Technical Report 2

Falls Church Tower

Falls Church, VA

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Structural Option

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Executive Summary

Technical Report 2 is a pro-con structural study of the existing typical floor system of Falls Church tower as well as three alternative floor systems. The purpose of this report is to verify the original design of the existing floor system and determine if it the best option when compared to the three alternative floor systems. This study uses current industry standards including ASCE 7, AISC and ACI to determine the viability of each floor system in question.

The existing floor system is a 7 in. post tensioned concrete slab. The typical bay size used to analyze the slab was 24 ft. by 28 ft. with the slab spanning the 24 ft. direction. The slab is supported by 16 in. by 32 in. columns which are arranged in alternating directions to resist lateral loads. This system was checked using criteria and methods described in chapters 5 and 7 of the Post-Tensioning Institute's post-tensioning manual.

The three other floor systems analyzed in this report include”

- Hollow Core Plank
- Two-Way Flat Plate
- Composite Steel Deck

Using the aforementioned standards, design calculations were carried out for each system. The design for each system is summarized in their respective sections along with the advantages and disadvantages of the systems. A comparison was then made between systems taking into account the total system depth, weight, constructability, cost, and feasibility among other criteria. From this comparison it was determined that the existing post tensioned system was the best choice for the building. This was due to the relatively low cost of the system as well as other factors such as its minimal system depth, vibration control, and the short lead time required. Additionally the post tensioned system was best suited fro the irregular shape of the building and the complex column layout.

Introduction

The Falls Church Tower is a luxury apartment building located in Falls Church, Virginia. The high rise apartment building stand eleven stories tall with penthouse on the main roof. Three and a half levels of parking are offered beneath the building and private pool sits adjacent to the plaza. The building encloses 364,000 square feet of gross floor area which excludes mechanical rooms, underground rooms, and garage space. The first floor contains the lobby, a residential gym, and a lounge as well as some living space with the remaining floors serving as strictly residential space. Overall the building contains 213 residential units with a wide view of the surrounding area courtesy of the building's curved facade. The structural system of the building is primarily concrete consisting of retaining walls, columns, post-tensioned slabs, and beams. The lateral system is composed of the aforementioned columns and slabs which form and ordinary concrete moment frame.

Foundation

The foundation system of Falls Church Tower was designed in accordance with the geotechnical report provided by Whitlock, Dairymple, Poston and Associates. The report indicated a soil bearing pressure of 4 ksf along the southern face of the tower and a bearing pressure of 10 ksf for the remainder of the structure.

The foundation system from levels B3 Ext. through B1 consist of retaining walls, spread footings, and a precast slab on grade. The retaining wall runs the full perimeter of the building with a thickness of 1'-4"on the B3 Ext. level and 1'-0" for B3 through B1. The footings under the retaining walls have a width ranging from 2' to 3'. The 2' width is used for sections of the buildings where the B1 retaining wall is offset towards the interior of the building by 3'-6". A section of a typical retaining wall can be seen in Figure 1-2 and Figure 1-3.

The column footings have a range of 6'x6' to 12'x12' throughout the structure. The larger footings (10'x10' to 12'x12') being located in the basement parking section beneath the plaza. A typical footing detail can be seen in Figure 1-1. The slab on grade is 5 ksi, normal weight concrete that is 5" thick with 6x6-W2.0xW2.0 welded wire fabric placed on a vapor barrier on top of 6" of #57 washed crushed stone

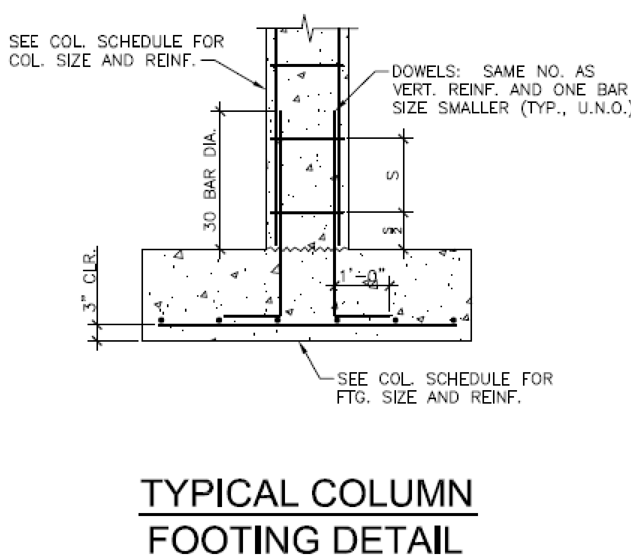


Figure 1-1

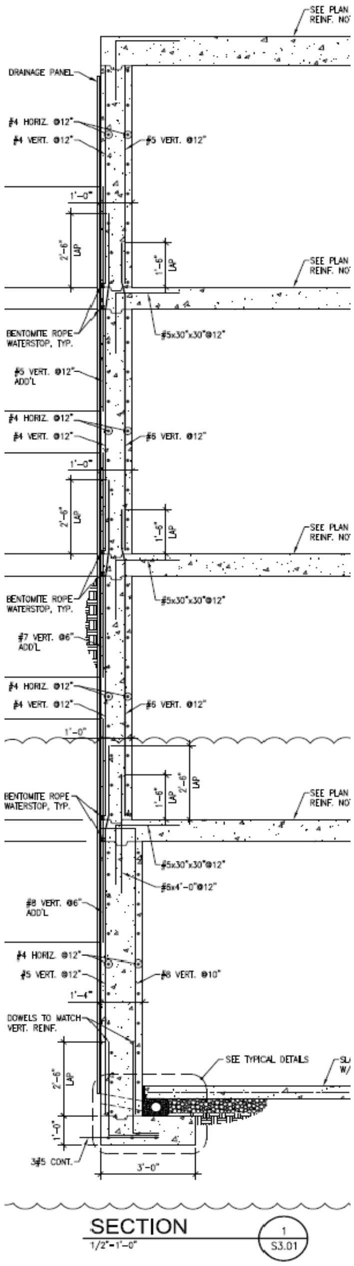


Figure 1-2

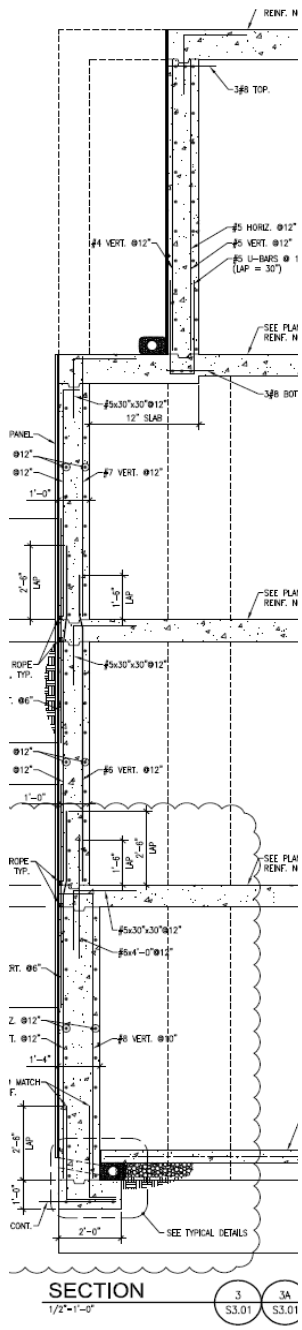


Figure 1-3

Gravity Load System

The main gravity load resisting system is composed of a flat plate supported by an intricate array of columns. Levels B3 Ext. through B1 plate systems are typically a 5 ksi, 9” thick, normal weight slab with a two way mat of #4 bottom bars at 12” on center except for slabs on grade which are 5 kis, 5” thick normal weight concrete. The penthouse roof and the elevator machine room roof use a 6” thick, one-way slab with the same properties and is support by a system of concrete beams. The plate systems from level 1 through the main roof utilize a 7” thick post tensioned slab. The typical tendons are two to three strands thick and spaced 5' on center. For a typical post tension layout plan refer to Figure 1-4.

The tower columns don't necessarily have a standard bay size due to the building's curved shape and the stair cases in both the east and west wings which interrupt any attempt at a rectilinear layout. The most typical bay size established throughout the building would be the 28'x24' bays located in the western half of the building's curved section. A standard column layout can be seen in Figure 1-5

In addition to the flat plate system the structural engineers also incorporated concrete beams into the design where necessary. As previously mentioned a system of beams is used to support the penthouse and mechanical room roofs. There are also strap (grade) beams used in the west section of B3 Ext. foundation and the east edge of B3 foundation which can be seen in Figure 1-6. Lastly, beams are used to frame all stairs and elevator shafts.

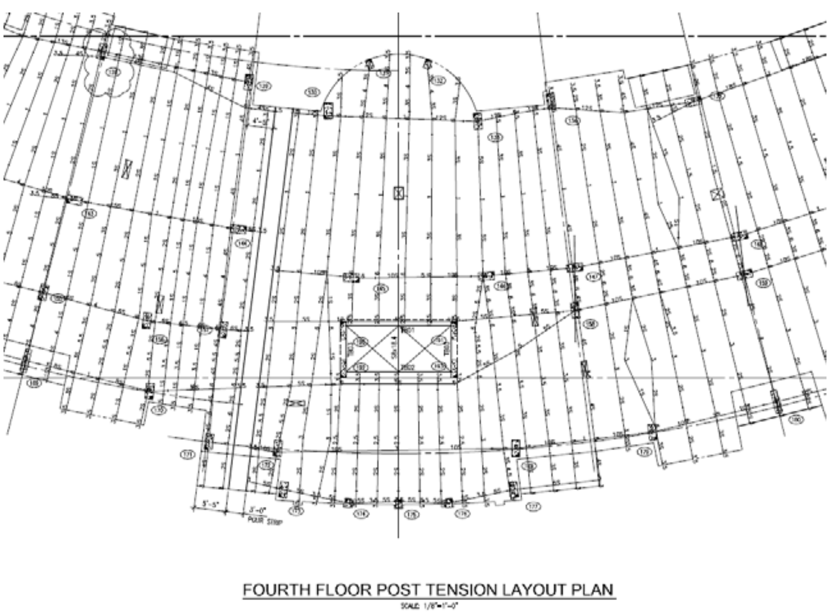


Figure 1-4
(for a larger view refer to Appendix A)

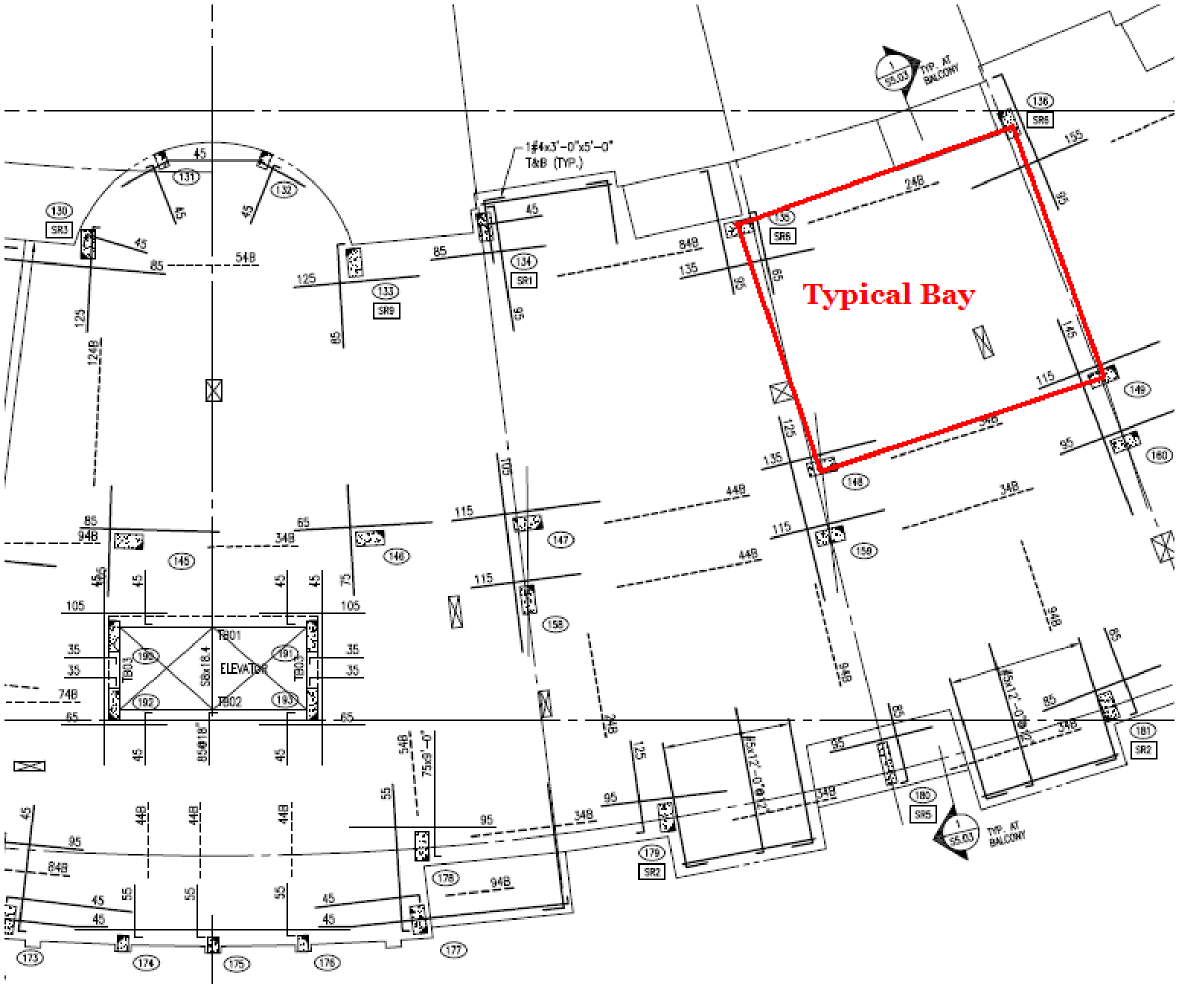


Figure 1-5

Lateral Load System

The lateral system of the building is an ordinary concrete moment frame. The tower columns' dimensions range from 12" to 24" on the short face and 12" to 48" on the long face. The two most typical columns that occur throughout the building are 16"x32" and 12"x36". The 16"x32" dimension is common for most of the interior columns whereas the 12"x36" columns are used to frame the stairs and elevator shafts.

Applicable Codes

Codes Used for Original Design

- International Building Code 2000
- Arlington County Building Code
- American Concrete Institute (ACI 318 and ACI 301)
- American Society for Testing and Materials
- American Institute of Steel Construction Manual

Codes Implemented for Thesis Analysis

- American Society of Civil Engineers (ASCE 7-05)
- International Building Code 2006

Resources

- AISC Steel Construction Manual (13^{th} Edition)
- ACI 318 – 08
- Vulcraft Deck Catalog
- Nitterhouse Hollow Core Plank Specifications

Materials and Properties

Concrete

• Footings	3000 psi
• Retaining Wall Footings	5000 psi
• Foundation Walls	
○ B3 and B3 Ext. Level	5000 psi
○ B2 and B1 Level	4000 psi
○ Site Retaining Wall	5000 psi
• Formed Slabs and Beams	5000 psi
• Columns	5000, 6000, and 8000 psi
• Slabs on Grade	5000 psi
• Pea-Gravel Concrete	2500 psi
• All Other Concrete	4000 psi

Reinforcing Steel

• Reinforcing Bars	ASTM A615
• Welded Wire Fabric	ASTM A185
• Reinforcing Bar Mats	ASTM A185
• Reinforcing Bars in Garage Slabs	ASTM A775
• Post Tension Steel	ASTM A416

Steel

• Wide Flange Members	ASTM A992
• Stiffener Plates	ASTM A572
• Other	ASTM A36

Design Loads

All of the design loads for Falls Church Tower were calculated using the values and methods provided in sections three and four. These values can be found in tables 1-1 and 1-2 below and include live load and dead load values. Snow loads have been excluded from this section but can be found in Appendix C. Live load reductions were not taken into consideration for this design.

Table 1-1: Gravity Live Loads

Live Load Areas	ASCE 7-05 Required Loading		Loads Used By Engineer
Private Rooms	40 psf	ASCE 7-05 Table 4-1	40 psf + 20 psf (Partition Allowance)
Public Rooms/Corridors	100 psf	ASCE 7-05 Table 4-1	100 psf
Tenant Storage	125 psf	ASCE 7-05 Table 4-1	125 psf
Roof	20 psf	ASCE 7-05 Table 4-1	30 psf
Stairways	100 psf	ASCE 7-05 Table 4-1	100 psf
Balconies	100psf	ASCE 7-05 Table 4-1	-
Theater	60 psf	ASCE 7-05 Table 4-1	-
Garage	40 psf	ASCE 7-05 Table 4-1	50 psf
Plaza	100 psf	ASCE 7-05 Table 4-1	350 psf
Mechanical	-		150 psf
Elevator Machine Room	-		125 psf

Table 1-2: Gravity Dead Loads

Dead Loads	Load Values
Floor Finish	16 psf
Slab: B3 - 1	109 psf
Slab: 2 - Main Roof	85 psf
MEP	15 psf
Steel	15 psf
Misc	10 psf
Roof Waterproofing	5.5 psf

Post Tensioned Floor System (Existing)

The Post Tensioned System for this project was designed for a typical bay of 28'x24' (rough dimensions given the curved shape of the building). The system consists of a variety of unbonded tendons typically spaced at 4.5 ft. The tendons are made up of ½ in. diameter strands of a minimum ultimate strength of 270 ksi with the number of strands per tendon ranging from two to ten strands.

Upon checking the existing system it was found that typical two strand tendon running through the column line is adequate to support a 7" slab with a tributary width of 4.5 ft. The only difference between the engineers design and the check carried out in this report was in the specified amount of bottom reinforcement. The original design has #4 bottom bars at 24 in. on center as sufficient reinforcing whereas the check carried out in this report calls for #7 bottom bars at 12 in. on center as the minimum reinforcing required to carry the loads. The reason for this discrepancy may be the specific loads used in either design as well as the rearranging of columns to simplify the check.

Design criteria such vibration and deflection were not checked in this report due the inherent complexity of post-tensioned systems as well as the unorthodox shape and layout of the building. Even so, it is common knowledge that post tensioned systems perform very well under deflections due the the balanced moment supplied by the stressed tendons.

Advantages

Post tensioned systems possess many advantages starting with their ability to span long distances which improves the flexibility of the the building's layout by minimizing the number of structural columns needed. Post tensioned systems also reduce the need for additional fire proofing due to the 7" of normal weight concrete that is the slab. From an architectural standpoint, post tension systems are more aesthetically pleasing with the smooth surface of the soffit which eliminates the need for a ceiling finish.

Disadvantages

The main disadvantages of post tensioned systems stem from their construction. The placement of tendons is extremely labor intensive and drawn out. Furthermore, the process of jacking the tendons is very delicate in that all the tendons must be jacked consistently and within a specific time frame to avoid uneven loading and ultimately the rupturing of a tendon through the slab which can be extremely hazardous for the laborers and set back the delivery date because of repairs.

In addition to the constructability issues, post tension system provide little in the way of mechanical flexibility in that most of the mechanical lines must be run through the walls. Additionally, early coordination between structural and mechanical engineers is necessary so as to avoid in field corrections such as cutting through a slab which could result in a ruptured tendon.

Hollow Core Plank System (Alternative)

The First alternative system researched for this report was the hollow core plank system. The specific system was obtained from the drawings and specifications section of the Nitterhouse website. The system chosen was a 6” x 4' section which conforms to the current by size of 28' x 24' by framing the planks into W14x30 beams located every 14'. The plank is composed of 5000 psi, normal weight concrete with 4 - ½ in. diameter strands per section and 2” of concrete topping. The beams and girders for this system were designed using the AISC Steel Construction Manual (13th Edition).

Advantages

Hollow core planks provide many advantages to construction and serviceability. Structurally speaking they are durable, low maintenance, and installation is relatively quick because of the lack of form work. Additionally, they can be erected year round due to the fact that they don't have an in-field curing time. From a serviceability standpoint, hollow core systems require little building insulation due to the stagnant air void in the slab.

Disadvantages

While the advantages of hollow core systems are numerous, their disadvantages are enough to cancel them out. One of the biggest disadvantages they possess is their module. Most hollow core systems come in 4' sections which limits the column orientation in any building intending. This module also makes implication in an irregularly shaped building a near impossibility. A good example of this being Falls Church Tower which has a curved facade and complex array of columns.

The other drawback to using hollow core is that they require steel beams for support which greatly increases the overall floor to floor height which also increases to the total coast of the building.

Flat Plate System

The second alternative floor system researched for this report was a two way reinforced flat plate system. The panels of the system are 24'x28' so as to conform with the current typical bay size. The plate is composed of 5000 psi, normal weight concrete with 60000 psi steel reinforcing.

The plate was designed using the direct design method from ACI 318-08. Upon completion of the design calculations it was determined that a 9 in. slab would suffice with top and bottom reinforcing. The required reinforcing calculated is #6 at 12 in. for the 24 ft. span direction and #8 at 12 in. for the 28 ft. span direction.

Advantages

The flat plate system eliminates the need for a ceiling finish due to the aesthetically pleasing smooth surface that is the bottom of the slab. This also maximizes the floor to ceiling height and reduces the ratio between floor to ceiling and floor to floor height. In this respect the flat plate system has a distinct advantage over steel systems which require a larger floor to floor height which increases the total cost of the building. Furthermore, the concrete possesses a two hour fire rating making additional fire protection unnecessary.

Disadvantages

The typical disadvantages with flat plate systems include shear and span restrictions. Punching shear is a concern due the moment transfer from slab to column. Fortunately , the calculated 9 in. slab depth proved adequate for supporting the shear loads of the building. I addition to the shear concerns, the flat plate system has span limitations which are common among most concrete structures. The typical span limit for flat plate systems is 25 ft. but the design calculations in Appendix D Show that the 24 ft. by 28 ft. bays of Falls Church Tower are acceptable for this floor system.

Composite Steel Deck System

The composite steel deck was third floor system checked for this report. The design was based on the typical 24 ft. by 28 ft. bays that have been used throughout this report. Support beams for the deck were placed at 7 ft. intervals along the 28 ft. span direction and framed into beam girders. The deck design utilized the 2006 catalog from Vulcraft and was based on a 2 hour fire rating. A 2VLI21 composite deck possesses a maximum construction span of 7 ft. 2 in. for a single span which is greater than the beam spacing of 7 ft. A minimum of 2 in. of topping is required for a 2 hour fire rating with sprayed fiber and was the topping thickness used to determine the deck. The deck meets both load and deflection requirements as shown in Appendix E.

The design of the steel beams and girders were performed using the AISC Steel Construction Manual. It was determined that a W12x22 performed adequately under the deflection and loading criteria presented by the deck and would be the most effective beam size. For the girders it was determined that a W18x35 was the most effective size for the exterior girder and a W21x44 was the most effective size for the interior girder.

Advantages

Steel structures possesses many advantages the most notable stemming from material properties and erection. Because steel has such a high strength to weight ratio it is able to carry larger loads and span longer distances than other systems while maintaining a lower weight. This greater reduces the seismic impact on buildings as well as the cost.

Disadvantages

Steel systems do however have their disadvantages, especially when it comes to ceiling height. Most steel beams have a fairly large depth which when added to the supported deck depth creates a substantially deep floor system. Add to that the fact that a ceiling finish must be installed due to the unappealing nature of exposed steel and you develop a low floor to ceiling height which must be offset by increasing the floor to floor height. Ultimately, this leads to a large increase in building cost.

Comparison

The comparison between the existing system and the alternative systems was based on the following criteria: slab weight, slab depth, system depth, vibration control, fire rating, additional fire proofing, constructability, formwork, floor to floor height, lead time, system cost, and feasibility. Table 2-1 illustrates the system comparison by highlighting positive aspects in blue and negative aspects in red.

	Post Tensioned	Hollow Core	Flat Plate	Composite Deck
Slab Weight	88 psf	49 psf	113 psf	44 psf
Slab Depth	7"	6"	9"	2"
System Depth	7"	24"	9"	23"
Vibration Control	Yes	No	Yes	Yes
Fire Rating	2 hr.	2 hr.	2 hr.	2 hr.
Additional Fire Proofing	No	No	No	Yes
Constructability	Hard	Easy	Medium	Easy
Formwork	Yes	No	Yes	No
Foor-Floor Height	-	Decreases	Increases	Increases
Lead Time	Short	Long	Short	Long
System Cost	$22.60/sf	$34.20/sf	$20.20/sf	$26.80/sf
Feasibility	Existing	Impossible	Most Possible	Possible

Table 2-1: Comparison Data

Conclusion

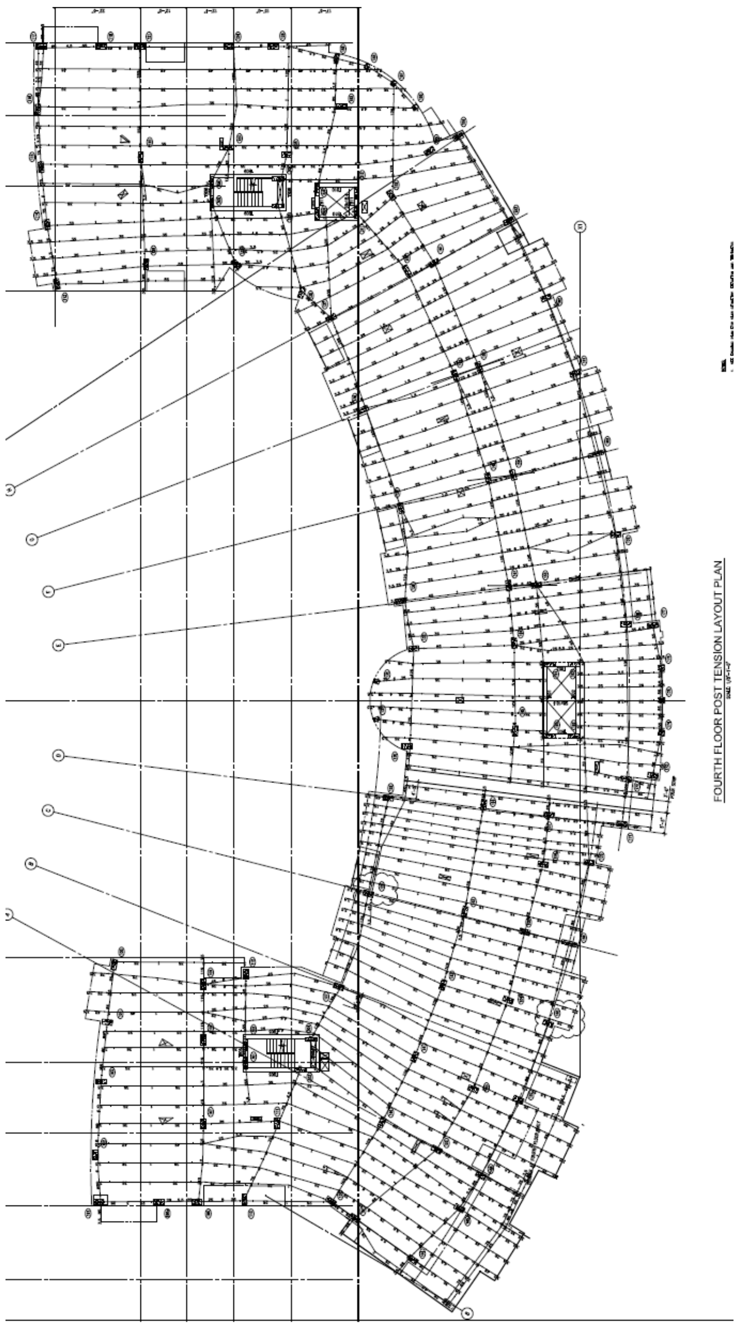
After analyzing the existing and alternate floor systems, it was determined that the existing post tensioned system is the best option for the building given the low cost and floor depth. Of the alternate floor systems explored the best choice for a replacement would be the flat plate system which has a lower cost than the post tensioned system and only a 2 in. increase in floor depth.

The hollow core system had positive attributes such as its low weight and ease of construction. However, the fact remains that the modular sections of this system make it impossible to incorporate into the existing shape of the building. The idea would be feasible if custom sections were ordered but this would only compound the already steep cost of the system. It must also be kept in mind that the implication of the hollow core system would mean rearranging the column grid for constructability purposes.

The composite steel deck system is a viable option but overall not the most appealing. With it low weight it reduces the impact of seismic loading and is fairly easy to construct. And while the price of the system is marginal it is offset buy the incurred costs of the increased floor height. Furthermore, as with the hollow core system, the column grid would have to be rearranged for constructability purposes.

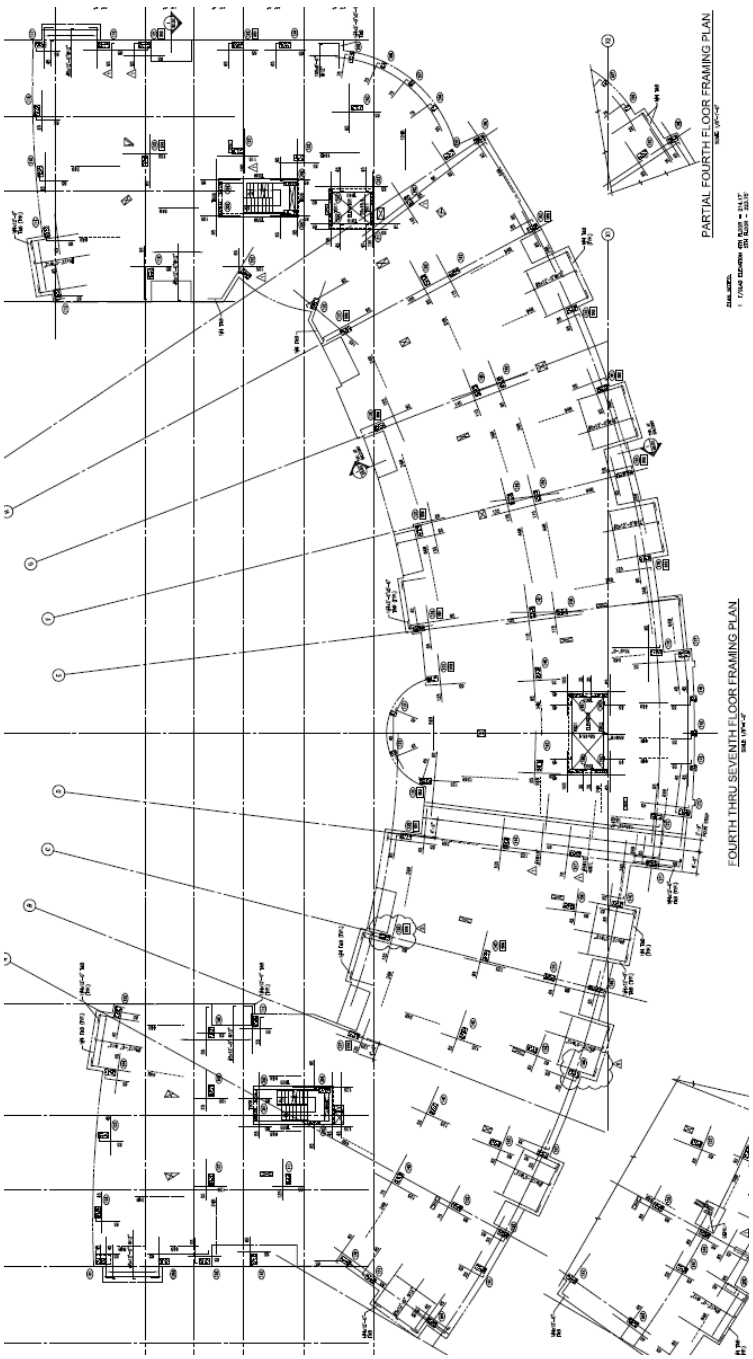
Appendix

Appendix A – Figures



Typical Post Tension Layout

Appendix A – Figures

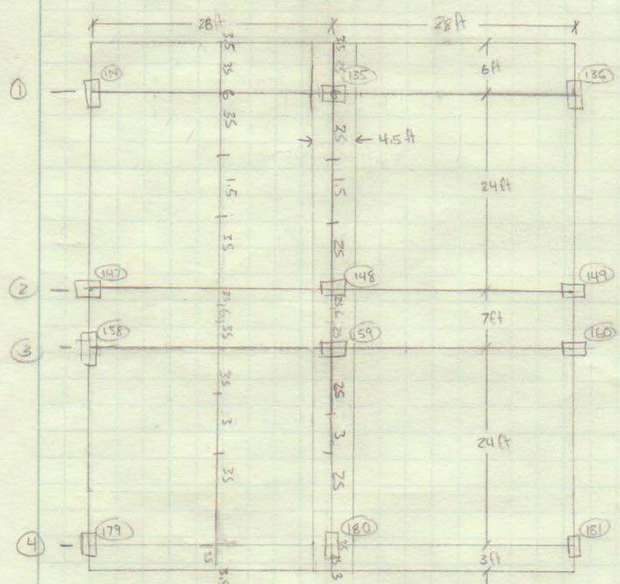


Typical Column Layout

Appendix B – Post Tension Design

Technical Assignment 2 | Existing Floor System | Floor 5 Bays E-G | 1

Assumptions: Bays are rectangular to simplify calculations
Columns line up to simplify calculations



Column	Size
134	16 x 32
135	16 x 32
136	16 x 32
147	16 x 32
148	16 x 32
149	16 x 32
158	16 x 32
159	16 x 32
160	16 x 32
179	16 x 32
180	12 x 48
181	16 x 32

Loads

Dead = 88 psf

Super Imp = 15 psf

Live = 60 psf (Privat Rooms)
100 psf (Corridors, Balconies)

Total = 163 (unfactored), 182 (factored) } Private Rooms

203 (unfactored), 280 (factored) } Corridors, Balconies

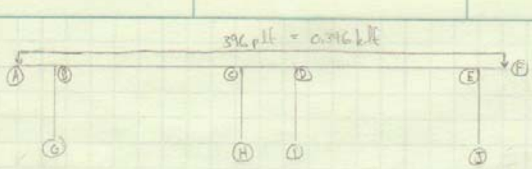
Given

$f'_c = 5000$ psi

Appendix B – Post Tensioned Design

2

Dead



Slab Beam Stiffness

$$I = \frac{(4.5\text{ft})(12\text{in}/\text{ft})(7\text{in})^3}{12} = 1544\ \text{in}^4$$

$E = 4000$ ksi

$K_{BA} = \frac{kEI}{l_1}$ (Table A-14 from Concrete Design Textbook)

$c_1 = 32\text{in}$ $c_2 = 16\text{in}$
$l_1 = 72\text{in}$ $l_2 = 54\text{in}$ $\Rightarrow$ $k = 6.324$
$c_1/l_1 = 0.44$ $c_2/l_2 = 0.29$

$$K_{BA} = \frac{(6.324)(4000\ \text{k/in}^2)(1544\ \text{in}^4)}{72\ \text{in}} = 568707\ \text{in-k/rad}$$

$K_{BC} = \frac{kEI}{l_1}$ $c_1 = 32\text{in}$ $c_2 = 16\text{in}$
$l_1 = 288\text{in}$ $l_2 = 54\text{in}$ $\Rightarrow$ $k = 4.574$
$c_1/l_1 = 0.11$ $c_2/l_2 = 0.29$ COF = 0.538

$$K_{BC} = \frac{(4.574)(4000)(1544)}{288} = 98087\ \text{in-k/rad}$$

K_{CB}: $c_1 = 16\text{in}$ $c_2 = 32\text{in}$
$l_1 = 288\text{in}$ $l_2 = 54\text{in}$ $\Rightarrow$ $k = 4.591$
$c_1/l_1 = 0.056$ $c_2/l_2 = 0.593$ COF = 0.538

$$K_{CB} = \frac{(4.591)(4000)(1544)}{288} = 98451\ \text{in-k/rad}$$

K_{CD}: $c_1 = 16\text{in}$ $c_2 = 32\text{in}$
$l_1 = 84\text{in}$ $l_2 = 54\text{in}$ $\Rightarrow$ $k = 6.049$
$c_1/l_1 = 0.190$ $c_2/l_2 = 0.593$ COF = 0.623

$$K_{CD} = \frac{(6.049)(4000)(1544)}{84} = 444746\ \text{in-k/rad}$$

K_{DC}: $c_1 = 32\text{in}$ $c_2 = 16\text{in}$
$l_1 = 84\text{in}$ $l_2 = 54\text{in}$ $\Rightarrow$ $k = 6.053$
$c_1/l_1 = 0.381$ $c_2/l_2 = 0.296$ COF = 0.623

$$K_{DC} = \frac{(6.053)(4000)(1544)}{84} = 445040\ \text{in-k/rad}$$

Appendix B – Post Tensioned Design

3

K_{DE}: $c_1 = 32$ in, $c_2 = 16$ in
$\ell_1 = 288$ in, $\ell_2 = 54$ in $\Rightarrow$ $k = 4.533$, COF $= 0.539$
$c_1/\ell_1 = 0.111$, $c_2/\ell_2 = 0.296$in

$$K_{DE} = \frac{(4.533)(4000)(1544)}{288} = 97208 \text{ in-k/rad}$$

K_{ED}: $c_1 = 32$, $c_2 = 16$ in
$\ell_1 = 288$, $\ell_2 = 54$ in $\Rightarrow$ $k = 4.533$, COF $= 0.539$
$c_1/\ell_1 = 0.111$, $c_2/\ell_2 = 0.296$in

$K_{DE} = 97208$ in-k/rad

K_{EF}: $c_1 = 32$ in, $c_2 = 16$in
$\ell_1 = 36$ in, $\ell_2 = 54$ in $\Rightarrow$ $k = 8.786$
$c_1/\ell_1 = 0.888$, $c_2/\ell_2 = 0.296$ in

$$K_{EF} = \frac{8.786(4000)(1544)}{36} = 1507287 \text{ in-k/rad}$$

Column Stiffness

$K_G = k\frac{EI}{l_c}$

$I_G = 43691$ in^4

$l_c = 9.583$ ft, $t_a = 3.5$ in
$l_u = 9$ ft, $t_b = 3.5$ in $\Rightarrow$ $k = 4.691$
$l_c/l_u = 1.065$, $t_a/t_b = 1.0$

$$K_G = \frac{2(4.691)(4000)(43691)}{115} = 14257703 \text{ in-k/rad}$$

K_H: $I_H = 10923$ in^4 $k = 4.691$

$$K_H = \frac{2(4.691)(4000)(10923)}{115} = 3564507 \text{ in-k/rad}$$

$K_I = K_J = K_G = 14257703$ in-k/rad

Appendix B – Post Tensioned Design

4

Dead Load Moments

	COF = 0.538					COF = 0.623			COF = 0.539		
	JOINT B			JOINT C			JOINT D			JOINT E		
	B-A	B-G	B-C	C-B	C-H	C-D	D-C	D-I	D-E	E-D	E-J	E-F
DF	0.038	0.955	0.007	0.024	0.868	0.108	0.030	0.963	0.007	0.006	0.899	0.095
FEM(A-K)	7.13		-19.01	19.01		-1.62	1.62		-19.01	19.01		-1.78
Dist 1	0.45	11.35	0.083	-0.417	-15.09	-1.88	0.52	16.75	0.121	-0.103	-15.49	-1.64
CO			-0.224	0.045		0.324	-1.17		-0.056	0.065		
Dist 2	0.009	0.214	0.002	-0.009	-0.32	-0.04	0.037	1.18	0.009	0.000	-0.058	-0.006
Final	7.59	11.56	-19.15	18.63	-15.41	-3.22	1.01	17.93	-18.93	18.97	-15.55	-3.43

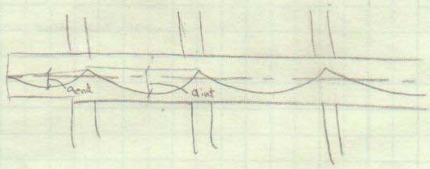
Balanced Loads

assume 65% of the dead loads are balanced by post-tensioning

$w_{bal} = 0.65 \times 103 = 66.95 \approx 67$ psf

Tendon Profile

use same profile as engineer



* a_{end} does not have a value due to the cantilevered balconies.

a_{int}: Span 1-2 = $7 - 1 - 1\frac{1}{2} = 4.5$ in

Span 2-3 = $a_{int} \approx 0$ due to a short span that creates a neglectable sag.

Span 3-4 = $7 - 1 - 3 = 3$ in

Tendon Force

$F = \dfrac{w_{bal}L^2}{8a}$ Span 1-2: $F = \dfrac{(0.067)(24)^2}{(8(4.5)/12)} = 12.86$ k/ft $F/A = \dfrac{12.86\text{ k/ft}}{7\text{in}(12\text{ in/ft})} = 0.153\text{ ksi} = 153\text{ psi}$

3-4: $F = \dfrac{(0.067)(24)^2}{(8(3)/12)} = 19.29$ k/ft $F/A = \dfrac{19.29\text{ k/ft}}{7(12)} = 0.230\text{ ksi} = 230\text{ psi}$

Critical Section

Min $V_{NET} = \omega_{NET}(L/2)(\text{Tributary width})$; $\omega_{NET} = DL + LL - \omega_{Bal} = 103 + 60 - 67 = 96\text{ psf} = 0.096\text{ ksf}$

$V_{NET} = (0.096)(4.5)(24/2) = 5.18$ K

* Maximum V_{NET} values will be used per column to simplify analysis

Appendix B – Post Tensioned Design

Live Load Moments (On All Spans) LL = 0.270 klf and 0.450 klf

	COF = 0.538					COF = 0.623			COF = 0.539			
	Joint B			Joint C			Joint D			Joint E		
	B-A	B-G	B-C	C-B	C-H	C-D	D-C	D-1	D-E	E-D	E-J	E-F
DF	0.038	0.955	0.007	0.024	0.868	0.108	0.030	0.963	0.007	0.006	0.899	0.095
FEM	8.1		-12.96	12.96		-1.84	-1.84		-12.96	12.96		-0.338
Dist 1	0.185	4.64	0.034	-0.267	-9.65	-1.2	0.339	10.71	0.078	-0.076	-11.35	-1.20
CO			-0.144	0.018		0.208	-0.748		-0.041	0.042		
Dist 2	0.005	0.138	0.001	-0.005	-0.196	-0.024	0.024	0.769	0.006		-0.038	-0.004
Final	8.29	4.78	-13.07 *	12.71 *	-9.85	-2.86	1.45	11.47	-12.92	12.93	-11.39	-1.54

Live Load Moments (On Spans 0-1, 1-2, 3-4) LL = 0.270 klf and 0.450 klf

	COF = 0.538					COF = 0.623			COF = 0.539			
	Joint B			Joint C			Joint D			Joint E		
	B-A	B-G	B-C	C-B	C-H	C-D	D-C	D-1	D-E	E-D	E-J	E-F
DF	0.038	0.955	0.007	0.024	0.868	0.108	0.030	0.963	0.007	0.006	0.899	0.095
FEM	8.1		-12.96	12.96		-1.84				12.96		
Dist 1	0.185	4.64	0.034	-0.267	-9.65	-1.2				-0.078	-11.65	-1.23
CO			-0.144	0.018		0.208	-0.748		-0.041			
Dist 2	0.005	0.138	0.001	-0.005	-0.196	-0.024	0.024	0.76	0.006			
Final	8.29	4.78	-13.07	12.71	-9.85	-2.96	-0.724	0.76	-0.035	12.88	-11.65	-1.23

* These moments will be used to determine support moments. Given the complex nature of the building, only span B-C will be checked

Appendix B – Post Tensioned Design

6

Section Modulus

$S = \frac{bh^2}{6} = \frac{54(7)^2}{6} = 441\ in^3 = 36.75\ in^3/ft$

Loading	Magnitude ksf	Moments (ft-k) at Support B	Moments (ft-k) at Support C
Dead Load	0.103	-19.15	-18.63
Live Load	0.060	-13.07	-12.71
Balanced Load	0.067	8.50	8.26
Net Load	0.096	-23.72	-23.08
V_{NET}		5.18	5.18
$V_c/3 = (5.18)(C^1/12)(1/3)$		4.60	2.30
Face Moment $= M_{net} + \frac{V_c}{3}$		-19.12	-20.78
M/S		0.521	0.565
F/A		-0.153	-0.153
Top of Slab Stress (ksi)		0.368	0.412

1. c = column width parallel to span length

Maximum Tensile Stress = 0.412 ksi

Supports	B	C
Balanced Moment	8.50	8.26
$-V_c/3$	-1.02	-0.92
Balanced Moment at Face	7.48	7.75
$Fe = 12.86 \cdot (0.5/12)$	-0.536	-0.536
Secondary Moments	6.94	7.21

Design Moments

Midspan Moment $= \frac{w_u l_n^2}{11}$

Dead Load $= \frac{0.103(24)^2(4.5)}{11} = 24.27$

Live Load $= \frac{0.06(24)^2(4.5)}{11} = 14.14$

$M_2 = M_{BAL} - Fe = \frac{0.067(24)^2(4.5)}{11} - 12.86\left(\frac{4.5}{12}\right) = 10.96$

Appendix B – Post Tensioned Design

7

	B	Midspan	C
1.2 D	22.98	29.12	22.36
1.6 L	20.91	22.62	20.34
M_2	6.94	10.96	7.21
M	50.83	62.70	49.91
V	8.8	8.8	8.8
$V_c/3$	4.6		2.3
$V_c/6$		-3.45	
M_F	46.23	59.25	47.61

Ultimate Tendon Force

$$f_{ps} = f_{se} + 10000 + \frac{f'_c}{300\rho_p}$$

$$f_{se} = 0.7(270) - 15 = 174 \text{ ksi}$$

$$\rho_p = A_{ps}/bd = \left(\frac{12.86}{26.6}\right)0.196/(54 \times 5.5) = 0.000319$$

$$f_{ps} = 174000 + 10{,}000 + \frac{5000}{300(0.000319)} = 236000 \text{ psi}$$

$$F_{ps} = \frac{236}{174}(12.86) = 17.44 \text{ K}$$

Design Capacity

$$A_s = 0.002(7)(54) = 0.756 \text{ in}^2/4.5\text{ ft} = 0.168 \text{ in}^2/\text{ft} \Rightarrow \text{use } \#4 @ 12'' \text{ o.c.}$$

$$A_s f_y = 0.9 \text{ in}^2(60) = 54$$

$$T_U = 54.0 + 17.44 = 71.44 \text{ K}$$

Depth of Compression Block

$$a = \frac{T_U}{0.85 f'_c b} = \frac{71.44\text{k}}{0.85(5)(54)} = 0.311 \text{ in} \Rightarrow d - a/2 = 5.5 - \frac{0.311}{2} = 5.35 \text{ in}$$

$$M_U = \phi T_U(d - a/2) = 0.9(71.44)(5.35/12) = 28.67 \text{ ft-K} < 59.25 \text{ ft} = M_{U\,required}$$

$$\frac{59.25 \text{ ft-K}}{28.67 \text{ ft-K}} = 2.07 > \therefore \text{ minimum steel is not adequate.}$$

Try #6@ 12 in o.c.

$$A_s f_y = (1.98)(60) = 118.8 \text{ K}$$

$$T_U = 118.81 + 17.44 = 136.24 \text{ K}$$

$$M_U = 53.17 \text{ ft-K} < 59.25 \therefore \text{ no good}$$

Try #7@ 12 in o.c.

$$A_s f_y = (2.7 \text{ in}^2)(60) = 162 \text{ K}$$

$$T_U = 162 + 17.44 = 179.44 \text{ K}$$

$$M_U = 68.76 \text{ ft-K} > 59.25 \therefore \text{ ok}$$

Appendix C – Hollow Core Plank Design

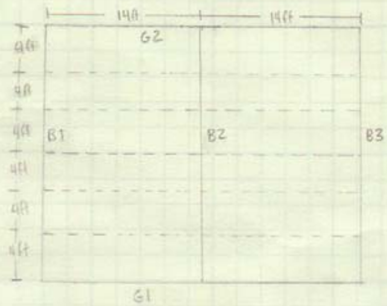
Hollowcore Plank

Criteria for Plank Selection

2hr fire rating
least amount of material

Using Hollowcore plank specifications from Nitterhouse:

6" × 4' Hollowcore
2" topping
4 - ½" Ø Strands
$f_{allowable} = 290 \text{ psf} < 154.5 = w_u$



Beam Sizing

B2 (Typ)

DL : Hollowcore = 48.75 psf
Misc = 10 psf
Member = 5 psf
DL = 63.75 psf

LL: $L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL} A_T}}\right)$; $A_T = (14)(24) = 336$
$K_{LL} = 1$
$K_{LL} A_T = 336 \text{ ft}^2 < 400 \text{ ft}^2$ ∴ no red.

LL = 60 psf

$w_u = 1.2(63.75) + 1.6(60) = 172.5 \text{ psf}$

$V_u = \frac{(0.173)(14)(24)}{2} = 29.1 \text{ K}$

$M_u = \frac{(0.173)(14)(24)^2}{8} = 174.4 \text{ ft K}$

Using Zx table from AISC Steel Construction Manual:

$M_u \le \phi M_{px}$

W14X30: $\phi M_p = 177 \text{ ft K} > 174.4 \text{ ft K}$ ∴ ok
$\phi V_n = 112 \text{ K} > 29.1 \text{ K}$ ∴ ok

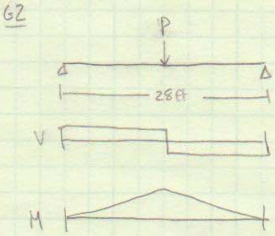
$\Delta_{LL} \le \frac{l}{360}$ $\Delta_{LL} = \frac{5 w_u L^4}{384 E I_x}$; $I_x = 291 \text{ in}^4$ $\frac{l}{360} = \frac{24(12)}{360} = 0.8 \text{ in}$

$\Delta_{LL} = \frac{5(0.06)(24)^4(1728)(14)}{384(29000)(291)} = 0.372 \text{ in} < 0.8 \text{ in}$ ∴ ok

Appendix C – Hollow Core Plank Design

2

G2



$V_u = P/2 \; ; \; P = 29.1\,K$

$V_u = \frac{29.1}{2} = 14.55\,K$

$M_u = \frac{V_u l}{2} = \frac{14.5(28)}{2} = 203\ \text{ft K}$

Using Z_x tables:

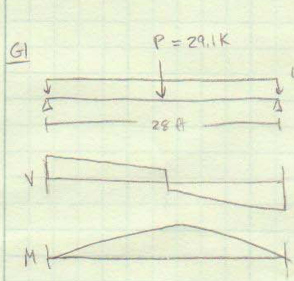
W14x34: $\phi M_p = 205\ \text{ft-K} > 203\ \text{ft K} \therefore$ ok $\quad I_x = 340\ \text{in}^4$

$\phi V_n = 120\,K > 14.55\,K \therefore$ ok

$\Delta_{LL} \le \frac{l}{360} \qquad \Delta_{LL} = \frac{PL^3}{48EI} \; ; \; P_{LL} = \frac{0.06(14)(24)}{2} = 10.08\,K \qquad \frac{l}{360} = \frac{28(12)}{360} = 0.93\ \text{in}$

$\Delta_{LL} = \frac{(10.08)(28)^3(1728)}{48(29000)(340)} = 0.81\ \text{in} < 0.93\ \text{in} \therefore$ ok

G1



$w_u = (0.588)(35) = 2.06\ \text{klf}$

$V_u = \frac{P + w_u L}{2} = \frac{29.1 + (2.06)(28)}{2} = 43.4\,K$

$M_u = \frac{w_u L}{2}\left(\frac{L}{2}\right)\frac{1}{2} + \frac{PL}{4} = \frac{(2.06)(28)^2}{8} + \frac{29.1(28)}{4} = 405.6\ \text{ft K}$

Using Z_x tables:

W21x50: $\phi M_p = 413 > 405.6 \therefore$ ok $\quad I_x = 984\ \text{in}^4$

$\phi V_n = 237 > 43.4 \therefore$ ok

$\Delta_{LL} \le \frac{l}{360} \qquad \Delta_{LL} = \frac{5 w_u L^4}{384 E I_x} \; ; \; w_u = 0.06(3.5 + 12) = 0.93\ \text{klf}$

$\frac{l}{360} = \frac{28(12)}{360} = 0.93 \qquad \Delta_{LL} = \frac{5(0.93)(28)^4(1728)}{384(29000)(984\ \text{in}^4)} = 0.451\ \text{in} < 0.93\ \text{in} \therefore$ ok

Appendix C – Hollow Core Plank Design

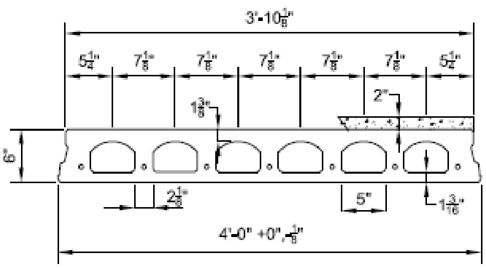
Prestressed Concrete
6"x4'-0" Hollow Core Plank

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

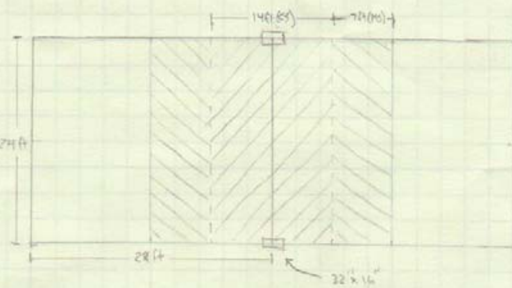
Appendix D – Flat Plate Design

Flat Plate System

1

$f'_c = 5000$ psi

$f_y = 60{,}000$ psi



Column Strip = 14 ft

Middle Strip = 7 ft

$l_n = 24\text{ft} - 16/12 = 22.67' = 22'8"$

Slab Depth $= l_n/30 = \frac{22.67}{30} = 0.756' = 9.06" \approx 9"$

Static Moment $= M_o = \frac{q_u l_2 l_n^2}{8}$; $q_u = 1.2(103\text{ psf}) + 1.6(60\text{ psf}) = 219.6\text{ psf} \approx 0.220\text{ ksf}$

$l_2 = 28\text{ft}$

$l_n = 22.67\text{ ft}$

$$M_o = \frac{(0.22)(28)(22.67)^2}{8} = 396\text{ ft-K}$$

Endspan Static Moment Distribution

	%	M_u
Interior Support	70	277 ft-K
Midspan	52	206 ft-K
Exterior Support	26	103 ft-K

$\alpha = \frac{I_B}{I_S}$; $I_B = 0$ (No Beam) $\Rightarrow \alpha = 0$

$\beta_t = 0$

Appendix D – Flat Plate Design

2

	%	
Interior Support	75	Column Strip $M_T = 0.75(277) = 208$ ft-k
	25	Middle Strip $M_T = 0.25(277) = 69$ ft-k
Midspan	60	Column Strip $M_T = 0.60(206) = 124$ ft-k
	40	Middle Strip $M_T = 0.40(206) = 82$ ft-k
Exterior Support	100	Column Strip $M_T = 1.00(103) = 103$ ft-k
	0	Middle Strip $M_T = 0$

Reinforcing

Minimum reinforcing $= 0.0018(9_{in})(12_{in}) = 0.1944\ in^2 \Rightarrow$ Use #4 @ 12 in o.c.

$$M_u = \frac{w_u l^2}{8} = \frac{(0.22)(24)^2}{8} = 15.84\ \frac{ft\text{-}k}{ft}$$

$$d_1 = 9 - 1.5 - \frac{0.5}{2} = 7.25\ in$$

$$a = \frac{(0.20)(60)}{0.85(5)(12)} = 0.235\ in \qquad c = \frac{0.235\ in}{0.8} = 0.294$$

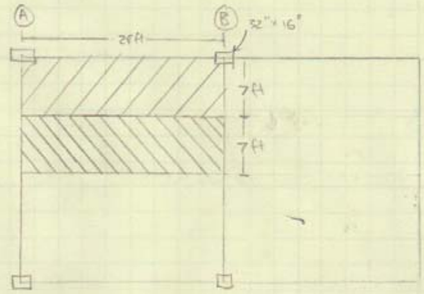
$$\phi M_n = 0.9(0.2)(60)\left(7.25 - \frac{0.235}{2}\right) = 77.03\ kin = 6.42\ k\text{-}ft < 15.84\ ft\text{-}K \therefore \text{no good}$$

$$A_s \text{ required} = \frac{15.84(12)}{(0.9)(60)\left(7.25 - \frac{0.235}{2}\right)} = 0.494\ in^2 \Rightarrow \text{Use \# 6 @ 12'' o.c.}\quad A_s = 0.6\ in^2$$

Appendix D – Flat Plate Design

3

Column Strip = 7 ft
Middle Strip = 7 ft
$l_n = 28 - 32/12 = 25.33$ ft
Slab Depth = 9"



Static Moment $= M_o = \frac{q_o l_2 l_n^2}{8}$; $q_o = 0.22$ ksf
$l_2 = 14$ ft
$l_n = 25.33$ ft

$$M_o = \frac{0.22(14)(25.32)^2}{8} = 247 \text{ ft K}$$

<u>Interior Span Static Moment Distribution</u>

	%	M_u
Support A	0.65	160.55
Midspan	0.35	86.45
Support B	0.65	160.55

$\alpha = \frac{I_B}{I_s}$; $I_B = 0 \therefore \alpha = 0$

$\beta_t = 0$

	%	
Support A	75	Column Strip $M_T = 0.75(160.55) = 120.41$ ft K
	25	Middle Strip $M_T = 0.25(160.55) = 40.14$ ft K
Midspan	60	Column Strip $M_T = 0.60(86.45) = 51.87$ ft-K
	40	Middle Strip $M_T = 0.40(86.45) = 34.58$ ft k
Support B	75	Column Strip $M_T = 0.75(160.55) = 120.41$ ft-K
	25	Middle Strip $M_T = 0.25(160.55) = 40.14$ ft-K

<u>Reinforcing</u>

Minimum Reinforcing $= 0.0018(9)(12) = 0.1944 \text{ in}^2 \Rightarrow$ Use #4 @ 12 in. o.c. $A = 0.2 \text{ in}^2$

$M_u = \frac{w_u l^2}{8} = \frac{0.22(28)^2}{8} = 21.56 \text{ ft-K} > \frac{120.41 \text{ ft K}}{7} = 17.20 \text{ ft-K}$

$d_t = 9 - 1.5 - 0.5 - \frac{0.5}{2} = 6.75$ $\quad a = \frac{(0.2)(60)}{0.85(5)(12)} = 0.235$

$\phi M_n = 0.9(0.2)(60)(6.75 - \frac{0.235}{2}) = 71.63 \text{ k-in} = 5.96 \text{ ft K} < 21.56 \text{ ft-K} \therefore$ no good

A_s required $= \frac{21.56(12)}{(0.9)(60)(6.75 - \frac{0.235}{2})} = 0.722 \text{ in}^2 \Rightarrow$ Use #8 @ 12" o.c top and bottom $A_s = 6.79 \text{ in}^2 \therefore$ ok

Appendix D – Flat Plate Design

4

Shear

$V_u = \omega_u A_c = (0.22)(672 - (\frac{16(32)}{144})) = 147.06\ k$ $\quad b_o = 2(2(6.75)+16) + 2(2(6.75)+32) = 150$

$\phi V_n = \phi(V_c + V_s)\ ;\ V_c = $

$0.17\left(1+\frac{2}{(32/16)}\right)\sqrt{5000}\ (150)(6.75) = 24342\ lbs = 24.3\ K$

$0.083\left(\frac{30(6.75)}{150}+2\right)\sqrt{5000}\ (150)(6.75) = 40773\ lbs = 40.7\ K$

min $0.33\sqrt{5000}(150)(6.75) = 23626\ lbs = 23.6\ K$

$\therefore$ 23.6K controls

$V_s = \frac{V_u}{\phi} - V_c = \frac{147.06}{0.75} - 23.6k = 172.48\ k$

$A_v = \frac{V_s s}{f_y d} = \frac{(172k)(8\ in)}{(60)(6.75in)} = 1.27\ in^2 \Rightarrow$ #10 @ 8 in o.c.

Appendix E – Composite Steel Deck Design

VULCRAFT

Restrained Assembly Rating	Type of Protection	Concrete Thickness & Type (1)	U.L. Design No. (2,3,4)	Classified Deck Type		Unrestrained Beam Rating
				Fluted Deck	Cellular Deck (5)	
2 Hr. (continued)	Sprayed Fiber	2" NW&LW	859 *	2VLI,3VLI	2VLP, 3VLP	1,1.5,2,3 Hr.
		2 1/2" NW&LW	822 *	2VLI,3VLI	2VLP, 3VLP	1 Hr.
			825 *	1.5VLI,2VLI,3VLI	2VLP, 3VLP	1,1.5,2 Hr.
			831 *	2VLI,3VLI	2VLP, 3VLP	1,1.5,2 Hr.
			832 *	1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5,2,3 Hr.
			833 *	1.5VLI,2VLI,3VLI	2VLP, 3VLP	1.5 Hr.
			847 *	2VLI,3VLI	3VLP	1,1.5,3 Hr.
			858 *	2VLI,3VLI	2VLP, 3VLP	1,1.5,2,4 Hr.
			861 *	12VLI,3VLI		1,1.5 Hr.
			870 *	1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,2 Hr.
			871 *	2VLI,3VLI	2VLP, 3VLP	1,1.5,2,3 Hr.
		2 1/2" LW	862 *	2VLI,3VLI		1 Hr.
		2 1/2" NW	864 *	3VLI	3VLP	1.5 Hr.
		3 1/4" LW	860 *	2VLI,3VLI		1,1.5,2 Hr.
	Unprotected Deck	3 1/4" LW	733 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5Hr.
			826 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5,2 Hr.
			840 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5 Hr.
			902 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5 Hr.
			907 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,2 Hr.
			913 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1 Hr.
			916 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5,2,3 Hr.
			918 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5 Hr.
			919 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5 Hr.
			920 #	2VLI,3VLI	2VLP, 3VLP	1.5 Hr.
		4 1/2" NW	902 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5 Hr.
			916 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5,2,3 Hr.
			918 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5 Hr.
			919 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5 Hr.
3 Hr.	Exposed Grid	3 1/4" NW	216 +	1.5VL,1.5VLI,2VLI,3VLI	2VLP, 3VLP	2,3 Hr.
	Cementitious	2" NW&LW	743 *	2VLI,3VLI	2VLP, 3VLP	1,1.5,2,3 Hr.
		2 1/2" LW	746 *	1.5VLI		1,1.5,2,3 Hr.
		2 1/2" NW&LW	703 *	1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1.5 Hr.
			708 *	1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1.5,3 Hr.
			739 *	1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5,2,3,4 Hr.
			755	1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5,2,3 Hr.
			759	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5,2,3 Hr.
			760 *	2VLI,3VLI		1,1.5,2,3,4 Hr.
		3 1/4" LW	754 *	1.5VLI,2VLI,3VLI		1.5,2 Hr.
		3 1/4" NW	742 *	1.5VLI,2VLI,3VLI		1,1.5 Hr.
	Sprayed Fiber	2" NW&LW	859 *	2VLI,3VLI	2VLP, 3VLP	1,1.5,2,3 Hr.
		2 1/2" NW&LW	816 *	1.5VLI,2VLI,3VLI	2VLP, 3VLP	1.5,2 Hr.
			831 *	2VLI,3VLI	2VLP, 3VLP	1,1.5,2 Hr.
			832 *	1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5,2,3 Hr.
			833 *	1.5VLI,2VLI,3VLI	2VLP, 3VLP	1.5 Hr.
			858	2VLI,3VLI	2VLP, 3VLP	1,1.5,2,4 Hr.
			871 *	2VLI,3VLI	2VLP, 3VLP	1,1.5,2,3 Hr.
		2 1/2" NW	864	3VLI	3VLP	1.5 Hr.
		3 1/4" LW	860 *	2VLI,3VLI		1,1.5,2 Hr.
	Unprotected Deck	4 3/16" LW	902 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5 Hr.
			916 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5,2,3 Hr.
			918 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5 Hr.
			919 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5 Hr.
		5 1/4" NW	902 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5 Hr.
			916 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5,2,3 Hr.
			918 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5 Hr.
			919 #	1.5VL,1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5 Hr.
4 Hr.	Cementitious	2 1/2" NW&LW	760	2VLI,3VLI		1,1.5,2,3,4 Hr.
			739	1.5VLI,2VLI,3VLI	1.5VLP, 2VLP, 3VLP	1,1.5,2,3,4 Hr.
		3 1/4" LW	754	1.5VLI,2VLI,3VLI		1.5,2 Hr.
	Sprayed Fiber	2 1/2" NW&LW	858	2VLI,3VLI	2VLP, 3VLP	1,1.5,2,4 Hr.
		3 1/4" LW	860	2VLI,3VLI		1,1.5,2 Hr.

NOTES:
1. Concrete thickness is thicness of slab above deck, in.
2. Refer to the U.L. "Fire Resistance Directory" for the necessary construction details.
3. Cellular deck finish shall be galvanized.
4. Fluted deck finish shall be galvanized unless noted otherwise.
   - + Denotes fluted deck finish is not critical when used in D2-- & D5-- Series designs. Deck finish shall be galvanized or phosphatized/painted.
   - * Fluted deck finish is critical for fire resitance. Fluted deck finish shall be galvanized or phospatized/painted. This paint is a special type of paint and is compatible with the spray-applied fire protection and is U.L. approved for use in the denoted D7-- & D8-- Series designs.
   - # Denotes fluted deck finish is not critical for fire resistance. Fluted deck finish shall be galvanized or phosphatized/painted.
5. Vulcraft cellular deck units are approved by U.L. for use as electrical raceways under U.L Standard 209.

COMPOSITE

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Appendix E – Composite Steel Deck Design

1

Composite Deck

Live Load = 60 psf
Misc. Dead Load = 10 psf

Super Imp. Live Load = 60 + 10 = 70 psf

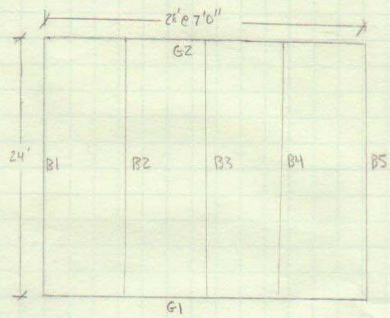
Using Vulcraft Deck Catalog:

1.5VL18 – Unshored clear span: 1 span = 7'6"
2 span = 9'11"
3 span = 10'2"

$f_{allowed}$ = 260 psf

Total Load = 70 + 33 = 103 psf < 260 psf ∴ ok

Span = 7'0"



Fire Protection: 2 hr rating
Protected with Sprayed Fiber
2" of topping
⇒ Use 2VLI

Use 2VLI 21: Unshored Spans – 1 Span = 7'2"
2 Spans = 9'5"
3 Spans = 9'8"

$f_{allowed}$ = 200 psf

Total Load = 70 + 39 = 109 psf < 200 psf ∴ ok

Beam Sizing

Dead Loads: Slab/Deck = 39 psf
Misc = 10 psf
Members = 5 psf
DL = 54 psf

Live Loads: $L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right)$

$A_T = 7'(24') = 168\ ft^2$

$K_{LL} = 1$

$K_{LL}A_T = 168\ ft^2 < 400\ ft^2$ ∴ no red.

Load = 60 psf

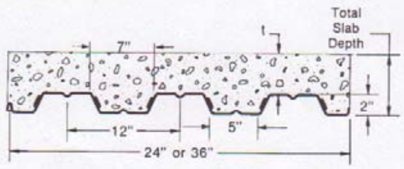
$w_u = 1.2(54) + 1.6(60) = 160.8$ psf

Appendix E – Composite Steel Deck Design

VULCRAFT

2 VLI

Maximum Sheet Length 42'-0
Extra Charge for Lengths Under 6'-0
ICBO Approved (No. 3415)



STEEL SECTION PROPERTIES Fy= 40 KSI

Deck Type	Design Thick.	Weight PSF	Ip in^4/ft	In in^4/ft	Sp in^3/ft	Sn in^3/ft
2VLI22	0.0295	1.62	0.332	0.329	0.274	0.277
2VLI21	0.0329	1.81	0.378	0.375	0.317	0.321
2VLI20	0.0358	1.97	0.418	0.415	0.355	0.360
2VLI19	0.0418	2.30	0.493	0.492	0.435	0.443
2VLI18	0.0474	2.61	0.557	0.557	0.512	0.518
2VLI17	0.0538	2.96	0.633	0.633	0.589	0.589
2VLI16	0.0598	3.29	0.704	0.704	0.653	0.653

(N=9) NORMAL WEIGHT CONCRETE (145 PCF)

Total Slab Depth	Deck Type	SDI Max. Unshored Clear Span			Superimposed Live Load, PSF Clear Span (ft.-in.)														
		1 Span	2 Span	3 Span	5'-6	6'-0	6'-6	7'-0	7'-6	8'-0	8'-6	9'-0	9'-6	10'-0	10'-6	11'-0	11'-6	12'-0	12'-6
	2VLI22	6'-6	8'-9	8'-10	274	239	211	164	145	129	115	104	94	85	78	71	65	59	54
4"	2VLI21	7'-2	9'-5	9'-8	294	255	224	200	155	138	123	111	100	91	83	76	69	64	58
	2VLI20	7'-8	9'-11	10'-3	310	269	236	210	188	146	130	117	106	96	87	80	73	67	62
(t=2")	2VLI19	8'-8	11'-0	11'-4	344	298	261	231	207	186	169	130	117	106	97	88	81	74	68
	2VLI18	9'-6	11'-10	12'-3	373	324	285	253	228	206	188	172	159	122	112	103	95	87	81
39 PSF	2VLI17	10'-4	12'-7	13'-0	400	351	308	273	245	221	201	184	170	157	120	111	102	94	87
	2VLI16	10'-11	13'-2	13'-5	400	376	330	292	261	235	214	195	180	166	154	118	109	100	93
	2VLI22	6'-2	8'-4	8'-5	319	278	217	190	168	150	134	121	109	99	90	83	76	69	63
4 1/2"	2VLI21	6'-9	8'-11	9'-3	341	297	261	204	180	160	144	129	117	106	97	88	81	74	68
	2VLI20	7'-3	9'-5	9'-9	361	313	275	244	190	169	152	136	123	112	102	93	85	78	72
(t=2 1/2")	2VLI19	8'-2	10'-5	10'-10	400	346	303	268	240	216	168	151	136	124	113	103	94	86	79
	2VLI18	9'-0	11'-3	11'-8	400	376	331	295	264	239	218	200	156	142	130	119	110	102	94
45 PSF	2VLI17	9'-9	12'-0	12'-5	400	400	358	318	284	257	234	214	197	153	140	129	118	109	101
	2VLI16	10'-4	12'-7	13'-0	400	400	383	339	303	274	248	227	209	193	150	137	126	117	108
	2VLI22	5'-11	7'-9	8'-0	364	285	247	217	192	171	153	138	125	113	103	94	86	79	72
5"	2VLI21	6'-5	8'-6	8'-10	389	338	266	233	206	183	164	147	133	121	110	101	92	84	78
	2VLI20	6'-11	9'-0	9'-4	400	356	313	246	217	193	173	156	141	128	116	106	97	89	82
(t=3")	2VLI19	7'-9	10'-0	10'-4	400	394	345	306	273	214	192	172	156	141	128	117	107	99	91
	2VLI18	8'-7	10'-9	11'-2	400	400	377	336	301	273	249	195	178	162	148	136	126	116	107
51 PSF	2VLI17	9'-3	11'-6	11'-10	400	400	400	362	324	293	266	244	192	175	160	147	135	125	116
	2VLI16	9'-10	12'-1	12'-5	400	400	400	386	346	312	283	259	238	187	171	157	144	133	123
	2VLI22	5'-8	7'-2	7'-4	400	320	278	244	216	192	172	155	140	127	116	106	97	89	81
5 1/2"	2VLI21	6'-2	8'-2	8'-5	400	379	298	261	231	205	184	166	150	136	124	113	104	95	87
	2VLI20	6'-7	8'-8	8'-11	400	400	351	276	244	217	194	175	158	143	131	119	109	100	92
(t=3 1/2")	2VLI19	7'-5	9'-7	9'-11	400	400	388	343	271	241	215	193	175	159	144	132	121	111	102
	2VLI18	8'-2	10'-4	10'-8	400	400	400	377	338	306	243	219	199	182	167	153	141	130	121
57 PSF	2VLI17	8'-10	11'-0	11'-5	400	400	400	400	364	329	299	237	215	196	180	165	152	140	130
	2VLI16	9'-4	11'-7	12'-0	400	400	400	400	388	350	318	290	230	210	192	176	162	150	138
	2VLI22	5'-5	6'-8	6'-10	400	355	308	270	239	213	191	172	156	141	129	118	108	99	90
6"	2VLI21	5'-11	7'-11	8'-1	400	381	331	290	256	228	204	184	166	151	137	126	115	105	97
	2VLI20	6'-4	8'-4	8'-7	400	400	350	306	271	241	215	194	175	159	145	132	121	111	102
(t=4")	2VLI19	7'-2	9'-3	9'-7	400	400	400	381	301	267	239	215	194	176	160	146	134	123	113
	2VLI18	7'-10	10'-0	10'-4	400	400	400	400	375	299	269	243	221	202	185	170	157	145	134
63 PSF	2VLI17	8'-6	10'-7	11'-0	400	400	400	400	400	364	331	263	239	218	199	183	169	156	144
	2VLI16	9'-0	11'-2	11'-6	400	400	400	400	400	388	352	322	255	233	213	195	180	166	154
	2VLI22	5'-1	6'-2	6'-4	400	390	339	297	263	234	210	189	171	155	141	129	118	108	99
6 1/2"	2VLI21	5'-9	7'-6	7'-6	400	400	363	318	281	250	224	202	183	166	151	138	126	116	106
	2VLI20	6'-1	8'-1	8'-4	400	400	385	337	297	264	237	213	193	175	159	145	133	122	112
(t=4 1/2")	2VLI19	6'-10	8'-11	9'-3	400	400	400	375	330	293	262	236	213	193	176	161	147	135	124
	2VLI18	7'-7	9'-8	9'-11	400	400	400	400	400	329	296	268	243	222	203	187	172	159	147
69 PSF	2VLI17	8'-2	10'-3	10'-7	400	400	400	400	400	400	320	289	262	239	219	201	185	171	158
	2VLI16	8'-8	10'-9	11'-2	400	400	400	400	400	400	387	309	280	256	234	215	198	183	169

Notes:
1. Minimum exterior bearing length required is 2.0 inches. Minimum interior bearing length required is 4.0 inches. If these minimum lengths are not provided, web crippling must be checked.
2. Always contact Vulcraft when using loads in excess of 200 psf. Such loads often result from concentrated, dynamic, or long term load cases for which reductions due to bond breakage, concrete creep, etc. should be evaluated.
3. All fire rated assemblies are subject to an upper live load limit of 250 psf.
4. Inquire about material availability of 17, 19 & 21 gage.

COMPOSITE

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Appendix E – Composite Steel Deck Design

2

Required Loads

$V_u = \frac{(160.8)(7')(24')}{2} = 13507 \text{ lbs} = 13.51 \text{ K}$

$M_u = \frac{w_u l^2}{8} = \frac{(160.8)(7)(24)^2}{8} = 81043 \text{ lbs} = 81.04 \text{ K}$

Using Z_x Table of AISC Steel Construction Manual:

$M_u \leq \phi M_{px}$

W12x19: $\phi M_p = 92.6 \text{ ft-K} > 81.04 \text{ ft-K} \therefore$ ok

$\phi V_n = 85.7 \text{ K} > 13.51 \text{ K} \therefore$ ok

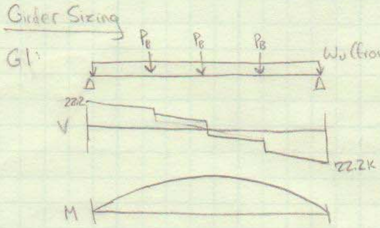
$\Delta_{LL} \leq \frac{l}{360}$ $\quad \Delta_{LL} = \frac{5 w_{LL} L^4}{384 E I_x}$; $I_x = 130 \text{ in}^4$ $\quad \frac{l}{360} = \frac{(24 \text{ ft})(12)}{360} = 0.8 \text{ in}$

$\Delta_{LL} = \frac{5(0.06)(24)^4(1728 \text{ in}^3/\text{ft}^3)(7)}{384(29000)(130 \text{ in}^4)} = 0.833 \text{ in} > 0.8 \text{ in} \therefore$ no good $I_{req} = 135.2 \text{ in}^4$

Use W12x22 $I_x = 156 \text{ in}^4$

Girder Sizing

G1:



w_u (from slab deck spanning 7' corridor)

$P_B = 13.51 \text{ K}$

$w_u = (49 \text{ psf})(3.5') = 171.5 \text{ plf} = 0.172 \text{ klf}$

$V_u = \frac{(0.172)(28) + 3(13.51)}{2} = 22.7 \text{ K}$

$M_u = \frac{(0.172)(28)}{2}\left(\frac{28}{2}\right)\left(\frac{1}{2}\right) + \frac{3(13.51)(7)}{2} + \frac{(13.51)(7)}{2}$

$= 16.86 + 141.9 + 47.3 = 206.1 \text{ ft-K}$

Using Z_x Tables:

W18x35: $\phi M_p = 249 \text{ ft-K} > 206.1 \text{ ft-K} \therefore$ ok $\quad I_x = 510 \text{ in}^4$

$\phi V_n = 159 \text{ K} > 22.7 \text{ K} \therefore$ ok

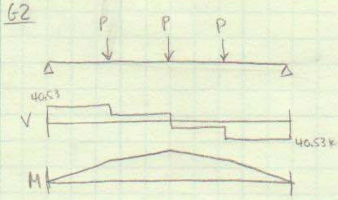
$\Delta_{LL} \leq \frac{l}{360}$ $\quad \Delta_{LL} = \frac{5 w_{LL} L^4}{384 E I_x}$; $w_{LL} = (0.06)(3.5 + 12) = 0.93 \text{ klf}$

$\Delta_{LL} = \frac{5(0.93)(28)^4(1728)}{384(29000)(510)} = 0.087 \text{ in} \ll \frac{l}{360} = \frac{28(12)}{360} = 0.93 \text{ in} \therefore$ ok

Appendix E – Composite Steel Deck Design

3

G2



$P = 13.51\,K$

$V_u = 3(13.51) = 40.53\,K$

$M_u = \frac{V_u L}{4} + \frac{PL}{8} = \frac{40.53(28)}{4} + \frac{13.51(28)}{8} = 331\ \text{ft-K}$

Using Z_x Tables:

$W21\times44:\ \phi M_p = 358\ \text{ft-K} > 331\ \text{ft-K}\ \therefore\ ok$ $\quad I_x = 843\ in^4$

$\phi V_n = 217\,K > 40.5\,K\ \therefore\ ok$

$\Delta_u \leq \frac{l}{360}$

$\frac{l}{360} = \frac{28(12)}{360} = 0.93\ in$

$\Delta_{LL} = \frac{5 w_u L^4}{384 EI} = \frac{5(0.06)(12)(28)^4(1728)}{384(29000)(843)} = 0.41\ in < 0.93\ in\ \therefore\ ok$