New Acute Care Hospital and Skilled Nursing Facility

San Francisco, CA

Technical Report 2

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Executive Summary

This report contains an analysis of four different floor systems for the New Acute Care and Skilled Nursing Facility, including the existing composite deck on composite beam system. The 3 alternative systems studied were:

- Composite deck with castellated beams
- Precast Pre-Stressed Hollow Core Planks
- Concrete Two-Way Flat Plate

The primary means by which these systems were compared was building weight, architectural impact, and serviceability. In addition to this, several other factors were taken into account, such as fire protection, constructability, and cost. The study concluded that both the original composite beams/composite deck and the castellated beam/composite deck system warranted further research. The hollow core plank system was ruled out on the basis that 4' width planks put too many restrictions on the architectural layout. The concrete flat plate system was rejected due to the increase in seismic loads that would occur due to its large self-weight.

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Introduction

The New Acute Care Hospital and Skilled Nursing Facility will serve as an addition to the existing Chinese Hospital located in the historic Chinatown district of San Francisco (See Fig. 1). The site lies on the north flank of Nob Hill, at an elevation of approximately 110' above sea level. Due to the slope of the site, the ground floor of the site is located partially below grade.

Figure 1: Site View of New Acute Care Hospital (blue) located adjacent to existing Chinese Hospital. Photo Courtesy of Google Maps.

This new addition will be connected directly to the existing Chinese Hospital, located at 845 Jackson Street. As part of the construction of this addition, the original portion of the hospital built in 1925 will be demolished. Then the new facility, which has seven stories above ground and one below will be constructed with a hard connection to a previous addition built in 1975. Therefore, the precast concrete panel exterior façade has been designed in a way that respects the 1975 design while providing a more modern look.

At approximately 92,000 SF, this new facility will provide additional patient rooms as well as well several new medical departments to serve the local community. Construction is expected to begin in 2010 and reach completion by Chinese New Year 2013.

Figure 2: Exterior view of New Acute Care Hospital and surrounding buildings

Structure Overview

The structure of the New Acute Care hospital rests on a mat foundation and consists primarily of composite steel decking with steel framing. A perimeter moment frame system is used to resist lateral loading.

Foundation System

According to the geotechnical report provided by Treadwell & Rollo, the soil conditions on the site can be described as "very stiff to hard sandy clay and clay with gravel," which rests on "intensely fractured, low hardness, weak, deeply weathered shale." Because of this, the New Acute Care Facility has been designed to bear on a 36" mat foundation. Columns rest on concrete pedestals, typically sized at 3'-0" x 3'-0". Since the base of

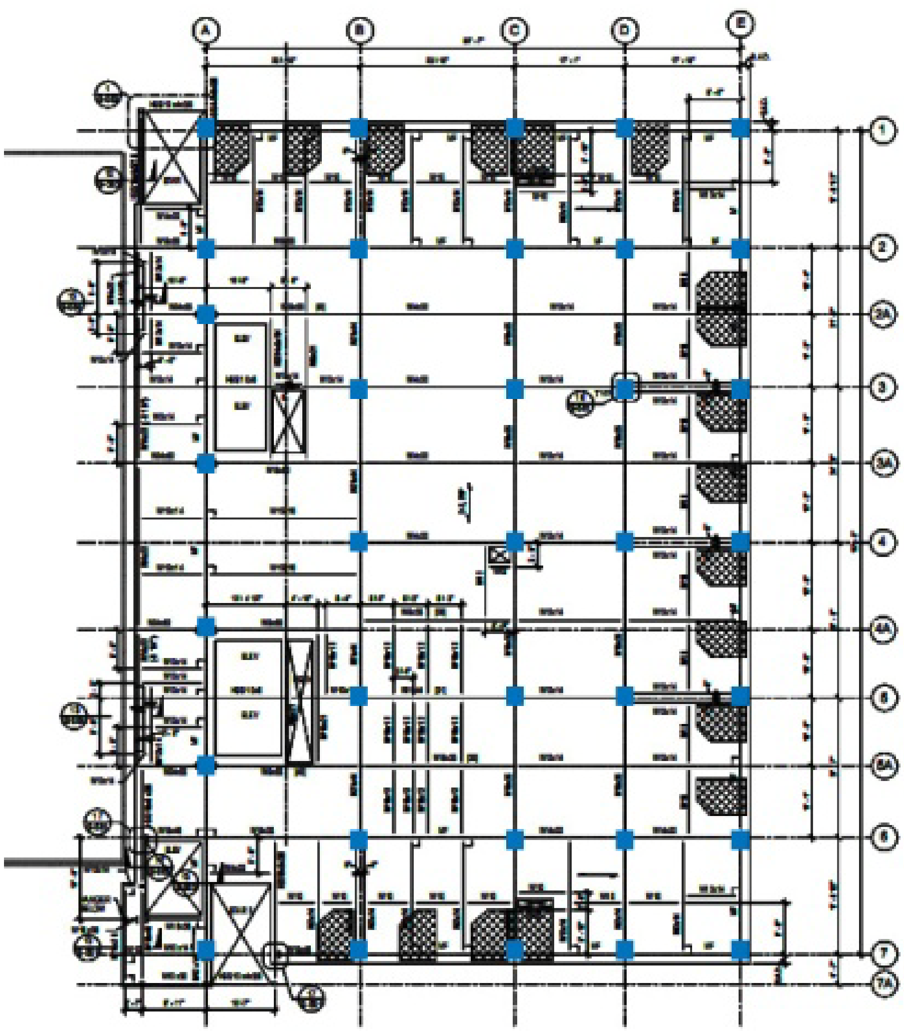

Figure 3: Typical Framing Plan with columns highlighted

the structure will lay below the water table, the foundation was also designed for hydrostatic uplift.

The close proximity to nearby structures, particularly the 1975 addition to the Chinese Hospital, provided a challenge to the designers. Underpinning was used to maintain the foundations of existing structures on either side of the building (see Fig.2).

Framing System

The New Acute Care Hospital uses steel columns (See Figure 3) to support the buildings gravity loads. These columns range in size from W14x445 near the base of the structure to W8x40's near the roof level. As the columns rise vertically through the structure they are spliced together, usually at a distance of 22'-0". Aside from those used in the

Figure 4: Typical Framing Plans with lateral system highlighted in blue

lateral system, most of the columns are connected to beams and girders using pinned connections.

Lateral System

As lateral loads move from through the frame of the structure, they are transferred to a series of special moment frames. These moment frames are used around the perimeter of the structure. As can be seen by the blue highlighting on Figure 4, there are 4 frames running east to west and two frames running north to south. See Figure 18 for a typical moment frame elevation.

Roof System

The roof system is supported in a similar manner to the floors below, with a concrete filled metal deck supported by beams and girders. However, beams at this level are typically spaced much closer together, at a distance of approximately 10-12 feet. The sizes of these roof beams generally vary from W10x12's to W24x104's.

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Materials Used

Concrete

Location	Weight	Strength f'c (ksi)
Foundation	Normal	4000
Drilled Piers	Normal	4000
Slab-on-Grade Walls, Columns, and Piers	Normal	4000
Fill in Metal Deck and Curbs at Ground Floor	Normal	4500
Fill in Metal Deck at First Floor and Above, Topping Slab, Curbs, and Pads	Light	4000
Fill in Stair Pans	Normal	2500
Fill in Over-Excavated Areas and Conduit Encasement	Normal	1500

Structural Steel

Type	Standard	Grade
W-Shapes	ASTM A992	Grade 50
Other Shapes	ASTM A992	Grade 50
Plates for Built-Up Members	ASTM A572	Grade 50
Steel Channels, Angles, Base Plates, Shear Tabs	ASTM A36	Grade 36
Structural Steel Plates	ASTM A572	Grade 50
Steel Bars	ASTM A529	Grade 50
Square or Rectangular Steel Tubes	ASTM A500	Grade B
Round Steel Tubes	ASTM A500	Grade C
Pipe Sections	ASTM A53	Grade B

Reinforcing Steel

	ASTM A615	Grade 60

Applicable Codes

Original Design Codes Used

In addition to the following codes, the California State Government requires that all new government and hospital buildings are approved by the Office of Statewide Health Planning and Development (OSHPD).

- 2007 California Administration Code
  - Part 1, Title 24, CCR
- 2001 California Building Code
  - Part 2, Title 24, CCR
  - (1997 UBC and 2001 CA Amendments)
- 2004 California Electrical Code
  - Part 3, Title 24, CCR
  - (2002 NEC and 2004 CA Amendments)
- 2001 California Fire Code
  - Part 4, Title 24, CCR
  - (2000 UMC and 2001 Amendments)

Design Codes Used in Thesis Analysis

- American Society of Civil Engineers (ASCE)
  - ASCE7-05, Minimum Design Loads for Buildings and Other Structures
- International Building Code, 2006 Edition
- American Institute of Steel Construction (AISC)
  - Steel Construction Manual, Thirteenth Edition (LRFD)
- American Concrete Institute
  - Building Code Requirements for Structural Concrete (ACI 318-08)

Design Loads

Gravity Loads

Live Load (psf)		
Live Load	As Designed	Per ASCE 7
Treatment Rooms	80*+20(partitions)	60
Patient Room	80*+20(partitions)	40
Other Rooms (offices)	80*+20(partitions)	50
Storage Areas		
Fixed Racks	125	125
Mobile Racks	250	250
Corridors	100	80
Mechanical Rooms	125	-
Roof (Mech)	125	100
Roof (Other)	20*	20

The designed live loads were found to be larger than the minimum live loads specified by ASCE7-05. It is likely that these values were higher based on the more stringent requirements of OSHPD as well as the experience of the designers.

Floor Dead Loads	
Material	(psf)
6 1/4" Concrete Deck	50
Finishes	1
MEP and Misc.	20
Total	71

Partition Wall Dead Loads (psf)	
Per ASCE7-05 12.7.2	10

Exterior Wall Dead Loads	
Material	(psf)
5" Concrete Panels	50
6" Metals Studs and Wallboard	0.38
6" Batt Insulation	0.9
Total	51.28

Roof Dead Loads	
Material	(psf)
80 Mil. TPO Roof Membrane	5.5
5/8" Dens Deck	2.5
6 1/4" Concrete Deck	60.4
Total	68.4

Dead load values were determined from a combination of sources including but not limited to ASCE7-05, design aids, and manufacturer specifications

Floor Systems

There were several factors that were considered in the selection of floor systems for further examination. Any viable system must be practical in three ways. First, since seismic concerns are a major issue for this project, the system must not be excessively heavy. Secondly, Figure 5 demonstrates how the architectural layout places tight constraints on the column grid. Therefore, alternate systems must be compatible with the current grid layout or one with *fewer* columns. In addition to this, since the hospital was designed to maintain floor to floor heights with the existing Chinese Hospital, the floor systems must maintain relatively low depths. Lastly, due to the demands of the complicated procedures that will be undergone in this facility, serviceability requirements will be of key importance.

In the design for each of the following floor systems, the dead load was taken as the self-weight of the floor system plus 21psf for MEP, misc dead loads, and finishes. The live load used was 125psf, which can conservatively be taken as the governing LL throughout the building. The effect of lateral loads were not investigated at this point.

Figure 5: Typical Composite Floor Plan Highlighting Column Location

Composite Deck (Existing System)

The New Acute Care hospital makes use of a composite floor system using a 3” Verco W3 Formlock deck with an additional 3 ¼” of concrete resulting in a total thickness of 6 ¼”. This slab then rests on W-shapes ranging from W10x12’s used as beams to sizes as large as W24x207’s which also serve in the buildings lateral system. ¾” Ø shear studs were used to achieve composite action.

There are several different bay sizes used in the New Acute Care Hospital. Larger bay typically exist towards the plan east side of the building while smaller bay sizes are typically used in the western portion of the structure. In most cases, the bays varied from approximately 18’-0”x 17’-0” to 23’-10”x24’-0”.

Spot checks were performed on several beams and girders in the existing design. General speaking, member sizes determined under the loads determined in this report were larger than those used by the designers.

Building Weight

One main advantage of the composite deck system was its ability to keep the building weight low. This was accomplished partially through the use of lightweight concrete in the composite decks. In addition, composite action in the beams drove down the required size of the beams. With dead loads of only 71psf, it is easy to see why this system would have been attractive to the designers.

Architectural Impact

The architectural layout demands a variety of fairly odd dimensioned bay sizes in order for columns to fit around all the various rooms used throughout the structure. Steel beams and girders with a cast-in-place concrete deck gave the designers flexibility to work with these odd dimensions. In addition, column sizes, which were generally only as large as W14’s, allowed them to be easily hidden in corners and in between partition walls.

The New Acute Care hospital was designed to match floor to floor heights with the existing Chinese Hospital next door. This constraint on maximum floor to floor height resulted in a typical floor-to-floor heights 12’-6”. Of that distance, the floor system and MEP equipment took up about 4’, which left only an 8’-6” height in occupied spaces. Therefore, any alternate floor system should aim for a shorter depth, roughly 24.25”, than this existing system.

Serviceability Requirements

Due to the complex operations and activities undertaken in hospital facilities, deflection and vibration were important concerns for the designers. Therefore, live load deflections were restricted to one-half inch for perimeter beams, and three-quarters of an inch for all interior beams and slabs. Vibration was restricted to 8,000 micro-inch per second in operating rooms and 16,000 micro-inches per second everywhere else.

Conclusions and Other Considerations

Advantages:

- Composite action results in small beam size
- 2 hr Fire protection when properly detailed.
- Quick Construction (no formwork or shoring required)
- Cheap construction cost (about $23.00/SF)

Disadvantages:

- Steel conflicts with MEP systems requiring larger floor depths

Although the current floor system could be improved in certain ways, its benefits more than outweigh its problems. The reasons for its selection are clear and it is a good choice for a floor system for this project.

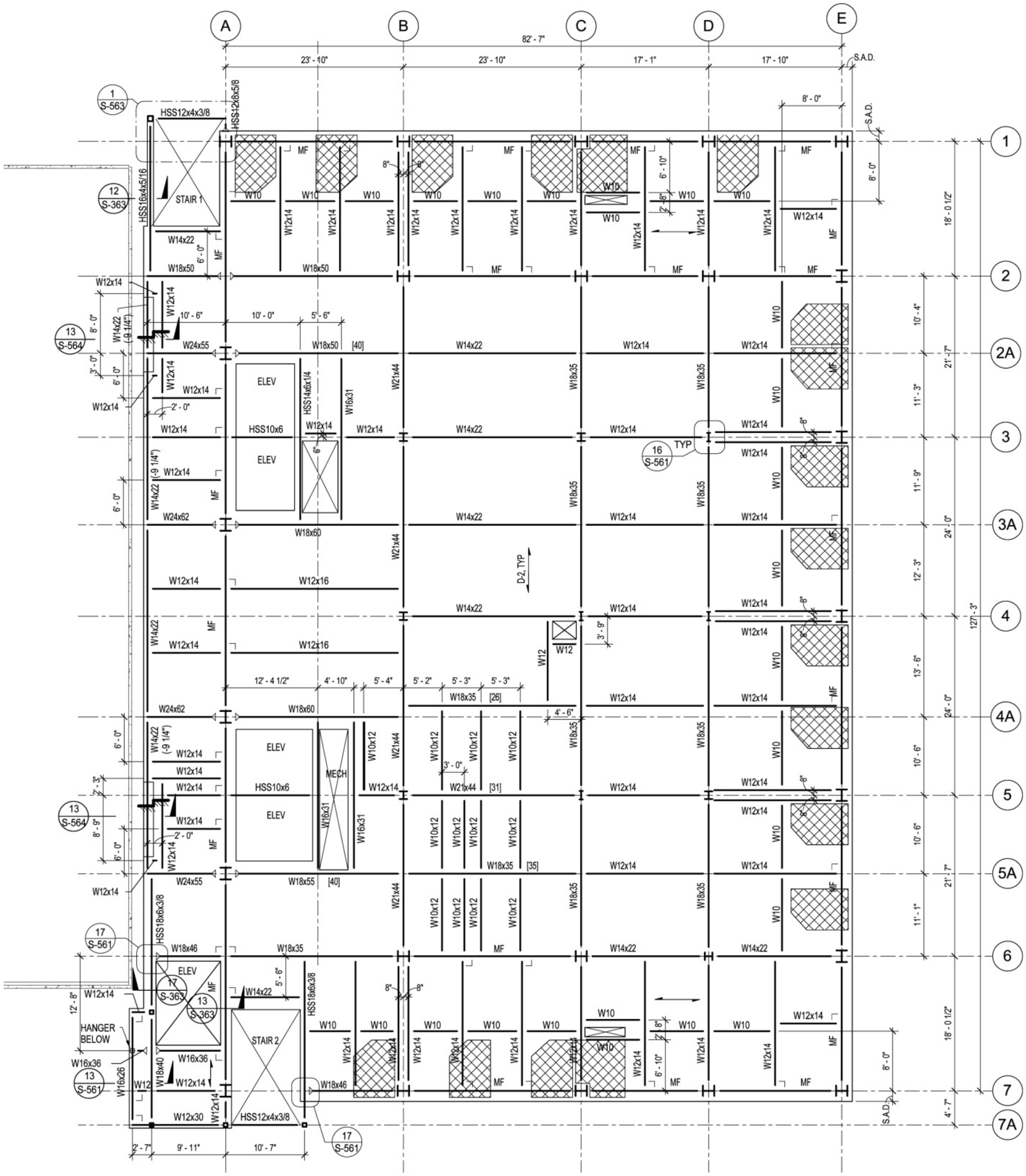


Figure 6: Original Framing Plan

Composite Castellated Beams with Composite Deck

Composite Castellated beams were designed using CMC Steel Products *SmartBeam Composite Castellated Design Program*. A representative 3 bays of the original layout were analyzed, two of which were combined to demonstrate the long-span capabilities of the system. A sample of the program output can be found in the appendix, and output for additional members is readily available upon request. It was assumed that the original composite deck design would work with these new beams.

Building Weight

Composite Castellated beams resulted in a floor system that weighed slightly more than the original system designed. Identical concrete deck materials were used (51 psf), and the castellated beams weighed 20 and 35 plf (compared to 14-35 plf in the original scheme). However, since the possibility exists for select columns to be removed, it is likely that the total building weight would be much less.

Architectural Impact

The existing floor plan would not be affected by the use of castellated beams. Like traditional W shapes, castellated beams give the flexibility to meet any of the existing spans. In fact, several spans were could be combined using these beams, thus freeing up the floor plan even more, provided the remaining column sizes are adjusted to accommodate the additional load.

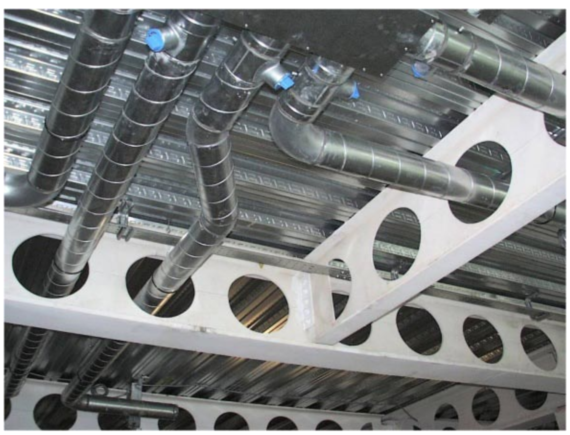


Figure 7: MEP system can be routed through castellations. Photo courtesy of CMC Steel Products

One of the greatest advantages of castellated beams is the floor depth could potentially be dramatically reduced through their use. Since MEP equipment can be routed through the castellation (See Figure 7) in the beams, it is possible that the floor depth could be cut down to as little as 36”, a 1’ reduction over the existing system. Reductions could be even greater if the original bay sizes were used. However, this would likely result in an increase in the labor cost associated with “snaking” the MEP equipment through the castellations.

Serviceability Requirements

In addition to shape selection, the *SmartBeam Composite Design Program* was also used to determine how much each beam would deflect. The maximum live load deflection over the 10 beams designed was 0.486", which is well below the 0.75" maximum required.

Castellated beams have also been found to perform well in terms of vibrations. According to a study performed by Structural Engineers, Inc. on floor vibration using SmartBeam Castellated Beams, "Excellent correlation was found between the measured and predicted natural frequencies of the floor framing….In addition, the overall response of the floors to walking and bouncing was found to be excellent." (See Floor Vibration Testing and Analysis of SMARTBEAM FLOORS).

Conclusions and Other Considerations

Advantages:

- Longer spans and fewer columns
- Improved vibration control
- 2 hr Fire Protection when properly detailed
- Increase in material cost (about $25.00/SF)

Disadvantages:

- Additional lead time due to fabrication
- Effect on lateral system must be evaluated

Castellated beams proved to be an extremely viable alternative to the existing W-shapes used due to the ease with which they could adapt, and even improve the existing column layout. Investigation into these beams will certainly be continued in the future.

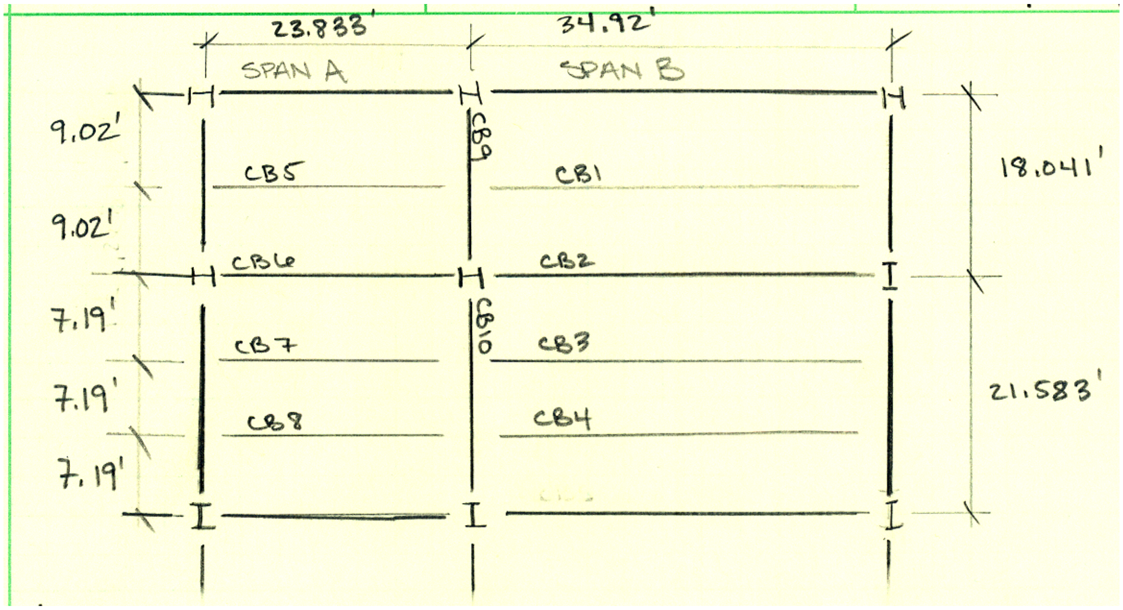


Figure 8: Representative Floor Layout for Castellated Beams

Smartbeam Design Summary

Beam #	Size	# of Studs	Δ_{LL} (in)
CB1	CB27x35	35	0.406
CB2	CB24x31	30	0.486
CB3	CB16x26	28	0.444
CB4	CB16x26	28	0.444
CB5	CB18x22	22	0.251
CB6	CB18x19	20	0.265
CB7	CB15x15/17	22	0.332
CB8	CB15x15/17	22	0.332
CB9	CB30x68	17 - 26	0.057
CB10	CB30x73/83	28-6-28	0.107

Pre-stressed Hollow Core Planks

The pre-stressed hollow core planks used in this proposal were designed using the Nitterhouse Concrete Products *Hollow Core* Brochure. The design table found in Figure 14 was used to determine that a 6" thick 4' wide plank with (4) ½" Ø strands would be most efficient for carrying the required load. Included in the designed dead load is an additional 2" leveling compound which will be used to produce a flatter finished floor.

Building Weight

Hollow Core plank systems were originally selected in an attempt to cut down on material and building weight. However, since they are built using normal weight concrete, they still weight 48.75psf, which is only a 2.5% reduction over the lightweight concrete slab used in the original design. In addition, since composite action was not employed, larger beam sizes were necessary. These new beams were typically twice as heavy per linear fit as the beams used in the original design. With this in mind, it can be concluded that the weight savings based on the use of hollow core precast planks is at best marginal, if not non-existent.

Architectural Impact

Pre-stressed Hollow Planks present several challenges to the architectural layout of the New Acute Care Hospital. The primary problem lies in that the planks are typically produced in four foot widths. The architectural layout puts tight constraints on bay dimensions, and augmenting the scheme to dimensions of 4 foot intervals would require at least some changes to the current layout.

The Hollow Core Plank assembly consists of 6" plank, W shapes as large as W16's, as well as a 2" leveling compound. This would bring the total height of the floor assembly to 24'-2", which is actually an increase in depth from the original system. However, with appropriate detailing, reductions in floor depth could be achieved. If additional angles and stiffener plates are attached to the wide flanges, the planks can be set so that the top of the concrete is in line with the top flange of the beam (see Figure 9). While this detail would bring the overall floor depth to only 16", it would also require additional cost and labor onsite.

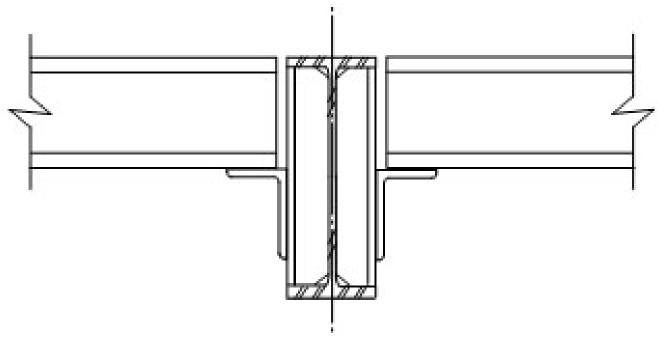


Figure 9: Additional Fabrication Details can be used to reduce floor depth.
Photo courtesy of Modern Steel Construction

Serviceability Requirements

The W-shapes used to support the precast planks were checked for deflection using ACI 318-08 limitations of L/480 for live loads and L/240 for total loads. In all cases studied, the live load deflection met the ACI requirements as well as the 0.75 inches required by the project. Additional studies would have to be performed to determine the deflection and vibration qualities of the planks themselves.

Conclusions and Other Considerations

Conclusion

Advantages:

- Reduction in construction time
- 2 hr Fire Protection when properly detailed
- Low noise transmission
- Consistent and reliable manufacturing process
- Low cost (about $13.00/SF)

Disadvantages:

- Larger W-Shapes
- Column grid adjustment
- Leveling compound necessary since planks are cambered.

While there are several advantages associated with h the use of Precast Pre-stressed Hollow Core Planks, the problems, such as the increase in building weight and the necessary adjustments to the column grid rule them out as a viable alternative system. No further investigation is required.

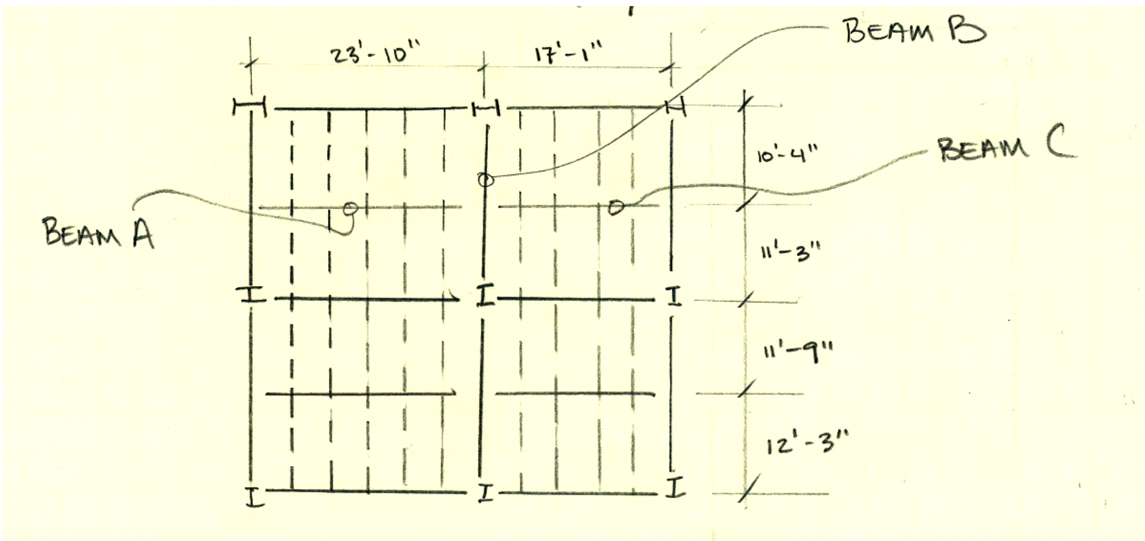


Figure 10: Representative Layout for Hollow Core Planks

Two-Way Flat Plate Concrete Floor

The last alternative system investigated was a two-way flat plate concrete floor system. This system was designed according to the Direct Design Method as laid forth by ACI 318-08. Although all the criterion for using this method were not met (Column offsets were greater than 10%), it was used anyway for the purposes of this preliminary design. It was assumed that the typical column size would be 30”x30” for this design, and that the original column grid would remain unchanged. The design was conducted on 24’-0”x23’-10” bay. Since this bay had the largest dimensions in the building, it was deemed to be the controlling case for the design.

Building Weight

The design of this flat plate system called for a slab thickness of 9”. This resulted in a floor dead load weight of 133.5psf, which is an 88% increase in weight over the original system. In addition to this, the 30”x30” columns used in the design would add a considerable amount of weight to the structure. Due to the nature of the seismic loads on the structure, this criterion alone rules out the use of this system.

Architectural Impact

One of the advantages of a concrete system is that it allows for large range of flexibility and variation in bay sizes and span length. In the case of the New Acute Care Hospital, no change in grid layout was necessary. However, the column sizes used would certainly be larger than those of the original design (typically W14s). These larger columns would result in some disruption of floor layout.

The flat plate system proved to be the best system in terms of floor depth. At a total thickness of only 9”, with no drop panels required for shear, the floor depth could be dramatically reduced. If 1’-6” of additional space were allowed for MEP systems, each floor would gain an additional 1’-9” of usable floor height.

Serviceability Requirements

The maximum deflection found to exist in this system was only 0.474”. This meets both the ACI requirements of L/480 as well as the 0.75” requirement specified by the project. In addition, due to their large mass, concrete systems are known to perform extremely well in terms of vibration.

Conclusion and Other Considerations

Advantages:

- Ease of Construction
- Low floor depth
- Flexibility of partitions
- 3 hr Fire Protection
- Reduction in cost (about $18.00/SF)

Disadvantages:

- Dramatic increase in weight
- Larger columns
- Longer construction time

The dramatic decrease in floor depth achieved through the use of the two-way flat plate, while impressive, was matched by a dramatic increase in floor weight. For this reason, this system will not be investigated further.

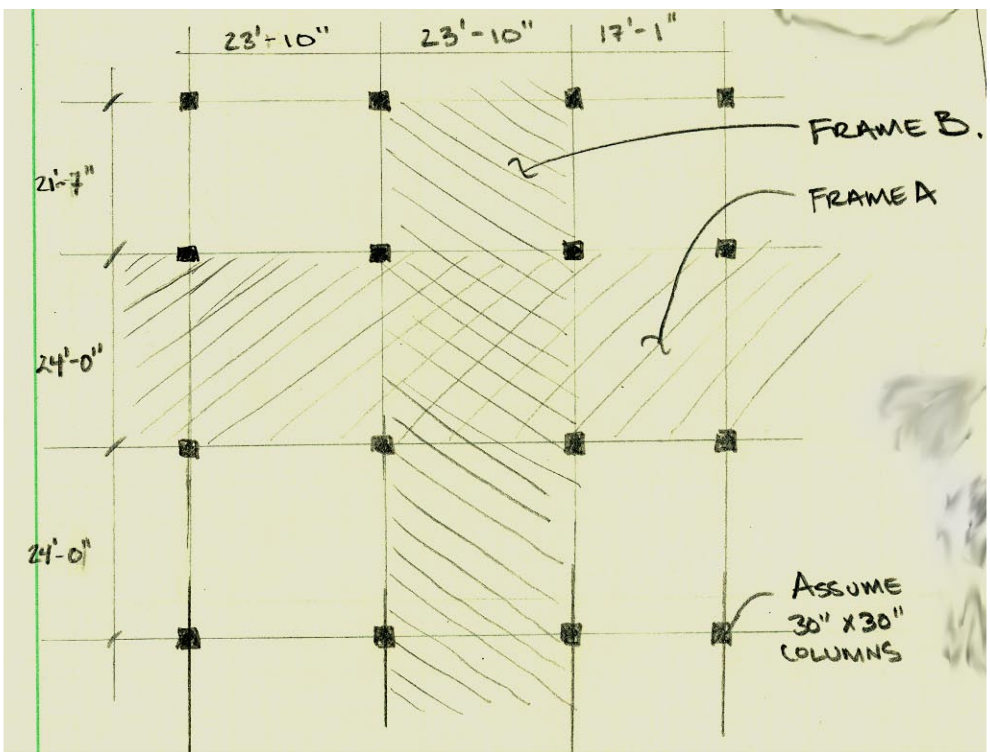


Figure 11: Representative Layout for Two-Way Flat Plate System

Conclusions

The analysis of the alternative floor systems for the New Acute Care Hospital showed that there were many options available to the designers. Each of these options had its own set of advantages and disadvantages. The original system used, a composite deck supported by composite beams, proved to be a logical choice by the designers, as it balanced low weight, compatibility with the architectural layout, and overall economy. Castellated beams, which can potentially be described as the wave of the future, were demonstrated to be a more than viable alternative and will surely be explored more in the future. While precast hollow core planks had plenty of benefits, particularly the ease of construction, it had too restrictive "built in" requirements on span dimensions. Lastly, the concrete flat plate system, which produced an extremely low floor depth, was too heavy to warrant further exploration.

Alternative Floor System Comparison									
Floor System	Weight		Architectural Impact		Deflection	Vibration	Fireprotection	Approximate Cost ($/SF)	Future Investigation
	Deck	Beams	Column Layout	Floor Depth					
Composite Beams with Composite Deck	50	14-31	Good	Good	Good	Good	2 hr	23.00	Yes
Castellated Beams with Composite Deck	50	20-35	Excellent	Excellent	Good	Excellent	2 hr	25.00	Yes
Precast Pre-Stressed Hollow Core Planks	48.75	26-44	Poor	Acceptable	Good	Unknown	2 hr	13.00	No
Flate Plate Concrete	133.5		Good/Poor	Excellent	Excellent	Good	3 hr	18.00	No

Figure 12: Floor System Comparison

APPENDIX

Appendix A: Composite Metal Deck with Steel Framing

JACOBS

Subject: TECH. 1 — Project: ______

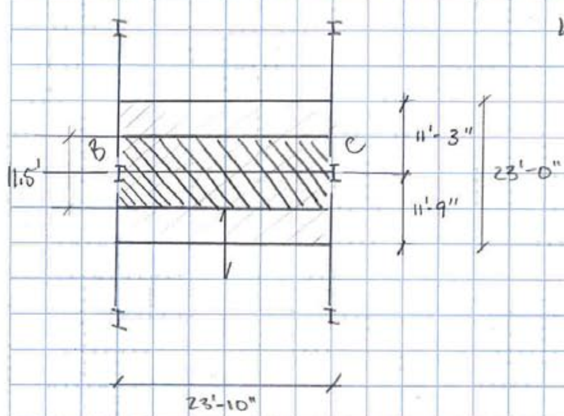
COMPOSITE BEAM SPOT CHECK — Sheet No. 1 Of 3

Authored by: TMA — Date: ______ — Checked by: ______ — Date: ______

SPOT CHECK #1: COMPOSITE STEEL BEAM

LOCATION: 3RD FLOOR

GRID LINE 3 BETWEEN B & C



LOADING

$w_D = 71.0$ psf

$w_L = 50$ OFFICES

LL REDUCTION

$A_T = 23.833' \times (11.5)$

$= 274$ SF $\therefore$ REDUCTION IS ALLOWED

$K_{LL} = 2$ (FOR INTERIOR BEAMS)

$AI = A_T K_{LL} = 2(247) = 548$

$L = L_o\left(0.25 + \frac{15}{\sqrt{AI}}\right) =$

$50\left(0.25 + \frac{15}{\sqrt{548}}\right) = \underline{44.5 \text{ psf}}$

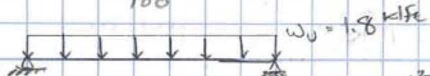
$44.5 > 25 = 0.5L_o$

GOOD

$w_u = 1.2w_D + 1.6w_L$

$= 1.2(71) + 1.6(44.5) = 156.4$ psf

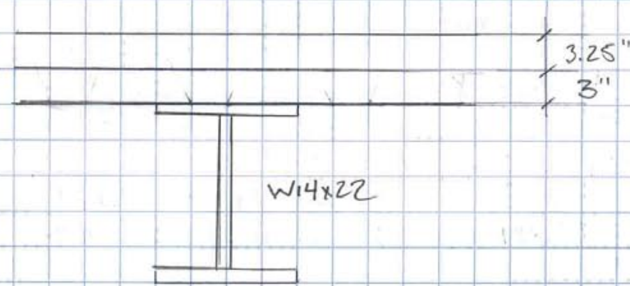
$\frac{156.4(11.5)}{100} = 1.80$ k/ft



$M_u = \frac{wL^2}{8} = \frac{(1.8)(23.833)^2}{8}$

$= 127.8$ ft-k

CHECK DESIGN OF W14x22 w/ 1 SHEAR STUD / FT



DETERMINE EFFECTIVE FLANGE WIDTH

$$b_{eff} \leq \frac{SPAN}{4} = \frac{(23.833')(12''/')}{4} = 71.5'' \Leftarrow \text{CONTROLS}$$

OR

$$\leq \text{DIST. TO ADJ BEAM} = (11.25')(12''/') = 135''$$

CHECK REQUIRED STRENGTH OF BARE STEEL UNDER CONST. DEAD + LIVE LOAD.

$$M_u = 1.2(71) + 1.6(\underset{\text{ASSUME}}{40}) = 149.2^{ft\text{-}k}$$

$$\phi M_p = 125^{ft\text{-}k} < 149.2^{ft\text{-}k}$$

∴ A W14X22 WOULD NOT BE ABLE TO SUPPORT THIS LOAD W/O SOME COMPOSITE ACTION.

A W14X26 ($\phi M_p = 151^{ft\text{-}k}$) WOULD WORK

CHECK DEFLECTION UNDER CONSTRUCTION LOADS FOR W14X26

$$w_D = 40\text{psf}(11.5')/1000$$

$$\Delta_{D.CONST} = \frac{5 w_D L^4}{384 EI} = \frac{5(0.46)(23.833')(1728)}{384(29{,}000)(245)} = 0.47''$$

$$\Delta_{ALLOW} = L/360 = \frac{(23.833' \times 12''/')}{360} = 0.79''$$

$0.47 < 0.79$ GOOD

NOW, CHECK DESIGNED BEAM SIZE UNDER FULL GRAVITY LOADS w/ COMPOSITE ACTION.

ASSUME $\Sigma Q_n = 81.2^{kips}$

$$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{81.2}{0.85(4)(71.5'')} = 0.33''$$

$$Y_2 = 3.25 - \frac{0.33}{2} = 3.085'' \approx 3''$$

→ TABLE 3-19 CONFIRMS THAT A COMPOSITE W14X22 CAN CARRY THE REQUIRED LOAD

PNA #7 $\phi M_n = 174^{ik} > M_u = 127.8^{ik}$

GOOD

JACOBS

Subject: TECH 1 Project:
COMPOSITE BEAM SPOT CHECK Sheet No. 3 of 3
Authored by Date Checked by Date

NOW CHECK THAT THE DESIGNED # OF SHEAR STUDS WORK.

1 3/4" ϕ SHEAR STUDS SPACED @ 1' INTERVALS.
LIGHTWEIGHT CONCRETE ON METAL DECK ⊥ TO BEAM.
↳ $f'_c = 4000^{psi}$

⇒ USING WEAK SHEAR STUD VALUES $Q_n = 17.2^{kip}$

$$\# \text{ OF STUDS REQ'D} = \frac{\Sigma Q_n}{Q_n}(2) = \frac{81.2(2)}{17.2} = 9.44$$

STUDS PROVIDED: 1 PER LF ∴ 23 STUDS > # REQUIRED
GOOD

THEREFORE, THE DESIGNED BEAM (A W14x22 w/ 1 SHEAR STUD/LF) WILL EASILY CARRY THE LOAD.

CHECK DEFLECTION

USING TABLE 3-20 $I_{LB} = 326 \text{ in}^4$

$$\Delta = \frac{5 w_L L^4}{384 E I} = \frac{5\underbrace{(0.575)}_{50\left(\frac{11.5}{1000}\right)}(23.833)^4(1728)}{384(29{,}000)(326)} = 0.44 \text{ in}$$

ACCORDING TO THE PROJECT REQ, $\Delta_{LL.max}$ FOR INTERIOR BEAMS IS 0.75^{in}

$0.44 \text{ in} < 0.75 \text{ in}$.

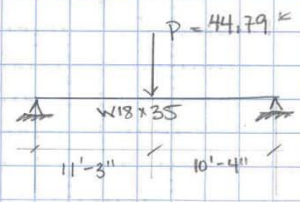
GOOD.

JACOBS

Subject TECH 1 — GIRDER SPOT CHECK | Project | Sheet No. 1 Of 3

Authored by | Date | Checked by | Date

GIRDER SPOT CHECK



$M_{MAX} = 43.98^k (11.25')$

$= 494.8'^k$

$b_{eff} = \frac{21.588'(12''/')}{4} = 64.77''$ CONTROLS

OR

$17.08'(12''/') = 205''$

TABLE 3-19 SHOWS THAT NO W18×35 WILL CARRY THE REQUIRED LOAD.

$P = P_{W14\times22} + P_{W12\times14}$

FIND LOAD FROM W14×22

$w_u = 1.2(71) + 1.6(63.7) = 187.12\left(\frac{11.5}{1000}\right) = 2.15\ ^{k}/'$

GIRDER LIES ON CORRIDOR

$LL = 80\,psf$

$A_I = 753.6\,SF > 400\,SF$ ∴ LL REDUCTION IS ALLOWABLE

$L = 80\left(0.25 + \frac{15}{\sqrt{753}}\right) = 63.7\ psf$

FOR W14×22

$P = \frac{(2.15 \times 23.833)}{2} = 25.6^k$

FOR W12×14

$P = \frac{(2.15 \times 17.08)}{2} = 18.36^k$

$P_{TOTAL} = 43.98$

TRY A W18×46 w/ $\frac{3}{4}''$ SHEAR STUDS SPACED @ 1'

ASSUME $\Sigma Q_n = 310^{KIPS}$

$a = \frac{\Sigma Q_n}{0.85 f'_c b} = \frac{310}{0.85(4)(64.77)} = 1.4$ $Y_2'' = 3.25 - \frac{1.4}{2} = 2.55$

@ PNA BFL, $\phi M_n = 508'^k > M_u$ GOOD.

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GIRDER SPOT CHECK Sheet No. 2 Of 3

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CHECK THAT 1 SHEAR STUD / LF WILL BE SUFFICENT)

- 3/4" ϕ SHEAR STUDS

- LIGHTWEIGHT CONCRETE $f'_c = 4^{ksi}$

USING TABLE 3-19 $Q_n = 18.3$

OF STUDS REQ'D $= \frac{\Sigma Q_n}{Q_n}(2) = \frac{310(2)}{18.3} = 33.4$ STUDS X

THEREFORE, THE NUMBER OF STUDS USED IN THE DESIGN (~22S IS INSUFFICENT.

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Subject TECH 1 Project

GIRDER SPOT CHECK Sheet No. 3 Of 3

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CHECK DEFLECTION

$I_{LB} = 1280 in^4$

$\omega_L = 80_{psf}\left(\frac{20.45'}{1000}\right) = 1.63$

$\Delta_{LL} = \frac{5 \omega_L L^4}{384 E I} = \frac{5(1.63)(21.833)^4(1728)}{384(29,000)(1280)}$

$= 0.22$

$0.22 < 0.75$ GOOD

$\Delta_{ALLOW} = 0.75$

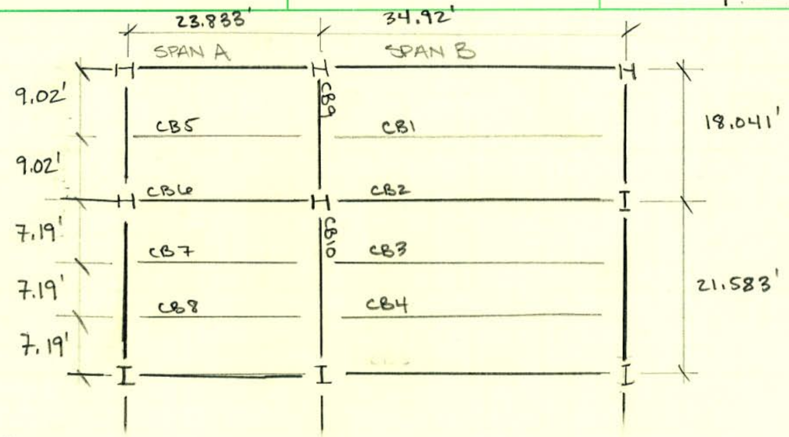
GOOD

TABLE 10–ALLOWABLE SUPERIMPOSED LOADS (psf), DIAPHRAGM SHEAR VALUES, q (plf), AND FLEXIBILITY FACTORS, F, FOR TYPE PLW3™-36 & W3-36 FORMLOK™ DECK PANELS WITH CONCRETE FILL[1,2,3,4,5,6,7,8]–Cont'd

TOTAL SLAB DEPTH & CONC. TYPE	DECK GAGE	NO. OF DECK SPANS	SPAN (ft-in.) 8'-0"	8'-6"	9'-0"	9'-6"	10'-0"	10'-6"	11'-0"	11'-6"	12'-0"	12'-6"	13'-0"	13'-6"	14'-0"	14'-6"	15'-0"
5½" Structural Sand Lightweight (110 pcf)	20	1	313	283	257	235	216	190	149	136	124	114	105	96	88	81	75
		2	313	283	257	235	216	199	184	171	124	114	105	96	88	81	75
		3	313	283	257	235	216	199	184	171	159	148	105	96	88	81	75
		q3	1470	1455	1440	1430	1420	1410	1400	1390	1385	1375	1370	1365	1355	1350	1345
		q4	1590	1565	1540	1520	1500	1485	1470	1455	1440	1430	1420	1410	1400	1390	1380
	19	1	362	327	297	272	249	230	213	184	156	137	126	116	107	99	92
		2	362	327	297	272	249	230	213	197	184	172	126	116	107	99	92
		3	362	327	297	272	249	230	213	197	184	172	161	151	134	99	92
		q3	1490	1470	1455	1440	1425	1415	1400	1390	1385	1375	1365	1360	1350	1345	1340
		q4	1650	1620	1590	1565	1540	1520	1500	1485	1470	1455	1440	1430	1420	1410	1400
	18	1	395	357	325	297	273	251	233	216	189	162	141	130	120	111	103
		2	395	357	325	297	273	251	233	216	201	188	176	164	120	111	103
		3	395	357	325	297	273	251	233	216	201	188	176	164	147	132	103
		q3	1510	1485	1465	1450	1435	1420	1410	1400	1385	1380	1370	1360	1355	1345	1340
		q4	1700	1665	1630	1605	1580	1560	1540	1520	1500	1485	1470	1455	1440	1430	1420
	16	1	400	400	389	356	327	301	279	259	241	225	203	162	150	140	130
		2	400	400	389	356	327	301	279	259	241	225	203	181	162	146	130
		3	400	400	389	356	327	301	279	259	241	225	203	181	162	146	132
		q3	1560	1535	1510	1490	1470	1455	1440	1425	1410	1400	1385	1375	1365	1360	1350
		q4	1830	1785	1740	1705	1670	1645	1620	1595	1570	1550	1530	1515	1500	1485	1470
6¼" Structural Sand Lightweight (110 pcf)	22	1	309	279	213	191	172	155	141	128	116	106	97	88	81	74	68
		2	309	279	254	232	172	155	141	128	116	106	97	88	81	74	68
		3	309	279	254	232	213	196	141	128	116	106	97	88	81	74	68
		q3	1705	1690	1680	1670	1660	1650	1645	1635	1630	1625	1620	1615	1610	1605	1600
		q4	1780	1760	1740	1725	1710	1695	1680	1670	1660	1650	1640	1630	1620	1615	1610
	21	1	341	309	281	256	194	176	160	145	133	121	111	102	93	86	79
		2	341	309	281	256	235	217	160	145	133	121	111	102	93	86	79
		3	341	309	281	256	235	217	201	186	133	121	111	102	93	86	79
		q3	1705	1690	1680	1665	1655	1645	1635	1630	1620	1615	1610	1605	1600	1595	1590
		q4	1810	1790	1770	1750	1730	1715	1700	1685	1670	1660	1650	1640	1630	1620	1610
	20	1	357	323	294	268	246	186	169	154	141	129	118	108	100	92	84
		2	357	323	294	268	246	227	210	154	141	129	118	108	100	92	84
		3	357	323	294	268	246	227	210	195	182	129	118	108	100	92	84
		q3	1710	1695	1680	1665	1655	1645	1635	1630	1620	1615	1605	1600	1595	1590	1585
		q4	1830	1805	1780	1760	1740	1720	1700	1690	1680	1665	1650	1640	1630	1620	1610
	19	1	400	372	338	309	284	262	235	183	168	154	142	131	121	111	103
		2	400	372	338	309	284	262	242	225	209	154	142	131	121	111	103
		3	400	372	338	309	284	262	242	225	209	196	183	131	121	111	103
		q3	1725	1710	1690	1675	1660	1650	1640	1630	1620	1610	1605	1595	1590	1580	1575
		q4	1890	1860	1830	1805	1780	1760	1740	1725	1710	1695	1680	1670	1660	1645	1630
	18	1	400	400	369	338	310	286	264	241	203	172	158	146	135	125	116
		2	400	400	369	338	310	286	264	246	229	214	158	146	135	125	116
		3	400	400	369	338	310	286	264	246	229	214	200	188	176	125	116
		q3	1745	1725	1705	1690	1670	1660	1645	1635	1625	1615	1605	1600	1590	1585	1575
		q4	1940	1905	1870	1845	1820	1795	1770	1750	1730	1715	1700	1690	1680	1665	1650

See Page 28 for footnotes. *(continued)*

Appendix B: Composite Deck on Castellated Beams



LOADING

SUPERIMPOSED DEAD – 71 PSF
SUPERIMPOSED LIVE – 125 PSF

MATERIAL PROPERTIES

CONCRETE STRENGTH ~ $f'_c = 4000$ psi
CONCRETE WEIGHT ~ 120 pcf (REINFORCED LIGHTWEIGHT)

DECK – 3" RIB HEIGHT w/ 3.25" ADDITIONAL CONCRETE,
STUDS – 3/4" ϕ 5" (UNLESS OTHERWISE NOTED)
ASSUME FULL COMPOSITE ACTION.

DETERMINE BEAM SIZES USING "SMARTBEAM: COMPOSITE CASTELLATED DESIGN PROGRAM"

- CB1 : SPAN 34.92' TRIB WIDTH = 9.02'

  LIVE LOAD = (125 PSF)(9.02 FT) = 1127.5 PLF
  DEAD LOAD = (71 PSF)(9.02 FT) = 640.42 PLF

  ∴ USE CB27x35 32 STUDS UNIFORMLY DISTRIBUTED.

- CB2 SPAN: 34.92 TRIB WIDTH = (9.02 + 7.19)/2 = 8.105'

  LIVE LOAD: (125)(8.105) = 1013.13 PLF
  DEAD LOAD: (71)(8.105) = 575.45 PLF

  ∴ USE CB 24x31 30 STUDS UNIFORMLY DISTRIBUTED.

- CB3 & 4 SPAN: 34.92 TRIB WIDTH = 7.19'

  LIVE LOAD: (125)(7.19) = 898.75 PLF
  DEAD LOAD: (71)(7.19) = 510.49 PLF

  ∴ USE CB 16x26 28 STUDS UNIFORMLY DISTRIBUTED

- **CB5** SPAN: 23.833' TRIB WIDTH = 9.02'

  LIVE LOAD: $(125)(9.02) = 1127.5$ PLF
  DEAD LOAD: $(71)(9.02) = 640.42$ PLF

  ∴ USE CB18x22 22 STUDS UNIFORMALLY SPACED

- **CB6** SPAN: 23.833' TRIB WIDTH = 8.105'

  LIVE LOAD = $(125)(8.105) = 1013.13$ PLF
  DEAD LOAD = $(71)(8.105) = 575.45$ PLF

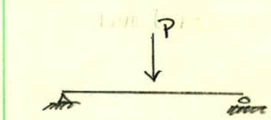
  ∴ USE CB18x19 20 STUDS UNIFORMLY DIST.

- **CB7 & 8** SPAN: 23.833' TRIB WIDTH = 7.19'

  LIVE LOAD = $(125)(7.19) = 898.75$ PLF
  DEAD LOAD = $(71)(7.19) = 510.49$ PLF

  ∴ USE CB15x15/17 22 5/8" ϕ STUDS UNIFORMLY DIST.
  → $\boxed{d_s \leq 2.5 t_f}$

- **CB9** SPAN: 18.041' TRIB. WIDTH: $\frac{23.83+34.92}{2} = 29.37'$



$P_u = [1.2(71)+1.6(125)]\,29.37\,(9.02) = 75554.4^{lbs}$

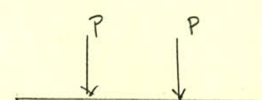
BEAM SELF WEIGHT $= [(22)(23.833) + 35(34.92)]\,\frac{1}{2} = 873.3^{lbs}$

$P = 75554.4 + 873.3 = 76.4^{k}$

PERCENT DL
$\frac{71}{125+75} = 36.22\%$

∴ CB30x68

- **CB10** SPAN: 21.583' TRIB WIDTH = 29.37'



$P_u = [1.2(71)+1.6(125)]\,29.37\,(7.19) = 60225^{lbs}$

BEAM SELF WEIGHT $= (28.5(34.92) + 16(23.833))\,\frac{1}{2} = 688.3^{lbs}$

$P = 60225 + 688.3^{lbs} = 60.9^{lbs}$

∴ USE CB30x73/83 28 | 6 | 28

SmartBeam™

Composite Castellated Design Program

CASTELLATED BEAM INFORMATION		
Job Name	New Acute Care Hospital	
Beam Mark #	CB1	
Span	34.920	ft
Spac. Left	9.020	ft
Spac. Right	9.020	ft
Mat. Strength-Fy	50	ksi
Round Duct Diam.	14.950	in
Duct W x H	8.500 in	14.555 in
Castellated Beam	CB27X35	
Root Beams (T/B)	W18X35	W18X35
d	17.7	17.7
bf	6	6
tf	0.425	0.425
tw	0.3	0.3
CASTELLATION PARAMETERS:		
e	7.000	in
b	5.500	in
dt	4.250	in
S	25.000	in
dg	26.900	in
phi	59.128	deg
ho	18.400	in
wo	18.000	in

LOADING INFORMATION				
Uniform Distributed Loads				
Live Load	1127.5	plf	Pre-comp %	0%
Dead Load	640.42	plf	Pre-comp %	80%
Concentrated Point Loads				
Load # (#)	Magnitude (kips)	Dist from Lft. End (ft)	Percent DL (%)	Percent Pre-Comp.
P1	0.00	0.00	0%	0%
P2	0.00	0.00	0%	0%
P3	0.00	0.00	0%	0%
P4	0.00	0.00	0%	0%
COMPOSITE INFORMATION				
Concrete & Deck:			Shear Studs:	
conc. strength - fc' (psi)		4000	stud dia. (in)	3/4"
conc. wt. - wc (pcf)		120	stud ht. (in)	5
conc. above deck - tc (in)		3 1/4	studs per rib	1
rib height - hr (in)		3	composite %	100%
rib width - wr (in)		6	Stud Spacing:	
			N=32,Uniformly Dist.	

RESULTS			WARNINGS
Failure Mode	Interaction	Status	
Bending	0.884	<=1.0 OK!!	
Web Post	0.847	<=1.0 OK!!	
Shear	0.823	<=1.0 OK!!	
Concrete	0.251	<=1.0 OK!!	
Pre-Comp.	0.448	<=1.0 OK!!	
Overall	0.884	<=1.0 OK!!	
Pre-Composite Deflec.	0.565"	=L/742	
Live Load Deflection	0.406"	=L/1031	

EXPAND'D. SXN. PROP'S		
Avg. wt.	35.0	plf
Anet	7.472	in^2
Agross	12.992	in^2
Ix net	1192.93	in^4
Ix gross	1348.67	in^4
Sx net	88.69	in^3
Sx gross	100.27	in^3
rx min	10.19	in
Iy	15.33	in^4
Sy	5.11	in^3
COMPOSITE SXN. PROP'S		
n	11.03	
beffec.	104.76	in
Actr	30.866	in^2
N.A. ht.	28.31	In Deck
Itr	3201.89	in^4
Ieffec.	3201.89	in^3
Sxconc	662.06	in^3
Sxsteel	113.09	in^3
CONSTRUCTION BRIDGING		
End Connection type		Double clip
Min. No. Of Bridging Rows		0
Max. Bridging. Spacing (ft)		36

CMC Steel Products

10/20/2009

Figure 13: SmartBeam Design Program Sample Output

Appendix C: Hollow Core Plank Analysis

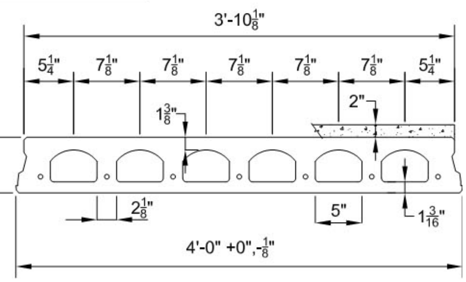
Prestressed Concrete
6"x4'-0" Hollow Core Plank

2 Hour Fire Resistance Rating With 2" Topping

PHYSICAL PROPERTIES
Composite Section

A_c = 253 in.^{2}	Precast b_w = 16.13 in.
I_c = 1519 in.^{4}	Precast S_{bcp} = 370 in.^{3}
Y_{bcp} = 4.10 in.	Topping S_{tct} = 551 in.^{3}
Y_{tcp} = 1.90 in.	Precast S_{tcp} = 799 in.^{3}
Y_{tct} = 3.90 in.	Precast Wt. = 195 PLF
	Precast Wt. = 48.75 PSF

DESIGN DATA



1. Precast Strength @ 28 days = 6000 PSI
2. Precast Strength @ release = 3500 PSI
3. Precast Density = 150 PCF
4. Strand = 1/2"Ø 270K Lo-Relaxation.
5. Strand Height = 1.75 in.
6. Ultimate moment capacity (when fully developed)...
   4-1/2"Ø, 270K = 67.4 k-ft at 60% jacking force
   6-1/2"Ø, 270K = 92.6 k-ft at 60% jacking force
   7-1/2"Ø, 270K = 95.3 k-ft at 60% jacking force
7. Maximum bottom tensile stress is $10\sqrt{f'c}$ = 775 PSI
8. All superimposed load is treated as live load in the strength analysis of flexure and shear.
9. Flexural strength capacity is based on stress/strain strand relationships.
10. Deflection limits were not considered when determining allowable loads in this table.
11. Topping Strength @ 28 days = 3000 PSI. Topping Weight = 25 PSF.
12. These tables are based upon the topping having a uniform 2" thickness over the entire span. A lesser thickness might occur if camber is not taken into account during design, thus reducing the load capacity.
13. Load values to the left of the solid line are controlled by ultimate shear strength.
14. Load values to the right are controlled by ultimate flexural strength or fire endurance limits.
15. Load values may be different for IBC 2000 & ACI 318-99. Load tables are available upon request.
16. Camber is inherent in all prestressed hollow core slabs and is a function of the amount of eccentric prestressing force needed to carry the superimposed design loads along with a number of other variables. Because prediction of camber is based on empirical formulas it is at best an estimate, with the actual camber usually higher than calculated values.

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

Strand Pattern		SPAN (FEET)																		
		12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30
4 - 1/2"ø	LOAD (PSF)	349	317	290	258	227	197	174	149	127	108	92	78	66	55					
6- 1/2"ø	LOAD (PSF)	524	478	437	377	334	292	269	237	215	188	165	142	122	104	88	73	61	49	39
7 - 1/2"ø	LOAD (PSF)	541	492	451	416	364	331	293	274	242	214	190	167	144	124	107	91	77	64	53

NITTERHOUSE
CONCRETE PRODUCTS

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

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

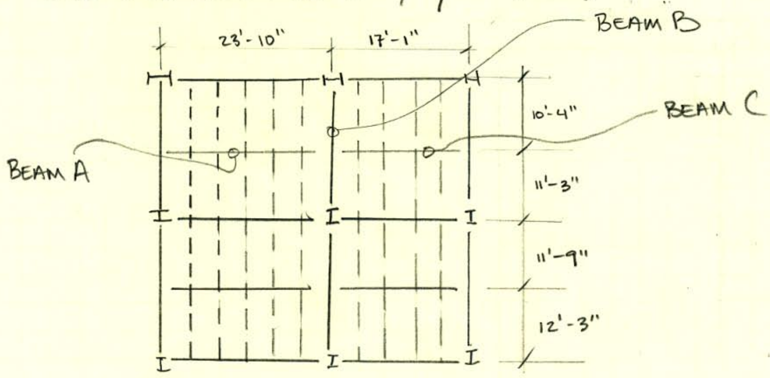
11/03/08

6F2.0T

Figure 14: Nitterhouse Concrete Product Plank Information

PRESTRESSED HOLLOW CORE PLANK

- USE 6"x4'-0" HOLLOW CORE PLANK
  2 HR FIRE RESISTANCE RATING w/ 2" TOPPING



LOADS

DEAD LOADS: 21 PSF (SUPERIMPOSED)
48.75 PSF (PLANK SELF-WEIGHT)

LIVE LOADS: 125 PSF (MAX ASCE 7-05)

SERVICE LOADS

$1.2D + 1.6L = 1.2(21) + 1.6(125) = 225.2$ PSF.

225.2 PSF < 317 PSF FROM TABLE, S = 13' CONSERVATIVE

GOOD

$w_U = 1.2(21 + 48.75) + 1.6(125) = 283.7$ PSF

REDESIGNED BEAM & GIRDER SIZES

DEFLECTION LIMITS

$\Delta_{LL} = L/480$ $\Delta_{TL} = L/240$

BEAM A TRIB WIDTH $= \frac{10'\text{-}4" + 11'3"}{2} = 10.8'$

$\Delta_{LL} = \frac{5(.125(10.38'))(23.83)^4(1728)}{384(29000)(I_{REQ})} = \frac{(23.83)(12)}{480} \Rightarrow I_{REQ} = 567.17$

0.59" < 0.75"

$\Delta_{TL} = \frac{5(.2837)(10.8)(23.83)^4(1728)}{384(29000)(I_{REQ})} = \frac{(23.83)(12)}{240.} \Rightarrow I_{REQ} = 643.63$

1.19"

$M_U = \frac{(.2837)(10.8)(23.83)^2}{8} = 207.5'^{\text{kips}}$

CHOOSE $\boxed{W21 \times 44}$ $I = 843\ in^4 > 613.83\ in^4$
$\phi M_n = 358^{k\text{-}ft} > 207.5\ ft\text{-}kips$

COMPARE w/ A W14x22.

BEAM C TRIB WIDTH = 10.8 LENGTH = 17'-1"

$$\Delta_{LL} = \frac{5(.125)(10.8)(17.083)^4(1728)}{384(29000)\,I_{REQ}} = \frac{(17.083)(12)}{480} \quad I_{REQ} = 208.88\ in^4$$

0.42" < 0.75"

$$\Delta_{TL} = \frac{5(0.2837)(10.8)(17.083)^4(1728)}{384(29000)\,I_{REQ}} = \frac{(17.083)(12)}{480} \quad I_{REQ} = 237.04\ in^4$$

$$M_U = \frac{(0.287)(10.8)(17.08)^2}{8} = 111.77^{ft\text{-}kips}$$

CHOOSE $\boxed{W14 \times 26}$ $I = 245\ in^4 > 237\ in^4$
$\phi M_p = 151^{'k} > 111.77^{'k}$

COMPARE w/ W12x14.

BEAM E TRIB WIDTH = 10.8 L = 17.83'

$$\Delta_{LL} = \frac{5(.125)(10.8)(17.83)^4(1728)}{384(29000)\,I_{REQ}} = \frac{(17.83)(12)}{480} \quad I_{REQ} = 237.6\ in^4$$

0.46" < 0.75"

$$\Delta_{TL} = \frac{5(.2837)(10.8)(17.83)^4(1728)}{384(29000)\,I_{REQ}} = \frac{17.83(12)}{240} \quad I_{REQ} = 269.6\ in^4$$

$$M_U = \frac{(.287)(10.8)(17.83)^2}{8} = 121.79^{ft\text{-}k}$$

CHOOSE $\boxed{W16 \times 26}$ $I = 301\ in^4 > 269.6\ in^4$

$\phi M_p = 166^{ft\text{-}k} > 121.79^{ft\text{-}k}$

COMPARE w/ W12x14.

GIRDER B - TRIB WIDTH $= \frac{(23.833 + 17.083)}{2} = 20.46\ ft$

TOTAL LOAD: $\frac{283.7(10.8)(20.46)}{2} = 31.3^k$ @ MIDSPAN.

LIVE LOAD: $\frac{125(10.8)(20.46)}{2} = 13.8^k$ @ MIDSPAN.

$$\Delta_{LL} = \frac{13.8(21.58)^3 1728}{48(29000)\, I_{REQ.}} = \frac{21.58(12)}{480} \Rightarrow I_{REQ} = 319.4\, in^4$$

$0.54'' < 0.75''$

$$\Delta_{TL} = \frac{31.3(21.58)^3 1728}{48(29000)\, I_{REQ.}} = \frac{21.58(12)}{240} \Rightarrow I_{REQ} = 362.4\, in^4$$

$$M_U = \frac{(31.3)(21.58)}{4} = 169.1^{ft\cdot k}$$

CHOOSE $\boxed{W16 \times 31}$

$I = 375\, in^4 > 362.4\, in^4$
$\phi M_p = 203^{ft\cdot k} > 169.1^{ft\cdot k}$

COMPARE w/ W18 x 35

MAX FLOOR SYSTEM THICKNESS

- 16" (W-SHAPE)
- 6" (PRECAST PLANK)
- 2" (LEVELING TOPPING)

24" = 2'-0"

?

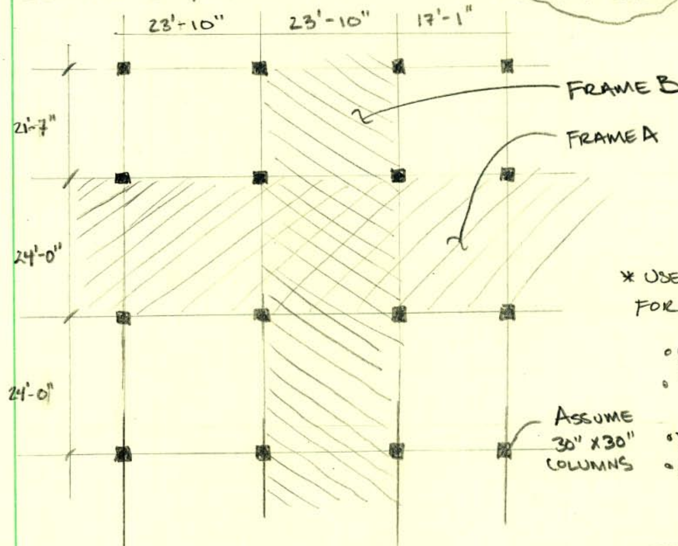
COMPARE w/ 18" W-SHAPE
6.25" CONCRETE DECK
24.25"

Appendix D: Concrete Flat Plate

- FLATE SLAB CONCRETE FLOOR SYSTEM DESIGN

DESIGN USING ACI 318.08

CRITICAL BAY = 23'-10" × 24'-0" ⇒ SAY 24'×24' TO SIMP. CALCS



CRITERIA FOR DIRECT-DESIGN METHOD § 13.6.1

- 3 SPANS PER DIRECTION ✓
- RECT. $\left(\frac{LONG}{SHORT} \le 2\right)$ ✓
- SUCCESSIVE SPAN LENGTHS DIFF ≤ 1/3 ✓
- COLUMN OFFSET ≤ 10% ✗
- LL ≤ 2DL ✓

* USE DD METHOD ANYWAY FOR PRELIMINARY DESIGN!

- USE NWC $f'_c = 4000$ psi
- $f_y = 60{,}000$ psi
- DL = SELFWEIGHT + 21 PSF
- LL = 125 PSF

MIN SLAB THICKNESS $\boxed{\text{ACI 318.08 TABLE 9.5c}}$

$$t = \frac{l_n}{33} = \frac{24 \times 12}{33} = 8.72" \Rightarrow \underline{9" \text{ SLAB}}$$

$$\therefore DL = 150(9/12) + 21 = 133.5 \text{ PSF}$$

$$1.2D + 1.6L = 1.2(133.5) + 1.6(125) = \underline{0.36 \text{ KSF}}$$

COLUMN STRIP WIDTH $\boxed{\S 13.2.1}$

$$0.25 l_2 = 0.25(24') = 6'$$
OR
$$0.25 l_1 = 0.25(24') = 6'$$

$\therefore$ <u>12' COLUMN STRIP</u>

$$M_o = \frac{q_u l_2 l_n^2}{8} = \frac{(.36)(21.6)(24)^2}{8} = 559.8 \approx \underline{560^k}$$

l_2 = AVG OF ADJ. SPANS $= \frac{2(23.833) + 17.83}{3} = 21.6'$

$\alpha = 0$

$I_b = 0$ SINCE NO INTERIOR BEAMS

<u>IN FRAME A</u> $\boxed{\S 13.6.3.2}$

M^+

M^- M^-

$M^- = 0.65 \quad M_o = 364'^k$

$M^+ = 0.35 \quad M_o = 196'^k$

transverse $\frac{l_2}{l_1} = \frac{24}{23.833} = 1.007 \approx 1.0$

USING $\boxed{\S 13.6.4.1}$

$\alpha \frac{l_2}{l_1} = 0$

$\therefore$ COLUMN STRIPS MUST SUPPORT 75% INTERIOR NEG. MOMENTS

75% OF M^- TO CS => $273'^k$
25% OF M^- TO MS => $91'^k$

USING $\boxed{\S 13.6.4.4}$ CS MUST CARRY 60% INTERIOR POSITIVE MOMENT

60% OF M^+ TO CS => $117.5'^k$
40% OF M^+ TO MS => $78.5'^k$

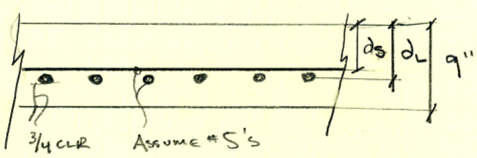
SUMMARY

TOTAL MOMENT	M^-	M^+	M^-	WIDTH
CS	$273'^k$	$117.5'^k$	$273'^k$	$12' \rightarrow 144''$
MS	$91'^k$	$78.5'^k$	$91'^k$	$12' \rightarrow 144''$

DESIGN
SLAB REINFORCEMENT

Max Spacing = $2t = 18''$ $\boxed{\S 13.3.2}$

Min Steel => $A_{s\,min} = 0.0018bt = 0.0018(144)(9) = 2.33\ in^2$
(TEMP & SHRINKAGE) for $f_y = 60$ ksi $\boxed{\S 7.12.2.1}$



$d_{SHORT} = 9'' - \frac{3}{4}'' - \frac{1}{2}(0.625) = 7.94''$

$d_{LONG} = 9'' - \frac{3}{4} - 0.625 - \frac{1}{2}(.625) = 7.31''$ (clr)

Design of CS reinforcement				
Item	Description	Interior Span		
		Left	Mid	Right
1	Mu ('k)	273	117.5	273
2	CS width (in) "b"	144	144	144
3	eff depth "d" (in)	7.31	7.31	7.31
4	Mn=Mu/phi	303.3333	130.5556	303.3333
5	Mn/b	25.27778	10.87963	25.27778
6	R=(Mn*12000)/(bd^2)	473.0468	203.6007	473.0468
7	p (from A.5a)	0.0085	0.0035	0.0085
8	As=pbd	8.94744	3.68424	8.94744
9	Asmin=.0018bt	2.3328	2.3328	2.3328
10	N=larger of (8,9)/A1s	29	12	29
11	Nmin=width strip/2t	8	8	8

p_{max}	0.0206
fy	60000
f'c	4000
d_{min}	5.001104
t	9
Rein. Area	0.31

CHECK d_{min} $\rho_{max} = 0.0206$ $\boxed{\text{NDD TABLE A.4}}$

$$d_{min} = \sqrt{\frac{M_n(1000)(12")}{\rho f_y b (1 - 0.59 \rho f_y / f'_c)}} = \sqrt{\frac{303.3(1000)(12)}{.0206(60{,}000)\left(1 - .59\left(\frac{.0206 \cdot 60{,}000}{3000}\right)\right)}}$$

$$= 5.2" < 7.31" \quad \underline{\text{GOOD}}$$

SUMMARY

M_{cs}^- = (29) #5 @ 5" SPACING
M_{cs}^+ = (12) #5 @ 12" SPACING
M_{cs}^- = (29) #5 @ 5" SPACING

Design of MS reinforcement

Item	Description	Interior Span		
		Left	Mid	Right
1	Mu ('k)	91	78.5	91
2	CS width (in) "b"	144	144	144
3	eff depth "d" (in)	7.31	7.31	7.31
4	Mn=Mu/phi	101.1111	87.22222	101.1111
5	Mn/b	8.425926	7.268519	8.425926
6	R=(Mn*12000)/(bd^2)	157.6823	136.0226	157.6823
7	p (from A.5a)	0.003	0.0025	0.003
8	As=pbd	3.15792	2.6316	3.15792
9	Asmin=.0018bt	2.3328	2.3328	2.3328
10	N=larger of (8,9)/A1s	11	9	11
11	Nmin=width strip/2t	8	8	8

p_{max}	0.0206
fy	60000
f'c	4000
d_{min}	2.887389
t	9
Rein. Area	0.31

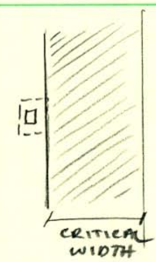
SUMMARY

M_{ms}^- = (11) #5's @ 13" SPACING
M_{cs}^+ = (9) #5's @ 16" SPACING
M_{ms}^- = (11) #5's @ 13" SPACING

SINCE DESIGN IS BASED ON A SQUARE BAY, THIS DESIGN WILL BE USED IN BOTH DIRECTIONS.

CHECK SHEAR

- WIDE BEAM ACTION
  - CRITICAL SECTION IS "d" FROM COLUMN FACE

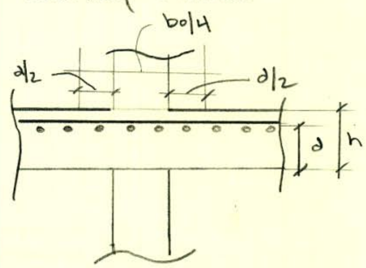


$$\frac{24'}{2} - \frac{7.31''}{12''/'} = 11.39' \text{ (CRITICAL SECTION WIDTH)}$$

- $w_u = 0.36^{ksf}$
- $V_u = w_u \cdot \text{AREA} = (0.36)(11.39')(23.833') = 97.7^k$
- $V_n = 2\sqrt{f'_c}\, bd = 2\sqrt{4000}\,(23.833')(7.31)(12''/') = 264^k$

$$\phi V_n = 0.75(264) = 198^k > V_u \quad \underline{\text{GOOD}}$$

- PUNCHING SHEAR



V_c IS SMALLEST OF $\boxed{\S 11.11.2}$

$= \boxed{4}\sqrt{f'_c}\, b_o d \implies$ GOVERNS

$= \boxed{\left(\frac{\alpha_s d}{b_o} + 2\right)}\sqrt{f'_c}\, b_o d \rightarrow \frac{40(7.93)}{151.75} + 2 = \boxed{4.09}$ ($\alpha_s = 40$ FOR INT. COLUMNS)

$= \boxed{\left(2 + \frac{4}{\beta}\right)}\sqrt{f'_c}\, b_o d \rightarrow 2 + \frac{4}{1} = \boxed{6}$ ($\beta = 1$ FOR)

$d = 9 - 3/4 - 1/2(0.625)$
$= 7.937$

$\frac{d}{2} = \frac{7.937}{2} = 3.96''$

$\frac{b_o}{4} = 30 + 2\left(\frac{d}{2}\right) = 37.93$

$b_o = 151.75''$

$V_c = 4\sqrt{4000}\,(151.75)(7.937) = \underline{304.8^k}$

$\phi V_c = 0.75(304.8) = \underline{228.6^k}$

$V_u = (0.36)\left[(23.833)(24) - \left(\frac{30''}{12''/'}\right)^2\right]$

$= 203.67^k < \phi V_c$ ∴ NO ADDITIONAL SHEAR REIN. NEEDED.

CHECK DEFLECTION

• ELASTIC DEFLECTION DUE TO SELFWEIGHT

$\Delta_{f,ref} = \frac{wl^4}{384 E I_{FRAME}}$

$E = 57000\sqrt{f'c} = 57000\sqrt{4000} = 3.6 \times 10^6$

$\Delta_{24,ref} = \frac{112.5(24')(24')^4(1728)}{384(3.6\times10^6)(17,496)}$

$I_{24} = \frac{(24)(12'')(9'')^3}{12} = 17,496 \text{ in}^4$

BOTH DIRECTIONS $= 0.064''$

SELF-WEIGHT $= (9/12)(150) = 112.5$ PCF

$\Delta_{f,col} = \Delta_{f,ref} \cdot \frac{M_{COL}}{M_{FRAME}} \cdot \frac{E_c I_{FRAME}}{E_c I_{COL}}$

$\Delta_{f,mid} = \Delta_{f,ref} \frac{M_{MID}}{M_{FRAME}} \cdot \frac{E_c I_{FRAME}}{E_c I_{MID}}$

FOR FRAME A

$I_{cs} = \frac{144(9'')^3}{12} = 8748 = I_{ms}$

$\frac{M_{COL}}{M_{FRAME}}$ 68% TO CS $(M^+ \,\&\, M^-)$

$\frac{M_{MID}}{M_{FRAM}}$ 32% TO MS $(M^+ \,\&\, M^-)$

$\Delta_{f,col} = (0.064)(0.68)\left(\frac{17496}{8748}\right) = 0.087''$

$\Delta_{f,mid} = (0.064)(0.32)\left(\frac{17496}{8748}\right) = 0.041''$

IN BOTH DIRECTIONS
ASSUMING SUPPORT ROTATIONS ARE NEGLIGIBLE.

• IMMEDIATE DEFLECTIONS DUE TO DL

$\Delta_{SHORT} = \Delta_{f,mid} + \Delta_{f,col} = 0.087 + 0.041 = \underline{0.128''}$

• DEFLECTION AFTER LONG PERIOD OF TIME

$\Delta_{LONG\ TERM} = 3\,\Delta_{SHORT} = 3(0.128) = \underline{0.384''}$

• IMMEDIATE DEFLECTION DUE TO 3/4 LL

$\Delta_{LL(3/4)} = 0.128\left(\frac{3}{4}\right)\left(\frac{125}{133.5}\right) = \underline{0.089''}$

$\frac{l}{480} = \frac{24'(12''/1')}{480} = 0.6''$

$\boxed{\text{ACI TABLE 9.5b}}$

$\Delta_{max} = \Delta_{LL} + 3\Delta_{DL} = 0.089'' + 0.384'' = 0.474 < 6''$

Appendix E: References

Alwood, T. (September 2007). Let's Be Plank... *Modern Steel Construction* .

Nitterhouse Concrete Products. (n.d.). Hollow Core Brochure.

CMC Steel Products. (2006). *Welcome to Long-Span Steel Solutions*. Retrieved October 21, 2009, from www.cmcsteelproducts.com.

Structural Engineers, Inc. (2000). *Floor Vibration Testing and Analysis of SMARTBEAM FLOORS.*

Appendix F: Plans

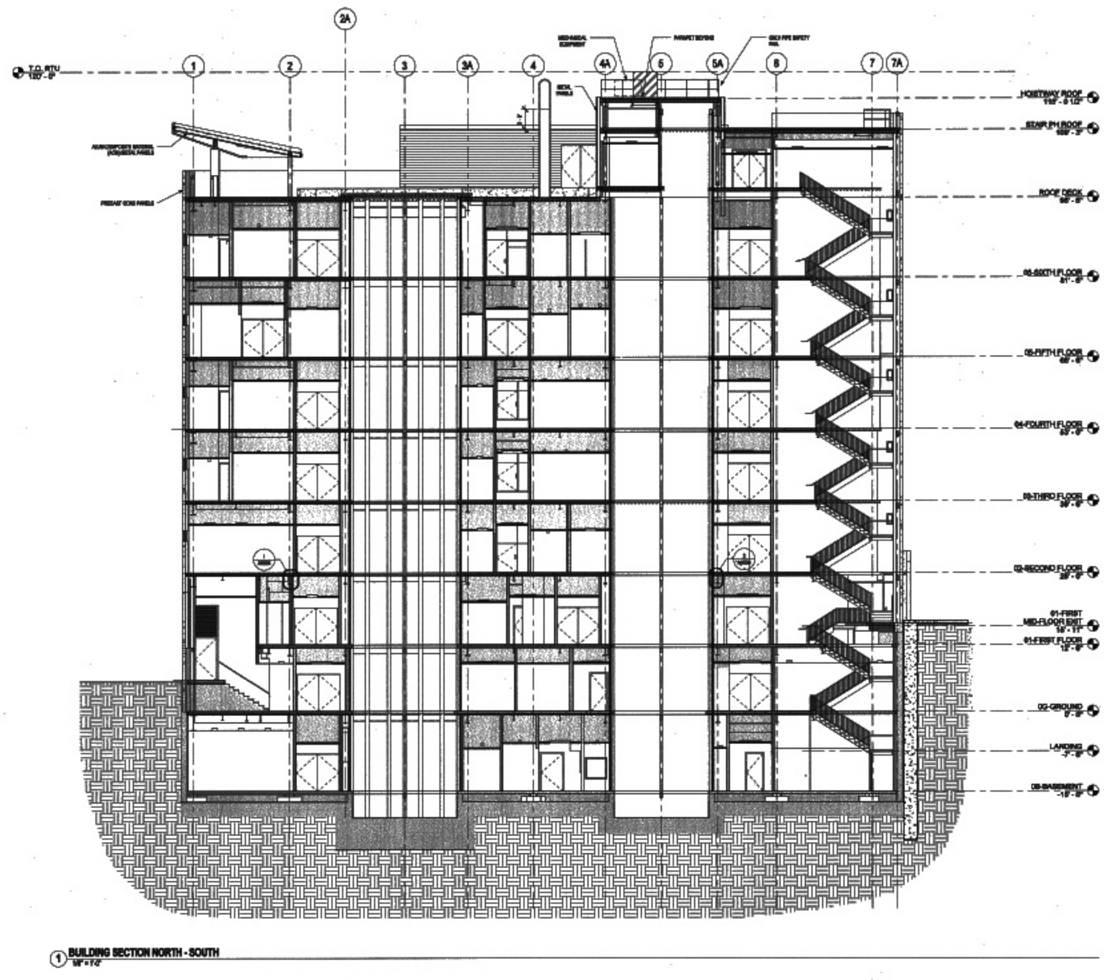


Figure 15: NS Buiding Section

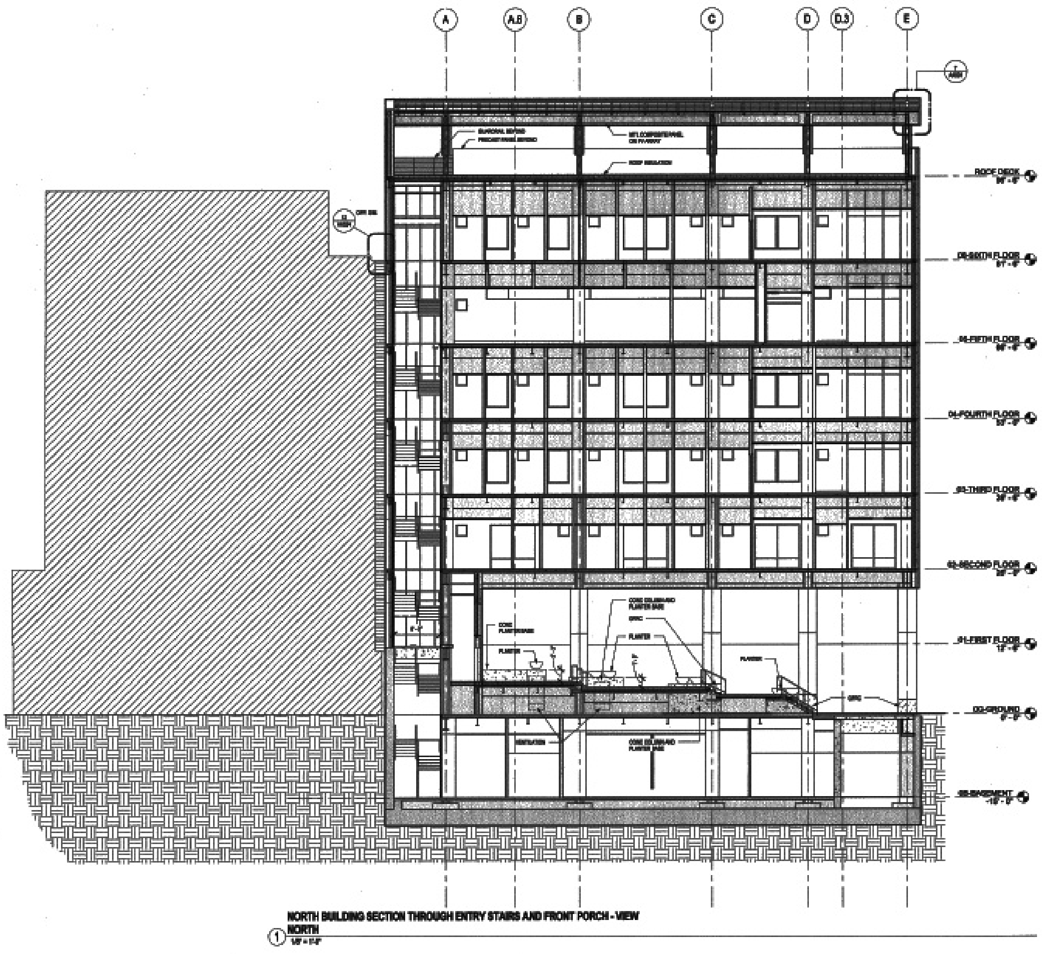


Figure 16: EW Building Section

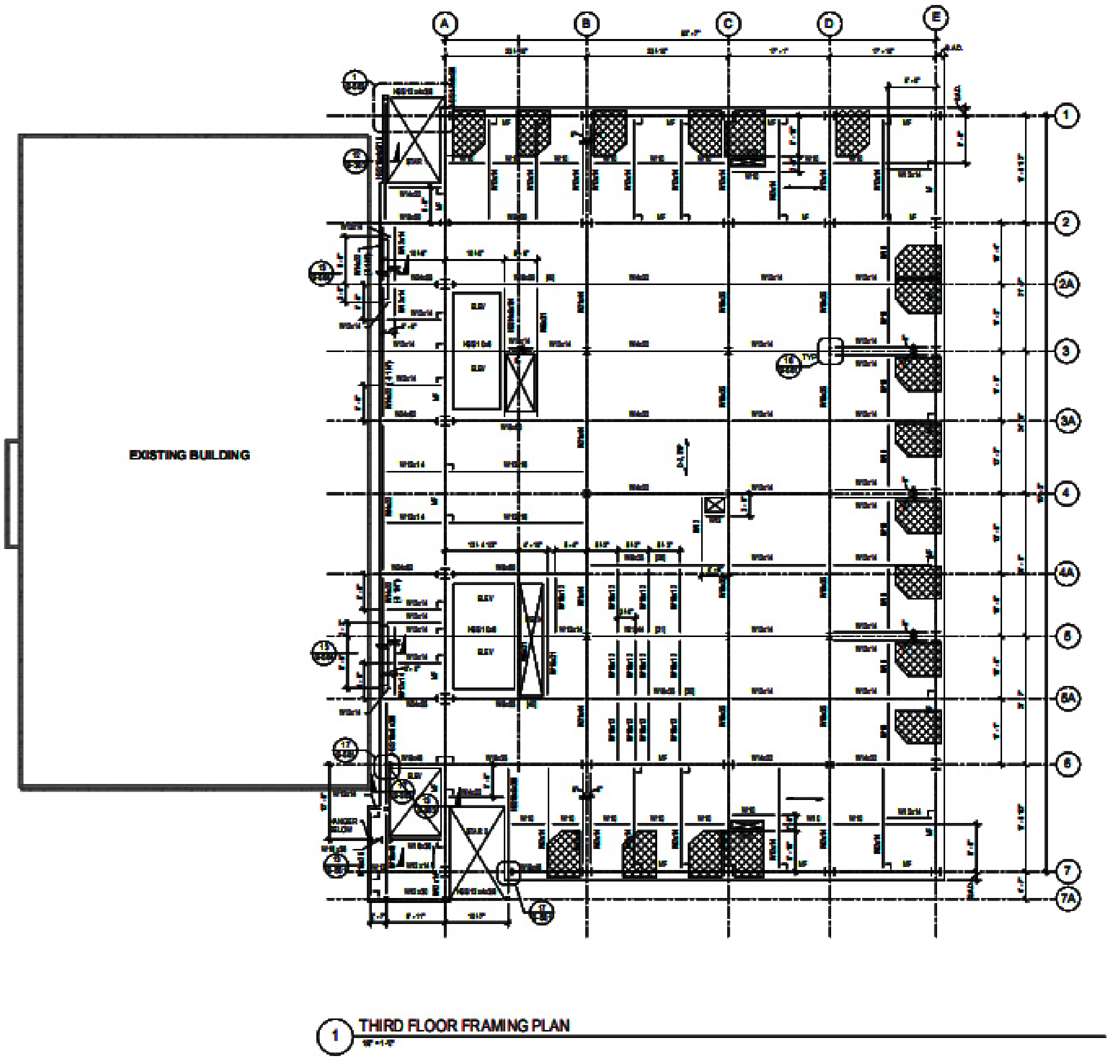


Figure 17: Typical Framing Plan

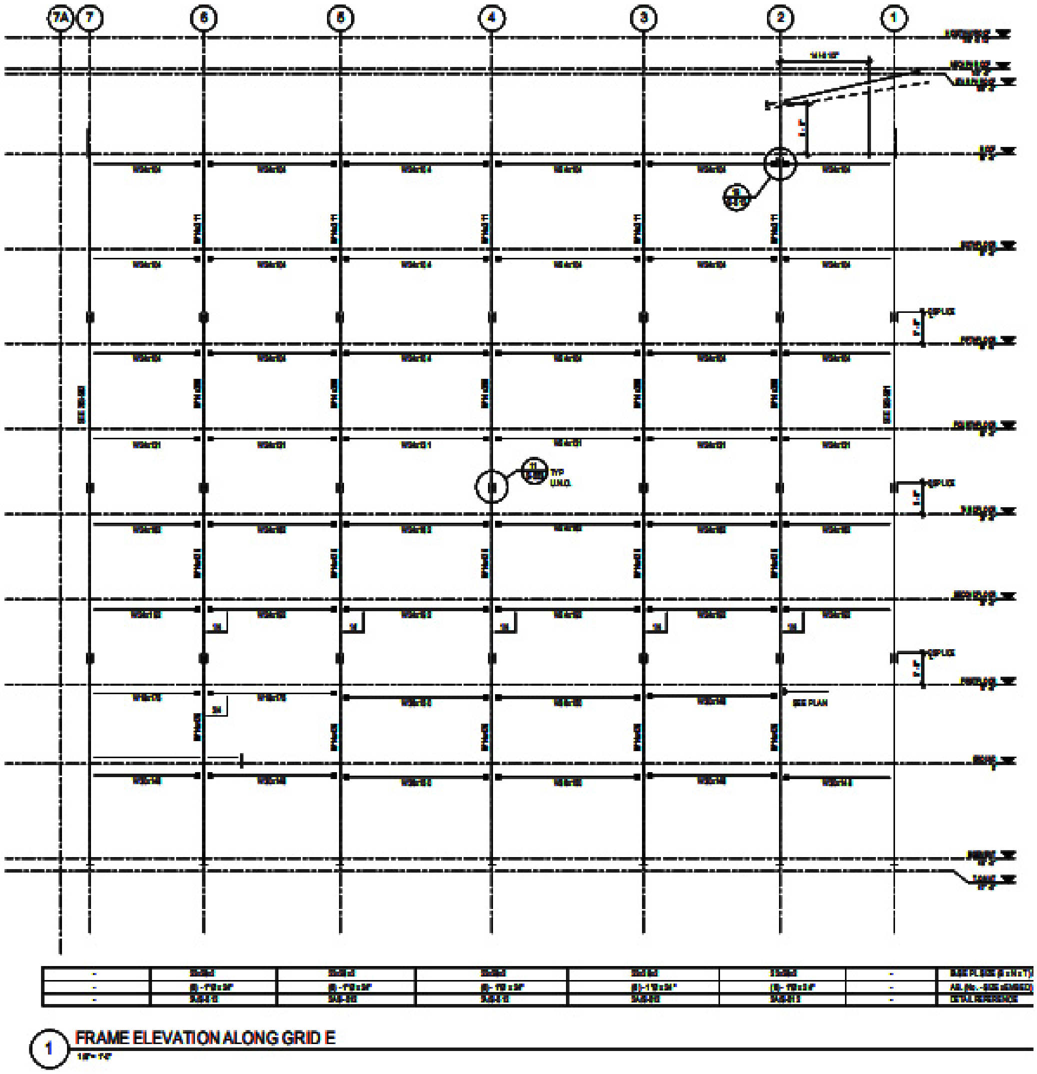


Figure 18: Typical Moment Frame Elevation