Mountain Hotel, Urban Virginia

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Structural Option

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Executive Summary

The objective of this report is to investigate three alternative floor systems that could be implemented into the Mountain Hotel, in which was original design called for pre-cast hollow core concrete planks resting on light gauge bearing walls. Three alternatives, Composite Deck on Beams, One-way Beam Joist, and a Flat Plate were designed to occupy the bay between column lines 6-8 and B - D located on the SE side of the Mountain Hotel in Virginia.

After Evaluation of the three systems it was determined that the most practical alternative floor system for implementation into the Mountain Hotel is a Flat Plate. By general comparison, the Flat Plate is the only alternative which allows for preservation of the existing architectural feature utilizing the flat underside of the floor structure as the ceiling of the floor below for every room in the building, and the only system which allows the building to keep its existing floor to ceiling heights without increasing the overall height of the structure. The thickness of the Flat Plate system requires no additional fireproofing to meet the IBC 2009 criteria. By comparison of the floor systems alone Flat Plate is the cheapest of the alternative systems.

The current system design utilizes light gauge bearing walls for its gravity and lateral system. All of these components will have to therefore be redesigned using concrete frames with an exchange to concrete shear walls for lateral resistance. Foundation bearing capacity will also have to be increased in order to accommodate the more than doubled loads from the new structure.

Although the other two systems, Composite Steel Deck, and One-way Concrete may be theoretically feasible, for design of a typically bay of the Mountain Hotel, they were not as practical as the Flat Plate system. Both would require additional ceiling considerations to deal with the unevenness of the structures underside. The overall impact of the Steel system was comparatively very expensive.

Through analysis of each system summarizing their attributes, it was able to be determined that a Flat Plate floor system was the most practical and realistic alternative to the existing pre-cast hollow core planks on light gauge walls designed into the Mountain Hotel.

Building Introduction

Figure 3-1
An aerial view from bing.com maps with the building superimposed on. Hotel is in Red, Garage in Yellow.

The new hotel is to be located in a wealthy urban area of Virginia (Location shown in Figure 3-1). The site chosen for construction of the new hotel is a prominent location previously occupied by a chain of parking lots, which border the main street of the town.

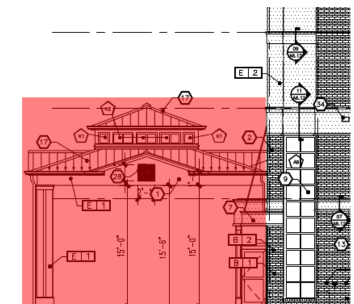


Figure 3-2
Porte Cochere attached to Hotel over main Entrance.

In order to match the new building into its surrounding architecture the first two floor facades are brick with large glazing panels, while the upper facade uses a palette of varying shades from brick red to white which enables it to match the brick and concrete of the surrounding buildings, including the adjacent concrete parking structure. However, in place of the brick or concrete, the upper stories of the hotel use a lighter more cost effective cladding, exterior insulation finishing system (EIFS) panels. The Porte Cochere on the west side, shown in Figure 3-2, will help funnel visitors into the main lobby where they can check-in and be directed to their rooms, other amenities, or sites of the town.

Guest rooms are located on the second through sixth floors totaling just over 40,000 square feet. Though the main function is to appease guests with a home away from home, it also contains meeting rooms for conferences, offices for hotel management, and a 40,000 square foot parking garage. Total building area is approximately 120,000 square feet.

Structural Overview

The hotel rests on reinforced concrete spread footings ranging from 12 to 42 inches in depth. Concrete piers transfer the load into the interior footings from the steel columns. The exterior concrete basement walls rest on strip footings, ranging from 12 to 24 inches, are load bearing and double as sheer walls for the lateral system. A500 Grade B hollow structural steel ranging from four to 16 inches, longer dimension, is used for the superstructure columns. Some of the floors are supported by wide flange beams, ranging from W8 to W21, while others are resting on steel stud bearing walls as shown in Figure 4-1. The lateral system employs a combination of reinforced concrete shear walls, specially reinforced masonry shear walls and light framed wall system with flat strap bracing extending from the ground floor to roof level in both the long and short directions. Floors ground through six are installed as a series of eight inch precast hollow-core planks ranging in length from 9' 2" to 25' 8". The roof is also built of four or eight inch hollow-core planks. Both the brick walls and EIFS system are attached to cold formed steel stud walls. The loading on the exterior facade is transferred through the wall framing to the floors and into the lateral system.

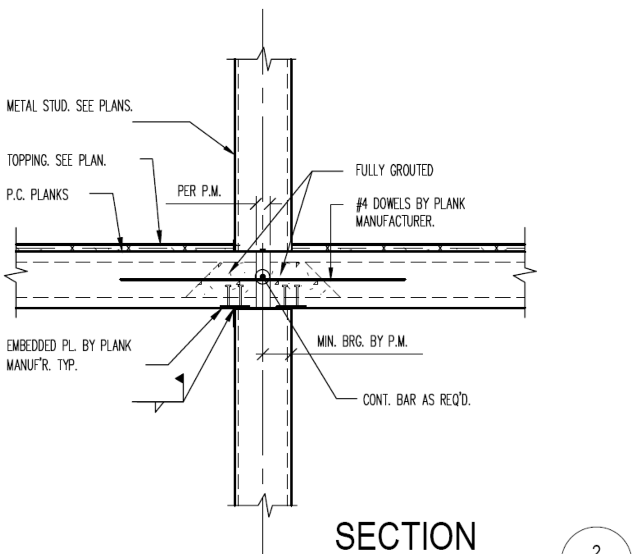


Figure 4-1
Hollow core plank ends spliced and resting on light gauge steel stud framed walls.

The garage is also supported on reinforced concrete spread footings 12 to 30 inches in depth, and strip footers 12 to 24 inches in depth. Piers transfer the load into the footings from the columns and the walls rest directly on the strip footings. Piers and beams are poured monolithic with the walls. Columns support two-way slabs and utilize drop panels, and edge beams.

Code Requirements

Standards and codes governing construction are as follows:

2009 ICC/ANSI A117.1

2009 International Building Code

2009 Virginia Uniform Statewide Building Code

2008 NEC – National Electric Code

2009 ICC – International Mechanical Code

2009 ICC – International Plumbing Code

2009 ICC – International Energy Conservation Code

- All concrete work shall be in accordance with ACI 301, ACI 318 and ACI 302 latest editions.
- All Masonry work shall be in accordance to: ACI 530/ASCE 5, "Building code requirements for Masonry structures"; ACI 530/ASCE 6, "specifications for masonry structures"
- Structural Steel Shall conform to the AISC "Specification for the design fabrication and erection of structural Steel for buildings", Latest edition, except chapter 4.2.1, code of standard practice
- All light gauge framing shall conform to "the specification for the design of cold-formed steel structural members", latest edition, by AISI
- All Wood framing shall conform to the "national design specification for wood construction" latest edition, published by the national forest products association,
- In addition to the requirements included in these structural notes, all construction and materials shall further conform to the applicable provisions of the following standards:
    1. American Society for Testing and Materials (ASTM)
    2. American Concrete Institute (ACI)
    3. National Concrete Masonry Association (NCMA)
    4. American Institute of Steel Construction (AISC)
    5. American Welding Society (AWS)
    6. American Iron and Steel Institute (AISI)
    7. Steel Structures Painting Council (SSPC)
    8. American Forest and Paper Association
    9. National Forest Products Association (NfoPA)

Governing the Parking Garage is all of the above with the exception of:

2006 International Building Code

2006 Virginia Uniform Statewide Building Code

Gravity System

Superstructure

This building uses several types of structural members to carry the various gravity induced loads to the earth. The hotel roof and all above grade floors utilize hollow core planks to support the dead loads of the structure as well as all the amenities people and other items. The planks typically rest on cold-formed steel stud shear walls which pass the load onto the floor below, and so on until it either reaches either a reinforced concrete shear wall or a wide flange beam which it can do so as high as the fourth floor, or as low as the first floor. W-shapes made to the ASTM standard A992 range in size from W6x15 to W33x130. ASTM A500 Hollow Structural Section (HSS), ranging from HSS 4x4x¼ HSS 12x12x½, columns hold the beams in place. Most of the HHS columns terminate in the lower floors; however there are several members that transfer load directly from the roof into the foundations. The Elevator and stair towers are an exception the typical framing types. They use specially reinforced masonry sheer walls to resist both gravity and lateral loads stretching from above the normal roof height and down into the foundation.

Substructure

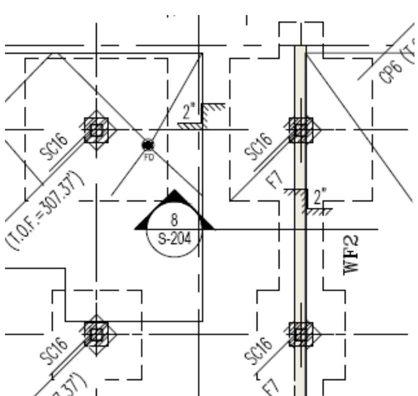


Figure 6-1
Shows a portion of the foundation plan using various structural elements.

The substructure uses a series of reinforced concrete shear walls to transfer the loads from the superstructure into the wall footings of the foundation (Figure 6-1). Under columns and column piers, there is a series of spread footings the largest of which is 16"x16"x42"deep. Footings maintain a minimum compressive strength of 3000psi. Other concrete members have an Fc of 5000psi. Footings rest upon soil which has a bearing pressure of 3000psf.

Typical Gravity Loads

Loads were calculated for the roof and each floor level and were subsequently compared to the design loads specified by the designer. Loads were checked over a single load path using LRFD load combinations from IBC 2009 (Appendix B) from the roof to the foundation. Members sized by author were within a 5% suggesting the designers may have employed Strength Design as well.

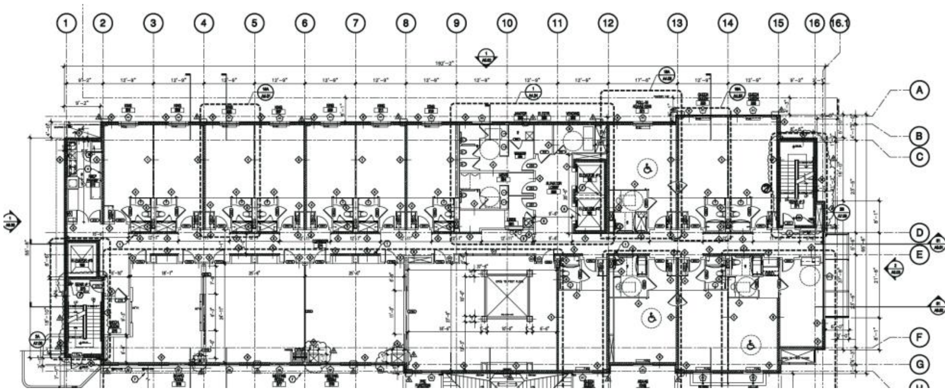


Figure 7-1
Shows Plan of Second Floor. Queen sized rooms shown in the upper left are the typical layout of entire floor for floors above. This floor also shows non-typical rooms: the ADA accessible rooms on the right and the King sized rooms in the lower left.

Floor Systems Analysis

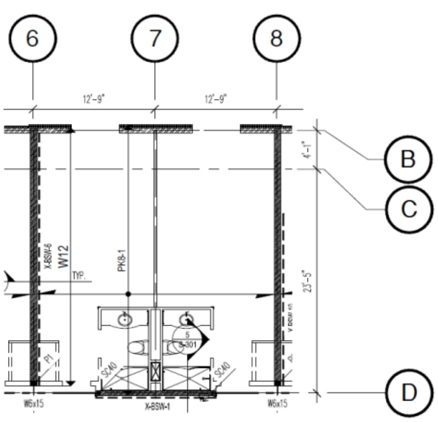


Figure 7-2
Typical Bay of Sheet S-003
Column lines overlaid to show dimensions

Visual analysis of the plans (second floor shown above Figure 7-1) reveals that a typical structural bay for the Mountain Hotel is 25' 6" x 27' 6". A typical bay was chosen between column lines 6 – 8 and D - E for comparative design and analysis (See Figure 7-2). The initial floor system was analyzed and compared to three proposed alternate systems for the Mountain Hotel. All systems were designed and evaluated solely under gravity load. The floor systems considered are: Precast Hollow Core Planks, Composite Deck on Wide Flange Beams, One-way Joist Beam System, and Flat Plate. Each floor system is detailed in its respective section noting advantages and disadvantages. Systems were evaluated based on: fire protection, durability, weight, susceptibility to vibration, cost (Costs data was obtained from RS Means Costworks Online database, tables located in Appendix G), depth, constructability, and aesthetic. A summary comparing the four floor systems can be found in Table XX. Calculations pertaining to the designs can be found in the appendices.

Precast Hollow Core Planks

Description:

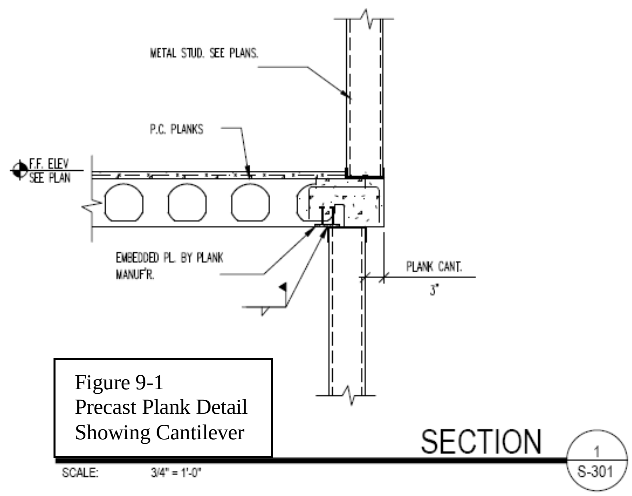


Figure 9-1
Precast Plank Detail
Showing Cantilever

The existing floor system used in all of the above grade floors of the Mountain Hotel is comprised of pre-stressed pre-cast hollow core planks spanning up to 25' 6" and rest on pre-engineered light gauge metal stud shear walls, which transfer the load via steel angles which are imbedded into the planks prior to casting. The exterior plank sticks past the centerline of the exterior support six inches (a three inch cantilever can be seen in the detail shown in Figure 9-1) in order to conform to the four foot module. The floor planks have a one inch gypcrete topping to tie the planks together and produce a continuous surface. The total thickness of the floor system is ten inches.

Advantages:

Precast Hollow Core Planks on light gauge bearing walls are a very efficient choice by almost every metric. It was the cheapest of all the options considered by over two dollars per square foot. The precast planks make construction quick and simple as planks only have to be lifted and set in place once they arrive at the site. Gravity and lateral elements and foundations can be designed using smaller members due to the low unit weight of the hollow core planks. These post-tensioned elements are able to carry significant load over large spans while taking up minimal floor cavity space and still maintaining fire rating criteria. The flat underside gives them the ability to double as an architectural ceiling.

Disadvantages:

Erection time is fast once the planks arrive on site, however the lead time to create the planks could create significant delays on a project, if not ordered within enough advance. Long pieces can have the potential to create problems for a tight site. Though it is possible to create planks in any width, a standard module size generally limits the bay sizes to multiples of four feet. Ability for thin floor systems also opens up vulnerability for issues with vibration, and sound isolation. Due to the lower lateral stiffness of the light gauge walls and the relatively higher weight of the concrete planks this system does not resist seismic forces particularly well.

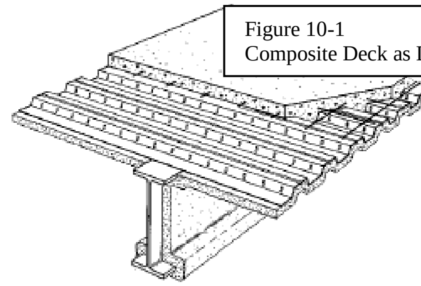


Figure 10-1
Composite Deck as Depicted by RSMeans

Composite Deck on Beams

Description:

Composite deck (depicted above in Figure 10-1) minimalizes need for forms and shoring while utilizing the bottom steel for increased moment capacity. The weight of the concrete, because of the 4.5 inch thickness required by a two-hour fire rating, combined with the 25' 6" span length and relatively small live load (40psf), makes composite beams impractical. Therefore the deck was placed on wide flange beams designed to resist the combined ultimate load. These beams connect into the light gauge bearing walls, therefore no girder or gravity system redesign is required.

Advantages:

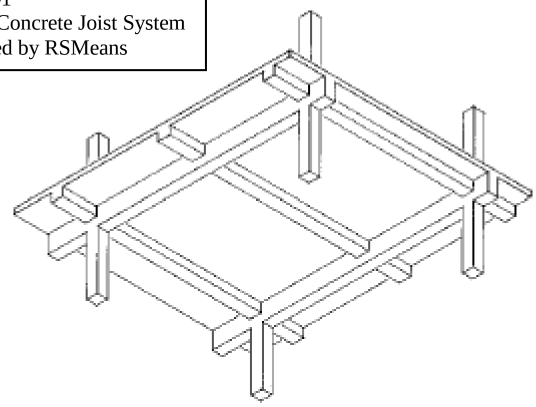
Significant advantages of steel systems include high strength to weight ratio, the convenience of the decking as permanent formwork and ability for a fast paced erection schedule. Chamber can be added to each beam to negate the deflection of dead load. Since steel is a very predictable material it can be designed to resist a greater percentage of its ultimate capacity.

Disadvantages:

W-Shapes give heightened moment capacity but typically sacrifices floor cavity depth. Thinner and lighter members are more susceptible to vibrations. Steel takes more skill to erect, and is generally mined farther from the construction site therefore increasing the lead times. Steel has a greater refinement and therefore cost is higher, especially that of the composite deck. Additional fireproofing is required to allow steel to last a minimum of two hours in a fire.

One-way Concrete

Figure 11-1
One-way Concrete Joist System
as Depicted by RSMeans



Description:

A one-way concrete joist system (depicted in Figure 11-1) has a simple load path from the slab to the beams to the girders to the columns to the foundations to the ground. Deflection criteria and fire rating control depths (five inches), and girders are sized to the assumed width of the columns (18") for ease of constructability. This system would require a redesign of the gravity and lateral systems to create integral moment frames. The second row of central columns was eliminated here in order to simplify design and to create a more uniform system. The larger bay was considered, as it was assumed to be the controlling case.

Advantages:

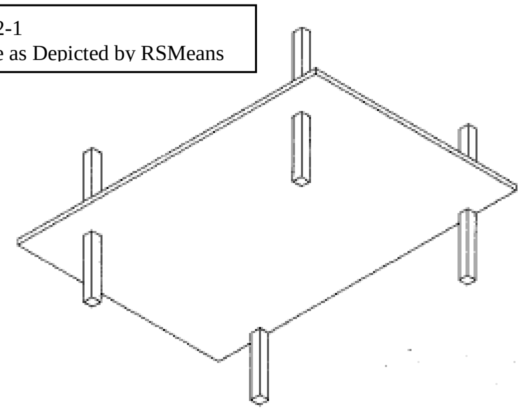
This system allows for the thinnest overall thickness of slab at five inches. Due to the 22 inch deep beams it is one of the stiffest solutions. Monolithic construction creates lateral strength and redundancy by providing moment frames. Concrete structures provide increased damping over steel for resisting seismic loading.

Disadvantages:

The largest inconvenience of this system is the large beam depths, which destroy the architectural appeal of the larger rooms. Concrete requires formwork which has a cost, and more importantly construction schedule is restricted by cure time. Foundations for a concrete structure have to resist more load due to the additional weight.

Flat Plate

Figure 12-1
Flat Plate as Depicted by RSMeans



Description:

Flat Plate (depicted in Figure 12-1) is a two-way concrete system designed to more efficiently utilize the concrete strength and limit the impact resulting from the thickness of a floor cavity. The depth was determined using the shear capacity of the slab without the aid of shear reinforcement, but deflection criteria in Table 9.5(c) ultimately controlled. A stipulation to use a flat plate would also require a redesign of the gravity and lateral systems with concrete columns and shear walls. The second row of central columns was also eliminated here in order to simplify design and to create a more uniform system. The larger bay was considered.

TABLE 9.5(c)—MINIMUM THICKNESS OF SLABS WITHOUT INTERIOR BEAMS*

	Without drop panels‡			With drop panels‡		
	Exterior panels		Interior panels	Exterior panels		Interior panels
f_y, psi†	Without edge beams	With edge beams§		Without edge beams	With edge beams§	
40,000	$\ell_n/33$	$\ell_n/36$	$\ell_n/36$	$\ell_n/36$	$\ell_n/40$	$\ell_n/40$
60,000	$\ell_n/30$	$\ell_n/33$	$\ell_n/33$	$\ell_n/33$	$\ell_n/36$	$\ell_n/36$
75,000	$\ell_n/28$	$\ell_n/31$	$\ell_n/31$	$\ell_n/31$	$\ell_n/34$	$\ell_n/34$

*For two-way construction, ℓ_n is the length of clear span in the long direction, measured face-to-face of supports in slabs without beams and face-to-face of beams or other supports in other cases.
†For f_y between the values given in the table, minimum thickness shall be determined by linear interpolation.
‡Drop panels as defined in 13.2.5.
§Slabs with beams between columns along exterior edges. The value of α_f for the edge beam shall not be less than 0.8.

Advantages:

This two-way system is very good at reducing the overall thickness compared to a one-way slab. Because of the flat underside of the slab there would be no adverse effect to the architectural ceiling. Flat plates have simple formwork and combined with the relative thickness results in a low cost solution. The concrete mass will provide good resistance to vibrations and deflections, and will have increased damping to combat seismic forces.

Disadvantages:

Compared to the existing, foundations would have to be designed for a much greater load. Flat plate are the most complicated to design of the proposed systems due to the complicated three dimensional load path. As a result, they are not very flexible for future design alterations.

Systems Summary

The chart provided below Figure 13-1 is a simple summary overviewing the strengths and weaknesses of the four systems previously described. The final row is based on a personal opinion about whether the system is worthwhile to implement as an alternative floor design with reasons listed if it has been determined that a system should be excluded from consideration.

Figure 13-1 Systems Summary	Systems			
	Existing	Alternatives		
Consideration	Precast Hollow Core Planks	Composit Steel Deck on W-Shapes on Shear Walls	One-way Concrete Joist System	Two-way Flat Plate
General Information				
Weight	57 psf	66.8 psf	79.5 psf	118 psf
Fire Rating	2-Hr	2-Hr	2-Hr	2-Hr
Fire Protection	Thickness of Planks Adaquate for Fire Protection	Requires Additional Fireproofing for underside of Deck and Beams	Thickness of Slab Controlled by Fire Protection Criteria	Thickness of Slab Adaquate for Fire Protection
Architectural				
Bay Size	25' 6" x 27' 6"	25' 6" x 27' 6"	25' 6" x 33' 0"	25' 6" x 33' 0"
Overall Depth	10"	14"	22"	9.5"
Slab Depth	10"	6"	5"	9.5"
Ceiling Height	8' 6"	8' 2"	8' 11"	8' 6.5"
Other	Exposed Ceilings	Requires a Ceiling	Can Expose Ceilings for Queen Rooms Only Without Obstructions	Exposed Ceilings
Structural				
Gravity System Considerations	No Change	Special Considerations for Attachment of Beams to Walls	Redesign using Concrete Columns	Redesign using Concrete Columns
Lateral System Considerations	No Change	No Change	Redesign using Concrete Moment Frames	Change From Light Gauge to Concrete Shear Walls
Foundation Considerations	No Change	Very Similar	Increase Foundation Size to Carry Larger Building Weight	Increase Foundation Size to Carry Larger Building Weight
Construction				
Assembly Cost	$13.23/sf	$23.9/sf	$15.78/sf	$15.24/sf
Formwork Required	None	Minimal	Yes	Yes
Constructability	Easy	Slightly Moderate	Slightly Difficult	Moderate
Lead Time	Long	Moderate	Moderate	Moderate
Servicability				
Vibration and Deflection Control	Slightly Moderate	Moderate	Great	Good
Feasible	Yes	No	No	Yes
Reason		Significant increase in price, requires a ceiling, reduces ceiling height	King and ADA Rooms would have a low ceiling height due to 22" deep beam in center of ceiling	

Conclusion

Three alternative systems were designed in an attempt to size them with proper loads and fit them within the existing parameters and function of the Hotel. Out of the three alternative systems, two can be eliminated because they are costly and or create undesirable conditions in the buildings.

The Composite Steel Deck system was eliminated first, because it has a significantly higher cost. It also created a need for a design of a drop ceiling to cover the appearance of the underside of the deck and the fireproofed beams. The initial design called only for painted ceilings which due to the flat underside of the planks gave it a pleasing aesthetic for a hotel room which the steel system did not share. It was also not desirable to lower ceiling heights or extend overall height of building to accommodate greater system depth.

At first glance the One-Way Concrete system appears to be of similar cost to that of the Flat Plate. When the beams are placed over the current position of the walls the system appears to create a greater floor to ceiling height in queen sized guest rooms. However, when a larger room such as requires for rooms containing king beds or requiring extra space for ADA accessibility, the double bay condition places an awkward 22 inch deep beam (17 inches below the ceiling height) and also restricts floor to ceiling height. This issue could be resolved by either creating a drop ceiling which in its current depth would be low, or extending the floor cavity and raising the overall height of the building, which would create significant additional cost. Neither of these was desirable and the design was therefore omitted from consideration.

This leaves only the Flat Plate system which besides additional weight can deliver performance similar to the existing system, at a similar cost. Flat Plate was therefore chosen as the most effective alternative floor system.

Appendix A – Design Loads

Roof Loads					
Snow Load					
	Ground Snow Load, Pg			30	PSF
	Flat Roof Snow Load, Pf = Ce*Pg			21	PSF
	Snow Exposure Factor, Ce			0.7	
	Snow Load Importance Factor, I			1.0	
	Snow Drift & Sliding Surcharge:				Per Code Requirements
Roof Load					
	Roof Live Load (Horizontal Projection)			37	PSF minimum
	Dead Load			81.5	PSF
Floor Loads (PSF)					+ Mechanical Unit Weight per MEP Drawings
	Dead Load	Live Load	Total Load		
Living Areas	81.5	40	121.5		
Common Areas	75.5/81.5	100	175.5/181.5		
Stairs	50	100	150		
Storage (Light)	81.5	125	206.5		
First Floor Only	106.5	100	206.5		
Wind Loads					
Basic Wind Speed (3- Second Gust)				90	MP
Occupancy Category				II	
Wind Importance Factor, 1				1.0	
Wind Exposure				B	
Internal Pressure Coefficient, Gcpi				±0.18	
Components and Cladding				21	PSF
Earthquake Loads					
Seismic Importance Factor, 1				1.0	
Mapped Spectral Response Accelerations				Ss = 15.5	% g
				S1 = 5.1	% g
Site Class				D	
Spectral Response Accelerations				Sds = 16.5	% g
				Sd1 = 8.2	% g
Seismic Design Category				B	
Design Base Shear				208	
Seismic Response Coefficients				0.028	
Response Modification Factors				4	
Foundation					
Footing Design Soil Bearing Pressure				3000	PSF
Back Fill Material Equivalent Fluid Pressure				55	PSF/FT
Deflection Limits					
		Live Load	Total Load		
Floor		SPAN/480	SPAN/360		
Floor Under Ceramic Tile		SPAN/720	SPAN/360		
Roof Trusses		SPAN/360	SPAN/240		
Roof Rafters		SPAN/240	SPAN/180		
Ceiling Joist		SPAN/360	SPAN/240		
Roof Ridge/Beam		SPAN/360	SPAN/120		

Appendix B – Gravity Loads

Snow Loads | Tech 1

Using ASCE 7-10

Figure 7-1 → $p_g = 25^{psf}$

Table 7-2 → $C_e = 1.0$

Table 7-3 → $C_t = 1.0$

Importance Factor → 1.0

eq 7.3-1 → $p_f = .7\,C_e C_t I_s p_g = (.7)(1.0)(1)(1)(25^{psf}) = 17.5^{psf}$

l_u in long direction $= 190^{ft}$
in short direction $= 62^{ft}$

Figure 7-9 → $h_{d(190)} = .43\sqrt[3]{190}\,\sqrt[4]{25+10} - 1.5 = 4.51^{ft} \times .75 = 3.38^{ft}$

$h_{d(62)} = .43\sqrt[3]{62}\,\sqrt[4]{25+10} - 1.5 = 2.64^{ft} \times .75 = 1.98^{ft}$

eq 7.7-1 → $\gamma = .13p_g + 14 = .13(25) + 14 = 17.3^{lb/ft^3}$

$h_b = p_f/\gamma = 17.5/17.3 = 1.01^{ft}$

$h_c = 9^{ft} - 1.01^{ft} = 7.99^{ft} \rightarrow 8^{ft}$

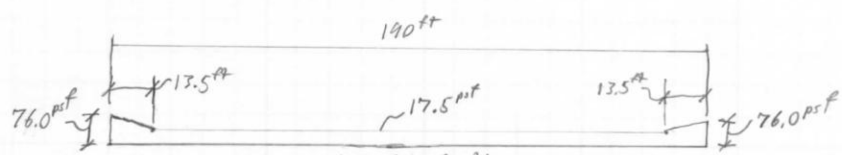
→ (drift width) $w_{(190)} = 4(3.38) = 13.5^{ft}$

$w_{(62)} = 4(1.98) = 7.92^{ft}$

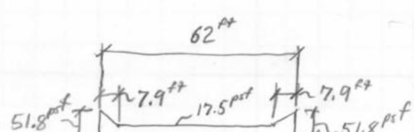
drift surcharge

$p_{d(190)} = 3.38^{ft}(17.3^{lb/ft^3}) = 58.5^{psf}$

$p_{d(62)} = 1.98^{ft}(17.3^{lb/ft^3}) = 34.3^{psf}$



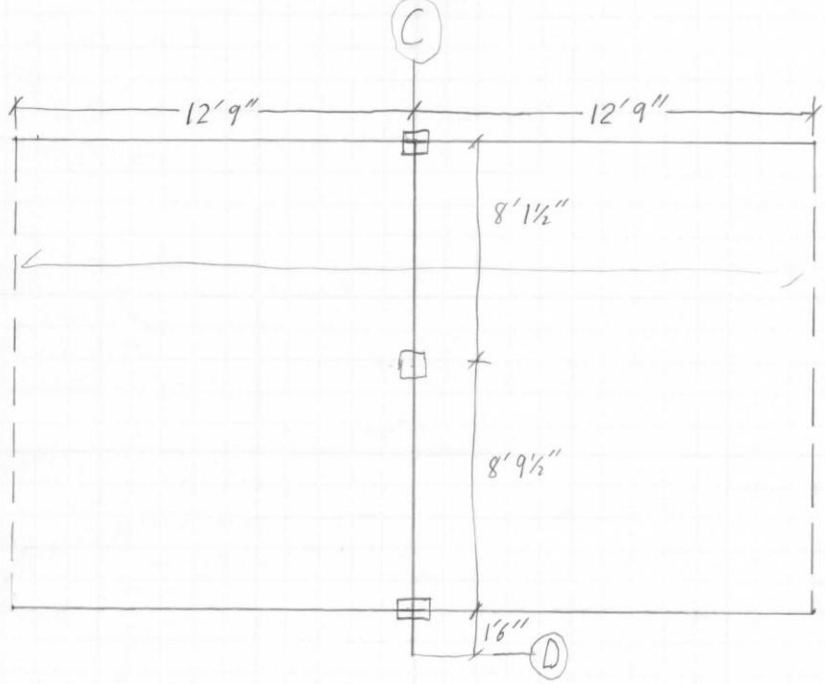
North-South Section



East-West Section

Gravity Check | Tech 1

Column Line 8 between C & D



Roof Loads: $1.2D + 1.6L_r = 1.2(81.5^{psf}) + 1.6(37^{psf}) = 157^{psf} \times 25'6" = 4^{k}/ft$

Floor Loads: $1.2D + 1.6L = 1.2(81.5) + 1.6(40) = 161.8^{psf} \times 25'6" = 4.13^{k}/ft$

Walls negligible weight, strength →
roof → W8 = 33^{ksi} @ 16" O.C. 800S200-43
6^{th} → W10 = 50^{ksi} @ 16" O.C. 800S200-68
5^{th} → W11 = 50^{ksi} @ 16" O.C 800S200-97
4^{th} → W12 = 50^{ksi} @ 16" O.C. 800S200-97x2
3^{rd} → W12 = 50^{ksi} @ 16" O.C.
2^{nd} → SC5 & W12
1^{st} → SC5 & W16x50 → 10' → $276^{k\text{-}ft}$
B → SC5 → HSS8x8x3/8 → @12' → 372^{k}, 14' → 353^{k}
Base Plate 3/4"x14"x14" → 1'4" below
Footing 31'x9'x28"
Longitudinal Bars
Top: #6 @ 12" O.C.
Bottom #7 @ 12" O.C.
Transverse Bars
Top #4 @ 12" O.C.
Bottom #6 @ 12" O.C.

Page 1 of 2

Gravity Check | Tech 1

Roof $= 4^{k/ft}$

$6^{th} = 4.13 + 4 = 8.13^{k/ft}$

$5^{th} = 2(4.13) + 4 = 12.26^{k/ft}$

$4^{th} = 3(4.13) + 4 = 16.39^{k/ft}$

$3^{rd} = 4(4.13) + 4 = 20.52^{k/ft}$

$2^{nd} = 5(4.13) + 4 = 24.65^{k/ft}$

$1^{st} = 6(4.13) + 4 = 28.78^{k/ft} \rightarrow \frac{wl^2}{8} = \frac{(28.78)(8'9\frac{1}{2}'')^2}{8} = 278^{k\text{-}ft} > 276^{k\text{-}ft}$ NG

$< 5\%$

$B = (28.78^{k/ft} + .05) \times \left(\frac{8'9\frac{1}{2}'' + 8'1\frac{1}{2}''}{2}\right) = 243.9^{k} < 353^{k}$ ✓

footing $\rightarrow \frac{243.9^{k}}{(14'' \times 14'')} = 1{,}244^{psi} < 3000^{psi}$ ✓

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Appendix C – Design Loads

Design Loads

PCI Design Handbook Seventh Edition (2010)

Chapter 3.7 → Spancrete Section Properties

→ 8"x4' → untopped = 63 lb/ft², with 2" topping = 88 lb/ft²

weight of 1" topping = $\frac{1}{2}(88-63) = 12.5$ lb/ft²

weight with 1" topping = $63+12.5 = 75.5$ lb/ft²

$W_u = 1.2D + 1.6L$ area = 25.6'x4' = 102.4 ft² → cannot reduce

$W_u = 1.2(75.5) + 1.6(65) = 194.6$ lb/ft² × 4 ft = 779 lb/ft

$M_u = \frac{wl^2}{8} = \frac{779(25.5)^2}{8} = 63.3^{k\text{-}ft}$

$M_n = \frac{f_y I}{y} = \frac{5000^{psi}(1806^{in^4})}{3.98^{in}} \times \frac{k}{1000^{lb}} \times \frac{ft}{12\ in} = 189^{k\text{-}ft}$

$M_u \le \phi M_n \rightarrow 63.3 \le .9(189) \rightarrow 63.3 \le 170$ OK

Appendix D – Table A.7

TABLE A.7
Maximum number of bars as a single layer in beam stems

Bar No. Inch-Pound	SI	$\frac{3}{4}$ in. Maximum Size Aggregate, No. 4 (No. 13) Stirrups[a] — Beam Width b_w, in. 8	10	12	14	16	18	20	22	24	26	28	30
5	16	2	4	5	6	7	8	10	11	12	13	15	16
6	19	2	3	4	6	7	8	9	10	11	12	14	15
7	22	2	3	4	5	6	7	8	9	10	11	12	13
8	25	2	3	4	5	6	7	8	9	10	11	12	13
9	29	1	2	3	4	5	6	7	8	9	9	10	11
10	32	1	2	3	4	5	6	6	7	8	9	10	10
11	36	1	2	3	3	4	5	5	6	7	8	8	9
14	43	1	2	2	3	3	4	5	5	6	6	7	8
18	57	1	1	2	2	3	3	4	4	4	5	5	6

Bar No. Inch-Pound	SI	1 in. Maximum Size Aggregate, No. 4 (No. 13) Stirrups[a] — Beam Width b_w, in. 8	10	12	14	16	18	20	22	24	26	28	30
5	16	2	3	4	5	6	7	8	9	10	11	12	13
6	19	2	3	4	5	6	7	8	9	9	10	11	12
7	22	1	2	3	4	5	6	7	8	9	10	10	11
8	25	1	2	3	4	5	6	7	7	8	9	10	11
9	29	1	2	3	4	5	6	7	7	8	9	9	10
10	32	1	2	3	4	5	6	6	7	7	8	9	10

[a]Minimum concrete cover assumed to be $1\frac{1}{2}$ in. to the No. 4 (No. 13) stirrup.
Source: Adapted from Ref. 3.8. Used by permission of American Concrete Institute.

TABLE A.8
Minimum number of bars as a single layer in beam stems governed by crack control requirements of the ACI Code

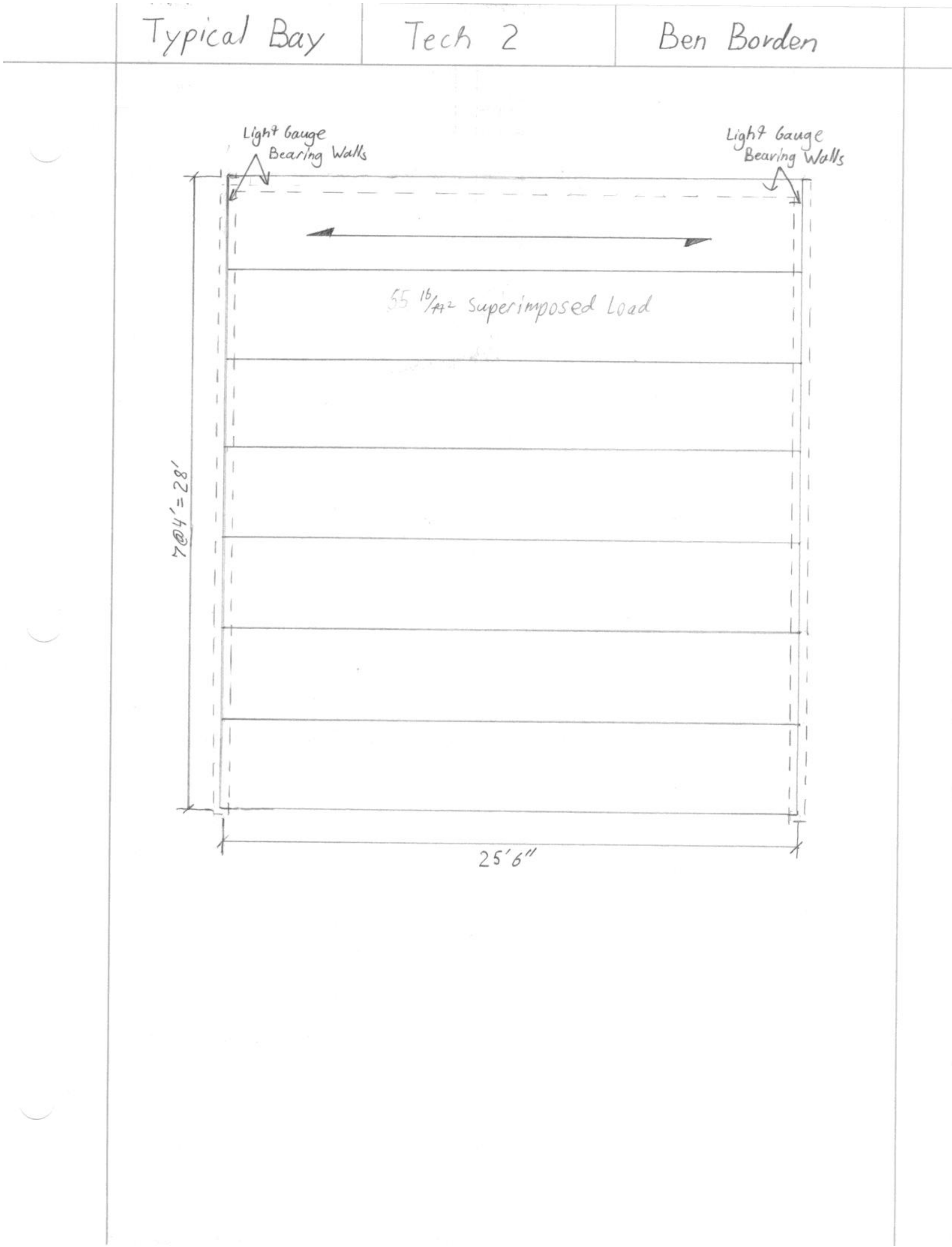
(a) 2 in. clear cover, sides and bottom

Bar No. Inch-Pound	SI	Minimum Number of Bars as a Single Layer of a Beam Stem — Beam Stem Width b_w, in. 8	10	12	14	16	18	20	22	24	26	28	30	32	34	36
3–14	10–43	1	1	2	2	3	3	3	3	3	4	4	4	4	4	5
18	57	1	1	2	2	2	3	3	3	3	3	4	4	4	4	4

(b) $1\frac{1}{2}$ in. clear cover, sides and bottom

Bar No. Inch-Pound	SI	Minimum Number of Bars as a Single Layer of a Beam Stem — Beam Stem Width b_w, in. 8	10	12	14	16	18	20	22	24	26	28	30	32	34	36
3–4	10–13	1	1	2	2	3	3	3	3	3	4	4	4	4	4	4
5–14	16–43	1	1	2	2	3	3	3	3	3	3	4	4	4	4	4
18	57	1	1	2	2	2	3	3	3	3	3	4	4	4	4	4

Appendix E – Precast Hollow Core Planks



Design Loads

PCI Design Handbook Seventh Edition (2010)

Chapter 3.7 → Spancrete Section Properties

→ 8"x4' → untopped = $63^{lb/ft^2}$, with 2" topping = $88^{lb/ft^2}$

weight of 1" topping $= \frac{1}{2}(88-63) = 12.5^{lb/ft^2}$

weight with 1" topping $= 63 + 12.5 = 75.5^{lb/ft^2}$

$W_u = 1.2D + 1.6L$ area $= 25.6 \times 4' = 102.4^{ft^2}$ → cannot reduce

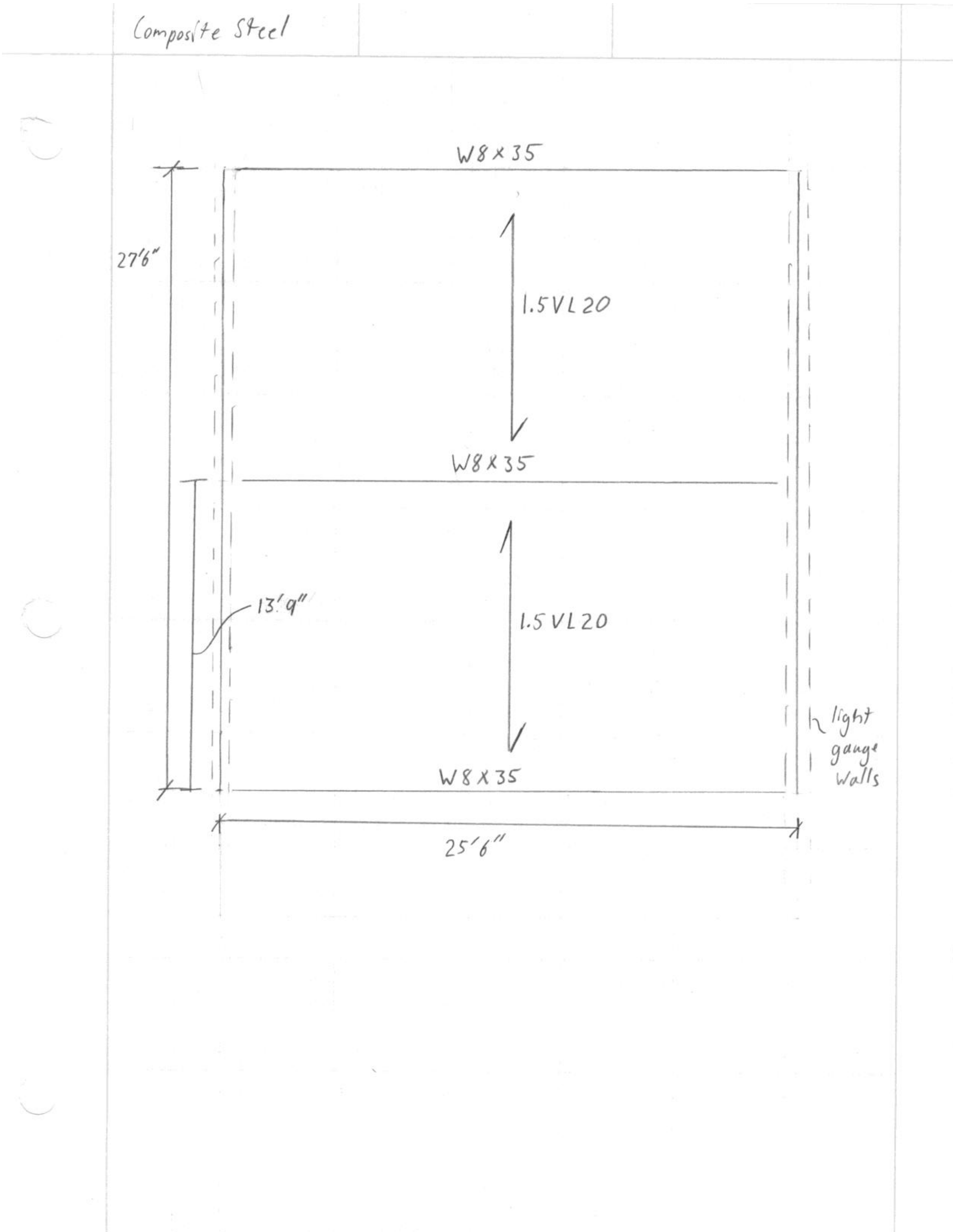
W_u $1.2(75.5) + 1.6(65) = 194.6^{lb/ft^2} \times 4^{ft} = 779^{lb/ft}$

$M_u = \frac{W\ell^2}{8} = \frac{779(25.5)^2}{8} = 63.3^{k\text{-}ft}$

$M_n = \frac{f_y I}{y} = \frac{5000^{psi}(1806^{in^4})}{3.98^{in}} \times \frac{k}{1000^{lb}} \times \frac{ft}{12\,in} = 189^{k\text{-}ft}$

$M_u \leq \phi M_n \rightarrow 63.3 \leq .9(189) \rightarrow 63.3 \leq 170$ OK

Appendix F – Steel Deck Calculations



Composite Steel

Vulcraft steel deck catalog

2hr fire rating, unprotected, NW → $4\frac{1}{2}^{in}$ thickness

Try 1.5VL

for 28′ → 4×7′0″ spans   3 span condition

SDI Max Span → 1.5VL20 → 7′6″

→ Superimposed Load @ 7′ = $383^{psf} \gg 40^{psf}$ OK

Design Composite Steel Beams

Slab and deck weight = 63^{psf}

$W_D = 63^{psf}$

$W_L = 40^{psf}$ → cannot reduce < 400^{ft^2}

$W_T = 1.2D + 1.6L = 1.2(63^{psf}) + 1.6(40^{psf}) = 140^{psf}$

$W_u = 140^{psf} \times 7^{ft} = 980^{lb/ft}$

$$M_u = \frac{wl^2}{8} = \frac{.98(25.5)^2}{8} = 79.7^{k\text{-}ft}$$

Table 3-10 → W8×35

Check Live Load Deflection

$W_{LL} = 40^{psf} \times 7^{ft} = .21^{k/ft}$

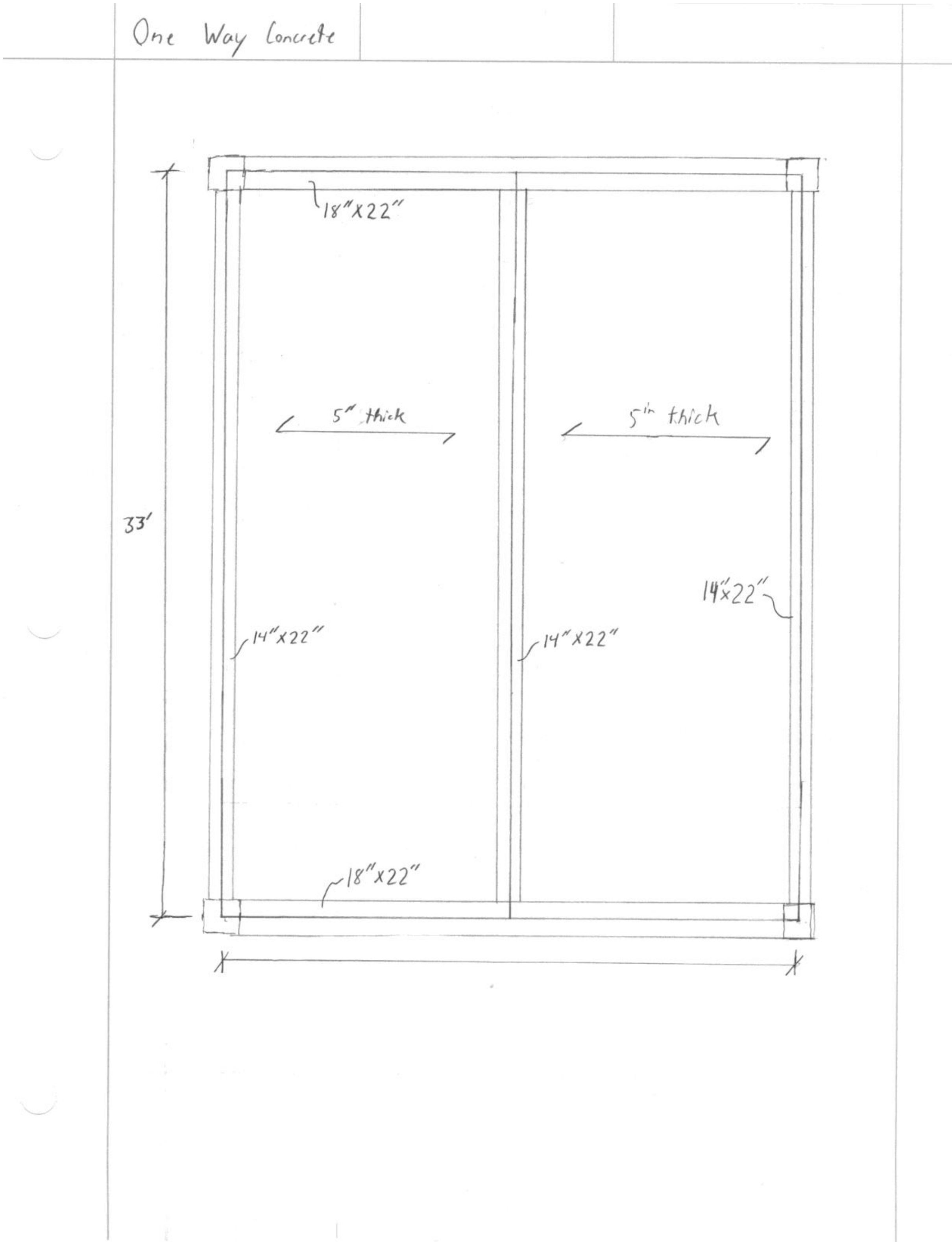
$$\Delta = \frac{5w_Ll^4}{384EI} = \frac{5(.21^{k/ft})(25.5')^4(12^{in/ft})^3}{384(29{,}000^{ksi})(127^{in^4})} = .542^{in} \rightarrow \frac{.542^{in}}{306^{in}} = \frac{1}{565}^{in}$$

OK unless ceramic tile

Floor System Weight

$$\frac{63^{psf} \times 27.5' \times 25.5' + 3(35^{lb/ft} \times 25.5')}{27.5' \times 25.5'} = 66.8^{psf}$$

Appendix G – One-way Concrete Calculations



One-way Concrete

2 hour fire rating → minimum slab thickness of 5^{in}

try 6^{in} w/ $12^{ft}9^{in}$ spans between beams due to architectural consideration

Misc DL

$W_D = 150^{lb/ft^3} \times \frac{5}{12}^{ft} \times 28^{ft} + 20^{lb/ft^2} \times 28^{ft} = 2.31^{k/ft}$

$W_L = 40^{lb/ft^2} \times 28^{ft} = 1.12^{k/ft}$

assumes 9" beams

$W_U = 1.2(2.31^{k/ft}) + 1.6(1.12^{k/ft}) = 4.56^{k/ft}$

$M_u^+ = \frac{W_u l_n^2}{16} = \frac{(4.56^{k/ft})(12')^2}{16} = 41.0^{k\text{-}ft}$

$A_s = \frac{M_u}{4d} = \frac{41.0^{k\text{-}ft}}{4(3.25)} = 3.15^{in^2}$

$M_u^- = \frac{W_u l_n^2}{11} = \frac{(4.56^{k/ft})(12^{ft})^2}{11} = 59.7^{k\text{-}ft}$

$A_s = \frac{M_u}{4d} = \frac{59.7^{k\text{-}ft}}{4(3.25)} = 4.59^{in^2}$

$s \geq_{min} \begin{cases} 15\left(\frac{40,000}{\frac{2}{3}(60,000)}\right) - 2.5(1.5) = 11.25^{in} \leftarrow \text{controls} \\ 12(40\,000/(\frac{2}{3}(60,000))) = 12^{in} \end{cases}$

$\rightarrow 31^{bars}$

$A_s/\#_{bars} = \frac{3.15}{31} = .101 \rightarrow \#3;\ A_s = .11^{in^2}$

$A_s/\#_{bars} = \frac{4.59^{in^2}}{31} = .148 \rightarrow \#4;\ A_s = .20^{in^2}$

$A_{s,max} = .85\beta_1 \frac{f'c}{f_y} \frac{\varepsilon_u}{\varepsilon_u + .004} bd$

$= (.85)^2 \frac{4}{60} \frac{.003}{.007}(28\times12)(3.25) = 22^{in^2} \rightarrow OK$

$A_{s,min} = max \begin{cases} \frac{3\sqrt{f'c}}{f_y} b_w d \rightarrow 3\sqrt{4000} = 190 < 200 \\ \frac{200 b_w d}{f_y} = \frac{200(28\times12)(3.25)}{60,000} = 3.64^{in^2} \end{cases}$

$\rightarrow$ use #4; $A_s = .20^{in^2}$

$a = \frac{A_s F_y}{.85 f'c\, b} = \frac{(6.2)(60)}{.85(4)(336)} = .326^{in}$ $\quad c = \frac{a}{\beta_1} = \frac{.326}{.85} = .384^{in}$

$\varepsilon_s = \frac{\varepsilon_u}{c}(d-c) = \frac{.003}{.384}(3.25 - .384) = .0224 \geq .005$

$\rightarrow \phi = .9$

$\phi M_n = \phi A_s f_y (d - a/2) = .9(6.2)(60)(3.25 - \frac{.326}{2}) = 1034^{k\text{-}in} \rightarrow 86.1^{k\text{-}ft} > M_u^+ \checkmark$

$> M_u^- \checkmark$

$A_t = .0018\, bh = .0018(153)(5) = 1.38^{in^2} \rightarrow \#3 @ 12^{in}$ o.c. $\rightarrow 1.43^{in^2} > 1.38 \checkmark$

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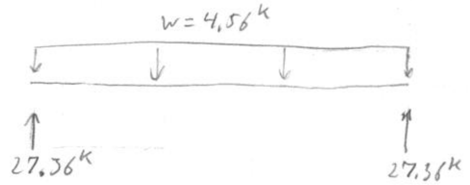
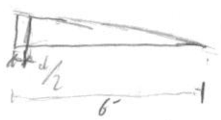
One-way Concrete

check shear





$V_u = \dfrac{27.36^k(72'' - \frac{3.25''}{2})}{72''} = 26.125^k$

$V_c = 2\lambda\sqrt{f'c}\,b_w d = 2(1)\sqrt{4000^{psi}}(28 \times 12'')(3.25'') = 138^k$

$V_u \geq \phi V_n = 1.0\,\phi V_c = (1)(.75)(138^k) = 103.5^k$ ✓

Design beams

$W_D = 150^{lb/ft^3}\left(\frac{5}{12}\right)'(12.75') + 20^{lb/ft^2}(12.75') = 1.05^{k/ft}$

$W_L = 40^{lb/ft^2} \times 12.75' = .51^{k/ft}$

$W_u = 1.2(1.05^{k/ft}) + 1.6(.51^{k/ft}) = 2.076^{k/ft}$

$M_u^- = \dfrac{W_u l_n^2}{9} = \dfrac{2.076^{k/ft}(31.5)^2}{9} = 228.9^{k\text{-}ft} \times 1.1 = 251.8^{k\text{-}ft}$ allowance for self-weight; assumes 18'' girders

$b_{eff} = \min \begin{cases} b_w + 16h_f = 80'' + b_w \rightarrow \text{assume } b_w = 10^{in} \rightarrow 90^{in} \leftarrow \text{controls} \\ \text{trib width} = 153'' \\ 1/4\,l = 94.5'' \end{cases}$

Table 9.5(a) $\rightarrow h \geq l/18.5 = 31.5/18.5 = 21.4^{in} \rightarrow 22^{in} \rightarrow d = 19.5^{in}$

$bd^2 = 20M_u \rightarrow b = \dfrac{20(251.8)}{19.5^2} = 13.2^{in} \rightarrow \text{try } 14^{in} \rightarrow b_{eff} = 94^{in}$

$W_{sw} = 150^{lb/ft^3}\left(\dfrac{17^{in} \times 14^{in}}{144}\right) = .248^{k/ft}$

$W_u' = 2.076^{k/ft} + 1.2(.248^{k/ft}) = 2.37^{k/ft}$

$M_u^- = \dfrac{W_u l_n^2}{9} = \dfrac{2.37^{k/ft}(31.5^{ft})^2}{9} = 261.3^{k\text{-}ft}$

$A_s = \dfrac{M_u}{4d} = \dfrac{261.3^{k\text{-}ft}}{4(19.5)} = 3.35^{in^2} \rightarrow (3)\#10 \rightarrow 3(1.27^{in^2}) = 3.81^{in^2}$

assume #3 stirrup $\rightarrow d = 19.4^{in}$

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One-way Concrete

$$a = \frac{A_s f_y}{.85 f'_c b} = \frac{(3.81^{in^2})(60)}{.85(4)(14^{in})} = 4.80^{in} \qquad c = a/\beta_1 = \frac{4.80^{in}}{.85} = 5.65^{in}$$

$$\varepsilon_s = \frac{\varepsilon_u}{c}(d-c) = \frac{.003}{5.65}(19.4-5.65) = .0073 > \varepsilon_y > \varepsilon_t \to \phi = .9$$

$$\phi M_n = \phi A_s f_y (d - a/2) = .9(3.81^{in^2})(60^{ksi})(19.4'' - \frac{4.80''}{2}) = 291.5^{k\text{-}ft} > M_u^{-int}$$

$$A_{s,min} = max\begin{cases} \frac{3\sqrt{f'_c}}{f_y} b_w d \\ \frac{200 b_w d}{f_y} \to \frac{200(14^{in})19.4^{in}}{60{,}000^{ksi}} = .905^{in^2} < A_s \end{cases}$$

$$M_u^{-ext\ beam} = \frac{w_u l_n^2}{24} = \frac{(2.37^{k/ft})(31.5^{ft})^2}{24} = \frac{2351.6^{k\text{-}ft}}{24} = 98.0^{k\text{-}ft} \to \frac{98^{k\text{-}ft}}{4(19.4)} = 1.23^{in^2} \to 3(\#6) \to 1.32^{in^2}$$

$$M_u^{-ext\ column} = \frac{w_u l_n^2}{16} = \frac{2351.6^{k\text{-}ft}}{16} = 147.0^{k\text{-}ft} \to \frac{147}{4(19.4)} = 1.89^{in^2} \to 2(\#9) \to 2^{in^2}$$

$$M_u^{+} = \frac{w_u l_n^2}{14} = \frac{2351.6^{k\text{-}ft}}{14} = 168.0^{k\text{-}ft} \to \frac{168}{4(19.4)} = 2.17^{in^2} \to 3(\#8) \to 2.37^{in^2}$$

$$a^{-ext\ beam} = \frac{A_s f_y}{.85 f'_c b} = \frac{1.32^{in^2}(60)}{.85(4)(14)} = 1.66^{in} \quad c = \frac{1.66}{.85} = 1.95 \quad \varepsilon_s = \frac{.003}{1.95}(19.4-1.95) = .0274 > \varepsilon_y > \varepsilon_t \to \phi = .9$$

$$a^{-int\ column} = \frac{2(60)}{.85(4)(14)} = 2.52^{in} \quad c = \frac{2.52}{.85} = 2.96 \quad \varepsilon_s = \frac{.003}{2.96}(19.4-2.96) = .0167 > \varepsilon_y > \varepsilon_t \to \phi = .9$$

$$a^{+} = \frac{(2.37)(60)}{.85(4)(94)} = .445 < 5^{in} \quad c = \frac{.445}{.85} = .524 \quad \varepsilon_s = \frac{.003}{.524}(19.4-.524) = .108 > \varepsilon_y \quad \varepsilon_t \to \phi = .9$$

$$\phi M_n^{-ext\ beam} = (.9)(1.32)(60)(19.4 - \frac{1.66}{2}) = 110.3^{k\text{-}ft} > M_u^{-ext\ beam} \checkmark$$

$$\phi M_n^{-ext\ column} = (.9)(2)(60)(19.4 - \frac{2.52}{2}) = 163.3^{k\text{-}ft} > M_u^{-ext\ column} \checkmark$$

$$\phi M_n^{-ext\ column} = (.9)(2.37)(60)(19.4 - \frac{.445}{2}) = 204.5^{k\text{-}ft} > M_u^{+} \checkmark$$

$$V_u^{ext} = \frac{w_u l}{2} = \frac{(2.37^{k/ft})(31.5)}{2} = 37.3^{k} \qquad V_u^{int} = 1.15 V_u^{ext} = 42.9^{k}$$

$$V_c = 2\lambda\sqrt{f'_c} \cdot b_w d = 2(1)\sqrt{4000}(14^{in})(19.4) = 34.4^{k}$$

$$V_s^{ext} = \begin{cases} \frac{V_u}{\phi} - V_c = \frac{37.3}{.75} - 34.4 = 15.3^{k} \leftarrow \text{controls} \\ 8\sqrt{f'_c} b_w d = 8\sqrt{4000}(14)(19.4) = 137.4^{k} > V_s \checkmark \end{cases} \qquad V_s^{int} = \frac{V_u}{\phi} - V_c = \frac{42.9}{.75} - 34.4 = 22.8^{k} \leftarrow \text{controls} \ < 137.4^{k} \checkmark$$

$$V_s \le 4\sqrt{f'_c} \cdot b_w d = 68.7 > V_s \to S_{max} = min\begin{cases} d/2 = 9.7'' \to 9'' \\ 24'' \end{cases} \qquad 68.7 > V_s \to S_{max} = 9^{in}$$

$$A_{v,min} = max\begin{cases} .75\sqrt{f'_c} \cdot b_w \cdot s/f_{yt} \\ 50 b_w s/f_{yt} \to 50(14)(9)/60000 = .105^{in^2} \end{cases} \to \text{use 2 legs} \times \#3 \to .22^{in^2}$$

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One-way Concrete

$s^{ext} = A_v \cdot f_{yt} \cdot \frac{d}{V_s} = (.22)(60)\frac{(19.4)}{(15.3)} = 16.7^{in} > 9^{in}$ $\quad s^{int} = (.22)(60)\frac{(19.4)}{(22.8)} = 11.2^{in} > 9^{in}$

$\phi V_n = .5(.75)(34.4) = 12.9^k$

Provide $\#3 \times [\,] @ 9^{in}$ from each end, first stirrup @ 2"

Design Girder

$W_u = 1.2\left[150^{lb/ft^3}\left(\frac{18" \times 22"}{144}\right)\right] = .495^{k/ft}$

$P_u = 37.3^k \leftarrow$ from beam (exterior)

$M_u^- = \frac{W_u l_n^2}{11} + \frac{P_u l_n}{8} = \frac{.495^{k/ft}(24')^2}{11} + \frac{37^k(24')}{8} = 136.9^{k\text{-}ft} \rightarrow \frac{136.9}{4(19.4)} = 1.76^{in^2} \rightarrow (3)\#7 \rightarrow 1.8^{in^2}$

$a = \frac{(1.8)(60)}{.85(4)(18)} = 1.76^{in}$ $\quad c = \frac{1.76}{.85} = 2.07^{in}$ $\quad \varepsilon_s = \frac{.003}{2.07}(19.4 - 2.07) = .0251 > \varepsilon_y$
$> \varepsilon_t \rightarrow \phi = .9$

$\phi M_n^- = (1.8)(60)\left(19.4 - \frac{1.76}{2}\right) = 166.7^{k\text{-}ft} > M_u^-$ ✓

$M_u^+ = \frac{W_u l_n^2}{16} + \frac{P_u l_n}{8} = \frac{.495^{k/ft}(24')^2}{16} + \frac{37^k(24')}{8} = 128.8^{k\text{-}ft} \rightarrow \frac{128.8}{4(19.4)} = 1.66^{in^2} \rightarrow (3)\#7 \rightarrow 1.8^{in^2}$

$\phi M_n^+ = \phi M_n^- = 166.7^{k\text{-}ft} > M_u^+$

$$A_{s,min} = \max\begin{cases} \frac{3\sqrt{f'c}\, b_w d}{f_y} \\ \frac{200 b_w d}{f_y} \rightarrow \frac{200(18)(19.4)}{60{,}000} = 1.164 < A_s \end{cases}$$

$V_u = \frac{W_u l_n}{2} + \frac{P_u}{2} = \frac{.495(24)}{2} + \frac{37.3^k}{2} = 24.6^k$

$V_c = 2\lambda\sqrt{f'c}\, b_w d = 2(1)\sqrt{4000}(18)(19.4) = 44.2^k$

$\phi V_n = .5(.75)\,44.2^k = 16.6^k$

$V_s = \frac{V_u}{\phi} - V_c = \frac{24.6}{.75} - 44.2 = -11.2 \rightarrow$ use min steel @ max spacing

$$S_{max} = \min\begin{cases} d/2 = 9.7 \rightarrow 9^{in} \\ 24" \end{cases}$$

$$A_{v,min} = \max\begin{cases} .75\sqrt{f'c} \cdot b_w \cdot s/f_{yt} \\ 50 b_w \cdot s/f_{yt} \longrightarrow 50(18)(9)/60{,}000 = .135 \rightarrow 2^{legs} \times \#3 \rightarrow .22^{in^2} \end{cases}$$

Provide $\#3 \times [\,] @ 9^{in}$ where $V_u > 16.6^k$ from each end. First stirrup @ 2^{in}

Effects of torsion could be significant and should be evaluated for girders if system is used

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One-way Concrete

Interior girder

$P_u = 37.3^k + \frac{2.37^{k/ft}(26')}{2} = 68.11^K$

$M_u^- = \frac{w_u l_n^2}{11} + \frac{P_u l_n}{8} = \frac{.495^{k/ft}(24^{ft})^2}{11} + \frac{68.11(24)}{8} = 230.25^{k\text{-}ft} \rightarrow \frac{230.25}{4(19.4)} = 2.97^{in^2}$

$\rightarrow (3)\ \#9 \div 3^{in^2}$
$> A_{s,min}$ ✓

$a = \frac{(3)(60)}{.85(4)(18)} = 2.94^{in} \quad c = \frac{2.94}{.85} = 3.46^{in} \quad \varepsilon_s = \frac{.003}{3.46}(19.4-3.46) = .0138 > \varepsilon_y$
$> \varepsilon_t \therefore \phi = .9$

$\phi M_n^- = (3)(60)\left(19.4 - \frac{2.94}{2}\right) = 269.0^{k\text{-}ft} > M_u^-$ ✓

$M_u^+ = \frac{w_u l_n^2}{16} + \frac{P_u l_n}{8} = \frac{(.495)(24)^2}{16} + \frac{68.11(24)}{8} = 222.2^{k\text{-}ft}$ → use same reinforcing as above

$\phi M_n^+ = \phi M_n^- = 269.0^{k\text{-}ft} > M_u^+$ ✓

$V_u = \frac{w_u l_n}{2} + \frac{P_u}{2} = \frac{.495(24)}{2} + \frac{68.11}{2} = 39.0^k$

$V_c = 44.2^k, \quad \phi V_n = 16.6^k$

$V_s = \frac{V_u}{\phi} - V_c = \frac{39.0}{.75} - 44.2 = 7.8^k < 8\sqrt{f'c}\,b_w d = 8\sqrt{4000}(18)(19.4) = 177^k > V_s$ ✓

$V_s \le 4\sqrt{f'c}\,b_w d = 88.3^k > V_s \rightarrow s_{max} = \min\begin{cases} d/2 \rightarrow 9^{in} \\ 24'' \end{cases}$

$A_{v,min} = \max\begin{cases} .75\sqrt{f'c}\,b_w\,s/f_{yt} \\ 50\sqrt{f'c}\,b_w\,s/f_{yt} \rightarrow 50(18)(9)/60{,}000 = .135^{in^2} \rightarrow \text{use } 2^{legs} \times \#3 \rightarrow .22^{in^2} \end{cases}$

$s = A_v f_{yt} \frac{d}{V_s} = (.22)(60)\frac{(19.4)}{(7.8)} = 32.8 > 9^{in}$

Provide #3 x [ ] @ 9" O.C. from each end, first stirrup @ 2"

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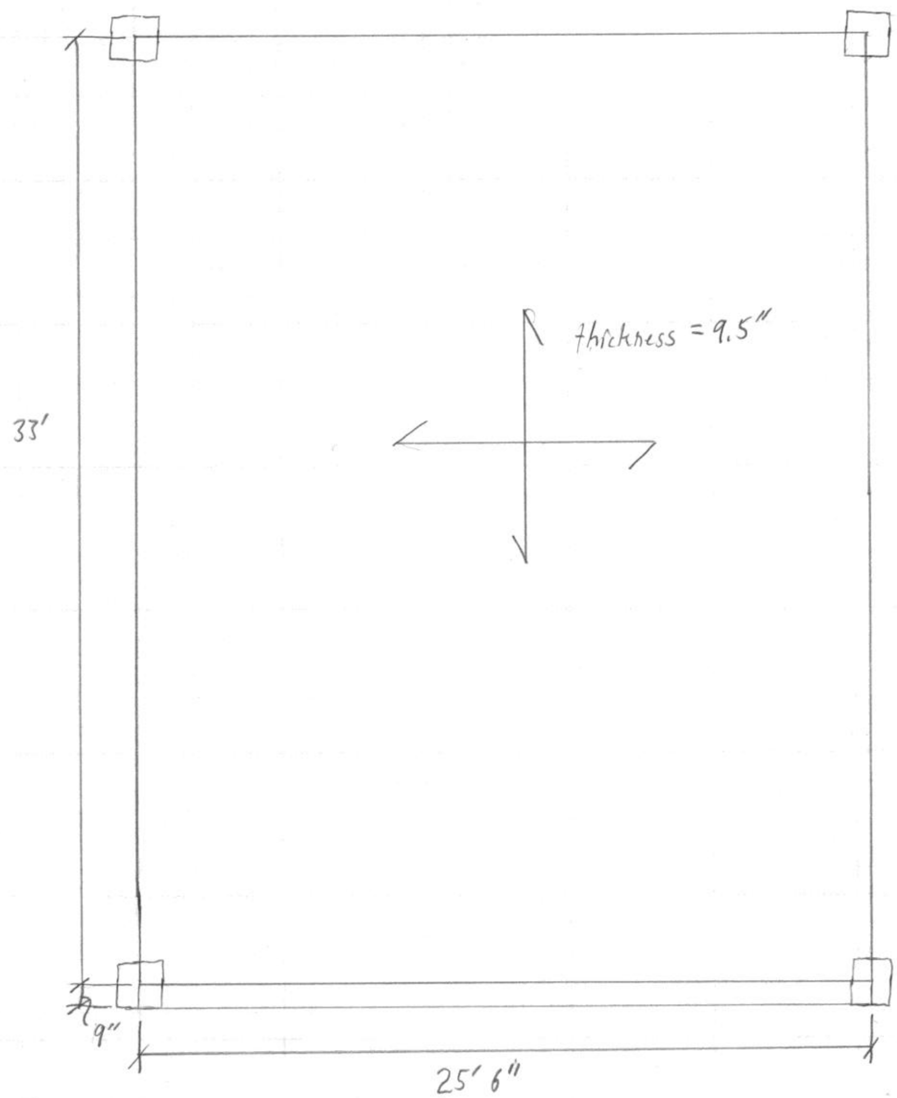
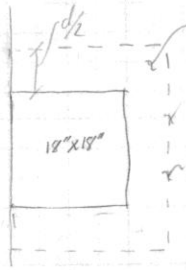
Appendix H – Flat Plate Calculations

Flat Plate



Flat plate



$(18+d)(18+\frac{d}{2}) = \frac{d^2}{2} + 27d + 324$

$b_o = 4(\frac{d}{2}) + 36$
$= 2d + 54$

$$q = 1.2\left[\left(\frac{h}{12}\right)(150^{lb/ft^3}) + 20^{psf}\right] + 1.6(40^{psf})$$

$$= 15h + 88 \quad \text{set } h = 1.5+d \rightarrow q = 15d + 110.5$$

$$V_u = (15d+110.5)\left[(25.5' \times 14.5') - \left(\frac{\frac{d^2}{2} + 27d + 324}{144}\right)\right]$$

$$= (15d+110.5)\left[\frac{-d^2}{288} + \frac{-27}{144}d + 368.125\right]$$

$$= \frac{-d^3}{19.2} + 2.4288d^2 + 5525.5d + 40857.375$$

$6 > 4 \leftarrow$ controlls

$\frac{30d}{2d+54} + 2 \rightarrow$ controlls if $d \leq 4.15$

$$\phi V_c = .75\left(4(1)\sqrt{4000}\,(2d+54)d\right)$$

$$= 379.5d^2 + 10245.8d$$

$V_u = \phi V_c$

$$\rightarrow 0 = \frac{d^3}{19.2} + 377.1d^2 + 4720.3d - 40857.375$$

$\rightarrow d = 5.89^{in}$ try 6^{in}

$q = 200.5^{psf}$

$$V_u = (200.5^{psf})\left[(25.5' \times 14.5') - (2' \times 1.75')\right] = 73.4^K$$

$$V_c \rightarrow \min\begin{cases} 2 + \frac{4}{\beta} = 6 \\ \left(\frac{\alpha_s \cdot d}{b_o} + 2\right) = \frac{30(6)}{66} + 2 = 4.73 \\ 4 \end{cases} \longrightarrow 4\lambda\sqrt{f'c}\,b_o d = 4(1)\sqrt{4000}\,(66)(6) = 100.2^K$$

$$\phi V_n = .75(1)(100.2) = 75.2^K > V_u$$

Table 9.5(c) $h \leq \frac{l_n}{33} = \frac{312}{33} = 9.45^{in} \rightarrow$ Use $9.5^{in} \rightarrow d = 8^{in}$

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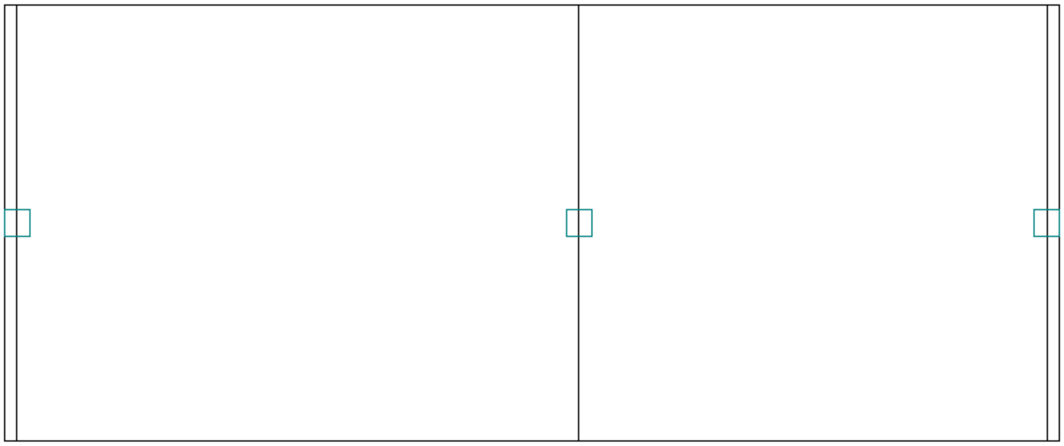
Flat Plate SPSlab Analysis

Short direction of building Left Bay is the typical bay analyzed.



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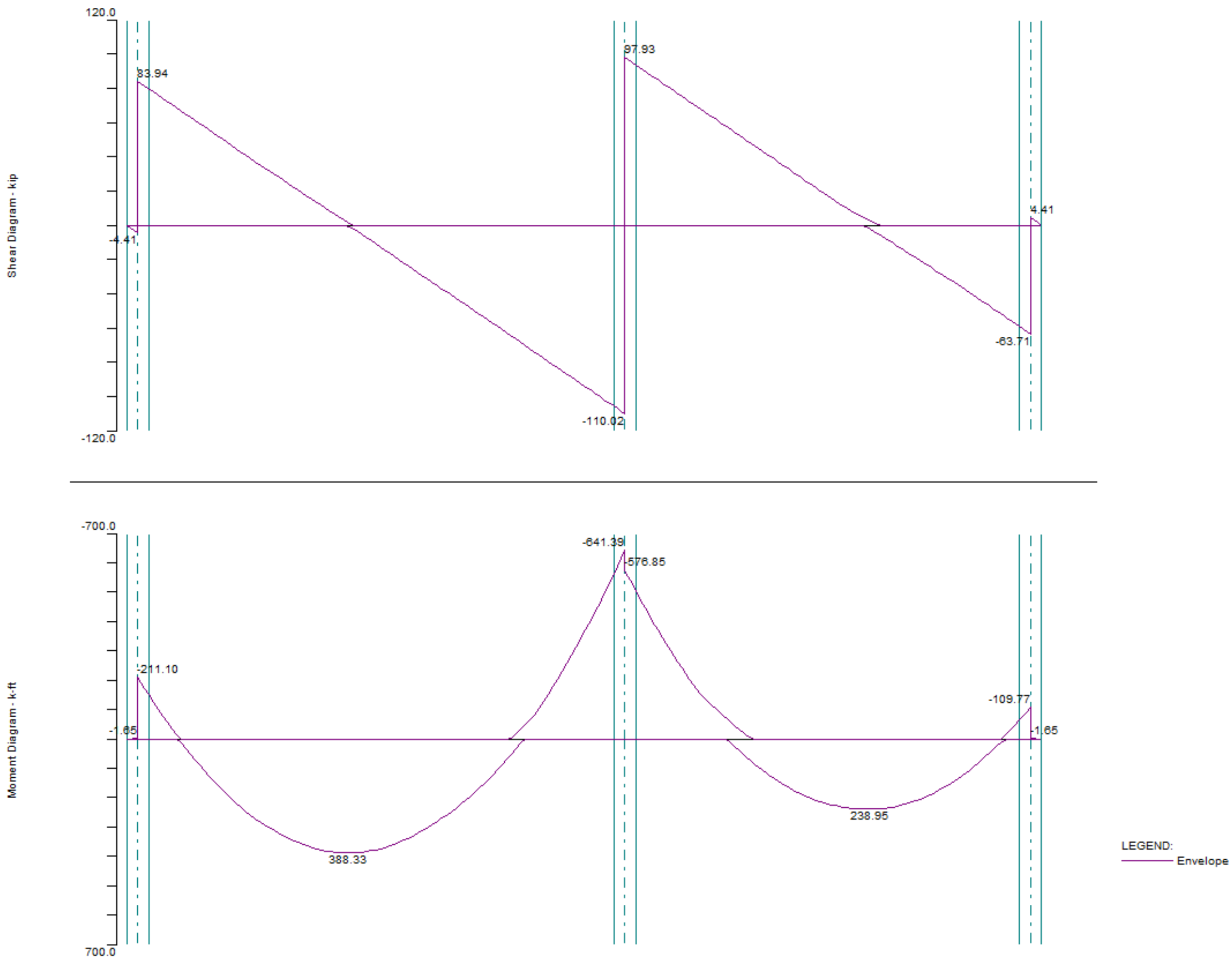

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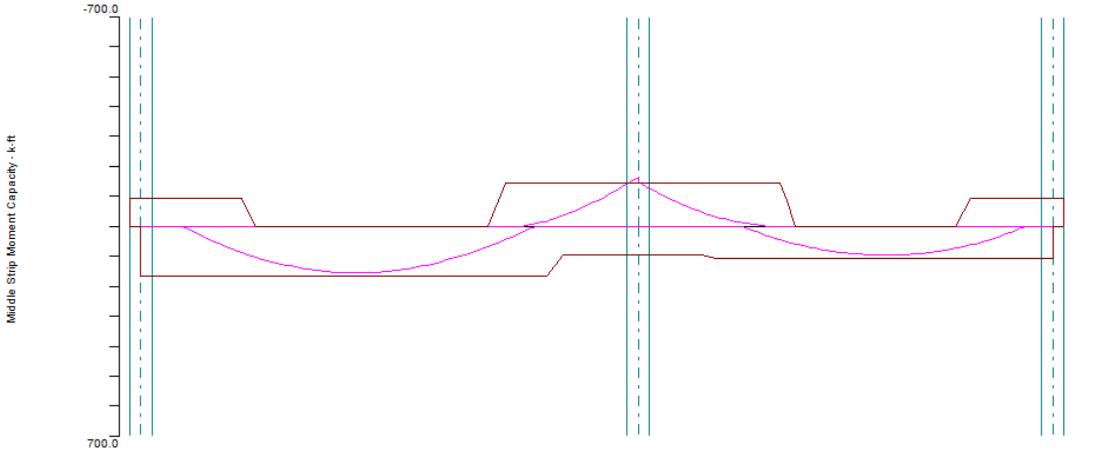
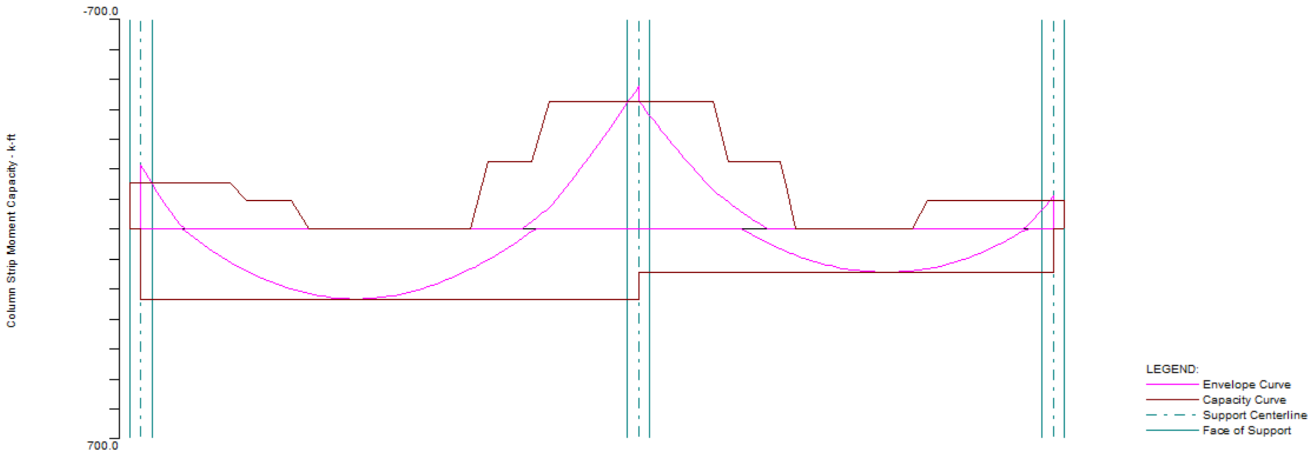
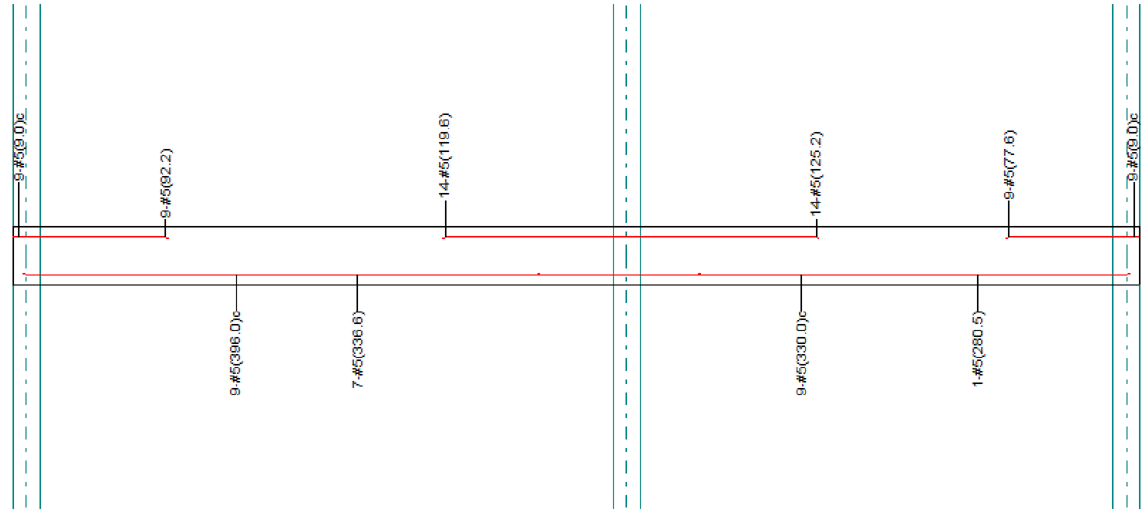
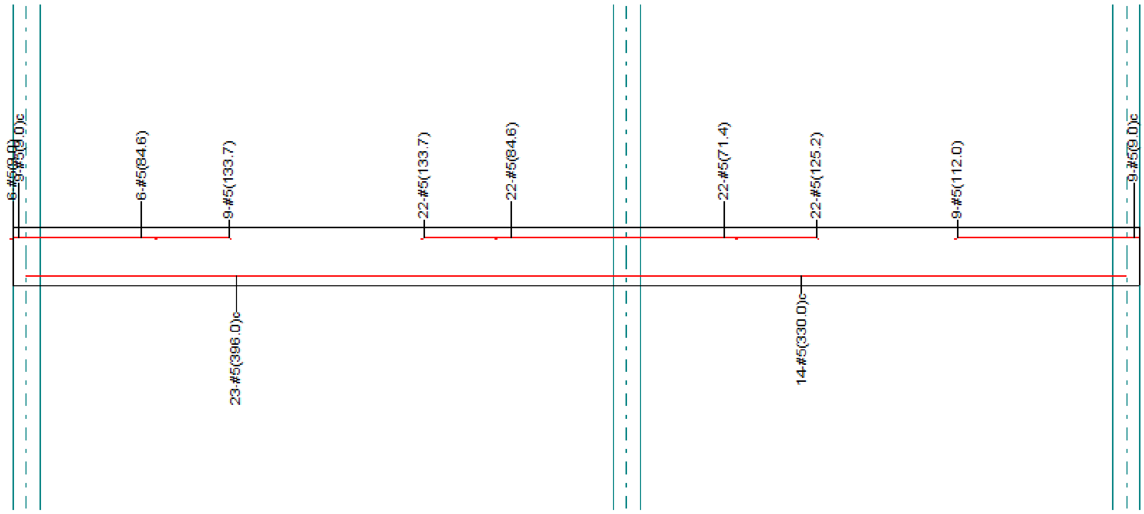




Rebar:

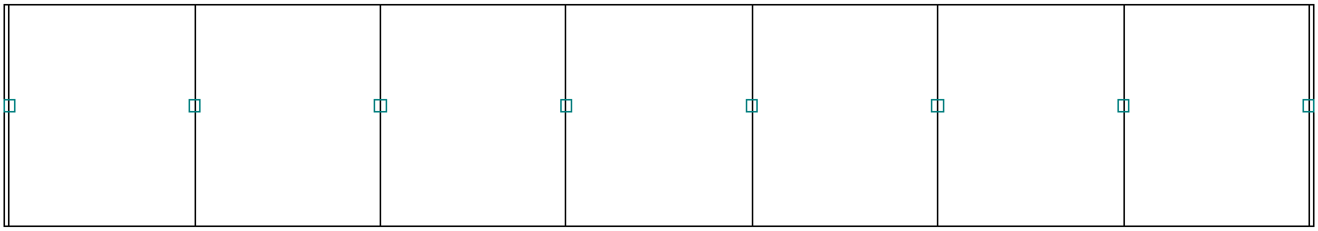


Middle Strip Flexural Reinforcement



Column Strip Flexural Reinforcement

Long Direction Central Column Line:



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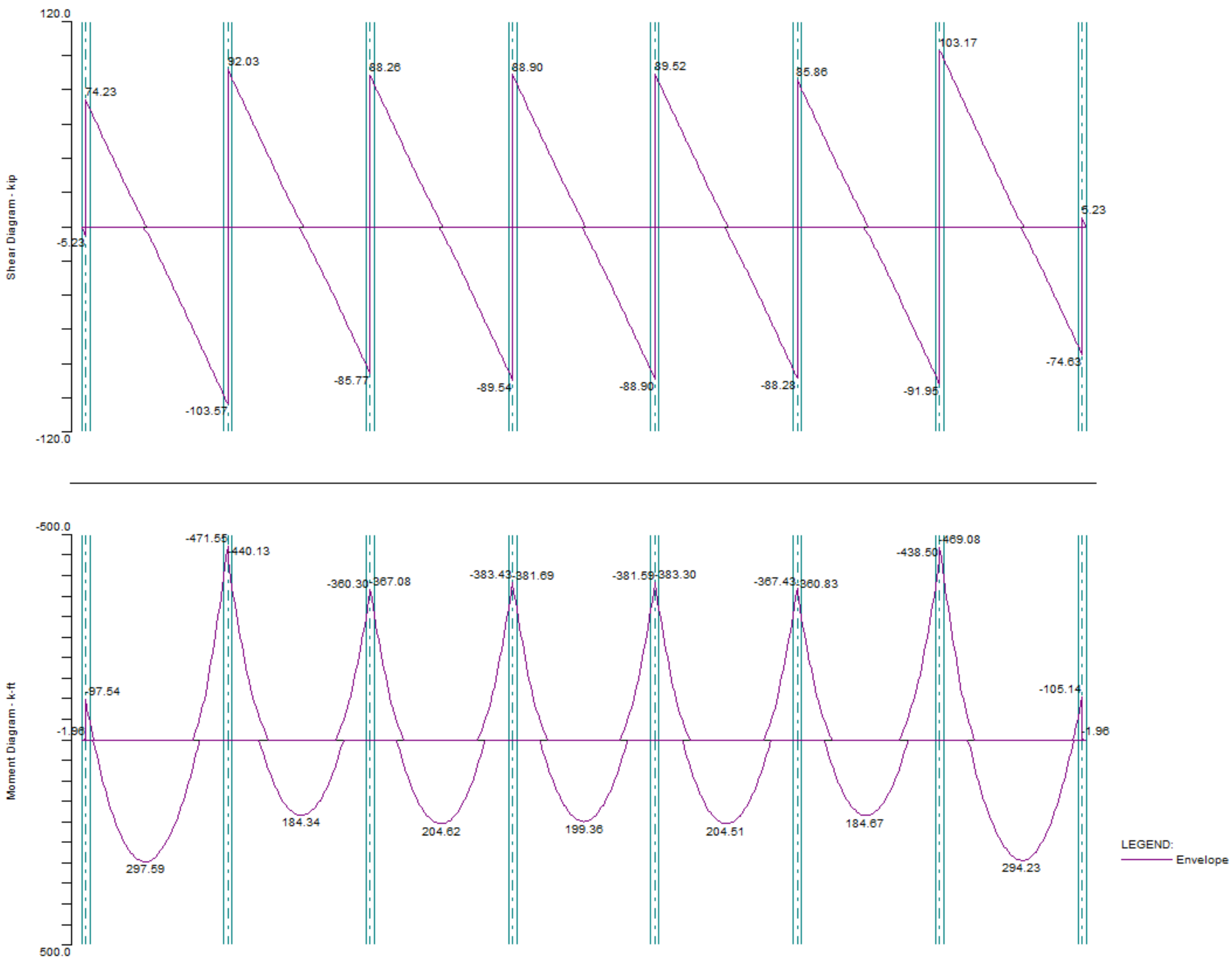
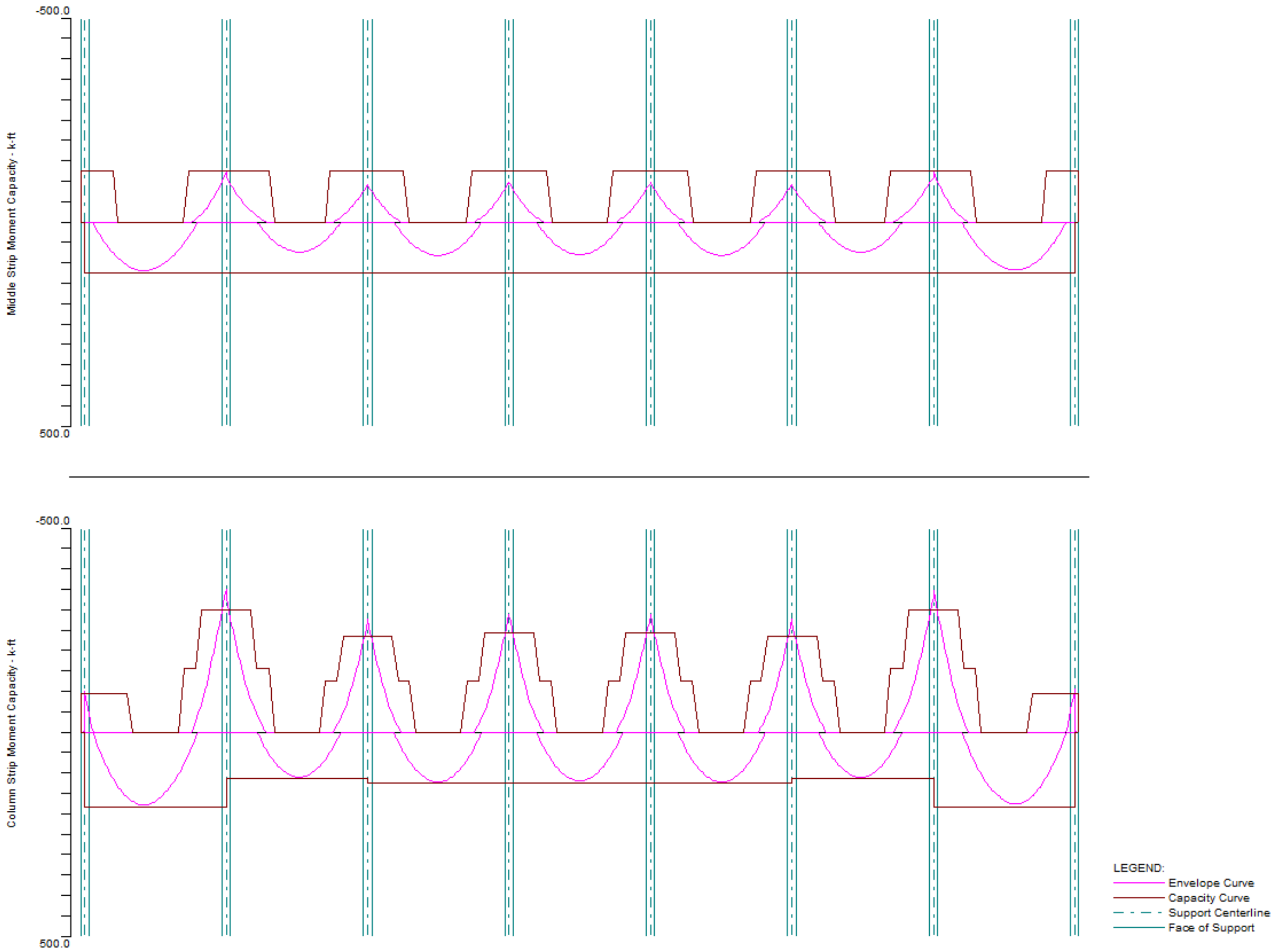
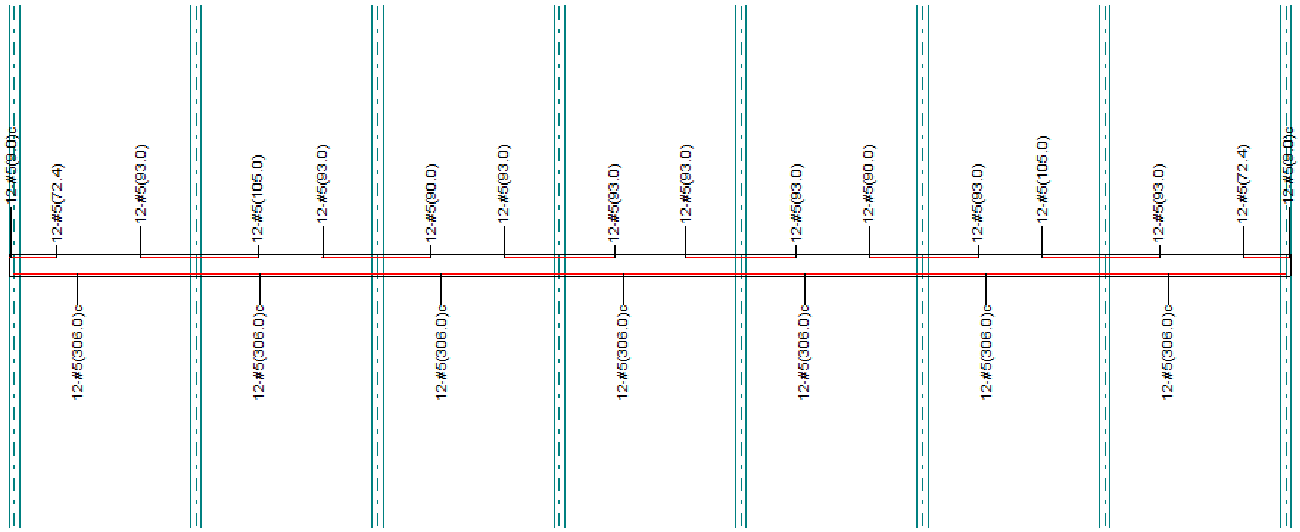


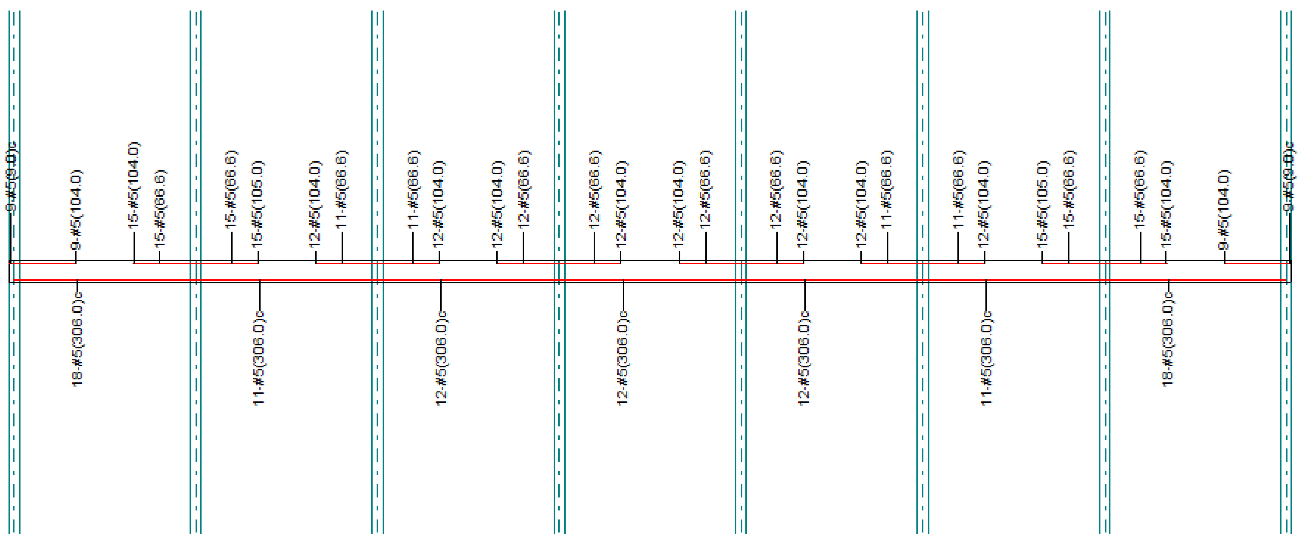
Moment Capacity:



Rebar:

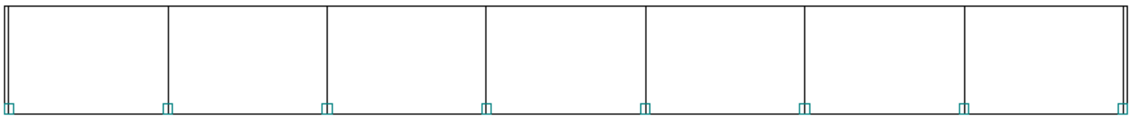


Middle Strip Flexural Reinforcement

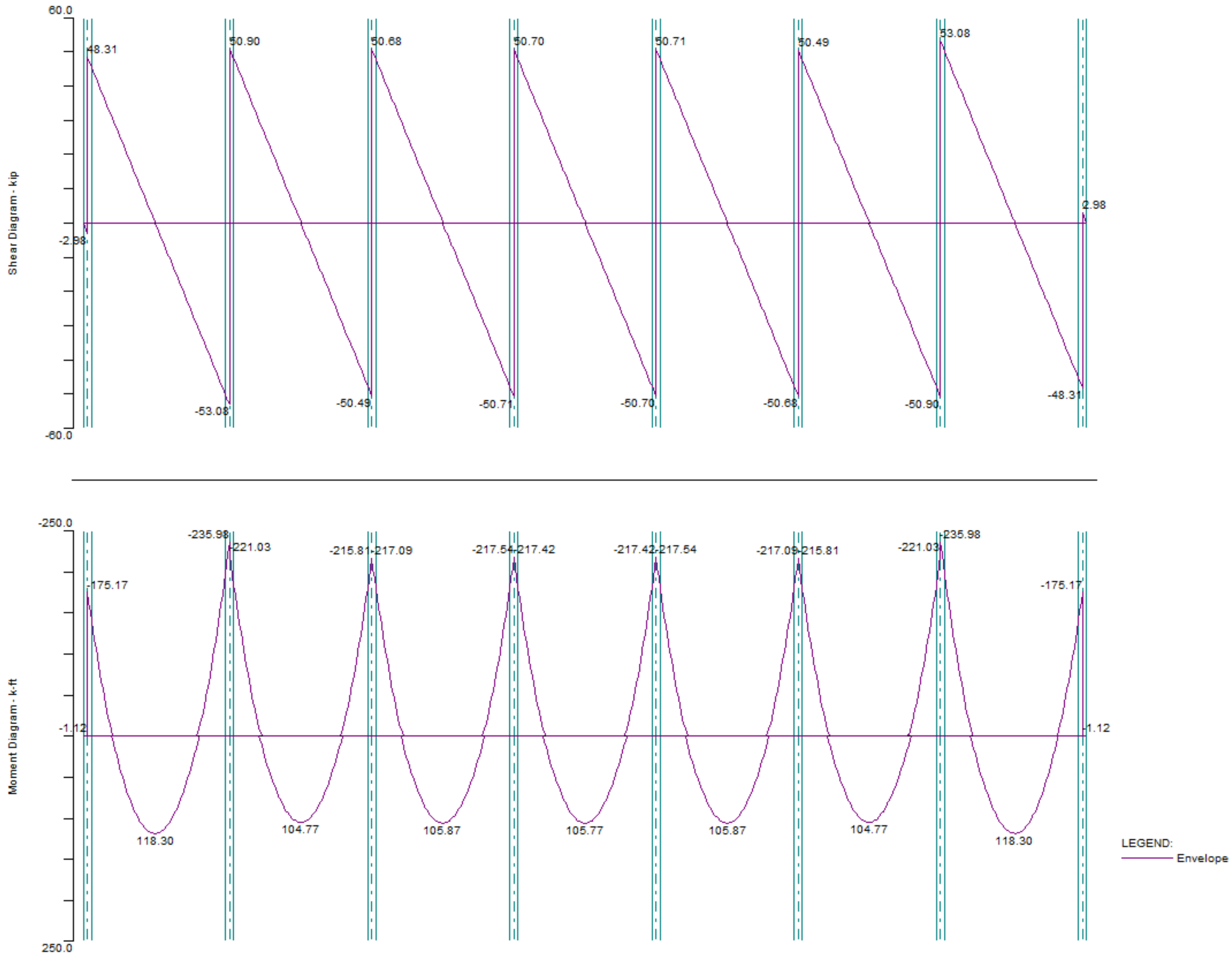


Column Strip Flexural Reinforcement

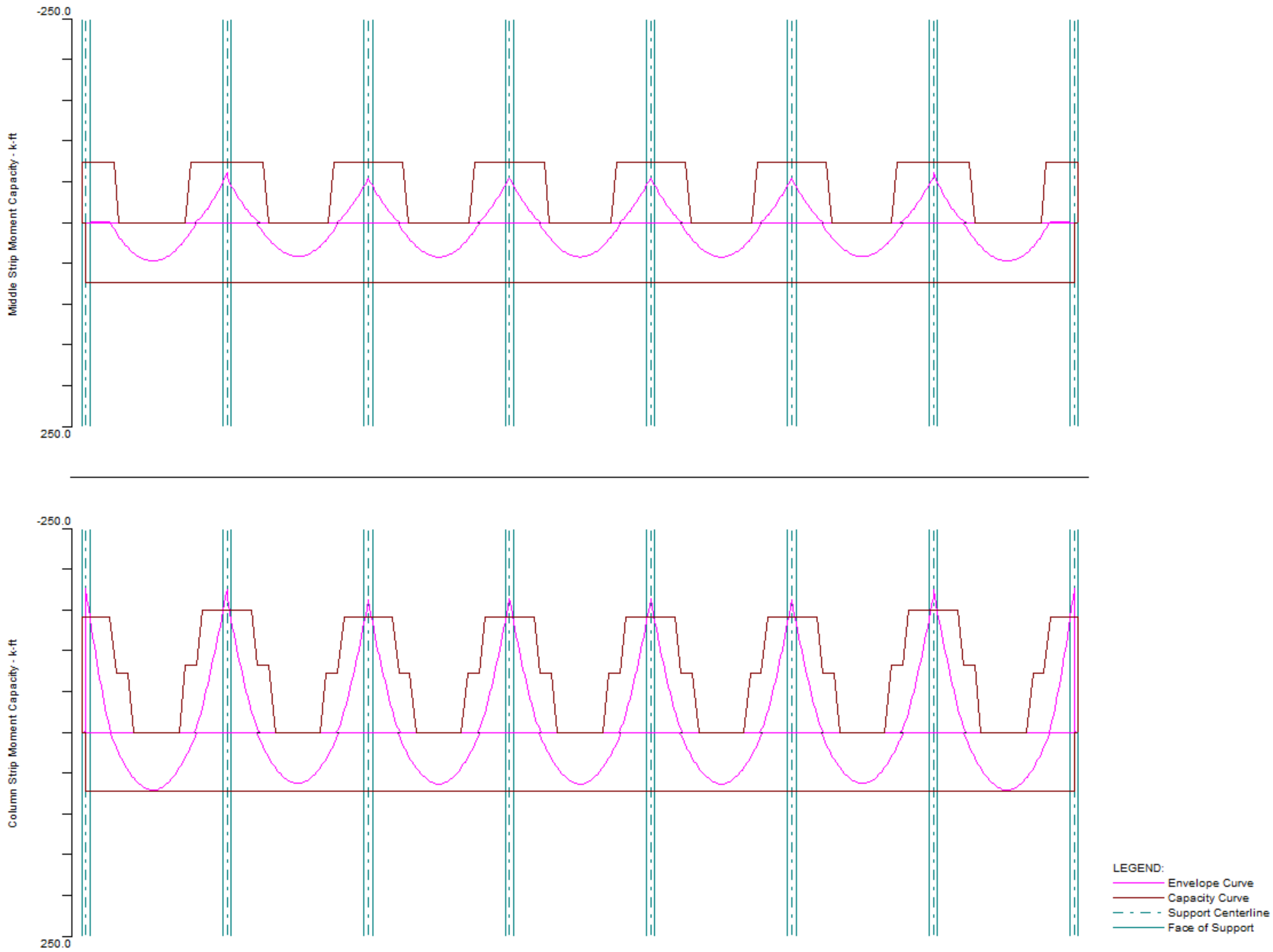
Long Direction Exterior Column Line:



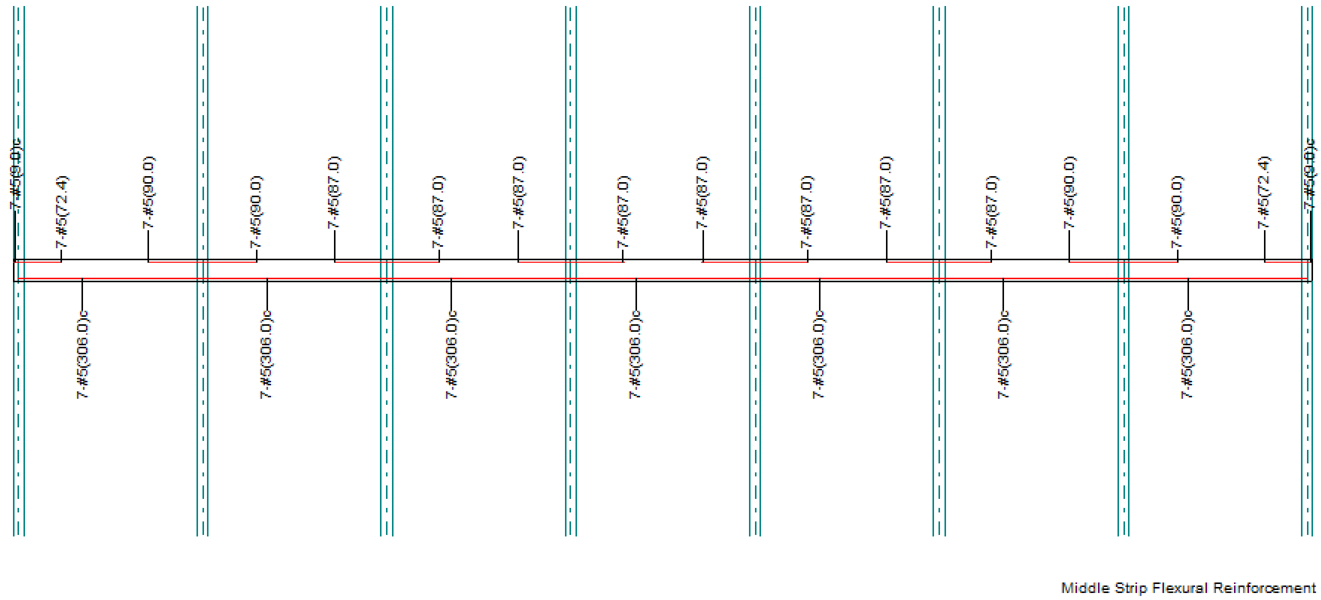
Design Moments:



Moment Capacity:



Rebar:


Middle Strip Flexural Reinforcement


Column Strip Flexural Reinforcement

Appendix I – Cost Evaluation

Assembly B10102303100 **Based on National Average Costs**

Precast concrete plank, 2" topping, 8" total thickness, 25' span, 40 PSF superimposed load, 115 PSF total load

Description	Quantity	Unit	Material	Installation	Total
C.I.P. concrete forms, elevated slab, edge forms, to 6" high, 4 use, includes shoring, e...	0.10000	L.F.	0.02	0.41	0.43
Welded wire fabric, sheets, 6 x 6 - W1.4 x W1.4 (10 x 10) 121 lb. per C.S.F., A185, incl...	0.01000	C.S.F.	0.15	0.36	0.51
Structural concrete, ready mix, normal weight, 3000 psi, includes local aggregate, san...	0.17000	C.F.	0.71	0.00	0.71
Structural concrete, placing, elevated slab, pumped, less than 6" thick, includes strike...	0.17000	C.F.	0.00	0.26	0.26
Concrete finishing, floors, basic finishing for unspecified flatwork, bull float, manual fl...	1.00000	S.F.	0.00	1.13	1.13
Concrete surface treatment, curing, sprayed membrane compound	0.01000	C.S.F.	0.08	0.09	0.17
Precast slab, roof/floor members, grouted, solid, 6" thick, prestressed	1.00000	S.F.	7.15	2.88	10.03
Total			$8.10	$5.13	$13.23

Assembly B10102540960 **Based on National Average Costs**

Floor, composite metal deck, 5" slab, 25'x25' bay, 29" total depth, 75 PSF superimposed load, 178 PSF total load

Description	Quantity	Unit	Material	Installation	Total
Welded wire fabric, sheets, 6 x 6 - W1.4 x W1.4 (10 x 10) 121 lb. per C.S.F., A185, incl...	0.01100	C.S.F.	0.17	0.40	0.56
Structural concrete, ready mix, normal weight, 3000 psi, includes local aggregate, san...	0.01100	C.Y.	1.23	0.00	1.23
Structural concrete, placing, elevated slab, pumped, less than 6" thick, includes strike...	0.01100	C.Y.	0.00	0.36	0.36
Concrete finishing, floors, for specified Random Access Floors in ACI Classes 1, 2, 3 an...	1.00000	S.F.	0.00	0.86	0.86
Concrete surface treatment, curing, sprayed membrane compound	0.01000	C.S.F.	0.08	0.09	0.17
Structural steel project, apartment, nursing home, etc, 100-ton project, 3 to 6 stories,...	7.31600	Lb.	10.24	3.15	13.39
Metal floor decking, steel, non-cellular, composite, galvanized, 3" D, 18 gauge	1.05000	S.F.	2.88	1.10	3.98
Metal decking, steel edge closure form, galvanized, with 2 bends, 12" wide, 18 gauge	0.04000	L.F.	0.16	0.10	0.25
Sprayed fireproofing, cementitious, normal density, beams, 1 hour rated, 1-3/8" thick...	0.66600	S.F.	0.39	0.66	1.05
Sprayed fireproofing, cementitious, normal density, corrugated or fluted decks, 1" thi...	1.00000	S.F.	0.87	1.18	2.05
Total			$16.00	$7.90	$23.90

Assembly B10102195500 **Based on National Average Costs**

Cast-in-place concrete beam and slab, 6" slab, one way, 12" column, 25'x25' bay, 40 PSF superimposed load, 129 PSF total load

Description	Quantity	Unit	Material	Installation	Total
C.I.P. concrete forms, beams and girders, exterior spandrel, plywood, 12" wide, 4 use...	0.11800	SFCA	0.11	1.21	1.32
C.I.P. concrete forms, beams and girders, interior, plywood, 12" wide, 4 use, includes...	0.21200	SFCA	0.23	1.78	2.01
C.I.P. concrete forms, elevated slab, flat plate, plywood, to 15' high, 4 use, includes s...	0.86600	S.F.	0.99	4.89	5.88
Reinforcing Steel, in place, elevated slabs, #4 to #7, A615, grade 60, incl labor for acc...	2.27600	Lb.	1.27	0.98	2.25
Structural concrete, ready mix, normal weight, 3000 psi, includes local aggregate, san...	0.60000	C.F.	2.50	0.00	2.50
Structural concrete, placing, elevated slab, pumped, 6" to 10" thick, includes strike of...	0.60000	C.F.	0.00	0.77	0.77
Concrete finishing, floors, for specified Random Access Floors in ACI Classes 1, 2, 3 an...	1.00000	S.F.	0.00	0.86	0.86
Concrete surface treatment, curing, sprayed membrane compound	0.01000	C.S.F.	0.08	0.09	0.17
Total			$5.20	$10.58	$15.78

Assembly B10102237400 **Based on National Average Costs**

Flat plate, concrete, 9.5" slab, 20" column, 25'x25' bay, 75 PSF superimposed load, 194 PSF total load

Description	Quantity	Unit	Material	Installation	Total
C.I.P. concrete forms, elevated slab, flat plate, plywood, to 15' high, 4 use, includes s...	0.98600	S.F.	1.12	5.57	6.69
C.I.P. concrete forms, elevated slab, edge forms, alternate pricing, to 6" high, 1 use, i...	0.03100	SFCA	0.02	0.20	0.22
Reinforcing Steel, in place, elevated slabs, #4 to #7, A615, grade 60, incl labor for acc...	3.02800	Lb.	1.70	1.30	3.00
Structural concrete, ready mix, normal weight, 3000 psi, includes local aggregate, san...	0.79100	C.F.	3.29	0.00	3.29
Structural concrete, placing, elevated slab, pumped, 6" to 10" thick, includes strike of...	0.79100	C.F.	0.00	1.02	1.02
Concrete finishing, floors, for specified Random Access Floors in ACI Classes 1, 2, 3 an...	1.00000	S.F.	0.00	0.86	0.86
Concrete surface treatment, curing, sprayed membrane compound	0.01000	C.S.F.	0.08	0.09	0.17
Total			$6.20	$9.04	$15.24