Final Report

Seneca Allegany Casino Hotel Addition

Salamanca, NY

Nicholas Reed

Structural Option

Advisor: Prof. Parfitt

April 3rd, 2013

Seneca Allegany Casino - Hotel Addition

Salamanca, NY

General Information

Function:	Casino/Hotel
Height:	11 Stories, 153 Ft.
Size:	165,000 Sq. Ft.
Construction:	Late 2007 - Fall 2012
Construction Cost:	$40 Million

Structure

Foundation:
Concrete strip and spread footings on steel piles driven to bedrock

Superstructure:
Composite metal deck on steel frame; diagonal bracing in N-S for wind; moment connections in E-W for seismic

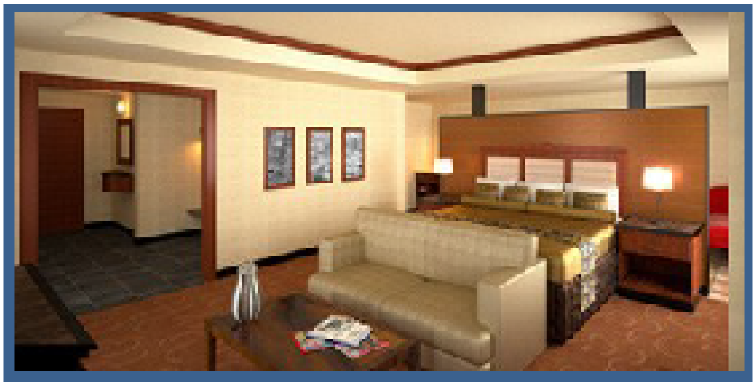
Project Team

Owner:	Seneca Nation of Indians
Architect:	JCJ Architecture
Structural/Civil:	Wendel
MEP:	M/E Engineering P.C.
CM:	Seneca CM Corporation

MEP

Mechanical/Plumbing:
3rd Floor - 2 VAV AHU's 12,000 cfm, boilers and water pumps
Roof - 1 VAV AHU 14,000 cfm

Lighting/Electrical:
480/277V 3 phase, 4 wire - Equipment
208/120V 3 phase, 4 wire - Data/Security
120/277V CFL, LED, Fluorescent



Hotel Room Rendering

Architecture

The SAC Hotel addition ties into an existing 11-story hotel tower, reproducing the precedent insulated glass façade. This allows for plenty of natural day-lighting. The lower 3 levels consist of insulated metal panels backed by metal framing studs.

Nicholas Reed Structural Option

Executive Summary

The Seneca Allegany Casino Hotel Addition is a 153 foot tall, 11 story hotel located within the Seneca Nation of Indians reserve in Salamanca, New York. This addition ties into an existing hotel tower and casino complex, adding a new floor of office space and 200 additional hotel rooms. Floors are comprised of normal weight concrete on composite metal deck supported by a steel framing system. To resist lateral loads, braced frames are used in the N-S direction and perimeter moment frames are used in the E-W direction. The whole addition rests on steel piles driven to bedrock.

Since the hotel addition makes use of a repetitive floor plan, a staggered truss system was deemed a possible design choice. The ultimate goal of this thesis was to properly implement a staggered truss system working as both the gravity system and lateral system in the N-S direction. Precast concrete planks were also implemented as the floor system, replacing the existing composite metal deck. Hand calculations were performed following *AISC Design Guide 14– Staggered Truss Framing Systems* to determine preliminary member sizes and stresses. Once member sizes were found, a RAM Elements model was created to check loads and deflections.

With the trusses spanning the entire width of the building, interior spaces were affected. The area most affected was the master bedroom in the VIP Suite at one end of the addition. This required that the addition's geometry be adjusted to fit the truss within the wall of the master bedroom. The master bedroom was shifted to "square-off" the end of the addition, thus creating more interior floor space. Options for this extra floor space included a new, separate hotel room, an additional guest room for the VIP Suite, and an elevator shaft.

The use of prefabricated members for the framing and floor system would allow for a faster erection process during construction. A new tower crane was selected in order to carry the heaviest member. This required an evaluation of the site plan during construction. Since the prefabricated members would be quite large, it was found that there would not be enough space on the existing site to store materials. Thus, members would have to be trucked in and lifted directly from the truck. A proper layout was created to show how the delivery trucks would reach the site and the tower crane.

A staggered truss system was found to be adequate for the SAC Hotel Addition, but required changes to the building's existing geometry to make the best use of the system. By "squaring-off" the NE corner of the addition, the existing retaining wall and two large drainage pipes behind the wall would have to be moved and redesigned. This would have been costly and time consuming. Had the SAC Hotel Addition been constructed with the staggered truss system in mind, and prior to the construction of the retaining wall, this would not have been an issue.

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Acknowledgements

I would like to thank the owners of the Seneca Allegany Casino, the Seneca Nation of Indians, for allowing me to use their new hotel addition as my senior thesis project. Special thanks to my cousin, Gary Paumen, for introducing me to the project and owner representative, Rob Chamberlain.

Thanks to:

- JCJ Architecture for providing all drawings of the hotel addition, and specifically Warren Sieber and Jay Hoelle for providing renderings
- M/E Engineering - Christopher Riggs and Rob Stewart

Very special thanks to Jim Boje of Wendel, who was integral in my success with this entire project. I was invited to see the project in person and Jim gave me a tour of the entire addition and casino complex himself, while providing me with plenty of guidance during both semesters of thesis. Thank you for always taking the time to answer my emails and give me detailed descriptions for all of my questions!

Thank you Professor Parfitt for being there to address all of my concerns during the spring semester and for putting my mind at ease any time I found a potential problem with this project.

I want to thank my family for putting up with all of my complaints over the years and always being there, while not fully knowing the extent of work put into this project.

Lastly, I want to thank my very good friend, Alex Oravitz, who kept me sane during my last year of school and who made sure that my reports were always grammatically correct.

Building Introduction

The Seneca Allegany Casino is a large complex located within the Seneca Indian Reserve in Salamanca, New York. The casino has undergone multiple construction phases over the years beginning with a pre-engineered metal building that housed the original casino, shown to the far right in Figure 1 below. With the construction of a new casino floor, parking deck, and hotel, the original casino was converted to an event center. This thesis will focus on the most recent phase of construction, highlighted in yellow in Figure 1, which is an additional 11-story, 200 room hotel tower.

Figure 1 - Seneca Allegany Casino
Satellite Photo - Bing.com Maps

The SAC Hotel Addition uses a structural steel framing system with composite metal deck bearing on steel pile foundations. This tower ties into an existing hotel tower and rests partly on a lobby built with the original hotel. The lobby was built to withstand the loads from the future hotel addition. Continuing the façade from the original hotel, the new addition is sheathed almost completely in insulated glass, shown in Figure 2.

Below the glass façade, the remaining portion of the hotel is covered in insulated metal panels. These floors of the hotel contain offices and mechanical and service rooms.

There are no surrounding structures near the complex, which allows plenty of direct sunlight for each hotel room. This also allows expansive views of the surrounding mountains and valley in which the casino is located.

Figure 2 - South Elevation
Photo Courtesy of Jim Boje, PE (Wendel)

Structural System

Foundation

Drawing 1 shows a plan view for the steel pile foundations, with the perimeter of the hotel addition outline in red. The piles are HP12x53's designed for a working capacity of 200 kips and driven to bedrock. The pile caps are designed for a compressive strength of 4000 psi, reinforced with #9 and #11 bars, and range 42" to 72" in thickness. The caps rest on piles and strip and spread footings rest on subgrade with an allowable bearing capacity of 2000 psf.

The perimeter foundation consists of strip and spread footings designed for a compressive strength of 3000 psi, ranging from 5' to 16' in width, reinforced with #5-#8 grade 60 steel bars. The perimeter uses concrete frost walls up to the ground floor slab on grade, while interior column footings make use of piers tied to columns with steel plates and Gr. 36 and Gr. 55 steel anchor bolts. A fixed connection was assumed for the E-W moment frames and a pinned connection for the N-S braced frames.

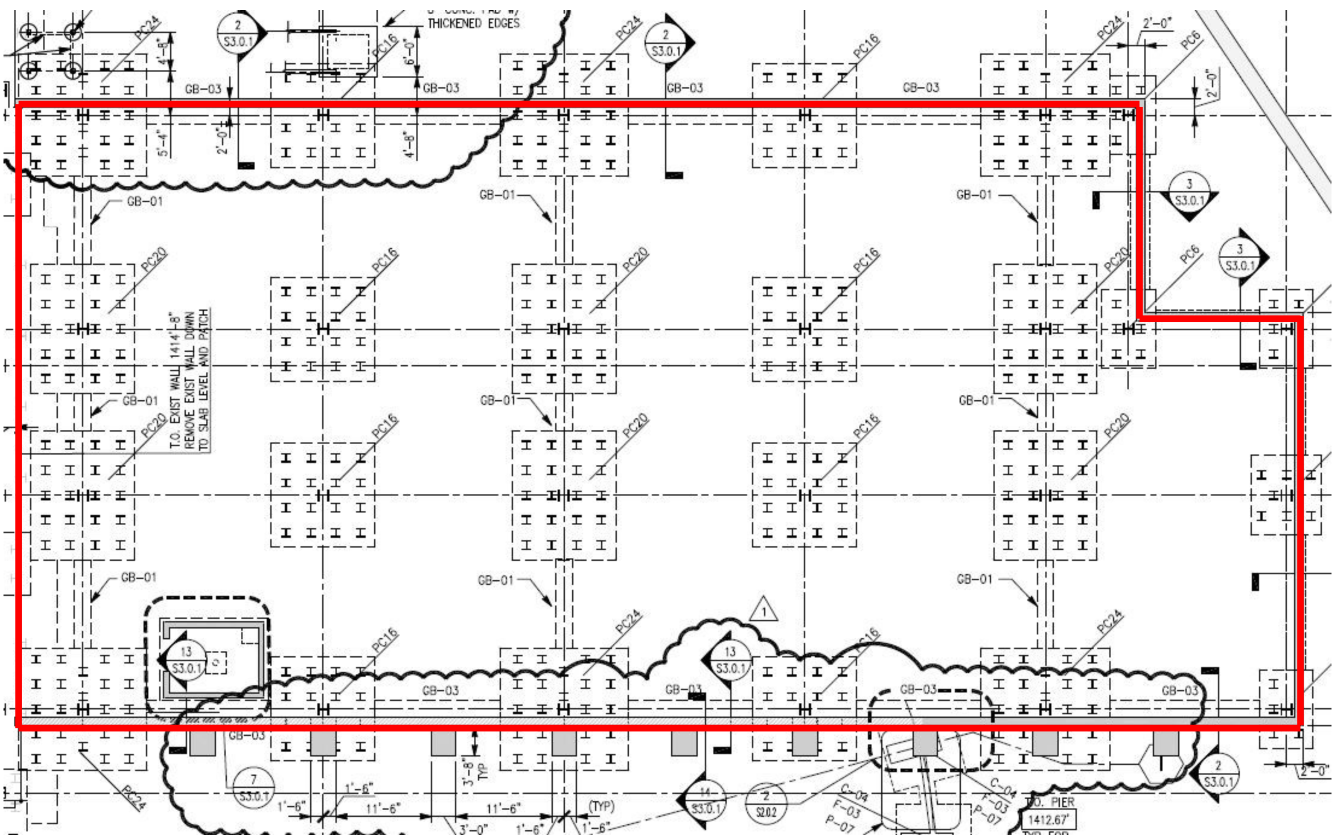


Figure 3 - Steel Pile/Pile Caps Plan
Drawings Courtesy of JCJ Architecture

Framing & Floors

Since this is a hotel tower, the bays are repetitive with the largest bay size a consistent 25'-9" by 29' from the lobby up through the 11th floor. The hotel rooms are located along the outer edges, between column lines 6.6 - 7.3 and 8.4- 9, shown here in Drawing 2. The middle section is the corridor, with a slightly smaller bay size of 20' by 29'.

The most significant change in member sizes occurs in the columns and girders as the elevation increases. All structural steel is 50 ksi. The majority of floor beams in the hotel rooms are W16x26, with the exception of the 3rd floor, where they are W16x31 and the mezzanine level, where they are W18x35. The corridor also is consistent with W12x16's on the 3rd through 10th floors. The exception in sizes for the corridor is on the 2nd floor with W14x22's and on the 11th floor with W12x19's.

The floor system consists of concrete slabs on metal deck; 20 gage for hotel rooms and 18 gage for roof, with a 6.5" total depth, normal weight concrete (145 pcf) with compressive strength of 3500 psi and 6x6/W2.9xW2.9 wire mesh. At splices between deck and span changes, #4 rebar spaced at 12" is used. 3/4" diameter shear studs are spaced evenly along beams and girders, with the number shown in plan. Figure 5 shows a typical deck section.

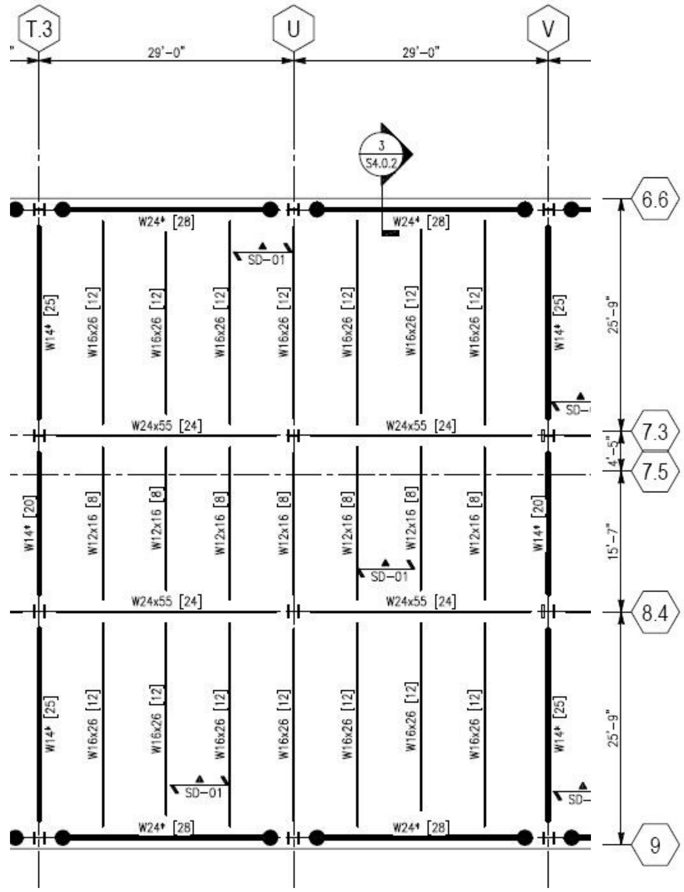


Figure 4 - Section of 4th—10th Floor Framing Plan
Drawings Courtesy of JCJ Architecture

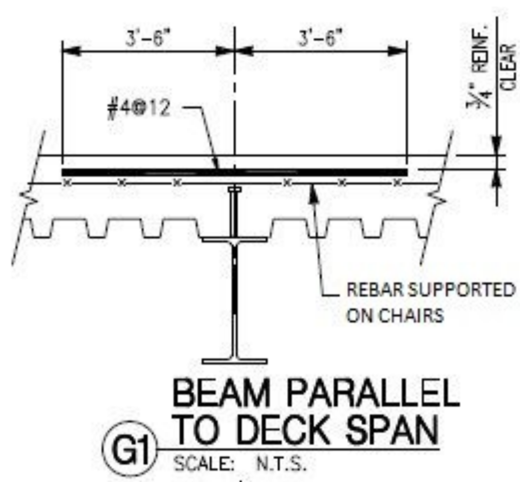


Figure 5 - Typical Composite Metal Deck Section
Drawings Courtesy of JCJ Architecture

Columns

The SAC Hotel addition uses wide flange columns throughout the entire addition. The weights of the columns decrease as the elevation increases, with a small range of sizes used. Figure 6 below shows the column schedule. All columns are in accordance with ASTM A992, 50 ksi steel.

Columns connect to the foundation by use of ASTM A572, 50 ksi base plates, and vary in attachments, whether it be with or without column piers, or directly to frost walls along the perimeter. Anchor bolts conform to ASTM F1554, 55 ksi.

STEEL COLUMN SCHEDULE

COLUMN MARK	COL. SIZE	BASE PLATE			ANCHOR BOLTS			REMARKS
		T (in.)	W (in.)	L (Ft.–in.)	QTY	SIZE (DIA)	ASTM F1554	
C–01	16"ø x0.50" PIPE	2"	24"	2'–0"	4	1 1/4"	GR55	.
C–02	W14x68	1"	22"	1'–10"	4	1"	GR36	.
C–03	W14x90	1 1/2"	22"	1'–10"	4	1"	GR36	.
C–04	W14x132	2"	28"	2'–4"	4	1 1/4"	GR55	20" WIDE BASE ℄ AT HOTEL LOBBY

Figure 6
Drawings Courtesy of JCJ Architecture

Lateral System

The lateral systems used in the SAC Hotel consist of moment frames in the long span (E-W) directions and diagonally braced frames in the short (N-S) directions. For the moment frames, moment connections occur at columns and girders, shown below in Figures 7 and 8.

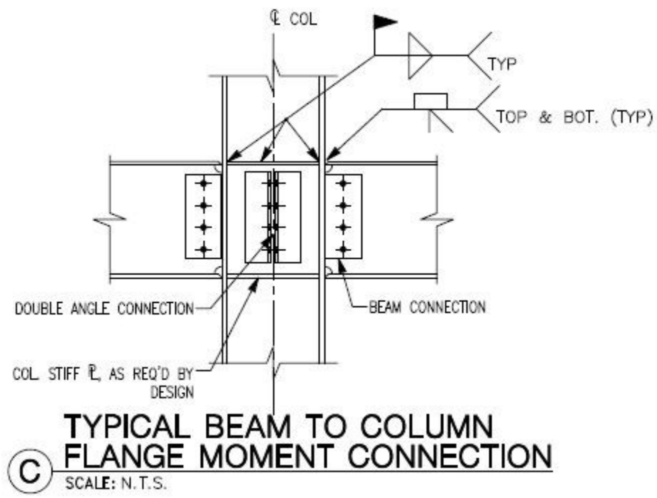


Figure 7 - Typical Moment Connection
Drawings Courtesy of JCJ Architecture

Figure 8 - Typical Moment Connection
Photo Courtesy of Jim Boje, PE (Wendel)

The diagonal bracing is used in specific column lines. Wide flange shapes are used, ranging in size from W14's at the lower floor levels to W10's for the 4th through 10th floor. Column line W has only one bay diagonally braced the entire height of the building to account for the stair-well. The bracing is tied into the frame by use of steel plates embedded in slab deck at beams and columns, shown by Figures 9 and 10.

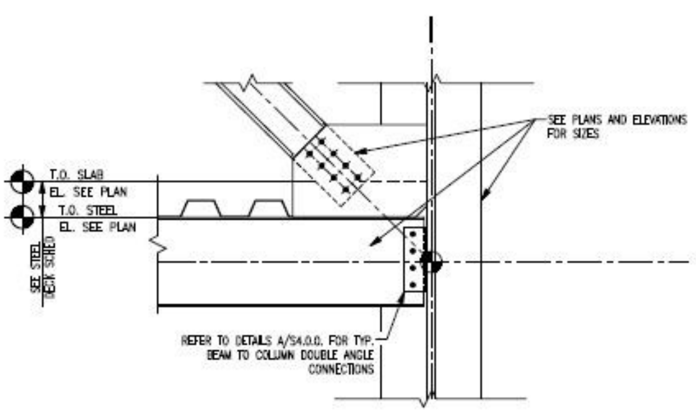


Figure 9 - Diagonal Brace Connection at Column
Drawings Courtesy of JCJ Architecture

Figure 10 - Diagonal Brace Connection at Column
Photo Courtesy of Jim Boje, PE (Wendel)

Roof

The roof structure is consistent with the hotel floor framing, with no change in bay sizes, or location of moment frames, and uses similar metal deck to the hotel floors, with a larger gauge of 18. Slightly larger W shapes are used to account for the extra roof snow load, (40 psf), with the majority of members being W18x35's. A 5' parapet surrounds the perimeter, framed with HSS 14x10x3/16 members embedded within. A detailed parapet section is shown in Figure 11, with the HSS outlined in red. The roof also supports window washing machines, with anchors embedded in the deck.

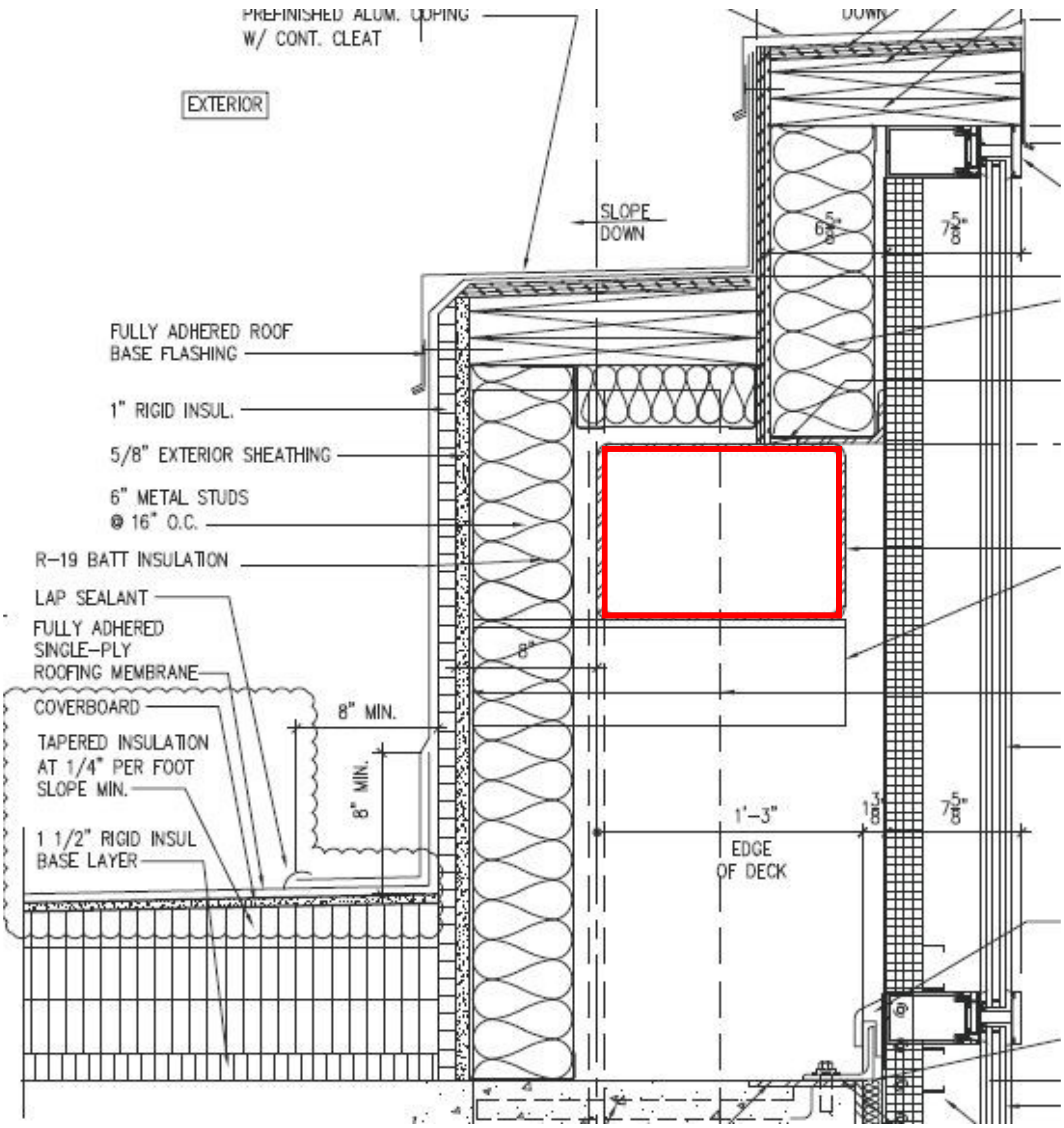


Figure 11 - Roof Parapet Section
Drawings Courtesy of JCJ Architecture

Expansion Joint

The addition to the SAC Hotel requires that the structure tie into the existing structure of the original 11-story hotel tower. This was accomplished using a 12” expansion joint beginning at the 4th floor and at each floor up through the roof level, shown below in Figure 12 and 13. The joint provides a flexible connection which allows the new addition to move independent of the existing tower, resisting wind and seismic loads through the moment and braced frames with no effect on the existing tower.

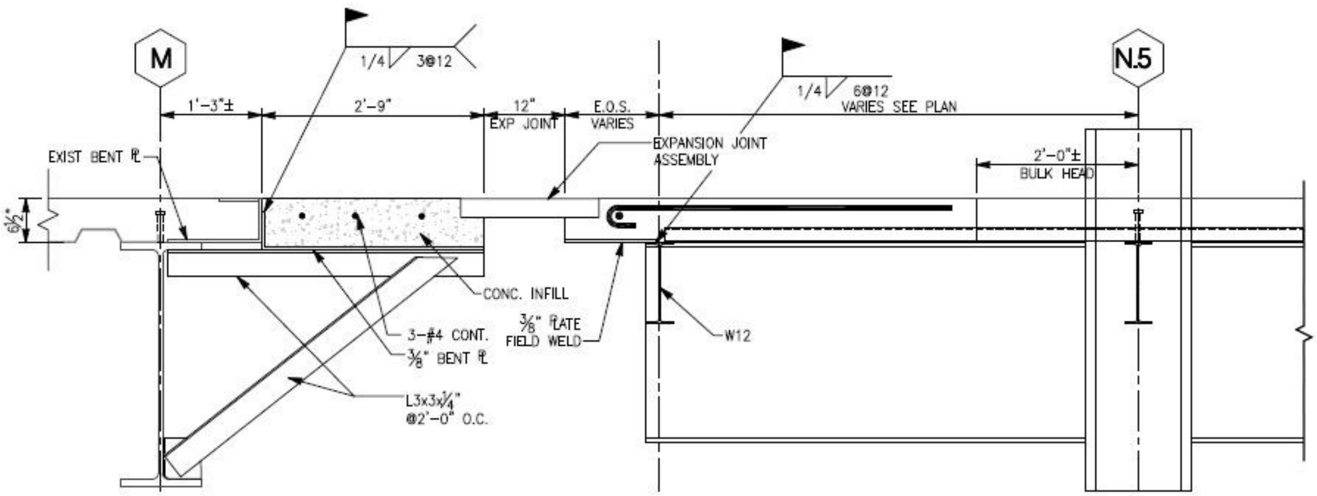


Figure 12 - Expansion Joint Section
Drawing Courtesy of JCJ Architecture

Figure 13 - Expansion Joint Section
Photo Courtesy Jim Boje, PE (Wendel)

Design Codes

Construction of the 2nd SAC Hotel tower began in 2008, and was put on hold until 2011. The following codes were used in the design process:

- 2006 International Building Code
- 2010 New York State Building Code
- ASCE 7-05
- ACI 318-08
- AISC, 13th edition
- Building code requirements for concrete masonry structures ACI-530 and ACI-530.1

For this report, the following code editions and design manuals were used for calculations:

- 2009 IBC
- ASCE 7-05
- AISC, 14th edition
- AISC Steel Design Guide 14 - Staggered Truss Framing Systems
- CRSI Design Manual 2008

Material Properties

Concrete

Pilecaps, Piers, and Grade Beams	4000 psi
Footings and Frost Walls	3000 psi
Interior Slabs	4000 psi
Concrete in Slabs on Metal Deck	3500 psi

Masonry

Hollow Masonry Units	ASTM C90, 1900 psi
Mortar	Type S, ASTM C270, 1800 psi
Grout	ASTM C476, 3000 psi

Metal Deck

Hotel Floors	2", 20 Gauge, NWC
Mezzanine and Roof	2", 18 Gauge, NWC

Reinforcement

Reinforcing Bars	ASTM 615, Grade 60
Welded Wire Fabric	ASTM A185
Lap Splices and Spacing	ACI 318

Structural Steel

Connections	Bolts, ASTM A325 or A490
Columns, Beams & Girders	50 ksi, ASTM A992
Tubular Shapes	46 ksi, ASTM A500, Grade B
Round Shapes	36 ksi, ASTM A53, Grade B
Plates	50 ksi, ASTM A572
All Other Steel	36 ksi, ASTM A36
Anchor Bolts	55 ksi, ASTM F1554 (U.O.N.)

Cold Formed Metal Framing

12, 14 and 16 Gage Studs	ASTM C955, Fy = 50 ksi
18 and 20 Gage Studs	ASTM C955, Fy = 33 ksi
Track, Bridging and Accessories	ASTM C955, Fy = 33 ksi

Gravity Loads

Below is an overview of the design loads used in this analysis of the SAC Hotel addition, including loads provided in the specifications and estimations used for calculations.

Dead Loads		
Superimposed	15 psf	Partitions/Façade Estimate
MEP	10 psf	Specs
Ceiling	5 psf	Specs
Precast Planks (w/ 2" topping)	86.25 psf	Nitterhouse Concrete Products

Live Loads		
	Design Loads	*ASCE 7-05*
Ground Floor	250 psf	
Typical Hotel Rooms	80 psf	40 psf
Hotel 2nd Floor	125 psf	
11th Floor Suites	125 psf	40 psf
Roof and Mezzanine	200 psf	20 psf
Corridors, Stairs, Lobbies	100 psf	100 psf
Mechanical Rooms	200 psf	

Note: Due to drastic differences in ASCE 7-05 values and the Design Loads listed in the specifications, the provided design loads were always used in calculations.

Snow Loads		
	Design Loads	*ASCE 7-05*
Roof Snow Load	40 psf	38.5 psf
Ground Snow Load	50 psf	CS
Drift Snow Load	-	20.5 psf

Note: CS in ASCE 7-05 stands for Case Study snow loads, which is why the 50 psf Design Load was used in calculations, taken from the specifications for the 2010 New York State Building Code.

Proposal Objectives

Structural Depth

Technical Report 2 was specifically focused on researching alternative designs for the gravity framing system in the SAC Hotel Addition. A staggered truss system was investigated and found to be a potential option due to the repetitive floor plan used in a majority of the hotel. The analysis of the staggered truss only took account of gravity loads from precast concrete planks, so a look at how the trusses would perform under lateral loads was required. Implementing the truss system would require a few concerns to be addressed, as well as considerations other than structural design. These include:

- The most effective layout of the trusses to carry gravity loads and work as the lateral system in the N-S direction, replacing the existing braced frames
- Change in overall building weight with use of precast planks
- Redesign of foundation due to change in overall building weight
- Impact on layout of interior spaces

Architectural Study

The repetitive floor plan of the SAC Hotel Addition allows for most trusses to be concealed within walls, but in a few areas, the truss would be exposed. These spaces will be shifted in order to keep all trusses concealed and to keep a consistent truss layout. To accomplish these shifts, one corner section of the addition will need to be "squared off" in order to make space for the moving of the rooms. The spaces affected will be discussed in more detail in later sections.

Construction Management Study

With the staggered truss system, precast concrete planks will be used for the floors. Using prefabricated members would allow for quicker erection and no time would be needed for the curing of concrete. A new site plan will be developed to communicate the flow of construction over time, detailing site access and crane locations.

Structural Design

Background

Staggered Truss systems make use of one-story deep trusses with Vierendeel panels for corridors. The trusses are encased within interior walls and allow for large open spaces since interior columns are not needed for support. A reduced number of interior columns allows for savings on foundation work by reducing the amount of concrete needed, formwork and construction time.

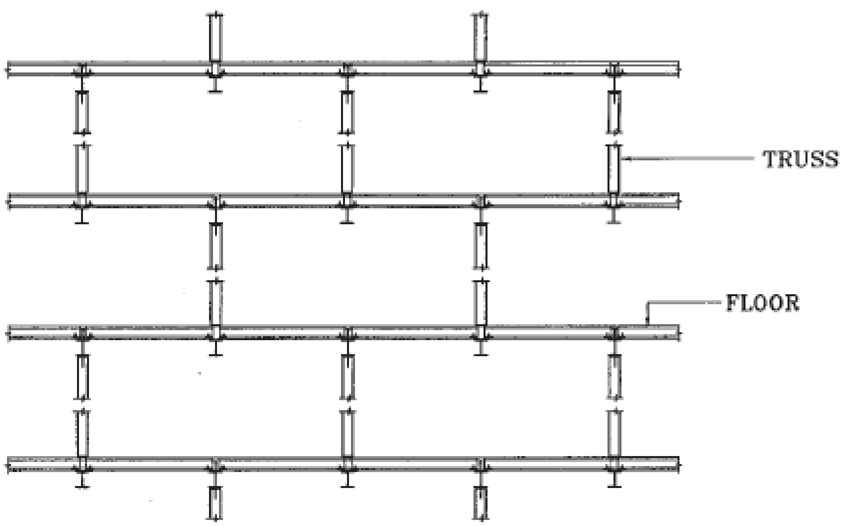


Figure 14 –Staggered Truss Vertical Arrangement
Image from AISC Design Guide 14

The "staggering" of the trusses is shown here in Figure 14. For each floor, the truss locations skip a typical column line for each bay. This is where large column-free areas are created, eliminating the need for interior column foundations. In this system, both the top and bottom chords of the members are loaded.

Shown below is a typical elevation of a truss that will be implemented into the SAC Hotel. The staggered truss spans the entire width of the building, 71.5', with the diagonal members located within the hotel room walls and the central corridor located in the Vierendeel panel.

AISC Design Guide 14 - Staggered Truss Framing Systems provided information on the overall system and design examples for sizing truss members. The guide suggests W-shapes for the top and bottom chords, and rectangular HSS-shapes for the web members.

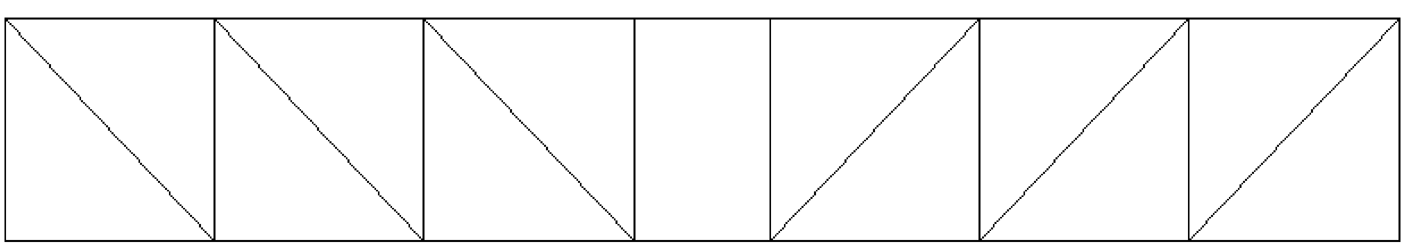


Figure 15 –Typical Truss

For the floor system, *AISC Design Guide 14* suggests the use of precast, hollow-core concrete planks. As stated before, the top and bottom chords of each truss carry the floor loads. The planks are connected to the chords with weld plates, then shear studs and reinforcing bars are grouted between planks, shown below in Figure 16.

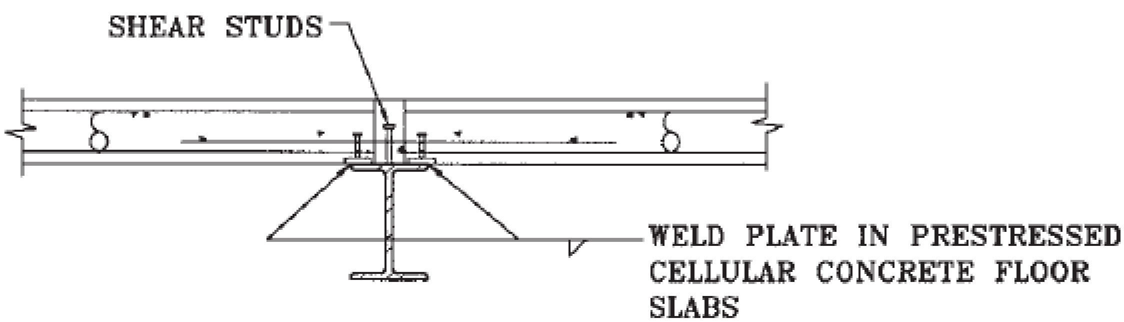


Figure 16 –Concrete Plank Floor System
Image from AISC Design Guide 14

Hollow-core planks will also be used in this redesign. A typical section of an 8 inch plank is shown below in Figure 17 from Nitterhouse Concrete Products. Two sizes of planks will be needed in order to carry the differing live loads from the SAC Hotel Addition's specs. This will be discussed in a later section.

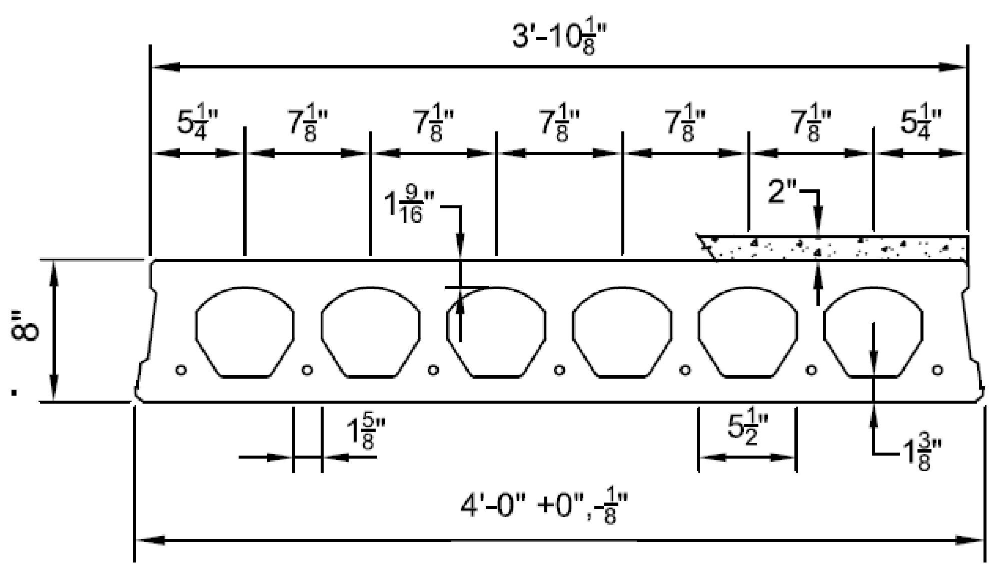


Figure 17 –Hollow-core Concrete Plank
Nitterhouse Concrete Products

Truss Layout

The SAC Hotel Addition's existing bay widths of 29' were maintained in order to keep the new design geometry as close as possible to the existing building. This way, the locations of the trusses would line up exactly with the existing braced frames in the N-S direction, requiring minimal rearranging of interior spaces. Shown below is a typical layout of the trusses.

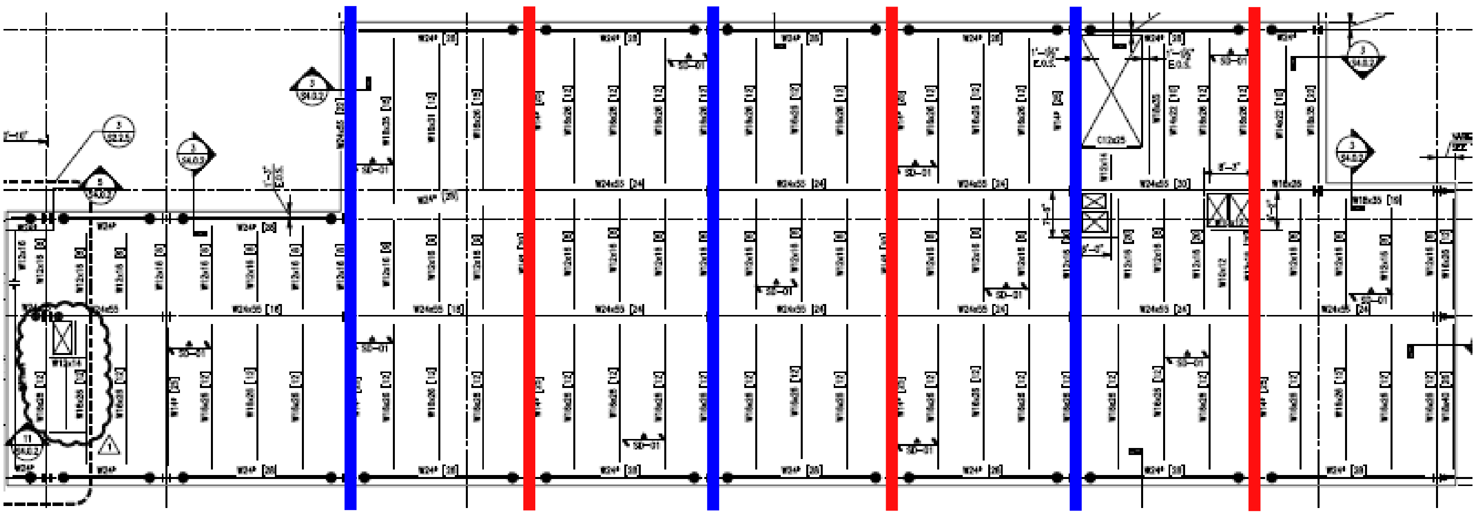


Figure 18 –Typical Truss Layout
Drawing Courtesy of JCJ Architecture

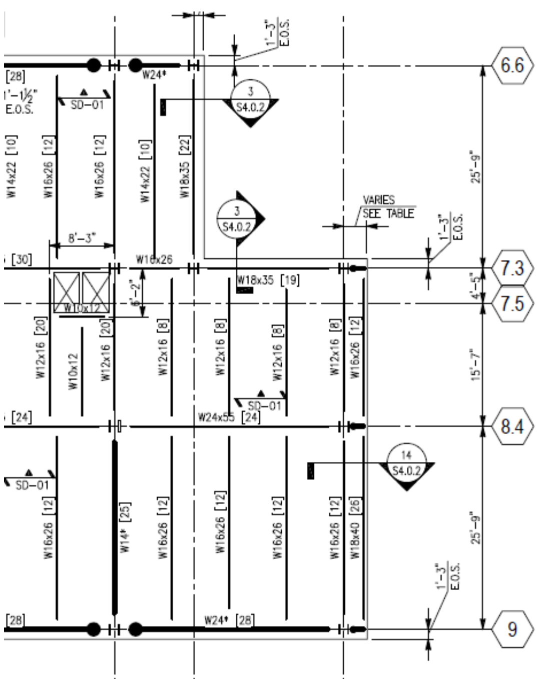


Figure 19 –Notched Corner
Drawing Courtesy of JCJ Architecture

Alternating colors for the trusses show alternating floor locations. This truss layout is consistent for the entire height of the building, with a few key areas that required changes to the floor plan and overall geometry of the building. Located in the far right of Figure 18 is a notched corner, shown here on the left between column lines 6.6 and 7.3. In order to make the best use of the trusses while not interfering with the existing room layout, this corner of the building had to be squared-off. This, and other areas affected, will be discussed later in the Architectural Study section.

Truss Design

Floor System

While the SAC Hotel Addition has a mostly consistent floor plan that would make good use of a staggered truss system, there are a few existing conditions that needed to be adjusted. The typical floors with hotel rooms have a floor to floor height of 11'4" and were designed with this height. The mezzanine, 2nd, 3rd, and 11th floors have varying floor heights ranging from 13' to 15'.

With the erection process in mind, having multiple sizes of trusses was undesirable. These 4 floors were instead assumed to be the same height, 15', to keep the number of different trusses limited to two.

Once it was decided that precast concrete planks would be used for the floor system, hand calculations were performed in order to determine an appropriate size of plank for each floor. These can be found in Appendix A and the loads used can be found on the tables below. It was determined that two sizes of planks would be required, since the live loads from the SAC Hotel's specs vary for some floors. An 8" plank can be used for the typical hotel floors, while a 10" plank will be used in all other floors. Specifications for each plank can be found in Appendix B.

Dead Loads		
Superimposed	15 psf	Partitions/Façade Estimate
MEP	10 psf	Specs
Ceiling	5 psf	Specs
8" Plank with 2" topping	86.25 psf	Nitterhouse Concrete Products
10" Plank with 2" topping	93 psf	Nitterhouse Concrete Products

Live Loads		
	Design Loads	*ASCE 7-05*
Ground Floor	250 psf	
Typical Hotel Rooms	80 psf	40 psf
Hotel 2nd Floor	125 psf	
11th Floor Suites	125 psf	40 psf
Roof and Mezzanine	200 psf	20 psf
Corridors, Stairs, Lobbies	100 psf	100 psf
Mechanical Rooms	200 psf	

Note: Due to drastic differences in ASCE 7-05 values and the Design Loads listed in the specifications, the provided design loads were always used in calculations.

Truss Members

In order to gain a better understanding of how a staggered truss system works, hand calculations were completed prior to creating a computer model. *AISC Design Guide 14* was followed extensively for this portion of the report. In truss members, the vertical and diagonal members are only subject to axial loads, while the top and bottom chords are subject to axial loads and moments.

After loads from the precast concrete planks were calculated, the uniform gravity loads were converted to concentrated loads that act on each joint of the truss. The design guide states that gravity loads produce shear in the top and bottom chord at the Vierendeel panel, but this could be ignored due to symmetry, thus the truss becomes statically determinate. With this assumption, there are two methods proposed by the design guide: the method of joints and the method of sections. The method of joints was used in this design and hand calculations for this method can be found in Appendix A.

Hand calculations took account of unfactored dead, live and lateral loads. Once member loads were determined, the design guide suggests a method of coefficients for determining actual loads on each member due to varying load cases. Excel spreadsheets were used to check each load case, which can be found in Appendix A, along with preliminary member sizes used in the computer model. It was found that 1.2D + 1.6L controlled member design.

2.3.2 Basic Combinations. Structures, components, and foundations shall be designed so that their design strength equals or exceeds the effects of the factored loads in the following combinations:

1. $1.4(D + F)$
2. $1.2(D + F + T) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R)$
3. $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.8W)$
4. $1.2D + 1.6W + L + 0.5(L_r \text{ or } S \text{ or } R)$
5. $1.2D + 1.0E + L + 0.2S$
6. $0.9D + 1.6W + 1.6H$
7. $0.9D + 1.0E + 1.6H$

Chapter 2 ASCE 7-05

* Note: Wind was only checked with hand calculations since wind was found to control the lateral design in the N-S direction in Technical Report 3. Seismic was checked with the computer model, which will be discussed in the following pages.

Truss Members

As stated previously, two different sizes of trusses had to be designed for the SAC Hotel Addition due to varying floor heights. The mezzanine, 2nd, 3rd and 11th floors were all designed to use one truss shape, with the 4th through 10th floors using the other.

Preliminary truss member sizes are as follows:

<u>Mezzanine, 2nd, 3rd, 11th floor truss</u>

- Top and bottom chords: W10 x 60
- Diagonal and vertical members: HSS14 x 10 x 5/8

<u>4th through 10th floor truss</u>

- Top and bottom chords: W10 x 33
- Diagonal and vertical members: HSS9 x 7 x 5/8

A computer model was constructed with RAM Elements to check controlling load cases for the building as a whole. According to code, floor members have an allowable deflection of L/240 or 3.6". Both sizes of truss members meet this requirement. Tables detailing node deflections can be found on the following page.

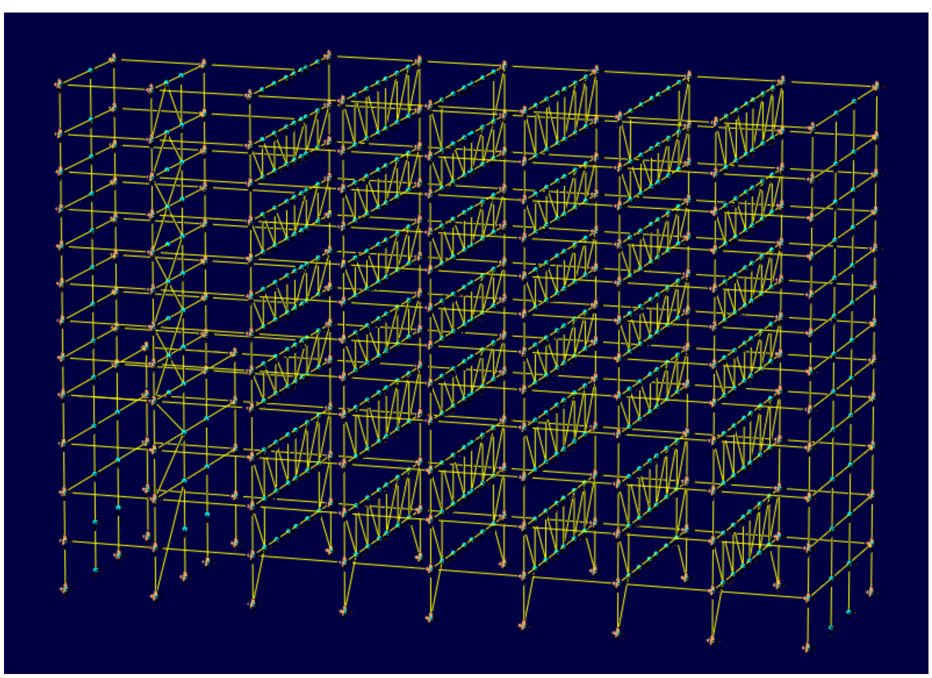
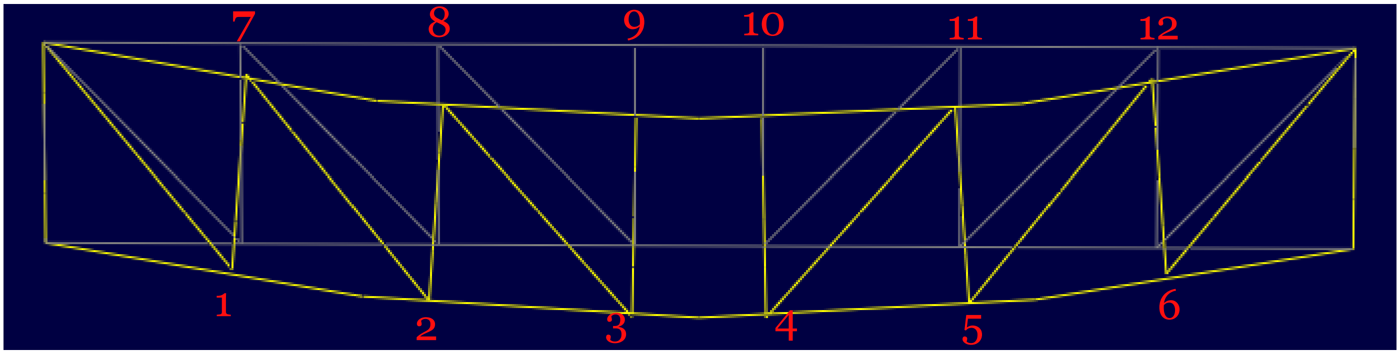


Figure 20 - RAM Elements Model

Truss Member Deflections



Large Truss			
Translations [in]			
Node	TX	TY	TZ
Load Combo: 1.2D+1.6W+1.0L			
1	-0.020	-0.110	0
2	-0.054	-0.436	0
3	-0.021	-0.608	0
4	0.021	-0.608	0
5	0.054	-0.436	0
6	0.020	-0.110	0
7	0.066	-0.207	0
8	0.056	-0.491	0
9	0.031	-0.633	0
10	-0.031	-0.633	0
11	-0.049	-0.541	0
12	-0.059	-0.265	0
Load Combo: 1.2D+1.6L			
1	-0.02758	-0.15051	0
2	-0.07393	-0.59424	0
3	-0.02809	-0.82991	0
4	0.02809	-0.82991	0
5	0.07393	-0.59424	0
6	0.02758	-0.15051	0
7	0.08977	-0.28279	0
8	0.07685	-0.66951	0
9	0.01888	-0.85277	0
10	-0.01888	-0.85277	0
11	-0.07685	-0.66951	0
12	-0.08977	-0.28279	0

Small Truss			
Translations [in]			
Node	TX	TY	TZ
Load Combo: 1.2D+1.6W+1.0L			
1	-0.16432	-0.44129	0
2	-0.16299	-0.96655	0
3	-0.05171	-1.227	0
4	0.05171	-1.227	0
5	0.16299	-0.96655	0
6	0.16432	-0.44129	0
7	0.09584	-0.53157	0
8	0.08173	-1.01846	0
9	0.02007	-1.24263	0
10	-0.02007	-1.24263	0
11	-0.08173	-1.01846	0
12	-0.09584	-0.53157	0
Load Combo: 1.2D+1.6L			
1	-0.20888	-0.56097	0
2	-0.20719	-1.22867	0
3	-0.06574	-1.55976	0
4	0.06574	-1.55976	0
5	0.20719	-1.22867	0
6	0.20888	-0.56097	0
7	0.12183	-0.67574	0
8	0.1039	-1.29467	0
9	0.02552	-1.57963	0
10	-0.02552	-1.57963	0
11	-0.1039	-1.29467	0
12	-0.12183	-0.67574	0

Truss Member Stresses

The hand calculations shown in Appendix A explain how axial forces work within the individual truss members, with the top chord and vertical members in compression, while the bottom chord and diagonal members are in tension. The following images are taken from the RAM Elements model showing how the designed trusses performed under gravity loads.

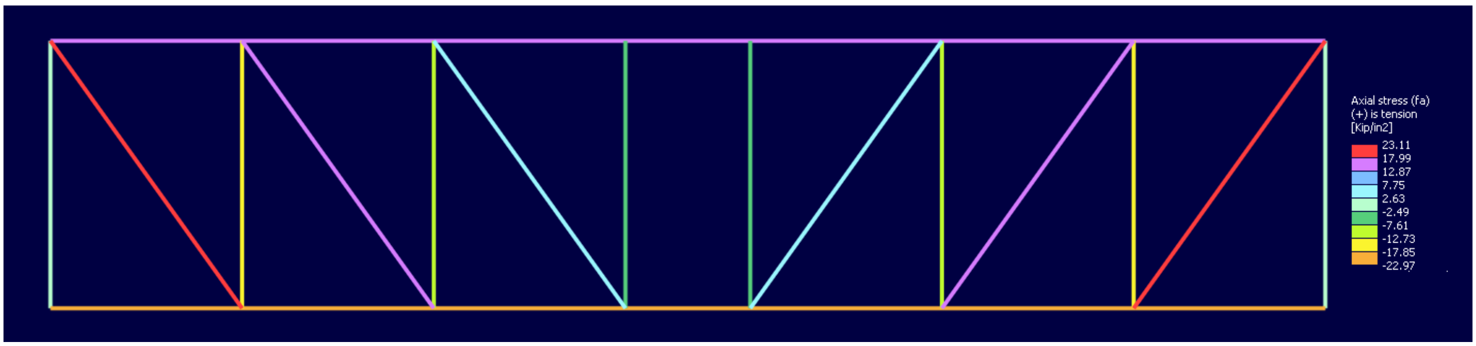


Figure 21 - Large Truss Stresses

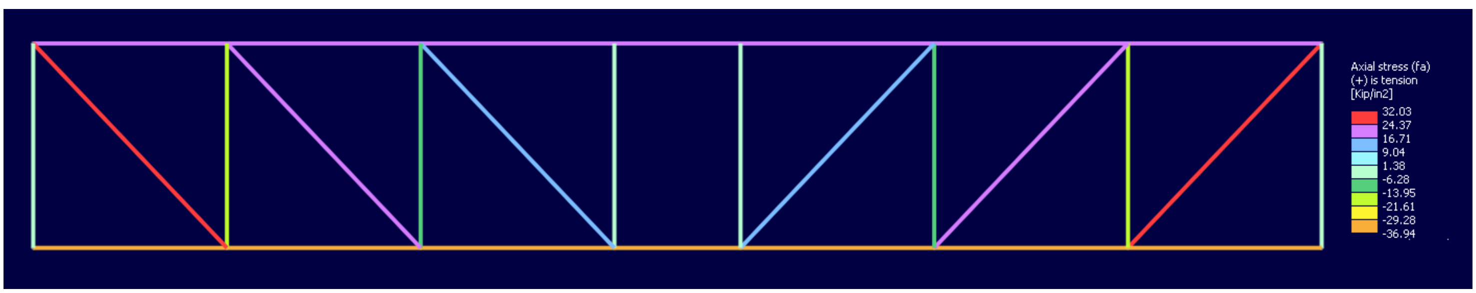


Figure 22 - Small Truss Stresses

In the figures above, the most stressed members are the exterior diagonal members (shown in red), which is to be expected in the design of trusses. Since these diagonals take the most load, they governed the design of all vertical and diagonal members save for the exterior columns. Exterior column design will be explained on the following page.

AISC Design Guide 14 suggests using the same size of HSS for all diagonal and vertical members during the design of trusses. The exterior diagonals in the larger truss in Figure 21 take significantly larger loads than the rest of the diagonals and verticals, which required an HSSx16 shape. Load cases per truss can be found in Appendix A. Savings could be made by stepping down in size as loads move towards the Vierendeel panel, as well as lowering the overall member weights.

Exterior Columns

While the staggered truss system eliminates the need for interior columns, edge columns are still required. The lack of interior columns greatly increases the tributary area and subsequent load that each edge column will carry, so a redesign was required. Sample hand calculations for column loads can be found in Appendix A. Tabulated below are the individual floor loads on each column, and the sizes of columns selected.

Column Loads				
Floor	A_t (ft^2)	DL (psf)	LL (psf)	RLL (psf)
Roof	1036.8	101	200	200
11	1036.8	101	80	38.6
10	1036.8	101	80	38.6
9	1036.8	101	80	38.6
8	1036.8	101	80	38.6
7	1036.8	101	80	38.6
6	1036.8	101	80	38.6
5	1036.8	101	80	38.6
4	1036.8	101	80	38.6
3	1036.8	101	125	125
2	1036.8	101	125	125
Mezz	1036.8	101	200	200

Column Capacities					
Floor	Pu (k)	∑Pu (k)	Member	ΦPn (k)	Unbraced Length (ft)
Roof	487	487	W12x79	809	15
11	190	677	W12x79	910	11.33
10	190	867	W12x96	1110	11.33
9	190	1057	W12x96	1110	11.33
8	190	1247	W12x136	1580	11.33
7	190	1437	W12x136	1580	11.33
6	190	1627	W12x170	1990	11.33
5	190	1817	W12x170	1990	11.33
4	190	2007	W12x230	2710	11.33
3	342	2349	W12x210	2450	15
2	342	2691	W14x283	3270	15
Mezz	466	3157	W14x283	3270	15
∑	**3157**				

Lateral Loads

The existing lateral framing for the SAC Hotel Addition consists of braced frames in the short (N-S) direction, and perimeter moment frames in the long (E-W) direction. With the new staggered truss system, the braced frames would be replaced. According to the *AISC Design Guide 14* and the *AISC Case Study: Building Success With the Staggered Truss*, structures up to 25 - 30 stories generally take both gravity and lateral loads.

The SAC Hotel Addition is 11 stories and was confirmed to take the lateral loads due to wind with the use of the RAM Elements model. Seismic loads were found to be the controlling case for the E-W moment frames in Technical Report 3 and were assumed to control with the staggered truss system as well. These loads were checked in the N-S direction to confirm that wind did indeed control, and are tabulated here.

Seismic (N-S) (kips)	
1.2D+L+E	
Roof	69.5
11	74.3
10	68.4
9	61.4
8	56.7
7	51.6
6	45.1
5	39.2
4	32.8
3	24.0
2	14.7
Mezz	8.6
	546.3

Wind (N-S) (kips)	
1.2D+1.6W+L	
Roof	113.4
11	85.9
10	83.1
9	83.4
8	84.5
7	83.9
6	82.6
5	83.5
4	110.7
3	112.6
2	111.6
Mezz	113.0
	1148.2

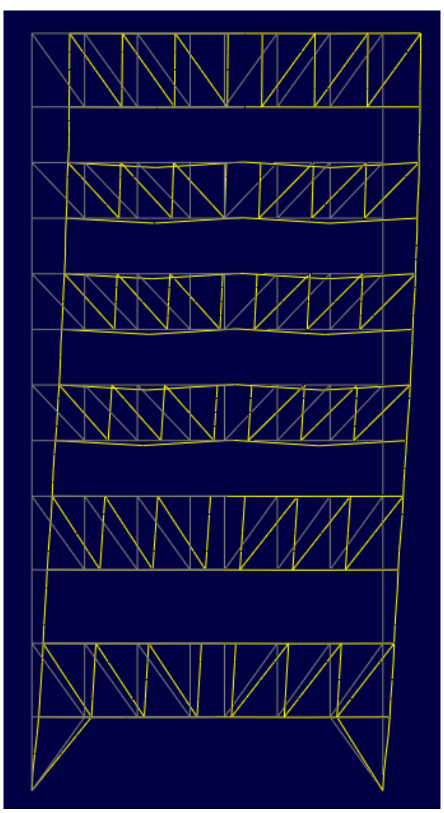


Figure 23 - Frame Deflection

With the controlling wind load case of 1.2D + 1.6W + 1.0L, the overall deflection of a frame was checked in RAM. The largest deflection occurred at one of the central frames with a value of 0.626" at the roof level. The new height of the SAC Hotel after increasing certain floor heights is 154'. This value is well under code limits of H/500, or 3.7".

Lateral Loads

Once new lateral loads were found using the computer model, a check of their effect on the top and bottom chords was required. Story forces create moments within the chords at the center of the Vierendeel panel while gravity loads also create moments at the ends of each truss.

AISC Design Guide 14 was followed to check moments in each chord per floor. The force at each floor is divided amongst the top and bottom chord, depending on truss location within the addition. Sample hand calculations of this procedure can be found in Appendix A. The table below details chord member sizes.

Chord Design								
	Story Force (kips)	Applied Load (kips)	Flr-to-Flr Height (ft)	V_{mid} (kips)	M_{mid} (ft-k)	Mu (ft-k)	Pu (kips)	Section
Roof	113.4	56.7	15	11.9	41.6	237.6	748.8	W10x60
11	85.9	99.65	11.33	15.8	55.3	130.1	492	W10x60
10	83.1	141.2	11.33	22.4	78.3	153.2	492	W10x68
9	83.4	182.9	11.33	29.0	101.4	176.3	492	W10x68
8	84.5	225.15	11.33	35.7	124.9	199.7	492	W10x77
7	83.9	267.1	11.33	42.3	148.1	223.0	492	W10x77
6	82.6	308.4	11.33	48.9	171.0	245.9	492	W10x88
5	83.5	350.15	11.33	55.5	194.2	269.0	492	W10x88
4	110.7	405.5	11.33	64.3	224.9	299.7	492	W10x100
3	112.6	461.8	15	96.9	339.1	535.1	748.8	W10x100
2	111.6	517.6	15	108.6	380.1	576.1	748.8	W12x170
Mezz	113	574.1	15	120.4	421.5	617.5	748.8	W12x170
Σ	1148.2							

	M_{end} (ft-k)
Small	74.84
Large	196

Foundation Impact

The SAC Hotel Addition makes use of steel piles driven to bedrock for the design of the foundation. Piles used are HP 12x53 with bearing capacities of 200 kips. Since the staggered truss system eliminates interior columns, the piles required to carry the loads from these interior columns can also be eliminated and are highlighted below.

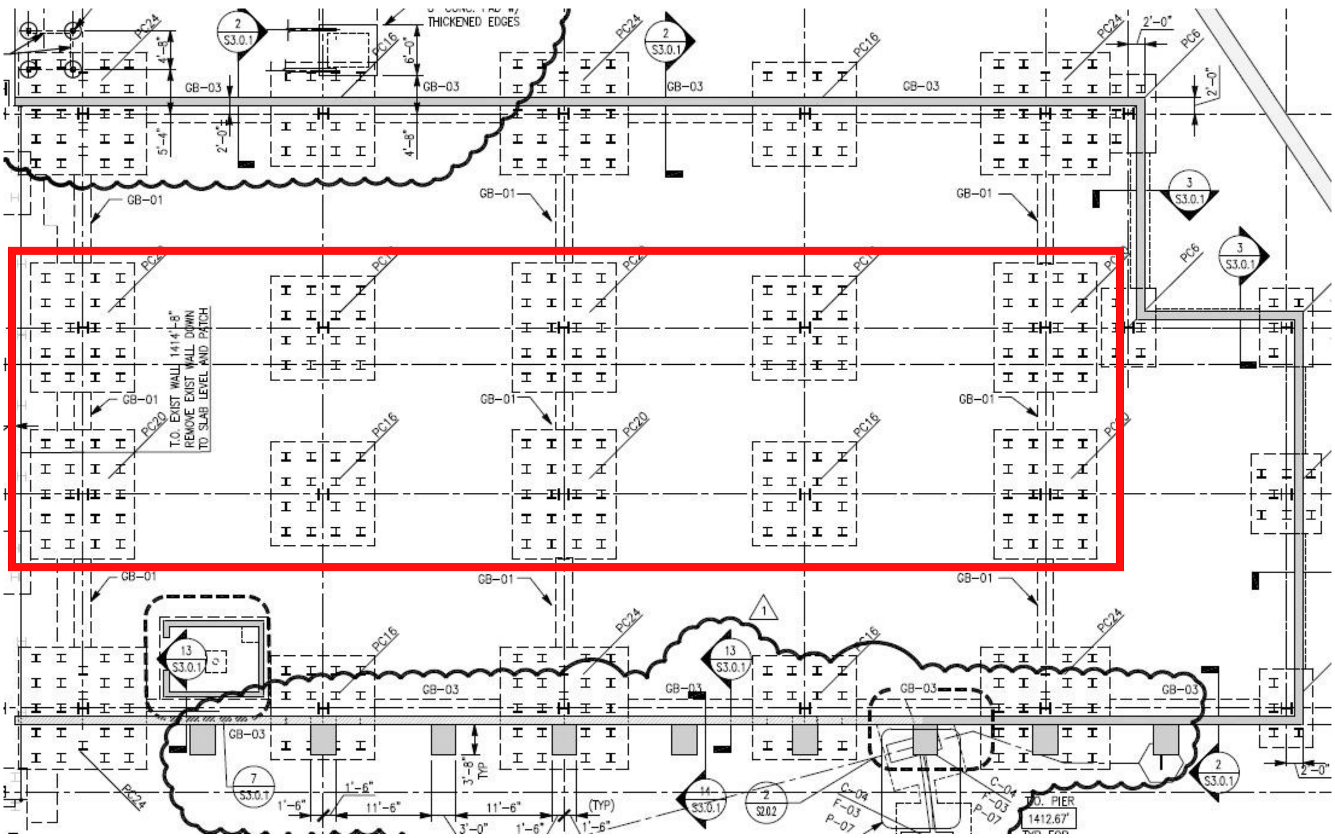


Figure 24 - Steel Pile/Pile Caps Plan
Drawings Courtesy of JCJ Architecture

Edge piles can also be reduced in size. The total load on each pile would be equal to the total load on the ground level column, found on page 24, which is 3,157 kips. With a capacity of 200 kips per HP 12x53, a typical pile would require 16 HP shapes.

In the existing foundation design, 424 HP piles are used.

Foundation Impact

The *Concrete Reinforcing Steel Institute 2008 Design Handbook* contains a chapter for pile-cap design. An HP 12x53 is equivalent to a 100-ton steel pile. The table in the handbook suggests 11 piles capable of supporting a total load of 3,404 kips with an overall concrete depth of 53”. The table referenced can be found in Appendix A.

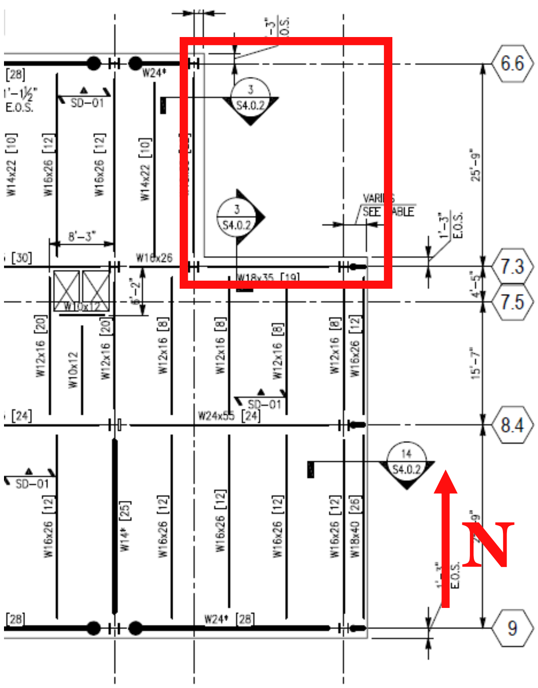


Figure 25 –Notched Corner
Drawing Courtesy of JCJ Architecture

As stated at the beginning of this report, this section shown to the left in red would require a redesign to make room for the staggered truss system. This will be explained in more detail in the following pages, but this new section would also require a redesign of the foundation. In the existing foundation, this edge column line has the smallest amount of piles overall, and the redesign will only remove a small amount. There will be 4 columns each requiring a pile-cap 40” deep with 4 HP 12x53 piles. These values can be seen in the same table previously stated to be in Appendix A.

With a reduction in piles and pile-cap sizes, the new piles with 11 HP 12x53’s will have a rectangular profile instead of the existing square profile. The long direction of the new pile-cap will be oriented in the N-S direction in order to better resist the lateral loads produced by wind.

The new total amount of steel piles required will approximately be 126, a 70% reduction from the existing amount. This is a considerable amount, but the addition was originally designed with ASD. This foundation check was done with LRFD.

*Note: The RAM Elements model showed that all exterior columns at the ground level would be in compression, thus uplift was not a concern in the new foundation design.

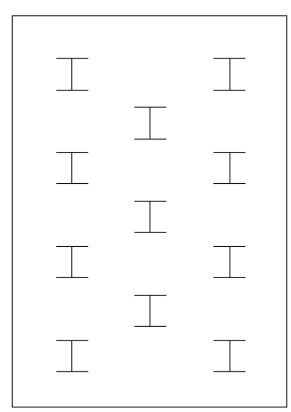


Figure 26 –New Pile-cap Geometry

Architectural Study

Background

With the staggered truss system replacing the existing braced frame system, interior spaces would be affected by certain truss locations in order to keep a consistent spacing and overall "staggering" of the trusses. The most important area affected is located in the NE corner of the existing SAC Hotel Addition. This area is a VIP Suite, encompassing 3 separate hotel rooms and a common area, and is outlined in red below. Figure 28 below shows a truss going directly through the master bedroom in the VIP Suite.

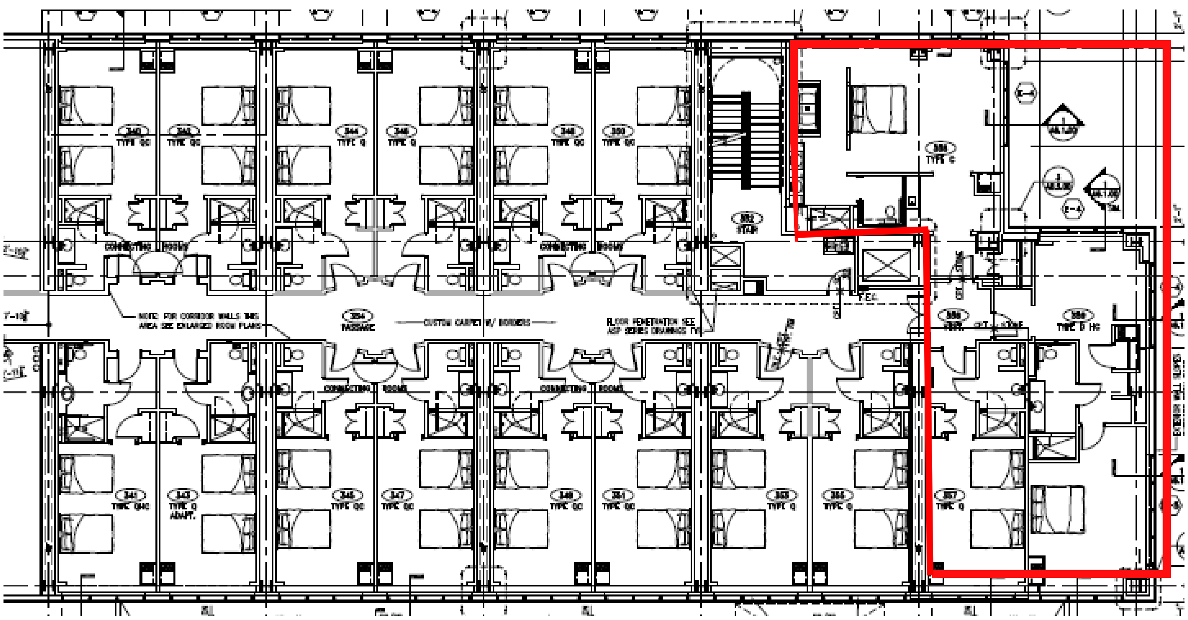


Figure 27 –VIP Suite

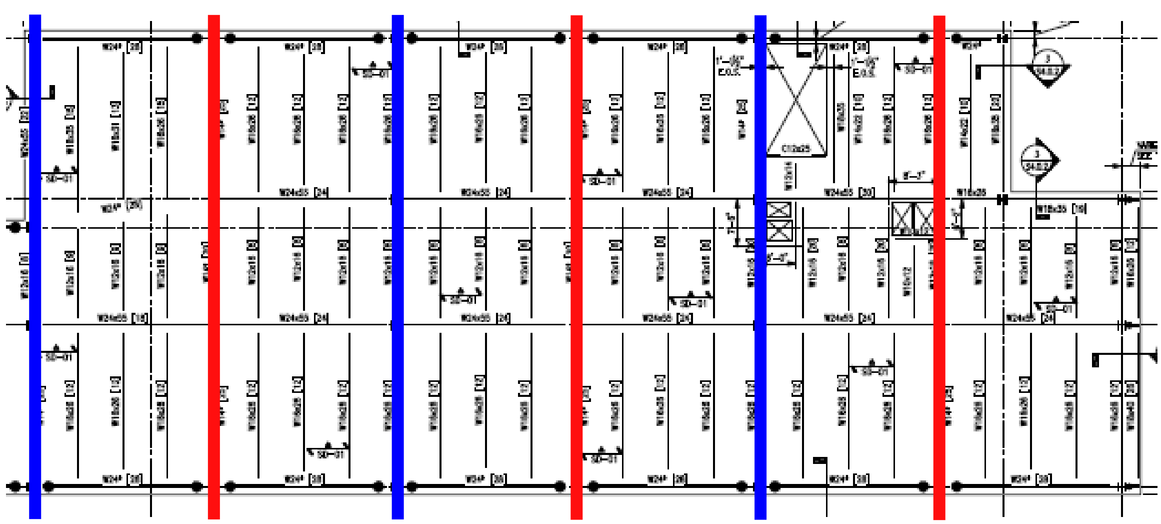


Figure 28 –Alternating Truss Locations

Drawings Courtesy of JCJ Architecture

NE Corner Redesign

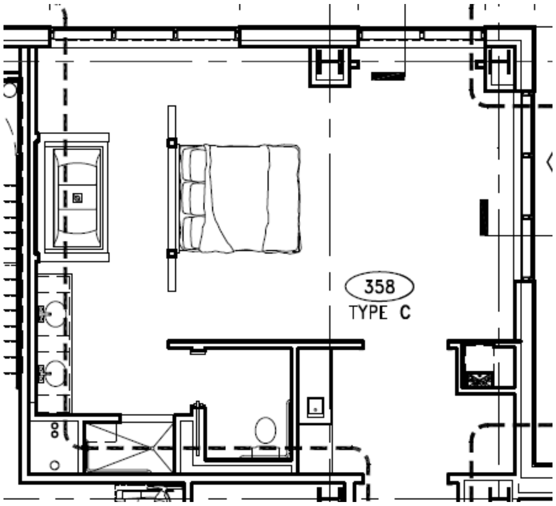


Figure 29 –Truss Obstructing VIP Master Bedroom
Drawing Courtesy of JCJ Architecture

The notched section of the SAC Hotel that the master bedroom is adjacent to needed to be "squared-off" to make room for the truss and to prevent any obstructions within the bedroom. This was accomplished by shifting the master bedroom to the right to match the furthest-extending exterior wall, shown below. The truss is then hidden within the bedrooms wall while maintaining the spacing of 58' between trusses per floor.

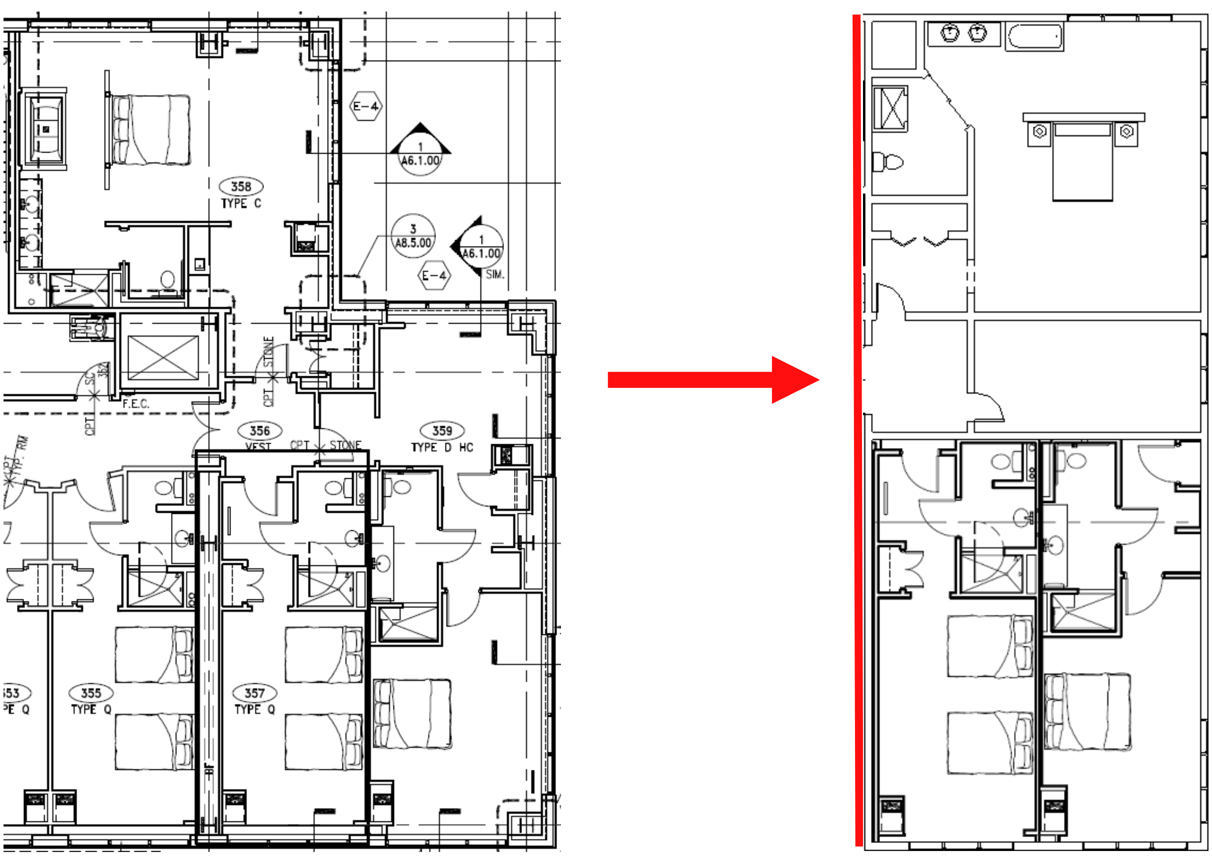
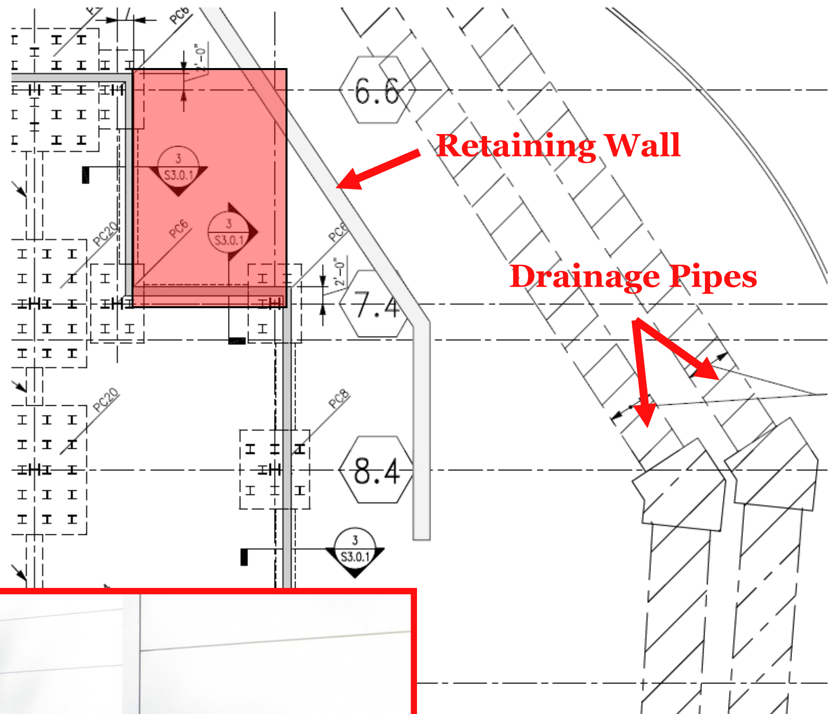


Figure 30 –Truss Hidden Within Wall of Master Bedroom

NE Corner Redesign

The squaring-off of the NE corner of the addition created a conflict with the existing site. During earlier construction phases of the SAC complex, a retaining wall was constructed, along with large drainage pipes to carry water run-off from the hills behind the hotel. The existing SAC Hotel Addition was purposefully designed with this corner notched in order to avoid reconstructing the retaining wall or diverting the drainage pipes.

This spot along the retaining wall also provides service access to the back of the addition, shown below. In order to proceed with the architectural study, it was assumed that this section could be squared off without this conflict.



Other design options to prevent the VIP Suite obstruction were considered, including: cantilevering the squared section over the retaining wall, completely moving the entire VIP Suite, and eliminating hotel rooms. Squaring the entire corner was deemed the best option.

Figure 31 –NE Corner Retaining Wall and Service Access
Photo Courtesy of Jim Boje, PE (Wendel)

VIP Suite

The VIP Suite's location at the far end of the SAC Hotel Addition required that there be no obstructions to the exterior windows. This is because the new addition has an excellent view of the surrounding valley, which can be seen in the photo below. Normally, the staggered truss system would have a truss located at the edge of the building on some floors. This required that the VIP suite be constructed with standard steel framing.

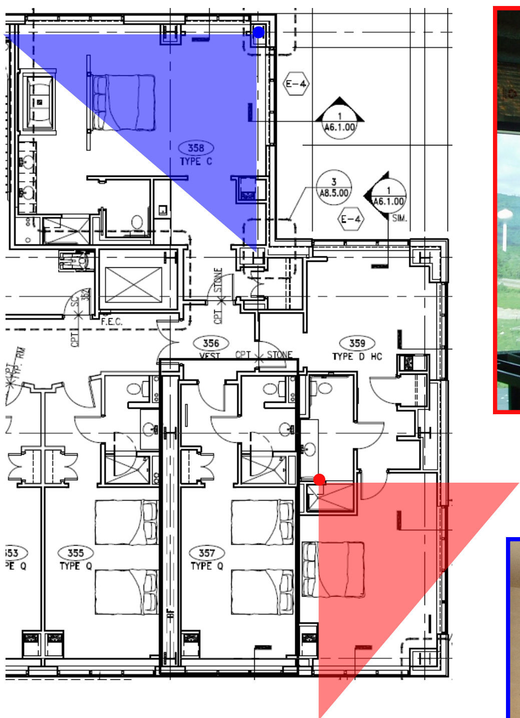
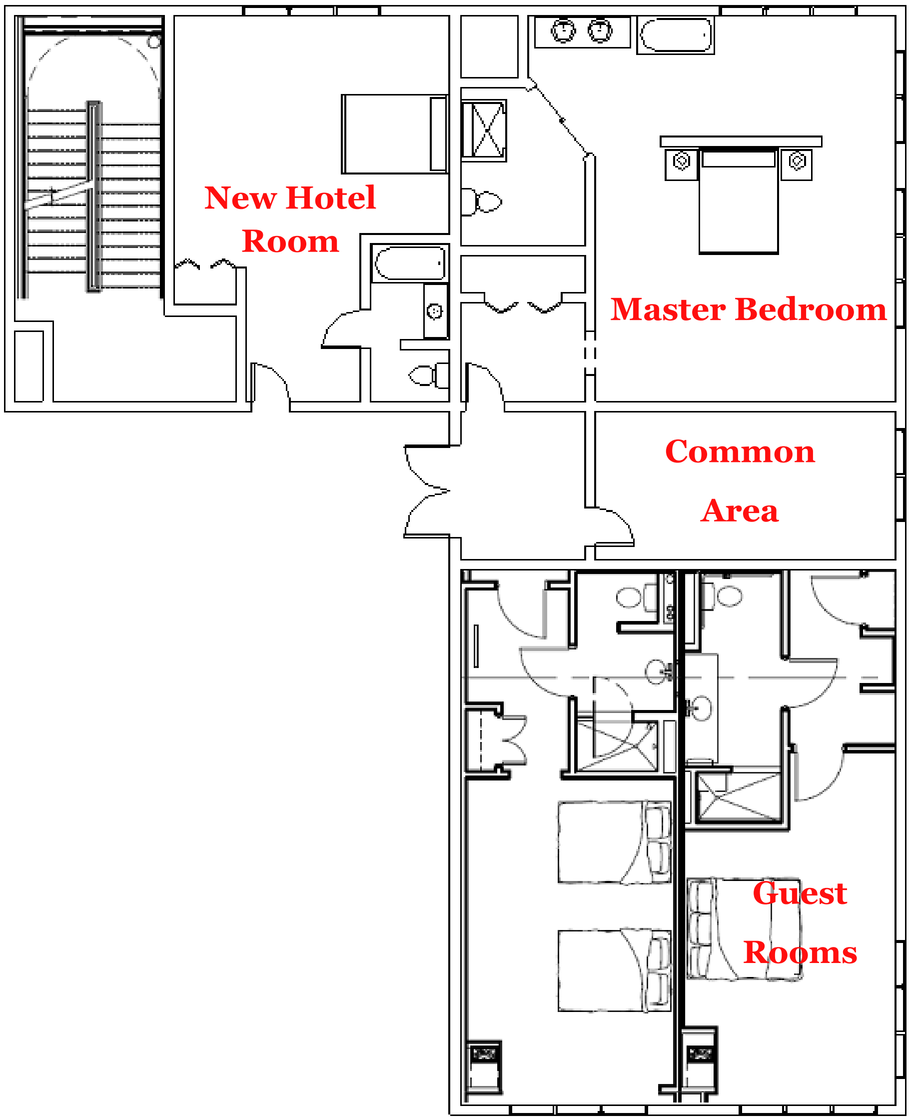


Figure 32 –VIP Suite View of Valley
Photo Courtesy of Jim Boje, PE (Wendel)

Figure 33 –VIP Suite Master Bedroom
Rendering Courtesy of JCJ Architecture

With the shift of the master bedroom shown on the previous page, more floor space was created per floor and a new layout of the entire VIP Suite needed to be created. Shown to the right is a rendering of the existing master bedroom. This specific layout of furniture was recreated as best as possible with each redesign. The following pages detail possible options for the new floor space created by the shift.

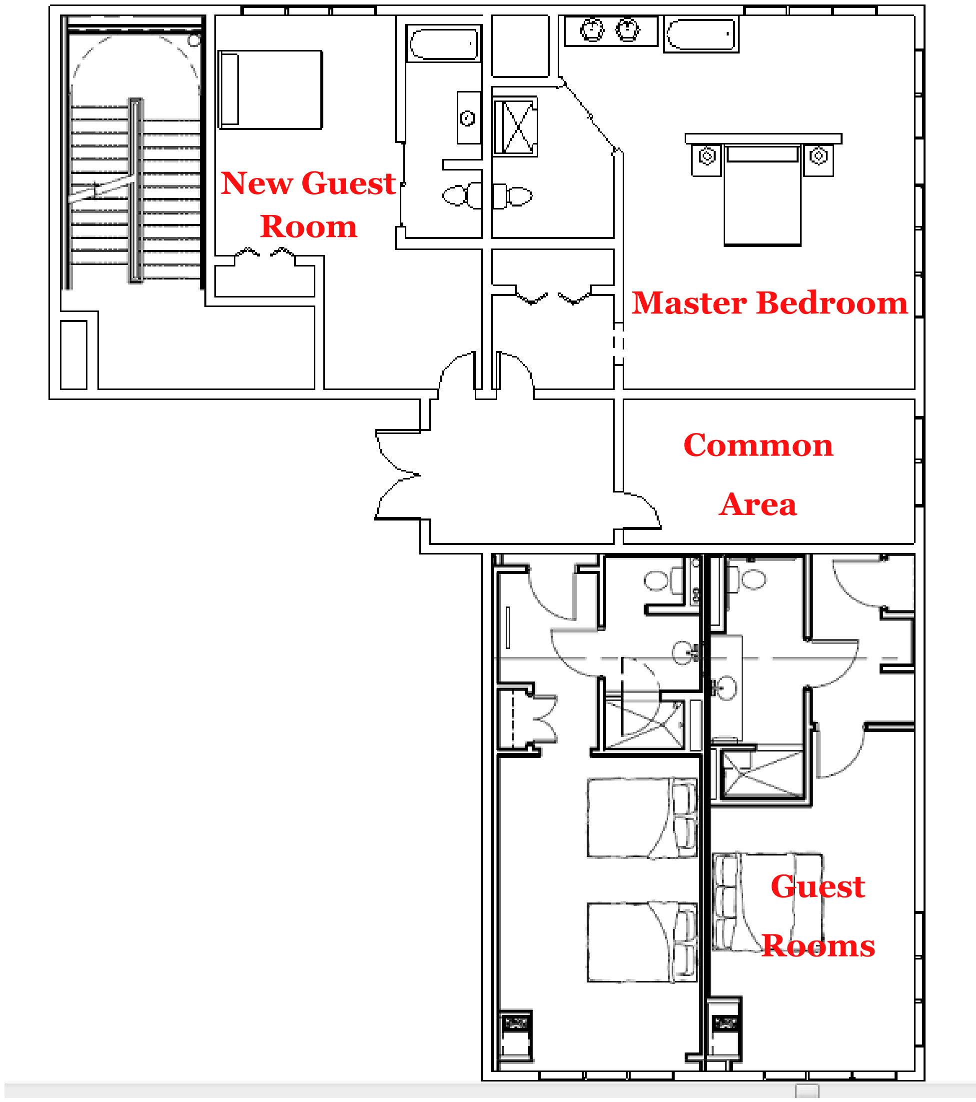
VIP Suite Redesigns



A possible solution for the newly created floor space could be an additional hotel room, separate from the VIP Suite, located right beside the existing stairwell. This would increase the existing amount of 200 hotel rooms to 211.

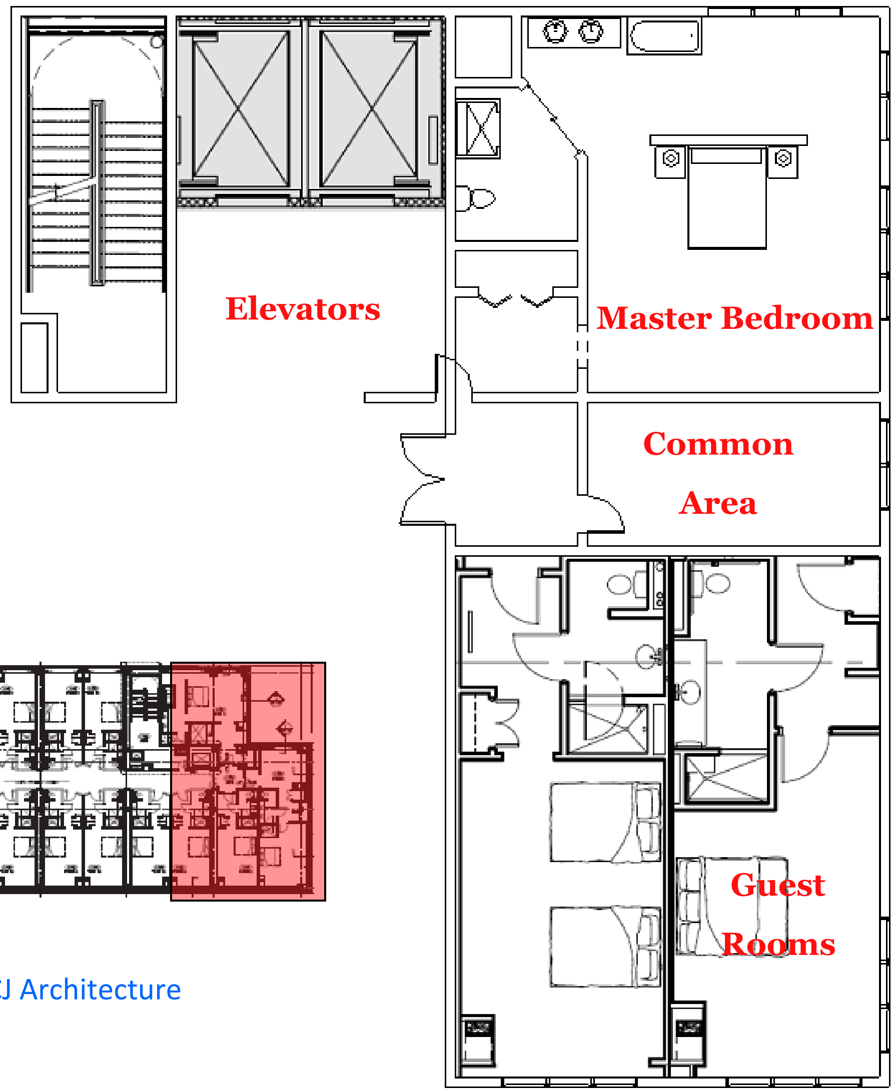
This redesign comes with a few disadvantages, such as the smaller room size. In order to maintain the private vestibule leading into the VIP Suite, the new hotel room's entrance had to be set back, leaving room for only one bed. Also, to keep the master bedroom's layout similar to the existing, and to allow the new bedroom to have an entrance, the plumbing fixtures are not aligned.

VIP Suite Redesigns



Another solution would be to increase the overall size of the suite with another guest room. Although this does not increase the number of rooms for monetary purposes, this does make better use of the new floor space. The fact that it is again only a room with a single bed does not create as much a problem when attached to the suite as a whole. Plumbing fixtures are also more easily serviced with less piping than the previous redesign.

VIP Suite Redesigns



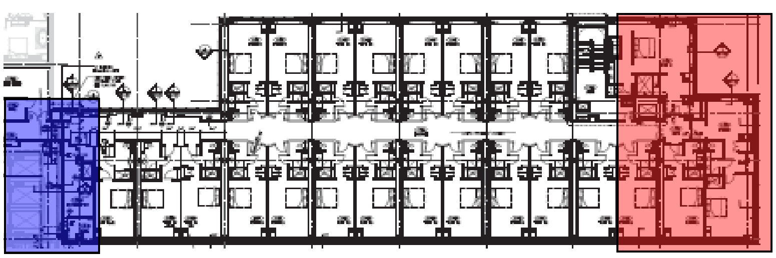


Figure 34
Drawing Courtesy of JCJ Architecture

Figure 34 above shows the VIP Suite in red. In blue, the figure shows the existing location of an elevator core that was built with the original hotel tower. There were no elevators included in the design of the new SAC Hotel Addition. The distance from these existing elevators to the other end of the addition where the VIP Suite is located, is about 230'.

This new floor plan incorporates an extra set of elevators next to the stairwell in order to better service the far end of the addition. The disadvantage here is that the staggered truss system alone would most likely not be able to carry the loads from both the stairwell and elevators, thus more bracing would be required, adding interior columns, and the advantage of the truss system itself would be lost.

Other Truss Conflicts

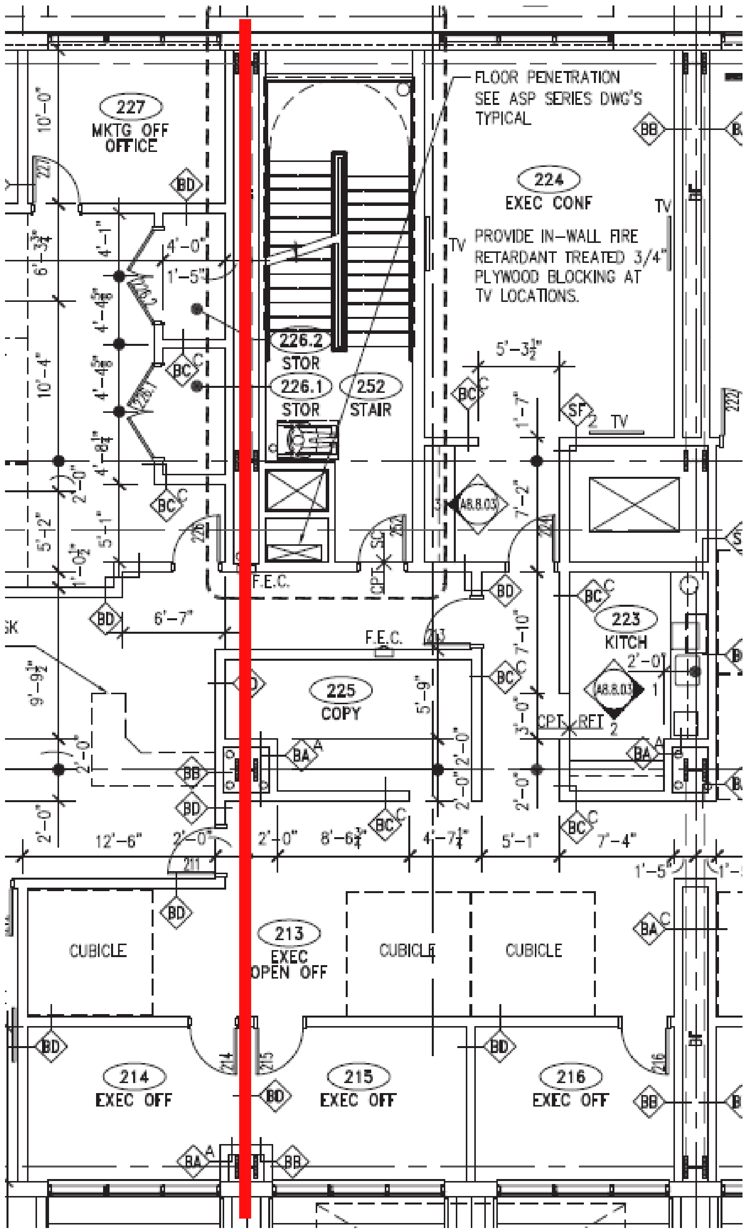


Figure 35 - 2nd Floor Office Conflict

Drawing Courtesy of JCJ Architecture

Two other areas would be affected by truss locations: the 2nd floor office areas and a mechanical room on the mezzanine level. In the figure above, a truss shown by the red line would go directly through an executive office and would also potentially obstruct the corridor leading to the office, depending on the truss' diagonal location. The mechanical room obstruction is show on the following page.

Other Truss Conflicts

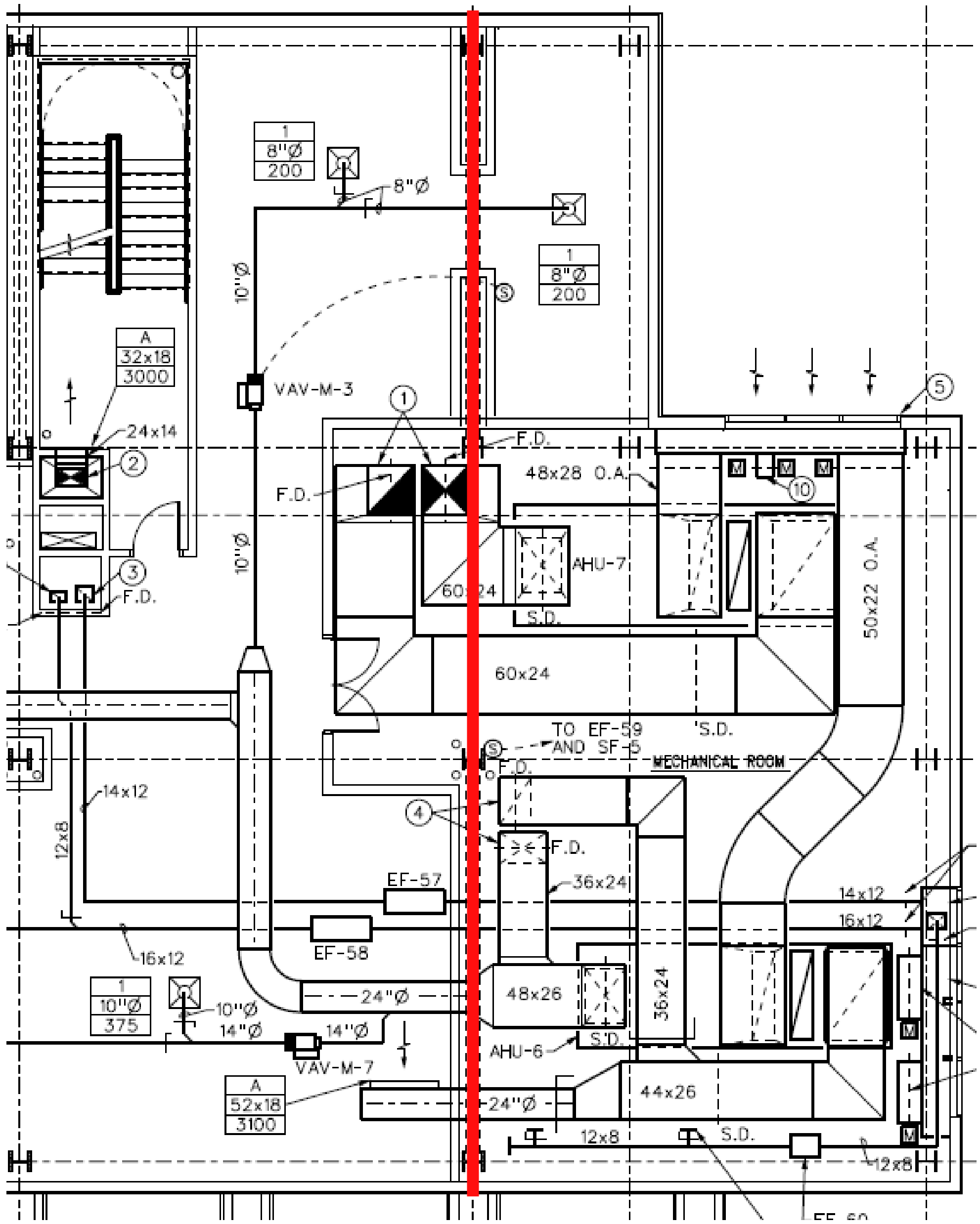


Figure 36 - Mezzanine Mechanical Room Conflict
Drawing Courtesy of JCJ Architecture

Figure 36 shows the location of a mechanical room on the mezzanine level. Shown in red, a truss would fall in the location of an existing air handling unit. On floors above the mezzanine, this truss may also affect duct work service for each floor. Had the SAC Hotel Addition been designed initially with the staggered truss system, this problem would most likely not occur with coordination with the mechanical design engineers.

Posts and Hangers

AISC Design Guide 14 suggests the use of posts and hangers at the topmost and bottommost levels of the building that do not have trusses, shown below. These posts and hangers could also cause obstructions on certain floors since normally there would not be trusses in those locations. Fortunately, the truss locations matched the existing braced frame locations, which were also hidden within the walls of the hotel rooms, thus the posts and hangers did not cause a problem in this redesign.

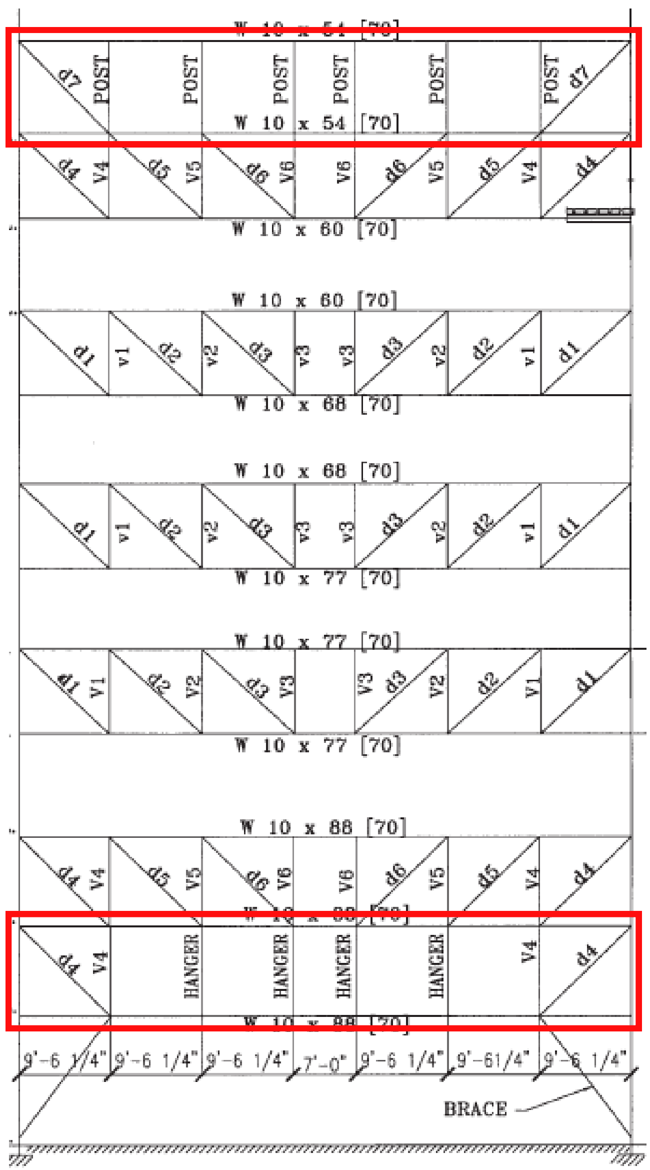


Figure 37 - Example Frame with Posts and Hangers

Drawing From AISC Design Guide 14

Construction Management Study

Background

By constructing the SAC Hotel Addition almost completely with prefabricated members, significant construction time will be saved. Erection of the entire building can be completed by a single crane capable of carrying the heaviest member load, and no time would be needed for the curing of concrete. This breadth will focus on selecting an appropriate crane for erecting the prefabricated members, as well as a new site plan for the coordination of the crane location and delivery routes for trucks carrying members.

Crane Selection

A few factors were included in determining an appropriate size of crane for the erection process: load capacity, jib length, and the load the crane could carry at max jib length. First, the individual member weights needed to be determined. There are 4 different prefabricated members used in this redesign: 2 different truss designs and 2 sizes of precast concrete planks. The tables below explain the calculation of each member's weight. It was determined that the largest truss, (15' in height), weighed the most.

Approximate Truss Member Weights									
Small Truss					Large Truss				
	Member	Weight (plf)	Length (ft)	Weight (lb)		Member	Weight (plf)	Length (ft)	Weight (lb)
Top Chord	W10x33	33	71.5	2359.5	Top Chord	W10x54	54	71.5	3861
Bottom Chord	W10x33	33	71.5	2359.5	Bottom Chord	W10x60	60	71.5	4290
Diagonals (6)	HSS9x7x5/8	59.32	15.62	5559.5	Diagonals (6)	HSS14x10x5/8	93.34	18.5	10361
Verticals (6)	HSS9x7x5/8	59.32	11.33	4032.6	Verticals (6)	HSS14x10x5/8	93.34	15	8401
			∑	**14311**				∑	**26913**

Precast Concrete Planks			
	Weight (plf)	Length (ft)	Weight (lb)
8"	245	29	7105
10"	272	29	7888

Site Plan

The image below shows a satellite view of the existing site for the SAC Hotel Addition. The new addition is highlighted in yellow. It can be seen here that there are multiple roads surrounding the site, which presents both a problem and an advantage. Since there is already a working casino and hotel, it would be undesirable to block any of these roads for a long amount of time. There is also not much space on the site to store or assemble members, thus the roads become an advantage in providing multiple trucking routes to deliver the prefabricated members.

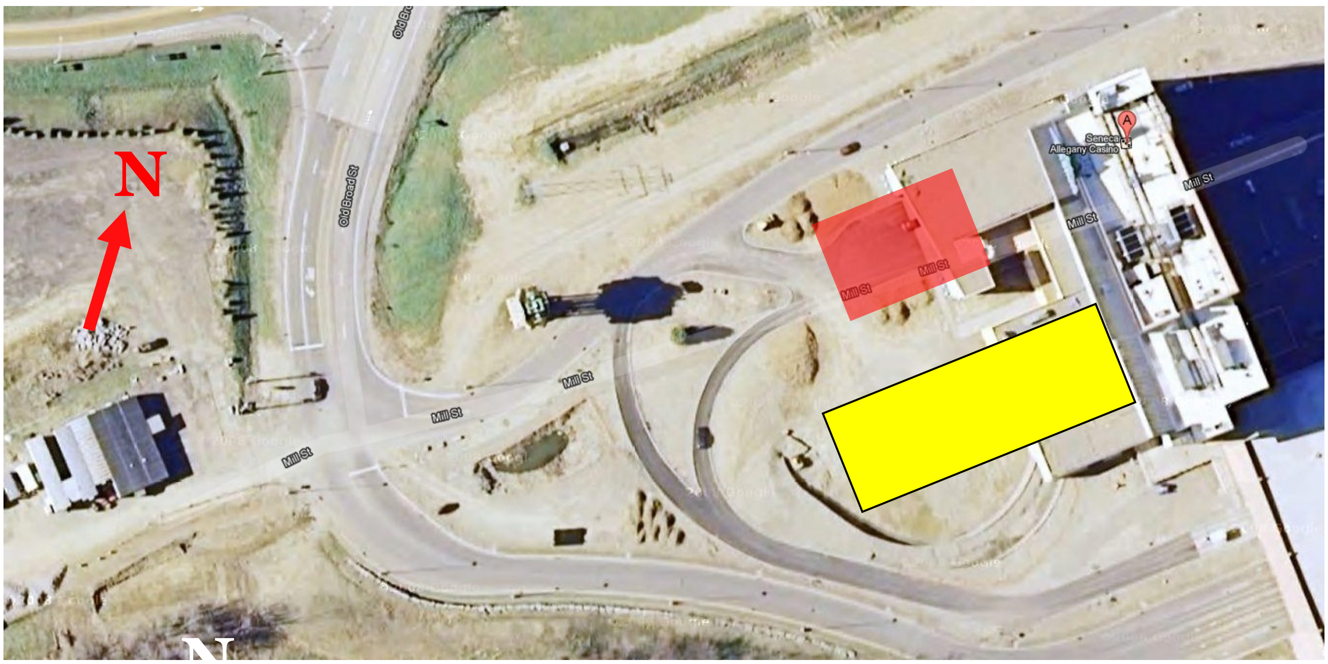
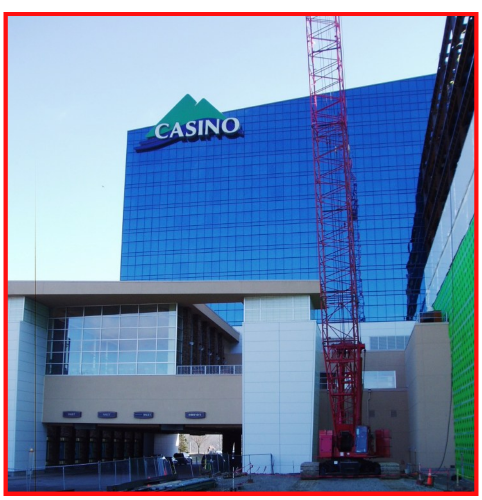


Figure 38 - Satellite Image of SAC Hotel Site
Google Maps



Highlighted in red above is a pick-up and drop-off entryway into the casino and existing hotel. This area provides plenty of room for delivery trucks to access the crane, which will be located along the north face of the new addition.

Figure 39 - Pick-up/Drop-off Casino Entry
Photo Courtesy of Jim Boje, PE (Wendel)

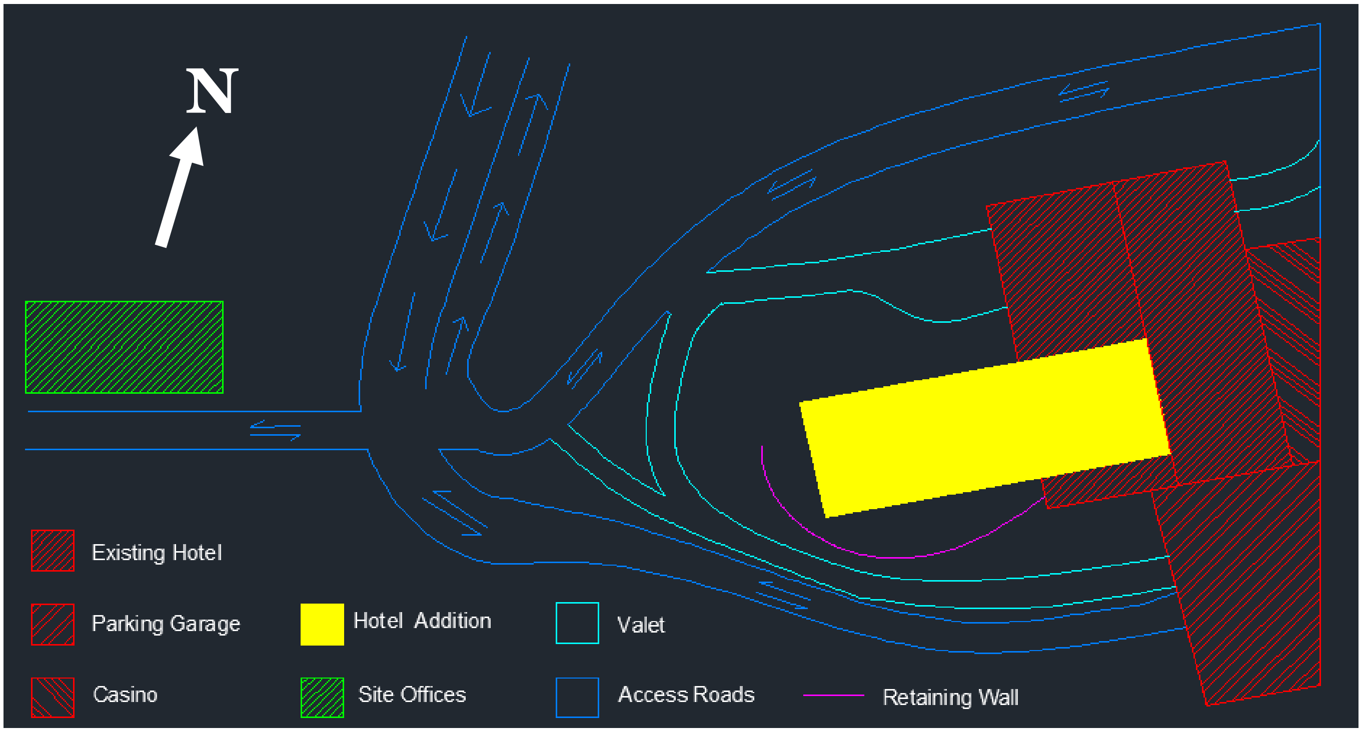
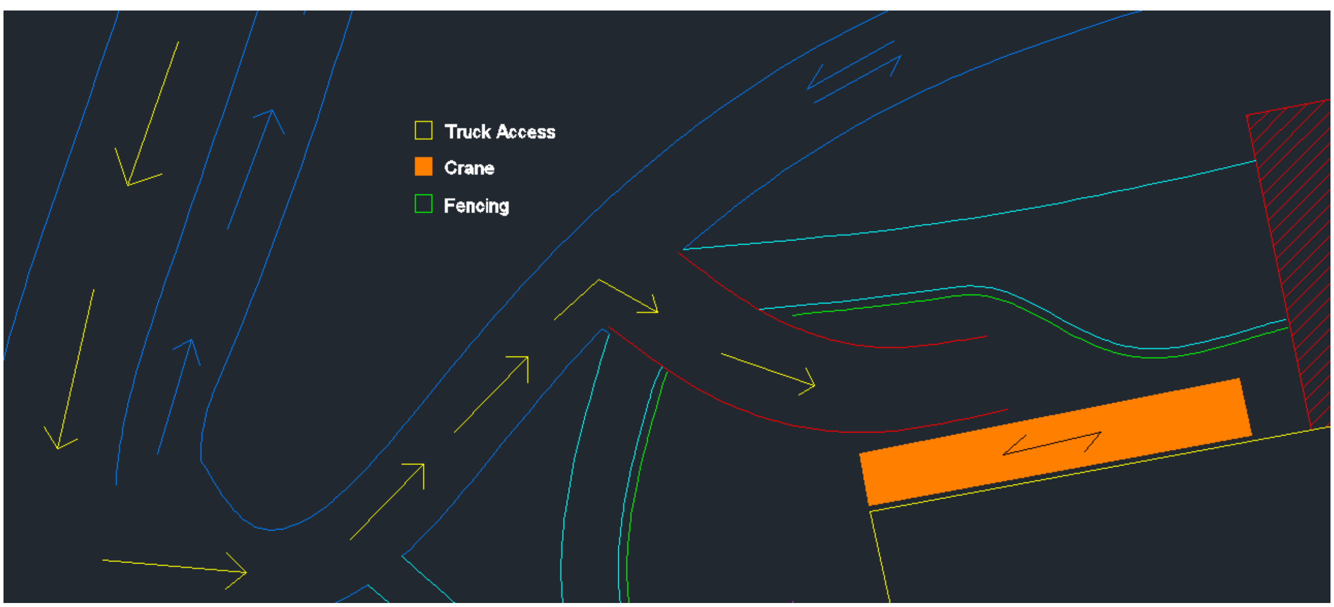


Figure 40 - Existing Site Plan

Figure 40 shows the various roadways more clearly. The SAC Hotel Addition's construction site can be accessed through the valet roadways, with the tower crane located along the north face of the addition (shown in orange in the bottom image). The crane selected for the project is a Comansa 2100. If the crane were to stay stationary at the central point of the addition, the farthest the crane's jib would have to reach would be 109'. The Comansa 2100 is capable of carrying 35,310 pounds at a jib length of 131.2 ft. In this situation, the crane will be able to move along the length of the building, thus it would never reach this max jib length or load.



Conclusion

The SAC Hotel Addition was successfully designed with a staggered truss system working as both the gravity system and lateral system in the N-S direction, replacing the existing braced frames. By replacing the existing concrete on metal deck floor system with precast concrete planks, the erection process of the addition could be completed much more quickly and efficiently without waiting for concrete to cure.

Varying floor heights in the existing addition's design required that two truss sizes be designed: one at a height of 11'4" and the other at a height of 15'. The redesign also required two sizes of precast concrete planks due to certain floor levels having large live load requirements. Both sizes of truss were checked for deflections individually at the code limit of l/360, and as a whole frame with a limit of H/500, and were found to be adequately designed.

New loads from a change in the floor and gravity system impacted the design of the foundation, originally consisting of steel piles. The truss system eliminated the need for interior columns, thus the amount of steel piles needed was reduced.

Hotels are typically designed with a repetitive floor plan in mind, which allowed for the use of the truss system. However, the specific geometry and room layouts of the SAC Hotel Addition conflicted with a few truss locations, requiring a look into the architectural impact of the re-design. To avoid having any trusses exposed, a corner of the addition was "squared-off" in order to create a uniform edge and to allow the shift of the VIP Suite's master bedroom. This created more available square footage per floor and options for that extra space were explored.

The new erection process required a look into a new site plan, as well as the selection of a different tower crane to carry the load of prefabricated members. The largest truss designed was found to be the heaviest member and controlled the selection of the crane. After the crane was selected, the flow of traffic during construction was examined. It was found that delivery trucks could access the construction site easily through existing roadways, and the crane could be situated along the north face of the addition during the entire process.

Appendices

Nick Reed | Final Report | Hollow Core Planks Capacity Checks

SDL = 15 psf
LL = 80 psf
RLL = 38.6 psf

w/ 8", 6-1/2"ϕ strands, 2" top
DL = 86.25 psf
Planks 4' wide

$1.2(101.25) + 1.6(38.6) = 183.3$ psf

$183.3 \times 4' = 733$ plf $= .733$ klf

$M = \frac{wl^2}{8} = \frac{.733(20)^2}{8} = 72.1$ ft·k < 130.6 ft·k (see Nitterhouse specs)

OK for hotel room floors

Corridor LL = 100 psf

$1.2(101.25) + 1.6(100) = 281.5$ psf $281.5(4') = 1126$ plf

$M = \frac{1.13(20)^2}{8} = 118.8$ ft·k < 130.6 ft·k

OK for corridor

$\boxed{\text{Use 8", 6-1/2" strand, 2" top for 4th-11th Floors}}$

Roof, Mezz, 3rd, 2nd Floors

SDL = 15 psf
LL = 200 psf

10", 7-1/2"ϕ, 2" top planks
DL = 93 psf

$1.2(108) + 1.6(200) = 450$ psf $450(4') = 1800$ plf

$M = \frac{1.8(20)^2}{8} = 189$ ft·k < 191.7 ft·k

OK

$\boxed{\text{Use 10", 7-1/2"}\phi\text{, 2" top for all other floors}}$

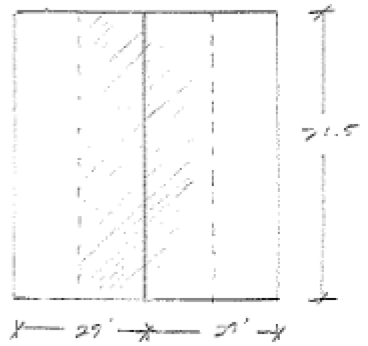
Nick Reed Final Report Truss Member Loads: (4th - 10th Floor) 1/2

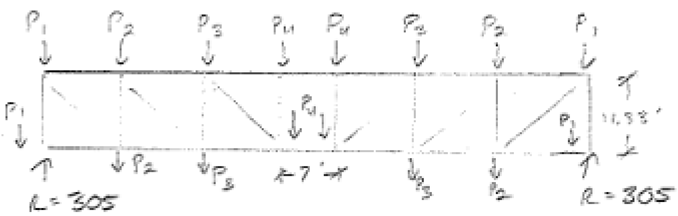
UNFACTORED

LL
100 psf (corridor)
80 psf (rooms per spec)

DL
MEP = 15 psf
Planks = 86.25 psf (8", 2" top, 6-1/2Ø)

Trib Area = (29'×71.5) = 2073.5 ft²



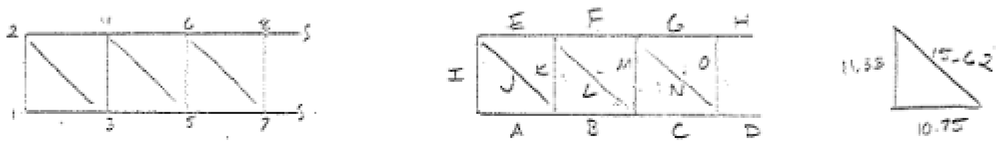


$w = (101+100) \times 29' = 5829\ plf = 5.83\ klf$

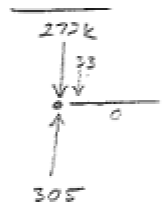
$P_1 = 5.83(11.33'/2) = 33k$

$P_2 = P_3 = 5.83(11.33) = 66k$

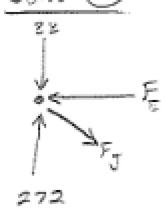
$P_4 = 5.83(11.33+7')/2 = 53.4k$



Joint ①



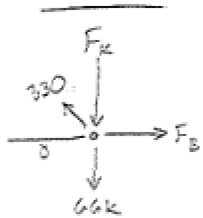
Joint ②



$$F_J = \frac{(272-33)(15.62)}{11.33} = 329.5\ k$$

$$F_E = \frac{(330)(10.75)}{15.62} = 227k$$

Joint ③



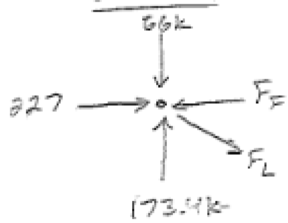
$$F_B = \frac{(330)(10.75)}{15.62} = 227k$$

$$F_K = \frac{(330)(11.33)}{15.62} - 66 = 173.4$$

Nick Reed Final Report Truss Member Loads (4th-10th Floor) ½

UNFACTORED

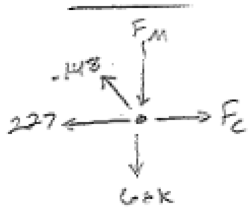
Joint 4



$$F_L = \frac{(173.4-66)(15.62)}{11.33} = 148\ k$$

$$F_F = \frac{(148)(10.75)}{15.62} + 227 = 328.9k$$

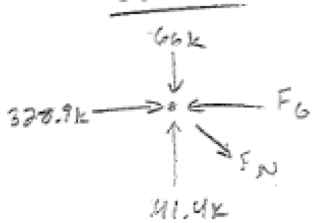
Joint 5



$$F_M = \frac{148(11.33)}{15.62} - 66k = 41.4k$$

$$F_C = \frac{148(10.75)}{15.62} + 227 = 328.9k$$

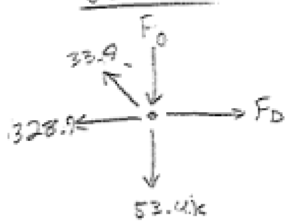
Joint 6



$$F_N = \frac{(66-41.4)(15.62)}{11.33} = 33.9k$$

$$F_G = \frac{(33.9)(10.75)}{15.62} + 328.9 = 352k$$

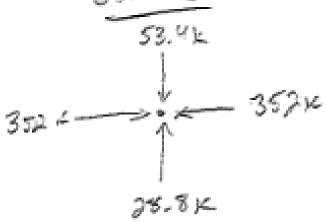
Joint 7



$$F_O = \frac{33.9(11.33)}{15.62} - 53.4 = 28.8k$$

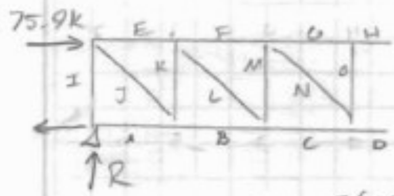
$$F_D = \frac{33.9(11.33)}{15.62} + 328.9 = 352k$$

Joint 8



Nick Reed | Final Report | Truss Member Loads (4th - 10th Floor) | 1/2

Lateral (Unfactored)



Wind Load on 4th story from Tech 3 = 75.9 k

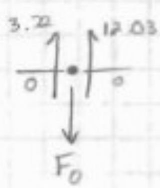
$$R = \frac{2(75.9)(11.33)}{71.5} = 24.1\text{k}$$

$$V_{@\,midspan} = \frac{\frac{1}{2}(75.9)(11.33)}{35.75} = 12.03\text{k}$$

$$M_{@\,joint\ 8} = 12.03\left(\tfrac{7}{2}\right) = 42.1\ \text{ft·k}$$

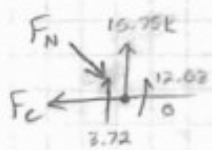
$$V_{@\,left\ of\ 8} = \frac{42.1}{11.33} = 3.72\text{k}$$

Joint 8



$F_O = 15.75\text{k}$ C

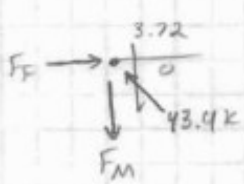
Joint 7



$$F_N = \frac{(15.75 + 12.03 + 3.72)(15.62)}{11.33} = 43.4\text{k}\ \ T$$

$$F_C = \frac{(15.75 + 12.03 + 3.72)(10.75)}{11.33} = 29.9\ \ C$$

Joint 6

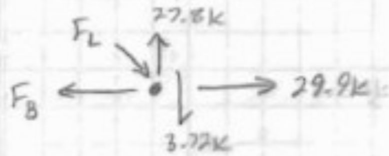


$$F_F = \frac{43.4(10.75)}{15.62} = 29.9\text{k}\ \ T$$

$$F_M = \frac{43.4(11.33)}{15.62} - 3.72 = 27.8\text{k}\ \ C$$

Nick Reed | Final Report | Truss Member Loads (4th - 10th Floor) Lateral (Unfactored)

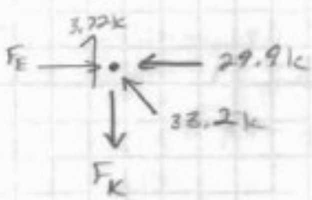
Joint 5



$$F_L = \frac{(27.8 - 3.72)(15.62)}{11.33} = 33.2 \text{ k } T$$

$$F_B = \frac{33.2\,(10.75)}{15.62} = 22.8 \text{ k } C$$

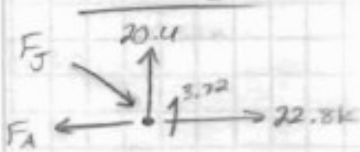
Joint 4



$$F_E = \frac{33.2\,(10.75)}{15.62} = 22.8 \text{ k } T$$

$$F_K = \frac{33.2\,(11.33)}{15.62} - 3.72 = 20.4 \text{ k } C$$

Joint 3



$$F_J = \frac{(20.4 + 3.72)(15.62)}{11.33} = 33.2 \text{ k } T$$

$$F_A = \frac{33.2\,(10.75)}{15.62} = 22.8 \text{ k } C$$

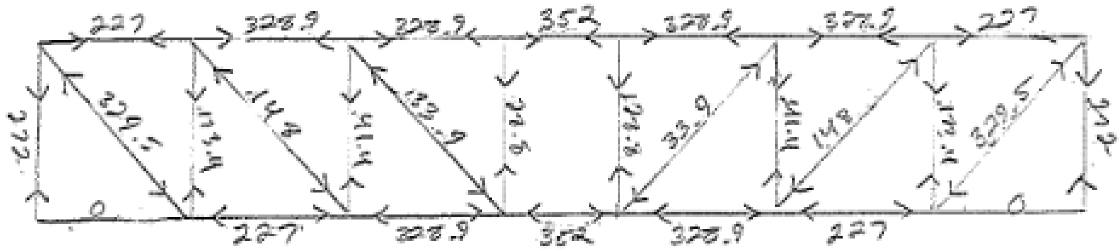
Joint 2

$$F_I = \frac{33.2\,(11.33)}{15.62} = 24.1 \text{ k } C$$

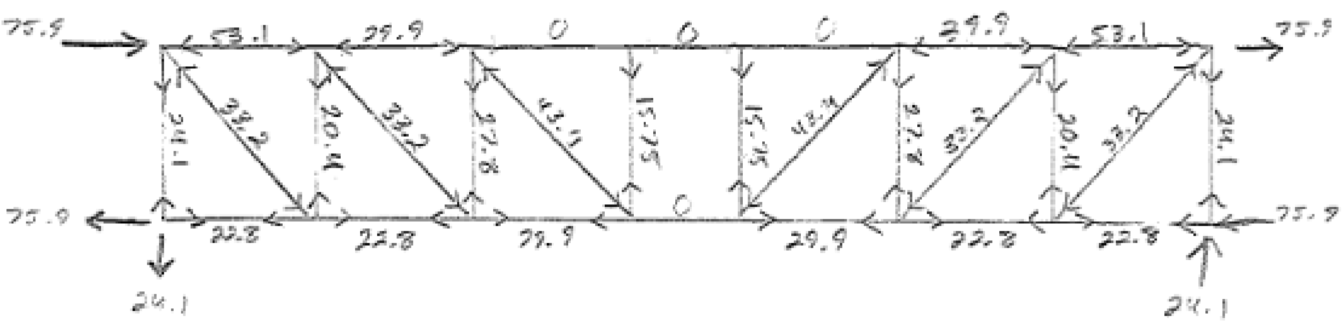
$$F_E = 75.9 - \frac{(33.2)(10.75)}{15.62} = 53.1 \text{ } T$$

Nick Reed Final Report Member Forces (4th - 10th)

Gravity Unfactored (kips)



Lateral Unfactored (kips)



Member Loads 4th-10th Floors

Load Combinations (kips)							Unfactored Forces (kips)			
Member	1.4D	**1.2D+1.6L**	1.2D+1.6W+L	1.2D+0.8W	0.9D+1.6W			Member	D+L	Lateral (Wind)
1	0	**0**	36.5	18.2	36.5	W10x33	BOT	1	0	22.8
2	159.7	**317.6**	286.3	155.1	139.1	W10x33	BOT	2	227	22.8
3	231.4	**460.1**	409.8	222.2	196.6	W10x33	BOT	3	328.9	29.9
4	247.6	**492.4**	387.4	212.3	159.2	W10x33	BOT	4	352	0
5	231.4	**460.1**	409.8	222.2	196.6	W10x33	BOT	5	328.9	29.9
6	159.7	**317.6**	286.3	155.1	139.1	W10x33	BOT	6	227	22.8
7	0	**0**	36.5	18.2	36.5	W10x33	BOT	7	0	22.8
8	159.7	317.6	**334.8**	179.4	187.6	W10x33	TOP	8	227	53.1
9	231.4	**460.1**	409.8	222.2	196.6	W10x33	TOP	9	328.9	29.9
10	231.4	**460.1**	362.0	198.3	148.7	W10x33	TOP	10	328.9	0
11	247.6	**492.4**	387.4	212.3	159.2	W10x33	TOP	11	352	0
12	231.4	**460.1**	362.0	198.3	148.7	W10x33	TOP	12	328.9	0
13	231.4	**460.1**	409.8	222.2	196.6	W10x33	TOP	13	328.9	29.9
14	159.7	317.6	**334.8**	179.4	187.6	W10x33	TOP	14	227	53.1
15	191.3	**380.5**	337.9	183.3	161.6	W12x79	COL	15	272	24.1
16	231.8	**461.0**	415.7	225.2	202.1	HSS9X7X5/8	DIAG	16	329.5	33.2
17	122.0	**242.6**	223.5	120.9	111.1	HSS9X7X5/8	VERT	17	173.4	20.4
18	104.1	**207.1**	216.0	115.8	120.1	HSS9X7X5/8	DIAG	18	148	33.2
19	29.1	**57.9**	90.0	47.2	63.2	HSS9X7X5/8	VERT	19	41.4	27.8
20	23.8	**47.4**	106.7	55.2	84.8	HSS9X7X5/8	DIAG	20	33.9	43.4
21	20.3	**40.3**	56.9	30.0	38.2	HSS9X7X5/8	VERT	21	28.8	15.75
22	20.3	**40.3**	56.9	30.0	38.2	HSS9X7X5/8	VERT	22	28.8	15.75
23	23.8	**47.4**	106.7	55.2	84.8	HSS9X7X5/8	DIAG	23	33.9	43.4
24	29.1	**57.9**	90.0	47.2	63.2	HSS9X7X5/8	VERT	24	41.4	27.8
25	104.1	**207.1**	216.0	115.8	120.1	HSS9X7X5/8	DIAG	25	148	33.2
26	122.0	**242.6**	223.5	120.9	111.1	HSS9X7X5/8	VERT	26	173.4	20.4
27	231.8	**461.0**	415.7	225.2	202.1	HSS9X7X5/8	DIAG	27	329.5	33.2
28	191.3	**380.5**	337.9	183.3	161.6	W12x79	COL	28	272	24.1

Loads (psf)	
Dead	101
Live	100

Load Coefficients	
ϕ1	0.703
ϕ2	1.40
ϕ3	1.10
ϕ4	0.603
ϕ5	0.452

Tension
Compression

Nick Reed | Final Report | Truss Member Loads Mezz, 11th Floor | 1/2

UNFACTORED

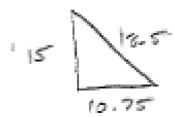
Roof LL, Mezz LL

200 psf (per spec)

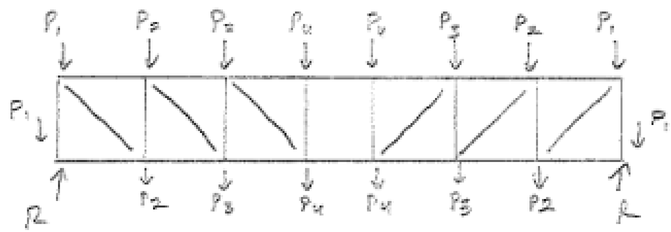
DL

MEP = 15 psf

Planks = 86.25 psf



Trib width = 29'



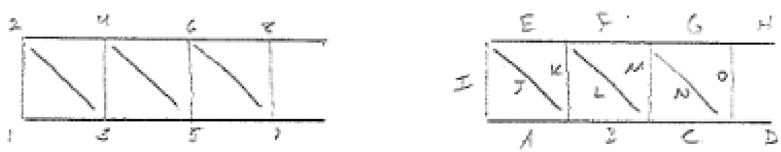
$w = (101 + 200) \times 29' = 8.73$ k/LF

$P_1 = 8.73(15/2) = 65.5$k

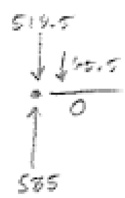
$P_2 = P_3 = 8.73(15) = 131$ k

R = 525k

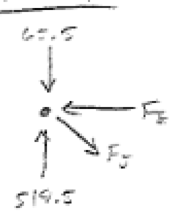
$P_4 = 8.73(15+7)/2 = 96$k



Joint ①



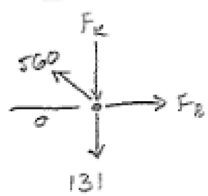
Joint ②



$F_J = \frac{(519.5-65)(18.5)}{15} = 560$k

$F_E = \frac{(560)(10.75)}{18.5} = 325$ k

Joint ③

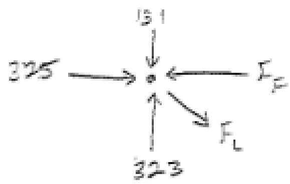


$F_B = \frac{(560)(10.75)}{18.5} = 325$k

$F_K = \frac{(560)(15)}{18.5} - 131 = 323$k

Nick Reed   Final Report   Truss Member Loads
Mezz, 11th Floor   2/2
UNFACTORED

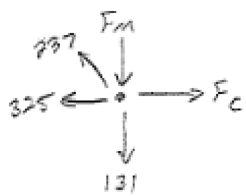
Joint ④



$$F_L = \frac{(323-131)(18.5)}{15} = 237\text{k}$$

$$F_F = \frac{(237)(10.75)}{18.5} + 325 = 463\text{k}$$

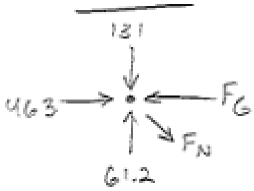
Joint ⑤



$$F_M = \frac{237(15)}{18.5} - 131 = 61.2\text{k}$$

$$F_C = \frac{237(10.75)}{18.5} + 325 = 463\text{k}$$

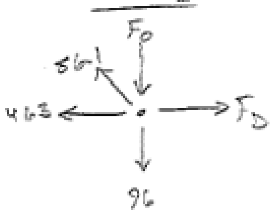
Joint ⑥



$$F_N = \frac{(131-61.2)(18.5)}{15} = 86.1\text{k}$$

$$F_G = \frac{(86.1)(10.75)}{18.5} + 463 = 513\text{k}$$

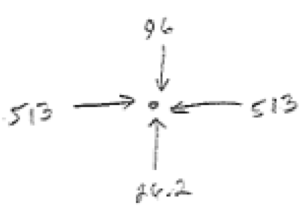
Joint ⑦



$$F_O = \frac{(86.1)(15)}{18.5} - 96 = 26.2\text{k}$$

$$F_D = \frac{(86.1)(10.75)}{18.5} = 513\text{k}$$

Joint ⑧

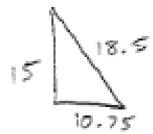


Nick Reed   Final Report   Truss Member Loads 2nd, 3rd   1/2

UNFACTORED

LL
125 psf (per spec)

DL
101 psf



Trib width = 29'

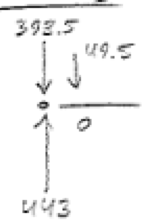
$w = (101 + 125) \times 29' = 6.6 \text{ klf}$

$P_1 = 6.6(15/2) = 49.5k$ $R = 443k$

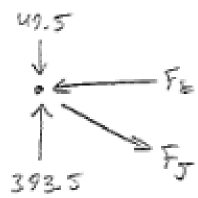
$P_2 = P_3 = 6.6(15) = 99k$

$P_4 = 6.6(15+7)/2 = 73k$

Joint ①



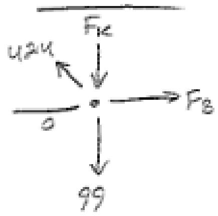
Joint ②



$F_J = \frac{(393.5 - 49.5)(18.5)}{15} = 424k$

$F_E = \frac{(424)(10.75)}{18.5} = 246k$

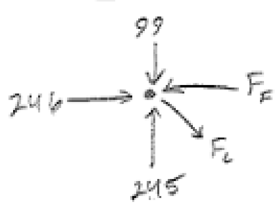
Joint ③



$F_B = \frac{(424)(10.75)}{18.5} = 246k$

$F_K = \frac{(424)(15)}{18.5} - 99 = 245k$

Joint ④



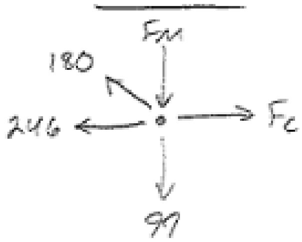
$F_L = \frac{(245 - 99)(18.5)}{15} = 180k$

$F_F = \frac{(180)(10.75)}{18.5} + 246 = 351k$

Nick Reed   Final Report   Truss Member Loads 2nd, 3rd   2/2

UNFACTORED

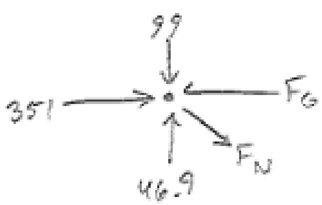
Joint 5



$$F_M = \frac{180(15)}{18.5} - 99 = 46.9K$$

$$F_C = \frac{180(10.75)}{18.5} + 246 = 351K$$

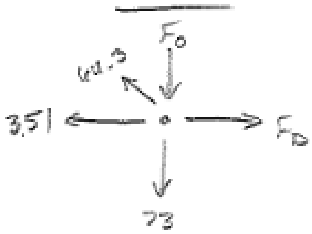
Joint 6



$$F_N = \frac{(99-46.9)(18.5)}{15} = 64.3k$$

$$F_G = \frac{(64.3)(10.75)}{18.5} + 351 = 388K$$

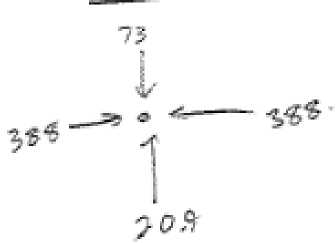
Joint 7



$$F_O = \frac{(64.3)(15)}{18.5} - 73 = 20.9K$$

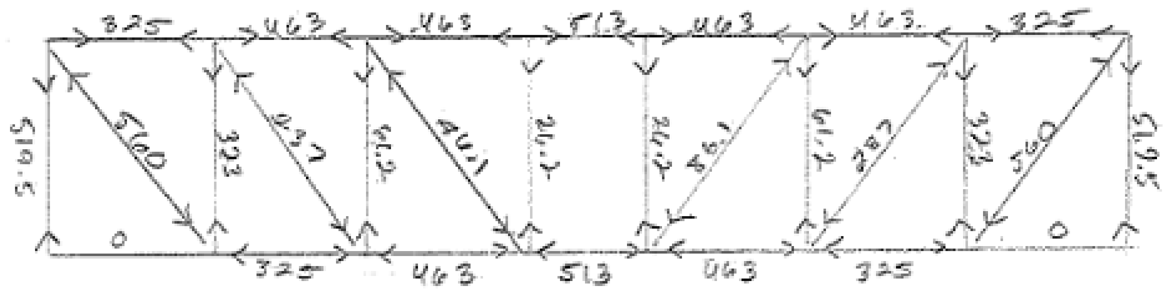
$$F_D = \frac{(64.3)(10.75)}{18.5} + 351 = 388K$$

Joint 8

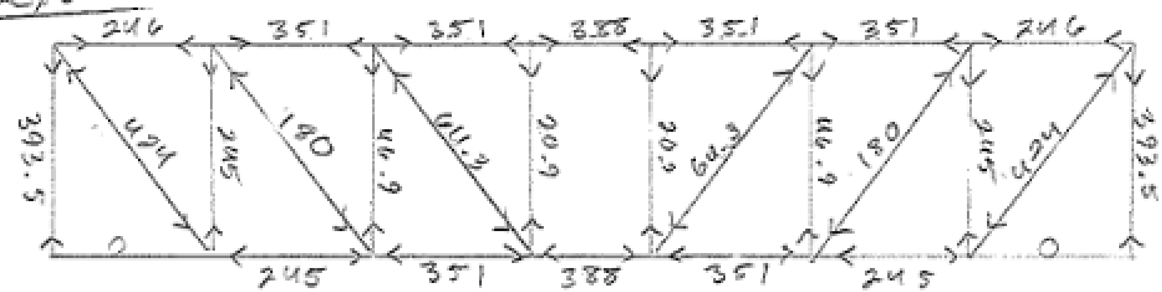


Nick Reed Final Report Member Loads

Mezz, 11th Floor (kips)



2nd, 3rd



Member Loads Mezzanine, 11th Floor

Load Combinations (kips)							Unfactored Forces (kips)			
Member	1.4D	1.2D+1.6L	1.2D+1.6W+L	1.2D+0.8W	0.9D+1.6W			Member	D+L	Lateral (Wind)
1	0	0	36.5	18.2	36.5	W10x60	BOT	1	0	22.8
2	159.5	474.4	278.8	155.0	139.0	W10x60	BOT	2	325	22.8
3	227.3	675.9	393.0	218.7	194.0	W10x60	BOT	3	463	29.9
4	251.8	748.8	382.4	215.9	161.9	W10x60	BOT	4	513	0
5	227.3	675.9	393.0	218.7	194.0	W10x60	BOT	5	463	29.9
6	159.5	474.4	278.8	155.0	139.0	W10x60	BOT	6	325	22.8
7	0	0	36.5	18.2	36.5	W10x60	BOT	7	0	22.8
8	159.5	474.4	327.2	179.2	187.5	W10x54	TOP	8	325	53.1
9	227.3	675.9	393.0	218.7	194.0	W10x54	TOP	9	463	29.9
10	227.3	675.9	345.1	194.8	146.1	W10x54	TOP	10	463	0
11	251.8	748.8	382.4	215.9	161.9	W10x54	TOP	11	513	0
12	227.3	675.9	345.1	194.8	146.1	W10x54	TOP	12	463	0
13	227.3	675.9	393.0	218.7	194.0	W10x54	TOP	13	463	29.9
14	159.5	474.4	327.2	179.2	187.5	W10x54	TOP	14	325	53.1
15	255.0	758.3	425.8	237.9	202.5	W12x79	COL	15	519.5	24.1
16	274.9	817.5	470.6	262.2	229.8	HSS14x10x5/8	DIAG	16	560	33.2
17	158.6	471.5	273.4	152.2	134.6	HSS14x10x5/8	VERT	17	323	20.4
18	116.3	346.0	229.8	126.3	127.9	HSS14x10x5/8	DIAG	18	237	33.2
19	30.0	89.3	90.1	48.0	63.8	HSS14x10x5/8	VERT	19	61.2	27.8
20	42.3	125.7	133.6	70.9	96.6	HSS14x10x5/8	DIAG	20	86.1	43.4
21	12.9	38.2	44.7	23.6	33.5	HSS14x10x5/8	VERT	21	26.2	15.75
22	12.9	38.2	44.7	23.6	33.5	HSS14x10x5/8	VERT	22	26.2	15.75
23	42.3	125.7	133.6	70.9	96.6	HSS14x10x5/8	DIAG	23	86.1	43.4
24	30.0	89.3	90.1	48.0	63.8	HSS14x10x5/8	VERT	24	61.2	27.8
25	116.3	346.0	229.8	126.3	127.9	HSS14x10x5/8	DIAG	25	237	33.2
26	158.6	471.5	273.4	152.2	134.6	HSS14x10x5/8	VERT	26	323	20.4
27	274.9	817.5	470.6	262.2	229.8	HSS14x10x5/8	DIAG	27	560	33.2
28	255.0	758.3	425.8	237.9	202.5	W12x79	COL	28	519.5	24.1

Loads (psf)	
Dead	108
Live	200

Load Coefficients	
ϕ1	0.491
ϕ2	1.46
ϕ3	0.75
ϕ4	0.421
ϕ5	0.316

Tension	
Compression	

Member Loads 2nd, 3rd Floor

Load Combinations (kips)							Unfactored Forces (kips)			
Member	1.4D	1.2D+1.6L	1.2D+1.6W+L	1.2D+0.8W	0.9D+1.6W			Member	D+L	Lateral (Wind)
1	0	**0**	36.5	18.2	36.5	W10x60	BOT	1	0	22.8
2	159.0	**346.6**	277.9	154.5	138.7	W10x60	BOT	2	245	22.8
3	227.8	**496.5**	393.7	219.2	194.3	W10x60	BOT	3	351	29.9
4	251.8	**548.9**	382.3	215.8	161.9	W10x60	BOT	4	388	0
5	227.8	**496.5**	393.7	219.2	194.3	W10x60	BOT	5	351	29.9
6	159.0	**346.6**	277.9	154.5	138.7	W10x60	BOT	6	245	22.8
7	0	**0**	36.5	18.2	36.5	W10x60	BOT	7	0	22.8
8	159.6	**348.0**	327.4	179.3	187.6	W10x54	TOP	8	246	53.1
9	227.8	**496.5**	393.7	219.2	194.3	W10x54	TOP	9	351	29.9
10	227.8	**496.5**	345.9	195.2	146.4	W10x54	TOP	10	351	0
11	251.8	**548.9**	382.3	215.8	161.9	W10x54	TOP	11	388	0
12	227.8	**496.5**	345.9	195.2	146.4	W10x54	TOP	12	351	0
13	227.8	**496.5**	393.7	219.2	194.3	W10x54	TOP	13	351	29.9
14	159.6	**348.0**	327.4	179.3	187.6	W10x54	TOP	14	246	53.1
15	255.4	**556.6**	426.3	238.2	202.7	W12x79	COL	15	393.5	24.1
16	275.1	**599.8**	470.9	262.4	230.0	HSS16x12x5/8	DIAG	16	424	33.2
17	159.0	**346.6**	274.1	152.6	134.8	HSS16x12x5/8	VERT	17	245	20.4
18	116.8	**254.6**	230.5	126.7	128.2	HSS16x12x5/8	DIAG	18	180	33.2
19	30.4	**66.3**	90.7	48.3	64.0	HSS16x12x5/8	VERT	19	46.9	27.8
20	41.7	**91.0**	132.8	70.5	96.3	HSS16x12x5/8	DIAG	20	64.3	43.4
21	13.6	**29.6**	45.8	24.2	33.9	HSS16x12x5/8	VERT	21	20.9	15.75
22	13.6	**29.6**	45.8	24.2	33.9	HSS16x12x5/8	VERT	22	20.9	15.75
23	41.7	**91.0**	132.8	70.5	96.3	HSS16x12x5/8	DIAG	23	64.3	43.4
24	30.4	**66.3**	90.7	48.3	64.0	HSS16x12x5/8	VERT	24	46.9	27.8
25	116.8	**254.6**	230.5	126.7	128.2	HSS16x12x5/8	DIAG	25	180	33.2
26	159.0	**346.6**	274.1	152.6	134.8	HSS16x12x5/8	VERT	26	245	20.4
27	275.1	**599.8**	470.9	262.4	230.0	HSS16x12x5/8	DIAG	27	424	33.2
28	256.7	**559.5**	428.3	239.3	203.5	W12x79	COL	28	395.5	24.1

Loads (psf)	
Dead	108
Live	125

Load Coefficients	
$\phi 1$	0.649
$\phi 2$	1.41
$\phi 3$	0.99
$\phi 4$	0.556
$\phi 5$	0.417

Tension
Compression

Nick Reed | Final Report | Column Loads

Sample calc

Columns on ends of truss

Trib Area = $29' \times \frac{71.5}{2} = 1036.8\ ft^2$

$1.2D + 1.6L + 0.5\ psf$

Roof

$D = 108\ psf$

$L = 200\ psf$

$S = 40\ psf$

$1.2(108) + 1.6(200) + 0.5(40) = 469.6\ psf$ $\quad 469.6(1036.8) = 487k$

Typ Floors (11th - 4th)

$D = 101$

$L = 80$

Reduce Live

$K_{LL} = 4$

$K_{LL} A_T = 4(1036.8) = 4147.2 > 400\ ft^2$ OK

$L = 80\left[.25 + \frac{15}{\sqrt{4147.2}}\right] = 38.6\ psf$

$1.2(101) + 1.6(38.6) = 183\ psf$ $\quad 183(1036.8) = 189.7k$

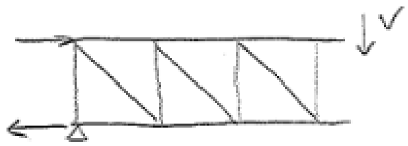
See spreadsheet

Nick Reed   Final Report   Chord Moments

Small Truss   prelim size W10x33

New load at 4th Floor, wind (Bottom chord)

700 k at 4th, 350 per chord



$V_{@mid} = \frac{\frac{1}{2}(350)(11.33)}{35.75} = 55.5k$

$M_{@mid} = 55.5(7/2) = 194.3$ ft·k

Uniform Load on Chord = 5.83 klf

$M_{ends} = 5.83(11.33)^2/10 = 74.84$ ft·k

$M_u = 74.84 + 193.45 = 268$ ft·k

Large Truss

Load at Mezz = 517.5 (Bottom chord)

$V_{@mid} = \frac{\frac{1}{2}(517.5)(15)}{35.75} = 109K$

$M_{@mid} = 109\,(7/2) = 382$ ft·k

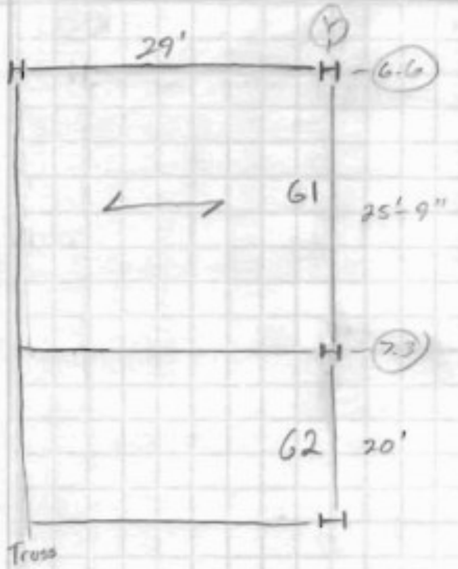
Uniform Load = 8.73 klf

$M_{ends} = 8.73(15)^2/10 = 196$ ft·k

$M_u = 578$ ft·k

See Spreadsheet

Nick Reed | Final Report | Squared-off section steel design (Roof)



SDL = 15 psf

Planks = 93 psf

Snow = 40 psf

Live = 200 psf

Col. 6.6

Trib Area = $\left(\frac{29'}{2}\right)\left(\frac{25.75}{2}\right) = 373.4\ ft^2$

$1.2(108) + 1.6(200) + .5(40) = 469.6\ psf$

$469(373.4) = 175\ k$

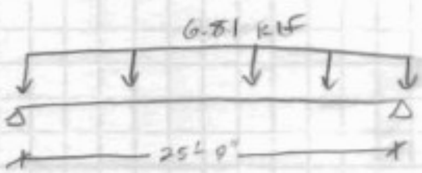
* See spreadsheet for other floor cols *

Col. 7.3

Trib Area = $\left(\frac{25.75+20}{2}\right)\left(\frac{29}{2}\right) = 331.7\ k$

$469.6(331.7) = 156\ k$

Beam G1



$469.6\ psf\left(\frac{29'}{2}\right) = 6.81\ klf$

$V_u = 88\ k$

$M_u = \frac{6.81(25.75)^2}{8} = 564\ ft\cdot k$

G2

6.81 klf

$V_u = 68\ k$

$M_u = \frac{6.81(20)^2}{8} = 341\ ft\cdot k$

Design References

CONCRETE REINFORCING STEEL INSTITUTE

$f_c' = 3{,}000$ psi; $w = 150$ pcf
$f_y = 60$ ksi
Minimum Pile Diameter = 10 in.
spaced at 3′-0″

100-TON STEEL PILES

capacity for unfactored, service ($D + L$)

Min. cover = 3″
$d_c = 10$″
Edge $E = 21$″
See Fig. 13-3

PILES	COLUMN		PILE CAP				REINFORCING BARS					SHEAR	
												$V_u/\phi V_n$	
No. of Piles per cap	Max. Load P_u (net)	Min. Size *	Long A **	Short B **	D	Con-crete **	Long A-Bars (1)	Min. A_s (2)	Short B-Bars (1)	Min. A_s (2)	Steel Wt. (3)	Beam One-Way	Slab Two-Way
	(kips)	(in.)	(ft-in.)	(ft-in.)	(in.)	(c.y.)	No.-Size	(in.²)	No.-Size	(in.²)	(tons)	Ratio	Ratio
2	621	13	6- 6	3- 6	41	2.9	6 H# 8	4.41	5 H# 4	N/A	0.069	0.991	N/A
3	933	18	6- 6	6- 2	42	4.1	6 H# 9	2.67	3-WAYS		0.130	0.542	0.923
			2- 1	2- 3									
4	1246	18	6- 6	6- 6	40	5.2	8 H# 9	7.91	8 H# 9	7.91	0.231	0.576	0.931
5	1548	20	7- 9	7- 9	43	8.0	13 H# 8	10.39	13 H# 8	10.39	0.315	0.532	0.956
6	1860	22	9- 6	6- 6	48	9.1	13 H# 8	10.41	10 H# 9	9.85	0.333	0.946	0.994
7	2148	27	9- 6	8- 9	55	14.1	13 H# 8	10.39	11 H# 9	11.29	0.389	0.421	0.981
8	2476	25	9- 6	8- 9	50	12.8	14 H# 9	14.15	15 H# 9	14.94	0.548	0.508	0.967
9	2778	27	9- 6	9- 6	56	15.6	17 H# 9	16.74	17 H# 9	16.74	0.665	0.459	1.000
10	3088	28	12- 6	8- 9	51	17.2	18 # 9	18.46	16 H# 9	15.67	0.660	0.727	0.973
11	3404	30	12- 6	8- 9	53	17.9	17 #10	22.02	20 H# 9	19.89	0.804	0.739	0.788
12	3702	31	12- 6	9- 6	58	21.3	18 #10	23.34	21 H# 9	20.71	0.875	0.732	0.695
13	4001	32	13- 11	9- 6	60	24.5	21 #10	26.89	23 # 8	18.63	0.883	0.583	0.595
14	4303	33	12- 6	11- 9	60	27.2	19 #10	23.67	24 # 9	24.58	0.950	0.622	0.997
15	4614	34	13- 11	12- 6	59	28.6	20 #10	25.98	27 # 9	27.76	1.128	0.570	0.803
			6- 2	8- 1									
16	4916	36	12- 6	12- 6	65	31.3	27 # 9	27.72	27 # 9	27.72	1.102	0.646	0.996
17	5254	37	13- 11	12- 6	59	28.6	26 #10	32.84	27 # 9	27.05	1.301	0.634	0.980
			6- 2	8- 1									
18	5558	38	13- 11	12- 6	58	31.1	29 #10	37.45	31 # 9	31.45	1.470	0.666	0.994
19	5844	39	14- 9	12- 6	64	36.4	29 #10	37.63	31 # 9	31.66	1.522	0.654	0.984
20	6132	40	15- 6	12- 6	69	41.3	24 #11	37.93	35 # 9	35.98	1.670	0.607	0.990

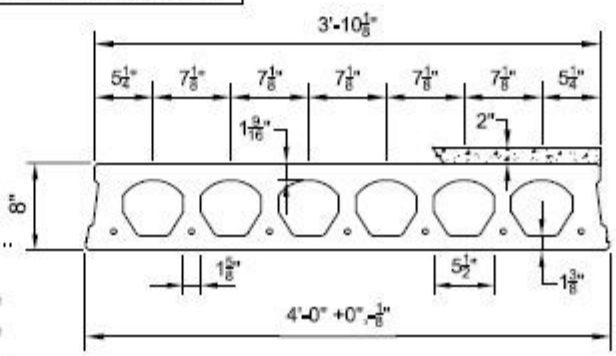
Prestressed Concrete
8"x4'-0" Hollow Core Plank

2 Hour Fire Resistance Rating With 2" Topping

PHYSICAL PROPERTIES
Composite Section

A_c = 301 in.^{2}	Precast b_w = 13.13 in.
I_c = 3134 in.^{4}	Precast S_{bcp} = 616 in.^{3}
Y_{bcp} = 5.09 in.	Topping S_{tct} = 902 in.^{3}
Y_{tcp} = 2.91 in.	Precast S_{tcp} = 1076 in.^{3}
Y_{tct} = 4.91 in.	Precast Wt. = 245 PLF
	Precast Wt. = 61.25 PSF

DESIGN DATA



1. Precast Strength @ 28 days = 6000 PSI
2. Precast Strength @ release = 3500 PSI
3. Precast Density = 150 PCF
4. Strand = 1/2"Ø 270K Lo-Relaxation.
5. Strand Height = 1.75 in.
6. Ultimate moment capacity (when fully developed)...
   4-1/2"Ø, 270K = 92.3 k-ft at 60% jacking force
   6-1/2"Ø, 270K = 130.6 k-ft at 60% jacking force
   7-1/2"Ø, 270K = 147.8 k-ft at 60% jacking force
7. Maximum bottom tensile stress is $10\sqrt{f'c}$ = 775 PSI
8. All superimposed load is treated as live load in the strength analysis of flexure and shear.
9. Flexural strength capacity is based on stress/strain strand relationships.
10. Deflection limits were not considered when determining allowable loads in this table.
11. Topping Strength @ 28 days = 3000 PSI. Topping Weight = 25 PSF.
12. These tables are based upon the topping having a uniform 2" thickness over the entire span. A lesser thickness might occur if camber is not taken into account during design, thus reducing the load capacity.
13. Load values to the left of the solid line are controlled by ultimate shear strength.
14. Load values to the right are controlled by ultimate flexural strength or fire endurance limits.
15. Load values may be different for IBC 2000 & ACI 318-99. Load tables are available upon request.
16. Camber is inherent in all prestressed hollow core slabs and is a function of the amount of eccentric prestressing force needed to carry the superimposed design loads along with a number of other variables. Because prediction of camber is based on empirical formulas it is at best an estimate, with the actual camber usually higher than calculated values.

SAFE SUPERIMPOSED SERVICE LOADS — IBC 2006 & ACI 318-05 (1.2 D + 1.6 L)

Strand Pattern		SPAN (FEET)																		
		17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35
4 - 1/2"ø	LOAD (PSF)	280	248	214	185	159	138	118	102	87	74	62	52	42						
6 - 1/2"ø	LOAD (PSF)	366	341	318	299	271	239	211	187	165	146	129	114	101	88	77	67	58	50	42
7 - 1/2"ø	LOAD (PSF)	367	342	320	300	282	265	243	221	202	181	161	144	128	114	101	90	79	70	61

This table is for simple spans and uniform loads. Design data for any of these span-load conditions is available on request. Individual designs may be furnished to satisfy unusual conditions of heavy loads, concentrated loads, cantilevers, flange or stem openings and narrow widths. The allowable loads shown in this table reflect a 2 Hour & 0 Minute fire resistance rating.

NITTERHOUSE
CONCRETE PRODUCTS
2655 Molly Pitcher Hwy. South, Box N
Chambersburg, PA 17202-9203
717-267-4505 Fax 717-267-4518

11/03/08

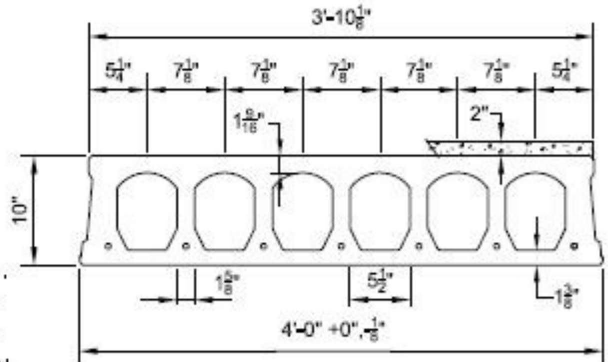
8SF2.0T

Prestressed Concrete
10"x4'-0" Hollow Core Plank

2 Hour Fire Resistance Rating With 2" Topping

PHYSICAL PROPERTIES
Composite Section

A_c = 327 in.^{2}	Precast b_w = 13.13 in.
I_c = 5102 in.^{4}	Precast S_{bcp} = 824 in.^{3}
Y_{bcp} = 6.19 in.	Topping S_{tct} = 1242 in.^{3}
Y_{tcp} = 3.81 in.	Precast S_{tcp} = 1340 in.^{3}
Y_{tcp} = 5.81 in.	Precast Wt. = 272 PLF
	Precast Wt. = 68.00 PSF



DESIGN DATA

1. Precast Strength @ 28 days = 6000 PSI
2. Precast Strength @ release = 3500 PSI
3. Precast Density = 150 PCF
4. Strand = 1/2"Ø and 0.6"Ø 270K Lo-Relaxation.
5. Strand Height = 1.75 in.
6. Ultimate moment capacity (when fully developed)...
   6-1/2"Ø, 270K = 168.1 k-ft at 60% jacking force
   7-1/2"Ø, 270K = 191.7 k-ft at 60% jacking force
7. Maximum bottom tensile stress is $10\sqrt{f'c}$ = 775 PSI
8. All superimposed load is treated as live load in the strength analysis of flexure and shear.
9. Flexural strength capacity is based on stress/strain strand relationships.
10. Deflection limits were not considered when determining allowable loads in this table.
11. Topping Strength @ 28 days = 3000 PSI. Topping Weight = 25 PSF.
12. These tables are based upon the topping having a uniform 2" thickness over the entire span. A lesser thickness might occur if camber is not taken into account during design, thus reducing the load capacity.
13. Load values to the left of the solid line are controlled by ultimate shear strength.
14. Load values to the right are controlled by ultimate flexural strength or fire endurance limits.
15. Load values may be different for IBC 2000 & ACI 318-99. Load tables are available upon request.
16. Camber is inherent in all prestressed hollow core slabs and is a function of the amount of eccentric prestressing force needed to carry the superimposed design loads along with a number of other variables. Because prediction of camber is based on empirical formulas it is at best an estimate, with the actual camber usually higher than calculated values.

SAFE SUPERIMPOSED SERVICE LOADS		IBC 2006 & ACI 318-05 (1.2 D + 1.6 L)																		
Strand Pattern		SPAN (FEET)																		
		26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44
6 - 1/2"ø	LOAD (PSF)	202	181	161	144	128	114	101	90	79	69	60	52	45	38					
7 - 1/2"ø	LOAD (PSF)	246	222	200	180	162	146	131	118	105	94	84	74	66	58					

NITTERHOUSE
CONCRETE PRODUCTS
2655 Molly Pitcher Hwy. South, Box N
Chambersburg, PA 17202-9203
717-267-4505 Fax 717-267-4518

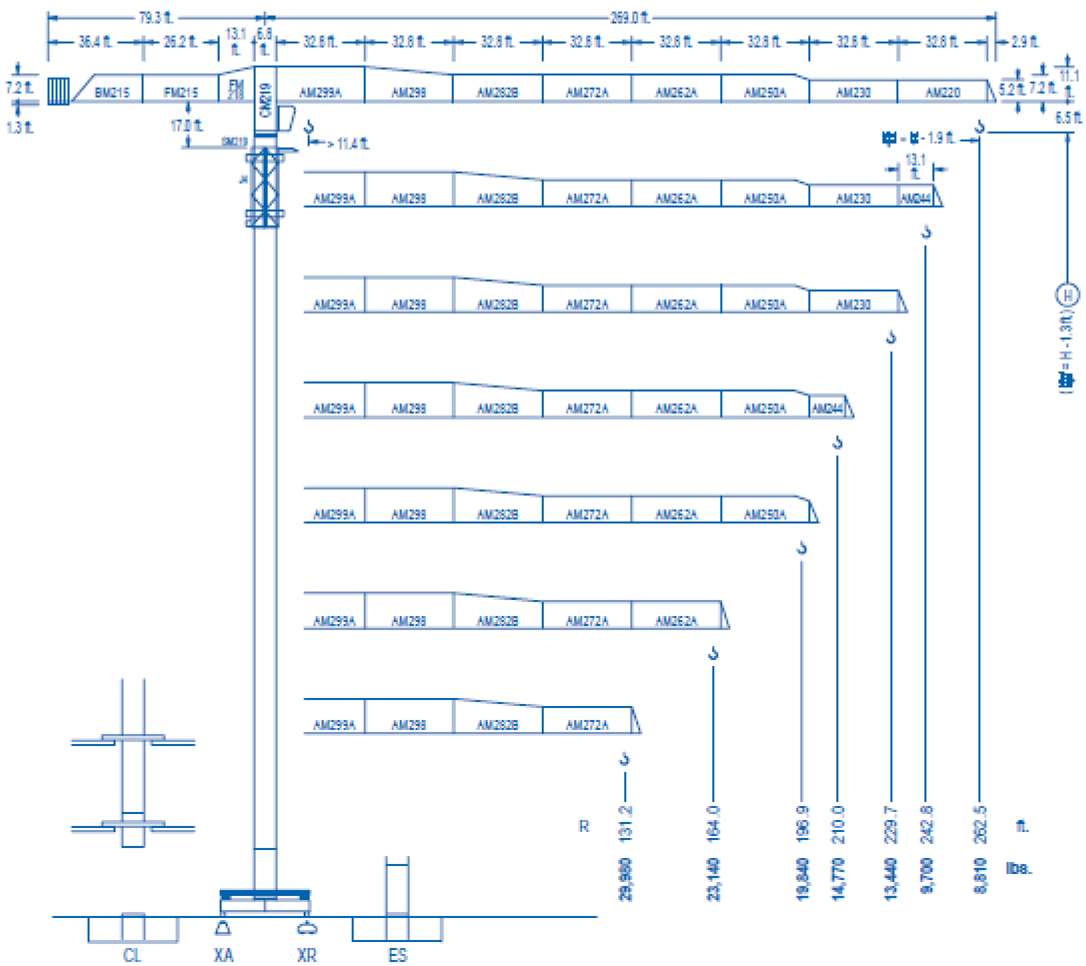
This table is for simple spans and uniform loads. Design data for any of these span-load conditions is available on request. Individual designs may be furnished to satisfy unusual conditions of heavy loads, concentrated loads, cantilevers, flange or stem openings and narrow widths. The allowable loads shown in this table reflect a 2 Hour & 0 Minute fire resistance rating.

11/03/08

10F2.0T

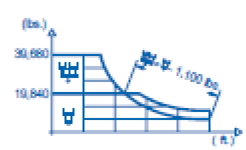
LINDEN COMANSA

LINDEN 2100 — 21 LC 550 — 39,680 lbs.



CE EN 14439 (C/25)

R (ft.)														
262.5	67.6 39,680	78.7 33,460	98.4 25,900	123.7 19,840	127.0 19,840	144.4 17,260	164.0 14,880	177.2 13,760	196.9 12,230	210.0 11,370	229.7 10,270	242.8 9,640	262.5 8,810	ft. lbs.
242.8	67.6 39,680	78.7 33,440	98.4 25,890	123.7 19,840	127.6 19,840	144.4 17,330	164.0 16,060	177.2 13,830	196.9 12,300	210.0 11,440	229.7 10,330	242.8 9,700	ft lbs.	
229.7		84.0 39,680	98.4 33,140	111.5 28,760	131.2 23,870	153.9 19,840	161.1 19,840	177.2 17,890	196.9 16,930	210.0 14,840	229.7 13,440	ft lbs.		
210.0		83.0 39,680	98.4 32,810	111.5 28,470	131.2 23,630	152.6 19,840	160.4 19,840	177.2 17,780	196.9 16,880	210.0 14,770	ft lbs.			
196.9			100.7 39,680	111.5 36,370	131.2 29,470	144.4 26,480	164.0 22,870	186.7 19,840	196.9 19,840	ft lbs.				
164.0			100.4 39,680	111.5 36,270	131.2 29,390	144.4 26,380	164.0 23,140	ft lbs.						
131.2			100.4 39,680	111.5 36,310	131.2 29,980	ft lbs.								



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