Technical Report 2

Seneca Allegany Casino Hotel Addition

Salamanca, NY

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Structural Option

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Executive Summary

The objective of Technical Report 2 is to design alternative floor systems for the Seneca Allegany Casino Hotel Addition and weigh the pros and cons to the existing floor system. This report begins with an analysis of the existing composite beam and deck system, including revisions from Technical Report 1. The following are the three systems researched, analyzed and designed in this report:

- Precast planks on steel beams
- Precast planks on staggered trusses
- One-way concrete slab and beams

Each system is evaluated based on multiple variables, such as structural impacts, cost, scheduling, construction and architectural impacts. Hand calculations are provided in determining whether or not a system is viable. They can be found in Appendix D. Lateral system alterations are considered, but are not designed differently from Technical Report 1's findings at this time. Technical Report 3 will be a more in depth focus on lateral analysis.

At the conclusion of this report on page 25, a comparison chart can be found which breaks down each design consideration for the alternative systems against the existing composite system. The comparison chart shows that the one-way slab can be eliminated as a design choice due to the location of the SAC Hotel in the northern US, the architectural impacts, cost, and construction time.

The staggered truss design will require a deeper analysis to be considered, but can not be immediately ruled out due to the useful nature of a truss design in a building such as the SAC Hotel. Overall, the precast plank on steel frame design seems to be the most viable due to the low cost, small system depth, quick time of construction and similar overall weight to the existing composite system.

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Building Introduction

The Seneca Allegany Casino has undergone multiple construction phases over the years, 5 in total, with the first being a pre-engineered metal building housing the original casino floor, built in 2004 and shown on the far right of Figure 1. Phase 2 consisted of an 8 level parking garage, built from precast concrete in 2005. Next came the first 11-story, 200 room hotel tower with a 2 story casino/restaurant addition, built in 2006 with a typical steel framing system. In 2007, Phase 4 was a renovation of phase 1, converting the original casino floor into an event center, which required new steel truss supports for partitions and concert lighting.

Figure 1 - Seneca Allegany Casino
Satellite Photo Courtesy of Bing.com

This thesis will focus on Phase 5, which is another 11-story, 200 room hotel tower with a structural steel framing system bearing on steel pile foundations. This tower ties into an existing portion of the Phase 3 tower, which was originally built to withstand the added gravity load of Phase 5. Construction started in 2008, but construction was halted until 2011, with a projected completion date of Fall 2012. Phase 5 is shown in yellow in Figure 1.

Figure 2 shows the hotel tower sheathed in an insulated glass façade, reflecting the same aesthetic of the original hotel tower. The casino is located within the Seneca Indian Reserve in Salamanca, NY, a mountainous region with an average elevation of 1400 ft. above sea level. This high elevation allows for plenty of natural light and there are no other surrounding structures to shade the casino complex. The lower 3 levels of the addition consist of insulated metal panels backed by metal framing studs.

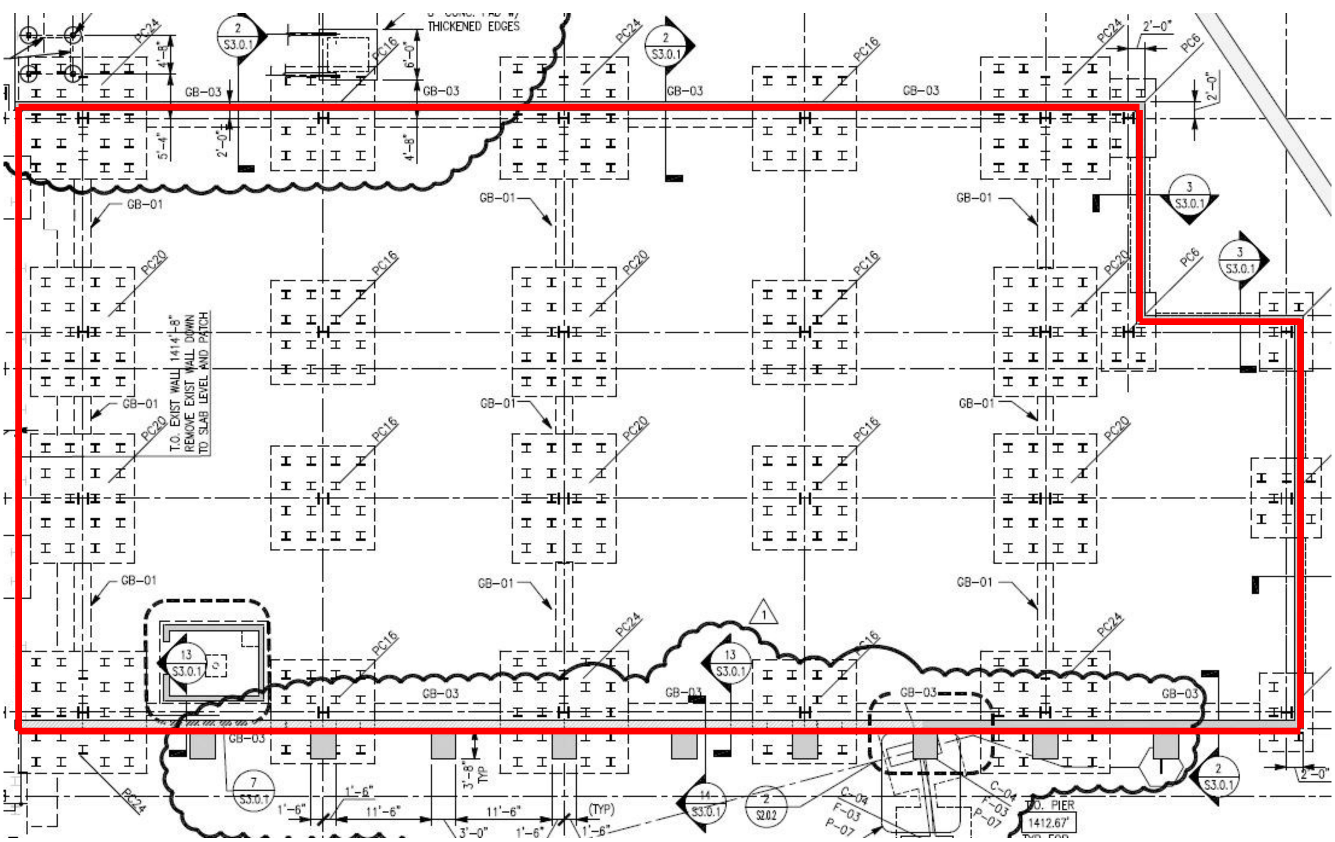
Figure 2 - South Elevation
Photo Courtesy of Jim Boje, PE (Wendel)

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. A section of a typical pile can be found in Appendix C.

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. Sections of the strip footing and typical interior column footings with piers can be found in Appendix C.

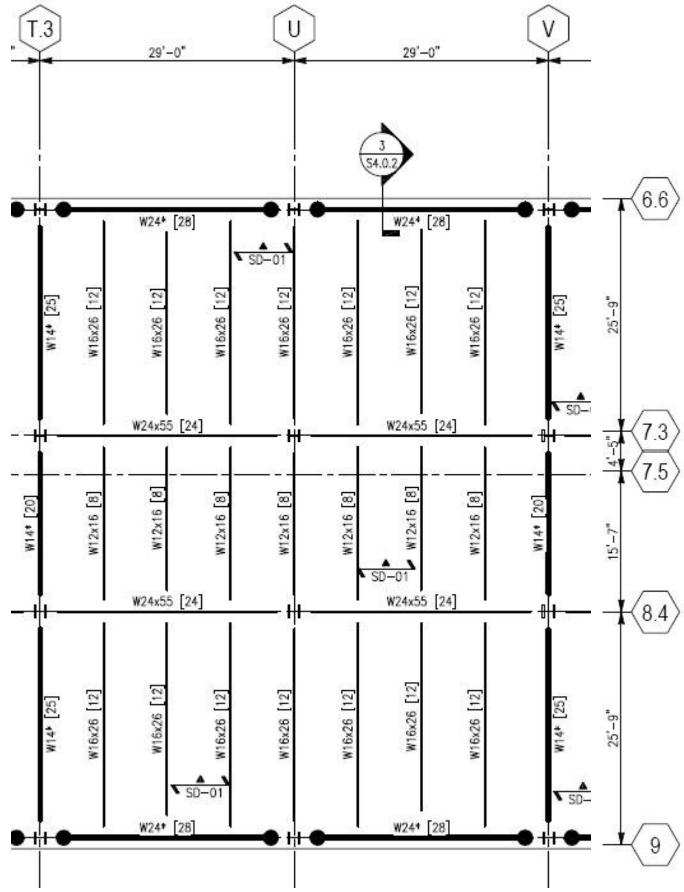


Drawing 1 - Steel Pile/Pile Caps Plan
Drawings Courtesy of JCJ Architecture

Framing & Floors

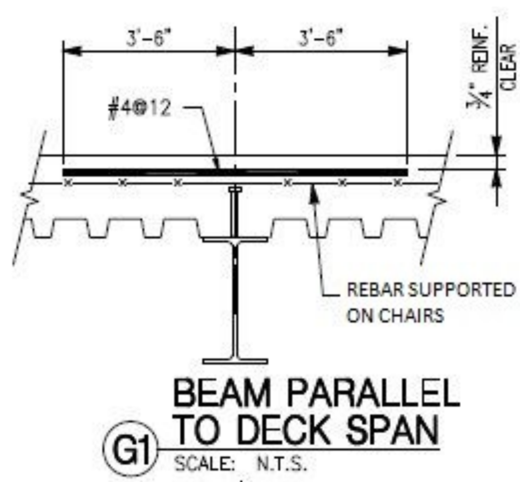
Since this is a hotel tower, the bays are repetitive with the largest bay size a consistent 25'-9" by 29' from the lobby up through the 11th floor. The hotel rooms are located along the outer edges, between column lines 6.6 - 7.3 and 8.4- 9, shown here in Drawing 2. The middle section is the corridor,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

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.



Drawing 2 - Section of 4th—10th Floor Framing Plan
Drawings Courtesy of JCJ Architecture

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 (see Appendix A). Drawing 3 shows a typical deck section.



Drawing 3 - Typical Composite Metal Deck Section
Drawings Courtesy of JCJ Architecture

Columns

The SAC Hotel addition uses wide flange columns throughout the entire addition. The weight of the columns decrease as the elevation increases, with a small range of sizes used. Table 1 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. Column elevations can be found in Appendix B.

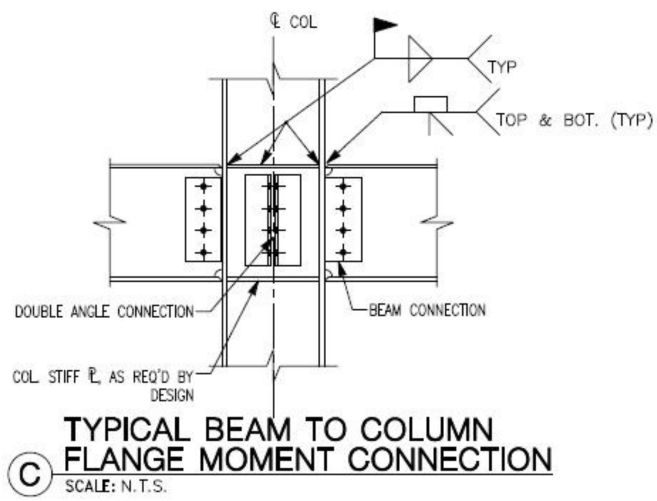
STEEL COLUMN SCHEDULE								
COLUMN MARK	COL. SIZE	BASE PLATE			ANCHOR BOLTS			REMARKS
		T (in.)	W (in.)	L (Ft.–in.)	QTY	SIZE (DIA)	ASTM F1554	
C–01	16"øx0.50" PIPE	2"	24"	2'–0"	4	1 1/4"	GR55	.
C–02	W14x68	1"	22"	1'–10"	4	1"	GR36	.
C–03	W14x90	1 1/2"	22"	1'–10"	4	1"	GR36	.
C–04	W14x132	2"	28"	2'–4"	4	1 1/4"	GR55	20" WIDE BASE ℄ AT HOTEL LOBBY

Table 1

Drawings Courtesy of JCJ Architecture

Lateral System

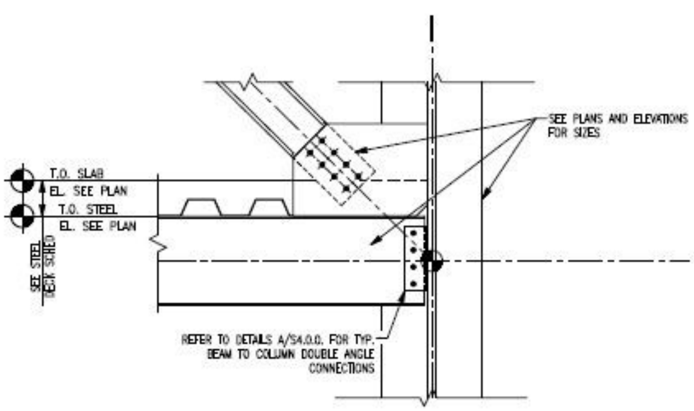
The lateral systems used in the SAC Hotel consist of moment frames in the long spans (E-W) directions and diagonally braced frames in the short (N-S) directions. For the moment frames, moment connections occur at columns and girders, shown below. Typical frame elevations can be found in Appendix B.



Drawing 4 - Typical Moment Connection
Drawings Courtesy of JCJ Architecture

Figure 3 - Typical Moment Connection
Photo Courtesy of Jim Boje, PE (Wendel)

The diagonal bracing is used in specific column lines, Q, S, T.3, V, W, and X. (Framing plan can be found in Appendix B.) 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 stairwell. The bracing is tied into the frame by use of steel plates embedded in slab deck at beams and columns, shown below. Sections of the diagonal bracing tied into the foundation can be seen in Appendix C.

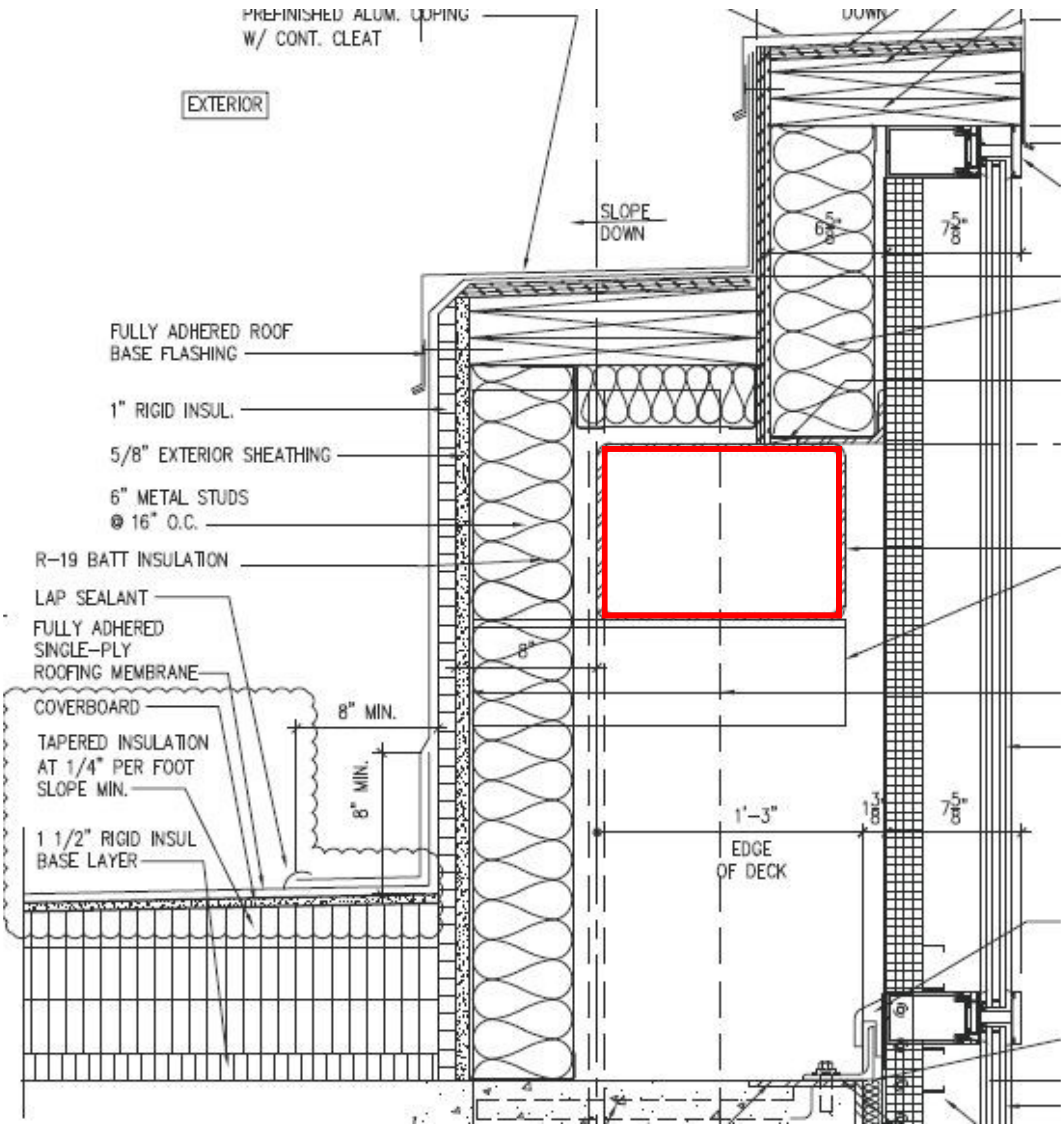


Drawing 5 - Diagonal Brace Connection at Column
Drawings Courtesy of JCJ Architecture

Figure 4 - Diagonal Brace Connection at Column
Photo Courtesy of Jim Boje, PE (Wendel)

Roof

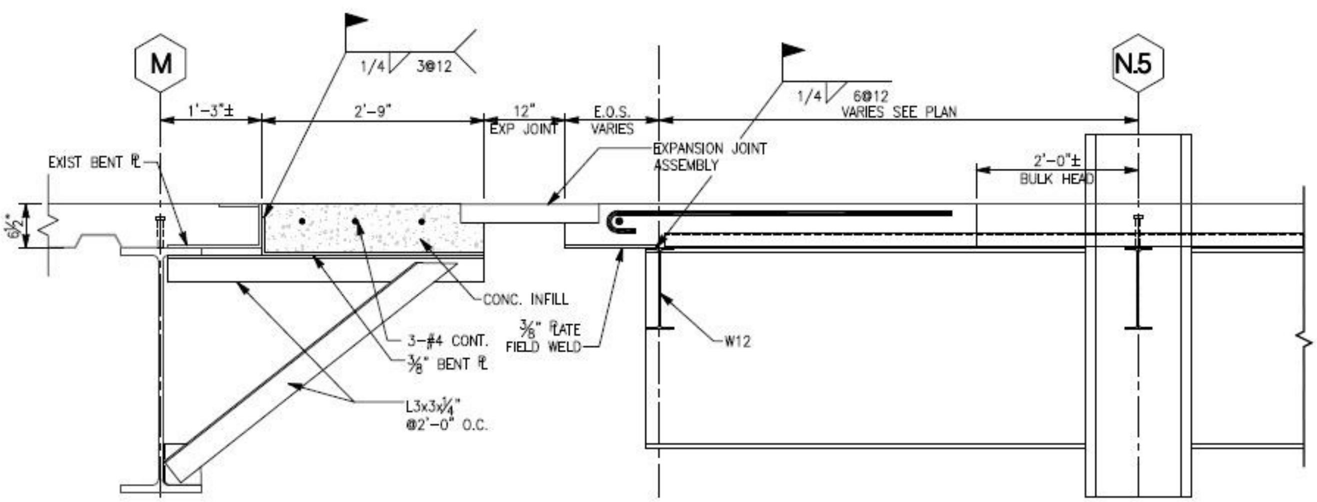
The roof structure is consistent with the hotel floor framing, with no change in bay sizes, or location of moment frames, and uses similar metal deck to the hotel floors, with a larger gauge of 18. Slightly larger W shapes are used to account for the extra roof snow load, (40 psf), with the majority 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 Drawing 6, with the HSS outlined in red. A more detailed roof framing plan can be found in Appendix A. The roof also supports window washing machines, with anchors embedded in the deck. The locations of these can also be seen in Appendix A.



Drawing 6 - 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. 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.



Drawing 7 - Expansion Joint Section
Drawing Courtesy of JCJ Architecture

Figure 5 - 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 technical report, the following code editions and specs were used for calculation checks:

- 2009 IBC
- ASCE 7-05
- AISC, 14th edition
- AISC, 14th edition - Staggered Truss Framing Systems Design Guide
- Vulcraft 2008 Decking Catalogue
- ACI 318-11
- Nitterhouse Precast Concrete Planks spec sheets

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
Metal Deck	69 psf	Vulcraft 2008 Deck Catalog

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

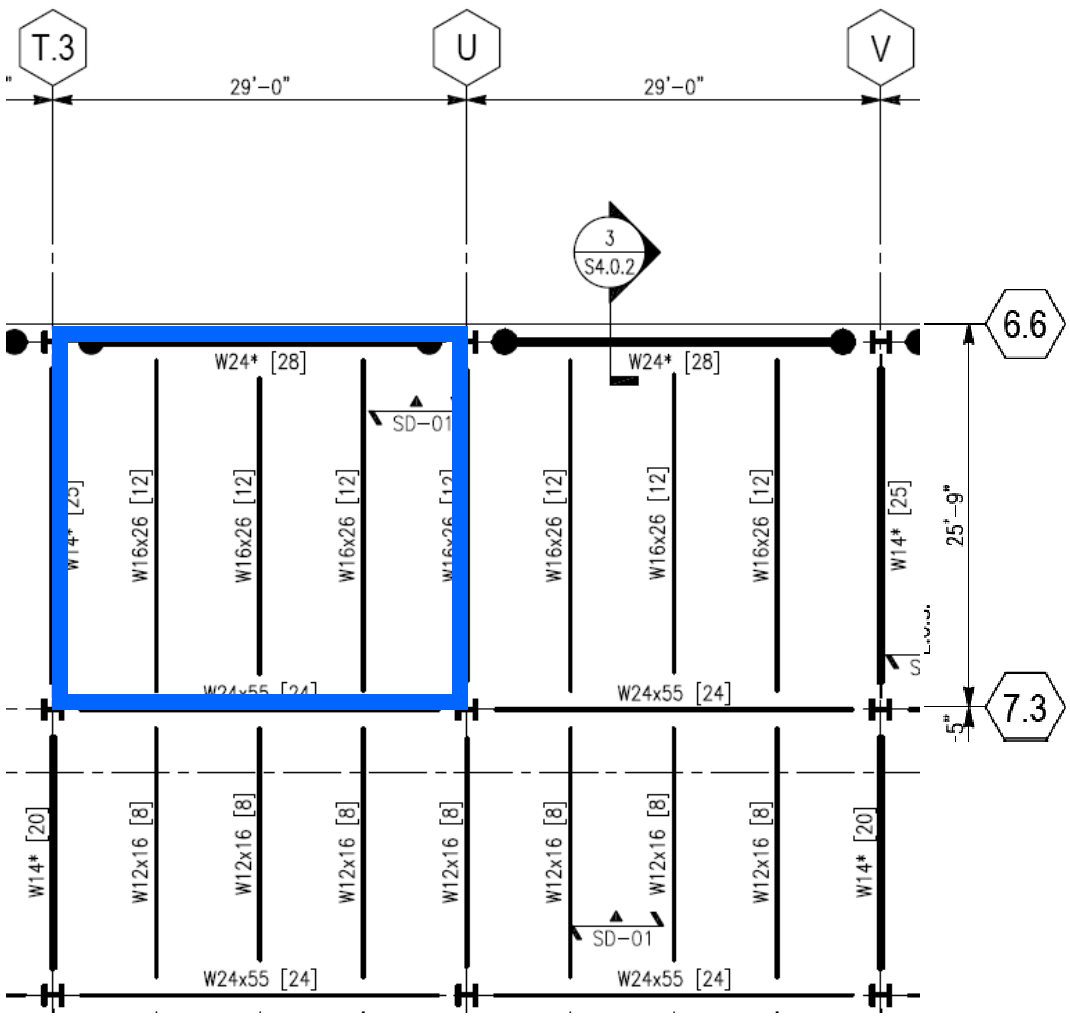
Note: CS in ASCE 7-05 stands for Case Study snow loads, which is why the 50 psf Design Load was used in calculations, taken from the specifications for the 2010 New York State Building Code.

Alternative System Designs

The purpose of this technical report is to design 3 alternative floor systems and compare them to the existing system in the SAC Hotel, based on variables such as cost of construction, architectural impact, weight, depth and others. All systems will be compared in a chart at the conclusion of this section.

The systems analyzed include:

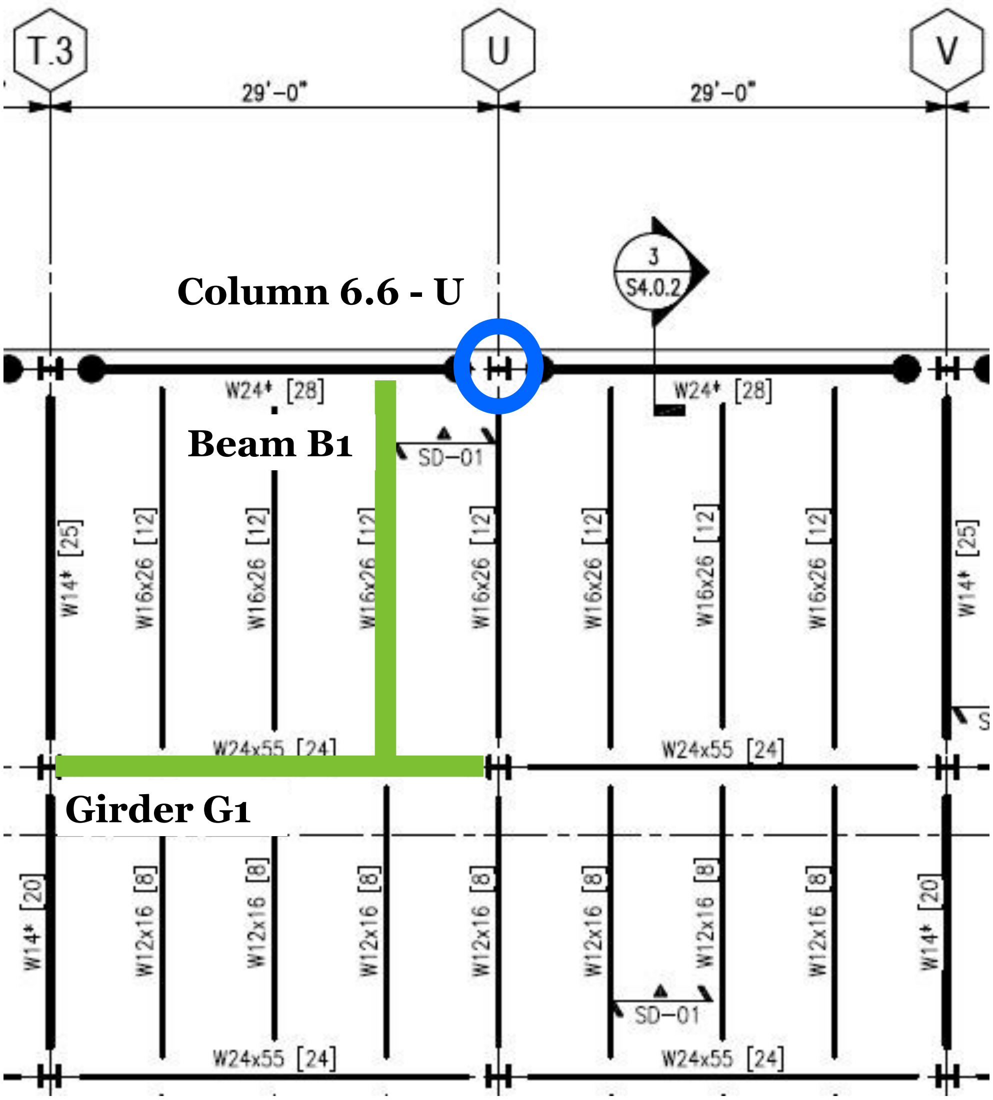
- Composite deck and beams
- Hollow core precast concrete planks on steel beams
- Staggered Truss
- One-way concrete slab



Drawing 8 - Typical Bay of 4th Floor used in analysis
Drawing Courtesy of JCJ Architecture

Existing Composite Deck and Beams

Prior to any calculations, it was found that the design of the SAC Hotel addition was accomplished using ASD over LRFD. This analysis was done using LRFD. Therefore, any discrepancies in final loads, moments, etc., could be a result of this fact. Drawing 9 below shows the members in a typical 29' by 25.75' bay, along with an exterior column, that were analyzed for this section of the report. It was found that live load reduction was not permitted for the beam due to small tributary areas, but was permitted for the girder.



Drawing 9 - Typical Bay of 4th Floor for Spot Checks
Drawing Courtesy of JCJ Architecture

Decking

In the SAC Hotel addition, the metal deck schedule calls for 2 different sizes of composite deck, 18 and 20 gage, with normal weight concrete. The bay in Figure 15 requires 20 gage, 6.5" in total depth. For this analysis, 2VLI20 deck was selected from the Vulcraft 2008 Deck Catalogue as a close approximation. The total weight of the 2VLI20 is 69 psf, with a clear span of 7'6", (>7.25' in the bay used for analysis), capable of carrying a 400 psf superimposed live load, while design specifications only require 80-125 psf superimposed live load.

Beams

Beams span in the N-W direction in the 29' by 25.75' bay, with W16x26's as the most common and repetitive members for the hotel room floors. For all load calculations, a superimposed dead load of 15 psf accounts for suspended mechanical equipment and ceilings. Initially, Beam B1 was checked as simply supported and was found to meet the required loads, with a slight overdesign most likely due to LRFD. Next, B1 was checked as fully composite to determine the number of shear studs required. It was found that 12 shear studs would be required, which reproduces what the SAC Hotel frame plans show. As far as deflections, the W16x26 met the requirements for live and total load deflections.

Girder

In Figure 15, Girder G1 spans 29' in the E-W direction, carrying beams that support the hotel rooms and the corridor. The girder was also checked with simply supported and then fully composite. It was found that the W24x55 was able to meet the required loads. After checking for fully composite, the required number of shear studs was found to be 24, which again reproduces what the frame plans show. Deflection checks showed that the girder could withstand live load deflections, while failing in total load.

Supporting calculations can be found in Appendix D.

Columns

A 4th floor exterior column from the bay in Drawing 9, column 6.6-U, was chosen for this spot check. From the 4th floor to the roof, shape sizes range from W14x211's to W14x120's. All columns were found to be more than adequately designed, although the effects of moment connections were not explored. Also, some loads could have been estimated as not conservative enough. Tables 2 and 3 below show the member sizes associated with floor levels and which loads were applied. Table 4-1 of the AISC manual was used to find the critical stresses associated with the effective lengths listed. Sample calculations can be found in Appendix D and frame elevations can be found in Appendix B.

Column Loads					
Floor	**A_t (ft^2)**	**DL (psf)**	**LL (psf)**	**Reduced LL (psf)**	**SL (psf)**
4	373.5	100	80	51	0
5	373.5	100	80	51	0
6	373.5	100	80	51	0
7	373.5	100	80	51	0
8	373.5	100	80	51	0
9	373.5	100	80	51	0
10	373.5	100	80	51	0
11	373.5	100	80	51	0
Roof	373.5	100	200	127.6	40

Table 2

Column Capacities					
Floor	**P_u (k)**	**$\sum P_u$ (k)**	**Member**	**ϕP_n (k)**	**Unbraced Length (ft.)**
4	75.30	730.92	W14x211	2580	11.33
5	75.30	655.62	W14x176	2150	11.33
6	75.30	580.32	W14x176	2150	11.33
7	75.30	505.03	W14x145	1770	11.33
8	75.30	429.73	W14x145	1770	11.33
9	75.30	354.43	W14x120	1450	11.33
10	75.30	279.13	W14x120	1450	11.33
11	75.30	203.84	W14x120	1450	11.33
Roof	128.54	75.29	W14x120	1450	13.67
∑	**730.92**				

Table 3

Advantages

The use of a composite system allows for both the concrete and steel to be designed to their fullest strength, ideally with the concrete in total compression and steel in total tension. Composite systems are also relatively lightweight, especially if light-weight concrete is used, (although the SAC Hotel uses normal weight concrete), and are easier for designing a typical foundation. Erection of this system can also be accomplished quickly if there are no scheduling issues and little formwork is needed because the concrete can be placed directly on the deck.

Disadvantages

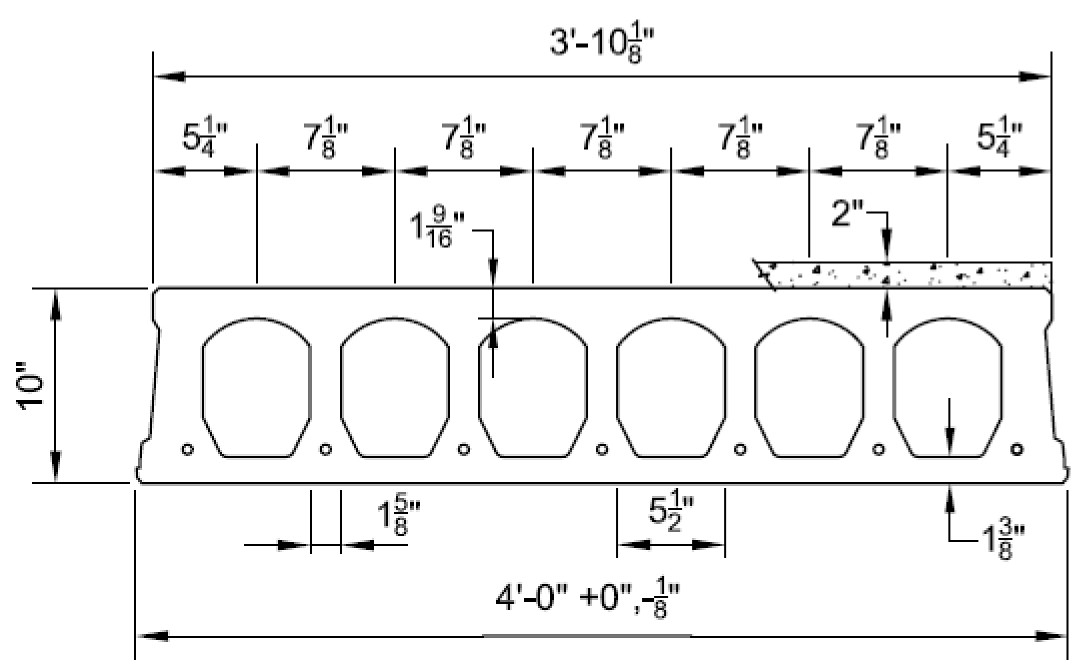
One of the main disadvantages of any steel system is fire proofing. With composite deck, both the underside of the deck and the steel beams themselves must be covered in fire proofing, thus increasing costs of overall construction and ceiling finishes. Also with composite systems, shear connectors are required to attach the deck to the beams. Making sure the connectors are placed properly can increase construction time due to inspections, and some connectors could be damaged during the placement of the slab, adding more cost. Depending on bay sizes, some composite systems could require rather large beams and girders, thus creating coordination problems with MEP or architecture companies.

Hollow Core Precast Planks on Steel Beams

The first system analyzed for this technical report utilized prestressed hollow core concrete planks. According to design specs, a 2-hour fire rating is required for the SAC Hotel. Initially, an 8” x 4’ plank with 2” topping with (7) 1/2” diameter strands from Nitterhouse was checked, but was found to fail under required loads. The next step was to increase the plank depth to a 10” x 4’ plank with 2” topping and (7) 1/2” strands, which did meet the required loads. Supporting calculations and Nitterhouse specs can be found in Appendix D.

Using precast planks required slight changes to the bay size, which would in turn affect column layouts. The shortest span available for the 10” plank was 26’, which was close to the 25’ x 9” direction. This would shorten the interior bay by 6” overall. The 29’ direction would also have to be shortened to 28’ to allow for (7) 4’ wide planks to be placed. Each bay typically spaces 2 hotel rooms, so this loss of a foot could result in each room losing 6” of floor space.

For the shortened span of 28’, a girder was designed as a simply supported beam. The girder accounts for the design specification loads as well as the self-weight of the precast planks. A W18x119 was found to meet all the strength requirements as well as total and live load deflections. Supporting calculations for the girder can be found in Appendix D.



Drawing 10 - Precast Panel Dimensions
Courtesy of Nitterhouse

Advantages

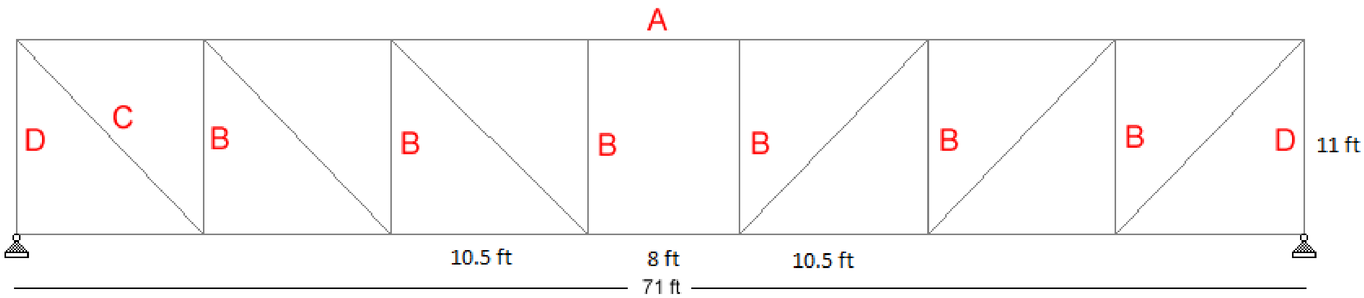
Precast planks do not require fireproofing on their underside and the smaller system depth allows for more room for MEP systems. Since the concrete is no longer cast in place, construction times are reduced due to no scheduling for finishing, curing or use of shoring, and planks can be placed quickly. With no shoring, construction could continue to the floors above. Overall, the system can be lightweight due to a smaller number of smaller steel members being used.

Disadvantages

The fact that this is a lightweight system could also be looked at as a disadvantage. Voids in the planks greatly reduce the weight, even with normal weight concrete. Paired with smaller steel members, this system could be greatly impacted by uplift forces as a result of wind or seismic since the SAC Hotel is 11 stories. In order for this system to be viable, a deeper investigation of lateral forces would need to be done. The system also still requires some fireproofing for the steel members, so the cost is not wholly eliminated.

Staggered Truss

Another system considered used the precast planks from the previous section on a staggered truss. This system uses a Vierendeel truss that is story deep and spans the entire width of the building from exterior column to exterior column, eliminating the need for interior columns. Staggered trusses are perfect for repetitive framing systems and room layouts, which the SAC Hotel utilizes. Shown below is the geometry of the selected truss, with member sizes found using STAAD Pro V8i.



Drawing 11 - Preliminary Truss Geometry

A: = W24 x 450 (Top and Bottom chord)

B: = W21 x 147

C: = W10 x 88

D: = W24 x 131

The corridor width was assumed to be 8', and in order to keep the outer 3 bays an equal length close to the height of the truss, the overall span of 72' was shortened to 71'. Distributed loads from the precast planks were placed on the top and bottom chords and members were sized based on overall total load deflections. Member sizes turned out to be relatively large and heavy, and the 26' span planks used in the previous section required the 29' long bay to be shortened 3' in order to place the planks perpendicular to the truss. Preliminary calculations for the truss member loads can be found in Appendix D. In order for this system to be considered viable, a closer investigation of individual member forces and interaction would be necessary.

Advantages

Due to the repetitive nature of the floor and framing layouts of the SAC Hotel, the staggered truss would be used to great advantage since the diagonal members would be hidden within walls and the middle section would be left open for the corridor. Members could be placed quickly since they are preassembled, which could free up construction time and space for materials on-site. Paired with precast planks, the entire framing system could be built in cold weather, which would be useful in a northern region like New York state. Much like the precast planks on steel members, this again could be another lightweight structure, only this time weight would be lost due to no interior columns. The use of the precast planks would also decrease the floor to floor height.

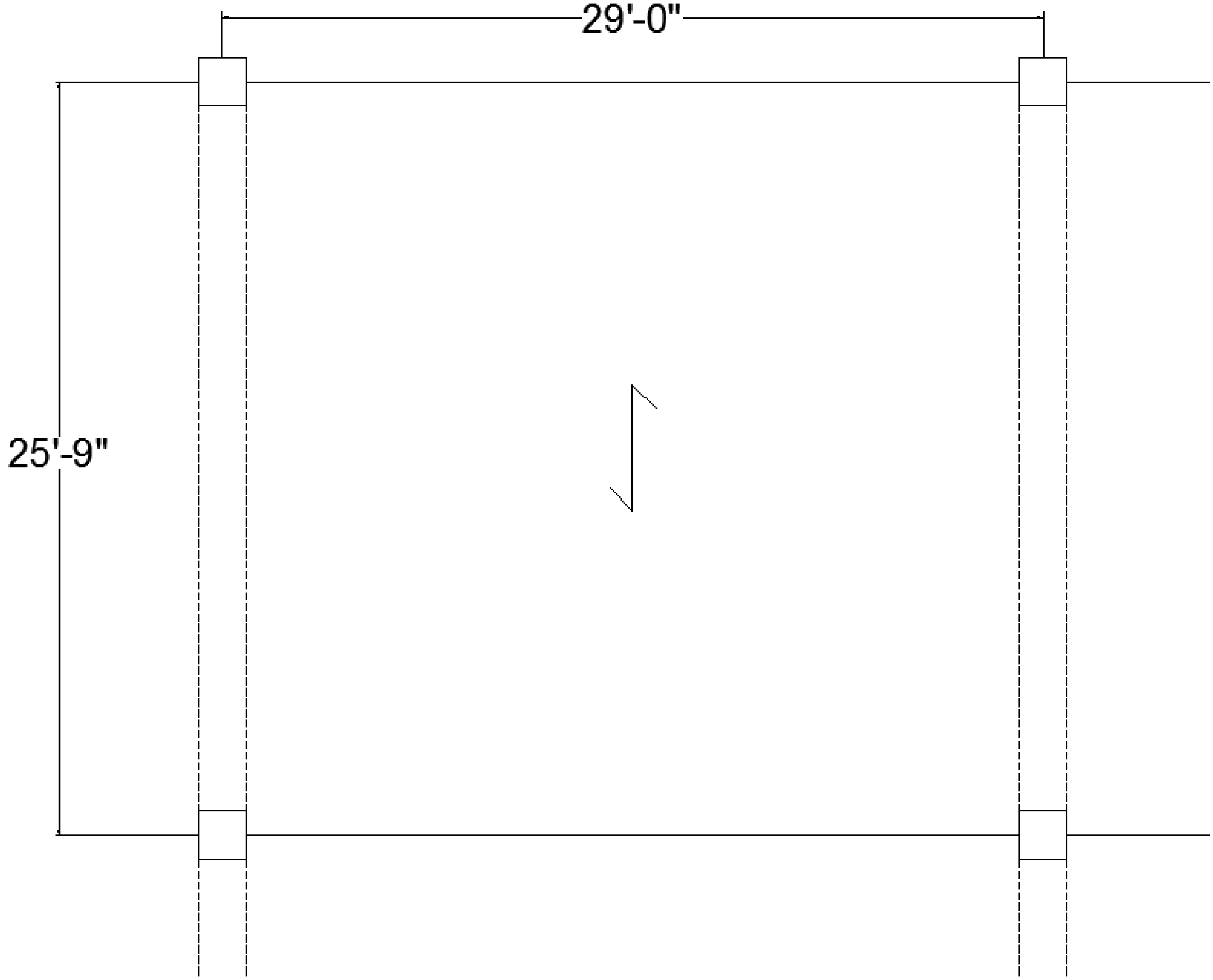
Disadvantages

For this system to be viable for the SAC Hotel, initial designs require heavy members that could drastically influence the foundation design. Even though interior columns are eliminated, the weight of the larger members may exceed the existing interior column weight overall, so this would need to be explored further. For project scheduling, the trusses would need to be prefabricated off-site, meaning a lead time would need to be established prior to construction. Considerable space on the construction site would need to be set aside for these large members as well, since site space would also be taken by the precast concrete planks.

One-Way Concrete Slab

The last system investigated was a one-way slab on beams. The bay size for this analysis was not changed from the 29' x 25.75'. In order to reduce the floor thickness, the one-way slab was designed with no interior beams, with the slab working in the 25.75' direction.

Through analysis, it was determined that a 13" slab reinforced with #5 bars spaced every 10" on center could be carried by a 22.5" deep beam reinforced with (5) #10 bars. Calculations for this system can be found in Appendix D. Columns were assumed to be 20" x 20", which in turn affected the assumption of a 20" wide beam to keep the system uniform.



Drawing 12 - Typical Bay

Advantages

A system of monolithic concrete construction would require no extra fire proofing and would provide great vibration control. Scheduling would not require a long lead time for materials since the concrete could be formed and placed directly on site. Construction costs could also be considerably reduced since concrete is relatively cheap and easily obtained.

Disadvantages

Although a long lead time would not be required, construction of a monolithic concrete system is slow. The concrete must be allowed time for curing to its ultimate strength and would require shoring for construction to continue to the above floors. With the added cost of shoring, there would also be the cost of the formwork. The overall system is much heavier than the existing composite system, so further investigation into the design and effects on the foundation would need to be done. Also, shrinkage and creep could lead to problems later on in the concrete systems life.

Systems Comparison

	Existing	Alternatives		
Comparison Criteria	Composite Beam/Deck	Precast Planks with Steel	Staggered Truss	One-way Slab and Beam
Weight	74 psf	68 psf	>250psf	201.5psf
Slab Depth	6.5"	10"	10"	13"
System Depth	32.5"	28"	24"	35.5"
Architectural Impact (Bay Size)	No	Yes	Yes	No
Fire Protection	Yes	Yes	Yes	No
Formwork	Minimal	No	No	Yes
Cost (per sq. ft.)	$25.07	$13.57	$13.57 + Truss Costs	$21.58
Lateral System (Alterations)	No	Yes	Yes	Yes
Foundation Alterations	No	No	Yes	Yes
Constructability	Average	Fast	Average	Slow
Lead Time	Moderate	Moderate	Long	Short
Viable System	Yes	Yes	Maybe	No

Table 4

Conclusion

Upon study of Table 5's conclusions, the one-way slab can be all but eliminated, mostly due to the largest system depth from the deep slab depth on top of the beam depth. Also, since the SAC Hotel is located more in the northern US, concrete construction can run into problems with weather and curing and installation times. The cost of shoring and formwork would be an issue as well. Foundations would have to redesigned to handle the larger loads and a new lateral system would have to be considered.

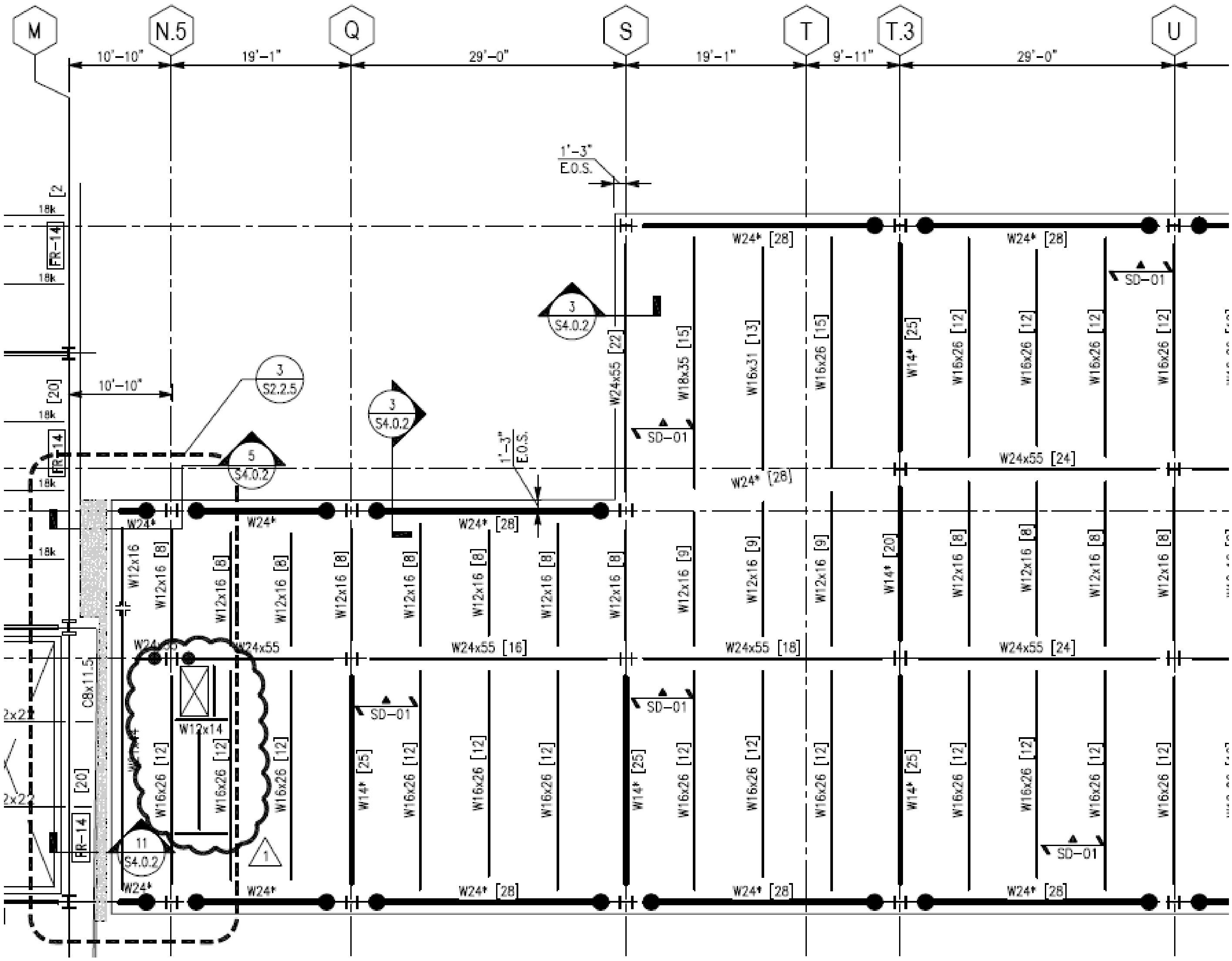
The staggered truss is still somewhat viable, depending on whether the individual members could be resized to cut down the overall weight, which would also impact the foundations. Because the layout of the SAC Hotel is perfect for a staggered truss, the system would need to be investigated much further.

The precast planks on steel beams seems to be the most attractive alternative due to the small system depth and cost. Altering the bay size by a minimal amount should not be too much of a concern, and the overall weight is relatively close to the existing, so foundation redesign could be minimal.

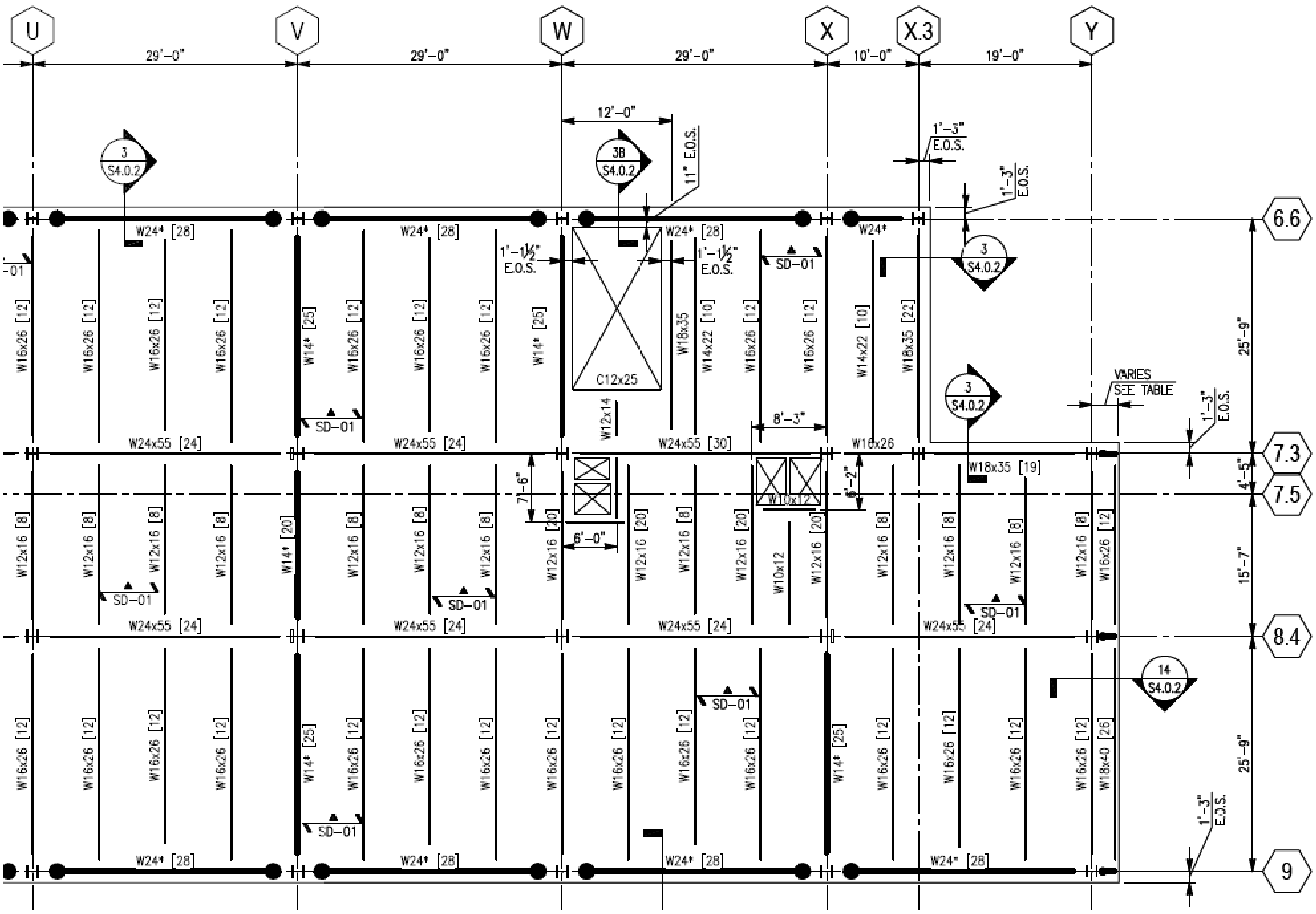
Lateral analysis will need to be investigated to determine a solid alternative.

Appendices

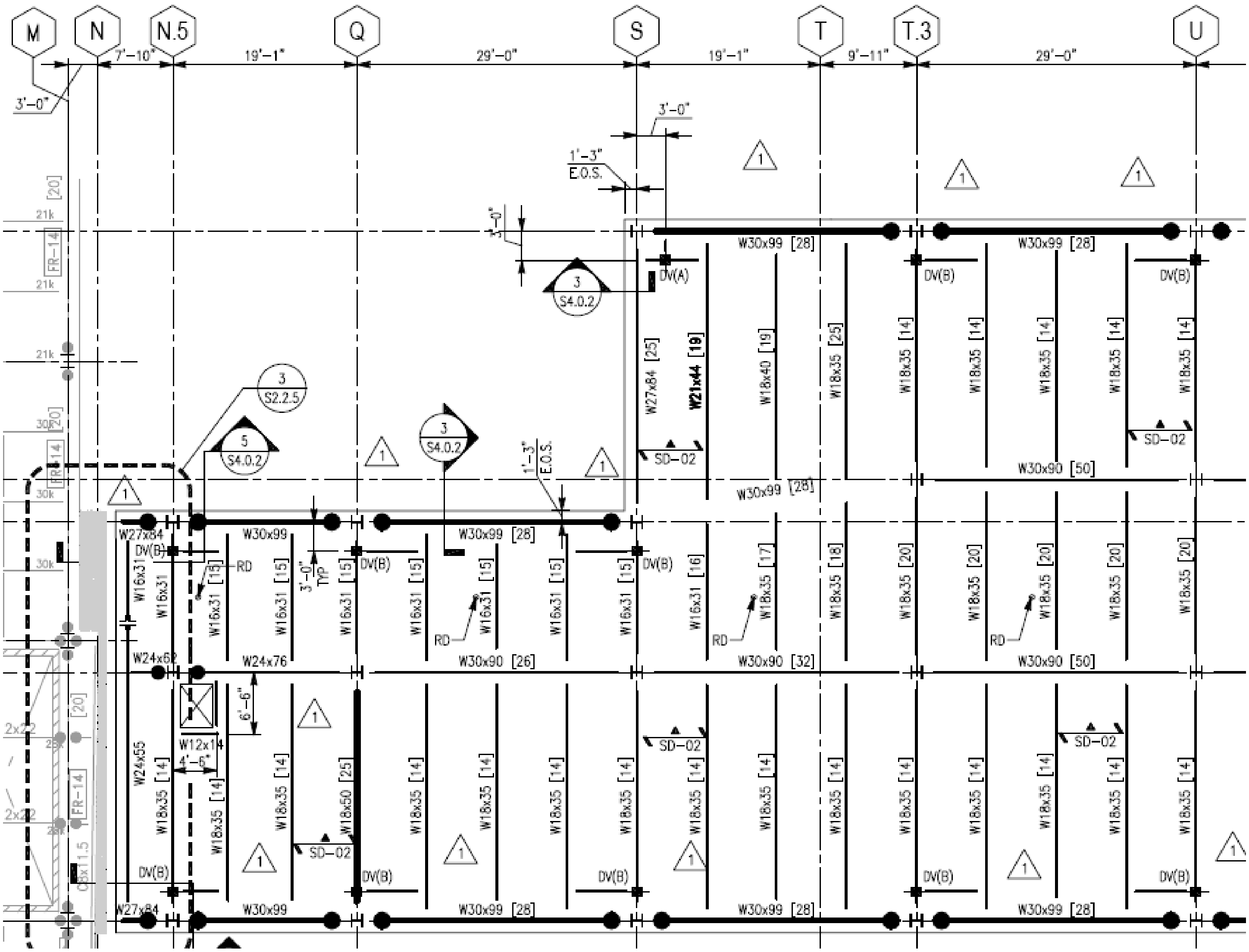
4th thru 10th Floor (Partial)



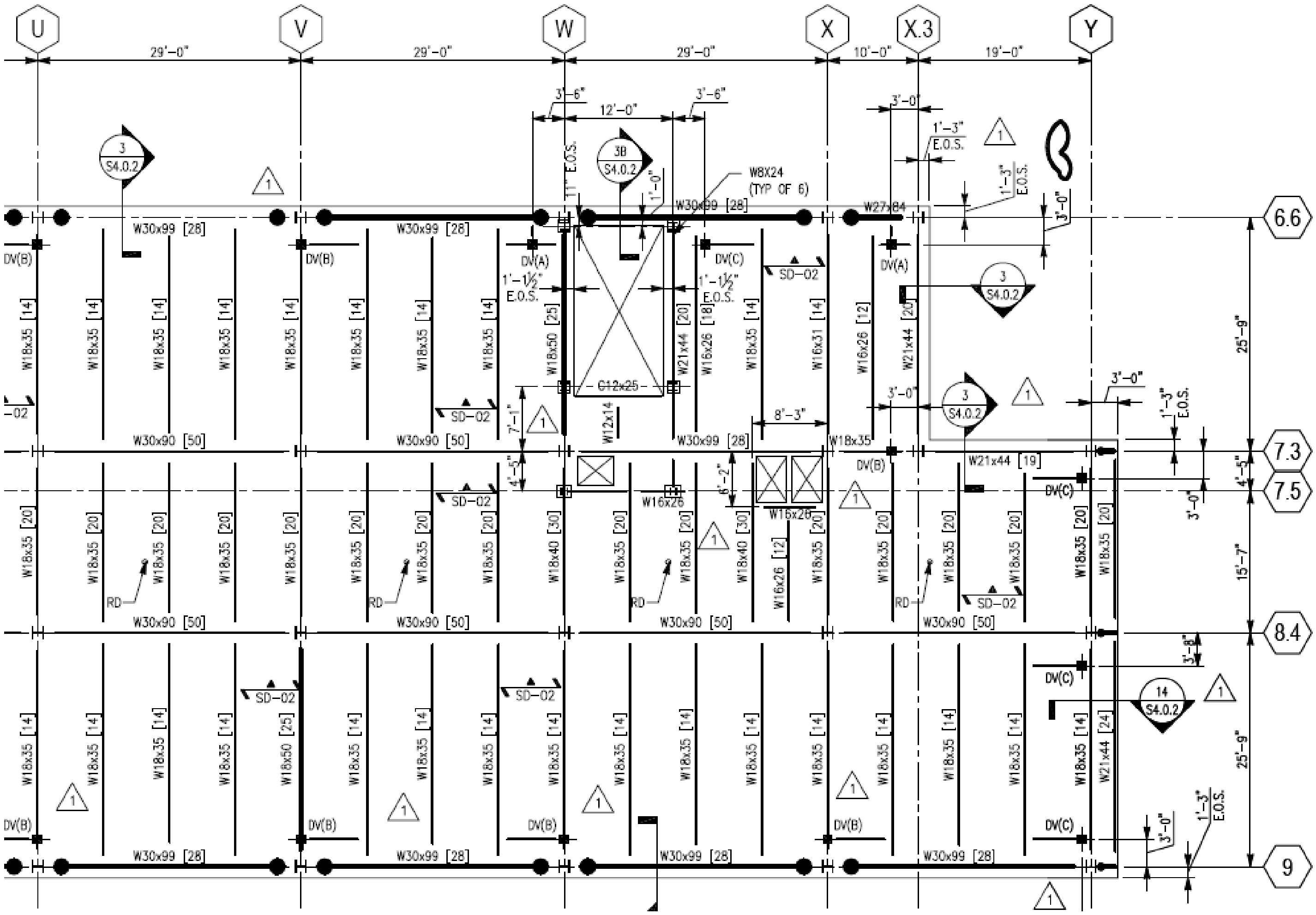
4th thru 10th Floor (Partial)



Roof (Partial)

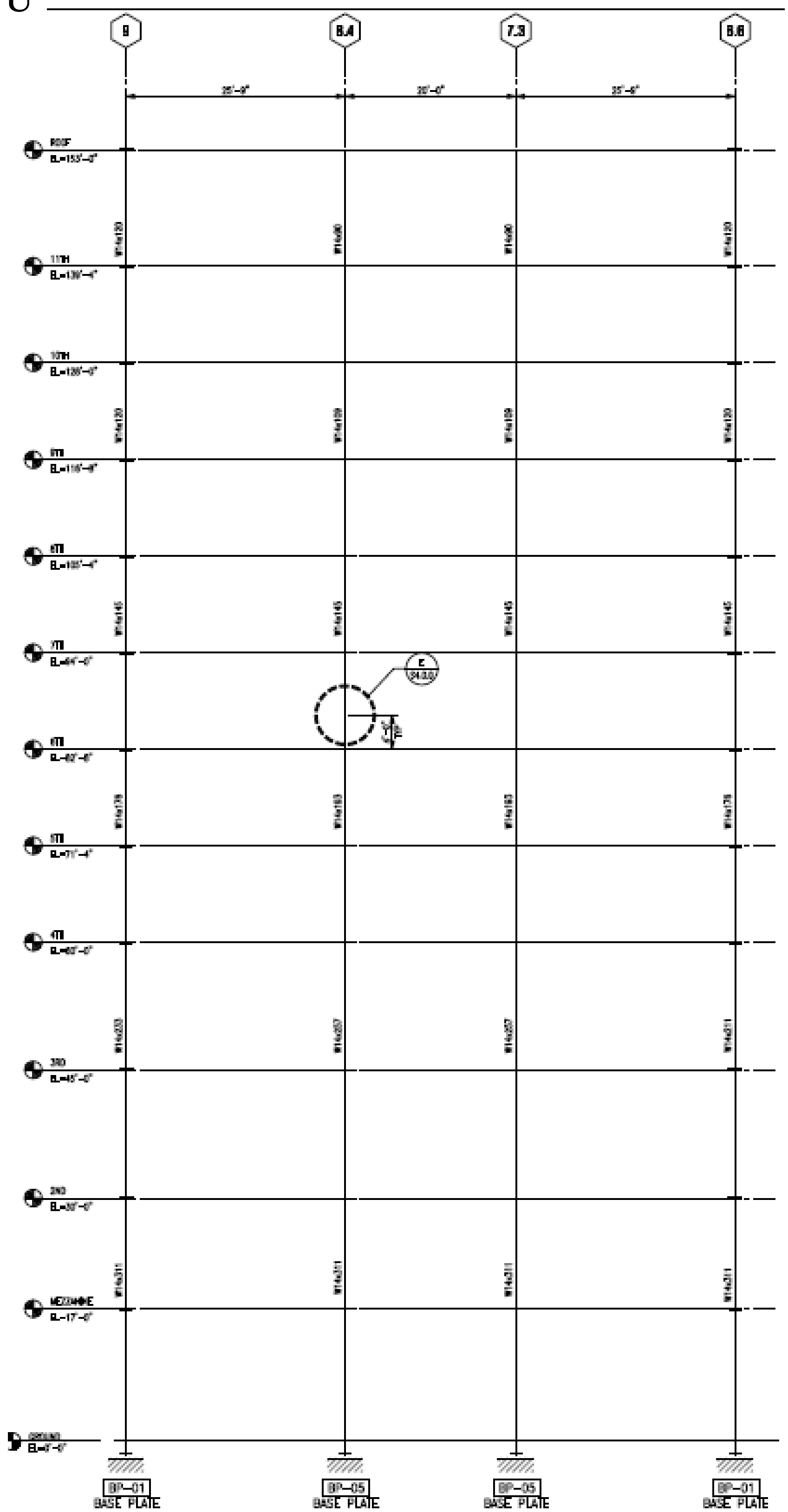


Roof (Partial)

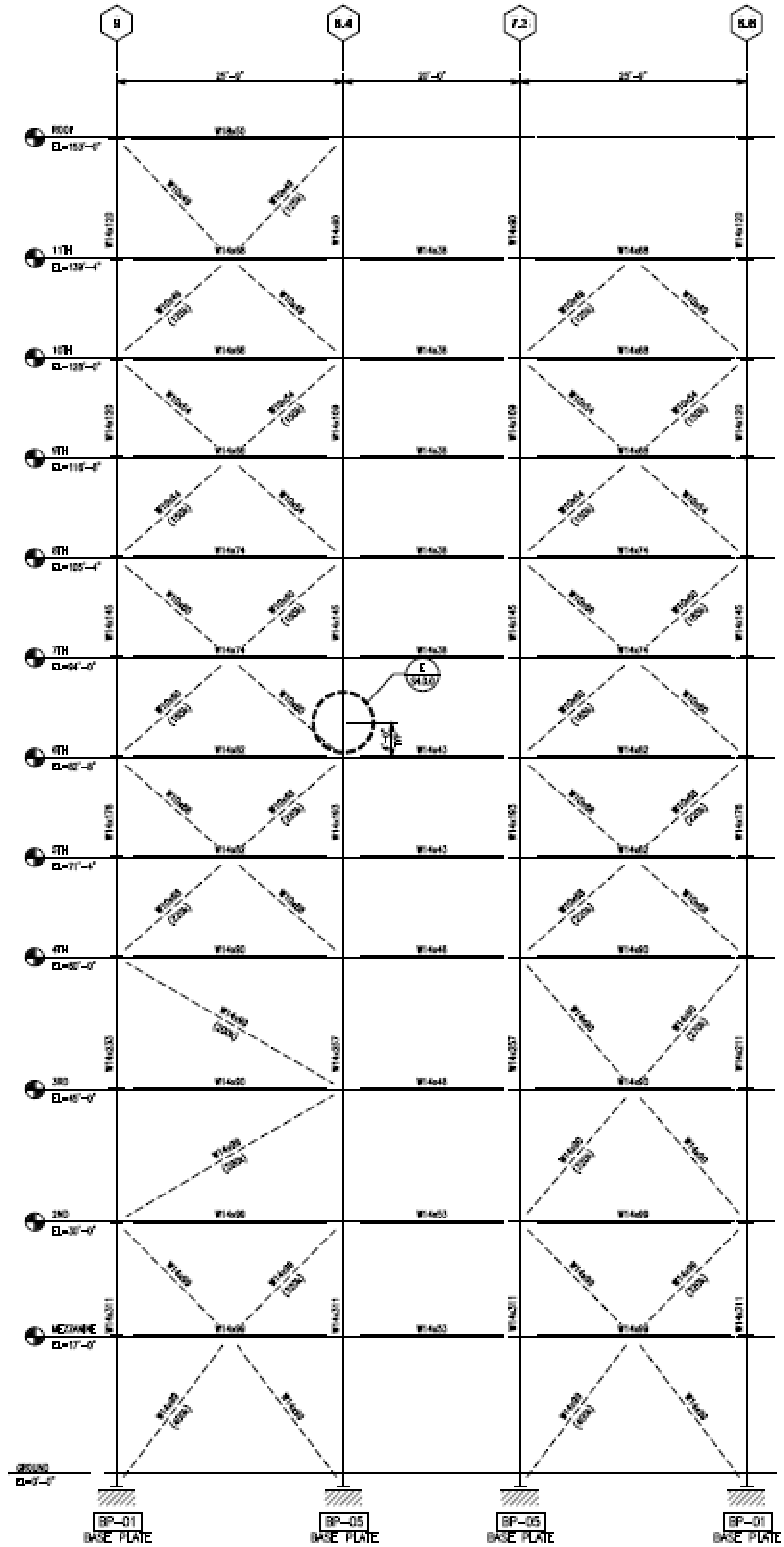


Appendix B - Elevations

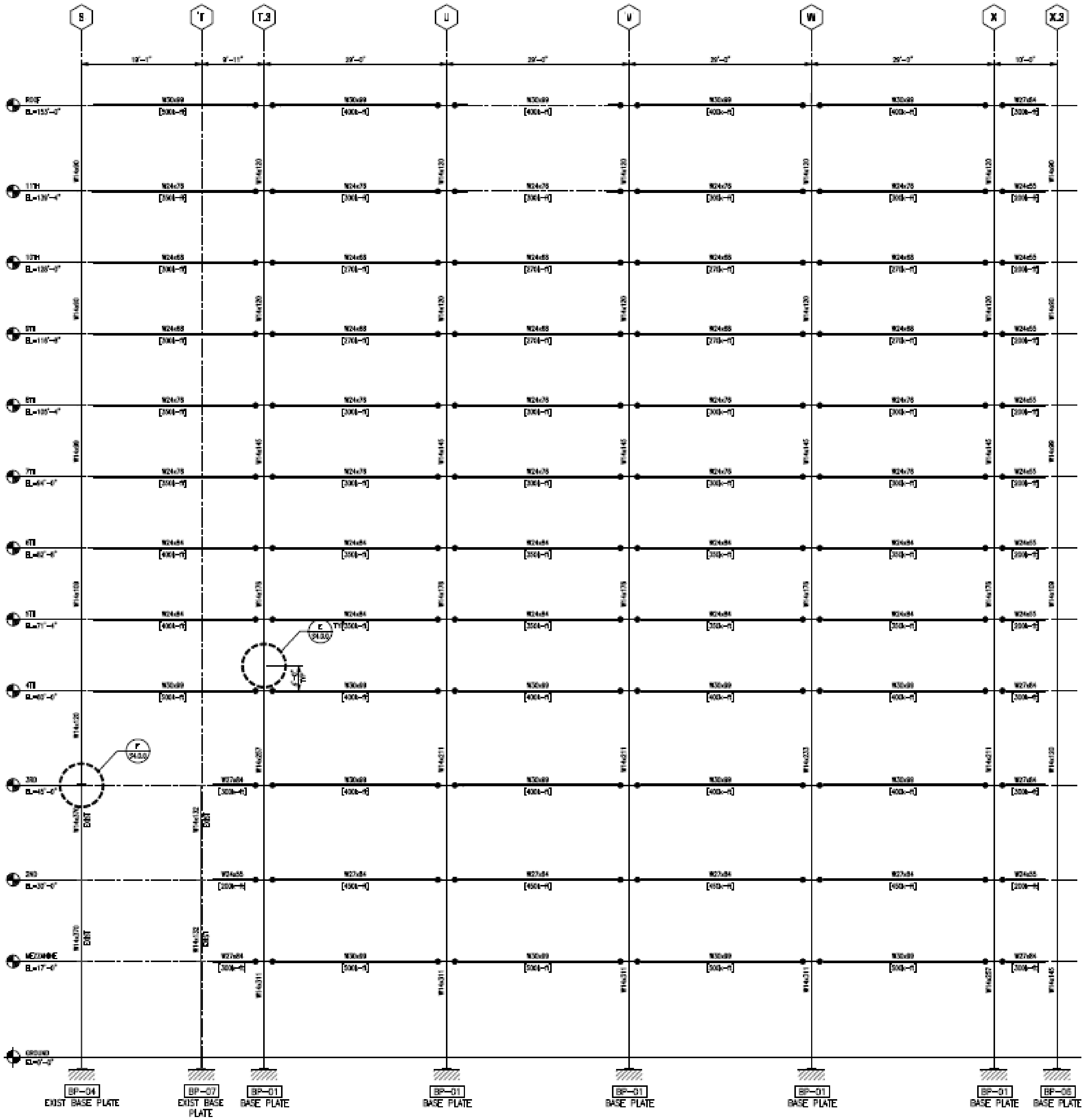
Column Line U



Column Line V

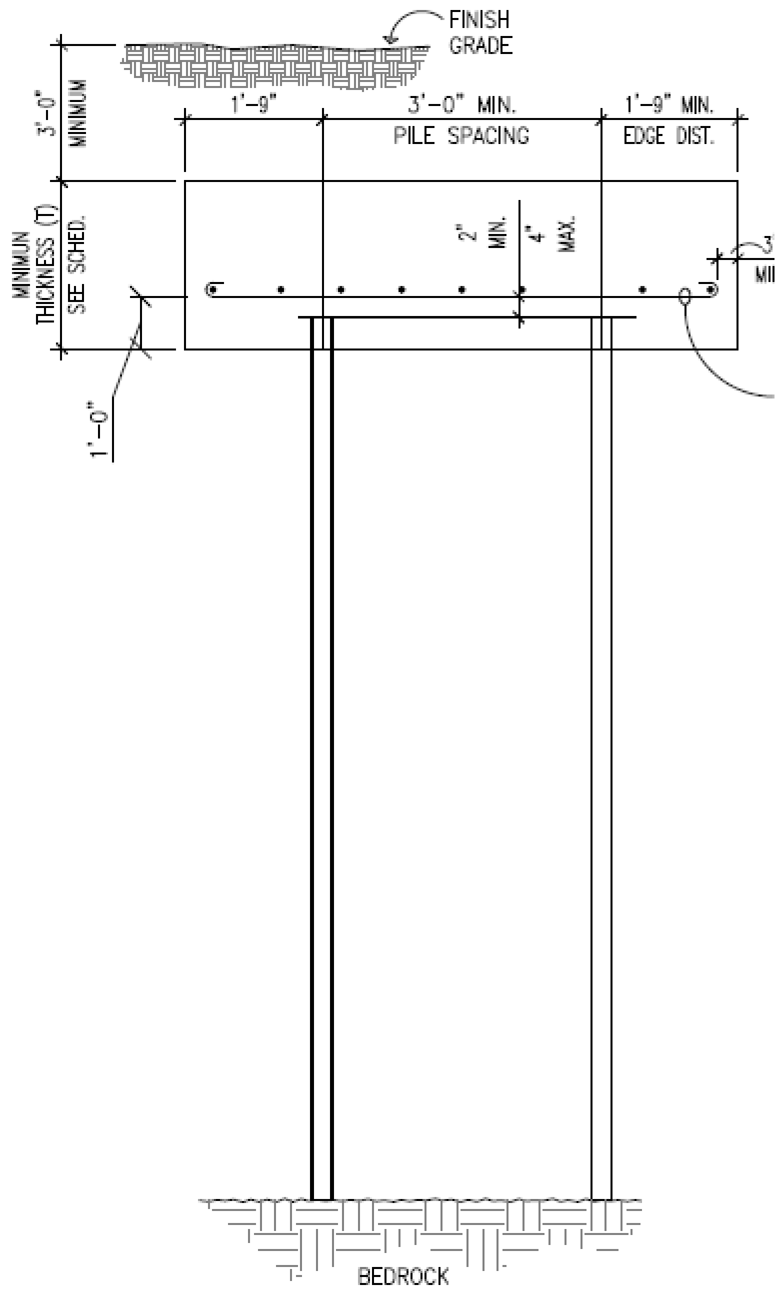


Column Line 6.6

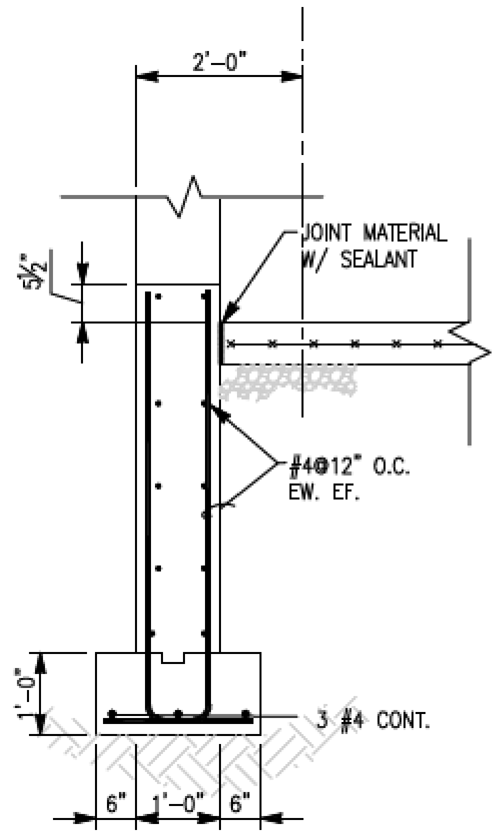


Appendix C - Sections

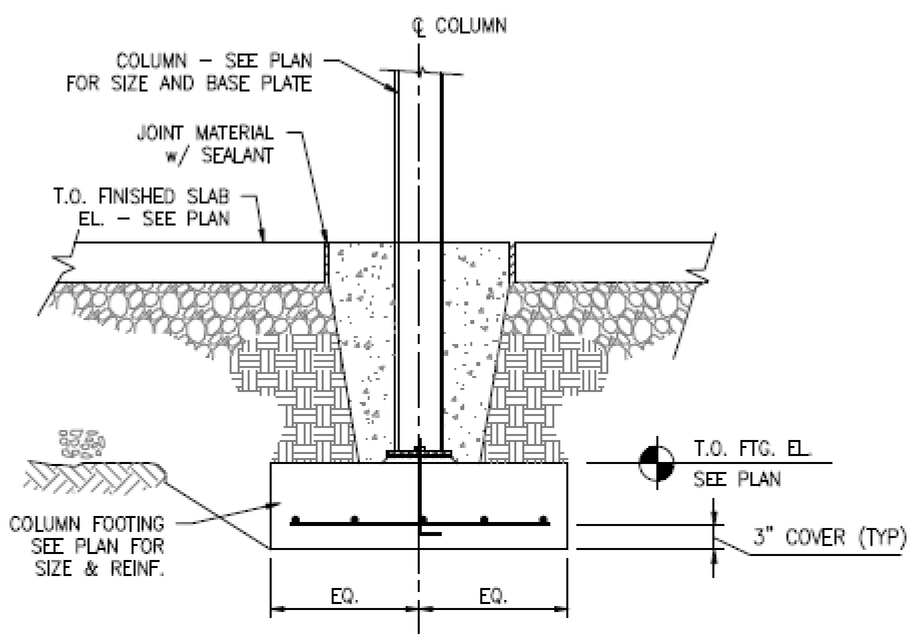
Steel Piles



Footings

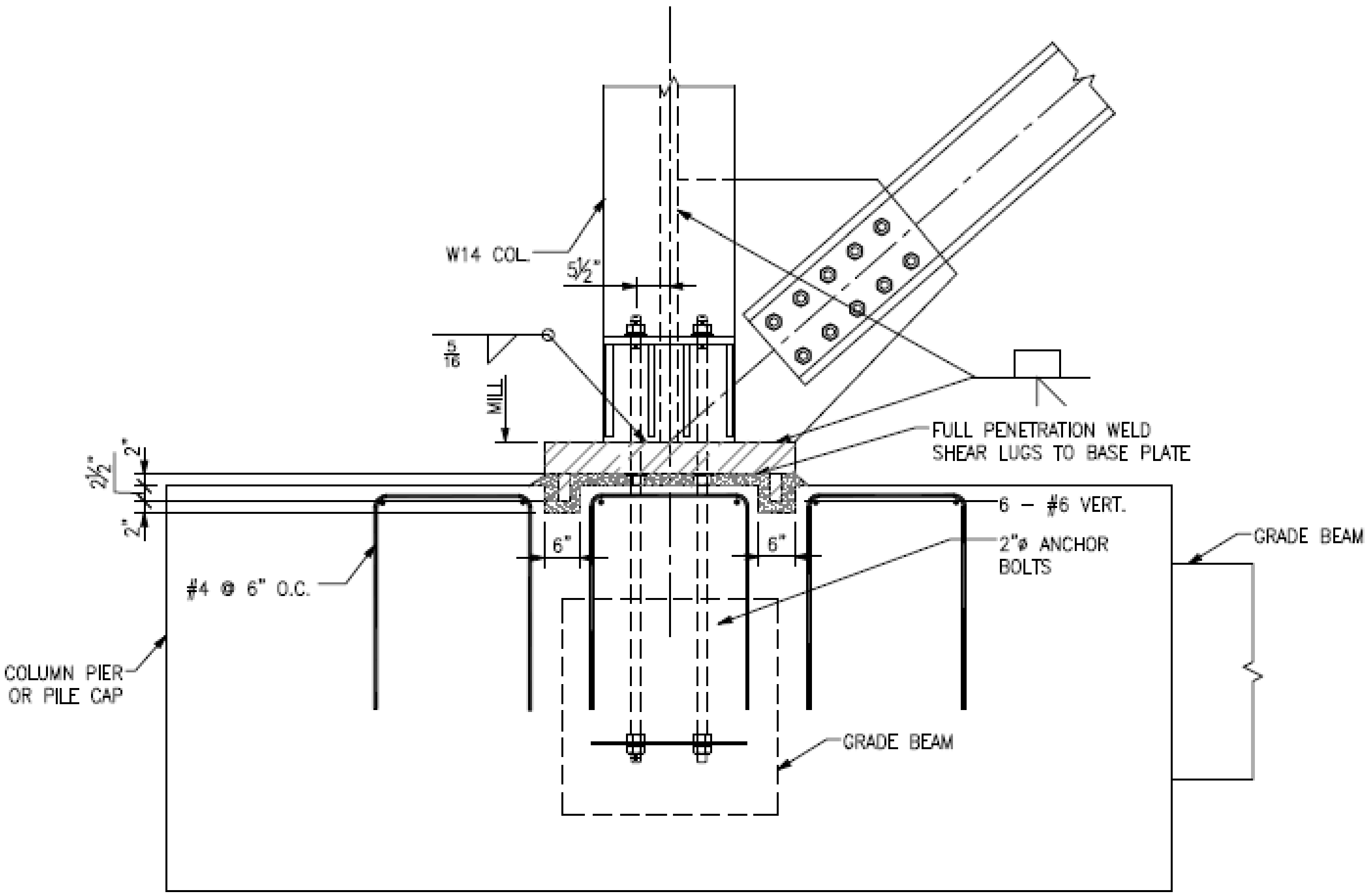


Typical Strip Footing



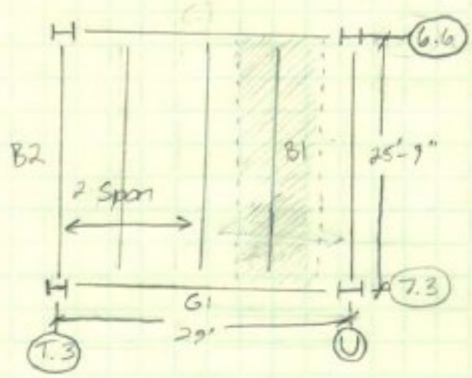
Typical Interior Footing Without Pier

Brace At Foundation



Nick Reed | Tech. Report 1 | Spot Checks | 1/4

Typical Bay (Hotel Room, Floor 6)



G1: W24x55

B1: W16x26

B2: W14x82

$\frac{14.5}{2} = 7.25'$

Composite Deck, 6.5" total depth
(2 span, NWC, 2VLI20, allow span = 8'9")

Loads

Deck = 69 psf

SDL = 10(susp. mech.) + 5(ceiling) = 15 psf

Beam Self WT = 5 psf

LL = 80 psf (includes partitions according to specs.)

Beam B1

$A_t = 25.75' \times 7.25'$

$A_t = 186.7\ ft^2$

$K_{LL} = 2 \Rightarrow K_{LL}A_T = 2(186.7) = 373.4\ ft^2$ No reduction needed

Total DL = 89

$W_U = 1.2(89) + 1.6(80) = 234.8\ psf$

$W_U = 234.8(7.25')/1000$

$W_U = 1.70\ klf$

$M_U = \frac{w_u l^2}{8} = \frac{1.70(25.75)^2}{8} = 140.9\ ft \cdot k < 166\ ft \cdot k$

Drawing show 1 stud/rib, assume 3/4" ϕ, Deck $\perp$ to beam, weak pos.

$f'_c = 3500\ psi \Rightarrow Q_n = 17.2\ k$ (Table 3-21)

b_{eff}

$b_1' = \min\left[\begin{array}{l} 25.75(12)/8 \\ \frac{7.25(12)}{2} \end{array}\right] = \min\left[\begin{array}{l} \boxed{38.63"} \\ 43.5" \end{array}\right]$ $\quad b_2' = b_1'$

$b_{eff} = 77.26"$

Nick Reed | Tech Report 2 | Tech 1 Spot Checks Revised | 1/3

Beam B1 (design Full composite)

Let $a = 1.0$ $Y2 = t - a/2 = 6.5 - 1/2 = 6''$

$A_{sc}F_u = \pi((3/8)^2(65)) = 28.72k$

$F_u = 65$ ksi
$R_p = 0.6$
$R_g = 1.0$

$Q_n = \min \begin{cases} 17.2 \\ (1.0)(.6)(28.72) = 17.23 \end{cases}$

W16x26 $M_u = 252$ ft·k $\quad 96/17.2 = 6 \times 2 =$ 12 studs

$\Sigma Q_n = 96$ k

Check a

$a = \dfrac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \dfrac{96}{0.85(3.5)(77.26)} = 0.42'' < 1.0''$ OK

Deflections

$W_{LL} = 80 \times 7.25 = .580$ klf

$I_{LB} = 795$ (Table 3-20)

$\Delta_{LL} = \dfrac{5wl^4}{384EI} = \dfrac{5(.580)(25.75)^4(1728)}{384(29000)(795)} = 0.25''$

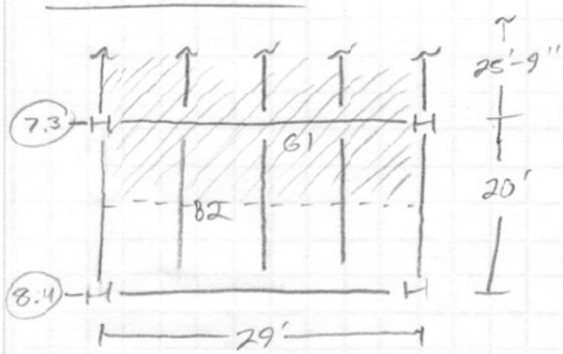
$\Delta_{allow} = \dfrac{L}{360} = \dfrac{25.75(12)}{360} = 0.86'' > .25$ OK

$\Delta_{TL} = \dfrac{5(1.70)(25.75)^4(1728)}{384(29000)(795)} = 0.73''$

$\dfrac{L}{240} = \dfrac{25.75(12)}{240} = 1.29'' > 0.73''$ OK

$\boxed{\text{W16x26 is adequate for loads}}$

Girder G1 (full composite and bay size/load correction)



$A_T = \left(\frac{20+25.75}{2}\right)(29') = 664\ ft^2$

LL reduction permitted

$K_{LL} = 2 \qquad B1: LL = 80psf$

$LL = 80\left(.25 + \frac{15}{\sqrt{2(664)}}\right) = 52.93$

B1 $\Rightarrow w_u = 1.39\ klf \qquad P_u = 17.9\ k$

B2 Loads

Deck = 69 psf

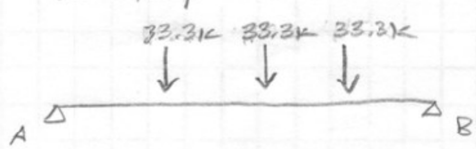
SDL = 15 psf

B.S.W = 5 psf

LL = 100 psf (corridor)

$W_u = [1.2(89psf) + 1.6(66.2psf)](7.25') = 1.54\ klf$

$P_u = \frac{1.54(20')}{2} = 15.4\ k$



Simplified reaction EQ's (Table 3-22a)

$R_A, R_B = 1.5(33.3) = 49.95\ k$

$M = 0.5(33.3)(29') = 483\ ft\text{-}k$

$LL = 100\left(.25 + \frac{15}{\sqrt{2(664)}}\right) = 66.2psf$

$483 < 503 \Rightarrow$ check composite

Nick Reed | Tech Report 2 | Tech 1 Spot Checks Revised | 2/3

From Tech 1, G1, $b_{eff} = 87''$

Let $a = 1.0''$, $Y2 = 6''$, $A_{sc}f_u = 20.72k$

Table 3-19, W24x55 $M_u = 735$ ft·k

$\Sigma Q_n = 203k$

$\#studs = 203/17.2 = 11.8 \times 2 = \boxed{24 \text{ studs}}$

Check a

$$a = \frac{203}{.85(3.5)(87)} = 0.78'' < 1.0 \quad \text{OK}$$

Deflections

$P_{LL} = 12.5k$

$I = 1350 \text{ in}^4$

$$\Delta_{LL} = \frac{0.05PL^3}{EI} = \frac{0.05(12.5)(29)^3(1728)}{(29000)(1350)} = 0.67''$$

$$\Delta_{allow} = \frac{L}{360} = \frac{29(12)}{360} = 0.97'' > 0.67'' \quad \text{OK}$$

$P_{TL} = 33.3k$

$$\Delta_{TL} = \frac{0.05(33.3)(29)^3(1728)}{(29,000)(1350)} = 1.79''$$

$$\Delta_{TL} \le \frac{L}{240} = \frac{29(12)}{240} = 1.45'' < 1.79'' \quad \text{NG}$$

Nick Reed | Tech Report 2 | Tech 1 Spot Checks Revised 3/3

Beam B1 wet concrete defl.

$$w_c = 69\,psf(7.25') + 35\,plf = 0.535\ klf$$

$$\Delta_{w_c} = \frac{5(.535)(25.75)^4(1728)}{384(29{,}000)(795)} = 0.23''$$

$$\Delta_{w_{c\,allow}} \leq \frac{l}{240} = \frac{25.75(12)}{240} = 1.29'' > 0.23'' \quad OK$$

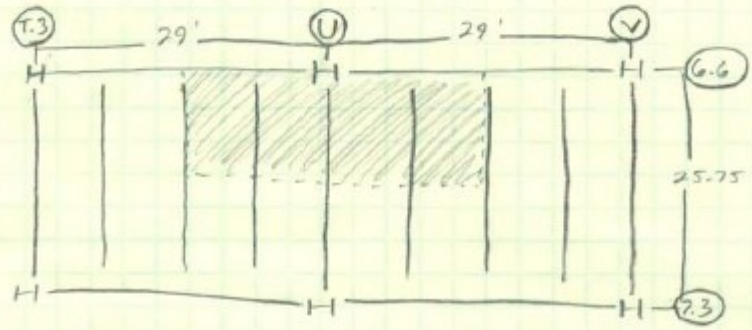
G1

$$\Delta_{w_c} = \frac{5(.535)(29)^4(1728)}{384(29000)(1350)} = 0.22''$$

$$\Delta_{w_{c\,allow}} \leq \frac{l}{240} = \frac{29(12)}{240} = 1.45'' > .22'' \quad OK$$

Nick Reed | Tech Report 1 | Spot Checks 4/4

Column (4th Floor)



$A_T = (29')(12.88') = 373.5\ ft^2$

$K_{LL} = 4$ (Ext. column w/o cant.)

$K_{LL}A_T = 4(373.5) = 1494\ ft^2 > 400\ ft^2$ Reduce LL

LL = 80 psf in hotel rooms

$$L = 80\left[.25 + \frac{15}{\sqrt{1494}}\right] = 51\ psf$$

Dead Loads

SDL = 10 + 5 + 15 (facade) = 30 psf

Deck = 69 psf

Total = 99 psf ≈ 100 psf

Snow = 40 psf

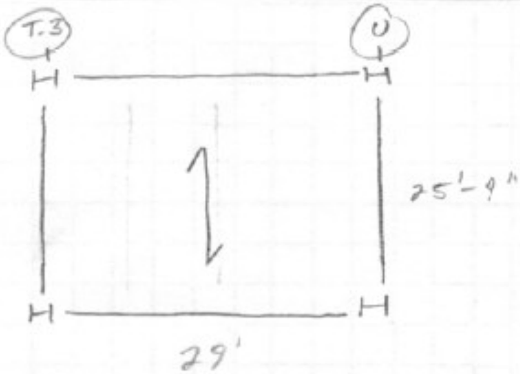
See spreadsheet for total loads on floors above 4th

Mech Live Load = 200 psf

$$L = 200\left[.25 + \frac{15}{\sqrt{1494}}\right] = 127.6\ psf$$

Load combo = 1.2D + 1.6L + 0.5S

Nick Reed | Tech Report 2 | Hollow Core Planks on steel | 1/2



2 HR Fire rating req.

Try 8"x4' Hollow core Plank w/ 2" top

SDL = 15 psf

DL = 61.25 psf (see specs)

LL = 80 psf

$w_u = 1.2(76.25) + 1.6(80) = 219.5$ psf

25' Span only supports 202 psf <u>NG</u> try larger size

<u>10"x4' w/ 2" top</u>

DL = 68 psf

$W_u = 1.2(68+15) + 1.6(80) = 228$ psf

From specs, shortest span available = 26'

26' span, 7-1/2"Ø strands carries 246 psf > 228 OK

* Using precast planks would require changing bay size, losing space in rooms as well as corridor *

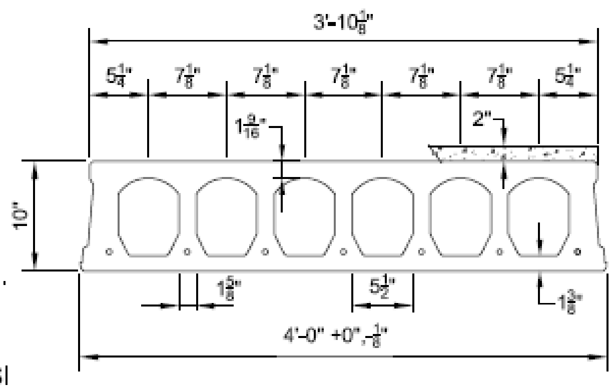
Nitterhouse plank specs can be found in appendices

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.

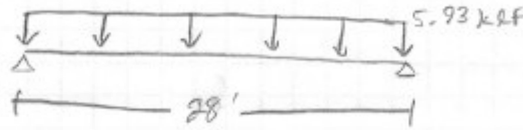
11/03/08

10F2.0T

Nick Reed | Tech Report 2 | Hollow Core Planks on Steel | 2/2

Girder design for planks

SDL = 15 psf
DL = 68 psf
LL = 80 psf



$w_u = 228 \text{ psf} \times 26' \text{ span}$

$w_u = 5.93 \text{ klf}$

$V_u = 83.02 \text{ k}$

$M_u = \frac{w l^2}{8} = \frac{5.93(28)^2}{8} = 581.1 \text{ ft-k}$

Try W18x106 (Table 3-10, unbraced 28')

$\phi M_n = 591 \text{ ft-k}$

Deflections

$I = 1910 \text{ in}^4$

$\Delta_{LL} = \frac{5 w l^4}{384 E I} = \frac{5(2.33)(28)^4(1728)}{384(29{,}000)(1910)} = 0.83''$

$\Delta_{LL} \le \frac{l}{360} = \frac{28(12)}{360} = 0.93 > 0.83$ OK

$\Delta_{TL} = \frac{5(5.93)(28)^4(1728)}{384(29000)(1910)} = 1.48''$

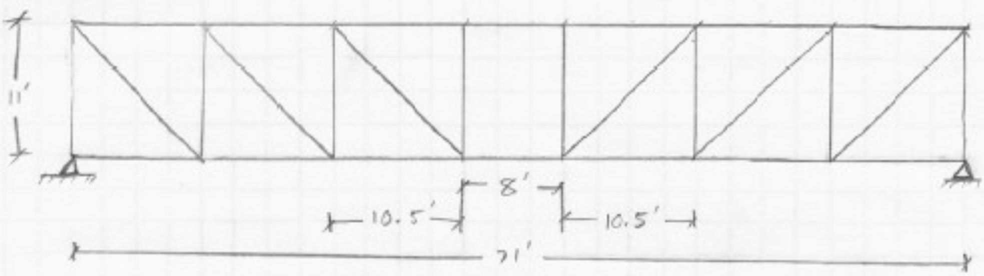
$\Delta_{TL} \le \frac{l}{240} = \frac{28(12)}{240} = 1.4 < 1.48$ NG

Try W18x119, $I = 2190 \text{ in}^4$

$\Delta_{TL} = \frac{5(5.93)(28)^4(1728)}{384(29{,}000)(2190)} = 1.29'' < 1.4''$ OK

$\boxed{\text{Use W18x119}}$

Nick Reed | Tech Report 2 | Staggered Truss



*Shorten building width by 1'

SDL = 15 psf

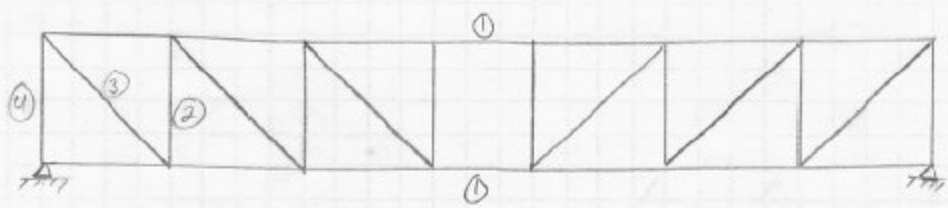
Planks = 68 psf

LL = 80 in rooms, 100 for corridor => 100 for simplification

$W_u = [1.2(83) + 1.6(100)]\,26' = 6.75\ \text{klf}$

Modeled in STAAD, w/ deflection controlling

$\Delta_{TL} = \frac{l}{240} = \frac{71(12)}{240} = 3.55''$



1 W24x450

2 W21x147

3 W10x88

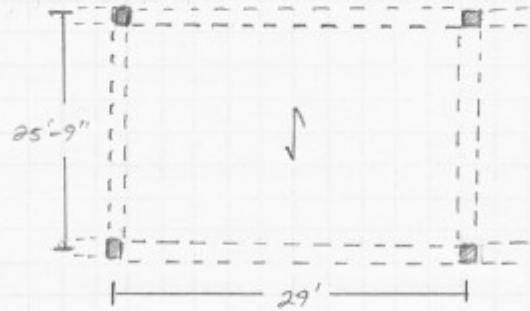
4 W24x131

Total Load Defl = 3.3"

*System requires very heavy members and significant bay size changes
Most likey not viable *

Nick Reed | Tech Report 2 | One-Way Slab 1/4

Assume: NWC
$f'_c = 4000$ psi
$f_y = 60$ ksi
Cols = 20"x20"
Clr Cover = 2"
Exterior, one end continuous



ACI 318-11, Table 9.5(a)

$$h_{min} = \ell/24 = \frac{25.75(12)}{24} = 12.9 \Rightarrow 13''$$

Loads:

SDL = 15 psf
LL = 80 psf (hotel room bay)
Slab SW = (150 pcf)(13/12) = 162.5 psf

Beam Design:

$$w_u = 1.2(15+162.5)+1.6(80) = 341 \text{ psf}$$

$$\frac{341 \times 25.75'}{1000} = 8.78 \text{ klf}$$

self wt check

$$M_u = \frac{w_u \ell_n^2}{14} = \frac{8.78(25.75-20/12)^2}{14} \times 1.1 = 400.1 \text{ k-ft}$$

Estimate size

$$M_u = \frac{bd^2}{20} \quad \text{Try } b = 20'' \qquad 20(400.1) = 20d^2$$

$$d = 20''$$

$$h = d + 2.5 = 22.5''$$

$$bd^2 = 20(20)^2 = 8000 \text{ in}^3$$

Nick Reed | Tech Report 2 | One-Way Slab 2/4

Beam Self Weight:

$$w = \frac{20(22.5)(150)}{144} = 469 \text{ plf}$$

$$w_u = 8.78 \text{ klf} + 1.2(.469) = 9.34 \text{ klf}$$

$$M_u = \frac{9.34(25.75 - 20/12)^2}{14} = 387 \text{ k-ft} \qquad 387 \times 20 = 7740 \text{ in}^3 < 8000$$

OK

Required Steel:

$$A_s = \frac{M_u}{4d} = \frac{387}{4(20)} = 4.84 \text{ in}^2 \qquad \text{Try } (5)\#9 = 5 \text{ in}^2$$

M_n:

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{5(60)}{.85(4)(20)} = 4.41'' \qquad d_b = 20 - 1.128 = 18.9''$$

$$c = \frac{a}{\beta_1} = \frac{4.41}{.85} = 5.19''$$

$$\varepsilon_s = \varepsilon_u\left(\frac{d-c}{c}\right) = .003\left(\frac{18.9 - 5.19}{5.19}\right) = 0.0079 > .00207 \text{ OK}$$

$$\varepsilon_t > .005 \Rightarrow \phi = 0.9$$

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

$$M_n = 5(60)(18.9 - 4.41/2)$$

$$M_n = 417.4 \text{ k-ft}$$

$$\phi M_n = 376 \text{ k-ft} < M_u \quad \text{NG} \qquad \text{Try } (5)\#10 \quad A_s = 6.35 \text{ in}^2$$

$$d_b = 20 - 1.27 = 18.73''$$

$$a = \frac{6.35(60)}{0.85(4)(20)} = 5.6'' \qquad c = \frac{5.6}{.85} = 6.59$$

$$\varepsilon_s = .003\left(\frac{18.73 - 6.59}{6.59}\right) = .0055 > .00207 \quad \text{OK} \qquad \varepsilon_t > .005 \rightarrow \phi = 0.9$$

$$M_n = 6.35(60)(18.73 - 5.6/2)$$

$$M_n = 505.8$$

$$\phi M_n = 455.2 \text{ k-ft} > M_u \quad \text{OK}$$

Nick Reed | Tech Report 2 | One-Way slab 3/4

Min Reinf:

$$A_{s\,min} = \frac{200}{F_y} bd = \frac{200}{60000}(20)(18.73) = 1.25\ in^2$$

$$A_s = 6.35 > 1.25\ in^2 \quad OK$$

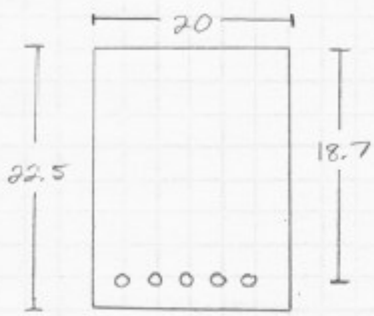
Max Reif:

$$\rho_{max} = 0.85\beta_1 \frac{f'_c}{F_y} \frac{\varepsilon_u}{\varepsilon_u + .005}$$

$$\rho_{max} = 0.85(.85)\frac{4}{60}\left(\frac{.003}{.008}\right)$$

$$\rho_{max} = .0181$$

$$\rho = \frac{A_s}{bd} = \frac{6.35}{(20)(18.73)} = .017 < .0181 \quad OK$$



Use a 20" x 22.5" beam with (5) #10 bars

Nick Reed | Tech Report 2 | One-Way Slab 4/4

Slab:

$t = 13''$ Try #4 bars

Check 1' wide strip

$A_{s,min} = 0.002bh = .002(12'')(13'') = 0.312\ in^2/ft$

Start with 10'' spacing o.c. (ACI 318-11, 10.6.4)

#4 A_s too small, Try #5 bars, $A_s = \frac{.31 \times 12}{10''} = .372\ in^2/ft$

Assume $d = 11.5''$

$\rho = \frac{A_s}{bd} = \frac{.372}{(12)(11.5)} = .0027$

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{.372(60)}{0.85(4)(12)} = 0.55$

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

$= 0.9(60)(11.5 - 0.55/2)$

$= 50.5\ k \cdot ft$

$W_u = 1.2(15 + 162.5) + 1.6(80) = \frac{341\ psf \times 1ft}{1000} = .341\ klf$

$M_u = \frac{w_u l_n^2}{8} = \frac{.341(25.75)^2}{8} = 28.3\ k \cdot ft$

$\phi M_n > M_u$ OK

13'' slab w/ #5 bars spaced 10'' o.c.

Appendix E - RSMeans Assemblies

Assembly B10102564000 **Based on National Average Costs**

Floor, composite metal deck, shear connectors, 6.25" slab, 25'x30' bay, 30.25" total depth, 200 PSF superimposed load, 252 PSF total load

Description	Quantity	Unit	Material	Installation	Total
Welded wire fabric, sheets, 6 x 6 - W1.4 x W1.4 (10 x 10) 121 lb. per C.S.F., A185, incl...	0.01100	C.S.F.	0.17	0.40	0.56
Structural concrete, placing, elevated slab, pumped, less than 6" thick, includes strike...	0.39500	C.F.	0.00	0.60	0.60
Structural concrete, ready mix, lightweight, 110 #/C.F., 3000 psi, includes local aggre...	0.39500	C.F.	2.86	0.00	2.86
Concrete finishing, floors, for specified Random Access Floors in ACI Classes 1, 2, 3 an...	1.00000	S.F.	0.00	0.86	0.86
Concrete surface treatment, curing, sprayed membrane compound	0.01000	C.S.F.	0.08	0.09	0.17
Weld shear connector, 3/4" dia x 4-7/8" L	0.19500	Ea.	0.14	0.39	0.54
Structural steel project, apartment, nursing home, etc, 100-ton project, 3 to 6 stories,...	7.20600	Lb.	10.09	3.10	13.19
Metal floor decking, steel, non-cellular, composite, galvanized, 3" D, 16 gauge	1.05000	S.F.	3.82	1.17	4.99
Metal decking, steel edge closure form, galvanized, with 2 bends, 12" wide, 18 gauge	0.03700	L.F.	0.15	0.09	0.24
Sprayed fireproofing, cementitious, normal density, beams, 1 hour rated, 1-3/8" thick...	0.67700	S.F.	0.39	0.67	1.06
Total			$17.70	$7.37	$25.07

Assembly B10102303300 **Based on National Average Costs**

Precast concrete plank, 2" topping, 10" total thickness, 25' span, 100 PSF superimposed load, 180 PSF total load

Description	Quantity	Unit	Material	Installation	Total
C.I.P. concrete forms, elevated slab, edge forms, to 6" high, 4 use, includes shoring, e...	0.10000	L.F.	0.02	0.41	0.43
Welded wire fabric, sheets, 6 x 6 - W1.4 x W1.4 (10 x 10) 121 lb. per C.S.F., A185, incl...	0.01000	C.S.F.	0.15	0.36	0.51
Structural concrete, ready mix, normal weight, 3000 psi, includes local aggregate, san...	0.17000	C.F.	0.71	0.00	0.71
Structural concrete, placing, elevated slab, pumped, less than 6" thick, includes strike...	0.17000	C.F.	0.00	0.26	0.26
Concrete finishing, floors, basic finishing for unspecified flatwork, bull float, manual fl...	1.00000	S.F.	0.00	1.13	1.13
Concrete surface treatment, curing, sprayed membrane compound	0.01000	C.S.F.	0.08	0.09	0.17
Precast slab, roof/floor members, grouted, hollow, 8" thick, prestressed	1.00000	S.F.	7.85	2.52	10.37
Total			$8.80	$4.77	$13.57

Assembly B10102196800 **Based on National Average Costs**

Cast-in-place concrete beam and slab, 6.5" slab, one way, 20" column, 25'x30' bay, 200 PSF superimposed load, 312 PSF total load

Description	Quantity	Unit	Material	Installation	Total
C.I.P. concrete forms, beams and girders, exterior spandrel, plywood, 12" wide, 4 use...	0.19800	SFCA	0.18	2.03	2.21
C.I.P. concrete forms, beams and girders, interior, plywood, 12" wide, 4 use, includes...	0.39000	SFCA	0.42	3.28	3.70
C.I.P. concrete forms, elevated slab, flat plate, plywood, to 15' high, 4 use, includes s...	0.85800	S.F.	0.98	4.85	5.83
Reinforcing Steel, in place, elevated slabs, #4 to #7, A615, grade 60, incl labor for acc...	4.78400	Lb.	2.68	2.06	4.74
Structural concrete, ready mix, normal weight, 3000 psi, includes local aggregate, san...	0.74500	C.F.	3.10	0.00	3.10
Structural concrete, placing, elevated slab, pumped, 6" to 10" thick, includes strike of...	0.74500	C.F.	0.00	0.96	0.96
Concrete finishing, floors, for specified Random Access Floors in ACI Classes 1, 2, 3 an...	1.00000	S.F.	0.00	0.86	0.86
Concrete surface treatment, curing, sprayed membrane compound	0.01000	C.S.F.	0.08	0.09	0.17
Total			$7.45	$14.13	$21.58

Note: Tables taken from RSMeans CostWorks and are as close as possible estimates to actual systems designed, except the concrete slab which only had a 6.5" thickness available for bay size used