FINAL REPORT

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CITY VISTA.

BUILDING 2. 5TH AND K STREET . WASHINGTON D. C.

JULIE DAVIS

AE482 : SENIOR THESIS

STRUCTURAL OPTION

DR. MEMARI

APRIL 9, 2008

CITY VISTA

BUILDING 2

460 L STREET WASHINGTON D.C.

JULIE DAVIS | STRUCTURAL OPTION

Architecture

Materials:

-Grey & Buff colored brick

- Champagne gold metal panels : penthouse and pedestrian bridge
- Precast stone panels

Architectural Statement:

- *Horizontal Element: strong* roof overhang
- *Vertical Elements:* protruding glass and crème colored brick vertical components
- *Narrowing Element:* building narrows at street level for to provide a more intimate appearance.

Construction Management

Construction Date

- *Start:* December 2005
- *Finish*: December 2007

Delivery Method:

- *Design-Bid-Build*: with Davis construction as the single prime contractor

Crane:

- Crane footing: incorporated into building foundation system as the bridge footings

General Info

PROJECT TEAM

- *Owner*: L5K LLC.
- *Architect:* Tortis Gallas
- *Structural Engineer:* SK&A
- *MEP:* GHT Limited
- *Contractor:* Davis

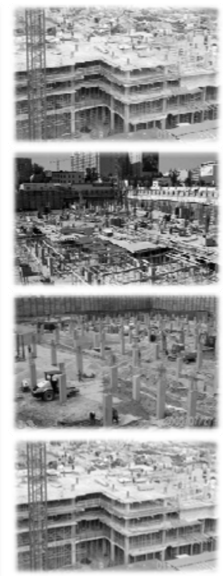
Size

- 11 Stories
- 324,296 sqft

Occupancy

- *Residential :* 149 Condos

Structural



Foundation System:

- Slab on grade construction with grade beams and a deep foundation system of augured cast in place *piles* drilled 60-65 feet below grade

Framing System:

- Cast in place concrete with *Post tension floor slabs*
- Conventional reinforcement for balconies
- (4) Shear walls

Pedestrian Bridge:

- *Steel* framing with moment connection
- Composite slab
- *F*lat rubber asphalt roofing system
- Composite beam construction where bridge connects to Building 2

Roof System:

- Post tension slab with ridged polystyrene insulation and ballast

MEP

Mechanical System:

- (2) Roof Top Units-208 V/3 phase 4000CFM,
- Heat Pumps – 208V/1 phase located in central locations varies from 600-1500 CFM
- Thru-the-wall heating *gas heating units* in condos: Gas Powered

Electrical System:

- (2) 3000A 208/120V SystemS
  1→ Lobby via sub-panels
  2→ Condos via 3000 A bus duct
- 230 KW diesel emergency generator
- *Contract Amount* = $3,000,000

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WASHINGTON D.C.
ADVISOR: DR. MEMARI

EXECUTIVE SUMMARY

City Vista, Building 2 is a mixed use complex located in downtown Washington D.C. Currently the building's gravity system is a flat plate post tension slab, tendons are unbounded and span in both directions. Slabs are supported by a grid of (52) cast in place columns. Resisting lateral forces are (4) cast in place shear walls. The building is supported with a deep foundation system consisting of over 270 augured cast in place piles and a 4" slab on grade. Building 2 is 324,298 sqft and 128'-5" tall at sections taken through the mechanical penthouse.

Developers in the D.C market buildings are driven by the 130'-0" height restrictions. Buildings are designed to maximize rentable space and minimize construction time while keeping the building under the 130 ft height restriction. With a post tension system floor to ceiling height is optimized while creating a finished ceiling with the underside of the slab.

Considering the competitive market, societies interest in "green" design, the finished ceiling incorporated into structural system and height restriction I have propose to redesign the post tensioned gravity system to a pre-cast system. Potentially a pre-cast system presents faster erection, possible leeds certification and similar if not the same floor to ceiling height

Before design began preliminary analysis and design consideration were formulated. Different pre-cast construction systems were compared and contrast. Issues concerning cost, shipping, fireproofing, optimization of member and connections were considered.

After preliminary analysis a hollow core floor system with 2" composite topping, interior inverted T-beams, exterior L-beams, and conventionally pre-cast columns was choose. A new column grid was created with span to depth ratios in mind. A lateral check was then conducted using E-tabs to verify the stability of the (4) shear walls. Cost, schedule and initial leeds analysis was performed along with architectural alterations to accommodate the new column grid.

After design was complete it was concluded that a pre-cast system would produce a building within the height limit, with a fast erection time, and finished ceiling if incorporated into the preliminary design phase of the project. The current architectural layout was conducive to the irregularly shaped bays that a post tension system allows. The new column grid creates awkward spaces within some condo layout.

Structurally the lateral system was not adequate after redesign to support the seismic base shear. This was un-expected because the building only increased in height by 3ft and weight by 3000 kips.

Economically the post tension system is cheaper after the cost analysis was done and it was discovered that the post tension system is $2,000,000 cheaper. After examining these results it is obvious this is a result of the beams added to the gravity system.

After completing the gravity, lateral, and constructability analysis it is concluded that in the D.C. building market a post tension building is more economical and provides more rentable space. I feel the pre-cast building could be optimized with longer spans but this would result in larger floor to ceiling heights and a heavier building and ultimately this redesign was driven by building height

ACKNOWLEDGEMENTS

My parents for all the encouragement and support they have given me over the past 5 years. Also for giving me the opportunity to come to Penn State and pursue my dream of being an architectural engineer.

My Fellow Classmates the past 5 years would not have been the same without every single one of you. Thank you for all the good memories, encouragement and help you have given me over the past 5 years.

The Penn State University A.E Faculty for the endless hours of instruction over the past 5 years. Without your knowledge and expertise I would never have been able to grasp the concepts of structural engineering.

Dr. Memari my thesis mentor for taking the time to grade all my work and guiding me through the thesis experience.

I would like to thank *Davis Construction* for supplying the plans, cad files, and building information

Special thanks to *Brian Polesmak* of Davis Construction for taking time out of his schedule to answer any questions I had pertaining to the construction of the building.

Waild Choueiri from S.K.A. engineering for his insight into thesis topics and answering all questions I had about the current structural system and proposed pre-cast system.

EXISTING CONDITIONS

City Vista is a three building mixed used complex in downtown Washington D.C. Building 2, is strictly residential and contains 149 condos along with a community room, library, steel frame pedestrian bridge, and outdoor patio.

Building #2: This is the building under examination in this report.

Building 1A and 1B: The other buildings in the complex.

Figure #1 : Site Plan of City Vista

Figure #2 : Rendering of City Vista Building #2 courtesy of Torti Gallas and Partners,

This 11 story 324,298 square foot post tension building reaches a height of 110'-6" and 128'-5" at sections that include the mechanical penthouse. Construction of Building 2 began in 2005 and was completed in 2007. City Vista was designed by world renounced architects Tortis Gallas and Partners. The architecture uses strong vertical and horizontal elements of glass and brick. For example, the skyline is a powerful horizontal statement through the use of metal roof overhangs extending several feet past the building's façade. The horizontal elements are counteracted with protruding glass and crème colored vertical brick components throughout the building's façade. At ground level the building narrows for a more intimate feel.

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ARCHITECTURAL LAYOUT

Figure #3: Typical floor layout

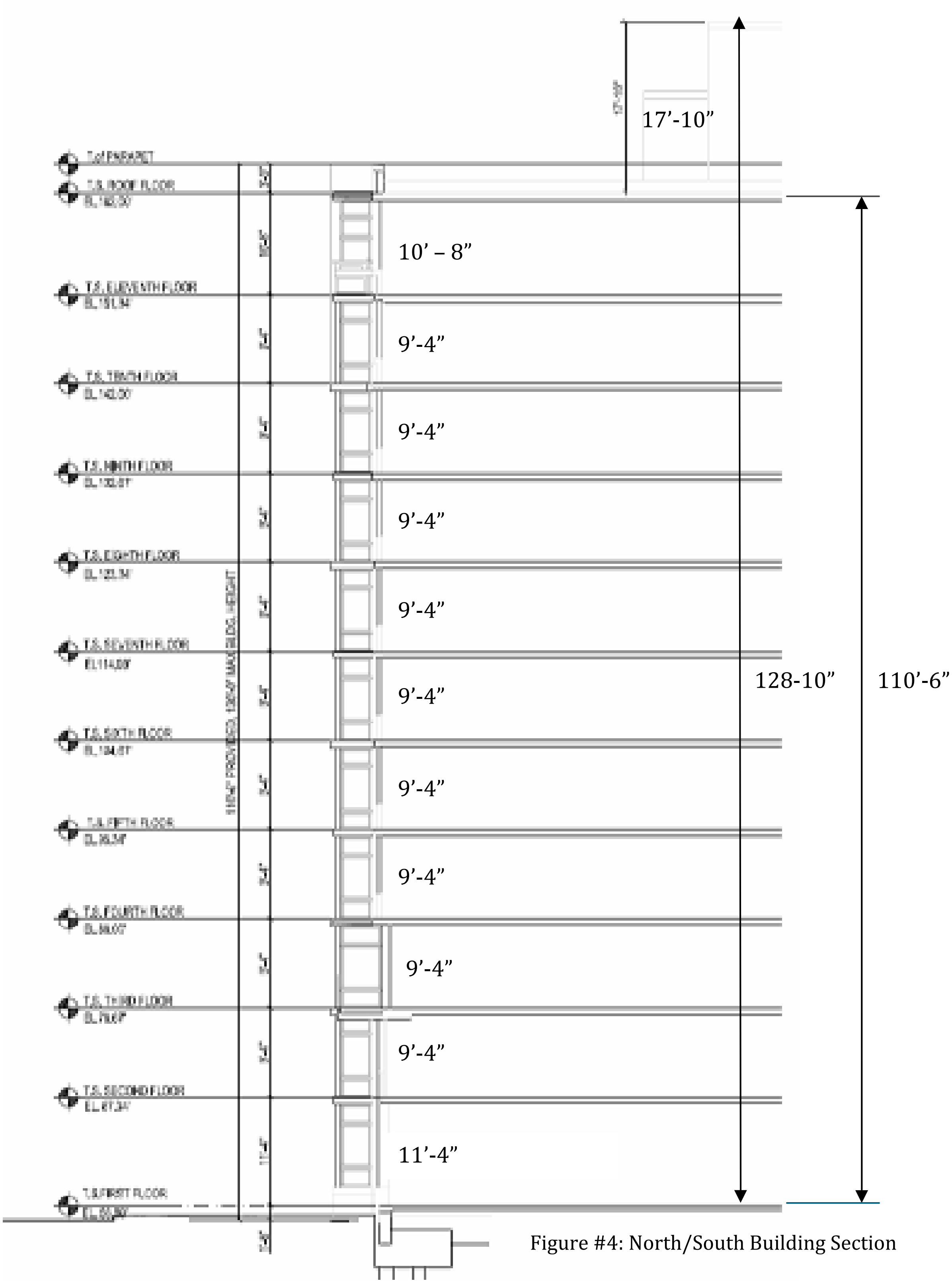


Figure #4: North/South Building Section

BUILDING ENVELOPE

Typical wall construction is 4" metal gauge studs, 1/2" sheathing, air space, and face brick, or floor to ceiling glass curtain walls. The penthouse uses 8" CMU back up wall, with a metal panel façade. Roof construction is a post tension roof slab, covered with ridged polystyrene insulation then sealed with a standing seam metal roofing system.

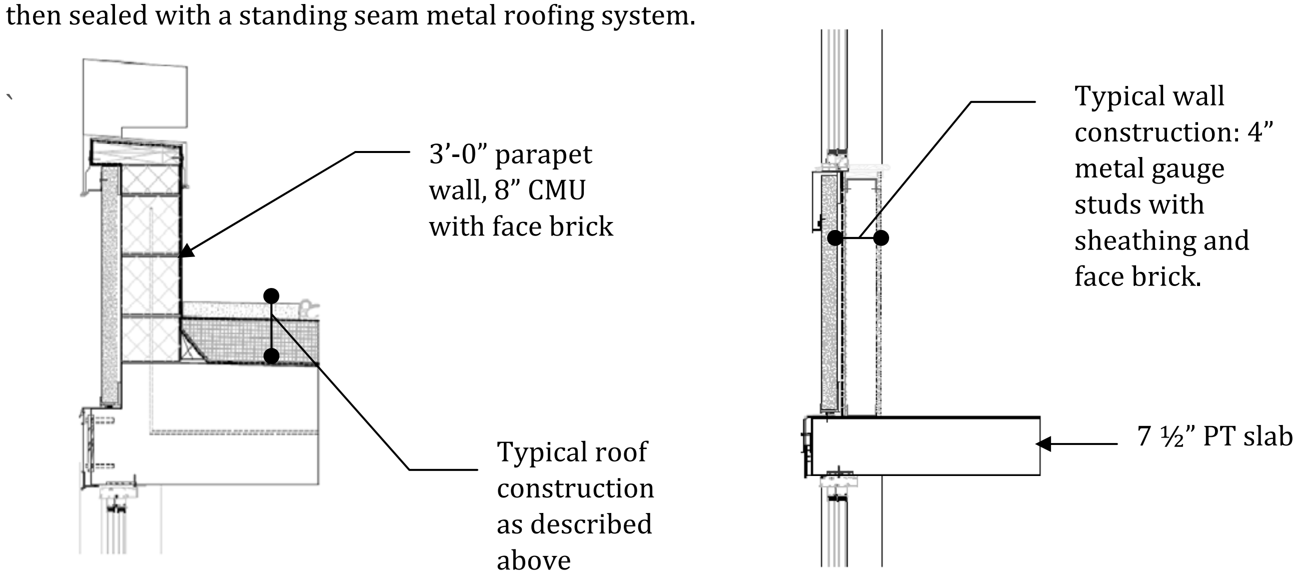


Figure #5: Typical Wall section at roof and floor

ELECTRICAL SYSTEM

Building 2 is fed by (2) 3000A 208/120V systems. The first system services the lobby via sub panels, and the second services the condos via a 3000 amp bus duct. The panel boards services corridor lighting, fire alarm equipment, and mechanical rooms. The bus duct feeds into a 120/208 volts, 3 phase, 600 amps, meter center at each floor and then is sent to individual 120/208 volt, 125 amp, 1 phase panel board located in each apartment. A 4 cycle 230 KW diesel emergency generator with a 50 gallon fuel tank is located on the roof. The generator operates the fire alarm system, fire pumps, elevators, and emergency lighting in the event of a power outage.

LIGHTING SYSTEM

All indoor public areas are lit by fluorescent 120V parabolic recessed lighting fixtures and wall sconces. The pedestrian bridge's architectural features can be seen at night by the 150W T10 halogen pendant fixtures. Condo units are equipped with a combination of fluorescent and incandescent surface and track lighting by Sea Gull Lighting. Fluorescent lighting is used in the kitchen of each condo, with a combination of (2) bulb 32W T-8 fixtures and track lighting with canopy style magnetic transformers. All other fixtures are incandescent with varying wattages depending on the rooms use.

MECHANICAL SYSTEM

On the roof are (2) 100% outside air rooftop units containing an indirect gas furnace, condensing unit, and compressor. Each unit is 208V/ 3 phase and pushed 4000 cfm of outside air. Public areas are serviced with 208V/ 3 phase split system heat pumps with cooling, reverse cycle heating and electrical heating capability. Each residential units contained its own through the wall gas heating units. This unit also used for cooling. Each individual unit is 208V/1 phase with variable CFM depending on the unit size.

FIRE PROTECTION

The building is 100% sprinkled with an automotive wet and dry sprinkling system. The dry system is located in areas that are not heated (i.e. loading deck). Stairwells walls have a 2 hr fire rating and are sprayed from top and bottom. Upon detection of smoke, water, and heat, the fire alarm system is activated and unlocks fire access doors, turns off corridor electrical units, and activates pressurized fans for towers. Building 2 construction is predominantly built with a 2 hour fire rating.

TRANSPORTATION

Building 2 has (2) elevators located in the building's central core. Both elevators are electric traction elevators with a capacity of 3,500 lbs. Elevators are controlled by the fire system and are equipped with emergency power. There are also (2) stairwells located at each end of the building.

CONSTRUCTION MANAGMENT

City Vista is a design-bid-build contract, with Davis Construction as the prime contractor. Truland Systems Corporation did all electrical and fire protection, work and Miller and Long performed all concrete work. Construction began in December 2005 and is set to end in December 2007. A tower crane was used during construction, whose footings double as a mat foundation for the steel pedestrian bridge connecting the 2 buildings.

CODE

Design of City Vista was govern by the following codes:

- *District of Columbia Building Code*
- ACI 318-89 : *Building Code Requirements for Structural Concrete*
- ACI 530-99 : *Building Code Requirements for Masonry Structures*
- *Post Tensioning Institute Standards*
- ASCE 7-05 *: American Society Of Civil Engineers 2005 edition*

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

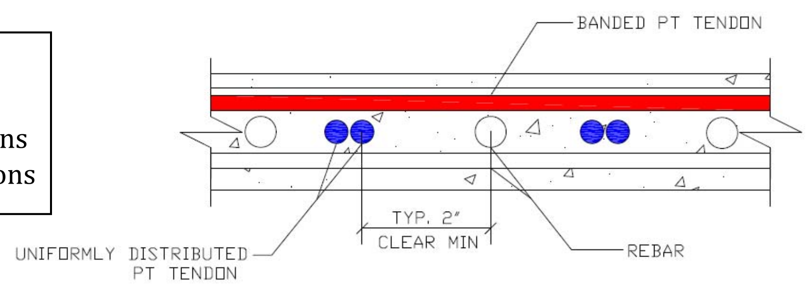


Figure #6: Typical floor section
Blue: Uniformly distributed tendon
Red: Banded Tendons

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).

<u>Shear Walls</u>*:* 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.

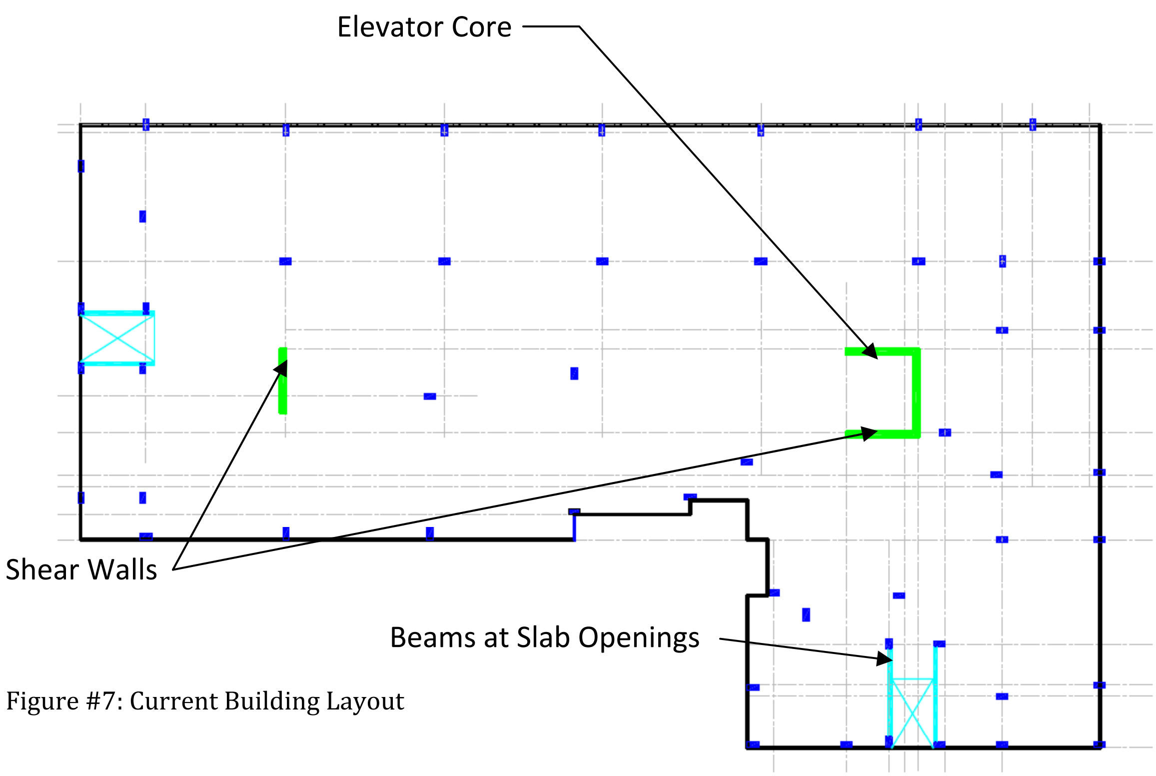


Figure #7: Current Building Layout

Advantages Post Tension:

1. Shallower floor plenum: Post tension building can reduce floor to ceiling height.
2. Reduction in materials: PT slabs require less rebar and are generally thinner so less concrete is needed.
3. Longer clear spans: PT slabs allow long continuous spans resulting in fewer columns which ultimately results in a lighter building.
4. Strength: Higher ultimate strength because of bond between concrete and strands.

Disadvantages Post Tension:

1. Cable Integrity: Cables can distress over time, although this is not a common problem
2. Remodeling: If the building is renovated in the future the floor slab can only be punctured once exact locations of tendons are known.
3. Shorting issues: As a result of PT ability to hold shrinkage cracks together tightly when shorting occurs large tension cracks can occur around perimeter of building.

PROBLEM

City Vista's presents designers with several challenges due to its design and location as a result many structural systems are not feasible for use at City Vista.

1. City Vista is located in Washington D.C where there is a height limit of 130 ft. Currently Building 2 is 110'-0" not including the mechanical penthouse. At sections that include the penthouse the building is 128'-6". It will be a challenge to stay within the 130' limit.
2. Because a flat plate system is used the underside of the floor slab is already a finished ceiling. When choosing alternate floor systems this should be taken into consideration.

When considering alternative structural solutions for City Vista, cost and construction time were as much a concern as floor to ceiling height. Washington D.C. construction business is very demanding and the faster and cheaper the construction the better.

PROPOSED SOLUTION

Taking into consideration the height restriction and competitive market, I propose a precast gravity system for City Vista. This system would consist of hollow precast floor planks with a slab thickness between 8-10 inches which keeps City Vista well within the height requirement. The current lateral system will be examined for stability after the gravity system is redesigned using precast inverted T-beams, hollow core planks and conventionally reinforced columns. Using a pre-cast system could potentially present faster erection time, a cheaper bottom line, possible LEED certification, and the same floor to ceiling height.

Advantages Pre-Cast:

1. Construction Time: Since pouring and curing is done offsite construction is significantly faster than other systems.
2. Efficiency of Members: Because members are poured and cured in controlled environments defects are limited.
3. Lightweight: Pre-cast components have good span to weight ratios, resulting in shallower foundation systems. And for this redesign the existing foundation will require little if any alterations.
4. Fireproofing: No fireproofing needed if pieces are designed with fire codes in mind.

Disadvantages Pre-Cast:

1. Lead Time: Since construction is done off site, a pre-cast system requires more planning so materials arrive on time.
2. Size Restrictions: Planks and beams come in standard sizes; therefore column grid will be restricted.

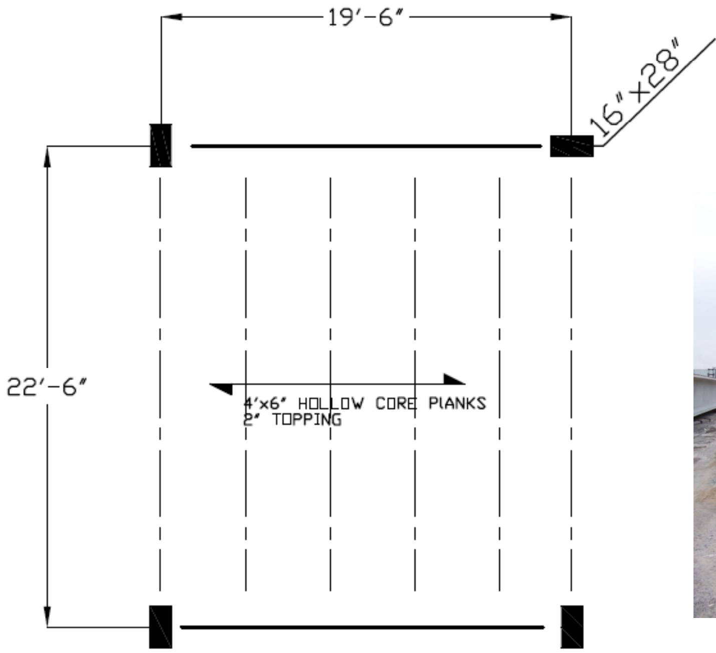


Figure #8 : Proposed typical bay for the new pre-cast system

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

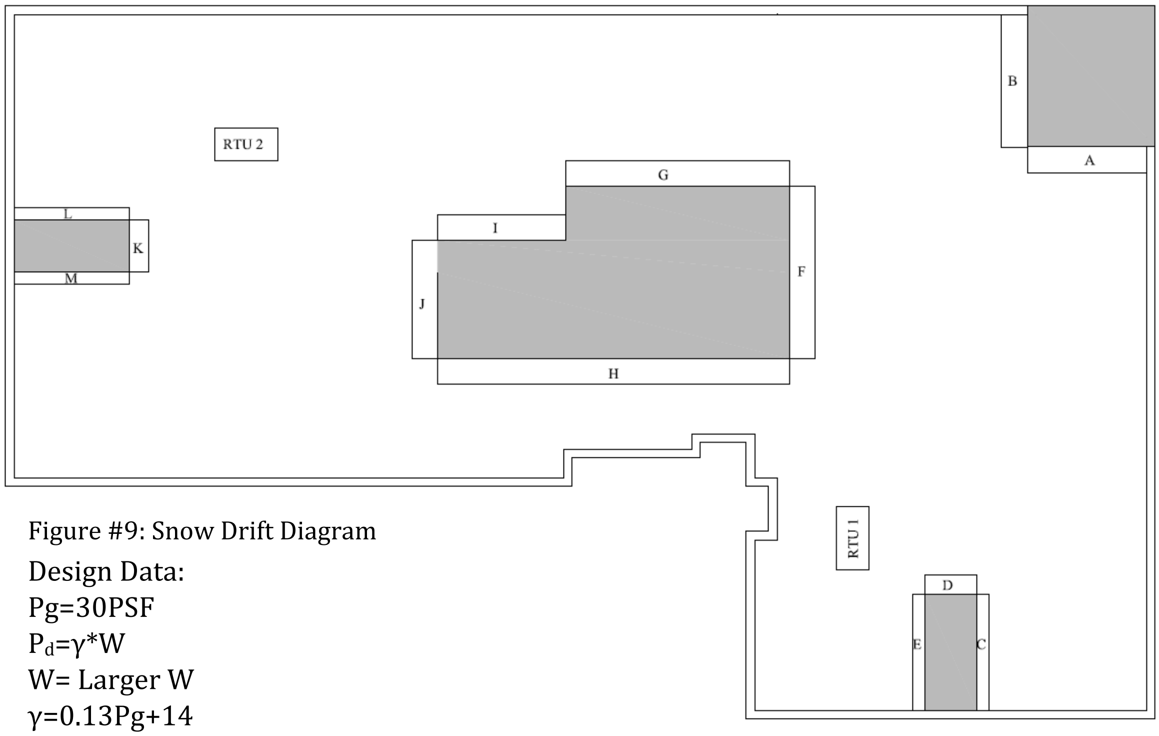


Figure #9: Snow Drift Diagram

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

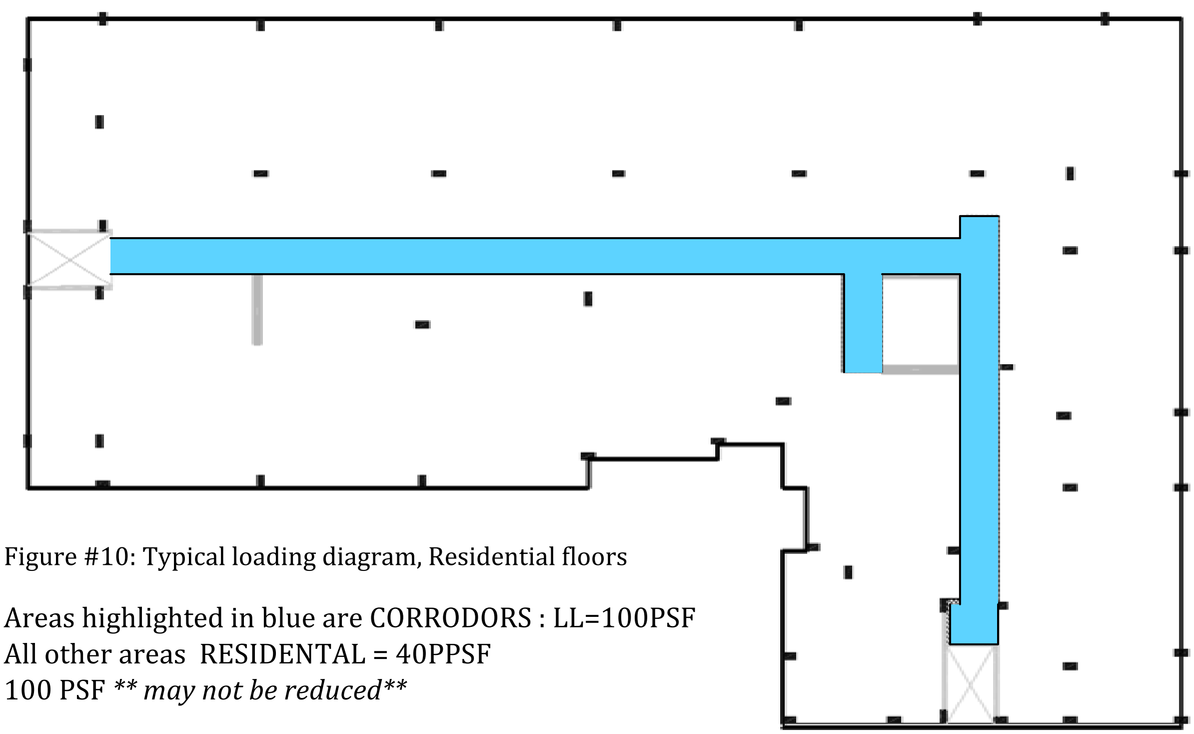


Figure #10: Typical loading diagram, Residential floors

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:

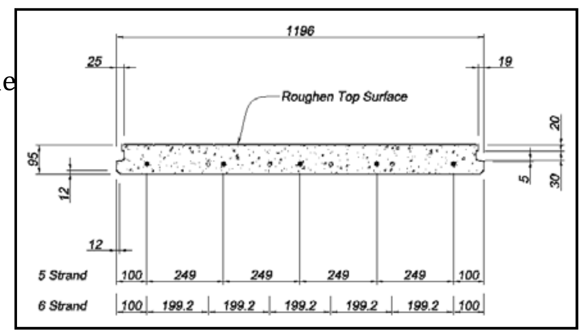
1. PCI Handbook 6th Edition
2. ACI 318-03
3. Several Manufacturers of Precast (Hanson, Nitterhouse, and Hollowcore)
4. PCI Manual for the design of hollow core slabs

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

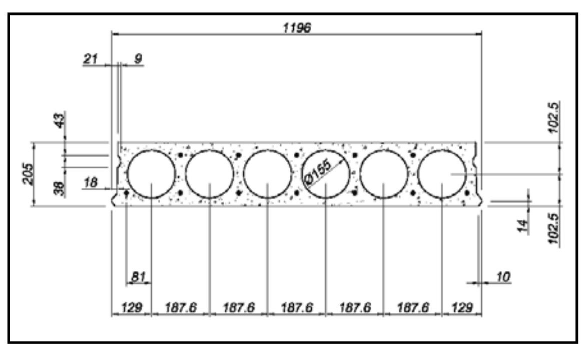
1. Hollow Core vs. Solid flat slab
2. Topping vs. No Topping
3. Ridged vs. Flexible Diaphragm
4. Expansion joints vs. No expansion joints
5. Tendons Considerations

1. Hollow core vs. solid flat slabs

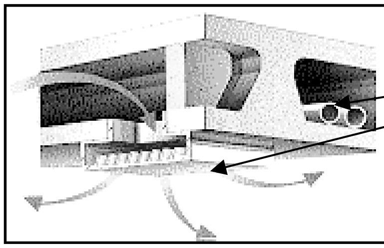
Solid flat slabs perform similar to cast in place solid slabs. Optimized when used for short spans of 12-24 feet and floor depth of 4-16 inches. This system provides a smooth underside so it can be used as a finished ceiling. The slabs have a fire rating of between ½ -2 hours depending construction. 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.

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.

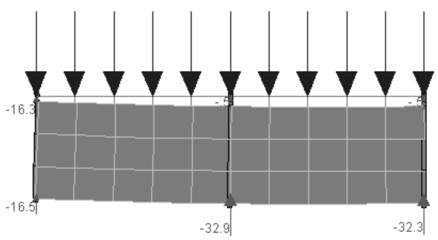


Figure #13. Ridged Diaphragm : You can see deformation of the diaphragm is limited. For a pre-cast system this is ideal.

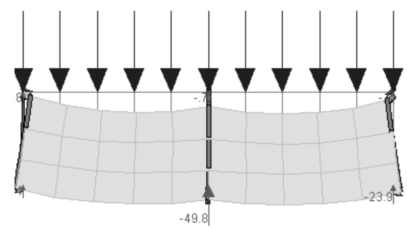


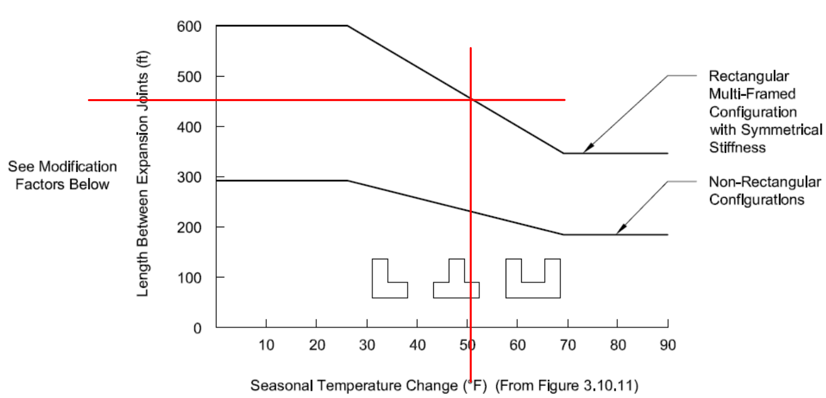
Figure #14: Flexible Diaphragm: You can see large deformations are experienced in the diaphragm.

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.

<u>Span to Depth Ratio</u>

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:

<u>Weight:</u>
No Permit: 20 Tons of Material
Permit: up to 100 Tons
Washington D.C.:

<u>Dimensions:</u>
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:
Wet Cast: Allows for many types of anchors because they are installed during casting.
Dry Cast: 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:

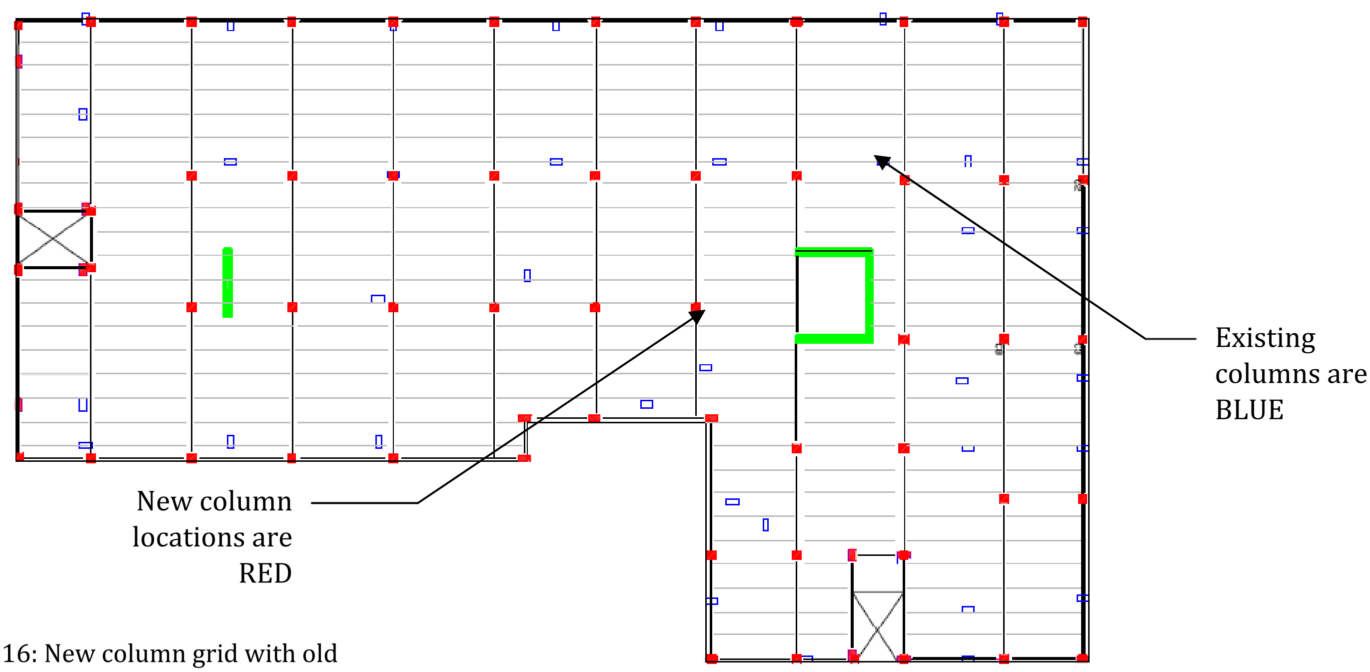


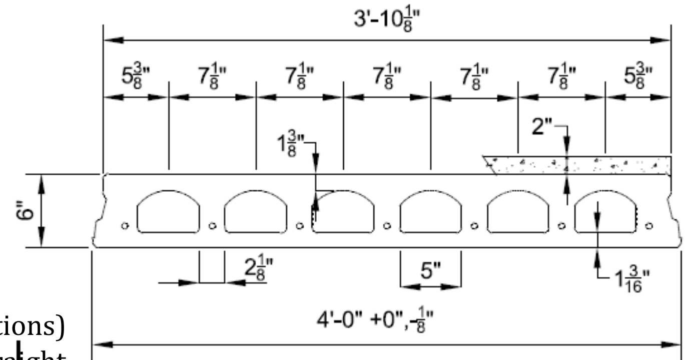
Figure #16: New column grid with old overlaid

MEMBER SELECTION:

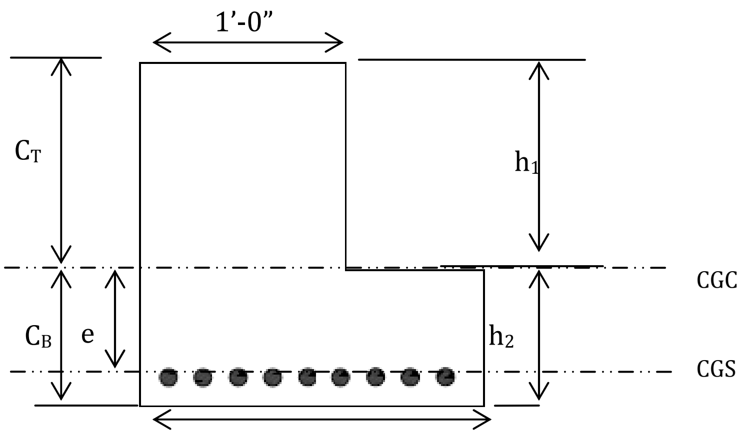
Hollow Core Planks:
Planks were selected from manufacturer Nitterhouse [www.nitterhouse.com]. This company was chosen because it services the D.C. area and has a good reputation for quality.
USE: 4' x 6" hollow core planks with 2" topping (2 hr fire rating) 7-1/2" Ø strands
(See Appendix #2 for Calculations)

PHYSICAL PROPERTIES

A_c=253 in^2	Precast S_{bc} = 370 in^3
I_c=1519 in^4	Topping S_{tc}= 551 in^3
Y_{bc}=4.10in	Wt=195plf
Y_{tc}=1.90in	Wt=48.75psf



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



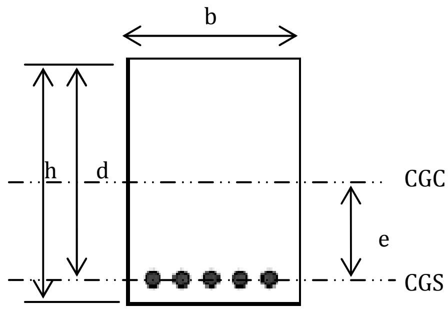
PROPERTIES 20LB20 :

A=304 in^2	S_b= 1,163 in^3
I=10,160 in^4	S_t= 902 in^3
h1=12 in	f'c=5000psi
h2=8 in	f_{pu}=270ksi
C_t = 11.26 in	A_{sp}= 9(0.153)= 1.377 in^2
C_b= 8.74 in	Po= 278.8 Kips
Wt=317 plf	
e =6.33 "	

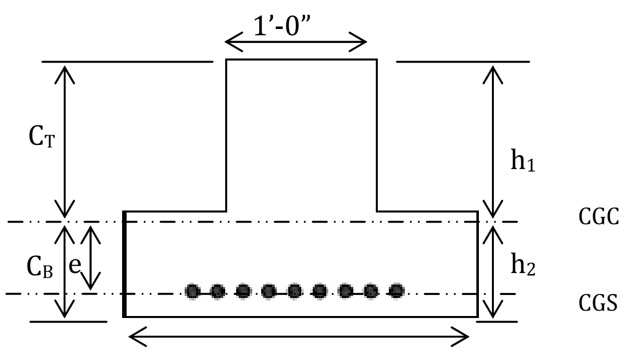
Beam Designation	Wu plf	TRAIL SIZE
LB-1	1147.3	20LB20
LB-2	1147.3	20LB20
LB-3	1147.3	20LB20
LB-4	1147.3	20LB20
LB-5	353.0	20LB20
LB-6	353.0	20LB20
LB-7	1558.5	20LB20
LB-8	1558.5	20LB20
LB-9	1270.8	20LB20
LB-10	1270.8	20LB20
LB-11	353.0	20LB20
LB-12	353.0	20LB20
LB-13	353.0	20LB20
LB-14	353.0	20LB20
LB-15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

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=192in^2
I=4096 in^4
h=16 in
S = 512 in^3
b = 12 in
Wt=200 plf
e = 5"
d=8"

f'c=5000psi
f_{pu}=270ksi
A_{sp}= 5(0.153)= 0.765 in^2
Po= 154.9Kips

PROPERTIES 28IT24 :

A=480 in^2
I=20,275 in^4
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 in^3
S_t= 1,408 in^3
f'c=5000psi
f_{pu}=270ksi
A_{sp}= 18(0.153)= 2.754 in^2
Po= 557.6 Kips

PROPERTIES 28IT20 :

A=368 in^2
I=11,688 in^4
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 in^3
S_t= 1,478 in^3
f'c=5000psi
f_{pu}=270ksi
A_{sp}= 9(0.153)= 1.377 in^2
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:

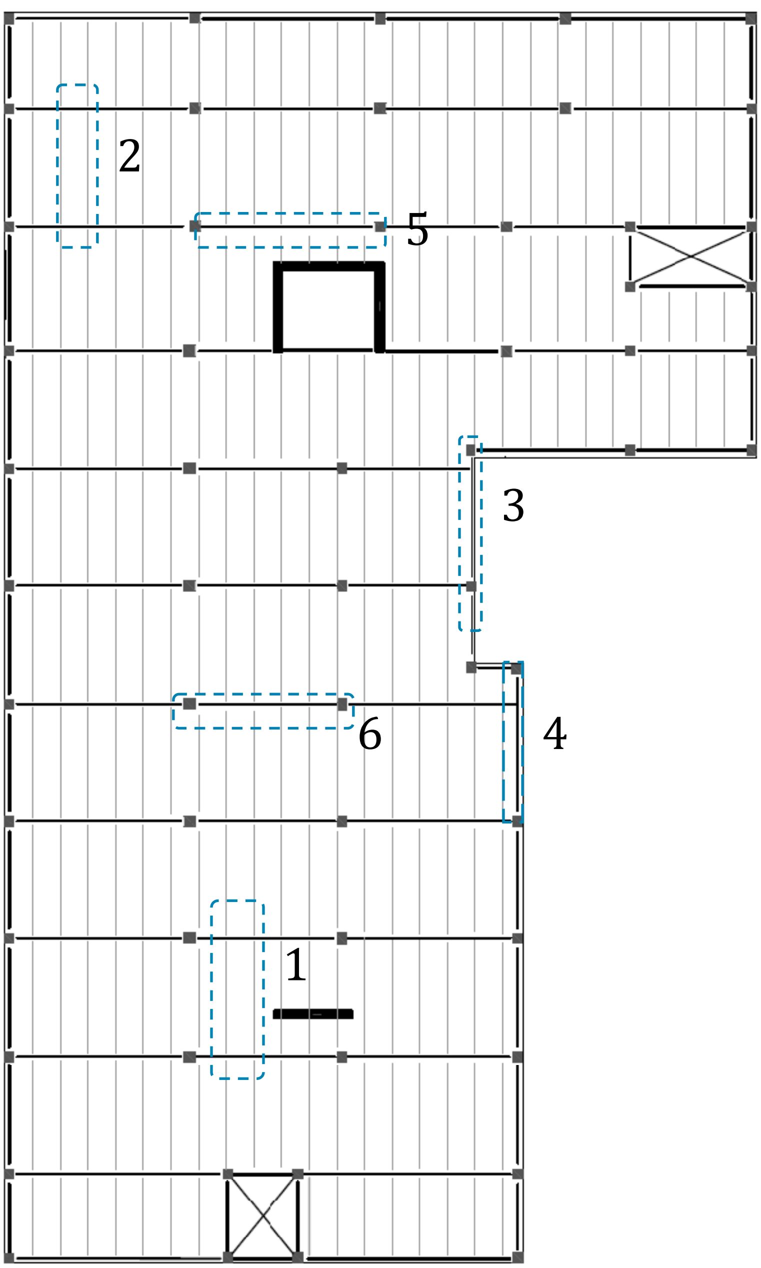
16"x16" | f'c =5000 psi | 4-#8 Bars

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.

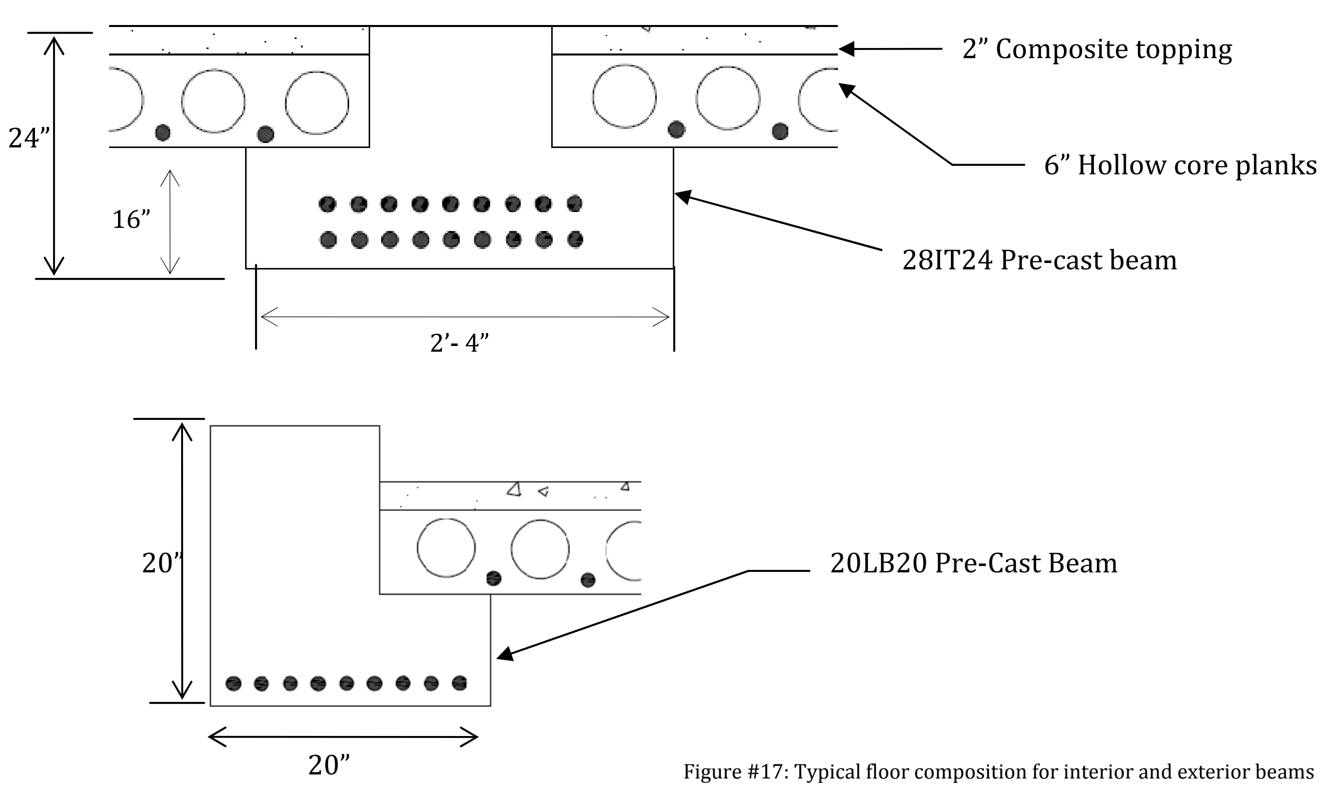


Figure #17: Typical floor composition for interior and exterior beams

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.

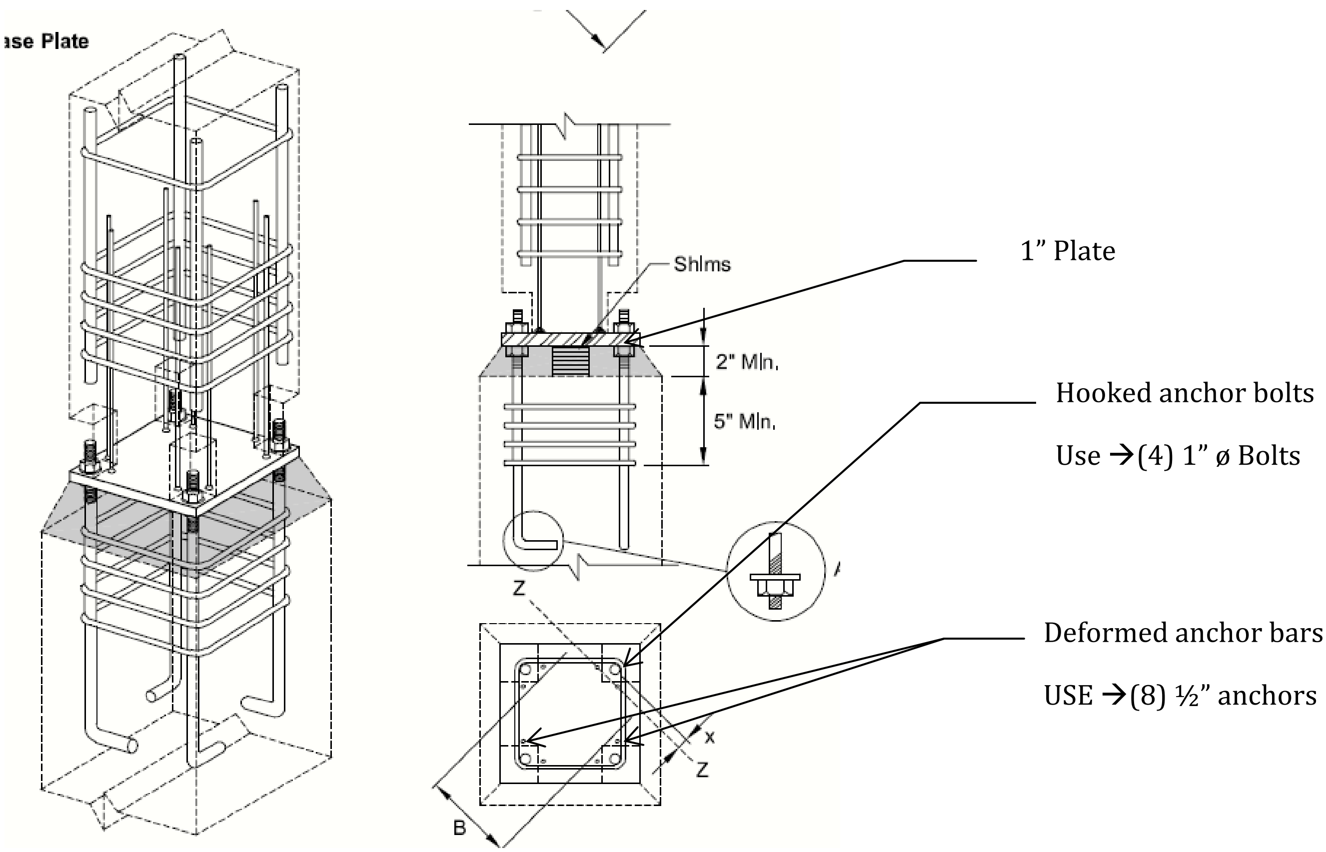


Figure #18 :Typical detail as per PCI 6th edition

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.

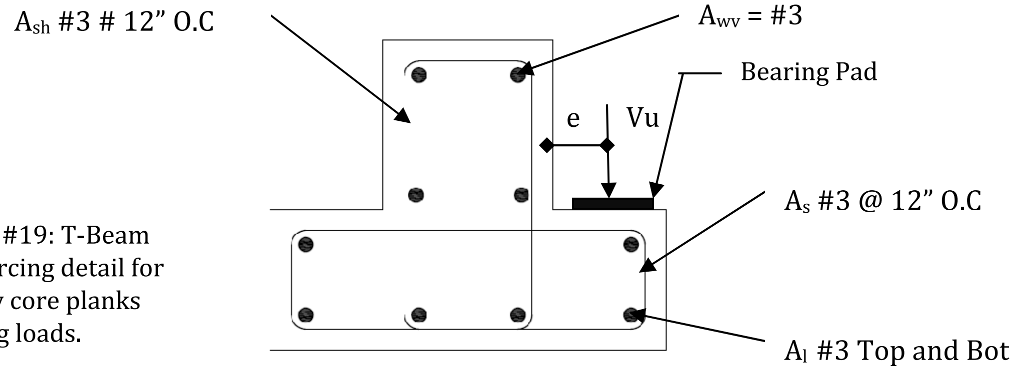
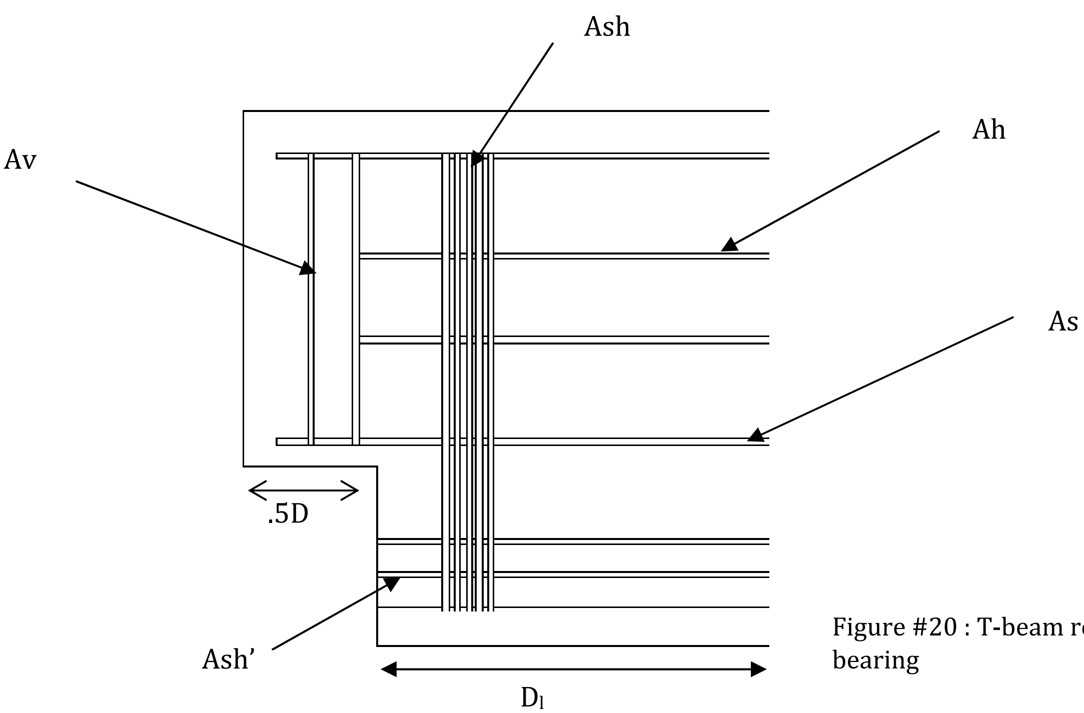


Figure #19: T-Beam Reinforcing detail for Hollow core planks bearing loads.

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

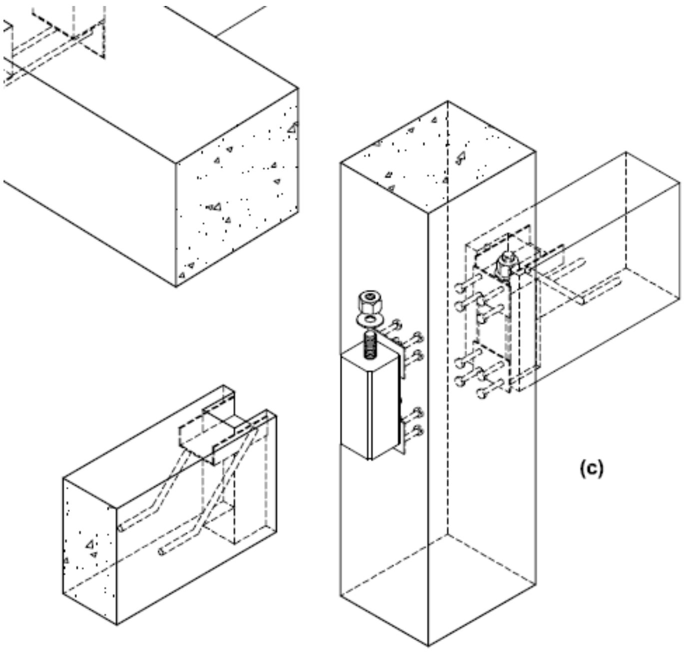


Figure #21 : Typical hanger detail as found in PCI 6th edition.

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.

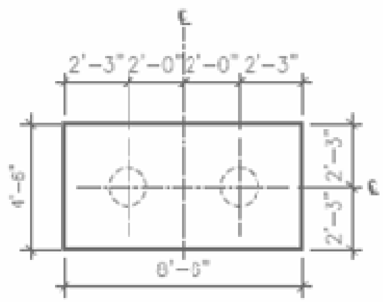
JULIE DAVIS
STRUCTURAL OPTION
APRIL 9, 2008

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

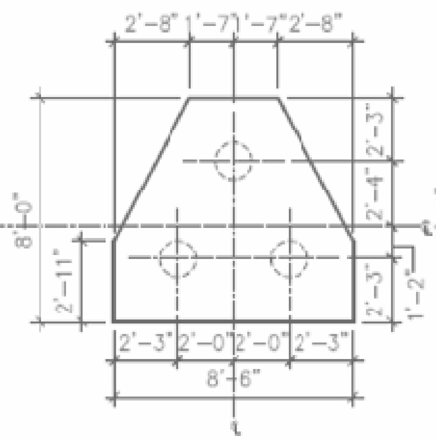
FOUNDATION CAPACITY:

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.

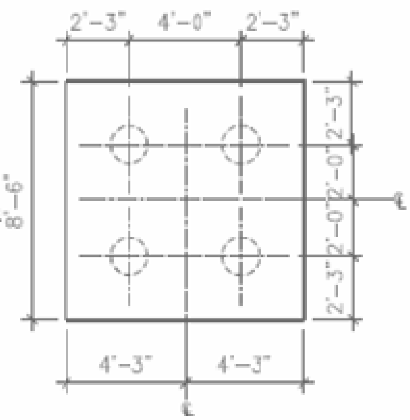
PT System



Pile Cap # 1



Pile Cap #2



Pile Cap #3

#1 : Column load 0-500 Kips

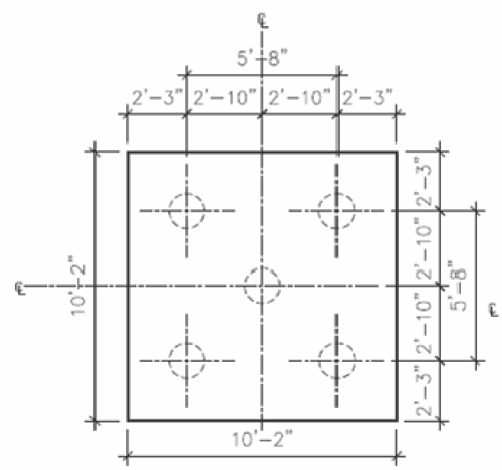
#2 : Column load 0-700 Kips

#3 : Column load 700-999 Kips

#4 : Column load 1000-1030 Kips

<u>Foundation Capacity Numbers</u>

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



Pile Cap #4

Pre-cast System,

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]

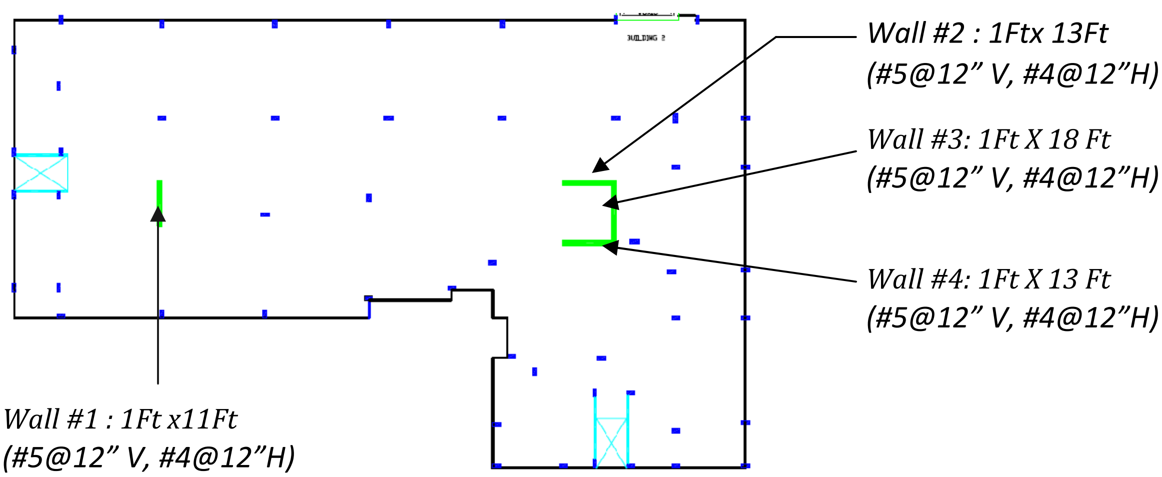


Figure #22: Shear Wall Locations, Size, and Reinforcing

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: $(0.9 - 0.2S_{ds})D + \rho E + 1.6H$

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STRUCTURAL OPTION
APRIL 9, 2008

FLEXURE CHECK

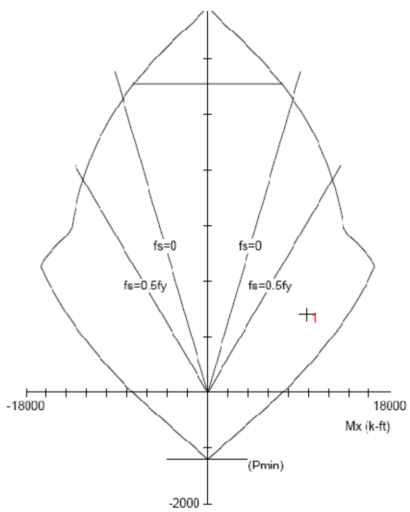

Figure #23: Interaction Diagram for Shear Wall #2 & Wall #4
Pu = 1,401.9 Kips
Mu= 9,806.8 Kip-ft
COMBO 50
Reinforcing : #8 @ 12" O.C (Floor 1-2)
#5 @ 12" O.C (floors 3-11)

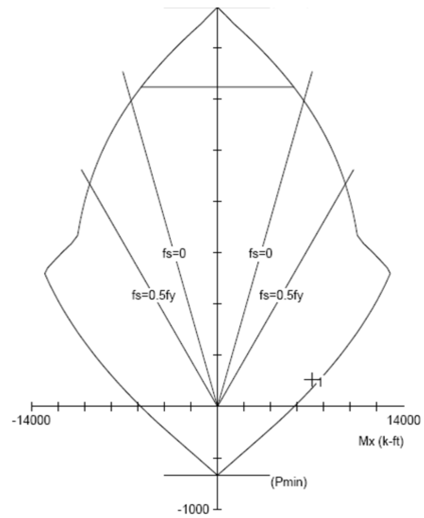

Figure #24 : Interaction Diagram for Shear Wall #3
Pu= 264.57 Kips
Mu=7,144 Kip-ft
COMBO 48
Reinforcing: #5 @ 12" O.C

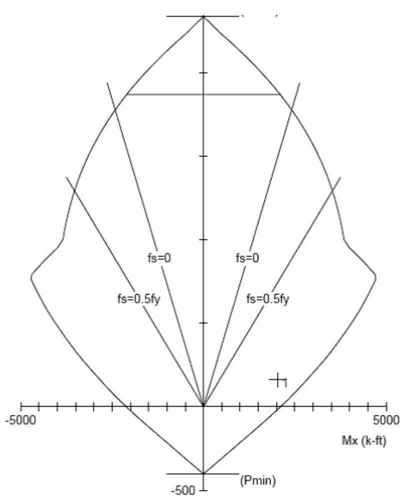

Figure #25 : Interaction Diagram for Shear wall # 1
Pu= 162.3 Kips
Mu= 2,035.7 Kip-Ft
COMBO 48
Reinforcing: #5 @ 12" O.C

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

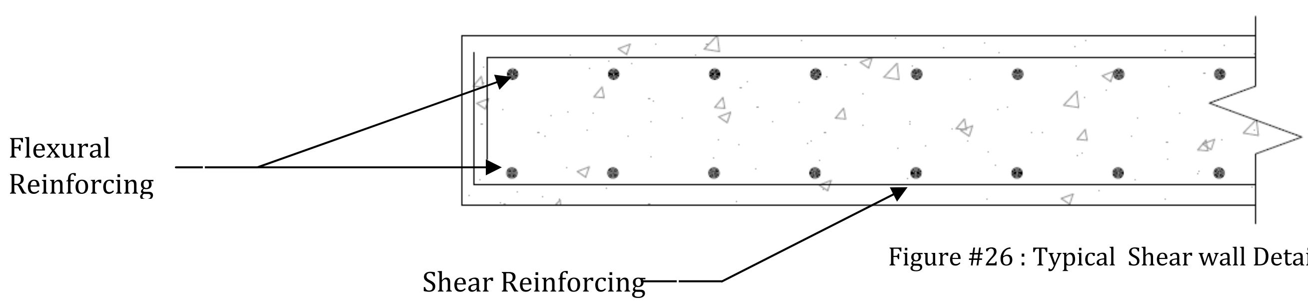

Figure #26 : Typical Shear wall Detail

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.

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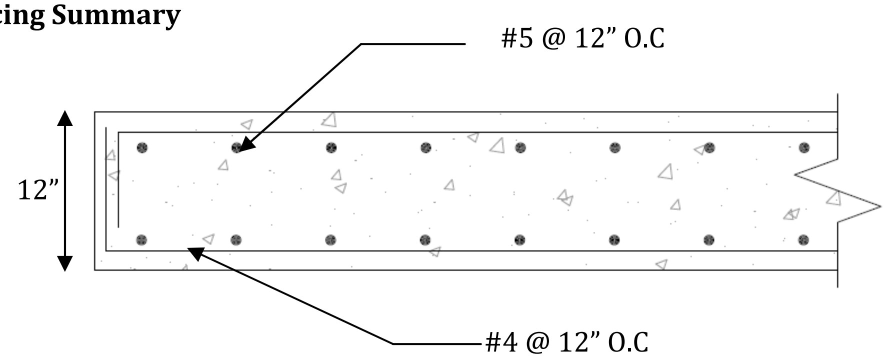
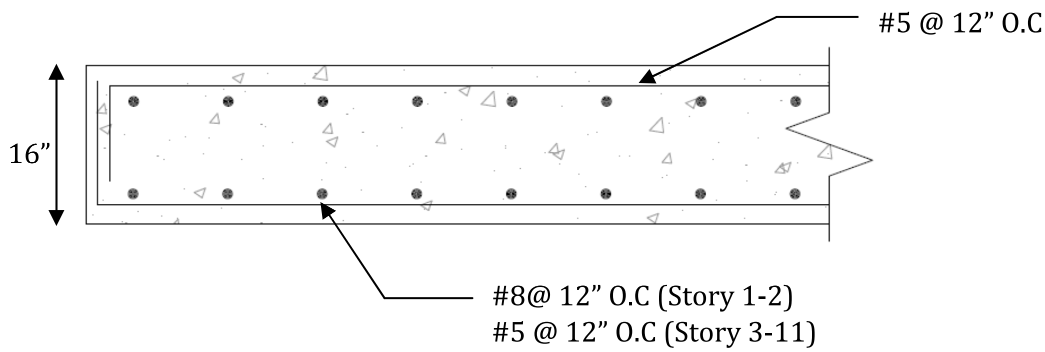
Wall Reinforcing Summary

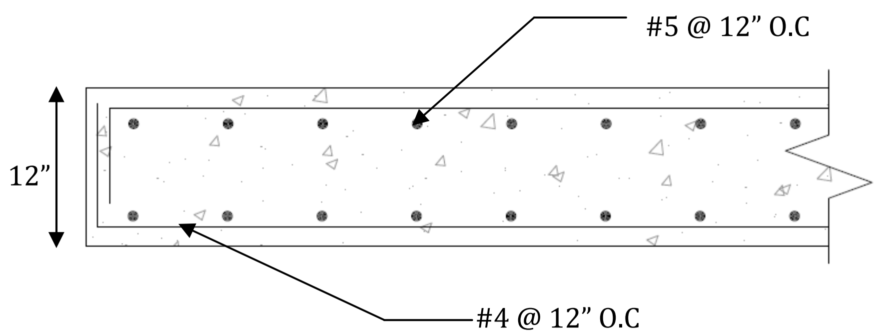
Wall #1 :



Wall #2&4:



Wall #3 :



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

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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 = $(\delta_e\text{-}\delta_e)C_d/I_e \leq \Delta$

	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

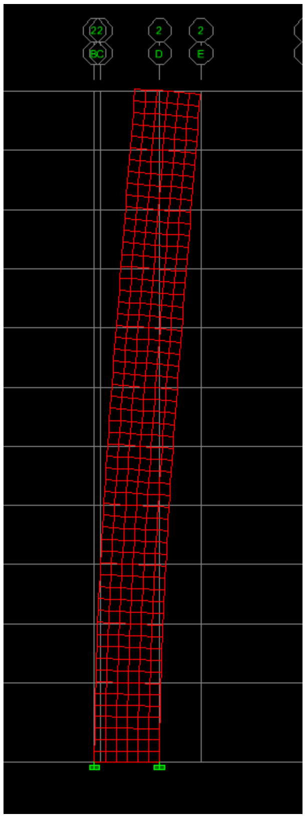
	Wall #2&4			Quake Y Direction		
Story	Story Drift	δei	δi	Total story Drift (δt)	Δb	δt≤Δb
11	0.54	2.43	1.89	3.78	2.40	NG
10	0.53	2.39	1.86	3.71	2.40	NG
9	0.52	2.34	1.82	3.64	2.40	NG
8	0.51	2.30	1.79	3.57	2.40	NG
7	0.48	2.16	1.68	3.36	2.40	NG
6	0.44	1.98	1.54	3.08	2.40	NG
5	0.43	1.94	1.51	3.01	2.40	NG
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**

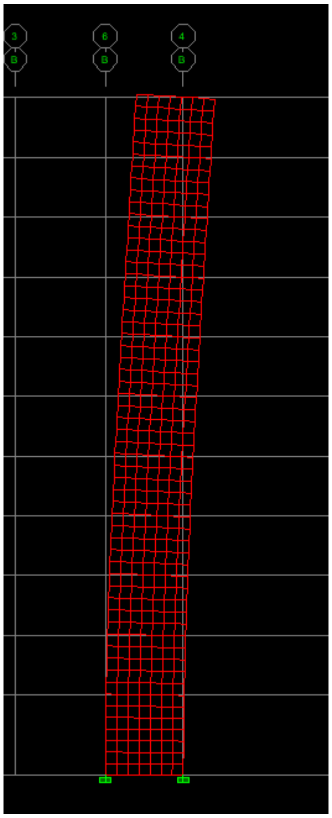
Y = 3.78 in NG

Displacement Summary



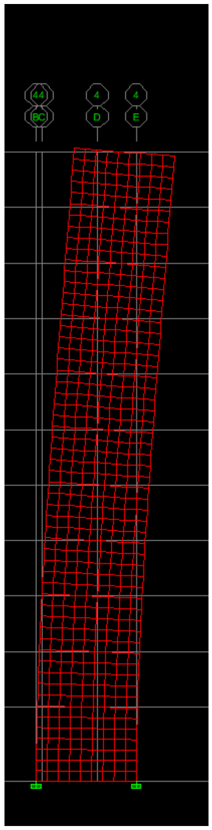
Wall #1 : Deformed shape under seismic forces in the X direction.

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.

BREADTH#1: CONSTRUCTABILITY

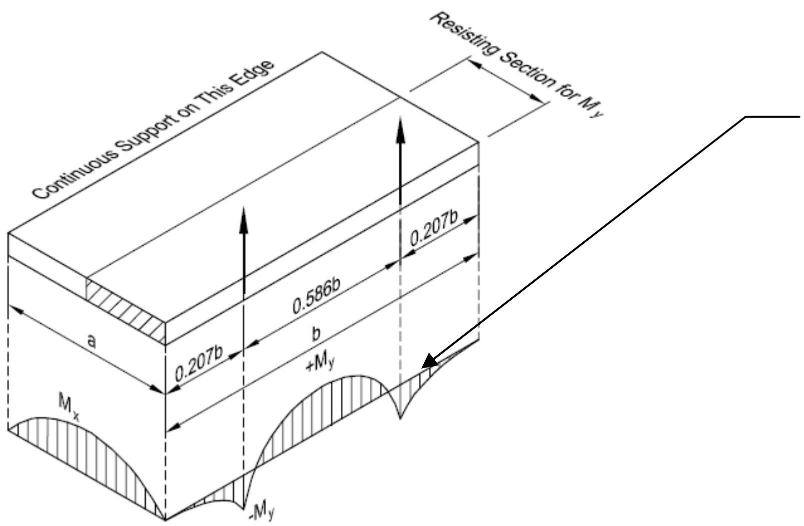
There are several constructability issues with the redesign of City Vista.

1. *ERECTION:* With pre-cast the erection of the structural system is much different from post tension. Members are set in place with a crane. This process caused forces in the member which can sometimes affect the design.
2. *LEEDS:* Pre-cast concrete allows for easier obtainment of a Leeds rating. An in-depth analysis was not performed, although advantages are discussed showing that LEEDS certification is more feasible with a pre-cast building.
3. *COST*: A Cost analysis was done to compare the gravity system cost of the PT and pre-cast system.
4. *SCHEDULING:* A simple schedule was also assembled to show potential time savings the precast system could provide.

ERECTION

Currently a saddle jib tower crane is being used at City Vista. After examining the cut sheets it is sufficient for erection of the pre-cast members.

Concerns:
Design of pre-cast members is influenced by storage and stripping, the number of pick points and location of the crane. All these variables create forces in the member need to be considered during design. For example the figure below shows a two point pick using a spreader beam. Predominantly line lifts will be used to assemble City Vista since all components have long spans and thin depths. As a result the inclined lines created by the two pick point creates a moment due to pΔ affects. Eccentric moments created by picks not at the center of the member are also an issue. As a safe practice a minimum **safety factor of 1.2** is applied to all pre-cast products, this factor accounts for stripping and dynamic forces.



Here is an example of the forces that can be seen during erection of the gravity system.

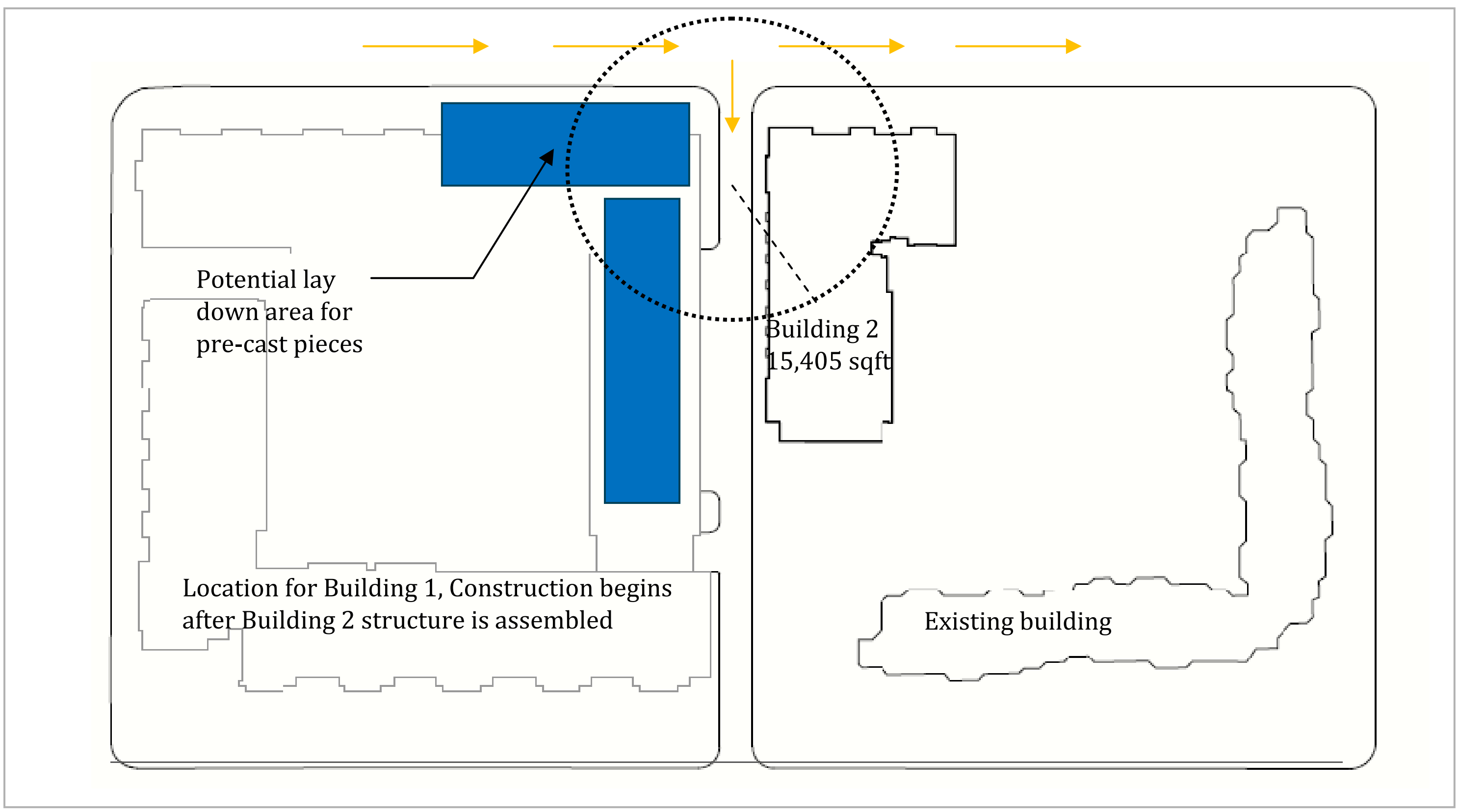
As shown in the figure a moment is seen in the component from lifting. For example a 29IT24 beam, self weight : 500plf , length: 26ft.

M_{xmax} = 49 in-kips

M_{ymax} = 46.73 in-kips

Although these moments are insignificant when compared to the flexural moments, although it could be a concern

Smooth erection at City Vista**:** Building 2 superstructure is erected before Building 1, as a result a lay down area is available (see diagram below). Currently the crane is located between the two buildings on the pedestrian bridge footings which double as crane footing.



LEED POTENTIAL

LEEDS ASSESMENT

Sustainability is the development that meets the needs of the present without compromising the ability of future generations to meet their own needs.

Pre-Cast concrete is a sustainable practice because it uses:

1. Integrated design
2. Materials efficiency
3. Reduces waste, site disturbance, and noise

Integrated design, when you examine a building as a whole not as individual parts. By doing this you can concentrate on energy efficiency, durability, environmental impacts, and cost.

Material efficiency is the combination of reducing energy and emissions created by building materials.

Reductions, is reducing the amount of material and toxic waste created when buildings are built.

WHY?

- *Operation Cost:* $0.60-1.50 sqft vs. $1.80 sqft of conventional buildings.
- Lower energy cost translates into smaller cooling equipment → lower first cost for equipment.
- Green design first cost ranges from 0-2% more than conventional buildings.
- This 2% increase → 10 times the initial cost in operation cost.

HOW?

- Material savings when precast panels are used for interior walls. This eliminates the need for drywall and additional framing.
- Eliminate duct work when hollow core planks voids are used as ducts.
- Concrete is a durable material therefore reducing maintenance.

LEED RATING AT CITY VISTA:

As discussed above a pre-cast building can obtain 23/26 points required for green certification, but exactly how is this accomplished by simply changing the method of concrete casting from onsite post tension to offsite pre-tension.

- **Material Recourses:** Precast components can be reused when building is renovated or demolished, reducing air and land pollution caused by demolition. Corrosion resistance which in return means less maintenance. This is because precast is made under ideal circumstances so things like steel cover are carefully monitored.
- **Sustainable site:** The heat island effect is minimized by concrete because pre-cast concrete provides a reflective surface.
- **Production:** Pre-cast plants create little waste. About 2.5% of the volume used in production is disposed of, and 95% of the water used is reused for other process. Steel formworks are also reused over and over again.
- **Recycled Content:** Concrete is a recycled material, and reinforcing bars are 90% recycled material. 95% of the waste water and steel formworks are reused

- **Energy:** Hollow core plank voids can be used as a passive solar system. This can be done by using the voids themselves as ducts changing the planks thermal mass. Energy reduction during production can be accomplished through the use of slag cement or silica fume. These items would be waste if not utilized in concrete products.
- **Local Materials:** Most suppliers are within 200 miles of the site.
- **Reuse:** At the end of the useful life of the building pre-cast pieces can be unassembled and reused.

LEEDS SUMMARY	
Sustainable Site	
Site Development, restore habitat	1
Site Development, maximize open space	1
Heat island effect	1
Energy and Atmosphere	
Prerequisite: Minimum Energy Performance	
Optimize Energy Performance	1-10
Material and Resources	
Reuse, maintain 75% existing shell	1
Reuse, maintain 25% existing shell	1
Construction waste management divert 50% by wt. or vol.	1
Construction waste management divert 75% by wt. or vol.	1
Recycled Content (10% of material on project, based on cost)	1
Recycled Content (20% of material on project based on cost)	1
Local/Regional material (minimum of 10%, based on cost)	1
Local/Regional material (minimum of 20%, based on cost)	1
Indoor Environmental Quality	
Construction Indoor air quality , during construction	1
Innovation and Design Process	
High volume supplementary cementious materials	1
Apply for other credits demonstrating performance	1
Apply for other credits demonstrating performance	1
Apply for other credits demonstrating performance	1
LEED accredited professional	1
TOTAL	23

Figure 27 #: LEED Checklist for pre-cast building , courtesy of www.PCI.org

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APRIL 9, 2008

COST ANALYSIS:

Pre-Cast Gravity System Cost Analysis:

Pre-Cast System — Page 1 of 1

Quantity	Assembly Number	Description	Unit	Ext. Material O&P	Ext. Installation O&P	Extended Total O&P	Labor Type
84.85	B10102152200	Concrete beam, precast, 12" x 16", 200 PLF, 15....	L.F.	$ 4,072.80	$ 1,654.58	$ 5,727.38	Standard Union
84.847	B10102152800	Concrete beam, precast, 12" x 16", 200 PLF, 20....	L.F.	$ 5,387.78	$ 1,092.83	$ 6,480.61	Standard Union
510	B10102144600	Concrete T beam, precast, 24" x 20", 360 PLF, ...	L.F.	$ 111,180.00	$ 8,445.60	$ 119,625.60	Standard Union
272.5	B10102145750	Concrete T beam, precast, 24" x 20", 865 PLF, ...	L.F.	$ 60,495.00	$ 5,014.00	$ 65,509.00	Standard Union
70.97	B10102152300	Concrete I beam, precast, 12" x 20", 300 PLF, 1....	L.F.	$ 8,232.52	$ 1,305.85	$ 9,538.37	Standard Union
224.58	B10102152900	Concrete I beam, precast, 12" x 20", 300 PLF, 2....	L.F.	$ 26,051.28	$ 3,096.96	$ 29,148.24	Standard Union
235.94	B10102154000	Concrete I beam, precast, 12" x 28", 435 PLF, 2....	L.F.	$ 32,559.72	$ 2,597.70	$ 35,157.42	Standard Union
29,481.64	B10102302900	Precast concrete plank, 2" topping, 8" total thic....	S.F.	$ 240,275.37	$ 135,910.36	$ 376,185.73	Standard Union
940	B10102071500	Precast concrete column, 20" sq, tied, eccentric....	V.L.F.	$ 81,780.00	$ 8,630.60	$ 90,410.60	Standard Union
200	B10102071350	Precast concrete column, 16" sq, tied, eccentric....	V.L.F.	[illegible]	[illegible]	[illegible]	Standard Union
				$ 583,934.47	$ 169,412.48	$ 753,346.95	

Pre-Cast System : a typical floor

Materials: $583,934.00
Installation: $ 169,412.00

TOTAL : $ 753,347.00 / FLOOR

Analysis was done using the program cost works by RSMeans. Values were drawn from Commercial/ New construction cost book released in 2008. A stand union labor was assumed and no mark ups were included.

Total Cost :

[$753,347 * 6] + [$647,372*5] =

** The two different floor prices take into consideration the double height columns **

Approx. TOTAL = **$ 7,756,942.00**

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Post Tension Gravity System cost Analysis:

PT System — Page 1 of 1

Quantity	Assembly Number			Description	Unit	Ext. Material O&P	Ext. Installation O&P	Extended Total O&P	Labor Type
324,218	B1010224200			Flat plate, concrete, 7" slab, 18" column, 20'x20...	S.F.	$ 1,637,704.90	$ 2,390,076.26	$ 4,027,781.16	Standard Union
5,920	B1010203450			Cast-in-place concrete column, 24" square, tied...	V.L.F.	$ 361,608.00	$ 761,036.64	$ 1,122,644.64	Standard Union
						$ 1,999,312.90	$ 3,151,112.90	$ 5,150,425.80	

<u>PT. System</u> : a typical floor

Materials: $18,120.00
Installation: $ 286,465.00

TOTAL : $ 604,584.00 / FLOOR

Analysis was done using the program cost works by RSMeans. Values were drawn from Commercial/ New construction cost book released in 2008. A stand union labor was assumed and no mark ups were included.

Approx. TOTAL = **$ 5,150,425.00**

The post tension system cost considerably less. This is due to the additional beams needed to support the hollow planks. The post tensioned slab and planks with topping price is competitive with one another. The same can be said when comparing the pre-cast and cast in place conventionally reinforced columns. Economically a post tensioned flat plate building is considerably cheaper.

SCHEDULE

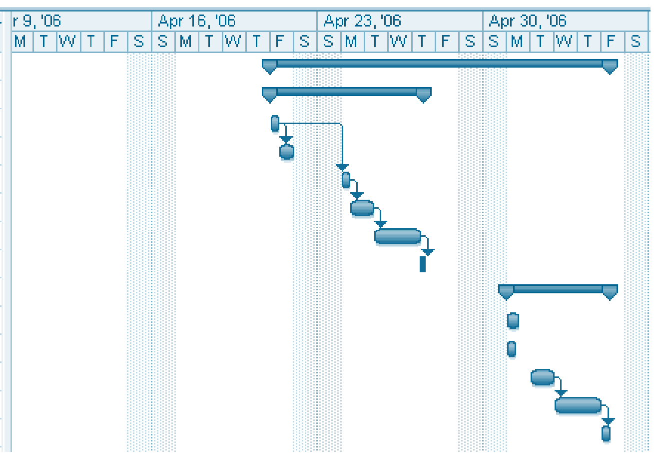
Pre-Cast System:
Typical erection of pre-cast is 400m^2 /week of pre-cast = 62,000 in^2 /week

Typical Floor :

Member	Quantity	Total Area
Column #1 [20x20]	24	66.69 in^2
Column #2 [16x16]	10	17.68 in^2
Column #3 [24x24]	23	92 in^2
L-Beams	31	9424 in^2
T-Beams	34	16 320 in^2
R-Beams	9	1728 in^2
Planks	200	50,600 in^2
TOTAL		**78,249 in^2**

A two floor schedule was done in Microsoft project to reflect this erection pace, while taking into consideration the double floor column height. This analysis shows erection pace of 2 floor in ***9 Days.***

Task Name	Duration	Start	Finish	Predecessor
Structural System	**10.25 days?**	**Fri 4/21/06**	**Fri 5/5/06**	
Floor 1	**4.5 days**	**Fri 4/21/06**	**Thu 4/27/06**	
Columns	0.25 days	Fri 4/21/06	Fri 4/21/06	
L-Beams	0.75 days	Fri 4/21/06	Fri 4/21/06	3
R-Beams	0.25 days	Mon 4/24/06	Mon 4/24/06	3
T-Beams	1 day	Mon 4/24/06	Tue 4/25/06	5
Hollow Core Planl	2 days	Tue 4/25/06	Thu 4/27/06	6
Topping	0.25 days	Thu 4/27/06	Thu 4/27/06	7
Floor 2	**4.25 days?**	**Mon 5/1/06**	**Fri 5/5/06**	
L-Beams	0.5 days?	Mon 5/1/06	Mon 5/1/06	
R-Beams	0.25 days	Mon 5/1/06	Mon 5/1/06	
T-Beams	1 day?	Tue 5/2/06	Tue 5/2/06	
Hollow Core Planl	2 days	Wed 5/3/06	Thu 5/4/06	12
Topping	0.25 days	Fri 5/5/06	Fri 5/5/06	13



Pre-Cast erection saves about 5.5 days in the schedule. Not a significant difference when considering the higher cost of the system.

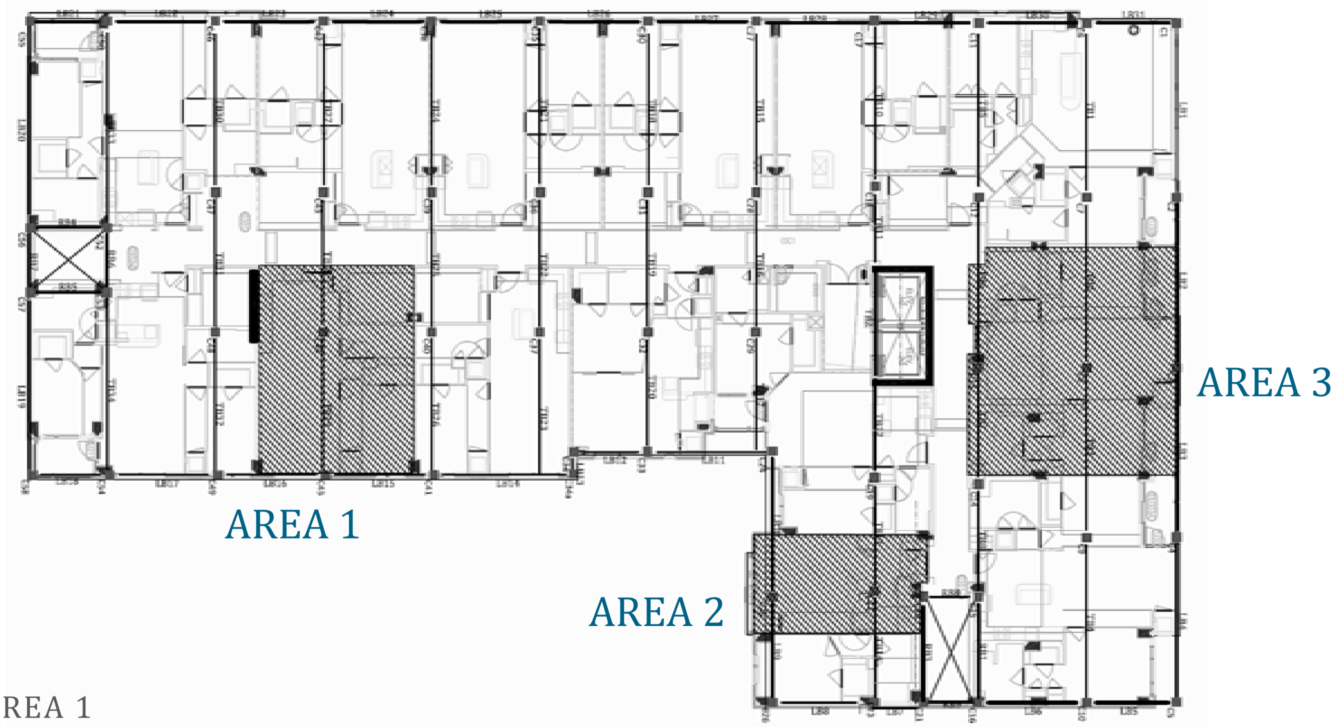
Post Tension System: Courtesy of Davis Construction

Activity ID	Activity Description	Orig Dur	Rem Dur	Early Start	Early Finish	Total Float
Building 2						
Concrete Structure						
Level 1						
02010010	Bldg 2 - Sitework / Foundations & SOG	80*	80*	29DEC05	20APR06	0
02010020	Install Test Piles	5	5	29DEC05	05JAN06	0
02010022	Install Test Probes, Test reports & Mobilize	14	14	06JAN06	25JAN06	0
02010024	Install Tower Crane Foundation	8	8	09FEB06	20FEB06	36
02010028	Erect Tower Crane	2	2	21FEB06	22FEB06	36
02010030	Start Auger Cast Piles - Bldg 2	0	0	26JAN06		0
02010035	Install Auger Cast Piles	20	20	26JAN06	22FEB06	0
02010040	Start Concrete Foundations - Bldg 2	0	0	23FEB06		0
02010045	F,R&P Pile Caps & Grade Beams	25	25	23FEB06	29MAR06	0
02010050	Selective Demo/Cut Site	25	25	24FEB06	30MAR06	0
02010055	Foundations / Slab on Grade - Bldg 2	41*	41*	23FEB06	20APR06	0
02010060	F,R&P Foundation Walls & Cols	25	25	03MAR06	06APR06	0
02010070	Backfill Foundation	10	10	10MAR06	06APR06	0
02010080	Rough-in Underground Plumbing	10	10	10MAR06	06APR06	0
02010085	Inspect Underground Plumbing	5	5	07APR06	13APR06	0
02010090	Rough-in Underground Electric	10	10	10MAR06	06APR06	0
02010095	Inspect Underground Electric	5	5	07APR06	13APR06	0
02010100	Prep & Pour Slab-on-Grade	5	5	14APR06	20APR06	0
02010110	Slab-on-Grade Complete - Bldg 2	0	0		20APR06	0
Level 2						
02020090	Start Concrete Structure - Bldg 2	0	0	21APR06		0
02020095	Concrete Structure (1st - Roof) - Bldg 2	60*	60*	21APR06	17JUL06	0
02020100	F,R&P Slabs, Walls & Cols - 2nd	7	7	21APR06	01MAY06	0
Level 3						
02030100	F,R&P Slabs, Walls & Cols - 3rd	5	5	02MAY06	08MAY06	0
Level 4						
02040100	F,R&P Slabs, Walls & Cols - 4th	5	5	09MAY06	15MAY06	0
Level 5						
02050100	F,R&P Slabs, Walls & Cols - 5th	5	5	16MAY06	22MAY06	3
Level 6						
02060100	F,R&P Slabs, Walls & Cols - 6th	5	5	23MAY06	30MAY06	5
Level 7						
02070100	F,R&P Slabs, Walls & Cols - 7th	5	5	31MAY06	06JUN06	8
Level 8						
02080100	F,R&P Slabs, Walls & Cols - 8th	5	5	07JUN06	13JUN06	12
Level 9						
02090100	F,R&P Slabs, Walls & Cols - 9th	5	5	14JUN06	20JUN06	15
Level 10						
02100100	F,R&P Slabs, Walls & Cols - 10th	5	5	21JUN06	27JUN06	16
Level 11						
02110100	F,R&P Slabs, Walls & Cols - 11th	5	5	28JUN06	05JUL06	16
Roof						
02RF0100	F,R&P Slabs, Walls & Cols - Main/PH Roof	8	8	06JUL06	17JUL06	16
02RF0195	Concrete Top-Out - Bldg 2	0	0		17JUL06	16
02RF0198	Dismantle Tower Crane 1 (NE)	0	0		05APR07	0

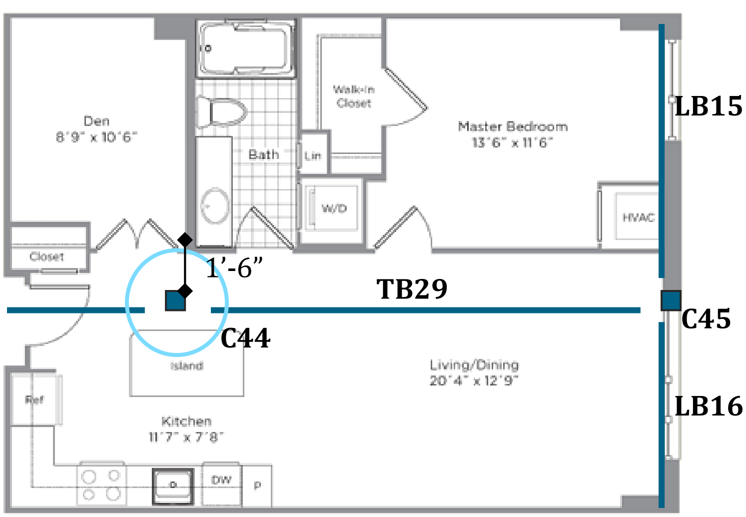
On average the current structure is assembled at a rate of 1 floor per week

BREADTH #2: ARCHITECTURAL ALTERATIONS

The hardest part about coordinating the existing architecture plans and the new column grid was the acquiring of the cad files. The cad files were not acquired until very late in the design process as a result there were small discrepancies between the plan created by hand in cad for analysis and the actual CAD files. The column grid was overlaid on a typical floor then 3 problem areas were chosen and architectural alterations were made. A section is also included showing the altered floor to ceiling height and ceiling appearance with the addition of T-Beams.



AREA 1

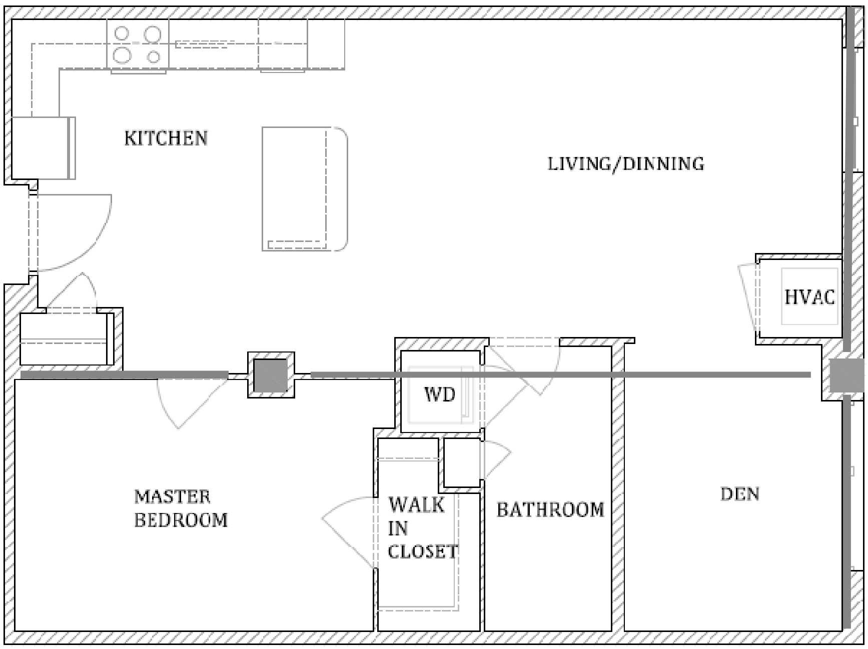


Existing Plan

Area 1 is a one bedroom/den combo layout is shown on the left with the new column grid in blue. It is evident that a 16" x16" column circled in blue, location is unacceptable. The column creates a tight an unusable space between it and the neighboring walls. Alterations were necessary to maximize the rentable space, and correct the flow issue created by the column.

SQFT $_{approx.}$= 780 sqft

Photo courtesy of Lowes developers www.cityvisatdc.com. [Figure N.T.S]

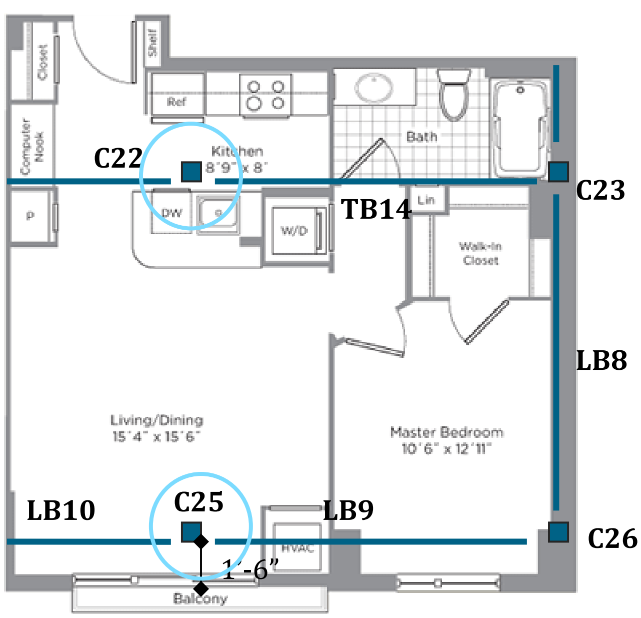


Area 1 Alterations

The floor plan was essentially mirrored to accommodate the column. Column C44 is now located within the wall between the kitchen and master bedroom. All spaces were kept to their original dimensions as shown above.

Fig # : New proposed floor plan for apt units 112, 212, 312, 412, 512, 612, 712, 812, 912, 1012, and 1112. [Figure N.T S]

AREA 2

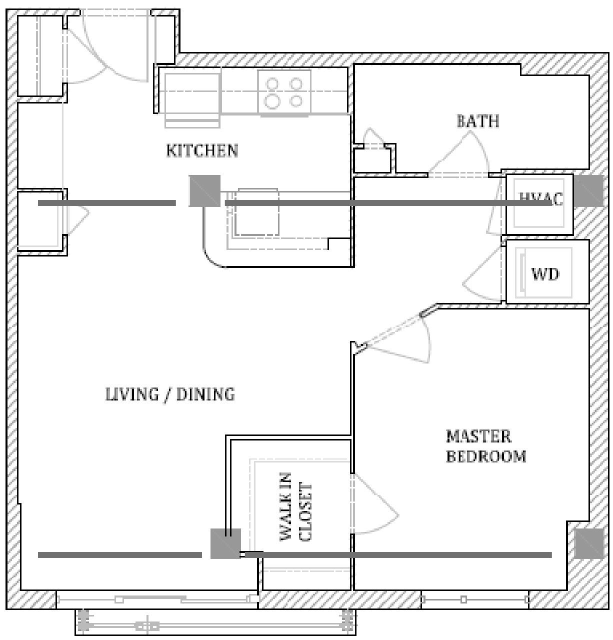


Existing Plan

Area 2 is part of a one bedroom condo, layout is shown on the left with the new column grid in blue. (2) Columns are in critical locations and the floor plan needs alterations.

SQFT $_{\text{approx.}}$= 650 sqft

Photo courtesy of Lowes developers
www.cityvisatdc.com [Figure N.T S]

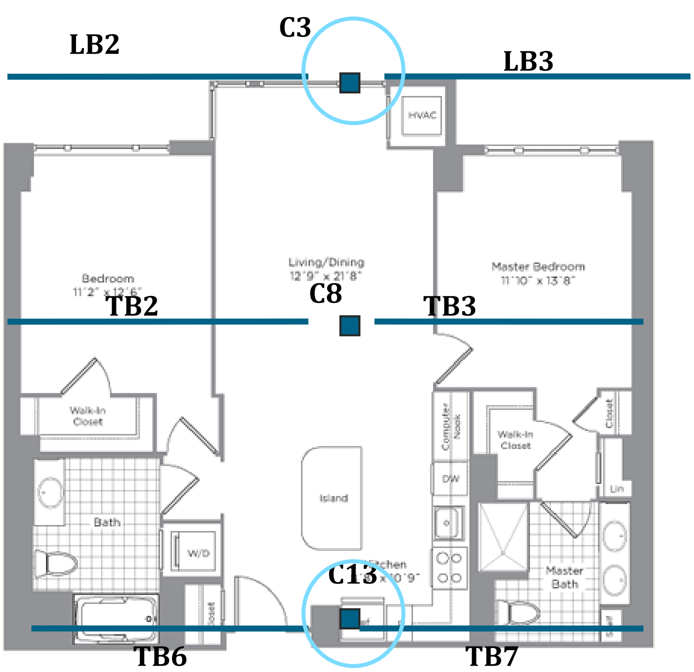


Area 2 Alterations

Unlike area 1 area 2 has (2) obstructions, as a result it was difficult to redesign the space to optimize both the window space and the rentable space. A decision had to be made between a smaller closet, or eliminating the awkward cubby between column C25 and the wall. Closet space is a definite plus for renters, so closet space was not reduced. It was decided the “cubby” space could be used as bookshelf or bench area.

Fig # : New proposed floor plan for apt units 102, 202, 302, 402, 502, 602, 702, 802, 902, 1002, and 1102 . [Figure N.T S]

AREA 3

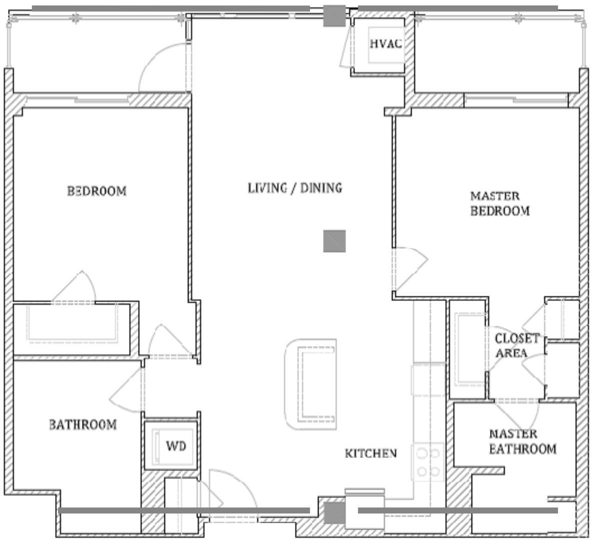


Existing Plan

Area 3 is a two bedroom condo, layout is shown on the left with the new column grid in blue.

SQFT $_{approx.}$= 1000 sqft

Photo courtesy of Lowes developers www.cityvisatdc.com [Figure N.T S]



Area 3 Alterations

Area three was the most difficult area to alter to accommodate the new column grid. Columns run down the center of the main living space. Column C3 is blocking the edge of glass curtain walls reducing its span from 9’-9” to 8’ -9”.

Fig # : New proposed floor plan for apt units 104, 204, 304, 404, 504, 604, 704, 804, 904, 1004, and 1104 [Figure N.T S]

SECTION

Currently City Vista uses varying ceiling height to add volume and create a separation between spaces in the condos as seen in figure #28. The underside of the slab in the new design is a grid of exposed pre-cast beams. This grid could be used to further develop the volume or other architectural interest in the ceiling, or a finished ceiling could be built under the beams. In general there is one grid line of beams running though each condo.

Figure #28 : Typical condo in City Vista Building 2 courtesy of The Lower enterprise www.cityvistadc.com

Floor to ceiling height was changed to accommodate the new system. A new north/south section is shown in figure #29. This section can be compared with figure #4 on page 6.

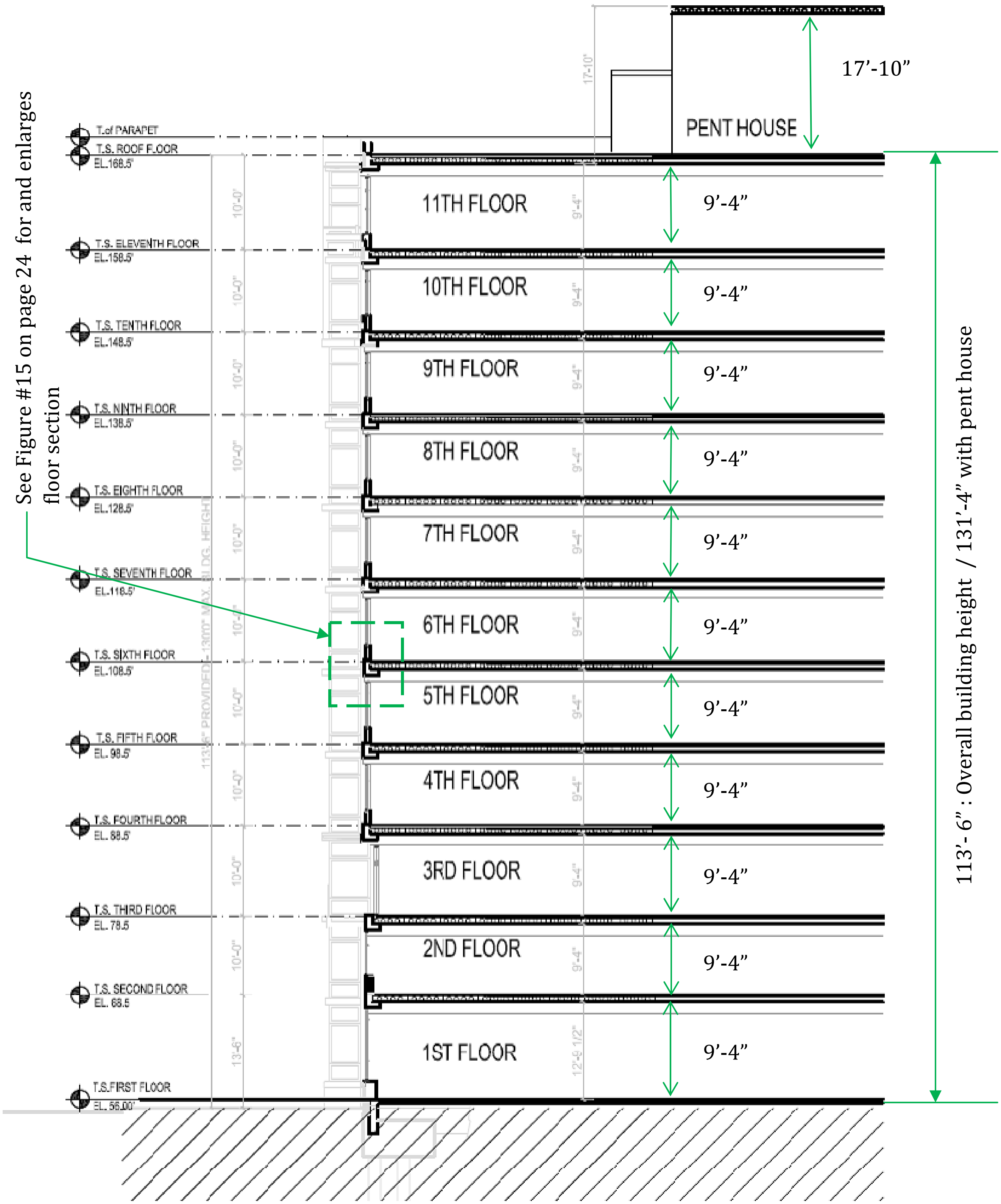


Figure #29 : New floor to ceiling heights

CONCLUSION

After closely examining the pre-cast gravity system I would not suggest it for this application. If it was incorporated during preliminary design I feel it would have better success, although still a PT system is more efficient and economic for an open plan condominium building with a height restriction.

Column Grid:

In the D.C. market the height restriction and overwhelming demand for rentable space control the design. The PT system provides a less restrictive column grid. The irregularity in column placement the post tension system allows is very conducive to architectural plans where column placement cannot govern. The pre-cast system does provide considerably smaller columns, but the requirement for a rectangular bay is an architectural issue no matter how it is laid out. Both systems use conventionally reinforced columns. The P.T column grid consists of (52) 28"x15" cast in place columns, mostly irregularly shaped bays are used. Typically bay sizes are in the range of 20ft x 22 ft. The pre-cast grid uses strict rectangular bays arranged to reduce the eccentric moment created by the hanger connection, (57) columns are needed decreasing in size every 4 floors.

Floor System:

The 7 ½" flat plate is about as efficient as they come for achieving small floor to ceiling height. The pre-cast system does provide more convenient for electrical and mechanical system because the hollow cores in the planks can double as conduits. The post tension system restricts the other trades because conduits, rebar, and tendons must fit inside the 7 ½" slab.

During redesign member, spans, and connection types were chosen to minimize the story height, although in the end an additional 8" was added to each story increasing the overall building height to 130'-6" at sections when the penthouse is considered. This is 6" over the height restriction in Washington D.C. The mechanical penthouse has a 17'-10" floor to ceiling height. In my analysis the possibility of decreasing this height was not considered, although it could be a possibility and the max building height could be obtained.

Building Weight:

The building weight did increase by approximately 3000 kips. The diaphragm (hollow core planks and 2" topping, weight = 73.75PSF) was lighter than the 93.75 PSF post tension slab although with the addition of T, R and L-beam each floor gained about 275 kips of additional dead weight. The composite topping contributed 25 PSF of the 73.75 PSF dead load. A ¾" leveling topping could have been used reducing the dead weight, but it was not because the composite topping presented more benefits to the overall serviceability of City Vista. The 2" topping benefits include: fireproofing, load distribution, diaphragm reinforcing and seismic capabilities. The additional mass did in return affect the story shear and displacement of the building.

Lateral System:

The lateral system was inadequate to support the building in the event of an earthquake. This was unsuspected due to the small height and weight increase in the building. Shear walls 2&4 used to resist lateral forces in the Y direction were significantly inadequate. The displacement was well above the minimum and walls were heavily reinforced compared to the walls used to resist forces in the X direction. Further investigation of this problem is needed before a decision can be made to either relocate or increase wall length. This is outside the scope of my thesis although it needs to be resolved before a final decision could be made by the developer to use a pre-cast system.

A possible explanation for the failure of shear walls 2 and 4 could be the modeling method in E-Tabs. The gravity system was assumed to take minimum of none of lateral forces. This may not have been the original assumption when the walls were originally designed. Another may be the simplification made to the model during analysis. Instead of drawing the entire gravity system in E-Tabs weightless floor diaphragms were modeled and then assigned a uniform mass to account for the planks, beams, and columns.

Constructability:

Both systems present constructability issues. A post tensioned system is dependent on weather during pours. Many admixtures and construction procedures have been created to combat this, although it is a concern. Pre-cast is not weather depend, although it requires a more intensive pre-design process. Considerable lead time is needed for the manufacturing and designing of the pre-cast members by the manufacturer. This limits last minute architectural and structural design changed.

Transportation of pre-cast members is crucial to erection. Laws and traffic in Washington D.C. restrict transportation of members. Shipping is restricted to Monday after noon to Friday before noon. During these times traffic patterns in down town D.C. are still an issue. In some instances pre-cast requires special permits for oversize loads or roadways to be closed to accommodate wide members. During design this was considered and no member in the new gravity system requires a permit.

Post tension system present shortening and cable integrity issues during erection and curing of the structure. Cables have the capability of rupturing during tensioning and blowing out entire bays. During curing the concrete tends to shrink creating tension cracks around the perimeter of the floor plates. Pre-cast is cured in a warehouse before transportation to site, therefore shrinking is not as big of a concern during curing.

If erection of the pre-cast system is not done correctly structural integrity could be in jeopardy. Connections between members must be executed to resisted lateral forces and distribute load correctly. The composite topping makes this procedure easier by creating a monolithic slab. Hangers connection are a concern due to high probability of manufacturing mistakes.

Cost:

The post tension gravity system cost considerably less than the pre-cast. Like building weight this is a result of the beams needed to support the planks. The post tensioned slab and planks with topping prices are competitive with each other. The same with the pre-cast and cast in place conventionally reinforced columns. If a strictly economical approach is taken the post tensioned system is about 2 million dollars cheaper when materials and installation was considered.

Pre-Cast System: $ **7,756,942.00**

Post Tension System: **$ 5,150,425.00**

Architecturally:

Architects Tortis Gallas uses an open plan for condos at City Vista. This system requires long clear spans so kitchen/ living room/ dining areas are not obstructed by columns. As seen in the architectural portion of the report, several floor plans are redesigned to accommodate for the new column grid, although even with alterations the rectangular grid is inadequate to support the open plan condos. The T-Beams does present architectural interest but may not be what the architect had in mind during design.

Recommendation:

As a result a pre-cast system would have been more successful if incorporated during preliminary design. Condos could still use an open plan if designed around rectangular bays. Although, it is very difficult to incorporate rectangular bays after a design for irregular bays is in place. To accommodate the current plan beam sizes would need to be increased resulting in larger member, a heavier building, and larger floor to ceiling heights.

REFERENCES:

Pre-Cast Design:

- PCI 6th edition
- Hollow core handbook :
  http://www.pci.org/view_file.cfm?file=MNL-126-981.PDF
- Nitterhouse Pre-cast products.
  http://Nitterhouse.com

Leeds:

- PCI Sustainability article:
  http://www.pci.org/publications/designers_notebook/dn_view.cfm?ascent_id=47
- U.S Green Building Council:
  http://www.usgbc.org/DisplayPage.aspx?CategoryID=19

Cost:

- RS-Means Cost Works:
  http://www.rsmeans.com/

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:

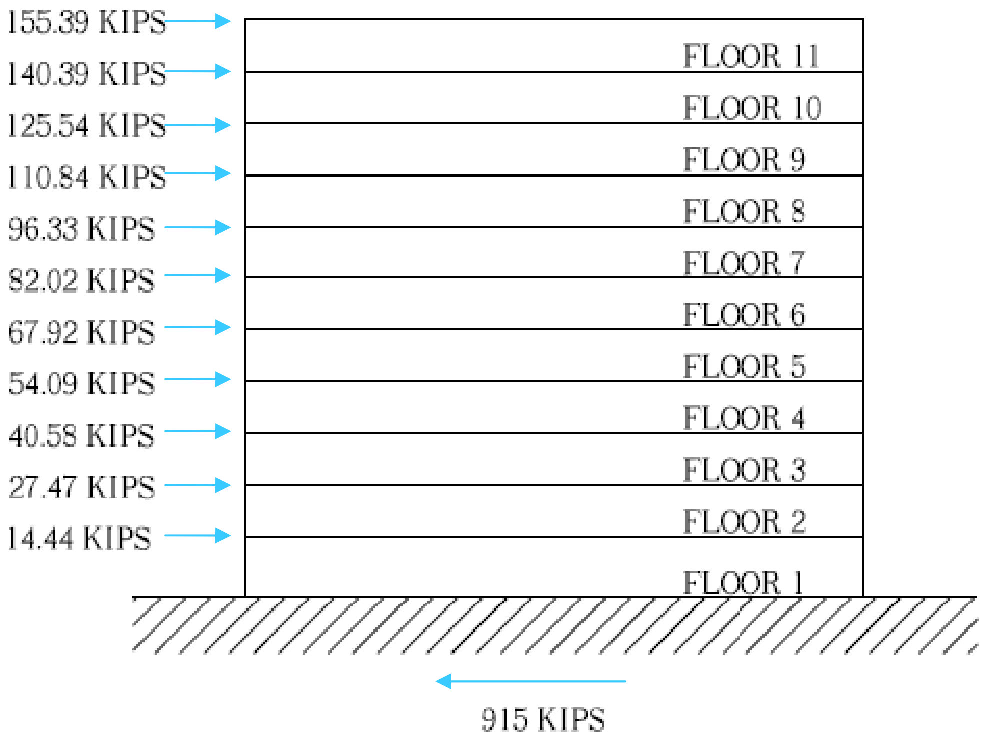
Rooftop Units = 8 Kips
Snow = 102.8 Kips

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

$Iz = c(33/z)^{1/6} = 0.3(33/76.8)^{1/6} = 0.38$
$Z = 0.6h = 76.8ft$
[Table 6-2] $Z_{min} = 30$
$c = 0.30$
$\varepsilon = 1/3$
$\ell = 320$
$Q = \begin{cases} \textbf{W-E} = 0.909 \\ \textbf{N-S} = 0.888 \end{cases}$

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

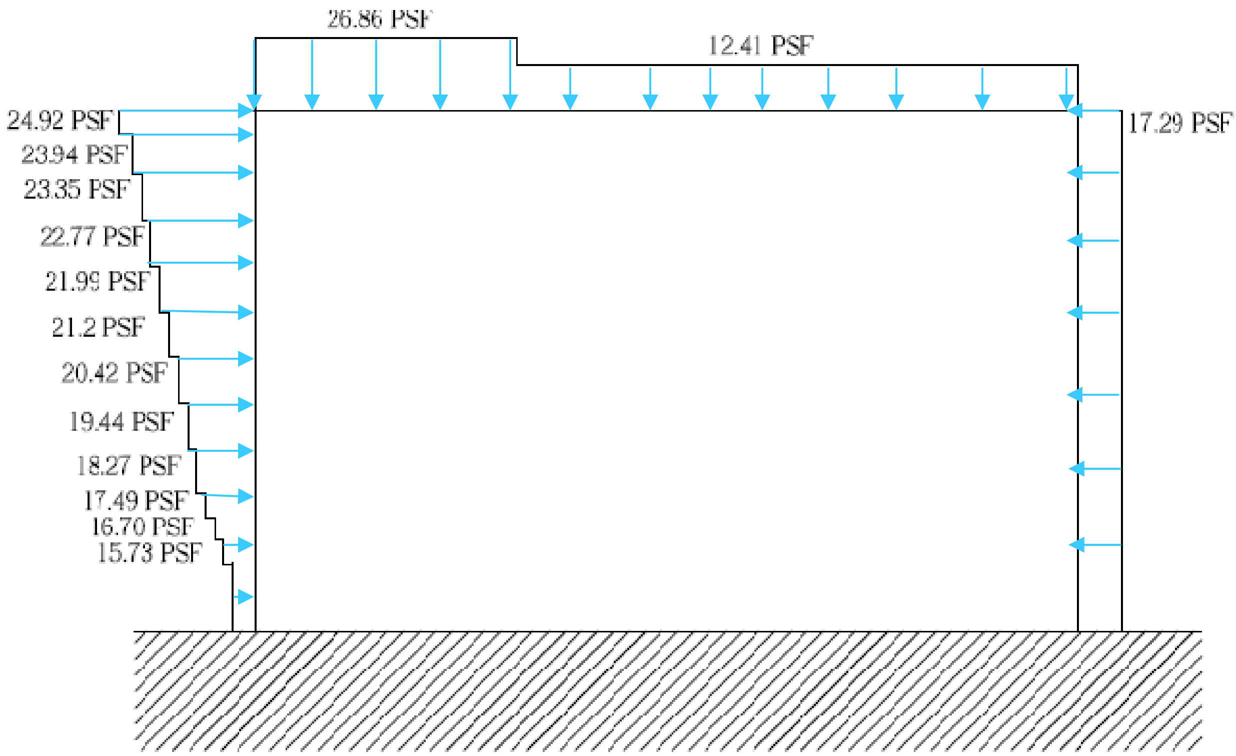
C_{pi} : FIG 6-5 = +/- 0.18
Cp: FIG 6.6 $\longrightarrow$ **W-E**: LEEWARD = -0.5 (L/B = .616)
WINDWARD= 0.8
N-S : LEEWARD = -0.3 (L/B = 1.6)
WINDWARD = 0.8

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

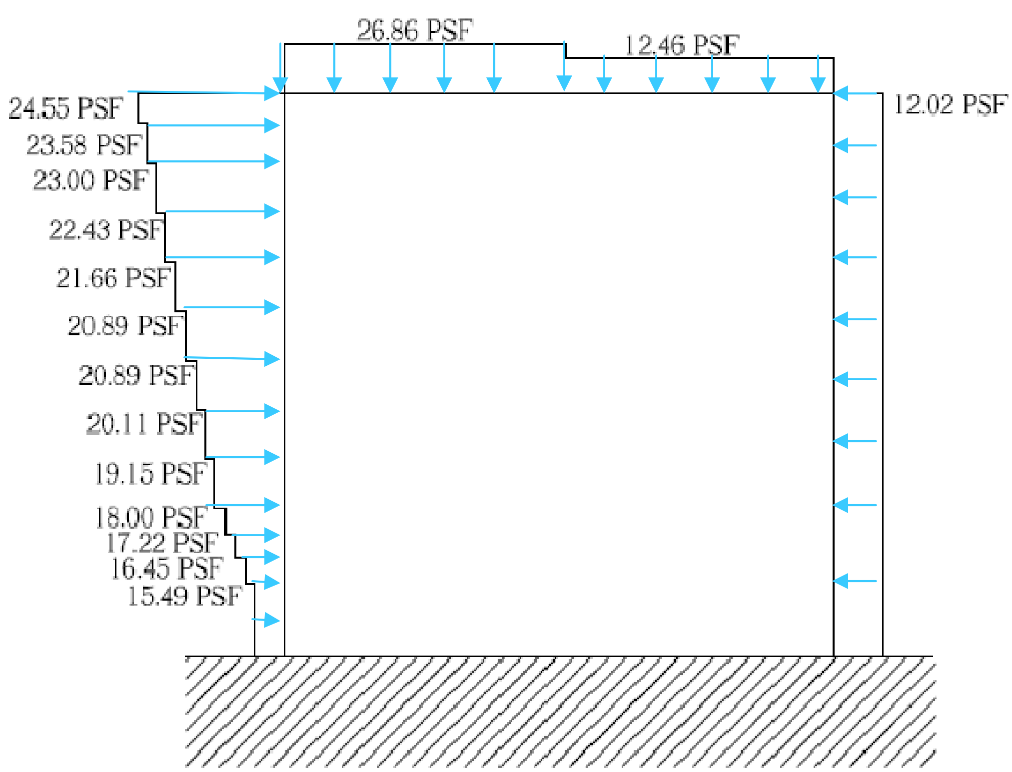
Roof:	
East/West	$C_{P\ Windward}$ = 0-h/2 → -1.3 $C_{P\ Windward}$ = >h/2 → -0.7
North/West	$C_{P\ Windward}$ = 0-h/2 → -1.3 $C_{P\ Windward}$ = >h/2 → -0.7

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	



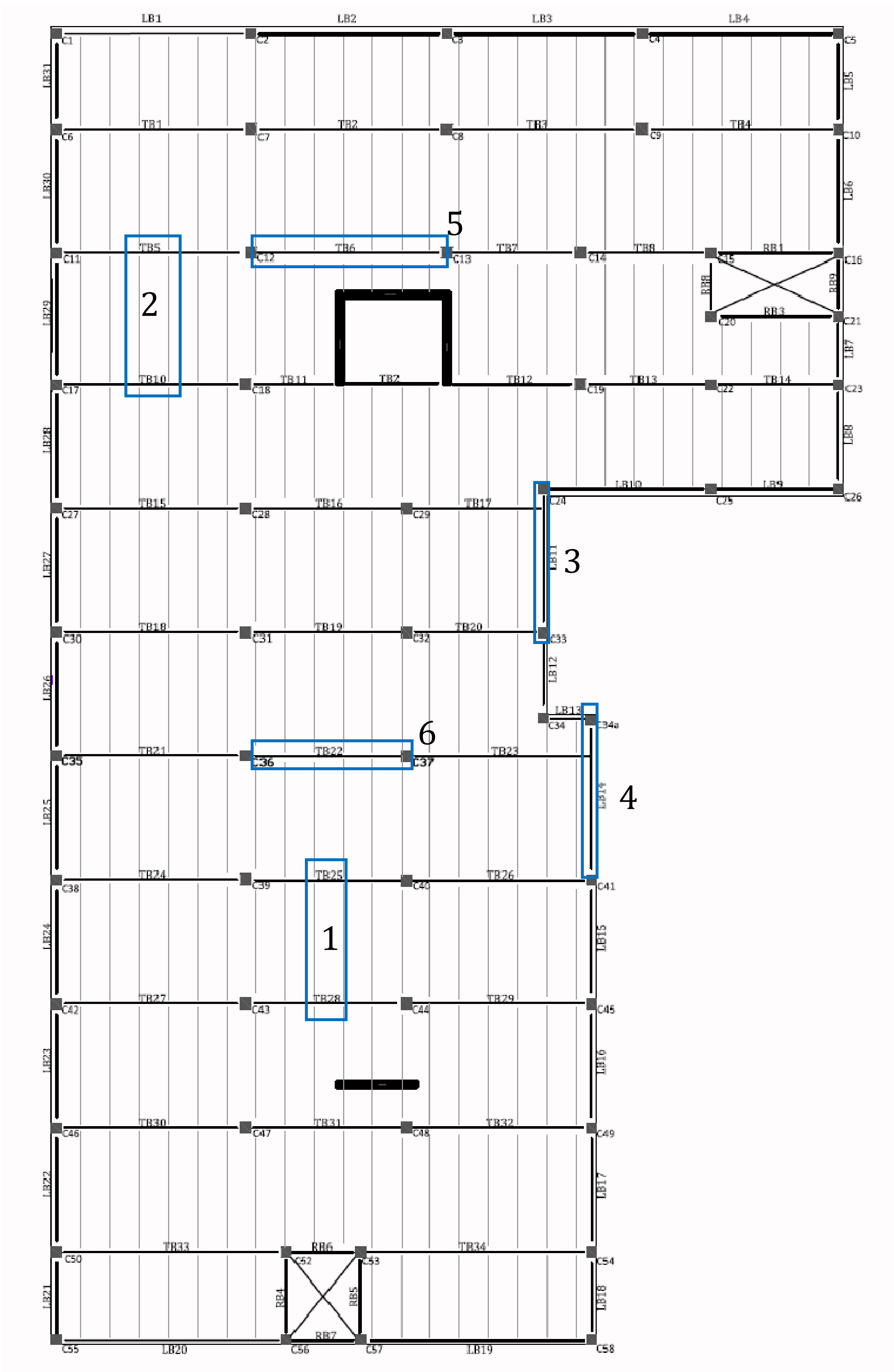
Wind Loading Diagram *East – West*



Wind Loading Diagram *North –South*

Appendix #2

GRAVITY SYSTEM


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
RB3
LB29
C20
C21
LB7
TB10
TB11
TB12
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TB14
C17
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C22
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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
LB14
4
LB25
TB24
TB25
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C38
C39
C40
C41
1
LB24
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

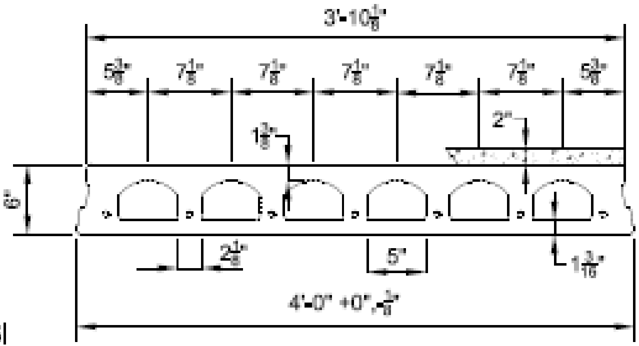
Prestressed Concrete
6"x4'-0" Hollow Core Plank

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	380	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

$-\varnothing M_n \geq M_u$
$C\omega_{pu}$=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√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√f'$_c$$b_w$d=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*$17^4$1728/384*4463*529 = 0.079"
Long Term = (0.079)(2.30) = 0.182"
$\Delta_{SUPERIMPOSED}$ = 5*.02*4*17^4*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: $P_o$el2/8EI – 5wl^4/384EI
Initial Camber = [173.5*2.25*(17*12)^2/8*3408*1519]-[5*.195*17^4*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 = Δ_{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
A_c= 253 in^2
I_c=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= $P_o/A \pm P_0e/S \pm M_D/S$ =
F_{TOP} =0.685-0.708+0.22 ***= 0.195 ≤ F_{ti} OK***
F_{BOT}=0.685+1.055-0.56 ***=1.10 ≤ F_{ci} 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-.113)= 192.3 Kips

CHECK:
$F = P/A \pm Pe/S \pm M_{Ser}*e/S \pm$ OR $\pm M_{Sus}*e/S$ =
$F_{TOP/Service}$ = 0.760-0.54+0.61 ***= 0.83 KS I < F_{Ci} OK***
$F_{TOP/Sustainable}$ = 0.760-0.54+0.96 ***= 1.18 KSI < F_{Ci} OK***
F_{BOT} = 0.760 + (0.54-0.96)(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}= 250ksi
$\text{Ø}M_n$= ØAsp*fps(d_p-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 → Ø=0.9
$\text{Ø}M_n$ =0.9*1.071*250*[6-1.23/2] = ***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

$\emptyset V_n \geq V_u$
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**

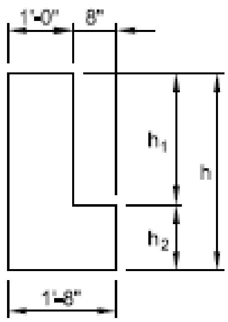
EXTERIOR BEAMS							
DESIGNATION	***Span***	***Trib W (ft)***	***1.2* Dead (psf)***	***1.6*Live (psf)***	***Wu psf***	***Wu plf***	***TRAIL SIZE***
LB-1	26.8	6.5	112.5	64	176.5	1147.3	20LB20
LB-2	26.8	6.5	112.5	64	176.5	1147.3	20LB20
LB-3	26.8	6.5	112.5	64	176.5	1147.3	20LB20
LB-4	26.8	6.5	112.5	64	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_e(end) in. y_e(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 2.44	6566 0.3 0.1	5131 0.4 0.2	4105 0.5 0.2	3345 0.6 0.2	2768 0.7 0.2	2318 0.8 0.2	1961 0.9 0.3	1674 1.0 0.3	1438 1.0 0.3	1243 1.1 0.3	1079 1.2 0.2							
20LB24	108-S	2.80 2.80	9577 0.3 0.1	7495 0.3 0.1	6006 0.4 0.1	4904 0.5 0.1	4066 0.5 0.1	3414 0.6 0.2	2896 0.7 0.2	2479 0.8 0.2	2137 0.9 0.2	1854 0.9 0.2	1617 1.0 0.1	1416 1.0 0.1	1244 1.1 0.1	1097 1.1 0.0	969 1.2 0.0			
20LB28	128-S	3.33 3.33			8228 0.4 0.1	6733 0.4 0.1	5596 0.5 0.2	4711 0.6 0.2	4009 0.6 0.2	3443 0.7 0.2	2979 0.8 0.2	2595 0.9 0.2	2273 0.9 0.2	2000 1.0 0.2	1768 1.1 0.2	1567 1.1 0.2	1394 1.2 0.1	1243 1.2 0.1	1110 1.2 0.0	992 1.3 0.0
20LB32	148-S	3.71 3.71				8942 0.4 0.1	7446 0.5 0.2	6281 0.5 0.2	5356 0.6 0.2	4611 0.7 0.2	4001 0.7 0.2	3495 0.8 0.2	3071 0.9 0.3	2712 1.0 0.3	2406 1.0 0.3	2143 1.1 0.2	1914 1.2 0.2	1715 1.2 0.2	1540 1.3 0.2	1386 1.3 0.1
20LB36	168-S	4.25 4.25					9457 0.4 0.2	7988 0.5 0.2	6823 0.5 0.2	5883 0.6 0.2	5113 0.7 0.2	4476 0.8 0.3	3941 0.8 0.3	3489 0.9 0.3	3103 1.0 0.3	2771 1.1 0.3	2483 1.1 0.3	2231 1.2 0.3	2011 1.2 0.3	1816 1.3 0.2
20LB40	188-S	4.89 4.89						9812 0.4 0.2	8386 0.5 0.2	7235 0.6 0.2	6293 0.6 0.2	5513 0.7 0.2	4858 0.8 0.2	4305 0.8 0.3	3832 0.9 0.3	3425 1.0 0.3	3073 1.0 0.3	2765 1.1 0.3	2495 1.1 0.3	2257 1.2 0.3
20LB44	198-S	5.05 5.05								8959 0.5 0.2	7803 0.6 0.2	6845 0.6 0.2	6042 0.7 0.2	5363 0.8 0.2	4783 0.8 0.2	4284 0.9 0.2	3851 0.9 0.2	3474 1.0 0.2	3143 1.1 0.2	2850 1.1 0.2
20LB48	218-S	5.81 5.81									9226 0.5 0.2	8100 0.6 0.2	7158 0.6 0.2	6360 0.7 0.2	5678 0.8 0.2	5092 0.8 0.2	4584 0.9 0.3	4140 0.9 0.3	3751 1.0 0.3	3408 1.1 0.3
20LB52	238-S	6.17 6.17										9634 0.6 0.2	8521 0.6 0.2	7578 0.7 0.2	6774 0.7 0.3	6082 0.8 0.3	5482 0.9 0.3	4958 0.9 0.3	4499 1.0 0.3	4094 1.0 0.3
20LB56	258-S	6.64 6.64											9954 0.6 0.2	8860 0.7 0.2	7927 0.7 0.3	7124 0.8 0.3	6427 0.8 0.3	5820 0.9 0.3	5287 1.0 0.3	4816 1.0 0.3
20LB60	278-S	7.33 7.33													9089 0.7 0.3	8173 0.7 0.3	7380 0.8 0.3	6688 0.9 0.3	6080 0.9 0.3	5544 1.0 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

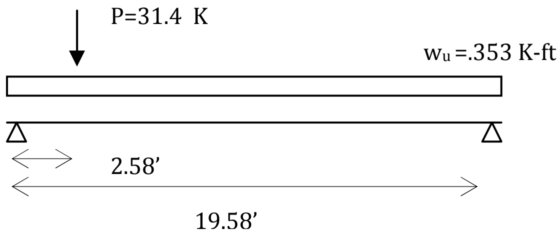
Moment Capacity: M_n = 196.75 Ft-Kips

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

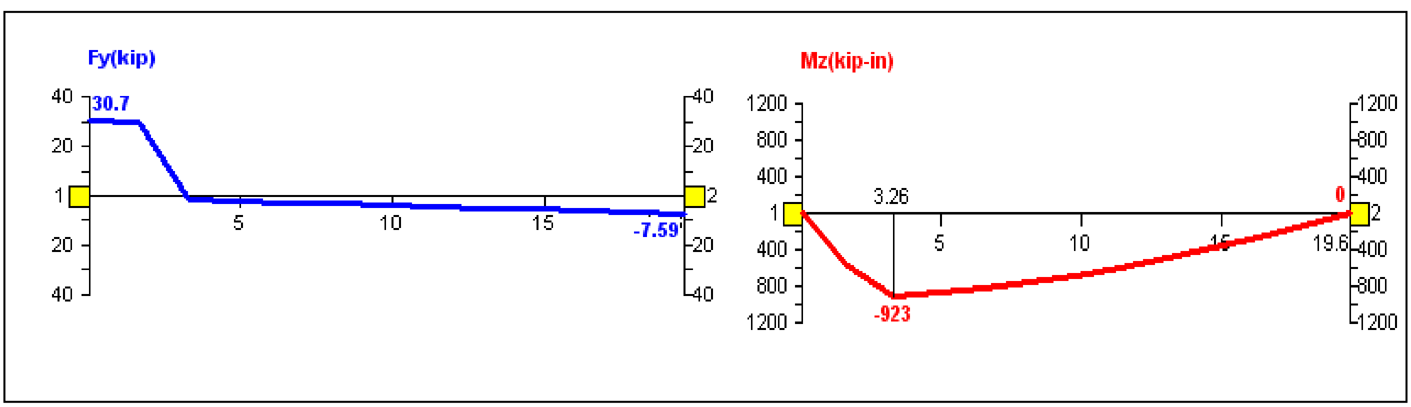
LOADING CONDITION:
Self 20LB20 = 317plf
Self TB18 = 500 plf
Wu = 166.4 PSF OR 2929.5 PLF
Span-Depth : 19.58/1.67 = 11.7 < 40 OK
P (TB18)=((2.829+.5)*18.83)/2=31.4 Kips
** Detailed loading conditions See spreadsheets**

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√f'$_c$ = 580psi
F_{ci}= 0.6(f'$_c$) = 3600 psi
Allowable @ transfer: F_t=3√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 ± P_OeC/I ± M_DC/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 ± P_eeC/I ± M_DC/I ± M_LC/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_ee+P_oI/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_rI/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)$^{1.5}$√5000 =4074 ksi
F_{se}= 170ksi
f_{pu}=270ksi

Loading Capacity: 3345 PLF

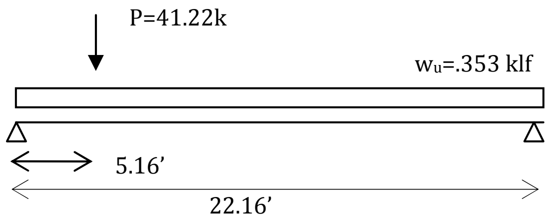
Moment Capacity: M_n = 205.3 Ft-Kips

Shear Capacity: V_n =37.0 Kips

Beam Properties
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

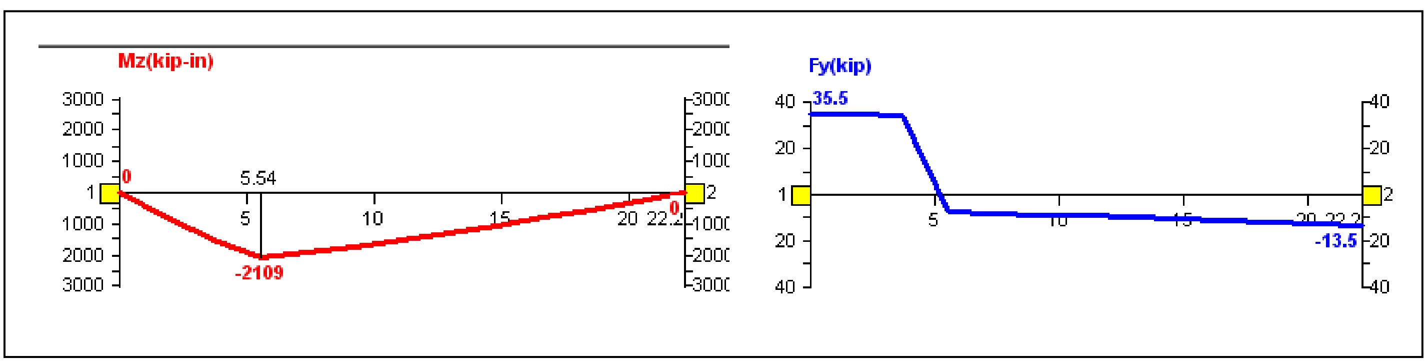
LOADING CONDITION:
LL=40 PSF
Super D=5 PSF
Self 12RB16 = 250plf
Self TB18 = 300plf
Wu = 176.5 PSF OR 353 PLF
Span-Depth : 22.16/1.33 = 16.66 < 40 OK
P (TB-24)=((2.733+.5)*25.5)/2=41.22 Kips
** Detailed loading conditions See spreadsheets**

1. Flexure and Shear



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

$ØV_n≥V_u$ OK



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:	$F_t=3\sqrt{f'_{ci}} =+180psi$
	$F_c=0.6f'_{ci} =-2160psi$

$A_{sp}= 9(0.153)= 1.377\ in^2$
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
$F_c= P_o/A \pm P_O eC/I \pm M_D C/I =$
F_{TOP} =-0.91+1.95-1.37= ***<u>=-0.33 KSI ≤ F_{ti} OK</u>***
F_{BOT}=-0.91-1.52+1.07= ***<u>= 1.36 KSI ≥- F_{ci} OK</u>***

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:
$F = P_e/A \pm P_e eC/I \pm M_D C/I \pm M_L C/I$
F_{TOP} =-0.77+1.66-1.37-0.95 = -1.43 ***<u>= -1.43 KSI ≥ F_c OK</u>***
F_{BOT}=-0.77-1.28+1.07+0.743 = ***<u>=-.237 KSI ≤- F_{ci} OK</u>***

6. 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= <u>483.8 ft-kips > 205.3 ft-kips OK</u>

Case #2: $M=Case\#1+f_r I/C$: cracking at bottom
M_{cr}=483.8+(530*10106/8.74)= <u>534.86 ft-kips > 205.3 ft-kips OK</u>

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

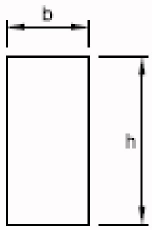
BLUE: Beams checked for structural integrity

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

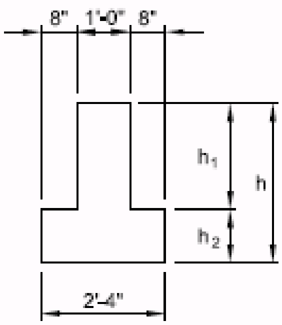
3553 – Safe superimposed service load, plf.
0.4 – Estimated camber at erection, in.
0.2 – Estimated long-time camber, in.

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

Desig-nation	No. Strand	y_s(end) in. y_s(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	3158	2620	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	5076	4040	3280	2711	2262	1905	1617	1381	1186	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	4730	4031	3451	2976	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

5

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:

$F = P_e/A \pm P_eeC/I \pm M_DC/I \pm M_LC/I$
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: $M=P_ee+P_oI/Ac$: 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

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

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

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

CHECK
$F_c= P_o/A \pm P_OeC/I \pm M_DC/I$ =
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

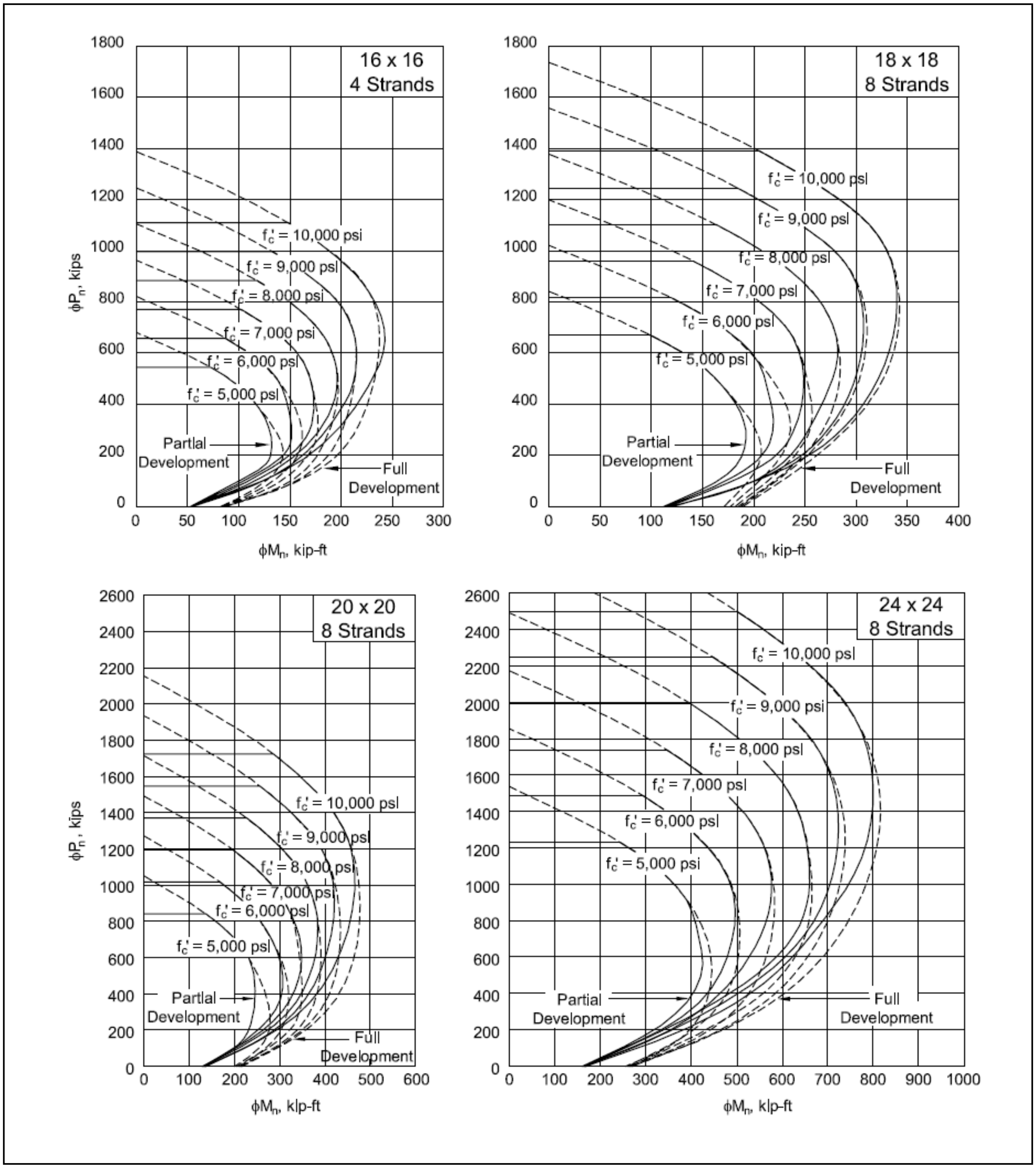
CHECK:
$F = P_e/A \pm P_e eC/I \pm M_D C/I \pm M_L C/I$
F_{TOP} =-0.98+2.31-0.98-0.71= ***= -0.36 KSI ≥ F_c OK***
F_{BOT}=-0.98-1.54+0.65+0.47 = ***= -1.4 KSI ≤ F_{ci} OK***

4.Cracking Moment

Case #1: $M=P_e e+P_o I/Ac$: 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

Column Sizing Charts from PCI 6th edition



Column Sizing [Fig 2.7.1 PCI handbook 6th edition]

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

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.

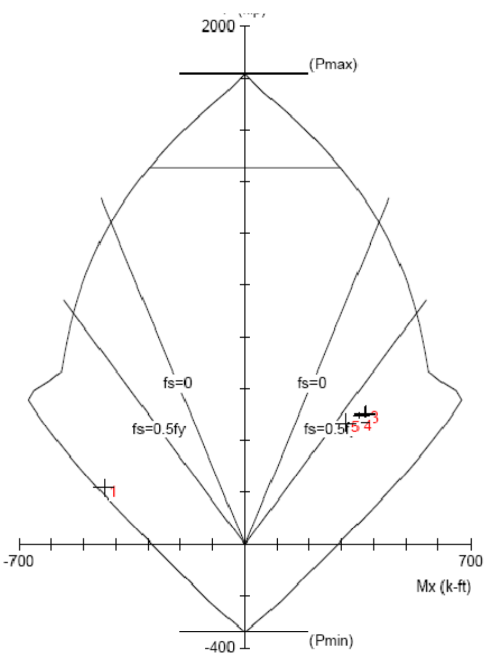


Fig 1: 24x24” Column w/ 4-#11

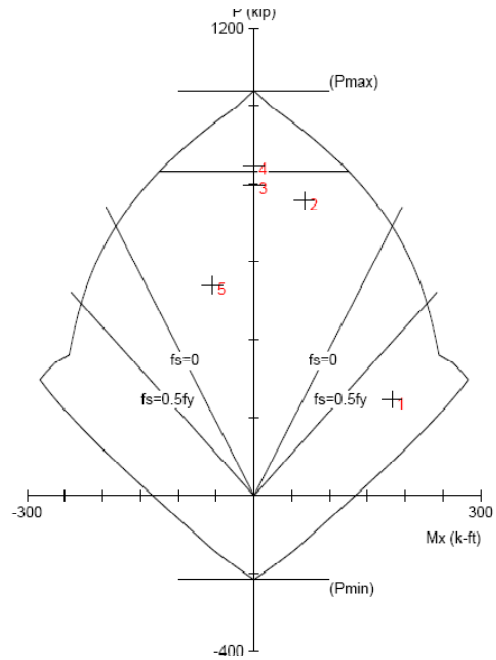


Fig 2: 18”x18” Column w/ 4-#9

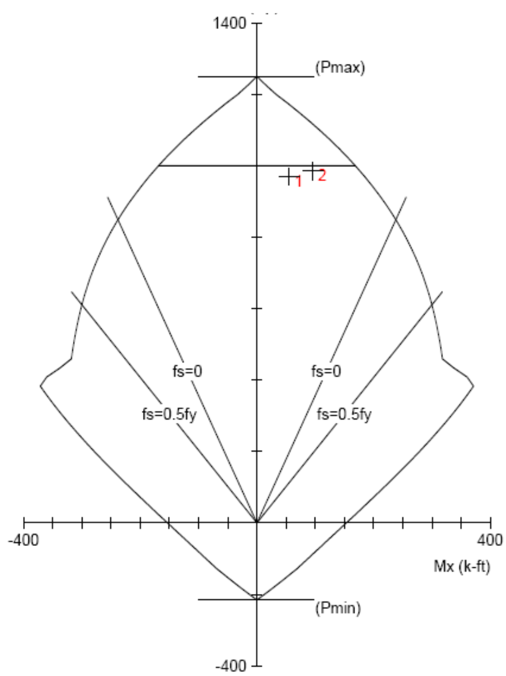


Fig 3: 20”x20” Columns w/ 4-#9

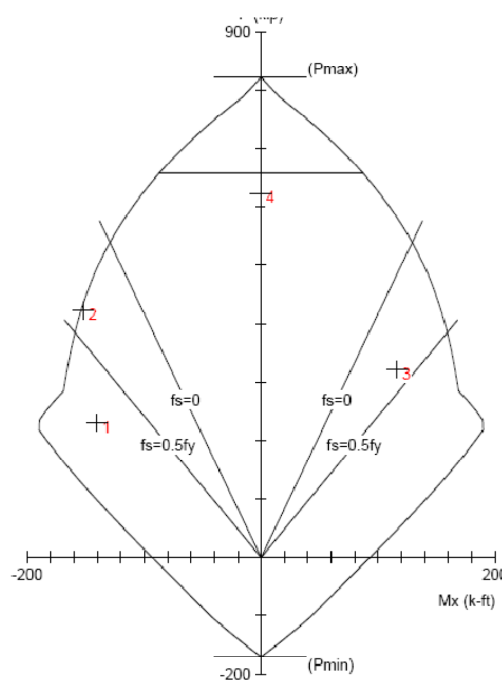


Fig 4: 18”x18” Column w/ 4-#8

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

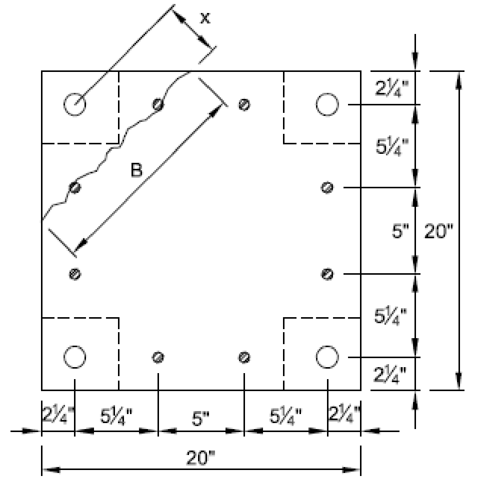
Creep = 169 x 10^{-6}*0.96*0.48*1.325 = 1.03 x 10^{-4}
Shrinkage = 354 x 10^{-6}*0.93*0.46 = 1.512 x 10^{-4}
Total Strain = 2.546 x 10^{-4}
Total Shortening = 2.546 x 10^{-4}(26)(12)= **0.079 in**

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} = **4.43 x 10^{-4}**

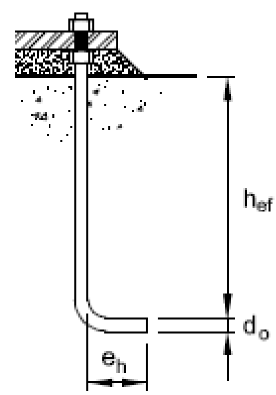
TOTAL STRAIN = 4.43E-4+150E-6= **5.93 x 10^{-4}**
TOTAL SHORTNING = 5.93E-4(12)(26)= **0.185 in**

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= $\sqrt{((T_u(4)x)/(\Phi BF_y)}$
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

Np=1.2f’$ce_d d_o c_{crp}$ ⟶ 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}=3.33λsqrt(f’c/h_{ef}) = 3.33(1)(√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 $\phi Vn = \phi\lambda\sqrt{f'c}h_l[2(b_l-b)+b_t+h_l+2d_e]$
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 ⟶ 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

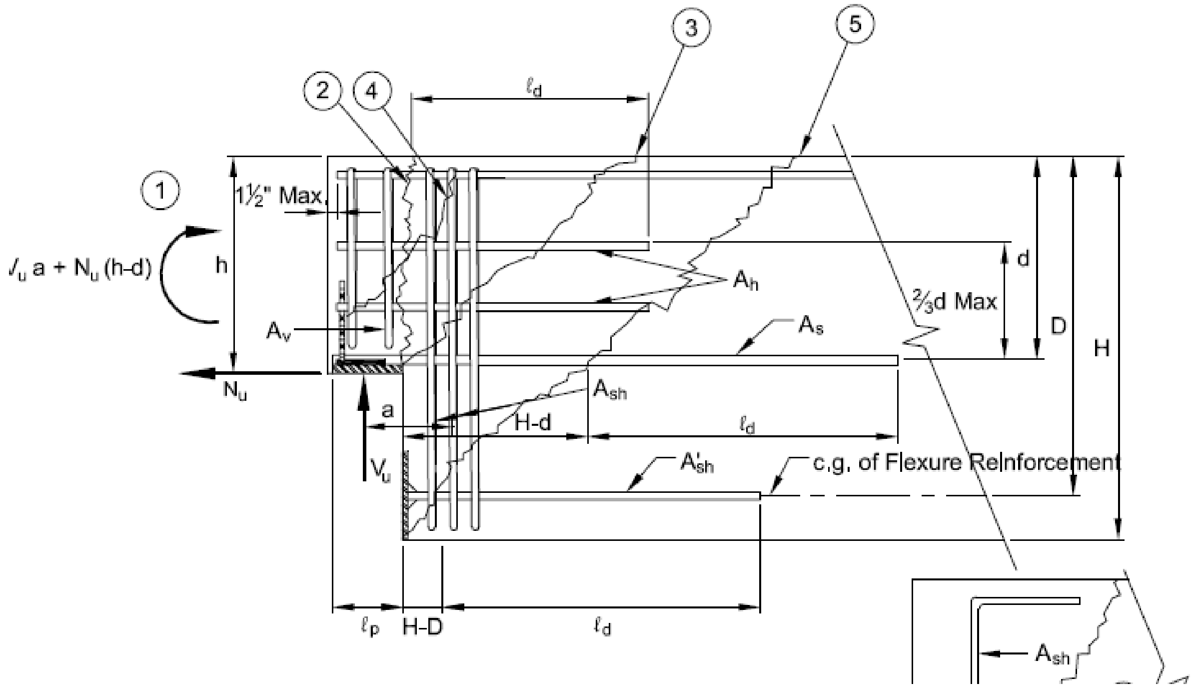
1. Flexure in extended end
   Shear span a=6" d=15"
   A_s=1/Φfy[Vu(a/d)+Nu(h/d) = 1/.75*60[42.41*(6/15)+8.48(24/15) = 0.62 in^2
2. Direct Shear :
   μ=1000λbhμ/Vu = 1000*1*28*24*1.4/42.41*1000 = 3.4
   $A_s=2Vu/3\Phi Fy\mu_e+Nu/\Phi fy$ = 0.373 in^2 < 0.62in^2
   A_s: USE → 2 #4 As = 0.62in^2
   $A_h=0.5(A_s-A_n)$=0.5(0.62-0.188)=0.216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



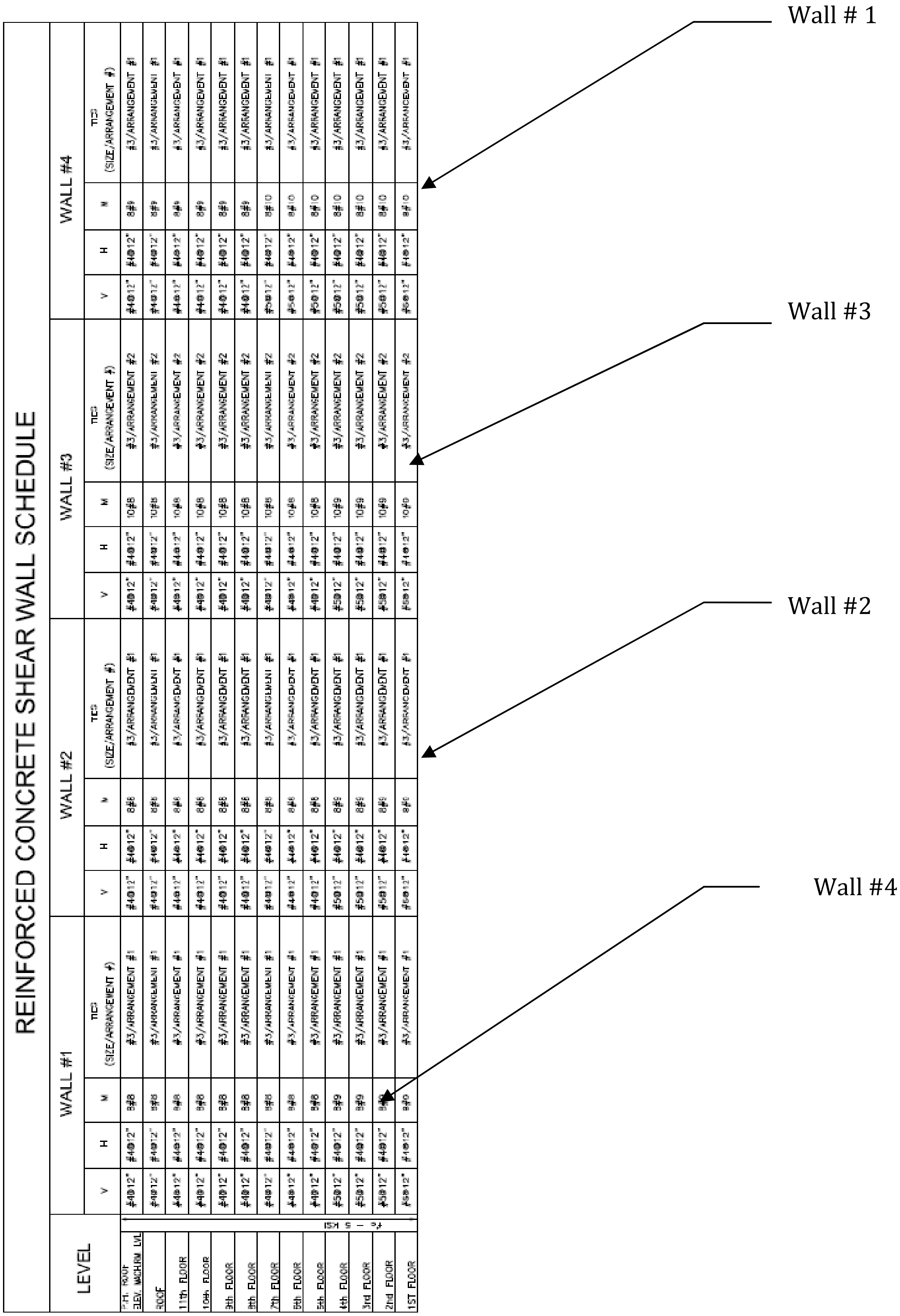
Shown above is the typical reinforcing for a dapped end beam from PCI 6th edition. The numbers represent the 5 modes of failure resulting from direct shear and diagonal tension.

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.



REINFORCED CONCRETE SHEAR WALL SCHEDULE

LEVEL	WALL #1 V	WALL #1 H	WALL #1 M	WALL #1 TIES (SIZE/ARRANGEMENT #)	WALL #2 V	WALL #2 H	WALL #2 M	WALL #2 TIES (SIZE/ARRANGEMENT #)	WALL #3 V	WALL #3 H	WALL #3 M	WALL #3 TIES (SIZE/ARRANGEMENT #)	WALL #4 V	WALL #4 H	WALL #4 M	WALL #4 TIES (SIZE/ARRANGEMENT #)
PH. ROOF ELEV. MACH.RM LVL	#4@12"	#4@12"	8#8	#3/ARRANGEMENT #1	#4@12"	#4@12"	8#8	#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#8	#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#8	#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#8	#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#8	#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#8	#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#8	#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#8	#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#8	#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#8	#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#8	#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#8	#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#9	#3/ARRANGEMENT #1	#5@12"	#4@12"	8#8	#3/ARRANGEMENT #1	#5@12"	#4@12"	10#9	#3/ARRANGEMENT #2	#5@12"	#4@12"	8#10	#3/ARRANGEMENT #1

f'c = 5 KSI

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.

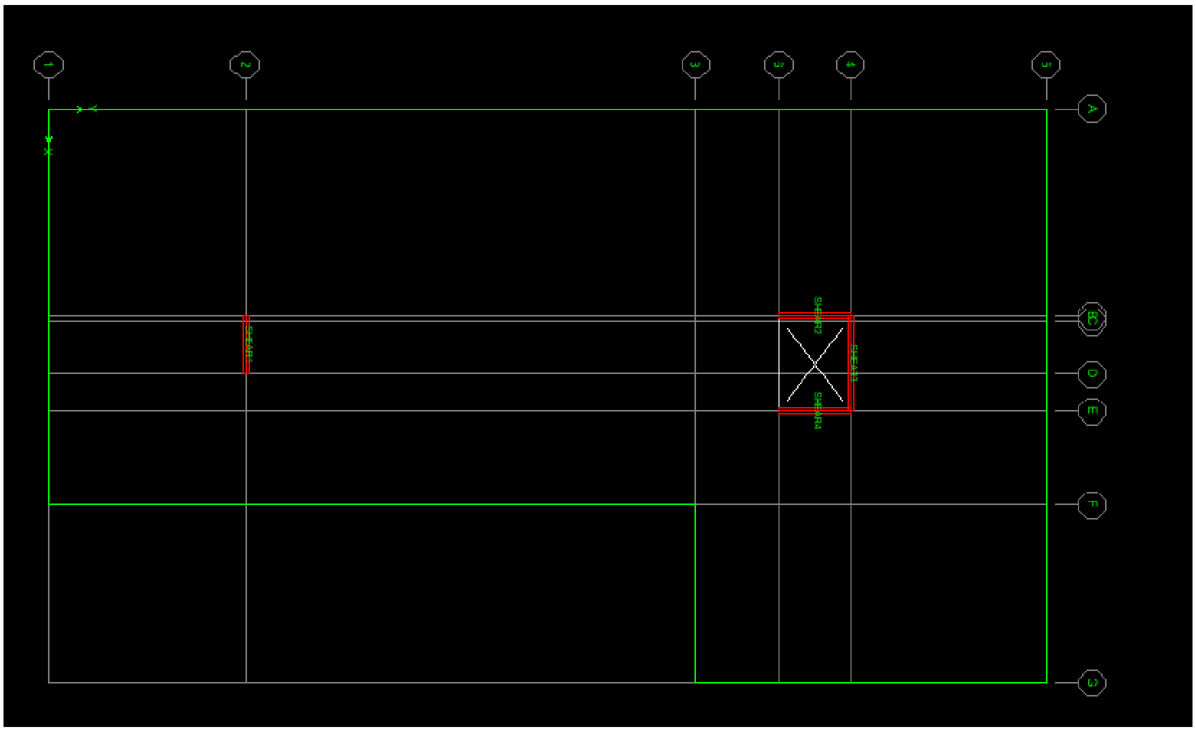


Figure #1: A snapshot of a typical floor that was input into E-Tabs for analysis

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

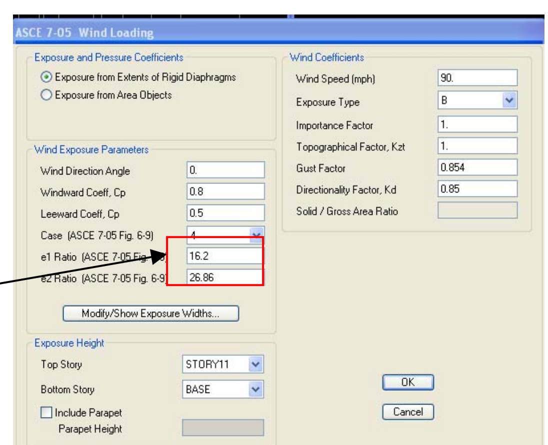
$\rho = 1.0$
$S_{ds} = 0.163$

(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

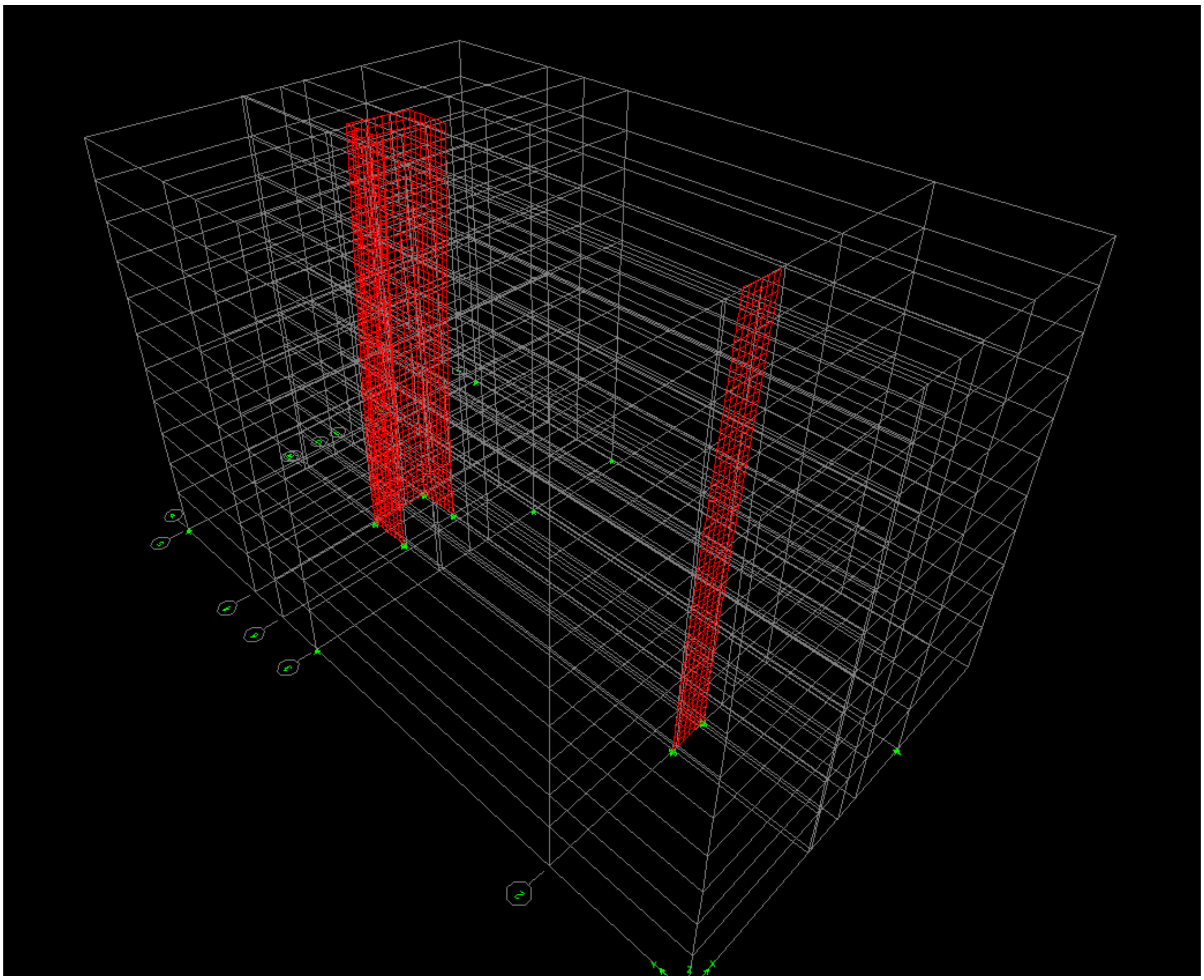


Figure #2: Deformed shape after analysis of seismic displacement in the X direction.

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

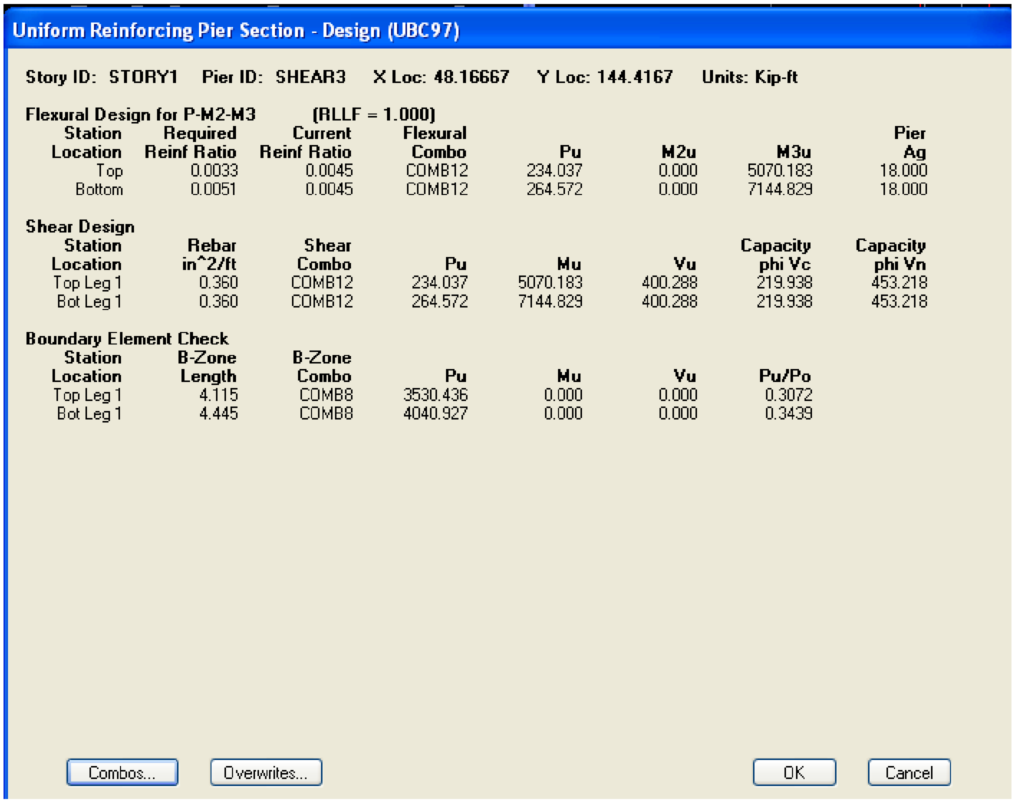
Station Location	Rebar in^2/ft	Shear Combo	Pu	Mu	Vu	Capacity phi Vc	Capacity phi Vn
Top Leg 1	0.562	COMB14	-1026.284	75371.383	266.690	56.123	266.690
Bot Leg 1	0.589	COMB14	-1180.360	93602.479	266.690	46.313	266.690

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

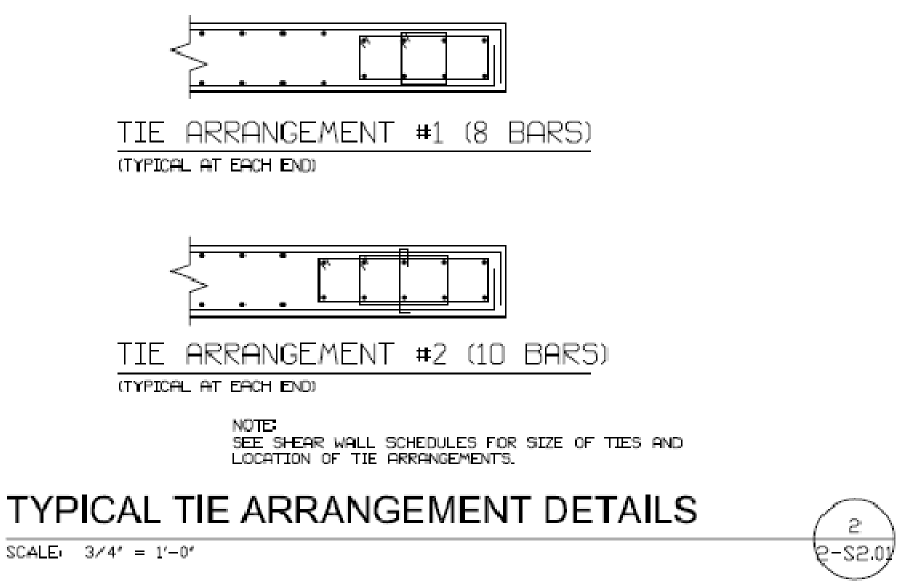
Station Location	Rebar in^2/ft	Shear Combo	Pu	Mu	Vu	Capacity phi Vc	Capacity phi Vn
Top Leg 1	0.360	COMB12	234.037	5070.183	400.288	219.938	453.218
Bot Leg 1	0.360	COMB12	264.572	7144.829	400.288	219.938	453.218

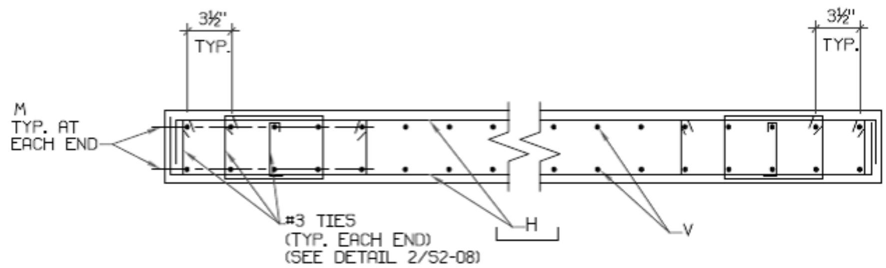
Boundary Element Check

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



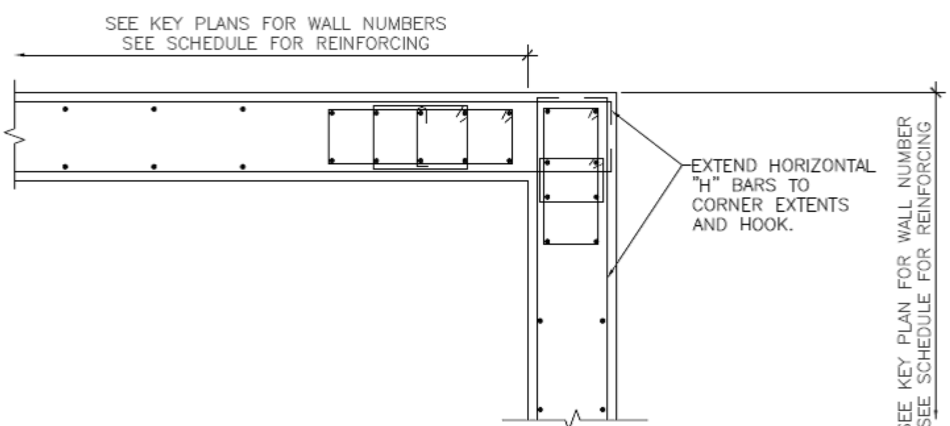


NOTES:
1. SEE KEY PLANS FOR SHEAR WALL DESIGNATION NUMBERS.
2. PROVIDE CORNER BARS WITH CLASS "B" SPLICES TO MATCH "H" BARS SHOWN ON SCHEDULE.
3. WHERE POSSIBLE, EXTEND "H" HORIZ. BARS INTO ADJACENT PERPENDICULAR WALL AND HOOK.
4. SEE SHEAR WALL SCHEDULES FOR FURTHER INFORMATION.

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