Senior Thesis Report:
Feasibility and Consequences of Staggered Truss Construction

River Tower at Christina Landing
Wilmington, DE



Joseph Bednarz **Structural Option** **The Pennsylvania State University**
Architectural Engineering *Dr. Thomas Boothby, Faculty Advisor*

River Tower at Christina Landing
Wilmington, DE

PROJECT OVERVIEW:
- 25-story condominium tower
- Enclosed parking garage interfaced in lower eight stories
- Ninth floor features roof terrace, pool, and observation platform
- *Total Size:* Approximately 433,200 Sq. Ft.
- *Projected Date of Construction:* September 2005 to April 2006
- *Project Delivery Method:* Design-Bid-Build
- *Projected Total Cost:* $55.5 Million

PROJECT TEAM:
- *Owner:* The Buccini Pollin Group
- *Architect:* Burt Hill Kosar Rittleman Associates
- *MEP Engineers:* Burt Hill Kosar Rittleman Associates
- *General Contractor:* Gilbane Construction Company
- *Civil Engineer:* Pennoni Associates, Inc.
- *Structural Engineer:* O'Donnell & Naccarato, Inc.
- *Elevator Consultant:* Lerch Bates & Associates, Inc.

ARCHITECTURAL:
- Phase II of riverside redevelopment plan
- Has since been redesigned for value engineering
- Brick/precast exterior, aluminum glass curtain wall
- Tall 9' ceilings (10' in penthouses), open floor plans
-Balconies provide spectacular views of city or river

STRUCTURAL:
- Steel piles support 12 in. thick first floor slab
- Concrete grade beams support perimeter
- 8 in. post-tensioned concrete floor slabs
- Concrete columns: typical 23'-0" by 28'-6" grid
- Concrete shear walls provide lateral resistance

MECHANICAL/PLUMBING:
- Two rooftop mounted cooling towers
- Constant heat value AHU on penthouse/mech. floor
- Electric radiant heat panels and central A/C control provided in each unit
-Four storage tankers with gas-powered water heaters

ELECTRICAL/LIGHTING:
- Both 480/277V and 208/120V power distribution
- 480Y/277V, 800kW, 1000kVa emergency generator
- 208/120V, 1 phase, 3 wire, 125A MCB to each unit
- Ceramic metal halides light parking lot and rooftop
- Fluorescents and halogen accent lighting illuminate lobbies and other public spaces

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Wilmington, DE

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Structural Option
Faculty Advisor: Dr. Boothby

April 3, 2006

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EXECUTIVE SUMMARY

Perhaps more than in any other type of structure, high-rise buildings always have a premium need for space: maintaining rentable floor space and sufficient floor thickness while meeting overall height requirements of local building codes. The River Tower at Christina Landing is no different. In the initial design, a flat plate, post-tensioned flooring system was able to minimize floor thickness, thereby controlling the overall building height. This maximizes useable floor area while satisfying code requirements at a critical benefit to the realtor. However, this flooring system results in sizeable columns, and a clustered column layout that hinders future renovation or versatility in the architectural layout.

In this report, the feasibility of a steel staggered truss system is considered, while detailing the existing conditions for the River Tower. Through structural analysis, cost estimates, and research into the existing and coordinating fire protection system of the building, it has been determined that the staggered truss system is not a cost effective solution for the River Tower at Christina Landing. This situation is not without its benefits, as building weight has been reduced, and the floor plan under the new system has the ability to be more flexible in the planning of architectural spaces. However, the River Tower's towering height and location in Wilmington, DE, where steel is at a high premium, negate the potential benefits of staggered truss construction. The existing post-tensioned flat plate construction remains the most efficient design for the River Tower at Christina Landing.

Existing Building System Background

Project Team

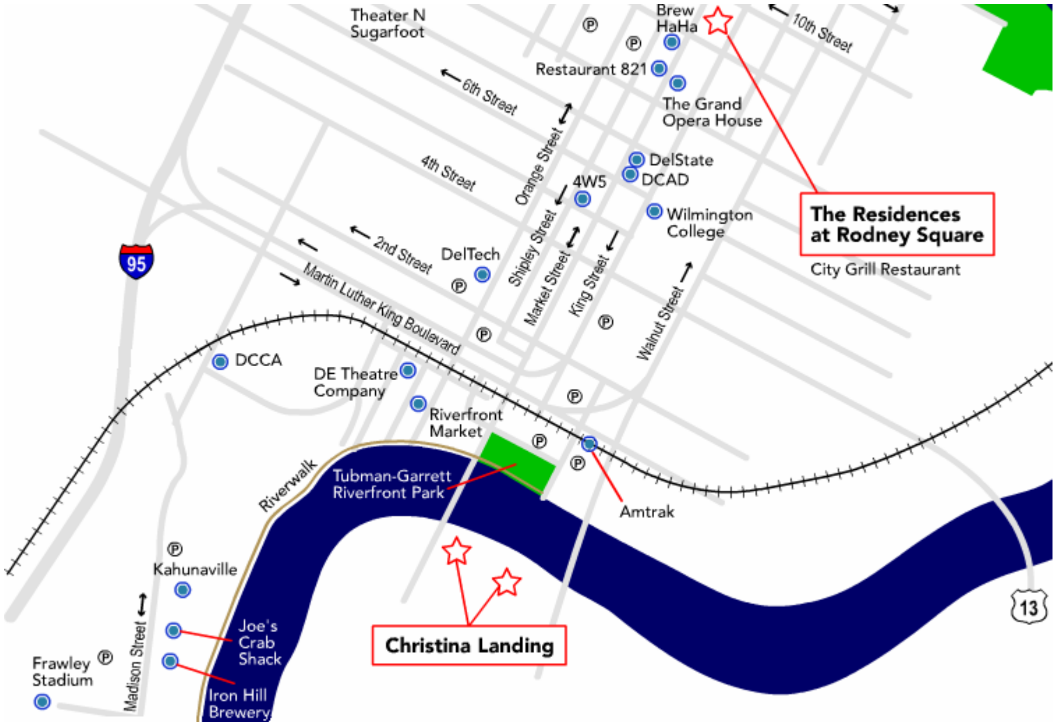
Owner The Buccini Pollin Group *http://www.bpgroup.net*	
Architect and MEP Engineers Burt Hill Kosar Rittleman Associates *http://www.burthill.com*	
General Contractor Gilbane Construction Company *http://www.gilbaneco.com*	
Civil Engineer Pennoni Associates, Inc. http://www.pennoni.com	
Structural Engineer O'Donnell & Naccarato, Inc. *http://www.o-n.com*	
Elevator Consultant Lerch Bates & Associates, Inc. *http://www.lerchbates.com*	

(Logos and images courtesy of each respective firm's websites)

Building Overview

The River Tower at Christina Landing is an integral part of the redevelopment of the Christina River waterfront in Wilmington, DE. Phase one of the riverfront project involves the construction of sixty-three luxury townhouses and a 22-story apartment tower connected by a 2-acre park adjacent to Christina River. Phase two of the redevelopment involves the construction of a second tower, the 25-story River Tower at Christina Landing (the "River Tower") on what is currently a parking lot for the first apartment tower.

Map of Wilmington, DE, Courtesy of http://www.mapquest.com



Local Map of Wilmington and Christina River, Courtesy of http://www.christinalanding.net

Building Architecture

The first floor of the River Tower contains retail space and various management and mechanical rooms on the south side of the building, with the entrance to the parking garage on the northern side of the floor. The second through seventh floors of the River Tower are comprised of a parking garage on the north side and six units on the southern side. The additional spaces required by the parking garage result in a wider base to the building, as the lowest eight floors have a wider footprint than the remaining seventeen floors. The eighth floor consists of a Great Room, Fitness Center, six luxury units, along with an outdoor terrace containing a rooftop pool, spa, garden, and observation deck. The other floors contain eight units a piece, each with master bedrooms and baths. These luxury units range in size from one- and two-bedroom dwellings, each unit having access to its own terrace. The largest units contain dens and multiple terraces.

Floor Area Usage

Floor/ Level	Residential Spaces		Parking Garage/ Other Use		Total Floor SF (*: Estimated)
	SF	% of SF	SF	% of SF	
1	11105	37.71	18344	62.29	29449
2	9812	33.08	19851	66.92	29663
3 to 6	9812	32.19	20674	67.81	30486
7	9812	24.80	29748	75.20	39560
8	19851	62.80	11759	37.20	31610
9 to 22	12186	100.00	0	0.00	12186
23	5724	61.50	3583	38.50	9307
24 to 25	0	0.00	1070	100.00	1070*

Exterior Rendering of the River Tower, Provided by Burt Hill Kosar Rittleman Associates

Building System Information

Building Envelope

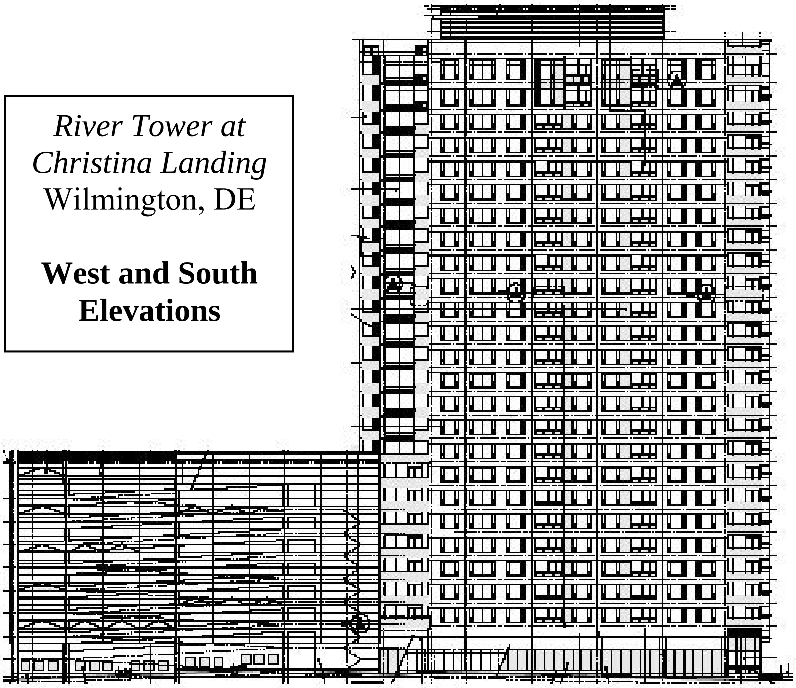
Brick-faced pre-cast panels line most of the exterior walls. The entrances on the eastern and southern side of the high rise possess an aluminum glass curtain wall system. The main entrance on the southern side has a canopy with aluminum composite panel system cladding. On the ground floor, a concrete block recessed wall painted black dominates the western elevation, partially obscured by a green-screen fence. An 8” composite aluminum panel reveal lines the building at its uppermost floors. An aluminum storefront glass system is used on the stair towers. Sliding doors open from the units to individual terraces (some penthouses have even two terraces), which are lined with an aluminum perforated railing systems, consistent with the open spaces of the parking garage. Horizontal and vertical metal panel systems hide the rooftop mechanical systems. The rooftop terrace, which houses the pool and observation deck, is lined with 6”x6” wood columns with vertical wood infill panels. An expansion joint exists where the edge of the parking garage roof/terrace meets the narrower part of the high rise beginning at level 8. The roofing system will be an adhered membrane/roofing flashing system coated with a water proofing sealer.

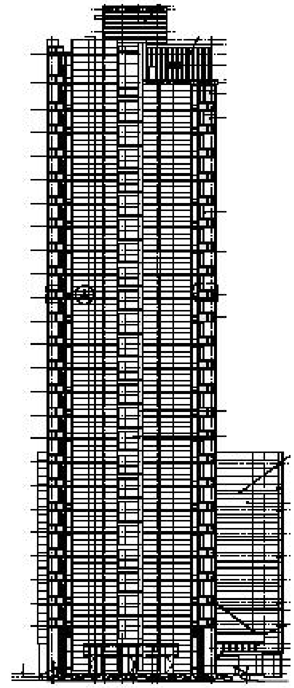
Zoning and Historical Use

The redevelopment site is a former tannery and oil storage yard. The site is now zoned as W-4 Waterfront Residential, Commercial District. A number of variances were required due to the normal height restrictions for an apartment building by the city of Wilmington. Normally limited to a maximum height of 72 feet, a variance and an amendment was needed to construct the River Tower. Another variance was necessary when the building coverage of 0.86 building/lot sq. ft. exceeded the normal 0.5 ratio. Finally, the floor area ratio of 5.97 required a variance of the accepted 2.00 ratio.

River Tower at Christina Landing
Wilmington, DE

West and South Elevations





Electrical Summary

The power distribution for the River Tower has both standard 480/277V and 208/120V. Two utility company transformers, one a 408V and the other a 208V, bring electrical service into the building. The 208V transformer feeds into a 208V, 100KAIC switchboard with two 2500A bolted pressure switches. Each pressure switch has TVSS protection and customer metering and supply 2500A residential bus duct risers rated at a minimum of 100KAIC.

The 480V transformer feeds into three main areas. This 480V transformer feeds directly into the fire pump control for fire suppression, and the transformer also sends electrical power into the retail space. The transformer also feeds into the main distribution panel, which is a 3000A, 480V switchboard, 65KAIC minimum rated, with a 3000A bolted pressure switch with GFCI protection, TVSS, utility and customer metering. This main distribution panel itself serves: a 1000A automatic transfer switch which feeds emergency power panels; a 480/277V, 3

phase, 4 wire, 600A MCB, 65 KAIC min. panel for the water booster pumps; a 1000A bus duct riser to the MDP in the penthouse; two 164kW chillers; as well as a 1200A, 480V distribution panel that serves the pool area, corridor lighting, among other things. There is a 480Y/277V, 800kW, 1000kVA emergency generator present to supply power to the fire pump control and critical loads in the building. Two automatic transfer switches, rated at 1000A and 600A, regulate this emergency power.

Some major risers include a 1200A mechanical bus duct and an 800A emergency riser leading to the mechanical penthouse on the 24^{th} floor. Two 2500A residence bus ducts service the 2-15^{th} and 16^{th}-23^{rd} floors respectively. Each condominium unit possesses an individual load center, all rated 208/120V, 1 phase, 3 wire, 125A MCB despite the differences in square footage. These load centers contain circuit breakers for receptacles, garbage disposals, dishwashers, and unit lighting.

Lighting Summary

The outdoor areas immediately surrounding the building will be lit using compact fluorescent triple tube downlights under canopies. Ceramic metal halide spotlights buried within the concrete will run along the setbacks along the perimeter of the building. Metal halide lamps, both surface and pole-mounted, will handle the rest of the exterior ground lighting. The main entrance to the building will be illuminated by recessed halogen PAR38 accent lighting. Recessed compact fluorescent downlights and wall mounted fluorescents will light the connection between the parking garage and main corridors of the ground floor lobby. Pole-mounted ceramic metal halide adjustable spotlights will illuminate most of the pool-area on the terrace level outside the eighth floor. The rest of the terrace will be lit by patio steplights and metal halide uplights for custom columns. The great room will have six pendant compact fluorescent triple tube uplight fixtures even distributed above the room. Halogen downlights will fill in the rest of the lighting requirements of the great room.

Mechanical Summary

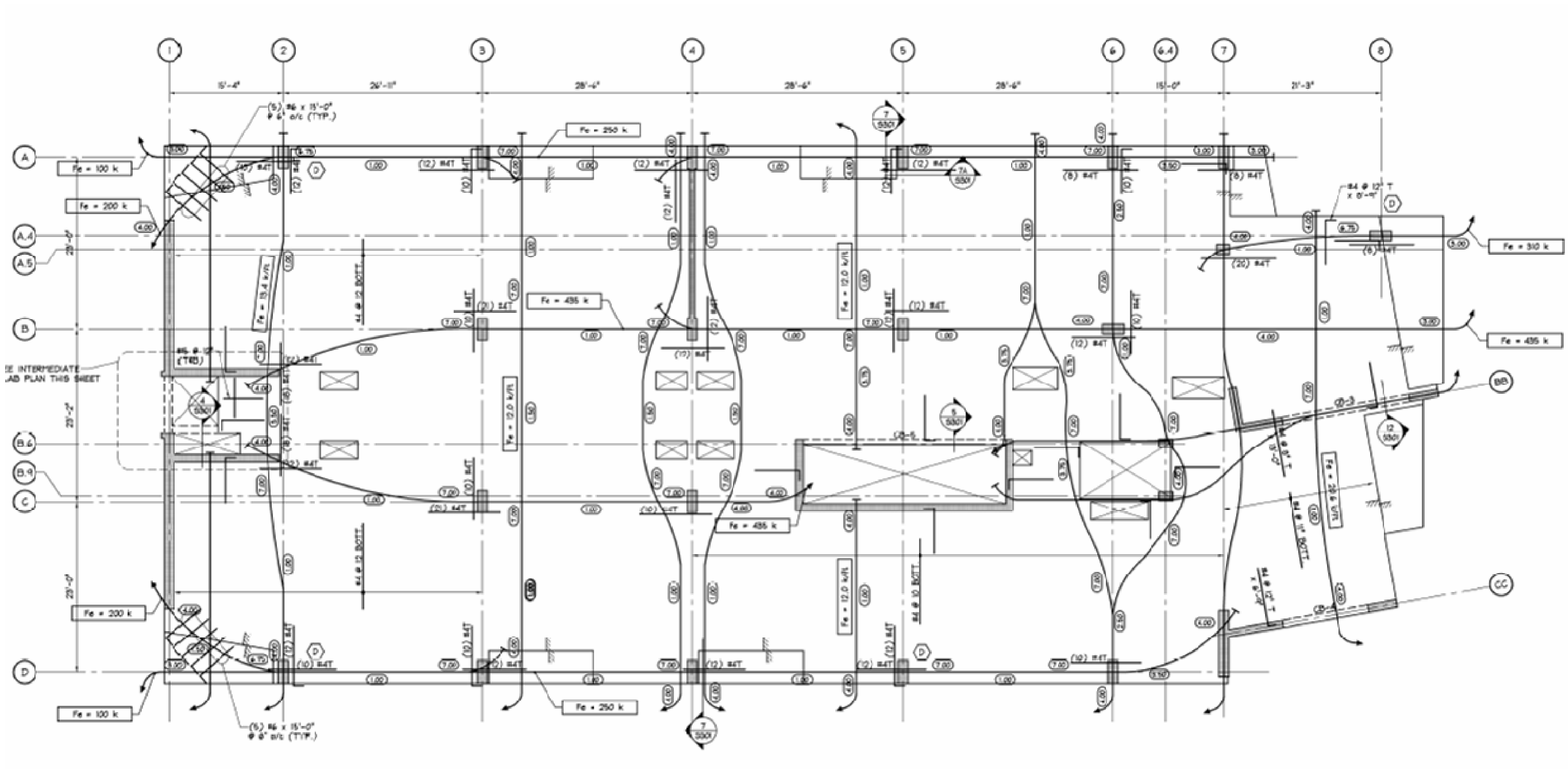
A constant volume heat recovery air handling unit located on the 23rd penthouse/ mechanical level provides forced ventilation to reduce indoor air pollution. Stairwell pressurization fans on the 24th level roof provide smoke ventilation in the event of a fire. 325-ton capacity cooling towers on the same level supply the building with the use of chilled water pumps. Air handlers service the first floor telecommunications room and the elevator equipment room, rated at 900 CFM and 1750 CFM respectively. 400W electric radiant panels heat individual units and rooms such as the fitness center and great room. Two exhaust fans run continuously on the roof and together with natural outdoor air ventilation, maintain natural room pressure. This counterbalances the negative pressure created by the exhaust fans present in each units' bathrooms.

Plumbing Summary

A domestic water booster located at the ground floor services the domestic water system in the River Tower. Four storage tankers are connected to the two main water heaters located on the 23rd floor. These water heaters are gas powered. Two cooling towers are located one level above, on the 25th floor. Storm water conductors are located throughout the perimeter of the building to collect rain water runoff. The pool equipment room is located on level 7.5, which can be accessed from the parking garage. There is no gas pipe access for individual units.

Major National Codes

<u>Primary Code:</u> BOCA 1996 with amendments adopted by the city of Wilmington
<u>Fire Protection:</u> BOCA-1999, Philadelphia Fire Protection Code, NFPA-13 Automatic Sprinklers



Typical Floor (Levels 9-21) of River Tower Condominium:

Provided by O'Donnell & Naccarato, Structural Engineers

Existing Structural System:
Post-Tensioned Slab with Concrete Columns and Shear Walls

Existing Conditions

Condominium Tower

The condominium building will be supported by a deep foundation system that will support the columns, walls, and slabs. The piles will be HP12×84 steel piles driven to 225 tons with a net bearing capacity of 200 tons. These piles will be grouped at columns and transfer load from columns using pile caps. A typical interior pile cap will be 7’-9”×11’-0” and 38 inches thick, with reinforcement in both directions. An exterior pile cap will be 7’-9”×7’-9” with 4 piles and a depth of 32 inches. Concrete grade beams span from column pile cap to pile cap and support the exterior walls of the building. The first floor slab will be a 12 inch thick concrete slab with #7 reinforcement at 12 inches on-center each way, top and bottom.

The condominium building floors will have 8 inch thick post-tensioned concrete slabs. The slabs span between columns spaced at 28’-6” in one direction and 23’-0” in the other direction. A typical interior column is 16”×52”, and its reinforcement and concrete strength decreases at upper floors. The exterior columns are 16”×36”. Concrete shear walls (varying 12-16 in., depending on location) provide lateral resistance and are located generally around elevators and stair towers and are scattered throughout the plan. The mechanical penthouse roof will be framed by steel beams spaced at 6 ft. on-center with 1 ½” deep, 22 gage roof deck spanning in between these beams. The mechanical area will be enclosed by metal panels with steel stud support. The cooling tower will similarly be enclosed with metal paneling, with a structural channel girt system to support it.

Parking Garage

For the parking garage, additional steel piles (80 ton HP12x53) will be added at approximately 20 feet on-center to support the lowest level slab. The exterior columns will have 9’-0”×9’-0”×3’-0” deep pile caps with (5) HP 12×84 piles. The interior walls will have a 6’-0” wide grade beam with HP12×84 piles on each side of the wall, spaced 8’-0” on-center. The slab spanning these piles and columns will be the same as the apartment building slab.

The floor framing of the parking garage will be 34 inch deep pre-topped double tees which span between 45 to 60 feet. An “L” shaped beam makes up the exterior of the building and support the pre-cast tees. These L beams will span approximately 48 feet from column to column. The interior support, including the support of the sloping tees, will be supported by 12 inch thick pre-cast light wall. The exterior pre-cast columns will be approximately 24”×36”. 12-inch thick shear walls located throughout the plan will resist the lateral loads on the parking garage. In the northern

Interface Between Condominium Tower and Parking Garage

For the first seven floors of the combined, the parking garage adjoins the condominium tower, and is separated by an expansion joint which spans the full depth of the parking garage. This parking garage was considered a stabile and completely independent entity from the tower in the scope of this report**,** and further research was concentrated on the condominium tower structure and its systems.

Structural Material Specifications

Concrete

- Foundations (Pile Caps and Grade Beams): 6,000 psi normal weight
- Slab on Grade: 4,000 psi normal weight
- Post Tensioned Slabs and Beams: 5,000 psi normal weight
- Columns: 5,000 and 6,000 psi normal weight
- Precast Garage Panels: 5,000 psi concrete

Concrete Reinforcing

- Deformed Reinforcing Bars: ASTM A615 Grade 60
- Welded Wire Fabric: ASTM A185

Structural Steel

- Wide Flange Shapes: ASTM A992
- M, S, Channels, Angle Shapes: ASTM A36
- Hollow Structural Steel: ASTM A500 Grade B
- Structural Pipe: ASTM A53 Grade B

Existing Gravity Loading

Live Loads

Area Type	Provided Design Values	Table 1606 of BOCA 1996 Code
Parking Garage	50 psf	50 psf (Passenger cars only)
Balconies	60 psf	60 psf (One- and two-story dwellings that do not exceed 100 sq. ft.)
Exit Stairs	100 psf	100 psf (Fire Escapes)
Tower Floors	40 psf	40 psf (Dwelling units and corridors)
Partitions	20 psf (where applicable)	20 psf minimum (by 1606.2.4 of code)
Terrace	100 psf	100 psf (Exterior balcony)
Mechanical Rooms	150 psf	
Elevator Machine Room	150 psf	

Live Load Calculation Results

Floor/Level	Primary Usage	Total LL per floor (kips)	(psf)	With 50% Reduction
1	Parking/Residential	1461.35	49.62	24.81
2	Parking/Residential	1486.68	49.70	24.85
3 to 6	Parking/Residential	1514.48	49.68	24.84
7	Parking/Residential	1968.19	49.75	24.88
8	Residential/Terrace	2148.59	67.97	33.99
9 to 22	Residential	597.11	49.0	24.5
23	Penthouse/Mechanical	926.05	99.5	49.75
24 to 25	Mechanical	160.5	150	75
Roof	-----	365.58	30	15

Dead Load Calculation Results

Self Weights Per Level					
Level	**Column (k)**	**Slab (k)**	**Shear Wall (k)**	**Total (k)**	**Total (psf)**
Roof	N/A	N/A	374.73	400*	373
24 to 25	20.13	104.86	374.73	499.72	467
23	42.73	912.09	374.73	1329.55	143
9 to 22	42.73	1194.23	384.1	1621.06	133
8	59.01	3097.78	483.54	3640.33	115
7	55.96	3876.88	483.54	4416.38	112
3 to 6	55.96	2987.63	483.54	3527.13	115.7
2	61.99	2906.97	483.54	3452.5	116.4
1	58.76	2886	483.54	3428.3	116
				54469.86	4234.2

Roof and Snow Loads

- Minimum Roof Live Load: 30 psf
- Ground Snow Load: 30 psf
- Snow Load Importance Factor: 1.0
- Snow Exposure Factor: 0.7
- Thermal Factor: 1.0
- Flat Roof Snow Load: 14.0 psf (Specified in construction documents as 20 psf minimum)

Drift and Deflection Criteria: As provided by O'Donnell & Naccarato, Structural Engineer:

- Lateral wind and seismic loads:
  - Interstitial drift: L/400 *(where L= floor-to-floor height)*
- Vertical gravity and live loads:
  - L/360 under live loads
  - L/240 under total load *(where L= span of member under consideration in both cases)*

Existing Lateral System

Simplified Building Schematic: Complete Footprint

Key:
- *- Tower Columns*
- *- Tower Shear Walls*
- *- Parking Garage Columns*
- *- Parking Garage Shear Walls*
- *- Expansion Joint*

Solid Lines: Tower Perimeter *(whole building)*
Dashed Line: Parking Garage Levels
(Base to Eighth Floor)

The River Tower at Christina Landing uses reinforced concrete shear walls as its primary lateral resistance, with help from the large rectangular concrete columns oriented perpendicular to the controlling wind loading in the wide direction. The greatest amount of lateral resistance is provided on the lower levels to account for the largest shear forces. Additional shear walls are located on the lower parking garage levels (the lower eight levels of the building), mostly near elevator and stairwell openings and the eastern walls, as shown in the diagram above. The shear walls located in the condominium tower, which stands the full 25 stories of the building, are relatively consistent in location and size, with occasional openings left for stairwells, elevators, and other architectural features. The thickness of these common stairwells is relatively

consistent, although the concrete strength for the shared parking garage/condominium levels (foundation to eighth floors) of this tower is 6000 psi. From the ninth to 25th floors, the concrete strength for the tower shear walls decreases to 5000 psi. Please consult the included concrete shear wall schedule below for more detailed information.

The parking garage areas are similar to the condominium tower in that reinforced concrete shear walls and thick columns provide the lateral resistance. The main structural system for the parking garage is a light precast concrete wall system with precast columns. The floor system consists of a pre-topped double-tee system, as noted previously.

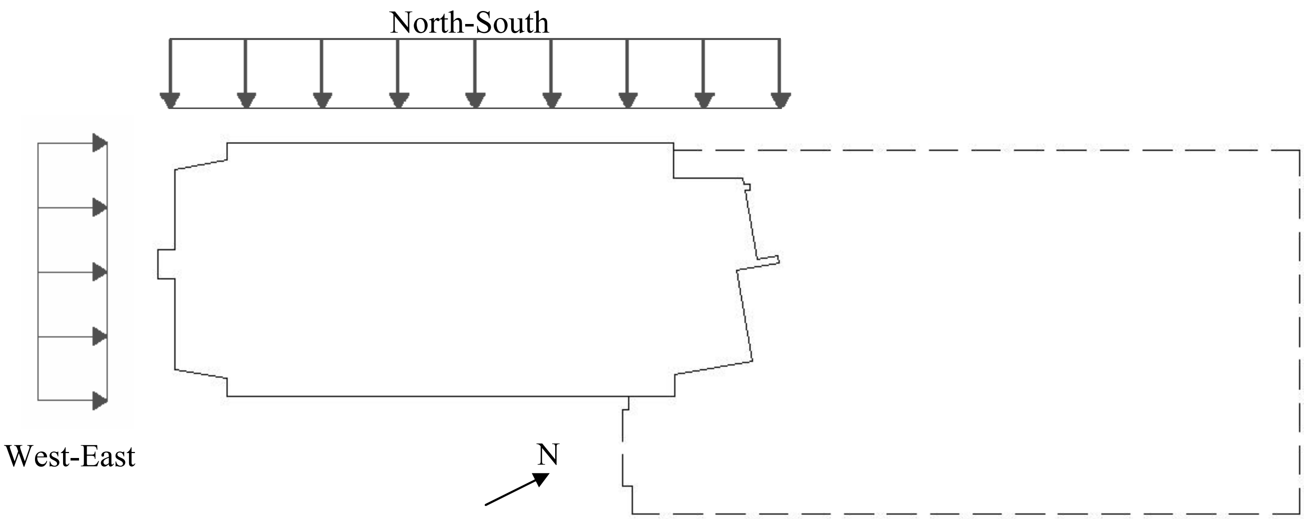
CONCRETE SHEAR WALL SCHEDULE

LEVEL \ MARK		SW1	SW2	SW3	SW4	SW5	SW6	SW7	SW8	SW9	SW10	SW11	SW12
15TH - ROOF	VERT. STL.	#4 @ 12	#8 @ 8	#6 @ 12	(22) #9	#7 @ 12	#9 @ 12	#9 @ 12	(18) #9	#7 @ 12	-	#9 @ 12	-
	HORIZ. STL.	#4 @ 12	#4 @ 12	#4 @ 14	#4 @ 12	#4 @ 12	#4 @ 12	#4 @ 12	#4 @ 12	#4 @ 12	-	#4 @ 12	-
	f'c	5000	5000	5000	5000	5000	5000	5000	5000	5000	-	5000	-
	THICKNESS	16	12	9	12	12	12	12	12	12	-	12	-
8TH - 15TH	VERT. STL.	#5 @ 12	#10 @ 8	#11 @ 10	(22) #11	#9 @ 12	#11 @ 8	#11 @ 8	(18) #11	#9 @ 12	-	#11 @ 8	-
	HORIZ. STL.	#4 @ 12	#4 @ 12	#4 @ 14	#4 @ 12	#4 @ 12	#4 @ 12	#4 @ 12	#4 @ 12	#4 @ 12	-	#4 @ 12	-
	f'c	5000	5000	5000	5000	5000	5000	5000	5000	5000	-	5000	-
	THICKNESS	16	12	9	12	12	12	12	12	12	-	12	-
FND. - 8TH	VERT. STL.	SEE DETAIL SW1/S200	#10 @ 8	#11 @ 11	(22) #11	#9 @ 12	#11 @ 8	#11 @ 8	(18) #11	#9 @ 12	#14 @ 13	#11 @ 8	#14 @ 13
	HORIZ. STL.		#4 @ 12	#4 @ 14	#4 @ 12	#4 @ 12	#4 @ 10	#4 @ 10	#4 @ 12	#4 @ 12	#5 @ 10	#4 @ 12	#5 @ 10
	f'c	6000	6000	6000	6000	6000	6000	6000	6000	6000	5000	6000	5000
	THICKNESS	16	12	9	12	12	12	12	12	12	24	12	24

Concrete Shear Wall Schedule from Sheet S200, Courtesy of O'Donnell & Naccarato

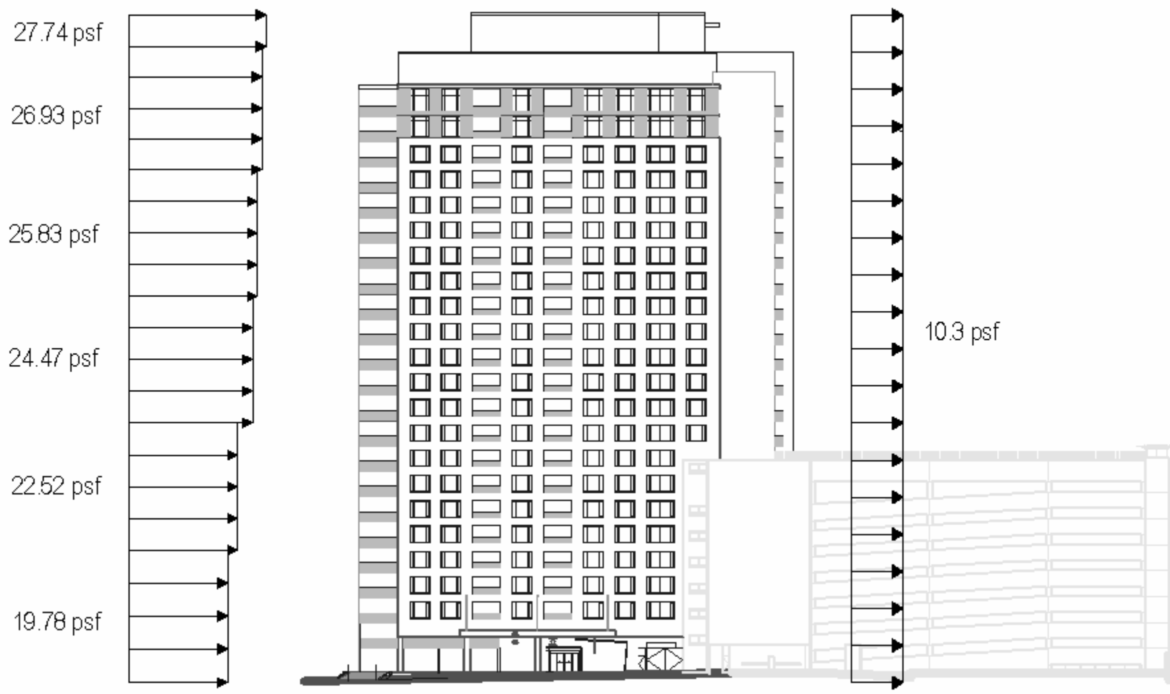
Wind Loading Criteria

- Wind Importance Factor: 1.04
- Wind Exposure: C
- Components and Cladding Loads: vary per code requirements
- Load Diagrams with results provided on next page

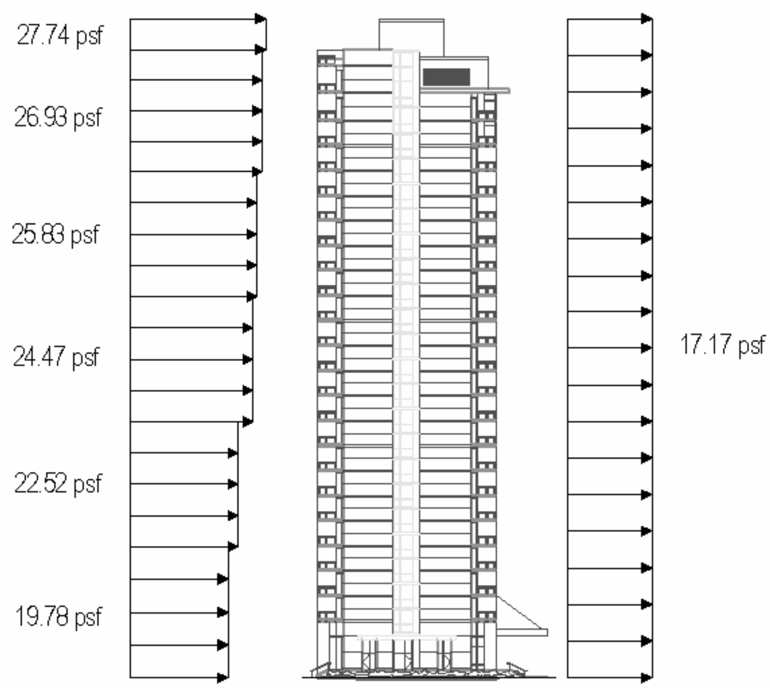


Wind Direction Schematic:
Condominium Tower shown in solid line
Parking garage shown in dashed line

When compared to the seismic loading results, the wind loading controlled as the primary source of lateral loading. This is to be expected, as the site of the building is in Wilmington, DE and along the riverfront. This riverfront location provides the reasoning behind the choosing of Wind Exposure category "C," which differs from the information on the project's Structural Narrative. This, however, provides larger loads and therefore, a more conservative analysis of the lateral system. Diagrams of the wind pressures in both major directions of the building are provided on the following page.



Wind Pressures (psf) in West-East Direction



Wind Pressures (psf) in North-South Direction

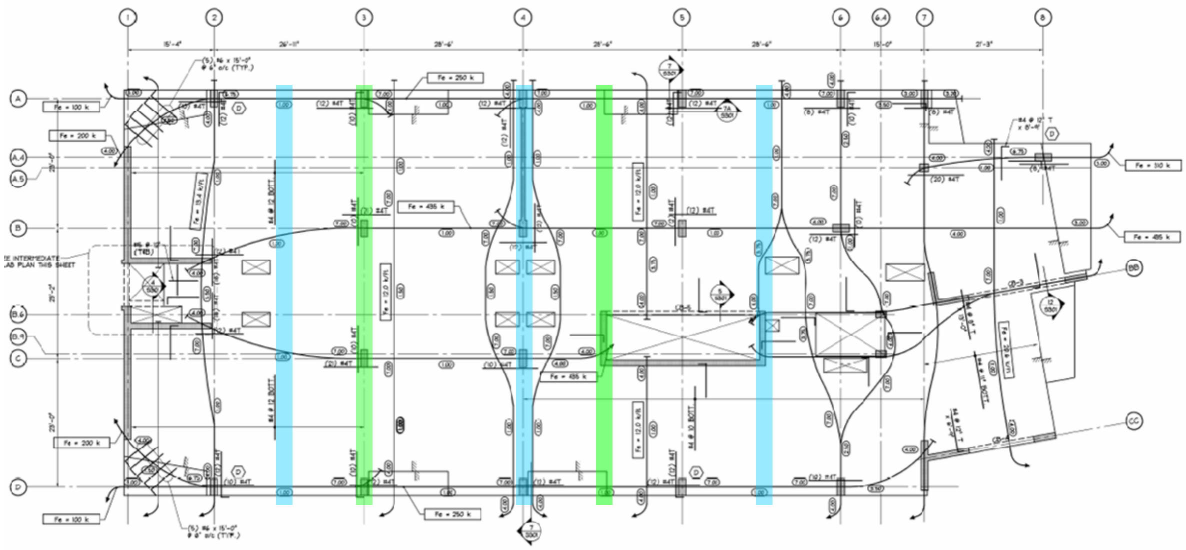
Seismic Loading Criteria

- Seismic Importance Factor: 1.0
- A_v (Velocity related acceleration coefficient) = 0.075
- A_a (Peak acceleration coefficient) = 0.05
- Seismic Design Category: B
- Basic Seismic Force Resisting System: Dual system with shear wall and intermediate concrete frame
- Response Modification Factor, R = 6
- Site Coefficient, S_4 = 2.0
- Analysis Procedure Used: Equivalent Force Method
- Base Shear = V = 849.73 kips

Structural Design and Theory

The riverfront location of the River Tower necessitated the use of piles as foundation support, as a spread foundation would not be sufficient in construction so close to the riverbed. The shear walls provide the lateral resistance for the structure, while the flat plate post-tensioned slabs distribute the gravity loads. Part of the reason for the choosing of a post-tensioned flat plate slab, as opposed to another type of two-way or a reinforced slab, is its improved resistance to punching shear. Whereas a reinforced flat plate system would most likely require drop panels or column capitals to provide this necessary shear resistance, the post-tensioning element provides this benefit without additional slab depth. This allows for speedier construction, and ultimately more cost- and space-efficient structures.

Proposed Structural System:
Staggered Truss System



Typical Floor (Levels 9-21) of River Tower Condominium:

Provided by O'Donnell & Naccarato, Structural Engineers

Existing floor plan shown for comparison
Proposed staggered truss locations shown in blue (even floors) and green (odd floors)

Introduction

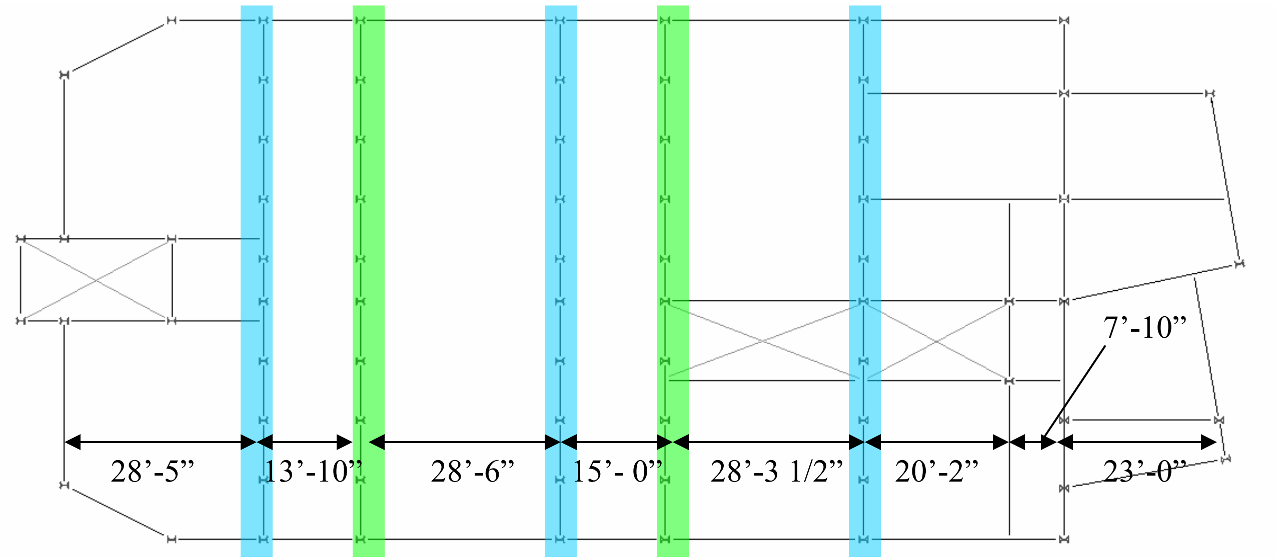
Previous technical research of the River Tower's systems confirmed the adequacy of the current post-tensioned concrete floor slab system. This system provides an efficient balance of minimal floor-to-floor height, system dead weight, and assembly installation cost. A significant reduction of system weight can be achieved using a steel framing system with precast concrete plank flooring. Ordinarily, floor thickness is a primary drawback to steel construction. However, staggered truss designs have provided a minimum amount of floor thickness using precast concrete plank flooring systems which can even rival cast-in-plate and post-tensioned flat plates in specific applications.

Utilizing the prefabrication of the trusswork and flooring systems, a staggered truss system has the potential to have very efficient erection and installation times. The elimination of most of the wet trades from the existing configuration allows for all-weather construction, which could further reduce erection times. Together with a reduction in system weight, the further spacing of the column layout can result in much greater flexibility in the design of interior spaces, and allow for more opportunity for renovation in the future. Shear walls would no longer be necessary, as trusses oriented perpendicular to the width of the building would be able to support the controlling lateral loads through its bracing. Further architectural benefits include using the precast plank and its topping as a finished floor and ceiling.

The non-symmetrical plan of the River Tower, though basically rectangular, necessitates the use of moment frames in these irregularly shaped areas where the trusswork could not logistically or architecturally be placed. These moment connections add a significant cost premium to the structure, and require complicated and expensive erection. The existing architectural layout of the River Tower was utilized to the fullest extent, which results in irregular spacing of the staggered trusses. The intent was to integrate the proposed structural system into the existing systems as much as possible. The staggered trusses were designed and placed where architects already have shared party walls between condominium units. This minimizes the number of interior columns, and results in smaller steel columns versus the existing often 16” by 52” concrete columns. This has the potential to open up individual units considerably, and result in designs with more interior freedom for designers and tenants alike.

Along with the logistics and cost benefit of changing the primary construction material from concrete to steel, there is also the issue of fire protection. This is especially critical in such a tall building, where evacuation procedures, fire and smoke control, and structural stability are controlling factors. These issues will be investigated, calculated, and analyzed in the following pages and in the accompanying appendix.

Structural Analysis



Proposed System Typical Floor Plan for Levels 9-21

Steel shown in black; even floor trusses highlighted in blue and odd floor trusses in green

Overview

As mentioned, the staggered trusses were designed in respect to the current architectural layout of the River Tower. These trusses were placed in the existing infill walls of the condominium units, as illustrated in the proposed system floor plan shown above. As this diagram displays, the architecture from the existing River Tower design has gone virtually unchanged, in order to accurately compare the adequacy of both systems. This has complicated the truss layout, which ordinarily relies on symmetry to work efficiency. The precast planks will span from truss to truss, left to right on the diagram, and have been sized for a 30' span. The precast plank chosen for this proposal is the 8"×4' SpanDeck by Nitterhouse Concrete Products. Please consult Appendix C for more details on this selection.

Consistencies in Design

Because the exterior factors of the building, such as footprint, height, and overall shape, were not affected by the proposed staggered truss system, the wind loading under this proposal is very similar to those of the existing conditions. Please consult page 18 of this report for a detailed wind loading diagram on the building. Updated calculations and the complete analysis procedure can be found in Appendix A. The live loading condition remains the same from the existing system, which appears on page 13 of this report and is detailed in Appendix C. Additional dead load has been introduced by the use of the 8” precast plank system, which adds an additional 82.5 PSF. The existing precast exterior wall system is self-supporting, and was used in the proposed system. Therefore, the staggered truss system was not required to support this loading. As stated previously, the parking garage that interfaces with the first seven floors of the River Tower has been assumed to be self-supporting as well, and is not within the scope of this analysis.

Proposed System Lateral Loading Conditions

Seismic Loading

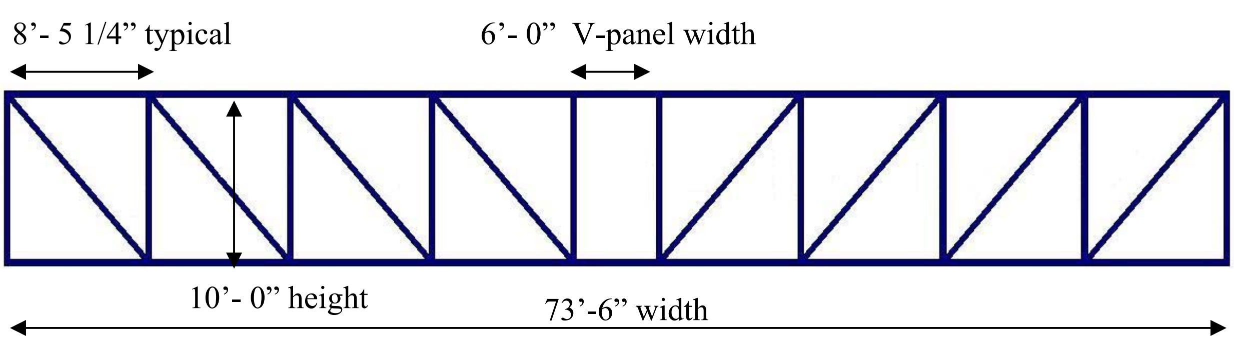
The proposed staggered truss system results in a lower system dead weight for the entire structure, resulting in an even lower seismic loading condition. As with the existing design, wind loading controlled over seismic loading in either direction. However, with the proposed structural configuration, there is an even greater discrepancy between these lateral conditions. The staggered truss system has been sized to withstand floor shear loads from the controlling case, that of the wind in the North-South direction. Lateral forces in the West-East direction are resisted by the moment frames on the ends of the floor plan. Detailed calculations can be found in Appendix B, and a summary of both controlling directions of seismic and wind loading is included on the following page. The controlling wind loads from the North-South Direction, which was consistent with the analysis of the existing building, was used in the lateral design of the staggered truss system.

Proposed System Wind and Seismic Forces Summary (kips)						
Floor	Wind (N-S direction)			Seismic		
	Lat Load	Story V	Φ_h	Lat Load	Story V	Φ_h
Roof/25	38.55	38.55	0.02	153.31	153.31	0.10
24	84.71	77.11	0.04	144.51	297.83	0.20
23	92.08	161.82	0.09	132.84	430.67	0.29
22	89.03	253.90	0.15	122.90	553.57	0.37
21	81.70	342.93	0.20	111.18	664.75	0.44
20	76.95	424.63	0.25	103.42	768.17	0.51
19	76.44	501.58	0.29	93.96	862.13	0.58
18	75.96	577.54	0.34	86.40	948.52	0.63
17	75.46	653.00	0.38	77.81	1026.33	0.69
16	74.93	727.94	0.42	70.86	1097.19	0.73
15	74.43	802.37	0.47	63.15	1160.35	0.78
14	73.93	876.30	0.51	56.46	1216.80	0.81
13	73.39	949.69	0.55	49.94	1266.75	0.85
12	72.78	1022.47	0.60	43.84	1310.59	0.88
11	72.15	1094.63	0.64	38.16	1348.74	0.90
10	71.37	1165.99	0.68	32.71	1381.45	0.92
9	70.57	1236.56	0.72	27.95	1409.40	0.94
8	69.75	1306.31	0.76	23.31	1432.71	0.96
7	68.85	1375.16	0.80	18.91	1451.62	0.97
6	69.50	1444.66	0.84	15.05	1466.67	0.98
5	69.97	1514.64	0.88	11.62	1478.29	0.99
4	68.79	1583.42	0.92	8.63	1486.92	0.99
3	67.44	1650.86	0.96	6.11	1493.03	1.00
2	65.88	1716.74	1.00	4.00	1497.03	1.00

Design of Staggered Truss System

AISC *Design Guide 14: Staggered Truss Systems* was used to determine initial sizes for the members of the staggered trusses. This design guide details the calculation of transverse shear through the rigid floor diaphragm made by the precast plank flooring system. This diaphragm transfers the shear forces taken from the staggered trusses into the remaining trusses. This creates a deep beam condition in the diaphragm that uses the trusses as “drag struts.” Moment frames were used in non-rectangular spaces and where the architectural layout did not warrant truss placement.

The trusses were designed with six-foot Vierendeel panels, to span where the main corridor was located. Fortunately for this design, these corridors were located exactly in the middle of the floor plan and allowed for a symmetrical geometry to be used. A basic diagram of the typical truss design is shown below. The remaining shape was divided into four equal quadrants 8’-5 ¼” in length on each side of the Vierendeel panel, to support the full 73’-6” width of the River Tower floor in the North-South direction. This shape and design was used at every truss location for ease of design. This truss configuration is known as a Pratt truss, which places the diagonal brace members in tension. This geometry was chosen because of its widespread use in existing staggered truss systems.



Typical Staggered Truss Dimensions

Summary of Hand Calculation Results

The staggered truss system was calculated to account for both direct shear and the torsional rigidity of this specified truss configuration. Because the shear force at each floor can be centralized at that particular floor's center of mass, and the trusses have their own different center of rigidity, there is an amount of torsion to account for in the lateral resistance from the resulting eccentricity. Accidental eccentricity was also considered, although because seismic loading did not control over wind loading, plan irregularity was not accounted for in these calculations, which appear in Appendix C of this report. Lateral loads were distributed under the assumption that the planks provided a rigid diaphragm, a simplified assumption. Transverse shear taken by the diaphragm formed by the precast floor planks was checked to ensure structural stability. The method of joints analysis was used to distribute separate gravity and lateral loads to each member of the truss. Load coefficients were then used to simplify load combinations based on LRFD and ASCE methods. Resulting calculations yielded various sized W10 members for the truss chords, large W12 and W14 sections for the truss columns, and HSS 10×4×1/2 for the diagonal members.

Summary of ETABS Analysis

ETABS Nonlinear was also used to verify these assumptions for truss member sizes, as well as the additional moment frames of the structure. The computer output from this program yielded much larger members than what was expected based on the hand calculations. Truss chord members, for example, were sized as W36 members by the analysis program, and even larger W14 sections were results for the columns. This occurred even though the loading matched the same criteria as the hand calculations. The discrepancy between these results can be explained by several factors. Staggered truss designs rely on the composite behavior between precast planks and the steel trusses, especially after grouting has been applied to secure this

connection.[1] It is quite possible that the computer model did not properly render this bonding condition.

Several alternate modeling possibilities were investigated, including changing the rigid diaphragm of the floor planks to a more realistic flexible diaphragm, as recommended by technical literature from the makers of *ETABS*.[2] This allows the program to more properly model the shear stresses that develop in the precast diaphragm and includes diaphragm deformation in its results. This flexible diaphragm condition resulted in slighter smaller chord sizes than previous models, but still larger than the expected hand-calculated sizes. With this reduction in chord size, the exterior truss columns failed or were massively sized, resulting in even more discrepancy with the hand calculations. A similar condition developed when three chord members were used in place of the typical one continuous member spanning the full 73.5 foot width of the structure. Again, relatively smaller chord sizes came at a result of much larger column sizes. These computer models may have not properly transmitted the lateral loads from the floor diaphragm to the trusses. The story drift

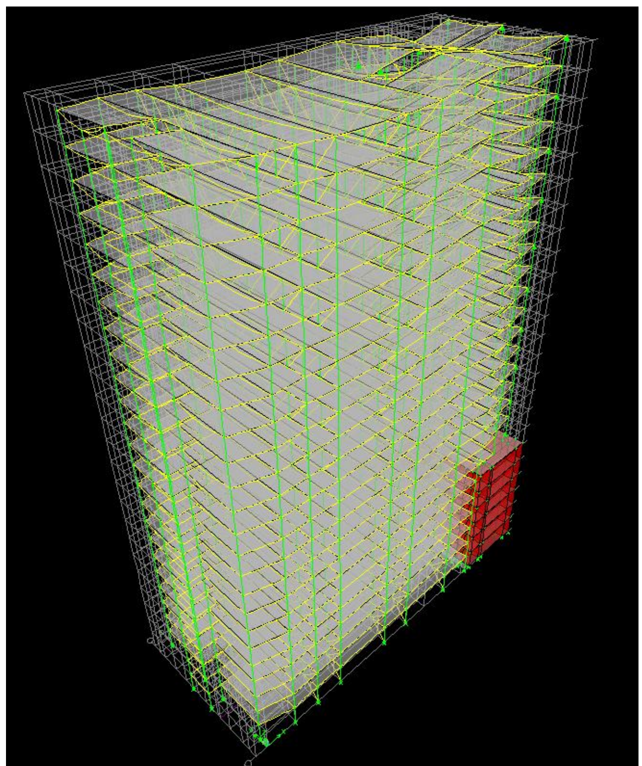


Image of the ETABS Model Deformed Shape, Parking Garage portion shown in red (not part of computer analysis)

[1] Wexler, Neil, and Feng-Bao Lin. *AISC Design Guide 14: Staggered Truss Framing Systems.* American Institute of Steel Construction, 2001.

[2] Habibullah, Ashraf. “Steel Frame Design: Staggered Truss Framing Systems Using ETABS.” Computer and Structures, 2005. <http://www.csiberkeley.com/Tech_Info/StaggeredTrussTechnicalNote.doc>.

results from *ETABS*, which are in the thousandths of an inch, support this notion.

The results of these computer outputs have been provided in Appendix D of this report. A realistic interpretation of these results, based on the inconsistency of the truss chords in particular, is that the final design member sizes likely lie somewhere in between the hand calculations and computer output. The consistency of the hand calculations with those of the design guide, along with previous projects, confirms the adequacy of that process. Of course, these existing projects were much smaller buildings, often no larger than five stories in height, which could also explain this difference in results. The computer analysis, modeled with a rigid diaphragm brought the most successful of the computer results, and was used to price this system in the following cost analysis section. Because of the theoretical nature of this research, it was judged that this was still an accurate assessment to compute system cost, with the size discrepancy accounted for in the final determination.

Impact on Foundation Requirements

The River Tower's current structural system of post-tensioned concrete slabs provides minimal floor-to-floor heights, minimizing the overall height of the building. This is crucial to reduce the overturning moment on the structure. Resistance to overturning is also aided by the overall building weight provided by the concrete slabs, columns, and shear walls. River Tower's location along the waterfront of the Christina River in Wilmington, DE, necessitates the use of deep piles based on the type of soil on the site. This will not change this designation, even with the reduced system weight of the proposed staggered truss system, but it will reduce the number of 200 ton HP12×84 steel piles. Despite the reduction in system weight, the building weight of the 25 stories counteracts the worst overturning moment brought by the controlling lateral load case of the wind in the wide direction. These figures appear after the wind calculations in Appendix A of this report.

Additional Concerns

Most of the floors of this 25 story tower are generally 10.25 feet in height, although particularly at the very lower and upper floors, this dimension changes. For the purposes of this analysis, all floors were assumed to be a consistent 10.25’ height, which would present minor changes to the analysis presented in this report. There were also instances where interior hallways and closets intersected with this truss placement. In this rare situation, proper architectural changes, though minor, would have to be considered. For purposes of this research, it was assumed that any doorways or openings besides the Vierendeel panels could be fit in between the braced members. In reality, this yields a 2’-10” wide doorway with the standard 6’-8” rough dimension, and does not take into account the thickness of chord or column members. Architectural changes were withheld from the scope of this report. These dimensions do not even account for the thickness of the chords, which stand to be at least 10 inches deep, which would further hinder access through these hallway portals within each truss. This does not become an issue for every condominium unit, but provides a significant architectural problem to correct. The thickness of the chords and other structural members can be isolated to the flange widths of these members, but with finishing and fireproofing procedures, this could further restrict ceiling heights. Similarly, the thickness of the HSS brace members in the trusses can result in unsatisfactory thickness in the infill walls which enclose these staggered trusses.

Fire Protection Systems

Existing System

Background

In the event of a fire in a large high-rise building such as the River Tower, a number of factors need to be considered in both the design and operation of such a large public space. Extinguishing the fire is only part of the story as three major objectives of the system dominate system design and operation: protection of life, protection of property, and continuity of operation. Within these objectives, evacuation, smoke control, structural protection, fire spread control, and electronic detection systems are all major issues to consider.

The general building classification for River Tower is mostly light hazard, and rated ordinary hazard (Group 1) for the storage and mechanical rooms, based on the BOCA 1999 Building Code. Most of the areas of River Tower are protected by an automatic sprinkler system. In addition, there are fire hose valves located at each level of the major stairwells. A fire pump room is located on the first level to service the fire protection system. Each stairwell has access to a standpipe for immediate access to water. The concrete slabs, beams, and columns provide passive fire protection by their very nature in the parking garage and loading dock areas. Strobes, fire alarms, smoke detectors, and standard equipment are located throughout the building, concentrated mainly in means of egress areas such as stairwells and corridors.

The post-tensioned concrete flooring and cast-in-place concrete columns of the original structural design provide more than enough inherent fire protection for this type of building. The River Tower falls under a Type 1A construction category, as it rises taller than the 160 ft height requirement of the code. The code provides reductions in fire ratings due to additional fire protective systems, such as automatic sprinkler systems that the River Tower does have, but the amount of floors is simply too great. The River Tower is required to have a two hour fire rating, which indicates that the protective system must sustain the fire for a minimum of two hours to

allow for proper evacuation. It is crucial that the structural system be properly protected, to ensure structurally stability during this evacuation process. The main goal is to extend the time it takes for the heat transfer from the fire to the steel member, thereby reducing its strength and potentially resulting in structural collapse.

Evacuation and Means of Egress

The main residential tower has two main stairwells, in addition to its main elevator core which consists of three elevator shafts. The easternmost stairwell, next to the elevator core, reaches all 25 floors of the building, but experiences a shift in axis as the tower interfaces with the parking garage levels from the eighth floor down to the ground floor. The westernmost stairwell which protrudes out of the western exterior wall rises from the first floor upward to the penthouse level on the 23^{rd} floor, but does not service the upper two floors. The parking garage that abuts the residential tower contains two stairwells, one in the northeastern corner, and the other on the extreme opposite of the garage in the southwestern corner. This southwestern stair also services the outdoor terrace on the eighth level that rests above the parking garage. Two elevators service the parking garage in the southeastern corner of the garage.

Based on the *NFPA Fire Protection Handbook*, the occupant load for a typical residential building consists of one person per gross 200 ft^2 of floor square footage. The typical residential tower square footage per floor of the River Tower is approximately 12,000 ft^2, which yields population per floor of approximately 60 people. With a flow rate of approximately 35 people per minute per 22 in width of stair, and 23 main occupied floors, and two main stairwells, the River Tower has approximately a 10 minute evacuation procedure.

Standpipe and Sprinkler System

The River Tower utilizes a wet pipe, combined standpipe/sprinkler system. This means that water is constantly flowing through the standpipe riser and sprinkler branch system. This

type of system is standard practice for high-rise systems. Fire department hoses only have the ability to reach approximately 75 feet above their water source level with their truck-mounted ladders. Buildings above this height are classified by code as “high-rise” buildings, and the River Tower certainly falls under this distinction. The condominium tower has a standpipe in each major stairwell of its condominium tower to provide instant access to fire hoses on the taller floors.

Smoke and Fire Control

In taller structures, stairwells become the primary means of egress in a fire or similar evacuative event. Elevators are not reliable and should not be used, as smoke especially has the ability to travel upward vertically through the shaft. The River Tower has air pressurization capabilities in these stairwells to ensure that smoke will not penetrate these critical areas and spread throughout the building. When evacuees or fire personnel open the fire doors to the stairwells, clear air is forced out into the floors, keeping smoke and carbon dioxide out. This is an additional fire protection system which helps control the spread of fire and smoke and confines it to the very minimum spaces necessary. Smoke control holds the same significance as fire control, as smoke damage to property and smoke inhalation of evacuees can prove just as devastating in the event of a fire.

Proposed Fire Protection System

Additional Needs for Fire Protection

Because only the structural members of the main residential tower have been changed, the existing fire rating of two hours for the River Tower will still apply. The 8" thick precast plank flooring with 2" concrete coating, provides this two hour rated fire protection in the floor system. The existing combined sprinkler and standpipe system will suffice since the architectural changes will be relatively minor. Therefore, these sprinklers will serve roughly the same square footage and will not need to be resized. The same applies to the standpipe dimensions, as it services the same areas as the previous structural system. The key change in this new building system is the introduction of structural steel members, which can lose up to 40% of its yield strength in fires reaching temperatures of 1000-1300 degrees F. The current active fire protection system of detectors, sprinklers, and other mechanical equipment should be sufficient even after the proposed structural changes. The proposed steel members will require an additional passive (i.e. no activation required) system to maintain structural stability and integrity in such high temperatures. A three hour fire rating requirement of interior bearing walls, columns, and trusses is required by BOCA 1999 for this structure.

Steel Column Fire Protection

Columns are able to retain their structural integrity "as long as the fire exposure does not cause the average temperature at any cross section to elevate above 1,000 degrees F."[3] The amount, or more specifically, the thickness, of fire protection needed for a column is related to the W/D ratio of the individual structural member. The amount of surface area exposed to the fire, along with the mass of the object, affects its ability to retain its current sectional properties when heated to such high temperatures. Several options exist to properly protect a column. Relative to the River Tower, prefabricated fireproof columns, gypsum wallboard, lath and plaster

[3] Ruddy, John, et al. *AISC Design Guide 19: Fire Resistance of Structural Steel Framing.* American Institute of Steel Construction, 2003.

enclosures, concrete enclosures, or the popular spray-applied fire resistive materials are all viable options. The new passive fire protective system must maintain a healthy balance of system thickness, cost, and aesthetic appeal.

Please consult Appendix E for calculations for a sample column chosen for comparative purposes between three common protective types: concrete encasement, spray-on fire resistive materials, and gypsum wallboard. With the sample W12×72 member, some interesting results occurred. The concrete encasement provided the thinnest thickness required, but has the most difficult installation. The gypsum wallboard provides the thickest protection needed, but has the most aesthetic benefits for an interior column. The spray-applied fire resistive material, found to be Isolatek 800, requires a thickness of approximately 1.75" inches, but would be the easiest and quickest application. Considering all of these factors, the gypsum board provides a flat and easily painted surface for an interior space, so that would be the most design-friendly protective surface.

Steel Beam Fire Protection

Similar passive fire protection needs exist for the proposed steel beams. Once again, system thickness, so as not to infringe on the architectural spaces of the tower's units, cost, constructability, and aesthetics are all controlling factors for the use of the various systems available. The steel beams have the added benefit of the inherent two-hour fire rating of the hollow-core slabs that they support. However, this flooring system only rests on top of the structure, and sufficient protection is needed on all four sides of these critical members. Architecturally, these beams will not be exposed to public view, and most likely hidden by drop ceilings or similar low-weight interior solutions. The ease of application and relatively minimum thickness requirements of spray-on fire treatment would be an efficient application on the undersides of this flooring system.

Staggered Truss Fire Protection

The placement of the staggered trusswork in existing infill walls between most condominium units simplifies the fire protective requirements. These infill walls are required to have a minimum one hour fire rating, which reduces the threat of fire spread between individual units. The three hour fire rating requirement of interior bearing walls, columns, and trusses from the code supersedes this requirement, and effectively satisfies both needs. The aesthetic nature of this type of fire protection is perhaps the most crucial of all the fire protective needs, as these walls will form the interior of the luxury condo units. With this in mind, gypsum wallboard should provide sufficient protection while maintaining a smooth, easily painted surface. In places where thickness is at a premium, such as the door opening cutouts running through the trusses in specific units, the use of intumescent coatings may prove more beneficial. These coatings do not have the ease of application of spray-applied materials, or the aesthetic appeal of gypsum. But the thickness of the coating in these potentially tight spaces outweighs the qualities of these other systems. A combination of these systems, used in specific applications, will result in the most cost-efficient fire protective system that best maintains the existing architectural spaces.

Construction Management

Constructability Issues

Besides the issues mentioned previously, the River Tower's original concrete construction adds potentially reducible installation costs and can result in a longer erection process when compared to the proposed staggered truss system. This lies mostly in the placing of forms, and the simple fact that concrete needs at least seven days to cure enough to continue construction. The existing system's reliance on the speed of the wet trades, and the appropriateness of their weather-related working conditions, is the cause of its relative lengthy construction process. However, the proposed system is not without its share of complexities. Most notably, steel construction for such a towering structure requires the use of large cranes to hoist these wide trusses up to the various floors of the high-rise condominium tower. The width of the trusses, 73'-6", is another consideration, as this width requires the need for splicing of the trusses for assembly on site and during erection. Finally, the complexity of the structural design of this system does not afford much leeway in the installed location of these trusses. This does not allow for much flexibility in the field, where unforeseen erection issues can arise.

Cost Analysis

The costs of both structural systems were estimated using primarily R.S. Means Unit Cost data, reflective for the River Tower's location in Wilmington, DE. These prices were used to gain a rough estimate of local values for the bare material, labor, and equipment costs for these two very distinctive systems. The steel system was calculated based on steel tonnage from the *ETABS* results but adjusted to account for the member sizes determined through hand calculations. This results in a rough estimate of a reduction of approximately $2.6 Million, based on steel tonnage. These costs contain a 5% waste factor estimation, and the steel costs contain an additional 10% factor to account for the expense of moment connections and prefabrication of the steel trusses.

The staggered truss system was determined to be approximately $181,650 more expensive than the existing post-tensioned flat plate system with the adjusted figures. Ordinarily, staggered truss systems provide a cost savings when compared to concrete flat plate systems. This change in expectation can be explained by several factors. Smaller cities like Wilmington, DE, do not have the proliferation of concrete contractors that larger cities in the region, such as Philadelphia, PA, and Washington, DC, have. There is not a premium for concrete in Wilmington but rather a premium for steel, which has seen rising costs in recent years due to material shortages. The moment connections required by the proposed system are very expensive and complicated to carry out in the field on any size building, not to mention a large high-rise. Finally, this immense height of the River Tower necessitates crane usage, as mentioned previously. The erection of steel on such a large structure, and its obligatory equipment, introduces this cost premium. Despite the rough estimate of the steel design, the very fact that this comparison is not overwhelmingly in the staggered truss system's favor indicates that the proposed design is not maximized in this particular application. The cost and difficulties of steel construction on a high-rise building negate the potential benefits brought about by staggered truss construction.

Conclusions and Recommendations

The proposed staggered truss system was determined to be the most efficient, in terms of cost and spatial requirements, of an alternative structural steel system in preliminary study. Allowing the maximum clearspan in between interior columns, this arrangement provides the greatest amount of flexibility in the use of interior spaces. Without the weighty columns and shear walls of the existing concrete system, the system dead weight has been lowered, alleviating the requirements on the foundation piles. However, the proposed staggered truss system is not without its shortfalls. Great care was made to preserve the architectural spaces of the existing system, but the combination of floor slab and thickness of braced members in the trusses results in tight spaces where current unit hallways are intersected by some of the placed staggered trusses. Of course, these issues are narrowed to only several units, on alternating stories, so a minor architectural redesign can diminish this effect on the floorplan.

However, theory can not totally predict a particular system's appropriateness for a specific project. In the instance of the River Tower, concrete proves to be the cost-efficient material due to the rising steel costs and expensive connections required with such a complex framing structure. In larger cities like Philadelphia or Washington, DC, concrete may be at a premium. The unit costs from R.S. Means are reflective of the building's location in Wilmington, DE, and confirm that the existing design of post-tensioned concrete slabs would be a much more viable option than the proposed staggered truss system. Either system results in fairly massive columns, due to the shear amount of floor support required. The additional architectural clear spaces created by the staggered truss layout are offset by the cost premiums of the project. The existing columns are spaced out enough to allow for fairly large luxury condo units, and the post-tensioning keeps the floor system at its very minimum thickness of eight inches. Despite evidence to the contrary on smaller buildings, the staggered truss configuration does not provide enough efficiency in cost or architectural considerations to merit further consideration for this particular building in Wilmington, DE.

Acknowledgements

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Appendices

A. Wind Load Calculations for Proposed System

B. Seismic Load Calculations for Proposed System

C. Staggered Truss Structural Hand Calculations for Selected Members

D. *ETABS* Output Data

E. Fire Protection Calculations

F. Cost Analysis Data and Calculations

G. List of Resources

Appendix A

Wind Loading for Proposed System
(Unchanged from Existing System)

Assumptions based on criteria listed on construction drawings and documents, and verified using the BOCA 1996 Building Code.

Coefficients and Categories

Exposure Category: B (BOCA 1996 1609.4)
Worst Case L/B Ratio: (73.5 ft)/(164 ft) = 0.448
Basic Wind Speed (V): 80 mph (Figure 1609.3 – Wilmington, DE)
Basic Velocity Pressure (P_v): 16.4 psf (Table 1609.7(3) based on V = 80 mph)
Wall Pressure Coefficients (C_p): For N-S Direction (Table 1609.7)
- Windward Walls: $C_p = 0.8$
- Leeward Walls: $C_p = -0.5$

Wall Pressure Coefficients (C_p): For W-E Direction (Table 1609.7)
- Windward Walls: $C_p = 0.8$
- Leeward Walls: $C_p = -0.3$

Importance Factor (I): 1.04 (Table 1609.5 and interpolation)
Internal Pressure Coefficients (GC_{pi}): +/- 0.25 (Table 1609.7(6))
Velocity Pressure Exposure (K_z and K_h): see below (Table 1609.7(4))
Gust Response Factors (G_h and G_z): see below (Table 1609.7(5))

Building Main Windforce-Resisting System:
- Windward wall design pressure, P

$$P = (P_v)(I)[(K_z)(G_h)(C_p) - (K_h)(GC_{pi})]$$

- Leeward wall design pressure, P

$$P = (P_v)(I)[(K_z)(G_h)(C_p) - (K_h)(GC_{pi})]$$

North-South Direction Wind Loading Data

Level	Elev. (ft)	K coeff	G coeff.	P (windward)	P (leeward)	Total P (psf)
Roof	279.22	1.84	1.09	27.51	-17.19	44.71
25	269.22	1.83	1.10	27.29	-17.19	44.49
24	259.39	1.81	1.10	27.07	-17.19	44.27
23	247.36	1.78	1.10	26.80	-17.19	44.00
22	236.00	1.76	1.10	26.47	-17.19	43.67
21	225.75	1.74	1.11	26.19	-17.19	43.38
20	215.50	1.71	1.11	25.89	-17.19	43.08
19	205.25	1.69	1.11	25.65	-17.19	42.84
18	195.00	1.67	1.11	25.33	-17.19	42.53
17	184.75	1.64	1.12	25.05	-17.19	42.24
16	174.50	1.62	1.12	24.76	-17.19	41.96
15	164.25	1.59	1.13	24.49	-17.19	41.68
14	154.00	1.56	1.13	24.15	-17.19	41.34
13	143.75	1.53	1.14	23.80	-17.19	40.99
12	133.50	1.50	1.15	23.43	-17.19	40.63
11	123.25	1.46	1.15	22.91	-17.19	40.10
10	113.00	1.43	1.16	22.54	-17.19	39.73
9	102.75	1.39	1.16	21.98	-17.19	39.18
8	92.50	1.35	1.17	21.52	-17.19	38.71
7	82.25	1.30	1.18	20.90	-17.19	38.09
6	72.00	1.25	1.19	20.26	-17.19	37.46
5	61.75	1.20	1.20	19.62	-17.19	36.81
4	51.50	1.14	1.21	18.81	-17.19	36.00
3	41.25	1.07	1.23	17.93	-17.19	35.12
2	31.00	0.99	1.26	16.98	-17.19	34.17
1	10.50	0.80	1.32	14.41	-17.19	31.60

North-South Direction Wind Loading Data (continued)

Level	Trib. Width	Trib Height	P (plf)	F (kips)
Roof	172.5	10.00	7711.74	77.12
25	172.5	9.83	7673.98	75.44
24	172.5	12.03	7636.05	91.86
23	172.5	11.36	7589.21	86.21
22	172.5	10.25	7532.70	77.21
21	172.5	10.25	7483.76	76.71
20	172.5	10.25	7432.01	76.18
19	172.5	10.25	7390.62	75.75
18	172.5	10.25	7335.71	75.19
17	172.5	10.25	7286.92	74.69
16	172.5	10.25	7237.52	74.18
15	172.5	10.25	7190.17	73.70
14	172.5	10.25	7131.55	73.10
13	172.5	10.25	7071.43	72.48
12	172.5	10.25	7008.58	71.84
11	172.5	10.25	6917.96	70.91
10	172.5	10.25	6853.59	70.25
9	172.5	10.25	6757.93	69.27
8	180.75	10.25	6996.77	71.72
7	180.75	10.25	6884.80	70.57
6	180.75	10.25	6770.36	69.40
5	180.75	10.25	6653.46	68.20
4	180.75	10.25	6507.11	66.70
3	180.75	10.25	6348.52	65.07
2	180.75	20.50	6177.04	126.63
1	180.75	10.50	5712.32	59.98
			Sum of F:	1960.35

East-West Direction Wind Loading Data

Level	Elev. (ft)	K coeff	G coeff.	P (windward)	P (leeward)	Total P (psf)
Roof	279.22	1.84	1.09	27.51	-10.32	37.83
25	269.22	1.83	1.10	27.29	-10.32	37.61
24	259.39	1.81	1.10	27.07	-10.32	37.39
23	247.36	1.78	1.10	26.80	-10.32	37.12
22	236.00	1.76	1.10	26.47	-10.32	36.79
21	225.75	1.74	1.11	26.19	-10.32	36.51
20	215.50	1.71	1.11	25.89	-10.32	36.21
19	205.25	1.69	1.11	25.65	-10.32	35.97
18	195.00	1.67	1.11	25.33	-10.32	35.65
17	184.75	1.64	1.12	25.05	-10.32	35.37
16	174.50	1.62	1.12	24.76	-10.32	35.08
15	164.25	1.59	1.13	24.49	-10.32	34.80
14	154.00	1.56	1.13	24.15	-10.32	34.46
13	143.75	1.53	1.14	23.80	-10.32	34.12
12	133.50	1.50	1.15	23.43	-10.32	33.75
11	123.25	1.46	1.15	22.91	-10.32	33.23
10	113.00	1.43	1.16	22.54	-10.32	32.85
9	102.75	1.39	1.16	21.98	-10.32	32.30
8	92.50	1.35	1.17	21.52	-10.32	31.83
7	82.25	1.30	1.18	20.90	-10.32	31.21
6	72.00	1.25	1.19	20.26	-10.32	30.58
5	61.75	1.20	1.20	19.62	-10.32	29.93
4	51.50	1.14	1.21	18.81	-10.32	29.12
3	41.25	1.07	1.23	17.93	-10.32	28.25
2	31.00	0.99	1.26	16.98	-10.32	27.30
1	10.50	0.80	1.32	14.41	-10.32	24.73

East-West Direction Wind Loading Data (continued)

Level	Trib. Width	Trib Height	P (plf)	F (kips)
Roof	72.1667	10.00	2729.92	27.30
25	72.1667	9.83	2714.12	26.68
24	72.1667	12.03	2698.25	32.46
23	72.1667	11.36	2678.66	30.43
22	72.1667	10.25	2655.02	27.21
21	72.1667	10.25	2634.54	27.00
20	72.1667	10.25	2612.89	26.78
19	72.1667	10.25	2595.57	26.60
18	72.1667	10.25	2572.60	26.37
17	72.1667	10.25	2552.19	26.16
16	72.1667	10.25	2531.53	25.95
15	72.1667	10.25	2511.71	25.75
14	72.1667	10.25	2487.19	25.49
13	72.1667	10.25	2462.04	25.24
12	72.1667	10.25	2435.75	24.97
11	72.1667	10.25	2397.83	24.58
10	72.1667	10.25	2370.90	24.30
9	72.1667	10.25	2330.88	23.89
8	104.417	10.25	3323.77	34.07
7	104.417	10.25	3259.09	33.41
6	104.417	10.25	3192.98	32.73
5	104.417	10.25	3125.45	32.04
4	104.417	10.25	3040.90	31.17
3	104.417	10.25	2949.29	30.23
2	104.417	20.50	2850.23	58.43
1	104.417	10.50	2581.77	27.11
			Sum of F:	756.34

Overturning Moments due to Controlling Wind Case: N-S			
Level	F (kips)	Elev (ft)	M (kip-ft)
Roof	77.12	279.22	21532.72
25	75.44	269.22	20308.67
24	91.86	259.39	23828.01
23	86.21	247.36	21325.75
22	77.21	236.00	18221.61
21	76.71	225.75	17316.96
20	76.18	215.50	16416.39
19	75.75	205.25	15548.48
18	75.19	195.00	14662.25
17	74.69	184.75	13799.16
16	74.18	174.50	12945.22
15	73.70	164.25	12105.09
14	73.10	154.00	11257.16
13	72.48	143.75	10419.3
12	71.84	133.50	9590.367
11	70.91	123.25	8739.549
10	70.25	113.00	7938.168
9	69.27	102.75	7117.37
8	71.72	92.50	6633.809
7	70.57	82.25	5804.313
6	69.40	72.00	4996.526
5	68.20	61.75	4211.223
4	66.70	51.50	3434.939
3	65.07	41.25	2684.235
2	126.63	31.00	3925.511
1	59.98	10.50	629.7831
Total Overturning Moment:			295,392.5 kip-ft

$T_{overturning} = C_{overturning}$ = Moment/Span = (295,392.5 kip/ft)/(182.72 ft) = 1,616.64 kip

C_{weight} = (Weight/2) = (42650.38 kips)/2 = 21,325.19 kips

Since $C_{weight} > C_{overturning}$, the weight of the building eliminates chance of overturning

Appendix B

Seismic Load Calculations for Proposed System
(Changes from Original System Noted Below)

Assumptions based on criteria listed on construction drawings and documents, and verified using the BOCA 1996 Building Code.

Seismic Hazard Exposure Group: II (Table 1610.1.5 – Substantial occupancy building)
Effective Peak Velocity-related Acceleration: $A_v = 0.075$
(Wilmington, DE – Figure 1610.1.3(1): halfway between 0.05 and 0.10 regions)
Effective Peak Acceleration Coefficient: $A_a = 0.05$ (Wilmington, DE – Figure 1610.1.3(2))
Seismic Performance Category: B (Table 1610.1.7 since $0.05 < A_v < 0.10$)
Seismic Resisting System: Combination of concentric braced frames (at the staggered trusses) and ordinary moment frames
- Response Modification Factor (R): 5.0 *(Compare to R = 8.0 of original system)*
- Deflection Amplification Factor (Cd): 4.5 *(Compare to R = 6.5 of original system)*

Site Coefficient: S_4, 2.0 (Table 1610.3.1)

Use Equivalent Lateral Force Procedure (Section 1610.3.5.2)

$V = (C_s)(W)$

Seismic Design Coefficient (C_s): (Section 1610.4.1.1)
min of $C_s = (1.2A_vS) / (RT)^{(2/3)}$ = See below
…and $(2.5A_a)/(R) = (2.5)(0.05)/(5.0) = \mathbf{0.025}$

Approximate Fundamental Period (T_a):
$T_a = (C_T)(h_n)^{(3/4)}$
$C_T = 0.02$ (Section 1610.4.1.2.1: Seismic resisting systems with concentrically braced frames) *(Unchanged from original system)*
$h_n = 279.22$ ft (Section 1610.4.1.2.1: Height from base to highest level of building)
$T_a = (0.02)(279.22)^{(3/4)} = 1.366$ seconds

Coefficient for Upper Limit on Calculated Period (C_a): 1.7 (Table 1610.4.1.2)
Fundamental Period (T): $T = (C_a)(T_a)$
$T = (1.7)(1.366) = 2.322$ seconds

$C_s = [(1.2)(0.075)(2.0)]/[(5.0)(2.322)]^{(2/3)} = \mathbf{0.0351} > 0.025 \rightarrow$ Use $C_s = 0.025$

$V = (C_s)(W_{total}) = (0.025)(42{,}650.381 \text{ kips}) = \mathbf{1066.26\ kips}$

Story Drift Based on Requirements from BOCA 1996 Building Code

Story Drift: $\Delta_a = 0.015(h_{sx})$			
Level	**h_{sx} (ft)**	**Δ_a (ft)**	**Δ_a (in)**
25	279.22	4.19	50.26
24	269.22	4.04	48.46
23	259.39	3.89	46.69
22	247.36	3.71	44.52
21	236.00	3.54	42.48
20	225.75	3.39	40.64
19	215.50	3.23	38.79
18	205.25	3.08	36.95
17	195.00	2.93	35.10
16	184.75	2.77	33.26
15	174.50	2.62	31.41
14	164.25	2.46	29.57
13	154.00	2.31	27.72
12	143.75	2.16	25.88
11	133.50	2.00	24.03
10	123.25	1.85	22.19
9	113.00	1.70	20.34
8	102.75	1.54	18.50
7	92.50	1.39	16.65
6	82.25	1.23	14.81
5	72.00	1.08	12.96
4	61.75	0.93	11.12
3	51.50	0.77	9.27
2	41.25	0.62	7.43

Appendix C

Staggered Truss Structural Calculations for Selected Members

Hollow Core Slab System Selection

- Superimposed Dead Load:
  7 psf for ceiling/mechanical (presumed)
  5 psf for collateral (listed on drawings)

- Worst Case Live Load (typical floor): 70 psf

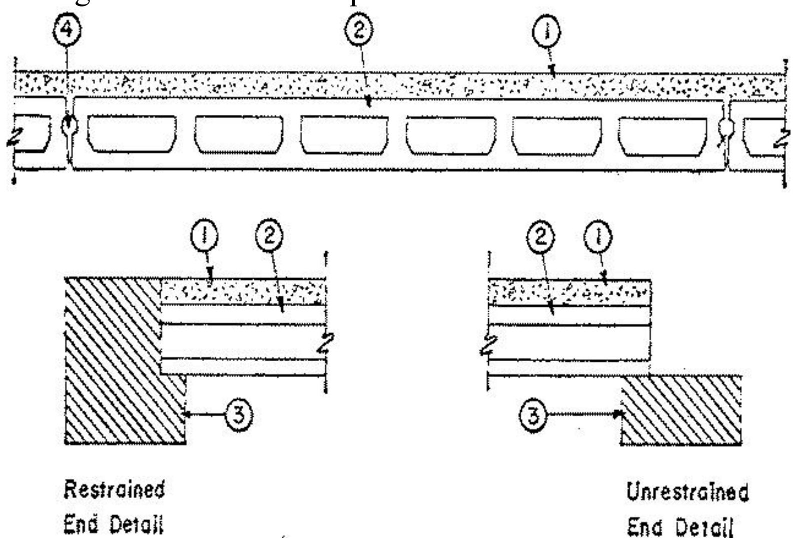
- **Total Superimposed Load (unfactored)** = 70 psf + 7 psf + 5 psf = **82 psf**
- Total Superimposed Load (factored) = 1.2(12 psf) + 1.4(70psf) = 114.8 psf

- *From Nitterhouse Concrete Products (see following data chart):*

  - Max Span = 28'-6" → 29' for design

  - Choose **8" × 4' Prestressed Concrete SpanDeck with 2" topping (U.L. J917)**
    - f'_c = 5000 psi at 28 days, 3000 psi at release
    - Precast density = 150 pcf (top and webs), 115 pcf (soffit)
    - Allowable Superimposed Load for 24' span = 112 psf (flexure) > 82 psf req'd
    - (4) ½" diameter, 270 ksi Low-Relaxation Strands at 2" height
    - Precast System Weight = 330 plf = 82.5 psf

Fire Rating from Underwriters' Laboratories
  - Restrained end: 2 in. concrete cover (1 in. gypsum board) required for 2 hour fire rating
  - Unrestrained end: 1 ½ hour rating with same cover requirements



U.L. Assembly Diagram Key

1.) Floor Topping (concrete, gypsum, or floor mat material)
2.) Precast Plank
3.) Min. 1.5" End Bearing Detail
4.) Grout: 3500 psi

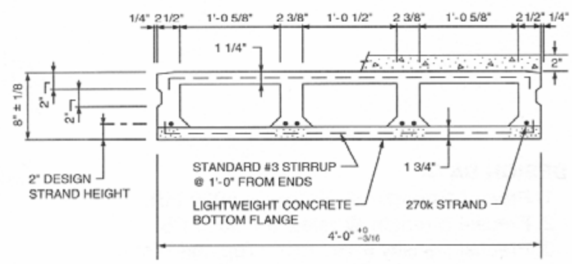
Courtesy Nitterhouse Concrete Products, Inc.:
http://www.nitterhouse.com/DrawingSpecs/DrawingsSpecs.html

Prestressed Concrete
8" x 4' SpanDeck – U.L. – J952
(2" C.I.P. TOPPING)

PHYSICAL PROPERTIES

Composite

A' = 295 in.^{2}	S'$_b$ = 468 in.^{3}
I' = 2624 in.^{4}	S'$_t$ = 1096 in.^{3} (At Top of SpanDeck)
Y$_{b'}$ = 5.61 in.	S'$_{tt}$ = 597 in.^{3} (At Top of Topping)
Y$_t$' = 2.39 in. (To Top of SpanDeck)	Wt.' = 330 PLF
Y$_{tt}$' = 4.39 in. (To Top of Topping)	Wt.' = 82.5 PSF



8" SPANDECK CROSS SECTION

UL FIRE RATED J952

DESIGN DATA

1. Precast Strength @ 28 days = 5000 PSI.
2. Precast Strength @ release = 3000 PSI.
3. Precast Density = 150 PCF (Top and Webs)
   = 115 PCF (Soffit)
4. Strand = 1/2"ø, 270K Lo-Relaxation.
5. Composite Strength = 3000 PSI.
6. Composite Density = 150 PCF.
7. Strand Height = 2.00 in.
8. Ultimate moment capacities (when fully developed) . . .
   4 – 1/2"ø, 270K = 88.3'K
   6 – 1/2"ø, 270K = 124.0'K
9. Maximum bottom tensile stress is $6\sqrt{f'c}$ = 424 PSI.
10. All superimposed load is treated as live load in the strength analysis of flexure and shear.
11. Flexural strength capacity is based on stress/strain strand relationships.
12. Shear values are the maximum allowable before shear reinforcement is required.
13. Deflection limits were not considered when determining allowable loads in this table.
14. Load values to the left of the solid line are controlled by ultimate strength. Load values to the right are controlled by service stress.
15. All loads shown refer to allowable loads applied after topping has hardened.

8" SPANDECK W/2" TOPPING	ALLOWABLE SUPERIMPOSED LOAD (PSF)																					
STRAND PATTERN	SPAN (FEET)																					
	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32
Flexure 4 – 1/2"ø	750	675	611	546	462	394	338	291	252	218	191	167	146	128	112	98	85	74	63	51	41	31	
Shear 4 – 1/2"ø	527	469	421	382	348	317	294	272	252	235	219	197	176	157	140	129	122	110	98	88	78	70	
Flexure 6 – 1/2"ø	1098	900	898	794	676	580	502	437	382	336	296	262	233	207	185	165	147	132	116	101	87	74	63
Shear 6 – 1/2"ø	542	483	434	393	359	329	303	280	261	243	227	212	199	188	178	167	152	137	124	112	101	91	86

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.

NITTERHOUSE
CONCRETE PRODUCTS

2655 Molly Pitcher Hwy. South, Box N
Chambersburg, PA 17201-0813
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REVISED 12/93

Staggered Truss Design

Even Floor Centroids	
Truss	x_i (ft)
2	29.25
4	71.583
6	114.917
Sum =	215.75
x_{even} = (215.75)/3 = 71.92ft	

Odd Floor Centroids	
Truss	x_i (ft)
3	43.083
5	86.627
Sum =	129.71
x_{odd} = (129.71)/2 = 64.85 ft	

Torsional Rigidity, Even Floors		
Truss	$Xbar_i$ (ft)	$Xbar_i^2$ (ft^2)
2T	-42.67	1820.44
4T	-0.33	0.11
6T	43.00	1849.00
	Sum =	3669.56

Torsional Rigidity, Odd Floors		
Truss	$Xbar_i$ (ft)	$Xbar_i^2$ (ft^2)
3T	-21.77	474.00
5T	21.77	474.00
	Sum =	948.00

Shear Force in Each Truss Due to Lateral Loads (Bottom Floor)								
Truss	x_i (ft)	V_s (k)	T = 23227.71 (ft-k)		T = -8207.40 (ft-k)		Design V_i	Φ_{ecc}
			V_{tors}	V_i	V_{tors}	V_i		
3T	43.08	905.91	-533.44	372.47	188.49	1094.40	1094.40	1.00
5T	86.63	905.91	533.44	1439.35	-188.49	717.42	1439.35	1.32
			T = 10433.07 (ft-k)		T = -21002.03 (ft-k)			
2T	29.25	603.94	-121.31	482.63	244.19	848.14	848.14	1.40
4T	71.58	603.94	-0.95	602.99	1.91	605.85	605.85	1.00
6T	114.92	603.94	122.26	726.20	-246.10	357.84	726.20	1.20

(Assuming each truss has approximately equal shear rigidity (GA))

$x = (\sum x_i GA_i)/(\sum GA_i)$

$Xbar = x_{even} - x_i$ or $x_{odd} - x_i$

Using the centroid of lower levels (1-7th floors):

e_{even} = Centroid – x_{even} = 69 ft – 71.92 ft = -2.92 ft

e_{odd} = Centroid - x_{odd} = 69 – 64.85 = 4.15 ft

After adding accidental torsional eccentricity (5% of total width):

e_{even} = -2.92 ft ± (0.05*173.5 ft) = 5.76 ft or -11.59 ft

e_{odd} = 4.15 ft ± (0.05*173.5 ft) = 12.82 ft or -4.53 ft

Base torsion calculations: Using base shear V = 1811.82 k (see Appendix A)

T = 1811.82*(5.76ft) = 10,433.07 ft-k
T = 1811.82*(-11.59ft) = -21,002.03 ft-k
T = 1811.82*(12.82ft) = 23,227.71 ft-k
T = 1811.82*(-4.53ft) = -8,207.40 ft-k

V_s = (1811.82 k)/2 = 905.51k for odd floors
V_s = (1811.82k)/3 = 603.94 k for even floors

Values in table above, where:

$V_{tors} = (T^*Xbar_i)/ \sum x_i^2$
$V_i = V_s + V_{tors}$

Transverse Shear in Diaphragm (Hollow Core Planks)

V_u = 1.7*(Φ_h)*(V)*(0.75) = 1.7*(1.0)*(726.2k)*(0.75) = **925.91 k**

Where: Max V_i = 726.6 k from above

$\Phi V_n = \Phi V_c + \Phi V_s$
ΦV_c = Φ*(2*√f'$_c$)*(bd) = (0.85*2*(√5000)*6in*0.8*73.5ft*12in/ft*(1k/1000lb) = **508.91 kip**
ΦV_s = Φ*A_{vf}*f_y*μ = (0.85)*(7.92in^2)*(60 ksi)*(1.4) = **565.49 kip**

Where: μ = 1.4 (coefficient of friction)
No. of planks = 73.5'/4ft wide planks ≈ 19
No. of joints = 19 – 1 = 18
A_{vf} = (18 joints)*(0.44 in^2) = 7.92 in^2 (using #6 bars)

ΦV_n = 508.91k + 565.49k = 1074.4k > 925.91k (OK)

Design of Truss Members

Gravity Loads on Typical Truss Members Using Method of Joints

Dead Loads for precast planks

- 8" precast plank with 2" topping = 82.5 psf
- Leveling compound = 5 psf
- Structural steel = 5 psf (estimate)
- Partitions / MEP = 12 psf

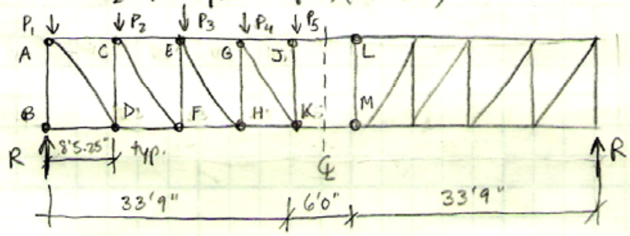
104.5 psf

Live Load (typical floor) 70 psf

Truss #2 ("regular" truss)

Trib width = $(28.4167' + 13.833')/2 = 21.125'$

$W_2 = (104.5\,psf + 70\,psf)(21.125') = 3.87$ kip/ft



$P_1 = (3.87\,klf)(8.354'/2) = 16.165$ kip

$P_2 = (3.87\,klf)(8.354') = 32.33$ kip

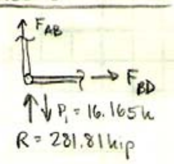
$P_3 = P_4 = 32.33$ kip

$P_5 = (3.87\,klf)\left(\frac{8.354'}{2} + \frac{6'}{2}\right) = 27.78$ kip

$R = (16.165k + 3(32.33k) + 27.75k) \times \underbrace{2}_{\text{top and bottom}} = 281.82$ kip

Method of Joints $(\Sigma F_x = 0; \Sigma F_y = 0)$

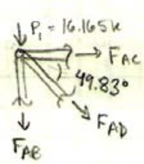
$\boxed{B}$



$F_{AB} = -281.81k + 16.165k = -265.65$ kip

$F_{BD} = 0$ kip

$\boxed{A}$

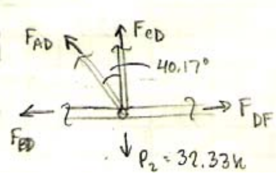


$F_{AC} = -F_{AD}\cos 49.83° = -0.645\,F_{AD} \rightarrow F_{AC} = -210.22$

$-16.165k - F_{AB} = F_{AD}\sin 49.83°$

$F_{AD} = \frac{-16.165 - (-265.65)}{0.764} = 325.92$ kip

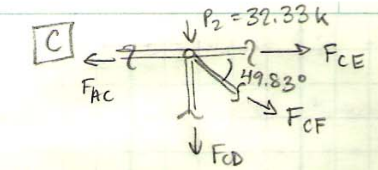
$\boxed{D}$



$F_{CD} = -F_{AD}\cos 40.17° + 32.33 = -216.67$ kip

$F_{DF} = \cancel{F_{BD}} + F_{AD}\sin 40.17°$

$= (325.92k)(0.645) = 210.22$ kip

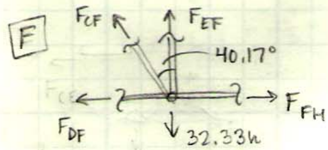


$$-32.33k - F_{CD} = F_{CF}\sin 49.83°$$

$$F_{CF} = \frac{-32.33 - (-216.67)}{0.764} = \underline{241.28 \text{ kip}}$$

$$F_{AC} = F_{CE} + F_{CF}\cos 49.83°$$

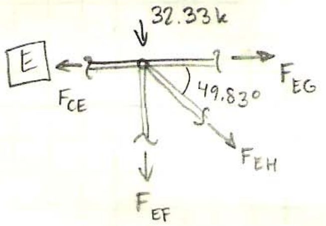
$$F_{CE} = -(241.38)(0.645) + (-210.22k) = \underline{-365.91 \text{ kip}}$$



$$F_{EF} = -F_{CF}\cos 40.17° + 32.33 = \underline{-152 \text{ kip}}$$

$$F_{FH} = F_{DF} + F_{CF}\sin 40.17°$$

$$F_{FH} = (210.22k) + (241.28k)(0.645) = \underline{365.85 \text{ kip}}$$



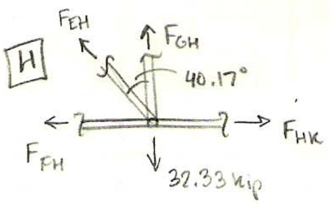
$$F_{EF} + F_{EH}\sin 49.83° = -32.33k$$

$$F_{EH} = \frac{-32.33k - (-152.0)}{0.764} = \underline{156.64 \text{ kip}}$$

$$F_{CE} = F_{EG} + F_{EH}\cos 49.83°$$

$$F_{EG} = F_{CE} - F_{EH}(0.645) = -365.91 - (156.64)(0.645)$$

$$\underline{F_{EG} = -466.94 \text{ kip}}$$

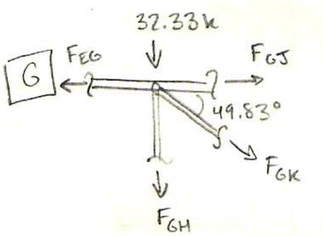


$$F_{GH} = -F_{EH}\cos 40.17° + 32.33k$$

$$F_{GH} = -(0.764)(156.64k) + 32.33 = \underline{-87.34 \text{ kip}}$$

$$F_{HK} = F_{FH} + F_{EH}\sin 40.17°$$

$$= (365.85k) + (156.64k)(0.645) = \underline{466.88 \text{ kip}}$$



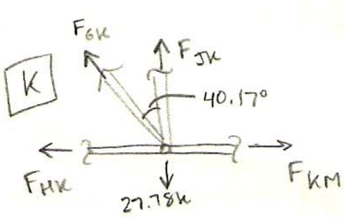
$$F_{GK}\sin 49.83° = -32.33 - F_{GH}$$

$$F_{GK} = \frac{-F_{GH}}{0.764} - \frac{32.33}{0.764} = \frac{-(-87.34) - 32.33}{0.764} = \underline{72.0 \text{ kip}}$$

$$F_{EG} = F_{GJ} + F_{GK}\cos 49.83°$$

$$F_{GJ} = F_{EG} - F_{GK}\cos 49.83°$$

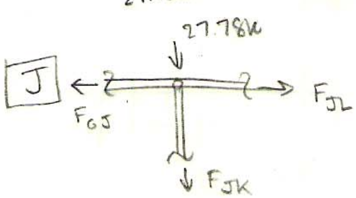
$$F_{GJ} = (-466.88) - (72.0k)(0.645) = \underline{-513.32 \text{ kip}}$$



$$F_{JK} = -F_{GK}\cos 40.17° + 27.78k = \underline{-27.28 \text{ kip}}$$

$$F_{KM} = F_{HK} + F_{GK}\sin 40.17°$$

$$= (466.88k) + (72.0k)(0.645) = \underline{513.32 \text{ kip}}$$



$$F_{JK} = -27.78 \text{ kip} \quad (\text{close due to rounding})$$

$$F_{JL} = F_{GJ} = \underline{-513.32 \text{ kip}}$$

Lateral Loads on Typical Truss Members Using Method of Joints

Lateral Loads - Truss #2 - regular truss → 73.5' width, 10' tall

Design V_i = 848.14 kips (from spreadsheet)

∴ Horiz. reactions = Design $V_i/2$ = 424.07 kips

$R = (424.07k)(2)(10')/(73.5') = 115.39\ k$

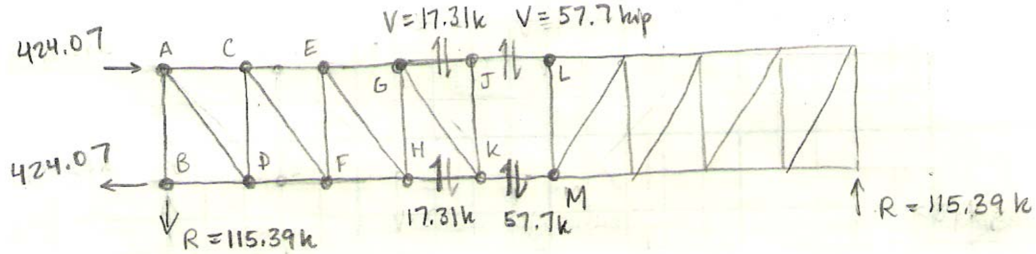
$V_{vierendeel} = \frac{1}{2}(424.07\ k)(10')/(\frac{73.5'}{2}) = \underline{57.70\ kip}$

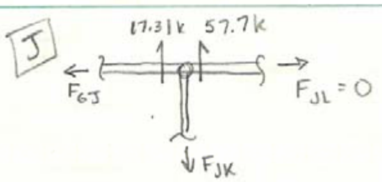
At joint J:

- $M = (57.70k)(6'/2)$ = 173.09 ft·kip (width of panel)
- In the panel GJ: (assuming M at other end = 0)

$V_{GJ} = (173.09\ ft \cdot k)/(10') = \underline{17.31\ kip}$

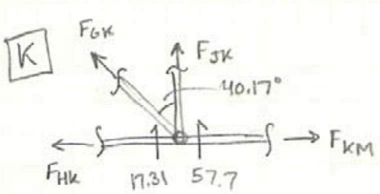
(Assume chord members in remaining members. and shear chord forces also = 0)





$F_{GJ} = F_{JL} = 0$

$F_{JK} = \underline{75.01k}$

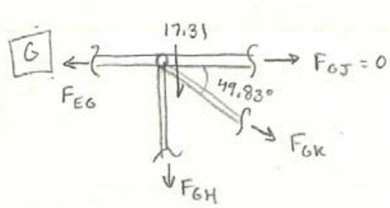


$F_{JK} = -F_{GK}\cos 40.17° - 75.01k$

$F_{GK} = \frac{-2(75.01\ k)}{0.764} = \underline{-196.35\ k}$

$F_{HK} = -F_{GK}\sin 40.17°$

$F_{HK} = -(-196.35k)(0.645) = \underline{126.64\ k}$

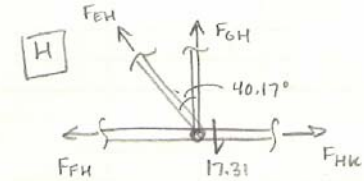


$F_{EG} = F_{GK}\cos 49.83°$

$F_{EG} = (0.645)(-196.35\ k) = \underline{-126.64k}$

$-17.31 = F_{GH} + F_{GK}\sin 49.83°$

$F_{GH} = -17.31 - (-196.35\ k)(0.764) = \underline{132.7k}$

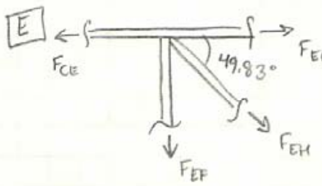


$$17.31 = F_{EH}\cos 40.17° + F_{GH}$$

$$F_{EH} = \frac{17.31 - (132.7k)}{0.764} = \underline{-151.04\,k}$$

$$F_{FH} + F_{EH}\sin 40.17° = F_{HK}$$

$$F_{FH} = (126.64k) - (-151.04k)(0.645) = \underline{224.06\,k}$$

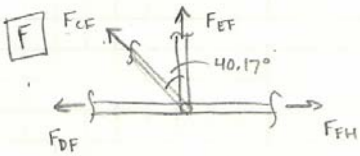


$$F_{CE} = F_{EG} + F_{EH}\cos 49.83°$$

$$F_{CE} = (-126.64k) + (-151.04k)(0.645) = \underline{-224.06\,k}$$

$$F_{EH}\sin 49.83° = -F_{EF}$$

$$F_{EF} = -(-151.04k)(0.764) = \underline{115.39\,k}$$

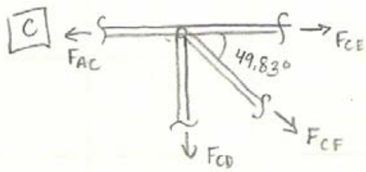


$$F_{EF} = -F_{CF}\cos 40.17°$$

$$F_{CF} = \frac{-(115.39\,k)}{0.764} = \underline{-151.04\,k}$$

$$F_{DF} + F_{CF}\sin 40.17° = F_{FH}$$

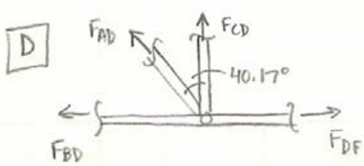
$$F_{DF} = 224.06 - (-151.04)(0.645) = \underline{321.48\,k}$$



$$F_{AC} = F_{CE} + F_{CF}\cos 49.83°$$

$$F_{AC} = -224.06\,k + (-151.04)(0.645) = \underline{-321.48\,k}$$

$$F_{CD} = -F_{CF}\sin 49.83° = -(-151.04)(0.764) = \underline{115.39\,k}$$

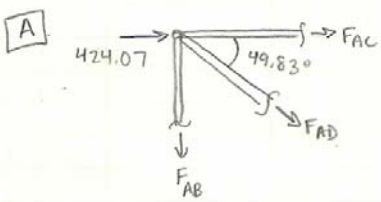


$$F_{CD} = -F_{AD}\cos 40.17°$$

$$F_{AD} = \frac{-115.39\,k}{0.764} = \underline{-151.04\,k}$$

$$F_{BD} + F_{AD}\sin 40.17° = F_{DF}$$

$$F_{BD} = 321.48k - (-151.04k)(0.645) = \underline{418.9\,k}$$



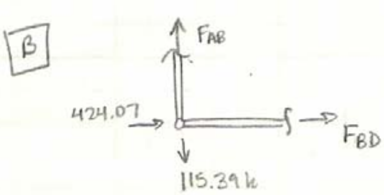
$$-424.07 = F_{AC} + F_{AD}\cos 49.83°$$

$$-424.07 = -321.48 + (-151.04)(0.645)$$

$$-424.07 \approx -419 \quad \text{(rounding error)}$$

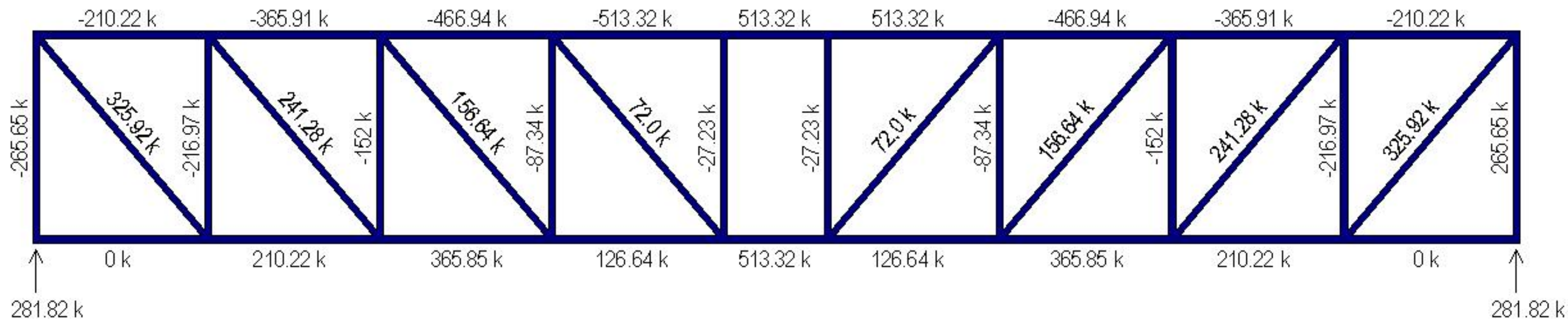
$$F_{AB} = -F_{AD}\sin 49.83°$$

$$F_{AB} = -(-151.04)(0.764) = \underline{+115.4\,k}$$

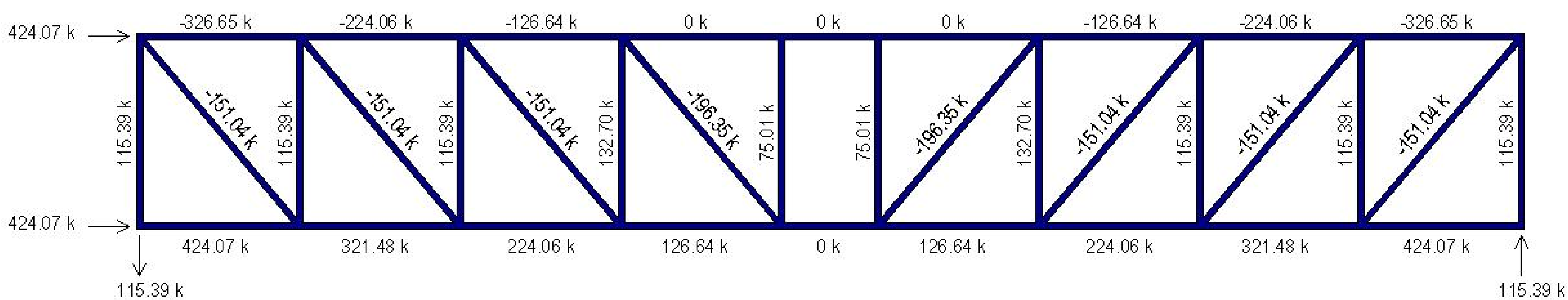


$$F_{AB} = +115.4\,k = 115.4\,k$$

$$F_{BD} = 424.07k \approx 418.9k \quad \text{(rounding error)}$$



Typical Truss Member Force Diagram: Under Gravity Loading



Typical Truss Member Force Diagram: Under Lateral Loading

Design of Diagonal Member D1 of Truss 3

Floor	WIND, kips		SEISMIC, kips		LOAD COMBINATIONS, kips					
	Φ_h	$\Phi_{ecc}\ \Phi_h\ F_w$	Φ_h	$\Phi_{ecc}\ \Phi_h\ F_E$	1	2	3	4	Governing Load	Member Sizes
25	0.04	6.56	0.10	5.27	273.84	339.04	277.82	272.59	339.04	HSS 10x4x1/2
24	0.09	13.13	0.20	10.23	273.84	339.04	288.32	277.55	339.04	HSS 10x4x1/2
23	0.13	19.55	0.29	14.79	273.84	339.04	298.59	282.11	339.04	HSS 10x4x1/2
22	0.18	27.37	0.37	19.01	273.84	339.04	311.10	286.33	339.04	HSS 10x4x1/2
21	0.23	34.70	0.44	22.83	273.84	339.04	322.84	290.15	339.04	HSS 10x4x1/2
20	0.27	41.27	0.51	26.38	273.84	339.04	333.36	293.70	339.04	HSS 10x4x1/2
19	0.32	47.80	0.58	29.61	273.84	339.04	343.80	296.93	343.80	HSS 10x4x1/2
18	0.36	54.25	0.63	32.57	273.84	339.04	354.12	299.89	354.12	HSS 10x4x1/2
17	0.40	60.65	0.69	35.25	273.84	339.04	364.36	302.57	364.36	HSS 10x4x1/2
16	0.44	67.01	0.73	37.68	273.84	339.04	374.53	305.00	374.53	HSS 10x4x1/2
15	0.49	73.32	0.78	39.85	273.84	339.04	384.63	307.17	384.63	HSS 10x4x1/2
14	0.53	79.59	0.81	41.79	273.84	339.04	394.67	309.11	394.67	HSS 10x4x1/2
13	0.57	85.81	0.85	43.50	273.84	339.04	404.62	310.82	404.62	HSS 10x4x1/2
12	0.61	91.98	0.88	45.01	273.84	339.04	414.49	312.33	414.49	HSS 10x4x1/2
11	0.65	98.10	0.90	46.32	273.84	339.04	424.27	313.64	424.27	HSS 10x4x1/2
10	0.69	104.13	0.92	47.44	273.84	339.04	433.93	314.76	433.93	HSS 10x4x1/2
9	0.73	110.11	0.94	48.40	273.84	339.04	443.50	315.72	443.50	HSS 10x4x1/2
8	0.77	116.01	0.96	49.20	273.84	339.04	452.93	316.52	452.93	HSS 10x4x1/2
7	0.81	122.11	0.97	49.85	273.84	339.04	462.69	317.17	462.69	HSS 10x4x1/2
6	0.85	128.11	0.98	50.37	273.84	339.04	472.30	317.69	472.30	HSS 10x4x1/2
5	0.89	134.02	0.99	50.77	273.84	339.04	481.75	318.09	481.75	HSS 10x4x1/2
4	0.93	139.83	0.99	51.06	273.84	339.04	491.04	318.38	491.04	HSS 10x4x1/2
3	0.96	145.50	1.00	51.27	273.84	339.04	500.12	318.59	500.12	HSS 10x4x1/2
2	1.00	151.04	1.00	51.41	273.84	339.04	508.98	318.73	508.98	HSS 10x4x1/2
Ground		151.04		51.41	273.84	339.04	267.32	267.32	339.04	HSS 10×4×1/2

Load Factors and Combinations and Truss Chord Design

DL = 105 psf
LL = 70 psf
RLL = 35 psf (due to 50% LL reduction)

$\Phi_W = 1.0$ for typical truss

$\Phi_{ecc} = 1.0$ for truss 3 and 4
2.81 for truss 5
1.04 for truss 2
1.56 for truss 6

(For Level 9)
$\Phi_{h\,wind} = 0.72$
$\Phi_{h\,seismic} = 0.85$

$$\Phi_{L1} = 1.4D = 1.4\left(\frac{105\text{ psf}}{105+70}\right) = 0.84$$

$$\Phi_{L2} = 1.2D + 1.6L = 1.2\left(\frac{105}{175}\right) + 1.6\left(\frac{35}{175}\right) = 1.04$$

$$\Phi_{L3} = 1.2D + 0.5L = 1.2\left(\frac{105}{175}\right) + 0.5\left(\frac{35}{175}\right) = 0.82$$

Load combos (for Diag. Member d_1 of Truss 3 at Level 2)

1) $1.4D = \Phi_{L1} F_G = (0.84)(326 \text{ kip}) = \underline{274\text{ k}}$ — from Method of Joints (Gravity)

2) $1.2D + 1.6L = \Phi_{L2} F_G = (1.04)(326\text{ k}) = \underline{339\text{ k}}$

3) $1.2D + 1.6W + 0.5L = \Phi_{L3} F_G + \Phi_{ecc}\Phi_h F_W (1.6)$

$= (0.82)(326\text{k}) + (1.0)(1.6)(1.00)(151.04\text{k}) = \underline{508.98\text{ k}}$ — from Method of Jts (Lat)

4) $1.2D + 1.0E + 0.5L = \Phi_{L3} F_G + \Phi_{ecc}\Phi_h F_E$

$$F_E = \underbrace{(151.04\text{k})}_{\text{force in chord}}\left(\frac{712.67}{1811.82\text{ k}}\right) = \underline{59.41\text{ k}}$$

712.67 — Seismic story shear @ 2nd floor
1811.82 k — wind shear at Lv. 2
(only want seismic portion of lat. load)

$= (0.82)(326\text{ k}) + (1.0)(1.0)(59.41\text{k}) = \underline{326.73\text{ k}}$

5) $\underline{0.9D} + (\underline{1.3W}$ or $\underline{1.0E}) <$ Case 3, 4 (Will not control)

See Excel for each floor calc

Truss Chords

$$\frac{P_u}{\Phi P_n} + \frac{8}{9}\left(\frac{M_{ux}}{\Phi_b M_{nx}}\right)(1.0)$$

$\Phi = 0.9$ for tension
$\Phi = 0.85$ for compression
$\Phi_b = 0.9$ for bending

Gravity $\quad w = 3.87$ kip/ft

$\Phi_w = 1.0$

$$M = \frac{w\ell^2}{10} = \frac{(3.87\text{ kip/ft})(10')^2}{10} = 38.7\text{ kip ft}$$

P = 513.32 kip (worst case from Method of Joints)

Truss Chord Design (Continued) and Column Design

Truss Chords (cont.)

$\Phi_{L1} = 0.84$
$\Phi_{L2} = 1.04 \leftarrow$ controls
$\Phi_{L3} = 0.82$

Gravity (cont.)

2) $1.2D + 1.6L$

$P_u = \Phi_{L2} P = (1.04)(513.32\text{ k}) = \underline{533.85\text{ k}}$

$M_u = \Phi_{L2} M = (1.04)(38.7\text{ kip}\cdot\text{ft}) = \underline{40.25\text{ k}\cdot\text{ft}} = M_{u_G}$ (every story)

3) $1.2D + 1.6W + 0.5L$

$P_u = \Phi_{L3} P = (0.82)(513.32\text{ k}) = \underline{420.92\text{ k}}$

$M_u = \Phi_{L3} M = (0.82)(38.7\text{ k}\cdot\text{ft}) = \underline{31.73\text{ ft}\cdot\text{k}}$

Wind

$M = 58.47\text{ ft}\cdot\text{kip}$

$\Phi_{ecc} M = (1.0)(58.47\text{ ft}\cdot\text{kip}) = \underline{58.47\text{ ft}\cdot\text{kip}}$

$M_u = 1.6(58.47\text{ ft}\cdot\text{kip}) = \underline{93.55\text{ ft}\cdot\text{kip}}$

Combos

Case #2

$P_u = \underline{533.85\text{ k}}$

$M_u = 40.25\text{ ft}\cdot\text{k}$

Case #3

$P_u = 420.92\text{ k}$

$M_u = 31.73\text{ ft}\cdot\text{k} + 93.55\text{ ft}\cdot\text{k} = \underline{125.28\text{ ft}\cdot\text{kip}}$

Column Design

- Exterior Column at truss 4 (greatest trib. area)
- Assume 50% LL reduction
- Carries load from 2 floors (3 when used with a hanger/post)

Axial Force

$\text{Trib. Area} = \left(\frac{28.5' + 28.5'}{2}\right)\left(\frac{73.5'}{2}\right) = \underline{1047.38\text{ ft}^2}$

$DL_1 = \left[(82.5\text{ psf})_{\text{hollow core}} + (5\text{ psf})_{\text{structural steel}}\right](1047.38\text{ ft}^2)\left(\frac{1\text{ kip}}{1000\text{ lb}}\right) = \underline{91.65\text{ k}}$

$DL_2 = 104.5\text{ psf}(1047.38\text{ ft}^2)\left(\frac{1\text{ kip}}{1000\text{ lb}}\right) = \underline{109.45\text{ kip}}$

$RLL = 35\text{ psf}(1047.38\text{ ft}^2)\left(\frac{1\text{ kip}}{1000\text{ lb}}\right) = \underline{36.66\text{ kip}}$

$DL_2 + RLL = 109.45\text{ k} + 36.66\text{ k} = \underline{146.11\text{ kip}}$

Two Floors

$DL_1 = 2(91.65\text{ k}) = 183.3\text{ kip}$

$DL_2 + RLL = 2(146.11\text{ k}) = 292.22\text{ kip}$

(Arch. cladding is self supporting)

Column Design (continued)

$M_{column} = M_{translation} + M_{rotation}$

$$M_{col} = \frac{6EI(\Delta_t + \Delta_b)}{l_c^2} + \left(\frac{-3EI\theta}{l_c}\right)$$

where $\theta = \dfrac{2\Delta_{TS}}{L}$

$$\therefore M_{col} = \frac{6EI}{l_c}\left(\frac{\Delta_t + \Delta_b}{l_c} - \frac{\Delta_{TS}}{L}\right)$$

Assume $\Delta_{TS} = 3/4''$ (deflection of truss at midspan due to weights of planks and structural steel)

$L = 73.5'$
$l_c = 10'$

$$\Delta_t = \frac{\Sigma P_i L_i}{EA_i}$$

With a top and bottom chord of W10×54 for example:

$$\Delta_t = \left(\frac{87.5\text{ psf}}{104.5 + 70\text{ psf}}\right)\frac{(8.44')(12\text{ in/ft})}{(29000\text{ ksi})(15.8\text{ in}^2)}\left(\frac{513.22}{2} + 513.22 + 466.94 + 365.91 + 210.22\right)$$

$\Delta_t = 0.201''$

$$\Delta_b = \left(\frac{87.5\text{psf}}{104.5+70\text{psf}}\right)\left[\frac{(8.44')(12\text{ in/ft})}{(29000\text{ksi})(15.8\text{in}^2)}\right]\left(0 + 210.22 + 365.85 + 466.88 + \frac{513.32}{2}\right)$$

$\Delta_b = 0.144''$

Try W12×65 column

$$M_{col} = \frac{6(29000\text{ ksi})(174\text{ in}^4)}{(10')(12\text{ in/ft})}\left[\frac{0.201'' + 0.144''}{10'(12\text{ in/ft})} - \frac{0.75''}{73.5'(12\text{ in/ft})}\right] = 510.82 \text{ in}\cdot\text{k}$$

$M_{col} = 42.56\text{ k}\cdot\text{ft} \sim 43\text{ kip}\cdot\text{ft}$

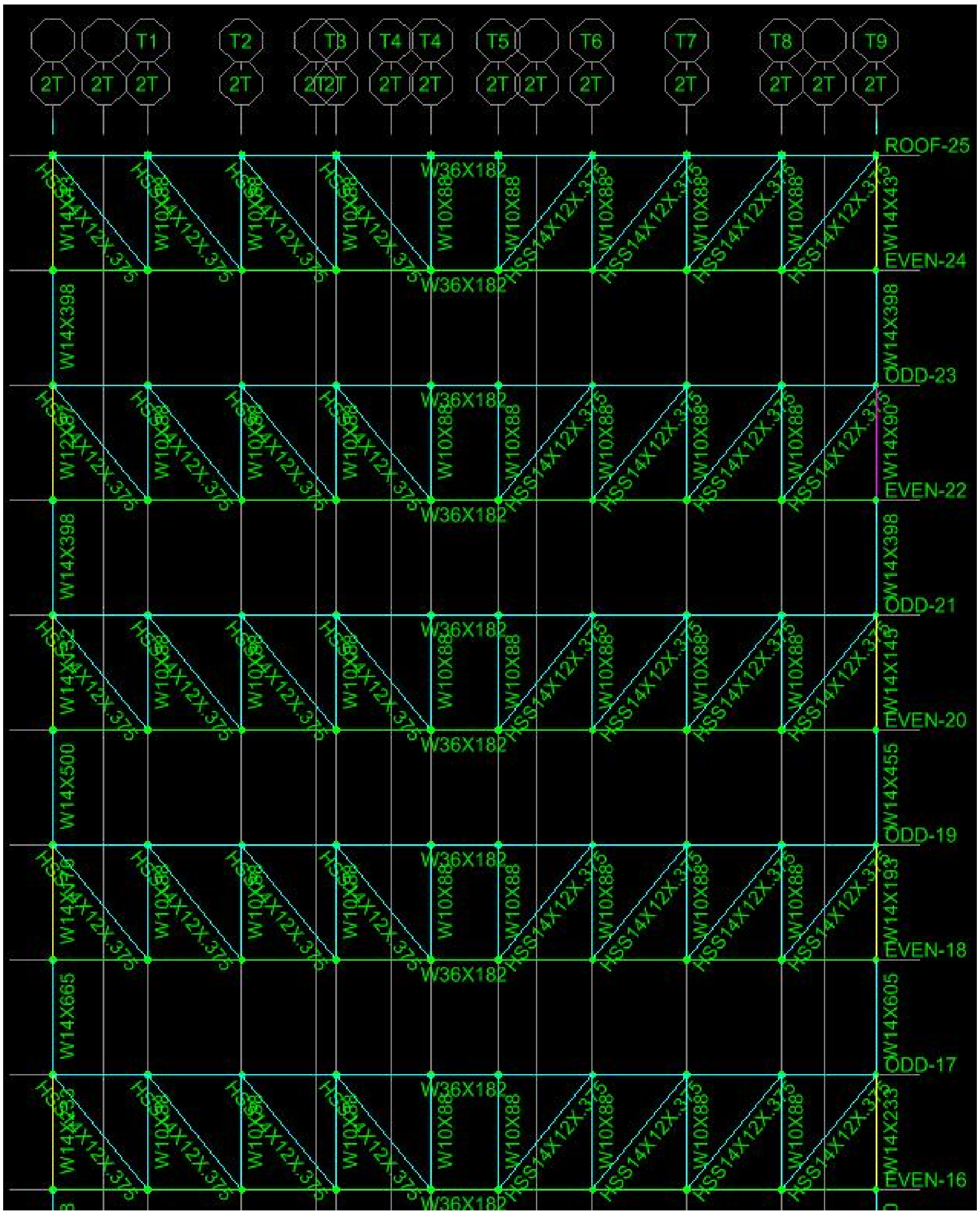
Design of Staggered Truss Chords - Truss 3						
Floor	Φ_h	M_{uw}	Total M_u	P_u	Section	LRFD Eq. 1-1a
25	0.04	4.07	44.32	533.85	W10x54	0.95
24	0.09	8.13	48.38	533.85	W10x54	0.97
23	0.13	12.11	52.36	533.85	W10x54	0.98
22	0.18	16.95	57.20	533.85	W10x54	1.00
21	0.23	21.49	61.74	533.85	W10x54	1.01
20	0.27	25.56	65.81	533.85	W10x60	0.92
19	0.32	29.61	69.86	533.85	W10x60	0.94
18	0.36	33.60	73.85	533.85	W10x60	0.95
17	0.40	37.56	77.81	533.85	W10x60	0.96
16	0.44	41.50	81.75	533.85	W10x60	0.97
15	0.49	45.41	85.66	533.85	W10x60	0.99
14	0.53	49.30	89.55	533.85	W10x60	1.00
13	0.57	53.15	93.40	533.85	W10x60	1.01
12	0.61	56.97	97.22	533.85	W10x68	0.90
11	0.65	60.76	101.01	533.85	W10x68	0.91
10	0.69	64.50	104.75	533.85	W10x68	0.92
9	0.73	68.20	108.45	533.85	W10x68	0.93
8	0.77	71.85	112.10	533.85	W10x68	0.94
7	0.81	75.63	115.88	533.85	W10x68	0.95
6	0.85	79.35	119.60	533.85	W10x68	0.96
5	0.89	83.01	123.26	533.85	W10x68	0.97
4	0.93	86.60	126.85	533.85	W10x68	0.98
3	0.96	90.12	130.37	533.85	W10x68	0.99
2	1.00	93.55	133.80	533.85	W10x68	1.00
25	0.04	4.07	44.32	533.85	W10x54	0.95
Ground						

Design of Column 4A										
	Axial Force				Moment	Load Combinations				Section
	Floor		Total			1.4D		1.2D + 1.6L		
	DL_1	DL_2 + RLL	DL_1	DL_2 + RLL	DL_1	P_u	M_u	P_u	M_u	
25	183.3	292.22	274.95	438.33	42.56761	384.93	59.59466	526.00	51.08114	W12x72
24			274.95	438.33		384.93		526.00		W12x72
23	183.3	292.22	458.25	730.55	84.4013	641.55	118.1618	876.66	101.2816	W12x120
22			458.25	730.55		641.55		876.66		W12x120
21	183.3	292.22	641.55	1022.77	108.1113	898.17	151.3559	1227.32	129.7336	W12x152
20			641.55	1022.77		898.17		1227.32		W12x152
19	183.3	292.22	824.85	1314.99	138.8509	1154.79	194.3913	1577.99	166.6211	W12x210
18			824.85	1314.99		1154.79		1577.99		W12x210
17	183.3	292.22	1008.15	1607.21	173.1454	1411.41	242.4036	1928.65	207.7745	W12x252
16			1008.15	1607.21		1411.41		1928.65		W12x252
15	183.3	292.22	1191.45	1899.43	219.5685	1668.03	307.3959	2279.32	263.4822	W12x305
14			1191.45	1899.43		1668.03		2279.32		W12x305
13	183.3	292.22	1374.75	2191.65	204.3103	1924.65	286.0345	2629.98	245.1724	W12x305
12			1374.75	2191.65		1924.65		2629.98		W12x305
11	183.3	292.22	1558.05	2483.87	372.5659	2181.27	521.5923	2980.64	447.0791	W14x370
10			1558.05	2483.87		2181.27		2980.64		W14x370
9	183.3	292.22	1741.35	2776.09	405.1869	2437.89	567.2617	3331.31	486.2243	W14x496
8			1741.35	2776.09		2437.89		3331.31		W14x496
7	183.3	292.22	1924.65	3068.31	494.4654	2694.51	692.2515	3681.97	593.3585	W14x455
6			1924.65	3068.31		2694.51		3681.97		W14x455
5	183.3	292.22	2107.95	3360.53	557.9905	2951.13	781.1866	4032.64	669.5885	W14x550
4			2107.95	3360.53		2951.13		4032.64		W14x550
3	183.3	292.22	2291.25	3652.75	631.8169	3207.75	884.5436	4383.30	758.1803	W14x550
2			2291.25	3652.75		3207.75		4383.30		W14x550
Ground	183.3	292.22	2474.55	3944.97	42.568					

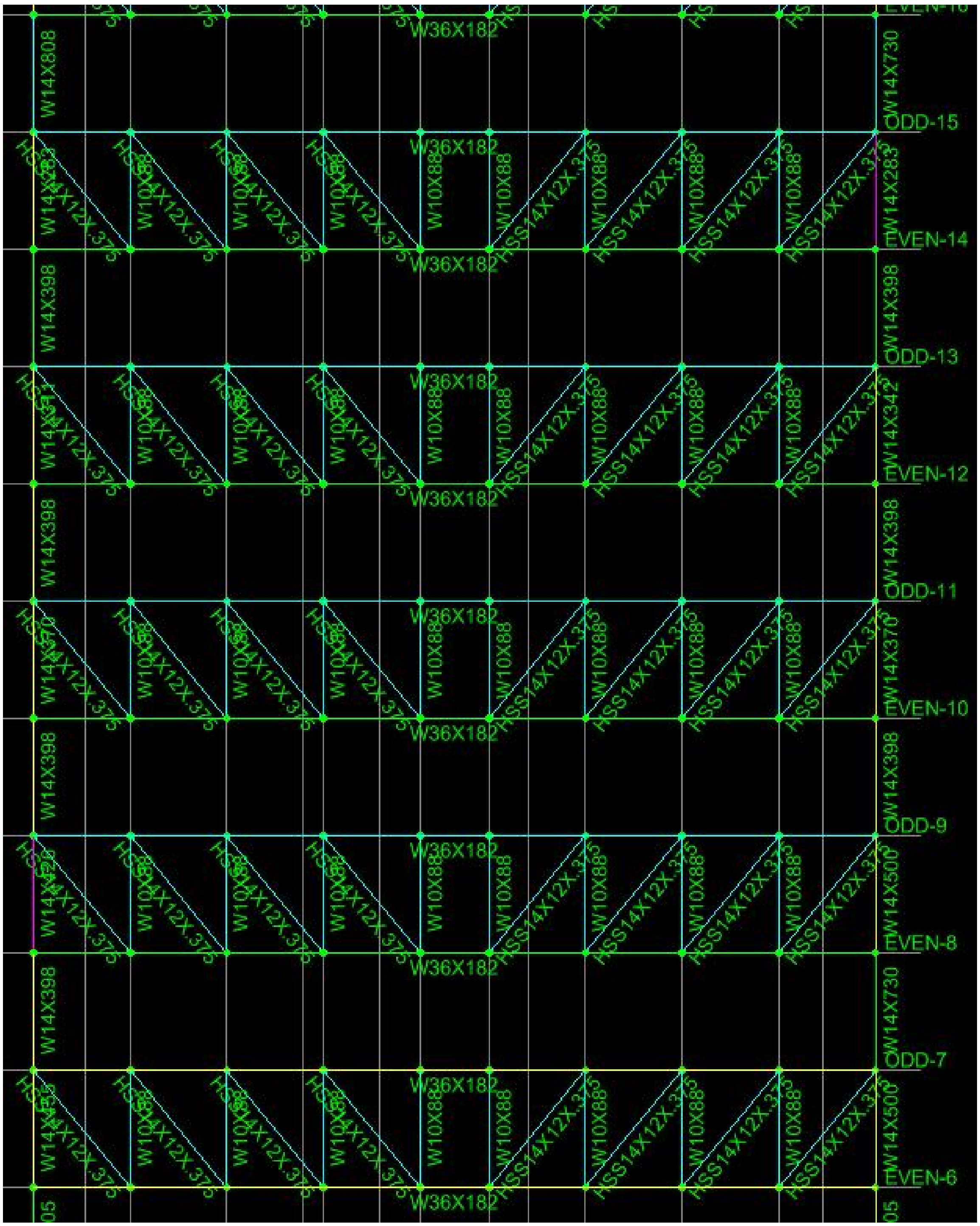
Appendix D

ETABS Output Data

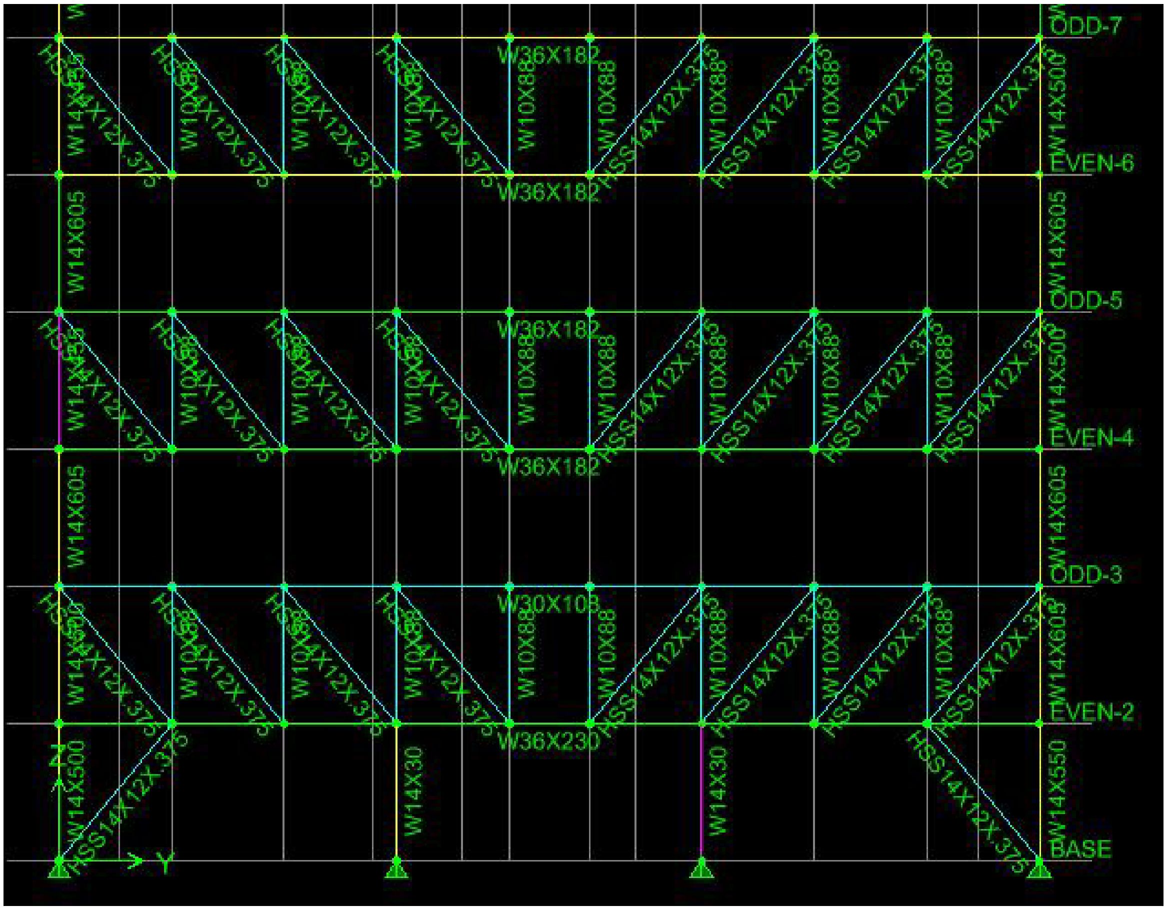
Sample Elevation: Truss 2 (16th Level to Roof)



Sample Elevation: Truss 2 (6th Level to 15th Level)



Sample Elevation for Truss 2 (Base to 7th Floor):



Elements for Levels 17 to 25

Story	ElementType	Material	TotalWeight	FloorArea	NumPieces
ROOF-25	Column	STEEL	46.788	1578105	74
ROOF-25	Beam	STEEL	119.066	1578105	55
ROOF-25	Brace	STEEL	21.839	1578105	28
ROOF-25	Floor	CONC	1643.755	1578105	
EVEN-24	Column	STEEL	81.172	1578105	56
EVEN-24	Beam	STEEL	113.451	1578105	54
EVEN-24	Brace	STEEL	12.48	1578105	16
EVEN-24	Floor	CONC	1643.755	1578105	
ODD-23	Column	STEEL	37.435	1578105	58
ODD-23	Beam	STEEL	126.739	1578105	55
ODD-23	Brace	STEEL	18.72	1578105	24
ODD-23	Floor	CONC	1643.755	1578105	
EVEN-22	Column	STEEL	82.229	1578105	56
EVEN-22	Beam	STEEL	111.908	1578105	54
EVEN-22	Brace	STEEL	12.48	1578105	16
EVEN-22	Floor	CONC	1643.755	1578105	
ODD-21	Column	STEEL	44.268	1578105	58
ODD-21	Beam	STEEL	124.541	1578105	55
ODD-21	Brace	STEEL	18.72	1578105	24
ODD-21	Floor	CONC	1643.755	1578105	
EVEN-20	Column	STEEL	85.222	1578105	56
EVEN-20	Beam	STEEL	112.692	1578105	54
EVEN-20	Brace	STEEL	12.48	1578105	16
EVEN-20	Floor	CONC	1643.755	1578105	
ODD-19	Column	STEEL	55.513	1578105	58
ODD-19	Beam	STEEL	122.823	1578105	55
ODD-19	Brace	STEEL	18.72	1578105	24
ODD-19	Floor	CONC	1643.755	1578105	
EVEN-18	Column	STEEL	91.34	1578105	56
EVEN-18	Beam	STEEL	110.266	1578105	54
EVEN-18	Brace	STEEL	12.48	1578105	16
EVEN-18	Floor	CONC	1643.755	1578105	
ODD-17	Column	STEEL	57.543	1578105	58
ODD-17	Beam	STEEL	125.166	1578105	55
ODD-17	Brace	STEEL	18.72	1578105	24
ODD-17	Floor	CONC	1643.755	1578105	

Elements for Levels 8 to 16

Story	ElementType	Material	TotalWeight	FloorArea	NumPieces
EVEN-16	Column	STEEL	96.389	1578105	56
EVEN-16	Beam	STEEL	110.346	1578105	54
EVEN-16	Brace	STEEL	12.48	1578105	16
EVEN-16	Floor	CONC	1643.755	1578105	
ODD-15	Column	STEEL	64.244	1578105	58
ODD-15	Beam	STEEL	124.805	1578105	55
ODD-15	Brace	STEEL	18.72	1578105	24
ODD-15	Floor	CONC	1643.755	1578105	
EVEN-14	Column	STEEL	91.765	1578105	56
EVEN-14	Beam	STEEL	110.117	1578105	54
EVEN-14	Brace	STEEL	12.48	1578105	16
EVEN-14	Floor	CONC	1643.755	1578105	
ODD-13	Column	STEEL	71.819	1578105	58
ODD-13	Beam	STEEL	124.706	1578105	55
ODD-13	Brace	STEEL	18.72	1578105	24
ODD-13	Floor	CONC	1643.755	1578105	
EVEN-12	Column	STEEL	95.659	1578105	56
EVEN-12	Beam	STEEL	109.454	1578105	54
EVEN-12	Brace	STEEL	12.48	1578105	16
EVEN-12	Floor	CONC	1643.755	1578105	
ODD-11	Column	STEEL	78.956	1578105	58
ODD-11	Beam	STEEL	124.382	1578105	55
ODD-11	Brace	STEEL	18.72	1578105	24
ODD-11	Floor	CONC	1643.755	1578105	
EVEN-10	Column	STEEL	97.514	1578105	56
EVEN-10	Beam	STEEL	109.206	1578105	54
EVEN-10	Brace	STEEL	12.48	1578105	16
EVEN-10	Floor	CONC	1643.755	1578105	
ODD-9	Column	STEEL	92.295	1578105	58
ODD-9	Beam	STEEL	124.236	1578105	55
ODD-9	Brace	STEEL	18.72	1578105	24
ODD-9	Floor	CONC	1643.755	1578105	
EVEN-8	Column	STEEL	96.865	1578105	55
EVEN-8	Beam	STEEL	113.072	1578105	59
EVEN-8	Brace	STEEL	25.967	1578105	21
EVEN-8	Floor	CONC	1643.755	1578105	

Elements for Base to Level 7

Story	ElementType	Material	TotalWeight	FloorArea	NumPieces
ODD-7	Column	STEEL	83.85	1651465	49
ODD-7	Beam	STEEL	116.972	1651465	40
ODD-7	Brace	STEEL	18.72	1651465	24
ODD-7	Floor	CONC	1643.755	1578105	
EVEN-6	Column	STEEL	88.603	1651465	47
EVEN-6	Beam	STEEL	101.824	1651465	39
EVEN-6	Brace	STEEL	21.792	1651465	17
EVEN-6	Floor	CONC	1643.755	1578105	
ODD-5	Column	STEEL	82.08	1651465	49
ODD-5	Beam	STEEL	101.651	1651465	40
ODD-5	Brace	STEEL	19.862	1651465	27
ODD-5	Floor	CONC	1643.755	1578105	
EVEN-4	Column	STEEL	87.851	1651465	47
EVEN-4	Beam	STEEL	91.63	1651465	39
EVEN-4	Brace	STEEL	17.667	1651465	19
EVEN-4	Floor	CONC	1643.755	1578105	
ODD-3	Column	STEEL	99.77	1651465	65
ODD-3	Beam	STEEL	98.218	1651465	39
ODD-3	Brace	STEEL	29.869	1651465	30
ODD-3	Floor	CONC	1643.755	1578105	
EVEN-2	Column	STEEL	79.914	1651465	35
EVEN-2	Beam	STEEL	110.359	1651465	39
EVEN-2	Brace	STEEL	8.83	1651465	13
EVEN-2	Floor	CONC	1643.755	1578105	
BASE	Floor	CONC	423.597	406679.6	

System Summary

Story	ElementType	Material	TotalWeight	FloorArea	NumPieces
SUM	Column	STEEL	1889.085	38721365	1333
SUM	Beam	STEEL	2737.631	38721365	1222
SUM	Brace	STEEL	414.139	38721365	499
SUM	Floor	CONC	40332.17	38721365	
TOTAL	All	All	53572.24	38721365	3054

Material List by Section

Section	ElementType	NumPieces	TotalLength	TotalWeight
W10X45	Column	22	2706	10.185
W10X49	Column	11	1353	5.514
W10X88	Column	107	13161	96.466
W12X87	Column	29	3567	25.842
W12X106	Column	22	2706	23.893
W12X120	Column	24	2952	29.49
W14X22	Column	233	28659	52.637
W14X22	Beam	9	533.25	0.906
W14X22	Brace	4	685.211	1.259
W14X26	Column	21	2583	5.621
W14X30	Column	129	15867	39.74
W14X30	Brace	6	946.477	2.371
W14X34	Column	12	1476	4.177
W14X34	Beam	5	1243.75	3.43
W14X38	Column	10	1230	3.899
W14X38	Beam	2	497.5	1.545
W14X43	Column	63	7749	27.631
W14X43	Beam	14	3627.75	12.468
W14X48	Column	15	1845	7.362
W14X48	Beam	3	862.75	3.354
W14X53	Column	6	738	3.258
W14X61	Column	63	7749	39.254
W14X68	Column	17	2091	11.835
W14X74	Column	12	1476	9.106
W14X74	Brace	1	184.859	1.14
W14X82	Column	11	1353	9.19
W14X90	Column	16	1968	14.759
W14X99	Column	23	2829	23.298
W14X109	Column	22	2706	24.506
W14X120	Column	19	2337	23.346
W14X132	Column	27	3321	36.466
W14X145	Column	28	3444	41.618
W14X159	Column	47	5781	76.402
W14X176	Column	48	5904	86.549
W14X176	Brace	1	184.859	2.71
W14X193	Column	29	3567	57.337
W14X211	Column	10	1230	21.582
W14X233	Column	50	6150	119.221
W14X233	Brace	2	369.719	7.167

Material List by Section (continued)

Section	ElementType	NumPieces	TotalLength	TotalWeight
W14X257	Column	14	1722	36.842
W14X283	Column	7	861	20.297
W14X311	Column	5	615	15.908
W14X342	Column	8	984	28.126
W14X370	Column	12	1476	45.53
W14X398	Column	14	1722	57.017
W14X426	Column	61	7503	265.419
W14X426	Brace	1	184.859	6.539
W14X455	Column	17	2091	79.295
W14X500	Column	15	1845	76.754
W14X500	Brace	1	184.859	7.69
W14X550	Column	11	1353	62.03
W14X605	Column	39	4797	241.644
W14X605	Brace	1	184.859	9.312
W14X665	Column	1	123	6.823
W14X730	Column	2	246	14.968
W14X808	Column	1	123	8.25
W18X50	Beam	703	141914.8	551.644
W18X60	Beam	46	14340.31	68.791
W21X55	Beam	47	12404.92	54.025
W21X62	Beam	18	6193.104	30.948
W21X68	Beam	21	6165.068	33.115
W24X62	Beam	5	1206	5.857
W24X76	Beam	86	27287.64	165.65
W24X84	Beam	26	7296.074	48.763
W27X94	Beam	34	11071.41	83.638
W27X102	Beam	21	8463.482	70.153
W27X114	Beam	1	882	8.197
W30X108	Beam	4	1766.964	15.697
W30X116	Beam	2	1221.482	11.774
W30X124	Beam	10	3049.287	30.22
W33X130	Beam	5	2648.964	28.168
W33X141	Beam	4	1633.482	18.822
W36X150	Beam	2	643.232	7.846
W36X170	Beam	14	11672	162.277
W36X182	Beam	140	88700.09	1320.342
HSS14X12X.375	Brace	482	76788.76	375.95
WALL1	Wall			8199.209
PLANK1	Floor			37609.53

Story Drift Summary for Controlling Lateral Load Case, Wind in N-S Direction

Story	Item	Load	Point	X	Y	Z	DriftX	DriftY
EVEN-24	Max Drift X	WINDY	60	1379	303.75	2829	0	
EVEN-24	Max Drift Y	WINDY	60	1379	303.75	2829		0
ODD-23	Max Drift X	WINDY	10	1723	0	2706	0.000041	
ODD-23	Max Drift Y	WINDY	1123	2046.088	466.452	2706		0.000051
EVEN-22	Max Drift X	WINDY	10	1723	0	2583	0.000041	
EVEN-22	Max Drift Y	WINDY	1123	2046.088	466.452	2583		0.000051
ODD-21	Max Drift X	WINDY	10	1723	0	2460	0.000029	
ODD-21	Max Drift Y	WINDY	1123	2046.088	466.452	2460		0.000046
EVEN-20	Max Drift X	WINDY	10	1723	0	2337	0.000029	
EVEN-20	Max Drift Y	WINDY	1123	2046.088	466.452	2337		0.000046
ODD-19	Max Drift X	WINDY	10	1723	0	2214	0.000024	
ODD-19	Max Drift Y	WINDY	1123	2046.088	466.452	2214		0.000042
EVEN-18	Max Drift X	WINDY	10	1723	0	2091	0.000024	
EVEN-18	Max Drift Y	WINDY	1123	2046.088	466.452	2091		0.000042
ODD-17	Max Drift X	WINDY	10	1723	0	1968	0.000021	
ODD-17	Max Drift Y	WINDY	1123	2046.088	466.452	1968		0.000039
EVEN-16	Max Drift X	WINDY	10	1723	0	1845	0.000021	
EVEN-16	Max Drift Y	WINDY	1123	2046.088	466.452	1845		0.000039
ODD-15	Max Drift X	WINDY	10	1723	0	1722	0.000019	
ODD-15	Max Drift Y	WINDY	1123	2046.088	466.452	1722		0.000036
EVEN-14	Max Drift X	WINDY	10	1723	0	1599	0.000019	
EVEN-14	Max Drift Y	WINDY	1123	2046.088	466.452	1599		0.000036
ODD-13	Max Drift X	WINDY	10	1723	0	1476	0.000017	
ODD-13	Max Drift Y	WINDY	1123	2046.088	466.452	1476		0.000032
EVEN-12	Max Drift X	WINDY	10	1723	0	1353	0.000017	
EVEN-12	Max Drift Y	WINDY	1123	2046.088	466.452	1353		0.000032
ODD-11	Max Drift X	WINDY	10	1723	0	1230	0.000012	
ODD-11	Max Drift Y	WINDY	1123	2046.088	466.452	1230		0.000031
EVEN-10	Max Drift X	WINDY	10	1723	0	1107	0.000012	
EVEN-10	Max Drift Y	WINDY	1123	2046.088	466.452	1107		0.000031
ODD-9	Max Drift X	WINDY	1200	1723	882	984	0.000035	
ODD-9	Max Drift Y	WINDY	67	-64.25	362	984		0.000066
EVEN-8	Max Drift X	WINDY	93	1955.078	439.328	861	0.001673	
EVEN-8	Max Drift Y	WINDY	93	1955.078	439.328	861		0.009267
ODD-7	Max Drift X	WINDY	2	194	0	738	0.000039	
ODD-7	Max Drift Y	WINDY	67	-64.25	362	738		0.000065
EVEN-6	Max Drift X	WINDY	49	1379	283	615	0.000029	
EVEN-6	Max Drift Y	WINDY	8	1585	0	615		0.000042

Story	Item	Load	Point	X	Y	Z	DriftX	DriftY
ODD-5	Max Drift X	WINDY	39	1379	202.5	492	0.000027	
ODD-5	Max Drift Y	WINDY	49	1379	283	492		0.00004
EVEN-4	Max Drift X	WINDY	1174	351	780.75	369	0.000029	
EVEN-4	Max Drift Y	WINDY	50	1585	283	369		0.000063
ODD-3	Max Drift X	WINDY	1192	194	882	246	0.000024	
ODD-3	Max Drift Y	WINDY	67	-64.25	362	246		0.000068
EVEN-2	Max Drift X	WINDY	2	194	0	123	0.000061	
EVEN-2	Max Drift Y	WINDY	1136	10	519	123		0.000159

Appendix E

Fire Protection Calculations

Thickness Check for Sample Truss Column: W12×72

Spray-Applied Fire Resistant Materials						
Section:		R	W/D	(W/D from LRFD Table 1-36: Case B)		
W12×72		3	1.02	where R = [C1(W/D)+C2]*h		
R = Fire Resistance Rating (hrs)				h = thickness of application		
	Grace MK6	Isolatek 800	Isolatek 280	Isolatek 280	Isolatek D-C/F	Isolatek D-C/F
C1	1.05	0.86	1.25	1.25	1.01	0.95
C2	0.61	0.97	0.53	0.25	0.66	0.45
h	1.785	1.624	1.662	1.967	1.775	2.114
Min./Max. W/D Requirements	OK	OK	OK	OK	OK	N/A
	OK	OK	OK	OK	OK	OK
Rank	4	1	2	5	3	6

Minimum thickness required for W12×72 Section: 1.662" ≈ 1.75" for R = 3 hr fire rating

Gypsum Wallboard Type X Board			
R = 130*[h(W'/D)/2]^0.75			
Section	W12x72		
Section Properties	bf =	12	in
	d =	12.3	in
	W =	72	plf
Gypsum Thickness to Check	h =	1.5	in
Weight of Column and Wallboard	W' =	97.3125	plf
Inside Perimeter of Wallboard	D =	48.6	in
Assembly Fire Rating	**R =**	**176.36**	**min**

1 ½ inch wallboard does not provide enough for R = 3 hrs = 180 min for W12×72 Section

Try 2 in thickness:

Gypsum Wallboard Type X Board			
R = 130*[h(W'/D)/2]^0.75			
Section	W12x72		
	bf =	12	in
Section Properties	d =	12.3	in
	W =	72	plf
Gypsum Thickness to Check	h =	2	in
Weight of Column and Wallboard	W' =	105.75	plf
Inside Perimeter of Wallboard	D =	48.6	in
Assembly Fire Rating	R =	232.90	min

2 inch thickness of wallboard gives R =3 (nearly 4) for W12×72 Section

Concrete Protection for Columns: Full Encasement			
Rough Concrete Encasement Dimensions	Width	13.5	in
	Depth	13.5	in
Column Section Properties	Section	W12x72	
	bf =	12	in
	d =	12.3	in
	As =	21.1	in2
	W =	72	plf
	tw =	0.43	in
	tf =	0.67	in
	T =	9.125	in
Interior Perimeter of Encasement (from Table 1-36)	D =	70.3	in
Average thickness of Conc. Encasement	h =	1.35	in
Average Interior Dimension of One Side of Square Conc. Box Protection	L =	12.15	in
Conc Density	Rhoc =	145	pcf
Ambient Spec Heat of Conc	cc =	0.2	Btu/lbF
Moisture Content of Concrete by Volume	m =	4	% volume
Thermal Capacity of Steel Column	H =	25.58835	
Fire endurance rating at zero moisture	Ro =	167.422	min
		2.790367	hrs
Fire endurance rating at actual moisture condition	R =	187.5127	
		3.125211	hrs

13.5" square concrete encasement around W12×72 Section provides R = 3 hr fire rating (Average thickness of 1.35" around steel section)

Appendix F

Cost Analysis Data and Calculations

Existing Design: Building Statistics

- **Building Name:** River Tower at Christina Landing
- **Location and Site:** 115 Christina Landing, Wilmington, DE, 19801
- **Building Occupant Name:** The Buccini Pollin Group
- **Occupancy or Function Type:**
    - **Primary Occupancy:** Condominium Building
    - **Accessory Occupancy:** Enclosed Parking Garage interfaced in lower eight stories
- **Size (Total Sq. Ft.):** Approximately 433,200 Sq. Ft.
- **Number of Stories Above Grade:** 25 stories
- **Dates of Construction:**
    - **Planned:** May 1, 2005 to November, 2006 (18 months)
    - **Actual:** September 1, 2005 to April 1, 2007 (18 months)
- **Costs:**

Original estimates:

- **Overall Project:** $55.5 million
- **Building Cost:** $46 million
- **Soft Costs:** $4 million (CM fees, permits, site services, laborers) and an additional $5 million of environmental remediation (contamination due to buried oil tanks from old tannery/oil storage yard)

- **Project Delivery Method:** Originally: Design-Bid-Build
After redesign: Fast Track/Design-Build

Cost Analysis Calculations

Existing Concrete System Estimates: Using R.S. Means Cost Data: Unit Costs								
Concrete Column Estimate: Using 36" square columns, max reinf.							Column CY =	129.08
Crew	Daily Output	Labor Hrs	Unit	Bare Materials	Bare Labor	Bare Equipment	Bare Total	Total Incl. O&P
C14A	17.80	11.22	CY	$ 400.00	$ 365.00	$ 48.50	$ 813.50	$ 1,075.00
	7.25	1415.16		$ 51,633.54	$ 47,115.60	$ 6,260.57	$ 105,009.71	$ 138,765.14
Shear Wall estimates (taken from Grade Walls, 15" thick, interpolated between 8' and 12' high)							Shear Wall CY =	442.100823
Crew	Daily Output	Labor Hrs	Unit	Bare Materials	Bare Labor	Bare Equipment	Bare Total	Total Incl. O&P
C14B	65.75	3.20	CY	$ 98.25	$ 103.00	$ 13.78	$ 215.03	$ 283.50
	1415.16	442.10		$ 43,436.41	$ 45,536.38	$ 6,092.15	$ 95,064.94	$ 125,335.58
Cast In Place Concrete 5000 psi Ready-Mix							Total Concrete CY =	1416.657735
Crew	Daily Output	Labor Hrs	Unit	Bare Materials	Bare Labor	Bare Equipment	Bare Total	Total Incl. O&P
			CY	$ 71.00			$ 71.00	$ 78.00
				$ 100,582.70			$ 100,582.70	$ 110,499.30
Prestressed Concrete (Large Job)							Total Concrete CY =	6686.364
Crew	Daily Output	Labor Hrs	Unit	Bare Materials	Bare Labor	Bare Equipment	Bare Total	Total Incl. O&P
C17B	10	8.2	CY	$ 530.00	$ 350.00	$ 30.00	$ 910.00	$ 1,150.00
	668.6641975	54830.46	0	$ 3,543,920.25	$ 2,340,324.69	$ 200,599.26	$ 6,084,844.20	$ 7,689,638.27
Totals								
	2091.08	56687.73		$ 3,739,572.89	$ 2,432,976.68	$ 212,951.98	$ 6,385,501.55	$ 8,064,238.29
							Add 5% waste:	$ 8,467,450.21

Proposed Steel System Estimates: Using R.S. Means Cost Data: Unit and Assembly Costs								
Steel projects: Apartments over 15 stories						Member	Weight (k) =	Tonnage =
						Column	1889.085	944.54
						Beam	2737.631	1368.82
						Brace	414.139	207.07
Crew	Daily Output	Labor Hrs	Unit	Bare Materials	Bare Labor	Bare Equipment	Bare Total	Total Incl. O&P
E6	13.9	9.209	TON	$ 1,900.00	$ 390.00	$ 129.00	$ 2,419.00	$ 2,900.00
Column	67.95	9.21	TON	$ 1,794,630.75	$ 368,371.58	$ 121,845.98	$2,284,848.31	$ 2,739,173.25
Beam	98.48	8698.29	TON	$ 2,600,749.45	$ 533,838.05	$ 176,577.20	$3,311,164.69	$ 3,969,564.95
Brace	14.90	12605.42	TON	$ 393,432.05	$ 80,757.11	$ 26,711.97	$ 500,901.12	$ 600,501.55
TOTALS:	181.33	1906.90		$ 4,788,812.25	$ 982,966.73	$ 325,135.15	$6,096,914.12	$ 7,309,239.75

<u>Accounting for Differential in Sizing:</u>

Approximately 200 plf differential in each exterior truss column:

(10 truss columns per floor)*(200 plf)*(10.25 ft height)*(24 floors) = 492 kip reduction in column weight

Approximately 150 plf differential in truss chords on each floor:

(5 chords per floor)*(150 plf)*(73.5 ft width)*(24 floors) = 1323 kip reduction in beam weight

Approximate adjustment on ETABS output = $2,630,000 (see next page)

Adjusted Steel System Estimates to Account for Sizing Discrepancy								
Steel projects: Apartments over 15 stories						Member	Weight (k) =	Tonnage =
						Column	1397.09	698.54
						Beam	1414.63	707.32
						Brace	414.14	207.07
Crew	Daily Output	Labor Hrs	Unit	Bare Materials	Bare Labor	Bare Equipment	Bare Total	Total Incl. O&P
E6	13.9	9.209	TON	$ 1,900.00	$ 390.00	$ 129.00	$ 2,419.00	$ 2,900.00
Column	50.25	6432.88	TON	$ 1,327,230.75	$ 272,431.58	$ 90,111.98	$1,689,774.31	$ 2,025,773.25
Beam	50.89	6513.66	TON	$ 1,343,898.50	$ 275,852.85	$ 91,243.64	$1,710,994.99	$ 2,051,213.50
Brace	14.90	1906.90	TON	$ 393,432.05	$ 80,757.11	$ 26,711.97	$ 500,901.12	$ 600,501.55
TOTALS:	116.04	618.89		$ 3,064,561.30	$ 629,041.53	$ 208,067.58	$3,901,670.41	$ 4,677,488.30

Assembly Cost for Precast Plank Flooring						
Total Area	Span	Total Depth	Superimposed Load	Materials	Installation	Total/SF
270,809 SF	30 ft	8 in	82 psf	$6.75	$3.75	$10.50
			Costs =	$1,827,960.75	$1,015,553.75	$2,843,494.50

Total Structure =	**$7,520,952.80**
Add 5% waste	$7,897,000.44
Add 10% Connections and Fabrication Costs	$8,649,095.72

Staggered Truss System is **$181,645.51 more expensive**

Appendix G

List of Resources

Structural References

BOCA National Building Code 1996. Thirteenth Edition. Building Officials and Code Administrators International, 1996.

Etabs 8 Nonlinear. Computer software. Version 8.2.6. Computers and Structures, Inc., 2003.

Habibullah, Ashraf. "Steel Frame Design: Staggered Truss Framing Systems Using ETABS." Computer and Structures, 2005. <http://www.csiberkeley.com/Tech_Info/ StaggeredTrussTechnicalNote.doc>.

Manual of Steel Construction: Load and Resistance Factor Design. Third Edition. American Institute of Steel Construction, Inc., 2001.

PCI Design Handbook: Precast and Prestressed Concrete. Fifth Edition. Precast/Prestressed Concrete Institute, 1999.

Precast/Prestressed Concrete Products and Building Systems. Nitterhouse Concrete Products, 2005. <http://www.nitterhouse.com>.

Structural Steel Framing Solutions for Multi-Story Residential Buildings. American Institute of Steel Construction. AISC Marketing, 2004.

United Steel Deck Design Manual and Catalog of Products. Catalog #303-16. United Steel Deck, Inc., 2002.

Wexler, Neil, and Feng-Bao Lin. *AISC Design Guide 14: Staggered Truss Framing Systems.* American Institute of Steel Construction, 2001.

Fire Protection Systems References

BOCA National Building Code 1999. Fourteenth Edition. Building Officials and Code Administrators International, 1999.

Bromann, Mark. The Design and Layout of Fire Sprinkler Systems. Second Edition. Lancaster, PA: Technomic Publishing Company, 2001.

Lindeburg, Michael R. Fire and Explosion Protection Systems: A Design Professional's Introduction. Second Edition. Belmont, CA: Professional Publications, 1995.

Ruddy, John, et al. *AISC Design Guide 19: Fire Resistance of Structural Steel Framing.* American Institute of Steel Construction, 2003.

Stein, Benjamin, and John S. Reynolds. Mechanical and Electrical Equipment for Buildings. Ninth Edition. New York: John Wiley & Sons, 2000.

Construction Management References

Costworks 2005. Computer software. Version 8.0. R.S. Means, 2004.

R.S. Means Assemblies Cost Data. 31^{st} Annual Edition. Reed Construction Data, 2006.