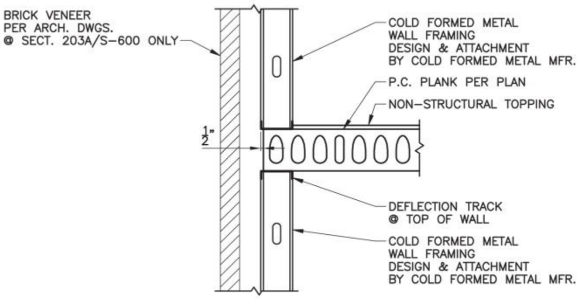




SECTION A- Beam carrying masonry wall



SECTION B- Plank on masonry wall



SECTION C- Plank resting on cold-formed steel at exterior

Figure 7: Second Story framing
Yellow indicates beams
Blue indicates columns

Lateral System

In the Hotel N.E.U.S, the lateral system consists is the same as the gravity system. Reinforced masonry shear walls provide the resistance to lateral loads applied to the building. The masonry is 8" wide with #5 bars at 24" on center. Cells with reinforcement are grouted solid. As with the gravity system, these walls are controlled by the fact that the first floor requires a space without obstructions. Therefore the shear walls are located in an irregular pattern shown in Figure 8. Due to the slenderness of the building, much more resistance is required perpendicular to the long side of the building.

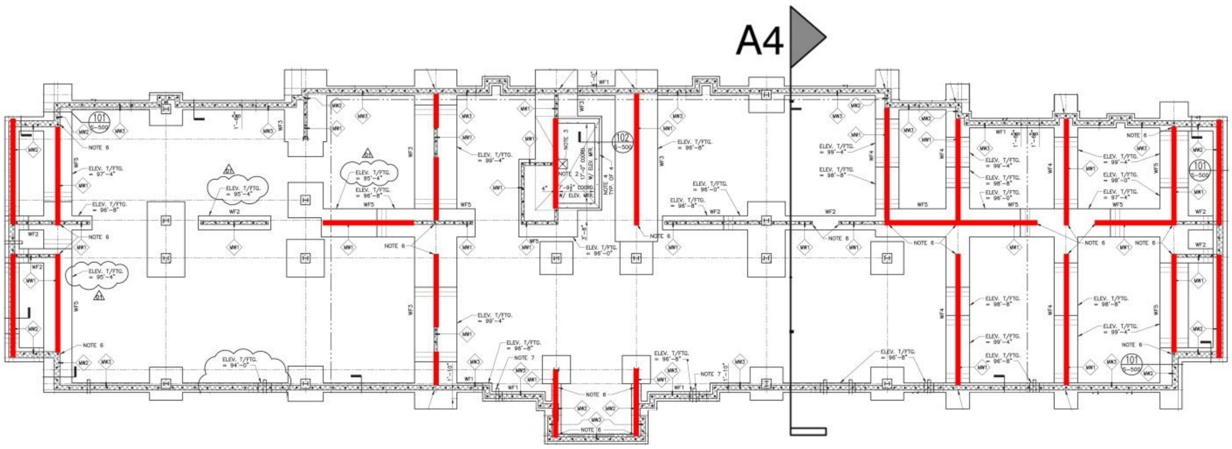


Figure 8: Location of shear walls on foundation plan

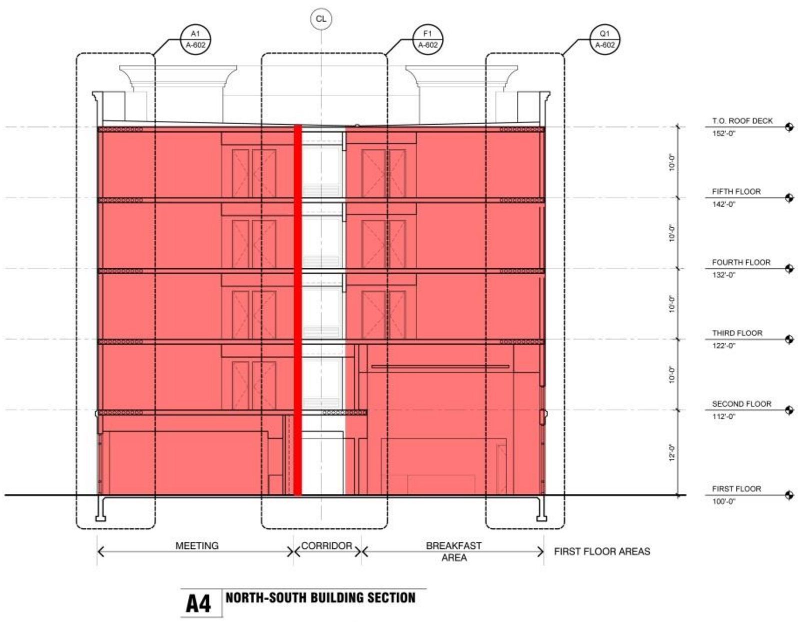
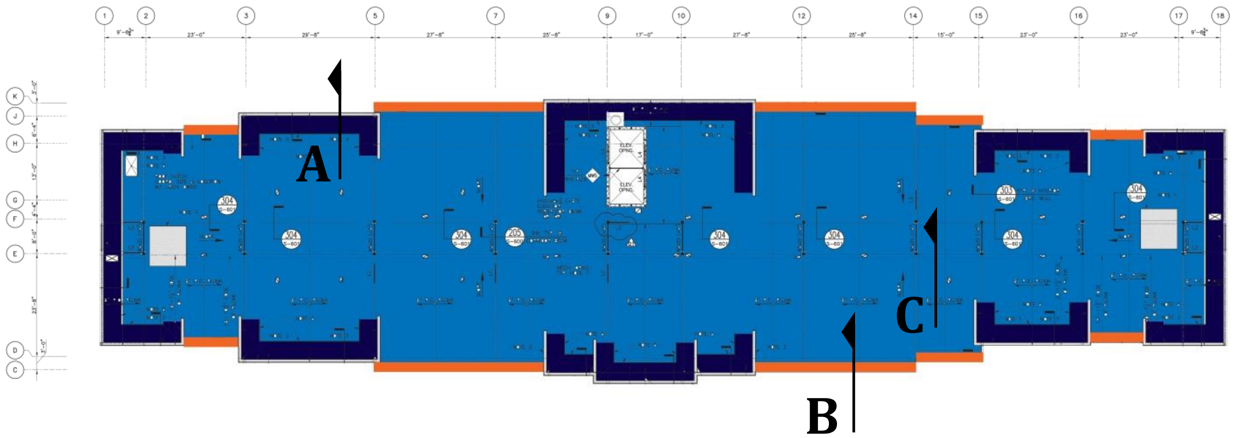
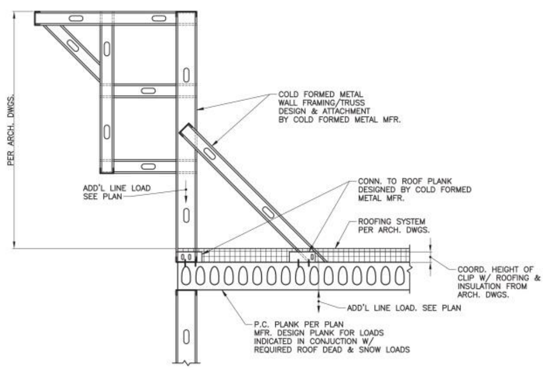


Figure 9: Section showing orientation of shear walls.

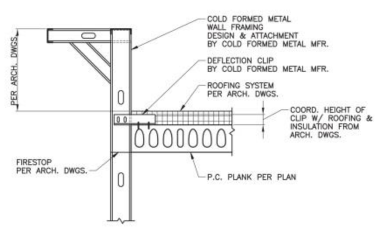
Roof System

As with the floor system, the roof is constructed of 8" Hollowcore Precast plank with insulation on top. A parapet constructed of cold-formed steel encloses the entire perimeter and is to 8'-8" high. Mechanical units weighing 4,000 lbs each are located at either end of the roof.

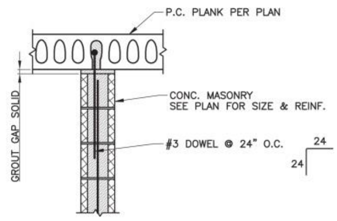




SECTION A- 8'-8" Cold-formed steel parapet wall



SECTION B- 5'-0" Cold-formed steel parapet wall



SECTION C- Roof plank on top of masonry wall

Figure 10: Roof layout.
Blue- 8" Hollowcore Precast Plank
Orange- 5'-0" Cold-formed steel parapet wall
Dark Blue- 8'-8" Cold-formed steel parapet wall

Materials

Listed in Figure 11 are the materials used in the construction of the Hotel N.E.U.S. They were gathered from the structural engineer's general notes and specifications.

Shallow Foundations Wall Footing Capacity	
Width	Allowable Bearing Pressure
2'-0"	4,100 PSF
3'-0"	4,600 PSF
4'-0"	4,500 PSF
5'-0"	3,800 PSF
6'-0"	3,250 PSF
7'-0"	2,800 PSF
8'-0"	2,500 PSF

Column Footing Capacity	
Width	Allowable Bearing Pressure
3'-0"	4,600 PSF
4'-0"	4,500 PSF
5'-0"	3,800 PSF
6'-0"	3,250 PSF
7'-0"	2,800 PSF
8'-0"	2,500 PSF
9'-0"	6,650 PSF
10'-0"	6,250 PSF
11'-0"	5,500 PSF

Reinforced Concrete	
Type	Design Compression Strength (f'c)
Foundations and Concrete Fill	3,000 PSI
Walls	4,000 PSI
Slabs and Grade	4,000 PSI
Reinforcement	
Deformed Bars	ASTM A625 GRADE 60
Deformed Bars (weldable)	ASTM A706, GRADE 60
Welded Wire Fabric	ASTM A185

Figure 11: Material Standards used in Hotel N.E.U.S.

Masonry	
Mortar	ASTM C270
	Type M for all F'm = 2,500 PSI, Type S for all structural masonry
Grout	F'c = F'm but no less than 2,000 PSI

Face Brick
ASTM C216, Grade SW, Type FBS absorption not more than 9% by dry weight per ASTM C67.

Structural Steel	
W shapes	ASTM 992
M, S, C, MC, and L shapes	ASTM A36
HP shapes	ASTM A572, GRADE 50
Steel Tubes (HSS shapes)	ASTM A500, GRADE B
Steel Pipe (Round HSS)	ASTM A500, GRADE B
Plates and Bars	ASTM A36
Bolts	ASTM A325, TYPE 1, 3/4" U.N.O.

Galvanized Structural Steel	
Structural Shapes and Rods	ASTM A123

Precast Concrete	
Type	Design Compression Strength (f'c)
Reinforcement (deformed)	ASTM A 615/A 615M, Grade 60
Welded Wire Reinforcement:	ASTM A 185
Pretensioning Strand	ASTM A 416/A 416M, Grade 250 or Grade 270, uncoated, 7-wire, low-relaxation strand wire or ASTM A 886/A 886M, Grade 270, indented, 7-wire, low-relaxation strand
Portland Cement	ASTM C 150

Figure 12: Material Standards used in Hotel N.E.U.S.

Design Codes

Because of the wide variety of materials used on this project there are also many different codes to abide by. These are listed in Figure 13. The codes used for analysis in this thesis are listed in Figure 14. For a list of other codes used see Appendix A.

Design Codes	
Reinforced Concrete	Building Code Requirements for Structural Concrete (ACI 318, latest)
	Specifications for Structural Concrete (ACI 301, latest)
Masonry	Building Code Requirements for Masonry Structures (ACI 530)
	Specifications for Masonry Structures (ACI 530.1)
Precast Concrete	Building Code Requirements for Structural Concrete (ACI 318, latest)
	Commentary (ACI 318R, latest)
	PCI Design Handbook - Precast and Prestressed Concrete (PCI MNL 120)
Structural Steel	Specification for Structural Steel Buildings (ANSI/AISC 360-05)
Metal Decking	Steel Roof Deck Specifications and Load Tables (Steel Deck Institute, latest edition)
Cold Formed Steel	Most current edition of the "North American Specification for the Design of Cold-Formed Steel Framing"
Wind and Seismic	ASCE 7-05
Loads	International Building Code 2009

Figure 13: Codes used by the engineer of record to design this structure

Thesis Analysis Codes	
Reinforced Concrete	Building Code Requirements for Structural Concrete (ACI 318-11)
Precast Concrete	PCI Design Handbook - Precast and Prestressed Concrete (PCI MNL 120)
Structural Steel	AISC Steel Manual 14th Edition, A
	AISC 360 2010 Specification for Structural Steel Buildings
Wind and Seismic	ASCE 7-05
Loads	International Building Code 2009
Masonry	Building Code Requirements forMasonry (ACI 530-05)

Figure 14: Codes used for thesis

Gravity Loads

The dead loads for this structure were either provided by the engineer of record or assumed by referencing structural handbooks. The plank weight was obtained using PCI Manual 120 and Masonry walls were determined using NCMA TEK 14-13B. The density was assumed as 105 lb/ft^3 as it was described as “medium” in the specifications. The topping is to level the surface since the camber of the plank will cause it to be uneven. These loads prove to be very similar to the overall load used by the engineer of record as the spot checks performed give good results.

Dead Loads	
Location	Load (psf)
8" Precast Plank	56
3/4" Topping	6
MEP/Misc.	5
Ceiling	3
Roof Insulation	12
C.F. Studs	5
Roof	20
Masonry Walls	43-53

Figure 15: Dead Loads

Live loads were listed in the general notes on sheet S001. All of them were in accordance with the International Building Code 2009. Due to the typical layout of floors in a hotel, 40 psf was used on the entire floor except for stairwells on floors two through five. The engineer of record used live load reduction when determining loads for the beams, columns, and column footings. However, there was no reduction for the wall footing.

Live Loads			
Location	Design Live Load (psf)	IBC 2009 Live Load (psf)	Reference Note
Public Areas	100	100	Residential - hotels and multifamily dwellings - public rooms and corridors serving them
Guest Rooms and Corridors	40	40	Residential - hotels and multifamily dwellings - private rooms and corridors serving them
Paritions	20	20	
Stairs	100	100	Stairs and exits - all other
Roof	20	20	Roofs - ordinary flat, pitched, and curved roofs

Figure 16: Live Loads

Thesis Objectives

Structural Depth

Problem Statement

The Hotel N.E.U.S. utilizes an unusual gravity system that is a hybrid of steel and masonry. The first floor has large open spaces that call for steel framing while the second through fifth floors are guest rooms where the masonry walls can be used as partitions. In Technical Report 2, the use of alternative gravity framing was explored to see whether there was a method that could use the same material from ground to roof and eliminating the need to provide special treatment to the ground floor. Therefore the first issue to address is to redesign the gravity floor system and framing to accommodate the spacious first floor while being able to successfully provide the same sized rooms in floors two through five.

The lateral system of the Hotel N.E.U.S. is composed of masonry shear walls. In Technical Report 3, an in depth investigation of the lateral loads and the ability of the shear walls to resist them was performed. It was found that there was an opportunity to provide more direct and torsional resistance since there was only one shear wall in that direction. The second item to address is finding a lateral system that will work well with the gravity framing and provide the necessary resistance in the long direction.

Solution

Since the Hotel N.E.U.S. is already partially constructed of steel it seems adequate to investigate how a full steel system could be utilized. For this project, the gravity system was redesigned with composite steel framing and concrete on metal deck. An efficient column grid was developed to accommodate the first floor spaces and partitioned guest room floors while maintaining the same room areas.

The lateral system was redesigned using braced frames in place of the shear walls in the short direction. The diagonal members can be enclosed in the partitioned walls in the same locations as the shear walls. In the long direction, moment frames were implemented to allow for windows and door openings.

ASCE 7-05 was used to calculate all the loads for the Hotel. A typical bay was selected and gravity connections will be designed for a beam, girder to column flange/web. For the lateral system, a typical moment and braced connection will be designed. A RAM model will be created in order to verify the size of members.

Breadth 1: Enclosure

The exterior façade of the Hotel N.E.U.S. is mainly constructed of Exterior Insulation and Finishing System (EIFS) which is known to have poor performance especially in wet regions such as the Northeast U.S. The existing enclosure was examined for advantages and disadvantages. A study of an alternative façade material and building enclosure was performed. The new material was selected in conjunction with the Architecture Breadth in order to create a fresh look for the building. A comparison of the two systems shows why the new one was selected and typical details were created for the updated enclosure.

Breadth 2: Architecture

By redesigning the framing and enclosure of the Hotel N.E.U.S., there will be an impact on the architecture. The aesthetics of hotels in today's world are made to represent their "brand" so you can recognize them from a distance and associate them with the qualities of that "brand". For this project, the Hotel N.E.U.S. is going to break away from this idea. The sleek style that was selected is delivered through the use of metal panels. The exterior is redesigned with all new colors, windows, parapet, and entrance overhang. Also, due to the new framing, the pool area now contains a braced frame. In order to conceal the structure, a new room design was investigated. A model was created in Sketch-Up and Revit to complete this study.

MAE Coursework

Knowledge gained from AE 530-*Computer Modeling of Building Structures* was used to create a RAM model for this study. STAAD Pro was also used to evaluate individual frames.

The information from AE 534-*Steel Connections* was used to design gravity and lateral connections.

AE 537- *Building Performance Failures and Forensic Techniques* provided knowledge of enclosures and overall building science that helped aid the decisions made for the breadth study.

AE 542-*Building Enclosures Science Design* was used to evaluate the existing enclosure along with alternatives.

Structural Depth

Introduction

Masonry construction typically dominates the scene for hotels. This is due to the cost effectiveness and how well it works for partitioned floors. However, the Hotel N.E.U.S. contains large open spaces on the first floor for things such as a swimming pool and breakfast area. To accommodate these spaces, steel framing is used only on the first floor. This makes for minimal shear wall placement. The lack of shear walls in the long direction of the building was a cause for concern in Technical Report III.

In order to keep these areas open and address the lateral issues, the framing was changed to steel. This increases the floor to floor but also decreases construction time. The column layout remains unchanged from the previous design. This means placement of foundations can remain intact and can be resized if needed. However, the overall building weight will be significantly decreased which may decrease the size of foundations. By decreasing the weight, wind loads may now control over seismic loads.

Gravity

The goal of this redesign was to try and keep the building's rooms and perimeter the same. Since the columns in the existing design were precisely located on the ground level to permit large areas they were unaltered. On floors 2 through 5, the bay sizes were easily met with steel. By spanning the beams in the short direction, girders are placed on the exterior of the building which will help with holding up the new façade investigated in Breadth 1. Girders are also located on each side of the 8'-0" wide hallway. Beams spanning the hallway have a much smaller tributary area and can be kept at minimal size. By doing this there is a maximum amount of hallway ceiling space for mechanical equipment to run the length of the building. Figure 18 shows the typical layout of floors 2-5 and Figure 17 shows the ground floor. For beam and column sizes refer to Appendix D.

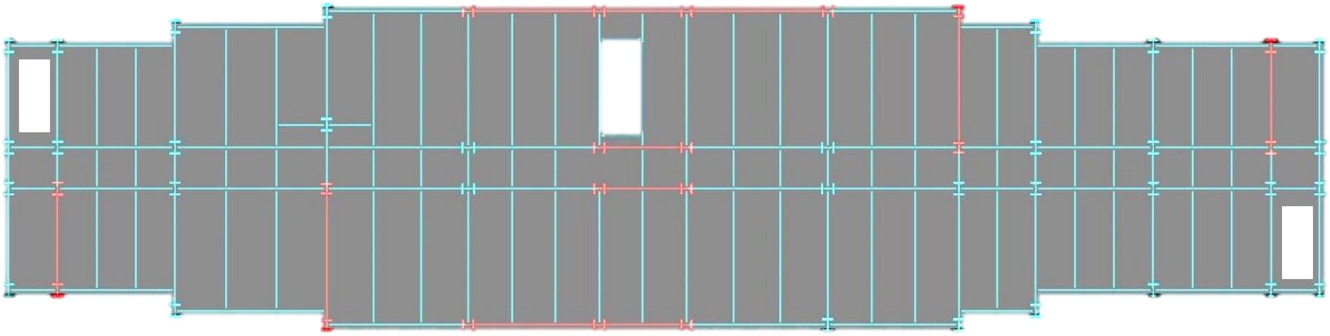


Figure 17: First Floor Plan

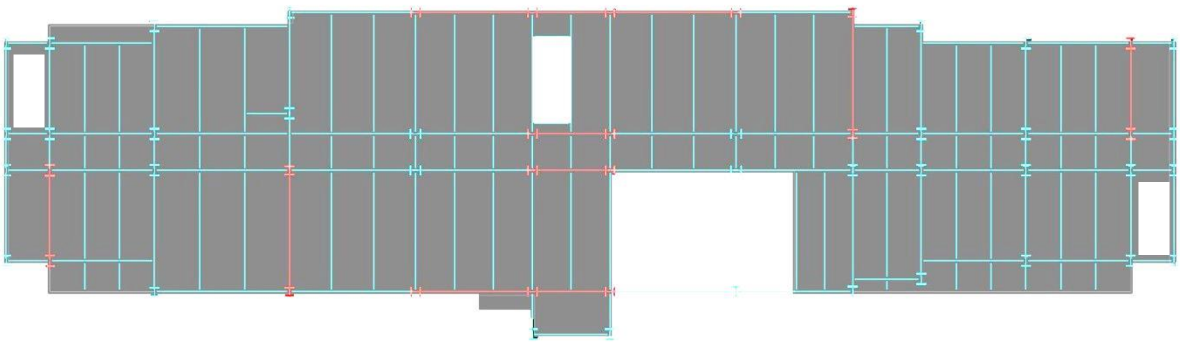


Figure 18: Second through Fifth Floor Plan

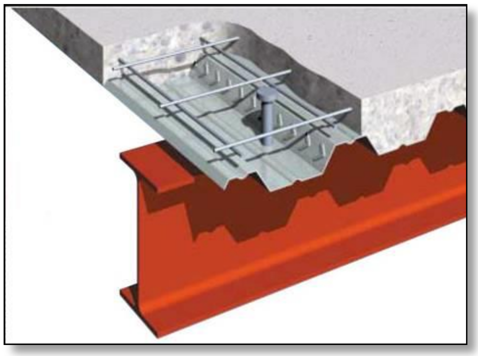


Figure 19: Composite Steel 3D section

The system selected for the gravity loads is composite steel. This allows for shallow beams that meet the span requirements for the bay sizes. It is constructed by pouring concrete over metal deck that acts in composite with steel beams through shear studs welded to the top of the beams. The poured concrete embeds the studs and transfers load to the beam more effectively than in there were no studs. Vulcraft 3VLI22 Deck was selected to meet fire rating and unshored construction span. This deck is 5.5" thick with 3" flutes.

RAM Structural System was the main computer modeling program used to check designs. A typical bay was selected and designed by hand and then compared to the computer designed output. The typical beam size was a W14x22 with 10 studs. A typical exterior girder was a W18x40 with 14 studs. These sizes were identical with RAM's design, with a few less studs being the only difference. A typical interior column was checked as well. The size obtained by hand was a W10x33 which was also matched with RAM's design. All hand calculations for gravity design can be found in Appendix D.

A spot of concern was that of the pool. Due to the building perimeter stepping back on the second floor, the column line falls in the middle of the pool room. To address this, a transfer girder spans across the room and picks up the column. This load is distributed to another column on the exterior of the pool room and since it's only one story high, the increase in size is no issue. See the next section for the connection of girder to column.

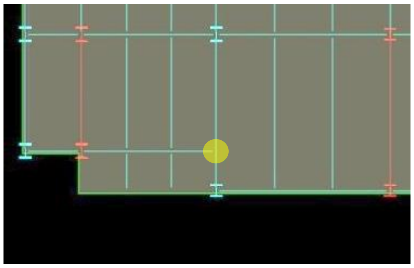


Figure 20: Location of Column on Transfer Girder

Connections

In a single bay of a building there can be several different connections. To apply the knowledge gained in AE 534- *Steel Connections*, three different gravity connections were designed for the Hotel N.E.U.S. See Appendix E for hand calculations and the limit states checked for each connection.

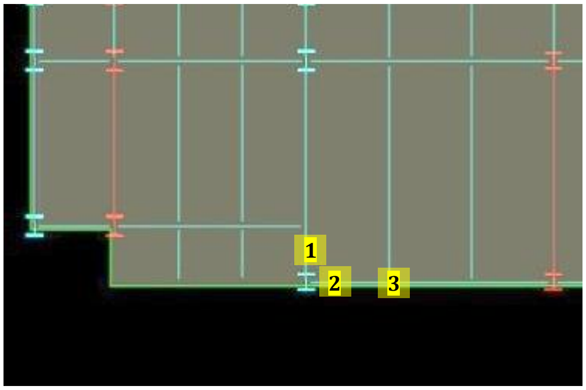


Figure 21: Location of Gravity Connections

1.) The first connection designed is the transfer girder above the swimming pool connecting to a column. A W24x68 girder connects to the flange of a W12x79 column. A double angle was selected to hold the large shear value associated with this layout. Figure 22 shows the designed connection.

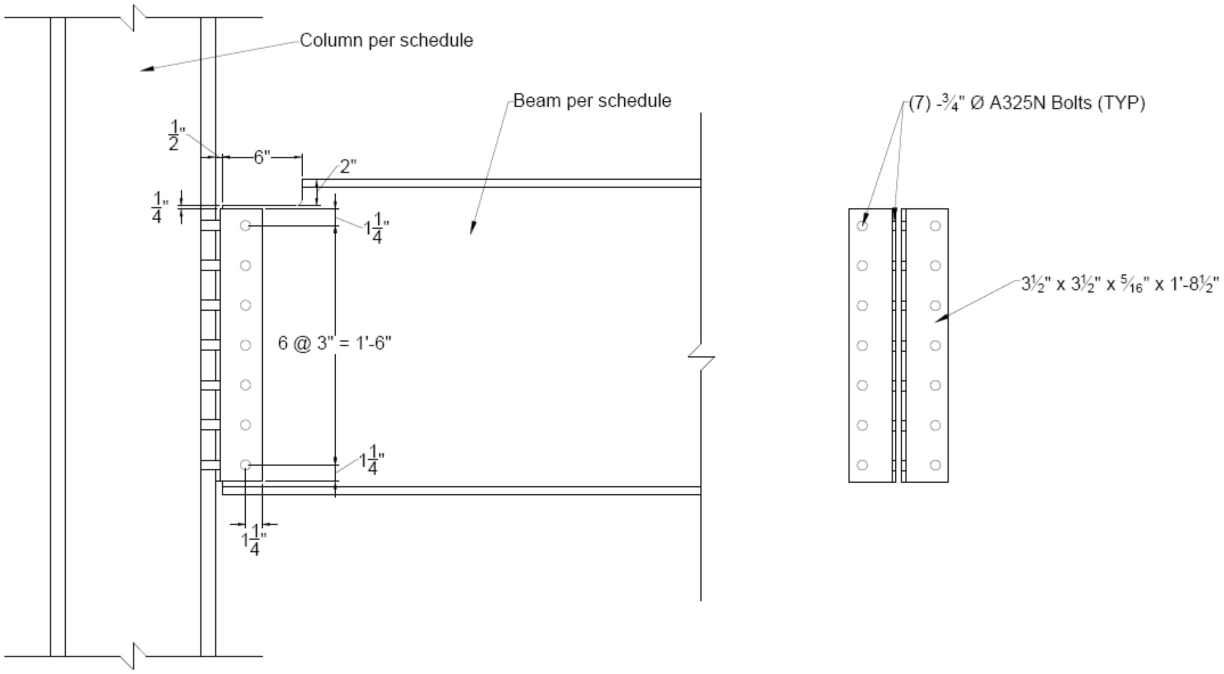


Figure 22: Girder to Column Flange connection

2.) A W18x40 exterior girder frames into the column web of the W12x79 mentioned above. To account for the flange width, an extended shear plate can be used. See Figure 23 for the second connection.

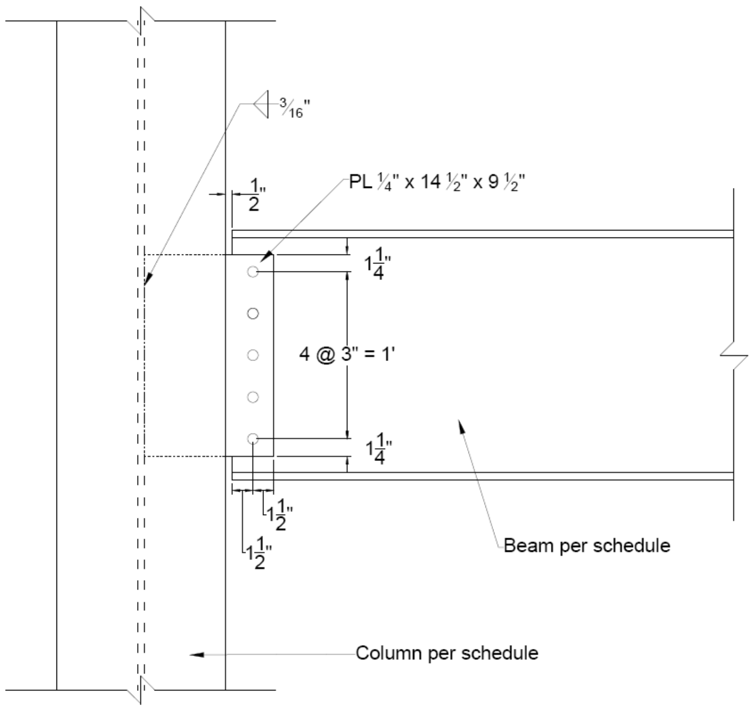


Figure 23: Girder to Column Web connection

3.) A typical W16x26 infill beam frames into W18x40 girders on both ends. For ease of construction a single angle will be welded to the beam and bolted to the girder web. See Figure 24 for the third connection type.

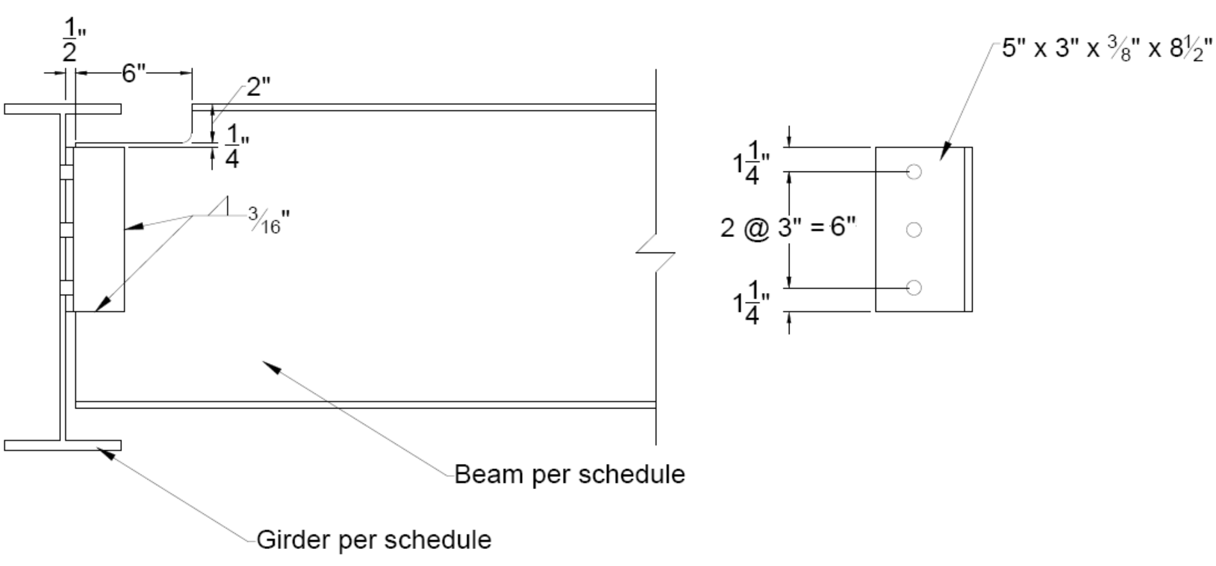


Figure 24: Beam to Column connection

Lateral

The first step in designing the lateral system is calculating the lateral loads. As stated earlier, the floor to floor height increased by 1'-0" for floors 2-5. The parapet was also decreased to 5'-0" along the entire perimeter for architectural reasons. This makes the total building height equal to 61'-0". Unsurprisingly, the wind controlled direct overturning in the short (N-S) direction. Earthquake still controls in the long (E-W) direction despite the decrease in base shear.

Wind

Wind loads for the Main Wind Force Resisting System were calculated using Method 2 in ASCE 7-05. The building was idealized as a rectangle with dimensions 258'x61'. A summary of the wind data and forces can be found in Table 1-5. See Appendix B for more calculations.

Wind Load Data		
Design Wind Speed	V	90
Directionality Factor	Kd	0.85
Occupancy Category	I	II
Importance Factor		1
Exposure Category		C
Topographic Factor	Kzt	1
Internal Pressure Coefficient	Gcpi	+/-0.18
Gust Factor	G	.85

Velocity Pressures							
Level	Elevation	K_z	K_{zt}	K_d	V^2	I	q_z
	61	1.1340	1	0.85	8100	1	20.0
Parapet	56	1.114	1	0.85	8100	1	19.6
5	45	1.065	1	0.85	8100	1	18.8
4	34	1.004	1	0.85	8100	1	17.7
3	23	0.924	1	0.85	8100	1	16.3
2	12	0.85	1	0.85	8100	1	15.0
Ground	0	0.85	1	0.85	8100	1	15.0

Table 1: Wind Data and Velocity Pressures

Wind Pressures N-S								
Location	Level	Distance (ft)	Velocity Pressure (psf)	External Pressure (psf)	Internal Pressure (psf)		Net Pressure (psf)	
			q_p / q_z / q_h	p_p / p_z / p_h (psf)	Positive (GCp)	Negative (GCp)	Positive	Negative
Windward		61	20.0	30.0	1.5		30.0	
	Parapet	56	19.6	13.4	2.70	-2.70	16.0	10.7
	5	45	18.8	12.8	2.70	-2.70	15.5	10.1
	4	34	17.7	12.0	2.70	-2.70	14.7	9.3
	3	23	16.3	11.1	2.70	-2.70	13.8	8.4
	2	12	15.0	10.2	2.70	-2.70	12.9	7.5
	Ground	0	15.0	10.2	2.70	-2.70	12.9	7.5
Leeward	Parapet	61	20.0	-20.0	-1.0		-20.0	
	G-5	56	15.0	-8.9	2.70	-2.70	-6.22	-11.61
Side	All	Total	15.0	-2.5	2.70	-2.70	0.15	-5.24
Roof	-	0-30.33	15.0	-11.5	2.70	-2.70	-8.76	-14.16
	-	30.33-60.67	15.0	-11.5	2.70	-2.70	-8.76	-14.16
	-	60.67-121.33	15.0	-6.4	2.70	-2.70	-3.67	-9.06
	-	>121.33	15.0	-3.8	2.70	-2.70	-1.12	-6.52

Table 2: Wind Pressures N-S

Wind Pressures E-W								
Location	Level	Distance (ft)	Velocity Pressure (psf)	External Pressure (psf)	Internal Pressure (psf)		Net Pressure (psf)	
			q_p / q_z / q_h	p_p / p_z / p_h (psf)	Positive (GCp)	Negative (GCp)	Positive	Negative
Windward		61	20.0	30.0	1.50		30.0	
	Parapet	56	19.6	13.4	2.70	-2.70	16.0	10.7
	5	45	18.8	12.8	2.70	-2.70	15.5	10.1
	4	34	17.7	12.0	2.70	-2.70	14.7	9.3
	3	23	16.3	11.1	2.70	-2.70	13.8	8.4
	2	12	15.0	10.2	2.70	-2.70	12.9	7.5
	Ground	0	15.0	10.2	2.70	-2.70	12.9	7.5
Leeward	Parapet	61	20.0	-20.0	-1.0		-20.0	
	G-5	56	15.0	-8.9	2.70	-2.70	-6.2	-11.6
Side	All	Total	15.0	-6.4	2.70	-2.70	-3.7	-9.1
Roof	-	0-28.5	15.0	-16.6	2.70	-2.70	-13.9	-19.2
	-	>h/2	15.0	-7.1	2.70	-2.70	-4.4	-9.8

Table 3: Wind Pressures E-W

Wind Forces N-S						
Level	Elevation (ft)	Tributary Area (ft^2)		Wind Force (k)	Story Shear (k)	Overturning Moment (ft-k)
		Above	Below			
	61	0	645	32.2	32.2	1966
Parapet	56	645	1419	63.8	96.1	3574
5	45	1419	1419	62.4	158.4	2806
4	34	1419	1419	60.5	218.9	2056
3	23	1419	1419	58.1	277.0	1336
2	12	1419	1548	57.9	334.9	695
Ground	0	1548	0	0.0	334.9	0
						12434

Table 4: Wind Forces N-S

Wind Forces E-W						
Level	Elevation (ft)	Tributary Area (ft^2)		Wind Force (k)	Story Shear (k)	Overturning Moment (ft-k)
		Above	Below			
	61	0	153	7.6	7.6	465
Parapet	56	153	336	15.1	22.7	845
5	45	336	336	14.7	37.5	663
4	34	336	336	14.3	51.8	486
3	23	336	336	13.7	65.5	316
2	12	336	366	13.7	79.2	164
Ground	0	366	0	0.0	79.2	0
						2940

Table 5: Wind Forces E-W

Seismic

The total building weight was reduced by almost 5000 kips for a total of 5605 kips. To calculate seismic loads, the Equivalent Lateral Force Procedure that is described in ASCE 7-05 was used. A summary of seismic data and forces can be found in Table 6. See Appendix C for more calculations.

Seismic Load Data		
Occupancy Category	-	II
Site Class	-	D
Seismic Load Importance Factor	I_e	1
Site Class Coefficient	S_s	0.125
	S_1	0.049
Spectral Response Coefficient	F_a	1.6
	F_v	2.4
	S_{DS}	0.1333
	S_{D1}	0.0784
Seismic Design Category	-	B
Long Period Transition Period	T_L	12
Response Modification Factor	R	3.25
Fundamental Period (N-S)	Ta	0.930
Response Modification Factor	R	3
Fundamental Period (E-W)	Ta	1.900

Table 6: Seismic Load Data

Total Building Weight					
Level	Area (ft^2)	Load (k)	Perimeter (ft)	Enclosure (k)	Total (k)
Ground	13133	972	640	96.00	1067.84
2	14370	1063	640	96.00	1159.38
3	14370	1063	640	96.00	1159.38
4	14370	1063	640	96.00	1159.38
5	14370	963	640	96.00	1058.79
				Total Weight(k)	**5604.77**

Table 7: Building Weight

Seismic Response Coefficient Cs								Base Shear	
Type	C_u	T_a	T	C_{smin}		C_{smax}	C_s	Weight	V (k)
N-S	1.7	0.409	0.696	0.008	0.010	**0.038**	**0.044**	5604.8	**210**
E-W	1.7	0.701	1.192	0.008	0.010	**0.022**	**0.044**	5604.8	**123**

Table 8: Base Shear

Vertical Force Distribution (N-S)								
Level	Weight (k)	Height (ft)	k	$w_x h_x^k$	Distribution Factor	Story Force (k)	Story Shear (k)	Overturning Moment (ft-k)
	w_x	h_x			C_{vx}	$F_x=C_{vx}V$		
5	1058.79	56	1	59292.24	0.31	65.55	65.55	3670.59
4	1159.38	45	1	52172.10	0.27	57.68	123.22	2595.38
3	1159.38	34	1	39418.92	0.21	43.58	166.80	1481.61
2	1159.38	23	1	26665.74	0.14	29.48	196.28	678.00
Ground	1067.84	12	1	12814.10	0.07	14.17	210.44	169.99
				190363.10	1.00			8595.57

Vertical Force Distribution (E-W)								
Level	Weight (k)	Height (ft)	k	$w_x h_x^k$	Distribution Factor	Story Force (k)	Story Shear (k)	Overturning Moment (ft-k)
	w_x	h_x			C_{vx}	$F_x=C_{vx}V$		
5	1058.79	56	1	59292.24	0.31	38.28	38.28	2143.87
4	1159.38	45	1	52172.10	0.27	33.69	71.97	1515.88
3	1159.38	34	1	39418.92	0.21	25.45	97.42	865.36
2	1159.38	23	1	26665.74	0.14	17.22	114.64	396.00
Ground	1067.84	12	1	12814.10	0.07	8.27	122.91	99.28
				190363.10	1.00			5020.39

Table 9: Seismic Forces

Summary

Although the seismic weight was reduced by a large amount, the seismic loads still control direct overturning in the East-West direction. The large tributary area of the North-South face causes wind loads to control in that direction. The forces were found to be within an acceptable percentage compared to those found by using RAM. Therefore the loads generated by the program were used in optimizing members with RAM.

Design Process

There were several goals associated with the design of the lateral system. The first was to successfully arrange the lateral resisting elements so that the layout of the rooms in the hotel was unaffected. Although steel is not the most cost effective alternative, keeping the floor plan, windows, and doors unchanged will help to keep the cost down. To achieve this, braced frames were used in the short (N-S) direction. Their placement can be concealed since they will be located where the existing plan calls for shear walls. The placement of the braces also considered the transfer of loads through the diaphragm. Ideally there would be two braces on either end of the building and the resistance provided would satisfy the load requirements. However, there are general rules for the spacing of shear walls and they were applied for the braces as well. In ACI 530 chapter 5, Empirical Design of shear walls states that shear walls should be not further apart than 5 times their length. This would not be met if the braces were placed at either end of the 268' long building. Taking this into consideration led to the layout shown in Figure 25. One brace on the first floor is not located where a shear wall used to be and the solution to this issue can be found in Breadth 2. A "K" brace was selected to allow for a doorway or entrance through the middle, specifically those near the swimming area.

In the long direction (E-W) ordinary moment frames were selected to resist lateral loads. These frames provide the most flexibility in floor plan and that is why they were selected. Windows and doors can penetrate infill walls between the frames with no issues. The second concern when designing this system was the drift for these frames. Braced frames are significantly stiffer than moment frames and do not suffer from the same problem. The moment frames were pinned at the base since it is difficult to achieve a true fixed base and because of the unusual soil conditions of this site. Also, due to the opening on the second floor (which can be seen in Figure 25), the frame on the front of the building was not extended to that bay. Since the diaphragm is absent at that floor, no load would be transferred to the frame. Therefore the decision was made to add two interior frames. These assisted in hindering the first story drift as well.

After investigating multiple layouts and configurations it was decided that the moment frames be located at the outermost wall in the long direction. The building steps back multiple times and the columns do not align in order to form frames until the middle region (see Figure 25 for floor plan). By placing the frames in the middle they avoided intersection with the braces as well. Although it can be done, intersecting a braced and moment frame was avoided to prevent adverse effects on that column. Stiffness would also be significantly decreased in one direction due to the column bending in its weak axis as well.

Another goal of this design was to keep the center of rigidity as close to the center as possible. In the existing design, the limited placement of the shear walls caused the center of rigidity to be offset a substantial amount. There was also only one shear wall in the long direction which was inefficient at resisting the loads. The placement of the braced frames and moment frames makes the center of rigidity very close to the center of the building, providing an efficient and fairly even distribution of forces. Figure 26 shows a three dimensional view of the lateral system with the center of rigidity at each floor shown as a blue dot. In Table 10 the center of rigidity is compared to the old layout's values and shows how the eccentricity was minimized.

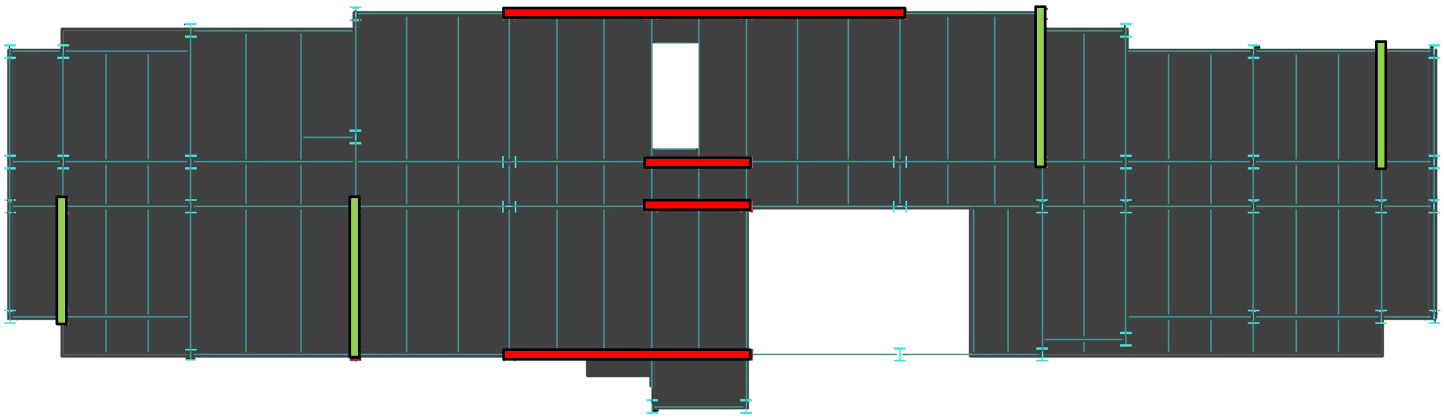


Figure 25: Location of Frames

- Braced Frame
- Moment Frame

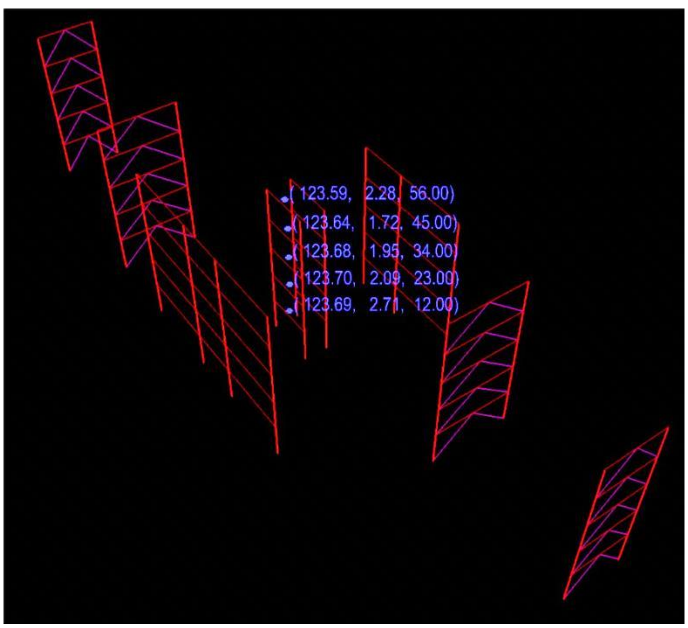
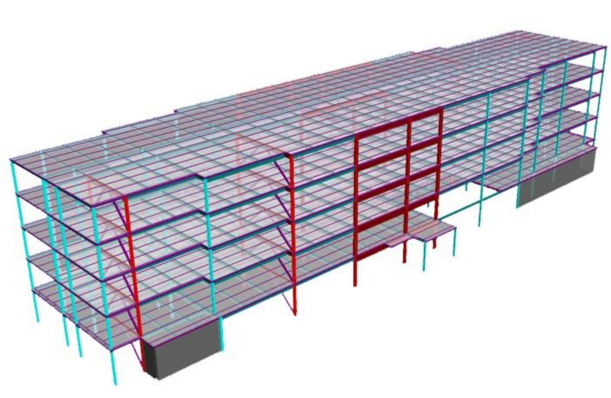


Figure 26: Center of Rigidity in 3D

Center of Rigidity Comparison				
Level	X		Y	
	New	Old	New	Old
5	123.59	161.57	2.28	-5.05
4	123.64	160.71	1.72	-4.5
3	123.68	159.13	1.95	-3.51
2	123.7	156.28	2.09	-1.72
1	123.69	151.16	2.71	1.37
*For the Y direction, 0 is equal to 34.667' from Column Line C or the "bottom" of the building"				

Table 10: Center of Rigidity Comparison



Braced Frames

In the short direction of the hotel braced frames were utilized. Braced frames transfer lateral forces from the diaphragm to the braces that run at an angle through the panel. They take purely axial loads, making the frame act like a cantilevered truss. By using this approach, the column and beam sizes are kept close to those used by the gravity system, saving on steel tonnage.

The design of the braces was performed by using RAM Structural System and hand calculations. By modeling the lateral system, over 200 load combinations were compiled and applied. The maximum member loads could then be obtained. These loads were then used and preliminary sizes were evaluated for strength. Hollow Structural Steel was selected to make up the braces and is a common shape for this scenario. A 6" square tube provides the strength and serviceability requirements needed. It is also slimmer than the 8" CMU shear walls that the existing plan calls for, therefore insulation and sheathing can be applied to the partition wall and easily conceals the brace within. Due to the long length of the brace the minimum size of the HSS is a 6x6x3/16 to prevent compression buckling. Figure 28 and 29 show the typical braces (2 of each are used in the building).

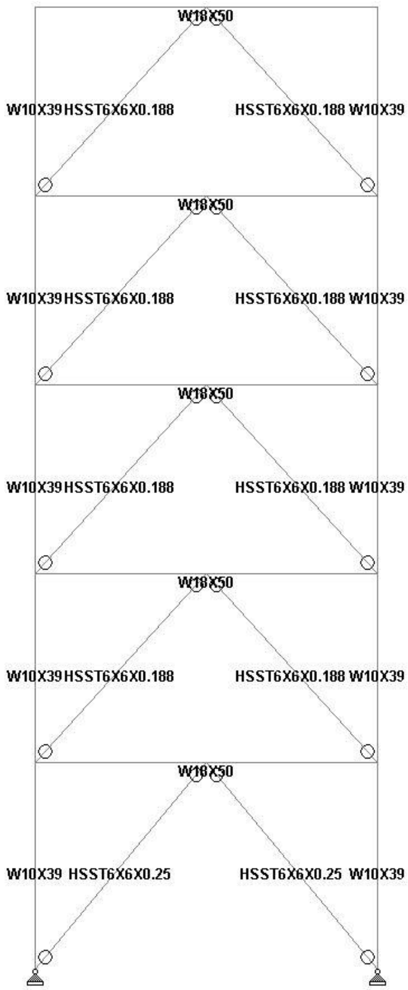


Figure 28: Exterior Brace

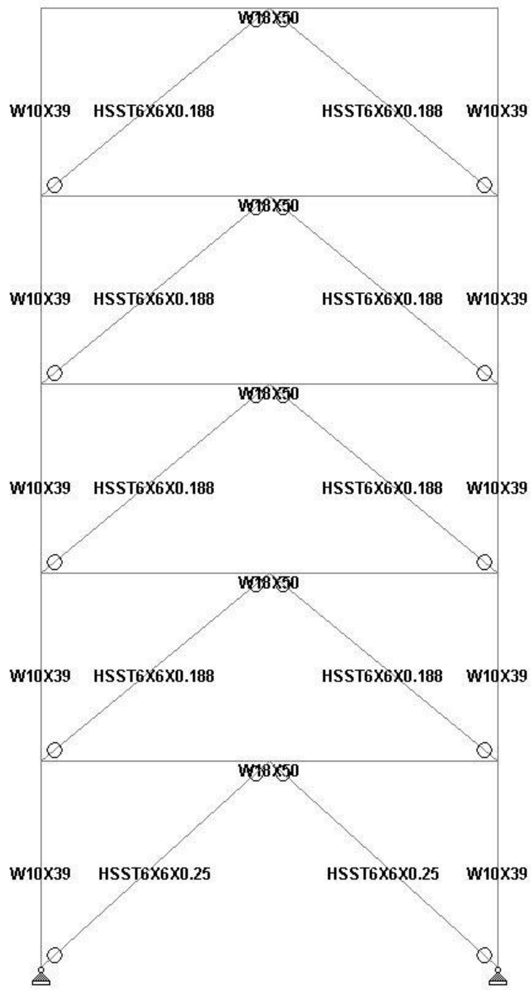


Figure 29: Interior Brace

Brace Connection

A typical braced connection was design to meet M.A.E. requirements. A braced connection can see compression and tension depending on the direction of the load. The uniform force method was used for design. This ensures that no moment will be induced at the interfaces of the connection. The limit states that were evaluated were as follows:

- Brace Limit States:
  - Tension Yield
  - Tension Rupture
- Gusset Plate Limit States:
  - Tension Yield
  - Block Shear
  - Base Metal Strength
  - Local Buckling
- Beam-Column Limit States:
  - Bolts
    - Shear Stress
    - Tensile Stress
  - Angle
    - Shear Yield
    - Shear Rupture
    - Block Shear
    - Bearing/Tear-out
    - Eccentric Weld Strength
- Gusset-Column Limit States:
  - Bolts
    - Shear Stress
    - Tensile Stress
  - Angle
    - Shear Yield
    - Shear Rupture
    - Block Shear
    - Bearing/Tear-out
    - Eccentric Weld Strength
- Gusset-Beam Limit States:
  - Weld Strength
  - Base Metal Strength

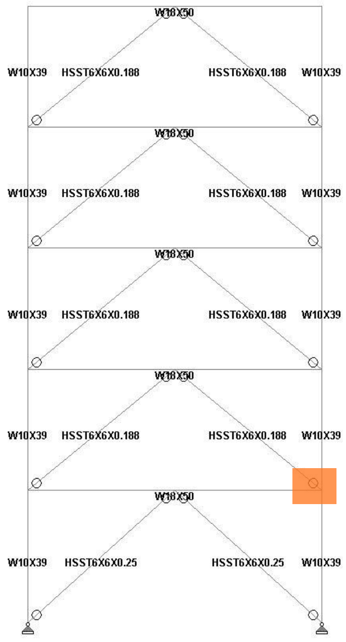


Figure 30: Location of designed braced connection

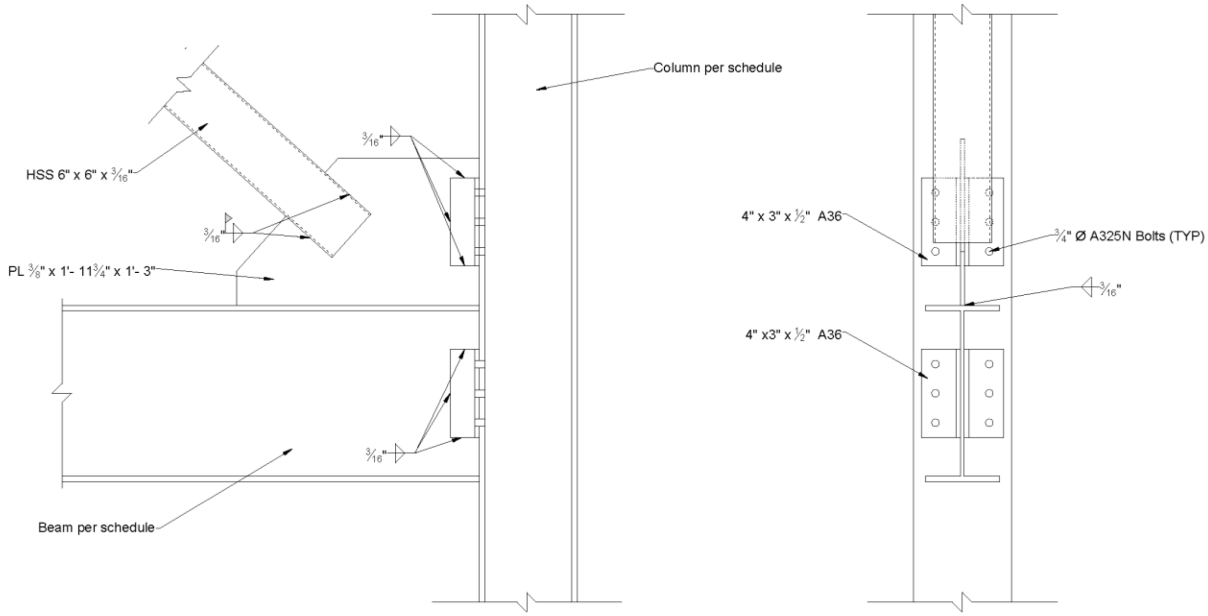


Figure 31: Brace connection

Moment Frames

The long direction of the Hotel was fitted with moment frames for lateral resistance. These proved to be critical since they were not governed by strength, but by drift. In order to design the frames, RAM, Staad Pro, and hand calculations were used.

To start off, the frames were modeled in Staad Pro. A 1 kip load was applied to the top to find the deflection. This deflection was then used calculate the stiffness. Since composite steel acts as a rigid diaphragm, loads are distributed to frames based on their stiffness. The calculated stiffness was used to distribute part of the total lateral load to each frame. The 3 bay frame was checked using the portal method and values were within 20%, proving that RAM allotted the moments to members in a proper manner.

In order to design a frame the Approximate Second Order Analysis (AISC Specification 8) was performed using the aid of AISC Design Guides. This method amplifies first order results from RAM. A leaning column was used to account for the mass on each floor that is used for P-Δ effects. It was found that the strength for preliminary sizes was fine. The controlling factor for design was the first floor drift. Moment frames struggle with drift as they are not nearly as stiff as braced frames. Since wind drift is a serviceability issue, the load factor can be reduced to 0.7. This still controlled over seismic drift because seismic is an ultimate load condition and is expected to move much more. RAM was used to check the upsized members for strength and drift. The frames were optimized with larger members at the base to account for the pinned foundation. The story shear at the top is the smallest and sizes do not need to be quite as large. Figures 32-34 show the optimized designs for the 3 moment frames in the hotel.

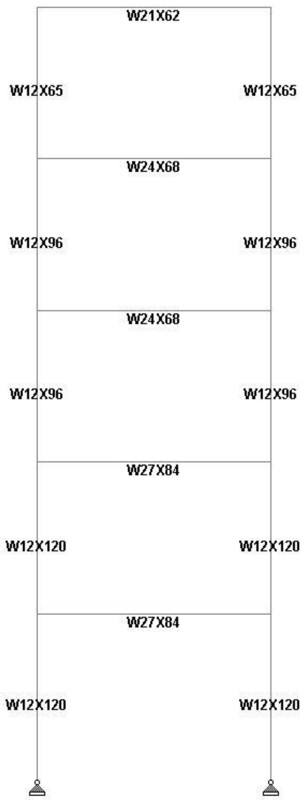


Figure 32: Interior 1 bay Moment Frame

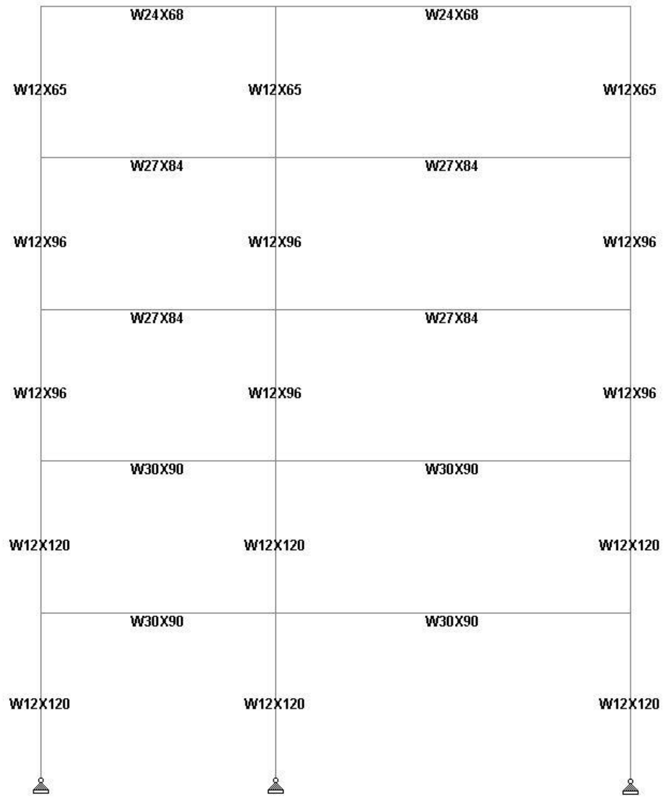


Figure 33: Exterior 2 bay Moment Frame

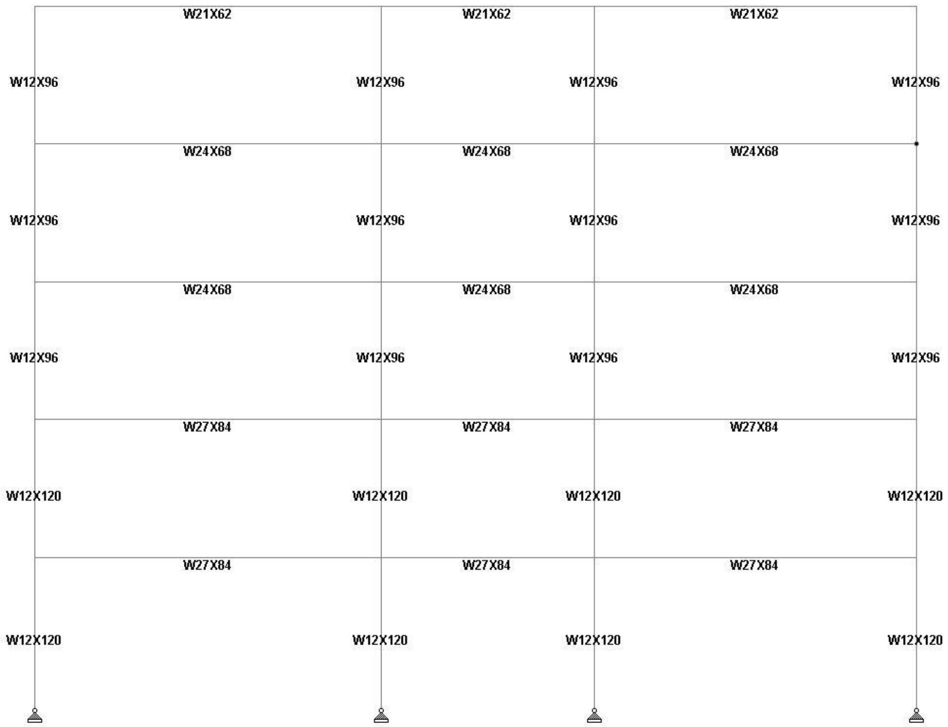


Figure 34: Exterior 3 bay Moment Frame

Moment Connection

A typical moment connection was designed to meet M.A.E. requirements. A flange bolted/web welded connection was used. This will allow for some prefabrication in the shop with quick assembly in the field. No stiffeners were required because the member sizes were increased to limit drift and the moments are not very large. The limit states that were evaluated were as follows:

- Beam-to-Column Flange Limit States:
  - Shear Yield
  - Shear Rupture
  - Block Shear
  - Bearing/Tear-out
  - Weld Strength
- Tension Plate Limit States:
  - Bolt Shear
  - Plate Bearing/Tear-out
  - Flange Bearing/Tear-out
  - Flange Bending
  - Tension Yield
  - Tension Rupture
  - Plate Block Shear
  - Flange Block Shear
  - Bearing
  - Weld Rupture
  - Column Flange Thickness
- Compression Plate Limit States:
  - Local Buckling
- Beam Flexural Strength:
  - Reduced Bending Capacity
- Column Limit States:
  - Flange Bending
  - Web Yielding
  - Web Crippling
  - Panel Zone Shear

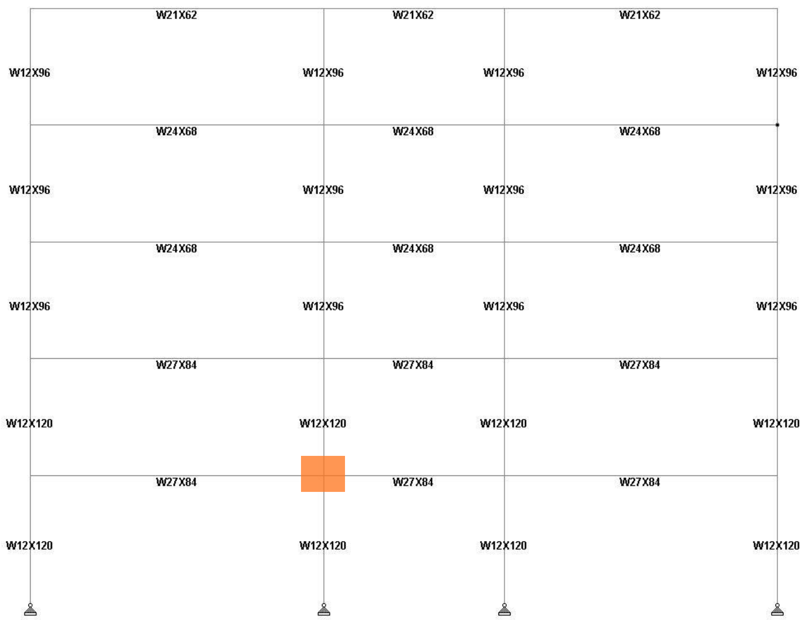


Figure 35: Location of designed moment connection

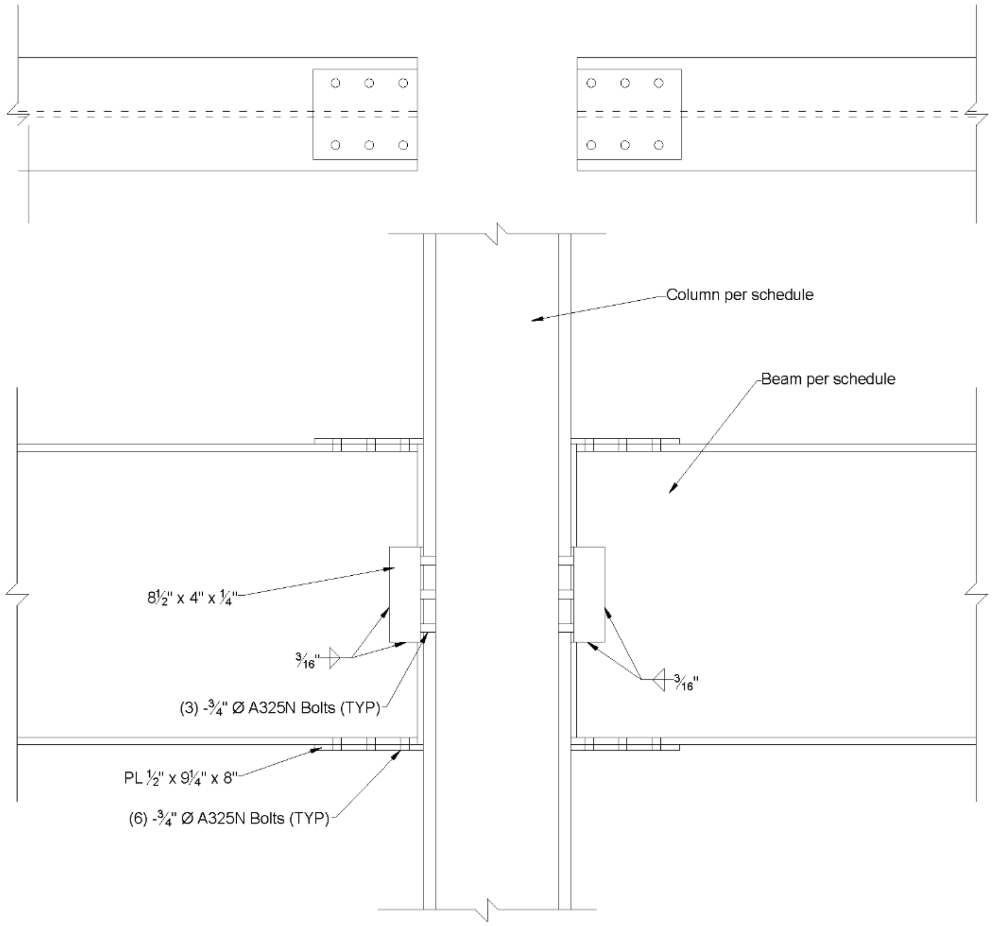


Figure 36: Moment Connection

Drift and Displacement

As was stated previously in this report, drift was the controlling factor in designing the lateral system. The drift and displacement was checked using RAM Structural System. These are serviceability issues for the building. For LRFD load combinations the factor for wind loads is 1.6 in ASCE 7-05. The 1.6 factor is for ultimate loads however, so it is reduced to 0.7 for the displacements. The allowable displacement for wind loads is L/400 which is not a code limit but more of an industry standard.

Lateral story drifts for seismic is limited to $0.02h_{sx}$ for occupancy category II by ASCE 7-05. These values were increased by the deflection amplification factor C_d. The X direction was amplified by 3 and the Y direction by 3.25.

All values for drift and displacement were found to be acceptable. Table 11 and 12 show tabulated values.

Wind Drift and Displacement					
Floor	Displacement		Drift		Allowable Displacement (in)
	X direction (in)	Y direction (in)	X direction (in)	Y direction (in)	
5	0.54924	0.63710	0.04477	0.10257	1.68
4	0.50447	0.53453	0.05847	0.12077	1.35
3	0.44600	0.41376	0.07073	0.13321	1.02
2	0.37527	0.28055	0.08812	0.13653	0.69
1	0.28715	0.14402	0.28715	0.14402	0.36

Table 11: Wind Drift and Displacement

Seismic Drift and Displacement									
Floor	Displacement						Drift		Allowable Drift (in)
	X direction (in)	Cd	Total	Y direction (in)	Cd	Total	X direction (in)	Y direction (in)	
5	1.19952	3.00	3.59856	0.5272	3.25	1.7134	0.31155	0.29055	2.64
4	1.09567	3.00	3.28701	0.4378	3.25	1.4229	0.44022	0.36442	2.64
3	0.94893	3.00	2.84679	0.3257	3.25	1.0584	0.52104	0.39868	2.64
2	0.77525	3.00	2.32575	0.2030	3.25	0.6598	0.58308	0.37931	2.64
1	0.58089	3.00	1.74267	0.0863	3.25	0.2804	1.74267	0.28044	2.88

Table 12: Seismic Drift and Displacement

Breadth 1: Enclosure Study

The purpose of this breadth is to examine the current enclosure of the Hotel N.E.U.S. and after weighing its pros and cons, select a new material in conjunction with the architecture breadth. A new system will provide suitable conditions and allow for the style of hotel to shift away from the many others like it.

Existing Conditions

The exterior of the Hotel N.E.U.S. consists of a brick veneer and Exterior Insulation and Finish System (EIFS). The large portion of the exterior area is covered with EIFS, shown in the elevations in Figure 37 and 38. This study focuses on this material because it has only become prominent in the past 50 years with issues surrounding its use in wet regions such as the northeast U.S. Brick has been used for centuries and is a standard building façade material.

Figure 37: North/South Elevation

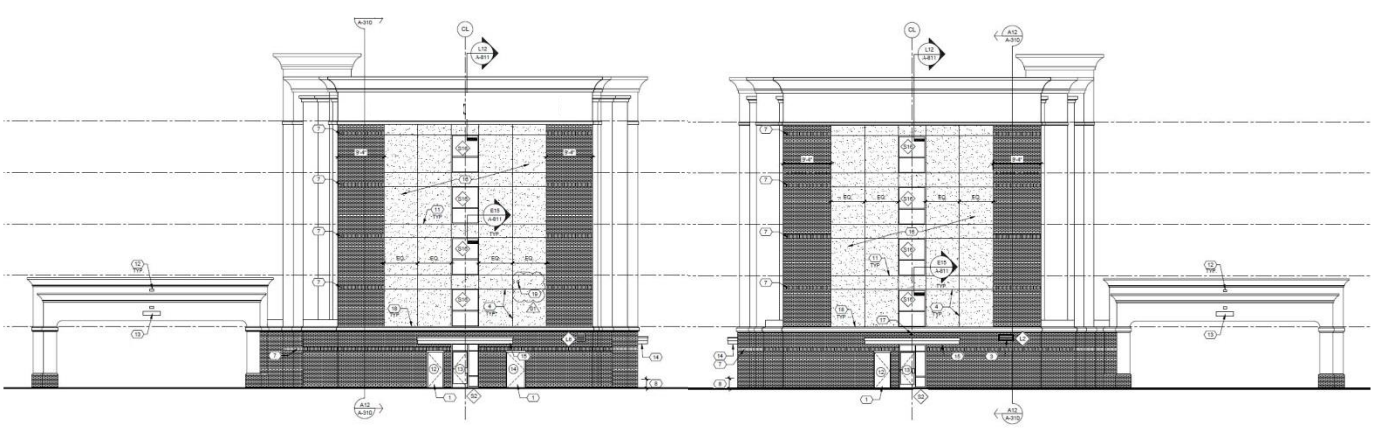


Figure 38: East/West Elevation

EIFS

Exterior Insulation and Finish Systems provide insulation and protection while being able to conform to any shape, color, and texture. It is a modernized version of traditional stucco. However it is an exterior cladding with different components which require more attention and care than stucco. The 3 parts of system are the insulation board, base coat, and finish coat. There are two types of EIFS system: a barrier wall and a wall drainage system. A wall drainage EIFS system is used in the Hotel N.E.U.S. It functions similar to a cavity wall, where a weather barrier is placed behind the insulation, allowing a way for moisture to gather and exit the system. Adhesive is applied in vertical strips prevent any hindrance in drainage. See Figure 39 for the makeup of a system.

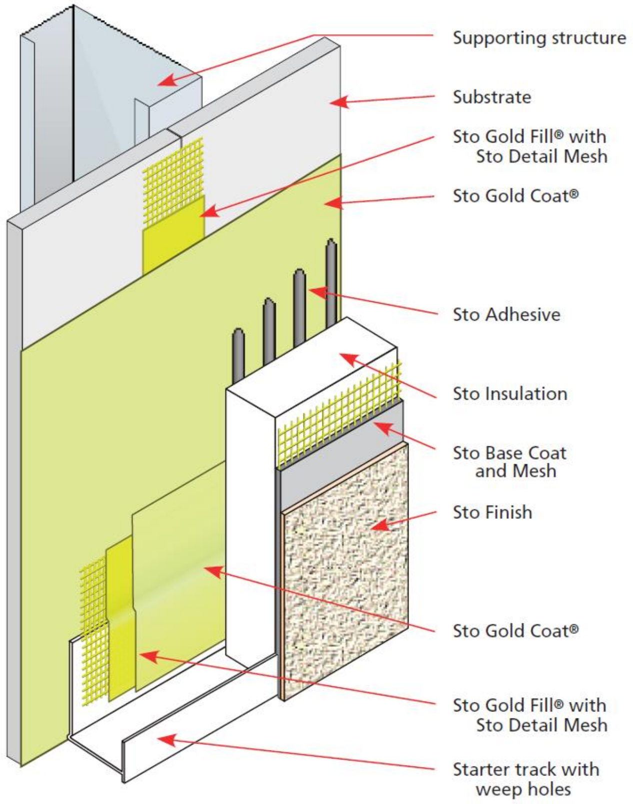
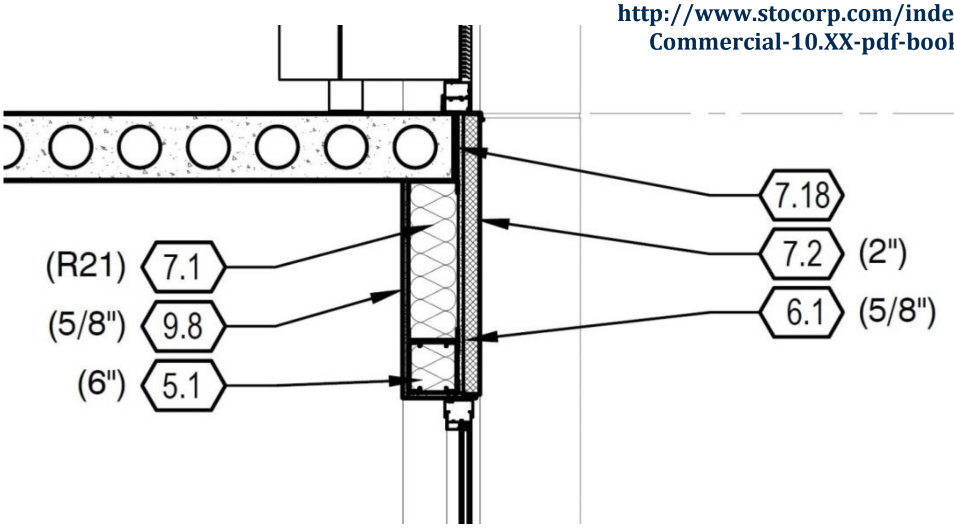


Figure 39: StoTherm NExT 3D section (source: http://www.stocorp.com/index.php/en/StoTherm-NExT-Commercial-10.XX-pdf-booklet/View-category.html



METALS
5.1 METAL STUD (SIZE)
WOOD, PLASTICS, AND COMPOSITES
6.1 FIBERGLASS-MAT FACED EXTERIOR GYPSUM SHEATHING (SIZE)
THERMAL AND MOISTURE PROTECTION
7.1 THERMAL BATT INSULATION (VALUE)
7.2 EXTERIOR INSULATION AND FINISH SYSTEM (EIFS) (SIZE)
7.18 FIRESTOPPING
FINISHES
9.8 FIBERGLASS-MAT FACED INTERIOR GYPSUM BOARD (SIZE)

Figure 40: Existing Typical Detail for Exterior Wall

Originally developed in the 1950’s in Europe, EIFS was marketed as the material that could insulate and protect old masonry structures. In 1969, Dryvit Systems, Inc. brought it to the United States. It was used almost exclusively in commercial building and was eventually adopted in residential construction. The problem with the U.S. was that most buildings were not heavy masonry construction like in Europe. Those walls could function fine without the application of EIFS. It was discovered in 1995 that poor construction and detailing was leading to water infiltration and damage in EIFS systems. Once water penetrates an EIFS system, it has no way to exit. Areas such as windows, doors, projections, roof and deck flashings were all associated with the water intrusion.

Benefits

EIFS is an efficient enclosure material. It reduces air infiltration by up to 55% more than brick of wood construction. It also saves energy by increasing the R value of wall, allowing for heating and cooling costs to be decreased. It weighs very little and is cheap as well. The ability to mold and shape the material, along with the other benefits, makes EIFS a popular exterior system. Drained systems provide a way for water to exit and prevent the buildup of moisture.

Performance Issues

The main issue revolving around EIFS is the infiltration of water. In a barrier system, the barrier must remain perfect at all times to halt any water from intruding. This is an unrealistic assumption and a flawed approach. Moisture in the system can lead to mold growth, corrosion of metal studs, discoloration of surfaces, loss of cohesion between building materials, and odors.

Cracking is unpreventable and therefore water will be able to enter the system. The damage caused is internal and cannot be seen, making it difficult to spot. It could take years to find an issue and by then the damage will have been done. Flashing, caulking, and expansion joints are especially important in buildings with many openings, such as hotels. EIFS systems should have yearly maintenance performed and cleaning needs to be completed based off manufacturer’s standards. It is recommended that barrier systems not be used in regions that receive less than 20 inches of precipitation annually and average monthly temperature remains above 45 degrees Fahrenheit. Unfortunately, there is also an absence of inspections related to the installation and standards are not always enforced, especially with projects that are fast and cheap. Drained systems can still have the same issues if the weather barrier is not installed properly.

New Enclosure

The selected alternative for the enclosure of the hotel are Metl Span Architectural Wall Panels. They provide a new architectural style while performing very well for the conditions of the Hotel. Polyurethane foam that is 2"-4" thick is injected between metal sheets to provide a durable insulator and first barrier layer with an assortment of colors and styles. The sizes allow for flexibility in design and since they are prefabricated, installation time is fast. The concealed clips attach the panels and allow for a flush appearance. They also have barrier side joints that hide the vapor sealant in grooves, providing protection from dirt and weather. When installed properly the amount of maintenance needed is minimal.

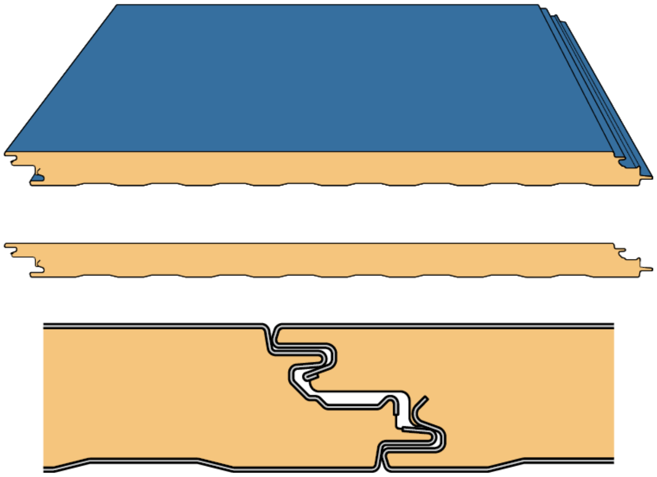


Figure 41: Metl Span Architectural Insulated Panel (source: http://www.metlspan.com/wp-content/uploads/2012/03/ArchitecturalWalldatasheet.pdf)

Drawbacks

Metal panels are similar to EIFS in that they require a weather barrier over an approved substrate at the exterior face of the wall. Due to this being a drainage system, the sealants at joints aren't as important as they would be in a face sealed cladding. Insulated metal panels are heavier and can cost more than EIFS too.

Despite being more durable, the thin metal sheets can be dented. Over time, the protective coating can be attacked cause unpleasant pitting appearance. Along with that, oil canning can detract from look of the panels if there are issues with fabrication, design, or installation.

Windows

In order to satisfy the horizontal panel layout new windows will be used as well. Metl Vision Window System is a flush frame design that integrates with the Metl Span panels to create a weather tight installation. Joints are fully sealed, sills and heads are dammed, and weep drainage is implemented to create this protection. This assembly eradicates typical interface problems with standard windows when used with panels.

Figure 42: Metl Vision Window (source: http://metlspan.com/products/architectural/metl-vision/)

The maximum window height is 20' which will allow for the same window size to be matched or increased while keeping space for the louver below. The glazing is designed to be installed from the interior. This prevents weather delays and lifting equipment and will further decrease construction schedule. All components are thermally broken to diminish thermal conductance and condensation resistance.

Thermal Performance

The outside temperature is almost always different from the inside. The ability of the building enclosure to prevent heating and cooling leak to the outside is important for comfort and energy cost.

Stud construction can allow for “thermal bridging” which means the studs offer less resistance to heat travelling through the material and can cause cold spots and condensation. The Metl Span Insulated Panels provide a continuous thermal barrier that eliminates the thermal bridge effect on the steel studs. Since the wall is not made purely of insulation or studs, the R value is calculated using the Isothermal Planes Method. This method averages the stud and insulation R values by the percent of wall area they occupy. In Figure 43, the temperature gradient through the wall on a cold winter day (outside = 5°F, inside = 68°F) is shown. The panel’s R value is provided by the manufacturer and includes the air films surrounding it. See Appendix G for calculations.

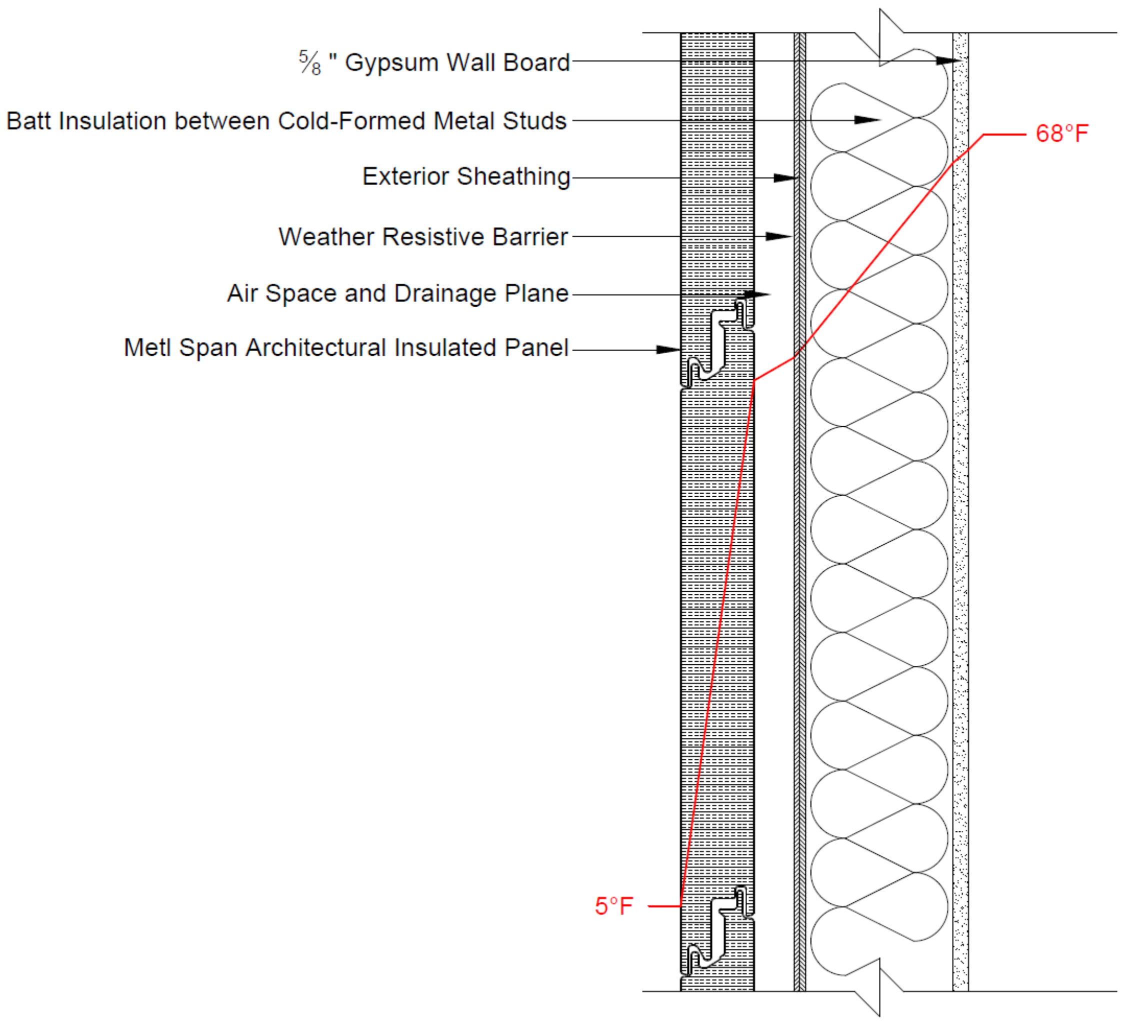


Figure 43: Temperature Gradient through an exterior wall

Detail

Detailing is very important in construction, especially for enclosures. Components such as the flashing and weather barrier need to be properly located and installed to prevent damage. Figure 44 shows a section through an exterior wall and Figure 45 shows a section at the top of a window. Due to the window jamb being located in line with the panels there is more wall area that has to be insulated. Since no details for this system are provided, the one in Figure 43 was created using other examples and knowledge from AE 542 *Building Enclosure Science Design,* and is not a final design.

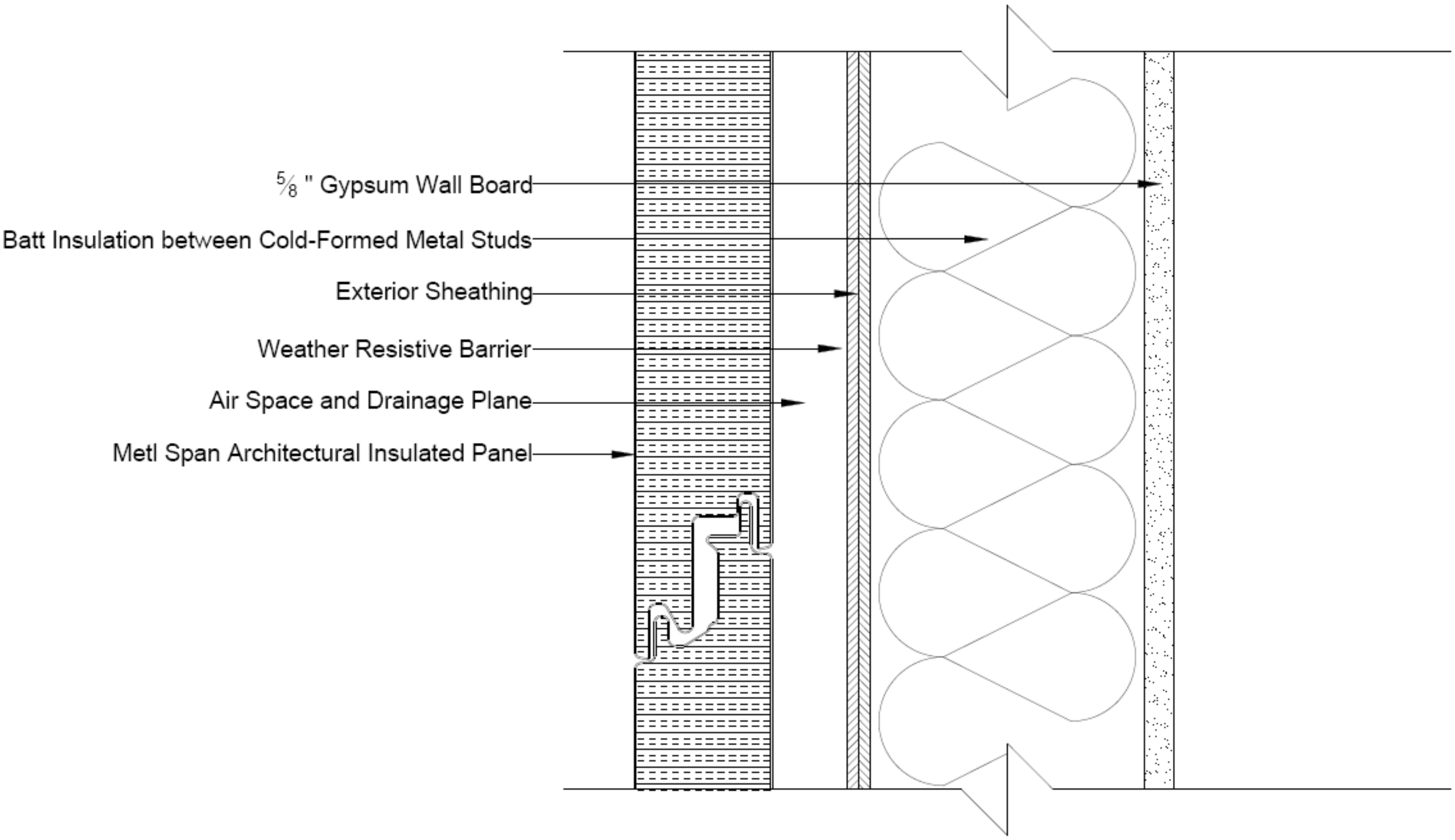


Figure 44: Typical Wall Detail

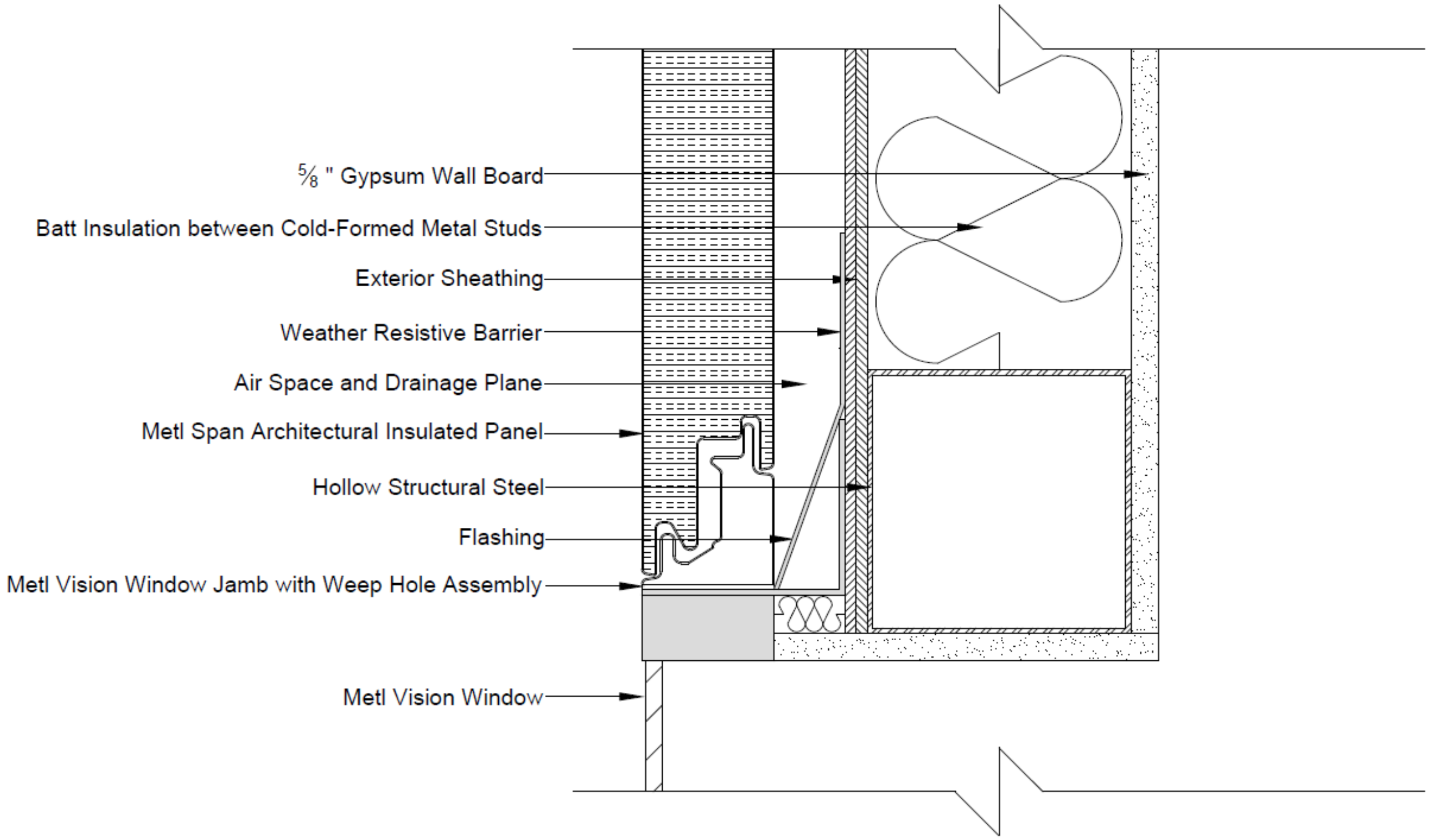


Figure 45: Typical Detail at window

Figure 46: Metl Span Architectural Flat panels and Metl Vision windows. There images were used as inspiration for this study. (source: http://metlspan.com/)

Comparison Matrix

A table was developed in order to have a side by side comparison of the old and new systems.

	EIFS	Insulated Metal Panel
Thickness	2"	3"
Waterproofing	Weather barrier and drained system	Weather barrier and drained system
Air Barrier	Provided by Base Coat and Substrate	Joint is formed on barrier side, hidden vapor sealant
Thermal	R value= 20 (interpolated from data sheet)	R value= 23 (average value)
Structural Integrity	Can crack and is easily punctured. Impact resistance is decent	Can be dented and have issues with pitting and oil canning. Metal is thin but strong.
Cost	Labor costs are higher than material	Matieral Costs are higher than labor
Installation	Exterior synthetic stucco finish must be applied by a skilled tradesman. Adhesive attaches rigid installation to wall.	All panels are prefabricated and can installed quick and easy. Hangers attached to studs support panels. Double tongue and groove connection.

Conclusion

By using knowledge gained from AE 542 *Building Enclosure Science Design* the enclosure was examined and evaluated. Due to the new architecture (found in Breadth 2), the Metl Span Insulated Architectural Wall Panels will be used along with Metl Vision windows. These will replace the existing StoTherm NExT drained EIFS system. The panels will reduce overall cost and construction time will be decreased. All ASTM and IBC requirements will be met and energy savings could potentially increase due to the larger thermal resistance value. Precast Concrete Panels were also considered for the enclosure but did not provide the desired aesthetics and fail to perform as well. They were not included in this report.

Breadth 2: Architecture

Introduction

The existing design of the Hotel N.E.U.S. has post-colonial architectural features which are represented by the use of recessed arches, cornices, and achieved through the use of brick and synthetic stucco. This style has been almost universally applied to small scale hotels in the United States.

This type of architecture uses warm and earthen colors such as brown, red, beige, and tan. These colors combined with the cornices and arches are intended to give off a feeling of "home". It is supposed to evoke a feeling of safety and comfort that one's own home provides. The construction materials are also cheap and readily available, therefore commonly used. In Figure 47, a perspective of Hotel can be seen.

There is a 4 story hotel located next to the N.E.U.S. that uses very similar styles (see Figure 48). In order to stand out and break away from this typical aesthetic "mold", a new material was selected to encompass the exterior.

Above-

Figure 47: Existing perspective view of the Hotel

Right-

Figure 48: Hotel located in adjacent lot

New Style

In conjunction with the enclosure breadth study, insulated metal panels were selected to replace the existing EIFS system. These metal panels provide lots of flexibility in the creation of the building's exterior with different sizes, colors, and finishes. Metal panels provide a sleek new look for the building with many benefits to the enclosure (which can be found in Breadth 1). A comparison between the existing façade (Figure 49) and new façade (Figure 50) can be found below.

The majority of the panels are white and green. They highlight the slight step back of the façade along the length. Even though the building isn't perfectly symmetrical, the color layout makes observer's mind's think that it is. The black accent strip in the middle of the building and is used to draw your eyes to the center and to the entrance, acting as a guide for newcomers.

Figure 49: Existing South elevation

Figure 50: New South elevation

Figures below show the existing side elevation and the newly configured side elevation. The color scheme from the front of the building is used again on the East and West sides.

Figure 51: Existing West elevation

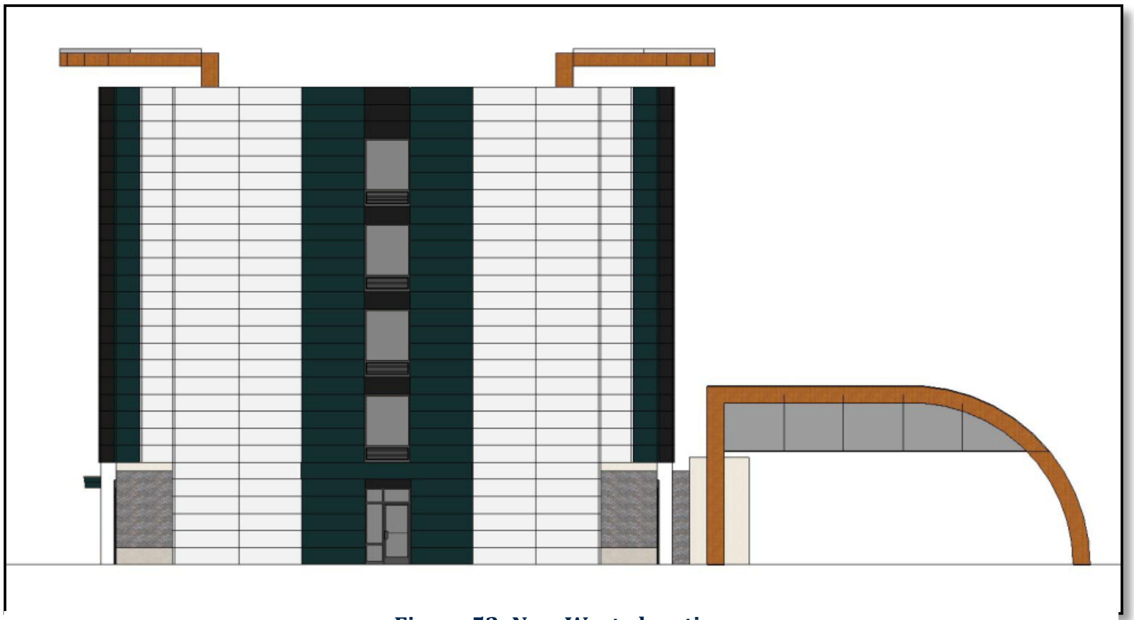


Figure 52: New West elevation

Along with the black accent strip, the entrance is highlighted by the canopy. Most hotels provide these to allow for drop offs without hindrance from the elements. The existing canopy was bulky and was of the same style as the façade. The new entryway (seen in Figure 54) is constructed of steel wrapped in a wood veneer that supports tinted glazing. These materials have made a large advance in today's construction. The mixture of an ancient material (wood) with a modern one (glass) makes for an elegant and fresh attraction before entering the building.

Figure 53: Perspective of new design

Figure 54: Close up of new entryway

Instead of cornices, an accent awning now overlays the parapet. Instead of just having the parapet end with nothing behind it, this awning gives a sense importance to the roof. The rounded lip contrasts the sharp cutbacks of the façade. This curve, along with those of the arches, and the new metal panels, makes for a smooth and comfortable feeling from the architecture.

Figure 55: Close up of roof awning

Figure 56: Bird's eye view of new perspective

Swimming Pool

One specific area of interest was the swimming pool on the first floor. Due to the new framing, a brace was needed to span the pool deck area. The brace location is highlighted in Figure 57 below.

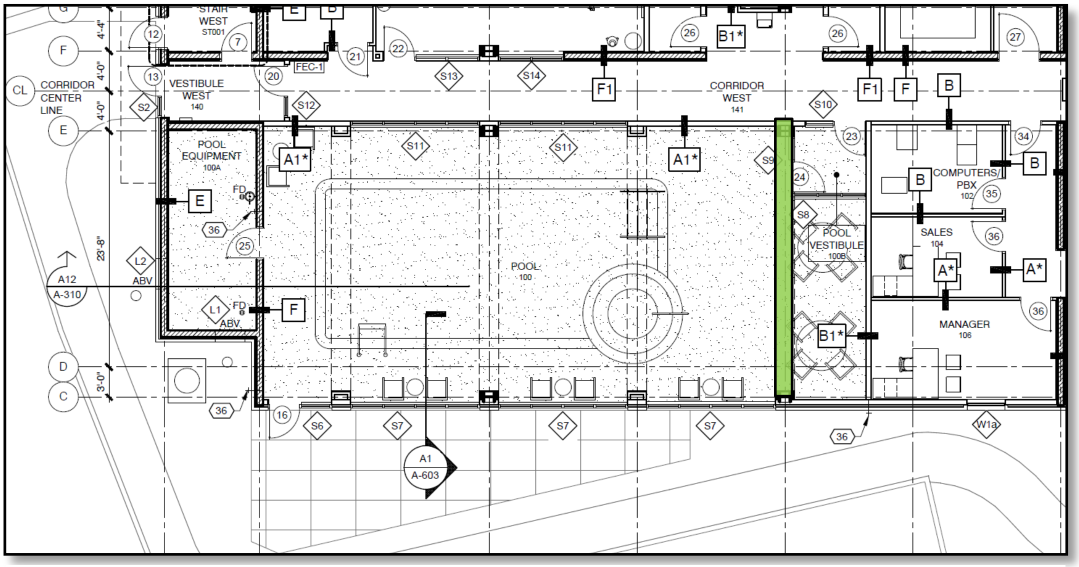


Figure 57: Plan view of pool area with new brace location highlighted in green.

To solve this issue, the pool entrance will be relocated to the other wall in the vestibule. Mold resistant drywall and protective coating will be used to create an archway through the K brace. This will enclose the brace without hindering the pool deck area used for guests. An interior view of the swimming pool can be seen in Figure 58.

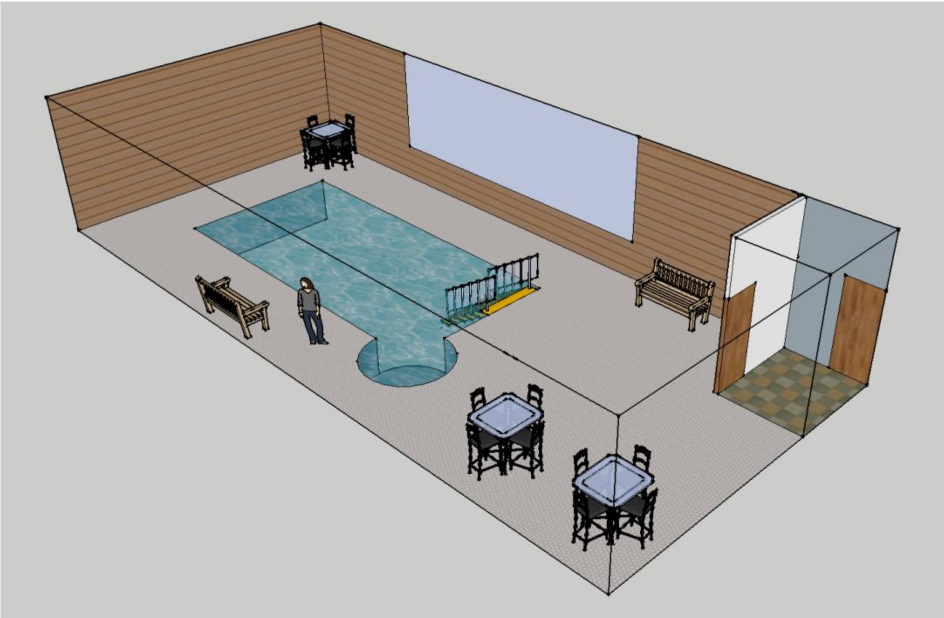


Figure 58: Interior view of swimming pool room

In Figure 59, the new brace can be seen with the vestibule door now relocated. Figure 60 shows how the wall would be constructed around the brace. Windows can be placed through the infill wall to allow for a more open atmosphere.

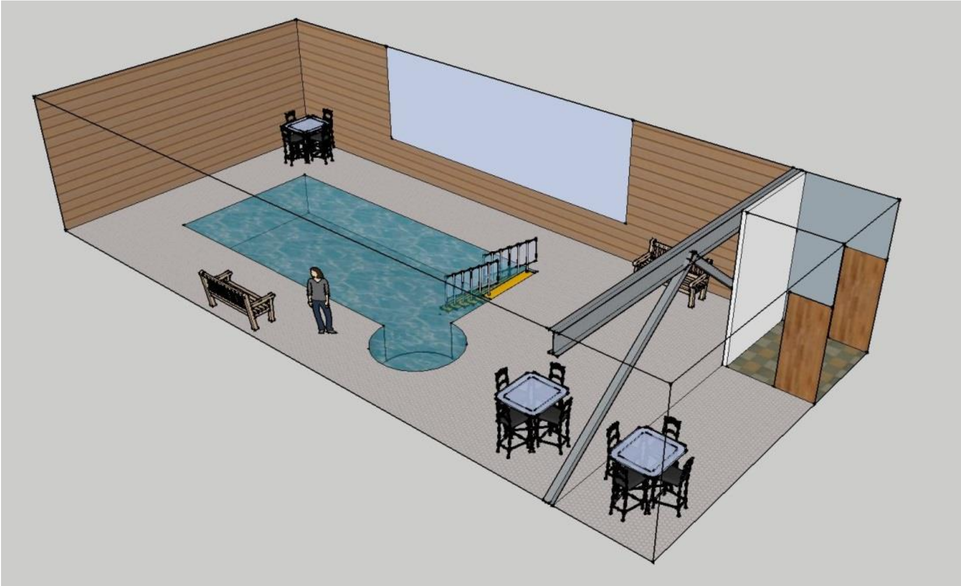


Figure 59: Interior view of swimming pool with brace exposed

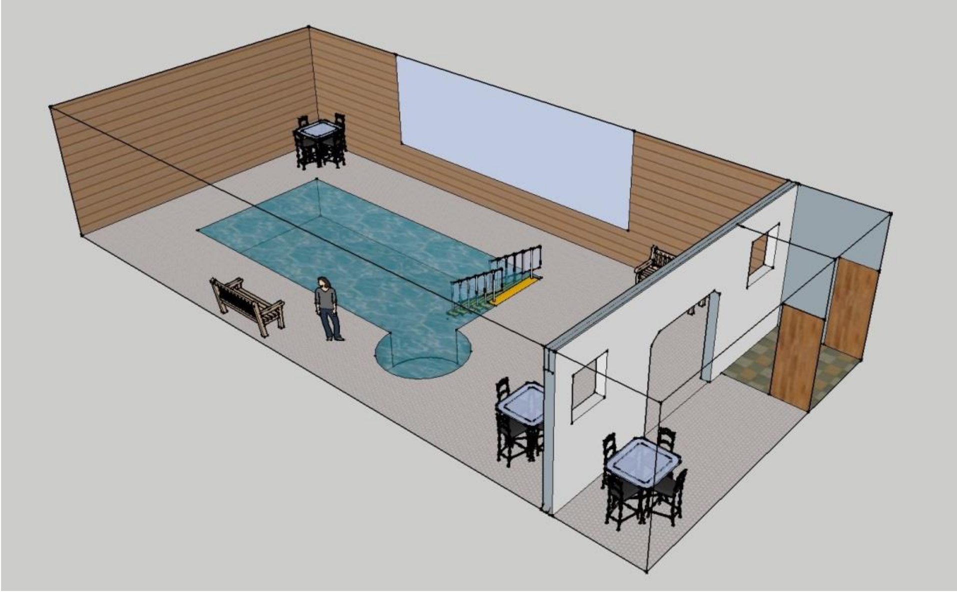


Figure 60: Interior view of swimming pool with brace concealed with infill wall

Conclusion

The design goals for this thesis study were all completed successfully. A summary of all parts are listed below:

The framing for the building was changed from masonry bearing walls and precast plank (with steel on the ground level) to steel beams and columns. The gravity system was redesigned using composite steel and concrete on metal deck. This allowed for an efficient placement of columns and no interruption of guest spaces. The floor plan was remained unchanged.

The lateral system was redesigned using braced frames in the short direction and moment frames in the long direction. These prevented any change in window/door layout and were sufficient in resisting loads and limiting drifts.

Steel was selected in order to perform a study of the difference in materials for low rise buildings. By changing the material to steel, the overall building weight will significantly decrease which lowers seismic loads. The construction timeline could also be decreased. A large benefit to steel construction is that the lateral system can achieve a balanced layout. It was found that steel performs very well in low rise buildings, but is more expensive. Masonry suffers with placement because it must run continuous from foundation to roof. The ability to resist lateral loads and limit drift is well met with shear walls but can be achieved with steel too. The difference between masonry and steel discovered through this report was a valuable learning tool.

A study of the enclosure showed that the selected materials were effective for the conditions of the northeast United States. In order to change the architectural style, metal panels were selected for the façade. They replaced the existing drained EIFS and were a suitable replacement for the enclosure.

To break away from the characteristic hotel style in today's construction industry, the architecture of the Hotel N.E.U.S. was overhauled. A study of old and new buildings was conducted and a new design was forged. The intent of the new aesthetics was to bring a fresh feeling to the building's exterior. A Sketch-Up model allowed for the ideas to be portrayed with elevations and perspectives.

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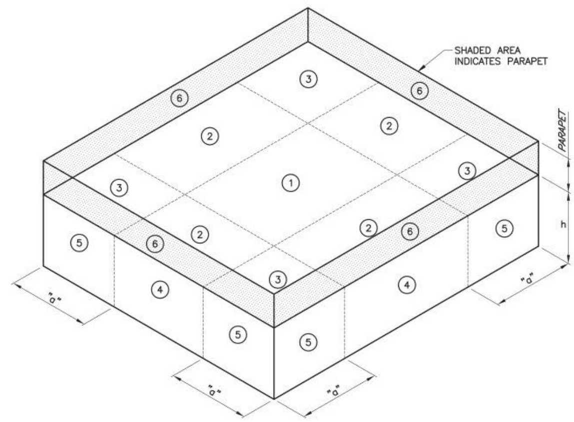
Appendices

Appendix A: Plans and Sections

COMPONENT AND CLADDING WIND PRESSURES						
TRIBUTARY AREA (SF)	ROOF ZONE			WALL ZONE		PARAPET
	1	2	3	4	5	6
10	−35	−54	−55	+24/−28	+24/−35	+71/−71
20	−33	−53	−52	+22/−27	+22/−32	+67/−67
50	−30	−48	−48	+21/−25	+21/−29	+62/−62
100	−28	−46	−45	+20/−24	+20/−27	+58/−58
200	−26	−43	−43	+20/−23	+20/−25	+54/−54
500	−24	−39	−39	+17/−21	+17/−21	+49/−49

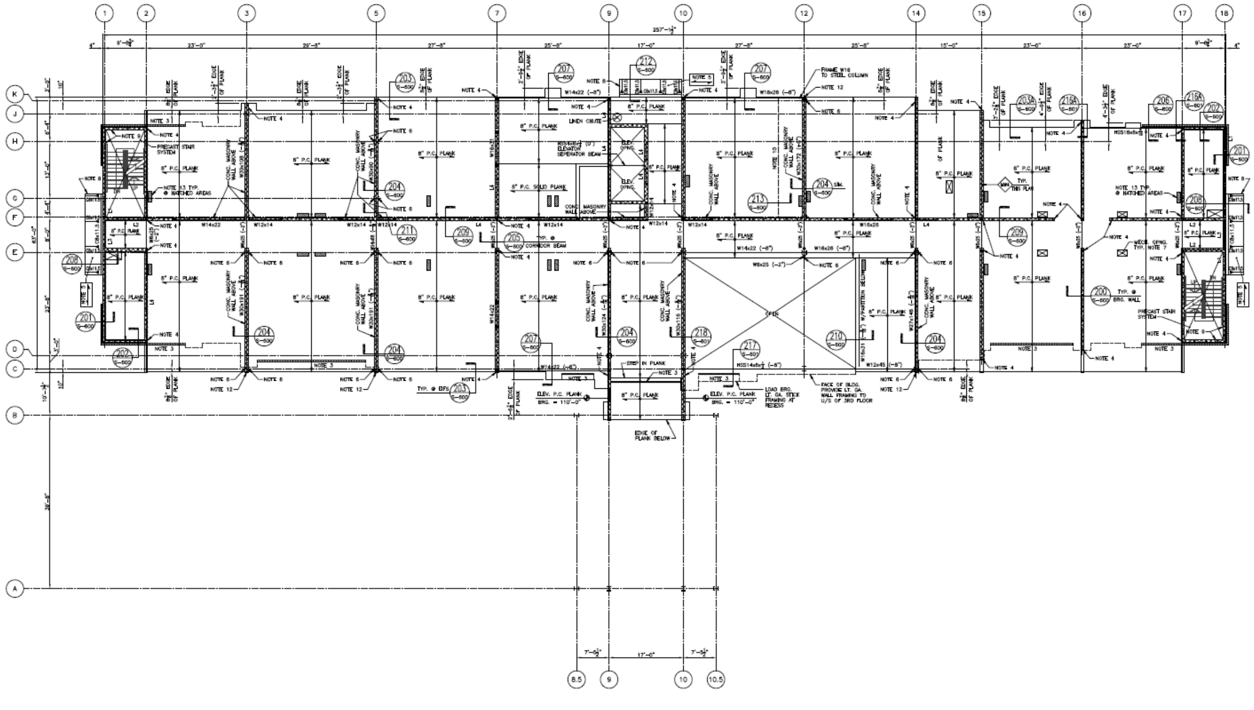
NOTES:

1. ALL LOADS ARE IN POUNDS PER SQUARE FOOT (PSF).
2. (+) DENOTES PRESSURE, (−) DENOTES SUCTIONS.
3. "a" SHALL BE 10% OF LEAST HORIZ. DIMENSION OR 0.4h, WHICHEVER IS SMALLER, BUT NOT LESS THAN 4% OF LEAST HORIZ. DIMENSION OR 3'−0".

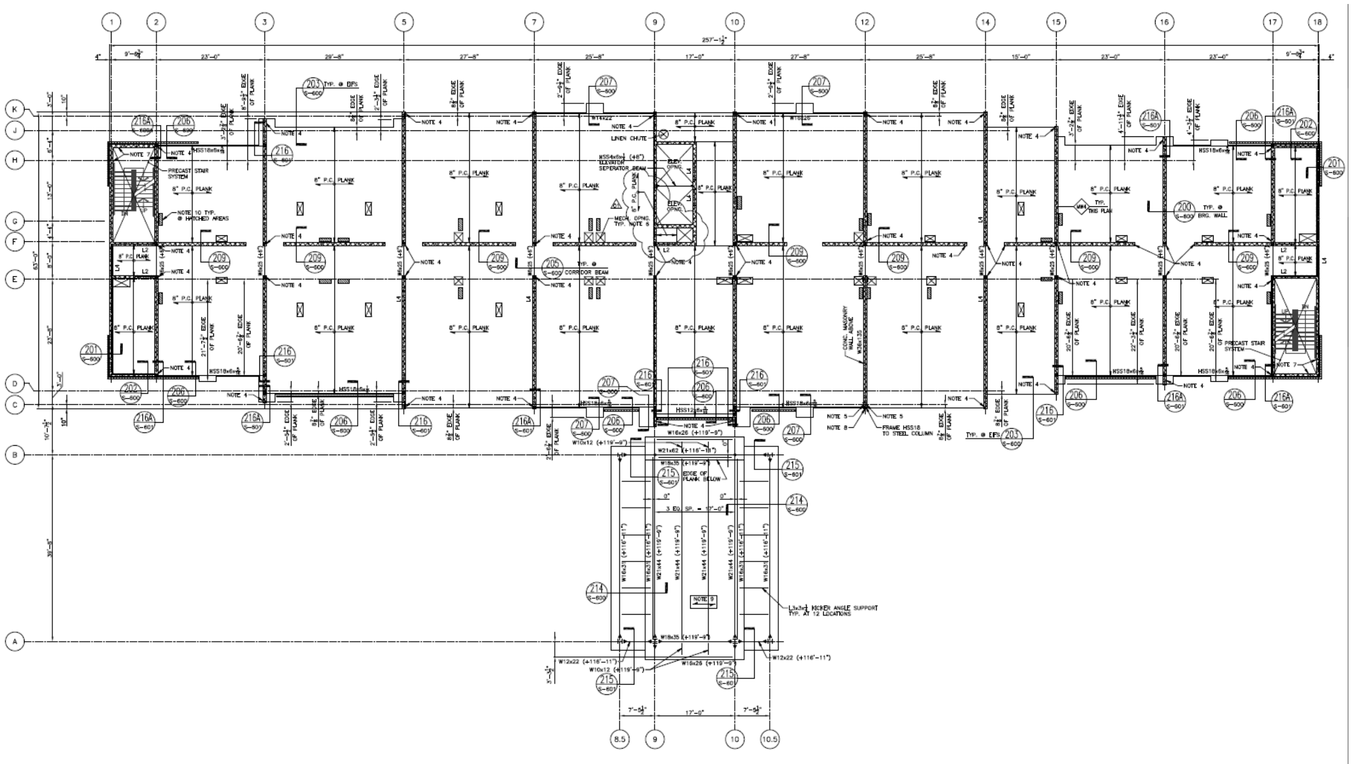


ROOF AND WALL ZONES

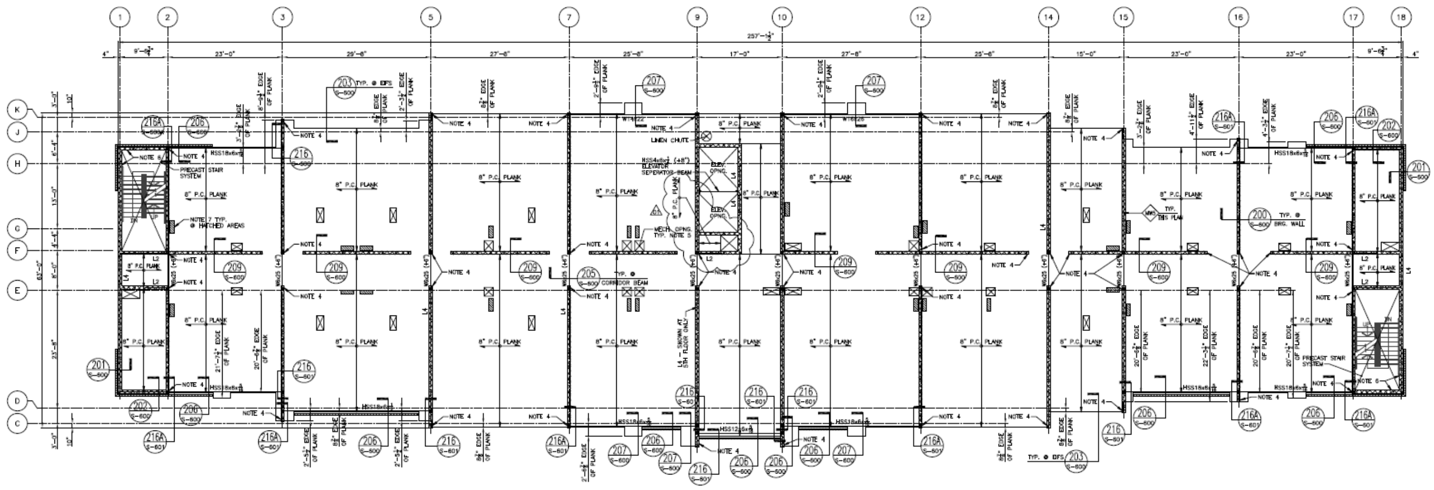
- ✓ IBC 2009
- ✓ International Mechanical Code (IMC 2009)
- ✓ International Plumbing Code (IPC 2009)
- ✓ International Fire Code (IFC 2009)
- ✓ National Fire Protection Associations (NFPA)
- ✓ ADA Accessibility Guidelines (ADAAG) and American National Standards Institute (ANSI)



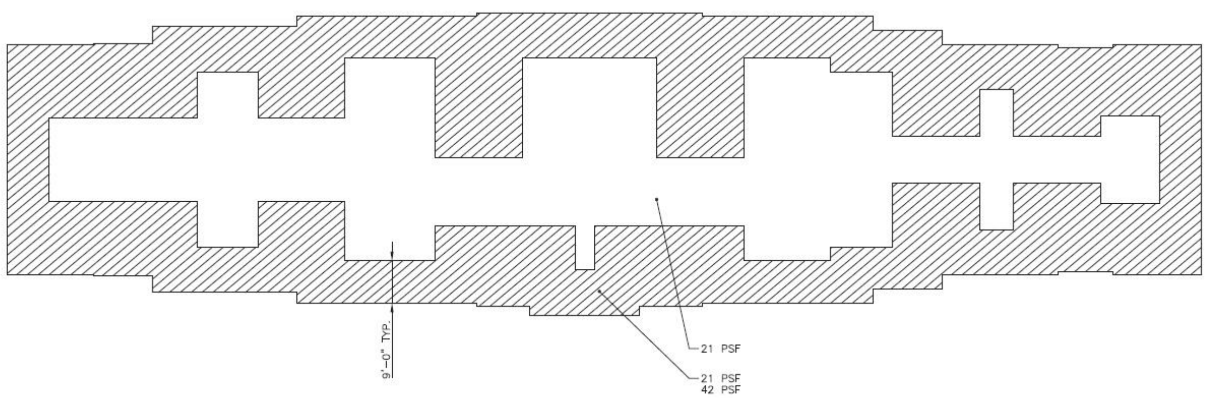
SECOND FLOOR FRAMING PLAN
SCALE: 1/8" = 1'-0"



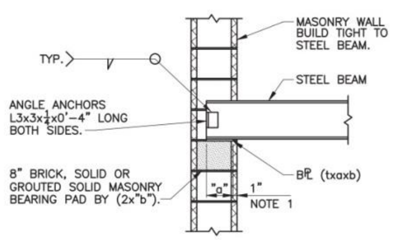
THIRD FLOOR FRAMING PLAN
SCALE: 1/8" = 1'-0"



FOURTH AND FIFTH FLOOR FRAMING PLAN



ROOF SNOW LOAD DIAGRAM
SCALE: $\frac{1}{16}$" = 1'-0"

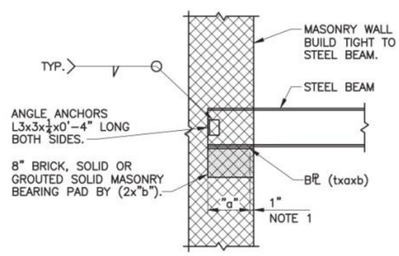


NOTES:

1. FOR BPL'S THAT ARE 1" SMALLER THAN THE MASONRY WALL, CENTER THE BPL ON THE WALL.

TYPICAL STEEL BEAM BEARING ON MASONRY WALL DETAIL

ALTERNATE DETAIL:
PROVIDE 2–$\frac{1}{2}$"ø ANCHOR BOLTS INTO GROUTED SOLID MASONRY BEARING W/ NO ANGLE ANCHORS.



NOTES:

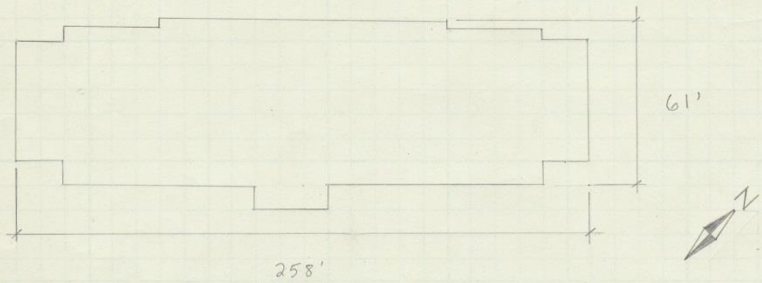
1. FOR BPL'S THAT ARE 1" SMALLER THAN THE MASONRY WALL, CENTER THE BPL ON THE WALL.

TYPICAL STEEL BEAM BEARING ON MASONRY END WALL DETAIL

ALTERNATE DETAIL:
PROVIDE 2–$\frac{1}{2}$"ø ANCHOR BOLTS INTO GROUTED SOLID MASONRY BEARING W/ NO ANGLE ANCHOR.

Appendix B: Wind Calculations

TECH 1 | WIND ANALYSIS 1 | JORDAN RUTHERFORD



PARAPET HEIGHT: 60'8" USE METHOD 2
E-W DIRECTION: 258'
N-S DIRECTION: 61'

WIND SPEED: (FIG 6-1) $V = 90$ MPH

DIRECTIONALITY FACTOR: (TBL 6-4) $K_d = 0.85$

OCCUPANCY CATEGORY: (TBL 6-1) II

IMPORTANCE FACTOR: (TBL 6-1) $I = 1.0$

EXPOSURE CATEGORY: (6.5.6.3) C

TOPOGRAPHIC FACTOR: (FIG 6-4) $K_{zt} = 1.0$

VELOCITY PRESSURE COEFFICIENTS: (TBL 6-3) VARIES W/ HEIGHT

INTERNAL PRESSURE COEFFICIENT: $GC_{pi} = \pm 0.18$

GUST FACTOR: (6.5.8.1) $G_f = 0.85$*

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.2\left(\frac{33}{0.6(60.67)}\right)^{1/6} = .197$$

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 500\left(\frac{0.6(60.67)}{33}\right)^{1/5} = 509.9$$

* CALCS PERFORMED BASED ON RIGID STRUCTURE, 0.85 USED TO BE CONSERVATIVE

$$Q = \sqrt{\frac{1}{1 + .63\left(\frac{258 + 60.67}{509.9}\right)^{.63}}} = .825 \qquad G_f = 0.925\left(\frac{1 + (1.7)(3.4)(.197)(.825)}{1 + (1.7)(3.4)(.197)}\right) = .8386$$

$$Q = \sqrt{\frac{1}{1 + .63\left(\frac{61 + 60.67}{509.9}\right)^{.63}}} = .892 \qquad G_f = 0.925\left(\frac{1 + (1.7)(3.4)(.197)(.892)}{1 + (1.7)(3.4)(.197)}\right) = .872$$

TECH 1 | WIND ANALYSIS 2 | JORDAN RUTHERFORD

BUILDING IS FULLY ENCLOSED

WALL PRESSURE COEFFICIENTS: (FIG 6-6)

	C_p		USE WITH
WINDWARD:	0.8		q_z
SIDEWALL:	-0.5	(L/B < 1) (FOR N-S)	q_h
	-0.2	(L/B > 4) (FOR E-W)	q_h
LEEWARD:	-0.7		q_h

ROOF PRESSURE COEFFICIENTS (FIG 6-6) q_h

			C_p	
	WINDWARD:	0 – h/2	-0.9	2h = 121 < 258'
		h/2 – h	-0.9	∴ USE ALL
$h/L = \frac{60.67}{258} = .22 < .5$		h – 2h	-0.5	h/2 = 30.33'
		> 2h	-0.3	

				AREA	
	WINDWARD:	0 – h/2	-1.3	(30.33)(258') = 7825 > 1000	X
		> h/2	-0.7 ← REDUCE x 0.8	(30.67)(258') = 7912 > 1000	✓
$h/L = \frac{60.67}{61} = .99 > .5$					

VELOCITY PRESSURES: $q_z = 0.00256\,K_z K_{zt} K_d V^2 I$

$q_h = 0.00256\,K_z K_{zt} K_d V^2 I$

DESIGN WIND PRESSURES: (6.5.12.2)

WINDWARD: $p_z = q_z(GC_p) - q_h(GC_{pi})$

LEEWARD:
SIDEWAYS
ROOF
$p_h = q_h(GC_p) - q_h(GC_{pi})$

PARAPET: $p_p = q_p(GC_{pn}) = q_p(1.5)$ WINDWARD
$q_p(-1.0)$ LEEWARD

☆ FOR VALUES, SEE EXCEL TABLES

Wind Load Data		
Design Wind Speed	V	90
Directionality Factor	Kd	0.85
Occupancy Category	I	II
Importance Factor		1
Exposure Category		C
Topographic Factor	Kzt	1
Internal Pressure Coefficient	Gcpi	+/-0.18
Gust Factor	G	.85
Wall Pressure Coefficients		
Windward	Cp	0.8
Side Wall (N-S)	Cp	-0.5
Side Wall (E-W)	Cp	-0.2
Leeward	Cp	-0.7
Roof Pressure Coefficients		
Windward (E-W)	0-h/2	-0.9
	h/2-h	-0.9
	h-2h	-0.5
	>2h	-0.3
Windward (N-S)	0-h/2	-1.3
	>h/2	-0.56

Velocity Pressures							
Level	Elevation	K_z	K_{zt}	K_d	V^2	I	q_z
	61	1.1340	1	0.85	8100	1	20.0
Parapet	56	1.114	1	0.85	8100	1	19.6
5	45	1.065	1	0.85	8100	1	18.8
4	34	1.004	1	0.85	8100	1	17.7
3	23	0.924	1	0.85	8100	1	16.3
2	12	0.85	1	0.85	8100	1	15.0
Ground	0	0.85	1	0.85	8100	1	15.0

Appendix C: Seismic Calculations

TECH 1 | SEISMIC ANALYSIS 1 | JORDAN RUTHERFORD

EQUIVALENT LATERAL FORCE METHOD

OCCUPANCY CATEGORY: (TBL 1-1) II

SITE CLASS: (GEOTECH. REPORT) D

SEISMIC LOAD IMPORTANCE FACTOR: (FIG 11.5-1) $I_e = 1.0$

SPECTRAL RESPONSE ACCELERATIONS: (FIG 22-1,2) $S_S = 0.125$
$S_1 = 0.049$

SITE CLASS COEFFICIENT: (TBL 11.4-1,2)

$F_a = 1.6 \quad S_{MS} = 1.6(0.125) = 0.2$
$F_v = 2.4 \quad S_{M1} = 2.4(0.049) = 0.1176$

SPECTRAL RESPONSE COEFFICIENT: (TBL 11.4-3,4)

$S_{DS} = 2/3\,(0.2) = 0.1333$
$S_{D1} = 2/3\,(0.1176) = 0.0784$

SEISMIC DESIGN CATEGORY: (TBL 11.6-1,2) B

BASE SEISMIC FORCE RESISTING SYSTEM: (TBL 12.2-1) $R = 2$
REINFORCED MASONRY SHEAR WALLS

APPROXIMATE FUNDAMENTAL PERIOD: (12.8.2.1)

$T_a = C_t h_n^x = 0.02(52)^{0.75} = 0.387$ FOR "OTHER" SYSTEMS (TBL 12.8-1)

OR

$T_a = \frac{0.0019}{\sqrt{C_W}} h_n =$ E-W: 0.1217 | N-S: 0.08
FOR MASONRY SHEAR WALLS 12.8-9

$$C_W = \frac{100}{A_B} \sum_{i=1}^{x} \left(\frac{h_n}{h_i}\right)^2 \frac{A_i}{\left[1 + 0.83\left(\frac{h_i}{D_i}\right)^2\right]} = 0.659 \;/\; 1.524$$

$A_b = 15725\ ft^2$

SEE EXCEL FOR A_i, D_i, h_i, t_i

TECH 1	SEISMIC ANALYSIS 2	JORDAN RUTHERFORD

SEISMIC RESPONSE COEFFICIENT: (12.8.1.1)

COEFF. FOR UPPER LIMIT ON PERIOD: (12.8-1)

$C_u = 1.7 \times T_a = 0.387 = T = 0.6579$

$T_L = 12\,s$ (FIG 22-15)

$$C_s = \frac{S_{DS}}{\frac{R}{I}} = \frac{0.1333}{\left(\frac{2}{1}\right)} = 0.067$$

$$C_{s\,MAX} \begin{cases} \dfrac{S_{D1}}{T\left(\frac{R}{I}\right)} & \dfrac{0.0784}{0.6579\left(\frac{2}{1}\right)} = 0.059 & \text{FOR } T < T_L \text{ OK} \\ \dfrac{S_{D1} T_L}{T^2\left(\frac{R}{I}\right)} & \dfrac{0.0784\,(12)}{(0.6579^2)\left(\frac{2}{1}\right)} = 1.087 & \text{FOR } T > T_L \text{ NG} \end{cases}$$

$$C_{s\,min} \begin{cases} \dfrac{0.5\,S_1}{\left(\frac{R}{I}\right)} = \dfrac{0.5\,(0.049)}{\left(\frac{2}{1}\right)} = 0.01225 \\ 0.01 \end{cases}$$

SEE EXCEL TABLE FOR DETAILED C_s PER DIR.

TECH 1 | SEISMIC ANALYSIS 3 | JORDAN RUTHERFORD

BASE SHEAR: (12.8.1)

DEAD LOAD:

P.C. PLANK: 56 psf
3/4" TOPPING: 6 psf
PARTITIONS: 15 psf (4.2.2)
MEP/MISC: 5 psf
CEILING: 3 psf
85 psf

ROOF:

P.C. PLANK: 56 psf
MEP/MISC: 5 psf
CEILING: 3 psf
INSULATION: 12 psf
76 psf

FLOOR AREAS	WEIGHT	WALL WEIGHT
2: 14871 ft²	1264 k	COMPLETED IN EXCEL
3: 14871 ft²	1264 k	
4: 14871 ft²	1264 k	
5: 14871 ft²	1130 k	

$V = C_s W = 0.067(10907) = 649$

VERTICAL FORCE DISTRIBUTION: (12.8.3)

$F_x = C_{vx} V$

$C_{vx} = \dfrac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$

CALCULATIONS DONE IN EXCEL

Floor Dead Loads	Load (psf)	Reference
5.5" 3VLI Com. Deck	51	VULCRAFT MNL
Beams/Columns	5	ESTIMATE
Paritions	10	12.14.8.1
MEP/Misc.	5	
Ceiling	3	
Total	74	
Roof Dead Load	**Load (psf)**	**Reference**
2C20, 4.5" NonComp	45	PCI MNL 120
Beams/Columns	2	ESTIMATE
MEP/Misc.	5	
Ceiling	3	
Insulation	12	
Total	67	
Enclosure	**Load (plf)**	**Reference**
Cold Formed Stud wall and Estimated Façade	150	Dri Design and ASCE 7-05

Appendix D: Gravity Design

Gravity Beam Design

RAM Steel v14.05.01.00
DataBase: full wind 4 braced frames smaller mfs 03/17/13 13:07:52
Building Code: IBC Steel Code: AISC 360-10 LRFD

Floor Type: 3rd **Beam Number = 248**

SPAN INFORMATION (ft): I-End (40.22,-27.88) J-End (40.22,-4.00)
Beam Size (Optimum) = W14X22 Fy = 50.0 ksi
Total Beam Length (ft) = 23.88

COMPOSITE PROPERTIES (Not Shored):

	Left	Right
Deck Label	5.5" 3VLI composite deck	5.5" 3VLI composite deck
Concrete thickness (in)	2.50	2.50
Unit weight concrete (pcf)	150.00	150.00
f'c (ksi)	4.00	4.00
Decking Orientation	perpendicular	perpendicular
Decking type	VULCRAFT 3.0VL	VULCRAFT 3.0VL

beff (in)	=	71.63	Y bar(in)	=	15.56
Mnf (kip-ft)	=	315.95	Mn (kip-ft)	=	218.09
C (kips)	=	95.43	PNA (in)	=	11.00
Ieff (in4)	=	546.02	Itr (in4)	=	838.92
Stud length (in)	=	4.50	Stud diam (in)	=	0.75

Stud Capacity (kips) Qn = 15.9 Rg = 1.00 Rp = 0.60
# of studs: Max = 23 Partial = 12 Actual = 12
Number of Stud Rows = 1 Percent of Full Composite Action = 29.41

LINE LOADS (k/ft):

Load	Dist	DL	CDL	LL	Red%	Type	PartL	CLL
1	0.000	0.623	0.504	0.396	6.0%	Red	0.198	0.000
	23.875	0.623	0.504	0.396			0.198	0.000
2	0.000	0.022	0.022	0.000	---	NonR	0.000	0.000
	23.875	0.022	0.022	0.000			0.000	0.000

SHEAR (Ultimate): Max Vu (1.2DL+1.6LL) = 20.12 kips 1.00Vn = 94.53 kips

MOMENTS (Ultimate):

Span	Cond	LoadCombo	Mu kip-ft	@ ft	Lb ft	Cb	Phi	Phi*Mn kip-ft
Center	PreCmp+	1.4DL	52.5	11.9	0.0	1.00	0.90	124.50
	Init DL	1.4DL	52.5	11.9	---	---		
	Max +	1.2DL+1.6LL	120.1	11.9	---	---	0.90	196.28
Controlling		1.2DL+1.6LL	120.1	11.9	---	---	0.90	196.28

REACTIONS (kips):

	Left	Right
Initial reaction	6.28	6.28
DL reaction	7.70	7.70
Max +LL reaction	6.80	6.80
Max +total reaction (factored)	20.12	20.12

DEFLECTIONS:

Gravity Beam Design

RAM Steel v14.05.01.00 Page 2/2
DataBase: full wind 4 braced frames smaller mfs 03/17/13 13:07:52
Building Code: IBC Steel Code: AISC 360-10 LRFD

Initial load (in)	at	11.94 ft	=	-0.667	L/D	=	430
Live load (in)	at	11.94 ft	=	-0.263	L/D	=	1089
Post Comp load (in)	at	11.94 ft	=	-0.318	L/D	=	901
Net Total load (in)	at	11.94 ft	=	-0.985	L/D	=	291

Gravity Beam Design

RAM Steel v14.05.01.00
DataBase: full wind 4 braced frames smaller mfs — 03/17/13 13:07:52
Building Code: IBC — Steel Code: AISC 360-10 LRFD

Floor Type: 3rd **Beam Number = 26**

SPAN INFORMATION (ft): I-End (30.33,-27.88) J-End (60.00,-27.88)

Beam Size (Optimum) = W18X40 Fy = 50.0 ksi
Total Beam Length (ft) = 29.67

COMPOSITE PROPERTIES (Not Shored):

	Left	Right
Deck Label	5.5" 3VLI composite deck	5.5" 3VLI composite deck
Concrete thickness (in)	2.50	2.50
Unit weight concrete (pcf)	150.00	150.00
f'c (ksi)	4.00	4.00
Decking Orientation	parallel	parallel
Decking type	VULCRAFT 3.0VL	VULCRAFT 3.0VL

beff (in)	=	50.50	Y bar(in)	=	16.68
Mnf (kip-ft)	=	590.34	Mn (kip-ft)	=	480.09
C (kips)	=	159.04	PNA (in)	=	14.00
Ieff (in4)	=	1350.48	Itr (in4)	=	1825.27
Stud length (in)	=	4.50	Stud diam (in)	=	0.75

Stud Capacity (kips) Qn = 19.9 Rg = 1.00 Rp = 0.75
\# of studs: Full = 60 Partial = 16 Actual = 16
Number of Stud Rows = 1 Percent of Full Composite Action = 27.10

POINT LOADS (kips):

Dist	DL	CDL	RedLL	Red%	NonRLL	StorLL	Red%	RoofLL	Red%	PartL	
9.890	7.70	6.28	4.72	8.1	0.00	0.00	0.0	0.00	Snow	2.36	0.00
19.780	7.70	6.28	4.72	8.1	0.00	0.00	0.0	0.00	Snow	2.36	0.00

LINE LOADS (k/ft):

Load	Dist	DL	CDL	LL	Red%	Type	PartL	CLL
1	0.000	0.200	0.000	0.000	---	NonR	0.000	0.000
	29.670	0.200	0.000	0.000			0.000	0.000
2	0.000	0.032	0.026	0.020	8.1%	Red	0.010	0.000
	29.161	0.032	0.026	0.020			0.010	0.000
3	29.162	0.032	0.026	0.020	8.1%	Red	0.010	0.000
	29.670	0.000	0.000	0.000			0.000	0.000
4	0.000	0.040	0.040	0.000	---	NonR	0.000	0.000
	29.670	0.040	0.040	0.000			0.000	0.000

SHEAR (Ultimate): Max Vu (1.2DL+1.6LL) = 25.50 kips 1.00Vn = 169.15 kips

MOMENTS (Ultimate):

Span	Cond	LoadCombo	Mu kip-ft	@ ft	Lb ft	Cb	Phi	Phi*Mn kip-ft
Center	PreCmp+	1.4DL	97.2	14.8	0.0	1.00	0.90	294.00
	Init DL	1.4DL	97.2	14.8	---	---		
	Max +	1.2DL+1.6LL	238.5	14.8	---	---	0.90	432.08
Controlling		1.2DL+1.6LL	238.5	14.8	---	---	0.90	432.08

Gravity Beam Design

RAM Steel v14.05.01.00 — Page 2/2
DataBase: full wind 4 braced frames smaller mfs — 03/17/13 13:07:52
Building Code: IBC — Steel Code: AISC 360-10 LRFD

REACTIONS (kips):

	Left	Right
Initial reaction	7.26	7.26
DL reaction	11.74	11.73
Max +LL reaction	7.13	7.12
Max +total reaction (factored)	25.50	25.47

DEFLECTIONS:

Initial load (in)	at	14.83 ft	=	-0.632	L/D	=	563
Live load (in)	at	14.83 ft	=	-0.287	L/D	=	1240
Post Comp load (in)	at	14.83 ft	=	-0.437	L/D	=	815
Net Total load (in)	at	14.83 ft	=	-1.069	L/D	=	333

Gravity Column Design

RAM Steel v14.05.01.00 Page 5/5
DataBase: full wind 4 braced frames smaller mfs 03/19/13 13:15:55
Building Code: IBC Steel Code: AISC 360-10 LRFD

Story level 2nd, Column Line 16-E

Fy (ksi) = 50.00 Column Size = W10X33
Orientation (deg.) = 90.0

INPUT DESIGN PARAMETERS:

		X-Axis	Y-Axis
Lu (ft)		12.00	12.00
K		1	1
Braced Against Joint Translation		Yes	Yes
Column Eccentricity (in)	Top	8.36	3.50
	Bottom	0.00	0.00

CONTROLLING COLUMN LOADS - Skip-Load Case 1:

			Dead	Live	Roof
Axial (kip)			105.98	49.21	7.26
Moments	Top	Mx (kip-ft)	-0.99	-0.58	0.00
		My (kip-ft)	-0.00	0.00	0.00
	Bot	Mx (kip-ft)	0.00	0.00	0.00
		My (kip-ft)	0.00	0.00	0.00

Single curvature about X-Axis
Single curvature about Y-Axis

CALCULATED PARAMETERS: (1.2DL + 1.6LL + 0.5RF)

Pu (kip)	=	209.53	0.90*Pn (kip)	=	292.24
Mux (kip-ft)	=	2.12	0.90*Mnx (kip-ft)	=	145.50
Muy (kip-ft)	=	0.00	0.90*Mny (kip-ft)	=	52.50
Rm	=	1.00			
Cbx	=	1.67			
Cmx	=	0.60	Cmy	=	0.60
Pex (kip)	=	2360.31	Pey (kip)	=	505.19
B1x	=	1.00	B1y	=	1.03

INTERACTION EQUATION

Pu/0.90*Pn = 0.717
Eq H1-1a: 0.717 + 0.013 + 0.000 = 0.730

Gravity Column Design Summary

RAM Steel v14.05.01.00
DataBase: full wind 4 braced frames smaller mfs 03/19/13 13:17:35
Building Code: IBC Steel Code: AISC 360-10 LRFD

Column Line 1--23.87ft

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	12.4	7.7	0.3	1	0.07 Eq (H1-1b)	90.0	50	W10X33
5th	22.4	2.8	0.2	1	0.07 Eq (H1-1b)	90.0	50	W10X33
4th	35.3	4.0	0.2	1	0.11 Eq (H1-1b)	90.0	50	W10X33
3rd	48.1	4.1	0.2	1	0.15 Eq (H1-1b)	90.0	50	W10X33
2nd	60.2	3.7	0.2	1	0.23 Eq (H1-1a)	90.0	50	W10X33

Column Line 1-E

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	13.6	6.4	0.0	12	0.07 Eq (H1-1b)	90.0	50	W10X33
5th	27.9	1.7	0.0	1	0.09 Eq (H1-1b)	90.0	50	W10X33
4th	43.1	2.4	0.0	1	0.14 Eq (H1-1b)	90.0	50	W10X33
3rd	57.7	2.4	0.0	1	0.19 Eq (H1-1b)	90.0	50	W10X33
2nd	72.2	2.1	0.0	1	0.26 Eq (H1-1a)	90.0	50	W10X33

Column Line 1-F

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	13.6	6.4	0.0	8	0.07 Eq (H1-1b)	90.0	50	W10X33
5th	29.2	3.3	0.0	1	0.09 Eq (H1-1b)	90.0	50	W10X33
4th	48.5	3.3	0.0	1	0.16 Eq (H1-1b)	90.0	50	W10X33
3rd	67.7	4.2	0.0	2	0.24 Eq (H1-1a)	90.0	50	W10X33
2nd	86.9	3.1	0.0	1	0.32 Eq (H1-1a)	90.0	50	W10X33

Column Line 1-23.87ft

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	12.4	7.7	0.3	1	0.07 Eq (H1-1b)	90.0	50	W10X33
5th	24.3	4.9	0.3	1	0.08 Eq (H1-1b)	90.0	50	W10X33
4th	40.6	4.9	0.3	1	0.13 Eq (H1-1b)	90.0	50	W10X33
3rd	56.8	5.1	0.3	1	0.18 Eq (H1-1b)	90.0	50	W10X33
2nd	73.1	4.7	0.2	1	0.28 Eq (H1-1a)	90.0	50	W10X33

Column Line 2-F

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	27.0	7.1	3.4	8	0.16 Eq (H1-1b)	90.0	50	W10X33
5th	60.8	3.9	2.6	1	0.20 Eq (H1-1b)	90.0	50	W10X33
4th	97.4	4.8	2.4	2	0.38 Eq (H1-1a)	90.0	50	W10X33
3rd	133.4	4.9	2.4	2	0.50 Eq (H1-1a)	90.0	50	W10X33
2nd	169.0	3.7	2.2	1	0.64 Eq (H1-1a)	90.0	50	W10X33

Column Line 2-23.87ft

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	25.8	8.8	3.3	6	0.16 Eq (H1-1b)	90.0	50	W10X33
5th	54.1	5.9	2.2	1	0.17 Eq (H1-1b)	90.0	50	W10X33

Gravity Column Design Summary

RAM Steel v14.05.01.00	Page 2/7
DataBase: full wind 4 braced frames smaller mfs	03/19/13 13:17:35
Building Code: IBC	Steel Code: AISC 360-10 LRFD

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
4th	87.2	5.8	2.1	2	0.35 Eq (H1-1a)	90.0	50	W10X33
3rd	119.7	5.9	2.0	3	0.45 Eq (H1-1a)	90.0	50	W10X33
2nd	151.2	5.4	1.8	1	0.58 Eq (H1-1a)	90.0	50	W10X33

Column Line 3-C

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
2nd	210.6	151.2	6.6	1	0.51 Eq (H1-1a)	90.0	50	W12X79

Column Line 3-D

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	39.8	15.0	5.2	1	0.27 Eq (H1-1b)	90.0	50	W10X33
5th	81.8	9.9	3.5	1	0.38 Eq (H1-1a)	90.0	50	W10X33
4th	129.5	9.4	3.3	1	0.53 Eq (H1-1a)	90.0	50	W10X33
3rd	176.5	9.2	3.2	1	0.68 Eq (H1-1a)	90.0	50	W10X33

Column Line 3-E

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	40.7	7.6	2.8	15	0.14 Eq (H1-1b)	90.0	50	W10X39
5th	99.9	5.5	2.3	3	0.33 Eq (H1-1a)	90.0	50	W10X39
4th	158.8	5.2	2.1	3	0.48 Eq (H1-1a)	90.0	50	W10X39
3rd	216.8	13.7	2.5	3	0.68 Eq (H1-1a)	90.0	50	W10X39
2nd	298.3	11.7	1.6	1	0.92 Eq (H1-2)	90.0	50	W10X39

Column Line 3-F

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	40.3	7.4	2.7	14	0.17 Eq (H1-1b)	90.0	50	W10X33
5th	99.1	5.3	2.3	2	0.39 Eq (H1-1a)	90.0	50	W10X33
4th	157.5	5.0	2.1	2	0.57 Eq (H1-1a)	90.0	50	W10X33
3rd	215.0	5.3	2.1	2	0.76 Eq (H1-1a)	90.0	50	W10X33
2nd	272.4	4.1	1.3	1	0.96 Eq (H1-2)	90.0	50	W10X33

Column Line 3-J

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	39.7	15.1	5.2	1	0.27 Eq (H1-1b)	90.0	50	W10X33
5th	81.5	9.9	3.4	1	0.38 Eq (H1-1a)	90.0	50	W10X33
4th	128.9	9.4	3.3	1	0.53 Eq (H1-1a)	90.0	50	W10X33
3rd	175.6	11.3	3.3	1	0.69 Eq (H1-1a)	90.0	50	W10X33
2nd	226.4	10.2	3.3	1	0.89 Eq (H1-1a)	90.0	50	W10X33

Column Line 5-G

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	37.3	13.4	0.0	12	0.15 Eq (H1-1b)	90.0	50	W10X33
5th	92.6	10.6	0.0	3	0.36 Eq (H1-1a)	90.0	50	W10X33
4th	147.2	9.9	0.0	3	0.53 Eq (H1-1a)	90.0	50	W10X33

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Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
3rd	200.9	10.1	0.0	5	0.71 Eq (H1-1a)	90.0	50	W10X33
2nd	254.0	6.6	0.0	1	0.91 Eq (H1-1a)	90.0	50	W10X33

Column Line 5-K

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	46.2	19.0	5.5	1	0.26 Eq (H1-1b)	90.0	50	W10X39
5th	94.5	12.3	3.6	1	0.37 Eq (H1-1a)	90.0	50	W10X39
4th	149.6	11.8	3.4	1	0.51 Eq (H1-1a)	90.0	50	W10X39
3rd	203.8	12.1	3.4	1	0.65 Eq (H1-1a)	90.0	50	W10X39
2nd	256.8	10.9	3.2	1	0.83 Eq (H1-1a)	90.0	50	W10X39

Column Line 7-E

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	49.6	0.9	2.6	1	0.13 Eq (H1-1b)	0.0	50	W10X39
5th	109.1	4.3	2.2	3	0.34 Eq (H1-1a)	0.0	50	W10X39
4th	173.7	3.8	2.1	3	0.51 Eq (H1-1a)	0.0	50	W10X39
3rd	237.9	3.9	2.1	3	0.69 Eq (H1-1a)	0.0	50	W10X39
2nd	301.0	0.6	1.9	1	0.89 Eq (H1-1a)	0.0	50	W10X39

Column Line 7-F

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	49.7	0.9	2.6	1	0.13 Eq (H1-1b)	0.0	50	W10X39
5th	109.2	4.3	2.2	4	0.34 Eq (H1-1a)	0.0	50	W10X39
4th	173.7	3.8	2.1	4	0.51 Eq (H1-1a)	0.0	50	W10X39
3rd	238.0	3.9	2.1	4	0.69 Eq (H1-1a)	0.0	50	W10X39
2nd	301.1	0.6	1.9	1	0.89 Eq (H1-1a)	0.0	50	W10X39

Column Line 113.34ft--40.00ft

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
2nd	12.1	6.1	0.8	1	0.06 Eq (H1-2)	90.0	50	W10X33

Column Line 130.33ft--40.00ft

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
2nd	11.7	5.8	0.8	1	0.06 Eq (H1-2)	90.0	50	W10X33

Column Line 12-C

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	42.9	9.8	2.2	10	0.18 Eq (H1-1b)	90.0	50	W10X33
5th	100.3	6.3	1.5	4	0.39 Eq (H1-1a)	90.0	50	W10X33
4th	158.9	8.1	1.4	5	0.58 Eq (H1-1a)	90.0	50	W10X33
3rd	216.6	3.8	1.1	1	0.82 Eq (H1-1a)	90.0	50	W10X33
2nd	230.8	3.8	1.0	1	0.87 Eq (H1-1a)	90.0	50	W10X33

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Column Line 12-E

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	49.7	0.9	2.6	1	0.16 Eq (H1-1b)	0.0	50	W10X33
5th	109.1	4.3	2.2	3	0.41 Eq (H1-1a)	0.0	50	W10X33
4th	173.6	3.9	2.1	3	0.62 Eq (H1-1a)	0.0	50	W10X33
3rd	237.0	3.1	2.2	5	0.82 Eq (H1-1a)	0.0	50	W10X33
2nd	259.9	1.9	0.8	1	0.92 Eq (H1-1a)	0.0	50	W10X33

Column Line 12-F

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	49.8	0.9	2.6	1	0.13 Eq (H1-1b)	0.0	50	W10X39
5th	109.3	4.3	2.2	4	0.34 Eq (H1-1a)	0.0	50	W10X39
4th	173.8	3.9	2.1	4	0.51 Eq (H1-1a)	0.0	50	W10X39
3rd	237.3	3.9	2.1	4	0.68 Eq (H1-1a)	0.0	50	W10X39
2nd	300.4	0.6	1.9	1	0.88 Eq (H1-1a)	0.0	50	W10X39

Column Line 14-C

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	35.4	12.8	4.9	1	0.24 Eq (H1-1b)	90.0	50	W10X33
5th	73.7	8.6	3.3	1	0.34 Eq (H1-1a)	90.0	50	W10X33
4th	116.7	8.2	3.1	1	0.48 Eq (H1-1a)	90.0	50	W10X33
3rd	158.9	8.1	3.1	1	0.61 Eq (H1-1a)	90.0	50	W10X33
2nd	197.0	7.4	2.4	1	0.76 Eq (H1-1a)	90.0	50	W10X33

Column Line 14-E

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	34.9	6.9	3.4	16	0.17 Eq (H1-1b)	90.0	50	W10X33
5th	84.4	5.2	2.7	4	0.35 Eq (H1-1a)	90.0	50	W10X33
4th	134.1	4.9	2.5	4	0.50 Eq (H1-1a)	90.0	50	W10X33
3rd	183.0	4.2	2.1	4	0.65 Eq (H1-1a)	90.0	50	W10X33
2nd	226.4	3.1	1.5	1	0.82 Eq (H1-1a)	90.0	50	W10X33

Column Line 15-D

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	27.3	13.9	2.0	1	0.14 Eq (H1-2)	90.0	50	W10X33
5th	58.4	9.7	1.5	1	0.19 Eq (H1-1b)	90.0	50	W10X33
4th	92.2	9.1	1.4	1	0.37 Eq (H1-1a)	90.0	50	W10X33
3rd	125.1	10.9	1.5	1	0.49 Eq (H1-1a)	90.0	50	W10X33
2nd	160.3	9.6	1.3	1	0.63 Eq (H1-1a)	90.0	50	W10X33

Column Line 15-E

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	29.8	6.7	2.1	15	0.13 Eq (H1-1b)	90.0	50	W10X33
5th	73.1	5.0	1.7	3	0.29 Eq (H1-1a)	90.0	50	W10X33

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Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
4th	115.9	4.7	1.6	5	0.43 Eq (H1-1a)	90.0	50	W10X33
3rd	158.0	5.0	1.6	3	0.56 Eq (H1-1a)	90.0	50	W10X33
2nd	200.3	3.9	0.9	1	0.73 Eq (H1-1a)	90.0	50	W10X33

Column Line 15-F

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	29.6	6.6	2.1	14	0.13 Eq (H1-1b)	90.0	50	W10X33
5th	72.7	4.9	1.7	2	0.29 Eq (H1-1a)	90.0	50	W10X33
4th	115.2	4.6	1.6	2	0.43 Eq (H1-1a)	90.0	50	W10X33
3rd	157.0	4.7	1.6	2	0.56 Eq (H1-1a)	90.0	50	W10X33
2nd	198.5	3.6	0.9	1	0.72 Eq (H1-1a)	90.0	50	W10X33

Column Line 15-J

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	27.3	13.9	2.0	1	0.14 Eq (H1-2)	90.0	50	W10X33
5th	58.3	9.7	1.4	1	0.19 Eq (H1-1b)	90.0	50	W10X33
4th	92.0	9.1	1.4	1	0.37 Eq (H1-1a)	90.0	50	W10X33
3rd	124.9	9.2	1.4	1	0.48 Eq (H1-1a)	90.0	50	W10X33
2nd	157.5	8.3	1.3	1	0.61 Eq (H1-1a)	90.0	50	W10X33

Column Line 16--23.87ft

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	28.9	6.5	1.3	6	0.12 Eq (H1-1b)	90.0	50	W10X33
5th	68.6	4.2	0.9	4	0.26 Eq (H1-1a)	90.0	50	W10X33
4th	108.5	3.9	0.8	4	0.39 Eq (H1-1a)	90.0	50	W10X33
3rd	147.6	3.8	1.0	4	0.51 Eq (H1-1a)	90.0	50	W10X33
2nd	195.3	2.4	0.0	1	0.68 Eq (H1-1a)	90.0	50	W10X33

Column Line 16-E

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	34.6	3.4	0.0	1	0.11 Eq (H1-1b)	90.0	50	W10X33
5th	76.6	3.2	1.2	4	0.29 Eq (H1-1a)	90.0	50	W10X33
4th	121.6	3.0	1.0	4	0.43 Eq (H1-1a)	90.0	50	W10X33
3rd	165.8	3.1	1.0	4	0.57 Eq (H1-1a)	90.0	50	W10X33
2nd	209.5	2.1	0.0	1	0.73 Eq (H1-1a)	90.0	50	W10X33

Column Line 16-F

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	34.6	3.4	0.0	1	0.11 Eq (H1-1b)	90.0	50	W10X33
5th	76.6	3.2	1.2	5	0.29 Eq (H1-1a)	90.0	50	W10X33
4th	121.6	3.0	1.0	5	0.43 Eq (H1-1a)	90.0	50	W10X33
3rd	165.8	3.1	1.0	5	0.57 Eq (H1-1a)	90.0	50	W10X33
2nd	209.5	2.1	0.0	1	0.73 Eq (H1-1a)	90.0	50	W10X33

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Building Code: IBC
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Steel Code: AISC 360-10 LRFD

Column Line 16-23.87ft

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	28.9	6.5	1.4	6	0.12 Eq (H1-1b)	90.0	50	W10X33
5th	68.5	4.2	0.9	2	0.26 Eq (H1-1a)	90.0	50	W10X33
4th	108.4	3.9	0.8	2	0.39 Eq (H1-1a)	90.0	50	W10X33
3rd	147.5	4.0	0.9	3	0.51 Eq (H1-1a)	90.0	50	W10X33
2nd	186.5	3.6	0.0	1	0.66 Eq (H1-1a)	90.0	50	W10X33

Column Line 17--23.87ft

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	25.7	8.8	3.3	10	0.16 Eq (H1-1b)	90.0	50	W10X33
5th	54.0	5.9	2.2	1	0.17 Eq (H1-1b)	90.0	50	W10X33
4th	87.0	5.7	2.1	4	0.35 Eq (H1-1a)	90.0	50	W10X33
3rd	119.5	5.9	2.6	5	0.46 Eq (H1-1a)	90.0	50	W10X33
2nd	154.4	5.4	2.3	1	0.60 Eq (H1-1a)	90.0	50	W10X33

Column Line 17-E

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	27.0	7.1	3.4	12	0.16 Eq (H1-1b)	90.0	50	W10X33
5th	60.7	3.9	2.6	1	0.19 Eq (H1-1b)	90.0	50	W10X33
4th	97.2	4.7	2.4	3	0.38 Eq (H1-1a)	90.0	50	W10X33
3rd	133.1	4.9	2.4	4	0.50 Eq (H1-1a)	90.0	50	W10X33
2nd	168.7	3.7	2.2	1	0.64 Eq (H1-1a)	90.0	50	W10X33

Column Line 18--23.87ft

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	12.4	7.6	0.3	1	0.07 Eq (H1-1b)	90.0	50	W10X33
5th	24.2	4.9	0.3	1	0.08 Eq (H1-1b)	90.0	50	W10X33
4th	40.4	4.9	0.3	1	0.13 Eq (H1-1b)	90.0	50	W10X33
3rd	56.6	5.1	0.3	1	0.18 Eq (H1-1b)	90.0	50	W10X33
2nd	72.8	4.7	0.2	1	0.28 Eq (H1-1a)	90.0	50	W10X33

Column Line 18-E

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	13.5	6.4	0.0	12	0.07 Eq (H1-1b)	90.0	50	W10X33
5th	29.1	3.3	0.0	1	0.09 Eq (H1-1b)	90.0	50	W10X33
4th	48.3	3.3	0.0	1	0.15 Eq (H1-1b)	90.0	50	W10X33
3rd	67.4	4.2	0.0	3	0.24 Eq (H1-1a)	90.0	50	W10X33
2nd	86.6	3.1	0.0	1	0.32 Eq (H1-1a)	90.0	50	W10X33

Column Line 18-F

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	13.5	6.4	0.0	8	0.07 Eq (H1-1b)	90.0	50	W10X33
5th	27.8	1.7	0.0	1	0.09 Eq (H1-1b)	90.0	50	W10X33

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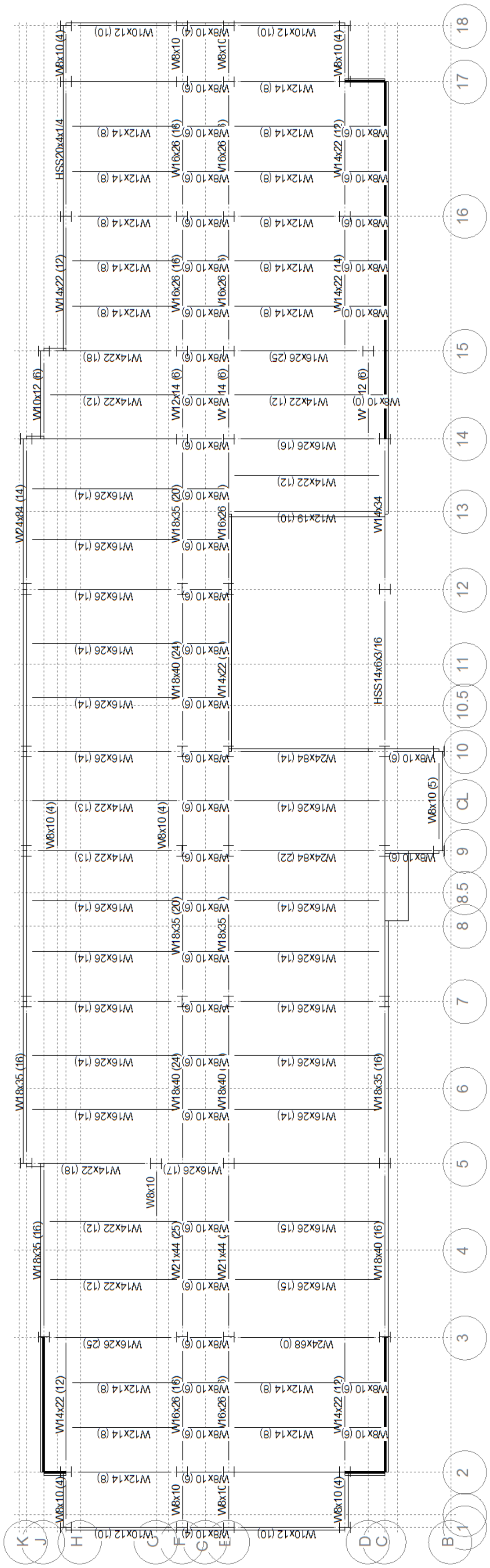
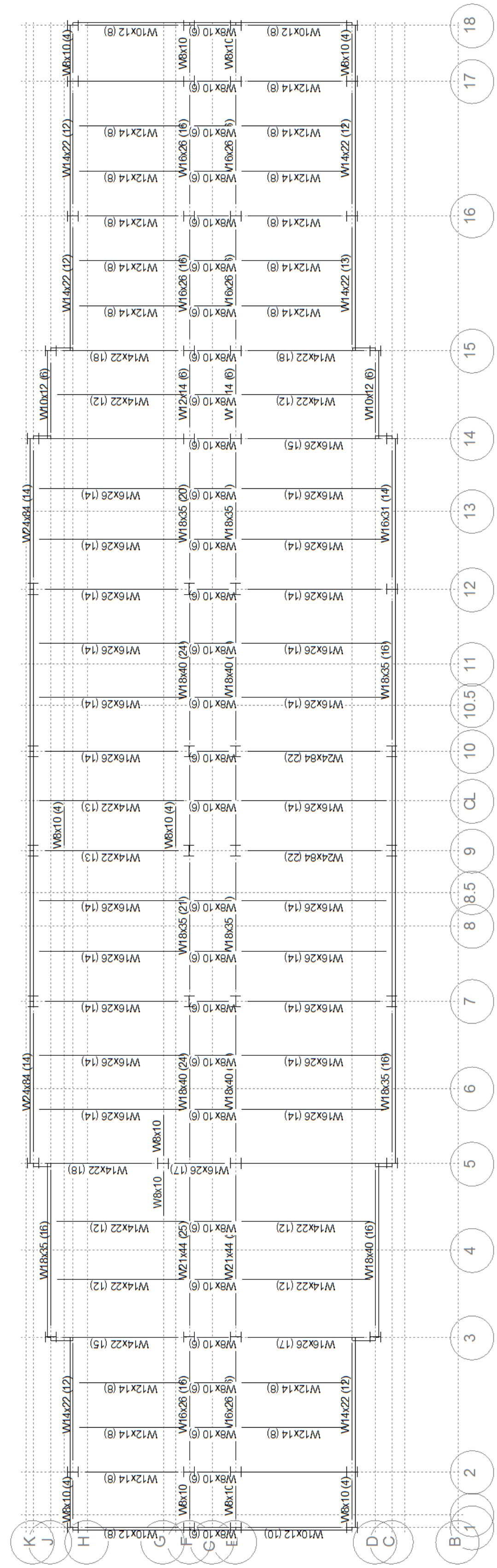
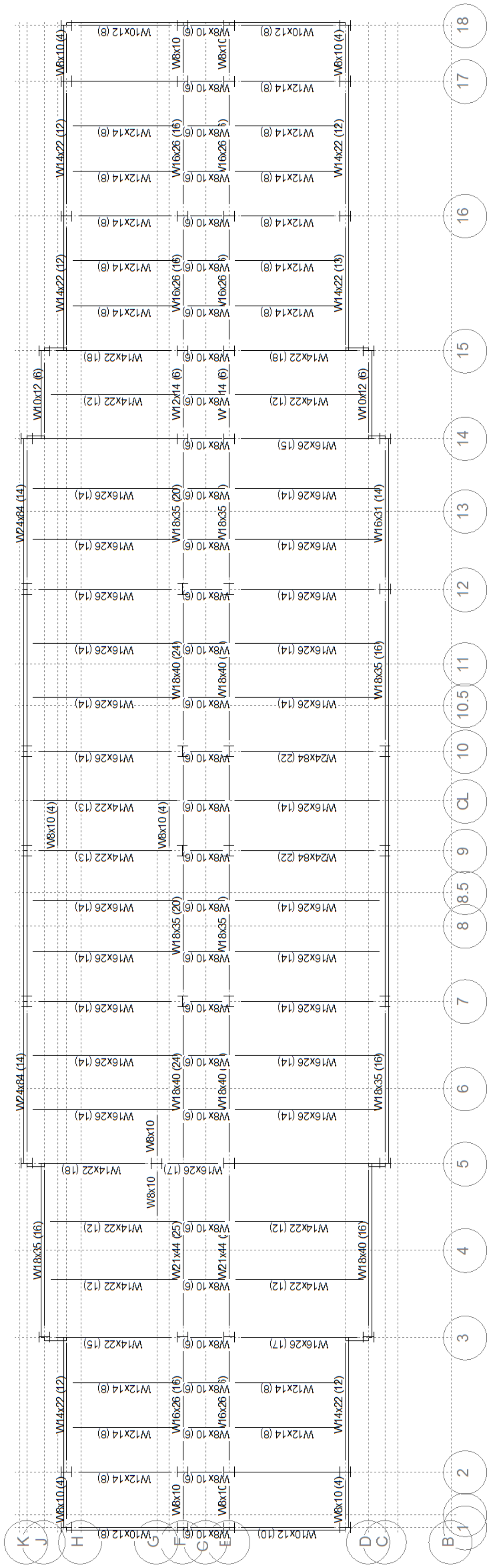
Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
4th	42.9	2.4	0.0	1	0.14 Eq (H1-1b)	90.0	50	W10X33
3rd	57.5	2.4	0.0	1	0.18 Eq (H1-1b)	90.0	50	W10X33
2nd	71.9	2.1	0.0	1	0.26 Eq (H1-1a)	90.0	50	W10X33

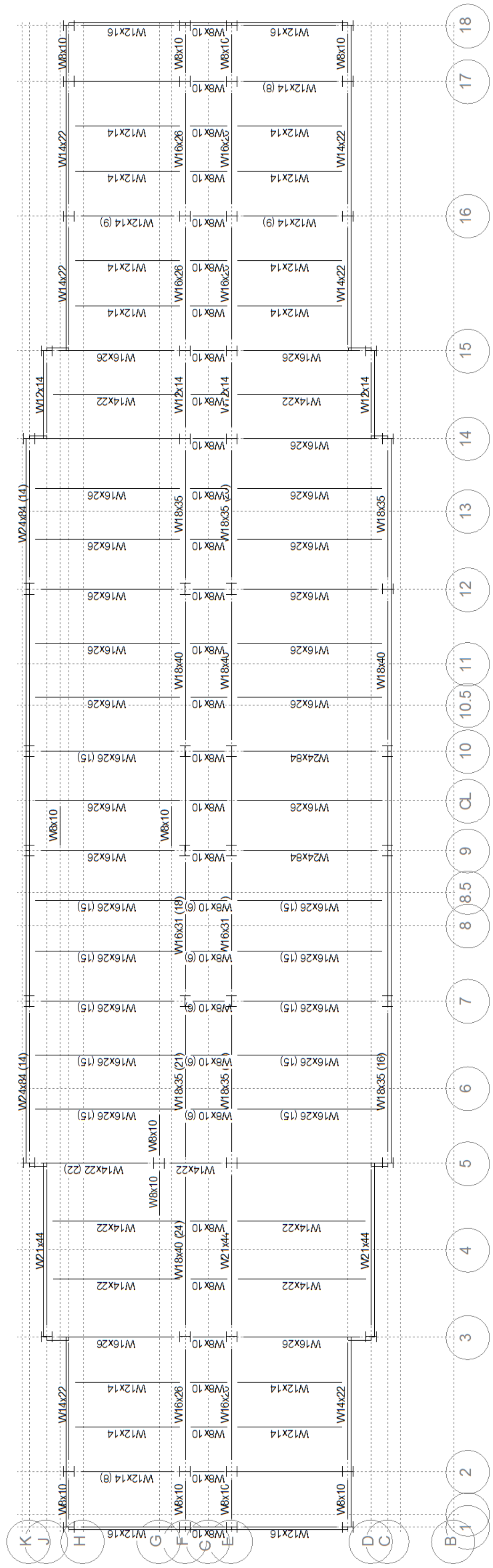
Column Line 18-23.87ft

Level	Pu	Mux	Muy	LC	Interaction Eq.	Angle	Fy	Size
Roof	12.4	7.6	0.3	1	0.07 Eq (H1-1b)	90.0	50	W10X33
5th	22.3	2.8	0.2	1	0.07 Eq (H1-1b)	90.0	50	W10X33
4th	35.1	4.0	0.2	1	0.11 Eq (H1-1b)	90.0	50	W10X33
3rd	47.9	4.1	0.2	1	0.15 Eq (H1-1b)	90.0	50	W10X33
2nd	60.0	3.7	0.2	1	0.23 Eq (H1-1a)	90.0	50	W10X33









TECH 1 | SNOW LOAD | 1 | JORDAN RUTHERFORD

FLAT ROOF SNOW LOAD:

GROUND SNOW LOAD: (FIG 7-1) 30 psf
IMPORTANCE FACTOR: (TBL 7-4) 1.0
EXPOSURE FACTOR: (TBL 7-2) 1.0 (CONSERVATIVE)
THERMAL FACTOR: (TBL 7-3) 1.0

$p_f = 0.7(1.0)(1.0)(1.0)(30) = \boxed{21\text{ psf}}$

SNOW DRIFT: (7.8 / 7.7.1)

SPECIFIC GRAVITY: $\gamma = 0.13(30) + 14 = 17.9$ pcf < 30 pcf OK ✓

BASE ACCUMULATION: $h_b = p_f / \gamma = 21/17.9 = 1.17'$

$h_c = 8.67' - 1.17' = 7.5'$

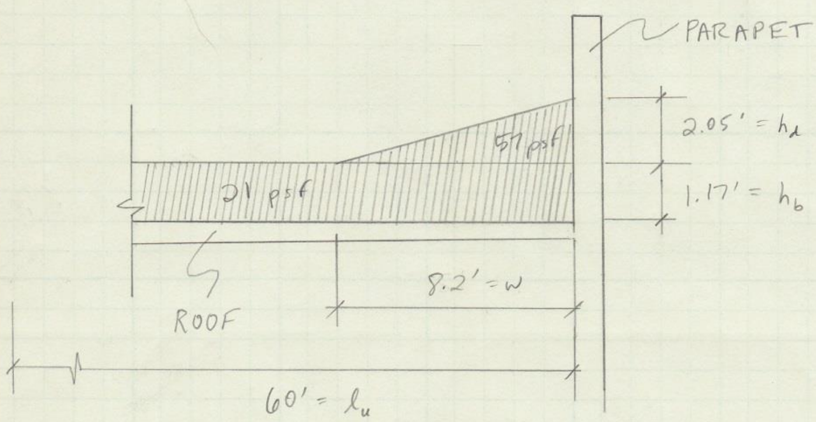
$h_c / h_b = 7.5'/1.17 = 6.41 > 0.2$ MUST CALC. DRIFT

WINDWARD DRIFT:

$h_d = 0.75(0.43\sqrt[3]{60}\sqrt[4]{30+10} - 1.5) = 2.05'$

$W = 4(2.05') = 8.2'$

$p_d = (17.9\text{ pcf})(2.05') = \boxed{57\text{ psf}}$



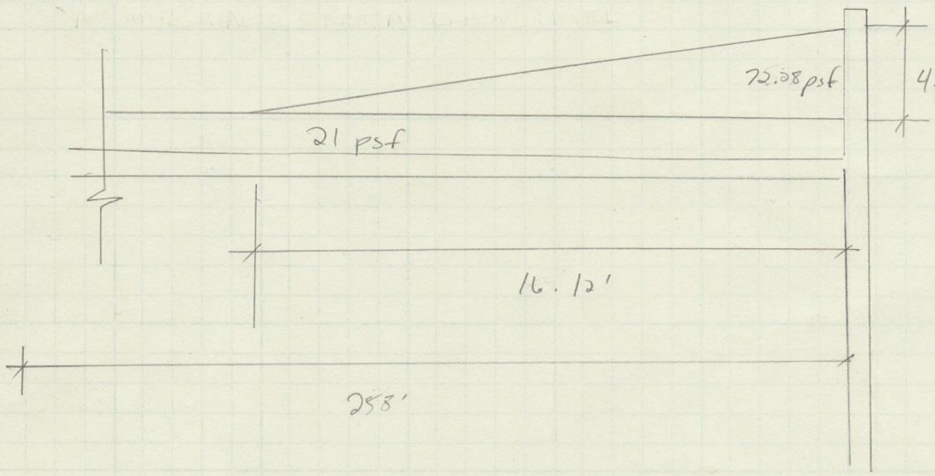
TECH 1 | SNOW LOAD 2 | JORDAN RUTHERFORD

DRIFT ALONG LENGTH

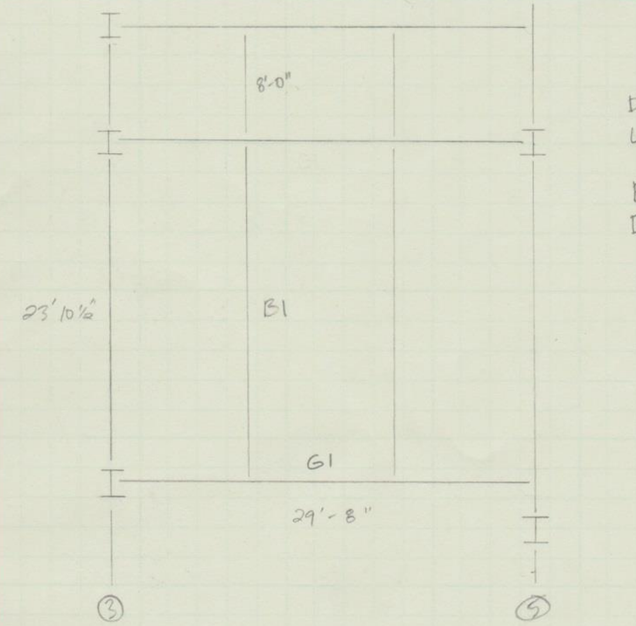
$$h_d = 0.75\left(.43\sqrt[3]{258}\sqrt[4]{30+10} - 1.5\right) = 4.03'$$

$$w = 4(4.03') = 16.12'$$

$$p_d = \gamma(4.03') = 72.28\ psf$$



GRAV. BEAM DESIGN | JORDAN RUTHERFORD | 1



DEAD LOAD: 51 psf
LIVE LOAD: 60 psf
SDL: 12 psf
EXT WALL: 200 - 400 plf
BM SLF WT: 5 psf

3VLI22 DECK

B1 CHECK:

LL REDUCTION: $L = 60\left(.25 + \frac{15}{\sqrt{472}}\right) = 56.4$ psf

b_{eff} = MIN | $(29.67)(12)/8 = 44.5\,(2) = \boxed{89"}$ USE
| $(9.89)(12)/2 = 59.34\,(2) = 119"$

$w_u = 1.2\left[(51+12+5)(9.89')\right] + 1.6\left[(56.4)(9.89')\right] = 1.7$ klf

$M_u = \frac{1.7 \text{ klf } (23.875')^2}{8} = 121$ k-ft

$V_u = \frac{1.7 \text{ klf } (23.875')}{2} = 21$ k

ASSUME $a = 1"$

$Y_2 = 5.5" - 1"/2 = 5"$

STUDS ARE ⊥, 3/4" ⌀, WEAK, 1 PER RIB = 17.2 k

SIZE BEAM:

TRY W14x22 $\quad \Sigma Q_n = 81.1 / 17.2 = 4.7 \longrightarrow 10$ STUDS

$\phi M_n = 185$

$I_{LB} = 381$

CHECK $a = \dfrac{81.1}{(.85)(4)(89)} = .268$ OK ✓

- CHECK UNSHORED STR:

$w_u = 1.2(51\,psf)(9.89') + 1.2(22\,plf) + 1.6(20\,psf)(9.89') = 0.95\,klf$

$M_u = \frac{1}{8}(0.95)(23.875')^2 = 67.7^{\,lk} < 125^{\,lk}$ OK ✓

- CHECK WET CONC DEFL:

$w_{wc} = (51\,psf)(9.89') + 22 = 0.526\,klf$

$\Delta_{wc} = \dfrac{5(.526)(23.875^4)(1728)}{384(29000)(199)} = 0.67" < 1.19" = \dfrac{23.875(12)}{240}$ OK ✓

- CHECK LL DEFL:

$\Delta_{LL} = \dfrac{5(0.558)(23.875^4)(1728)}{384(29000)(381)} = 0.369" < 0.796" = \dfrac{23.875(12)}{360}$ OK ✓

- CHECK TL DEFL.

$w_u = (51 + 12 + 56.4)(9.89) + 22 = 1.2\,klf$

$\Delta_{TL} = \dfrac{5(1.2)(23.875^4)(1728)}{384(29000)(381)} = 0.794" < 1.19"$ OK ✓

W14x22 w/ 10 STUDS | W14x22 w/ 12 STUDS (RAM) OK ✓

SIZE GIRDER:

TRY 18 x 40 $\Sigma Q_n = 148\text{ k} / 21.5 = 6.9 \rightarrow 14$ STUDS
$\phi M_n = 424\ ^{\text{'k}}$
$I_{LB} = 1070\text{ in}^4$

- CHECK UNSHORED STR.

$P_u = 1.2(51\text{psf})(9.89')(11.94') + 1.6(20\text{psf})(9.89')(11.94') = 11\text{ k}$

$w_u = 1.2(40\text{plf}) = 0.048\text{ klf}$

$M_u = \frac{0.048(29.67')^2}{8} + 11\text{k}(9.89') = 114\ ^{\text{'k}} < 294\ ^{\text{'k}}$ OK ✓

- CHECK WET CONC. DEFL.

$P_{uwc} = (51\text{psf})(9.89)(11.94) + (22\text{plf})(11.94') = 6.3\text{ k}$

$\Delta_{wc} = \frac{6.3\text{ k}(29.67^3)(1728)}{28(29000)(612)} = .572'' < 1.48''$ $\frac{29.67(12)}{240}$ OK ✓

- CHECK LL DEFL.

$P_{LL} = (48.4\text{psf})(9.89')(11.94') = 5.8\text{ k}$

$\Delta_{LL} = \frac{5.8(29.67^3)(1728)}{28(29000)(1070)} = .301'' < .989''$ $\frac{29.67(12)}{360}$ OK ✓

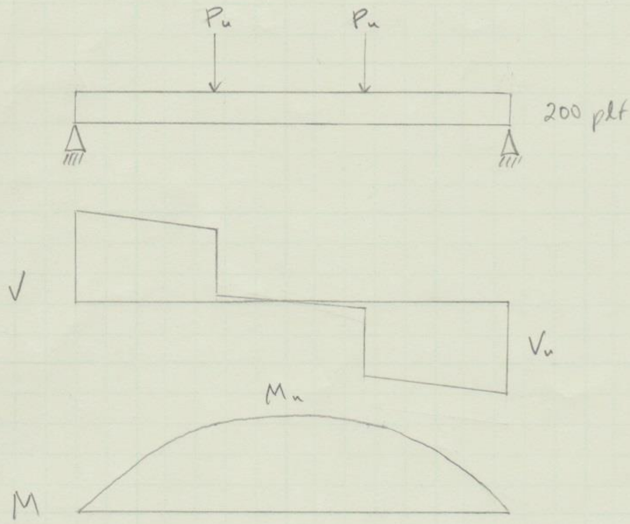
- CHECK TL DEFL

$P_{TL} = 1.2(51+12)(9.89)(11.94) + 1.6(48.4)(9.84)(11.94) = 18\text{ k}$

$\Delta_{TL} = \frac{18(29.67^3)(1728)}{28(29000)(1070)} = .935'' < 1.48''$ OK ✓

W 18 x 40 w/ 14 STUDS | W18 x 40 w/ 16 STUDS (RAM) OK ✓

G1 CHECK:



LL REDUCTION: $L = 60\left(.25 + \frac{15}{\sqrt{708}}\right) = 48.8\ plf$

$w_u = 1.2(200\ plf + 40\ plf) = 0.288\ klf$

$M_u = \frac{0.288(29.67')^2}{8} = 32^{'k}$

$P_u = [1.2(51+12) + 1.6(48.8)](9.89')(11.94') + (22plf)(11.94') = 18.4\ k$

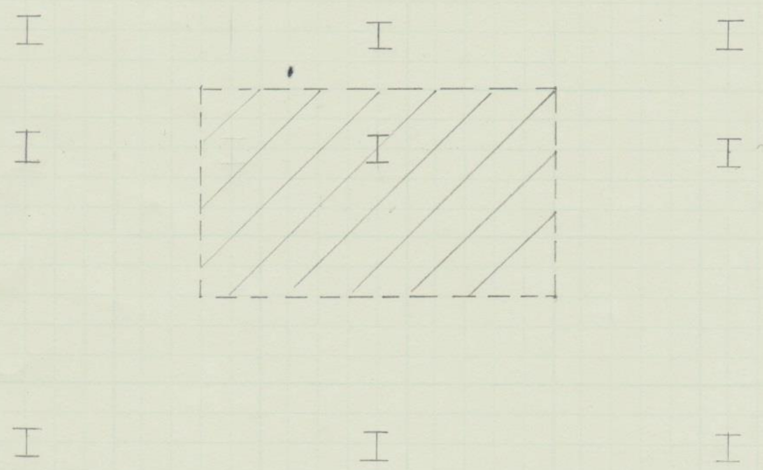
$M_{u_{TOT}} = 32^{'k} + (18.4k)(9.89') = 214^{'k}$

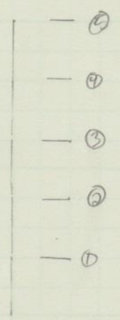
ASSUME $a = 1"$

$Y_2 = 5.5" - \frac{1}{2}(1") = 5"$

STUDS ARE 3/4"ϕ, ∥, WEAK, 1 PER RIB. = 21.5 k

GRAV. COLUMN | JORDAN RUTHERFORD | 1





DEAD LOAD : 60 psf
LIVE LOAD : 60 psf
ROOF DEAD : 65 psf
SNOW : 21 psf

$$LL_F = \max\begin{cases} \left(.25 + \dfrac{15}{\sqrt{(23)(14)(4)(3)}}\right) \\ 0.4 \end{cases} = .49\,(60) = 29.5 \text{ psf}$$

$\Delta_{max} = .28''$

$P_D = 1.2[(60)(3) + (65)(1)](23)(14) = 94.7$ k

$P_L = 1.6[(29.5)(3)](23)(14) = 45.6$ k

$P_S = 0.5[21](23)(14) = 3.4$ k

$P_u = 144$ k

$M_r = P\Delta = 144k\,(.28/12) = 3.36\ ^{k}$

TRY W10 x 33

$P_r = 144\,k$

$P_c = 292\,k$ @ 12' UNBRACED , $M_c = 128\ ^{k}$

$\frac{P_r}{P_c} = \frac{144}{292} = .49 > 0.2$ H1-1a

$\frac{P_r}{P_c} + \frac{8}{9}\frac{M_r}{M_c}$

$\frac{144}{292} + \frac{8}{9}\left(\frac{3.36}{128}\right) = .516 < 1.0$ ok ✓

VERY SMALL EFFECTS FROM P-Δ

AXIAL CONTROLLED GRAVITY COLUMN

Appendix E: Gravity Connections

Connection 1: Girder to Column Flange

- Beam Limit States:
  - Shear Yield
  - Shear Rupture
  - Coped Beam Flexure Strength
  - Block Shear
- Angle Limit States:
  - Shear Yield
  - Shear Rupture
  - Block Shear
  - Bolts
    - Shear
    - Bearing/Tear-out (angle and web)

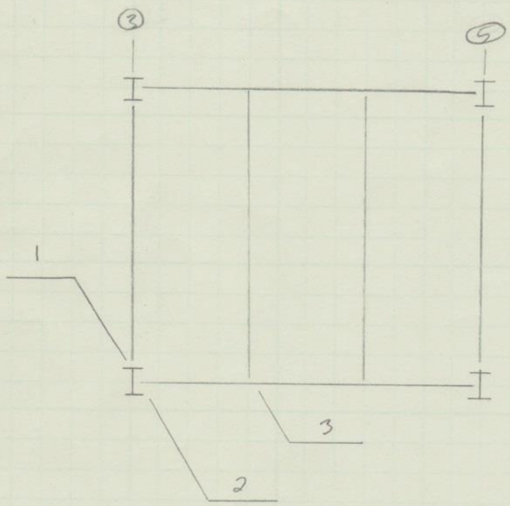
Connection 2: Girder to Column Web

- Plate Limit States:
  - Flexural Strength
  - Shear Yield
  - Shear Rupture
  - Block Shear
  - Combined Loading
  - Plate Buckling
  - Bolts
    - Shear
    - Bearing/Tear-out
- Weld Limit States:
  - Minimum Weld Size
  - Eccentric Weld Strength

Connection 1: Beam to Girder Web

- Bolt Limit States:
  - Shear
  - Bearing/Tear-out
- Angle Limit States:
  - Flexural Yield
  - Flexural Rupture
  - Shear Yield
  - Shear Rupture
  - Block Shear
- Weld Limit States:
  - Eccentric Weld Strength
  - Minimum Weld Size
- Beam Limit States:
  - Coped Beam Flexural Strength
  - Block Shear

DEPTH – GRAV. CONN. | AE 481 | JORDAN RUTHERFORD | 1



INFO:

ALL BEAMS – A992
ALL ANGLES/PLATES – A36
ALL BOLTS – 3/4"ø A325N

CONNECTION 1: GIRDER TO COLUMN FLANGE

THIS GIRDER SUPPORTS A BEAM ON ON SIDE AND A GLUMN ABOVE. TO SUPPORT THE LARGE SHEAR A DOUBLE ANGLE WILL BE USED.

COL.: W12×79
BM: W24×62
SHEAR: 189 k

TRY 7 ROWS OF BOLTS

BEAM LIMITS:

SHEAR YIELD: $\phi V_n = 1.0(0.6)(50)(21.7)(0.43) = 280\text{ k} > 189\text{ k}$ OK ✓

SHEAR RUPTURE: $\phi V_n = 0.75(0.6)(65)(21.7)(0.43) = 273\text{ k} > 189\text{ k}$ OK ✓

COPED BEAM FLEX: $c/d = 6/23.7 < 1$ $\quad f = 2(.253) = .506$

$c/h_o = 6/21.7 < 1$ $\quad k = 2.2(21.7/6)^{1.65} = 18.35$

$F_{BC} = 26210(.43/21.7)^2(.506)(18.35) = 96\text{ ksi} > 50\text{ ksi}$

$\phi M_n = 0.9(50)(48.3) = 2174\text{ "k}$

$\phi V_n = \dfrac{2174\text{ "k}}{6.5"} = 334\text{ k} > 189\text{ k}$ OK ✓

2

BLOCK SHEAR: TABLE 9-3a 64 k/in TENS. RUPT.
Len = 1.75" 9-3b 439 k/in SHEAR YIELD
Lev = 1.5" 9-3c 404 k/in SHEAR RUPT.

$\phi V_n = (64 + 404)(.59) = 276\ k > 189\ k$ ok ✓

ANGLE LIMITS:

SHEAR YIELD: $\phi V_n = 1.0(0.6)(36)(0.3125)(20.5)(2) = 277k > 189k$ OK ✓

SHEAR RUPTURE: $\phi V_n = 0.75(0.6)(58)(0.3125)(20.5)(2) = 334k > 189k$ OK ✓

BLOCK SHEAR: TABLE 9-3a 35.3
Len = 1.25" 9-3b 312
Lev = 1.25" 9-3c 354

$\phi V_n = (35.3 + 312)(5/16)(2) = 212\ k > 189\ k$ OK ✓
ON COL. SIDE 200 k

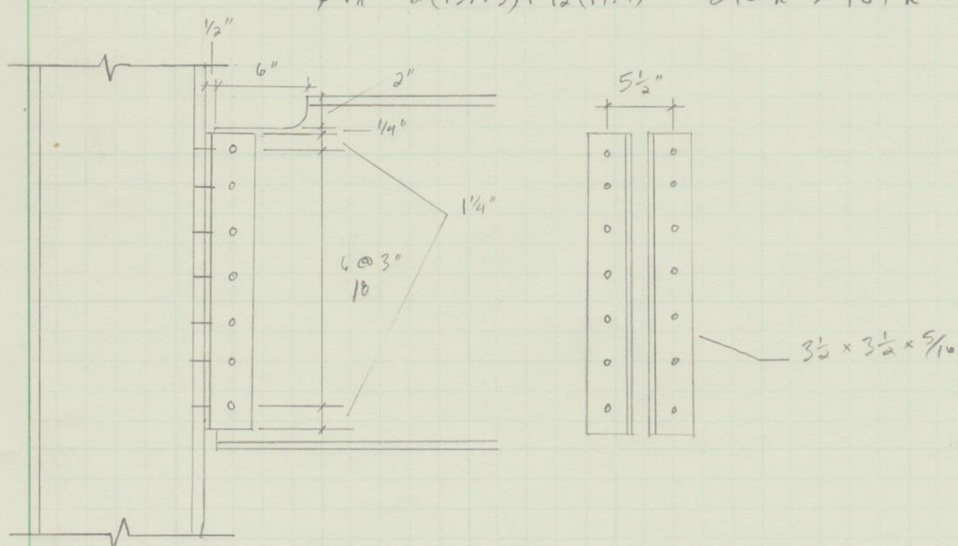
BOLT LIMITS:

SHEAR: 17.9 k

BEARING - ANGLE 78.3 (.3125) = 24.5
WEB 87.8 (.43) = 37.8

T/O - ANGLE 44 (.3125) = 13.75
WEB 49.4 (.43) = 21.2

$\phi V_n = 2(13.75) + 12(17.9) = 242\ k > 189\ k$ OK ✓



3

CONNECTION 2: GIRDER TO COLUMN WEB

THIS GIRDER FRAMES INTO A COL. WEB, THERE AN EXTENDED SHEAR TAB WILL BE USED.

COL.: W12 x 79
WEB: W18 x 40
SHEAR: 27 k

PLATE LENGTH = $\frac{12.4 - .47}{2}$ = 5.965" + 0.5" + 3" = 9.5"

ECCENTRICITY = 9.5" - 1.5" = 8"

BOLTS: $\frac{27k}{17.9k}$ = 1.508 → TRY 5 BOLTS

PLATE LIMITS:

MOMENT STR.: $M_{MAX} = (54/0.9)(.442)(17.1) = 453.5''^{k}$

$t_{MAX} = \frac{6(453.5)}{36(14.5)^2} = 0.36''$ TRY 1/4"

SHEAR YIELD: $\phi V_n = 1.0(0.6)(36)(.25)(14.5) = 78.3\,k > 27\,k$ OK ✓

SHEAR RUPTURE: $\phi V_n = 0.75(0.6)(58)(.25)(14.5 - 5(7/8)) = 66.1\,k > 27\,k$ OK ✓

BLOCK SHEAR: TABLE 9-3a 46.2
9-3b 215
9-3c 243

$\phi V_n = (46.2 + 215)(.25) = 65.3\,k > 27\,k$ OK ✓

INTERACTION: $V_r = 27\,k$
$V_c = 62.6\,k$
$M_r = 30k(8") = 240$
$M_c = 0.9(36)\frac{(.25)(14.5)^2}{4} = 425.76$

$$\left(\frac{30}{62.6}\right)^2 + \left(\frac{240}{425.76}\right)^2 = 0.547 < 1$$ OK ✓

PLATE BUCKLING: $\phi M_n = 0.9(50)\frac{(.25)(14.5)^2}{6} = \frac{394.2''^{k}}{8''} = 49.25\,k > 27\,k$ OK ✓

4

BUCKLING (CONT.):

$$\lambda = \frac{14.5\sqrt{36}}{10(.25)\sqrt{475+280\left(\frac{14.5}{8}\right)^2}} = 1.08$$

$Q = 1.34 - 0.486(1.08) = .815$

$F_{cr} = 0.815(50) = 40.75 \text{ ksi}$

$\phi M_n = .9(40.75)\frac{(.25)(14.5^2)}{6} = \frac{321^{\text{in-k}}}{8''} = 40\text{ k} > 27\text{ k}$ OK ✓

BOLT LIMITS:

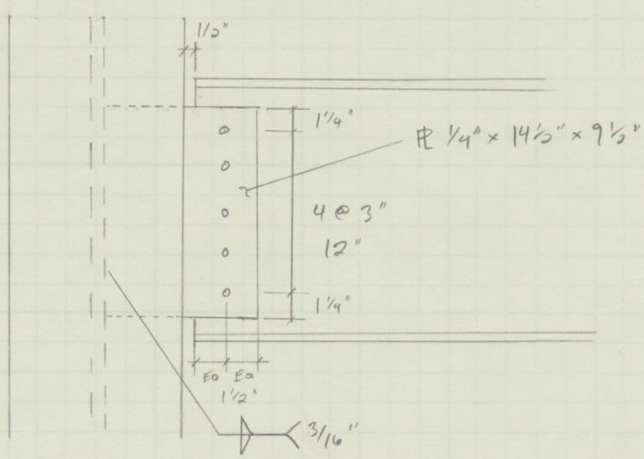
SHEAR = 17.9 k
BEARING = 78.3 (1/4) = 19.6 k
87.8 (.315) = 27.6 k

$\phi V_n = (17.9\text{k})(2.04) = 36.5\text{ k} > 27\text{ k}$ OK ✓

WELD LIMITS:

MIN WELD: $t_{min} = 1/4 - 1/16 = 3/16'' = 5/8(.25)$ OK ✓

$l = 14.5''$
$e = 7.75''$
$a = \frac{7.75''}{14.5''} = .534$
$k = 0$
$C = 2.198$ $\phi V_n = 0.75(2.198)(3)(14.5)(2) = 143.4\text{ k} > 27\text{ k}$ OK ✓



5

CONNECTION 3: BEAM TO GIRDER

A SIMPLE GRAVITY CONNECTION FOR A BEAM FRAMING INTO A GIRDER. A SINGLE ANGLE WILL BE USED.

BEAM: W16 × 26
GIRDER: W18 × 40
SHEAR: 25k

3/4" Ø BOLTS → $t = 3/8"$

$e = 3" + \frac{1}{2}(.315") = 3.158"$ → 5" × 3" × 3/8" ANGLE

BOLT LIMITS:

SHEAR = 17.9
BEARING = 78.3 (3/8) = 29.4
TEAROUT = 44 (3/8) = 16.5

REQ C = $\frac{25k}{16.5k}$ = 1.52 → TRY 3 BOLTS

ANGLE LIMITS:

FLEXURAL YIELD: $\phi V_n = 0.9(36)\frac{(3/8)(8.5^2)}{4}\left(\frac{1}{3.158}\right) = 69.5k > 25k$ OK ✓

FLEXURAL RUPTURE: $\phi V_n = \frac{0.75(58)(5.55)}{3.158} = 76.45k > 25k$ OK ✓

SHEAR YIELD: $\phi V_n = 1.0(0.6)(36)(8.5)(3/8) = 68.9k > 25k$ OK ✓

SHEAR RUPTURE: $\phi V_n = 0.75(0.6)(58)(8.5)(3/8) = 83.2k > 25k$ OK ✓

?

BLOCK SHEAR: $A_{gv} = (7.25)(3/8) = 2.72\ in^2$
$A_{nv} = (7.25 - 2.5(7/8))(3/8) = 1.9\ in^2$
$A_{nt} = (1.842 - 0.5(7/8))(3/8) = .527\ in^2$

$0.6(2.72)(36) = 58.75$
$0.6(1.9)(58) = 66.12$

$0.75(58.75 + (58)(.527)) = 70k > 25k$ OK ✓

6

WELD LIMITS:

	0.3	.353	0.4
0.3	2.15		2.36
.3209			
0.4	1.85		2.05

$kl = 3''$
$l = 8.5''$
$k = .353$
$x = .0467$
$xl = 0.0467(8.5) = .397$
$al = 3'' + \frac{1}{2}(.25'') - .397 = 2.728''$
$a = \frac{2.728}{8.5} = .3209$

$C = 2.197$

$D_{MIN} = \frac{25k}{0.75(2.197)(1.0)(8.5)} = 1.78$ USE 1/4" TO MEET MIN

BEAM LIMITS:

COPED BEAM FLEX: $c/d = 6/15.7 < 1$ $f = 2(3.82) = .764$
$c/h_o = 6/13.7 < 1$ $k = 2.2(13.7/6)^{1.65} = 8.6$

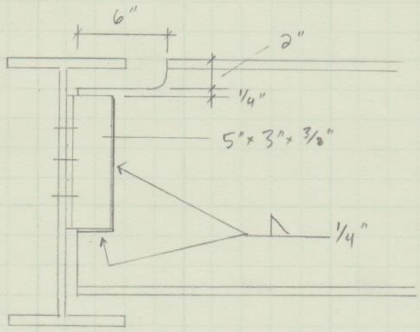
$F_{BC} = 26210(.25/13.7)^2(.764)(8.6) = 57 \text{ ksi} > 50 \text{ ksi}$

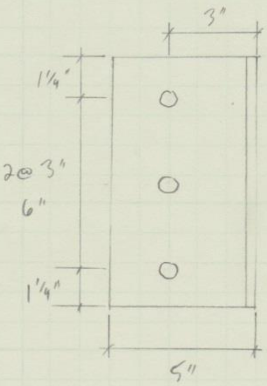
$\phi M_n = 0.9(50)(11.6) = 522 \text{ ''k}$

$\phi V_n = \frac{522 \text{ ''k}}{6.5''} = 80 k > 25 k$ OK ✓

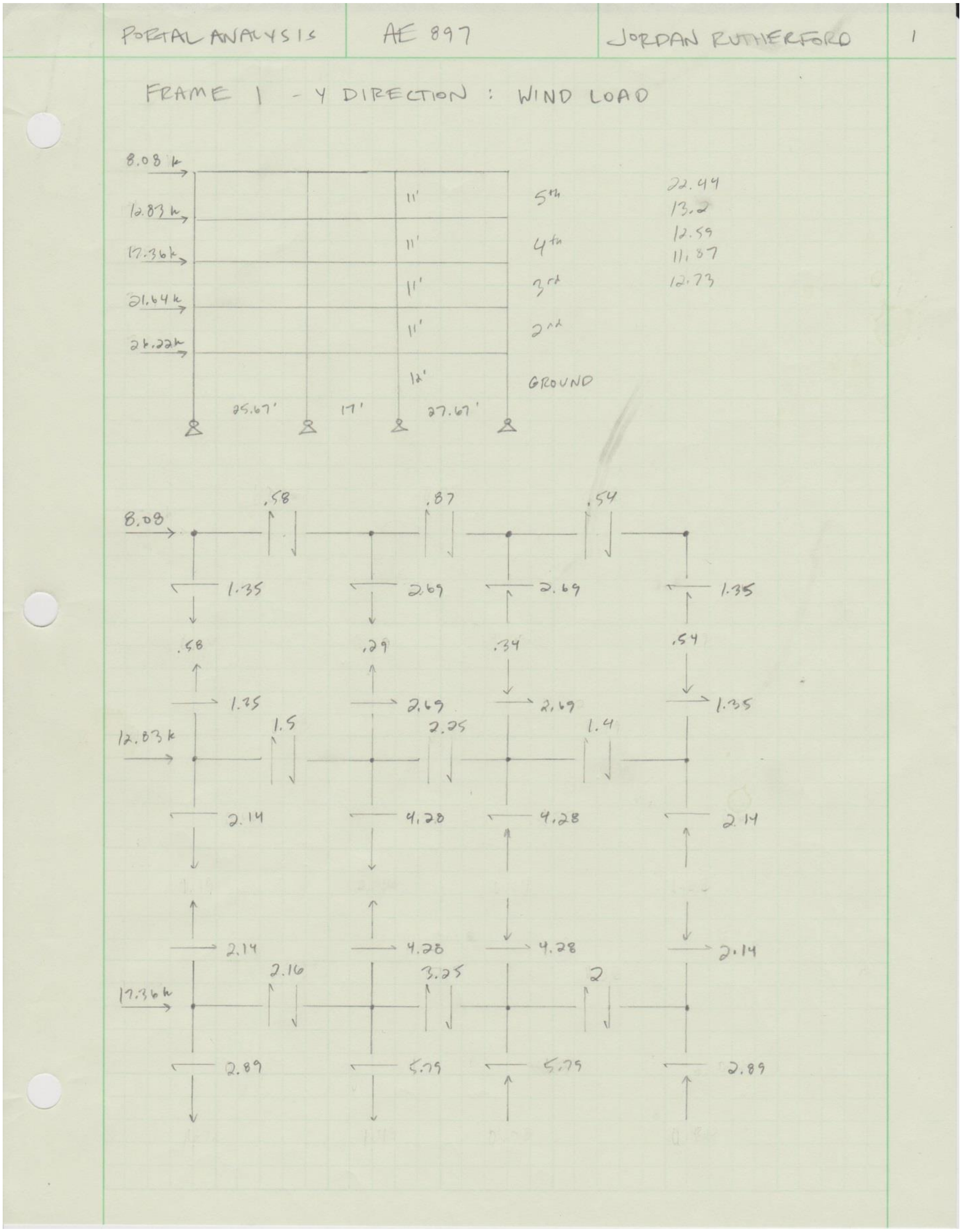
BLOCK SHEAR: $0.6(50)(8.75)(0.25) = 65.6 k$
$0.6(65)(8.75)(0.25) = 85.3 k$

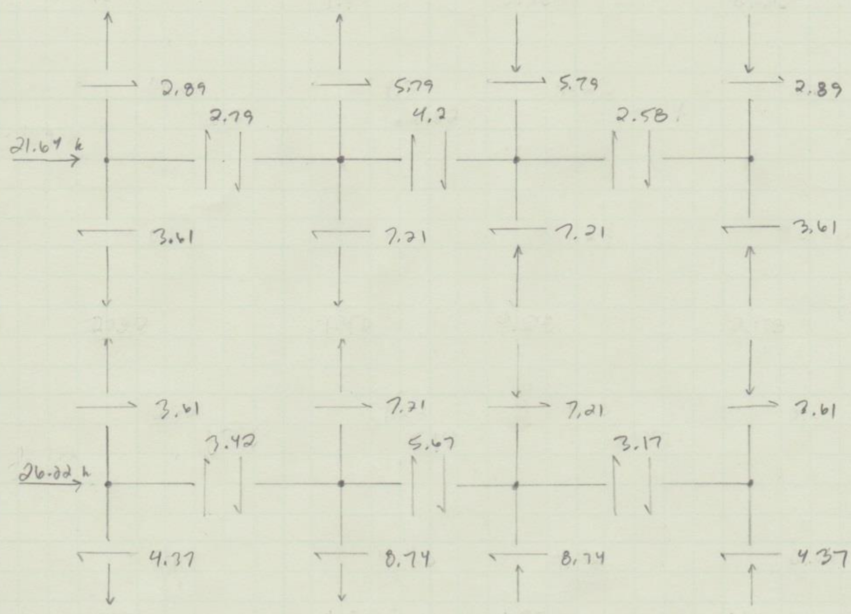
$\phi V_n = 0.75(65.6 + 1.0(65)(2.25)(0.25)) = 76.6k > 25k$ OK ✓





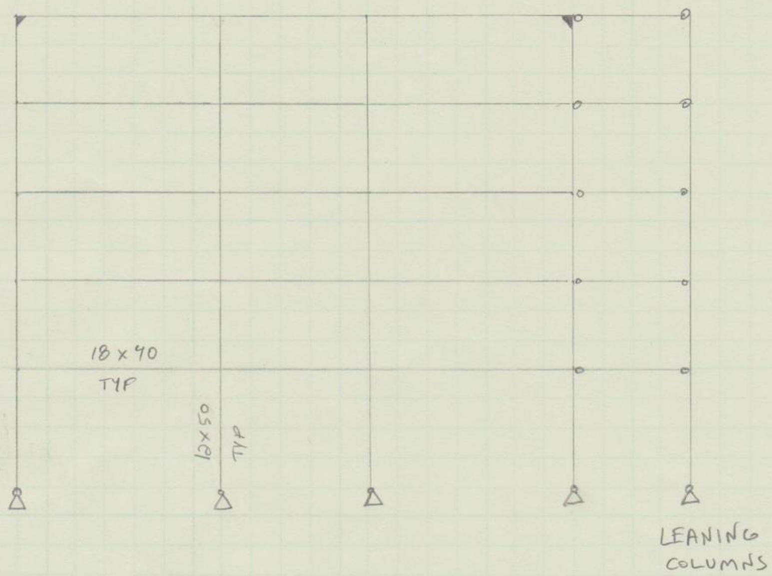
Appendix F: Lateral Design





BASE COLUMN INT: 8.74 (6) = 52.44	60.1	12.7%
2nd LEVEL INT COL: 7.21(5.5) = 39.7	41.44	4.2%
5th LEVEL EXT BM: .58 (25.67)(1/2) = 7.4	9.9	25 %
4th LEVEL EXT BM: 1.5(25.67)(1/2) = 19.25	19.32	> 5 %

DIRECT ANALYSIS METHOD   JORDAN RUTHERFORD   1



ANALYZE AN INT. COL. BY DIRECT ANALYSIS METHOD

ASSUME $B_2 < 1.7$, REDUCED STIFFNESS

CONTROLLING LOAD COMBO : $1.2D + 0.5L_p + 0.5S_p \pm 1.6W1$

LC 45   FIRST ORDER ANALYSIS w/ REDUCED STIFFNESS $P_u = 240$ k $M_u = 135$ ftk $\Delta_{1st} = 0.33$

TRIB AREA = $(97')(13.33') = 1293$ ft²

LOAD TO FRAME = $(63 \text{ psf})(1293 \text{ ft}^2) = 81.5$ k
$(200 \text{ plf})(97') = 19.4$ k
101 k DEAD

$(60 \text{ psf})(1293 \text{ ft}^2) = 77.6$ k
$(100 \text{ plf})(97') = 9.7$ k
87.3 k LIVE

$(55 \text{ psf})(1293 \text{ ft}^2) = 71$ k ROOF DEAD

$(40 \text{ psf})(1293 \text{ ft}^2) = 51.7$ k SNOW

LEANING COLUMN LOADS

FLOOR	DEAD	LIVE
5	995 - 71 = 924	306 - 51.7 k = 254 k
4	1158 - 101 = 1057	545 - 87.3 k = 458 k
3	1158 - 101 = "	545 - " = "
2	1158 - 101 = "	545 - " = "
1	1165 - 101 = "	545 - " = "

DETERMINE B_1

$P_r = 200\ k$

$P_{e1} = \dfrac{\pi^2 EI}{(KL)^2} = \dfrac{\pi^2(0.8)(29000\ ksi)(1380\ in^4)}{[(1.0)(12')(12)]^2} = 15238\ k$

$B_1 = \dfrac{C_m}{1 - \frac{\alpha P_r}{P_{e1}}} = \dfrac{1}{1 - \frac{240}{15238}} = 1.016$

DETERMINE B_2

$P_{STORY} = 7238\ k$

$R_m = 1 - 0.15 \dfrac{P_{MF}}{P_{STORY}} = 1 - \dfrac{(240 + 202 + 158 + 202)}{7238} = .98$

802

$P_{e\,STORY} = R_m \dfrac{HL}{\Delta} = \dfrac{.98(140\ k)(12')(12'')}{.33} = 54880$

$B_2 = \dfrac{1}{1 - \frac{7238}{54880}} = 1.15 < 1.5$ METHOD OK ✓
> 1.0

AMPLIFIED AXIAL LOAD:

$P_r = P_{nt} + B_2 P_{lt} = 240 + 1.15(0) = 240\ k$

$P_y = AF_y = 42.7(50) = 2135$

$\dfrac{\alpha P_r}{P_y} = \dfrac{1.0(240)}{2135} = .112 < .5 \quad \therefore \tau_b = 1.0$

$P_c = 1750\ k\ @\ 12'$

$\dfrac{P_r}{P_c} = \dfrac{240}{1750} = .137 < .2$ H1-1b

AMPLIFIED MOMENT:

$M_r = (1.0)(0) + (1.15)(135) = 155 \text{ ftk}$

$M_c = 975 \text{ ftk @ } 12'$

H1-1b: $\frac{1}{2}\frac{P_r}{P_c} + \frac{M_r}{M_c}$

$\frac{1}{2}\frac{240}{1750} + \frac{155}{975} = 0.0686 + .159 = .228 < 1.0$

IN ORDER TO MEET DRIFT REQUIREMENTS, COLUMN AND BEAM SIZES ARE SIGNIFICANTLY INCREASED IN MOMENT FRAMES. THE PINNED BASE CREATES A CRITICAL FIRST STORY DRIFT, THE STRENGTH REQUIREMENTS ARE EASILY MET.

FLEXURAL BUCKLING:

$G_{TOP} = \dfrac{\frac{1710}{12} + \frac{1710}{11}}{\frac{3610}{25.67} + \frac{3610}{17}} = .84$

$k = 1.9$

$\frac{kl}{r} = \frac{1.9(12')(12'')}{6.33''} = 43 < 113$

$F_e = \frac{\pi^2(29000)}{43^2} = 155$

$F_{cr} = [0.658^{50/155}]\, 50 = 43.7$

$P_n = 43.7(42.7) = 1865 \text{ k} >> 240 \text{ k}$ ok ✓

COMPACT CRITERIA: $.38\sqrt{\frac{29000}{50}} = 9.15 > 7.11 = \frac{15.5}{1.09(2)}$ ok ✓

$3.76\sqrt{\frac{29000}{50}} = 90.5 > 8.58 = \frac{14.8 - (1.09)(2)}{.68}$ ok ✓

FIRST FLOOR EXT BEAM:

$$C_b = \frac{12.5(1)}{2.5(1) + (.75)(3) + (1)(4) + (.75)(3)} = 1.14$$

$M_w = 68$
$M_D = 86.6$
$M_L = 91.6$

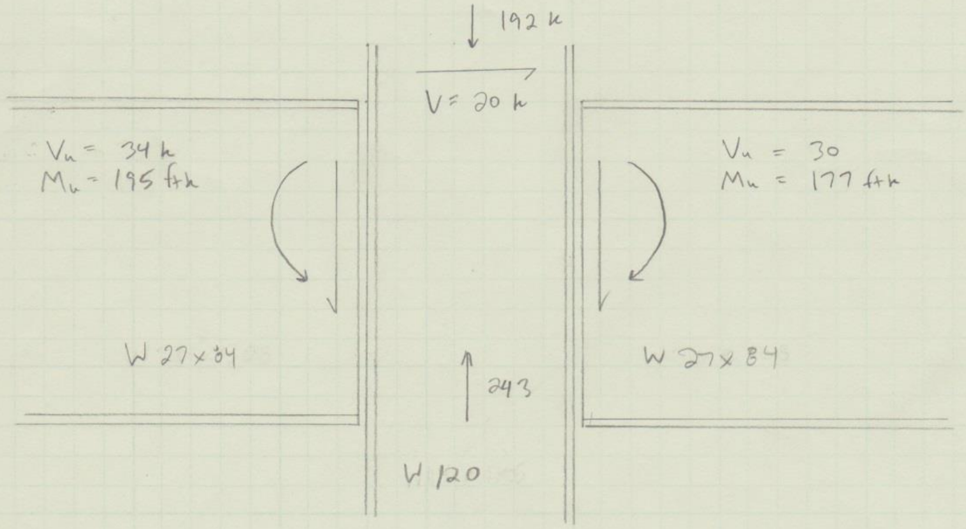
$M_u = 68(1.15) + 1.2(86.6) + 0.5(91.6) = 228$ ftk

$\phi M_n = \left| \begin{array}{l} 1060 \text{ ftk} \\ 1.14(980) \end{array} \right. \Rightarrow 1060$ ftk > 228 ftk OK ✓

BEAM IS GOOD FOR APPLIED MOMENTS. AS EXPECTED, THE LARGE SECTIONS REQUIRED FOR DRIFT CONTROL OVER STRENGTH.

MOMENT CONN.	AE 482	JORDAN RUTHERFORD	1

TYPICAL MOMENT CONNECTION



- USE A FLANGE BOLTED/WEB BOLTED CONNECTION
- BEAMS/COLS A992
- PLATES A36
- 3/4" ⌀ A325N BOLTS

• BEAM-TO-COLUMN FLANGE

$V_u = \frac{30 k}{17.9 k} = 1.7 \Rightarrow$ USE 3

$L = (2)(3") + 2(1.25") = 8.5"$

SHEAR RUPTURE: TRIAL THICKNESS $\frac{30 k}{.75(.6)(58)(8.5 - 3(.875))} = 0.195 \rightarrow$ USE 1/4"

SHEAR YIELD: $1.0(0.6)(36)(1/4)(8.5) = 45.9 k > 30 k$ OK

BLOCK SHEAR: TBL 9-3a = 35.3
b = 121
c = 139

$\phi V_n = \frac{1}{4}(35.3 + 121) = 39 k > 30 k$ OK

2

BOLT SHEAR: TABLE 7-1 = 17.9 k
BEARING: TABLE 7-4 = 78.3 (1/4) = 19.6 k
TEAR-OUT: TABLE 7-5 = 44 (1/4) = 11 k

$\phi V_n = 11k + 2(17.9k) = 46.8k > 30k$ OK✓

WELD STRENGTH: MIN WELD = 3/16"

$\phi V_n = 1.392(2)(3)(8.5) = 71k > 30k$ OK✓

- PLATE (WELDED TO COL. FLANGE / BOLTED TO BEAM FLANGE)

TENSION PLATE:

$F_u = \frac{195\ kft\ (12)}{26.7} = 87.6k$ TRY 1/2"

BOLT SHEAR = 17.9 k

PLATE BEARING = 78.3
T.O. = 44 $\left(\frac{1}{2}\right) = 22$

FLANGE BEARING = 87.8
T.O. = 49.4 $(.64) = 31.6$

$\#\ OF\ BOLTS = \frac{78.5k}{17.9k} = 4.4 \rightarrow$ USE 6

$L = 2(3") + 1.25" + 2" = 9.25"$

$W = 5.5" + 1.25"(2) = 8"$

TENSION YIELD: $F_u = \frac{195\ kft\ (12)}{26.7 + .5} = 86k$

$\phi F_n = 0.9(36)(8)(0.75) = 145k > 86k$ OK✓

TENSION RUPTURE: $A_n = 0.5"(8" - 2(.875")) = 3.125\ in^2$

$A_e = \min \begin{cases} 0.85(8")(1/2") = 3.13\ in^2 \\ 3.125\ in^2 \end{cases}$

$\phi F_n = 0.9(58)(3.13) = 163k > 86k$ OK✓

3

BLOCK SHEAR: (PLATE)
TBL 9-3a : 35.3
9-3b : 170
9-3c : 194

$\phi R_n = (35.3 + 170)(\frac{1}{2}) = 102\text{ k} > 87.6\text{ k}$ ok ✓

(FLANGE) : $A_{nt} = (10.5 - 8)(.5) + 1.25 = 2.5" > 1.25"$ FOR PLATE
$A_{gv} = 2(3") + (1.25 - .5) = 6.75$
$A_{nv} = 6.75 - 2.5(7/8) = 4.56$

$0.6 F_y A_{gv} = 0.6(50)(6.75) = 203$
$0.6 F_u A_{nv} = 0.6(65)(4.56) = 178$

$\phi R_n = 0.75(178 + 1.0(58)(2.5)) = 242\text{ k} > 87.6\text{ k}$ ok ✓

BEARING: $\phi R_n = 0.75(1.2)(1.0 - \frac{1}{2}(.8125)(58)(1.64) = (11.4)(2) + (17.9) = 95\text{ k} > 87.6\text{ k}$ ok ✓

WELD RUPTURE $D = \dfrac{77\text{ k}}{2(1.5)(1.392)(8)} = 2.3 \rightarrow 5/16"$
USE 5/16 MAX FOR 1/2" ok ✓

COL FLANGE $t_{MIN} = \dfrac{3.09(5)}{65} = .237" < 1.09"$ ok ✓

COMPRESSION FLANGE:

TRY 1/2" x 9.25

FLEXURAL BUCKLING: $k = 0.65$
$l = 1.5" + 1" = 2"$
$r = 0.289(1/2") = 0.1445$

$\dfrac{kl}{r} = \dfrac{0.65(2)}{.1445} = 9 < 25 \therefore F_{cr} = F_y$

$\phi F_{cr} A_g = 0.9(36)(8")(1/2") = 130\text{ k} > 108\text{ k}$ ok ✓

LOCAL BUCKLING: $\dfrac{b_f}{t_p} = \dfrac{10.5}{.5} = 21 \leq 42 = \dfrac{253}{\sqrt{36}}$ ok ✓

$\dfrac{b}{t_p} = \dfrac{1.25}{0.5} = 2.5 \leq 15.8 = \dfrac{95}{\sqrt{36}}$ ok ✓

4

- BEAM FLEXURAL STRENGTH:

$A_{fg} = (10)(.64) = 6.4 \text{ in}^2$

$A_{fn} = 6.4 - 2(7/8)(.64) = 5.28 \text{ in}^2$

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

$F_u A_{fn} = (65)(5.28) = 343 \text{ k} < 320 \text{ k} = 1.0(50)(6.4) = Y_t F_y A_{fg}$

BENDING CAPACITY NOT REDUCED ✓

- COLUMN LIMIT STATES

$T_u = C_u = \frac{194(12)}{26.7 - .64} = 89.7 \text{ k}$

FLANGE BENDING: $\phi R_n = 0.9(14.8)(1.09^2)(50) = 791 \text{ k} > 89.7 \text{ k}$ OK ✓

WEB YIELDING: $\phi R_n = 1.0[5(1.69) + 1.09](50)(.68) = 324 > 89.7 \text{ k}$ OK ✓

WEB CRIPPLING:

$$\phi R_n = 0.75(0.8)(0.68^2)\left[1 + 3\left(\frac{1.09}{14.8}\right)\left(\frac{.68}{1.09}\right)^{1.5}\right]\sqrt{\frac{29000(50)(1.09)}{.68}}$$

$\phi R_n = 469 \text{ k} > 80 \text{ k}$ 1524.5 OK ✓

NO STIFFENERS REQUIRED

PANEL ZONE SHEAR:

$0.4 P_y = 0.4(50)(42.7) = 854 \text{ k} > 240 \text{ k}$

$R_v = 0.6 F_y d_c t_{wc} = 0.6(50)(14.8)(.68) = 302 \text{ k}$

$V = \frac{195(12)}{26.7 - .64} + \frac{177(12)}{26.7 - .64} - 20 = 1171 \text{ k} < 302 \text{ k}$

NO DOUBLER PLATE REQUIRED

STORY	FRAME #	Horiz. L	Height	Length	Pu	Pn	SIZE	Tu	Tn	SIZE
5	1	9.94	11	14.82	26.67	112	3/16	15.17	130	3/16
	2	9.94	11	14.82	29.37	112	3/16	17.29	130	3/16
	3	13.34	11	17.29	47.22	112	3/16	22.2	130	3/16
	4	13.34	11	17.29	35.27	112	3/16	17.34	130	3/16
	5	9.94	11	14.82	25.64	112	3/16	14.22	130	3/16
	6	9.94	11	14.82	25.93	112	3/16	14.56	130	3/16
4	1	9.94	11	14.82	39.51	112	3/16	26.23	130	3/16
	2	9.94	11	14.82	41.84	112	3/16	27.29	130	3/16
	3	13.34	11	17.29	60.3	112	3/16	28.16	130	3/16
	4	13.34	11	17.29	48.52	112	3/16	26.21	130	3/16
	5	9.94	11	14.82	39.12	112	3/16	26.43	130	3/16
	6	9.94	11	14.82	39.41	112	3/16	26.24	130	3/16
3	1	9.94	11	14.82	51.64	112	3/16	29.14	130	3/16
	2	9.94	11	14.82	53.44	112	3/16	39.01	130	3/16
	3	13.34	11	17.29	67.99	112	3/16	36.41	130	3/16
	4	13.34	11	17.29	57.98	112	3/16	35.71	130	3/16
	5	9.94	11	14.82	51.93	112	3/16	39.02	130	3/16
	6	9.94	11	14.82	52.09	112	3/16	38.75	130	3/16
2	1	9.94	11	14.82	63.29	112	3/16	49.69	130	3/16
	2	9.94	11	14.82	64.37	112	3/16	50.35	130	3/16
	3	13.34	11	17.29	73.43	112	3/16	42.72	130	3/16
	4	13.34	11	17.29	65.87	112	3/16	43.71	130	3/16
	5	9.94	11	14.82	64.36	112	3/16	51.4	130	3/16
	6	9.94	11	14.82	63.5	112	3/16	51.09	130	3/16
1	1	9.94	12	15.58	78.59	112	3/16	65.26	130	3/16
	2	9.94	12	15.58	78.75	112	3/16	65.71	130	3/16
	3	13.34	12	17.94	75.57	112	3/16	52.97	130	3/16
	4	13.34	12	17.94	75.52	112	3/16	54.51	130	3/16
	5	9.94	12	15.58	80.89	112	3/16	67.81	130	3/16
	6	9.94	12	15.58	80.69	112	3/16	67.35	130	3/16

	15	16	18	
3/16	112	106	94.2	130
1/4	146	138	122	171
5/16	177	167	148	210
3/8	206	195	172	248
1/2	260	245	215	318
5/8	305	286	249	382

X Direction Framing

Frame	Displacement	Stiffness	% of Load
1	0.032	31.3	38
2	0.08	12.5	15
3	0.08	12.5	15
4	0.04	25.0	31
	Totals	81.3	100

Y Direction Framing

Frame	Displacement	Stiffness	% of Load
1	0.281	3.6	24
2	0.258	3.9	26
3	0.258	3.9	26
4	0.281	3.6	24
	Totals	14.9	100

BRACE FRAME | JORDAN RUTHERFORD

BRACE: MAX COMP. = 78.11 k
TENS. = 54.8 k

SELECTED SIZE: 6 × 6 × 3/16 (FOR SLENDERNESS)
TENSION YIELD = 165 k
RUPTURE = 130 k > 54.8 k OK

COMPRESSION = 94.2 k @ 18' L_b > 78 k OK

COLUMN: MAX COMP. = 384 k
TENS. = 85 k

SELECTED SIZE W10 × 49
TENSION YIELD = 648 k
RUPTURE = 527 k

COMPRESSION = 512 k

$$\lambda_f = \frac{10}{2(.56)} = 8.9 < 9.15 = .38\sqrt{\frac{29000}{50}}$$

$$\lambda_w = \frac{10}{.34} = 29 < 90.5 = 3.76\sqrt{\frac{29000}{50}}$$

BEAM: MAX COMP. = 47 k
TENS. = 47 k
MOMENT = 52 ft k

SELECTED SIZE W18 × 50
TENSION YIELD = 662
RUPTURE = 536 > 47

$$\text{COMPRESSION} = F_e = \frac{\pi^2(29000)(800)}{[(26.47)(12)/(7.78)]^2} = 152 > 113$$

$$F_{cr} = .877 F_e = 133$$

$$\phi P_n = 0.9(133)(14.7) = 1760\text{ k} >> 47\text{ k}$$ OK

MOMENT: $\phi M_n = 379 > 52$ OK

BRACE CONNECTION 1

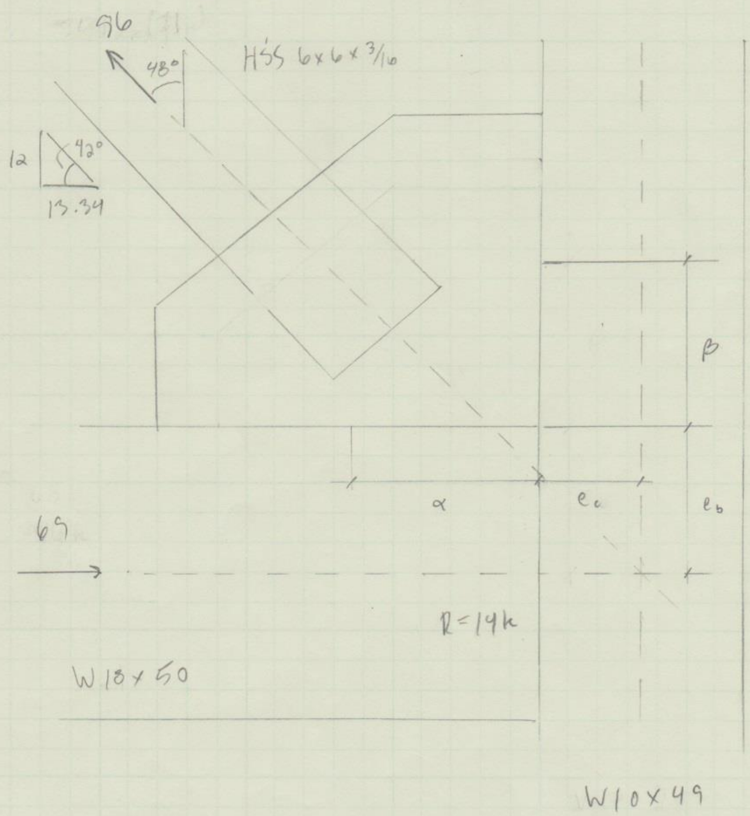
BRACE DESIGN:

SMALLEST SIZE: 6 × 6 × 1/8

NONSLENDER COMPACT CRITERIA: FOR 1/8" t, 4" w < 6" OK ✓
3/16" t, 6" w ≤ 6" OK ✓

REFER TO TABLE FOR MAX LOADS + SIZE SELECTION

BRACED CORNER CONNECTION:



BRACE: ASTM A500 GR B
$A = 3.98 \text{ in}^2$
$d = 6''$
$t = .174''$

$e_b = 18/2 = 9$
$e_c = 10/2 = 5$

2

BRACE LIMITS

TENSION YIELD: $0.9(46)(3.98) = 165\ k > 50\ k$

TENSION RUPTURE: $\bar{x} = \frac{6^2 + 2(6)(6)}{4(6+6)} = 2.25$

$U = 1 - \frac{\bar{x}}{L} = 1 - \frac{2.25}{6} = 0.625$

$A_n = A_g - 2(t_p + 1/16)t = 3.98 - (2)(3/8 + 1/16)(.174) = 3.83\ in^2$

$A_e = 3.83(0.625) = 2.4\ in^2$

$\phi P_n = \phi F_u A_e = 0.9(58)(2.4) = 125\ k > 50\ k$

PLATE LIMITS

WHITMORE SECTION: ASSUMING $l = 6"$

$6 \tan 30 = 3.46"$

$w = 3.46(2) + 6' = 12.9"$

TENSION YIELD: $P_n = F_y A_g$

$50\ k = .9(36)(12.9)\ t \Rightarrow .12 \therefore$ USE $3/8"$

WELD LIMITS

$t_{HSS} = .174$, MIN WELD $t = 1/8"$, TRY $3/16$

$\phi R_n = 1.392(3)(6)(4) = 100\ k > 71\ k$

BASE GUSSET: $\phi R_n = 0.75(0.6)(58)(.375)(6)\ 2 = 117\ k > 71\ k$
BRACE: $\phi R_n = 0.75(0.6)(58)(.174)(6)\ 4 = 109\ k > 71\ k$

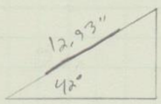
BLOCK SHEAR: $A_{nv} = 2(6)(.375) = 4.5\ in^2$
$A_{nt} = (6)(.375) = 2.25\ in^2$

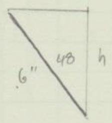
$\phi R_n = 0.75[0.6(36)(4.5) + 1.0(58)(2.25)] = 171\ k > 71\ k$

3

SIZE GUSSET:

MIN h ALONG COL.: $\sin 42 = \frac{x}{12.93}$ $x = 8.7''$





$\cos 48 = \frac{h}{6}$ $h = 4''$

TRY 15"

UNIFORM FORCE METHOD TO PREVENT MOMENT FROM DEVELOPING AT ANY INTERFACE

$\alpha - \beta \tan \theta = e_b \tan \theta - e_c$

$\left[L/2 + 1/2\right] - \underset{(1.11)}{7.5 \tan 48} = \underset{(1.11)}{9 \tan 48} - 5$

$L_1 = 25.63$

$PL = 25.63 \times 15 \times 3/8$

LOCAL BUCKLING: $\lambda = \frac{0.5(20)}{\left(\frac{.375}{\sqrt{12}}\right)\pi} \sqrt{\frac{36}{29000}} = 1.04 < 1.5$

$F_{CR} = 0.658^{(1.04)^2}(36) = 22.98$

$\phi P_n = 0.9(0.375)(12.93)(22.98) = 100\text{ k} > 44\text{ k}$

4

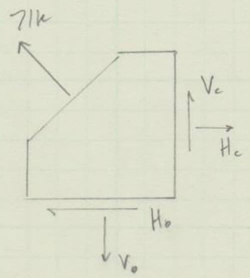
FORCES:

$r = \sqrt{(\alpha+e_c)^2 + (e_b+\beta)^2} = \sqrt{(14.75+6.35)^2 + (9+10)^2} = 28.4$

BEAM - GUSSET:

$H_b = \frac{\alpha}{r} P = \frac{14.65}{28.4}(96\ k) = 49.5\ k$

$V_b = \frac{e_b}{r} P = \frac{9}{28.4}(96\ k) = 30.4\ k$



COL - GUSSET:

$H_c = \frac{e_c}{r} P = \frac{6.35}{28.4}(96\ k) = 21.5\ k$

$V_c = \frac{\beta}{r} P = \frac{10}{28.4}(96\ k) = 33.8\ k$

BEAM - COLUMN

$\Sigma F_x = 46\ k - \underbrace{49.5\ k}_{H_b} + \underbrace{21.5\ k}_{H_c} = 18\ k = H_{bc}$

$\Sigma F_y = 14\ k + 30.4\ k = 44\ k = V_{bc}$

CONNECTION: A325N 3/4" ϕ BOLTS $\phi F_{nt} = 67.5$
TRY 3 $\phi F_{nv} = 40.5$

SHEAR STRESS: $f_v = \frac{44\ k}{6(.442)} = 17.4$ ksi

AVAILABLE TENS. STR. $F_{nt}' = 1.3F_{nt} - \left(\frac{F_{nt}}{\phi F_{nv}}\right) f_v$

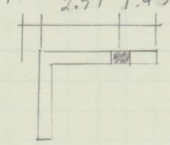
$F_{nt}' = 1.3(90) - \left(\frac{90}{40.5}\right)17.4 = 78.3 > 90$ OK ✓

$\phi r_{nt} = 0.75(78.3)(.442) = 26$

CALCULATE $r_{ut} = \frac{49.5}{6} = 8.25 < 26$ OK ✓

5

DETERMINE IF THERE IS PRYING: TRY 4 x 3 x .5



$L = 2(3) + 2(1.5) = 9$

$p = \left|\begin{matrix}5.5 \\ 9/3\end{matrix}\right. = 3$

$b = 2.57 - \frac{1}{2}(\frac{1}{4}) = 2.32$

$b' = 2.445 - \frac{1}{2}(3/4) = 1.945$

$r_{ut} b' = 8.25(1.945) = 16$

$\phi M_{n_1} = 0.9(58)(3)(.5^2)/4 = 9.78 < 16$ PRY

$\phi M_{n_2} = 9.78(1 - \frac{7/8}{3}) = 6.93 + 9.78 = \frac{16.7}{1.945''} = 8.6 > 8.25$ OK

ANGLE LIMITS:

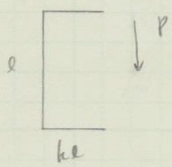
SHEAR YIELD: $\phi V_n = 1.0(0.6)(36)(9)(.5) = 97.2$ k

SHEAR RUPTURE: $\phi V_n = .75(0.6)(58)(9 - 3(7/8))(.5) = 83$ k

BEARING/T.O: $\phi R_n = 78.3(.5) = 39$ k
$44(.5) = 22$ k

BLOCK SHEAR: $9 - 3a = 68$ $\phi R_n = (68 + 121)(.5) = 94.5 >$
$b = 121$
$c = 139$

ECCENTRIC WELD:



$l = 9''$
$kl = 2.5''$
$k = 0.2778$

TABLE 8-8

	k		
	0.2	0.2778	0.3
x	.029	$\boxed{.05}$	.056

$xl = 0.05(9) = .45$
$al = 3'' - .45'' = 2.55''$
$a = \frac{2.55}{9} = .283$

C = 2.75 FROM BILINEAR INTERPOLATION

$D_{MIN} = \frac{11.5}{.75(2.75)(1)(9)} = .6 \Rightarrow$ USE 3/16 WELD

GUSSET-TO-BEAM

$D_{MIN} = \frac{10\text{ k}}{1.392(25'')(2)} = .144 \Rightarrow$ USE 3/16 WELD

6

GUSSET - COLUMN:

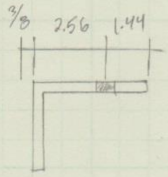
SHEAR STRESS: $f_v = \frac{33.8k}{6(.442)} = 12.7$

AVAILABLE TENS. STR. $F_{nt}' = 1.3(90) - \left(\frac{90}{40.5}\right)12.7 = 88.6 < 90$

$\phi r_{nt} = 0.75(88.6)(.442) = 29.4$

$r_{ut} = \frac{21.5}{6} = 3.6 < 29.4$

DETERMINE IF THERE IS PRYING:



$L = 2(3) + 2(1.5) = 9$

$p = 3"$

$b = 2.56 - \frac{1}{2}(.5) = 2.31$

$b' = 2.31 - \frac{1}{2}(.75) = 1.935$

$r_{ut}b' = 2.65(1.935) = 5.13$

$\phi M_n = 0.9(58)(3)(.5^2)/4 = 9.78 > 5.13$

ANGLE LIMITS:

SHEAR YIELD: $\phi V_n = 1.0(0.6)(36)(9)(.5) = 97.2$

RUPTURE: $\phi V_n = 0.75(0.6)(58)(9 - 3(7/8))(.5) = 83.2$

BEARING/T.O.: $\phi R_n = 78.3\,(.5) = 39$

$44\,(.5) = 22$

BLOCK SHEAR: 9-3 a = 40.8

b = 121

c = 137

$\phi R_n = (40.8 + 121)(.5) = 80.9$

ECCENTRIC WELD:

$l = 9"$

$kl = 2.5"$

$k = 0.2778$

$xl = 0.05(9) = .45$

$al = 3" - .45" = 2.55"$

$a = \frac{2.55}{9} = .283$

$C = 2.75 \Rightarrow D_{min} = \frac{25}{.75(2.75)(1)(9)} = 1.3 \Rightarrow$ USE 3/16"

Appendix G: Enclosure

Thermal Gradient with Studs

Layer Material	Conductivity (BTU*in/°F*ft2*h)	Thickness (in)	Conductance (BTU/°F*ft²*h)	Resistance (°F*ft2*h/BTU)	Temperature Change (°F)	Temperature (°F)	Temperature (°F)	Permiability (ng/Pa*s*m)	Permeance (ng/Pa*s*m²)	Resistance (Pa*s*m²/ng)	$P_{w,sat}$	ΔP_w	P_w	Relative Humidity (%)
	k	t	C	Rt	ΔT	T	T	μ	M	Rv				RH
			k/t	1/C		R/ΣR*(ΔT)	R/ΣR*(ΔT)		μ/t	1/M			RH*Pw,sat	
Interior temp						68	293.2				2348		939	40
Interior Film	N.A.	N.A.	N.A.	0.680	1.58				15000	6.67E-05		0.33		
						66.42	292.3				2223		939	42
Drywall	0.16	0.625	1.78	0.562	1.31			40	64	0.0156		76.37		
						65.11	291.5				2125		863	41
Studs @ 16"O.C.	N.A.	6.000	N.A.	1.179	2.75			0	0	0.0000		0.00		
						62.36	290.0				1930		863	45
Ext. Sheathing	0.035	0.500	1.60	0.625	1.46			3.5	7	0.1429		698.25		
						60.91	289.2				1833		164	9
Air Space	N.A.	1.000	N.A.	1.000	2.33				7200	0.0001		0.68		
						58.58	287.9				1687		164	10
Metal Panel	N.A.	3.000	N.A.	23	53.58			0	0	0		0.00		
						5.00	258.2				192		164	85
Exterior temp						5	258.2				192		164	85
		R Total		27.0					R Total	0.1587				
		Overall Coef. Of Heat		0.037										

Thermal Gradient with Batt Insulation

Layer Material	Conductivity	Thickness	Conductance	Resistance	Temperature Change	Temperature (°F)	Temperature (°F)	Permiability	Permeance	Resistance	$P_{w,sat}$	ΔP_w	P_w	Relative Humidity (%)
	k	t	C	Rt	ΔT	T	T	μ	M	Rv				RH
			k/t	1/C		R/ΣR*(ΔT)	R/ΣR*(ΔT)		μ/t	1/M			RH*Pw,sat	
Interior temp						68	293.2				28327		11331	40
Interior Film	N.A.	N.A.	8.00	0.680	0.89				15000	6.67E-05		3.38		
						67.11	292.7				27243		11327	42
Drywall	N.A.	0.625	1.78	0.562	0.74			40	64	0.0156		793.14		
						66.37	292.2				26374		10534	40
Batt Insulation	1.8	6.000	0.05	22.222	29.11			120	20	0.0500		2538.04		
						37.26	276.1				6333		7996	126
Ext. Sheathing	0.035	0.500	1.60	0.625	0.82			3.5	7	0.1429		7251.55		
						36.44	275.6				6057		745	12
Air Space	N.A.	1.000	N.A.	1.000	1.31				7200	0.0001		7.05		
						35.13	274.9				5636		738	13
Metal Panel	N.A.	3.000	N.A.	23	30.13			0	0	0		0.00		
						5.00	258.2				868		738	85
Exterior temp						5	258.2				868		738	85
		R Total		48.1					R Total	0.2087				
		Overall Coef. Of Heat		0.021										

Thermal Gradient with Average R value (25%)

Layer Material	Conductivity	Thickness	Conductance	Resistance	Temperature Change	Temperature (°F)	Temperature (°F)	Permiability	Permeance	Resistance	$P_{w,sat}$	ΔP_w	P_w	Relative Humidity (%)
	k	t	C	Rt	ΔT	T	T	μ	M	Rv				RH
			k/t	1/C		R/ΣR*(ΔT)	R/ΣR*(ΔT)		μ/t	1/M			RH*Pw,sat	
Interior temp						68	293.2				28327		11331	40
Interior Film	N.A.	N.A.	8.00	0.680	1.43				15000	6.67E-05		3.38		
						66.57	292.4				26603		11327	43
Drywall	N.A.	0.625	1.78	0.562	1.18			40	64	0.0156		793.14		
						65.39	291.7				25247		10534	42
Stud/Batt	1.8	6.000	0.09	4.067	8.56			120	20	0.0500		2538.04		
						56.83	286.9				17060		7996	47
Ext. Sheathing	0.035	0.500	1.60	0.625	1.32			3.5	7	0.1429		7251.55		
						55.51	286.2				16028		745	5
Air Space	N.A.	1.000	N.A.	1.000	2.10				7200	0.0001		7.05		
						53.41	285.0				14489		738	5
Metal Panel	N.A.	3.000	N.A.	23	48.41			0	0	0		0.00		
						5.00	258.2				868		738	85
Exterior temp						5	258.2				868		738	85
		R Total		29.9					R Total	0.2087				
		Overall Coef. Of Heat		0.033										

Thermal Gradient with Average R value (10%)

Layer Material	Conductivity	Thickness	Conductance	Resistance	Temperature Change	Temperature (°F)	Temperature (°F)	Permiability	Permeance	Resistance	$P_{w,sat}$	ΔP_w	P_w	Relative Humidity (%)
	k	t	C	Rt	ΔT	T	T	μ	M	Rv				RH
			k/t	1/C		R/ΣR*(ΔT)	R/ΣR*(ΔT)		μ/t	1/M			RH*Pw,sat	
Interior temp						68	293.2				28327		11331	40
Interior Film	N.A.	N.A.	8.00	0.680	1.27				15000	6.67E-05		3.38		
						66.73	292.4				26794		11327	42
Drywall	N.A.	0.625	1.78	0.562	1.05			40	64	0.0156		793.14		
						65.68	291.9				25581		10534	41
Stud/Batt	1.8	6.000	0.09	7.899	14.74			120	20	0.0500		2538.04		
						50.95	283.7				12851		7996	62
Ext. Sheathing	0.035	0.500	1.60	0.625	1.17			3.5	7	0.1429		7251.55		
						49.78	283.0				12132		745	6
Air Space	N.A.	1.000	N.A.	1.000	1.87				7200	0.0001		7.05		
						47.91	282.0				11052		738	7
Metal Panel	N.A.	3.000	N.A.	23	42.91			0	0	0		0.00		
						5.00	258.2				868		738	85
Exterior temp						5	258.2				868		738	85
		R Total		33.8					R Total	0.2087				
		Overall Coef. Of Heat		0.030										

Isothermal Planes Method

$$R_{T(avg)} = 25.867 + \frac{1}{0.03375 + 0.21212} = 29.934 \ \text{°F*ft}^2\text{*h/BTU}$$

$$R_{T(avg)} = \frac{1}{0.03375 + 0.21212} = 4.06717$$

$$U_{(avg)} = 0.03341 \ \text{BTU/°F*ft}^2\text{*h}$$

Isothermal Planes Method

$$R_{T(avg)} = 25.867 + \frac{1}{0.04043 + 0.08617} = 33.7654 \ \text{°F*ft}^2\text{*h/BTU}$$

$$R_{T(avg)} = \frac{1}{0.04043 + 0.08617} = 7.89865$$

$$U_{(avg)} = 0.02962 \ \text{BTU/°F*ft}^2\text{*h}$$

0.10156