Hyatt Place North Shore
Pittsburgh, PA

Technical Assignment #2

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Executive Summary:

In this report, the existing floor system of the Hyatt Place North Shore was analyzed, along with 3 alternatives that could provide benefit to the overall design and success of the building. The purpose was to explore floor systems and how they affect MEP systems, architecture, the rest of the structure, overall cost, and the functional use of the building. Based upon these issues, the following 3 alternatives were proposed:

1. Precast Concrete Planks on Steel Frame
2. Composite Steel Frame
3. Concrete 2-Way Flat Plate

Through analysis the first system to be crossed off the list was the composite steel frame due to the depth of the system, overall cost, and functional need of the building. The remaining systems all could serve as good options for a hotel structure. The issue lies in the site conditions along the Allegheny River. Soft soils make overall weight an important issue, so the precast plank on masonry walls and the concrete 2-way flat plate systems were crossed off due to the weight of the structure that supports the floor system.

The precast plank on steel frame provides the best of both weight and efficiency. Through the use of the girder-slab system, the precast planks can be tucked up into the space taken by the girder that supports it. This is done using a beam with a wide bottom flange for the plank to bear on and a narrow top flange. The system is also easily constructed and has a reasonable price tag.

Introduction:

The Hyatt Place Hotel is part of an agreement between the Pittsburgh Steelers and Pirates that began back in 2003 with the goal to bring commercial development to the North Shore. The 108,000 SF, 178 room hotel is conveniently close to both of the teams' stadiums, Rivers Casino, and Pittsburgh in general.

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

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

Figure 2: South Elevation

Exterior elevations are mainly comprised of brick veneer cavity wall system with rigid insulation and structural CMU backup along with cast stone window headers, some strips of aluminum, metal plates, caste stone, and polished block in a way to complement the modern look of the interior. The parapet wall also varies in height from 3 feet to 9 feet creating interesting snow and wind loadings on the roof that will be examined in the Building Load Summary section of the report on page 13. The roof is a typical TPO membrane roof system.

Structural System Overview

The Hyatt Place North Shore is a 7 story reinforced concrete masonry bearing structure located on soft soils along the Allegheny River that utilizes precast concrete planks for ease of construction and headroom. Steel beams are used to create an open space on the ground floor for a large meeting room and in other various places where the layout makes it impossible for the concrete planks to rest on the typical masonry bearing walls, shown in *Figure 3.* The reinforced concrete masonry bearing walls also serve as the lateral force resisting system with the aid of the precast concrete planks acting as a rigid diaphragm.

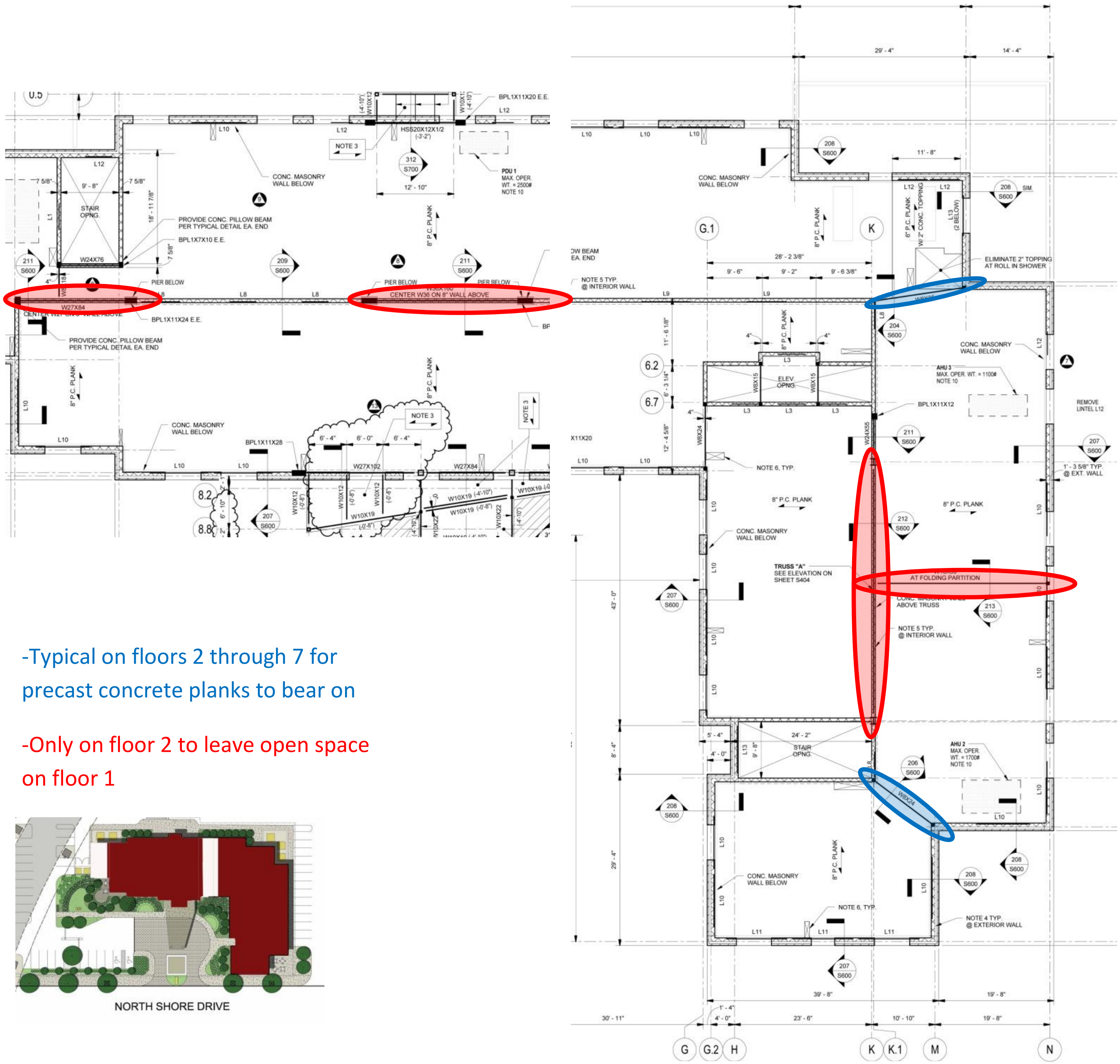


Figure 3: View of steel beams used

Foundation:

The Hyatt Place North Shore has a 15,500 SF footprint located on soil along the Allegheny River that has a maximum allowable bearing capacity of 1,500 psf. Spread footings have been provided for the front canopy, 5'-0"x5'-0"x1'-0" concrete spread footing with a maximum load of 25 kips, and site wall foundations only. There are 121 – 18" diameter end bearing 140 ton auger-cast piles that have a minimum depth of 1'-0" into bedrock to support the building. They have a 285 kip vertical capacity and a 16 kip lateral capacity. Piles are typically expected to be 70 feet deep, but this varies per pile. As shown in *Figure 4,* pile caps are 4'-0" thick. There are 2 to 4 piles supporting each pile cap. All concrete used for shallow foundations and piers have a strength of 3000 psi and the concrete for grade beams, pile caps, and slabs on grade are 4000 psi. The first floor is a 4" concrete slab on grade with W/ 6x6-W1.4xW1.4 welded wire fabric.

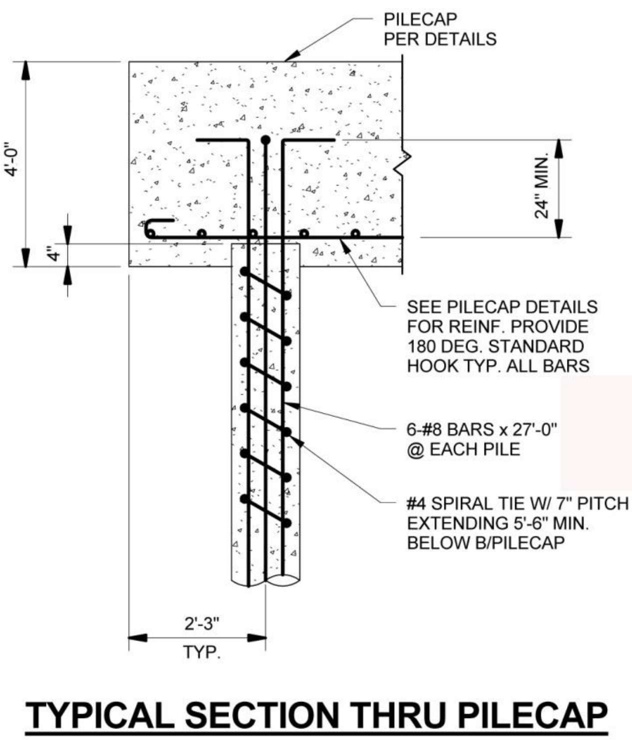


Figure 4: Section through typical pile cap

Gravity System

Walls:

Nearly all of the walls in the Hyatt Place North Shore are reinforced concrete masonry walls that resist gravity and lateral loads. The only exceptions are partition walls between the hotel rooms and other random walls not along the perimeter of the building. The walls vary in thickness and spacing of grout and reinforcing, *Table 1* shows the wall types and location. The compressive strength of the CMU units is 2800 psi and the bricks are 2500 psi, both normal weight. The grout used has a compressive strength of 3000 psi and the steel reinforcement is sized and placed as stated in *Table 1*. *Figure 5* shows the orientation of the walls on a typical upper level plan, the capacity of each of these wall types can be determined. *Table 2 & 3* along with *Figure 6* show the typical lintel in a masonry bearing wall.

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

Table 1: Reinforced concrete masonry bearing wall schedule

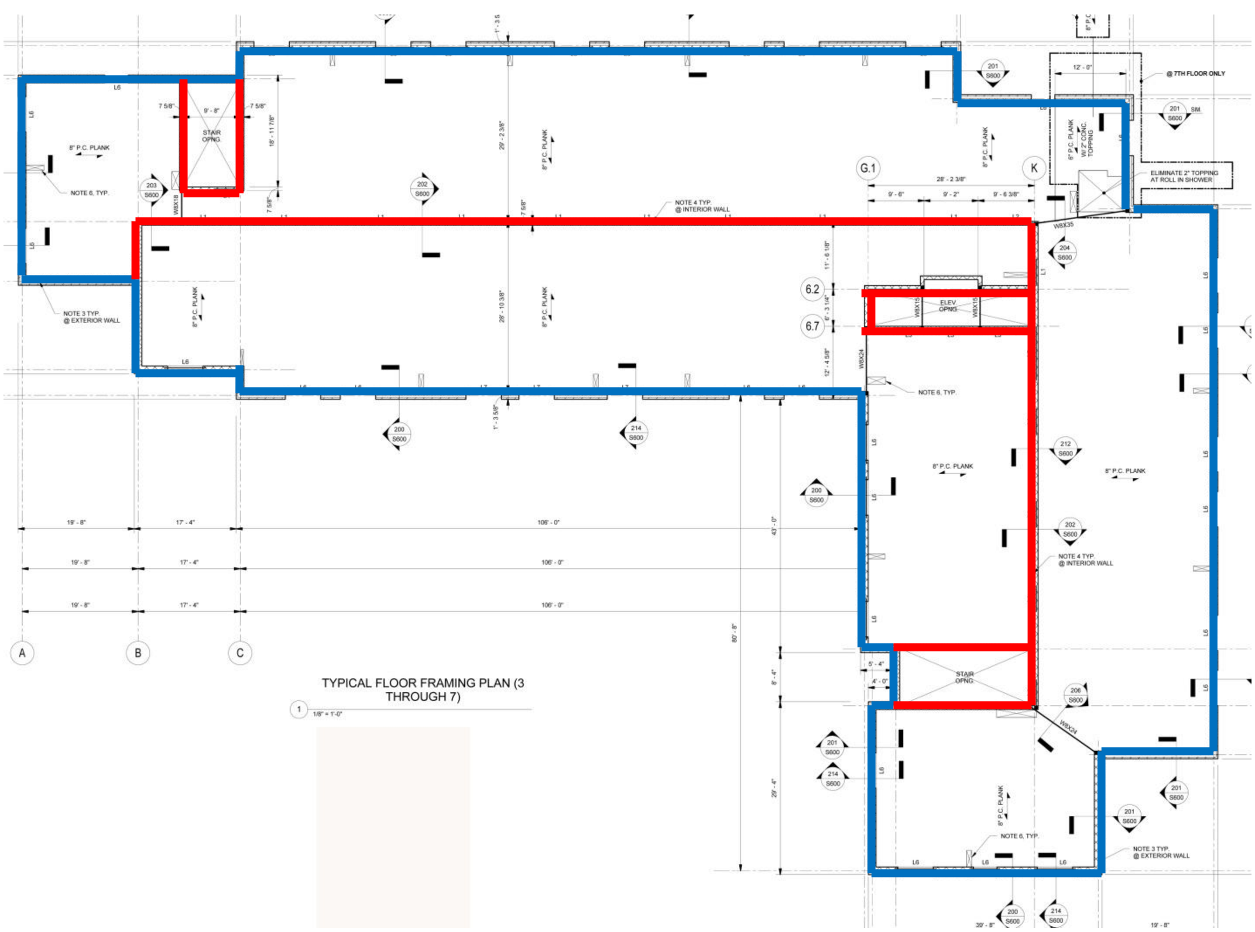


Figure 5: Typical load bearing wall layout, floors 3 through 7

PRECAST LINTEL SCHEDULE F0R LOAD BEARING MASONRY WALLS						
			LOADING LBS/FT			
MARK	SIZE	MAX. M.O.	LIVE	DEAD	REMARKS	MARK
L1	8"	3'-4"	2000	1800	SEE "TYP. LINTEL DETAIL 1"	L1
L2	8'	6'-4"	2000	1800	SEE "TYP. LINTEL DETAIL 1"	L2
L3	10" VERIFY W/ELEV. MFR.	3'-6"	500	500	SEE "TYP. LINTEL DETAIL 1"	L3
L4	8"	6'-0"	1400	400	SEE "TYP. LINTEL DETAIL 2"	L4
L5	8"	6'-0"	1400	400	SEE "TYP. LINTEL DETAIL 4"	L5
L6	8"	6'-0"	1000	1000	SEE "TYP. LINTEL DETAIL 2"	L6
L7	8"	6'-0"	1000	1000	SEE "TYP. LINTEL DETAIL 4"	L7
L8	8"	6'-0"	1000	1000	SEE "TYP. LINTEL DETAIL 1"	L8
L9	8"	3'-4"	1000	1000	SEE "TYP. LINTEL DETAIL 1"	L9
L10	16"	6'-4"	2100	1000	SEE "TYP. LINTEL DETAIL 3"	L10
L11	16"	9'-4"	2100	1000	SEE "TYP. LINTEL DETAIL 3"	L11
L12	8"	5'-0"	1500	1000	SEE "TYP. LINTEL DETAIL 2"	L12
L13	16"	7'-0"	2600	1000	SEE "TYP. LINTEL DETAIL 2"	L13

PRECAST LINTEL FOR LOAD BEARING MASONRY WALLS NOTES:

1. MASONRY OPENINGS SHOWN IN SCHEDULE ARE MAXIMUM ALLOWED FOR LINTEL. SEE ARCH. DWGS. FOR ACTUAL MASONRY OPENINGS DIMENSIONS.
2. PROVIDE MIN. 8" BEARING ON BRICK OR SOLID CONC. BLOCK.
3. PRECAST LINTEL MFR. TO DESIGN PRECAST LINTELS FOR LOADS SHOWN IN SCHEDULE. SEE GENERAL NOTES FOR ADD'L INFO. LOADS ARE UNFACTORED.
4. SEE BRICK SUPPORT LINTEL SCHEDULE FOR ANGLE SIZE NEEDED FOR MASONRY OPENING.
5. LINTEL MUST BE DESIGNED FOR A MAXIMUM TOTAL LOAD DEFLECTION LESS THAN 0.3" OR SPAN/600.

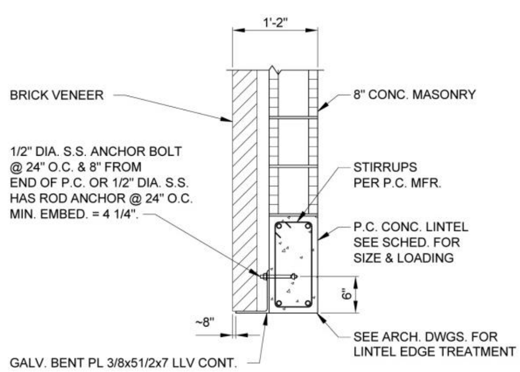
Table 2: Precast Lintel schedule for load bearing masonry walls

BRICK LINTEL SCHEDULE			
WALL THICKNESS	MASONRY OPNG. UP TO 4'-0"	MASONRY OPNG. 4'-0"+ TO 6'-0"	MASONRY OPNG. 6'-0"+ TO 8'-0"
4" WALL	BENT PL5/16x5 1/2x3 1/2 LLH	BENT PL5/16x5 1/2x4 LLH	BENT PL5/16x5 1/2x5 1/2

NOTES:

1. PROVIDE MINIMUM 6" BEARING ON BRICK.

Table 3: Brick lintel schedule



TYPICAL LINTEL DETAIL "3"

Figure 6: Typical lintel detail

Columns:

With the masonry structure, the only 2 columns in the building are W12x136s located on the first floor and are used to transfer the load in the large transfer girder down to the foundation, *Figure 7.* There are also concrete masonry piers also on the first floor that support transfer beams in the lobby space and make it possible to have more window space on the first floor.

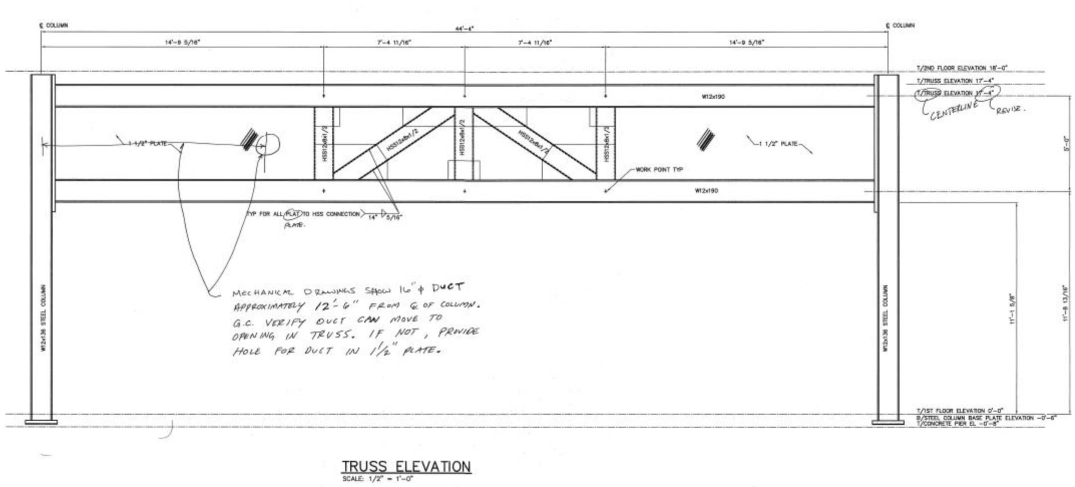


Figure 7: Transfer girder in first floor meeting space

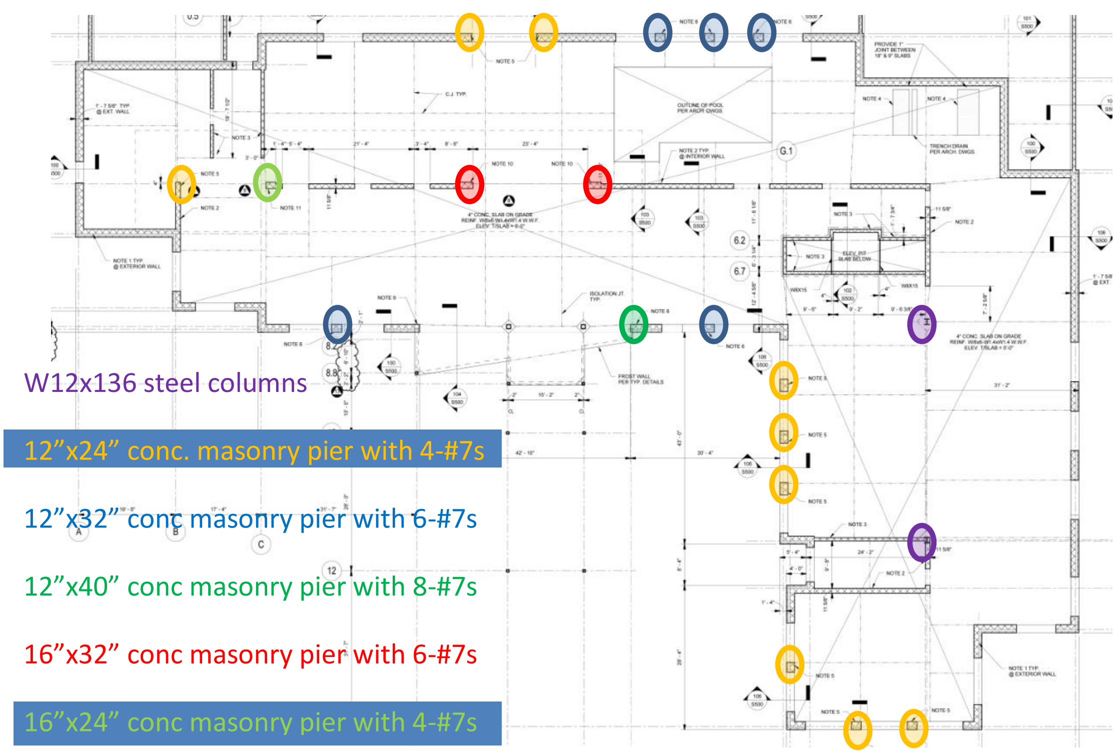


Figure 8: Location of masonry piers on first floor

Floors:

The Hyatt Place North Shore floor system is 8" thick untopped precast concrete planks. This system simplifies design and expedites construction. The system efficiently carries the loading over relatively long spans ranging from 27'-6" to 30'-6". The concrete compressive strength of the floors is f'c=5000 psi. Extra strength is also added by prestressing the units. *Figure 9* shows a typical connection with masonry bearing walls.

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

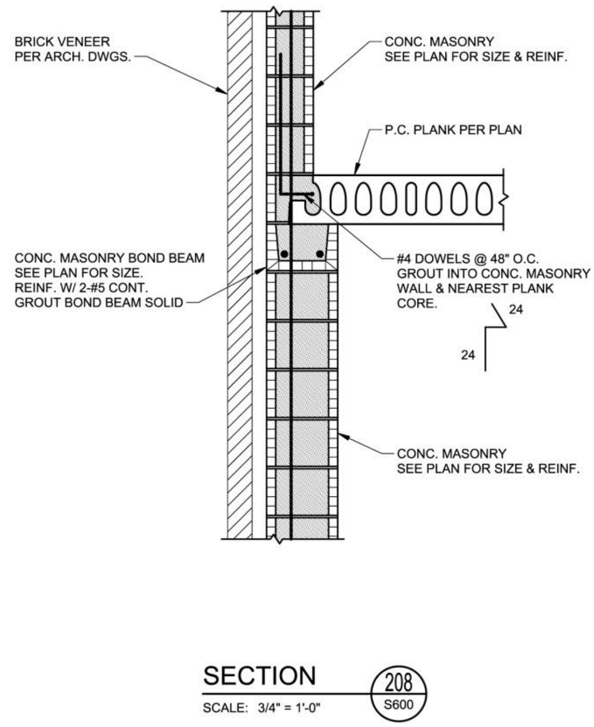


Figure 9: Typical plank and masonry wall connection

As previously stated on page 4 and denoted in *Figure 3*, steel beams are used in places where there is an opening in the interior bearing wall on the first floor and on all floors as needed for the planks to bear on. The members used are W8x18, W8x24, W8X35, W36x160, and W27x84. The large steel truss spanning 44'-4" over the meeting rooms 2 – W12x190s that are spaced 5' apart with HSS members and 1 ½" steel plate webbing.

Lateral System

The lateral system for the structure is simply the gravity system. The reinforced masonry bearing walls depicted in *Figures 5 & 6* on page 7 act as shear walls and the precast concrete planks act as a semi-rigid diaphragm compared to cast-in-place concrete floor. The existing system only has a leveling material added, for planks to be considered fully rigid there must be a 2" structural concrete topping. The loads travel into the diaphragm and then into the bearing walls and down to the foundation and the auger piles that are capable of resisting 16 kips of lateral force per pile.

Codes and Design Standards

Codes:

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

- The International Building Code 2006 – Amendments City of Pittsburgh
- The Building Code Requirements for Structural Concrete (ACI 318-05), American Concrete Institute
- PCI MNL 120 "PCI Design Handbook – Precast and Prestressed Concrete"
- The Building Code Requirements for Masonry Structures (ACI 530), American Concrete Institute
- Specifications for Masonry Structures (ACI 530.1), American Concrete Institute
- Specifications for Structural Steel Buildings (ANSI/AISC 360-150), American Institute of Steel Construction
- Minimum Design Loads for Buildings and Other Structures (ASCE 7-05), American Society of Civil Engieers
- RS Means Assemblies Cost Data
- RS Means Facilities Construction Data
- Live load deflection criteria used: L/360
- Total load deflection criteria used: L/240

Materials

Concrete:

Shallow Foundations and Piers	3000 psi
Grade Beams and Pile Caps	4000 psi
Slabs on Grade	4000 psi
Precast Concrete Planks	5000 psi

Rebar:

Deformed Bars Grade 60	ASTM A615
Welded Wire Fabric	ASTM A185

Masonry:

Concrete Masonry Units	2800 psi
Bricks	2500 psi
Grout	3000 psi

Structural Steel:

W Shapes	ASTM A992,	Fy = 50 ksi	Fu = 65 ksi
Channels	ASTM A572 Grade 50	Fy = 50 ksi	Fu = 65 ksi
Tubes (HSS Shapes)	ASTM 500 Grade B	Fy = 46 ksi	Fu = 58 ksi
Pipe (Round HSS)	ASTM 500 Grade B	Fy = 46 ksi	Fu = 58 ksi
Angles and Plates	ASTM A36	Fy = 36 ksi	Fu = 58 ksi

Gravity Loads

Load conditions determined from ASCE 7-05

Dead Loads:

Reinforced Concrete	150 pcf
Steel	490 pcf
Reinforced Masonry Walls	*Figure 5*
MEP	10 psf
Partitions	15 psf
Miscellaneous	5 psf
Roof	20 psf

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						Weight (psf)		
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Table 1: Reinforced concrete masonry bearing wall schedule

Live Loads:

Floor Live Loads		
Area	Design Load (psf)	ASCE 7-05 Load (psf)
Public Areas	100	100
Lobbies	100	100
Public Corridors	100	100
Room Corridors	60	40
Hotel Rooms	60	40
Stairs	100	100
Mechanical*	150	125
Fitness Room	100	100
*on grade		

Table 4: Floor live loads

Snow Load:

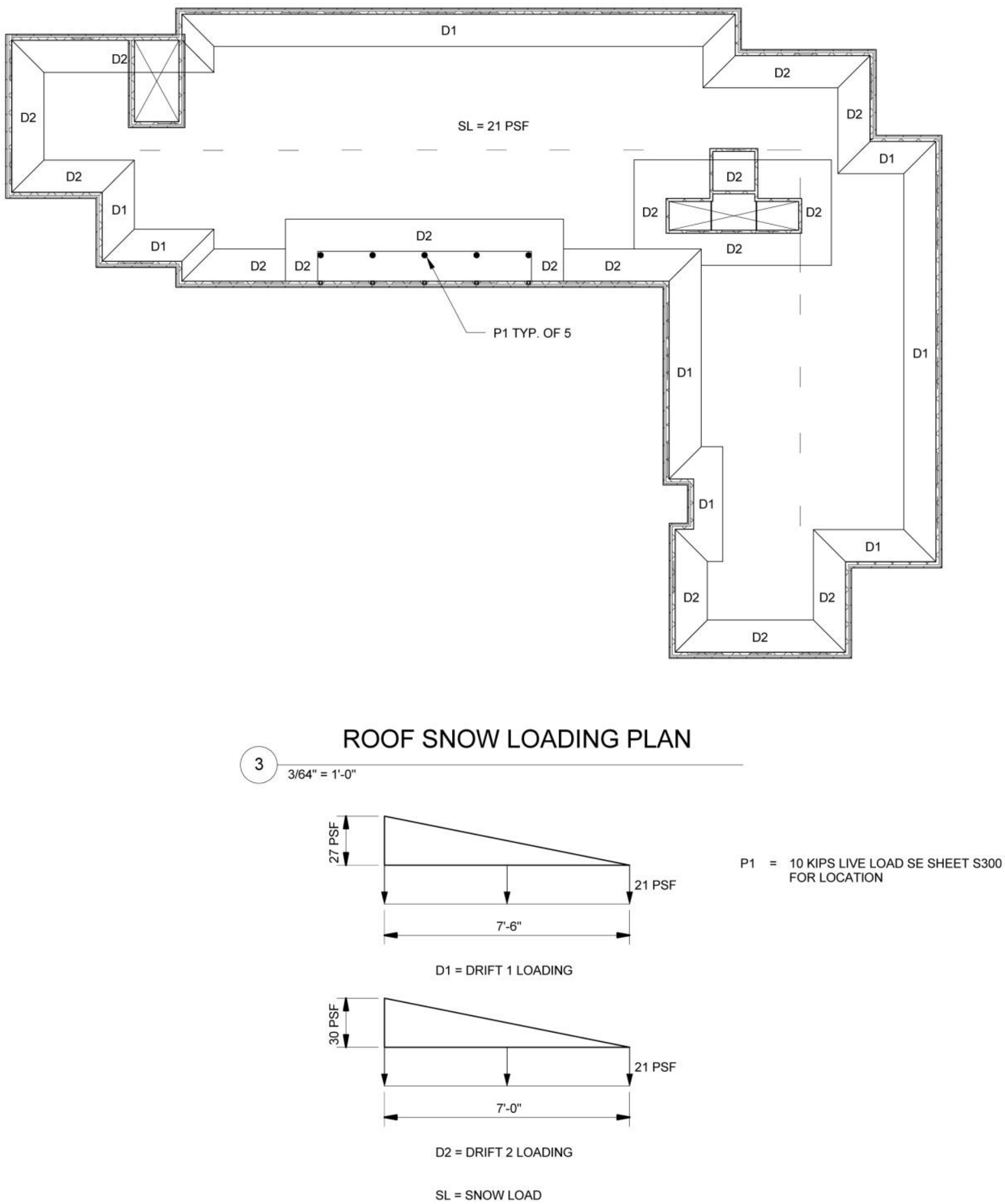


Figure 10: Roof snow loading plan as calculated by AES

Flat Roof Snow Load:

Determined using ASCE 7-05

Flat Roof Snow Load				
		AES	ASCE 7-05	
Ground Snow Load	P_g=	30	25	psf
Snow Exposure Factor	C_e=	1.0		
Snow Load Importance Factor	I_s=	1.0		
Thermal Factor	C_t=	1.0		
Flat Roof Snow Load	P_f=	21	17.5	psf

Table 5: Calculation of flat roof snow load

The roof system uses the same 8" precast concrete planks as the lower levels of the structure, therefore the roof is significantly overdesigned and can handle a much greater snow load than the tabulated value.

Drift Calculation:

Calculation of drift depth from *figure 16*

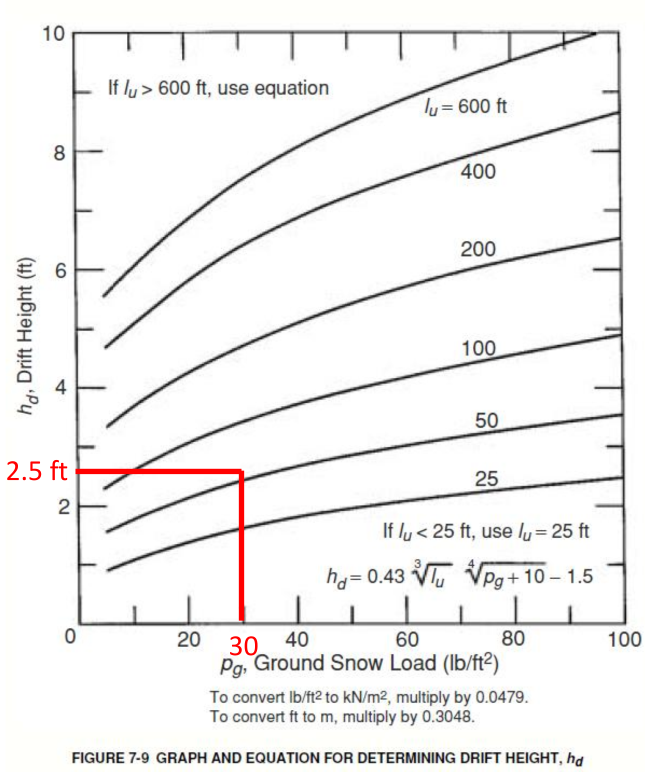


Figure 11: Graph and equation for determining drift height

Snow Density = .13(P_g) + 14

= .13(25) + 14 = **17.25 lb/ft^3**

Balanced Height= P_g/Snow Density = 25/17.25 = **1.4 ft**

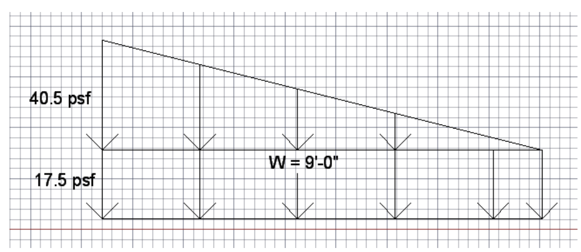
<u>Typical Parapet Wall Drift Height</u>

Drift Height = 2.5ft – from *Figure 16*

Max allowable =.75 h_d = .75*2.5 = **2.25ft**

Drift Weight = 2.25ft * 18 lb/ft^3 = **40.5 psf**

Drift Width = 4*h_d = 4*2.25 = **9 ft**



Floor Systems

Figure 12 shows the area that different floor systems will be analyzed over. The size of the bay depends on the type of system used.

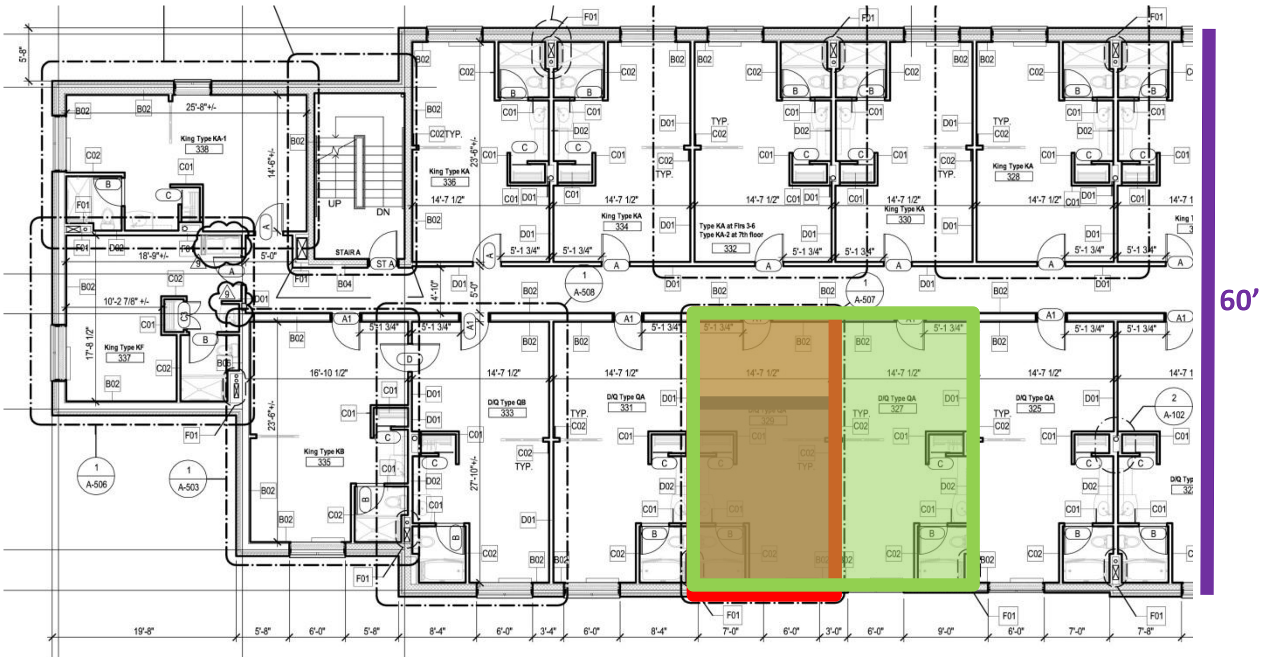
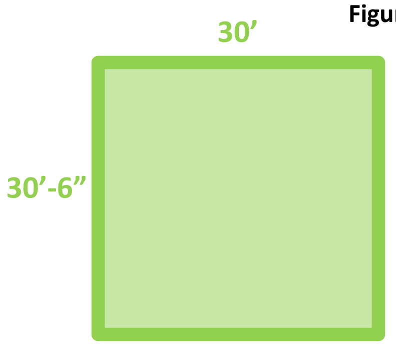
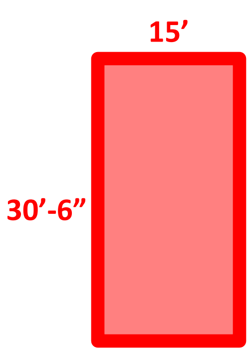


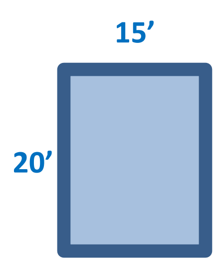
Figure 12: Graph and equation for determining drift height



- **Existing: precast plank on bearing wall system**
- **Alternative #2: composite steel frame**



- **Alternative #1: composite steel and precast plank**



- **Alternative #3: concrete two way flat plate**

Existing: Precast Concrete Plank on Masonry Walls

The Hyatt Place North Shore existing floor system is 8" thick untopped precast concrete planks. This system simplifies design and expedites construction. The system efficiently carries the loading over relatively long spans ranging from 27'-6" to 30'-6". The concrete compressive strength of the floors is f'c=5000 psi. Extra strength is also added by prestressing the units. *Figure 12* shows a typical connection with masonry bearing walls.

The typical load of 83 psf is less than the allowable value of 87.75 psf for a 30'-6" span found by interpolating the table provided by the manufacturer, *Table 6*.

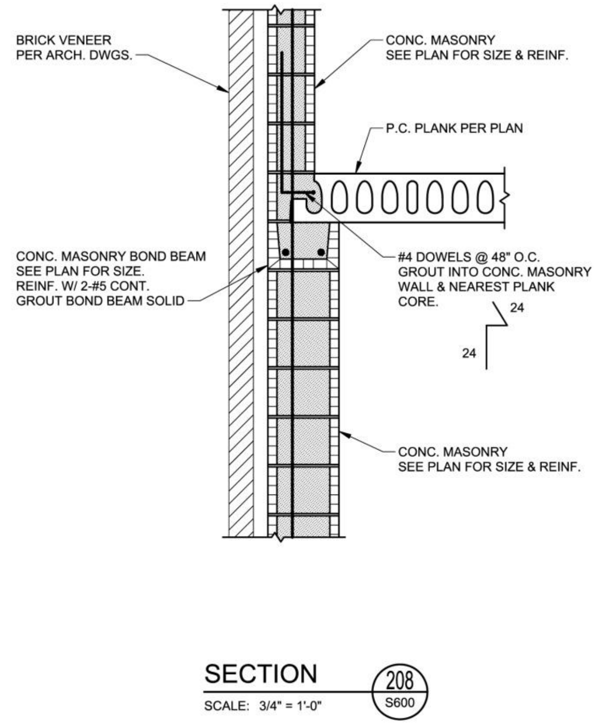


Figure 12: Typical plank and masonry wall connection

Summary

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

Loading: Dead Load (self weight) = 63 psf

Leveling Toping = 13 psf

Superimposed = 30 psf

Live Load = 40 psf

Total System Weight: Slab = 76 psf
Masonry Bearing walls = 47 psf
Total = 123 psf

Thickness: 9" (from leveling toping to bottom of plank)

Cost: 20.7 $ per SF

Advantages

The existing precast concrete plank system on masonry bearing walls has many advantages. It provides for quick construction in the field since the concrete doesn't need time to cure and doesn't need spray on fireproofing to achieve the desired 2 hour rating. Also the system provides a flat ceiling that only needs an architectural coating to have a finished product, this is also desirable in hotel construction. Structurally, the system is able to span long distances with a low floor to floor height and weight, due to pretensioning.

Disadvantages

There aren't many disadvantages to the precast concrete plank system when spanning long distances with moderate loads. Construction can sometimes be difficult due to the fact that the planks don't always have the exact same camber, making it difficult to line up the edges and properly connect them. But the main disadvantage is of the overall structural system, because the masonry bearing walls make the structure heavy. Due to this fact, the next system explored is precast concrete plank on steel frame.

PITTSBURGH FLEXICORE CO., INC.
8" x 48" Spiroll Corefloor Load Table

8" x 48" Hollowcore (Untopped)
CLEAR SPAN IN FEET

Designation	14'	16'	18'	20'	22'	24'	26'	28'	30'	32'	34'	36'	38'
8S38-1.75	257	186	137	102	75	55	40	X	X	X	X	X	X
8S48-1.75	350	258	194	148	113	87	67	51	38	X	X	X	X
8S58-1.75	369	314	241	186	146	114	90	71	55	42	32	X	X
8S68-1.75	381	325	281	232	184	146	117	94	76	60	48	37	X
8S78-1.75	393	335	290	255	214	172	140	113	92	75	61	49	38

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

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

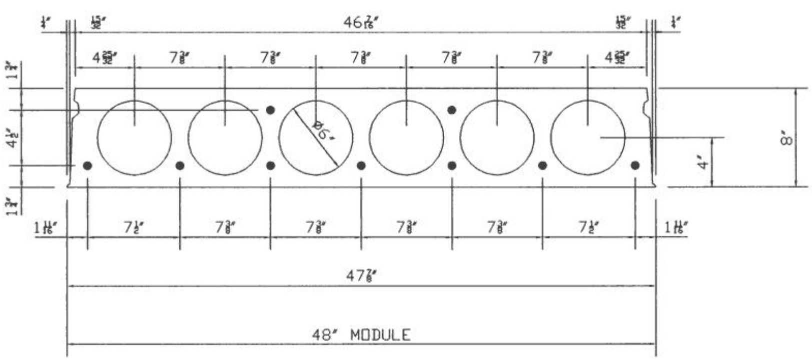


Table 6: Precast concrete plank design values

Alternative #1: Composite Steel and Precast System

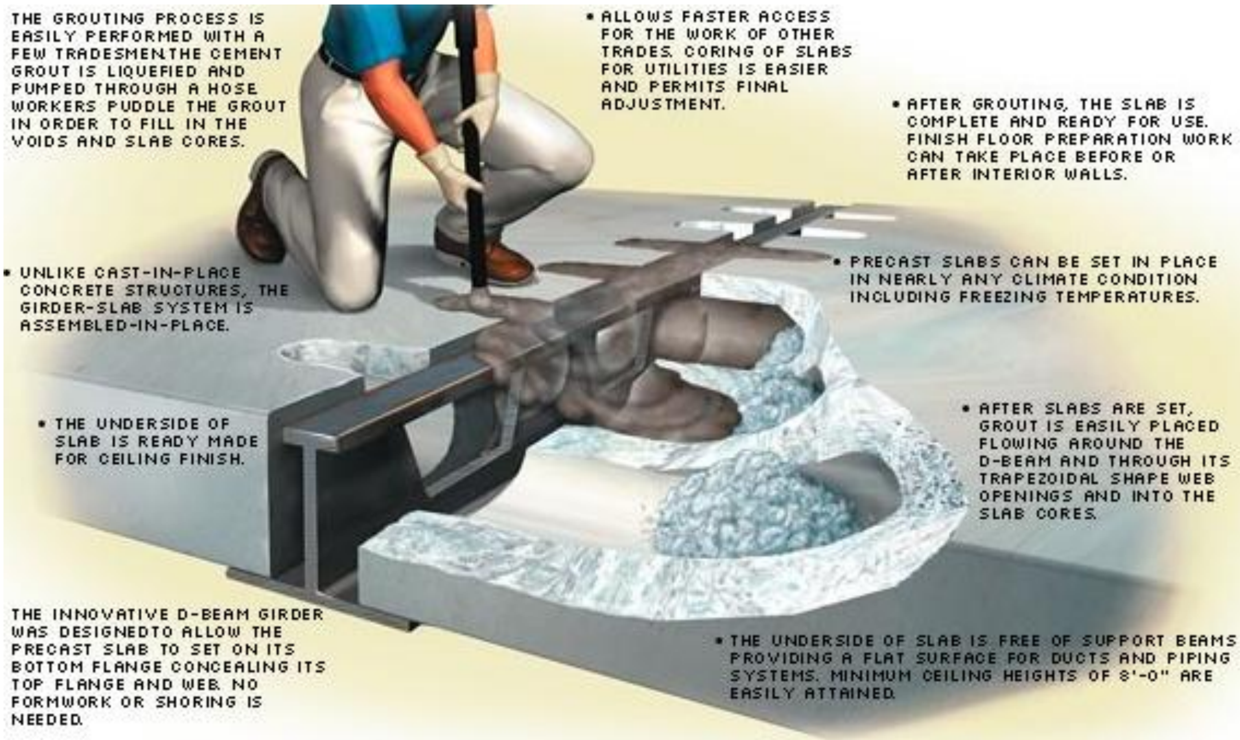


Figure 12: Composite steel and precast system

The first possible system analyzed was picked to try and improve upon the system as a whole. One weakness of the precast concrete plank on masonry bearing walls was that the bearing walls are very heavy and the building is located on soft soil. One possible way to reduce the overall weight of the structure is to redesign as a steel frame. But there are drawbacks to the steel frame, floor to floor height and fireproofing. In the past 10 years, Girder-Slab Technologies, LLC. have developed a composite steel and precast system that solves both of these issues, *Figure 12*. Their solution to floor thickness is to create a girder, as depicted in *Figure 13*, that has a wider bottom flange for precast concrete plank to bear on, the beam is known as an open-web dissymmetric beam or “D-Beam”. After the planks are placed and grout is filled in, the girder slab system develops composite action, thus enabling a smaller girder to carry more load.

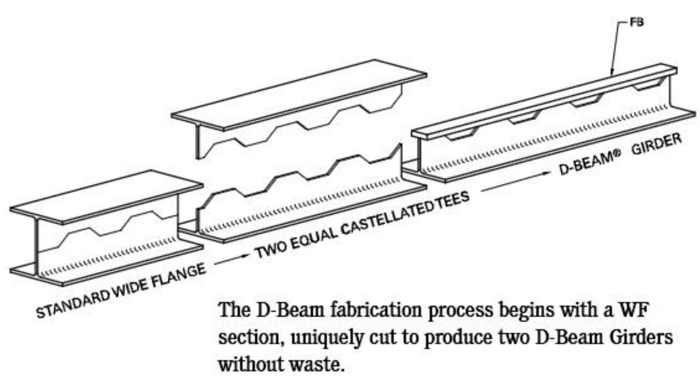


Figure 13: Composite steel and precast system

System Statistics: It was decided to use an 8" precast concrete plank with 2" topping for this system. Topping was used in order to make sure that the floor acts as a rigid diaphragm. The engineers from AES assumed the existing 8" precast concrete plank structure to be a rigid diaphragm due to how the planks are tied together along their edges with steel and grout, but it is more certain to act rigidly when there is a 2" concrete topping to tie the planks together. The same manufacturer was used as the existing system. *Table 6* shows that 106 psf can be supported by the system which is more than the 95 psf unreduced load on the planks. *Tables 7* & *8* show property tables that Girder-Slab Technologies, LLC. created for the D-Beam.

Summary

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

Grout: f'c = 4000 psi

Steel: DB 9x46 29000 ksi

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

Loading: Dead Load (self weight) = 63 psf

Structural Toping = 25 psf

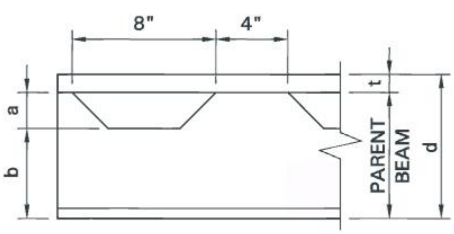
Superimposed = 30 psf

Live Load = 40 psf

Total System Weight: Slab = 88 psf
Steel Frame = 4.7 psf
Total system = 92.7 psf

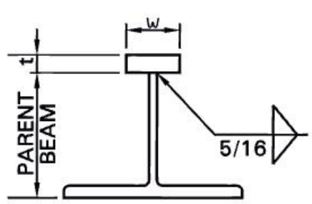
Cost: $20.8 per SF

Designation	Web Included		Depth	Web	Parent Beam			Top Bar
	Weight	Avg. Area	d	Thickness t_w	Size	a	b	w x t
	lb/ft	in²	in	in		in	in	in x in
DB 8 x 35	34.7	10.2	8	.340	W10 x 49	4	3	3 x 1
DB 8 x 37	36.7	10.8	8	.345	W12 x 53	2	5	3 x 1
DB 8 x 40	39.8	11.7	8	.340	W10 x 49	3	3.5	3 x 1.5
DB 8 x 42	41.8	12.3	8	.345	W12 x 53	1	5.5	3 x 1.5
DB 9 x 41	40.7	11.9	9.645	.375	W14 x 61	3.375	5.25	3 x 1
DB 9 x 46	45.8	13.4	9.645	.375	W14 x 61	2.375	5.75	3 x 1.5



D-Beam® Reference Calculator is Available on Website. www.girder-slab.com

Table 7: D-Beam dimension table



Designation	Steel Only / Web Ignored						Transformed Section / Web Ignored				
	Ix	C bot	C top	S bot	S top	Allowable Moment Fy=50 KSI f_b=0.6 Fy	Ix	C bot	C top	S bot	S top
	in⁴	in	in	in³	in³	kft	in⁴	in	in	in³	in³
DB 8 x 35	102	2.80	5.20	36.5	19.7	49	279	4.16	4.40	67.1	63.5
DB 8 x 37	103	2.76	5.24	37.3	19.7	49	282	4.16	4.42	67.7	63.8
DB 8 x 40	122	3.39	4.61	36.1	26.5	66	289	4.26	4.30	67.9	67.2
DB 8 x 42	123	3.35	4.65	36.9	26.5	66	291	4.26	4.32	68.4	67.5
DB 9 x 41	159	3.12	6.51	51.0	24.4	61	332	4.27	5.35	77.7	62.1
DB 9 x 46	195	3.84	5.79	50.8	33.7	84	356	4.43	5.20	80.6	68.6

Table 8: D-Beam property table

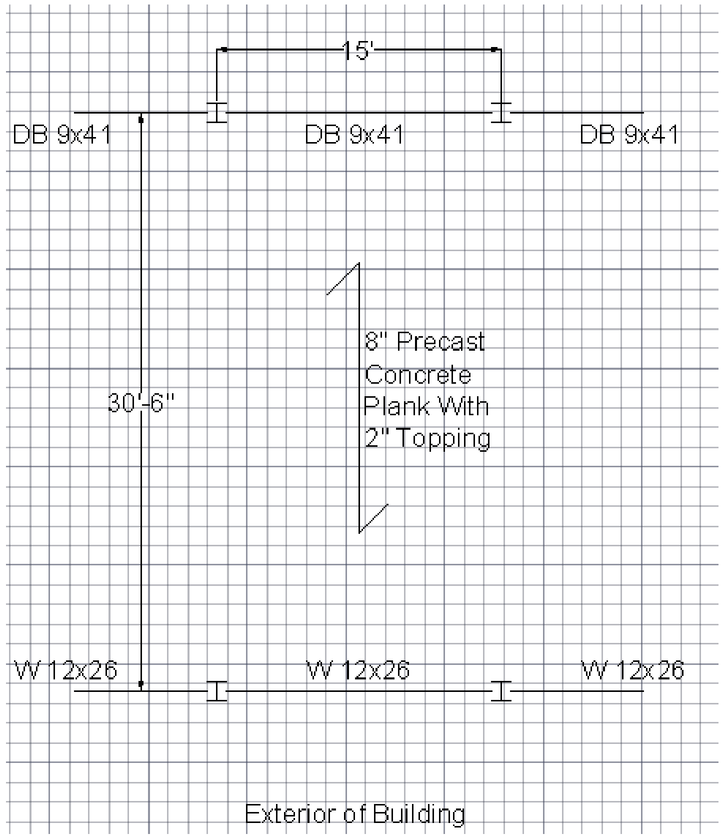


Figure 14: Typical layout

The columns were laid out to be where existing masonry bearing walls were, so architectural layout was not affected. In most cases they are located at the intersection of partition wall and bearing wall so as to have the least affect. Column location and design will be further explored later in the report.

Advantages

There are numorous good things about the composite steel and precast system, all of the same things that were good from the existing system, except with a lower overall structural weight. The slab sits on the bottom flange of the girder, keeping low floor to floor heights and a flat surface for a ceiling that is easy to finish coat. Building construction is simple and fast, limiting construction costs. The precast concrete planks meet the 2 hour fire rating desired.

Disadvantages

There isn't space between beams and girders to run HVAC components, but this isn't a problem with how HVAC is ran in most hotels. The steel and precast concrete planks need extra lead time because they must be fabricated off site and transported.

Alternative #2: Composite Steel Frame

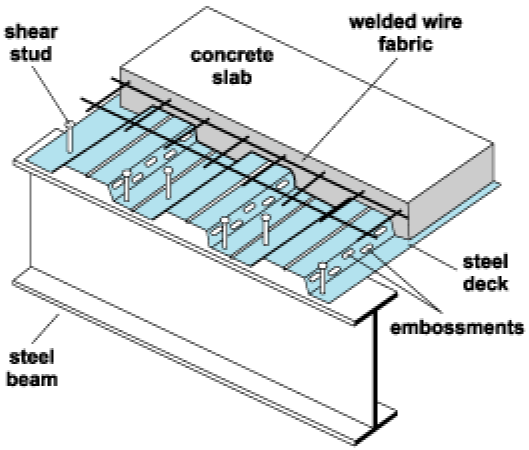


Figure 15: Composite floor and beam system

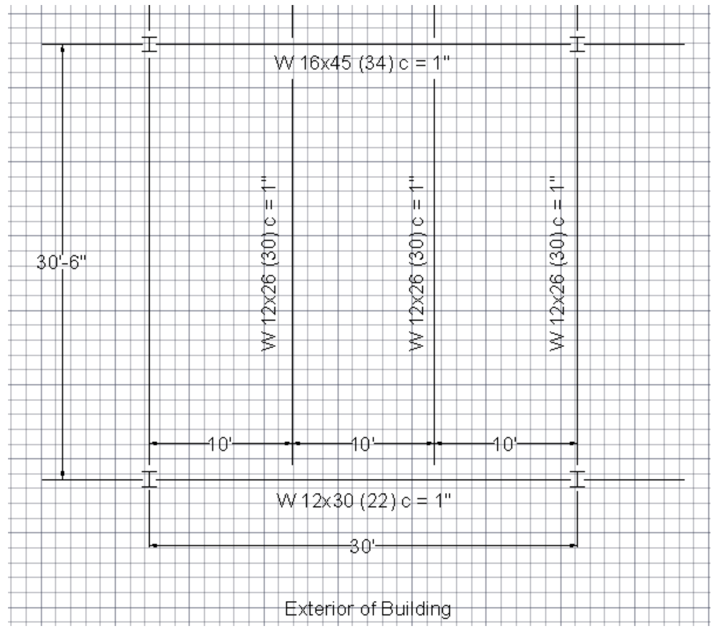


Figure 16: Typical bay

Summary

Materials: **Concrete:** 4.5" slab
normal weight (145 pcf)
f'c = 4000 psi

Steel Deck: 2VLI17 fy = 40 ksi
3 span – 10'-7"

Steel Reinforcement: fy = 60 ksi
6x6-W2.1xW2.1

Steel W members: fy = 50 ksi

Steel Studs: ¾" dia. 3.5" long

Loading: Dead Load (self weight) = 69 psf
Superimposed = 30 psf

Live Load = 40 psf

Total System Weight: Slab = 69 psf
Steel Frame = 7.6 psf
Total system = 76.6 psf

Thickness: 22.5" from bottom of girder to top of slab (would need hung ceiling also)

Cost: 24.9 $ per SF

The typical bay size for this system was determined to be 30'-6" by 30'-0" because the longest span is 30'-6" and each room is 15' wide so there will be columns where the bearing walls were at every other hotel room partition wall, thus architecture is little affected in plan view.

First step is to pick a composite deck from the Vulcraft catalog that meets the needed 10' unshored construction span. The composite slab transfers floor load to the nearest beam and over to nearest girder, then column and down to foundation. The beams and girders have ¾" diameter 3.5" long shear studs that are used to transfer the compression load be transferred into the concrete slab and thus allowing a lower beam size once composite action is in effect. But beams must be designed to carry wet concrete and construction loads without composite action. This ends up being a problem with this system due to the fact that the live load is small and a large portion of the load is from dead load. To compensate for this issue, beams and girders were all given a 1" camber. This solves the deflection issue without having to pick a much larger member than needed for composite action, but also adds in more cost to have the beams cambered. Sizing calculations can be found in Appendix C.

Advantages

The system has a low total weight, which is adventageous on the soft soils located under the building. There is plenty of space left in between beams for MEP systems, but in hotels this isn't as much of a concern as other building types. The system also leaves a large amount of open space.

Disadvantages

The first main disadvantage of this system is the thickness of the floor system and the need for a hung ceiling, both of which are not standards of hotel design. There are many systems that can be designed to take up less space and provide a flat easy to finish ceiling. Also, construction would be fairly labor entensive. Metal deck and welded wire fabric must be laid out and shear studs attached all before concrete is poured. Changes in the layout of MEP systems would also have to be considered based upon where the beams and columns are laid out.

Alternative #3: Two-Way Concrete Flat Plate System

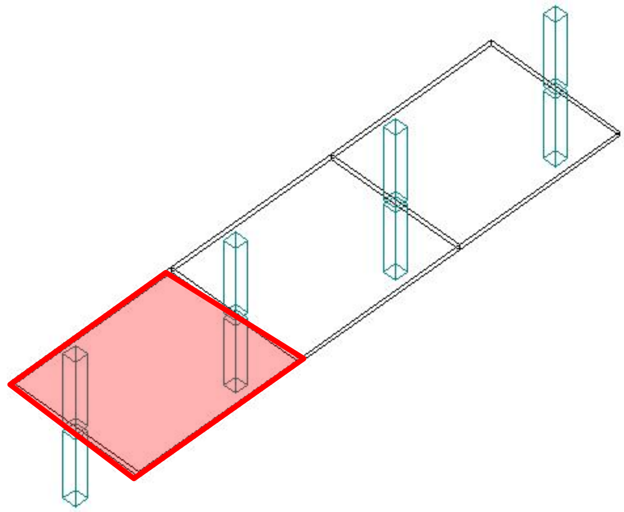


Figure 17: System overview

Summary

Materials: **Concrete:** 7.5" slab
normal weight (145 pcf)
f'c = 4000 psi

Steel Reinforcement: fy = 60 ksi

Loading: Dead Load (self weight) = 93.75 psf
Superimposed = 30 psf

Live Load = 40 psf

Total System Weight: Slab = 93.75 psf
Columns = 38.6 psf
Total system = 131.9 psf

Thickness: 7.5" from top of slab to bottom of slab

Cost: 13.36 $ per SF

The last system to be analyzed as an alternative is a two-way concrete flat plate system. This system was chosen to be investigated based on its thin flat slab that is ideal for easy hotel construction. As see in *Figure 17*, the bay size was redone in order to make a flat plate system more feasible without the need of either a super thick slab or the more complicated post tension system. By dividing the 60' span up into 3 equal 20' bays that span the width of the 15' wide rooms, a feasible layout is achieved, but there will have to be columns sticking out of the partition walls. If this system proves to be a good solution, rectangular columns that won't intrude as much on hotel rooms will be investigated.

Portland Cement Association's "Concrete Floor Systems Guide to Estimating and Economizing" was used to determine initial slab thickness and column size based upon the bay size and loading, *Table 9*. The direct design method in the American Concrete Institute (ACI) was used to determine the moments in the slab column and middle strips and then size reinforcement and check punching shear at the columns. The moments were verified with spSlab and were within the capacity of the slab, *Figure 18*. Details of the calculation using direct design method can be found in Appendix D.

Flat Plate			f'_c = 4,000 psi SIDL = 20 psf LL = 50 psf		
Bay Size (ft)	Slab Thickness (in.)	Square Column Size (in.)	Concrete (ft^3/ft^2)	Reinforcement (psf)	Formwork (ft^2/ft^2)
15 × 15	6.0	14	0.50	2.20	1.00
15 × 20	7.5	18	0.63	1.94	1.00
15 × 25	9.5	20	0.79	2.50	1.00
15 × 30	11.5	22	0.96	3.08	1.00
20 × 20	7.5	20	0.63	2.06	1.00
20 × 25	9.5	22	0.79	2.62	1.00
20 × 30	12.0	24	1.00	3.15	1.00
25 × 25	9.5	26	0.79	2.74	1.00
25 × 30	11.0	30	0.92	3.22	1.00
30 × 30	12.0	32	1.00	3.46	1.00

Table 9: Flat plate sizing table

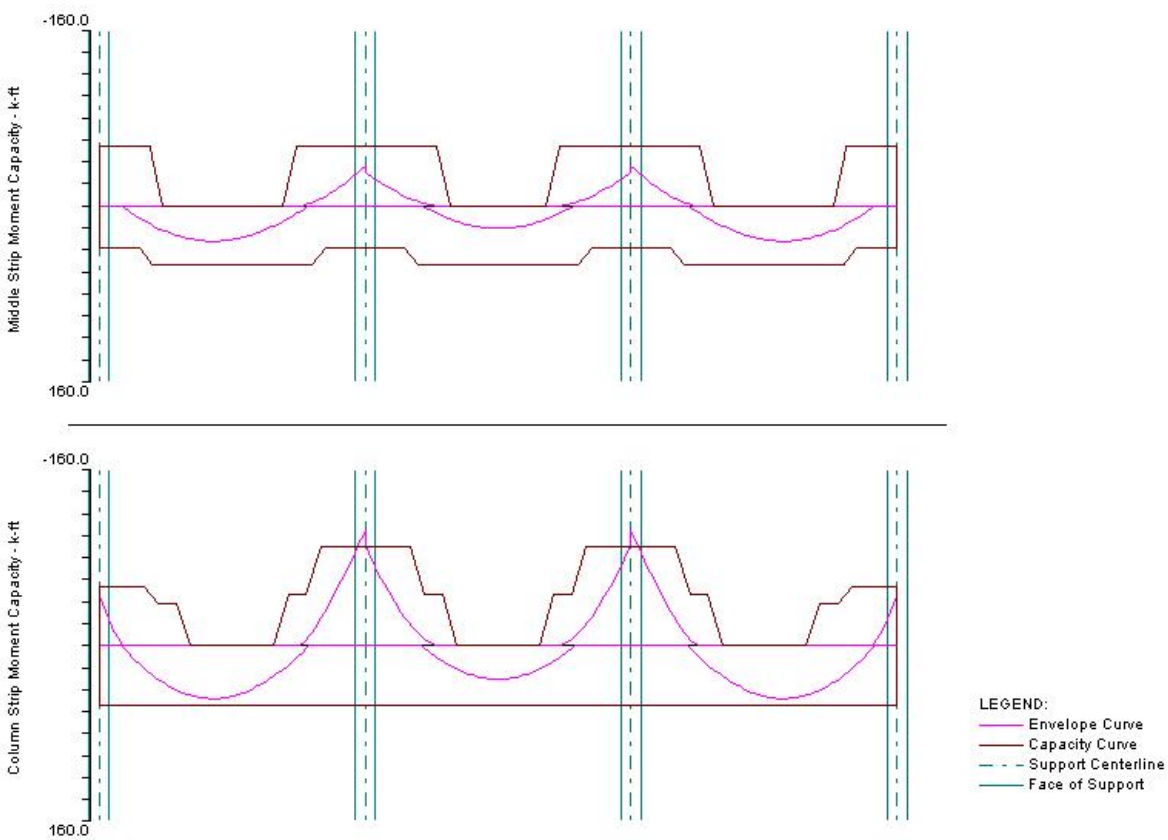


Figure 18: spSlab moment curves

Advantages

The concrete two-way flat plate system is commonly used in hotels and similar structures due to its lower live loads, simpler grouped HVAC systems, and low floor to floor height. The slab is only 7.5 inches thick and the underside is flat, fireproof, and ready for application of architectural finish. Construction is simple when compared to post tensioning and other concrete systems that have more irregular shaped concrete forms. Having smaller bays increases the number of columns, but decreases their size and thus making them easier to hide in architectural walls.

Disadvantages

The system will slightly intrude on the architectural layout. Some rooms in the east wing of the hotel tower will need to be shifted at least 6 inches due to the fact that the maximum column offset of the system is 1'-6" and the current layout would require an offset of 2'-0". Also columns may not be able to be sized slender enough to completely fit them inside the partition walls. Also some columns will probably need to be transferred to keep the open space on the first floor. Construction will take more time due to the fact that forms must be made and the concrete has to cure.

Floor System Comparison

There are many different types of floor systems that could work for the Hyatt Place North Shore. There are also many different things to consider when choosing the best option for the location. Every project has a different list of needs and desirable traits, like how most hotels have low floor to floor heights and flat ceilings that are ready for spray on architectural finishes. The floor systems analyzed were compared based on cost, weight, floor depth, fireproofing, vibration, ease of construction, foundation needs, gravity and lateral force resisting systems, and changes to architecture.

Cost:

Cost is often if not always a driving factor in design of systems, it is all of the trades' role to do their best to keep costs down and design systems that efficiently work well together. It is important to pick a floor system that works well with the architectural and MEP systems. In this case, cost can be limited by having a system that doesn't require a hung ceiling or fireproofing. The system also affects construction costs and timeline of the project. An assemblies estimate was done using R.S. Means 2011 data to get a conceptual estimate for the systems. This estimate is not as accurate as a unit price method, but much simpler and better in the conceptual phase of a building.

Floor Cost (per square foot)				
System	**Material Cost**	**Labor Cost**	**Location Factor**	**Total Cost**
Precast Plank on Masonry Walls	7.9	2.5	1.009	**10.5**
Precast Plank on Steel Frame	8.8	4.7	1.009	**13.6**
Composite Steel Frame	16.2	8.5	1.009	**24.9**
Concrete 2-Way Flat Plate	4.9	8.4	1.009	**13.4**

Table 10: Flat plate sizing table

From the cost investigation seen in *Table 10*, the cheapest floor (slab only) system is the existing precast plank. It differs from the precast plank on steel because the existing structure doesn't have a 2 inch concrete topping. With further investigation into the two precast plank systems, the planks on steel frame would prove cheaper in the end due to the ease of constructability of a steel frame compared to masonry bearing walls. This can also be said for the flat plate system compared to the masonry walls. In the end, the only system out of the running at this point is the composite steel frame.

Weight

Weight is something that definitely affects the structural system as a whole and also cost in the end. Heavier floors need progressively larger columns and then foundations to carry them. Also a large difference in mass can lead to large differences in the lateral load due to seismic forces, so weight affects both the gravity and lateral system.

The majority of the floor systems have similar slab weights, because even the steel system has concrete as one component of the slab. The difference between the slab weights is through the use of steel. The precast planks have higher strength concrete and steel tendons in them that make it possible to span with less concrete material, leading to "hollow core" planks. The lightest slab system was the composite deck. The deck and concrete work together well and they only have to span 10 feet in between beams, so beams and girders must also be counted into the equation. Likewise, since we are looking for an overall light superstructure, the gravity system supporting the floor system was approximated in order to get a better idea which system as a whole is the best option.

Floor Weight (psf)				
System	**Slab Weight**	**Beams/Girders**	**Columns/Walls**	**Total**
Precast Plank on Masonry Walls	76.0	None	47.0	**123.0**
Precast Plank on Steel Frame	88.0	2.2	2.5	**92.7**
Composite Steel Frame	69.0	6.0	1.6	**76.6**
Concrete 2-Way Flat Plate	93.3	None	38.6	**131.9**

Table 11: Floor system weights

Table 11 shows the composite steel system is the best option according to weight, but it has already been ruled out because of its cost. The main thing learned from this table is that a great majority of weight can be saved by having steel columns. So far the precast plank on steel frame is the best system option. It is extra important to save weight on this site since the building is placed on soil along the Allegheny River that has a bearing capacity of 1500 psi.

Floor Depth:

The desired depth of floor system varies by application. Some buildings require lots of room to run MEP services though. In hotels most of the MEP is ran up through shafts, and uses are stacked on top of each other, thus it is possible to do without the space. Some cities also have a limit on the total building height, so a thin floor system is desired to get as many floors in as possible. In hotels a thin floor system is desired, all of the systems except for composite steel frame meet this need. The precast plank on steel frame uses a D-Beam that has a wider bottom flange in order to have the precast planks rest at the bottom of the girder, tucking the slab system into the girder rather than the girder being below the slab. A summary of the systems is in *Table 12*.

System Depth		
System	Slab Depth	Total Depth
Precast Plank on Masonry Walls	8”	8”
Precast Plank on Steel Frame	10”	10”
Composite Steel Frame	6.5”	22.5”
Concrete 2-Way Flat Plate	7.5”	7.5”

Table 12: System depths

Fireproofing:

Concrete is naturally a good fire barrier, the existing plank system and concrete flat plate system will need no fireproofing at all. The precast plank on steel frame will need fireproofing on the steel portions. The Composite steel frame would need its whole underside coated in fireproofing. Fireproofing adds extra cost to the system, thus the composite steel frame has another strike against it, which was noticed in the cost investigation earlier.

Vibration:

Another sometimes important factor with floor systems is vibration. Some types of buildings like hospitals and labs definitely need to pay attention to how much a floor can vibrate because of the work done on that floor. In hotels vibration is not as much of a big deal, but the majority of the possible systems do well with vibration. In general a light weight floor systems that span long distances have issues with vibration.

Construction:

Precast concrete planks tend to simplify and expedite construction. The planks are already cured and ready to hold load once bearing on their resting place. Sometimes complications arise getting the planks' edges to lock into each other due to differences in camber because of conditions when they were formed and cured. There is ample room on site to store planks and steel members. Putting the planks on steel frame would further simplify the construction, laying concrete masonry units is time consuming. Lead time would have to be considered for both the steel and planks though.

Both the composite floor system and the concrete flat plate are common to construction workers although laying forms, rebar, and welding shear studs is labor intensive. The concrete flat plate system is the simplest cast-in-place concrete system because there are no drop panels or beams, leaving a flat surface formwork. Both of these systems require time for the concrete to cure before full strength.

Gravity System and Foundations:

The size of gravity members and foundations is directly affected by the weight of the floor system, and the affect is exponential. The possibility to reduce the overall weight of the structure and number of foundation piles needed is very tempting. The soils on the site are soft and the bedrock is deep. The composite steel frame the best weight solution, but the precast concrete planks on steel frame provides a good mix of weight and efficiency.

Lateral System:

All of the alternative floor systems require the lateral system to change. Designing both steel and concrete systems for lateral load is more complicated than laying out masonry shear walls. The location and design of lateral elements is something that can be done, and will be explored in later reports.

Architecture:

All of the alternative floor systems move from masonry shear walls to concrete or steel columns. The location of the columns has effect on the architecture. All columns can be located along walls, but some will be hard to contain within the wall. The concrete flat plate system requires a small shift in room location. The more columns there are the smaller they need to be. But also the more columns there are, the more problems arise when they come to the ground floor and trying to keep open space. Although it is easier to work around columns than 6 stories of heavy shear wall.

Conclusion:

Overall System Comparison				
	Precast Plank on Masonry Walls	**Precast Plank on Steel Frame**	**Composite Steel Frame**	**Concrete 2-Way Flat Plate**
Cost ($ per SF)	**10.5**	13.6	24.9	13.4
Weight (psf)	123	92.7	76.6	131.9
Total Depth	8"	10"	22.5"	7.5"
Fireproofing	**1**	2	3	**1**
Vibration	**1**	**1**	2	**1**
Construction	2	**1**	2	2
Foundations	NA	2	**3**	1
Lateral	NA	3	3	3
Architecture	NA	**2**	**2**	3
Sum of Good Marks	**3**	**3**	2	2
1 = Not Needed/Easy	2 = Some/Moderate	3 = Most/Hard	*The best of each category is in bold	

Table 13: Overall system comparison

Table 13 provides all of the comparisons in one table. From this table it is seen that the plank systems have more benefits than the other two systems, but there isn't enough detail to get a true feel for which system is best. Upon further investigation, the best alternative still proves to be the precast plank on steel frame. If a change is to be made it is between the precast plank on steel frame and the concrete 2-way flat plate, but in the end the large weight difference is the deciding factor.

The precast planks on the innovative D-Beam provide the best of both worlds. There is an efficient thin slab system and a light framework. Fireproofing and vibration are of minimal concern, and weight is reduced enough to most likely be able to reduce the number of foundation piles extending 70 feet into the earth. The architectural layout and MEP systems will have to minimally changed.

The composite steel frame can be ruled out quickly because of the depth, need for fireproofing, and cost. The precast plank on masonry wall and concrete 2-way flat plate systems can be ruled out largely on weight issues. Both systems are good for hotels, but the site location is the deciding factor.

This exercise was a good way to learn more about systems and what goes into choosing the best one. There are many options when it comes to floor and vertical structural systems that are dependent on many different factors. In the end it is something that should be explored by all trades working on the project and discussed with the owner to make sure his needs are met.

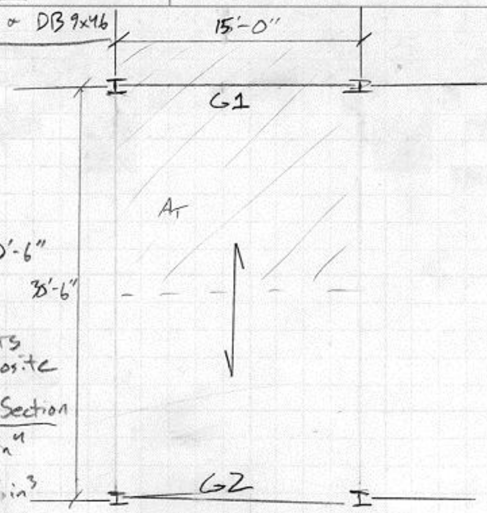
Appendices:

Appendix A: Precast Plank on Steel Frame Calculations

Kyle Tennant — Tech. 2 — (1)

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

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



Properties

Steel Section	Transformed Section (once acts composite)
$I_{x_s} = 159\ in^4$	$I_{x_t} = 332\ in^4$
$S_t = 24.4\ in^3$	$S_t = 62.1\ in^3$
$S_b = 51.0\ in^3$	$S_b = 77.7\ in^3$
$M_{Allow} = 61.0$ k ft	$b = 5.25$
	$t_w = .375$

Live Load = 40 psf

$$LL\ reduction = 40\left(.25 + \frac{15}{\sqrt{2(30.6\times 6)}}\right) = 29.2\ psf$$

(construction) Interior Girder G1

Initial Load (before composite)

DB 9x46 needed for Δ_{DL}

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

Deflection

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

Total Load (once composite)

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

$$M_{Total\ Load} = 51.5 + 72.1 = 123.6\ k\,ft$$

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

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

(next page)

Kyle Tennant | Tech 2 | ②

Composite Steel and Precast Plank Continued

Check Compressive Stress on Concrete

→ transformed steel section must be converted to concrete section

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

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

Check Bottom Flange Tension Stress (check for total load)

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

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

Check Shear

Load $= 60 + 30 + 29.2 + 25 = 144.2 \text{ psf} = .144 \text{ klf}$

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

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

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

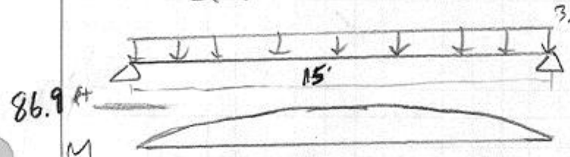
Exterior Girder G2

Loading

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

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

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



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

$l_b = 15'$ ↓ Table 3-10 15' unbraced

W12x26 → $\phi M_n = 87.7$ ft-k ✓

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

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

Kyle Tennant | Tech 2 | 3

Composite Steel and Precast Plank Continued

Girder G 2 Design Continued

Checks

Shear

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

LL Deflection

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

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

TL Deflection

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

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

$\boxed{W12\times26}$

Precast Concrete Plank

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

Loads:

DL: SDL = 30 psf

LL: 40 psf

Total = 70 psf

go to manufacturer's table unfactored

Allowable

Given Pittsburgh Flexicore Co., Inc.

Use T8S78-1.75 on 30' 6" span

can carry 106 psf

106 psf > 70 psf ✓

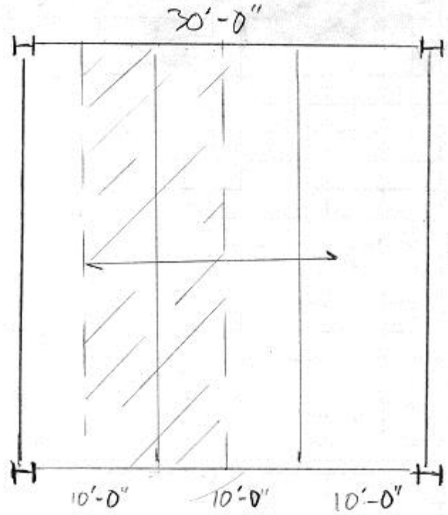
Used the stronger plank to make sure long term deflection won't be a problem

Deflection + Moment are considered in table tabulation

Appendix B: Composite Steel Frame Calculations

Kyle Tennant | Tech 2 ①

Composite Deck on Composite Beam Floor System



Loading

Live Load = 40 psf

Live Load Reduction $= L_0\left(.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right)$

$A_T = 10(30) = 300$ $K_{LL} = 300(2) = 600 > 400$ ✓

$L = 40\left(.25 + \frac{15}{\sqrt{600}}\right) = \underline{34.5\ psf}$

.86 > .5

Dead Loads

Partitions = 15 psf

MEP = 15 psf

Self Weight = ?

Materials

Normal Weight Concrete 145 pcf

Metal Deck

Steel W-members

Composite Metal Deck Design

Vulcraft manual → pg 46

2VLI17 → 2.96 psf

SDI Max Unshored Clear Span

1 span 8'2" | 2 span 10'3" | 3 span 10'7"

↑ specify 3 span

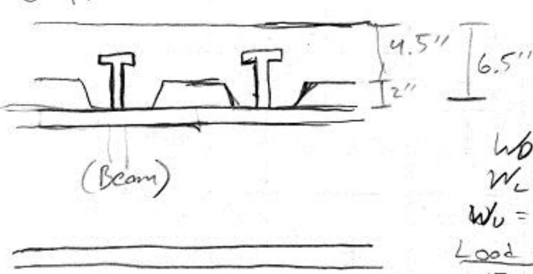
Superimposed LL @ Clear span 10'-0"

↓

239 psf > 34.5 + 15 + 15 ✓

Floor Statistics

69 psf (66 conc. + 3 steel)



3/4" stud $\xrightarrow{T3\text{-}21}$ $Q_n = 17.2k$

Total Weight

$W_D = 69 + 15 + 15 = 99$

$W_L = 34.5\ psf$

$W_U = 99(1.2) + 34.5(1.6) = 174\ psf$

Load on Beam

174 psf × 10 ft = 1,740 plf

↑ trib width

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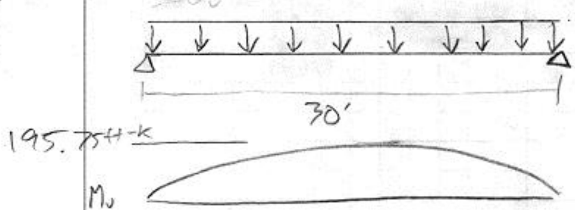
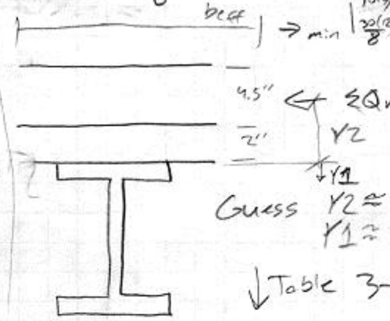
Design Composite Beam



$W_u = 1{,}740$ plf $= 1.74$ klf

$M_u = \frac{1.74(30)^2}{8} = 195.75$ ft-k

$b_{eff} \rightarrow \min \left| \frac{10(12)}{2} = 60'' ;\ \frac{30(12)}{8} \times 2 = 90'' \right.$ ✓



Guess $Y2 \approx 5.5$
$Y1 \approx 3$ (category)

↓ Table 3-19

W10X22 → $\phi M_p = 214$ ✓

$I = 118\ in^4$ $\quad \Sigma Q_n = 221$ kips

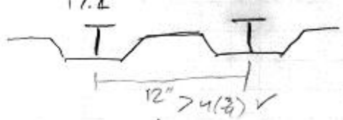
$a = \frac{221}{.85(4)(60)} = 1.08$

$Y2 = 6.5 - \frac{1.08}{2} = 5.96 > 5.5$ ✓

Shear Studs over whole length

$\Sigma Q_n = 259$ kips $\quad$ ¾" Φ Stud → $Q_n = 17.1$ k

$n = \frac{259}{17.2} = 15.05 = 15(2) = 30$



1 per rib → 30 studs ✓

Checks

Unshored Length

$W_{LL} = 20(10) = 200$ plf (construction workers) $\quad W_{DL} = 69(10) + 22 = 712$ plf (Floor, Beam)

$W_u = 1.2(712) + 1.6(200) = 1.17$ klf ✓ → $M_u = \frac{1.17(30)^2}{8} = 131.6 < 214$ ✓

or $W_u = 1.4(712) = .996$ klf

Check LL Deflection

$W_{LL} = 34.5(10) = .345$ klf $\quad I_{LB} \xrightarrow{\text{Table 3-20}} = 413$

$\Delta_{LL} = \frac{5(.345)(30^4)(1728)}{384(29000)(413)} = .52'' < \frac{30(12)}{360} = 1''$ ✓

Check Wet Concrete (before composite)

$W_{WL} = 69(10) + 22 = .712$ klf $\quad \frac{L}{240} = \frac{30(12)}{240} = 1.5''$ because majority is DL

$\Delta_{WL} = \frac{5(.712)(30^4)(1728)}{384(29000)(118)} = 3.79'' > 1.5$ ✗ → need $I = 160.4\ in^4$

1" camber

W12x26 $\quad I = 204\ in^4$

$Y2 = 5.5$, $Y1 = 3$ → $\phi M_p = 281$ ft-k

$\Sigma Q_n = 259$ kips

$a = 1.3 \quad Y2 = 6.5 - \frac{1.3}{2} = 5.85 > 5.5$ ✓

$\boxed{W12 \times 26 (30)\ c = 1''}$

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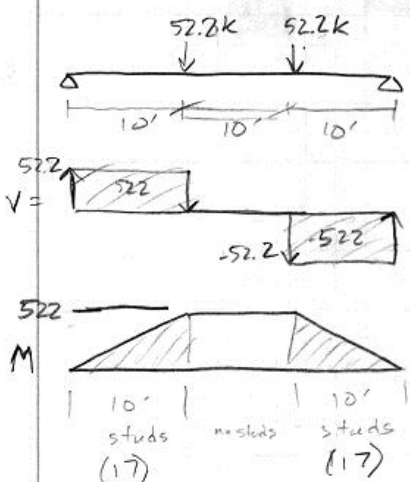
3

Design Composite Girder

Interior Girder



$P_u = 2\left(\frac{1}{2}(1.74\,klf \times 30)\right) = 52.2\,kips$

$M_u = 522$ ft-k

Assume → a ≈ 4 → Y2 ≈ 5.5"

Table 3-19 ↓

W16×45 → $\phi M_p = 542$ ✓ $\Sigma Q_n = 365$

Trying to keep low thickness

b_{eff} min | $30(12)/2 = 180"$; $\frac{30(12)}{8}(2) = 90"$ ✓

$a = \frac{365}{.85(4)(90)} = 1.2$ ∴ Y2 ✓

Studs

$\frac{365}{21.5}(2) = 34$

deck parallel

spacing ≥ 4(¾) = 3" (allowed)

$\frac{10(12)}{17} = 7"$ spacing > 3" ✓ (actual)

Checks

Unshored Length

$P_u = 1.17\,klf\,(30\,ft) = 35.1\,k$ $M_u = 35.1(10) = 351$ ft-k < 522 ✓

LL Deflection

$I_{LB} = 1450$ $P_u = .345(30) = 10.35\,k$

$\Delta_{LL} = \frac{10.35(30^3)(1728)}{28(29000)(1450)} = .42" < \frac{30(12)}{360} = 1"$ ✓

Wet Concrete Load

$W_{wc} = .712(30) = 21.36\,k$ $I = 586$ $\frac{30(12)}{240} = 1.5"$

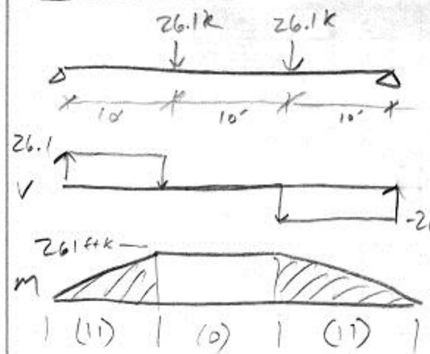
$\Delta_{wc} = \frac{21.36(30^3)(1728)}{28(29000)(586)} = 2.09" > 1.5"$ ✗ → camber beam 1" ✓

$\boxed{W16 \times 45\ (34)\ c = 1"}$

Kyle Tennant | Tech2

Design Composite Girder

Exterior Girder $P_U = \frac{1}{2}(1.74 \times 30) = 26.1$ kips



$M_U = 261$ ft-k Assume $Y1 \approx 4$

$Y2 \approx 5.5''$

Table 3-17

W12x30 $\Rightarrow \phi M_p = 294$ ft-k

$\Sigma Q_n = 225$ k

$I_{LB} = 643$

$I = 238$ in^4

Studs

$\frac{225}{21.5}(2) = 21 \rightarrow 22$

✓ meets spacing

Checks

Unshored Length

$P_U = 1.17(15) = 17.6$ k $M_U = 176$ ft k < 294 ft k ✓

LL Deflection

$I_{LB} = 643$ $P_U = .345(15) = 5.2$ kips

$\Delta_{LL} = \frac{5.2(30^3)(1728)}{28(29000)(643)} = .46'' < 1''$ ✓

Wet Concrete

$W_{WC} = .712(15) = 10.7$ k $I = 238$ in^4 $\frac{30(12)}{240} = 1.5''$

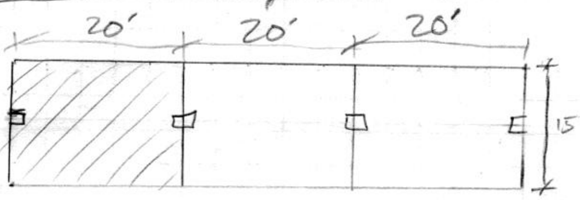
$\Delta_{WC} = \frac{10.7(30^3)(1728)}{28(29000)(238)} = 2.5'' > 1.5''$ with 1" camber ✓

$\boxed{\text{W12X30 (22) } c = 1''}$

Appendix C: Concrete 2-Way Flat Plate Calculations

Kyle Tennant | Tech2 | Flat Plate System

Flat Plate System (2-way concrete slab)



Used Concrete Floor Systems Guide to Estimating and Economiz to estimate slab thickness, column size, and set a range for amount of needed reinforcement.

Slab Thickness: 7.5" $\ell_1 = 20'$
Columns: 18" square $\ell_2 = 15'$
Floor to floor height: 8'8" $\ell_n = 18.5'$

steel → $f_y = 60$ ksi
concrete → $f'_c = 4000$ psi

Dead Load SDL = 15 psf
partitions = 15 psf
self weight ≈ $.63(145) = 91.35$ psf $+ 1.94$ (reinforcement) $= 93.29$ psf
or $\frac{7.5(12)}{12(12)}(150) = 93.75$ psf

Live Load 40 psf

Direct Design Method → 3 spans ✓ → same span lengths ✓

Long to Short Span Ratio $= \frac{20}{15} = 1.3 < 2$ ✓

Column offset max of $.1(15) = 1.5$ ft
↳ East wing need a 2 ft offset → will need small architectural change

Check Min. Slab Depth against ACI 9.5(c)
$\ell_n/30 = \frac{(18.5)(12)}{30} = 7.4" < 7.5"$ ✓

Static Moment

not reducible → $K_{LL} = 1$ for 2-way
$\alpha A_T = 300 < 400$ ✗

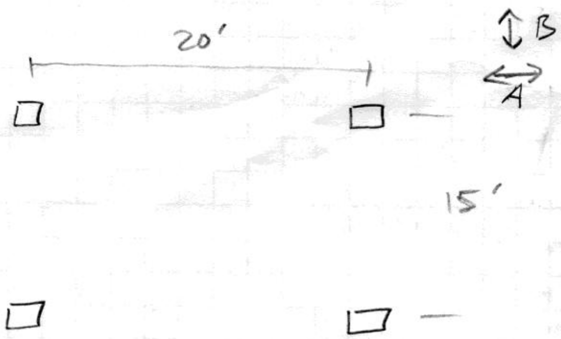
$M_{oA} = \frac{q_u \ell_2 \ell_n^2}{8}$ $q_u = 1.2(93.75 + 30) + 1.6(40) = .212$ ksf
$\ell_2 = 15'$ $\ell_n = 18.5'$

$= \frac{.212(15)(18.5^2)}{8}$

(20' span) $M_{oA} = 136.0$ k-ft

M_{oB} $\ell_2 = 20'$ $\ell_n = 13.5'$
$= \frac{.212(20)(13.5^2)}{8}$

(15' span) $M_{oB} = 96.6$ k-ft



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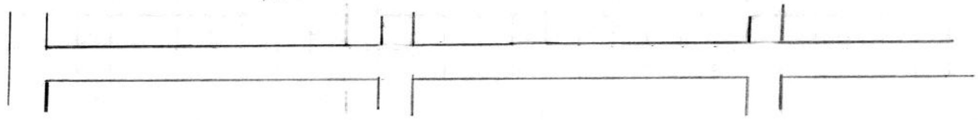
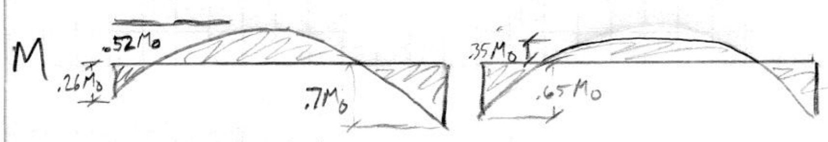
Distribution of M_o (For slab without beams or dro

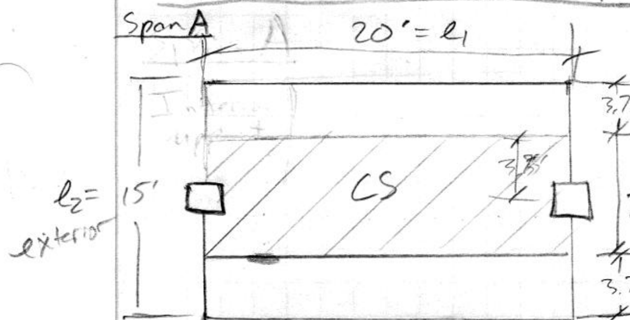
↳ ACI 13.6.3.3





$M_o = 136$ ft-K Span Direction A (20') →Interior and exterior support		M_u	$M_o = 96.6$ ft-K Span Direction B (15') → only interior supports	M_u
Interior Support	.7	−95.2	.65	−62.8
Midspan	.52	70.7	.35	33.8
Exterior Support	.26	−35.4		

Find Column Strip & Middle Strip Moments



$C.S. = \frac{15}{2} = 7.5'$

$M.S. = 7.5 - 3.75 = 3.75'$ (det

→ no beam → $\alpha_{f1} = 0$

$\beta_t = 0$

$\frac{l_2}{l_1} = \frac{15}{20} = .75$

M.S. takes remained of momen

	Span A		→ factor	M_u	Width	Moment/foot width
ACI 13.6.4.1	Interior Support (95.2 k-ft)	Column Strip	.75	−71.4	7.5	−9.5 → #5 @ 18"
		Middle Strip	.25	−23.8	7.5	−3.2 → #4 @ 18"
ACI 13.6.4.4	Midspan (+70.7 k-ft)	Column Strip	.60	42.4	7.5	5.7 → #4 @ 12'
		Middle Strip	.40	28.3	7.5	3.8 → #4 @ 18'
ACI 13.6.4.2	Exterior Span (−35.4)	Column Strip	1.00	−35.4	7.5	−4.7 → #4 @ 18"
		Middle Strip	.0		7.5	0 ← use min reinf #4 @ 18

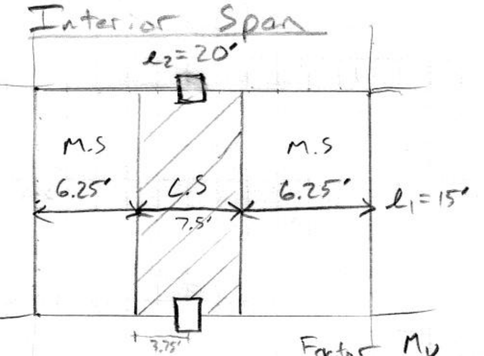
calculated reinfor on page 4

Checked Span A in spSlab and got similar answ which spSlab showed to be within the capacity of the system

Kyle Tennant | Tech 2 | Flat Plate System ③

Column Strip and Middle Strip Moments for Span B

Interior Span



$C.S. = \frac{15}{4} = 3.75'(2) = 7.5'$

$M.S. = 10 - 3.75 = 6.25(2) = 12.5'$

$\alpha_{f1} = 0 \quad \beta_t = 0$

$\frac{\ell_2}{\ell_1} = \frac{20}{15} = 1.33$

ACI 13.6.4.1
ACI 13.6.4.4

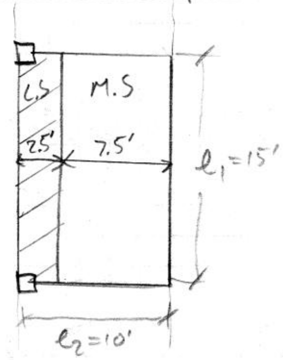
		Factor	M_U	Width	Moment/foot width
Interior Support (-62.8)	Column Strip	.75	-47.1	7.5	-6.3 → #4 @ 12"
	Middle Strip	.25	-12.7	12.5	-1.3 → #4 @ 18"
Midspan (53.8)	Column Strip	.60	20.3	7.5	2.7 → #4 @ 18"
	Middle Strip	.40	13.5	12.5	1.1 → #4 @ 18"

Exterior Span

New $M_o = \frac{.212(10)(13.5^2)}{8} = 48.3$ k-ft

$Int. = .65(48.3) = -31.4$ k-ft

$Midspan = .35(48.3) = 16.9$ k-ft



$C.S. = \frac{10}{4} = 2.5'$

$M.S = 10' - 2.5' = 7.5'$

$\alpha_{f1} = 0 \quad \beta_t = 0$

$\frac{\ell_2}{\ell_1} = \frac{10}{15} = .67$

		Factor	M_U	Width	Moment/foot width
Interior Support (-31.4)	Column Strip	.75	-23.6	2.5	9.4 → #5 @ 18"
	Middle Strip	.25	-7.9	7.5	1.0 → #4 @ 18"
Midspan (16.9)	Column Strip	.6	10.1	2.5	4.1 → #4 @ 18"
	Middle Strip	.4	6.8	7.5	.9 → #4 @ 18"

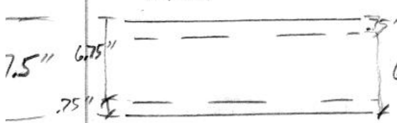
Kyle Tennant | Tech 2 | Flat Plate System | 4

Design Reinforcement

→ Min reinforcement ratio .0018 of gross concrete area
→ max spacing of 3 x (slab thickness) or 18"

Use #4s & 5s
#4: .20 in², d = .5 in
#5: .31 in², d = .625 in

$A_{s\,min} = .0018(7.5)(12) = .16\ in^2/ft$ (min)



min cover in slabs = .75"

$d = 6.75'' - 1.5(.625) = 5.8''$

$A_s = \frac{M_u}{\phi f_y d}$ $\phi f_y = 60\ ksi$, $d = 5.8''$

→ Max moment/foot width needed to meet is 9.5 k-ft/ft

Checks some possible arrangements of reinforcing

#4 @ 18" → $A_s = .2 / \left(\left(\frac{7.5}{12}\right)\left(\frac{12}{18}\right)\right) = .21 > .16$ ✓ $A_s = \frac{A}{bh}$

$a = \frac{.21(60)}{.85(4)(12)} = .31$ $a = \frac{A_s f_y}{.85 f'_c b}$

$\phi M_n = .9(.21)(60)(5.8 - \frac{.31}{2}) / 12$ $\phi M_n = \phi A_s f_y (d - a/2)$

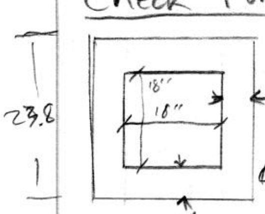
$\boxed{\#4 @ 18''}$ → = $\boxed{5.33\ k\text{-}ft/ft}$

$\boxed{\#4 @ 12''}$ → $A_s = .32$ $a = .47$ $\boxed{\phi M_n = 8.0\ k\text{-}ft/ft}$

$\boxed{\#5 @ 18''}$ → $A_s = .50$ $a = .73$ $\boxed{\phi M_n = 12.2\ k\text{-}ft/ft}$

These reinforcement arrangements cover all needs... most regions only need #4 @ 18"

Check Punching Shear



Interior column

$d/2 = \frac{5.8}{2} = 2.9''$

$b_0 = 23.8(4) = 95.2''$

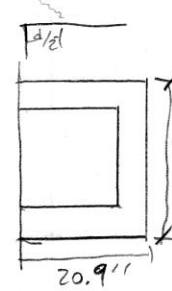
Concrete strength =

$\phi V_c = .75(4)\sqrt{f'_c}\, b_0 d$
$= .75(4)(\sqrt{4000})(95.2)(5.8)$
$= 104.8$ kips

$W_u = .212$ ksf
$A_T = 20 \times 15 = 300$
$A_{column} = (23.8/12)^2 = 3.9$

$V_u = W_u (A_T - \text{column area})$
$= .212(300 - 3.9)$
$= 62.8$ kips

$\phi V_c > V_u$ ✓



$b_0 = 23.8 + 20.9(2)$
$= 65.6$

$\phi V_c = .75(4)\sqrt{4000}(65.6)(5.8)$
$= 72.2$ kips

$A_T = (\frac{20}{2} + \frac{9}{12})(15)$
$= 161.25\ ft^2$

column $= \left(\frac{23.8}{12}\right)\left(\frac{20.9}{12}\right) = 3.45\ ft^2$

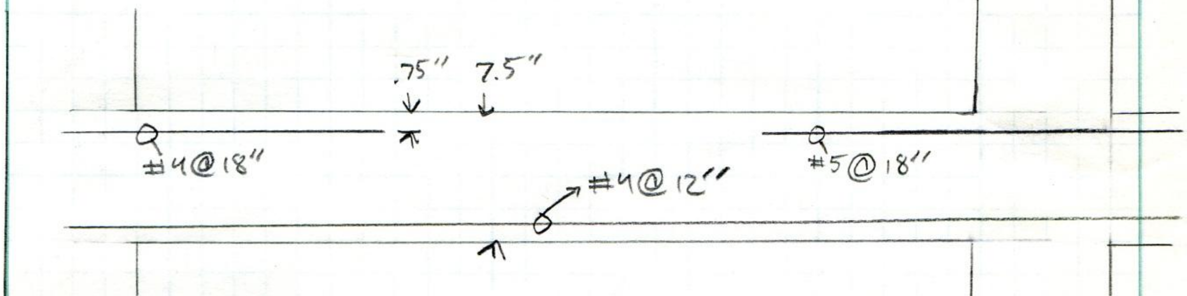
$V_u = .212(161.25 - 3.45)$
$= 33.5$ kips

$\phi V_c > V_u$ ✓

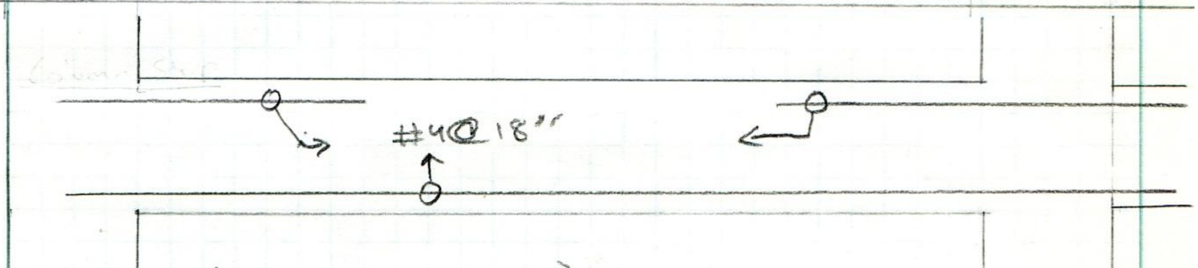
Kyle Tennant | Tech 2 | Flate Plate System | 5

Location of Shear Reinforcement → did not determine bar cutoffs
→ full bottom reinforcement as suggested

Span A (20' span direction)

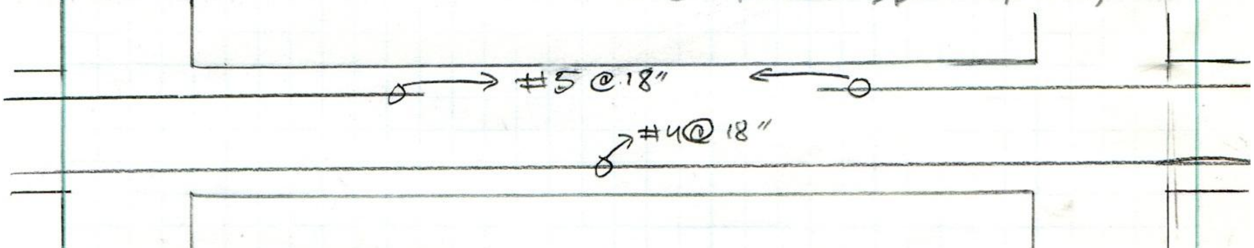


Column Strip

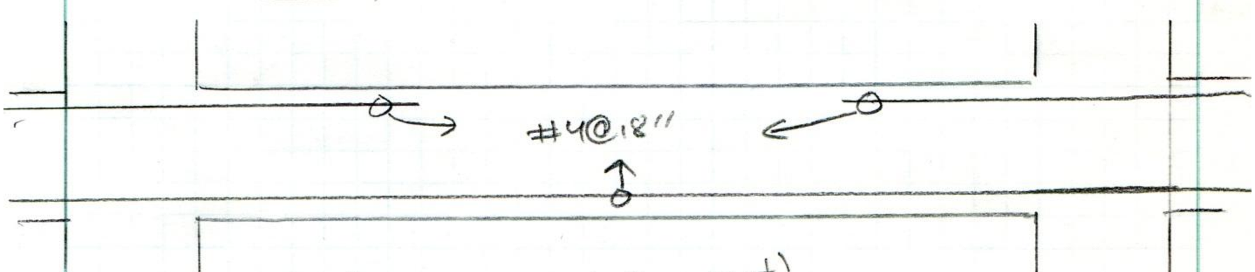


Middle Strip (all min. reinforcement)

Span B (15' span direction) → have to look at Interior span and Exterior span separately



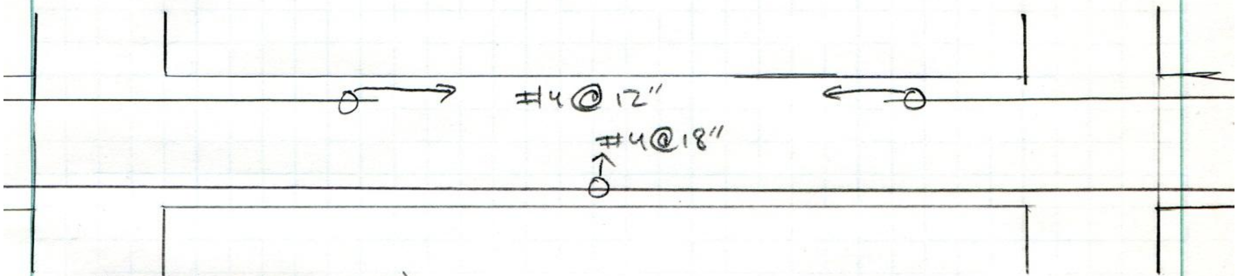
Column Strip (Exterior)



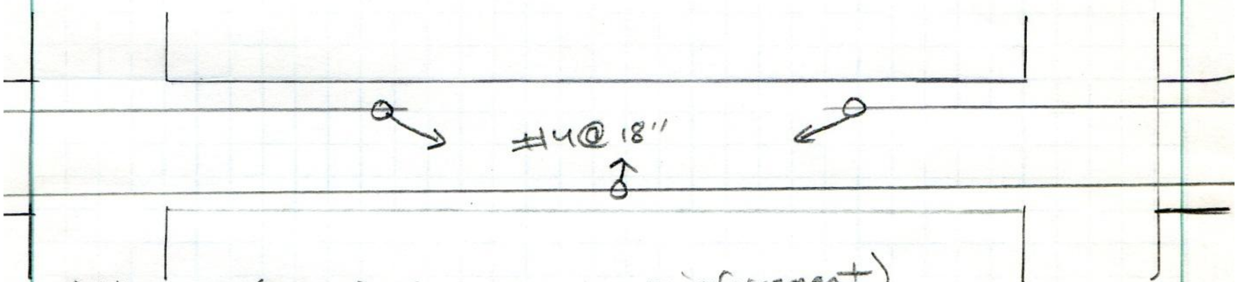
Middle Strip (Exterior) (all min. reinforcement)

Kyle Tennant | Tech 2 | Flate Plate System (6)

Span B



Column Strip (Interior)



Middle strip (Interior) (all minimum reinforcement)

Appendix D: Comparison Calculations – System Weights

Kyle Tennant | Tech 2 | Weight | 1

Total system Weight

→ find total weight over the 30'-6" × 30'-0" area or typical bay area and then find an average per square foot.

Precast Concrete Plank on Masonry Walls (existing)

Slab – 63 psf + 13 psf leveling topping = 76 psf

Beams – none

Girders – none → masonry bearing walls ≈ 90 psf (8 ft) = 720 plf

Columns – masonry walls ↗

30'-6" × 30'-0" → Area = 915 ft²

$$\frac{915(76) + 720(60)}{915} = 123 \text{ psf}$$

Floor only	Vertical System	Total
76 psf	47 psf	123 psf

Composite Steel and Precast System

Slab – 63 psf + 25 psf (2" concrete topping) = 88 psf

Beams – none

Girders – 15' DB 9×41 15' 12×26

$$41(15) + 26(15) = \frac{1005 \text{ lbs}}{15 \times 30.5} = 2.2 \text{ psf}$$

Columns – approximate a column size

Supporting → 4th floor
4,5,6,7 + roof
more work done on comp. system

$A_T = 15' \times 30.5' = 457.5 \text{ ft}^2$

Interior
$L_r = 24$ psf
$P_L = 43.9$ K
$P_D = 215.7$ K
$P_S = 11.4$ K
$P_U = 334.8$ K
W10×33
$\phi P_n = 365$ ✓
4-1
KL = 8

Exterior $A_T = 228.8$ ft²
$L_r = 29.8$
$P_L = 27.3$ K
$P_D = 107.9$ K
$P_S = 5.7$
$P_U = 176$ K
W10×33 (stuck with because its already small)

$$\text{Total} \approx \frac{4(33)(8)}{457.2} = 2.5 \text{ psf}$$

Total system = 88 + 2.2 + 2.5 = 92.7 psf

Total System Weights Continued

Composite Steel Frame

Slab = 69 psf

Beams = W12×26 (4 in 30'6"×30'-0" bay)

$26(30.5)(4) = \frac{3172}{30.5\times30} = 3.5\ psf$

Girders W16×45 & W12×30

$\frac{45(30)+30(30)}{30.5\times30} = 2.5\ psf$

Columns approximate column size

$A_T = 30.5\times30 = 915\ ft^2 \quad L_r = 40\left(.25+\frac{15}{\sqrt{4(915)}}\right) = 19.9\ psf$ > 40% ✓

→ Check a middle floor → so column supporting the 4th floor
↳ 4 floor + roof

$P_L = 19.9(915)(4) = 72.8K$

$P_D = 75(915)(4)+30(915)(4) = 384.3K$

$P_S = 25(915)(1) = 22.8K$

$P_U = 1.2(384.3)+1.6(72.8)+.5(22.8) = 589.\ Kips \xrightarrow{\text{Table 4-1}}$ get an aprox size for only gravity

KL = 8 ft
W14×53 $\phi P_n = 585 \approx 589$ ✓

Exterior Columns $A_T = 457.5\ ft^2$

$L_r = 24$

$P_L = 43.9K$

$P_D = 192.2K$

$P_S = 11.4K$

$P_U = 306.58K \xrightarrow{\text{Table 4-1}} W10\times33 \quad \phi P_n = 365$ ✓

$Weight = \frac{8.5(53)(2)+8.5(33)(2)}{915} = 1.6\ plf$

Total Weight $= 69+3.5+2.5+1.6 = 76.6\ plf$

Kyle Tennant | Tech 2 | System Weights | (3)

Two-Way Concrete Flat plate System (use design old)

Slab - 145(.63) + 1.94 = 93.3 psf (7.5" thick)

concrete, reinforcement — solid ∴ heavier than precast planks

Beams - none

Girders - none

Columns - 18"×18" → 2.25 ft² (8.5 ft)(150) = 2.9 k per column (6 to 11)

2.9(4) = 11.6 k / (20×15) = 38.6 psf

Total = 93.3 + 38.6 = 131.9 psf

heavier than existing

close because of the # of columns (made smaller bays than masonry) → compared to 47.2 psf from masonry walls

Cost Estimation

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Determined cost based on structural sytem and location.
↳ material and labor are considered — Pittsburgh, PA

System	Materials ×	Labor ×	Location =	Total
1) Precast Plank on Masonry walls (no topping) 30' span → Load → 70psf	Plank → 7.9; Masonry Walls (8") 3.38; 11.3	2.49; 6.7; 9.2	1.009	20.65 $/SF
15x30 bay — 2) Composite Steel and Precast Plank (topping) 30' span L = 70psf	Plank → 8.8 12.5; Girder → 7.45 / 2; because → beams included and I don't have beams	4.68 6.2; 3.01 / 2	1.009	18.8 $/SF
3) Composite Deck and Composite Beam 30'x30' bay; # 2300	16.20	8.50	1.009	24.9 $/SF
4) Concrete 2-Way Flat Plate 20x15 bay; S_{upp} = 70 psf; # 3400	4.89	8.35	1.009	13.36 $/sf

* Assemblies estimate is not a substitute for unit price estimate. It is designed to be used as an early conceptual estimate for systems to bring project within the owner's budget