JULIE DAVIS
STRUCTURAL OPTION
APRIL 9, 2008

STRUCTURAL DEPTH

EXISTING STRUCTURAL SYSTEM

Foundation System:

Building 2's foundation system is a 4" slab on grade with a deep foundation system. Drilled at a depth of 60-65' below grade there are over 250 16" diam. augured cast in place piles. This foundation system was chosen due to the median to stiff clay located up to 22' below grade. Piles have an assumed service capacity of 125 tons and typically are reinforced with 1 #8 x 15'-0" LG. Piles under shear walls are reinforced at 25' with 4-#8 vert. and #4 ties. The slab is thickening at interior CMU walls and location of increased service loads. Grade beams at a width of 1'- 0" are placed around the buildings perimeter at varying depths.

Floor System:

A two way post-tension slab is used for all floors. The tendons are unbounded and span in both directions with a minimum of (2) tendons above columns. Banded tendons are used north to south and uniformed tendons east to west. Bundle size varies but is restricted to a minimum of 4 tendons per bundle. The 7 ½" slab is reinforced two-ways with #4@24" bottom mesh reinforcement and #5 top bars at various locations. Rebar is also provided around the perimeter. Where tendons and rebar intersect chairs should be placed with #4 ties for lateral stability. Tendons stressing will be done with a hydraulic jack, anchorage blockouts are grouted and tendons cut 1" from slab edge, stressing sequence is as follows;

1. Stress 50% banded tendons
2. Stress 50% of uniform tendons
3. Stress remaining 50% banded tendons
4. Stress remaining 50% uniform tendons

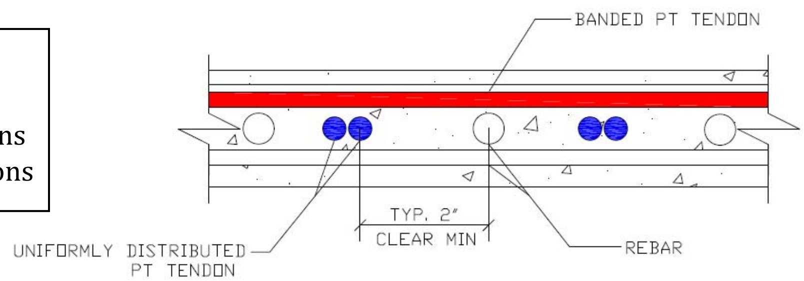

Figure #6: Typical floor section
Blue: Uniformly distributed tendon
Red: Banded Tendons

Balconies are conventionally reinforced with #4 @ 12" O.C and 2-#5 top & bottom.

Roof System:

Post-tension roof slab is to be 10" deep with #5@24" reinforcing. A 1 ½" galvanized metal roof deck placed on top continuous over 3 spans. This is followed by an asphalt membrane, ridged insulation and ballast

The flat plate post tension slab is supported by a grid of (52) cast in place gravity concrete columns. Columns have an f'c of 5000 psi and take some lateral forced but predominately support gravity loads. Cold form metal studs are used for most wall construction with the exception of stairwells, mechanical rooms and storage areas which are masonry construction.

Lateral System:

The lateral system consists of (4) concrete shear walls, three of which surround the elevator shaft (i.e. the central core).

Shear Walls*:* Shear wall footings are to be reinforced at a depth of 25'-0" with vertical bars and ties. Typical shear wall reinforcing is #4@12" vertical and horizontal, 8#8 in the middle and #3 ties in various arrangements. An F'_C of 5000 and F_y of 60,000 are used in each shear wall.

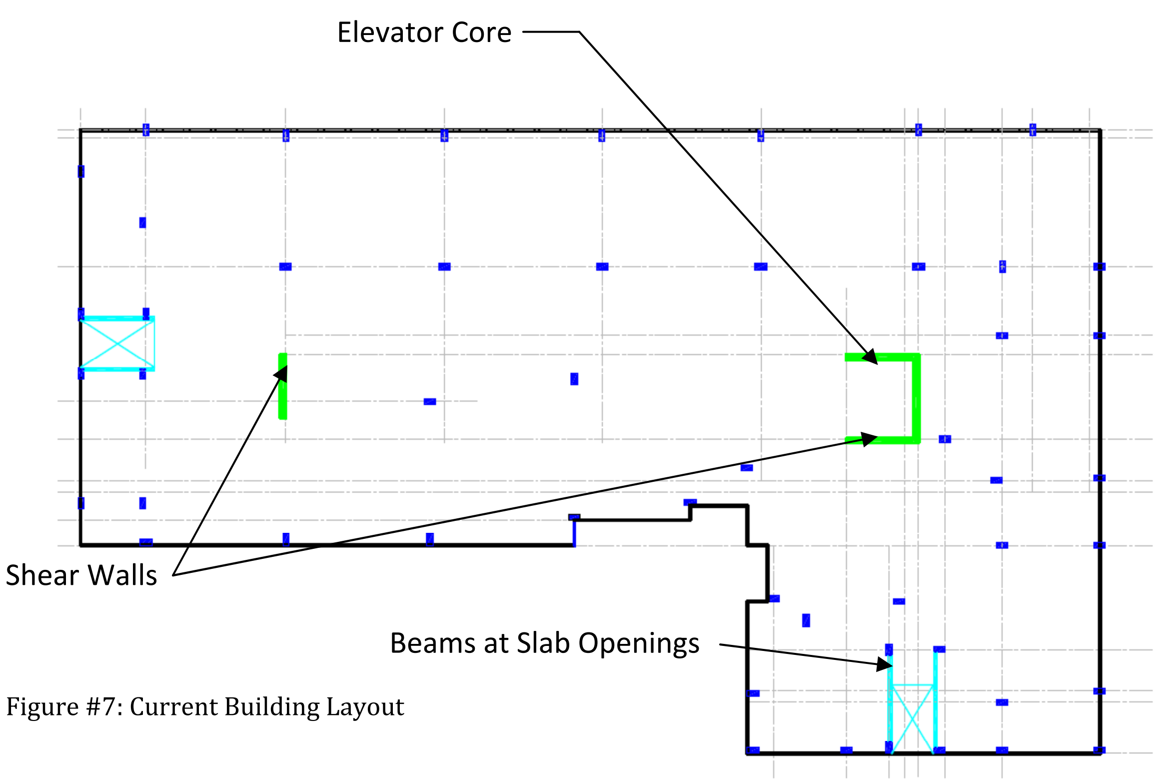

Figure #7: Current Building Layout

LOADING CONDITIONS

The original loading conditions specified by ASCE07-05 are as follows:

DEAD LOAD

7 ½" Post Tension Slab	150 PCF
Beams	VARIES
Façade #1 (4" Brick, 8" CMU)	95 PSF
Façade #2 (4" Brick, Glass, Cold form)	35 PSF
Walls	
Superimposed Dead Loads:	
Partitions	20 PSF
Mechanical/Electrical	5 PSF

LIVE LOAD

Residential Units:	40 PSF
Lobbies/Corridors:	100 PSF
Balconies:	100 PSF
Mechanical/Storage:	125 PSF
Canopy:	60 PSF
Public Areas:	100 PSF
Snow:	30 PSF
Elevator Rooms:	150 PSF

Roof:

Live*:*

Ordinary flat Roof = 20PSF

Equipment:

100% outside air rooftop units

Units @ 4000lbs a piece

Dimension :

Live Load Reduction :

$L_r=L_0R_1R_2$

$R_1= 1.2-0.001A_t$

$R_2=1.2-0.05F$

Snow:

$Pf=0.7C_eC_sIP_g$

(0.7)(1.0)(1.0)(1.0)(30) = 21PSF

Snow Drift				Leeward		Windward			
	H_c (ft)	Lu_T (ft)	Lu_B (ft)	H_d (ft)	W (ft)	H_d (ft)	W (ft)	γ	P_d (PSF)
A	10.58	27	84	1.7	6.8	1.51	6.04	17.9	30.43
B	10.58	15.667	163	1.3	5.2	2.32	9.28	17.9	41.528
C	10.58	9.5	29.58	1.3	5.2	0.9	3.6	17.9	23.27
D	10.58	25.5	23.4	1.3	5.2	0.9	3.6	17.9	23.27
E	10.58	9.5	24.8	1.3	5.2	0.9	3.6	17.9	23.27
F	17.83	55.5	40.2	1.9	7.6	1.125	4.5	17.9	34.01
G	17.83	22	36	1.3	5.2	1	4	17.9	23.27
H	17.83	22	46	1.3	5.2	1.2	4.8	17.9	23.27
I	17.83	14.8	42	1.3	5.2	1.2	4.8	17.9	23.27
J	17.83	55.5	70.4	1.9	7.6	1.4	5.6	17.9	34.01
K	10.58	25.5	123	1.3	5.2	2.6	10.4	17.9	46.54
L	10.58	9.5	35.3	1.3	5.2	1.35	5.4	17.9	24.165
M	10.58	9.5	31.8	1.3	5.2	1.35	5.4	17.9	24.165

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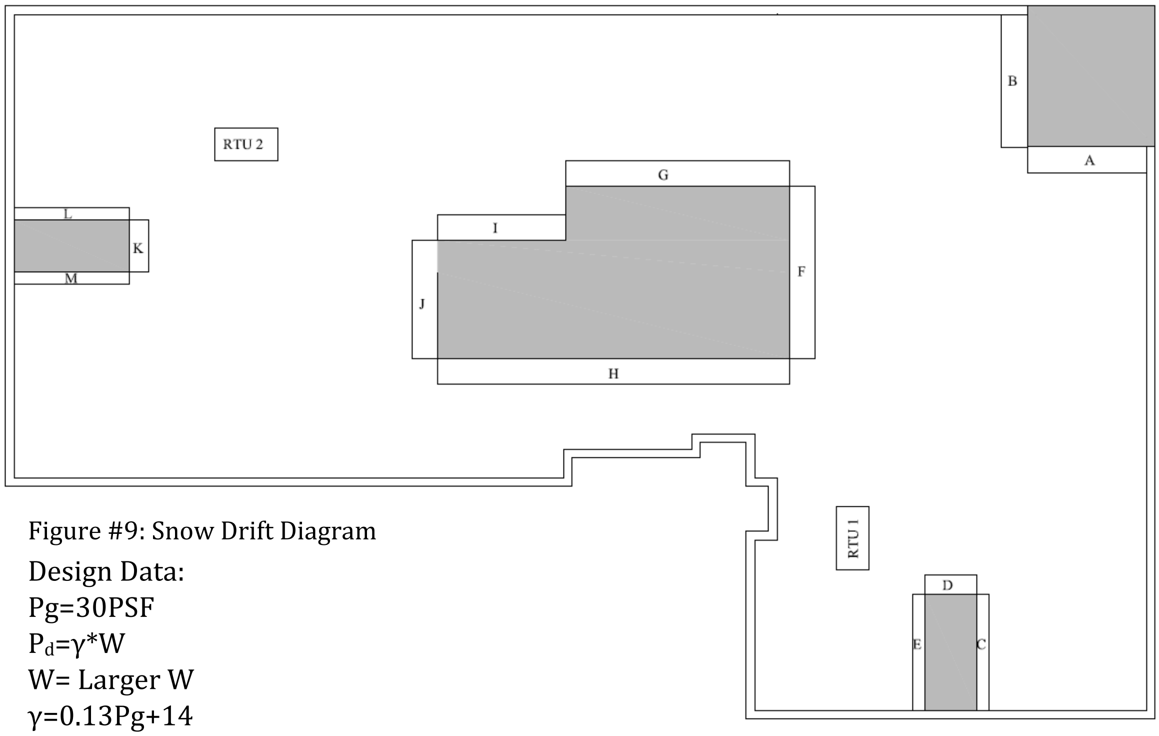


Figure #9: Snow Drift Diagram

Design Data:
Pg=30PSF
$P_d = \gamma * W$
W= Larger W
$\gamma = 0.13Pg + 14$

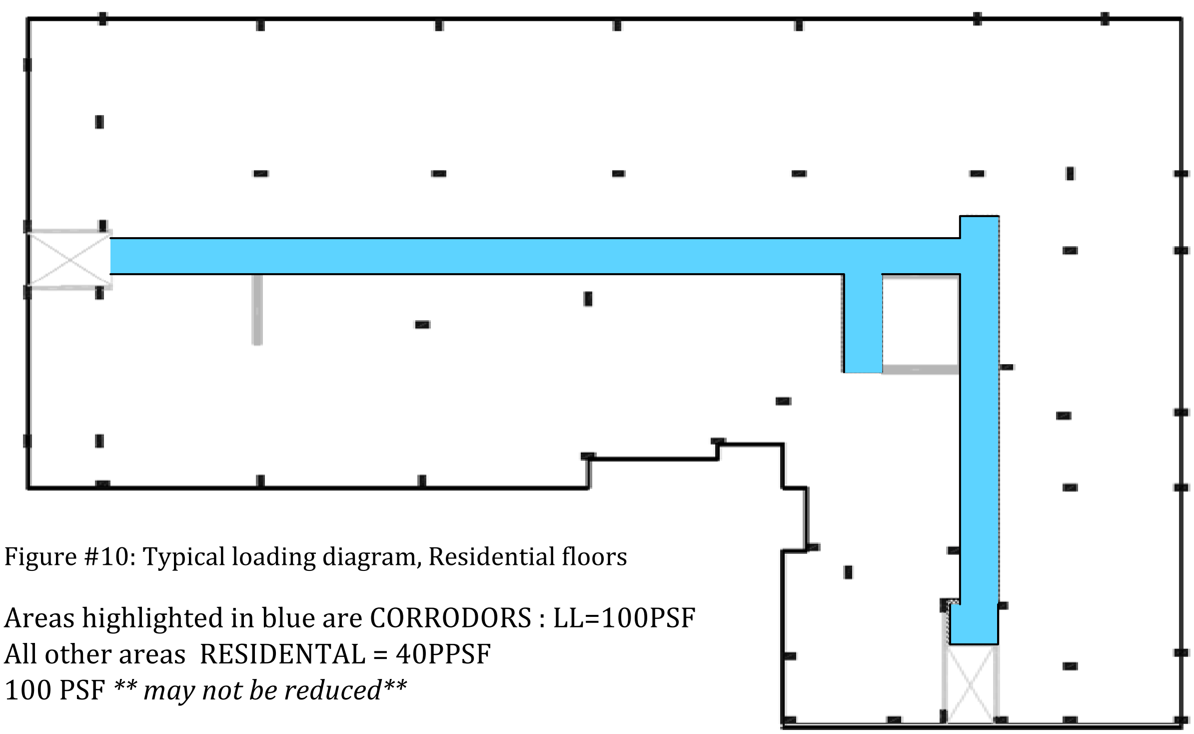


Figure #10: Typical loading diagram, Residential floors

Areas highlighted in blue are CORRODORS : LL=100PSF
All other areas RESIDENTAL = 40PPSF
100 PSF *** may not be reduced***

PRELIMINARY ANALYSIS

To begin analysis decisions were made to optimize performance of the precast system. Many sources were used to determine the design criteria:

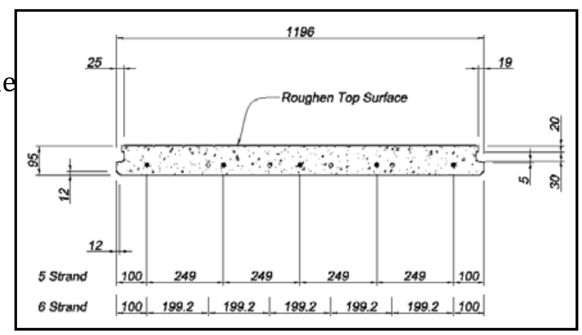
1. PCI Handbook 6th Edition
2. ACI 318-03
3. Several Manufacturers of Precast (Hanson, Nitterhouse, and Hollowcore)
4. PCI Manual for the design of hollow core slabs

The following topics were considered and a decisions was made with regards to the overall constructability, fire rating, mechanical and electrical requirements, erection time, seismic requirements, slab thickness, and

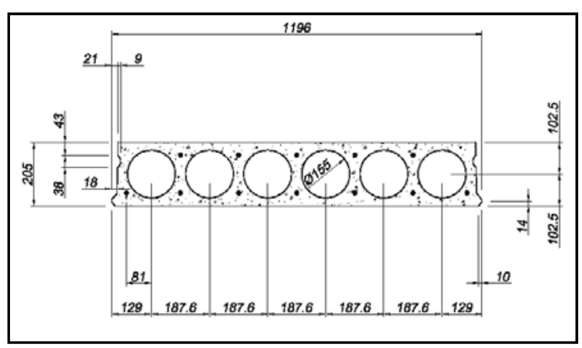
1. Hollow Core vs. Solid flat slab
2. Topping vs. No Topping
3. Ridged vs. Flexible Diaphragm
4. Expansion joints vs. No expansion joints
5. Tendons Considerations

1. Hollow core vs. solid flat slabs

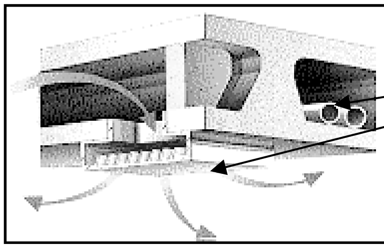
Solid flat slabs perform similar to cast in place solid slabs. Optimized when used for short spans of 12-24 feet and floor depth of 4-16 inches. This system provides a smooth underside so it can be used as a finished ceiling. The slabs have a fire rating of between ½ -2 hours depending construction. Mechanical and electrical fittings can be embedded during casting. Deflections are easily controlled with this system, and depending on the manufacturer a 2 inch reinforced concrete topping could be required.



Hollow core slabs are utilized in intermediate spans of 12-40 feet and depths of 4-16 inches. The continuous voids reduce weight and provide space to run electrical and mechanical equipment. If designed properly the voids can be engineered as a passive solar system. Floor covering can be applied directly to the planks or a concrete topping can be applied. The concrete topping is ½ to 2" in width and can be non-structural or structural composite concrete. The underside of the plank can be utilized as a finished ceiling. Depending thickness a 1 to 4 hour fire rating can also be accomplished with hollow precast planks. A hollow core system also provides excellent sound transmission, fast onsite construction, durability, and precision casting is an added bonus.



Conclusion *Hollow Core Slab:* For the redesign of City Vista I would like to accomplish spans of up to 20+ feet, as little interference with other trades, fast erection, and adequate fire rating. As a result I have chosen a hollow core system which is optimized when used for intermediate spans, has better fire rating than the flat slab, and is friendlier to the space requirements of other trades.



Selling factor: Longer spans, mechanical and electrically friendly, and sound absorption.

2. Concrete vs. No concrete topping

Topping: Using a composite topping will provide added stiffness and strength with a min. topping thickness of 2 inches, and f'c between 3000-4000 psi. The added topping weight can cause significant deflection , camber can be included in the original design to combat this. Diaphragm design can also be simplified with the use of a composite topping. When the shear between the planks and topping is limited to 80 psi the topping can contain all the diaphragms reinforcing. This will eliminate the need to keyhole and grout all the planks to one another.

Un-topped: *(PCI 3.8.4.5)* An un-topped system may be used if the shear strength is proven to meet ACI requirements. This system usually requires a reinforced perimeter and grouted joints to achieve required shear strength. An untopped system is also inherently lighter, which could potentially reduce base shear. Un-topped systems still consist of a ¾" leveling concrete slab.

Dead Load Comparison for Hollow Core Planks

Plank Size	Dead Load (PSI)
6 in	0.350
6 in Topped	0.564
8 in	0.446
8 in Topped	0.568
10 in	0.527
10 in Topped	0.789
12 in Topping	0.848
Topping=60mm which is approx. 2.3"	
** Information based on data from Hanson Precast and is converted from metric so answers are approx. **	

Figure #11: Dead load comparison between topped & un-topped

Minimum Cover to Achieve Fire Resistance

Plank Size	60 Min	120 Min	180 Min	240 Min
6"	-	1 "	2.5"	3"
8"	-	-	1"	2"
10"	-	-	-	1"
12"	-	-	-	-
** Information based on data from Hanson Precast and is converted from metric so answers are approx. **				

Figure #12: Required thickness to achieve specific fire ratings

Conclusion: A *topped composite slab* is industry standard. I will design for a 2" composite slab, although if the additional strength isn't needed I will only apply a ¾" topping to level the floor. The 2" topping will result in additional camber consideration.

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3. *Ridged vs. Flexible Diaphragm*

Ridged diaphragm distributes horizontal forces to vertical elements proportionate to their relative stiffness. In a ridged system deflections of the diaphragm have little effect on the overall system. In high seismic zones a stiff ridged diaphragm is ideal and requires much less analysis. Chord requirements (Chord: the tension or compressive elements creating a flange for the diaphragm used to develop flexural integrity) for this system are larger than a flexible system, but potentially creates a safer distribution of forces.

Flexible diaphragms are used mainly for distribution of story shear when the lateral deformation is twice the average story drift. To keep elastic analysis simple a factor of 2R/5 is applied. This system is less demanding on vertical elements with smaller chord requirements, and smaller shear and moment diagrams, although the stability of the floor can be unsafe compared to a ridged system.

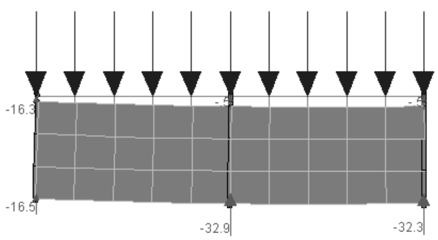


Figure #13. Ridged Diaphragm : You can see deformation of the diaphragm is limited. For a pre-cast system this is ideal.

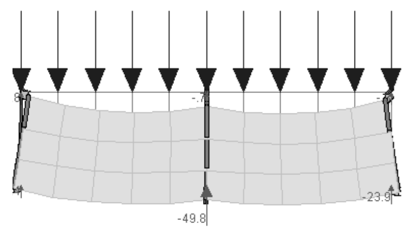


Figure #14: Flexible Diaphragm: You can see large deformations are experienced in the diaphragm.

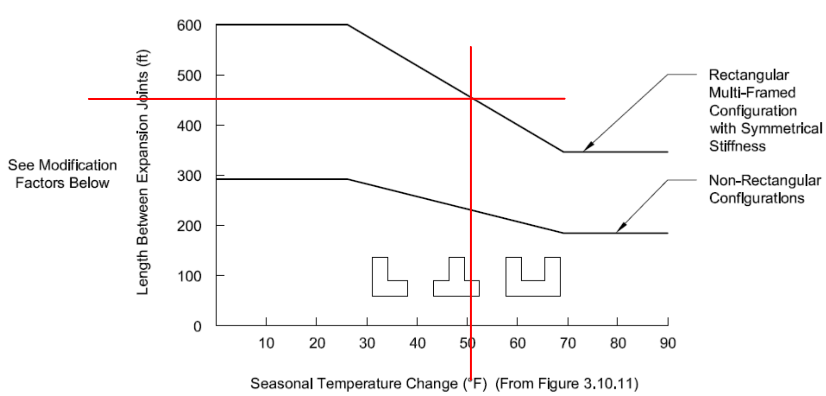
Courtesy of the University of Virginia : [urban.arch.virginia.edu/~km6e/arch721/content/lectures/lec-03/home.html]

Conclusion: A *ridged diaphragm* will be used, this system will be provide simpler analysis, performs better in seismic zones and is inherently stronger and safer.

5. Expansion joints vs. No expansion Joints

Expansion joint requirements are specified in *(PCI FIG 3.10.20)*

Maximum Seasonal Climatic Temp. Change: 50^0



Conclusion: *No expansion* joints needed. Max building length is 179'-4"

7. Tendons:
Prestressed vs. Non-Prestressed
Prestressed concrete members achieve higher span to depth ratio and better resistance to cracking. Conventionally reinforced is only used in small construction projects where small loads are experienced.

Post tension vs. Pre-tensioned
Post-tension tendons are placed in conduits during casting then tensioned in the field after erection and grouted. They are placed in conduits so they do not bond to concrete during curing. Post tensioning is usually used in conjunction with pre-tensioning when a component cannot sustain the full stressing before stripping, or to stop cracking during production.

Conclusion: *Pre-tensioned,* 270ksi low relaxation (7) wire strands, 2-3 strands per bundle and varying diameters.

Strand placement: The number of depressions: varies from 0-2. (2) points of inflection can potentially create a higher capacity. Depending on fire rating thicker cover is required.
Debonding: When there is an area of high stress concentration and as a result pragmatic cracking occurs, which could cause failure. In prestensioned concrete there are areas of high stress at the edges. The tendons begin to separate from the concrete causing cracking and a decrease in capacity.

DESIGN CONSIDERATION

When determining preliminary framing dimensions the following were considered:

1. Framing Dimensions
2. Span to Depth Ratio
3. Connection Concepts
4. Mechanisms for control of volume change
5. Optimization of member sizes
6. Basic Design Data

Framing Dimensions*:* Select modular dimensions from standard pieces. Optimize this selection by choosing the least amount of components that satisfy structural, architectural, cost, production, shipping, and erection requirements.

Optimization of Members:

Beams: Due to the height restriction in Washington D.C. choosing a beam depth is the first concern. Therefore, a T-Beam is the most sufficient, depending the size of the beam 8-16 inches could be added to the floor slab. The column to beam connection creates and moment due to eccentricity. To minimize or subtract this spans should of equal length and loading. Other ways to optimize member are:

1. Maximize repetitive and modular dimensions for plan layout and member dimensions.
2. Use simple spans whenever possible.
3. Minimize the number of different member types and sizes.
4. Minimize the number of different reinforcing patters in the particular member type.

Columns: Because this is a condominium building space is money. As a result smaller column sizes are desired.

Span to Depth Ratio

Hollow-core floor slabs	30 to 40
Hollow-core roof slabs	40 to 50
Stemmed floor slabs	25 to 35
Stemmed roof slabs	35 to 40
Beams	10 to 20

Architectural: City Vista is a condominium building whose previous gravity system used longer spans and less columns than a precast system. The column grid was also skewed to accommodate the architectural plan. When planning a new column layout it will be a challenge to create a uniform grid while considering the open architectural plan. Although additional columns will reduce column sizes.

Cost Info

Hollow Core Planks	$ 8.15 (8" plank 20 ft span)
Beams	$ 143.00 (12x16 T beam 20 ft span)
Columns	$ 69.00 (14 ft 14x14 column)

Figure #15: All info is from RSM and only includes material cost

Transportation:

Weight:
No Permit: 20 Tons of Material
Permit: up to 100 Tons
Washington D.C.:

Dimensions:
No Permit: 8ft x 40ft
Permit: 13.5ft x 70ft
Washington D.C.: 13.5':tall 8': wide (2' overhang limit)
-Per load permits available but expensive

Permit Restriction:	Limited to Monday: After Noon Friday: Until Noon M-F: Not during rush hour (8-9 am & 5-6pm)

Connection Concepts: Pre-Cast connections can be complicated, as a result during preliminary design connection options should be examined.

Erection Connections:
<u>Wet Cast:</u> Allows for many types of anchors because they are installed during casting.
<u>Dry Cast:</u> Limited to shallow anchors because anchors are inserted with grout after production and curing.
Member Connections:
Beams connect to columns using hanger. These connections are expensive, provide little bearing area, and are susceptible to fabrication errors although, these connections work well in areas where floor to ceiling height is an issue.

Control of volume change: Once preliminary design has been established charts located in the PCI handbook should be used to establish totally shortening/expansion due to creep and shrinkage. These strains will be used when designing connections.

Basic design data:
1. Occupancy of the structure : Residential | R-2
2. Fire ratings
   TYPE: II
   Walls: 2 Hours
   Floors: 2 hours
   Cover: Concrete topping 2 → 2.5”

Min Slab Thickness: 6 inches
Slab Width: 8” (6” plank 2” topping)

PRELIMINARY DESIGN

Preliminary Column Layout:

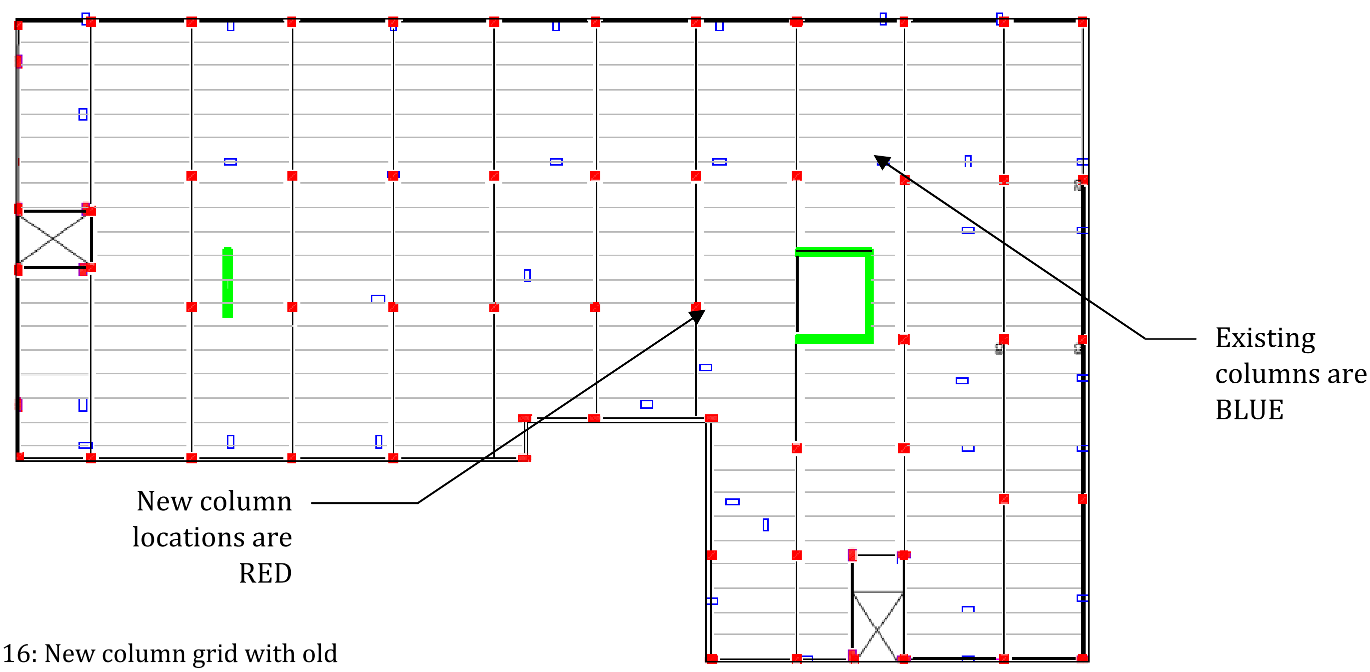


Figure #16: New column grid with old overlaid

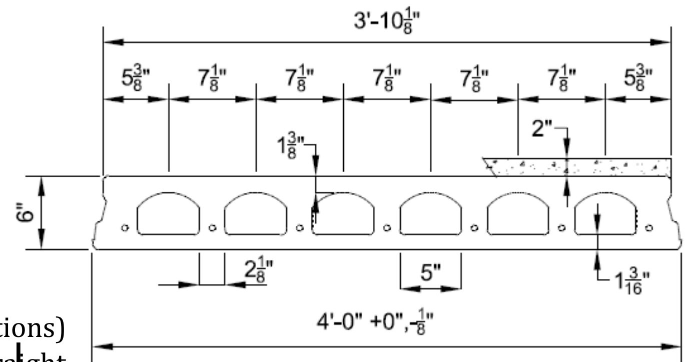
MEMBER SELECTION:

Hollow Core Planks:
Planks were selected from manufacturer Nitterhouse [www.nitterhouse.com]. This company was chosen because it services the D.C. area and has a good reputation for quality.
USE: 4' x 6" hollow core planks with 2" topping (2 hr fire rating) 7-1/2" Ø strands
(See Appendix #2 for Calculations)

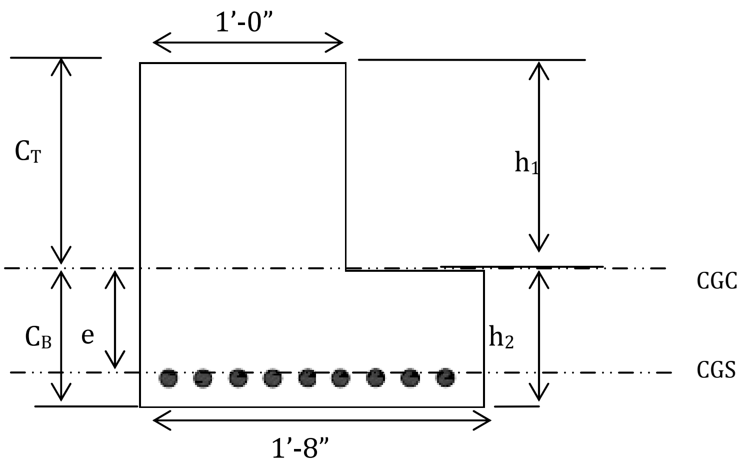
PHYSICAL PROPERTIES

A_c=253 in^2
I_c=1519 in^4
Y_{bc}=4.10in
Y_{tc}=1.90in

Precast S_{bc} = 370 in^3
Topping S_{tc}= 551 in^3
Wt=195plf
Wt=48.75psf



***Exterior-Beams**:* (See Appendix #2 for full calculations)
20LB20/ 98-S: (9) ½"Ø low relaxation strands – straight



PROPERTIES 20LB20 :

A=304 in^2
I=10,160 in^4
h1=12 in
h2=8 in
C_t = 11.26 in
C_b= 8.74 in
Wt=317 plf
e =6.33 "

S_b= 1,163 in^3
S_t= 902 in^3
f'c=5000psi
f_{pu}=270ksi
A_{sp}= 9(0.153)= 1.377 in^2
Po= 278.8 Kips

Beam Designation	Wu plf	TRAIL SIZE
LB-1	1147.3	20LB20
LB-2	1147.3	20LB20
LB-3	1147.3	20LB20
LB-4	1147.3	20LB20
LB-5	353.0	20LB20
LB-6	353.0	20LB20
LB-7	1558.5	20LB20
LB-8	1558.5	20LB20
LB-9	1270.8	20LB20
LB-10	1270.8	20LB20
LB-11	353.0	20LB20
LB-12	353.0	20LB20
LB-13	353.0	20LB20
LB-14	353.0	20LB20
LB-15	353.0	20LB20
LB-16	353.0	20LB20

Beam Designation	Wu Plf	Trial Size
LB-16	353.0	20LB20
LB-17	353.0	20LB20
LB-18	353.0	20LB20
LB-19	1059.0	20LB20
LB-20	1059.0	20LB20
LB-21	353.0	20LB20
LB-22	353.0	20LB20
LB-23	353.0	20LB20
LB-24	353.0	20LB20
LB-25	353.0	20LB20
LB-26	353.0	20LB20
LB-27	353.0	20LB20
LB-28	353.0	20LB20
LB-29	353.0	20LB20
LB-30	353.0	20LB20
LB-31	353.0	20LB20

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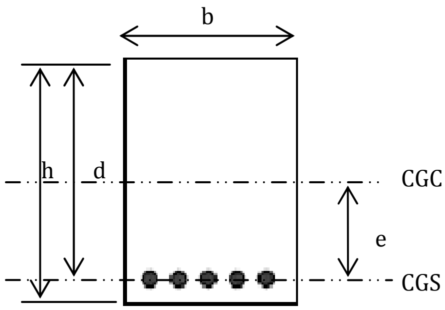
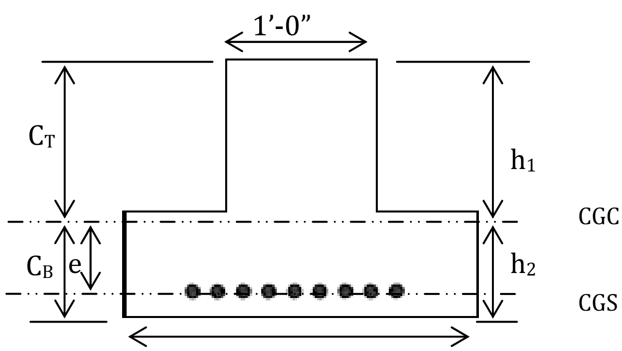
CITY VISTA
WASHINGTON D.C.
ADVISOR: DR. MEMARI

Interior Beams: (See Appendix #2 for calculations)
12RB16 / 58-S: (8) ½"ø low relaxation strands - straight



28IT20 / 98-S: (9) ½"ø low relaxation strands – straight
28IT24 / 188 -S: (18) ½"ø low relaxation strands – straight



PROPERTIES 12RB20 :

A=192in²
I=4096 in⁴
h=16 in
S = 512 in³
b = 12 in
Wt=200 plf
e = 5"
d=8"

f'c=5000psi
f_{pu}=270ksi
A_{sp}= 5(0.153)= 0.765 in²
Po= 154.9Kips

PROPERTIES 28IT24 :

A=480 in²
I=20,275 in⁴
h1=12 in
h2=12 in
C_t = 14.4 in
C_b= 9.60 in
Wt=500 plf
e =6.87"

Yb= 9.60in
S_b= 2,112 in³
S_t= 1,408 in³
f'c=5000psi
f_{pu}=270ksi
A_{sp}= 18(0.153)= 2.754 in²
Po= 557.6 Kips

PROPERTIES 28IT20 :

A=368 in²
I=11,688 in⁴
h1=12 in
h2=8 in
C_t = 12.09in
C_b= 7.91 in
Wt=383 plf
e =5.47"

Yb= 7.91in
S_b= 967 in³
S_t= 1,478 in³
f'c=5000psi
f_{pu}=270ksi
A_{sp}= 9(0.153)= 1.377 in²
Po= 278.8 Kips

Interio Beams

DESIGNATION	Wu plf	TRAIL SIZE
TB-1	2665.15	28IT24
TB-2	2665.15	28IT24
TB-3	2665.15	28IT24
TB-4	2665.15	28IT24
TB-5	2797.29	28IT24
TB-6	-	28IT24
TB-7	3855.00	28IT20
TB-8	3855.00	28IT20
TB-10	2797.29	28IT24
TB-11	4692.99	28IT20
TB-12	2915.77	28IT20
TB-13	2915.71	28IT20
TB-14	2118.00	28IT20
TB-15	2733.40	28IT20
TB-16	•	28IT24
TB-17	2829.49	28IT20
TB-18	2733.40	28IT24
TB-19	•	28IT24
TB-20	2319.49	28IT20
TB-21	2733.40	28IT24
TB-22	•	28IT24
TB-23	2738.76	28IT24
TB-24	2733.40	28IT24
TB-25	•	28IT24
TB-26	3202.36	28IT24
TB-27	2733.40	28IT24
TB-28	•	28IT24
TB-29	2734.46	28IT24
TB-30	2733.40	28IT20
TB-31	•	28IT24
TB-32	2734.46	28IT24
TB-33	2331.74	28IT28
TB-34	2331.74	28IT28
RB-1	1563.20	12RB16
RB-2	2316.25	12RB16
RB-3	485.44	12RB16
RB-4	513.57	12RB16
RB-5	513.57	12RB16
RB-6	2316.25	12RB16
RB-7	•	12RB16
RB-8	545.00	12RB16
RB-9	•	12RB16

* Designate beams with special loading conditions. Analysis for these beams can be seen in Appendix #2

Columns

A summary of the column sizing and loading can be found in appendix #2. Columns were sized every 4 floors to accommodate both the decrease in gravity loads and the ability to cast columns to span 2 stories. Columns were designed and then checked in PCA column.

The following columns were selected:

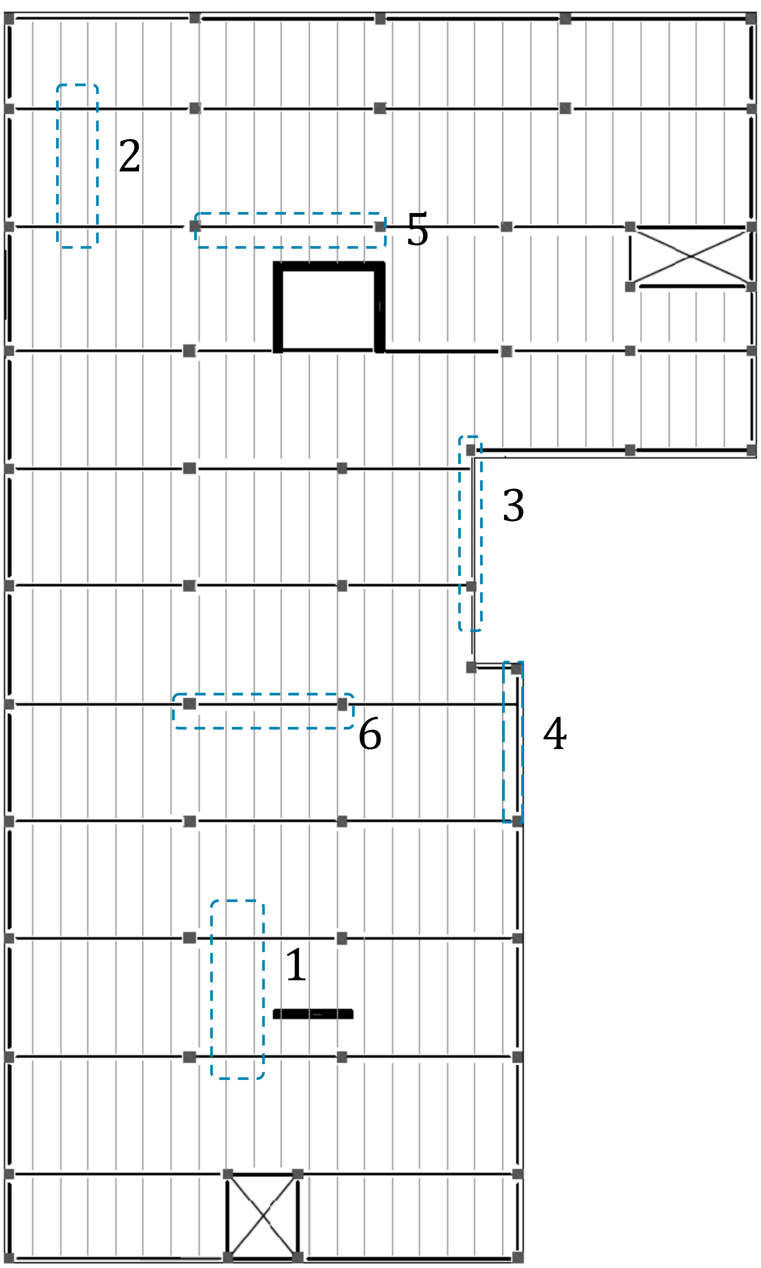
16"x16" | f'c =5000 psi | 4-#8 Bars

18"x18" | f'c = 5000 psi | 4-#9 Bars

20"x20" | f'c = 5000 psi | 4-#9 Bars

24"x24" | f'c = 5000 psi | 4-#11 Bars

FINAL LAYOUT / MEMBER CHECK



Highlighted pre-cast members are either worst case scenarios or special loading conditions and detailed checked were done for the following: (results can be seen in appendix #2)

1. Flexure
2. Shear
3. Transfer Stress
4. Pre-Stress Losses
5. Serviceability
6. Deflection and Camber

1: *Hollow Core Planks:* Corridor Loading

2: *Hollow Core Planks*: Residential Loading

3: *Exterior L-Beam:* Special loading condition

4:*Exterior L-Beam:* Special loading conditions

5. *Interior T-Beam:* Combined loading conditions (LL 40-100)

6. Interior T-Beam: Combined loading conditions (LL 40-100)

PRELIMINARY CHECK

GRAVITY SYSTEM ISSUES:

Floor Section: Due to pre-cast construction the floor depth and composition has changed. Initially City Vista was a 7 ½" flat plate slab. Now it is an 8" slab with beams framing in every bay.

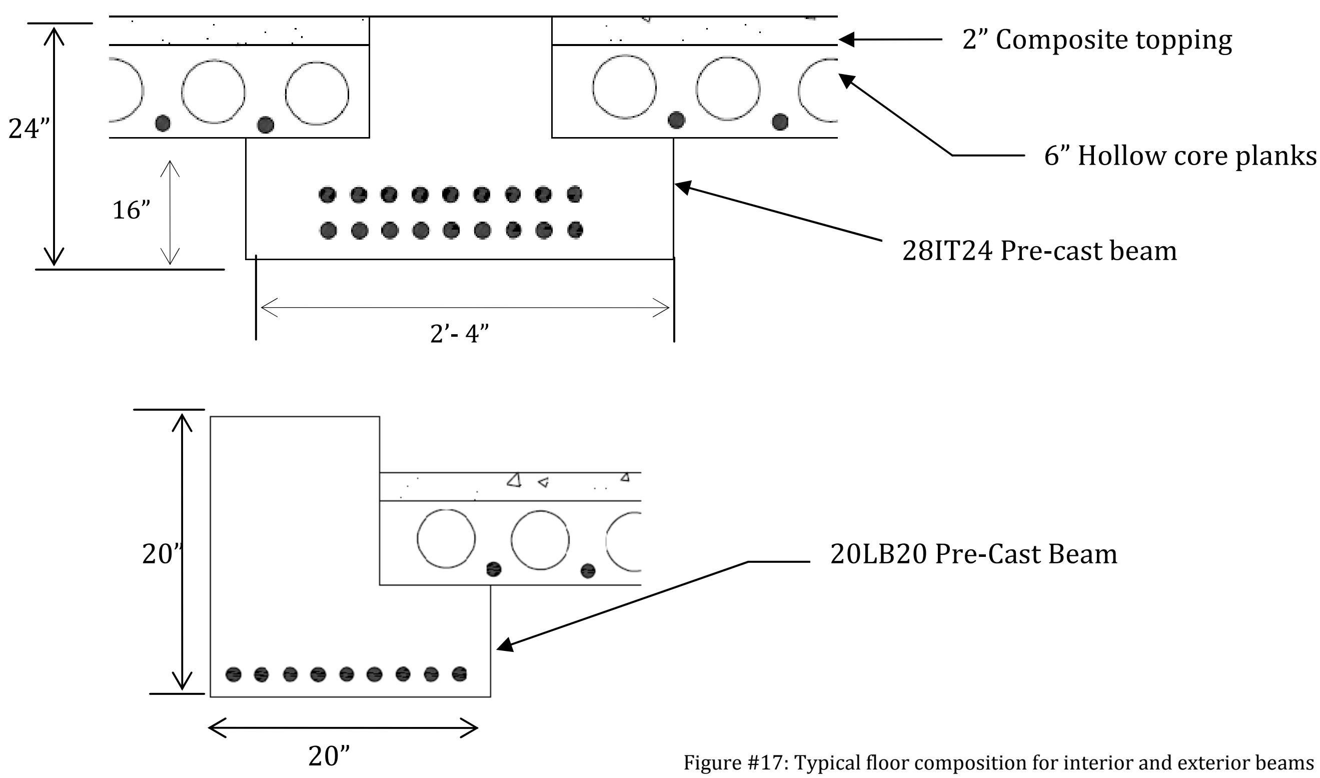


Figure #17: Typical floor composition for interior and exterior beams

Load Distribution:
ISSUE: If load is not applied to the center of a plank the plank has the tendency to twist and deflect, grouting forces neighboring beams to take some of the deflections. Shear forces are now created causing torsion and now the system act as a 2 way slab that transfers bending moment .
SOLUTION: The 2"composite topping will create a monolithic slab so forces will be distributed between planks. Also no significant point loads are seen by the hollow core planks, therefore load distribution won't be an issue

Creep and Shortning:
Creep occures over time by continuous loading, causing pre-stress losses, deflection, and stresses in non bearing members. Concrete shortens as it cures. These factors and others were considered in the pre-stress losses calculations found in appendix #3.

TOTAL STRAIN = 4.43E-4+150E-6= **<u>5.93 x 10^{-4}</u>**
TOTAL SHORTNING = 5.93E-4(12)(26)= **<u>0.185 in</u>**

CONNECTIONS:

Column to Foundation: Currently City Vista Building 2 is slab on grade construction with augured piles. Columns sit on pedestal pile caps 48" deep and 4'-6" to 12'-6" in length. For the new pre-cast system I am proposing the current augured pile pedestal system with the addition of base plates and anchors to secure the pre-cast columns. (For calculations see Appendix #3) The following calculations were done for a column C18 a 20"x20" column.

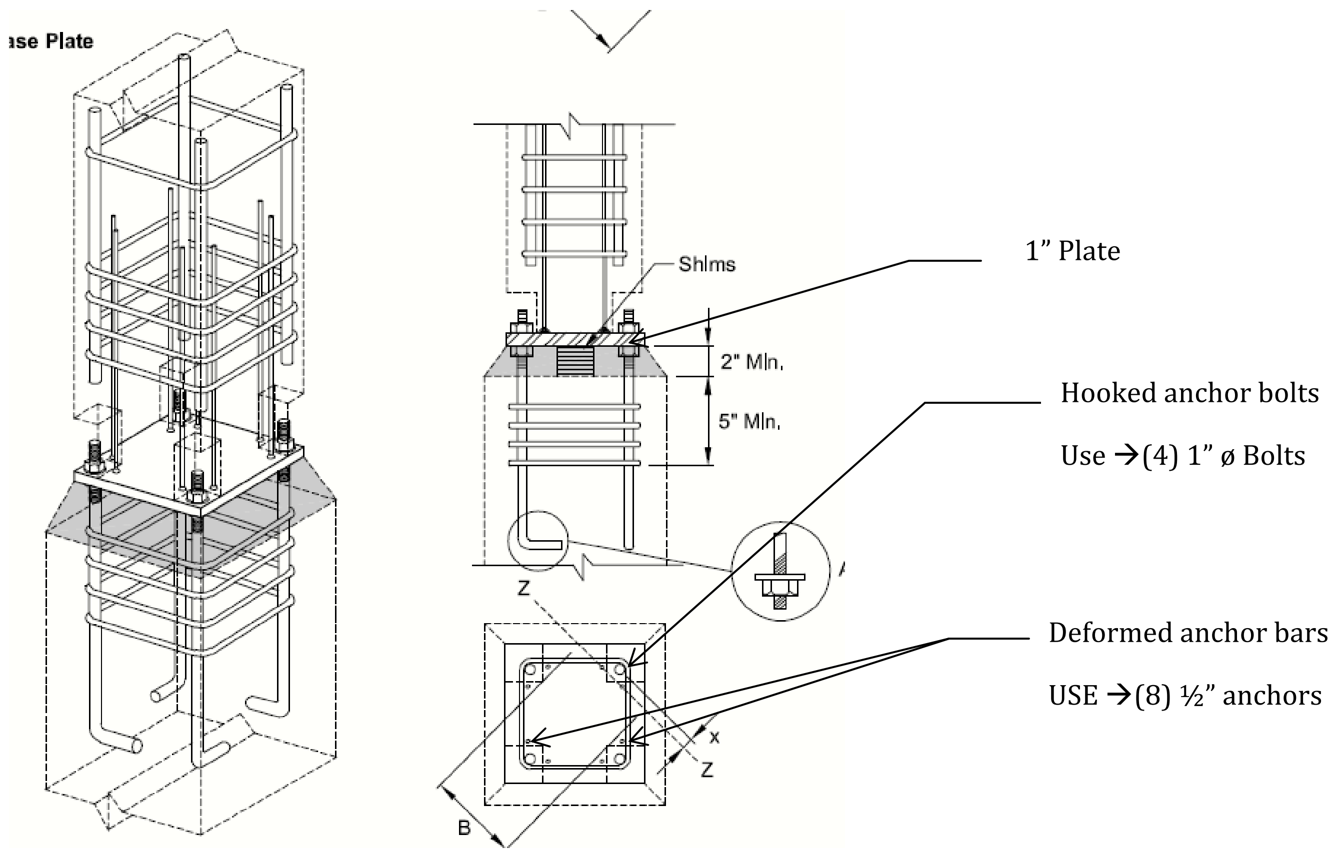


Figure #18 :Typical detail as per PCI 6th edition

Hollow Core to Beam: Both the L-Beam and T-Beam use ledgers to support the hollow core planks, therefore the ledgers must be reinforced to support the load. A bearing pad is placed on ledgers to help distribute the load. Uneven loading on T-beams create torsion, a remedy is additional reinforcing in the ledgers. (All calculations in Appendix #3) Bearing pads are to be ½" past edge, and hollow core planks are required to bear a minimum of 3 in or $l_n/180$ past ledge.

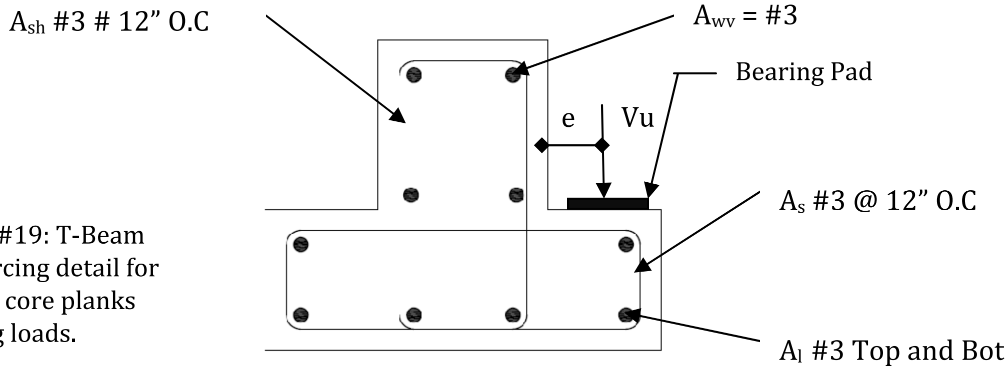
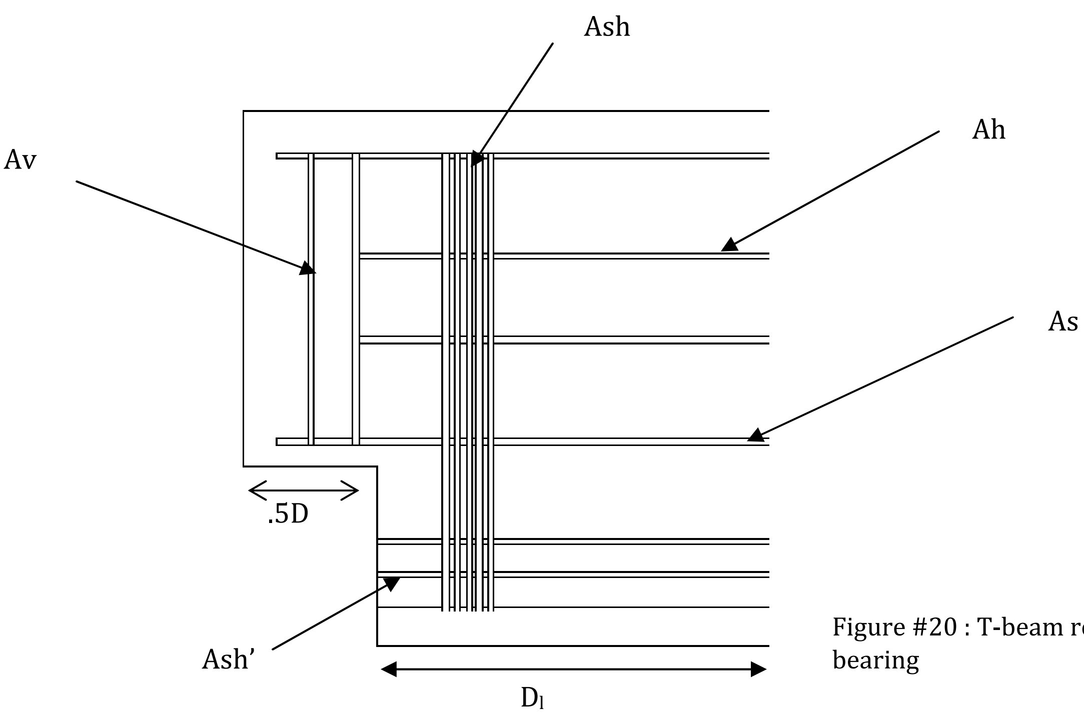


Figure #19: T-Beam Reinforcing detail for Hollow core planks bearing loads.

Beam to Column: Beams will be poured with dapped ends to connect to columns with hanger connections. Hangers are the most expensive and prone to errors during production. Although in a situation where floor to ceiling height is tight it is the best option to minimize space. Corbels may be easier to manufacture and erect, but take up considerably more space. Reinforcing will be added for direct shear, flexure, tension, and diagonal. All calculations are located in Appendix #3.



Av= 2 #4
Ash'= 5 #4
Ash= 4 #5
Ah= 2 #4
As = 2 #4

Development Length :
As, Ah and Ash'
= 26.6"

Figure #20 : T-beam reinforcing detail for column bearing

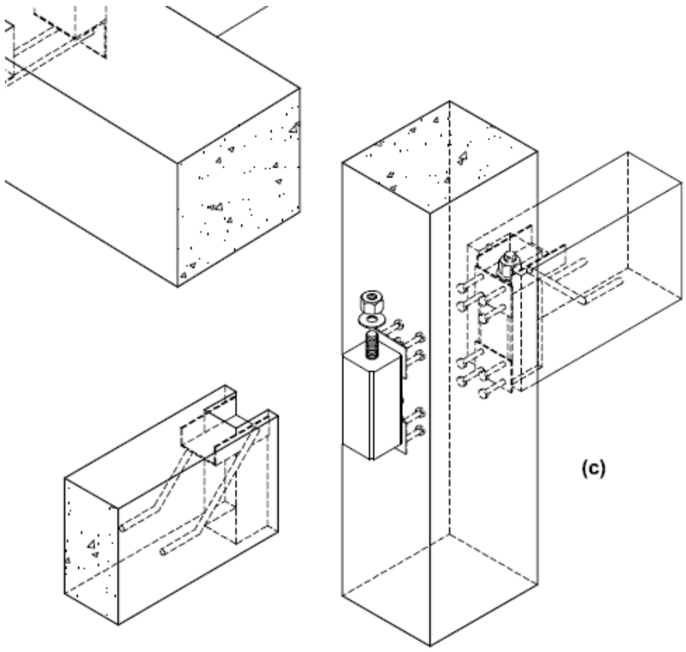


Figure #21 : Typical hanger detail as found in PCI 6th edition.

The actual hanger and bolts have not been designed. In this exercise only the reinforcing relating directly to the T, L and R beams manufacturing.

Full calculations can be found in appendix #3

DIAPHRAM

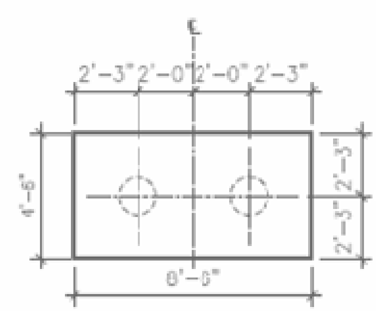
In the pre-design stages a composite topping was selected for its load distribution, additional strength, connection of planks, and fire rating. When the horizontal stress between the topping and hollow core planks is limited to 80 psi the diaphragm can be contained in the topping eliminating keyways and grouting in hollow core planks.

If Horizontal Shear: VQ/I < 80 PSI; the chords struts and drags can all be contained with-in the topping. The actual design of the diaphragm is out of the realm of knowledge, and is not included in the scope of this gravity analysis.

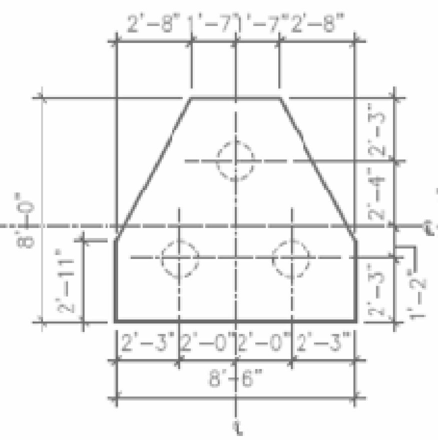
FOUNDATION CAPACITY:

The current foundation system will be compatible with the pre-cast system as well. The new system may require additional pile caps but not many. The new system is 3,000 kips heavier than the PT system, so additional piles will need to be added, although this will be accounted for by the additional pile caps and pedestal footings for the 5 additional columns.

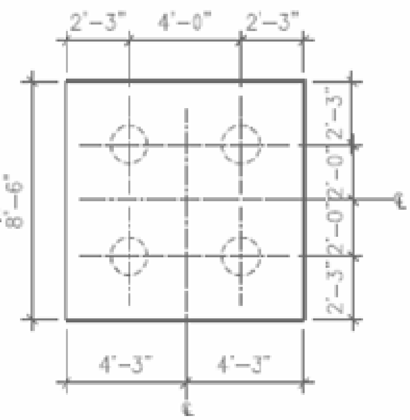
PT System



Pile Cap # 1



Pile Cap #2



Pile Cap #3

#1 : Column load 0-500 Kips

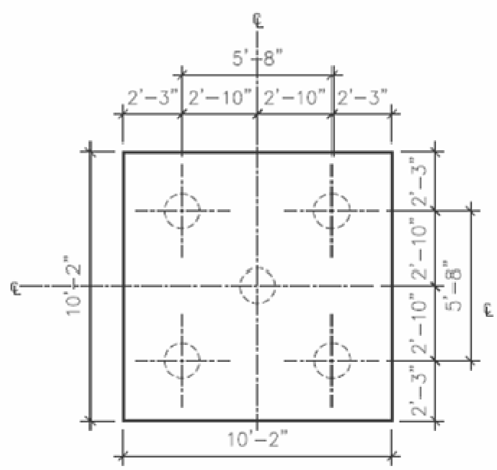
#2 : Column load 0-700 Kips

#3 : Column load 700-999 Kips

#4 : Column load 1000-1030 Kips

Foundation Capacity Numbers

1 Pile = 125 Tons
1 Kips = 0.446 Tons
[1 ton = 2.24 kips]



Pile Cap #4

<u>Pre-cast System,</u>

The maximum column load is 984 kips, therefore all the columns in the new system fall into one of the four pile cap categories. The only alteration to the foundation system will be the column to pedestal connections shown in the connection section on page 27.

LATERAL CHECK

The new floor system created a slightly taller and heavier building; therefore new seismic and wind calculations were formulated. A lateral check was then conducted for lateral stability. PCA Column was used flexural strength, shear reinforcing and building drift was also examined. The (4) walls were not expected to need any alteration due to only a 3' increase in wall height and 3000 kip increase in weight. [For more detail see appendix #4]

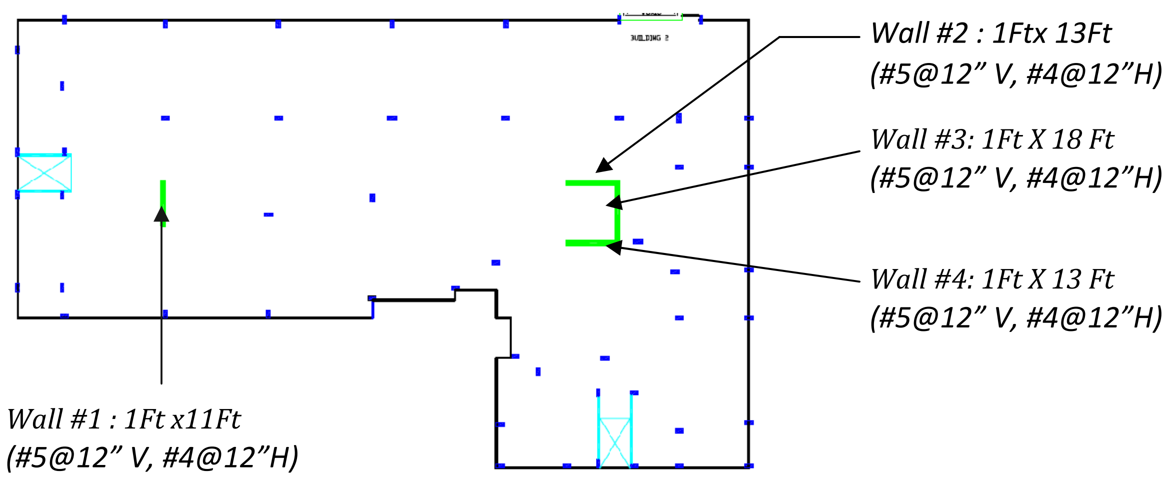


Figure #22: Shear Wall Locations, Size, and Reinforcing

Load Combinations **: Strength Design**
[ASCE 7-05 2.3.2 & 12..2.3]
#1 : 1.4D
#2: 1.2D + 1.6L + 0.5S
#3: 1.2D + 1.6S + 0.8W
#4: 1.2D + 1.6W + L + 0.5S
#5: $(1.2+0.2S_{ds})D + \rho E + L + 0.2S$ → ρ [Design Category D, H/L<1 1.0
$S_{ds} = .163$
#6: 0.9D + 1.6W + 1.6H
#7: $(0.9 - 0.2S_{ds})D + \rho E + 1.6H$

FLEXURE CHECK

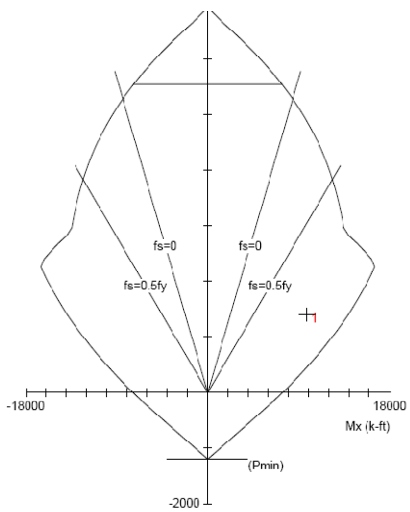


Figure #23: Interaction Diagram for Shear Wall #2 & Wall #4
Pu = 1,401.9 Kips
Mu= 9,806.8 Kip-ft
COMBO 50
Reinforcing : #8 @ 12" O.C (Floor 1-2)
#5 @ 12" O.C (floors 3-11)

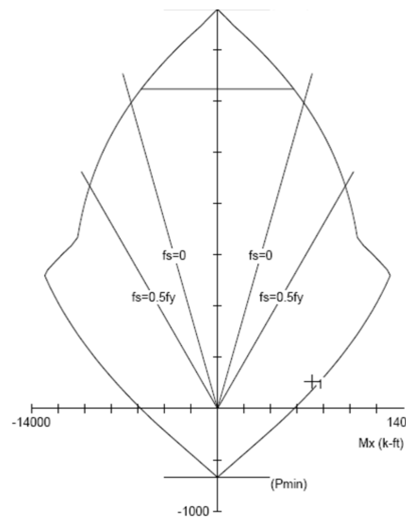


Figure #24 : Interaction Diagram for Shear Wall #3
Pu= 264.57 Kips
Mu=7,144 Kip-ft
COMBO 48
Reinforcing: #5 @ 12" O.C

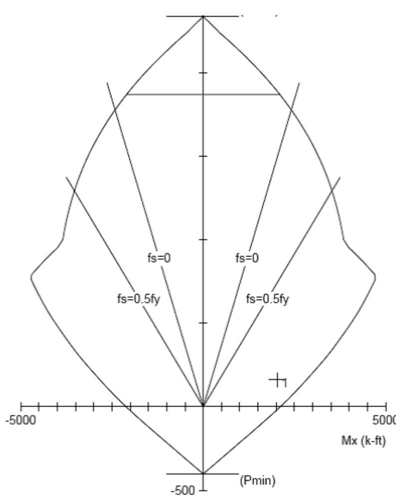


Figure #25 : Interaction Diagram for Shear wall # 1
Pu= 162.3 Kips
Mu= 2,035.7 Kip-Ft
COMBO 48
Reinforcing: #5 @ 12" O.C

Combo 48= 0.867D + 1.0 Quake X(-)Y eccentricity
Combo 50= 0.867D + 1.0 Quake Y(-) X eccentricity

All shear walls are adequate in flexure. Shear walls #1 and #3 reinforcing were consistent with the original schedule (see appendix #4). Shear walls #2 and #4 were not and needed larger reinforcing for the first 2 stories.

SHEAR CHECK

Controlling Factors : From code

(2) Curtain of Reinforcing needed when : $V_u \geq 2A_{cv}\sqrt{f'c}$ = 20.36 kips
Boundary Element needed when: Pu/Po ≤ 0.30
Wall Thickness ≥ lu/16 = 7.5" < 12" OK

Wall #1

Combo 30 :1.233 D + 1.0L + 0.2S + 1.0 Quake X (-) Y eccentricity
63.8 > 20.36 : Use 2 Curtains

Reinforcing: #4 @ 12" O.C A_s= 0.20 * 2 curtains = 0.40 in^2 > 0.36 in^2 Required
Boundary Element : 0.0328 < 0.30 None Required

Wall #2&4:

Combo 48 : 0.867D + 1.0 Quake X(-)Y eccentricity
272.48 > 20.36 : Use 2 Curtains

Reinforcing*:* #5 @ 12" O.C A_s = 0.31 *2 curtains = 0.62 in^2 ≥ 0.62 in^2 Required
Boundary Element: 0.79 < 0.30 None Required

Wall #3:

Combo 30 : 1.233 D + 1.0L + 0.2S + 1.0 Quake X (-) Y eccentricity
400.288 > 20.36 : Use 2 Curtains

Reinforcing: #4 @ 12" O.C A_s = 0.20 * 2 curtains = 0.40 in^2 ≥ 0.36 in^2 Required
Boundary Element: 0.34 > 0.30 Required 4 ft each side

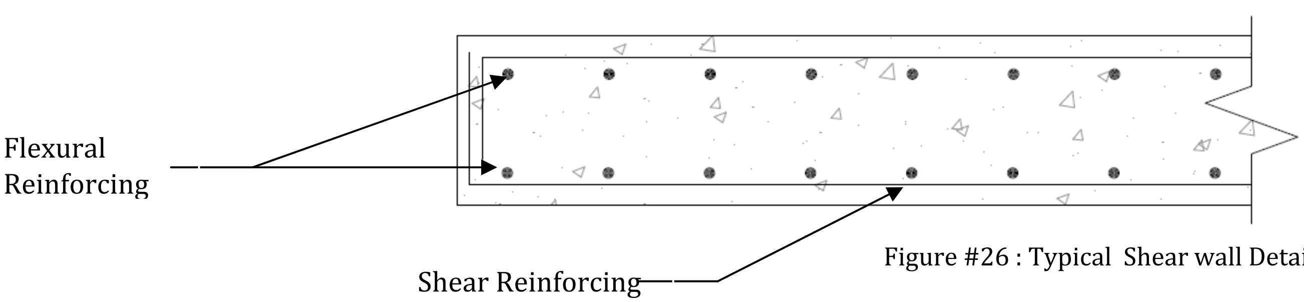
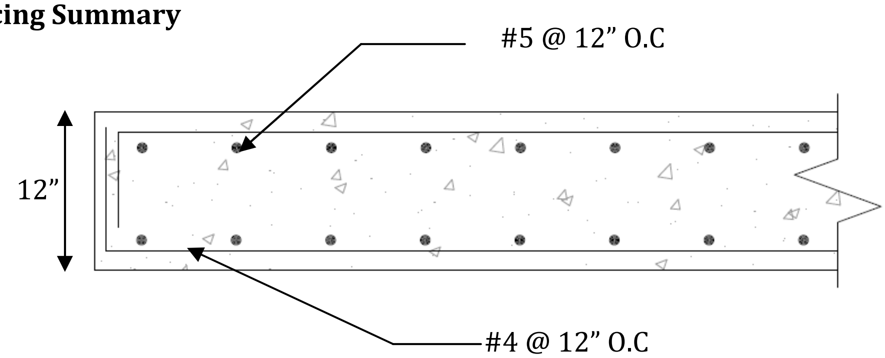


Figure #26 : Typical Shear wall Detail

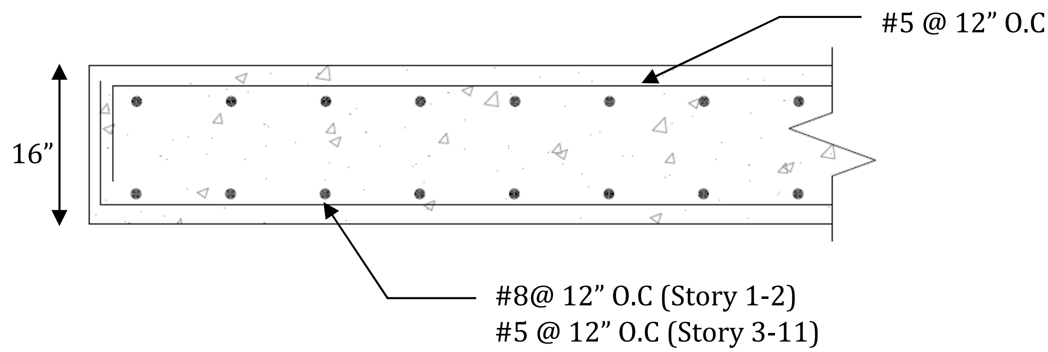
Again shear walls #1 and #3 were consistent with the original reinforcing schedule (see appendix #4) specified by the engineer. Shear walls #2 and #4 were not and required larger reinforcing. Bars were increasing from #4 to #5. This was expected after flexural reinforcing was increased.

Wall Reinforcing Summary

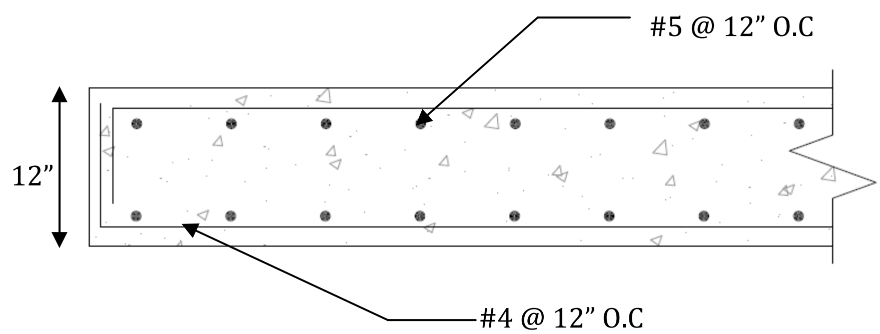
Wall #1 :



Wall #2&4:



Wall #3 :



For full reinforcing detail and the existing reinforcing schedule see appendix #4

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STRUCTURAL OPTION
APRIL 9, 2008

DRIFT CHECK

An E-tabs serviceability model was created to check the displacement and story drift . PΔ affects were taken into consideration during E-Tab's analysis, with the non-iterative mass base method. Seismic base shear controlled, therefore seismic displacement was examined. Story drift and displacement values are taken from the E-Tabs output. To comply with ASCE7-03 story drift was multiplied by an amplification factor (C_d) and then divided by the importance factor (I_e) before comparing it to the story drift limit (Δ_b) specified in *ASCE7-03 Table 12.12-1.* In the charts below story drift and δ_{ei} (story displacement) are taken from the E-Tabs output. δ_i is the amplified displacement.

Story Drift Requirement = $(\delta_e\text{-}\delta_e)C_d/I_e \le \Delta$

	Wall #3			Wall #1			Quake X Direction		
Story	Story Drift	δei	δi	Drift	δei	δi	Total Story Drift (δt)	Δb	δt≤Δb
11	0.26	1.17	0.91	0.26	1.17	0.91	1.82	2.40	OK
10	0.26	1.16	0.90	0.259	1.17	0.91	1.81	2.40	OK
9	0.26	1.16	0.90	0.258	1.16	0.90	1.81	2.40	OK
8	0.26	1.15	0.89	0.255	1.15	0.89	1.79	2.40	OK
7	0.25	1.11	0.86	0.247	1.11	0.86	1.73	2.40	OK
6	0.23	1.05	0.82	0.234	1.05	0.82	1.64	2.40	OK
5	0.21	0.96	0.75	0.214	0.96	0.75	1.50	2.40	OK
4	0.19	0.85	0.66	0.188	0.85	0.66	1.32	2.40	OK
3	0.15	0.69	0.54	0.153	0.69	0.54	1.07	2.40	OK
2	0.11	0.51	0.40	0.1137	0.51	0.40	0.80	2.40	OK
1	0.07	0.32	0.25	0.0715	0.32	0.25	0.50	3.12	OK

Symbol Key

δ_{ei} = Elastic displacement taken from E-Tabs model

$\delta_i = [C_d\delta_{ei}/I_e]$ Amplified displacement

$\Delta_b = 0.020h_{sk}$ Allowable story drift

	Wall #2&4			Quake Y Direction		
Story	Story Drift	δei	δi	Total story Drift (δt)	Δb	δt≤Δb
11	0.54	2.43	1.89	3.78	2.40	NG
10	0.53	2.39	1.86	3.71	2.40	NG
9	0.52	2.34	1.82	3.64	2.40	NG
8	0.51	2.30	1.79	3.57	2.40	NG
7	0.48	2.16	1.68	3.36	2.40	NG
6	0.44	1.98	1.54	3.08	2.40	NG
5	0.43	1.94	1.51	3.01	2.40	NG
4	0.41	1.85	1.44	2.87	2.40	NG
3	0.39	1.76	1.37	2.73	2.40	OK
2	0.25	1.13	0.88	1.75	2.40	OK
1	0.165	0.74	0.58	1.155	3.12	OK

Both the X and Y direction met displacement and drift requirement

Total Displacement: **X = 1.82 in OK**

Y = 3.78 in NG

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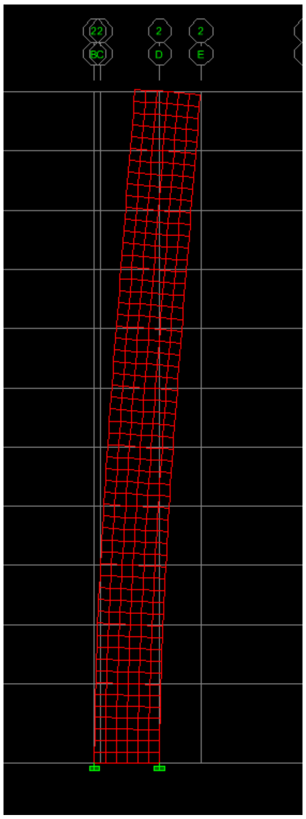
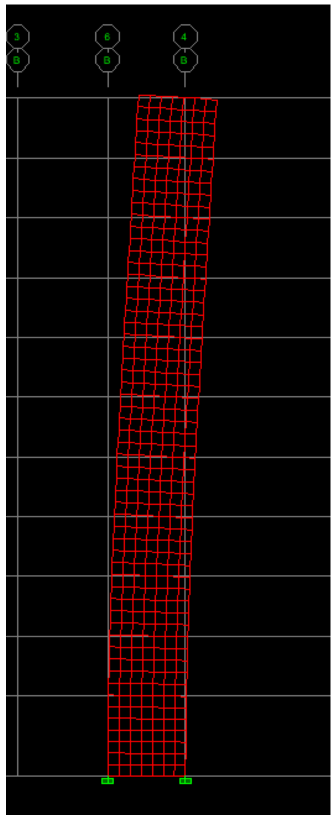
APRIL 9, 2008

Displacement Summary



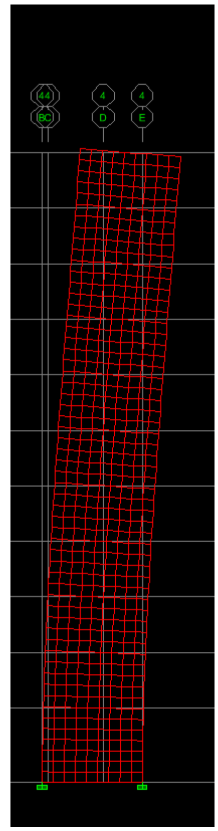
Wall #1 : Deformed shape under seismic forces in the X direction.

Wall Adequate



Wall #2 & 4 : Deformed shape under seismic forces in the Y direction

Wall Inadequate



Wall #3 : Deformed shape under seismic forces in the X direction

Wall Adequate

After examination of the lateral system it can be concluded that shear walls #2 and 4 are inadequate and will need to be redesigned. Even after thickening these walls to 16" the walls were unable to resist seismic displacement. This was partly expected after reinforcing was increased to support flexure and shear in both walls. To correct this problem an increase in length or different placements of the (2) walls will need to be examined further.

Appendix #1

SESMIC AND WIND CALCULATIONS

REVEISED BUILDING

Seismic *(ASEC7-05 : Chapter 11-12)*

Latitude / Longitude :
RESULTS FROM SOFTWARE :
$S_s = 0.153$
$S_1 = 0.05$
Fa = 1.6 (Table 11.4-1)
Fv = 2.4 (Table 11.4-2)
$Sm_s = FaS_s = 0.2448g$
$Sm_1 = FvS_1 = 0.12g$
$SD_S = 0.163g$
$SD_1 = 0.08g$

Site Classification : D
Design Category: B
Occupancy Category: II
Building Height : 128'-0"
Seismic Use Group: Group Importance Factor: 1.0

Table 12.8-1 $C_U = 1.7$ [$S_{D1} \leq 0.1$]
Table 12.8-2; $C_T = 0.02$ x=0.75
$Ta = C_t H_n^x = (0.02)(113)^{0.75} = 0.69$
T : Fundamental Period of Structure = $C_u T_a = (1.7)(0.69) = 1.17$
T_L = [Fig 22-15] Long-Period transition period = 8 Sec

Table 12.2-1: *Ordinary plain concrete shear walls* R = 5.0
$\Omega = 2.5$
$C_D = 4.5$

$$C_S = \begin{cases} S_{DS}/(R/I) = (0.163)/(5/1) = 0.0326 \\ S_{D1}/T(R/I) = (0.08)/(1.17(5/1)) = \underline{0.013} \\ S_{D1}T_L/(T^2(R/I)) = (1.292*8)/(1.292^2(5/1)) = 1.24 \end{cases}$$

Building Weight
(W) = DEAD LOAD + 20% SNOW LOAD + ROOFTOP UNITS+20PSF PARTITION

Building Weight Summary

Floors	*Plank (psf)*	*Partitions*	*2" Topping (psf)*	*Area*	*Total (Kips)*	*Beams (Kips)*	*Shear (Kips)*	*Columns (Kips)*	*Walls (Kips)*
1	48.75	20	25	15405	1444	566.82	92.76	225	101
2	48.75	20	25	15405	1444	566.82	92.48	215.4	193
3	48.75	20	25	15405	1444	566.82	92.48	215.4	184
4	48.75	20	25	15405	1444	566.82	92.48	215.4	184
5	48.75	20	25	15405	1444	566.82	92.48	215.4	184
6	48.75	20	25	15405	1444	566.82	92.48	215.4	184
7	48.75	20	25	15405	1444	566.82	92.48	215.4	184
8	48.75	20	25	15405	1444	566.82	92.48	215.4	184
9	48.75	20	25	15405	1444	566.82	92.48	215.4	184
10	48.75	20	25	15405	1444	566.82	92.48	215.4	184
11	48.75	20	25	15405	1444	566.82	92.48	215.4	92
Pent	78.75	20	25	1738	215	41.82	92.48	25.6	337.7
				TOTAL	16101	6276.84	1017.6	2404.4	2195.7

Additional Weight:

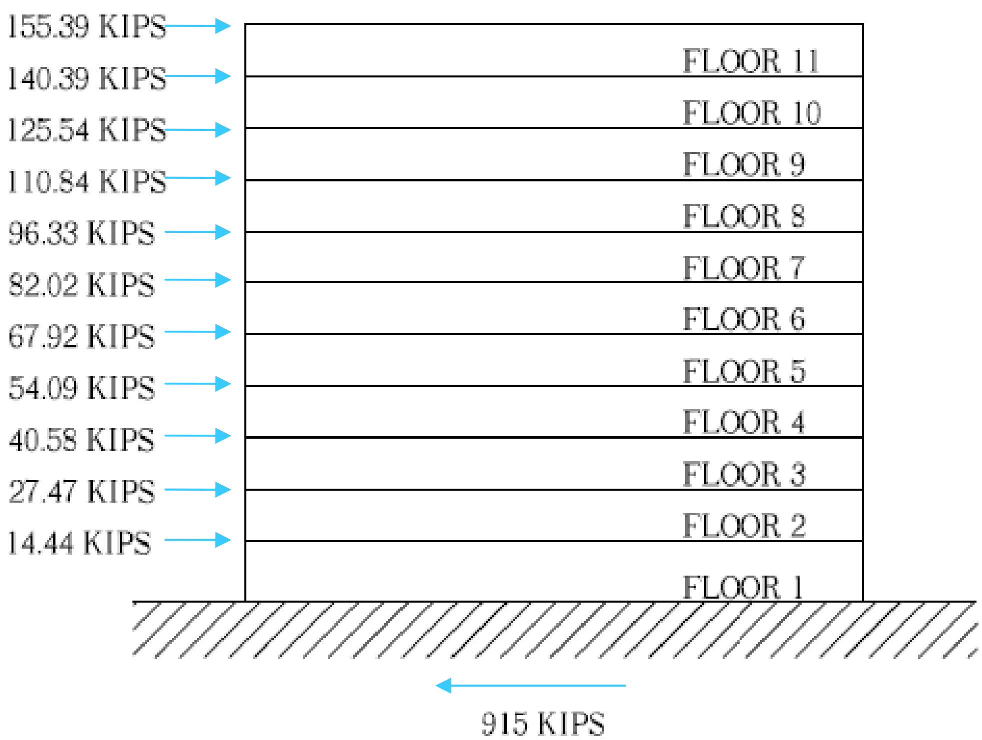
Rooftop Units = 8 Kips
Snow = 102.8 Kips

TOTAL BUILDING WEIGHT = 28,076 Kips

Base Shear: $V = C_S W = 0.0326*28,076 = 915.27$ Kips

Overturning Moment = 73,601.43 Kip-Ft

Seismic Loading								
K = 1.1	Level	W_x	H_x	$W_xH^{1.1}$	C_{vx} (k)	F_x (kips)	Vx (kips)	Mx (kip-Ft)
11	12	2321	113.46	422670.75	0.17	155.39	155.39	17630.10
10	11	2321	103.46	381878.20	0.15	140.39	295.78	14524.70
9	10	2321	93.46	341478.60	0.14	125.54	421.32	11732.73
8	9	2321	83.46	301509.75	0.12	110.84	532.16	9251.02
7	8	2321	73.46	262018.02	0.11	96.33	628.49	7076.07
6	7	2321	63.47	223100.47	0.09	82.02	710.50	5205.70
5	6	2321	53.47	184755.06	0.07	67.92	778.43	3631.75
4	5	2321	43.47	147124.07	0.06	54.09	832.51	2351.17
3	4	2321	33.48	110392.39	0.04	40.58	873.10	1358.73
2	3	2321	23.48	74721.06	0.03	27.47	900.57	644.99
1	2	2248	13.47	39273.60	0.02	14.44	915.00	194.48
						Overturning Moment		73601.43 Kip-Ft
						Base Shear		915.00 Kips



Wind: *(ASCE7-05 : Chapter 6)*

$$p=qGC_p-q_i(GC_{pi})$$

General Info:
Rigid Building T= 0.76 Sec < 1 Sec
Exposure Category = B
Enclosure Category = Enclosed Building
Basic Wind Speed: V = 90 mph
Importance Factor : I = 1.0
Mean Roof Height = 128'-0"

Calculations:
K_z: Table 6-3
K_{zt} : 1.0
K_d: Table 6-4 / *Building Main wind force resisting System* = 0.85

$$G = 0.925\,(1+1.7gI_zQ)/(1+1.7*gI_z) \begin{cases} \textbf{N-S} = 0.854 \\ \textbf{W-E} = 0.867 \end{cases}$$

$I_z = c(33/z)^{1/6} = 0.3(33/76.8)^{1/6} = 0.38$
$Z= 0.6h = 76.8ft$
[Table 6-2] $Z_{min}= 30$
$c = 0.30$
$\varepsilon= 1/3$
$\ell=320$

$$Q = \begin{cases} \textbf{W-E} = 0.909 \\ \textbf{N-S} = 0.888 \end{cases}$$

$L_z=\ell/(z/33)^{\varepsilon} = 320/(76.8/33)^{.33} = 423.9$
$g_q= 3.4$
$g_v =3.45$

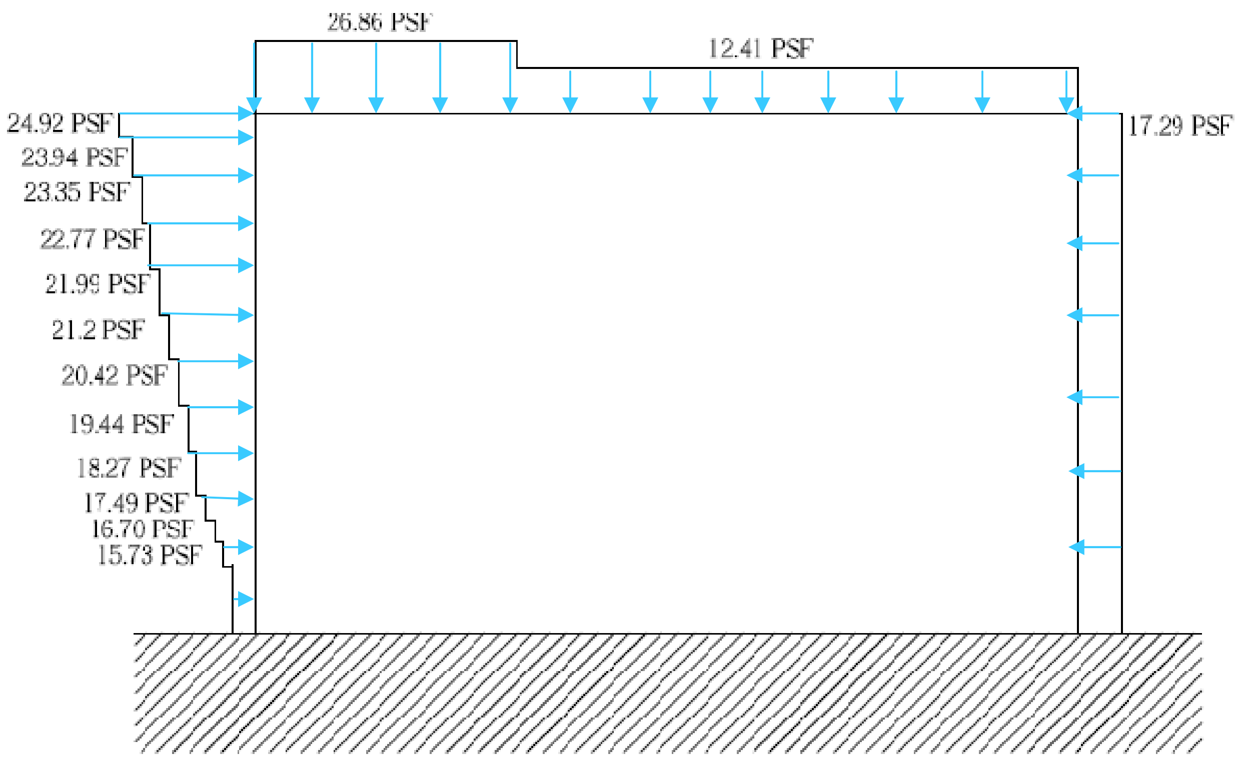
C_{pi} : FIG 6-5 = +/- 0.18
Cp: FIG 6.6 $\longrightarrow$ **W-E**: LEEWARD = -0.5 (L/B = .616)
WINDWARD= 0.8
N-S : LEEWARD = -0.3 (L/B = 1.6)
WINDWARD = 0.8

GC_{pi} = +/- 0.18	
East/West	$C_{P\ Windward}$ = 0.80 $C_{P\ Leeward}$ = -0.50
$C_{P\ Side}$ = -0.70	
North/South	$C_{P\ Windward}$ = 0.80 $C_{p\ Leeward}$ = -0.30 $C_{P\ Side}$ = -0.70

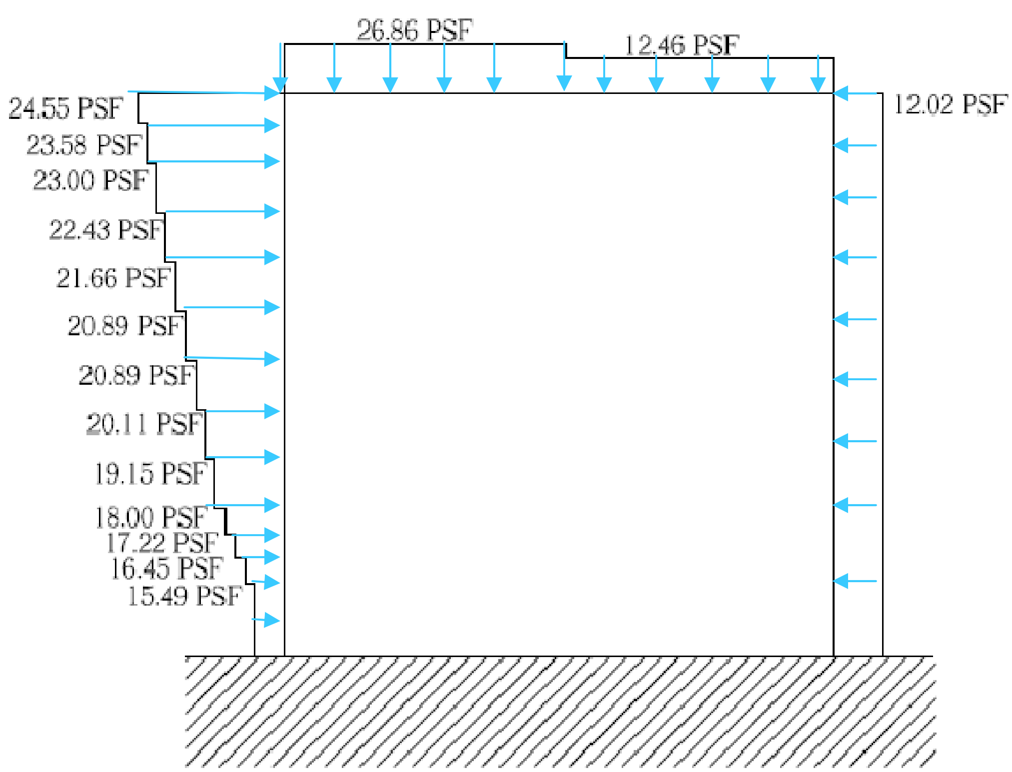
Roof:	
East/West	$C_{P\ Windward}$ = 0-h/2 → -1.3 $C_{P\ Windward}$ = >h/2 → -0.7
North/West	$C_{P\ Windward}$ = 0-h/2 → -1.3 $C_{P\ Windward}$ = >h/2 → -0.7

Wind From W-E								
Windward		Leeward		TOTAL	Area (ft^2)	P (kips)	Shear	Moment
h	P	h	p					
0-15	15.73	0-15	-17.29	33.02	2700	89.154	829.52	0.00
20	16.7	20	-17.29	33.99	900	30.591	740.37	611.82
25	17.49	25	-17.29	34.78	900	31.302	709.78	782.55
30	18.27	30	-17.29	35.56	900	32.004	678.47	960.12
40	19.44	40	-17.29	36.73	1800	66.114	646.47	2644.56
50	20.42	50	-17.29	37.71	1800	67.878	580.36	3393.90
60	21.2	60	-17.29	38.49	1800	69.282	512.48	4156.92
70	21.99	70	-17.29	39.28	1800	70.704	443.20	4949.28
80	22.77	80	-17.29	40.06	1800	72.108	372.49	5768.64
90	23.35	90	-17.29	40.64	1800	73.152	300.38	6583.68
100	23.94	100	-17.29	41.23	1800	74.214	226.17	7421.40
116	24.92	120	-17.29	42.21	3600	151.956	151.96	18234.72
					Base Shear=	830 Kips		
					Moment=	55507.59	Ft Kips	

Wind From N-S								
Windward		Leeward		TOTAL	Area (ft^2)	P(Kips)	Shear	Moment
h	P	h	p					
0-15	15.49	0-15	-12.02	27.51	1665	45.80415	436.4853	0
20	16.45	20	-12.02	28.47	555	15.80085	390.6812	316.017
25	17.22	25	-12.02	29.24	555	16.2282	374.8803	405.705
30	18	30	-12.02	30.02	555	16.6611	358.6521	499.833
40	19.15	40	-12.02	31.17	1110	34.5987	341.991	1383.948
50	20.11	50	-12.02	32.13	1110	35.6643	307.3923	1783.215
60	20.89	60	-12.02	32.91	1110	36.5301	271.728	2191.806
70	21.66	70	-12.02	33.68	1110	37.3848	235.1979	2616.936
80	22.43	80	-12.02	34.45	1110	38.2395	197.8131	3059.16
90	23	90	-12.02	35.02	1110	38.8722	159.5736	3498.498
100	23.58	100	-12.02	35.6	1110	39.516	120.7014	3951.6
120	24.55	120	-12.02	36.57	2220	81.1854	81.1854	9742.248
					Base Shear =	437 Kips		
					Moment=	29448.97	Ft-Kips	



Wind Loading Diagram *East – West*



Wind Loading Diagram *North –South*

Appendix #2
GRAVITY SYSTEM

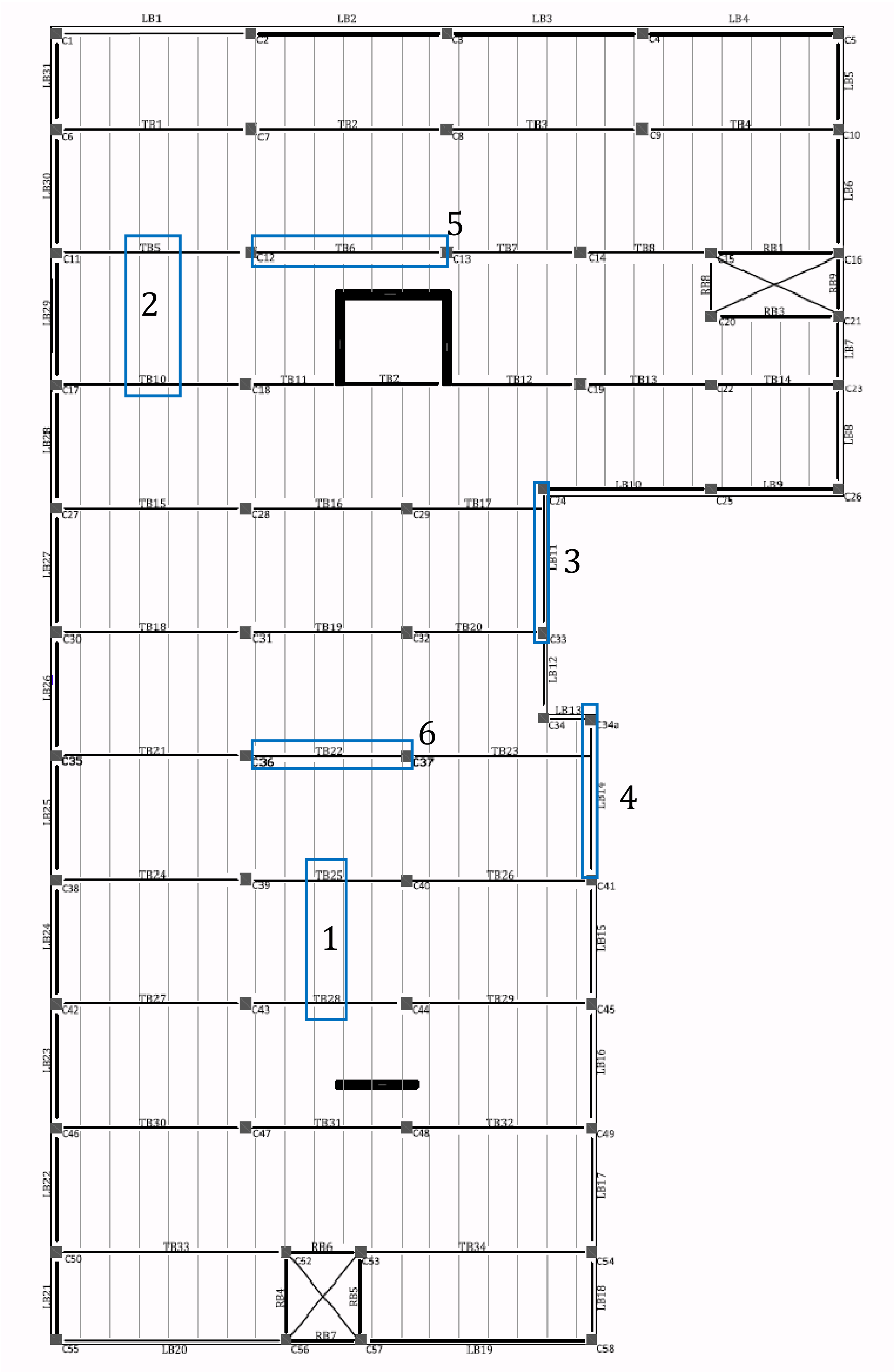
FLOOR PLAN

HOLLOW CORE

EXTERIOR BEAMS

INTERIOR BEAMS

COLUMNS

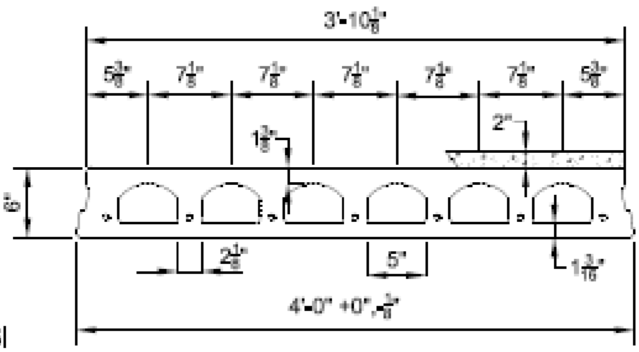

LB1
LB2
LB3
LB4
C1
C2
C3
C4
C5
LB31
LB5
TB1
TB2
TB3
TB4
C6
C7
C8
C9
C10
LB30
LB6
5
TB5
TB6
TB7
TB8
RB1
C11
C12
C13
C14
C15
C16
2
RB8
RB9
LB29
RB3
C20
C21
LB7
TB10
TB11
TB12
TB13
TB14
C17
C18
C19
C22
C23
LB28
LB8
LB10
LB9
C24
C25
C26
TB15
TB16
TB17
C27
C28
C29
LB27
LB11
3
TB18
TB19
TB20
C30
C31
C32
C33
LB26
LB12
LB13
C34
C34a
6
TB21
TB22
TB23
C35
C36
C37
LB25
LB14
4
TB24
TB25
TB26
C38
C39
C40
C41
LB24
1
LB15
TB27
TB28
TB29
C42
C43
C44
C45
LB23
LB16
TB30
TB31
TB32
C46
C47
C48
C49
LB22
LB17
TB33
RB6
TB34
C50
C52
C53
C54
LB21
RB4
RB5
LB18
RB7
C55
LB20
C56
C57
LB19
C58

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 S_{bc} = 370 in.^{3}
I_c = 1519 in.^{4}	Topping S_{tc} = 551 in.^{3}
Y_{bc} = 4.10 in.	Precast S_{tc} = 799 in.^{3}
Y_{tc} = 1.90 in.	Wt.= 195 PLF
	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 = 67.5 k-ft
   7-1/2"Ø, 270K = 104.2 k-ft
7. Maximum bottom tensile stress is $7.5\sqrt{f'c}$ = 580 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 2003 & ACI 318-02 (1.2 D + 1.6 L)

Strand Pattern		SPAN (FEET) 11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29
4 - 1/2"ø	LOAD (PSF)	227	187	360	306	268	229	194	165	141	120	102	86	73	61	50				
7 - 1/2"ø	LOAD (PSF)	367	305	495	455	418	387	340	312	275	243	215	189	167	147	130	114	97	83	70

NITTERHOUSE CONCRETE PRODUCTS
2655 Molly Pitcher Hwy. South, Box N
Chambersburg, PA 17201-0813
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.

05/14/07

6F2.0T

HOLLOW CORE CHECKS:

Pre-cast	F'_c = 6000psi F'_{ci} = 3500psi E_c =4463ksi	E_c=33(145)$^{1.5}$√6000 =4463ksi (11.2.2) E_{ci}=33(145)$^{1.5}$√3500=3408ksi F_{se}= 170ksi
Topping	F'_c =5000psi E_c = 4074 ksi	
Service loads :	F_{ti}=7.5√f'$_c$ = 580psi F_{ci}= 0.6(f'$_c$) = 3600 psi	
Allowable @ transfer:	F_t=3√f'$_{ci}$ =+180psi F_c=0.6f'$_{ci}$ =-2160psi	

1 ***Corridor Design Check***

Span =16.33'
Length =17'
Slab Thickness: 6" plank + 2" topping = 8"
LOADING:
Topping W=25PSF
LL=100PSF
Super=20PSF
Span-Depth : 17/.667 = 25.5 < 40 OK

Plank by Nitterhouse: www.nitterhouse.com
-6"x4' Hollowcore plank 2 hour rating w/ 2" topping
A_c= 253 in^2
I_c=1519 in^4
T_{bc}=4.10 in
Y_{tc}=1.90in
D_p=6"
Wt=48.75 psf
S_T=799 in^3
S_B = 307in^3

1. Preliminary Design Loads :

W=1.2(20)+1.6(100)= 184psf
6" Slab+ 2" Topping and 7-1/2"Ø strands

Capacity = 189 psf M Capacity = 104.2 k-ft

2.Transfer Stresses: 7-1/2" Ø 270ksi low relaxation strands

A_{sp}= 7(0.153)= 1.071in^2
e=2.25"
L= 17"
Wu=1.2*(48.75+25)*4ft=354 plf
M_D= (295)(17^2)/8=10.6 ft-kips → 127 in-kips
Po=0.6$A_{ps}F_{pu}$ = (1.071)(270)(0.60) = 173.5 Kips
Pi= 0.153*270*.75*7=216.87

CHECK
F_c= P_o/A ± P_Oe/S ± M_D/S =
F_{TOP} =0.685-0.708+0.15= ***= 0.127 KSI ≤ F_{ti} OK***
F_{BOT}=0.685+1.055-0.41 = ***=1.33 KSI ≤ F_{ci} OK***

3. **Prestress Losses**:

P_i=216.87kips	f_{pu}=270ksi
Pe=Pi-RA_{ps}	f_{pi}= P_i/A_{ps} = /1.071
R= ES+CR+SH+RE	f_{pu}/f_{pi}=0.749
$E_c=33(145)^{1.5}\sqrt{6000}$ =4463ksi	A_g=384in^2
$E_{ci}=33(145)^{1.5}\sqrt{3500}$=3408ksi	I_g=2048in^3
R.H.= 75%	e = 2.25”
V/S= 384/884= 4.36	

Elastic Shortening: $K_{es}E_{ps}F_{cir}/E_{ci}$ = (1)(28.5E3)(0.82)/3400 = 6.80 KSI
F_{cir}= $K_{cir}(P_i/A_g + P_ie^2/I_g)-M_ge/I_g$ (Mg=1.2(48.75+25)*4*18^2/8)= 153.5 in-kips
=0.9[(216/384)+(216*2.25^2)/2048)]-(153.5*2.25/2048) = 0.82ksi

= 6.80 KSI

Concrete Creep: $K_{cr}(E_{ps}/E_c)(f_{cir}-f_{cds})$ = 2(28.5E3/4400)(.78 -.16) = 7.6 kips
$f_{cds}=M_{sd}e/I_g$ = (153.5*2.25)/2048 = 0.16 Kips

= 7.64 KSI

Shrinkage: (8.2E-6)$K_{sh}E_{ps}$(1-0.06V/S)(100-RH) = (8.2E-6)(1.0)(28.5E3)(1-0.06(4.36))(100-75) =

= 4.31 KSI

Steel Relaxation: R.E=(K_{re}-J(SH+CR+ES))C = (5000-0.040(6.80+7.64+4.31)1.441 =
K_{re}=5000 J=0.040 C= 1+9(0.749-.7) = 1.441 ***= 3.0 KSI***
TOTAL LOSSES AT MIDSPAN = 6.80+7.64+4.31+3.0 = 21.75 KSI → 10%

4. **Service Load Stresses:**

$M_{Sustained}$ = 222.8 in-kips
$M_{Service}$ = 153.5 in-kips
P=0.75$A_{ps}F_{pu}$ = (.75)(7)(41.3)(1-.10)= 195.1 Kips

CHECK:
F = P/A ±Pe/S ±M_{Ser}*e/S± OR ±M_{Sus}*e/S =
$F_{TOP/Service}$ = 0.77-0.55+1.0 ***= 1.22 KSI < F_{Ci} OK***
$F_{TOP/Sustainable}$= 0.77-0.55+2.4 ***= 2.62 KSI < F_{Ci} OK***
F_{BOT} = 0.77 + (0.55-2.4)(799/511) ***= -2.1 KSI > -F_t OK***

5. **Flexure Check:**

PCI 6th edition FIG 4.12.3
M_u= 104.2 kip-ft
$f_{se}\geq 0.5f_{pu}$ [140≥135]
Bonded YES

-$\emptyset M_n \geq M_u$
$C\omega_{pu}$=1.13(1.071*270,000)/(48*6*6000) + (8/6)(0) = 0.189
f_{ps}= 250ksi

$ØM_n= ØAsp*fps(d_p-a/2)$
$a=A_{sp}*f_{ps}/0.85f'_{cb}$ = 1.071*250/0.85*6*48 = 1.09
c=1.09/0.75 = 1.46
1.46/6 = .24 < 0.375 → Ø=0.9
$ØM_n$ =0.9*1.071*250*[6-1.46/2] = ***105.82 kip-ft > 104.2 kip-ft OK***

-$ØM_n > 1.2M_{cr}$
P: from part 4
e=2.25"
$1.2M_{cr}=1.2(P/A + Pe/S_b +7.5\sqrt{f'c}) * S_b$ =
1.2(.762+0.543+.581)*370 = ***83.7 kip-ft < 105.82 kip-ft OK***

6. **Shear Check**
PCI 6th edition: FIG 4.12.5
$ØV_n \geq V_u$
$x=50d_b$=50(.5)=25"
x/ℓ=25"/23*12=0.09
d= 6.981 bw=18"
Vu=4.6 Kips
Mu= 222.8 in-kips
Vn = Vc
Vc= (0.6√f'c + 700* Vud/Mu) bwd = [46.47+700(4.6*6.981/222)bwd
Vc=.147bwd
$ØVn=Ø2\sqrt{f'_c}b_wd$=14.6 Kips ***14.6 Kips ≥ 18.56 Kips OK***

7. **Deflections**
Hollow core design handbook Table 2.4.1

Deflection :
$\Delta_{TOPPING}$ = 5*.025*4*17⁴1728/384*4463*529 = 0.079"
Long Term = (0.079)(2.30) = 0.182"
$\Delta_{SUPERIMPOSED}$ = 5*.02*4*17⁴*1728/384*4463*2048 = .016"
Long Term = (.016)(3) = 0.05"
Live Deflection = (100/20)(.05) = 0.25"
**Muilt from Table 2.4.1 of Hollow core design paper **

Δ_{FINAL}= -.079 - .016 - .25 = **-0.336"**

Camber: $P_oel2/8EI – 5wl^4/384EI$
Initial Camber = [173.5*2.25*(17*12)²/8*3408*1519]-[5*.195*17⁴*1728/384*3408*1519]
0.39"-0.1" = 0.49"
Erection Camber =0.39(1.80)-0.1(1.85)= 0.517"
Final Camber = 0.39(2.45)-0.1(2.70) = 0.68"
Camber = **0.68"**

TOTAL = $\Delta_{CAMBER}-\Delta_{DEFLECTION}$ =0.68-0.336=**0.344"**
Limit = L/360 = 17*12/360 = 0.567" **0.344" < 0.567" OK**

2 *Residential Design Check*

Span =16.67
Length =18”
Slab Thickness: 6” plank + 2” topping = 8”
LOADING:
Topping W=25PSF
LL=20PSF
Super=20PSF
Span-Depth : 18/.667 = 26 < 40 OK

Plank by Nitterhouse: www.nitterhouse.com
-6”x4’ Hollowcore plank 2 hour rating w/ 2” topping
Ac= 253 in^2
Ic=1519 in^4
Tbc=4.10 in
Ytc=1.90in
D_p=6”
Wt=48.75 psf

1. Preliminary Design Loads :

W=1.2(20)+1.6(40)= 88 psf
6” Slab+ 2” Topping and 7-1/2”Ø strands

Capacity = 165 psf
M Capacity = 104.2 k-ft

2. Transfer Stresses: 7-1/2” Ø 270ksi low relaxation strands

A_{sp}= 7(0.153)= 1.071in^2
e=2.25”
L= 17”
Wu= 1.2(48.75+25) = 88.5 psf*4ft=354plf
M_d= (354)(18^2)/8=14.3 ft-kips → 171.6 in-kips
P_o=0.75$A_{ps}F_{pu}$ = (1.071)(270)(0.60) = 173.5 Kips
P_i= 0.153*270*.75*7=216.87

CHECK
F_c= P_o/A ± P_Oe/S ± M_D/S =
F_{TOP} =0.685-0.708+0.22 ***= 0.195 ≤ F_{ti} OK***
F_{BOT}=0.685+1.055-0.56 ***=1.10 ≤ $F_{c\ i}$ OK***

3. Prestress losses

Elastic Shortening: $K_{es}E_{ps}F_{cir}/E_{ci}$ = (1)(28.5E3)(0.91)/3400 = 7.57 KSI
F_{cir}= $K_{cir}(P_i/A_g + P_ie^2/I_g)-M_ge/I_g$ (Mg=48.75+25*4*18^2/8)= 171.6 in-kips
=0.9[(216.87/384)+(216.87*2.25^2)/2048)]-(171.6*2.25/2048) = 0.91

= 7.57 KSI

Concrete Creep: $K_{cr}(E_{ps}/E_c)(f_{cir}-f_{cds})$ = 2(28.5E3/4400)(.91-.24) = 8.6 kips
f_{cds}=$M_{sd}e/I_g$ = (218*2.25)/2048 = 0.24 Kips

=8.67 KSI

Shrinkage: $(8.2E\text{-}6)K_{sh}E_{ps}(1\text{-}0.06V/S)(100\text{-}RH) = (8.2E\text{-}6)(1.0)(28.5E5)(1\text{-}0.06(4.36))(100\text{-}75) =$

= 4.31 KSI

Steel Relaxation: $R.E=(K_{re}\text{-}J(SH+CR+ES))C = (5000\text{-}0.040(7570+8679+4310)1.441 =$
$K_{re}=5000$ $J=0.040$ $C= 1+9(0.749\text{-}.7) = 1.441$

= 3.815 KIS

TOTAL LOSSES AT MIDSPAN = 7.757+8.679+4.310+3.815= 24.5 KSI → 11.3%

4. Service Load Stresses:

$M_{Sustained} = 343$ in-kips
$M_{Service} = 218.7$ in-kips
$P=0.75A_{ps}F_{pu} = 216.87(1\text{-}.113)= 192.3$ Kips

CHECK:
$F = P/A \pm Pe/S \pm M_{Ser}*e/S\pm$ OR $\pm M_{Sus}*e/S =$
$F_{TOP/Service} = 0.760\text{-}0.54+0.61$ ***= 0.83 KS I < F_{Ci} OK***
$F_{TOP/Sustainable} = 0.760\text{-}0.54+0.96$ ***= 1.18 KSI < F_{Ci} OK***
$F_{BOT} = 0.760 + (0.54\text{-}0.96)(799/551)$ ***= 0.103 KSI > $-F_t$ OK***

5. Flexure Check:

PCI 6th edition FIG 4.12.3
$M_u= 104.2$ kip-ft
$f_{se}\geq 0.5f_{pu}$ [140≥135]
Bonded YES

$-\text{Ø}M_n \geq M_u$
$C\omega_{pu}=1.13(1.071*270{,}000)/(48*6*6000) + (8/6)(0) = 0.189$
$f_{ps}= 250$ksi
$\text{Ø}M_n= \text{Ø}Asp*fps(d_p\text{-}a/2)$
$a=A_{sp}*f_{ps}/0.85f'_{cb} = 1.071*250/0.75*6*48 = 1.23$
$c=1.23/0.75 = 1.64$
$1.64/6 = .27 < .0375$ → $\text{Ø}=0.9$
$\text{Ø}M_n =0.9*1.071*250*[6\text{-}1.23/2] =$ ***108.13 kip-ft > 104.2 kip-ft OK***

$-\text{Ø}M_n > 1.2M_{cr}$
P: from part 5
e=2.25"
$1.2M_{cr}=1.2(P/A + Pe/S_b +7.5\sqrt{f'c}) * S_b =$
$1.2(.762+0.543+.581)*370 =$ ***83.7 kip-ft < 108.13 kip-ft OK***

6. **Shear Check**
PCI 6th edition FIG 4.12.5

$ØV_n \geq V_u$
x=50d_b=50(.5)=25"
x/ℓ=25"/18*12=0.12
d= 6.981
bw=18"
Vu=6.35 Kips
Mu=28.5 Kip-ft=343 Kip-in
Vn = Vc
Vc= (0.6√f'c + 700* Vud/Mu) bwd = [46.47+700(6.35*6.981/343)bwd
Vc=136.9 bwd
ØVn=Ø2√$f'_c b_w$d= ***14.6 Kips ≥ 6.35 Kips OK***

7. **Composite Deflections :**
Hollow core design handbook Table 2.4.1
Deflection:
$\Delta_{TOPPING}$ = 5*.025*4*$18^4$1728/384*4463*529 = 0.100"
Long Term = (0.100)(2.30) = 0.23"
$\Delta_{SUPERIMPOSED}$ = 5*.02*4*18^4*1728/384*4463*2048 = .0206"
Long Term = (.0206)(3) = 0.0618"
Live Deflection = (100/20)(.0206) = 0.13"
Δ_{FINAL}= -.23- .0618 - .13 = **-0.422"**
**Muilt from Table 2.4.1 of Hollow core design paper **

Camber: [$P_o$el2/8EI – 5wl^4/384EI]
Initial Camber = [173.5*2.25*$(18*12)^2$/8*3408*1519]-[5*.195*$18^4$1728/384*3408*1519]
0.43"-0.0889" = 0.34"
Erection Camber =0.43(1.80)-0.0889(1.85)= 0.93"
Final Camber = 0.43(2.45)-0.0889(2.70) = 0.813"
Camber = **0.813"**

TOTAL = Δ_{CAMBER}-$\Delta_{DEFLECTION}$ = 0.813-0.422 = **1.391"**
Limit = L/360 = 18*12/360 = 0.6" **0.391" < 0.6" OK**

EXTERIOR BEAMS

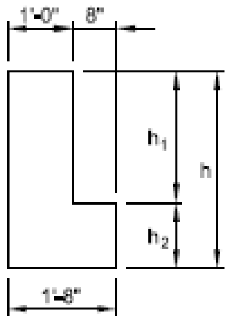
DESIGNATION	Span	Trib W (ft)	1.2* Dead (psf)	1.6*Live (psf)	Wu psf	Wu plf	TRAIL SIZE
LB-1	26.8	6.5	112.5	64	176.5	1147.3	20LB20
LB-2	26.8	6.5	112.5	64	176.5	1147.3	20LB20
LB-3	26.8	6.5	112.5	64	176.5	1147.3	20LB20
LB-4	26.8	6.5	112.5	64	176.5	1147.3	20LB20
LB-5	13.2	2	112.5	64	176.5	353.0	20LB20
LB-6	16	2	112.5	64	176.5	353.0	20LB20
LB-7	14.4	8.83	112.5	64	176.5	1558.5	20LB20
LB-8	14.4	8.83	112.5	64	176.5	1293.6	20LB20
LB-9	17.6	7.2	112.5	64	176.5	1270.8	20LB20
LB-10	23	7.2	112.5	64	176.5	1270.8	20LB20
LB-11	19.58	2	112.5	64	176.5	353.0	20LB20
LB-12	11.25	2	112.5	64	176.5	353.0	20LB20
LB-13	6.6	2	112.5	64	176.5	353.0	20LB20
LB-14	22.16	2	112.5	64	176.5	353.0	20LB20
LB-15	17	2	112.5	64	176.5	353.0	20LB20
LB-16	17	2	112.5	64	176.5	353.0	20LB20
LB-17	17	2	112.5	64	176.5	353.0	20LB20
LB-18	12.16	2	112.5	64	176.5	353.0	20LB20
LB-19	32	6	112.5	64	176.5	1059.0	20LB20
LB-20	32	6	112.5	64	176.5	1059.0	20LB20
LB-21	12.16	2	112.5	64	176.5	353.0	20LB20
LB-22	17	2	112.5	64	176.5	353.0	20LB20
LB-23	17	2	112.5	64	176.5	353.0	20LB20
LB-24	17	2	112.5	64	176.5	353.0	20LB20
LB-25	17	2	112.5	64	176.5	353.0	20LB20
LB-26	17	2	112.5	64	176.5	353.0	20LB20
LB-27	17	2	112.5	64	176.5	353.0	20LB20
LB-28	17	2	112.5	64	176.5	353.0	20LB20
LB-29	18	2	112.5	64	176.5	353.0	20LB20
LB-30	17	2	112.5	64	176.5	353.0	20LB20
LB-31	13.167	2	112.5	64	176.5	353.0	20LB20

DESIGN CHECKS

DESIGNATION	Load (plf)	Self (plf)	ØMn (kip-ft)	ØVn (kip)	Mu (kip-ft)	Vu(kip)	ØMn≥Mu	ØVn≥Vu
LB-1	2318	317	187.30	27.96	103.00	15.37	OK	OK
LB-2	2318	317	187.30	27.96	103.00	15.37	OK	OK
LB-3	2318	317	187.30	27.96	103.00	15.37	OK	OK
LB-4	2318	317	187.30	27.96	103.00	15.37	OK	OK
LB-5	6556	317	128.51	38.94	7.69	2.33	OK	OK
LB-6	6556	317	188.81	47.20	11.30	2.82	OK	OK
LB-7	6556	317	152.94	42.48	40.40	11.22	OK	OK
LB-8	6556	317	152.94	42.48	33.53	9.31	OK	OK
LB-9	5131	317	178.81	40.64	49.21	11.18	OK	OK
LB-10	2768	317	164.73	28.65	84.03	14.61	OK	OK
LB-11	4105	317	-	-	-	-	OK	OK
LB-12	6566	317	93.49	33.24	5.58	1.99	OK	OK
LB-13	6566	317	32.18	19.50	1.92	1.16	OK	OK
LB-14	3345	317	-	-	-	-	OK	OK
LB-15	5131	317	166.82	39.25	12.75	3.00	OK	OK
LB-16	5131	317	166.82	39.25	12.75	3.00	OK	OK
LB-17	5131	317	166.82	39.25	12.75	3.00	OK	OK
LB-18	6566	317	109.22	35.93	6.52	2.15	OK	OK
LB-19	2768	317	318.87	39.86	135.55	16.94	OK	OK
LB-20	2768	317	318.87	39.86	135.55	16.94	OK	OK
LB-21	6566	317	109.22	35.93	6.52	2.15	OK	OK
LB-22	5131	317	166.82	39.25	12.75	3.00	OK	OK
LB-23	5131	317	166.82	39.25	12.75	3.00	OK	OK
LB-24	5131	317	166.82	39.25	12.75	3.00	OK	OK
LB-25	5131	317	166.82	39.25	12.75	3.00	OK	OK
LB-26	5131	317	166.82	39.25	12.75	3.00	OK	OK
LB-27	5131	317	166.82	39.25	12.75	3.00	OK	OK
LB-28	5131	317	166.82	39.25	12.75	3.00	OK	OK
LB-29	5131	317	187.02	41.56	14.30	3.18	OK	OK
LB-30	5131	317	166.82	39.25	12.75	3.00	OK	OK
LB-31	6566	317	128.06	38.90	7.65	2.32	OK	OK

L-BEAMS

Normal Weight Concrete



$f'_c = 5{,}000$ psi
$f_{pu} = 270{,}000$ psi
½ in. diameter
low-relaxation strand

Designation	h in.	h_1/h_2 in./in.	A in.^{2}	I in.^{4}	y_b in.	S_b in.^{3}	S_t in.^{3}	wt plf
20LB20	20	12/8	304	10,160	8.74	1,163	902	317
20LB24	24	12/12	384	17,568	10.50	1,673	1,301	400
20LB28	28	16/12	432	27,883	12.22	2,282	1,767	450
20LB32	32	20/12	480	41,600	14.00	2,971	2,311	500
20LB36	36	24/12	528	59,119	15.82	3,737	2,930	550
20LB40	40	24/16	608	81,282	17.47	4,653	3,608	633
20LB44	44	28/16	656	108,107	19.27	5,610	4,372	683
20LB48	48	32/16	704	140,133	21.09	6,645	5,208	733
20LB52	52	36/16	752	177,752	22.94	7,749	6,117	783
20LB56	56	40/16	800	221,355	24.80	8,926	7,095	833
20LB60	60	44/16	848	271,332	26.68	10,170	8,143	883

1. Check local area for availability of other sizes.
2. Safe loads shown include 50% superimposed dead load and 50% live load. 800 psi top tension has been allowed, therefore, additional top reinforcement is required.
3. Safe loads can be significantly increased by use of structural composite topping.

Key

6566 – Safe superimposed service load, plf.
0.3 – Estimated camber at erection, in.
0.1 – Estimated long-time camber, in.

Table of safe superimposed service load (plf) and cambers (in.)

Desig-nation	No. Strand	y_s(end) in. / y_s(center) in.	Span, ft 16	18	20	22	24	26	28	30	32	34	36	38	40	42	44	46	48	50
20LB20	98-S	2.44	6566	5131	4105	3345	2768	2318	1961	1674	1438	1243	1079							
		2.44	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.0	1.1	1.2							
			0.1	0.2	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.2							
20LB24	108-S	2.80	9577	7495	6006	4904	4066	3414	2896	2479	2137	1854	1617	1416	1244	1097	969			
		2.80	0.3	0.3	0.4	0.5	0.5	0.6	0.7	0.8	0.9	0.9	1.0	1.0	1.1	1.1	1.2			
			0.1	0.1	0.1	0.1	0.1	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.1	0.0	0.0			
20LB28	128-S	3.33			8228	6733	5596	4711	4009	3443	2979	2595	2273	2000	1768	1567	1394	1243	1110	992
		3.33			0.4	0.4	0.5	0.6	0.6	0.7	0.8	0.9	0.9	1.0	1.1	1.1	1.2	1.2	1.2	1.3
					0.1	0.1	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.1	0.1	0.0	0.0
20LB32	148-S	3.71				8942	7446	6281	5356	4611	4001	3495	3071	2712	2406	2143	1914	1715	1540	1386
		3.71				0.4	0.5	0.5	0.6	0.7	0.7	0.8	0.9	1.0	1.0	1.1	1.2	1.2	1.3	1.3
						0.1	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.2	0.2	0.2	0.1
20LB36	168-S	4.25					9457	7988	6823	5883	5113	4476	3941	3489	3103	2771	2483	2231	2011	1816
		4.25					0.4	0.5	0.5	0.6	0.7	0.8	0.8	0.9	1.0	1.1	1.1	1.2	1.2	1.3
							0.2	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.2
20LB40	188-S	4.89						9812	8386	7235	6293	5513	4858	4305	3832	3425	3073	2765	2495	2257
		4.89						0.4	0.5	0.6	0.6	0.7	0.8	0.8	0.9	1.0	1.0	1.1	1.1	1.2
								0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3
20LB44	198-S	5.05								8959	7803	6845	6042	5363	4783	4284	3851	3474	3143	2850
		5.05								0.5	0.6	0.6	0.7	0.8	0.8	0.9	0.9	1.0	1.1	1.1
										0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2
20LB48	218-S	5.81									9226	8100	7158	6360	5678	5092	4584	4140	3751	3408
		5.81									0.5	0.6	0.6	0.7	0.8	0.8	0.9	0.9	1.0	1.1
											0.2	0.2	0.2	0.2	0.2	0.2	0.3	0.3	0.3	0.3
20LB52	238-S	6.17										9634	8521	7578	6774	6082	5482	4958	4499	4094
		6.17										0.6	0.6	0.7	0.7	0.8	0.9	0.9	1.0	1.0
												0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3
20LB56	258-S	6.64											9954	8860	7927	7124	6427	5820	5287	4816
		6.64											0.6	0.7	0.7	0.8	0.8	0.9	1.0	1.0
													0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3
20LB60	278-S	7.33													9089	8173	7380	6688	6080	5544
		7.33													0.7	0.7	0.8	0.9	0.9	1.0
															0.3	0.3	0.3	0.3	0.3	0.3

3

BEAM LB-11

20LB20 / 98-S: (9) ½"Ø low relaxation strands – straight
f'_c=5000psi f_{pu}=270ksi
$E_c=33(145)^{1.5}\sqrt{5000}$ =4074 ksi
F_{se}= 170ksi
f_{pu}=270ksi

Loading Capacity: 4105 PLF

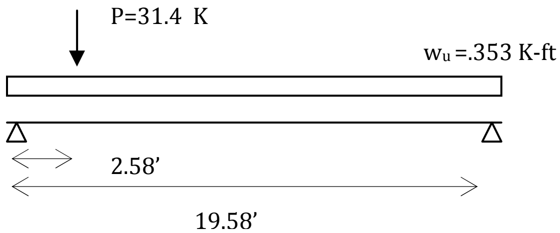
Moment Capacity: M_n = 196.75 Ft-Kips

Shear Capacity: V_n =40.2 Kips

<u>Beam Properties</u>
L=19.58 ft
A= 304 in^2
I = 10,160 in^4
Y_{bc} =8.74 in
S_t =902 in^3
S_b=1163 in3
h_1 = 12 h_2= 8 in
wt = 317 plf

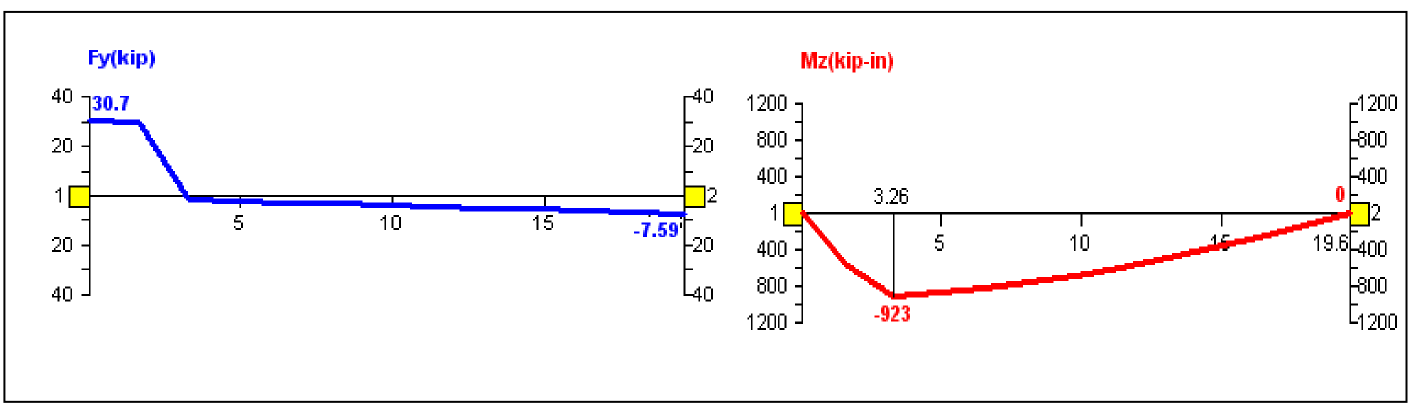
LOADING CONDITION:
Self 20LB20 = 317plf
Self TB18 = 500 plf
Wu = 166.4 PSF OR 2929.5 PLF
Span-Depth : 19.58/1.67 = 11.7 < 40 OK
P (TB18)=((2.829+.5)*18.83)/2=31.4 Kips
** Detailed loading conditions See spreadsheets**

1. Flexure and Shear



<u>Using Stadd-Pro:</u>
V_u: 30.7 Kips
M_u: 76.92 kip-ft
<u>$ØM_n ≥ Mu$ OK</u>

<u>$ØV_n ≥ V_u$ OK</u>



2.Transfer Stresses:

9-1/2" Ø 270ksi low relaxation strands

Service loads :	$F_{ti}=7.5\sqrt{f'_c}$ = 580psi
	F_{ci}= 0.6(f'_c) = 3600 psi
Allowable @ transfer:	$F_t=3\sqrt{f'_{ti}}$ =+180psi
	$F_c=0.6f'_{ci}$ =-2160psi

A_{sp}= 9(0.153)= 1.37 in^2
e=6.33" ct=11.26" cb=8.74"
L= 19.58"
USING STADD
M_D=658 kip-in
M_L= 264 kip-in
Po= 0.153*270*.75*9=278.8
I=8000 in^4

CHECK
$F_c= P_o/A \pm P_O eC/I \pm M_D C/I$ =
F_{TOP} =-0.92+2.48-0.92= ***= 0.64 KSI ≤ F_{ti} OK***
F_{BOT}=-0.92-1.93+0.71= ***= 2.14 KSI ≥- F_{ci} OK***

3. Pre-Stress Losses

** Pre-Stress loss assumption of 15%** [this is conservative full calculations were done for hollow core planks yielding losses of 10-11%]

Pe = (1-0.15)278.8=236.9 Kips

CHECK:
$F = P_e/A \pm P_e eC/I \pm M_D C/I \pm M_L C/I$
F_{TOP} =-0.78+1.66-0.92-0.26= ***= -0.30 KSI ≥ -F_{ti} OK***
F_{BOT}=-0.78-1.28+0.71+0.23 = ***= 1. 3 KSI ≤ F_{ci} OK***

4. Cracking Moment

Case #1: $M=P_e e+P_o I/Ac$: zero stress at bottom
M_{cr}=278.8 *6.33+236.98*10106/304*11.26= 483.8 ft-kips < 196.75 ft-kips OK

Case #2: M=Case#1+$f_r I/C$: cracking at bottom
M_{cr}=483.8+(530*10106/8.74)= 534.86 ft-kips <196.75 ft-kips OK

4 ***BEAM LB-14***

20LB20/ 98-S: (5) ½"Ø low relaxation strands – straight
f'_c=5000psi f_{pu}=270ksi
$E_c=33(145)^{1.5}\sqrt{5000}$ =4074 ksi
F_{se}= 170ksi
f_{pu}=270ksi

Loading Capacity: 3345 PLF

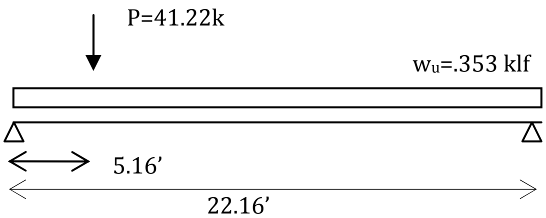
Moment Capacity: M_n = 205.3 Ft-Kips

Shear Capacity: V_n =37.0 Kips

<u>Beam Properties</u>
L=22.16 ft
A= 304 in^2
I = 10,160 in^4
Y_{bc} = 8.74 in
St =902 in^3
Sb=1163 in3
H1=12" H2=8"
wt = 317plf

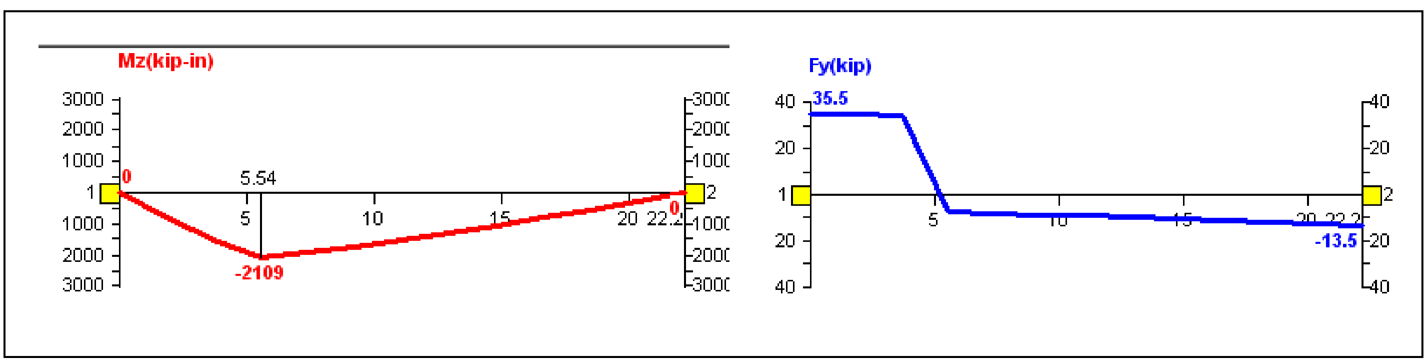
LOADING CONDITION:
LL=40 PSF
Super D=5 PSF
Self 12RB16 = 250plf
Self TB18 = 300plf
Wu = 176.5 PSF OR 353 PLF
Span-Depth : 22.16/1.33 = 16.66 < 40 OK
P (TB-24)=((2.733+.5)*25.5)/2=41.22 Kips
** Detailed loading conditions See spreadsheets**

1. Flexure and Shear



<u>Using Stadd-Pro:</u>
V_u:35.5 Kips
M_u: 175.75 Kip-Ft
<u>$ØM_n≥Mu$ OK</u>

<u>$ØV_n≥V_u$ OK</u>



2.Transfer Stresses:

9-1/2" Ø 270ksi low relaxation strands

Service loads : F_{ti}=7.5√f'$_c$ = 580psi

F_{ci}= 0.6(f'$_c$) = 3600 psi

Allowable @ transfer: F_t=3√f'$_{ci}$ =+180psi

F_c=0.6f'$_{ci}$ =-2160psi

A_{sp}= 9(0.153)= 1.377 in^2
e=6.33" ct=11.26" cb=8.74"
L= 22.16'
From Stadd
M_D= 1245 in-kips
M_L= 864 in-kips
Po= 0.153*270*.75*9=278.8
I=10160 in^4

CHECK

F_c= P_o/A ± P_OeC/I ± M_DC/I =

F_{TOP} =-0.91+1.95-1.37= ***=-0.33 KSI ≤ F_{ti} OK***

F_{BOT}=-0.91-1.52+1.07= ***= 1.36 KSI ≥- F_{ci} OK***

5. Pre-Stress Losses

** Pre-Stress loss assumption of 15%** [this is conservative full calculations were done for hollow core planks yielding losses of 10-11%]

Pe = (1-0.15)278.8=236.98 Kips

CHECK:

F = P_e/A ± P_eeC/I ± M_DC/I ± M_LC/I

F_{TOP} =-0.77+1.66-1.37-0.95 = -1.43 ***= -1.43 KSI ≥ F_c OK***

F_{BOT}=-0.77-1.28+1.07+0.743 = ***=-.237 KSI ≤- F_{ci} OK***

6. Cracking Moment

Case #1: M=P_ee+P_oI/Ac : zero stress at bottom

M_{cr}=278.8 *6.33+236.98*10106/304*11.26= 483.8 ft-kips > 205.3 ft-kips OK

Case #2: M=Case#1+f_rI/C : cracking at bottom

M_{cr}=483.8+(530*10106/8.74)= 534.86 ft-kips > 205.3 ft-kips OK

INTERIOR BEAMS

DESIGNATION	Span	Trib W (ft)	1.2* Dead (psf)	1.6*Live (psf)	KLL	AT	Lr	Wu psf	Wu plf	TRAIL SIZE
TB-1	26.8	15.1	112.5	64	2	404.68	64	176.50	2665.15	28IT24
TB-2	26.8	15.1	112.5	64	2	404.68	64	176.50	2665.15	28IT24
TB-3	26.8	15.1	112.5	64	2	404.68	64	176.50	2665.15	28IT24
TB-4	26.8	15.1	112.5	64	2	404.68	64	176.50	2665.15	28IT24
TB-5	26.8	17.5	112.5	64	2	469.00	47	159.85	2797.29	28IT24
TB-6	26.8	-	-	-	-	-	-	-	-	28IT24
TB-7	18.125	10.5	112.5	*	*	*	*	*	3855.00	28IT20
TB-8	18.125	10.5	112.5	*	*	*	*	*	3855.00	28IT20
TB-10	26.8	17.5	112.5	64	2	469.00	47	159.85	2797.29	28IT24
TB-11	12.3	17.5	112.5	160	2	215.25	156	268.17	4692.99	28IT20
TB-12	18.125	17.5	112.5	64	2	317.19	54	166.62	2915.77	28IT20
TB-13	18.128	17.5	112.5	64	2	317.24	54	166.61	2915.71	28IT20
TB-14	17.66	12	112.5	64	2	211.92	63	176.50	2118.00	28IT20
TB-15	26	17	112.5	64	2	442.00	48	160.79	2733.40	28IT20
TB-16	22	•	112.5	•	•	•	•	•	•	28IT24
TB-17	18.83	17	112.5	64	2	320.11	54	166.44	2829.49	28IT20
TB-18	26	17	112.5	64	2	442.00	48	160.79	2733.40	28IT24
TB-19	22	•	112.5	•	•	•	•	•	•	28IT24
TB-20	18.83	15.6	112.5	64	2	293.75	56	168.11	2319.49	28IT20
TB-21	26	17	112.5	64	2	442.00	48	160.79	2733.40	28IT24
TB-22	22	•	•	•	•	•	•	•	•	28IT24
TB-23	25.5	17	112.5	64	2	433.50	49	161.10	2738.76	28IT24
TB-24	26	17	112.5	64	2	442.00	48	160.79	2733.40	28IT24
TB-25	22	•	•	•	•	•	•	•	•	28IT24
TB-26	25.9	20.25	112.5	64	2	524.48	46	158.14	3202.36	28IT24
TB-27	26	17	112.5	64	2	442.00	48	160.79	2733.40	28IT24
TB-28	22	•	•	•	•	•	•	•	•	28IT24
TB-29	25.9	17	112.5	64	2	440.30	48	160.85	2734.46	28IT24
TB-30	26	17	112.5	64	2	442.00	48	160.79	2733.40	28IT20
TB-31	22	•	•	•	•	•	•	•	•	28IT24
TB-32	25.9	17	112.5	64	2	440.30	48	160.85	2734.46	28IT24
TB-33	32	14.58	112.5	64	2	466.56	47	159.93	2331.74	28IT28
TB-34	32	14.58	112.5	64	2	466.56	47	159.93	2331.74	28IT28
RB-1	17.66	8.5	112.5	64	2	150.11	71	183.91	1563.20	12RB16
RB-2	14.67	8.5	112.5	160	2	124.70	160	272.50	2316.25	12RB16
RB-3	17.66	2.00	112.5	64	2	35.32	130	242.72	485.44	12RB16
RB-4	14.00	2.00	112.5	64	2	28.00	144	256.79	513.57	12RB16
RB-5	14.00	2.00	112.5	64	2	28.00	144	256.79	513.57	12RB16
RB-6	10.17	8.50	112.5	160	2	86.42	160	272.50	2316.25	12RB16
RB-7	10.67	*	*	*	*	*	*	*	*	12RB16
RB-8	10.67	2.00	112.5	160	2	21.34	160	272.50	545.00	12RB16
RB-9	10.67	*	*	*	*	*	*	*	*	12RB16

* : Wu found by hand due to loading combinations

• : These beams are the same indepts analysis can be found in appendix 2

: Mu and Vu were found in STADD due to loading conditions

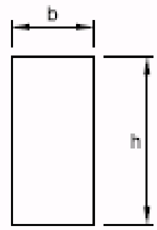
BLUE: Beams checked for structural integrity

Green: Beam which additional reinforcing for column and hollow core bearing is designed for in Appendix #3

DESIGN CHECKS							
DESIGNATION	Load (plf)	Ø Mn (kip-ft)	Ø Vn (kip)	Mu (kip-ft)	Vu(kip)	ØMn≥Mu	ØVn≥Vu
TB-1	3374	272.63	40.69	239.28	35.71	OK	OK
TB-2	3374	272.63	40.69	239.28	35.71	OK	OK
TB-3	3374	272.63	40.69	239.28	35.71	OK	OK
TB-4	3374	272.63	40.69	239.28	35.71	OK	OK
TB-5	3374	272.63	40.69	251.14	37.48	OK	OK
TB-6	3374	272.63	40.69	261.90	42.30	OK	OK
TB-7	5078	187.67	41.42	158.30	34.94	OK	OK
TB-8	5078	187.67	41.42	158.30	34.94	OK	OK
TB-10	3374	272.63	40.69	251.14	37.48	OK	OK
TB-11	6511	110.82	36.04	88.75	28.86	OK	OK
TB-12	5076	187.60	41.40	205.00	30.70	OK	OK
TB-13	5076	187.66	41.41	119.77	26.43	OK	OK
TB-14	5076	178.10	40.34	82.57	18.70	OK	OK
TB-15	3374	256.59	39.48	230.97	35.53	OK	OK
TB-16	4882	265.82	48.33	199.80	39.50	OK	OK
TB-17	5078	202.56	43.03	125.41	26.64	OK	OK
TB-18	3374	256.59	39.48	230.97	35.53	OK	OK
TB-19	4882	265.82	48.33	199.80	39.50	OK	OK
TB-20	5076	202.48	43.01	102.80	21.84	OK	OK
TB-21	3374	256.59	39.48	230.97	35.53	OK	OK
TB-22	4882	265.82	48.33	199.80	39.50	OK	OK
TB-23	3374	246.82	38.72	222.61	34.92	OK	OK
TB-24	3374	256.59	39.48	230.97	35.53	OK	OK
TB-25	4882	265.82	48.33	199.80	39.50	OK	OK
TB-26	3374	254.62	39.32	268.52	41.47	OK	OK
TB-27	3374	256.59	39.48	230.97	35.53	OK	OK
TB-28	4882	265.82	48.33	199.80	39.50	OK	OK
TB-29	3374	254.62	39.32	229.29	35.41	OK	OK
TB-30	3374	256.59	39.48	230.97	35.53	OK	OK
TB-31	4882	253.62	50.72	199.80	39.50	OK	OK
TB-32	3374	254.62	39.32	229.29	35.41	OK	OK
TB-33	2976	342.84	42.85	298.46	37.31	OK	OK
TB-34	2976	342.84	42.85	298.46	37.31	OK	OK
RB-1	2772	97.26	22.03	60.94	13.80	OK	OK
RB-2	2772	67.11	18.30	62.31	16.99	OK	OK
RB-3	2772	97.26	22.03	18.92	4.29	OK	OK
RB-4	2772	61.12	17.46	12.58	3.59	OK	OK
RB-5	2772	61.12	17.46	12.58	3.59	OK	OK
RB-6	3553	41.32	16.26	29.93	11.77	OK	OK
RB-7	3553	45.51	17.06	*	*	OK	OK
RB-8	3553	45.51	17.06	7.76	2.91	OK	OK
RB-9	3553	45.51	17.06	*	*	OK	OK

RECTANGULAR BEAMS

Normal Weight Concrete



$f'_c = 5{,}000$ psi
$f_{pu} = 270{,}000$ psi
½ in. diameter
low-relaxation strand

Section Properties							
Designation	**b in.**	**h in.**	**A $in.^2$**	**I $in.^4$**	**y_b in.**	**S $in.^3$**	**wt plf**
12RB16	12	16	192	4,096	8.00	512	200
12RB20	12	20	240	8,000	10.00	800	250
12RB24	12	24	288	13,824	12.00	1152	300
12RB28	12	28	336	21,952	14.00	1568	350
12RB32	12	32	384	32,768	16.00	2048	400
12RB36	12	36	432	46,656	18.00	2592	450
16RB24	16	24	384	18,432	12.00	1536	400
16RB28	16	28	448	29,269	14.00	2091	467
16RB32	16	32	512	43,691	16.00	2731	533
16RB36	16	36	576	62,208	18.00	3456	600
16RB40	16	40	640	85,333	20.00	4267	667

1. Check local area for availability of other sizes.
2. Safe loads shown include 50% superimposed dead load and 50% live load. 800 psi top tension has been allowed, therefore, additional top reinforcement is required.
3. Safe loads can be significantly increased by use of structural composite topping.

Key

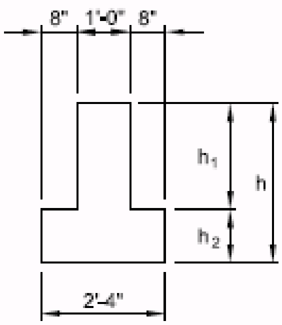
3553 – Safe superimposed service load, plf.
0.4 – Estimated camber at erection, in.
0.2 – Estimated long-time camber, in.

Table of safe superimposed service load (plf) and cambers (in.)

Desig-nation	No. Strand	y_e(end) in. y_e(center) in.	Span, ft																	
			16	18	20	22	24	26	28	30	32	34	36	40	42	44	46	48	50	52
12RB16	58-S	3.00	3553	2772	2212	1799	1484	1239	1045											
		3.00	0.4	0.5	0.6	0.8	0.9	1.0	1.1											
			0.2	0.2	0.2	0.2	0.3	0.3	0.3											
12RB20	88-S	3.00	6103	4825	3867	3159	2626	2201	1868	1600	1380	1198	1046							
		3.00	0.4	0.5	0.6	0.7	0.9	1.0	1.1	1.3	1.4	1.5	1.7							
			0.2	0.2	0.3	0.3	0.4	0.4	0.4	0.5	0.5	0.5	0.5							
12RB24	108-S	3.60	8950	7018	5636	4613	3835	3230	2749	2362	2045	1782	1562	1375	1216	1079	960			
		3.60	0.4	0.4	0.5	0.7	0.8	0.9	1.0	1.1	1.3	1.4	1.5	1.6	1.8	1.9	2.0			
			0.2	0.2	0.3	0.3	0.3	0.4	0.4	0.5	0.5	0.6	0.6	0.6	0.6	0.7	0.6			
12RB28	128-S	4.00		9781	7866	6448	5370	4532	3866	3329	2890	2525	2220	1962	1741	1552	1387	1244	1118	1006
		4.00		0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.3	1.4	1.5	1.7	1.8	1.9	2.0	2.1	2.2
				0.2	0.2	0.3	0.3	0.4	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.7	0.8	0.8	0.8	0.8
12RB32	138-S	4.77				8320	6936	5859	5005	4316	3752	3284	2892	2561	2278	2034	1823	1639	1477	1334
		4.77				0.5	0.6	0.7	0.8	0.9	1.0	1.1	1.2	1.3	1.4	1.5	1.6	1.7	1.8	1.9
						0.2	0.3	0.3	0.3	0.4	0.4	0.4	0.5	0.5	0.5	0.5	0.6	0.6	0.6	0.6
12RB36	158-S	5.07					9015	7624	6521	5631	4902	4298	3792	3364	2999	2684	2411	2173	1964	1780
		5.07					0.5	0.6	0.7	0.8	0.9	1.0	1.1	1.2	1.3	1.4	1.5	1.6	1.7	1.8
							0.2	0.3	0.3	0.4	0.4	0.4	0.5	0.5	0.6	0.6	0.6	0.6	0.7	0.7
16RB24	138-S	3.54		9397	7547	6177	5136	4325	3682	3164	2739	2387	2092	1843	1629	1446	1287	1149	1027	
		3.54		0.4	0.5	0.6	0.8	0.9	1.0	1.1	1.2	1.4	1.5	1.6	1.7	1.8	1.9	2.0	2.1	
				0.2	0.2	0.3	0.3	0.4	0.4	0.5	0.5	0.5	0.5	0.6	0.6	0.6	0.6	0.6	0.5	
16RB28	148-S	3.71				8730	7272	6137	5237	4510	3915	3423	3010	2660	2362	2105	1883	1688	1518	1368
		3.71				0.5	0.6	0.7	0.8	0.9	1.1	1.2	1.3	1.4	1.5	1.6	1.7	1.8	1.9	1.9
						0.2	0.2	0.3	0.3	0.3	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.3
16RB32	188-S	4.67					9340	7891	6741	5813	5054	4425	3897	3451	3070	2742	2458	2210	1992	1800
		4.67					0.6	0.7	0.8	0.9	1.0	1.1	1.2	1.3	1.5	1.6	1.7	1.8	1.9	2.0
							0.3	0.3	0.4	0.4	0.5	0.5	0.5	0.6	0.6	0.6	0.7	0.7	0.7	0.7
16RB36	208-S	5.40						9946	5805	7343	6391	5603	4942	4383	3905	3494	3138	2827	2555	2314
		5.40						0.6	0.7	0.8	0.9	1.0	1.1	1.2	1.3	1.4	1.5	1.6	1.7	1.8
								0.3	0.3	0.4	0.4	0.4	0.5	0.5	0.5	0.6	0.6	0.6	0.6	0.6
16RB40	228-S	6.00								9122	7949	6976	6160	5470	4881	4374	3935	3552	3215	2918
		6.00								0.7	0.8	0.9	1.0	1.1	1.2	1.3	1.4	1.5	1.6	1.7
										0.3	0.4	0.4	0.4	0.5	0.5	0.5	0.5	0.6	0.6	0.6

INVERTED TEE BEAMS

Normal Weight Concrete



f'_c = 5,000 psi
f_{pu} = 270,000 psi
½ in. diameter
low-relaxation strand

Section Properties								
Designation	h in.	h_1/h_2 in./in.	A in.^{2}	I in.^{4}	y_b in.	S_b in.^{3}	S_t in.^{3}	wt plf
28IT20	20	12/8	368	11,688	7.91	1,478	967	383
28IT24	24	12/12	480	20,275	9.60	2,112	1,408	500
28IT28	28	16/12	528	32,076	11.09	2,892	1,897	550
28IT32	32	20/12	576	47,872	12.67	3,778	2,477	600
28IT36	36	24/12	624	68,101	14.31	4,759	3,140	650
28IT40	40	24/16	736	93,503	15.83	5,907	3,869	767
28IT44	44	28/16	784	124,437	17.43	7,139	4,683	817
28IT48	48	32/16	832	161,424	19.08	8,460	5,582	867
28IT52	52	36/16	880	204,884	20.76	9,869	6,558	917
28IT56	56	40/16	928	255,229	22.48	11,354	7,614	967
28IT60	60	44/16	976	312,866	24.23	12,912	8,747	1,017

1. Check local area for availability of other sizes.
2. Safe loads shown include 50% superimposed dead load and 50% live load. 800 psi top tension has been allowed, therefore, additional top reinforcement is required.
3. Safe loads can be significantly increased by use of structural composite topping.

Key

6511 – Safe superimposed service load, plf.
0.2 – Estimated camber at erection, in.
0.1 – Estimated long-time camber, in.

Table of safe superimposed service load (plf) and cambers (in.)

Desig-nation	No. Strand	y_s(end) in. y_s(center) in.	Span, ft																	
			16	18	20	22	24	26	28	30	32	34	36	38	40	42	44	46	48	50
28IT20	98-S	2.44	6511	5878	4848	3288	2711	2282	1935	1617	1361	1160	1022							
		2.44	0.2	0.3	0.4	0.4	0.5	0.5	0.6	0.7	0.7	0.7	0.8							
			0.1	0.1	0.1	0.1	0.1	0.1	0.0	0.0	0.0	0.0	–0.1							
28IT24	188-S	2.73	9612	7504	5997	4882	4034	3374	2850	2427	2081	1795	1555	1351	1178	1029				
		2.73	0.2	0.3	0.3	0.4	0.4	0.5	0.6	0.6	0.7	0.7	0.7	0.8	0.8	0.8				
			0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.0	0.0	–0.1	–0.2				
28IT28	138-S	3.08			8353	6822	5657	4750	4051	3451	2970	2582	2252	1973	1735	1530	1352	1197	1061	
		3.08			0.3	0.3	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.8	0.8	0.9	0.8	0.8	
					0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.0	0.0	–0.1	–0.2	–0.2	
28IT32	158-S	3.47				9049	7521	5333	5389	4628	4006	3490	3057	2691	2379	2110	1876	1673	1495	1337
		3.47				0.3	0.4	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.8	0.9	0.9	0.9	0.9
						0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.0	0.0	0.0	–0.1
28IT36	168-S	3.50					9832	8295	7075	6092	5287	4619	4060	3587	3183	2835	2534	2271	2040	1836
		3.50					0.3	0.4	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.8	0.9	0.9	0.9
							0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.0	0.0	0.0	–0.1
28IT40	198-S	4.21							8638	7440	6460	5647	4966	4390	3898	3474	3107	2787	2506	2258
		4.21							0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.8	0.8	0.9	0.9
									0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1
28IT44	208-S	4.40								9186	7989	6997	6165	5462	4861	4344	3896	3505	3162	2859
		4.40								0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.7	0.8	0.8	0.8
										0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.0
28IT48	228-S	4.55									9719	8525	7523	6676	5953	5330	4791	4320	3907	3542
		4.55									0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.8	0.9
											0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1
28IT52	248-S	5.17										9987	8823	7838	6998	6274	5647	4100	4619	4196
		5.17										0.5	0.5	0.6	0.6	0.6	0.7	0.7	0.8	0.8
												0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1
28IT56	268-S	5.23												9307	8319	7469	6731	6088	5524	5026
		5.23												0.5	0.6	0.6	0.7	0.7	0.8	0.8
														0.2	0.2	0.2	0.2	0.2	0.2	0.2
28IT60	288-S	5.57													9645	8668	7820	7081	6432	5859
		5.57													0.6	0.6	0.7	0.7	0.8	0.8
															0.2	0.2	0.2	0.2	0.2	0.2

BEAM TB-6

Beam TB-6
28IT24/ 188-S: (18) ½"Ø low relaxation strands – straight
f'_c=5000psi f_{pu}=270ksi
E_c=33(145)$^{1.5}$√5000 =4074 ksi
F_{se}= 170ksi
f_{pu}=270ksi

Loading Capacity: 3374 PLF

Moment Capacity: M_n = 299 Ft-Kips

Shear Capacity: V_n = 45 Kips

Beam Properties
A=480in^2
L=26'-8"
I =20,275 in^4
c = in
h1=12" h2=12"
St = 1408 in^3
Sb=2112
Wt=500 plf

LOADING CONDITION:
Self 28IT24 = 500plf
Wu = differs
Span-Depth : 26.67/2 = 13.35 < 40 OK
** Detailed loading conditions See spreadsheets**

1.Flexure and Shear

Mu= 261.9 ft-Kips < .9M_n OK
Vu= 42.3 Kips < .9V_n OK

2.Transfer Stresses:
10-1/2" Ø 270ksi low relaxation strands

Service loads : F_{ti}=7.5√f'_c = 580psi
F_{ci}= 0.6(f'_c) = 3600 psi
Allowable @ transfer: F_t=3√f'_{ci} =+180psi
F_c=0.6f'_{ci} =-2160psi

A_{sp}= 18(0.153)= 2.75 in^2
e=6.87" ct=14.4" cb=9.6"
L= 26.8"
From Stadd
M_D= 1537.2 ft-kips
M_L= 1605 ft-kips
Po= 0.153*270*.75*18=557.68K

CHECK
F_c= P_o/A ± P_OeC/I ± M_DC/I =
F_{TOP} =-1.16+2.72-0.09= ***= 1.47 KSI ≤ F_{ti} OK***
F_{BOT}=-1.16-1.82+0.06= ***=-2.92 KSI ≥- F_{ci} OK***

3.Pre-Stress Losses

** Pre-Stress loss assumption of 15%** [this is conservative full calculations were done for hollow core planks yielding losses of 10-11%]

Pe = (1-0.15)557.68 = 474 Kips

CHECK:

$F = P_e/A \pm P_e eC/I \pm M_D C/I \pm M_L C/I$
F_{TOP} =-0.98+2.31-1.09-1.13= ***= -0.89 KSI ≥ F_c OK***
F_{BOT}=-0.98-1.54+0.72+0.75 = ***= -1.05 KSI ≤ F_{ci} OK***

4.Cracking Moment

Case #1: $M=P_e e+P_o I/Ac$: zero stress at bottom
M_{cr}=474*6.87+557*20275/480*14.4= 434.7 kip-Ft

Case #2: M=Case#1+$f_r I/C$: cracking at bottom
Mcr=434.7+(530*20275/9.6)=527.9 kip-ft

6 **Beam RB-22**

Beam RB-23
28IT24/ 188-S: (10) ½"Ø low relaxation strands – straight
f'_c=5000psi f_{pu}=270ksi
$E_c=33(145)^{1.5}\sqrt{5000}$ =4074 ksi
F_{se}= 170ksi
f_{pu}=270ksi

Loading Capacity: 7882 PLF

Moment Capacity: M_n = 269.4 Ft-Kips

Shear Capacity: V_n = 38.5 Kips

Beam Properties
$A=480in^2$
L=26'-8"
$I=20,275\ in^4$
c = in
h1=12" h2=12"
$St = 1408\ in^3$
Sb=2112
Wt=500 plf

LOADING CONDITION:
Self 28IT24 = 500plf
Wu = differs
Span-Depth : 22/2 = 12 < 40 OK
** Detailed loading conditions See spreadsheets**

1.Flexure and Shear

Mu=199.8 ft-Kips < $.9M_n$ OK
Vu= 39.7 Kips < $.9V_n$ OK

2.Transfer Stresses:
18-1/2" Ø 270ksi low relaxation strands

Service loads : $F_{ti}=7.5\sqrt{f'_c}$ = 580psi
$F_{ci}= 0.6(f'_c)$ = 3600 psi
Allowable @ transfer: $F_t=3\sqrt{f'_{ci}}$ =+180psi
$F_c=0.6f'_{ci}$ =-2160psi

A_{sp}= 18(0.153)= 2.75 in^2
e=6.87" ct=14.4" cb=9.6"
L= 22"
From Stadd
M_D=1388 ft-kips
M_L=1008 ft-kips
Po= 0.153*270*.75*18=557.6K

CHECK
$F_c= P_o/A \pm P_O eC/I \pm M_D C/I$ =
F_{TOP} =-1.16+2.72-0.082= ***= 1.47 KSI ≤ F_{ti} OK***

F_{BOT}=-1.16-1.82+0.054=- ***=-3.03 KSI ≥- F_{ci} OK***

3.Pre-Stress Losses

** Pre-Stress loss assumption of 15%** [this is conservative full calculations were done for hollow core planks yielding losses of 10-11%]

Pe = (1-0.15)557.6=473.966 Kips

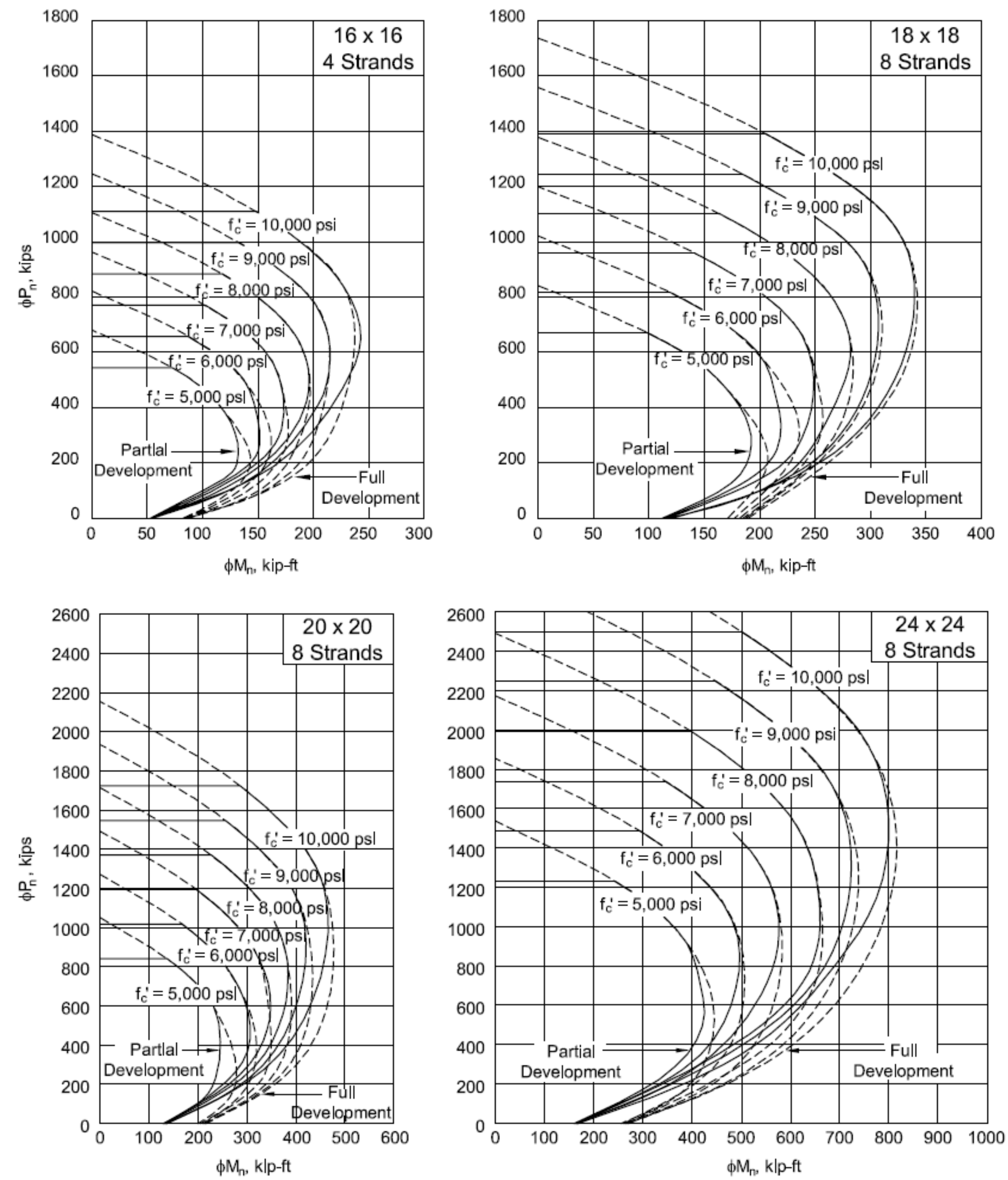
CHECK:
$F = P_e/A \pm P_eeC/I \pm M_DC/I \pm M_LC/I$
F_{TOP} =-0.98+2.31-0.98-0.71= ***= -0.36 KSI ≥ F_c OK***
F_{BOT}=-0.98-1.54+0.65+0.47 = ***= -1.4 KSI ≤ F_{ci} OK***

4.Cracking Moment

Case #1: $M=P_ee+P_oI/Ac$: zero stress at bottom
M_{cr}=474*6.87+557*20275/480*14.4= 434.7 kip-Ft

Case #2: $M=Case\#1+f_rI/C$: cracking at bottom
Mcr=434.7+(530*20275/9.6)=527.9 kip-ft

Column Sizing Charts from PCI 6th edition



<table>
<tr><th colspan="9">Column Sizing [Fig 2.7.1 PCI handbook 6th edition]</th></tr>
<tr><th colspan="3">Per Floor</th><th colspan="3">Level 1-4</th><th colspan="3">Level 5-9</th></tr>
<tr><th>Designation</th><th>Pu(kips)</th><th>Mu(k-ft)</th><th>Φ Pn (kips)</th><th>Φ Mn (k-ft)</th><th>TRIAL</th><th>Φ Pn (kips)</th><th>Φ Mn (k-ft)</th><th>TRIAL</th></tr>
<tr><td>C1</td><td>21.7</td><td>-17.51</td><td>217.07</td><td>-175.12</td><td>16x16 | f'c=5000psi | 4-#8</td><td>130.24</td><td>-105.07</td><td>16x16 | f'c=5000psi | 4-#8</td></tr>
<tr><td>C2</td><td>39.2</td><td>0.00</td><td>392.43</td><td>0.00</td><td>16x16 | f'c=5000psi | 4-#8</td><td>235.46</td><td>0.00</td><td>16x16 | f'c=5000psi | 4-#8</td></tr>
<tr><td>C3</td><td>39.2</td><td>0.00</td><td>392.43</td><td>0.00</td><td>16x16 | f'c=5000psi | 4-#8</td><td>235.46</td><td>0.00</td><td>16x16 | f'c=5000psi | 4-#8</td></tr>
<tr><td>C4</td><td>39.2</td><td>0.00</td><td>392.43</td><td>0.00</td><td>16x16 | f'c=5000psi | 4-#8</td><td>235.46</td><td>0.00</td><td>16x16 | f'c=5000psi | 4-#8</td></tr>
<tr><td>C5</td><td>21.7</td><td>-43.67</td><td>217.07</td><td>-436.74</td><td>24x24 | f'c=5000psi | 4-#11</td><td>130.24</td><td>-262.04</td><td>18x18 | f'c=5000psi | 4-#11</td></tr>
<tr><td>C6</td><td>47.2</td><td>19.35</td><td>471.91</td><td>193.47</td><td>16x16 | f'c=5000psi | 4-#8</td><td>283.15</td><td>116.08</td><td>16x16 | f'c=5000psi | 4-#8</td></tr>
<tr><td>C7</td><td>84.8</td><td>0.00</td><td>848.22</td><td>0.00</td><td>18x18 | f'c=5000psi | 4-#9</td><td>508.93</td><td>0.00</td><td>16x16 | f'c=5000psi | 4-#8</td></tr>
<tr><td>C8</td><td>84.8</td><td>0.00</td><td>848.22</td><td>0.00</td><td>18x18 | f'c=5000psi | 4-#9</td><td>508.93</td><td>0.00</td><td>16x16 | f'c=5000psi | 4-#8</td></tr>
<tr><td>C9</td><td>84.8</td><td>0.00</td><td>848.22</td><td>0.00</td><td>18x18 | f'c=5000psi | 4-#9</td><td>508.93</td><td>0.00</td><td>16x16 | f'c=5000psi | 4-#8</td></tr>
<tr><td>C10</td><td>47.3</td><td>-21.79</td><td>472.77</td><td>-217.86</td><td>20x20| f'c=5000psi | 4-#9</td><td>283.66</td><td>-130.72</td><td>16x16 | f'c=5000psi | 4-#8</td></tr>
<tr><td>C11</td><td>49.8</td><td>36.93</td><td>498.48</td><td>369.30</td><td>24x24 |f'c=5000psi | 4-#11</td><td>299.09</td><td>221.58</td><td>18x18 | f'c=5000psi | 4-#11</td></tr>
<tr><td>C12</td><td>97.0</td><td>5.57</td><td>969.62</td><td>55.74</td><td>20x20| f'c=5000psi | 4-#9</td><td>581.77</td><td>33.45</td><td>16x16 | f'c=5000psi | 4-#8</td></tr>
<tr><td>C13</td><td>80.3</td><td>0.00</td><td>803.41</td><td>0.00</td><td>18x18 | f'c=5000psi | 4-#9</td><td>482.05</td><td>0.00</td><td>16x16 | f'c=5000psi | 4-#8</td></tr>
<tr><td>C14</td><td>79.5</td><td>0.00</td><td>794.79</td><td>0.00</td><td>18x18 | f'c=5000psi | 4-#9</td><td>476.87</td><td>0.00</td><td>16x16 | f'c=5000psi | 4-#8</td></tr>
<tr><td>C15</td><td>55.6</td><td>-15.32</td><td>555.99</td><td>-153.20</td><td>18x18 | f'c=5000psi | 4-#9</td><td>333.60</td><td>-91.92</td><td>16x16 | f'c=5000psi | 4-#8</td></tr>
<tr><td>C16</td><td>18.3</td><td>-5.58</td><td>182.55</td><td>-55.81</td><td>16x16 | f'c=5000psi | 4-#8</td><td>109.53</td><td>-33.48</td><td>16x16 | f'c=5000psi | 4-#8</td></tr>
<tr><td>C17</td><td>50.4</td><td>37.29</td><td>503.87</td><td>372.89</td><td>24x24 |f'c=5000psi | 4-#11</td><td>302.32</td><td>223.74</td><td>18x18 | f'c=5000psi | 4-#11</td></tr>
<tr><td>C18</td><td>78.6</td><td>-7.37</td><td>786.26</td><td>-73.68</td><td>18x18 | f'c=5000psi | 4-#9</td><td>471.76</td><td>-44.21</td><td>16x16 | f'c=5000psi | 4-#8</td></tr>
<tr><td>C19</td><td>62.3</td><td>0.00</td><td>623.24</td><td>0.00</td><td>16x16 | f'c=5000psi | 4-#8</td><td>373.94</td><td>0.00</td><td>16x16 | f'c=5000psi | 4-#8</td></tr>
<tr><td>C20</td><td>6.8</td><td>5.10</td><td>67.57</td><td>51.04</td><td>16x16 | f'c=5000psi | 4-#8</td><td>40.54</td><td>30.63</td><td>16x16 | f'c=5000psi | 4-#8</td></tr>
<tr><td>C21</td><td>6.2</td><td>-0.10</td><td>61.56</td><td>-1.05</td><td>16x16 | f'c=5000psi | 4-#8</td><td>36.94</td><td>-0.63</td><td>16x16 | f'c=5000psi | 4-#8</td></tr>
<tr><td>C22</td><td>54.1</td><td>-5.50</td><td>540.66</td><td>-55.01</td><td>18x18 | f'c=5000psi | 4-#9</td><td>324.40</td><td>-33.00</td><td>16x16 | f'c=5000psi | 4-#8</td></tr>
<tr><td>C23</td><td>26.7</td><td>-10.16</td><td>267.39</td><td>-101.63</td><td>16x16 | f'c=5000psi | 4-#8</td><td>160.43</td><td>-60.98</td><td>16x16 | f'c=5000psi | 4-#8</td></tr>
<tr><td>C24</td><td>56.4</td><td>-11.33</td><td>564.03</td><td>-113.30</td><td>18x18 | f'c=5000psi | 4-#9</td><td>338.42</td><td>-67.98</td><td>16x16 | f'c=5000psi | 4-#8</td></tr>
<tr><td>C25</td><td>36.0</td><td>-3.15</td><td>360.13</td><td>-31.46</td><td>16x16 | f'c=5000psi | 4-#8</td><td>216.08</td><td>-18.88</td><td>16x16 | f'c=5000psi | 4-#8</td></tr>
<tr><td>C26</td><td>18.1</td><td>-7.19</td><td>180.78</td><td>-71.89</td><td>16x16 | f'c=5000psi | 4-#8</td><td>108.47</td><td>-43.14</td><td>16x16 | f'c=5000psi | 4-#8</td></tr>
<tr><td>C27</td><td>47.4</td><td>35.20</td><td>474.18</td><td>352.05</td><td>24x24 |f'c=5000psi | 4-#11</td><td>284.51</td><td>211.23</td><td>18x18 | f'c=5000psi | 4-#11</td></tr>
<tr><td>C28</td><td>98.5</td><td>9.62</td><td>984.81</td><td>96.15</td><td>20x20| f'c=5000psi | 4-#9</td><td>590.89</td><td>57.69</td><td>16x16 | f'c=5000psi | 4-#8</td></tr>
<tr><td>C29</td><td>65.0</td><td>-0.42</td><td>650.36</td><td>-4.23</td><td>16x16 | f'c=5000psi | 4-#8</td><td>390.22</td><td>-2.54</td><td>16x16 | f'c=5000psi | 4-#8</td></tr>
<tr><td>C30</td><td>47.4</td><td>35.20</td><td>474.18</td><td>352.05</td><td>24x24 |f'c=5000psi | 4-#11</td><td>284.51</td><td>211.23</td><td>18x18 | f'c=5000psi | 4-#11</td></tr>
<tr><td>C31</td><td>98.5</td><td>9.62</td><td>984.81</td><td>96.15</td><td>20x20| f'c=5000psi | 4-#9</td><td>590.89</td><td>57.69</td><td>16x16 | f'c=5000psi | 4-#8</td></tr>
<tr><td>C32</td><td>60.1</td><td>-3.71</td><td>601.04</td><td>-37.10</td><td>18x18 | f'c=5000psi | 4-#9</td><td>360.63</td><td>-22.26</td><td>16x16 | f'c=5000psi | 4-#8</td></tr>
<tr><td>C33</td><td>32.2</td><td>-11.63</td><td>321.83</td><td>-116.28</td><td>16x16 | f'c=5000psi | 4-#8</td><td>193.10</td><td>-69.77</td><td>16x16 | f'c=5000psi | 4-#8</td></tr>
<tr><td>C34</td><td>4.0</td><td>4.16</td><td>40.16</td><td>41.57</td><td>16x16 | f'c=5000psi | 4-#8</td><td>24.10</td><td>24.94</td><td>16x16 | f'c=5000psi | 4-#8</td></tr>
<tr><td>C34a</td><td>5.0</td><td>3.13</td><td>50.47</td><td>31.34</td><td>16x16 | f'c=5000psi | 4-#8</td><td>30.28</td><td>18.81</td><td>16x16 | f'c=5000psi | 4-#8</td></tr>
<tr><td>C35</td><td>47.4</td><td>35.21</td><td>474.31</td><td>352.13</td><td>24x24 |f'c=5000psi | 4-#11</td><td>284.59</td><td>211.28</td><td>18x18 | f'c=5000psi | 4-#11</td></tr>
<tr><td>C36</td><td>98.5</td><td>9.61</td><td>984.94</td><td>96.07</td><td>20x20| f'c=5000psi | 4-#9</td><td>590.96</td><td>57.64</td><td>16x16 | f'c=5000psi | 4-#8</td></tr>
<tr><td>C37</td><td>75.7</td><td>6.68</td><td>756.86</td><td>66.77</td><td>18x18 | f'c=5000psi | 4-#9</td><td>454.11</td><td>40.06</td><td>16x16 | f'c=5000psi | 4-#8</td></tr>
<tr><td>C38</td><td>47.4</td><td>35.20</td><td>474.18</td><td>352.05</td><td>24x24 |f'c=5000psi | 4-#11</td><td>284.51</td><td>211.23</td><td>18x18 | f'c=5000psi | 4-#11</td></tr>
<tr><td>C39</td><td>98.5</td><td>9.62</td><td>984.81</td><td>96.15</td><td>20x20| f'c=5000psi | 4-#9</td><td>590.89</td><td>57.69</td><td>16x16 | f'c=5000psi | 4-#8</td></tr>
<tr><td>C40</td><td>68.6</td><td>1.99</td><td>686.49</td><td>19.86</td><td>18x18 | f'c=5000psi | 4-#9</td><td>411.89</td><td>11.92</td><td>16x16 | f'c=5000psi | 4-#8</td></tr>
<tr><td>C41</td><td>42.3</td><td>-15.28</td><td>422.59</td><td>-152.83</td><td>16x16 | f'c=5000psi | 4-#8</td><td>253.55</td><td>-91.70</td><td>16x16 | f'c=5000psi | 4-#8</td></tr>
<tr><td>C42</td><td>47.4</td><td>35.20</td><td>474.18</td><td>352.05</td><td>24x24 |f'c=5000psi | 4-#11</td><td>284.51</td><td>211.23</td><td>18x18 | f'c=5000psi | 4-#11</td></tr>
<tr><td>C43</td><td>98.5</td><td>9.62</td><td>984.81</td><td>96.15</td><td>20x20| f'c=5000psi | 4-#9</td><td>590.89</td><td>57.69</td><td>16x16 | f'c=5000psi | 4-#8</td></tr>
<tr><td>C44</td><td>73.9</td><td>5.50</td><td>739.20</td><td>55.00</td><td>18x18 | f'c=5000psi | 4-#9</td><td>443.52</td><td>33.00</td><td>16x16 | f'c=5000psi | 4-#8</td></tr>
<tr><td>C45</td><td>46.5</td><td>-20.20</td><td>464.74</td><td>-202.04</td><td>20x20| f'c=5000psi | 4-#9</td><td>278.84</td><td>-121.23</td><td>16x16 | f'c=5000psi | 4-#8</td></tr>
<tr><td>C46</td><td>47.4</td><td>35.21</td><td>474.31</td><td>352.13</td><td>24x24 |f'c=5000psi | 4-#11</td><td>284.59</td><td>211.28</td><td>18x18 | f'c=5000psi | 4-#11</td></tr>
<tr><td>C47</td><td>96.0</td><td>7.93</td><td>959.75</td><td>79.27</td><td>20x20| f'c=5000psi | 4-#9</td><td>575.85</td><td>47.56</td><td>16x16 | f'c=5000psi | 4-#8</td></tr>
<tr><td>C48</td><td>95.1</td><td>-8.59</td><td>950.51</td><td>-85.88</td><td>20x20| f'c=5000psi | 4-#9</td><td>570.30</td><td>-51.53</td><td>16x16 | f'c=5000psi | 4-#8</td></tr>
<tr><td>C49</td><td>46.5</td><td>-20.20</td><td>464.74</td><td>-202.04</td><td>20x20| f'c=5000psi | 4-#9</td><td>278.84</td><td>-121.23</td><td>16x16 | f'c=5000psi | 4-#8</td></tr>
<tr><td>C50</td><td>52.6</td><td>39.42</td><td>525.71</td><td>394.15</td><td>24x24 |f'c=5000psi | 4-#11</td><td>315.43</td><td>236.49</td><td>18x18 | f'c=5000psi | 4-#11</td></tr>
<tr><td>C52</td><td>48.3</td><td>-28.28</td><td>483.20</td><td>-282.80</td><td>24x24 |f'c=5000psi | 4-#11</td><td>289.92</td><td>-169.68</td><td>18x18 | f'c=5000psi | 4-#11</td></tr>
<tr><td>C53</td><td>47.0</td><td>31.34</td><td>469.57</td><td>313.45</td><td>24x24 |f'c=5000psi | 4-#11</td><td>281.74</td><td>188.07</td><td>18x18 | f'c=5000psi | 4-#11</td></tr>
<tr><td>C55</td><td>24.6</td><td>18.23</td><td>246.47</td><td>182.27</td><td>18x18 | f'c=5000psi | 4-#9</td><td>147.88</td><td>109.36</td><td>16x16 | f'c=5000psi | 4-#8</td></tr>
<tr><td>C56</td><td>23.0</td><td>-14.01</td><td>230.16</td><td>-140.11</td><td>16x16 | f'c=5000psi | 4-#8</td><td>138.10</td><td>-84.06</td><td>16x16 | f'c=5000psi | 4-#8</td></tr>
<tr><td>C57</td><td>25.1</td><td>16.76</td><td>250.77</td><td>167.58</td><td>18x18 | f'c=5000psi | 4-#9</td><td>150.46</td><td>100.55</td><td>16x16 | f'c=5000psi | 4-#8</td></tr>
<tr><td>C58</td><td>24.1</td><td>-11.93</td><td>240.77</td><td>-119.30</td><td>16x16 | f'c=5000psi | 4-#8</td><td>144.46</td><td>-71.58</td><td>16x16 | f'c=5000psi | 4-#8</td></tr>
</table>

Column Sizing [Fig 2.7.1 PCI handbook 6th edition]			
Level 10-11			
Designation	*Φ Pn (kips)*	*Φ Mn (k-ft)*	*TRIAL*
C1	43.41	-70.05	16x16 | f'c=5000psi | 4-#8
C2	78.49	0.00	16x16 | f'c=5000psi | 4-#8
C3	78.49	0.00	16x16 | f'c=5000psi | 4-#8
C4	78.49	0.00	16x16 | f'c=5000psi | 4-#8
C5	43.41	-174.69	18x18 | f'c=5000psi | 4-#11
C6	94.38	77.39	16x16 | f'c=5000psi | 4-#8
C7	169.64	0.00	16x16 | f'c=5000psi | 4-#8
C8	169.64	0.00	16x16 | f'c=5000psi | 4-#8
C9	169.64	0.00	16x16 | f'c=5000psi | 4-#8
C10	94.55	-87.14	16x16 | f'c=5000psi | 4-#8
C11	99.70	147.72	18x18 | f'c=5000psi | 4-#11
C12	193.92	22.30	16x16 | f'c=5000psi | 4-#8
C13	160.68	0.00	16x16 | f'c=5000psi | 4-#8
C14	158.96	0.00	16x16 | f'c=5000psi | 4-#8
C15	111.20	-61.28	16x16 | f'c=5000psi | 4-#8
C16	36.51	-22.32	16x16 | f'c=5000psi | 4-#8
C17	100.77	149.16	18x18 | f'c=5000psi | 4-#11
C18	157.25	-29.47	16x16 | f'c=5000psi | 4-#8
C19	124.65	0.00	16x16 | f'c=5000psi | 4-#8
C20	13.51	20.42	16x16 | f'c=5000psi | 4-#8
C21	12.31	-0.42	18x18 | f'c=5000psi | 4-#11
C22	108.13	-22.00	16x16 | f'c=5000psi | 4-#8
C23	53.48	-40.65	16x16 | f'c=5000psi | 4-#8
C24	112.81	-45.32	16x16 | f'c=5000psi | 4-#8
C25	72.03	-12.59	16x16 | f'c=5000psi | 4-#8
C26	36.16	-28.76	16x16 | f'c=5000psi | 4-#8
C27	94.84	140.82	18x18 | f'c=5000psi | 4-#11
C28	196.96	38.46	16x16 | f'c=5000psi | 4-#8
C29	130.07	-1.69	16x16 | f'c=5000psi | 4-#8
C30	94.84	140.82	18x18 | f'c=5000psi | 4-#11
C31	196.96	38.46	16x16 | f'c=5000psi | 4-#8
C32	120.21	-14.84	16x16 | f'c=5000psi | 4-#8
C33	64.37	-46.51	16x16 | f'c=5000psi | 4-#8
C34	8.03	16.63	16x16 | f'c=5000psi | 4-#8
C34a	10.09	12.54	16x16 | f'c=5000psi | 4-#8
C35	94.86	140.85	18x18 | f'c=5000psi | 4-#11
C36	196.99	38.43	16x16 | f'c=5000psi | 4-#8
C37	151.37	26.71	16x16 | f'c=5000psi | 4-#8
C38	94.84	140.82	18x18 | f'c=5000psi | 4-#11
C39	196.96	38.46	16x16 | f'c=5000psi | 4-#8
C40	137.30	7.94	16x16 | f'c=5000psi | 4-#8
C41	84.52	-61.13	16x16 | f'c=5000psi | 4-#8
C42	94.84	140.82	18x18 | f'c=5000psi | 4-#11
C43	196.96	38.46	16x16 | f'c=5000psi | 4-#8
C44	147.84	22.00	16x16 | f'c=5000psi | 4-#8
C45	92.95	-80.82	16x16 | f'c=5000psi | 4-#8
C46	94.86	140.85	18x18 | f'c=5000psi | 4-#11
C47	191.95	31.71	16x16 | f'c=5000psi | 4-#8
C48	190.10	-34.35	16x16 | f'c=5000psi | 4-#8
C49	92.95	-80.82	16x16 | f'c=5000psi | 4-#8
C50	105.14	157.66	18x18 | f'c=5000psi | 4-#11
C52	96.64	-113.12	16x16 | f'c=5000psi | 4-#8
C53	93.91	125.38	18x18 | f'c=5000psi | 4-#11
C55	49.29	72.91	16x16 | f'c=5000psi | 4-#8
C56	46.03	-56.04	16x16 | f'c=5000psi | 4-#8
C57	50.15	67.03	16x16 | f'c=5000psi | 4-#8
C58	48.15	-47.72	16x16 | f'c=5000psi | 4-#8

Full column checks, and columns loading can be obtained upon request. PCA Column was used to check columns on level 1-4 interaction diagrams are on the next page.

PCA Column: Interaction diagrams for columns on ground level.

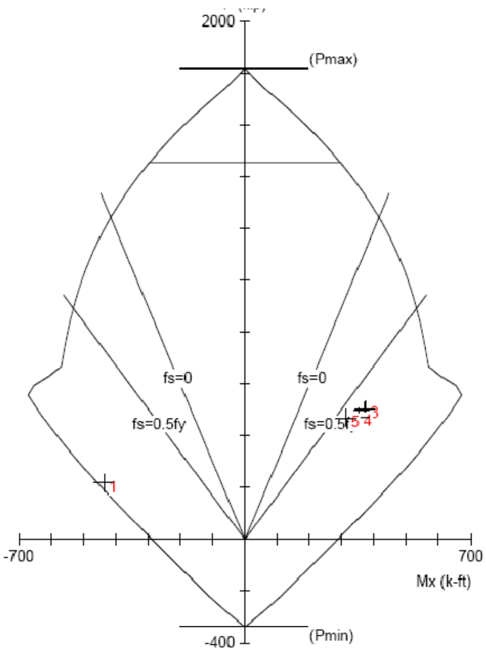


Fig 1: 24x24" Column w/ 4-#11

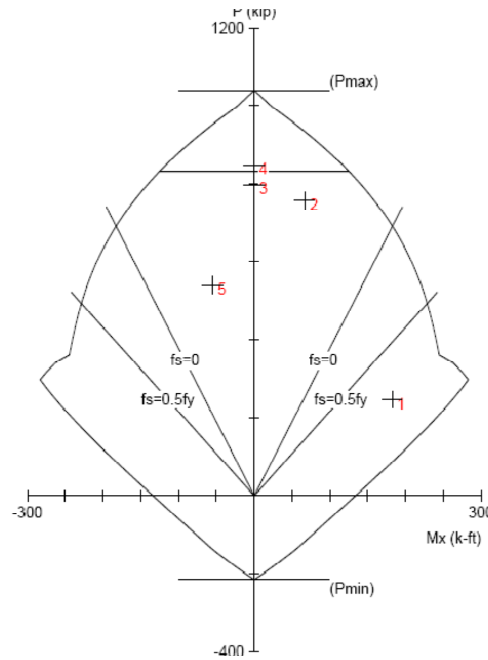


Fig 2: 18"x18" Column w/ 4-#9

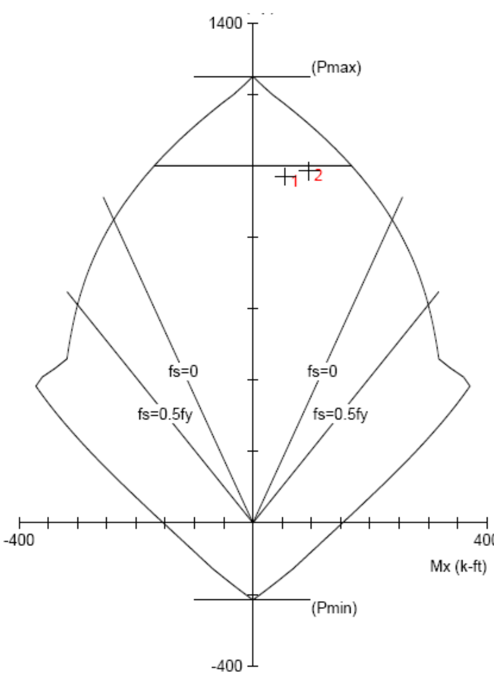


Fig 3: 20"x20" Columns w/ 4-#9

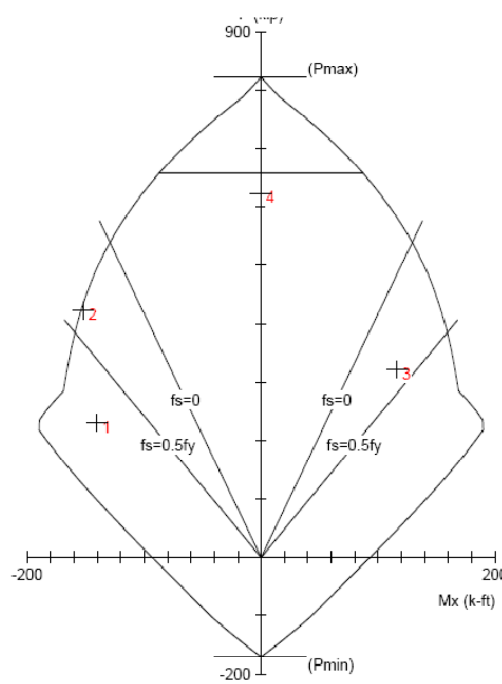


Fig 4: 18"x18" Column w/ 4-#8

Appendix #3

GRAVITY SYSTEM DETAILING

CREEP & STRAIN

CONNECTIONS

VOLUME CHANGE:

PCI 6th edition: Fig 3.10.13-3.10.16
Location: Washington D.C
Normal Weight Beam 12RB24
10 -1/2" diam. 270 low relaxation strands
Length 26 ft
Aps = 0.153*10= 1.53 in^2
Po=1.53*270*.75*(1-.15)=263.35 kips
Po/A= 914.5 psi

V/S = 4 in
F'c=5000 psi
R.H=75%
Design Temp. Change = 50°F

At 60 Days :
Creep Strain: 169 x 10^{-6}
Shrinkage Strain: 354 x 10^{-6}
Creep correction factor: 1.325
Relative humidity correction (creep): 0.96
Relative humidity correction (shrinkage): 0.93
Volume-to-surface ratio correction (creep): 0.48
Volume-to-surface ratio correction (shrinkage): 0.46

Creep = 169 x 10^{-6}*0.96*0.48*1.325 = 1.03 x 10^{-4}
Shrinkage = 354 x 10^{-6}*0.93*0.46 = 1.512 x 10^{-4}
Total Strain = 2.546 x 10^{-4}
Total Shortening = 2.546 x 10^{-4}(26)(12)= **<u>0.079 in</u>**

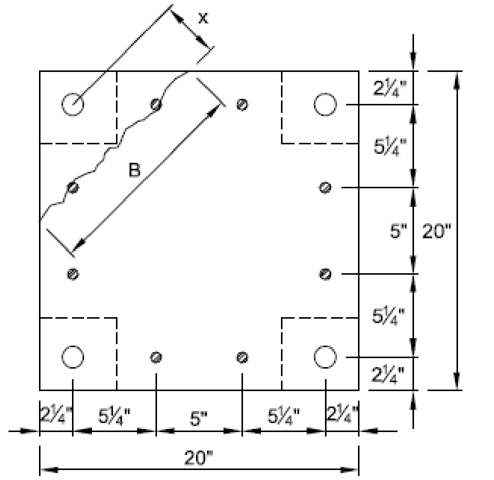
At final
Creep Strain: 315 x10^{-6}
Shrinkage Strain: 560 x 10^{-6}
Creep correction factor: 1.325
Volume-to-surface ratio correction (creep): 0.77
Volume-to-surface ratio correction (shrinkage): 0.75
Temperature Strain = 150 x 10^{-6}

Creep = 3.08x10^{-4}
Shrinkage = 3.906x10^{-4}
Total = 6.98 x 10^{-4}
Difference = 6.98 x 10^{-4} - 2.546 x 10^{-4} = **<u>4.43 x 10^{-4}</u>**

TOTAL STRAIN = 4.43E-4+150E-6= **<u>5.93 x 10^{-4}</u>**
TOTAL SHORTNING = 5.93E-4(12)(26)= **<u>0.185 in</u>**

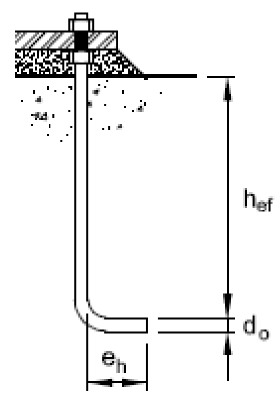
COLUMN TO FOUNDATION CONNECTION

A 20”x20” column located in the corner with type “P2” column cap details
Column C18
Factored Axial Load : 790 Kips
Column f’c=5000
Pedestal f’c= 4000psi
Base plate & Anchor bolts= 36ksi
Reinforcing bars=60ksi
Φ=1.0
ϕT_u= 200Ag=.2(20^2)=80 Kips or 20 Kips/Bolt



Base Plate Thickness: t= $\sqrt{((T_u(4)x)/(\Phi B F_y)}$
B=13.79in
X=3.71in
T= $\sqrt{20(4)(3.71)/(1.0)(13.79)(36)}$ = 0.77in
USE ⟶ **1” Plate**

Deformed Anchors:
As=Tu/ϕFy = 1.33in^2
Try ⟶ [8(0.2)=1.6>1.33 OK] **(8)-1/2”**



Anchor Bolts:
[Fig 6.16.3] 1” Diam Bolts A=0.785 in^2 Tension=25.6 Kips Shear = 13.7 Kips
4(25.5)=102.4 > 80 OK
[h_{ed}<11] Hooked anchor bolts

$N_p=1.2f'_c e_d d_o c_{crp}$ ⟶ N_p=20Kips
C_{crp}=1.0 [concrete assumed uncracked]
D_o=1.0

E_n= 3.97in

Spacing 15.5in <3h_{ef} [Fig 6.5

Breakout :
$C_{bs}=3.33\lambda sqrt(f'c/h_{ef})$ = 3.33(1)($\sqrt{4000}$/6.683)=80.6psi
An=36(36) 1296 in^2
Ψ=0.7+0.3(10.25/1.5*6.83)=1.0
C_{crb}(Designed for corner cracking)= 0.8
$N_{cb}=C_{bs}A_nC_{ceb}\Psi_{ed}$ = 83.6 Kips < 102.4 OK

Detail of Column Connection to Column Cap

TB-15 : Evenly loaded	
Vu= 5.46	
Nu=0.2	
Vu=1.09	λ=1.0
d_e=4in	d = 11in
b=12in	b_l = 20in
h=24in	h_l= 12in
Fy=60ksi	b_t=8in
F'c=5000psi	s=48in

d_e=Bt/2= 4in
S > bt+hl = 16"
d_e<2(bl-b)+bt+bl=44"
Eq: 4.5.1.2 $øVn=ø\lambda\sqrt{f'c}h_l[2(b_l-b)+b_t+h_l+2d_e]$
0.75*1.0*√5000*12[2(20-12)+8+1+2*4 = 21 >5.46 OK
Shear Span a = ¾(b_l-b)=3/4(20-12)= 6in

<u>Flexural Reinforcing</u> : As = 1/øfy[Vu(a/d)+Nu(h/d)]
1/(.75*60)(5.46(6/11)+1.09(24/11) = 0.11 in^2
Spacing = 6h_l > s/2 >12in [lesser of these 3]

<u>Longitudinal Reinforcing</u> :
Attach ledger to web Ash = Vu/øfy(m)
M: Table 2.5.4.1

h_l/h = 0.5
b_l/b= 1.667 $\longrightarrow$ M= 1.19

A_{sh}=[5.48/.75*60]1.19 = 0.145 in^2

A_l= 200(b_l-b)d_l/fy= 0.29in^2

<u>Additional reinforcing due to e:</u>
Vue= 5.46* 5 = 27.3
Awl= Vue/2øfyDw = 27.3/2*.75*60*10.5= 0.028 in^2
A_{wl} = #3

USE :
A_{wv}: #3
A_l: 1-#3 top & 1-#3 bot
A_{sh}: #3 bars @12"O.C
A_s: #3 Bars @ 12" O.C
with 2 additional bars at the beam end to provide reinforcing for stems placed near the end

301: Dapped ends
28IT24
$Vu \leq \Phi Vn$
Vu=42.41 Kips
Nu=0.2Vu=0.2(42.41)=8.48 kips
F'c=5000
Fy=60,000

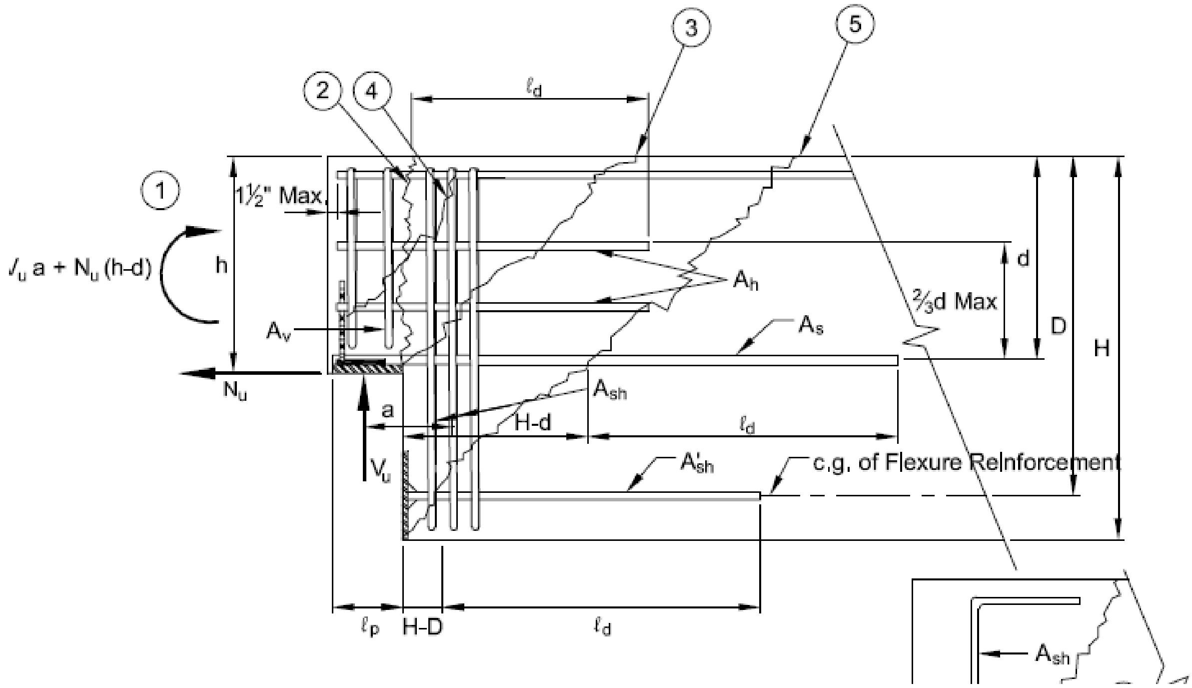
1. Flexure in extended end
   Shear span a=6" d=15"
   $A_s=1/\Phi fy[Vu(a/d)+Nu(h/d) = 1/.75*60[42.41*(6/15)+8.48(24/15)$ $\underline{= 0.62\ in^2}$
2. Direct Shear :
   $\mu=1000\lambda bh\mu/Vu = 1000*1*28*24*1.4/42.41*1000 = 3.4$
   $A_s=2Vu/3\Phi Fy\mu_e+Nu/\Phi fy = 0.373\ in^2 < 0.62 in^2$
   $\underline{A_s}$: USE → 2 #4 As = $0.62in^2$
   $A_h=0.5(A_s-A_n)=0.5(0.62-0.188)=0.216\ in^2$
   $V_{uMAX}=1000\lambda^2 Acr = 1000(1)(18)(16.75) = 226$ Kips > 42.41 OK
   $\underline{A_h}$: USE → 2 #4 Ah=.4 in2
3. Diagonal tension at re-entrant corner
   $A_{sh}=Vu/\Phi fy = 0.94\ in^2$
   $\underline{A_{sh}}$: USE → 4 #5 Ash = 1.24 in2
   $\underline{A_{sh}}$': USE → 5 #4 Ash = 1 in2
4. Diagonal Tension in extended end:
   Concrete capacity = $2\lambda\sqrt{f'c}bd = 55.15$ kips
   $A_v=1/2fy[Vu/\Phi-2bd\lambda\sqrt{f'c}] = 0.011\ in^2$
   USE → Stirrup
   $\Phi Vn=\Phi(A_vf_y+A_hf_y+2\lambda\sqrt{f'c}bd) =$
   $\Phi Vn>Vu$

Anchorage: A_s Design Aid 11.2.9:

Fy=60000 f'c=5000 ld=

#5 Bars: Ld=21in

Ash' Ld =17" Extension = H-d+ld = 24-14.4 + 17 = 26.6"
As Ld = 17" Extension = 26.6"
Ah Ld = 17" Ex = 26.6"



Shown above is the typical reinforcing for a dapped end beam from PCI 6th edition. The numbers represent the 5 modes of failure resulting from direct shear and diagonal tension.

Appendix #4

LATERAL CHECK

ETABS

Existing Shear wall reinforcing : A different numbering system was used during my analysis wall numbers on the left is the alternate numbering used for the lateral check.

Wall # 1

Wall #3

Wall #2

Wall #4

REINFORCED CONCRETE SHEAR WALL SCHEDULE																
LEVEL	WALL #1				WALL #2				WALL #3				WALL #4			
	V	H	M	TIES (SIZE/ARRANGEMENT #)	V	H	M	TIES (SIZE/ARRANGEMENT #)	V	H	M	TIES (SIZE/ARRANGEMENT #)	V	H	M	TIES (SIZE/ARRANGEMENT #)
P.H. ROOF ELEV. MACH.RM LVL	#4@12"	#4@12"	8#8	#3/ARRANGEMENT #1	#4@12"	#4@12"	8#6	#3/ARRANGEMENT #1	#4@12"	#4@12"	10#8	#3/ARRANGEMENT #2	#4@12"	#4@12"	8#9	#3/ARRANGEMENT #1
ROOF	#4@12"	#4@12"	8#8	#3/ARRANGEMENT #1	#4@12"	#4@12"	8#6	#3/ARRANGEMENT #1	#4@12"	#4@12"	10#8	#3/ARRANGEMENT #2	#4@12"	#4@12"	8#9	#3/ARRANGEMENT #1
11th FLOOR	#4@12"	#4@12"	8#8	#3/ARRANGEMENT #1	#4@12"	#4@12"	8#6	#3/ARRANGEMENT #1	#4@12"	#4@12"	10#8	#3/ARRANGEMENT #2	#4@12"	#4@12"	8#9	#3/ARRANGEMENT #1
10th FLOOR	#4@12"	#4@12"	8#8	#3/ARRANGEMENT #1	#4@12"	#4@12"	8#6	#3/ARRANGEMENT #1	#4@12"	#4@12"	10#8	#3/ARRANGEMENT #2	#4@12"	#4@12"	8#9	#3/ARRANGEMENT #1
9th FLOOR	#4@12"	#4@12"	8#8	#3/ARRANGEMENT #1	#4@12"	#4@12"	8#6	#3/ARRANGEMENT #1	#4@12"	#4@12"	10#8	#3/ARRANGEMENT #2	#4@12"	#4@12"	8#9	#3/ARRANGEMENT #1
8th FLOOR	#4@12"	#4@12"	8#8	#3/ARRANGEMENT #1	#4@12"	#4@12"	8#6	#3/ARRANGEMENT #1	#4@12"	#4@12"	10#8	#3/ARRANGEMENT #2	#4@12"	#4@12"	8#9	#3/ARRANGEMENT #1
7th FLOOR	#4@12"	#4@12"	8#8	#3/ARRANGEMENT #1	#4@12"	#4@12"	8#6	#3/ARRANGEMENT #1	#4@12"	#4@12"	10#8	#3/ARRANGEMENT #2	#5@12"	#4@12"	8#10	#3/ARRANGEMENT #1
6th FLOOR	#4@12"	#4@12"	8#8	#3/ARRANGEMENT #1	#4@12"	#4@12"	8#6	#3/ARRANGEMENT #1	#4@12"	#4@12"	10#8	#3/ARRANGEMENT #2	#5@12"	#4@12"	8#10	#3/ARRANGEMENT #1
5th FLOOR	#4@12"	#4@12"	8#8	#3/ARRANGEMENT #1	#4@12"	#4@12"	8#6	#3/ARRANGEMENT #1	#4@12"	#4@12"	10#8	#3/ARRANGEMENT #2	#5@12"	#4@12"	8#10	#3/ARRANGEMENT #1
4th FLOOR	#5@12"	#4@12"	8#9	#3/ARRANGEMENT #1	#5@12"	#4@12"	8#9	#3/ARRANGEMENT #1	#5@12"	#4@12"	10#9	#3/ARRANGEMENT #2	#5@12"	#4@12"	8#10	#3/ARRANGEMENT #1
3rd FLOOR	#5@12"	#4@12"	8#9	#3/ARRANGEMENT #1	#5@12"	#4@12"	8#9	#3/ARRANGEMENT #1	#5@12"	#4@12"	10#9	#3/ARRANGEMENT #2	#5@12"	#4@12"	8#10	#3/ARRANGEMENT #1
2nd FLOOR	#5@12"	#4@12"	8#9	#3/ARRANGEMENT #1	#5@12"	#4@12"	8#9	#3/ARRANGEMENT #1	#5@12"	#4@12"	10#9	#3/ARRANGEMENT #2	#5@12"	#4@12"	8#10	#3/ARRANGEMENT #1
1ST FLOOR	#5@12"	#4@12"	8#0	#3/ARRANGEMENT #1	#5@12"	#4@12"	8#0	#3/ARRANGEMENT #1	#5@12"	#4@12"	10#0	#3/ARRANGEMENT #2	#5@12"	#4@12"	8#10	#3/ARRANGEMENT #1

f'c = 5 KSI

INPUT

Additional Mass: was added to each diaphragm to account for the planks, beams, topping, and partitions = (22377.84/11)/386)15405*12 = <u>4.6E-6</u>

F22 : Was changed from 1 to 0.5 for the concrete in the shear walls to account for the cracked section

P-Delta : affects were included in the analysis with a non-iterative (mass based) method

Dynamic analysis: was also included considering all 12 modes.

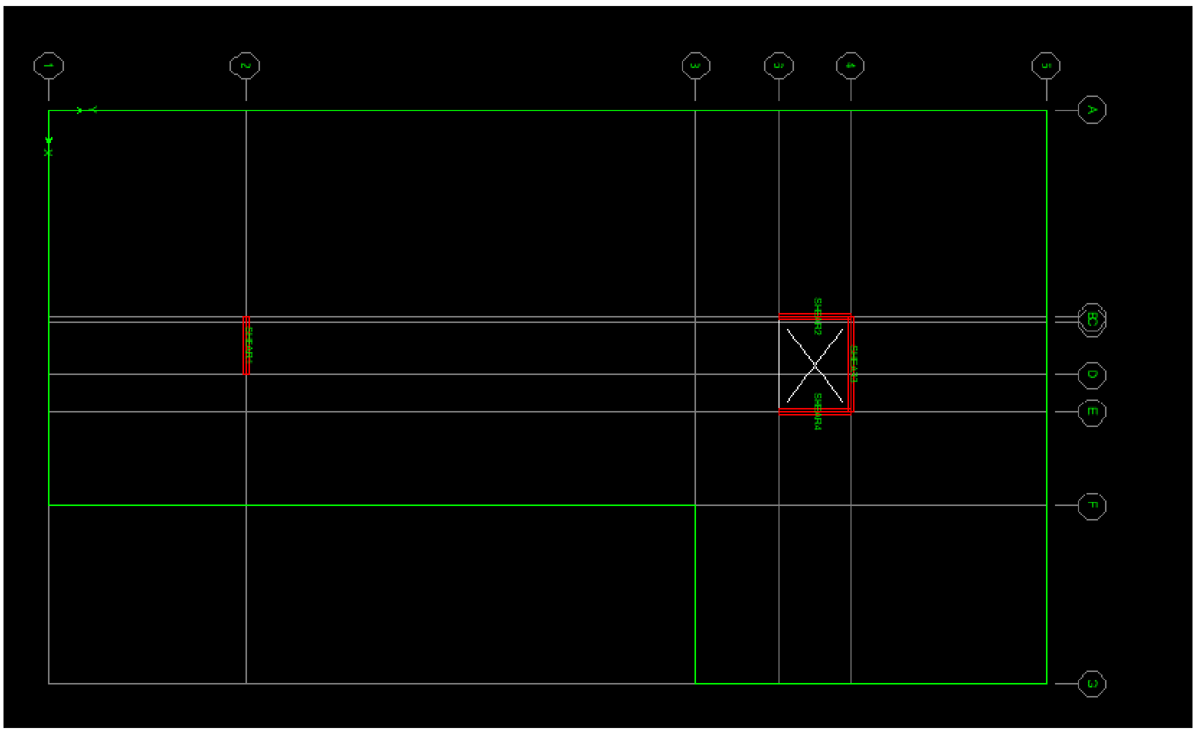


Figure #1: A snapshot of a typical floor that was input into E-Tabs for analysis

Load Combinations (Strength): The following load combinations were put into E-Tabs to determine reinforcing in all shear walls.

#1 : 1.4D
#2: 1.2D + 1.6L + 0.5S
#3: 1.2D + 1.6S + 0.8W
#4: 1.2D + 1.6W + L + 0.5S
#5: (1.2+0.2Sds)D + ρE + L + 0.2S
#6: 0.9D + 1.6W + 1.6H
#7: (0.9 – 0.2Sds)D + ρE + 1.6H

$\rho = 1.0$

$S_{ds} = 0.163$

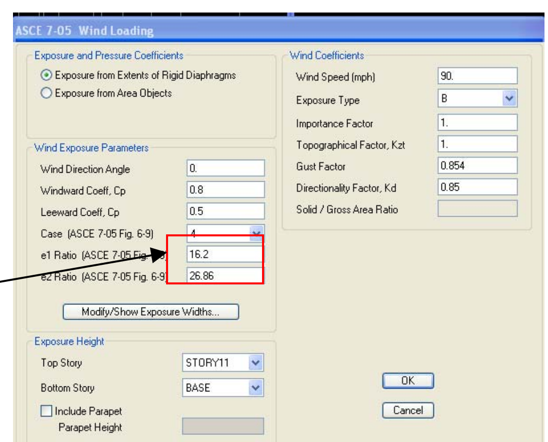
(50) Load Combinations Inputted into E-Tabs once all combos were considered

1. 1.4D
2. 1.2D + 1.6L + 0.5S
3. 1.2D + 1.6S + 0.8 Wind
4. 1.2D + 1.6S + 0.8 Wind2
5. 1.2D + 1.6S + 0.8 Wind3
6. 1.2D + 1.6S + 0.8 Wind4
7. 1.2D + 1.6S + 0.8 Wind5
8. 1.2D + 1.6S + 0.8 Wind6
9. 1.2D + 1.6S + 0.8 Wind 7
10. 1.2D + 1.6S + 0.8 Wind 8
11. 1.2D + 1.6S + 0.8 Wind 9
12. 1.2D + 1.6S + 0.8 Wind 10
13. 1.2D + 1.6S + 0.8 Wind 11
14. 1.2D + 1.6S + 0.8 Wind 12
15. 1.2D + 1.6Wind + L + 0.5S
16. 1.2D + 1.6Wind2 + L + 0.5S
17. 1.2D + 1.6Wind3 + L + 0.5S
18. 1.2D + 1.6Wind4 + L + 0.5S
19. 1.2D + 1.6Wind5 + L + 0.5S
20. 1.2D + 1.6Wind6 + L + 0.5S
21. 1.2D + 1.6Wind7 + L + 0.5S
22. 1.2D + 1.6Wind8 + L + 0.5S
23. 1.2D + 1.6Wind9 + L + 0.5S
24. 1.2D + 1.6Wind10 + L + 0.5S
25. 1.2D + 1.6Wind11 + L + 0.5S
26. 1.2D + 1.6Wind12 + L + 0.5S
27. 1.233D + 1.0QuakeX + L + 0.2S
28. 1.233D + 1.0QuakeY + L + 0.2S
29. 1.233D + 1.0QuakeXeY + L + 0.2S
30. 1.233D+ 1.0QuakeXenY + L + 0.2S
31. 1.233D + 1.0QuakeYeX + L + 0.2S
32. 1.233D + 1.0QuakeYenX + L + 0.2S
33. 0.9D + 1.6Wind
34. 0.9D + 1.6Wind 2
35. 0.9D + 1.6Wind 3

35. 0.9D + 1.6 Wind
36. 0.9D + 1.6Wind 4
37. 0.9D + 1.6Wind 5
38. 0.9D + 1.6Wind 6
39. 0.9D + 1.6Wind 7
40. 0.9D + 1.6Wind 8
41. 0.9D + 1.6Wind 9
42. 0.9D + 1.6Wind 10
43. 0.9D + 1.6Wind 11
44. 0.9D + 1.6Wind 12
45. 0.867D +1.0 Quake X
46. 0.867D + 1.0 Quake Y
47. 0.867D + 1.0 Quake XeY
48. 0.867D+ 1.0 Quake XenY
49. 0.867D + 1.0 Quake YeX
50. 0.867D + 1.0 Quake YenX

WIND: Because seismic controls wind values were calculated in E-Tabs

E1 = 0.15(108)
E2 = 0.15(179)



SEISMIC: Was calculated as seen in **appendix #1** and then input by hand for each of the 6 cases.

OUTPUT

Controlling Combination:

Combo 50 = 0.867D+1.0Quake Y (-) X eccentricity : Controls in Flexure
Combo 48 = 0.0867D + 1.0 Quake Y(-)X eccentricity : Controls in Flexure and Shear
Combo 30 = 1.22D +1.0L +1.0 Quake X (-) Y eccentricity: Controls in Shear

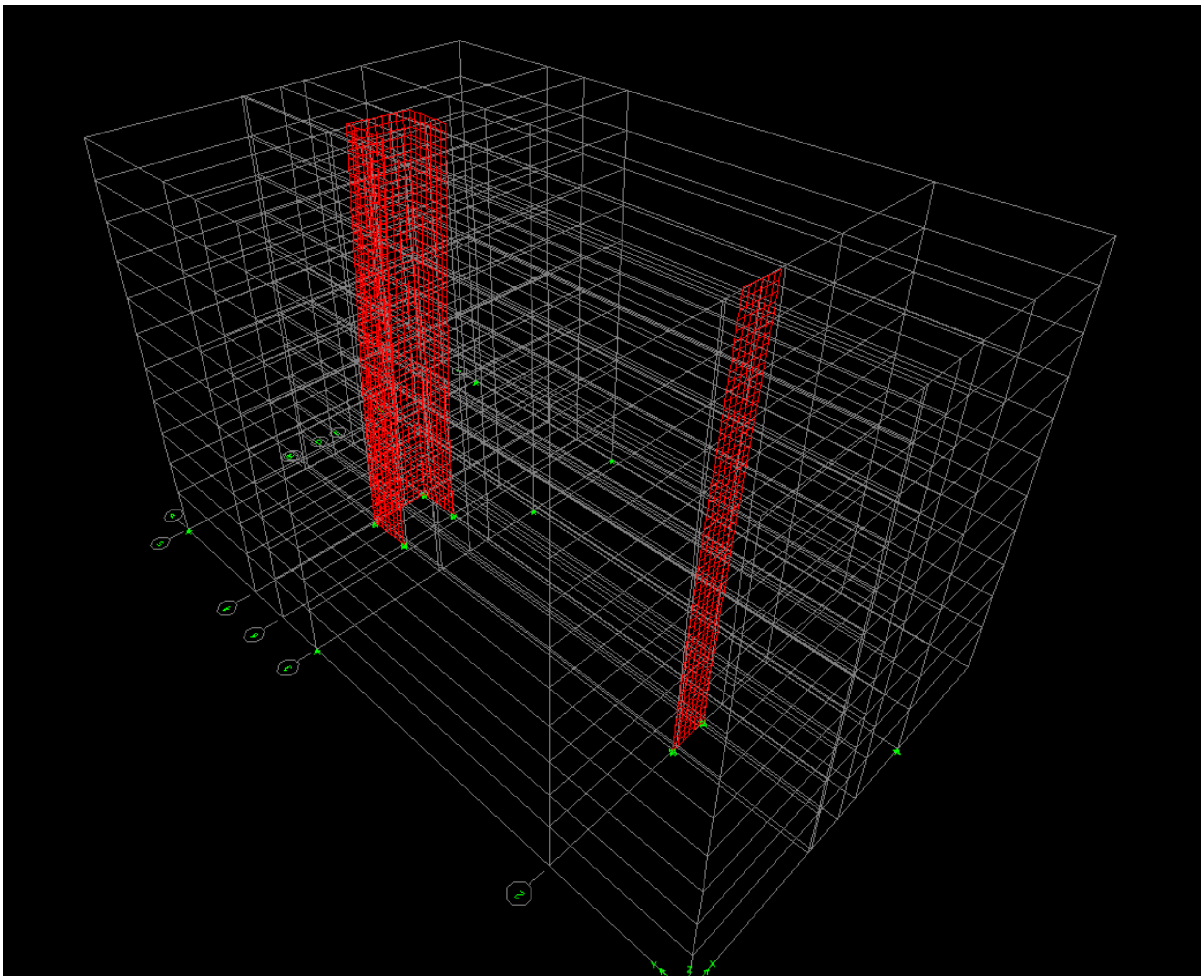


Figure #2: Deformed shape after analysis of seismic displacement in the X direction.

Wall #1 Output: (Units Kip-in)

Uniform Reinforcing Pier Section - Design (UBC97)

Story ID: STORY1 Pier ID: SHEAR1 X Loc: 44.66667 Y Loc: 35.33333 Units: Kip-ft

Flexural Design for P-M2-M3 (RLLF = 1.000)

Station Location	Required Reinf Ratio	Current Reinf Ratio	Flexural Combo	Pu	M2u	M3u	Pier Ag
Top	0.0025	0.0047	COMB14	143.046	0.000	0.000	11.000
Bottom	0.0034	0.0047	COMB12	162.274	0.000	2035.736	11.000

Values input into PCA column for flexural check

Shear Design

Station Location	Rebar in^2/ft	Shear Combo	Pu	Mu	Vu	Capacity phi Vc	Capacity phi Vn
Top Leg 1	0.360	COMB12	143.046	1177.897	63.819	100.970	215.018
Bot Leg 1	0.360	COMB12	162.274	2035.736	63.819	66.752	180.800

Boundary Element Check

Station Location	B-Zone Length	B-Zone Combo	Pu	Mu	Vu	Pu/Po
Top Leg 1	Not Needed	COMB8	203.432	0.000	0.000	0.0293
Bot Leg 1	Not Needed	COMB8	230.777	0.000	0.000	0.0328

Combos... Overwrites... OK Cancel

Wall #2&4 Output: (Units Kip-in)

Uniform Reinforcing Pier Section - Design (UBC97)

Story ID: STORY2 Pier ID: SHEAR2 X Loc: 470 Y Loc: 1655 Units: Kip-in

Flexural Design for P-M2-M3 (RLLF = 1.000)

Station Location	Required Reinf Ratio	Current Reinf Ratio	Flexural Combo	Pu	M2u	M3u	Pier Ag
Top	0.0165	0.0035	COMB14	-1026.284	0.000	75371.383	2489.763
Bottom	0.0198	0.0035	COMB14	-1180.360	0.000	93602.479	2489.763

Shear Design

Station Location	Rebar in^2/ft	Shear Combo	Pu	Mu	Vu	Capacity phi Vc	Capacity phi Vn
Top Leg 1	0.562	COMB14	-1026.284	75371.383	266.690	56.123	266.690
Bot Leg 1	0.589	COMB14	-1180.360	93602.479	266.690	46.313	266.690

Boundary Element Check

Station Location	B-Zone Length	B-Zone Combo	Pu	Mu	Vu	Pu/Po
Top Leg 1	Not Needed	COMB14	-1026.284	75371.383	266.690	0.0797
Bot Leg 1	Not Needed	COMB14	-1180.360	93602.479	266.690	0.0886

Combos... Overwrites... OK Cancel

Wall #3 Output: (Unit Kip-in)

Uniform Reinforcing Pier Section - Design (UBC97)

Story ID: STORY1 Pier ID: SHEAR3 X Loc: 48.16667 Y Loc: 144.4167 Units: Kip-ft

Flexural Design for P-M2-M3 (RLLF = 1.000)

Station Location	Required Reinf Ratio	Current Reinf Ratio	Flexural Combo	Pu	M2u	M3u	Pier Ag
Top	0.0033	0.0045	COMB12	234.037	0.000	5070.183	18.000
Bottom	0.0051	0.0045	COMB12	264.572	0.000	7144.829	18.000

Shear Design

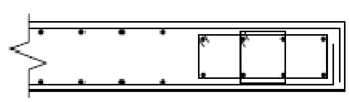
Station Location	Rebar in^2/ft	Shear Combo	Pu	Mu	Vu	Capacity phi Vc	Capacity phi Vn
Top Leg 1	0.360	COMB12	234.037	5070.183	400.288	219.938	453.218
Bot Leg 1	0.360	COMB12	264.572	7144.829	400.288	219.938	453.218

Boundary Element Check

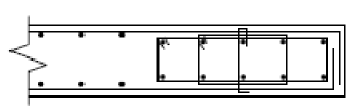
Station Location	B-Zone Length	B-Zone Combo	Pu	Mu	Vu	Pu/Po
Top Leg 1	4.115	COMB8	3530.436	0.000	0.000	0.3072
Bot Leg 1	4.445	COMB8	4040.927	0.000	0.000	0.3439

Combos... Overwrites... OK Cancel

REINFORCING DETAILS



TIE ARRANGEMENT #1 (8 BARS)
(TYPICAL AT EACH END)



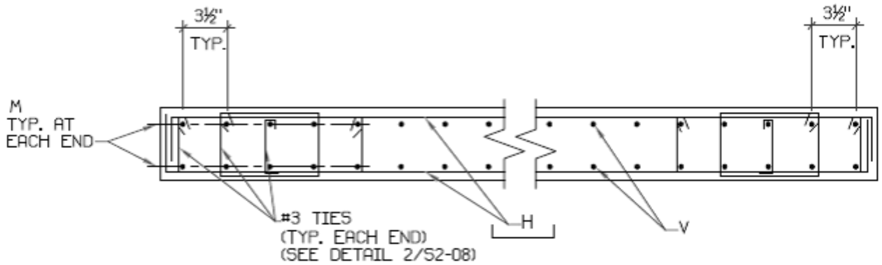
TIE ARRANGEMENT #2 (10 BARS)
(TYPICAL AT EACH END)

NOTE:
SEE SHEAR WALL SCHEDULES FOR SIZE OF TIES AND LOCATION OF TIE ARRANGEMENTS.

TYPICAL TIE ARRANGEMENT DETAILS

SCALE: 3/4" = 1'-0"

2
2-S2.01

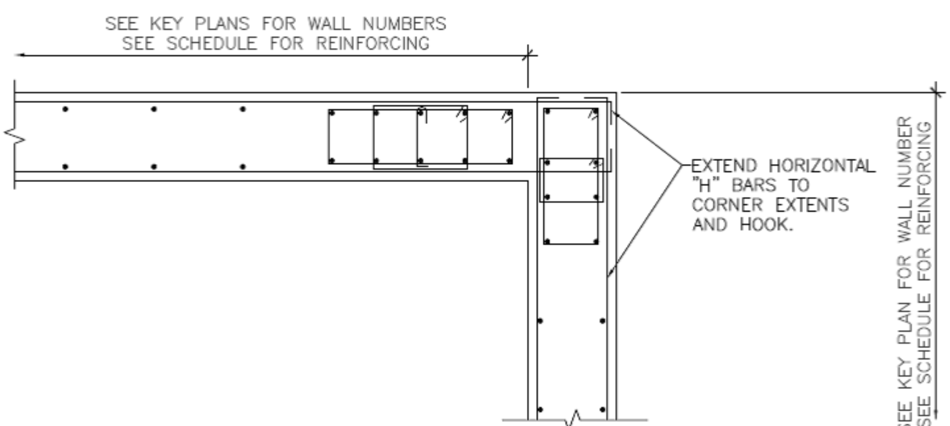


NOTES:
1. SEE KEY PLANS FOR SHEAR WALL DESIGNATION NUMBERS.
2. PROVIDE CORNER BARS WITH CLASS "B" SPLICES TO MATCH "H" BARS SHOWN ON SCHEDULE.
3. WHERE POSSIBLE, EXTEND "H" HORIZ. BARS INTO ADJACENT PERPENDICULAR WALL AND HOOK.
4. SEE SHEAR WALL SCHEDULES FOR FURTHER INFORMATION.

TYPICAL SHEAR WALL DETAIL

SCALE: 3/4" = 1'-0"

1
2-S2.01



TYPICAL CORNER DETAIL

SCALE: 3/4" = 1'-0"

3
2-S2.01