Contents

Executive Summary

The purpose of Technical Report 2 is to explore alternative floor systems and compare them the existing system in the Hotel N.E.U.S. A typical bay that spans 26'-8" between column line C and E and 27'-8" between column line 5 and 7 was selected analysis. These values are rounded to whole numbers and a 28'x27' bay is used for all calculations and models. A comparison of general conditions (weight, cost, depth), serviceability (deflection, vibration), architectural (fire rating, fire protection, ceiling, mecanical), structural (foundation, lateral system, building height), and construction impact (schedule, constructability) is performed between all four systems. The existing floor in the Hotel N.E.U.S. is composed of precast 8" hollowcore planks that sit on 8" thick masonry bearing walls. The three alternates considered in this report consist of:

- Composite Deck on Composite Steel Beams and Girders
- One way concrete slab with beams
- Precast Hollowcore Planks on Staggered Truss

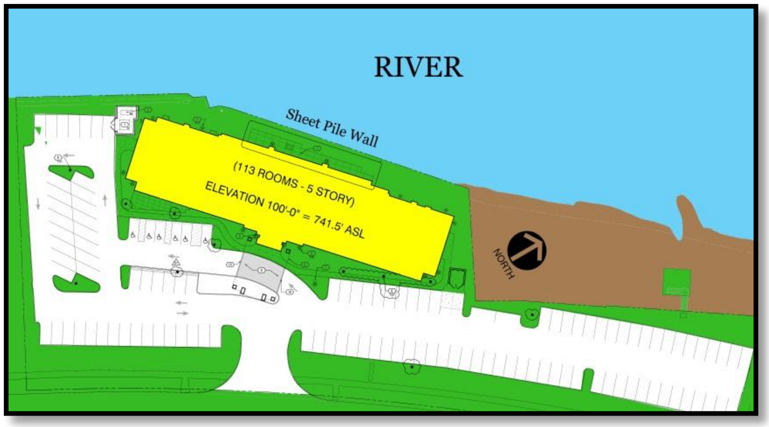
The composite steel system design results in W12x26 beams that divide the 28' span into three 9'-4" sections. The two interior beams are supported by a W18x35 girder. To accomplish a 2 hour fire rating, a Vulcraft 3", 22 gauge interlocking deck with 2.5" of topping was selected to achieve a 2 hour fire rating with sprayed fiber. This system can significantly reduce the amount of foundations required and allows for the required large open spaces on the ground floor. However, due to the low live loads, a large amount of the composite strength is not utilized. Deflections control the size and a low stud count is required. A drop ceiling would also be required due to the beam depth. This system is a viable option for this building.

The floor layout for this building lends itself to a one way concrete system. To limit the thickness of the slab a 16"x18" beam with (6)#6 bottom bars, (4)#7 top bars, and #3 stirrups split the 28' span in half. A 6" slab with which sit on 16"x18" girders with (8)#6 bars and #3 stirrups. The inherent fire protection provided by concrete does not require a drop ceiling and has a relatively shallow depth at 16". By locating the beam at the middle of the bay, the partition will be located directly underneath and ceiling height will be at a maximum. Shear walls or a moment frame can be used for lateral forces, allowing for a potential decrease in foundation size. The drawback to concrete is the increased column sizes which are hard to conceal in walls. Overall this option provides a feasible alternate to the existing.

The Hotel N.E.U.S. has the prescriptive layout for a staggered truss system. A story high Vierendeel truss spans the entire width of the building eliminating the need for interior columns. Using STAAD Pro V8i, a 62' long, 10' truss was modeled and analyzed for gravity loads only. The results were a W12x53 top and bottom flange with 6x6x0.5 HSS composing the vertical and diagonal members. The floor is hung from the top and bottom flange and is constructed of the same 8" Hollowcore Plank as the existing floor allowing for high ceilings. This system weighs less than an equivalent concrete frame and would significantly reduce the amount of foundation since there are no interior columns. A disadvantage to the staggered truss for this building is that there will be many partition walls that could hide interior columns, therefore the large open spaces on floors 2 through 5 are no necessarily needed. It would also cost more due to unique truss fabrication and transportation. This system is possible but not likely to be used.

Introduction

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



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

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

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

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

Structural Overview

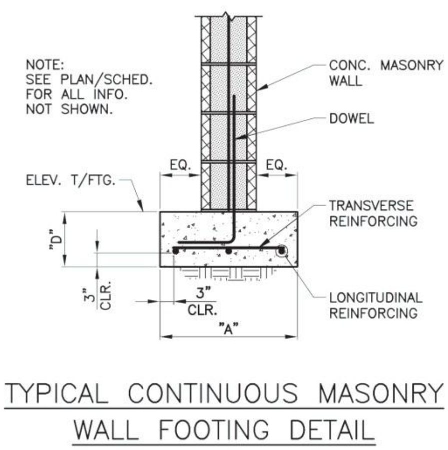
Foundations

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

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

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

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



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

Figure 1: Continuous Masonry Wall Footing detail and schedule

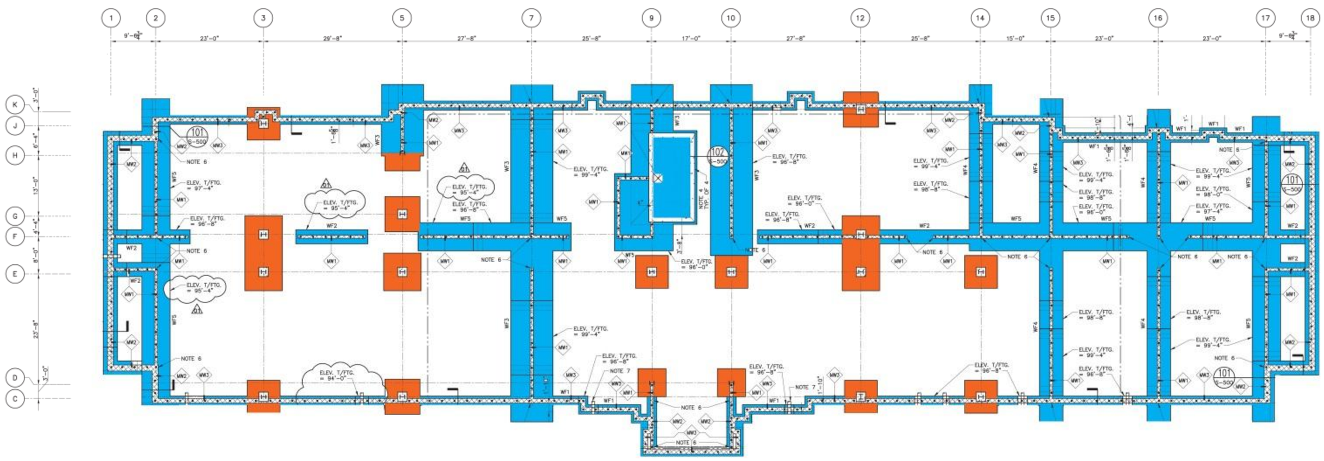


Figure 2: Foundation Plan.
Blue- wall footings
Orange- Column Footings

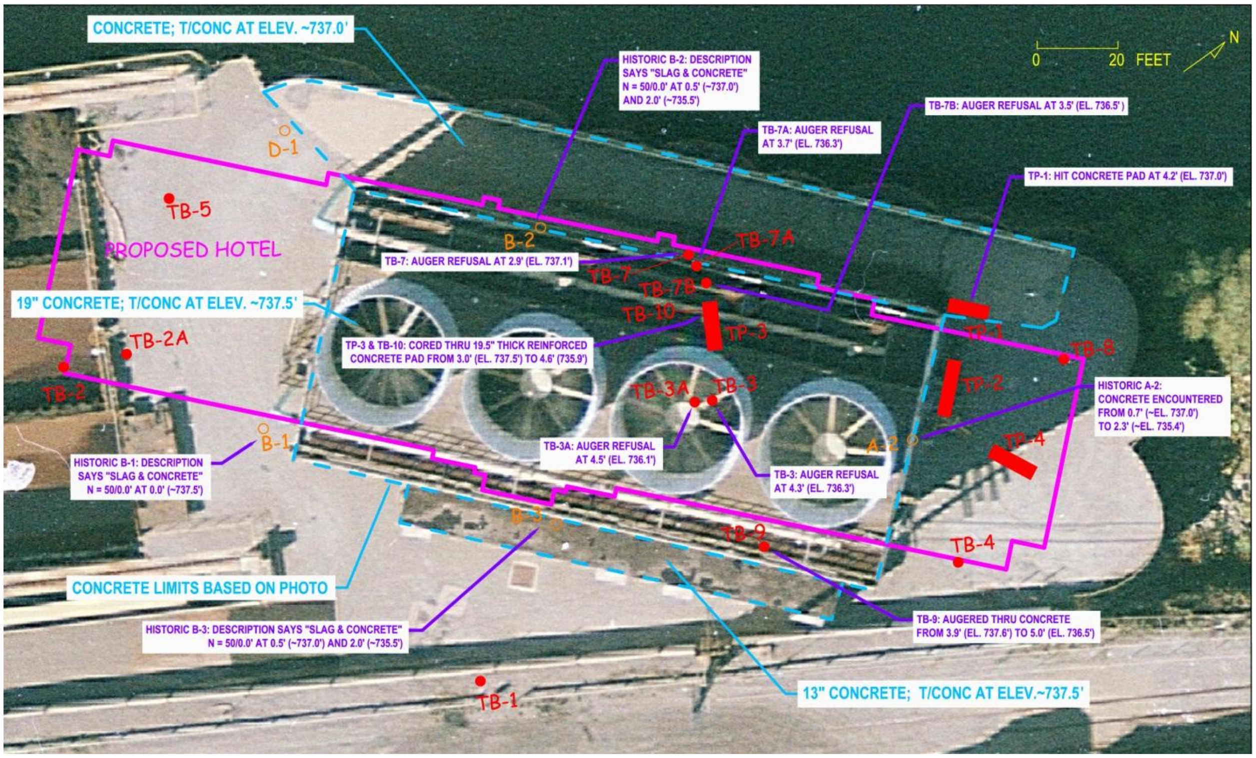


Figure 3: Site map showing test borings, existing mat foundation, hotel footprint, and location of former cooling towers.

Floor System

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

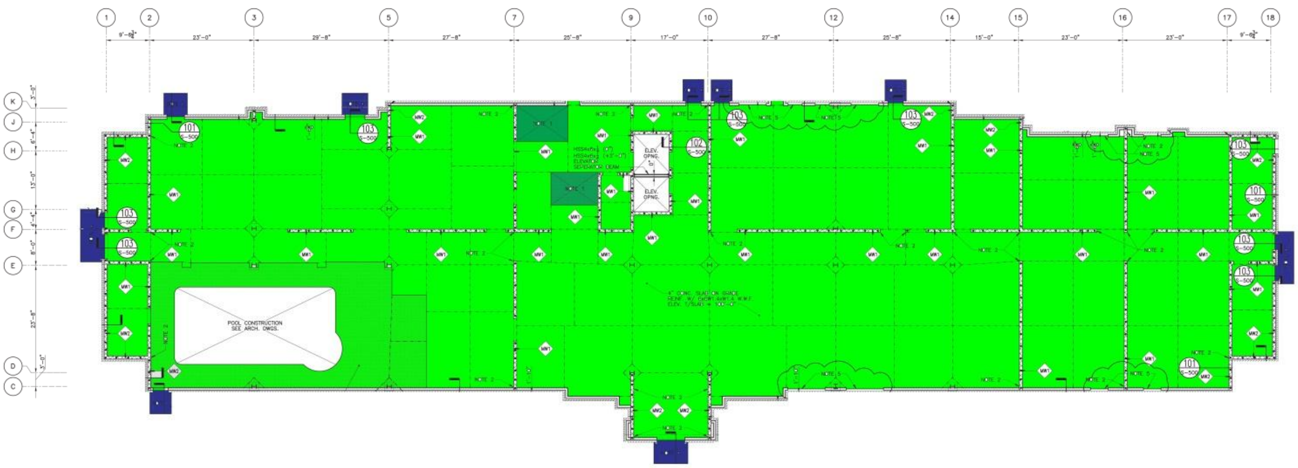


Figure 4: Slab on grade. Light green- 4" Conc. Slab on grade w/ 6x6W1.4xW1.4 W.W.F.

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

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

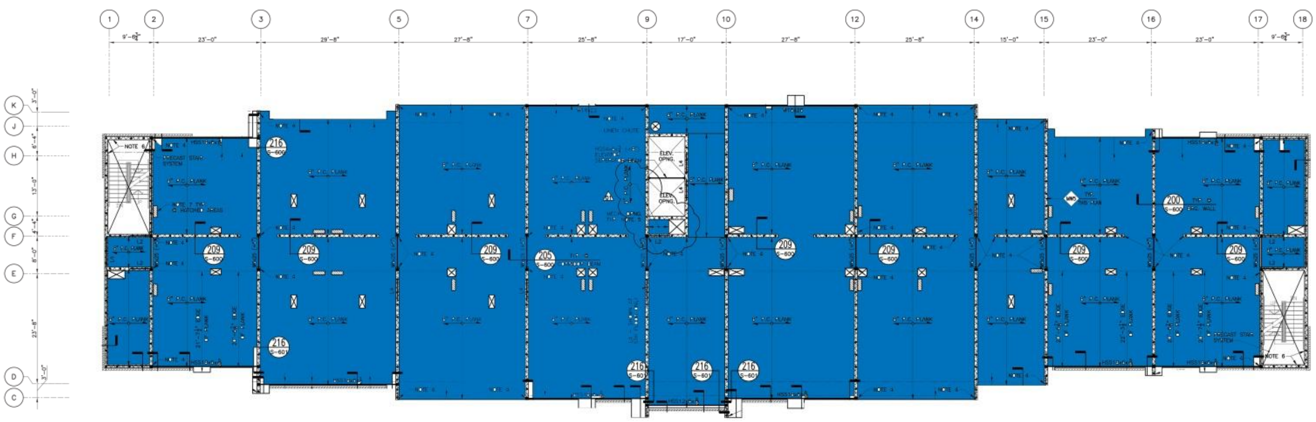


Figure 5: Typical Floor plank layout

Framing System

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

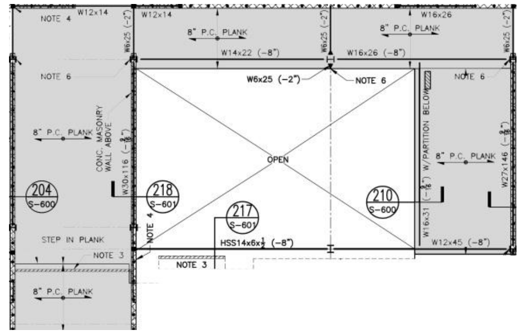
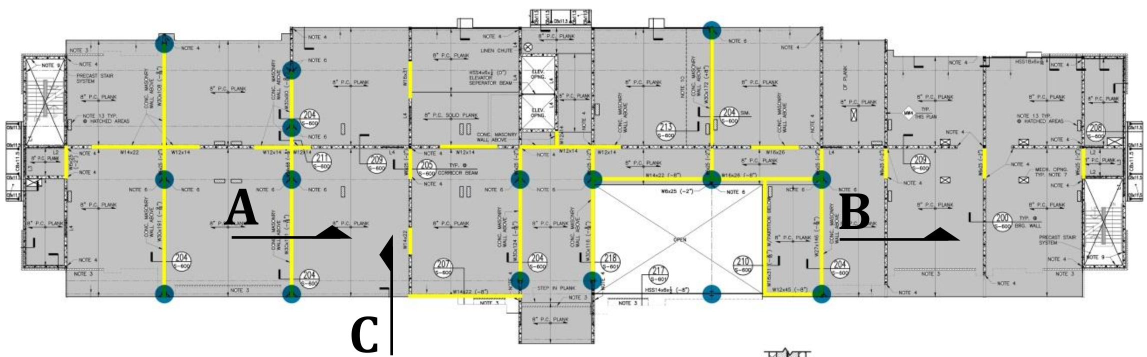
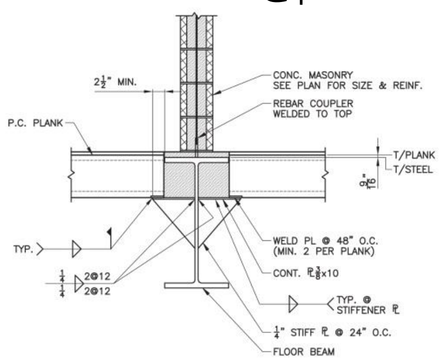
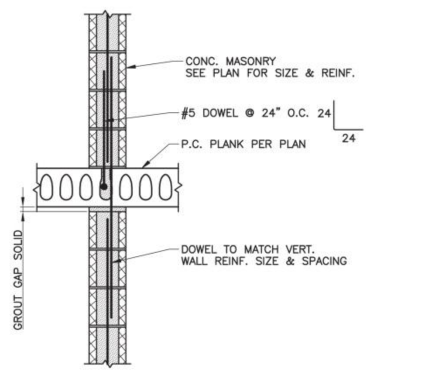


Figure 6: Open section on second floor

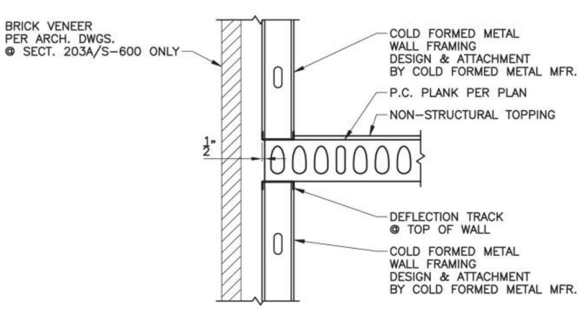




SECTION A- Beam carrying masonry wall



SECTION B- Plank on masonry wall



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

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

Lateral System

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

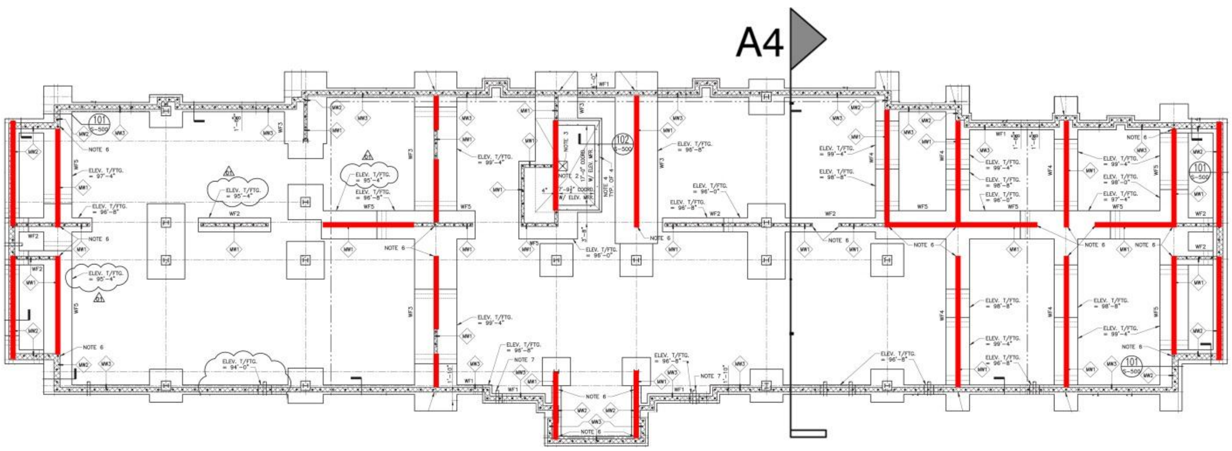


Figure 8: Location of shear walls on foundation plan

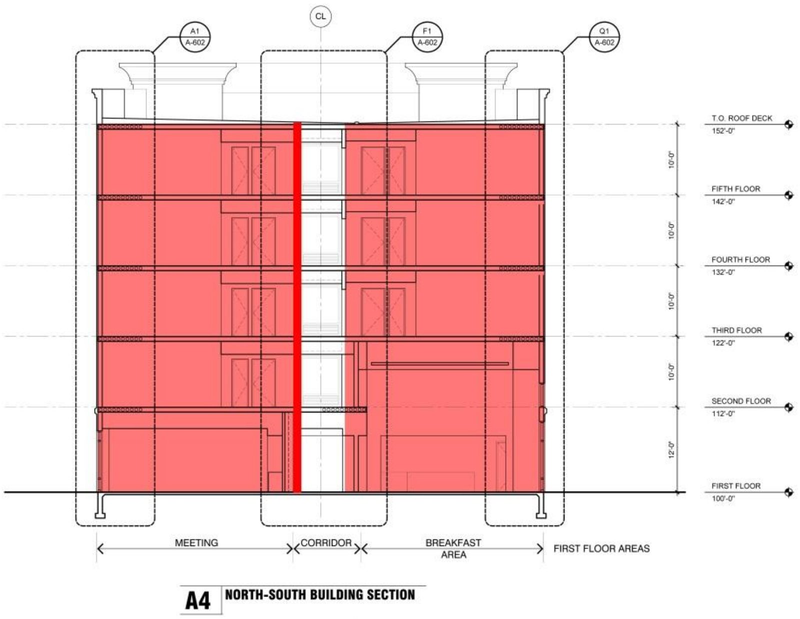
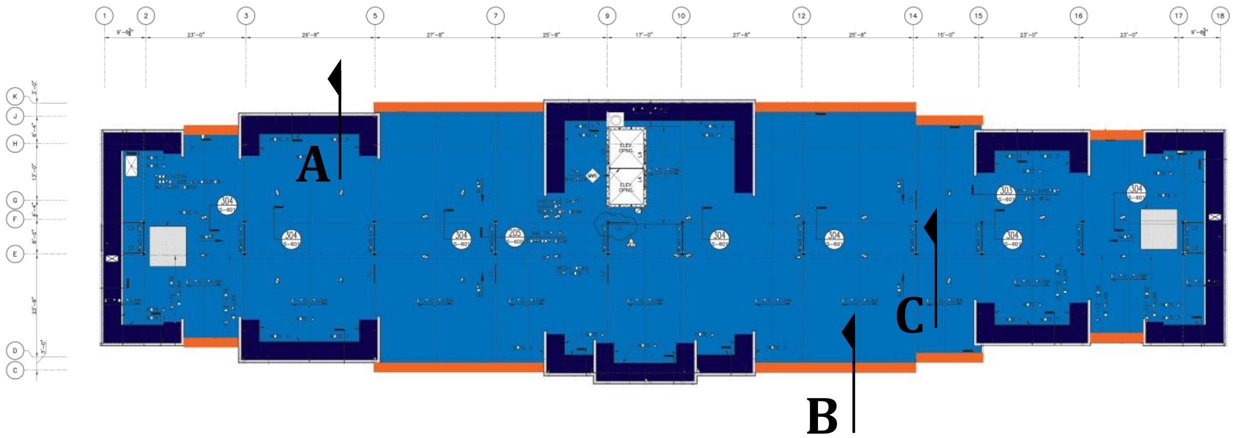
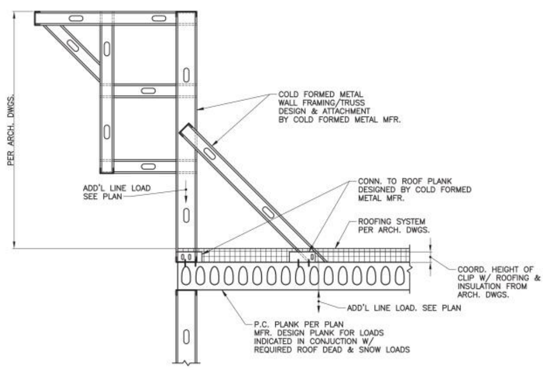


Figure 9: Section showing orientation of shear walls.

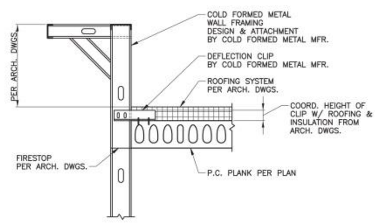
Roof System

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

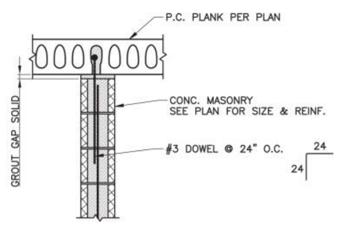




SECTION A- 8'-8" Cold-formed steel parapet wall



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



SECTION C- Roof plank on top of masonry wall

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

Materials

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

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

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

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

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

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

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

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

Galvanized Structural Steel	
Structural Shapes and Rods	ASTM A123

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

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

Design Codes

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

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

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

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

Figure 14: Codes used for thesis

Gravity Loads

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

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

Figure 15: Dead Loads for Hotel N.E.U.S.

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

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

Figure 16: Live Load comparison and references

Floor Systems Analysis

In order to analyze the existing and alternate floor system a typical bay in the Hotel N.E.U.S. was selected. Due to its slender design and step backs in the floor plans, the bays vary in size by several feet depending on their location. A "typical" bay (highlighted in green in Figure 35) was selected. The span between column lines C and E (highlighted in blue) is 26'-8" and is rounded to 27'-0". The span between column lines 5 and 7 (highlighted in yellow) is 27'-8" and is rounded to 28'-0".

The three systems designed have standards based off of the area this bay encloses. The corridor of the Hotel N.E.U.S. is located adjacent to the bay in the North-South direction. Because of this, no loads outside of the bay (except for the façade) were considered due to the large difference in spans. By performing hand calculations and computer modeling, the systems were designed and compared. The criteria for comparison were general conditions (weight, cost, depth), serviceability (deflection, vibration), architectural (fire rating, fire protection, ceiling), structural (foundation, lateral system, building height), and construction impact (schedule, constructability).

The existing systems and alternates include:

- Precast Hollowcore Plank on Masonry Bearing Walls (existing)
- Composite Deck on Composite Beams and Girders (option 1)
- One way Concrete slab with Beams and Girders (option 2)
- Staggered Truss with Precast Hollowcore Plank (option 3)

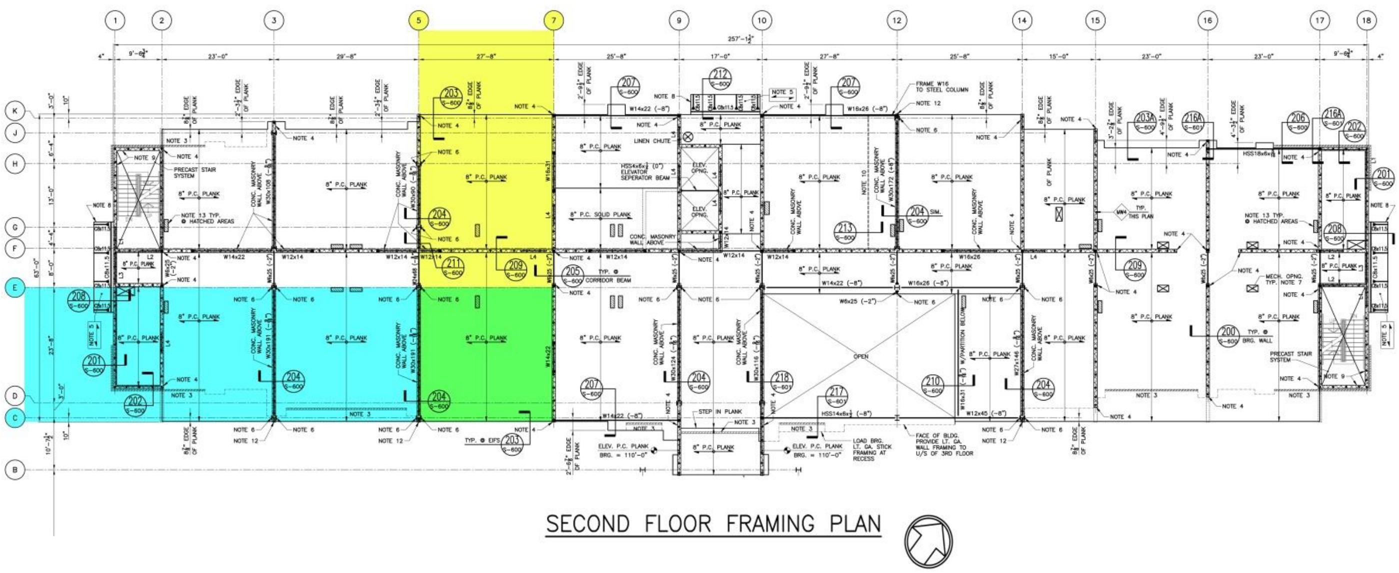
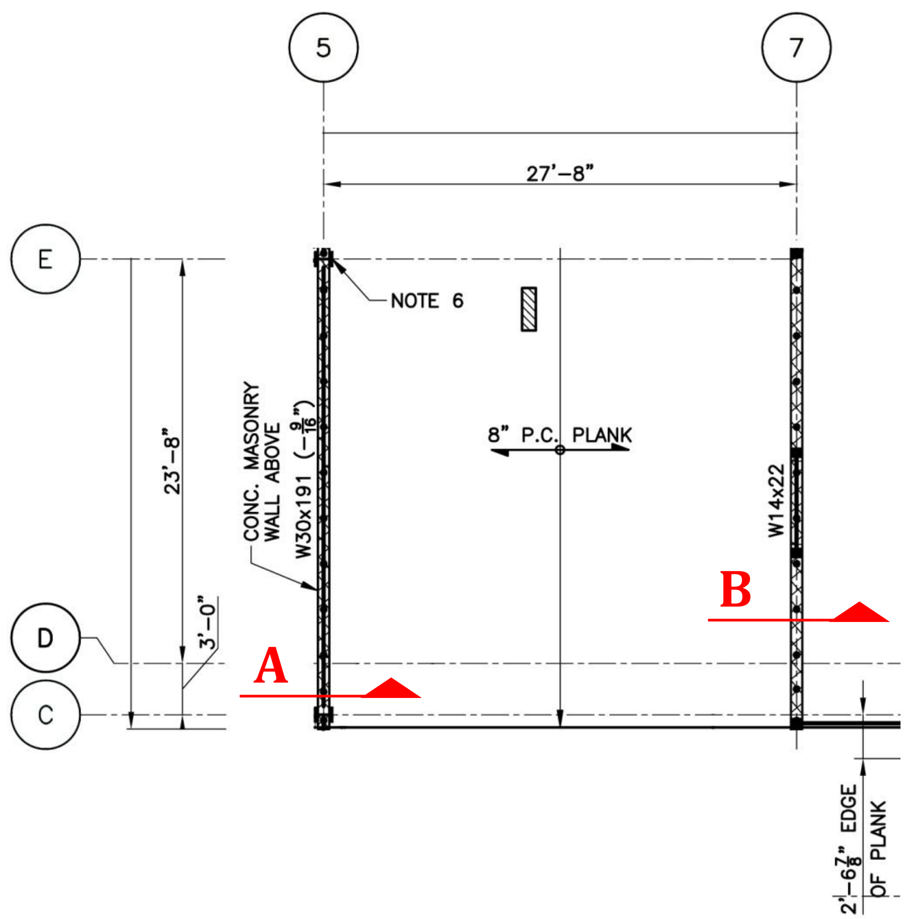
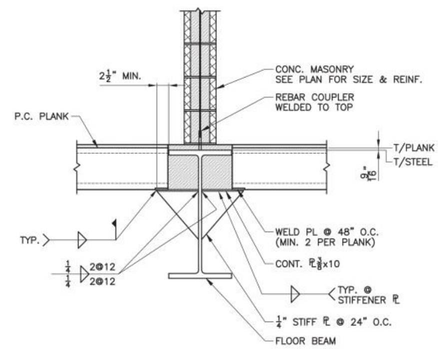


Figure 17: Typical Bay

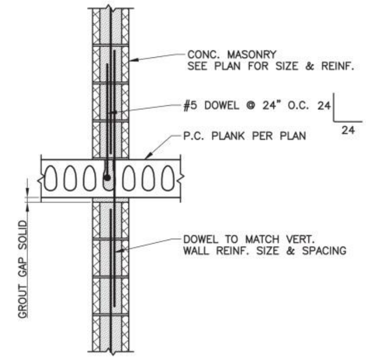
Hollowcore Precast Plank on Masonry Bearing Wall

The existing floor system of Hotel N.E.U.S. consists of 8" precast hollowcore plank that spans between masonry bearing walls. The first floor has many large open spaces such as a swimming pool and breakfast area. In order to achieve these spaces a steel frame on the ground floor support masonry walls on floors 2 through 5. The bay that was selected is on the second floor where the plank and masonry is supported by a W30x191 beam along column line 5 and a masonry wall along column line 7. A W12x96 column and W12x87 support the large beam. In Technical Report 1 an analysis of the prestressing forces in the plank was performed along with a check of the beam and exterior column. Although the span was slightly different in the calculations, the result still holds that the plank in the existing system is adequately selected. Results show that the beam and column were found to be sufficient to carry the loads applied. The results from this analysis can be found in Appendix B. The masonry wall was not evaluated because it is controlled by the lateral system requirements.





SECTION A- Beam carrying plank



SECTION B- Masonry Wall and Plank

Figure 18: Precast Plank and Steel/Masonry Wall Bay Plan

General

The precast plank system weighs approximately 79 pounds per square foot. It is the second heaviest system behind concrete. This is due to long masonry bearing walls and 8" planks. The cost for this assembly was based off of RS Means Construction Data for 2012 and found to be 17.4 dollars per square foot. This includes the planks, masonry, and reinforcement. The values for masonry were taken conservatively as it is hard to assign a cost to vertical elements in a horizontal plane. Therefore the total cost given may be high. This system was selected since it is cheap and efficient in creating partitioned rooms. In the main area of the Guest Room the depth is 8". A high ceiling height with a low floor to floor height is the most desirable for the Guest Rooms. The mechanical ducts coming into the room are located within the ceiling at the entrance that is 2'6" deep. This data was used to compare with the other systems. See Figure 19 for a section through a typical room.

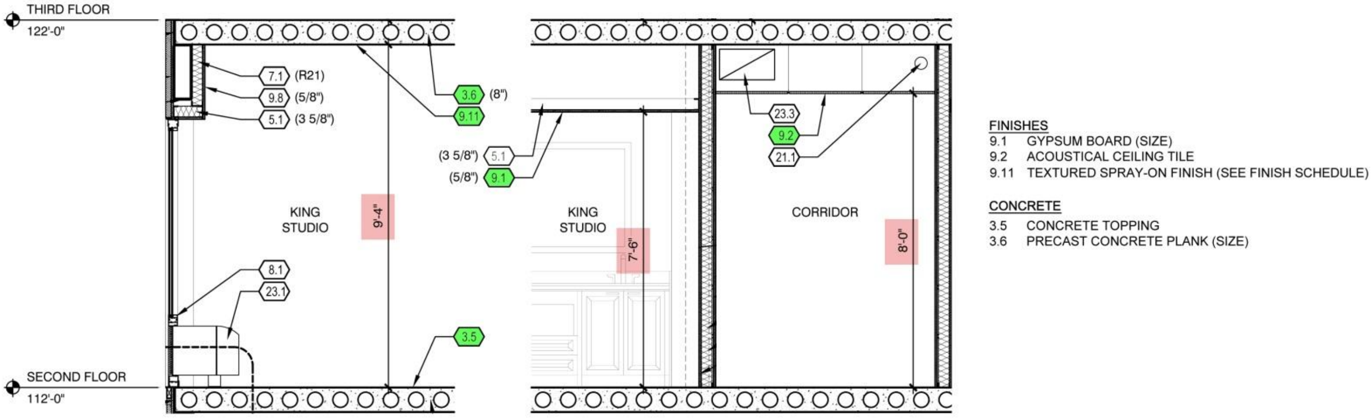


Figure 19: Section through Guest Room

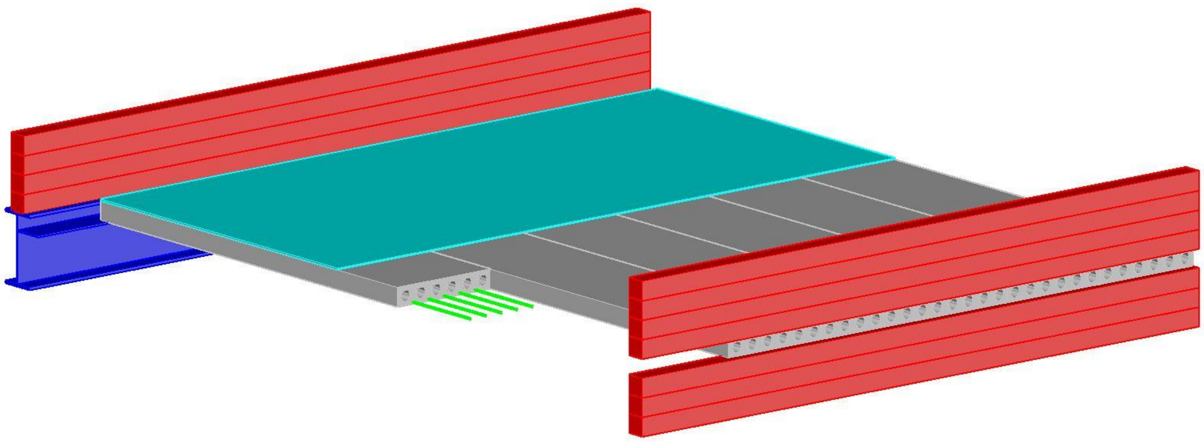


Figure 20: 3D representation of precast plank on beam and masonry wall

Serviceability

Due to the prestress and eccentricity of the strands in precast plank there is a camber in each piece. The load tables provided for different sections give approximate values for the camber required to meet deflection. This allows for longer spans and shallower depths. The service loads for the typical bay selected require a 4HC8 plank with a 68-S strand pattern. The estimated camber at erection is 0.8" and 0.9" for long term. However, these planks support partitions and that needs to be taken into account when dealing with camber and deflection. See Appendix B for an analysis of the prestress in a plank.

A ¾" topping is placed on the planks. This topping and the addition of partitions provide damping for this system which provides satisfactory vibration control.

Strand Pattern Designation
76-S

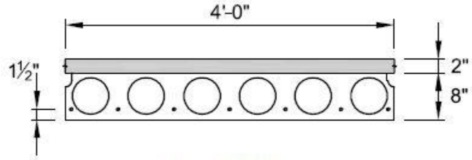
S = straight
Diameter of strand in 16ths
No. of Strand (7)

Safe loads shown include dead load of 10 psf for untopped members and 15 psf for topped members. Remainder is live load. Long-time cambers include superimposed dead load but do not include live load.

Capacity of sections of other configurations are similar. For precise values, see local hollow-core manufacturer.

Key
458 – Safe superimposed service load, psf
0.1 – Estimated camber at erection, in.
0.2 – Estimated long-time camber, in.

HOLLOW-CORE
4'-0" x 8"
Normal Weight Concrete



f'_c = 5,000 psi
f_{pu} = 270,000 psi

Section Properties

		Untopped		Topped	
A	=	215	in.^{2}	311	in.^{2}
I	=	1,666	in.^{4}	3,071	in.^{4}
y_b	=	4.00	in.	5.29	in.
y_t	=	4.00	in.	4.71	in.
S_b	=	417	in.^{3}	581	in.^{3}
S_t	=	417	in.^{3}	652	in.^{3}
wt	=	224	plf	324	plf
DL	=	56	psf	81	psf
V/S	=	1.92	in.		

4HC8

Table of safe superimposed service load (psf) and cambers (in.) **No Topping**

Strand Designation Code	Span, ft																													
	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40
66-S	458	415	378	346	311	269	234	204	179	158	140	124	110	98	87	77	69	61	54	48	43	38	33	29						
	0.1	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.2	0.2	0.2	0.2	0.1	0.0	0.0	–0.1	–0.2	–0.3	–0.5	–0.6						
	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.2	0.1	0.0	–0.1	–0.2	–0.3	–0.5	–0.7	–0.9	–1.2	–1.4						
76-S	470	424	387	355	326	303	276	242	213	188	167	149	133	119	106	95	86	77	69	62	55	50	44	39	35	31	26			
	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.4	0.4	0.4	0.3	0.3	0.3	0.3	0.2	0.2	0.1	0.0	–0.1	–0.2	–0.4	–0.5	–0.7	–0.9			
	0.2	0.2	0.3	0.3	0.3	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.3	0.3	0.2	0.1	0.0	–0.1	–0.2	–0.4	–0.6	–0.8	–1.1	–1.4	–1.7	–2.0			
58-S	464	421	384	352	323	300	280	260	244	229	211	194	177	160	144	130	118	107	97	88	80	72	66	60	54	48	42	37	32	28
	0.2	0.2	0.3	0.3	0.3	0.4	0.4	0.5	0.5	0.5	0.5	0.6	0.6	0.6	0.6	0.6	0.6	0.5	0.5	0.5	0.4	0.3	0.2	0.1	0.0	–0.4	–0.3	–0.5	–0.7	–0.9
	0.3	0.3	0.4	0.4	0.5	0.5	0.6	0.6	0.6	0.7	0.7	0.7	0.7	0.7	0.7	0.6	0.6	0.5	0.4	0.3	0.2	0.0	–0.2	–0.4	–0.6	–0.9	–1.2	–1.6	–2.0	–2.4
68-S	476	430	393	361	332	309	286	269	253	235	223	209	200	180	165	153	142	132	121	110	101	92	84	77	70	63	56	51	45	40
	0.3	0.3	0.3	0.4	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.7	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.7	0.7	0.6	0.5	0.4	0.2	0.1	–0.1	–0.3
	0.3	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.8	0.9	0.9	1.0	1.0	1.0	1.0	0.9	0.9	0.9	0.8	0.7	0.6	0.4	0.2	0.0	–0.2	–0.5	–0.8	–1.1	–1.5
78-S	488	442	402	370	341	318	295	275	259	241	229	215	203	195	180	168	157	144	135	126	118	110	101	92	84	77	70	64	58	52
	0.3	0.3	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.9	0.9	1.0	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.0	0.9	0.8	0.7	0.6	0.5	0.3
	0.4	0.5	0.5	0.6	0.7	0.8	0.8	0.9	1.0	1.0	1.1	1.2	1.2	1.2	1.3	1.3	1.3	1.3	1.3	1.2	1.2	1.1	1.0	0.8	0.7	0.5	0.3	0.0	–0.3	–0.7

Figure 21: PCI load table

Architectural

Fire protection is inherent in precast members because they are not combustible. A ¾" topping is used to level out the effects of camber while also aiding the plank in providing a 2 hour fire rating. In Figure 22, the dark blue area above the entrance and bathroom is 7'-6" high finished with gypsum board. This allows for mechanical ducts to enter from the hallway. In light blue the ceiling height is 9'-4" which is the maximum that can be achieved. This is desirable in a hotel room as it makes it feel larger while keeping the floor to floor height limited to lower the cost of the building. The undersides of the planks have a textured finish. This is used as a benchmark to compare to the alternate systems.

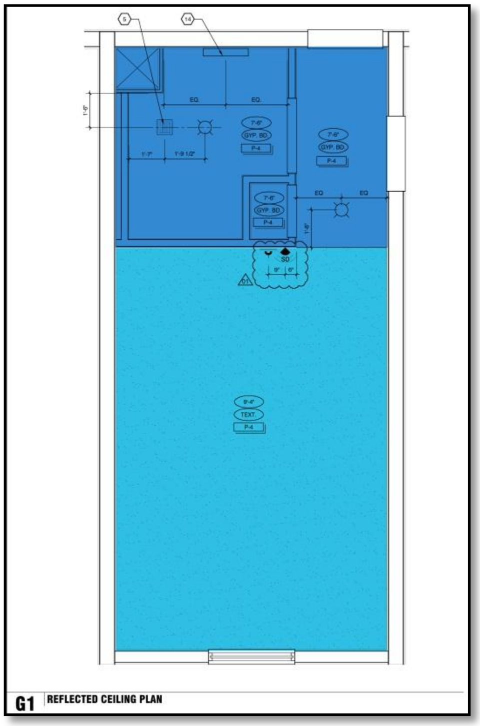


Figure 22: Typical Ceiling Heights

Structural

The foundation of the existing system is composed of spread footings for the steel columns and continuous wall footings for the lateral force resisting shear walls.

Construction

The ground floor of the Hotel N.E.U.S. has a height of 12'-0". Floors 2 through 5 are all 10'-0" high. Construction started in October of 2011 and is ongoing. It is slated to be completed in November of 2012, giving the project a construction schedule of just over a year. The precast floor and masonry walls allow for quick erection of the structure. The steel located on the ground level is minimal, although some members are large and would require a crane. Overall this system was assigned a constructability rating of "easy" since it can be accomplished quickly and cost efficiently.

PROS:	CONS:
• Fast Construction • Minimal depth and long spans • Durable • Inherent fire protection • Bearing walls serve as lateral system as well	• Openings in plank require coordination between trades • Heavy system (masonry)

Composite Steel with Composite Deck

The first alternate system designed was composite steel. This was selected due to the fact that there was already some steel on the ground floor of the building. Since the existing gravity system is a hybrid between steel and masonry, this will keep a similar material type for the gravity and lateral systems. Composite steel can cut down on section sizes due to the attachment of shear studs on the top of the beam that are integrated into the poured concrete. In this design a Vulcraft 3", 22 gauge interlocking deck with 2.5" of concrete was selected for its ability to span the beams without shoring. With sprayed fiber fire protection on the deck this assembly satisfies a 2 hour fire rating. The beams were design as W12x26 with 12 studs uniformly spaced. The result of the girder design was a W18x35 with 14 studs uniformly spaced. Typically a girder would not need as many studs in the center span due to no change in moment, however the girder along column line C has an extra load due to the building façade. Since the girder along column line E would see load extra load from outside of the bay, both girders are assumed to be the same. Since loads are relatively low for the Hotel N.E.U.S, deflection was a controlling factor over composite strength, leading to relatively low stud counts.

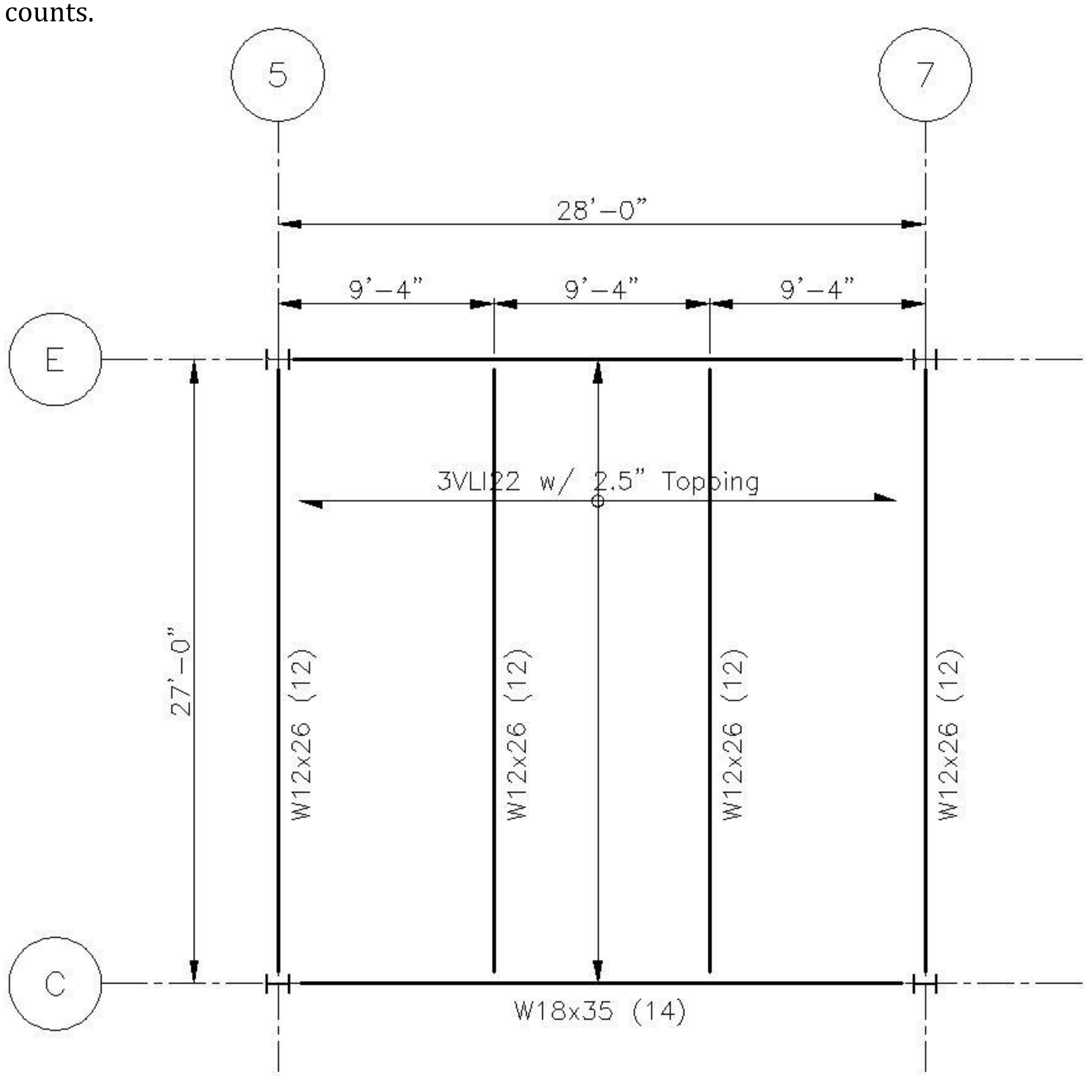
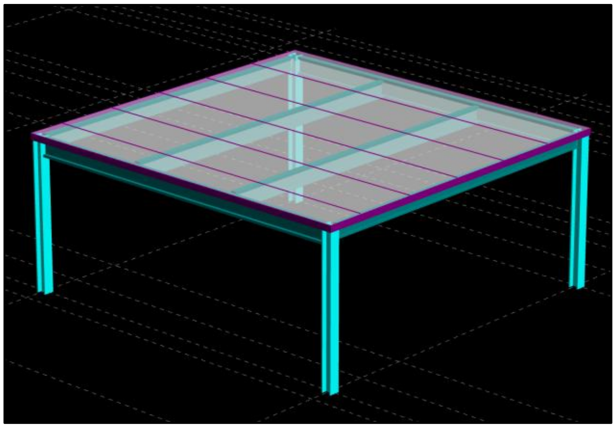


Figure 23: Composite Steel Bay Plan

General

The composite steel system weighs approximately 55 pounds per square foot. It is the lightest system considered in this report. The cost of per square foot was estimated to be about 17.8 dollars. This does not include changes to schedule and does not include labor. Compared to the other three systems composite steel was determined to be the second cheapest option behind precast plank on masonry walls. A structure depth of 17.75" measures the largest and is a significant increase from the precast plank.



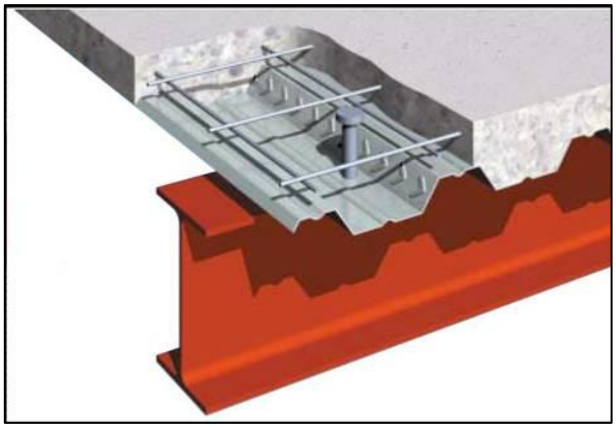


Figure 24: LEFT-3D RAM model of composite steel system
RIGHT- 3D section of system
Image from http://sydney.edu.au/engineering/civil/people/tahmasebinia.shtml

Serviceability

The maximum deflection for the infill beams was found to be 1.21" for services loads and is within the accepted limitation.

Steel construction is typically worse than other systems when dealing with long spans and shallow members. No vibration analysis was performed, but the 5.5" thick slab combined with a finished floor could help prevent complaints. The majority of the walking will be concentrated in the corridor of the building where the span is 8'-0". This will allow for a stiffer floor and will not translate much to the typical bay that is focused on in this report.

Architectural

The selection of the deck was based on the SDI max unshored construction span and for the ability to achieve a 2 hour fire rating with sprayed fiber fireproofing on the underside. Along with this, the beams and girders would need a sprayed fireproofing. A drop ceiling is required to cover these conditions. This would cause the ceiling height in the bedroom to change from 9'-4" for plank to about 8'-6" for steel. This is the deepest ceiling of all the options but is necessary to for aesthetic reasons.

Structural

A composite steel system provides the lowest overall weight. The foundations could be significantly reduced since the existing system uses shear walls and requires continuous footings. A braced frame could be utilized since it can be concealed within walls in both directions. A moment frame could be used as well but would cost more.

By continuing the steel framing from the ground level the lateral system can be uniform across the floor plan, unlike the masonry shear walls that cannot be located in certain areas on the ground floor. To achieve this, moment frames would be required to span areas such as the pool to keep it free from obstructions. The center of rigidity would then be centered unlike the existing system. Refer to the Lateral System section of this report to observe the imbalanced shear walls.

Construction

Spray on fireproofing can increase construction time for the project. However, steel erection is fairly quick and efficient. The schedule impact would be minimal in comparison to precast planks and masonry. If a ceiling height of 9'-4" is a requirement in Guest Rooms than the floor to floor height must be increased. This calls for larger columns and an overall increase in building cost.

Feasibility

Since steel framing was already used for part of the existing design, continuing it throughout the building is certainly a viable option. There is a height limitation in the zone the Hotel N.E.U.S. is located but increasing the height of 5 floors by 1' would not bring it close to the regulation. A composite steel system is deemed *POSSIBLE* for an alternate.

PROS:	CONS:
• Light weight • Reduced Steel cost • Smaller foundations • Shallower beams (compared to noncomposite steel)	• Increased floor to floor height • Requires drop ceiling • Larger deflections compared to concrete/plank

One way Concrete slab with beams and girders

The second alternative floor system evaluated was a one way concrete slab with beams and girders. The bay size is essentially square which usually calls for a two way flat slab. However, the Hotel N.E.U.S. is long and slender, having a corridor between two of the typical bays. The beam/girder size was selected to match the estimated columns for ease of forming and pouring the concrete. To limit the thickness of the slab a beam spans the center of column lines 5 and 7, forming two sections. The resulting slab is 6" thick, supported by 18"x16" beams and girders. The girder carries less load than the beam due to the one way action of the slab so the size was selected to maintain regularity. A one way slab would likely span the corridor and rest on the girder but this load is not considered in this analysis. See Appendix D for the design of steel reinforcement.

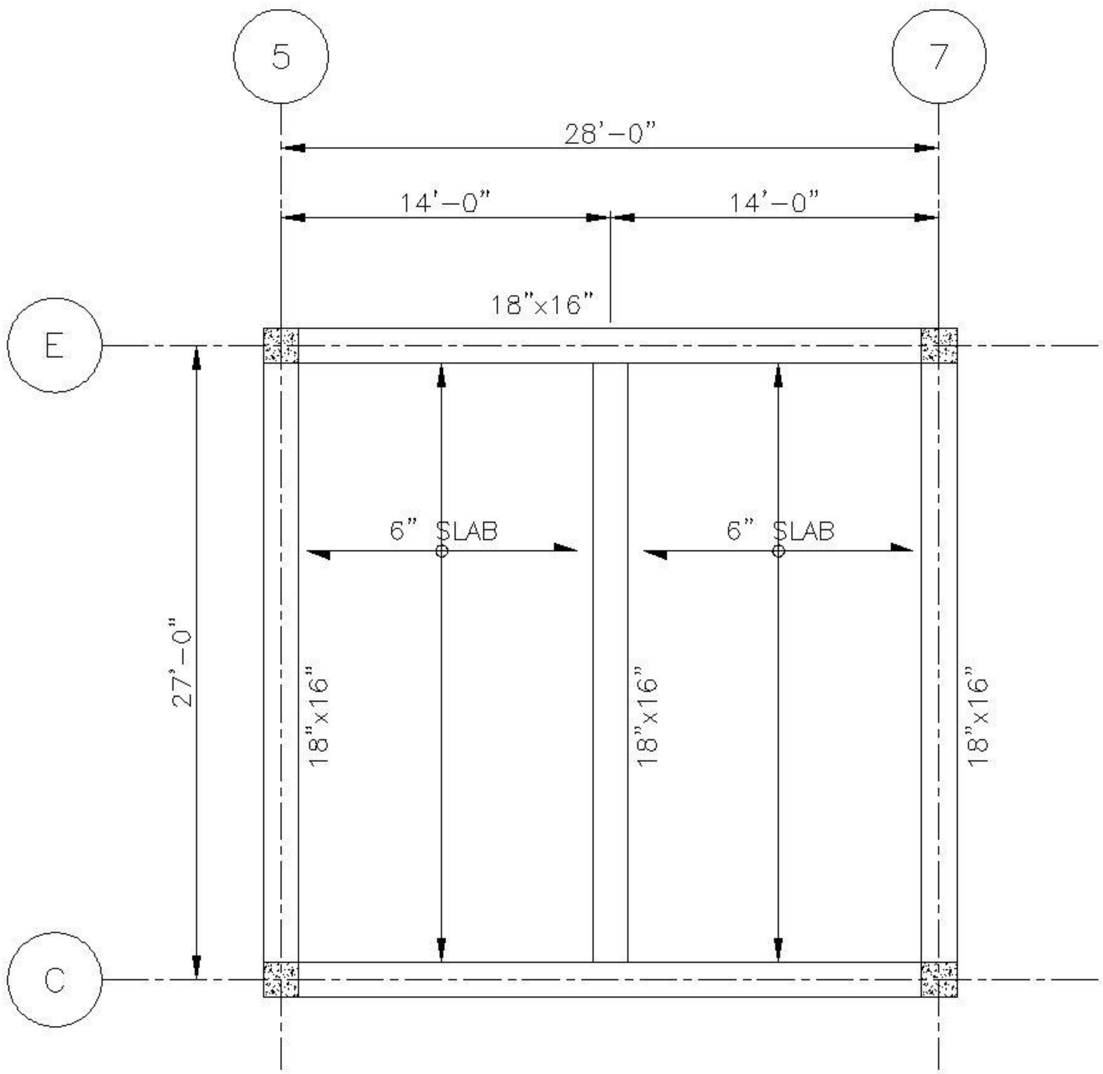


Figure 25: One Way Slab Bay Plan

General

The one way slab system weighs in at about 98.7 pounds per square foot, making the heaviest overall system. The design resulted in an overall depth of 16". The beams are slightly less deep than those of the composite steel but still twice that of precast plank. A cost of 20.8 dollars per square foot was estimated. The extra costs are associated with formwork and the labor. A finishing cost was estimated and added to the total.

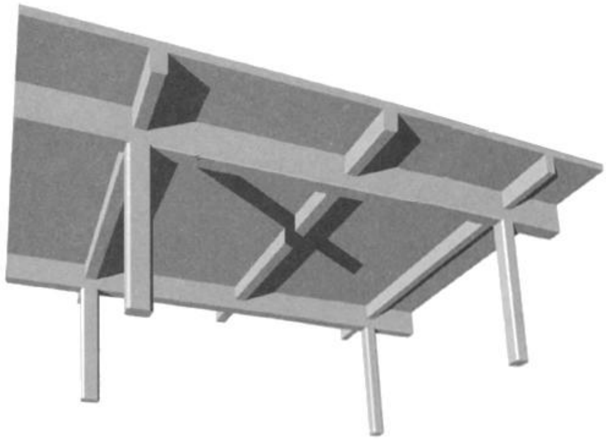


Figure 26: 3D representation of 1 way slab

Serviceability

According to Table 9.5a in ACI 318-11, deflections do not need to be calculated if the span coefficients are used. In this design the limiting thickness was L/28. This depth was rounded to the 6" used and therefore the requirements for deflections are satisfied.

Vibration is not an issue in this system because of the stiffness provided by reinforced concrete. A soft material such as concrete can help limit direct impact sounds.

Architectural

A 2 hour fire rating is achieved through the inherent properties of the concrete. This is beneficial as no extra costs and time must be spent on fire protection. Also, a drop ceiling is only required if the architecture/interior designer feels the need for one. The structure can be left exposed and finished similar to the precast plank. The ceiling height will be 9'-6" at the bottom of the slab and 8'-8" where the beams are located. There is 1'-2" between the bottom of the beam and the ceiling in the bathroom which should allow for mechanical systems to pass through. However, the drop ceiling in the hallway would likely have to be increased from the current 8'-0". A drawback of this system is the large columns and the inability to enclose them in walls.

Structural

There are two options for the lateral system associated with reinforced concrete. The first and more probable option is using reinforced concrete shear walls, similar to the masonry shear walls in the existing design. These could be placed in the same locations with continuous wall footings. The spread footings would increase slightly due to the amplified weight of the building. A reinforced concrete moment frame could be considered as well, but would cost more and is not as necessary for the region it is located.

Construction

A one way concrete system would take longer to construct then all other options. Forming, pouring, and letting the concrete cure are all tedious processes and need to be done correctly. The beams and girders are the same size and can therefore be formed and poured all at once, decreasing some of that extra time. Construction started in October of 2011, therefore cold temperatures could be encountered when placing the concrete and could call for admixtures that increase the cost.

Feasibility

The one way concrete system provides a sturdy structure to support the building loads. Although it is heavier the construction type is similar to the existing design, allowing shear walls and the same floor to floor heights. This option is deemed *POSSIBLE* for an alternate.

PROS:	CONS:
• Minimal vibration issues • No need for fire protection • Simple layout for shear walls • Low floor to floor heights	• Heavy system • Large columns are hard to conceal • Not good for winter construction • Larger foundations

Staggered Truss with Precast Plank

The Hotel N.E.U.S. has the prescriptive layout for a staggered truss system. This utilizes a story high Vierendeel truss spanning the width of the building or 62' maximum for this building. A central corridor is permitted by the rectangular panels in the truss. Precast planks are hung from the top and bottom chord eliminating the need for interior columns. The trusses run along alternating column lines on each floor so the bottom chord is always in between top chords on the same floor. The computer modeling program STAAD Pro V8i was used to model the truss and get preliminary forces, moments, and member sizes. The chords are made from wide flange beams and hollow structural steel sections are used for the vertical and diagonal components. By using the same spans as the existing system the same 8" hollowcore planks were used as the floor system. In Figure 27 the typical bay is shown with the layout of the trusses selected.

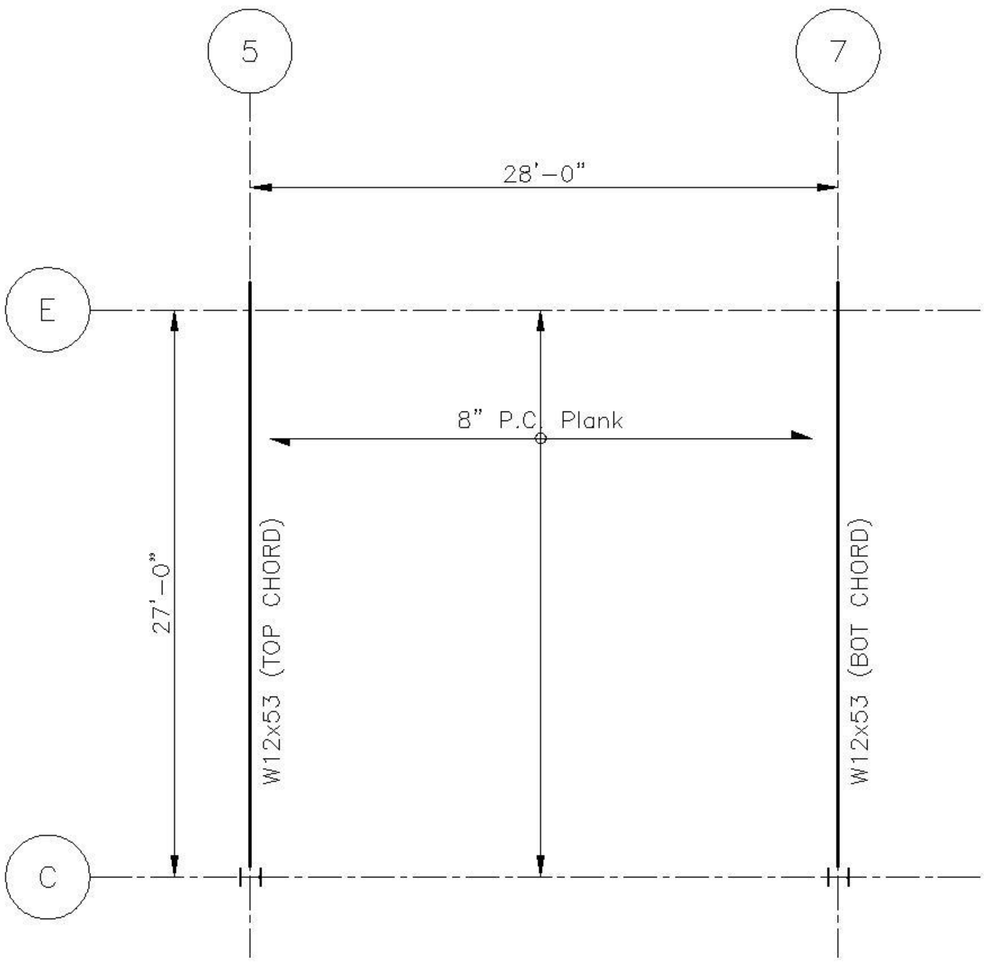


Figure 27: Staggered Truss Bay Plan

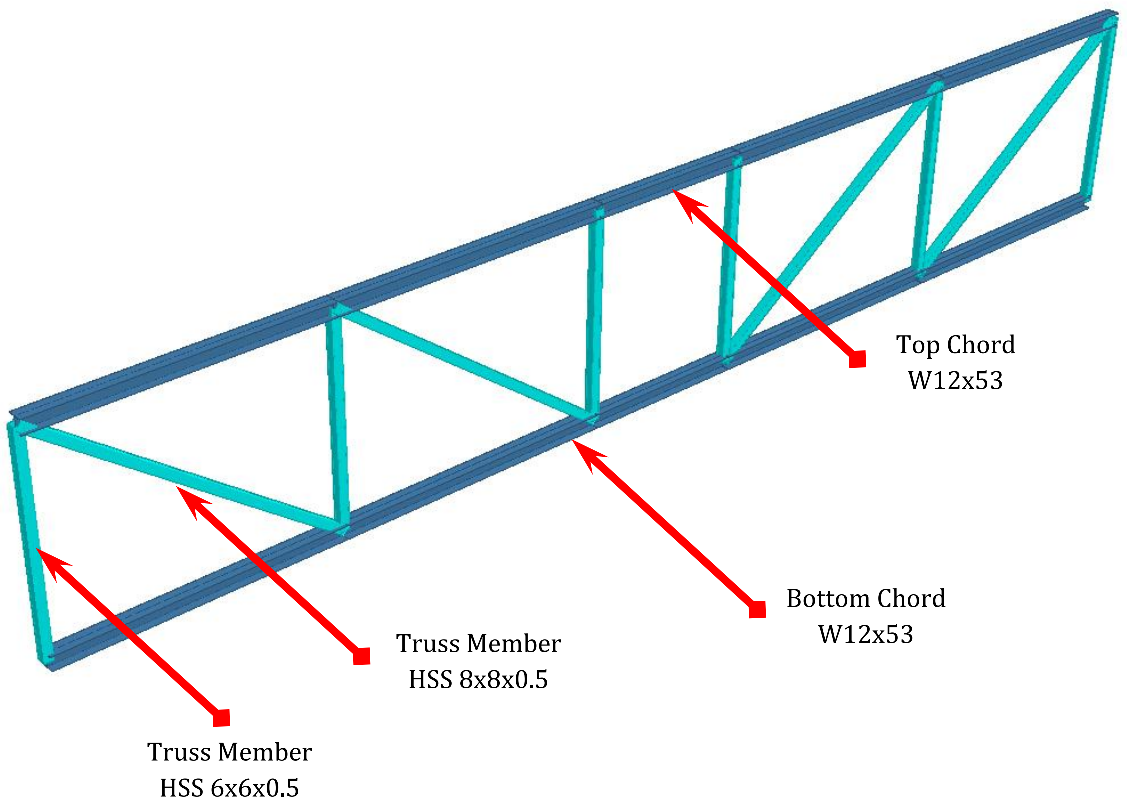


Figure 28: Design of Staggered Truss

Design

The computer model of the Vierendeel truss gave axial loads and moments for gravity loads only. The top chord is controlled by compression with a maximum load of 466 kips along the middle three members. Tension controls the bottom chord with a tensile load of 260 kips. A W12x53 is selected to construct the chords can withstand the tensile and compressive loads. Bending moments applied are not nearly as large due to relatively short spans and can be resisted by the W12x53.

The vertical elements on the exterior suffer the highest compressive loading. An HSS 6x6x0.5 can suitably resist the applied compressive load of 278 kips. The exterior diagonal members encounter a tensile loading of 419 kips. To resist both yielding and rupture an HSS 8x8x0.5 must be used.

The loading diagrams can be found in Figure 29, 30, and 31 on the following page.

More diagrams and a complete list of the loads can be found in Appendix E.

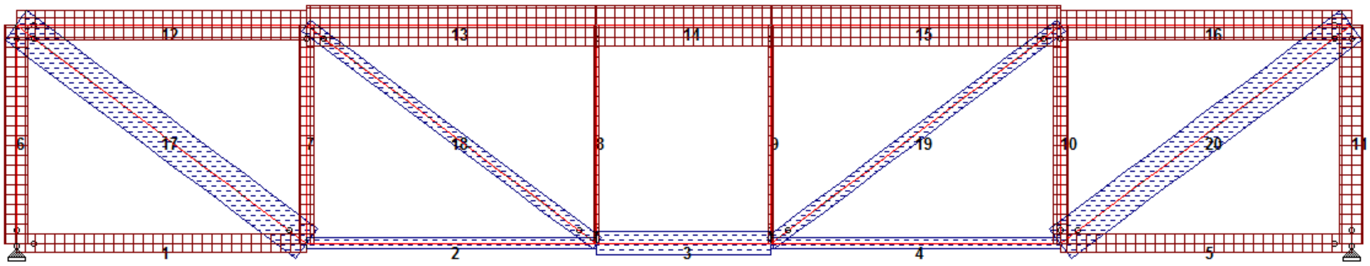


Figure 29: Axial Loading (Compression- Red, Tension- Blue)

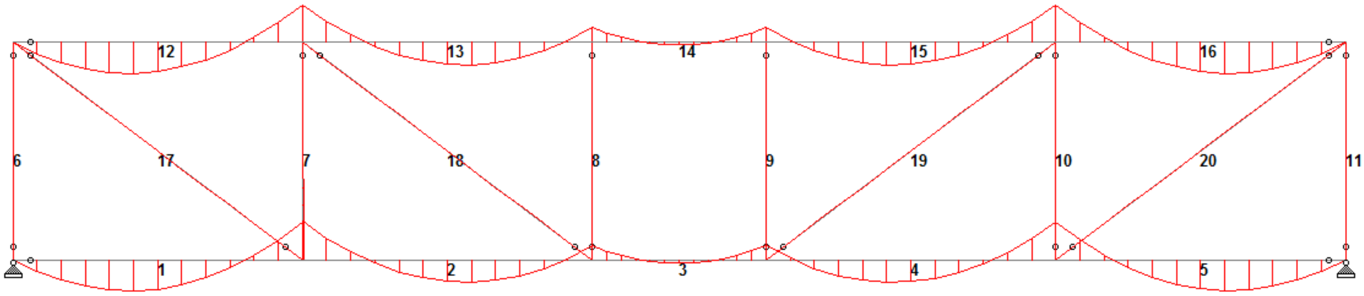


Figure 30: Moment Diagram

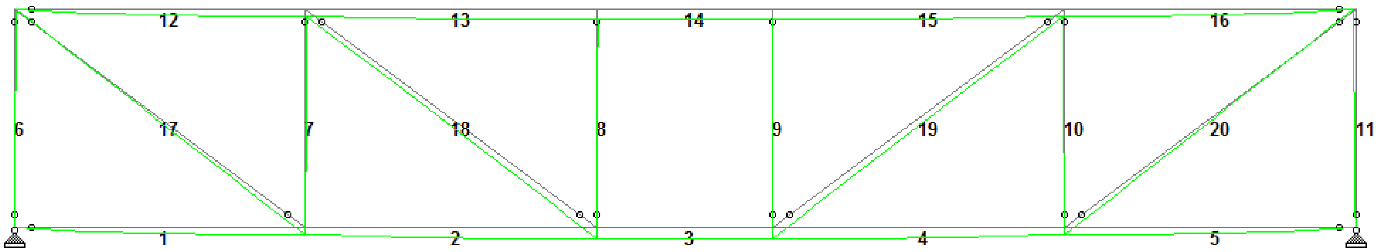


Figure 31: Displacement Diagram

General

A weight of 68 pounds per square foot was calculated for the staggered truss system. This is the lightest option behind composite steel. Precast planks forming the floor are lightweight and while the trusses are heavy they are not located on every floor for each column line. The depth is 8" at the center of the bay and 12" at the column lines. This alternative came in at the highest cost of 21.2 dollars per square foot. However, in an attempt to get a value, the members of the truss were taken as individual parts. In reality the truss would be constructed at a single price and fabricated offsite. The transportation and assembly would cost extra due to its unique sizes.

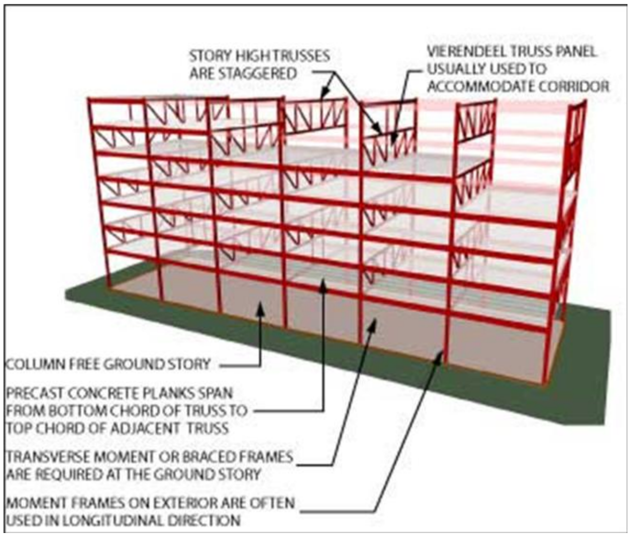


Figure 32: Staggered Truss system. Image from http://www.structuremag.org/article.aspx?articleID=690

Serviceability

The identical precast plank from the existing system can be used due to the spans staying the same. Therefore the camber induced into the plank allows it satisfy deflection criteria. The truss deflection is 2.422" which is within the acceptable limit for total load. Due to the lightweight plank being supported by steel the system could be vulnerable to vibration. A concrete topping on the plank would help along with partition weight and a finished floor.

Architectural

To achieve a 2 hour fire rating similar to the other options a sprayed fireproofing would need to be applied to the trusses. The precast concrete has inherent fire resisting capabilities. Since the beams are located where rooms are divided they would need to be enclosed in partitions to prevent the use of a drop ceiling. Therefore the same 9'-4" ceiling height of the existing floor system can be accomplished. This system allows for the most flexibility in architecture since it requires no interior columns. The clear space between trusses on a level is two bay widths or approximately 50'.

Structural

Lateral forces are resisted by the trusses in the transverse direction of the building. A moment frame on the exterior resist loads in the longitudinal direction. The system is efficient due to its inherent stiffness.

The foundations would need to be increased around the perimeter since the buildings weight is supported by exterior columns only. Lateral effects need to be resisted as well which can increase their size. Spread footings would be used since spacing between columns is on average 25'.

Construction

Lateral forces are resisted by the trusses in the transverse direction of the building. A moment frame on the exterior resist loads in the longitudinal direction. The system is efficient due to its inherent stiffness.

The foundations would need to be increased around the perimeter since the buildings weight is supported by exterior columns only. Lateral effects need to be resisted as well which can increase their size. Spread footings would be used since spacing between columns is on average 25'.

Feasibility

The flexibility in the floor plan is a huge benefit to a staggered truss system. However, the Hotel N.E.U.S. has a floor plan that becomes narrower as it extends longitudinally, calling for different trusses to be constructed. Although it could be done, the cost would be increased due to these conditions. It does not gain as much from the open floor plan due the module bedroom sizes remaining the same and bay sizes being configured to contain them. Therefore this system is deemed *POSSIBLE* but *NOT LIKELY.*

PROS:	CONS:
• Flexible layout due to no interior columns • Fast construction • Good for winter construction (dry system) • Inherent stiffness performs well against lateral forces	• Extra fees involved in truss fabrication and transport • Long lead time • Larger deflection due to span length

System Comparison

	Systems Comparison				
	Consideration	Precast Plank on Masonry Bearing Wall	Composite Deck w/ Steel Beams and Girders	One way slab w/ Beam and Girder	Precast Plank on Staggered Truss
General	Weight	79 psf	55 psf	96.7	68 psf
	Cost	17.4	17.8	20.8	21.2
	Depth	8"	17.75"	16"	8"
Serviceability	Deflection	Deflection for Span is met by Tables (includes Camber)	1.21"	Deflection for Span is met by table 9.5a in ACI 318-11	Plank Defl. Is met by Tables/2.422" for Truss
	Vibration	Good	Average	Great	Average
Architectural	Fire Rating	2 HR	2 HR	2 HR	2 HR
	Fire protection	Inherent	Sprayed Fiber	Inherent	Inherent (Excluding Truss)
	Ceiling	Finished Gypsum	Drop Ceiling	Drop Ceiling/Finish Concrete	Finished Gypsum
Structural	Foundation	Spread Footings and Continuous Wall Footings	Spread Footings	Spread Footings and Continuous Wall Footings	Spread Footings
	Lateral System	Masonry Shear Walls	Braced Frame/Moment Frame	Concrete Shear Walls/Concrete Shear Walls	Moment Frame (Longitudinal), Moment/Braded (First Floor)
Construction	Building Height	10' Floor-to-Floor	May increase due to mechanical space	Can stay the same with finished concrete/May require drop ceiling	10' Floor-to-Floor
	Schedule	N/A	Potentially decrease construction time	Slight increase in schedule	Fast, allows for winter construction
	Constructability	Easy	Medium	Hard	Medium
	Feasability	**N/A**	**Possible**	**Possible**	**Possible/Not likely**

Figure 33: System Comparison Matrix

Conclusion

Technical Report 2 investigated three alternate floor systems to compare with the existing system. A typical bay was selected to best represent the floor throughout the Hotel N.E.U.S. The different options were designed using hand calculations and computer modeling software. The criteria for the comparison included general conditions and serviceability along with architectural, structural, and construction impacts. The most important issue was the allowance of partitioned rooms and small floor to floor heights with maximum ceiling space.

The existing system was composed of 8" precast plank on masonry bearing walls. Some steel was used on the ground floor. There was an analysis of these components performed in Technical Report 1 and are included in Appendix B. It was determined to be the cheapest system with the highest ceiling height. The plank is sufficient to span the bays and the masonry walls provide lateral resistance. Therefore this system was very efficient for the design of the Hotel N.E.U.S.

Option 1 consisted of Composite Steel framing. This system cost slightly more than the existing but came with the need to increase the building floor to floor height and add drop ceilings. The architecture would not be impacted by this switch. Foundation sizes could be reduced by eliminating the use of shear walls for a lateral system. The construction time would not be effected by the implementation of this option. Overall composite steel presented itself as a possible alternative.

Option 2 was comprised of a One Way Concrete slab with beams and girders. The weight per square foot of this system was the greatest of all possibilities. Its cost was higher than precast and steel construction. Due to the stiffness and thickness of the members, vibration and deflection are not much of an issue. The system depth is twice that of the existing system but would not require an increase in building height. A shear wall system could be used with concrete. However since it is heavy the foundations may increase along with the building's base shear. This system could also require extra time and cost during the winter for admixtures and curing. A one way system presented itself as a possible alternative.

Option 3 was designed as a Staggered Truss system with precast planks. This system was the most expensive because the trusses would require special fabrication and transportation. The depth of the floor and open areas provided high flexibility in the placing of partitions and windows. Lateral resistance is achieved efficiently through exterior moment frames and the inherent stiffness of the design. The construction time could potentially be decreased, but the trusses would require a long lead time. Also, since the Hotel N.E.U.S. has a floor plan that steps back along its length, the cost would be increased due to the unique truss sizes. This system is very efficient and allows for a singular corridor which is what the Hotel N.E.U.S. needs. However, the long column free spans are not needed since the room sizes are already determined and the structure can be contained within them. A staggered truss system is certainly a possible alternative but is less likely due to the increased cost and lead time required.

Appendices

Appendix A: Plans and Sections

MISCELLANEOUS LINTEL SCHEDULE				
WALL THICKNESS	MASONRY OPNG. UP TO 4'–0"	MASONRY OPNG. 4'–0" TO 6'–0"	MASONRY OPNG. 6'–0" TO 8'–0"	MASONRY OPNG. 8'–0" TO 13'–0"
4" WALL	L$3\frac{1}{2}$x$3\frac{1}{2}$x$\frac{5}{16}$	L4x$3\frac{1}{2}$x$\frac{5}{16}$	L5x$3\frac{1}{2}$x$\frac{5}{16}$	C7x8.9 + PL$\frac{5}{16}$x$5\frac{1}{2}$
6" WALL	⊥L$3\frac{1}{2}$x$2\frac{1}{2}$x$\frac{5}{16}$	⊥L$3\frac{1}{2}$x$2\frac{1}{2}$x$\frac{5}{16}$	⊥L$3\frac{1}{2}$x$2\frac{1}{2}$x$\frac{3}{8}$	----
8" WALL	⊥L$3\frac{1}{2}$x$3\frac{1}{2}$x$\frac{5}{16}$	⊥L4x$3\frac{1}{2}$x$\frac{5}{16}$	⊥L5x$3\frac{1}{2}$x$\frac{5}{16}$	----
10" WALL	L5x$3\frac{1}{2}$x$\frac{1}{4}$(*) + L4x$3\frac{1}{2}$x$\frac{1}{4}$(*)	L5x$3\frac{1}{2}$x$\frac{1}{4}$(*) + L4x$3\frac{1}{2}$x$\frac{1}{4}$(*)	L5x5x$\frac{5}{16}$(*) + L4x4x$\frac{5}{16}$(*)	----
12" WALL	⊥LL$3\frac{1}{2}$x$3\frac{1}{2}$x$\frac{5}{16}$	⊥LL4x$3\frac{1}{2}$x$\frac{5}{16}$	⊥LL5x$3\frac{1}{2}$x$\frac{5}{16}$	----

NOTES:

1. PROVIDE MINIMUM 6" BEARING ON BRICK, SOLID OR GROUTED SOLID CONCRETE BLOCK.
2. THIS SCHEDULE IS FOR THOSE OPENINGS NOT SHOWN ON THE STRUCTURAL DRAWINGS. REFER TO ARCH. & MECH. DRAWINGS FOR LOCATION AND SIZE OF OPENINGS FOR NON–BEARING MASONRY WALLS.
3. ALL EXTERIOR LINTELS SHALL BE HOT DIP GALVANIZED OR COLD GALVANIZED W/ ZRC GALVANIZING COMPOUND.
4. ALL ANGLES LONG LEG VERT. UNLESS NOTED BY (*). WHEN NOTED BY (*) USE LONG LEG HORIZ.
5. SEE LINTEL DETAIL "3".

MISCELLANEOUS MASONRY LINTEL SCHEDULE FOR NON–LOAD BEARING WALLS

LINTEL SCHEDULE					
MARK	SIZE	B PL (txaxb)	MAX. M.O.	REMARKS	MARK
L1	⊥⊥L$3\frac{1}{2}$x4x$\frac{5}{16}$	----	4'–0"	SEE "TYP. LINTEL DETAIL 1"	L1
L2	⊥L$3\frac{1}{2}$x4x$\frac{5}{16}$	----	4'–0"	SEE "TYP. LINTEL DETAIL 2"	L2
L3	⊥⊥L$3\frac{1}{2}$x6x$\frac{5}{16}$	----	5'–6"	SEE "TYP. LINTEL DETAIL 1"	L3
L4	⊥L$3\frac{1}{2}$x6x$\frac{5}{16}$	----	5'–6"	SEE "TYP. LINTEL DETAIL 2"	L4

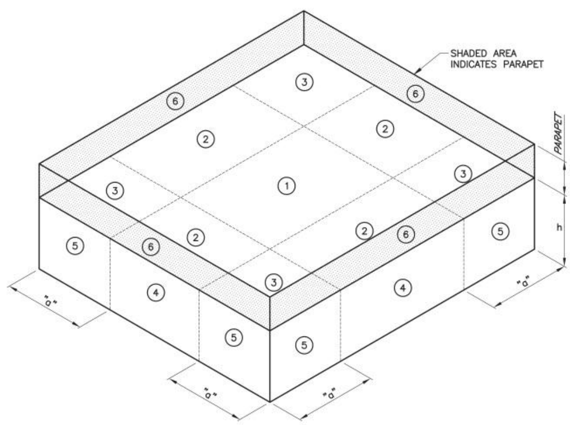
LINTEL NOTES:

1. PROVIDE MINIMUM 6" BEARING ON LOAD BEARING BRICK OR SOLID CONCRETE BLOCK @ EACH END.
2. ALL EXTERIOR LINTELS SHALL BE HOT DIP GALVANIZED.
3. ALL ANGLES LONG LEG VERT. UNLESS NOTED BY (*). WHEN NOTED BY (*) USE LONG LEG HORIZ.
4. FOR LINTEL BEAMS OVER 8" IN DEPTH, PROVIDE MASONRY ANCHORS FROM BEAM WEB TO MASONRY @ 8" O.C. VERT. & @ 16" O.C. HORIZ.
5. SIZE OF LINTEL OPENING AND BEARING ELEVATION TO BE COORD. W/ ARCH. DWGS.

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

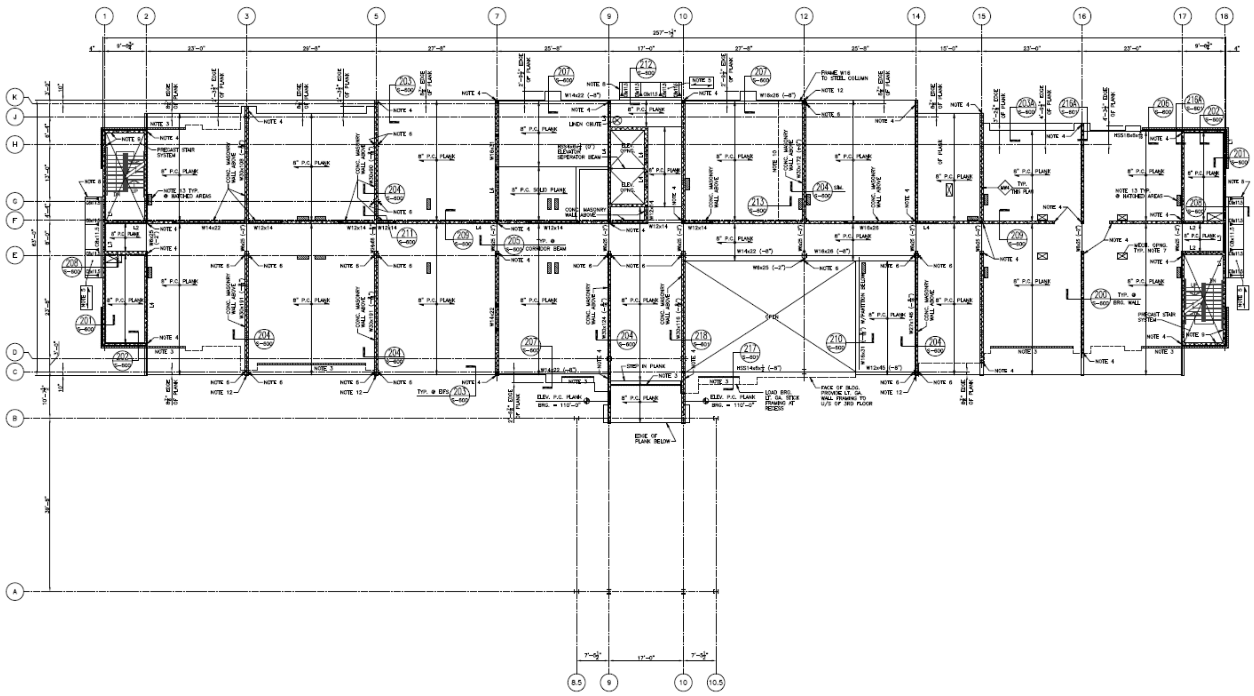
NOTES:

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

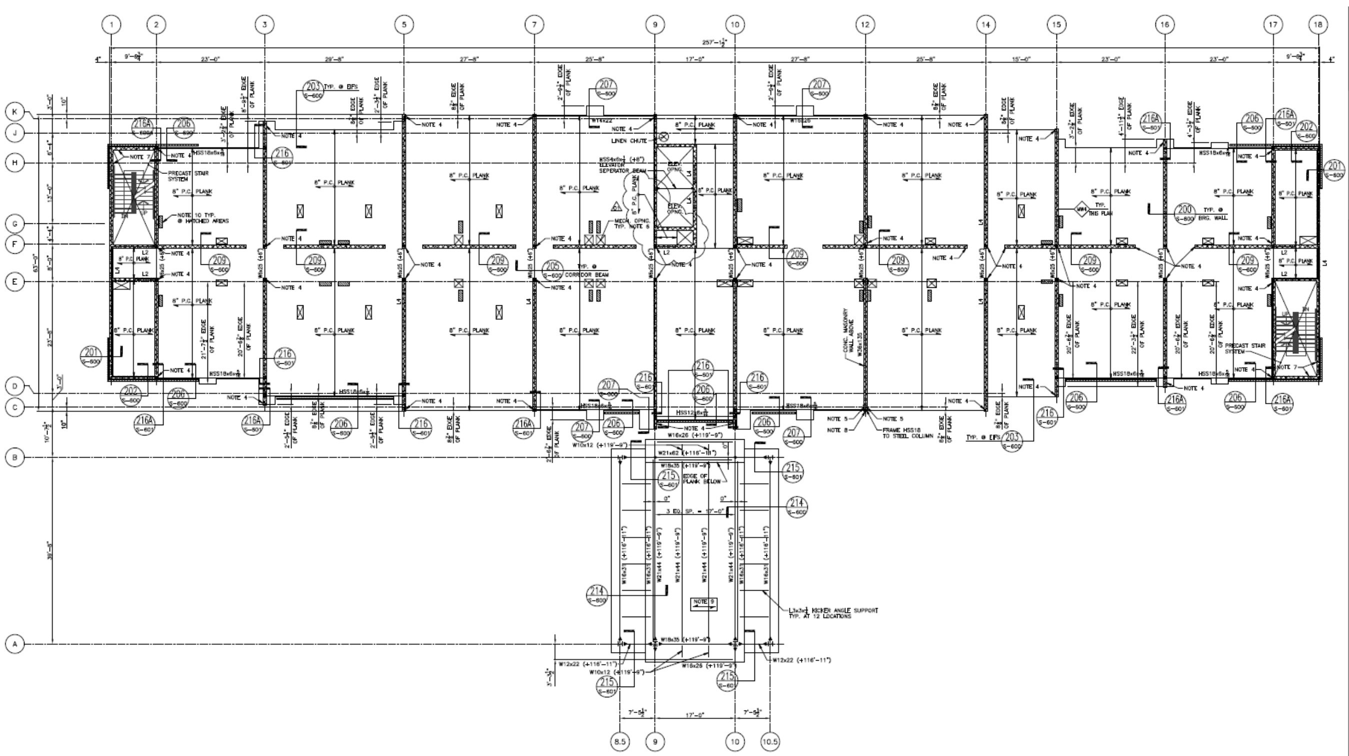


ROOF AND WALL ZONES

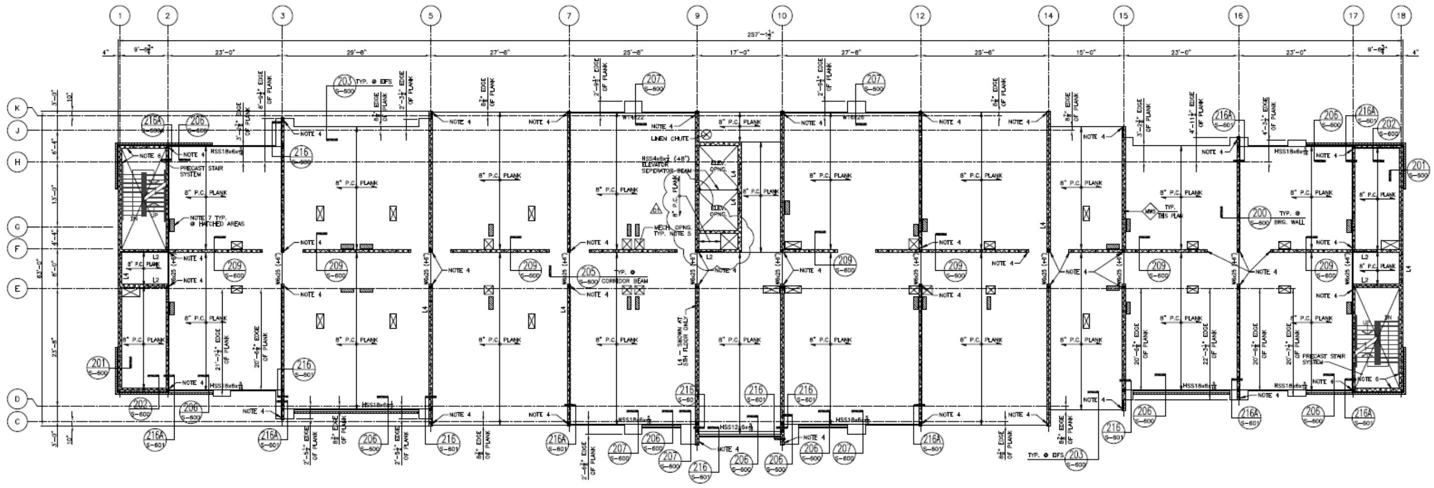
- ✓ International Mechanical Code (IMC 2009)
- ✓ International Plumbing Code (IPC 2009)
- ✓ International Fire Code (IFC 2009)
- ✓ National Fire Protection Associations (NFPA)
- ✓ ADA Accessibility Guidelines (ADAAG) and American National Standards Institute (ANSI)



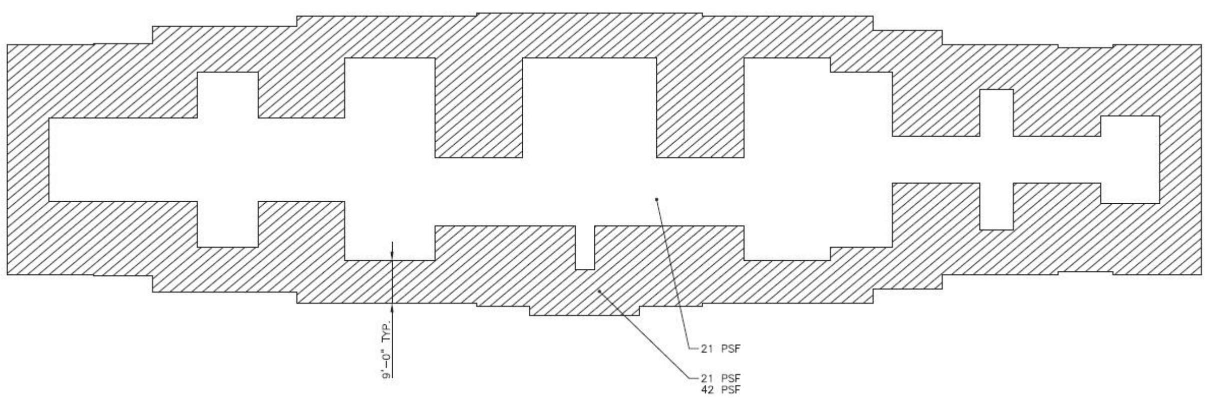
SECOND FLOOR FRAMING PLAN
SCALE: 1/8" = 1'-0"



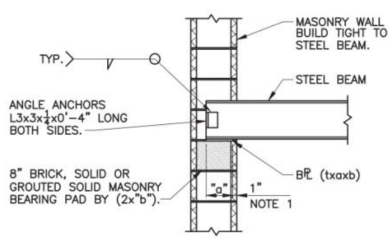
THIRD FLOOR FRAMING PLAN
SCALE: 1/8" = 1'-0"



FOURTH AND FIFTH FLOOR FRAMING PLAN



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

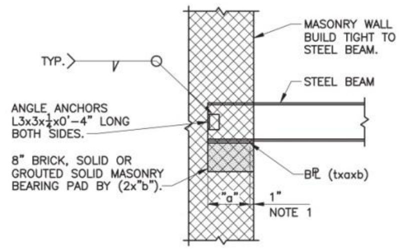


NOTES:

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

TYPICAL STEEL BEAM BEARING ON MASONRY WALL DETAIL

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

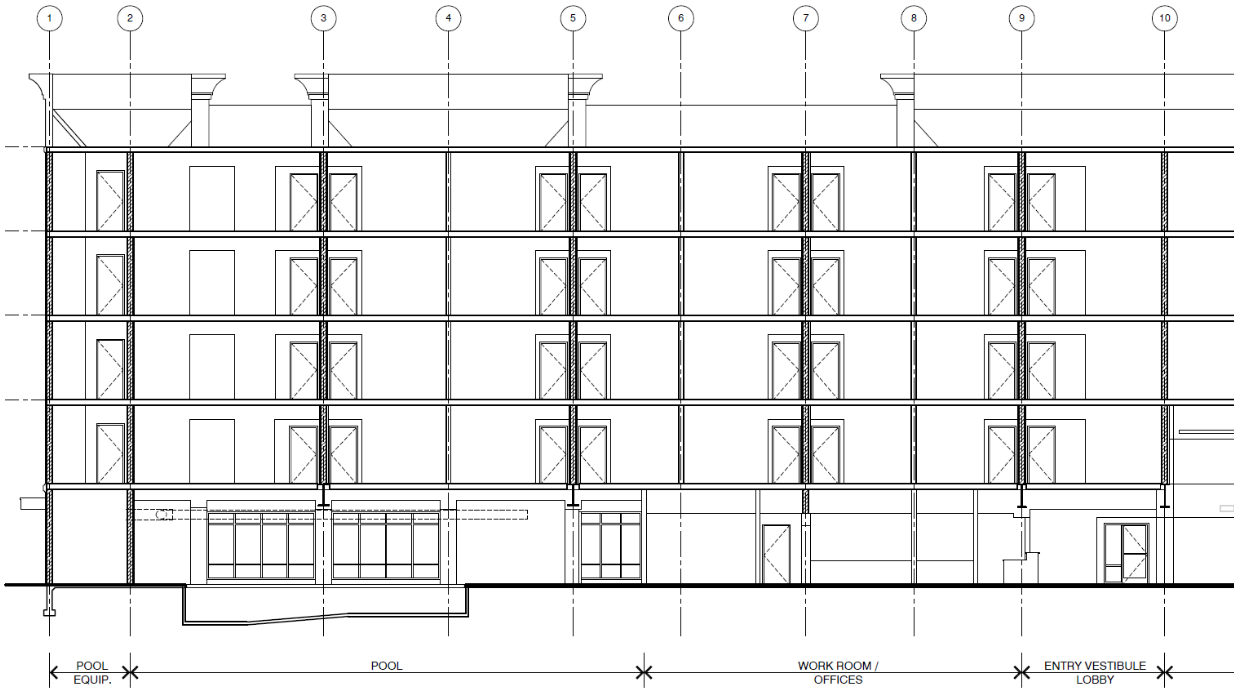


NOTES:

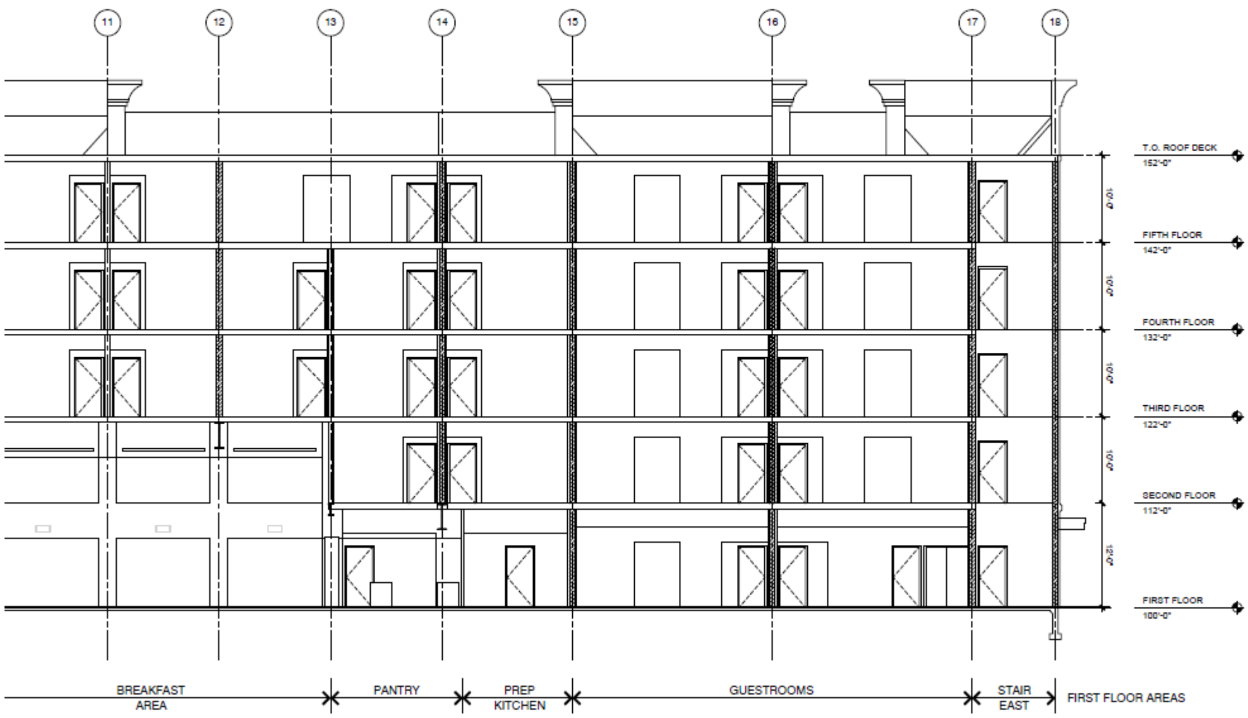
1. FOR BPL'S THAT ARE 1" SMALLER THAN THE MASONRY WALL, CENTER THE BPL ON THE WALL.

TYPICAL STEEL BEAM BEARING ON MASONRY END WALL DETAIL

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

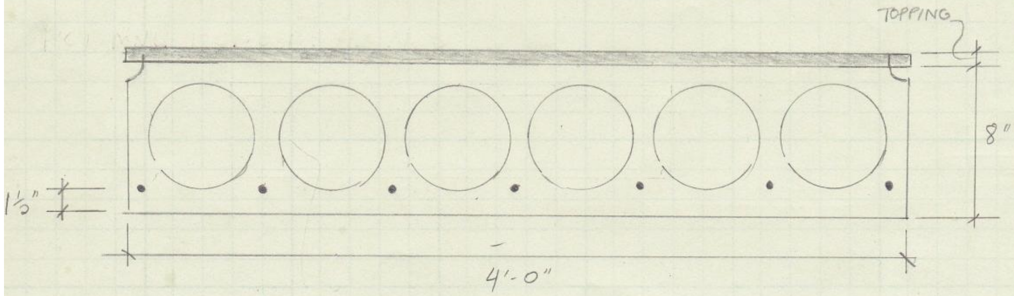


A12 **EAST-WEST BUILDING SECTION**
1/8" = 1'-0"



Appendix B: Existing System Evaluation

TECH 1 | PLANK SPOT CHECK 1 | JORDAN RUTHERFORD



$f'_c = 5000$ psi
$f'_{pu} = 270000$ psi

CHECK 6 STRAND
$n = 6$
DIA. = 6/16"
A_{STRAND} 0.085 in²

$A = 215$ in²
$I = 1666$ in⁴
$y_b = 4"$
$y_t = 4"$
$S_b = 417$ in³
$S_t = 417$ in³
wt = 224 plf
DL = 56 psf
V/s = 1.92 in

• LOADS APPLIED: GUEST ROOM 223 (SERVICE)

LIVE LOAD:	40 psf
PARTITIONS:	20 psf
MEP/MISC:	5 psf
CEILING:	3 psf
3/4" TOPPING:	6 psf
SELFWEIGHT:	56 psf
	130 psf

PLANK IS SIMPLE SPAN:

$$M = \frac{wl^2}{8} = \frac{520\,(25.667)^2}{8} = 42.821 \text{ ft-k}$$

$$M = 42.821 \text{ ft-k} \cdot \frac{12"}{ft} = 513.86 \text{ k-in}$$

$$w = (130 \text{ psf})(4') = 520 \text{ plf}$$

• EFFECTS WITHOUT PRESTRESSING

$$f_t = \frac{M}{S_t} = \frac{513.83 \text{ k-in}}{417 \text{ in}^3} = -1.232 \text{ ksi}$$

$$f_b = \frac{M}{S_b} = \frac{513.83 \text{ k-in}}{417 \text{ in}^3} = 1.232 \text{ ksi}$$

TECH 1	PLANK SPOT CHECK 2	JORDAN RUTHERFORD

• PRESTRESSING:

$P_e = 0.6(270000\,psi)(0.085\,in^2)(6) = 82.62\,k$

EFFECT OF P_e

$\frac{P_e}{A} = \frac{82.62\,k}{215\,in^2} = -0.3843\,ksi$

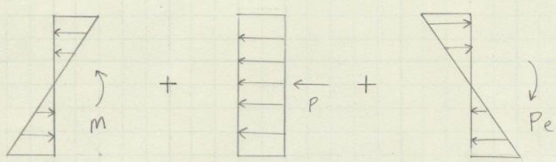
• EFFECTS OF M

$e = 4'' - 1.25'' = 2.75''$

$P_e(e) = 82.62\,k\,(2.75'') = 227.205\,k\text{-}in$

$f_t = \frac{M}{S} = \frac{227.205\,k\text{-}in}{215\,in^3} = 1.057\,ksi$

$f_b = \frac{M}{S} = \frac{227.205\,k\text{-}in}{215\,in^3} = -1.057\,ksi$



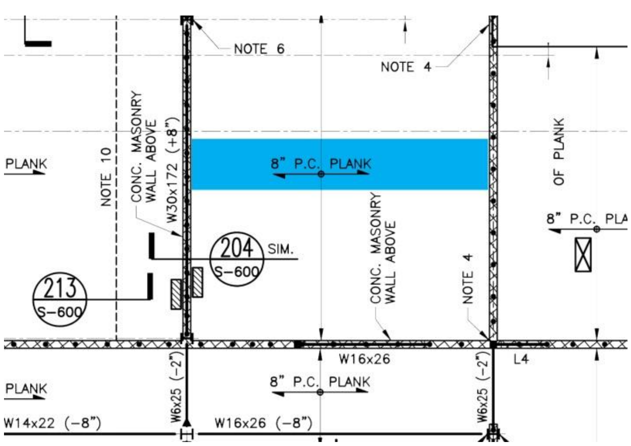
$f_t = -1.232\,ksi - 0.3843\,ksi + 1.057\,ksi = -0.5593\,ksi$

$f_b = 1.232\,ksi - 0.3843\,ksi - 1.057\,ksi = -0.2093\,ksi$

$f_t = -0.5593\,ksi < 3\,ksi = 0.6f'_c$ (ACI 318-11 18.4)

$f_b = -0.2093\,ksi < 0.424\,ksi = 6\sqrt{f'_c}$ (ACI 318-11 18.4)

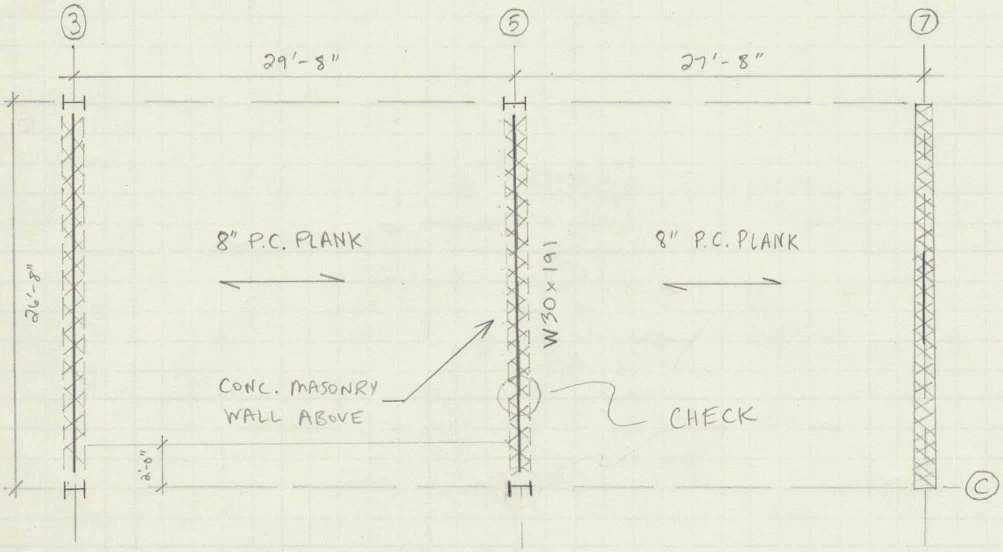
RESULT: OVERPRESTRESSED, BUT DUE TO ASSUMPTION OF TOPPING ADDING TO LOAD BUT NOT TO SECTION MODULUS, PLANK IS OK ✓



A prestressed analysis was used to determine whether the plank used in Guestroom 223. The planks are 8" Hollowcore precast concrete with prestressed strands and is 25'-8" long. The values used in this check were obtained from the PCI Manual 120-04. These values may differ slightly from those of the manufacturers listed in the specifications.

A plank with 6 strands at 6/16" was found to be overprestressed for the loads it has to carry. The reason for performing this analysis was to understand the effects of the prestressed strands. However in practice, many engineers will use the load tables to save time on projects. In Figure 24 you can see the table of safe loads and highlighted is the span of the plank in Guestroom 223. A total of 130 psf exists on the plank, thus a 48-S plank can be used to satisfy the capacity requirements.

TECH 1 | BEAM SPOT CHECK 1 | JORDAN RUTHERFORD



BEAM C5-E5

LOADS:

DEAD: 8" PLANK : 56 psf
3/4" TOPPING : 6 psf
MEP/MISC : 15 psf

8" PLANK : 56 psf
MEP/MISC : 10 psf
INSULATION : 10 psf

LIVE: GUEST ROOM : 40 psf
PARTITIONS : 20 psf
ROOF : 57 - 21 psf
USE 36 psf

WALL : 49 psf

TRIBUTARY AREA: $\left(\frac{29.667'}{2}\right)(24.667') + \left(\frac{27.667'}{2}\right)(26.667') = \boxed{734.8\ ft^2}$

LL REDUCTION: $L_{max} \left| \begin{array}{l} L_o\left(.25 + \frac{15}{\sqrt{K_{LL} A_t}}\right) = 60\left(.25 + \frac{15}{\sqrt{1469.6}}\right) = \boxed{38.47\ psf} \\ 0.4 L_o \qquad 0.4(60) = 24\ psf \end{array} \right.$

LINEAR LOAD:

$$w_u = 1.2\left[(3)(77)(42.5) + (76)(42.5) + (49)(40)\right] + 1.6\left[(3)(38.47)(42.5) + (36)(42.5)\right]$$

$$w_u = 18009 + 10295 = 28304\ plf$$

MOMENT:

$$M_u = \frac{(25974\ plf)(26.667')^2}{8} = 2316\ ft\text{-}k < 2530\ ft\text{-}k \quad \text{OK} \checkmark$$

AISC 14th (TABLE 3-2)

TECH 1	BEAM SPOT CHECK 2	JORDAN RUTHERFORD

DEAD/LIVE LOAD DEFLECTION:

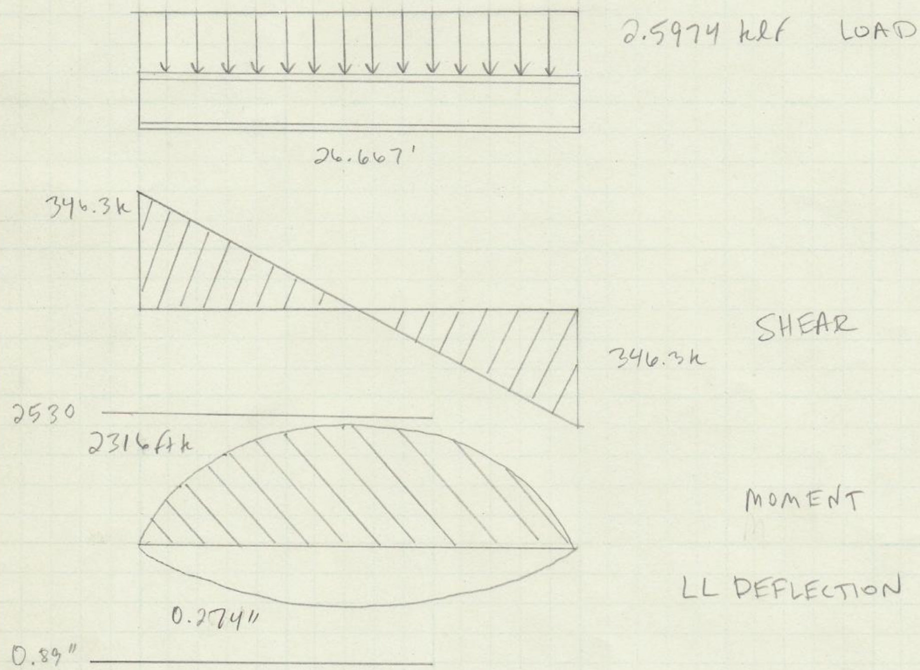
$$w_{LL} = (3)(38.47)(42.5) + (36)(42.5) = 6432 \text{ plf}$$

$$\Delta_{LL} = \frac{5 w_{LL} \ell^4}{384 E I}$$

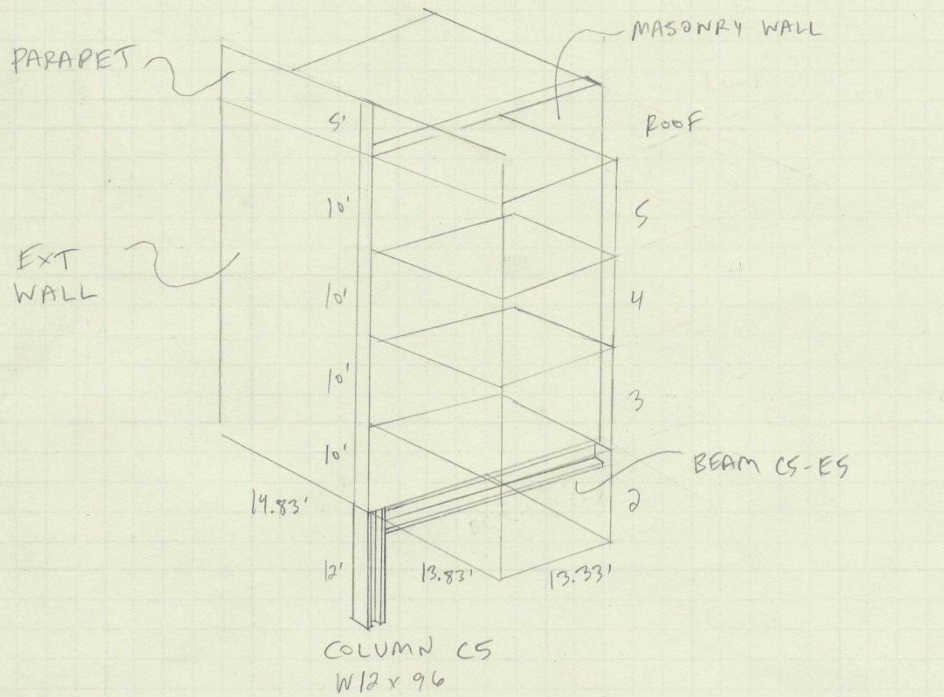
$$\Delta_{LL} = \frac{5(6.43\text{ klf})(26.667')^4(1728)}{384(29000\text{ ksi})(9200\text{ in}^4)} = 0.274'' < 0.89'' = \frac{(26.667')(12)}{360}$$

$$w_{DL} = (3)(77)(42.5) + (76)(42.5) + (49)(40) = 15008 \text{ plf}$$

$$\Delta_{DL} = \frac{5(15.01)(26.667)^4(1728)}{384(29000)(9200)} = 0.64'' < 1.333'' \quad \frac{(26.667')(12)}{240}$$



TECH 1	COLUMN SPOT CHECK	1	JORDAN RUTHERFORD



COLUMN C5:

FLOOR LOADS:

DEAD:
- 8" PLANK : 56 psf
- 3/4" TOPPING : 6 psf
- MEP/MISC : 15 psf
- WALL : 49 psf

LIVE:
- GUEST ROOM : 40 psf
- PARTITIONS : 20 psf

ROOF LOADS:

DEAD:
- 8" P.C. PLANK : 56 psf
- 8" INSULATION : 10 psf
- MEP/MISC : 10 psf

LIVE: 57 — 21

USE 41 psf AVERAGE

EXT WALL:
- INSULATION : 5 psf
- C.F. STUDS : 1.5 psf
- 16" o.c.

TRIB AREA: $\left(\frac{29.667'}{2}\right)\left(\frac{26.667'}{2}\right) + \left(\frac{27.667'}{2}\right)\left(\frac{26.667'}{2}\right) = 383\ ft^2$

LL REDUCTION: L_{MAX}

$$60\left(.25 + \frac{15}{\sqrt{765}}\right) = \boxed{47.55\ psf}$$

$$0.4(60) = 24\ psf$$

TECH 1	COLUMN SPOT CHECK 2	JORDAN RUTHERFORD

AXIAL LOAD:

ROOF: $1.2[(76)(383)] + 1.6[(36)(383)] = 56990.4k$

FLOORS 2-4: $1.2[3(77)(383) + (49)(40)(\frac{26.667}{2}) + (6.5)(45)(\frac{29.667}{2} + \frac{27.667}{2})$
$+$
$1.6[3(47.55)(383)] = 235006\ k$

$P_u = \boxed{292\ k}$ (VERY CLOSE TO 295 k TO BASE PLATE ON COLUMN SCHEDULE)

CHECK BUCKLING:

W12×96 ASSUMING k=1

AREA = 28.2 in²
$r_y = 3.09$

$\frac{kL}{r} = \frac{(1)(12)(12)}{4} = 36 < 113 = 4.71\sqrt{\frac{29000}{50}}$

$F_e = \frac{\pi^2 E}{(\frac{kL}{r_y})^2} = \frac{\pi^2(29000)}{\left(\frac{(1)(12)(12)}{3.09}\right)^2} = 131.8$

$F_{cr} = [0.658^{50/131.8}]\,50 = 42.65\ ksi$

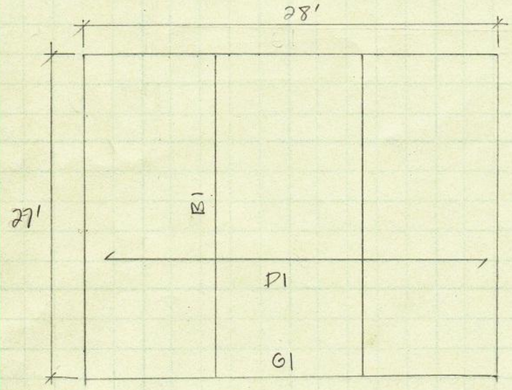
$P_n = A_g F_{cr} = (28.2\ in^2)(42.65\ ksi) = 1202\ k$

$\phi P_n = 0.9\,(1202\ k) = \boxed{1082k > 288.9\ k}$ OK ✓

1080 k
(4-1)

Appendix C: Composite Steel Design

TECH 2 | COMP. STEEL | JORDAN RUTHERFORD | 1



3 SPACES @ 9'-4"

DEAD LOAD: 51 psf
LIVE LOAD: 60 psf
SDL : 12 psf
EXT WALL: 17 psf
BM SLFWT: 5 psf

D1:

FOR 2HR FIRE RATING SPRAYED: 2½ NW TOPPING

USE $\boxed{\text{VULCRAFT 3VLI22}}$ (REFER TO PG. 54, 71 VULCRAFT MNL)

SDI MAX UNSHORED 3 SPAN: 10'-11" > 9'-3" OK ✓

SUPERIMPOSED LIVE LOAD: 72 psf < 124 psf OK ✓
@ 9'-3" @ 9'-6"

B1:

LL REDUCTION: $L = 60\left(.25 + \frac{15}{\sqrt{504}}\right) = 55.1$ psf

$b_{eff} = \min \begin{cases} (28)(12)/8 = 42" \times 2 \\ (9.33)(12)/2 = 56" \end{cases}$ USE 84"

$w_u = 1.2[(51+5+12)(9.33)] + 1.6[(55.1)(9.33)] = 1.58$ klf

$M_u = \frac{1.58 \text{ klf } (27')^2}{8} = 144$ ft-k

$V_u = \frac{1.58 \text{ klf } (27')}{2} = 21.33$ k

ASSUME $a = 1"$

$Y_2 = 5.5" - .5" = 5"$

STUDS: ⊥, 3/4, WEAK, 1 PER RIB = 17.2 k

TECH 2	COMP. STEEL	JORDAN RUTHERFORD	2

- SOLVE FOR I_{LB} + I

$\Delta_{LL} = \frac{(27')(12)}{360} = 0.9''$

$w_{LL} = (55.1)(9.33) = 0.514$ klf

$I_{LB} = \frac{5(0.514)(27^4)(1728)}{384(29000)(0.9)} = 235$ in^4

$\Delta_{TL} = \frac{(27)(12)}{240} = 1.35''$

$w_{TL} = (51 + 12 + 5)(9.33) + (55.1)(9.33) = 1.15$ klf

$I = \frac{5(1.15)(27^4)(1728)}{384(29000)(1.35)} = 351$ in^4

- STUDS: 3/4", $\perp$, WEAK, 1 PER RIB $Q_n = 17.2$ k

- SIZE

W12x26 $\Sigma Q_n = 95.6/17.2 = 5.5 \Rightarrow$ 12 STUDS
$\phi M_n = 204$ ft-k
$I_{LB} = 392$ in^4 (CAMBER)

- CHECK UNSHORED STRENGTH

$w_u = 1.2(51\text{psf})(9.33') + 1.2(26\text{plf}) + 1.6(20\text{psf})(9.33') = 0.91$ klf

$M_u = 0.91\text{klf}(27')^2/8 = 83$ ft-k $<$ 140 ft-k OK ✓

- CHECK WET CONC. DEFL.

$w_{WC} = (51\text{psf})(9.33') + 22\text{ plf} = 0.501$ klf

$\Delta_{WC} = \frac{5(.501)(27^4)(1728)}{384(29000)(204)} = 1.01'' < 1.35'' = \frac{27(12)}{240}$ OK ✓

- CHECK LL DEFL.

$\Delta_{LL} = \frac{5(0.514)(27^4)(1728)}{384(29000)(392)} = 0.59'' < 0.9''$ $\frac{24(12)}{360}$ OK ✓

- CHECK TL DEFL.

$\Delta_{TL} = \frac{5(1.15)(27^4)(1728)}{384(29000)(392)} = 1.21'' < 1.35''$ OK ✓

$\boxed{\text{W12 x 26 w/ 12 STUDS}}$

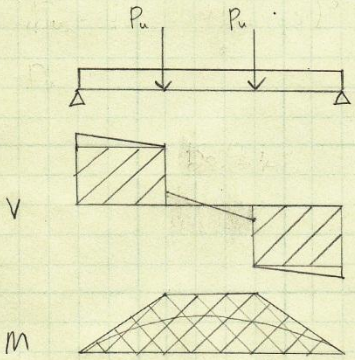
TECH 2	COMP. STEEL	JORDAN RUTHERFORD	3

G1

LL REDUCTION: $L = 60\left(.25 + \frac{15}{\sqrt{672}}\right) = 49.7\ psf$

$w_u = \underset{\text{WALL EXT}}{1.2(7psf)(10')} + \underset{\text{SLF WT}}{1.2(2psf)(35')} = .168\ klf$ $\quad M = \frac{.168(28)^2}{8} = 16.5\ ft\text{-}k$

$P_u = [1.2(51+12) + 1.6(49.71)](9.33')(13.5') + \underset{\text{BEAM}}{(22\ plf)(13.5')} = 19.5\ k$



$182 + 16.5 = 198\ ft\text{-}k$

NOTE: THIS IS AN EXTERIOR GIRDER, THEREFORE IT INCLUDES THE FACADES WEIGHT. THE INTERIOR GIRDER LABELED "G2" WOULD HAVE SOME ADDED LOADED DUE TO THE CORRIDOR BETWEEN THE BAY OPPOSITE THIS ONE. THEREFORE IT WILL MOST LIKELY BE A SIMILAR SIZE. FOR THAT REASON, GIRDER "G1" WAS SELECTED FOR THIS ANALYSIS.

- SOLVE FOR I TO LIMIT LL DEFL.

$\Delta_{LL} = \frac{(28')(12)}{360} = 0.933''$

$I_{LB} = \frac{6.3(28^3)(1728)}{28(29000)(0.933)} = 315\ in^4$

$\Delta_T = \frac{(28')(12)}{240} = 1.4'$

$I = \frac{14.8(28^3)(1728)}{28(29000)(1.4)} = 474\ in^4$

- STUDS: 3/4", ||, WEAK, 1 PER RIB : $Q_n = 21.5\ k$
- SIZE: ASSUME $a = 1''$ $\quad 5.5'' - .5'' = 5'' = Y2$

W14x26 $\quad \Sigma Q_n = 96.1/21.5 = 4.5 \Rightarrow 10$ STUDS

$\phi M_n = 223\ ft\text{-}k$

$I_{LB} = 465\ in^4$ (CAMBER)

TECH 2	COMP STEEL	JORDAN RUTHERFORD	4

- CHECK UNSHORED STRENGTH

$P_u = [1.2(51 psf + 26 psf) + 1.6(20 psf)](9.33')(13.5') = 15.67 k$

$M_u = \frac{15.67(28')}{2} = 219 ft k > 151 ft k$ NG X

TRY W18×35 $M_u = 248 ft\text{-}k$ $\phi M_p = 249 ft k$ OK ✓

$\Sigma Q_n = 129k / 21.5 = 6 \Rightarrow 12$ STUDS

$\phi M_n = 363 ft k$

$I_{LB} = 906 in^4$

- CHECK WET CONC. DEFL.

$P_{u,WC} = (51 psf)(9.33')(13.5') + (40 plf)(13.5') = 6.96 k$

$\Delta_{WC} = \frac{6.96(28^3)(1728)}{28(29000)(510)} = 0.64'' < 1.4''$ OK ✓

- CHECK LL DEFL

$P_{u,LL} = 6.3$

$\Delta_{LL} = \frac{6.3(28^3)(1728)}{28(29000)(910)} = 0.32'' < 0.933''$ OK ✓

- CHECK TL DEFL

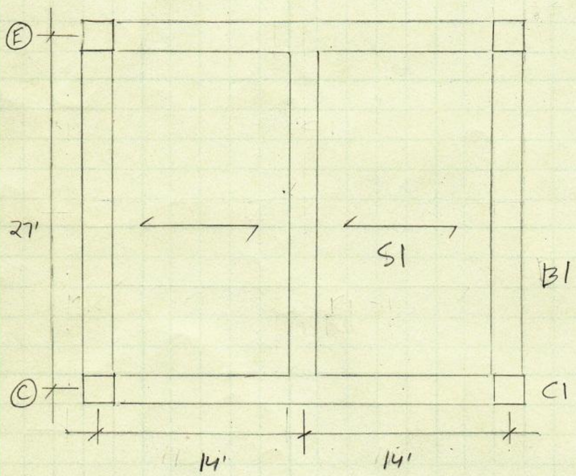
$P_{u,TL} = 14.8 k$

$\Delta_{TL} = \frac{14.2(28^3)(1728)}{28(29000)(910)} = 0.79'' < 1.4''$ OK ✓

$\boxed{\text{W18×35 W/ 12 STUDS}}$

Appendix D: One Way Concrete Design

TECH 2 | 1 WAY SLAB W/ BEAM | JORDAN RUTHERFORD | 1



$f'_c = 4000$ psi
$f_y = 60\,000$ psi

ASSUMING: 12" x 12" COL

S1:

• MINIMUM THICKNESS (ACI 318-11 TABLE 9.5a):

SOLID 1 WAY SLAB, BOTH ENDS CONT. $h = \frac{l}{28} = \frac{(14)(12)}{28} = \boxed{6"}$

$d = 5"$ FOR 3/4" CC (7.7.1)

• LOADS

SELFWEIGHT: $(\frac{6}{12})(150\,pcf) = 75$ psf
SDL: 12 psf
LIVE: 60 psf

$w_u = 1.2[(87\,psf)(1')] + 1.6(60)(1') = .166$ klf

$M_{u+} = \frac{0.166\,klf\,(14')^2}{12} = 2.71$ ft-k

$M_{u-} = \frac{0.166\,klf\,(14')^2}{24} = 1.36$ ft-k

• REINFORCEMENT

ASSUMING $\rho \approx 1\%$ $A_s = \frac{M_u}{4d} = \frac{2.71}{4(5)} = 0.1355$ in² < 0.2 in²
#4 @ 12" O.C.

TEMPERATURE: $A_t = 0.0018(12")(6") = 0.1296$ in² < 0.13 in²
#4 @ 18" O.C.

SPACING: $S \leq 15' - 2.5(3/4") = 13 \frac{1}{8}"$

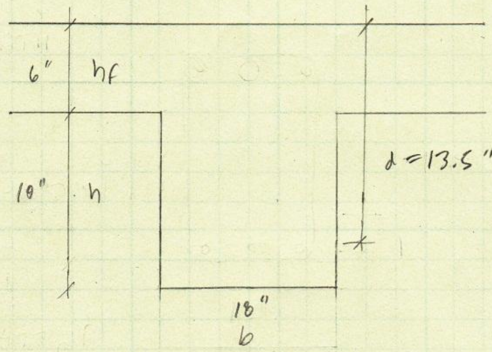
TECH 2 | 1 WAY SLAB W/BEAM | JORDAN RUTHERFORD | 2

B1

ASSUME $b = 12"$

$h_t = \frac{(27)(12)}{18.5} = 17.5" \Rightarrow$ USE $18"$

$d = 16" - 1.5" - .5"_{\#4} - .5"_{\#4} = 13.5"$



- LOADS:

  SLAB $= \frac{6}{12}(150) = 75$ psf

  SDL $= 12$ psf

  LIVE $= 60$ psf

  SELF $= \frac{(12)(12)}{144}(150) = .15$ klf

CONSERVATIVE

$w_u = 1.2[(75+12)(14') + .15] + 1.6[(60)(14)] = 3$ klf

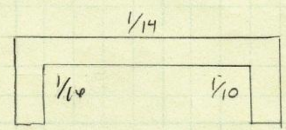
USING MOMENT COEFFIENT (ACI 318-11 8.3.3)

$M_u^+ = \frac{w_u l_n^2}{14} = \frac{3(25.5')^2}{14} = 140$ ft-k

NOTE: COLUMN IS ASSUMED AS $12" \times 12"$ WHICH IS CONSERVATIVE

$M_{u\,ext}^- = \frac{w_u l_n^2}{16} = \frac{3(25.5')^2}{16}$ 122 ft-k

$M_{u\,INT}^- = \frac{w_u l_n^2}{10} = \frac{3(25.5')^2}{10}$ 195 ft-k



- EFFECTIVE WIDTH: (318-11 8.12)

$$b_{eff} \le \begin{cases} (27)(12)/4 = & \boxed{81"} \\ 18 + 16"(6") = & 114" \\ 18" + (14' \times 12 - 18") = & 168" \end{cases}$$

$M_{uTBm} = 0.9(.85)(4)(81)(6)(13.5 - 6/2) = 1300$ ft-k $\gg 140$ ft-k

$\therefore$ USE b_{eff} AS b AND DESIGN AS RECT

$A_s = \frac{M_u}{4d} = \frac{140}{4(13.5)} = 2.59$ in^2

$\boxed{\text{USE } (6)\#6 \quad A_s = 2.64 \text{ in}^2}$

$A_{s\,min} = \frac{3\sqrt{4000}}{60000}(12)(13.5) = .512$ in^2

$\frac{200(12)(13.5)}{60000} = .54$ in^2

< 2.64 in$^2 < 3.34$ in$^2 = .0206(12)(13.5)$

OK ✓

TECH 2	1 WAY SLAB W/ BEAM	JORDAN RUTHERFORD	3

- CHECK ASSUMPTIONS:

$a = \frac{(2.64)(60000)}{.85(4000)(81)} = .575 < 6"$ OK ✓ $\qquad c = \frac{.575}{.85} = .677$

$d = 16" - 1.5" - \underset{\#3}{.375"} - .75"/2 = 13.75"$

$\varepsilon_t = \frac{.003}{.677}(13.75" - .677) = .058 >> .002$ OK ✓

$\phi M_n = \frac{(2.64)(60)(13.75 - \frac{.575}{2})}{12} = 178$ ft-k > 140 ft-k OK ✓

SPACING: MIN # OF BARS PER LAYER = 3 OK ✓
MAX # OF BARS PER LAYER = 8 OK ✓

$b_{min} = 2(1.5") + 2(.375") + 6(.75") + 5(1") = 13.25" < 18"$ OK ✓

- REINFORCEMENT (-)

$A_s = \frac{M_u}{4d} = \frac{195}{4(15.6875)} = 3.1 \text{ in}^2$ $\boxed{\text{USE (4) \#7 } 3.16 \text{ in}^2}$

$d = 16" - 1.5" - .375" - \frac{.875"}{2} = 13.69"$

$b_{min} = 2(1.5") + 2(.375") + 4(.875") + 3(1") = 10.25" < 18"$ OK ✓

- SHEAR REINFORCEMENT (INTERIOR FACE)

$V_c = 2\sqrt{f'_c}\, b_w d = 2\sqrt{4000}(18)(13.75) = 31.3k$

$\phi V_n = 0.5(0.75)(31.3k) = 11.7$ k

$V_u = \frac{3(25.5')1.15}{2} = 43.9$ k

$V_u @ d = 43.9k - (3)(1.15') = 40.45k$

$V_s = \frac{V_u}{\phi} - V_c = \frac{40.45}{.75} - 31.3 = 22.6$ k

$V_s \le 8\sqrt{f'_c}\, b_w d = 8\sqrt{4000}(18)(13.75) = 125$ k OK ✓

$V_s \le 4\sqrt{f'_c}\, b_w d = 4\sqrt{4000}(18)(13.75) = 62.6$ k OK ✓

$S_{MAX} = \frac{d/2'}{24"} = 6.875" \rightarrow 6"$

$A_{v\,MIN}$ | $.75\sqrt{4000}(18)(6)/60000 = .085 \Rightarrow$ USE (1) #3
MAX | $50(18)(6)/60000 = .09$

TECH 2	1 WAY SLAB W/ BEAM	JORDAN RUTHERFORD	4

$S_{MIN} = .22(60)(13.75)/22.6 = 8.03 \rightarrow 8''$

USE ⊔ #3 @ 6" SPACING STARTING 2" FROM FACE

- SHEAR REINFORCEMENT (EXT FACE)

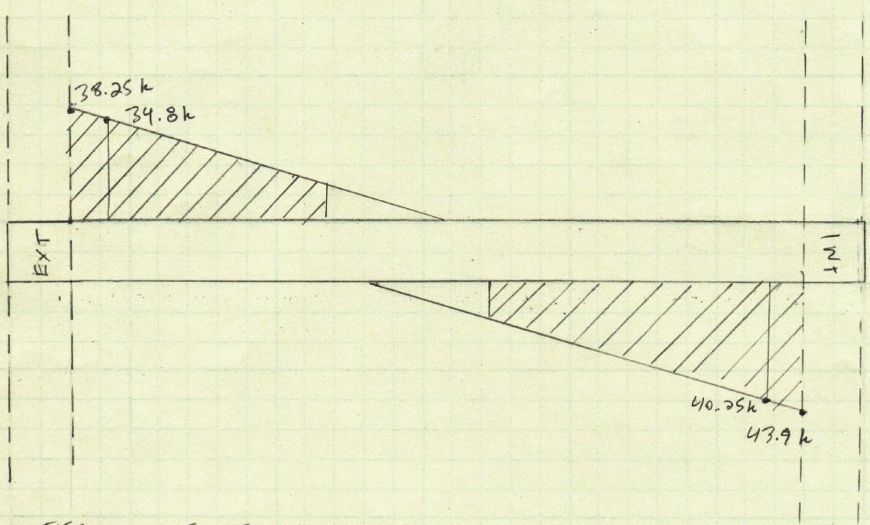
$V_u = 3(25.5)/2 = 38.25$

$V_u @ d = 38.25k - (3)(1.15') = 34.8k$

$V_s = \frac{34.8}{.75} - 31.3 = 15.1k$

$S_{MIN} = .22(60)(13.75)/15.1 = 12''$

USE ⊔ #3 @ 6" SPACING STARTING 2" FROM FACE



TERMINATE REINFORCEMENT:

$43.9k - 3(x) = 11.7k$

$x = 10.73'$

$38.25k - 3(x) = 11.7k$

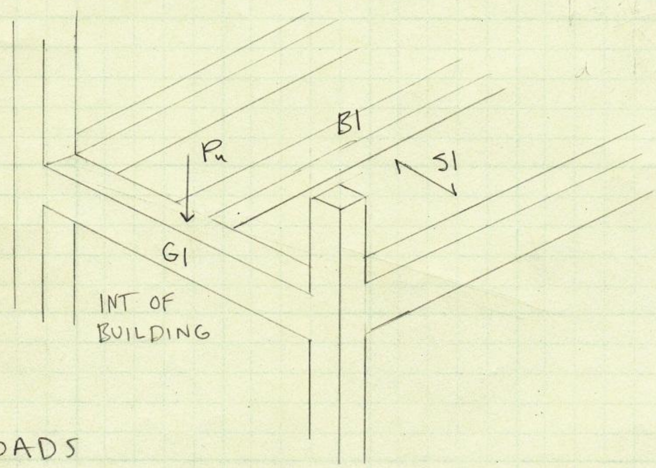
$x = 8.85'$

FOR EASE OF CONSTRUCTION, CONTINUE ⊔ #3 AT 6" O.C. THROUGH WHOLE LENGTH, STARTING 2" FROM FACE OF SUPPORT

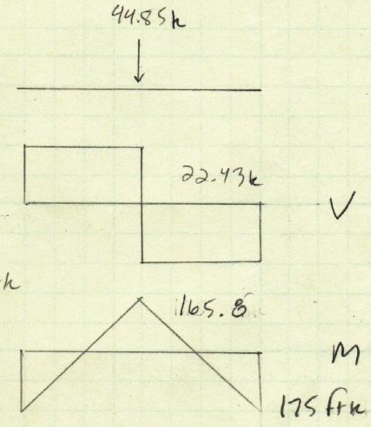
TECH 2	1 WAY SLAB W/ BEAM	JORDAN RUTHERFORD	5

G1

USE SAME DIMENSIONS AS BEAM



• LOADS



$V_u = \frac{(44.85)}{2} = 22.43\ k$

$w_{SLFWT} = \frac{(18)(16)(150)}{144} = .300\ klf$

$M_u^- = \frac{P_u l}{8} + \frac{w l_n^2}{12} = 157 + 17.6 = 175\ ft\text{-}k$

$M_u^+ = 157 + \frac{w l_n^2}{24} = 165.8\ ftk$

• REINFORCEMENT

$A_s^+ = 165.8 / (4)(13.5) = 3.07\ in^2 \Rightarrow$ USE (7) #6 $3.08\ in^2$ ok ✓

$A_s^- = 175 / (4)(13.5) = 3.24\ in^2 \Rightarrow$ USE (8) #6 $3.52\ in^2$ ok ✓

$b_{min} = 2(1.5") + 2(.375") + 8(.75") + 7(1") = 16.75" < 18"$ ok ✓

MAX # OF BARS PER LAYER = 8 OK ✓
MIN # OF BARS PER LAYER = 3 OK ✓

ACTUAL $d = 16" - 1.5" - .375" - (.75"/2) = 13.75"$

TECH 2	1 WAY SLAB W/ BEAM	JORDAN RUTHERFORD	6

• SHEAR REINFORCEMENT

$V_c = 31.3\,k$

$\phi V_n = 11.7\,k$

$V_u = 22.43\,k$

$V_u @ d = 22.43k - (.3)(1.15) = 22.1\,k$

$V_s = \frac{22.1}{.75} - 31.3k = -1.83$

$V_s < 4\sqrt{f'_c}\,b_w d < 8\sqrt{f'_c}\,b_w d$

$S_{MAX} = \begin{cases} d/2'' = 6.875'' \rightarrow 6'' \\ 24'' \end{cases}$

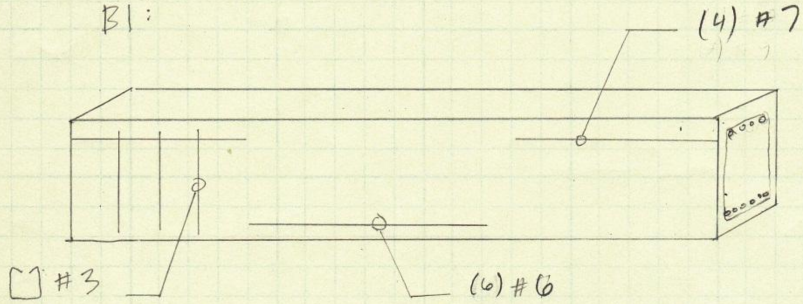
$A_{v\,MIN} \,|\, 0.085 \Rightarrow$ USE U #3

$\quad_{MAX} \,|\, 0.09$

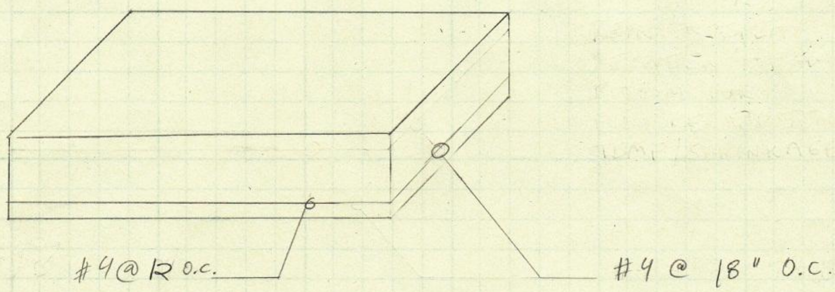
TECH 2 | 1 WAY SLAB W/ BEAM | JORDAN RUTHERFORD | 7

SUMMARY

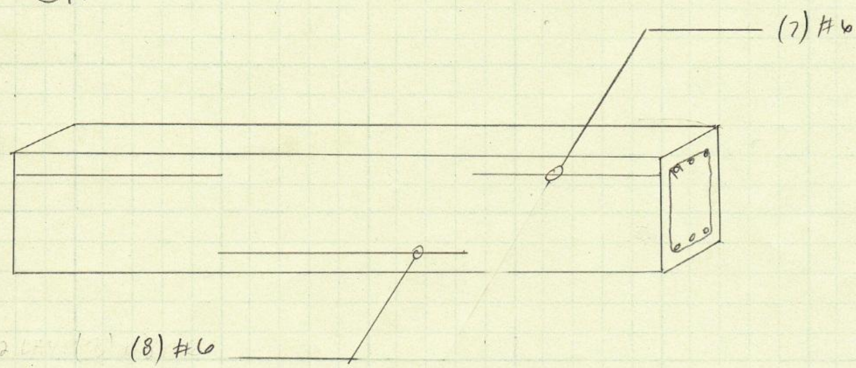
B1:



S1:

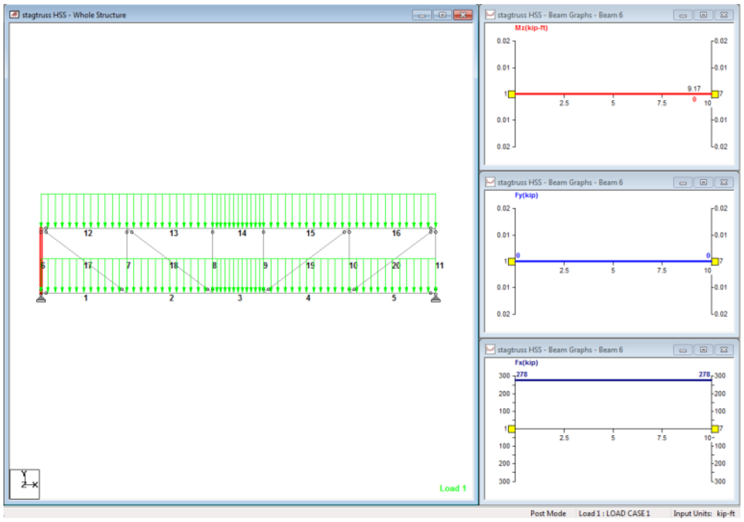


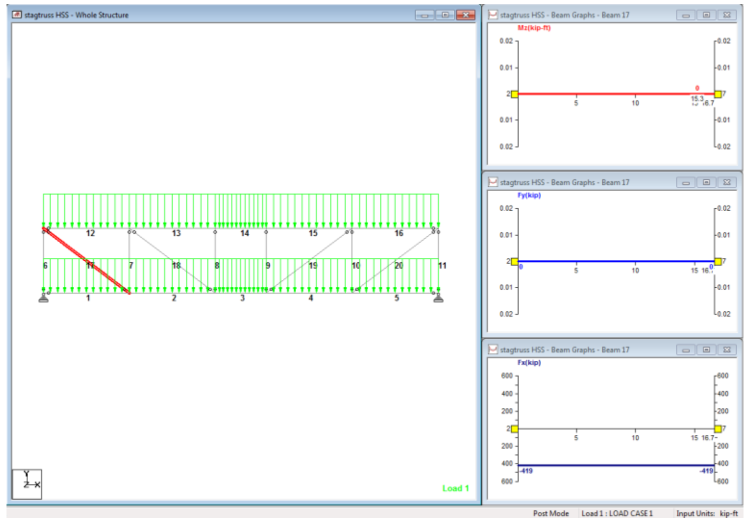
G1:

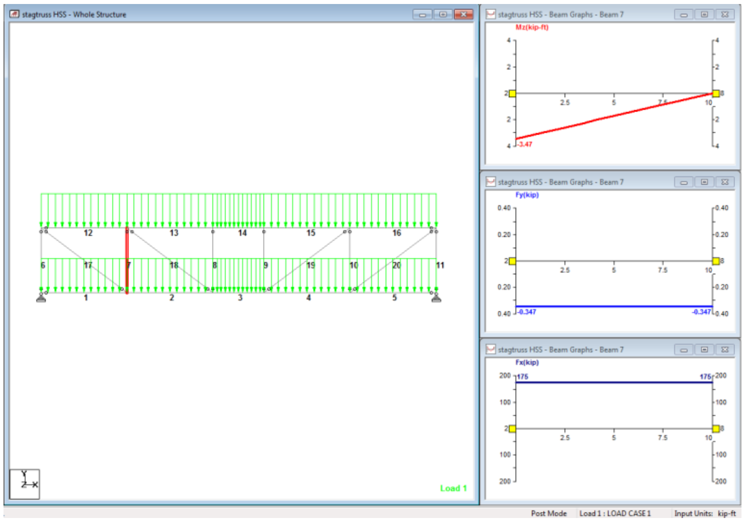


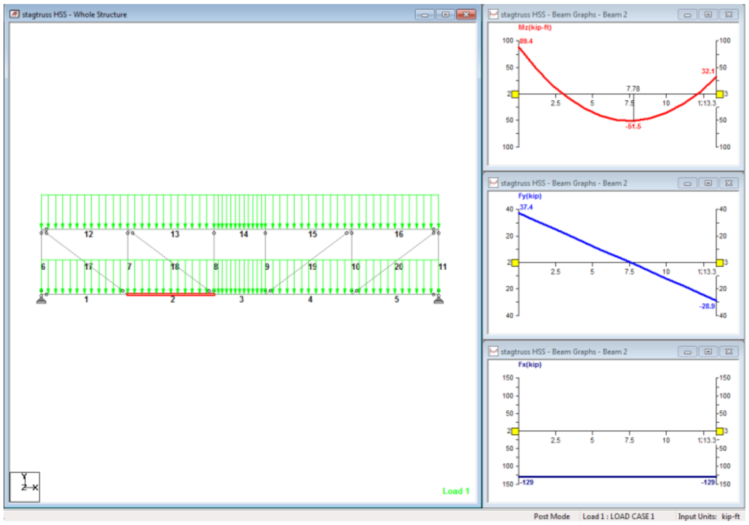
Appendix E: Staggered Truss Design

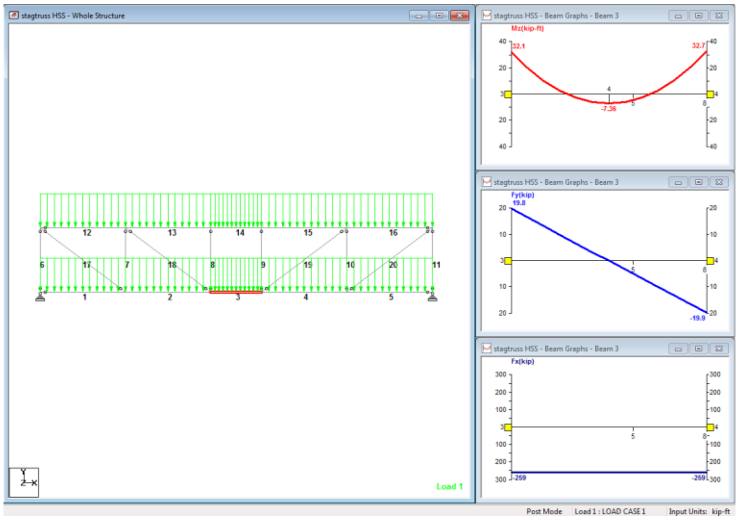
STAAD RESULTS					
Beam	Env	Fx kip	Mz kip-ft	Member	
1	+ve	206.705	85.942	W12x53	Lower forces, sizes used to match top flange
	-ve	0	0		
2	+ve	0	89.408	W12x53	
	-ve	-128.937	0		
3	+ve	0	32.133	W12x53	
	-ve	-259.333	0		
4	+ve	0	32.662	W12x53	
	-ve	-128.738	0		
5	+ve	206.705	87.471	W12x53	
	-ve	0	0		
6	+ve	278.132	0	HSS 6x6x0.5	Controlling Members
	-ve	0	0		
7	+ve	174.519	0	HSS 6x6x0.375	
	-ve	0	0		
8	+ve	49.078	0	HSS 6x6x0.125	
	-ve	0	0		
9	+ve	49.011	0	HSS 6x6x0.125	
	-ve	0	0		
10	+ve	174.722	0	HSS 6x6x0.375	
	-ve	0	0		
11	+ve	278.247	0	HSS 6x6x0.5	
	-ve	0	0		
12	+ve	335.296	87.011	W12x45	
	-ve	0	0		
13	+ve	466.038	87.011	W12x53	
	-ve	0	0		
14	+ve	466.038	34.069	W12x45	
	-ve	0	0		
15	+ve	466.038	33.938	W12x53	
	-ve	0	0		
16	+ve	335.444	86.96	W12x45	
	-ve	0	0		
17	+ve	0	0	HSS 8x8x0.5	
	-ve	-419.158	0		
18	+ve	0	0	HSS 6x6x0.25	
	-ve	-162.965	0		
19	+ve	0	0	HSS 6x6x0.25	
	-ve	-163.258	0		
20	+ve	0	0	HSS 8x8x0.5	
	-ve	-419.342	0		

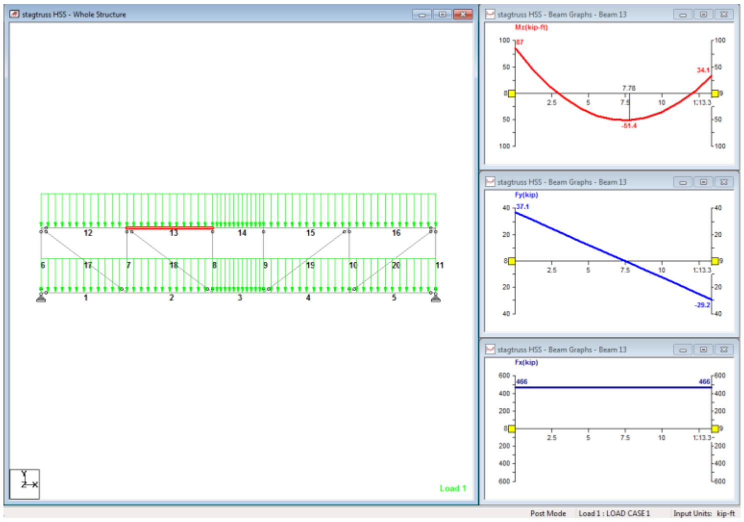

stagtruss HSS - Whole Structure
stagtruss HSS - Beam Graphs - Beam 6
Mz(kip-ft)
Fy(kip)
Fx(kip)
278
278
Load 1
Post Mode
Load 1 : LOAD CASE 1
Input Units: kip-ft

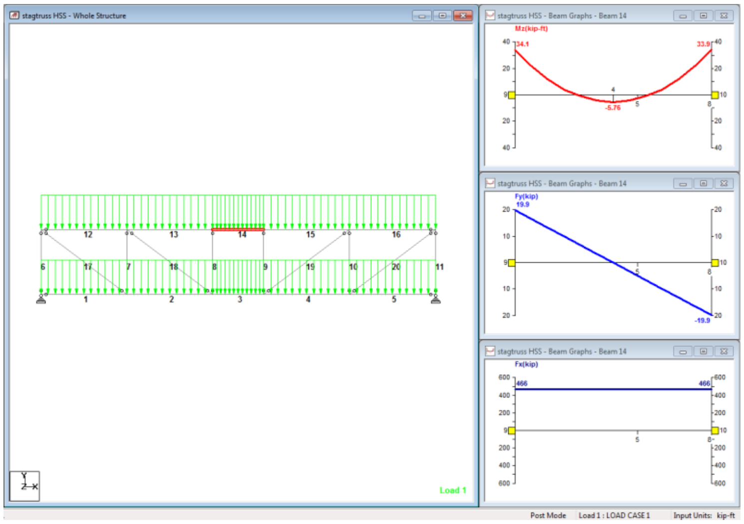

stagtruss HSS - Whole Structure
stagtruss HSS - Beam Graphs - Beam 17
Mz(kip-ft)
Fy(kip)
Fx(kip)
-419
-419
Load 1
Post Mode
Load 1 : LOAD CASE 1
Input Units: kip-ft


stagtruss HSS - Whole Structure
stagtruss HSS - Beam Graphs - Beam 7
Mz(kip-ft)
-3.47
Fy(kip)
-0.347
-0.347
Fx(kip)
175
175
Load 1
Post Mode
Load 1 : LOAD CASE 1
Input Units: kip-ft

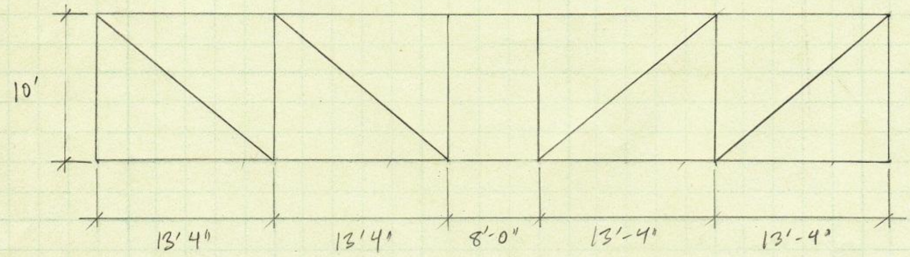

stagtruss HSS - Whole Structure
stagtruss HSS - Beam Graphs - Beam 2
Mz(kip-ft)
Fy(kip)
Fx(kip)
Load 1
Post Mode
Load 1 : LOAD CASE 1
Input Units: kip-ft


stagtruss HSS - Whole Structure
stagtruss HSS - Beam Graphs - Beam 3
Mz(kip-ft)
Fy(kip)
Fx(kip)
Load 1
Post Mode
Load 1 : LOAD CASE 1
Input Units: kip-ft


stagtruss HSS - Whole Structure
stagtruss HSS - Beam Graphs - Beam 13
Mz(kip-ft)
Fy(kip)
Fx(kip)
Load 1
Post Mode
Load 1 : LOAD CASE 1
Input Units: kip-ft


stagtruss HSS - Whole Structure
stagtruss HSS - Beam Graphs - Beam 14
Mz(kip-ft)
Fy(kip)
Fx(kip)
Load 1
Post Mode
Load 1 : LOAD CASE 1
Input Units: kip-ft

TECH 2	STAGGERED TRUSS	JORDAN RUTHERFORD

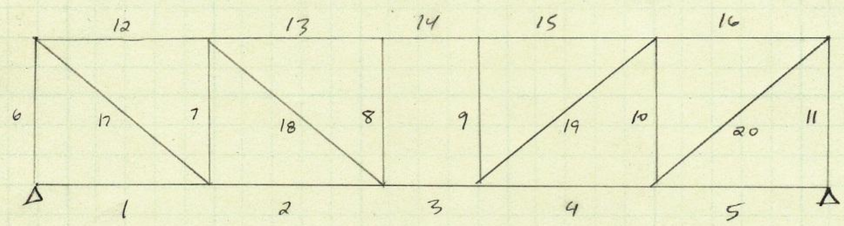


DUE TO EXISTING LAYOUT OF BUILDING WITH MASONRY BEARING WALLS, PRECAST HOLLOWCORE PLANK CAN BE USED TO SPAN BETWEEN TRUSSES.

LOADS ON TOP AND BOTTOM CHORD:

DEAD: 56 psf
SDL: 12 psf
LIVE: 40 psf
PARTITION: 20 psf

$w_u = 1.2[(56+12)(14')] + 1.6[(40+20)(14')] = 2.486 \text{ klf} \times 2 \Rightarrow \boxed{4.97 \text{ klf}}$



AXIAL FORCES

6/11	= 278 k	<	332 k	HSS 6 x 6 x 1/2
7/10	= 174 k	<	195 k	HSS 6 x 6 x 3/8
8/9	= 49 k	<	81.8 k	HSS 6 x 6 x 1/8
17/20	= -419 k	<	439 K	HSS 8 x 8 x 1/2
18/19	= -162 k	<	171 k	HSS 6 x 6 x 1/4
12/16	= 335 k	<	369 k	W12 x 45
13/15	= 466 k	<	526 k	W12 x 53
14	= 466 k	<	499 k	W12 x 45

Appendix F: Systems Comparison

Cost Data from RSS Means *Construction Costs 2012*											
	Items	**Unit**	**Quantity**	**Material**	**Labor**	**Equipment**	**Total**	**Total with Overhead and Profit**	**Total with Location Factor (1.015)**	**Cost Per Square Foot ($)**	
Existing	**Precast Hollowcore Planks, 8"**	S.F.	1	7.15	1.09	0.57	8.81	10.35	10.5	10.5	17.4
	C.M.U., NW 8"x16", 2000 psi, 8" thick	S.F.	1	2.49	4.07	-	6.56	8.9	6.8	6.8	
	Reinf. Walls, #3-#7	Ton	0.0704	930	395	-	1325	1650	1674.8	0.2	
Composite Steel	**W12x26**	L.F.	108	36	3.07	1.7	40.77	46.5	47.2	6.7	17.8
	W18x35	L.F.	56	48	4.07	1.69	53.76	62	62.9	4.7	
	Steel Deck, 3" 22 Gauge	S.F.	1	1.8	0.5	0.4	2.34	2.91	3.0	3.0	
	C.I.P. Concrete, 4000 psi	C.Y.	12.833	103	-	-	103	113	114.7	1.9	
	3/4" dia Shear Stud	Ea	76.000		-	-				0.5	
	Sprayed mineral fiber f.p., beams, 2 hr rating	S.F.	N/A		-	-	-	1	1.0	1.0	
One Way Slab/Beam	**C.I.P. Concrete, 4000 psi**	C.Y.	19.4167	103	-	-	103	113	114.7	2.9	20.8
	C.I.P. Beams, pumped	C.Y.	7.611	-	40	12.85	52.85	75.5	76.6	0.8	
	Slabs 6"-10"	C.Y.	11.806	-	15.1	4.82	19.92	28.5	28.9	0.5	
	A615, Grade 60, #3-#7 Bars	Ton	1.369	980	980	-	1960	2650	2689.8	4.9	
	Reinf. Elevated Slabs, #4-$7	Ton	0.355	1050	540	-	1590	2025	2055.4	1.0	
	Formwork, Flat Plate, Job Built plywood, up to 15', 4 use	S.F.C.A.	637.5	1.01	3.58	-	4.59	6.62	6.7	5.7	
	Formwork, Interior Beam/Grider, Job Built plywood, 24" 4 use	S.F.C.A.	382.185	0.65	5.06	-	5.71	8.56	8.7	4.4	
Stag. Truss	**Precast Hollowcore Planks, 8"**	S.F.	1	7.15	1.09	0.57	18.81	10.35	10.5	10.5	21.2
	W12x53	L.F.	56	69	3.6	2	74.6	84	85.3	6.3	
	HSS 6x6x0.25, 12' Long	Ea	6	320.28	60.55	30.83	411.66	486.88	494.2	3.9	
	Sprayed mineral fiber f.p., beams, 2 hr rating	S.F.	N/A		-	-	-	1	0.5	0.5	

Cost data obtained from RS MEANS *CONSTRUCTION COSTS* 2012. Margin of error is +/- 15%.
All total prices are calculated as a total and divided by the area to determine the cost per square foot.
An extra 0.75 was added to concrete for finishing.
Shear studs and fireproofing were user estimated values.
For lack of a better method, the truss was split into individual parts that would be located in the typical bay selected.

TECH 2 | SYSTEM DEPTH | JORDAN RUTHERFORD | 1

- COMPOSITE STEEL

  SLAB/DECK: 5.5"
  BEAM: 12.25"
  GIRDER: 16"

  TOTAL: $\boxed{17.75"}$

- ONE WAY

  SLAB: 6"
  (INC SLAB) BEAM: 16"

  TOTAL: $\boxed{16"}$

- STAGGERED TRUSS

  PLANKS: 8"

  BOT FLANGE: 12.1"

  TOTAL: $\boxed{8"}$

- PRECAST PLANK

  PLANKS: 8"

  TOTAL: $\boxed{8"}$

TECH 2 | SYSTEM WEIGHTS | JORDAN RUTHERFORD | 1

- COMPOSITE STEEL

DECK : 51 psf
BEAM : 22 plf / 9.33' = 2.36 psf
GIRDER : 40 plf / 28' = 1.42 psf

TOTAL : $\boxed{54.78 \text{ psf}}$

- ONE WAY

SLAB : $\left(\frac{6''}{12}\right)(150 \text{ pcf}) = 75 \text{ psf}$

BEAM : $\left(\frac{16'' \times 12''}{12 \times 12}\right)(150 \text{ pcf}) / 14' = 14.3 \text{ psf}$

GIRDER : $\left(\frac{10'' \times 12''}{12 \times 12}\right)(150 \text{ pcf}) / 27' = 7.4 \text{ psf}$

TOTAL : $\boxed{96.7 \text{ psf}}$

- STAGGERED TRUSS

PLANK : 56 psf
TOPPING : 6 psf

TRUSS : T + B : 53 plf (62')(2) = 6572

HSS : $35 \text{ plf} \left[(10')(6) + (16.64')(4)\right] = 4430$

$\frac{11002}{(28')(62')} = 6.34 \text{ psf}$

TOTAL : $\boxed{68.34 \text{ psf}}$

- PRECAST PLANK

PLANK : 56 psf
TOPPING : 6 psf

MASONRY : 47 psf (10')(27') = 12690 lbs

$\frac{12690 \text{ lbs}}{(27')(28')} = 16.8 \text{ psf}$

TOTAL : $\boxed{78.8 \text{ psf}}$

TECH 2	COST ANALYSIS	JORDAN RUTHERFORD	1

COST DATA FROM RSS MEANS 2012

LOCATION FACTOR: 1.015 (WEIGHTED AVERAGE) (pg 742)

• PRECAST HOLLOWCORE PLANKS 03 41 13.5

		UNIT	MAT	LABOR	EQP	TOTAL w/o	TOTAL w/FEES
0100	HOLLOW, 8" THICK	S.F.	7.15	1.09	.57	8.81	10.75

• PRECAST STRUCTURAL PRETENSIONED CONCRETE 03 41 33.6

2450	32"×10', 60' SPAN	E.A	4725	249	131	5105	5775

• CAST IN PLACE CONCRETE 03 31 05

.35	0300	NW CONC. READY MIX, 4000 psi	CY	103			103	113
.7	0050	BEAMS, PUMPED	CY		40	12.85	52.85	75.5
.7	1500	6"-10" THICK	CY		15.1	4.82	19.92	28.5

• CONCRETE FINISHING 3 35 29.3

0250 .75

• CONCRETE MASONRY UNIT 4 22 10.16

0350	NW, 8"×16", C90 2000 psi, 8" THICK	S.F.	2.49	4.07	6.56	8.9

• REINFORCEMENT BARS (UNCOATED) 3 21 10.6

0100	A615, GRADE 60 #3 - #7	TON	980	980	1960	2650
0700	WALLS #3 - #7	TON	930	395	1325	1650
0400	ELEVATED SLABS #4 - #7	TON	1050	540	1590	2025

• STEEL DECKING 05 31 13.5

5700	3" 22 GAUGE	S.F.	1.8	0.5	0.4	2.34	2.91