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Structural Option
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Structural Technical Report #1: Structural Concepts/Structural Existing Conditions

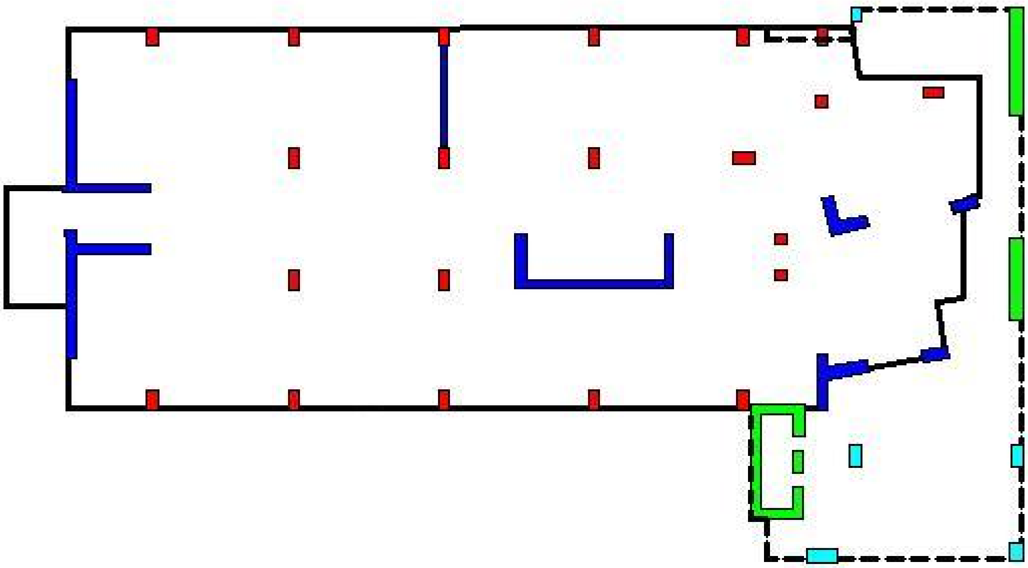
Executive Summary

The following report is a detailed summary of the structural system of the River Tower at Christina Landing. The River Tower is part of the latest phase of redevelopment along the banks of the Christina River in Wilmington, DE. The redevelopment site consists of luxury townhouses, a 22-story apartment building, and will now add the River Tower, a 25-story condominium tower. This tower has since been redesigned for value engineering, and has added two stories to create a 27-story condominium tower. The drawings that I have procured, however, reference the original 25-story design, and this design will be the focus of my analysis and research.

This initial report will provide an introduction to the overall existing design of the structural system of River Tower at Christina Landing. The applicable codes and standards used in the design of this system are listed, along with some typical framing plans and elevations to illustrate the overall framing layout of the building. Loading diagrams showing the required seismic and wind loads, determined through code research and calculation, are given as well. Finally, a spot check for gravity loads in a typical floor slab, as well as a simplified check for one of the shear walls as an example, is provided in this report.

In my preliminary analysis of the River Tower, I have found that most of the design criteria do not deviate from BOCA 1996, as my research has produced the same values as those provided on the construction documents and drawings. The structural system for this building presents an especially challenging analysis due to my lack of experience with post-tensioned concrete flat plate slab design. I attempted to analyze a 12 inch section of the slab as a post-tensioned beam, as I was fairly aware of that procedure. However, my over-simplification appears to have rendered this analysis inclusive. While I hope to learn more about this system as my research work progresses, the following analyses are mere first attempts and will be improved upon and added to with time. With a better understanding of the analysis of the post-tensioning slab system, I will be able to provide a more accurate check in upcoming reports.

Introduction to Structural System



Simplified Building Schematic for Typical Floor
(Levels 9 through 22):

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

Solid Lines: Tower Perimeter (whole building)
Dashed Line: Garage Addition (Up to 8th Floor)

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 HP12x84 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"x11'-0" and 38 inches thick, with reinforcement in both directions. An exterior pile cap will be 7'-9"x7'-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"x52", and its reinforcement and concrete strength decreases at upper floors. The exterior columns are 16"x36". 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"x9'-0"x3'-0" deep pile caps with (5) HP 12x84 piles. The interior walls will have a 6'-0" wide grade beam with HP12x84 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"x36". 12-inch thick shear walls located throughout the plan will resist the lateral loads on the parking garage.

Governing Codes and Standards

Primary Building Code
BOCA 1996 Building Official and Code Administration with City of Wilmington Amendments

Loads and Serviceability Requirements
American Society of Civil Engineers, "Minimum Design Loads for Buildings and Other Structures" (ASCE 7-02)

Concrete
American Concrete Institute, Building Code Requirements for Reinforced Concrete (ACI318-02)

Masonry
Building Code Requirements for Masonry Structures (ACI530-02/ASCE 5-02)

Structural Steel
American Institute of Steel Construction, Specification for Structural Steel Buildings

Light Steel Framing
American Iron and Steel Institute, Specifications for the Design of Cold-Formed Steel Structural Members, Specification for Structural Steel Buildings
Allowable Stress Design and Plastic Design (AISI CFSD-ASD)

Precast Concrete
Precast/Prestressed Concrete Institute, Design Handbook-Precast and Prestressed Concrete: Code of Standard Practice for Precast/Prestressed Concrete (PCI MNL-120)

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 Structural Loading

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*:* please see Appendix A for detailed calculations

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

Live Load Reductions

- "Live loads of 100 psf or less shall be reduced in accordance with the Code established procedure, except at the parking garage levels or roof, where live loads will not be reduced…" (consistent with BOCA 1996 1606.7.2.2). For simplicity, I did not consider these reductions on any level, although levels 8-22 would have been eligible.

Dead Load Calculation Results: please see Appendix B for detailed calculations

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

**Please see Appendix B for more clarification on assumptions on roof self weight. In addition to those self weights listed here, there has been an estimation of 20 psf for partition loads where applicable.*

Roof and Snow Loads

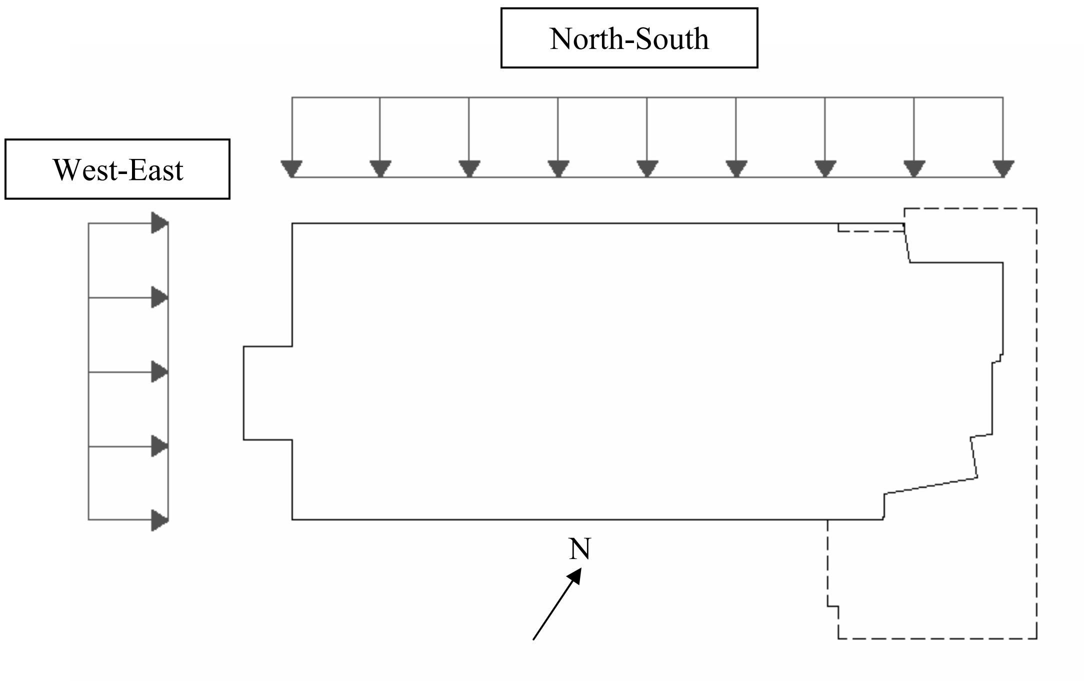
- Minimum Roof Live Load: 30 psf
- Ground Snow Load: 30 psf
- Snow Load Importance Factor: 1.0
- Snow Exposure Factor: 0.7
- Thermal Factor: 1.0
- Flat Roof Snow Load: 14.0 psf (Specified in construction documents as 20 psf minimum)
- *Please consult Appendix C for detailed Snow Load Calculations*

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

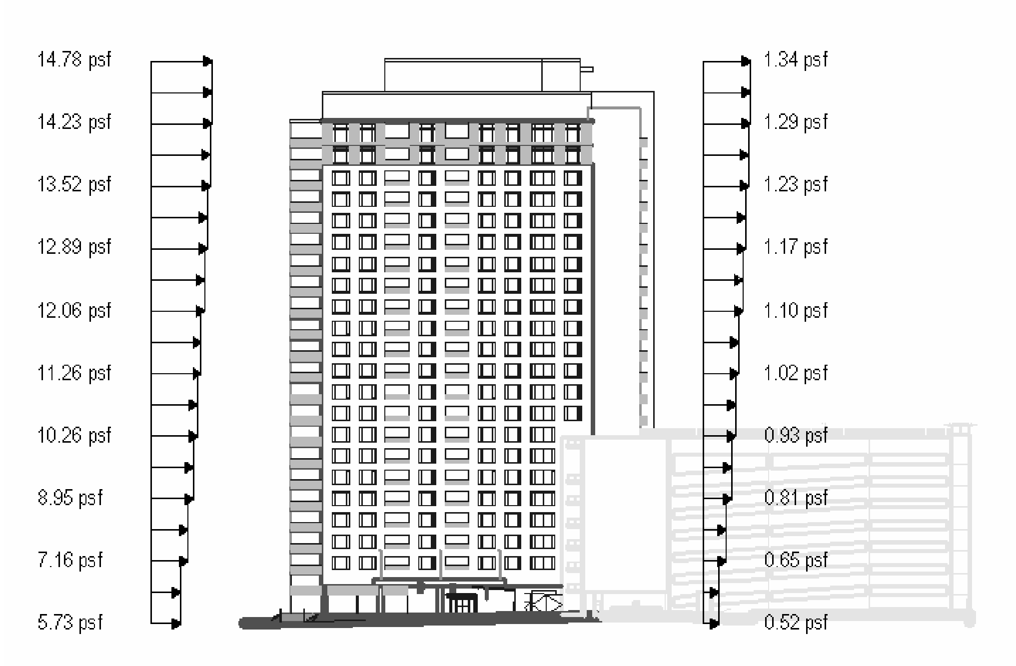
- Lateral wind and seismic loads:
  - Interstitial drift: L/400 *(where L= floor-to-floor height)*
- Vertical gravity and live loads:
  - L/360 under live loads
  - L/240 under total load *(where L= span of member under consideration in both cases)*

Wind Loads

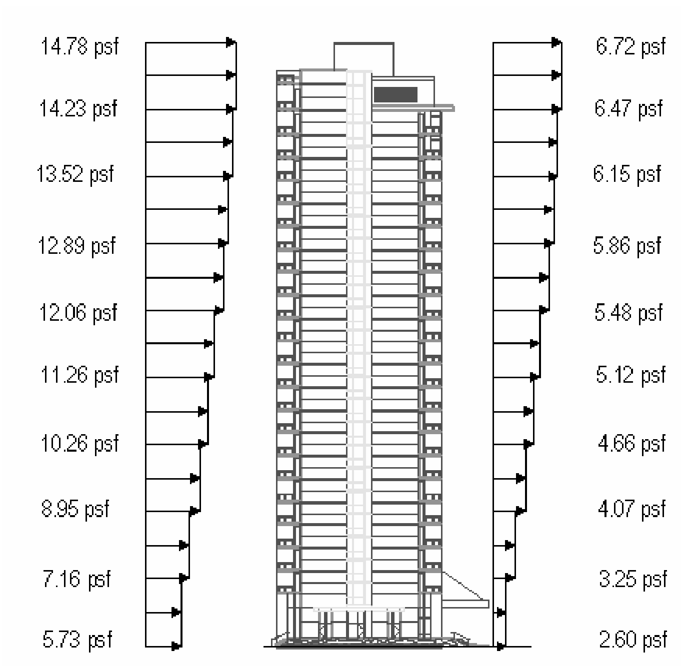
- Basic Wind Speed: 80 mph
- Wind Importance Factor: 1.04
- Wind Exposure: B
- Internal Pressure Coefficient: +/-0.25
- Components and Cladding Loads: vary per code requirements
- *Load Diagrams with results provided on next page*
- *Please consult Appendix D for detailed Wind Load Calculations*



Wind Direction Schematic:
Only tower considered for simplicity;
Parking garage (dashed) not considered



Wind Pressures (psf) in West-East Direction



Wind Pressures (psf) in North-South Direction

Seismic Loads

- 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 iteration
- Response Modification Factor, R = 6
- Site Coefficient, S_4 = 2.0
- Analysis Procedure Used: Equivalent Force Method
- Base Shear = V = 849.73 kips
- *Please see Appendix E for detailed Seismic Load Calculations and results*

Structural Design and Theory

From my own personal discussions with the structural engineers on this project, the Residences at Christina Landing tower, a 23-story apartment building adjacent to this structure, served as the initial inspiration for the River Tower's design. The Residences at Christina Landing was designed structurally by the Kling engineering firm, and in fact was the subject of Ms. Pamela A. Morris' senior thesis project of 2004-2005. This structure, which is in the process of being completed, made use of two-way precast concrete floor slabs.

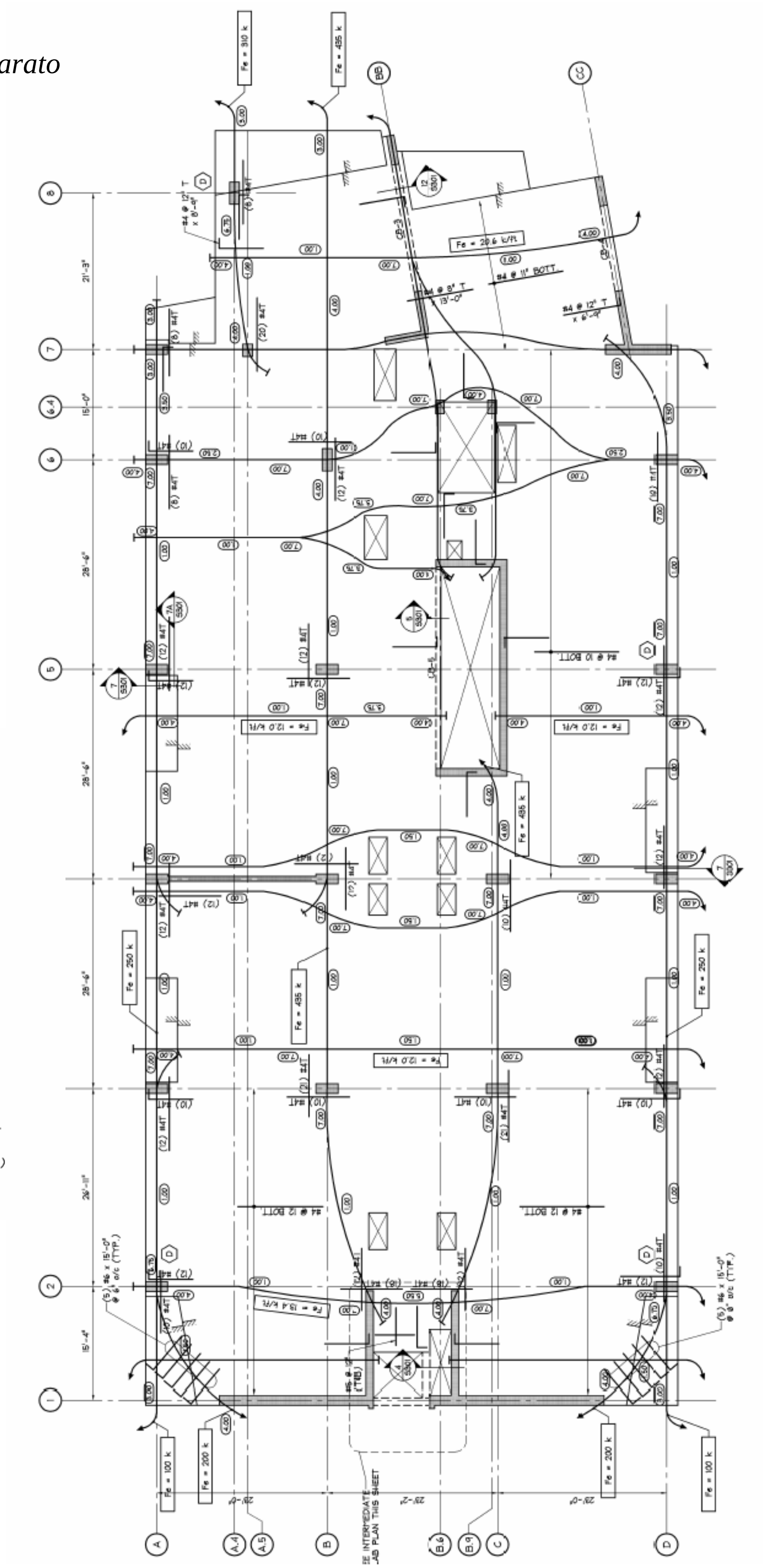
The riverfront location of the River Tower necessitated the use of piles as foundation support, as a spread foundation would not be sufficient in construction so close to the riverbed. The shear walls provide the lateral resistance for the structure, while the flat plate post-tensioned slabs distribute 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.

Preliminary Spot Check and Lateral Analysis

I attempted to perform a spot check analysis on the post-tensioned slab based on my dead and live load calculations. This proved difficult as my knowledge and experience with both post-tensioned and reinforced slabs are severely limited. Because of the post-tensioning in the slab, the Equivalent Frame Method was recommended by ACI. I then tried to check the slab using Ultimate Flexural Strength Analysis, considering a small 12" section of the slab as a beam. However, this over-simplification (and perhaps poor assumptions along the way) has that compressive reinforcement is needed in the interior bay of the tenth floor that I was analyzing, as shown on the following page. Because this does not match the actual design, I reasoned that my assumptions and methods are not applicable in this case. Unfortunately, this particular system has not been part of my curriculum as of yet, so I will further investigate these methods of analysis and their applicability in future reports. Please consult Appendix F for the Spot Check Calculations in greater detail.

The lateral resistance check of the shear walls, also taken from the tenth floor for consistency, appeared to verify the actual design as the reinforcement and deflection of the wall met the minimum criteria of which I was aware. Again, this was a first attempt at such an analysis, and a more proficient method will be used in the future to confirm the legitimacy of the actual designs. My spot check for gravity loading and lateral resistance checks are only preliminary investigations into the analysis, and will be updated in future reports. Please consult Appendix G for the Lateral Resistance Check calculations in greater detail.

Typical Floor Plan:
From Sheet S104
Courtesy of O'Donnell & Naccarato



9th - 21st FLOOR FRAMING PLAN
SCALE: 1/8" = 1'-0"
NOTES:
1) FINISHED FLOOR ELEVATIONS:
9th FLOOR EL. (+102.75')
10th FLOOR EL. (+113.00')
11th FLOOR EL. (+123.25')
12th FLOOR EL. (+133.50')
13th FLOOR EL. (+143.75')
14th FLOOR EL. (+154.00')
15th FLOOR EL. (+164.25')
16th FLOOR EL. (+174.50')
17th FLOOR EL. (+184.75')
18th FLOOR EL. (+195.00')
19th FLOOR EL. (+205.25')
20th FLOOR EL. (+215.50')
21st FLOOR EL. (+225.75')
2) FLOOR SLAB TO BE 8" (5000 psi) CONCRETE POST-TENSIONED SLAB w/ #4 @ 12" IN BANDED P.T. DIRECTION AND #4 @ 20" IN UNIFORM P.T. DIRECTION, CONT. BOTT., UNLESS NOTED OTHERWISE ON PLAN.
3) OPENINGS FOR DUCTS, UTILITIES, PIPES, ETC. TO BE QUANTIFIED AND COORDINATED WITH ARCH. DRAWINGS.
4) "SW1" INDICATES CONCRETE SHEAR WALL. SEE SCHEDULE FOR SIZE & REINF.
5) TENDON PROFILES SHOW ON PLAN, 4.00 ,ARE MEASURED FROM BOTTOM OF SLAB TO THE C.G.S. OF 1/2"φ TENDON.
6) " " INDICATES STUDRAIL SHEAR REINFORCEMENT OF SLAB AT COLUMN. SEE SHEET S300 FOR STUDRAIL DETAILS AND STUDRAIL LAYOUT FOR EACH TYPE.
7) PROVIDE #5 x 6'-0" @ 12" o/c AT ELEVATOR AND STAIR OPENINGS (U.N.O.)
8) ALL TENDON DRAPES ARE FULL DRAPES (U.N.O.)

APPENDICES

A. Design Live Load Calculations

B. Design Dead Load Calculations

C. Snow Load Calculations

D. Wind Load Calculations

E. Seismic Load Calculations

F. Spot Check Calculations

G. Preliminary Lateral Load Analysis Calculations

APPENDIX A

Design Live Load Calculations

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

Roof Live Load: 20 psf
Public Spaces: (Corridors, public rooms): 100 psf
Typical Tower Floor: (Residential floors): 40 psf
Typical Parking Garage Floor: 50 psf
Mechanical Load: 150 psf

Floor/ Level	Residential Spaces		Parking Garage/ Other		Total Floor Square Footage
	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*
Roof	0	0.00	12186	100.00	12186**

Assumptions:
- For simplicity, mechanical loads were judged to apply to half of the spaces on the mechanical/penthouse level 23, and mechanical loads were applied to the full area of level 24.
- Public spaces were assumed to be 15% of residential spaces on typical floors, with the exception of the eighth floor.
- For simplicity, the eighth floor's "other" spaces were assumed to be the rooftop terrace, and was assigned the public space loading of 100 psf to be conservative.
- The roof load was applied to the typical tower floor area for simplicity.
- Live load reductions for live loads of 100 psf or less are not permitted for public garages or for roofs, per BOCA 1996 1606.7.2.2. The reduction would be permitted for the eighth through 22nd floors in this case, but was not considered for simplicity.

*The 25th floor is an extension of the Elevator Machine Room on the 24th floor and was assumed to have the same total area. The elevator room machine load given in the construction documents was used in this live load calculation.

**The largest tower floor area was used in the roof live load calculation for simplicity and to be conservative.

Live Load Calculations:

First Floor:
LL = (29449 total sf)*[(0.15*0.377*(100 psf public)) + (0.85*0.377*(40 psf residential)) +
+ 0.623*(50 psf parking)] = 1461.35 kips
LL = (0.15*0.377*100 psf) + (0.85*0.377*40psf) + (0.623*50 psf) = 49.62 psf

Second Floor:
LL = (29663 total sf)*[(0.15*0.331*(100 psf public)) + (0.85*0.331*(40 psf residential)) +
+ 0.669*(50 psf parking)] = 1486.68 kips
LL = 49.70 psf

Third through Sixth Floors:
LL = (30486 total sf)*[(0.15*0.322*(100 psf public)) + (0.85*0.322*(40 psf residential)) +
+ 0.678*(50 psf parking)] = 1514.48 kips per floor
LL = 49.68 psf

Seventh Floor:
LL = (39560 total sf)*[(0.15*0.248*(100 psf public)) + (0.85*0.248*(40 psf residential)) +
+ 0.752*(50 psf parking)] = 1968.19 kips
LL = 49.75 psf

Eighth Floor:
LL = (31610 total sf)*[(0.15*0.628*(100 psf public)) + (0.85*0.628*(40 psf residential)) +
+ 0.372*(100 psf public/terrace)] = 2148.59 kips
LL = 67.97 psf

Ninth through 22nd Floor:
LL = (12186 total sf)*[(0.15*(100 psf public)) + 0.85*(40 psf residential)] = 597.11 kip per floor
LL = 49 psf

23rd Floor:
LL = (9307 total sf)*[(0.15*0.5*(100 psf public)) + (0.85*0.5*(40 psf residential)) +
+ 0.5*(150 psf mechanical)] = 926.05 kips
LL = 99.5 psf

24th and 25th Floor:
LL = (1070 total sf)*(150 psf mechanical/elevator room) = 160.5 kips
LL = 150 psf

Roof Load:
LL = (12186 total sf)*(30 psf roof) = 365.58 kips
LL = 30 psf

APPENDIX B

Design Dead Load Calculations

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

Column Self-Weight Calculations

Ground Level					
Width (in)	Depth (in)	Area (ft^2)	Height (ft)	Weight (pcf)	Self Wt. (kips)
16.00	36.00	4.00	10.25	147.00	6.03
16.00	52.00	5.78	10.25	147.00	8.71
14.00	24.00	2.33	10.25	147.00	3.52
24.00	28.00	4.67	10.25	147.00	7.03
16.00	36.00	4.00	10.25	147.00	6.03
16.00	36.00	4.00	10.25	147.00	6.03
16.00	20.00	2.22	10.25	147.00	3.35
16.00	36.00	4.00	10.25	147.00	6.03
16.00	36.00	4.00	10.25	147.00	6.03
16.00	36.00	4.00	10.25	147.00	6.03
				Σ (k) =	58.76

Level 2					
Width (in)	Depth (in)	Area (ft^2)	Height (ft)	Weight (pcf)	Self Wt. (kips)
16.00	36.00	4.00	10.25	147.00	6.03
16.00	52.00	5.78	10.25	147.00	8.71
14.00	24.00	2.33	10.25	147.00	3.52
24.00	28.00	4.67	10.25	147.00	7.03
16.00	36.00	4.00	10.25	147.00	6.03
16.00	36.00	4.00	10.25	147.00	6.03
16.00	20.00	2.22	10.25	147.00	3.35
16.00	36.00	4.00	10.25	147.00	6.03
16.00	36.00	4.00	10.25	147.00	6.03
16.00	36.00	4.00	10.25	147.00	6.03
14.00	22.00	2.14	10.25	147.00	3.22
				Σ (k) =	61.99

Levels 3 to 7					
Width (in)	Depth (in)	Area (ft^2)	Height (ft)	Weight (pcf)	Self Wt. (kips)
16.00	36.00	4.00	10.25	147.00	6.03
16.00	52.00	5.78	10.25	147.00	8.71
14.00	24.00	2.33	10.25	147.00	3.52
24.00	28.00	4.67	10.25	147.00	7.03
16.00	36.00	4.00	10.25	147.00	6.03
16.00	36.00	4.00	10.25	147.00	6.03
16.00	20.00	2.22	10.25	147.00	3.35
16.00	36.00	4.00	10.25	147.00	6.03
16.00	36.00	4.00	10.25	147.00	6.03
14.00	22.00	2.14	10.25	147.00	3.22
				Σ (k) =	55.96

Level 8					
Width (in)	Depth (in)	Area (ft^2)	Height (ft)	Weight (pcf)	Self Wt. (kips)
16.00	36.00	4.00	10.25	147.00	6.03
16.00	36.00	4.00	10.25	147.00	6.03
24.00	28.00	4.67	10.25	147.00	7.03
16.00	36.00	4.00	10.25	147.00	6.03
16.00	20.00	2.22	10.25	147.00	3.35
16.00	36.00	4.00	10.25	147.00	6.03
16.00	36.00	4.00	10.25	147.00	6.03
12.00	48.00	4.00	10.25	147.00	6.03
12.00	48.00	4.00	10.25	147.00	6.03
14.00	22.00	2.14	10.25	147.00	3.22
14.00	22.00	2.14	10.25	147.00	3.22
				Σ (k) =	59.01

Level 9 to 23					
Width (in)	Depth (in)	Area (ft^2)	Height (ft)	Weight (pcf)	Self Wt. (kips)
16.00	36.00	4.00	10.25	147.00	6.03
16.00	36.00	4.00	10.25	147.00	6.03
16.00	36.00	4.00	10.25	147.00	6.03
16.00	20.00	2.22	10.25	147.00	3.35
16.00	36.00	4.00	10.25	147.00	6.03
12.00	48.00	4.00	10.25	147.00	6.03
12.00	48.00	4.00	10.25	147.00	6.03
14.00	22.00	2.14	10.25	147.00	3.22
				Σ (k) =	42.73

Levels 24 and 25					
Width (in)	Depth (in)	Area (ft^2)	Height (ft)	Weight (pcf)	Self Wt. (kips)
16.00	20.00	2.22	10.0	147.00	3.27
12.00	16.00	1.33	10.0	147.00	1.96
12.00	48.00	4.00	10.0	147.00	5.88
12.00	48.00	4.00	10.0	147.00	5.88
14.00	22.00	2.14	10.0	147.00	3.14
				Σ (k) =	20.13

Shear Wall Self-Weight Calculations

Foundation to Eighth Floor					
Thick (in)	Length (ft)	Area (ft^2)	Height (ft)	Weight (pcf)	Self Wt. (kips)
16.00	44.00	58.67	10.25	147.00	88.40
12.00	28.00	28.00	10.25	147.00	42.19
9.00	19.00	14.25	10.25	147.00	21.47
12.00	18.00	18.00	10.25	147.00	27.12
12.00	30.00	30.00	10.25	147.00	45.20
12.00	12.00	12.00	10.25	147.00	18.08
12.00	9.00	9.00	10.25	147.00	13.56
12.00	18.00	18.00	10.25	147.00	27.12
12.00	42.00	42.00	10.25	147.00	63.28
24.00	15.00	30.00	10.25	147.00	45.20
12.00	25.00	25.00	10.25	147.00	37.67
24	18	36.00	10.25	147.00	54.24
				Σ (k) =	483.54

9th to 22nd Floors					
Thick (in)	Length (ft)	Area (ft^2)	Height (ft)	Weight (pcf)	Self Wt. (kips)
16.00	44.00	58.67	10.25	147.00	88.40
12.00	28.00	28.00	10.25	147.00	42.19
9.00	19.00	14.25	10.25	147.00	21.47
12.00	18.00	18.00	10.25	147.00	27.12
12.00	30.00	30.00	10.25	147.00	45.20
12.00	12.00	12.00	10.25	147.00	18.08
12.00	9.00	9.00	10.25	147.00	13.56
12.00	18.00	18.00	10.25	147.00	27.12
12.00	42.00	42.00	10.25	147.00	63.28
12.00	25.00	25.00	10.25	147.00	37.67
				Σ (k) =	384.10

23rd Level to Roof					
Thick (in)	Length (ft)	Area (ft^2)	Height (ft)	Weight (pcf)	Column Wt. (kips)
16.00	44.00	58.67	10.00	147.00	86.24
12.00	28.00	28.00	10.00	147.00	41.16
9.00	19.00	14.25	10.00	147.00	20.95
12.00	18.00	18.00	10.00	147.00	26.46
12.00	30.00	30.00	10.00	147.00	44.10
12.00	12.00	12.00	10.00	147.00	17.64
12.00	9.00	9.00	10.00	147.00	13.23
12.00	18.00	18.00	10.00	147.00	26.46
12.00	42.00	42.00	10.00	147.00	61.74
12.00	25.00	25.00	10.00	147.00	36.75
				Σ (k) =	374.73

Dead Load Calculations per Floor

First Floor:
W_{slab} = (8” slab)*(147 pcf)*(1ft/12”)*(29449 sf)*(1 kip/1000 lb) = 2886.0 kips
$W_{columns}$ = 58.76 kips
$W_{shear\ wall}$ = 483.54 kips
W_{first} = 3428.30 kips = 0.116 ksf

Second Floor:
W_{slab} = (8” slab)*(147 pcf)*(1ft/12”)*(29663 sf)*(1 kip/1000 lb) = 2906.97 kips
$W_{columns}$ = 61.99 kips
$W_{shear\ wall}$ = 483.54 kips
W_{second} = 3452.50 kips = 0.1164 ksf

Third through Sixth Floors (values per floor):
W_{slab} = (8” slab)*(147 pcf)*(1ft/12”)*(30486 sf)*(1 kip/1000 lb) = 2987.63 kips
$W_{columns}$ = 55.96 kips
$W_{shear\ wall}$ = 483.54 kips
$W_{3\text{-}6th}$ = 3527.13 kips per floor = 0.1157 ksf per floor

Seventh Floor:
W_{slab} = (8” slab)*(147 pcf)*(1ft/12”)*(39560 sf)*(1 kip/1000 lb) = 3876.88 kips
$W_{columns}$ = 55.96 kips
$W_{shear\ wall}$ = 483.54 kips
$W_{seventh}$ = 4416.38 kips per floor = 0.112 ksf

Eighth Floor:
W_{slab} = (8" slab)*(147 pcf)*(1ft/12")*(31610 sf)*(1 kip/1000 lb) = 3097.78 kips
$W_{columns}$ = 59.01 kips
$W_{shear\ wall}$ = 483.54 kips
W_{eighth} = 3640.33 kips = 0.115 ksf

Ninth through 22nd Floors (values per floor):
W_{slab} = (8" slab)*(147 pcf)*(1ft/12")*(12186 sf)*(1 kip/1000 lb) = 1194.23 kips
$W_{columns}$ = 42.73 kips
$W_{shear\ wall}$ = 384.10 kips
$W_{9\text{-}22nd}$ = 1621.06 kips per floor = 0.133 ksf per floor

23rd Floor:
W_{slab} = (8" slab)*(147 pcf)*(1ft/12")*(9307 sf)*(1 kip/1000 lb) = 912.09 kips
$W_{columns}$ = 42.73 kips
$W_{shear\ wall}$ = 374.73 kips
W_{23rd} = 1329.55 kips = 0.143 ksf

24th to 25th Floor:
W_{slab} = (8" slab)*(147 pcf)*(1ft/12")*(1070 sf)*(1 kip/1000 lb) = 104.86 kips
$W_{columns}$ = 20.13 kips
$W_{shear\ wall}$ = 374.73 kips
$W_{24\text{-}25th}$ = 499.72 kips = 0.467 ksf

Roof:
Because the roof consists of a small amount of mostly steel framing, which is relatively light compared to the mostly concrete construction of the other floors, this floor self-weight was estimated.
W_{roof} = $(374.73\ \text{kips})_{shear\ walls}$ + $(25\ \text{kips})_{steel,\ etc.}$ ≈ 400 kips = 0.373 ksf

APPENDIX C

Snow Load Calculations

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

For flat and low-sloped roofs (BOCA 1996 1608.4):

$P_f = (C_e)(I)(P_g)$

$C_e = 0.7$ (Table 1608.4 - All other structures)
$P_g = 20$ psf (Table 1608.3(1) - Wilmington, DE)
$I = 1.0$ (Table 1609.5 - All other structures)

Roof Snow Load: $P_f = (0.7)(20\text{ psf})(1.0) =$ **14.0 psf**

APPENDIX D

Wind Load Calculations

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

Assumptions:
- Because of the stabilizing nature of the parking garage and for the simplicity of these preliminary calculations, I considered a worst case building length/width ratio which only took the tower dimensions into account.

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

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

For North-South Direction:

Level	Elev. (ft)	K coeff	G coeff.	P (windward)	P (leeward)	Total P (psf)
Roof	279.22	1.34	1.17	14.78	-6.72	8.06
25	269.22	1.32	1.18	14.63	-6.65	7.98
24	259.39	1.30	1.18	14.46	-6.57	7.89
23	247.36	1.27	1.19	14.23	-6.47	7.76
22	236.00	1.25	1.20	13.98	-6.35	7.63
21	225.75	1.22	1.20	13.76	-6.25	7.50
20	215.50	1.20	1.20	13.52	-6.15	7.37
19	205.25	1.17	1.21	13.29	-6.04	7.25
18	195.00	1.15	1.22	13.08	-5.95	7.14
17	184.75	1.12	1.23	12.89	-5.86	7.03
16	174.50	1.09	1.23	12.65	-5.75	6.90
15	164.25	1.06	1.24	12.33	-5.61	6.73
14	154.00	1.03	1.25	12.06	-5.48	6.58
13	143.75	1.00	1.26	11.79	-5.36	6.43
12	133.50	0.97	1.27	11.53	-5.24	6.29
11	123.25	0.94	1.28	11.26	-5.12	6.14
10	113.00	0.91	1.29	10.97	-4.99	5.98
9	102.75	0.87	1.31	10.66	-4.84	5.81
8	92.50	0.83	1.32	10.26	-4.66	5.60
7	82.25	0.78	1.34	9.79	-4.45	5.34
6	72.00	0.74	1.36	9.39	-4.27	5.12
5	61.75	0.69	1.39	8.95	-4.07	4.88
4	51.50	0.64	1.42	8.47	-3.85	4.62
3	41.25	0.58	1.46	7.88	-3.58	4.30
2	31.00	0.51	1.51	7.16	-3.25	3.90
1	10.50	0.37	1.65	5.73	-2.60	3.12

For West-East Direction:

Level	Trib. Width (ft)	Trib. Height (ft)	P (plf)	F (kips)
Roof	164	10.00	1321.81	13.22
25	164	9.83	1308.81	12.87
24	164	12.03	1293.61	15.56
23	164	11.36	1273.28	14.46
22	164	10.25	1250.52	12.82
21	164	10.25	1230.54	12.61
20	164	10.25	1209.38	12.40
19	164	10.25	1188.66	12.18
18	164	10.25	1170.47	12.00
17	164	10.25	1153.38	11.82
16	164	10.25	1131.94	11.60
15	164	10.25	1103.28	11.31
14	164	10.25	1079.05	11.06
13	164	10.25	1055.03	10.81
12	164	10.25	1031.84	10.58
11	164	10.25	1007.04	10.32
10	164	10.25	981.51	10.06
9	164	10.25	953.46	9.77
8	164	10.25	917.99	9.41
7	164	10.25	875.87	8.98
6	164	10.25	839.77	8.61
5	164	10.25	800.78	8.21
4	164	10.25	757.51	7.76
3	164	10.25	705.11	7.23
2	164	20.50	640.30	13.13
1	164	10.50	512.30	5.38
			Sum of F:	284.16

For West-East Direction:

Level	Elev. (ft)	K coeff	G coeff.	P (windward)	P (leeward)	Total P (psf)
Roof	279.22	1.34	1.17	14.78	-1.34	13.43
25	269.22	1.32	1.18	14.63	-1.33	13.30
24	259.39	1.30	1.18	14.46	-1.31	13.15
23	247.36	1.27	1.19	14.23	-1.29	12.94
22	236.00	1.25	1.20	13.98	-1.27	12.71
21	225.75	1.22	1.20	13.76	-1.25	12.51
20	215.50	1.20	1.20	13.52	-1.23	12.29
19	205.25	1.17	1.21	13.29	-1.21	12.08
18	195.00	1.15	1.22	13.08	-1.19	11.90
17	184.75	1.12	1.23	12.89	-1.17	11.72
16	174.50	1.09	1.23	12.65	-1.15	11.50
15	164.25	1.06	1.24	12.33	-1.12	11.21
14	154.00	1.03	1.25	12.06	-1.10	10.97
13	143.75	1.00	1.26	11.79	-1.07	10.72
12	133.50	0.97	1.27	11.53	-1.05	10.49
11	123.25	0.94	1.28	11.26	-1.02	10.23
10	113.00	0.91	1.29	10.97	-1.00	9.97
9	102.75	0.87	1.31	10.66	-0.97	9.69
8	92.50	0.83	1.32	10.26	-0.93	9.33
7	82.25	0.78	1.34	9.79	-0.89	8.90
6	72.00	0.74	1.36	9.39	-0.85	8.53
5	61.75	0.69	1.39	8.95	-0.81	8.14
4	51.50	0.64	1.42	8.47	-0.77	7.70
3	41.25	0.58	1.46	7.88	-0.72	7.17
2	31.00	0.51	1.51	7.16	-0.65	6.51
1	10.50	0.37	1.65	5.73	-0.52	5.21

For West-East Direction:

Level	Trib. Width	Trib. Height	P (plf)	F (kips)
Roof	164	10.00	2203.02	22.03
25	164	9.83	2181.34	21.44
24	164	12.03	2156.02	25.94
23	164	11.36	2122.13	24.11
22	164	10.25	2084.20	21.36
21	164	10.25	2050.90	21.02
20	164	10.25	2015.63	20.66
19	164	10.25	1981.11	20.31
18	164	10.25	1950.78	20.00
17	164	10.25	1922.29	19.70
16	164	10.25	1886.56	19.34
15	164	10.25	1838.81	18.85
14	164	10.25	1798.41	18.43
13	164	10.25	1758.39	18.02
12	164	10.25	1719.74	17.63
11	164	10.25	1678.40	17.20
10	164	10.25	1635.86	16.77
9	164	10.25	1589.11	16.29
8	164	10.25	1529.98	15.68
7	164	10.25	1459.78	14.96
6	164	10.25	1399.61	14.35
5	164	10.25	1334.63	13.68
4	164	10.25	1262.51	12.94
3	164	10.25	1175.18	12.05
2	164	20.50	1067.17	21.88
1	164	10.50	853.84	8.97
			Sum of F:	473.60

APPENDIX E

Seismic Load Calculations

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: Dual-system with intermediate moment frame of reinforced concrete with reinforced concrete shear walls (Table 1610.3.3 – No height limitations)
- Response Modification Factor (R): 8.0
- Deflection Amplification Factor (Cd): 6.5

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 $(2.5A_a)/(R) = (2.5)(0.05)/(8.0) = 0.0156$

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 System with shear walls)
$h_n = 279.22$ 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

$C_s = [(1.2)(0.075)(2.0)]/[(8.0)(2.322)]^{(2/3)} = 0.0257 > 0.0156 \rightarrow$ Use $C_s = 0.0156$

$V = (0.0156)(54469.86 \text{ kips}) =$ **849.73 kips**

Vertical Distribution of Seismic Forces: $F_x = (C_{vx})(V)$ (Section 1610.4.2)
$C_{vx} = (w_x h^k_x) / (\Sigma\, w_i h^k_i)$

(k determined through linear interpolation: 1.911)

Level	w_x (k)	h_x (ft)	$w_x h_x^k$	C_{vx}	F_x (k)
Roof	375	279.22	17611207	0.028729	24.41
25	499.72	269.22	21889292	0.035707	30.34
24	499.72	259.39	20388130	0.033259	28.26
23	1329.55	247.36	49540908	0.080815	68.67
22	1621.06	236.00	55215501	0.090072	76.54
21	1621.06	225.75	50725707	0.082747	70.31
20	1621.06	215.50	46417675	0.07572	64.34
19	1621.06	205.25	42292167	0.06899	58.62
18	1621.06	195.00	38349985	0.062559	53.16
17	1621.06	184.75	34591977	0.056429	47.95
16	1621.06	174.50	31019041	0.050601	43.00
15	1621.06	164.25	27632130	0.045076	38.30
14	1621.06	154.00	24432261	0.039856	33.87
13	1621.06	143.75	21420524	0.034943	29.69
12	1621.06	133.50	18598088	0.030339	25.78
11	1621.06	123.25	15966220	0.026045	22.13
10	1621.06	113.00	13526297	0.022065	18.75
9	1621.06	102.75	11279826	0.0184	15.64
8	3097.78	92.50	17635236	0.028768	24.44
7	4416.38	82.25	20089819	0.032772	27.85
6	3427.13	72.00	12090260	0.019723	16.76
5	3427.13	61.75	9016690.8	0.014709	12.50
4	3427.13	51.50	6375032.3	0.010399	8.84
3	3427.13	41.25	4172442.2	0.006806	5.78
2	3452.5	31.00	2435760.2	0.003973	3.38
1	3428.3	10.50	305879.51	0.000499	0.42
		$\Sigma\, w_i h^k_i$ =	613018057	$\Sigma\, F_x$ (k) =	849.73

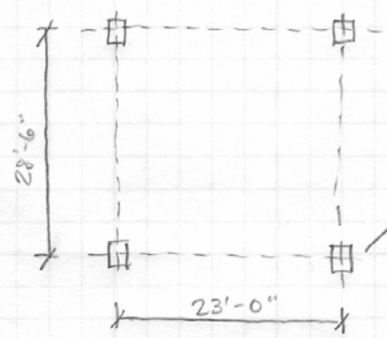
Allowable Story Drift: (Table 1610.3.8 – Exposure Group II, "All other buildings")

Story Drift: $\Delta_a = 0.015(h_{sx})$			
Level	**h_{sx} (ft)**	**Δ_a (ft)**	**Δ_a (in)**
Roof	279.22	4.19	50.26
25	269.22	4.04	48.46
24	259.39	3.89	46.69
23	247.36	3.71	44.52
22	236.00	3.54	42.48
21	225.75	3.39	40.64
20	215.50	3.23	38.79
19	205.25	3.08	36.95
18	195.00	2.93	35.10
17	184.75	2.77	33.26
16	174.50	2.62	31.41
15	164.25	2.46	29.57
14	154.00	2.31	27.72
13	143.75	2.16	25.88
12	133.50	2.00	24.03
11	123.25	1.85	22.19
10	113.00	1.70	20.34
9	102.75	1.54	18.50
8	92.50	1.39	16.65
7	82.25	1.23	14.81
6	72.00	1.08	12.96
5	61.75	0.93	11.12
4	51.50	0.77	9.27
3	41.25	0.62	7.43
2	31.00	0.47	5.58
1	10.50	0.16	1.89

APPENDIX F

Spot Check Calculations

Typical Interior Span



Because of prestressing, Equivalent Frame Method of analysis is necessary (ACI 18.12)

Typical Column → 14" x 36" (Assume all similar)

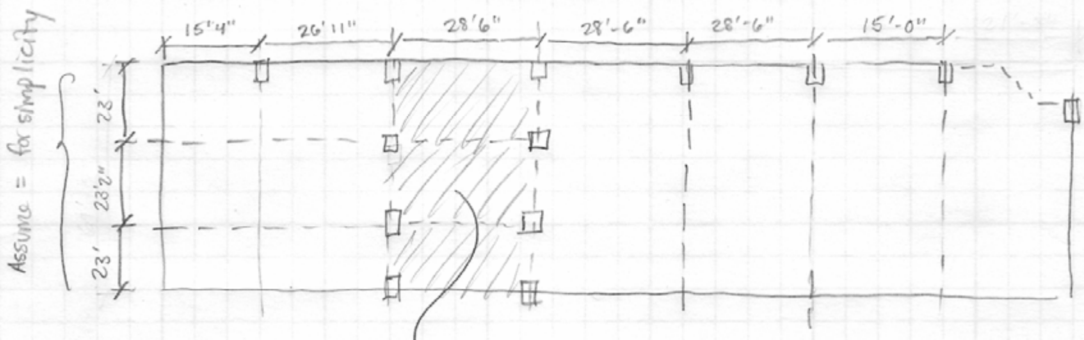
$f_y = 60$ ksi

$f'_c = 5000$ psi

8" slab

$\ell_1 = 23'-0"$

$\ell_2 = 28'-6"$



Bay to be checked (interior panel)

Shortspan

Minimum thickness (ACI 9.5.3)

Interior Panel without drop panels → $t = \frac{\ell_n}{33}$

$$t = \frac{(28.5' - 36"/12^{in}/ft)}{33} = 0.77' = 9.27" \rightarrow \text{Use } 9.5" \text{ slab}$$

DL/

Assume SDL = 25 psf (partitions / finishes)

Total DL = $25\text{ psf} + (147\text{ pcf})(9.5")\left(\frac{1\text{ ft}}{12"}\right) = 116.38$ psf

$w_u = 1.2DL + 0.5LL$

Assume LL = 70 psf (worst case tower floor)

Fully factored $w_u = 1.2(116.38\text{ psf}) + 1.6(70\text{ psf}) = 251.66$ psf (ACI 8.9)

Use moment distribution for moment calcs

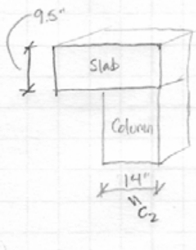
$$K_{slab} = \frac{4E_cI_c}{\ell}$$

$$I_c = \ell_2(t^3/12) = (28.5')\left(\frac{12"}{1ft}\right)\frac{(9.5")^3}{12} = 24435.19\text{ in}^4$$

$$K_{slab} = \frac{4E_c(24435.19\text{ in}^4)}{23' \times 12^{in}/ft} = \boxed{354.13E_c}$$

$$K_{\substack{column\\exterior}} = \frac{4E_c\,(36'' \times (14'')^3)(\frac{1}{12})}{(10.25')(12^{in}/_{ft})} = 267.71E_c$$

$$K_{torsion} = \Sigma\ \frac{9E_{cs}C}{\ell_2\,(1 - c_2/\ell_2)^3}$$

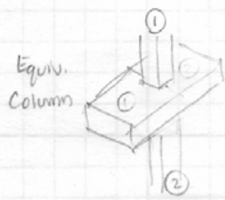


$$C = \Sigma\ \tfrac{1}{3}x^3 y\,(1 - 0.63\,(x/y))$$

$$C = (1 - 0.63 \times \tfrac{9.5''}{14''})\,(9.5'')^3\,(\tfrac{14''}{3}) = 2290.62\,in^4$$

Torsional Stiffness

$$K_t = \frac{9E_c\,(2290.62\,in^4)}{(28.5')(12^{in}/_{ft})\,(1 - 3'/28.5')^3} = 84.16E_c$$

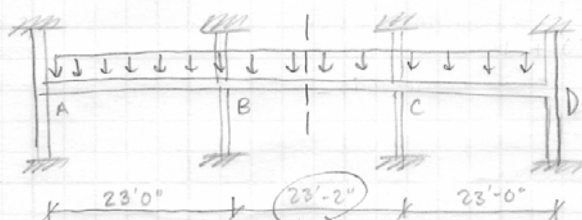


$$\frac{1}{K_{ec}} = \frac{1}{2(267.71E_c)} + \frac{1}{2(84.16E_c)} \longrightarrow K_{ec} = \boxed{128.06E_c}$$

By ACI 13.7.6.2 $\longrightarrow LL/DL = \frac{70psf}{(98psf + 25psf)} = 0.57 < 0.75$

· Use Full Factored Load on entire slab

$W_u = 251.66\,psf\,(28.5') = 7.17\,klf$



For simplicity, take this as 23'

$$FEM_{slab\,AB} = \frac{W_u\ell^2}{12} = \frac{(7.17\,klf)(23')^2}{12} = 316.08\ kip\cdot ft = FEM_{CD}$$

$$FEM_{slab\,BA} = -316.08\ kip\cdot ft = FEM_{DC}$$

- Use Symmetry to simplify Moment Distribution

- Assume interior bay is 23'-0"

$FEM_{23'-2''} = 320.68\ kip\cdot ft$ VS $316.08\ kip\cdot ft$ ~ negligible

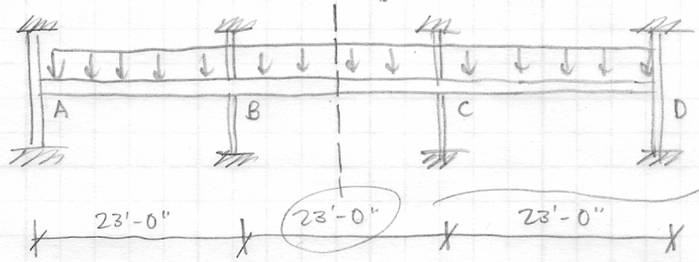
Distribution Factors

Exterior $DF_{slab} = \frac{K_{ij}}{\Sigma K_{ij}} = \frac{354.13}{128.06 + 354.13} = 0.734$

$DF_{column} = \frac{1 - 0.734}{2} = 0.133$

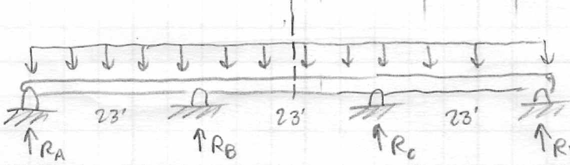
Interior $DF_{slab} = \frac{354.13}{128.06 + 1.5(354.43)} = 0.536$

due to symmetry



Assume Equal Spans for Simplicity (23'-2" really)

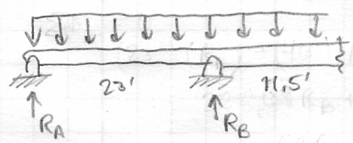
	Col Above	Col Below	Slab	Slab	Col Above	Col Below	Slab
DF	0.133	0.133	0.734	0.536	0.098	0.098	0.268
FEM (kip-ft)	—	—	-316.08	+316.08	—	—	-316.08
D	+42.04	+42.04	+232.0				
C.O.				+116			
D				-62.18	-11.37	-11.37	-31.09
C.O.			-31.09				
D	+4.13	+4.13	+22.82				
C.O.				+11.41			
D				-6.12	-1.12	-1.12	-3.06
C.O.			-3.06				
D	+0.41	+0.41	+2.25				
C.O.				+1.125			
				-0.603	-0.110	-0.110	-0.302
M final	+46.58	+46.58	+93.16	+375.71	-12.68	-12.68	-350.53



$$R_A + R_B + R_C + R_D = (7.17\,\text{klf})(23')(3)$$

$$R_A = R_D \text{ and } R_B = R_C \text{ (symmetry)}$$

$$2R_A + 2R_B = 494.73\text{ kips} \rightarrow R_A + R_B = 248.87\text{ kips}$$

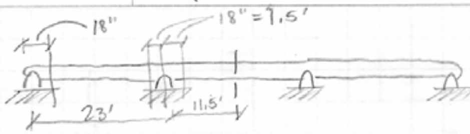


$$\Sigma M_A = (7.17\,\text{klf})(23' + 11.5')\left(\frac{23+11.5}{2}\right) = 23 R_B$$

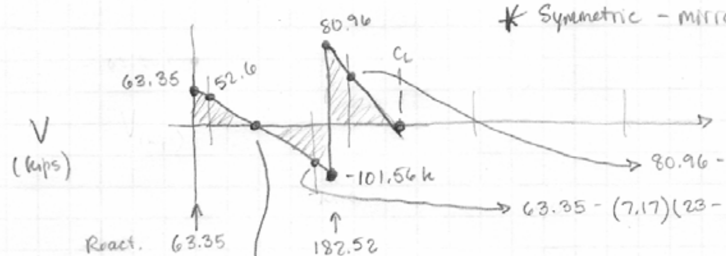
$$R_C = R_B = 185.52\text{ kips}$$

$$R_D = R_A = 63.35\text{ kips}$$

Spot Check | Floor 10 (typical floor)



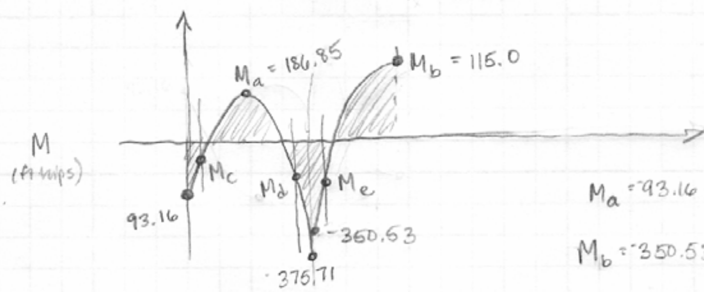
✱ Symmetric - mirrored about C_L



$80.96 - (7.17)(1.5) = 70.21$

$63.35 - (7.17)(23 - 1.5) = -90.81$

$63.35 - 7.17x = 0 \longrightarrow x = 8.84'$



$M_a = -93.16 + (\frac{1}{2})(63.35)(8.84) = \underline{186.85 \text{ ft}\cdot\text{kip}}$

$M_b = -350.53 + \frac{1}{2}(80.96)(11.5') = \underline{115 \text{ ft}\cdot\text{kip}}$

$M_c = -93.16 + \frac{1}{2}(63.35 - 52.6)(1.5) +$
$+ (52.6)(1.5) = \underline{-6.2 \text{ ft}\cdot\text{kip}}$

$M_d = 186.85 + \frac{1}{2}(-90.81)(11.5' - 1.5') =$
$= \underline{-267.2 \text{ ft}\cdot\text{kip}}$

$M_e = 115 - \frac{1}{2}(11.5 - 1.5)(70.21) =$
$\hookrightarrow = \underline{-236.05 \text{ ft}\cdot\text{kip}}$

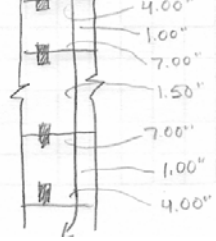
Summary

Exterior Span
- At face of left support → M_u = -6.2 ft·kip
- Near midspan → M_u = 186.85 ft·kip
- At face of right support → M_u = -267.2 ft·kip

Interior Span
- At face of left support → M_u = -236.05 ft·kip
- At midspan → M_u = 115 ft kips

From sheet S104:

F_e = 12 kip/ft



← Tendon Profiles

f'_c = 5000 psi

f'_{ci} = 3000 psi minimum (from specs)

Check midspan only for simplicity (interior span)

Tendons $\begin{cases} f_{pu} = 270 \text{ ksi} & \text{(from specs)} \\ \text{Unbonded tendons} & \text{(from specs)} \end{cases}$

Design as 12" wide beam for simplicity

Total load at interior

$M_{max} = 236.05 \text{ ft·kip} = 2832.6 \text{ in·kip}$

$M_{min} = 115 \text{ ft·kips} = 1380 \text{ in·kip}$

~~Assume $\Omega = 0.8$~~

$\bar{\sigma}_{cs} = 0.45 f'_c = 0.45(5000 \text{ psi}) = \underline{2250 \text{ psi}}$

$\bar{\sigma}_{ci} = 0.6 f'_{ci} = 0.6(3000 \text{ psi}) = \underline{1800 \text{ psi}}$

$\bar{\sigma}_{ti} = -6\sqrt{f'_{ci}} = -6\sqrt{3000 \text{ psi}} = \underline{-328.63 \text{ psi}}$

$\bar{\sigma}_{ts} = -7.5\sqrt{f'_c} = -7.5\sqrt{5000 \text{ psi}} = \underline{-530.33 \text{ psi}}$

$$Z_t \geq \frac{M_{max} - (M_{min} \times \Omega)}{\bar{\sigma}_{cs} - \Omega\bar{\sigma}_{ti}} = \frac{[(236.05 \text{ ft·kip}) - (115 \text{ ft·kip})(0.841)]\left(\frac{12''}{\text{ft}}\right)\left(\frac{1000 \text{ lb}}{1 \text{ kip}}\right)}{2250 \text{ psi} - (0.841)(-328.63 \text{ psi})}$$

$Z_t \geq 661.82 \text{ in}^3$

$$Z_b \geq \frac{M_{max} - (M_{min})(\Omega)}{\Omega\bar{\sigma}_{ci} - \bar{\sigma}_{ts}} = \frac{[(236.05 \text{ ft·kip}) - (115 \text{ ft·kip})(0.841)]\left(\frac{12000 \text{ in·lb}}{1 \text{ ft·kip}}\right)}{(0.841)(1800 \text{ psi}) - (-530.33 \text{ psi})}$$

$Z_b \geq 817.96 \text{ in}^3$

Determine Ω value

Typical Losses for low-lax post-tensioning $= 30 \text{ ksi} = f_{pi} - f_{pe}$

From ACI 18.5.1 $\rightarrow$ $f_{pi} = 0.7 f_{pu}$ for post-tensioning

$f_{pi} = 0.7(270 \text{ ksi}) = 189 \text{ ksi}$

$\therefore f_{pe} = 189 \text{ ksi} - 30 \text{ ksi} = 159 \text{ ksi}$

Actual $\Omega = \frac{f_{pe}}{f_{pi}} = \frac{159 \text{ ksi}}{189 \text{ ksi}} = 0.841$

$k_b = Z_t / A_c = \frac{661.82 \text{ in}^3}{(9.5'' \times 12'')} = \underline{5.81''}$

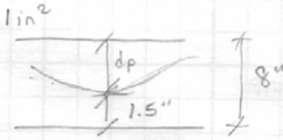
$k_t = -Z_b / A_c = \frac{-817.96 \text{ in}^3}{(9.5'' \times 12'')} = \underline{-7.18''}$

$y_t = h/2 = \frac{9.5''}{2} = 4.75''$

Ultimate Strength Analysis – Flexure → using actual design from S104 sheet

$f_{pe} = 159\text{ ksi} \geq 0.5 f_{pu} = 0.5(270\text{ ksi}) = 135\text{ ksi}$ ✓

Unbonded tendons → $A_{ps} = \frac{(12\text{ kip/ft})(23')}{189\text{ ksi}} = \frac{F}{f_{pi}} = 1.91\text{ in}^2$



From Eq. 18-4, 18-5 in ACI:

$$f_{ps} = f_{pe} + 10 + \frac{f'_c}{k\rho_p} \quad (\text{ksi})$$

Span-to-depth ratio: $L/d_p = \frac{23\text{ ft}(12''/\text{ft})}{(8'' - 1.5'')} = 42.46 \quad \therefore k = 300,\ C = 30$

$$\rho_p = \frac{A_{ps}}{b \cdot d_p} = \frac{1.91\text{ in}^2}{(12'')(8''-1.5'')} = 0.0245$$

$$f_{ps} = (159\text{ ksi}) + 10 + \frac{(5\text{ ksi})}{(300)(0.0245)} = \underline{169.68\text{ ksi}}$$

Does $f_{ps} \leq f_{pe} + C = 159\text{ ksi} + 60$? (✓)

$f_{ps} \leq f_{py} = 0.9 f_{pu} = 0.9(270\text{ ksi}) = 243\text{ ksi}$ (✓)

These equations referenced from Naaman's "Prestressed Concrete Analysis and Design." Chapter 5. (Based off ACI)

$$c = \frac{A_{ps} f_{ps} + A_s f_y - A_s'(f_y')}{0.85 f'_c \beta_1 b}$$

$\beta_1 = 0.80$ for $f'_c = 5000\text{ psi}$

This bay does not appear to have any compressive reinforcement in the slab (reference: S104)

$\therefore A_s = A_s' = 0$

$$c = \frac{(1.91\text{ in}^2)(169.68\text{ ksi})}{0.85(5\text{ ksi})(0.80)(12'')} = 7.94''$$

Rectangular section → $b = b_w$

$$d_e = \frac{A_{ps} f_{ps} d_p + \cancel{A_s f_y d_s}}{A_{ps} f_{ps} + \cancel{A_s f_y}} = d_p = 6.5''$$

$a = \beta_1 c = (0.80)(8.03'') = 6.42''$

$c/d_e = c/d_p = \frac{7.94''}{6.5''} = 1.22 > 0.6 \rightarrow$ Use $\Phi = 0.65$

$c/d_e > 0.4 \rightarrow \therefore$ Apparently compression reinforcement is necessary by my calculations

I am assuming that this overly-simplified process is not suitable to provide a reasonable spot check. I will investigate the proper analysis procedure in future reports.

APPENDIX G

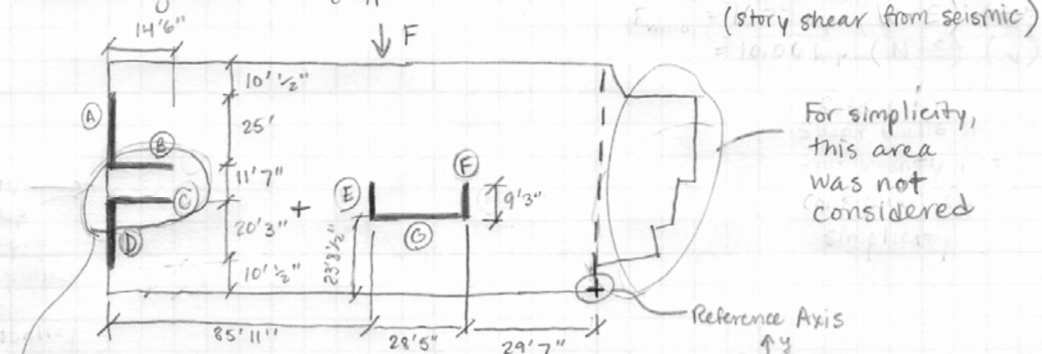
Lateral Load Preliminary Analysis: Shear Wall Check

Despite the large amount of floors (25 stories), I used the distribution of rigidity procedure to analyze a sample shear wall. It was brought to my attention that this method does not account for overturning moments in taller buildings, but since this is only a preliminary check (considering only a single level), this analysis should still prove valuable.

Distribution by Rigidity

Story height = h = 10.25'

Concerning Level 10 (typical floor) → $F_x = 18.75$ kips (story shear from seismic)



these shear wall shapes were simplified from their actual irregular shapes

Stiffnesses

(G) $h/L = \frac{10.25'}{28.5'} = 0.359$

(A) $h/L = 10.25'/25' = 0.41$

All shear walls are "regular"

* For simplicity, each wall was considered as an individual, unconnected entity

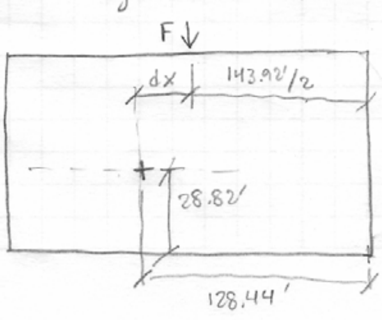
$$k = R = \frac{Et}{4(h/L)^3 + 2.78(h/L)}$$

$$E_c = w^{1.5} \cdot 33\sqrt{f'c} = (147pcf)^{1.5}(33)\sqrt{5000psi} = 4158.87 \text{ ksi}$$

See Excel spreadsheet for k calculations and $\bar{x}$ and $\bar{y}$

$\bar{x} = 128.44'$ (←)

$\bar{y} = 28.82'$ (↑)



$M = F \cdot dx$

$dx = 128.44' - \frac{143.92'}{2} = 56.48'$ (→)

$M = (18.75k)(56.48') = 1059$ ft-kips

Please see spreadsheet for forces on each wall

Through distribution by rigidity method:

	t (in)	h (in)	L (in)	k = R	x_i (ft)	y_i (in)	$k_i x_i$	$k_i y_i$
A	16	123	300	47010.01	143.92	0	6765681	0
B	12	123	174	14773.41	0	41.875	0	618636.7
C	12	123	174	14773.41	0	30.2917	0	447511.8
D	12	123	243	25913.19	143.92	0	3729427	0
E	16	123	111	7807.207	58	0	452818	0
F	12	123	111	5855.405	29.583	0	173220.5	0
G	12	123	341.04	41927.84	0	23.708	0	994025.2
			$\Sigma k_x =$	86585.82				
			$\Sigma k_y =$	71474.67				

$x_{bar} = \Sigma k_i x_i / \Sigma k_i = 128.44$ ft
$y_{bar} = \Sigma k_i y_i / \Sigma k_i = 28.82$ ft

Torsional forces				
	k_i	d_i (ft)	$k_i d_i^2$	$F_{torsion}$
A	47010.01	15.47926	11263956	6.581879
B	14773.41	-13.0512	2516400	-1.74397
C	14773.41	-1.46787	31831.363	-0.19615
D	25913.19	-15.4793	6208998.1	-3.62811
E	7807.207	-70.4407	38738560	-4.97426
F	5855.405	-98.8577	57224009	-5.23573
G	41927.84	5.115831	1097324.1	1.940117
		$\Sigma k_i d_i^2 =$	117081079	

Direct forces (k)	
F_A	-10.1799
F_B	0
F_C	0
F_D	-5.61145
F_E	-1.69064
F_F	-1.26798
F_G	0

Total Forces (k)	
A	-3.59805
B	-1.74397
C	-0.19615
D	-9.23956
E	-6.6649
F	-6.5037
G	1.940117

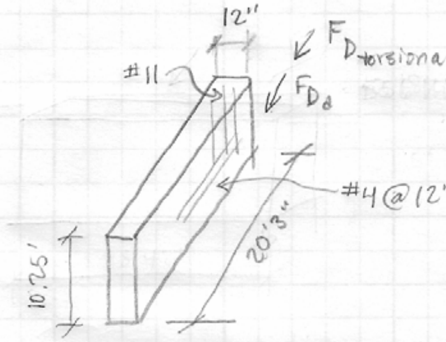
Considering Shear Wall "D"

$t = 12''$
$L = 20'3''$

$F_{D\,total} = -9.24$ kip

$F_{D\,direct} = -5.611$ kip

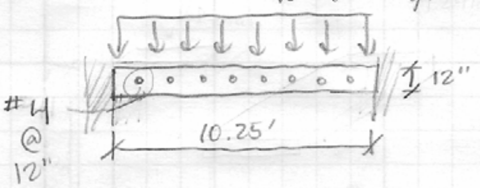
$F_{D\,torsion} = -3.628$ kip

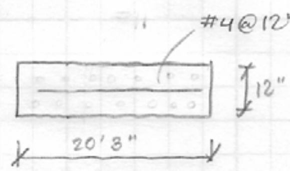


True Design

Shear Wall D has Vertical Reinf. of (22) #11
and Horizontal Reinf. of #4@12" spacing

Treat as really deep beam with distributed load
$w = (9.24\,k)(10.25') = 0.90$ kip·ft





$$M_u = \frac{w\ell^2}{8} = \frac{(0.90\text{ kip·ft})(10.25')^2}{8} = 11.84\text{ kip·ft}$$

$$\text{Req'd } M_n = \frac{M_u}{\Phi} = \frac{11.84\text{ ft·kip}}{0.9} = 13.15\text{ kip·ft}$$

$$\text{Req'd } A_s = \frac{\text{Req'd } M_n}{f_y(d - a/2)} \approx \frac{(13.15\text{ kip·ft})}{(60\text{ ksi})(0.9)(6'')} = 0.04\text{ in}^2$$

put reinf. in middle of wall/beam

The design reinforcement of #4@12" works

$$A_{s\,\#4} = 0.2\text{ in}^2 > 0.04\text{ in}^2 \checkmark$$

In fact, any size reinforcement would work,
but #4 @ 12" is the general minimum size and placement

Check Deflection

$$\Delta = \frac{Ph^3}{3EI} + \frac{2.78Ph}{A_{wall}E} = \frac{(9.24k)(10.25' \times \frac{12in}{ft})^3}{3(4158.87)(1860867\text{ in}^4)} + \frac{2.78(9.24k)(10.25' \times 12^{in}/ft)}{(20.25' \times 12''/ft)(4158.87\text{ksi})}$$

$$I = \tfrac{1}{12}(12'')(10.25' \times 12^{in}/ft)^3 = 1860867\text{ in}^4$$

$$\Delta = 0.0047'' < L/360 = \frac{10.25'(12^{in}/ft)}{360} = 0.342'' \checkmark$$