The Regent

950 N. Glebe Road
Arlington, VA

Architect: Cooper Carry Architects

Structural Technical Report 1
Structural Concepts/Structural Existing Conditions Report

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Option:	**Structural**
Date:	**October 5, 2005**
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Executive Summary

This report includes a detailed description and preliminary structural analysis of The Regent, which is currently under construction in Arlington, VA. The Regent is a 12-story office building which has retail space on the first level and a 3-story parking garage below grade.

The scope of this report includes providing a detailed description of the existing structural system and completing a preliminary structural analysis based on calculated design loads developed from model codes and design standards.

This report includes the identification of design codes and standards, identification of live loads and dead loads, the development of lateral loads and their distribution to the lateral load resisting elements, and a detailed description of the structural system above and below grade. In addition, structural members were spot checked for comparison between the existing design and the preliminary structural analysis design. This report summarizes the results of the lateral load analysis and lateral load distribution as well as the results from the spot checked members. The Appendix includes all of the detailed calculations and the framing plans.

After completing a preliminary investigation and analysis of the structure, it has been determined that the existing structural systems, design loads, and member sizes are in the ballpark of the calculated design loads and member sizes selected as a result of the preliminary structural analysis done in this report. In the case of the composite beam design, the moment capacity of the existing design significantly exceeded the calculated design load. Therefore, in this case, the existing design was determined to be conservative or some other analysis may have controlled the design, which resulted in the conservative design. In the case of the Plaza slab, it was determined to be under-reinforced for the midspan column strip, but not significantly. The slab reinforcement was adequate for the interior support column and middle strips and the midspan middle strip. In the case of the lateral element diagonal member, the calculated loads were equal to or greater than the loads listed in the structural plans, but the existing design of the lateral member was significantly more conservative than required by the analysis done in this report.

Technical Reports 2 and 3 will go into further detail and analysis of all of the loads on the structure, including loads out of the scope of this report, and a more accurate lateral distribution will be performed taking into account actual building and lateral framing characteristics and the foundation systems. Once these more in-depth investigations and analyses of the structural system are completed for Technical Reports 2 and 3, it is anticipated that the existing

design should better coincide with the design resulting from the more detailed analyses.

Codes and Code Requirements

The 2000 ICC International Building Code (IBC 2000) was used for the structural design of The Regent. IBC 2000 incorporates many of the design load procedures of ASCE 7. The design procedures for the lateral forces, wind and seismic, were taken from ASCE 7-02 Chapters 6 and 9 respectively. ASCE 7-02 was also used for calculating the snow loads and roof live loads. The live loads were taken from Table 1607.1 of IBC 2000. The equations, tables, and procedures used to calculate the design loads in this report were taken from ASCE 7-02. LRFD was used for the structural design.

Gravity Loads

- **Dead Loads**
  - Roof
    - 3" - 22 Gage Metal Deck — 5 PSF
    - Insulation — 3 PSF
    - Misc. DL — 10 PSF
    - Roofing — 20 PSF
  - Typical Floor
    - 3 ¼" lt. wt. slab on 3" - 20 gage metal deck (United Steel Deck design manual p. 40) — 46 PSF
    - Concrete Ponding *included because of the long steel spans and cambers — 10 PSF
    - Misc. DL (mechanical ducts, sprinklers, ceiling, plumbing, etc.) — 15 PSF
  - Construction Loads
    - 3 ¼" lt. wt. slab on 3" -20 gage metal deck — 46 PSF
    - Concrete Ponding — 10 PSF

- **Live Loads** (IBC 2000, Table 1607.1)
  - **Corridors** — **100 PSF**
  - **Stairs** — **100 PSF**
  - **Mechanical Spaces** — **150 PSF**
  - **Offices** — **100 PSF**
    - Lobbies and 1st Floor Corridors — 100 PSF *Critical Case
    - Offices — 50 PSF
    - Corridors above 1st Floor — 80 PSF
  - **Retail – 1st Level** — **100 PSF**
  - **Terrace Above 1st Floor Retail** — **100 PSF**
    - Deck (Roof/Patio) – same as occupancy served (Office) — 100 PSF
    - Balcony – exterior — 100 PSF
  - **Loading Dock** — **350 PSF**
    - *Designed for Arlington Fire Dept. Tower 75-1987 (total weight = 66,320#) — 350 PSF *Critical Case
  - **Parking Garage (Garages having trucks and busses)** — **50 PSF**
    - IBC 2000 1607.6
    - Truck and bus access provided to loading dock on 1st level
  - **Mechanical** — **150 PSF**
  - **Plaza Deck (Fire Truck Loading)** — **350 PSF**
    - Vehicular Driveways — 250 PSF
    - *Designed for Arlington Fire Dept. Tower 75-1987 (total weight = 66,320#) — 350 PSF *Critical Case

- **Snow Loads**
  - Snow Load — 30 PSF*
    *See Appendix for detailed Snow Load Calculations and Assumptions

- Construction Live Loads (unreducible) — 20 PSF

- Roof Live Loads (as calculated per ASCE 7-02) — 12 PSF — 30 PSF*
  *Since the 30 PSF snow load > 12 PSF, the roof live load = snow load = 30 PSF

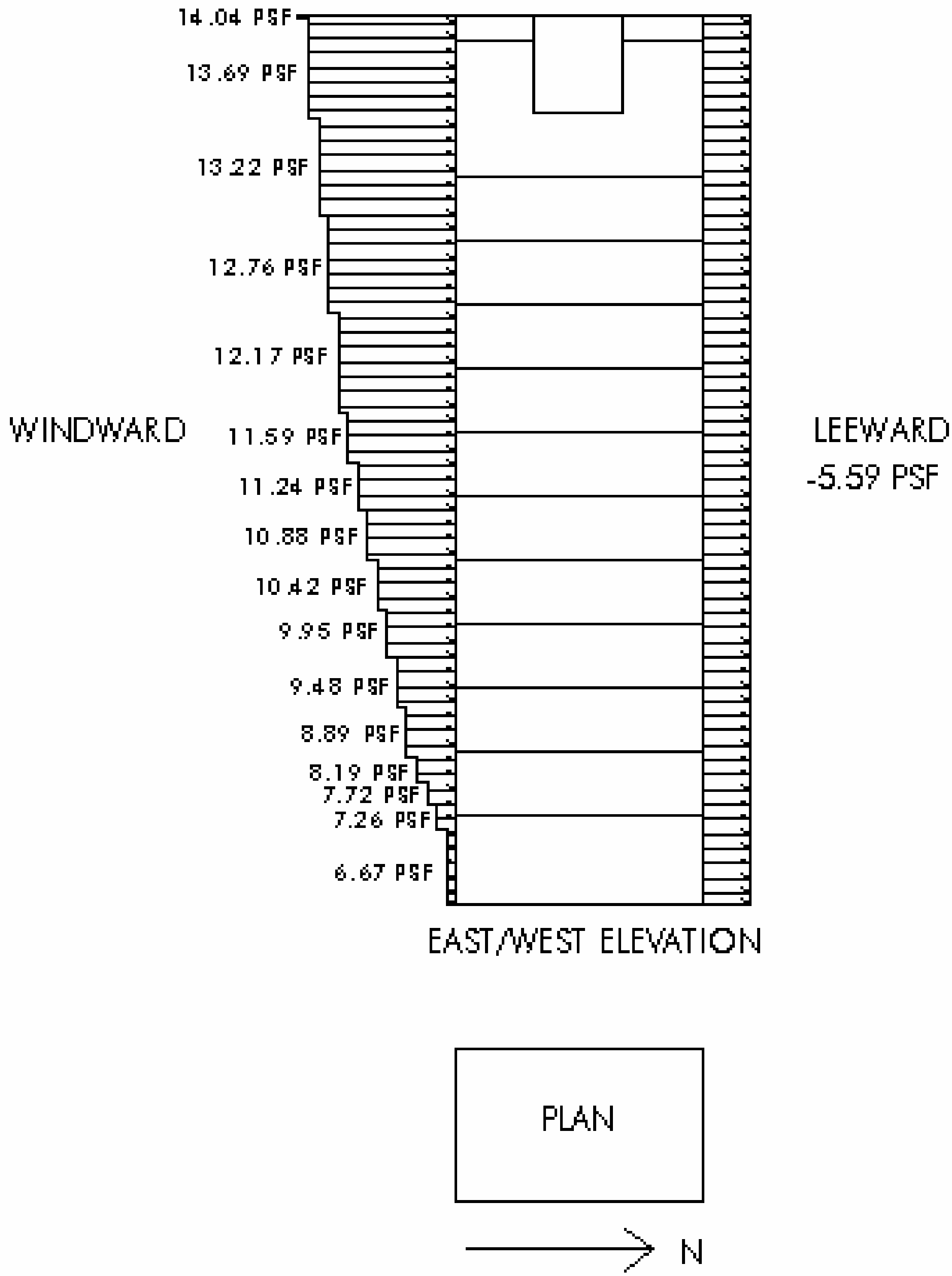
Lateral Loads

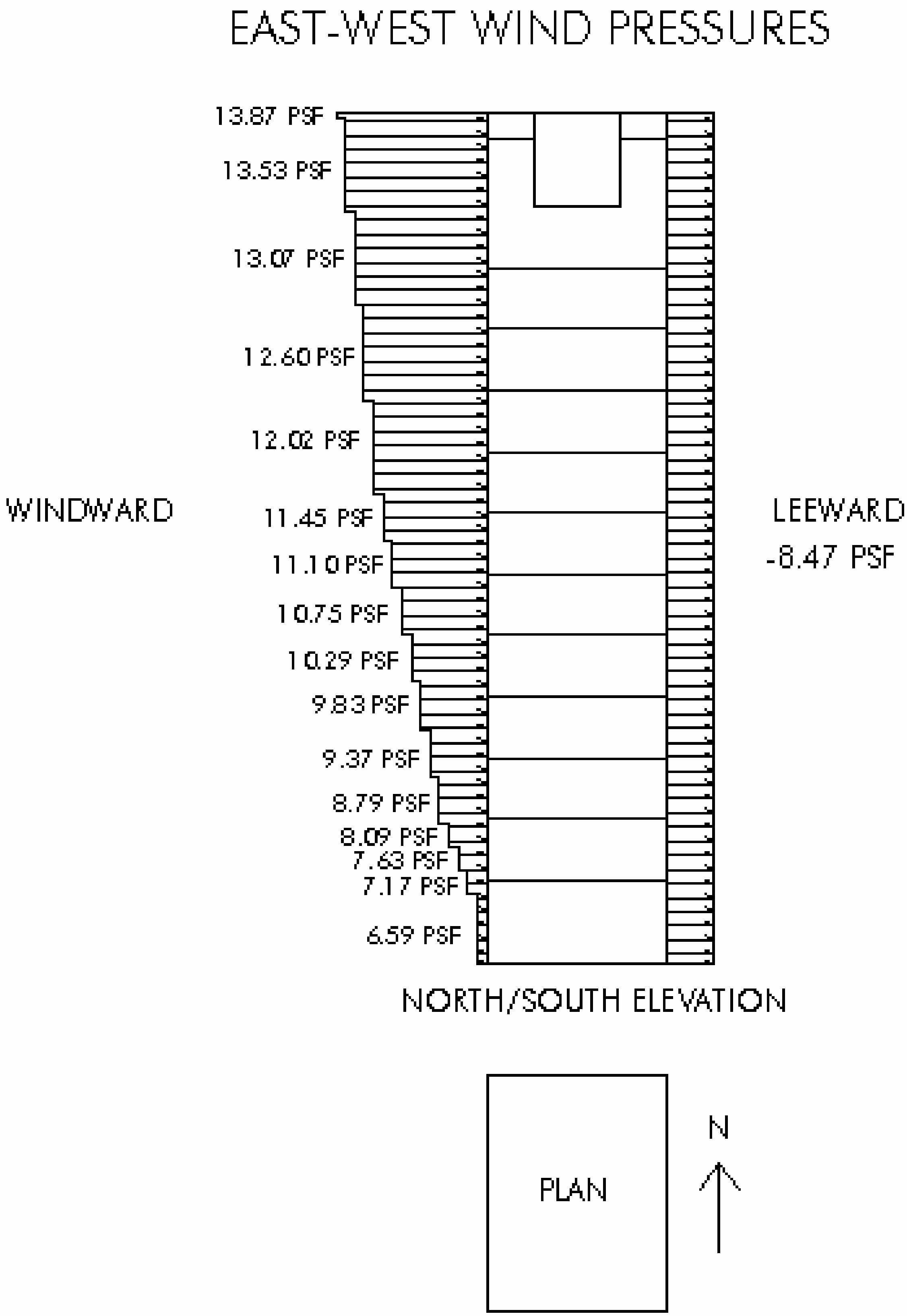
- **Wind Loads**
  *See Appendix for detailed Wind Load Calculations and Assumptions

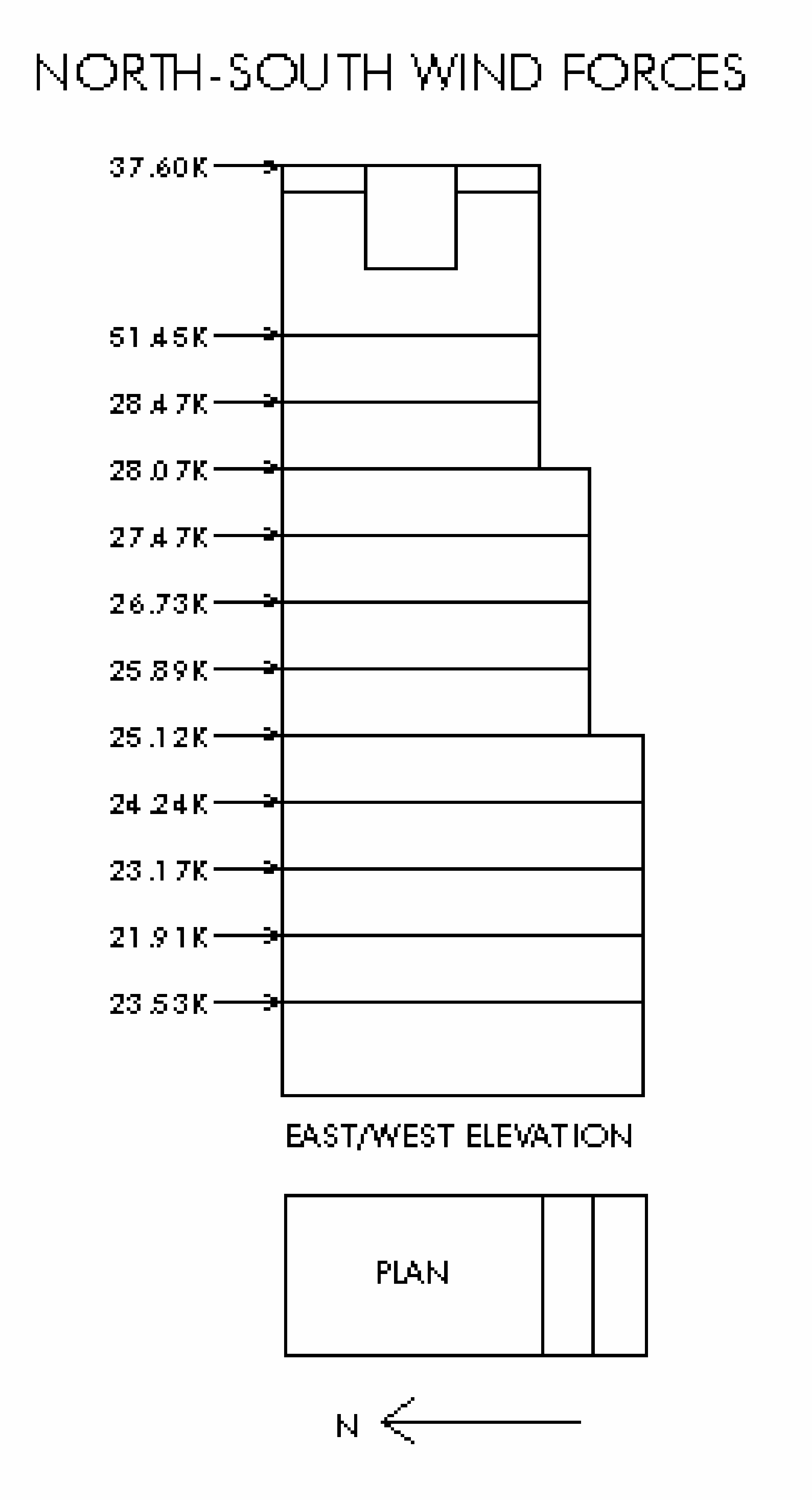
Wind Pressures

z	Kz	qz	N-S Windward Pressure (PSF)	E-W Windward Pressure (PSF)	N-S Leeward Pressure (PSF)	E-W Leeward Pressure (PSF)	P_{total} (N-S) (PSF)	P_{total} (E-W) (PSF)
0-15	0.57	10.05	6.67	6.59	-5.59	-8.47	12.26	15.06
20	0.62	10.93	7.26	7.17	-5.59	-8.47	12.85	15.64
25	0.66	11.63	7.72	7.63	-5.59	-8.47	13.31	16.10
30	0.70	12.34	8.19	8.09	-5.59	-8.47	13.78	16.56
40	0.76	13.40	8.89	8.79	-5.59	-8.47	14.48	17.26
50	0.81	14.28	9.48	9.37	-5.59	-8.47	15.07	17.84
60	0.85	14.98	9.95	9.83	-5.59	-8.47	15.54	18.30
70	0.89	15.69	10.42	10.29	-5.59	-8.47	16.01	18.76
80	0.93	16.39	10.88	10.75	-5.59	-8.47	16.47	19.22
90	0.96	16.92	11.24	11.10	-5.59	-8.47	16.83	19.57
100	0.99	17.45	11.59	11.45	-5.59	-8.47	17.18	19.92
120	1.04	18.33	12.17	12.02	-5.59	-8.47	17.76	20.49
140	1.09	19.21	12.76	12.60	-5.59	-8.47	18.35	21.07
160	1.13	19.92	13.22	13.07	-5.59	-8.47	18.81	21.54
180	1.17	20.62	13.69	13.53	-5.59	-8.47	19.28	22.00
200	1.20	21.15	14.04	13.87	-5.59	-8.47	19.63	22.34

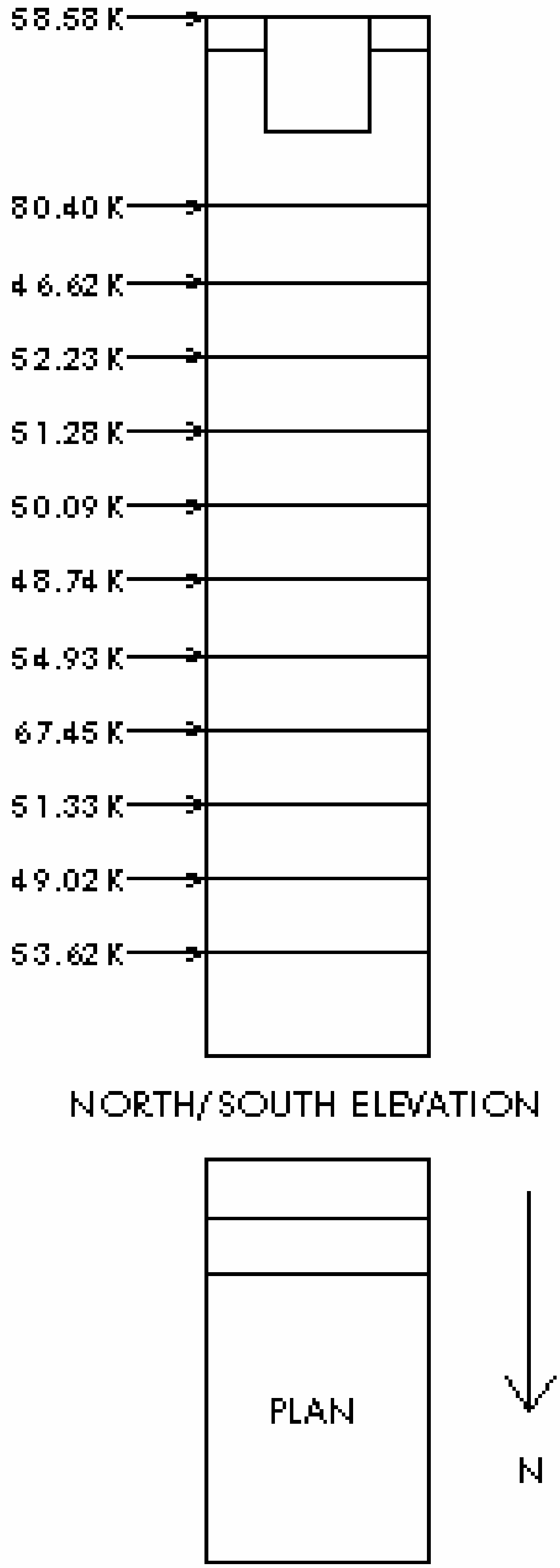
NORTH-SOUTH WIND PRESSURES




EAST-WEST WIND PRESSURES
13.87 PSF
13.53 PSF
13.07 PSF
12.60 PSF
12.02 PSF
11.45 PSF
11.10 PSF
10.75 PSF
10.29 PSF
9.83 PSF
9.37 PSF
8.79 PSF
8.09 PSF
7.63 PSF
7.17 PSF
6.59 PSF
WINDWARD
LEEWARD
-8.47 PSF
NORTH/SOUTH ELEVATION
PLAN
N


NORTH-SOUTH WIND FORCES
37.60K
51.45K
28.47K
28.07K
27.47K
26.73K
25.89K
25.12K
24.24K
23.17K
21.91K
23.53K
EAST/WEST ELEVATION
PLAN
N

EAST-WEST WIND FORCES



- **Seismic Loads**

  *See Appendix for detailed Seismic Load Calculations and Assumptions

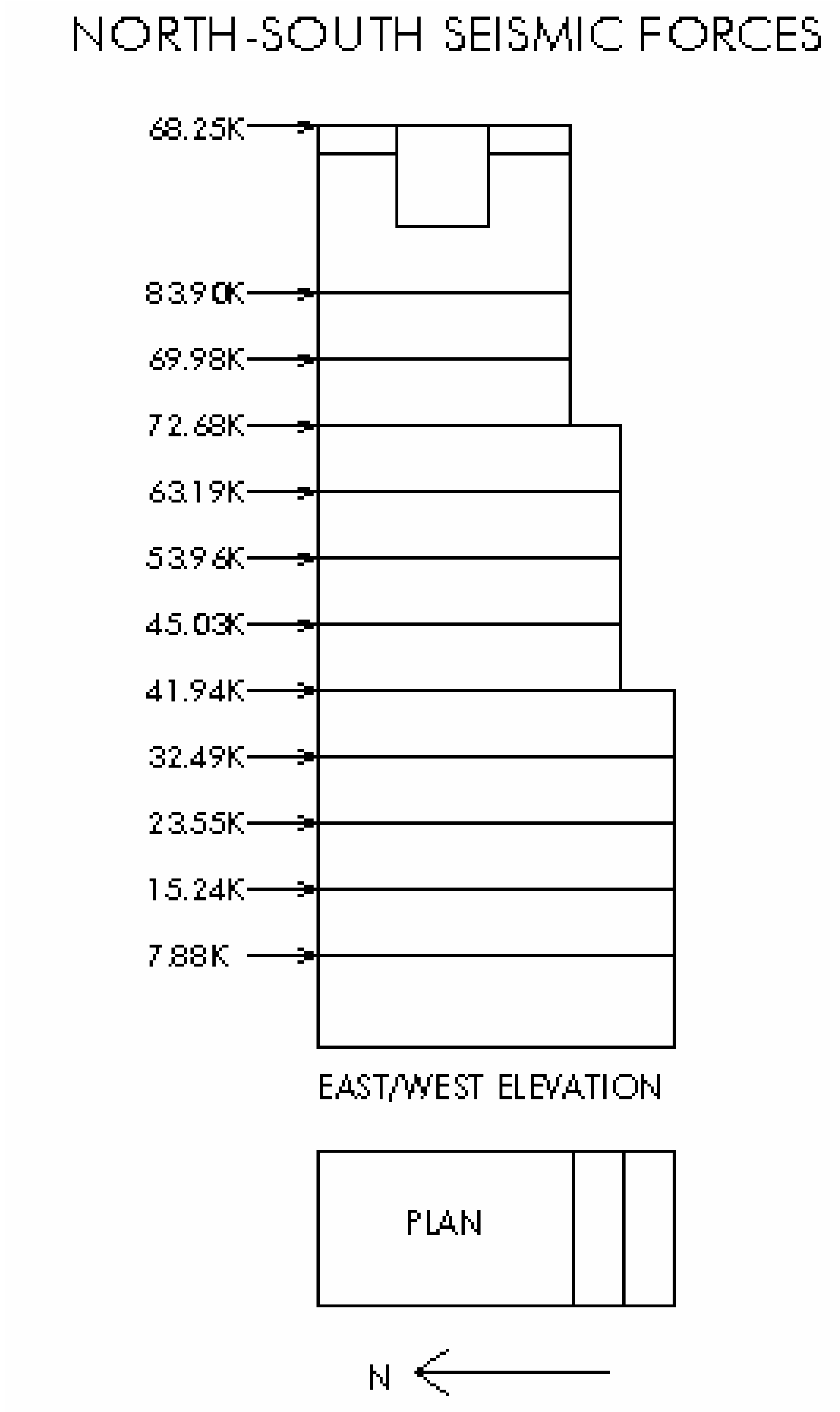
Base Shear and Overturning Moments

Level	w_x	h_x	$w_x h_x^{1.243}$	$w_x h_x^{1.243}$	C_{vx} (N-S)	C_{vx} (E-W)	F_x (N-S)	F_x (E-W)
12 (roof)	1026	180.75	655686	655686	0.118	0.118	68.25	68.25
11	1617	148	806019	806019	0.145	0.145	83.90	83.90
10	1512	135	672290	672290	0.121	0.121	69.98	69.98
9	1781	122	698247	698247	0.126	0.126	72.68	72.68
8	1781	109	606995	606995	0.109	0.109	63.19	63.19
7	1781	96	518355	518355	0.093	0.093	53.96	53.96
6	1781	83	432592	432592	0.078	0.078	45.03	45.03
5	2050	70	402912	402912	0.073	0.073	41.94	41.94
4	2050	57	312109	312109	0.056	0.056	32.49	32.49
3	2050	44	226238	226238	0.041	0.041	23.55	23.55
2	2050	31	146392	146392	0.026	0.026	15.24	15.24
1	2083	18	75682	75682	0.014	0.014	7.88	7.88
			5553516	5553516	1.000	1.000	578.10	578.10

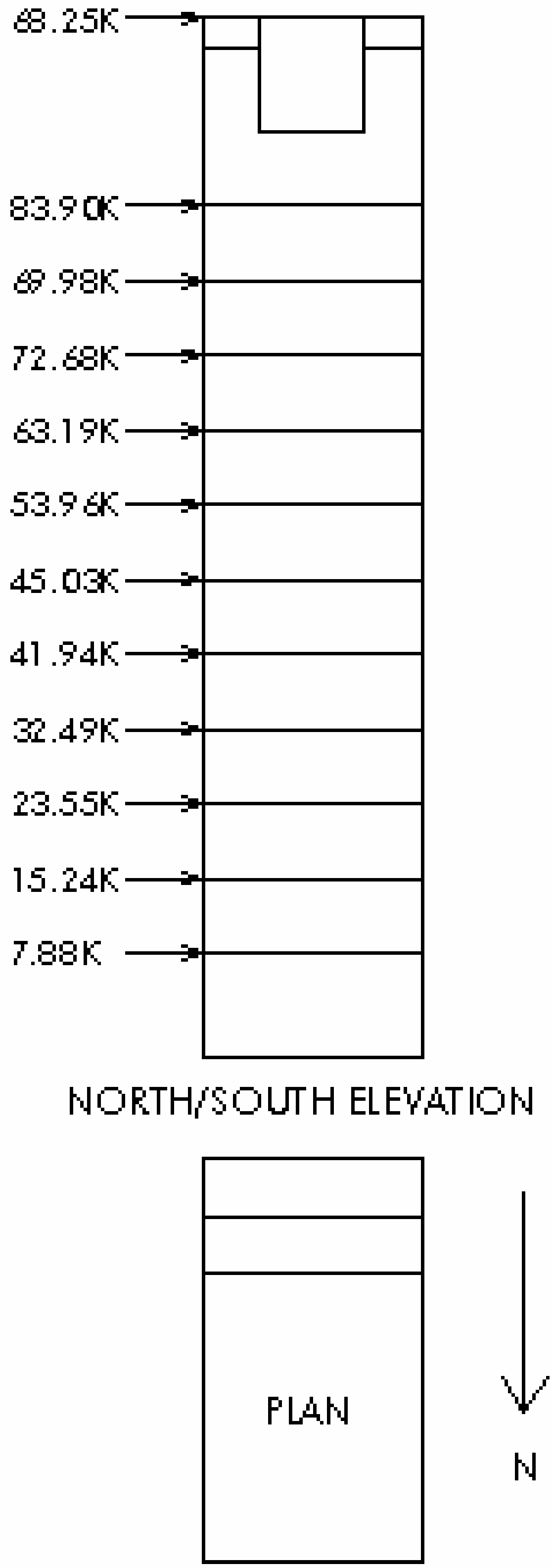
k (N-S)	1.243	
k (E-W)	1.243	
V (N-S)	578.1	k
V (E-W)	578.1	k

Base Shear		
N-S	578.10	k
E-W	578.10	k

Overturning Moment		
Overturning Moment (N-S)	65313.0733	ft-k
Overturning Moment (E-W)	65313.0733	ft-k


NORTH-SOUTH SEISMIC FORCES
68.25K
83.90K
69.98K
72.68K
63.19K
53.96K
45.03K
41.94K
32.49K
23.55K
15.24K
7.88K
EAST/WEST ELEVATION
PLAN
N

EAST-WEST SEISMIC FORCES



Other General or Special Loadings

In addition to the live loads, dead loads, roof live loads, snow loads, wind loads, and seismic loads already calculated, other special loadings considered were snow drift and construction loads. Additional general or special loadings may be considered in Technical Report 2 and/or Technical Report 3, but at this stage only these loads were considered.

Snow Drift

The Regent's roof is relatively flat with slopes of less than 3º and the roof slopes down toward the roof drainage systems. The Regent is located in Arlington, VA, where the minimum required snow load as per ASCE 7-02 is only 20 PSF. The actual design snow load specified in the structural notes is 30 PSF. Since the roof is relatively flat with a drainage system, and since the design roof snow load exceeded the code minimum by 150%, snow drift loads were not calculated.

Construction Loads

The construction loads considered for The Regent are listed below.

Dead Loads:

3 ¼" light weight concrete on 3" – 20 gage metal deck	46 PSF
Concrete Ponding	10 PSF

Live Loads:

Construction Live Load (unreducible)	20 PSF

Typical Framing Plans and Elevations

Framing Plans

*See Appendix for Framing Plans

Elevations

Architect: Cooper Carry Architects

The Regent's Southeastern corner and East Elevation looking across Glebe Road

Architect: Cooper Carry Architects

The Regent's Northern Elevation as seen from Glebe Road across North Fairfax Drive

Detailed Description of the Structure

Foundations

The foundations for The Regent consist of square footings ranging in size from 4' x 4' to 9' x 9' with depths ranging from 24" to 50" respectively. They are located on a 30' x 30' square grid. The two allowable bearing pressures for the square footings are 25 ksf and 40 ksf. The southwest quarter of the building has allowable bearing pressures of 25 ksf, while the other three quarters of the building have a 40 ksf allowable bearing pressure. The larger square footings are located in the central core of the building below the elevator shafts. There are also continuous 24" wide, 12" deep concrete footings under the 12" thick continuous walls. The slab on grade is 4" thick reinforced with 6 x 6, 10/10 WWF. The concrete strength for all foundations, walls, and slabs on grade is a minimum of 3000 psi.

Concrete Parking Garage Below Grade

There is a 3-level concrete parking garage below grade. The typical bay size for the three levels of below grade parking is 30' x 30'. The most common column sizes are 16" x 24"and 28" x 36" and the most common beam sizes are 12" x 24", 12" x 18", 8" x 18", and 18" x 30". All of the columns are of design strength f'c = 5000 psi, although a few are f'c = 7000 psi and the 28-day design strength of the beams is f'c = 4000 psi. The parking garage slabs are 8" thick with a typical drop panel size of 10' x 10' x 5 ½" and a 28-day strength of 4000 psi.

Plaza and 1st Floor Slabs

The Plaza level slab is 12" thick with 10' x 10' x 12" drop panels. The design loads for the Plaza level include a 350 PSF live load which accounts for the weight of a fire truck loading.

The first floor slab is 9" thick with 10' x 10' x 5 ½" drop panels. The Plaza and 1st floor slabs are both of strength f'c = 4000 psi.

Steel Framing Above Grade

There are two typical bay sizes for the steel superstructure above grade; 30' x 30' and approximately 43'-46' x 30'. From North to South the columns are at a 30' spacing. From East to West the columns spacings are approximately 46', 30' and 43' respectively. The most common column sizes are W14 x 145, W14 x 99, and W14 x 176.

The most common beam sizes are W18 x 50, W18 x 46, and W16 x 26 with cambers ranging from ¾" to 2", which are designed to 75% dead load. The most common girder sizes are W18 x 65, W24 x 55, W24 x 62, and W24 x 55.

The typical floor slab is 3 ¼" light weight concrete with an f'c = 3000 psi and is reinforced with 6 x 6 10/10 WWF on top of a 3" – 20 gage composite steel deck for a total slab thickness of 6 ¼". The typical floor slab construction continues to the retail roof terrace areas.

The roof deck construction is 3" x 22 gage, deep rib, type N, painted roof deck. There are a few full moment connections at certain corners of the roof and penthouse roof.

The perimeter precast panels and columns covers have a gravity connection to the columns only. They have no gravity connections to the beams.

The W-shapes are ASTM A572 (Grade 50) or ASTM A992. The structural tube shapes are ASTM A500 (Grade B).

Lateral Load Resisting System

The lateral load resisting system for The Regent consists of five braced frames at the core of the building (see the Braced Frame Location Plan in the *Lateral Load Resisting Elements* section). There are two braced frames, #4 and #5, that span along the building's north / south axis, and three braced frames, #1, #2, and #3, that span along the building's east / west axis. The braced frames are approximately 30' in width and run the full height of the building from the first floor to the penthouse roof.

The typical diagonal steel members used in the braced frames are HSS 8" x 8"'s, 10" x 10"'s, and 12" x 12"'s with thicknesses ranging from 3/8" to 5/8". The braced frame columns are all 14" wide flange members ranging in size from W14 x 233's and W14 x 257's near the base to W14 x 53's to W14 x 72's at the top.

Structural System Selection

There are several possible reasons why a steel framing system was used above grade. Since The Regent is a spec office building, the office floor levels need to be open floor plans, with minimal column interruptions. Since larger spans (>40 ft) are common in this building, steel members are able to accommodate the larger spans, while not significantly increasing the column and beam sizes. Also, The Regent's entire northern façade is a curtain wall system. A steel structure is typically used with curtain wall systems. Braced frames were probably used instead of shear walls because of the taller spans (>180 ft), narrow locations, and for consistency of materials.

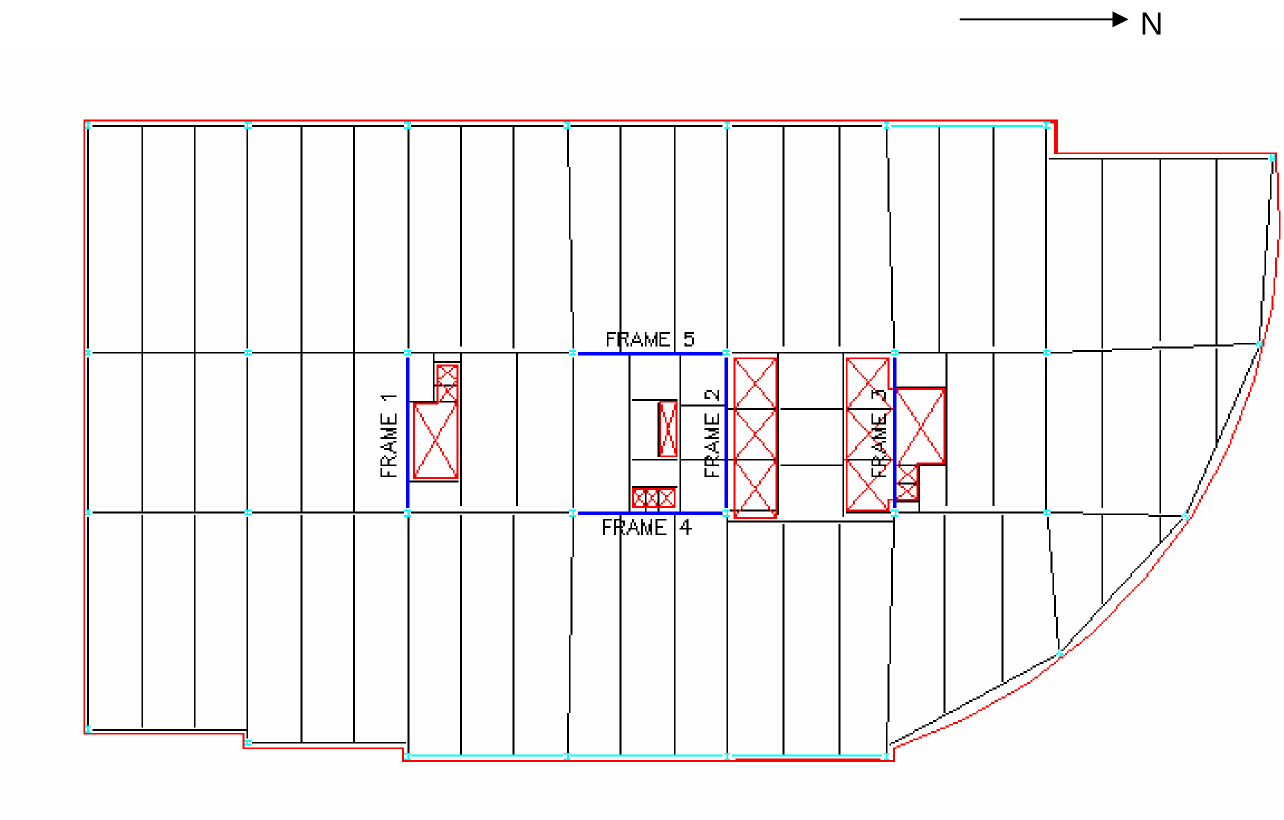
There are several possible reasons why concrete was used for the parking structure below grade. Parking garages in the Washington D.C. area are typically concrete structures. Concrete allows the thickness of the floors to be thinner than if a steel flooring system was used. Also, the structure is below grade where there are moisture issues to be concerned with. Steel could easily rust and become a maintenance problem. Other concrete structural systems, such as hollow core planks, precast, or prestressed systems could be other possible structural systems.

Alternative structural systems for The Regent's superstructure and below grade parking structure will be analyzed in Technical Report 2.

Lateral Load Resisting Elements

The lateral load resisting elements for The Regent are a combination of five centrally located braced frames. There are two braced frames, Frames #4 and #5, which span north to south and resist lateral loads from the east / west direction. The other three braced frames, Frames #1, #2, and #3, run from east to west and resist lateral loads in the north / south direction. Frames #1, #3, and #5 have chevron style bracing and Frames #2 and #4 have single diagonal bracing. All of the braced frames are approximately 30' wide and span the entire height of the building from the first level to the penthouse roof. All of the braced frame columns are W14's and all of the horizontal steel members are W18's. All of the diagonal members are HSS 8x8's, HSS 10x10's or HSS 12x12's.

Braced Frame Location Plan



Load Combinations Involving Wind Loads (W) and Seismic Loads (E)

ASCE 7-02 (Sec. 2.3.2)

1.2D + 1.6(Lr or S or R) + (L or **0.8W**)
1.2D + **1.6W** + L + 0.5(Lr or S or R)
1.2D + **1.0E** + L + 0.2S
0.9D + **1.6W** + 1.6H
0.9D + **1.0E** + 1.6H

Check 1.6W vs. 1.0E

Red = Controlling E-W Lateral Force, Blue = Controlling N-S Lateral Force

	1.6W (N-S)	1.6 (E-W)	1.0E (N-S/E-W)
Roof	60.16	**93.72**	**68.25**
12	82.32	**128.64**	**83.90**
11	45.55	**74.59**	**69.98**
10	44.91	**83.57**	**72.68**
9	43.95	**82.05**	**63.19**
8	42.77	**80.14**	**53.96**
7	41.42	**77.98**	**45.03**
6	40.19	**87.89**	**41.94**
5	**38.78**	**107.92**	32.49
4	**37.07**	**82.13**	23.55
3	**35.06**	**78.43**	15.24
2	**37.64**	**85.79**	7.88

After reviewing all of the load combinations for ASCE 7-02, it was determined that wind will control the lateral design in the east / west direction and seismic will control the north / south direction from the roof down to the 6th floor at which point wind will control. Only the load combinations involving wind and seismic were considered to calculate the worst case lateral loading since they are the only two loads considered in a lateral direction.

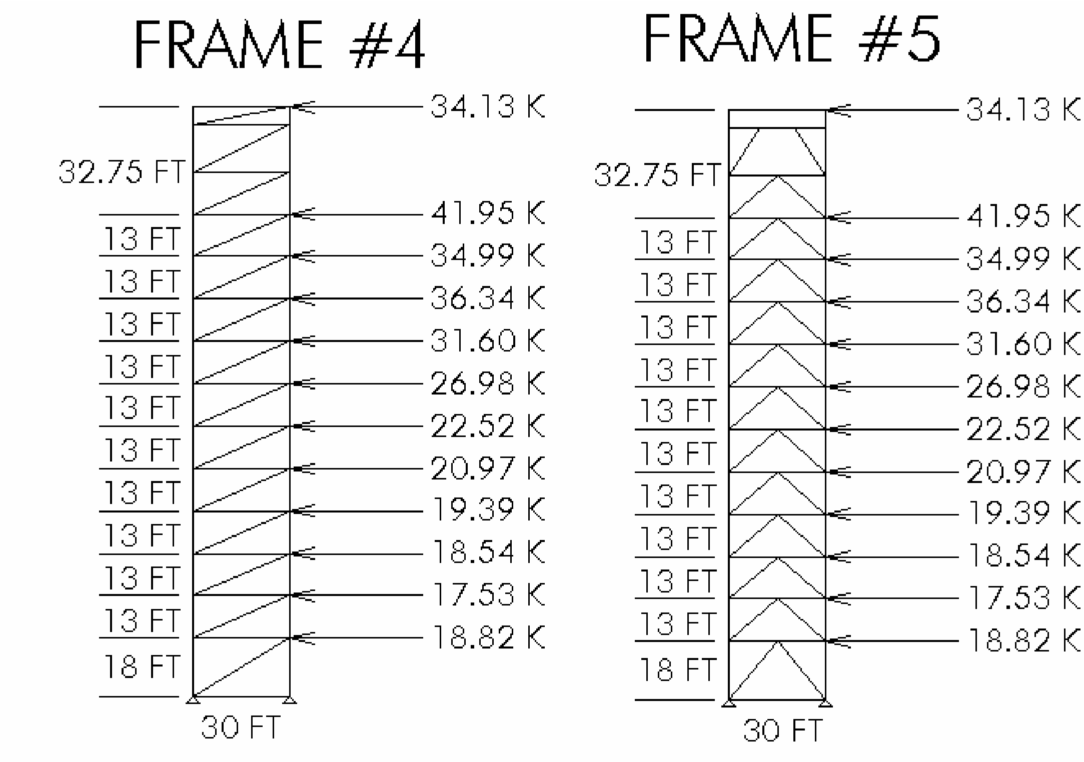
North / South Lateral Forces

When there are lateral forces acting in the north / south direction, Frames #4 and #5 will take the lateral loads. For the purposes of this report, Frames #4 and #5 are assumed to be approximately equidistant from the center of the load and the center of mass, of similar steel shapes and sizes, of similar height and width, and each frame is assumed to be of the same relative stiffness. As a result, each frame will take half of the lateral loading in the north / south direction. In the north / south direction, wind is the controlling lateral force from the roof level down to and including the 6th level. For the 5th through 2nd levels, the controlling lateral force is seismic.

The following table illustrates the lateral force distribution to Frames #4 and #5.

Level	Controlling Lateral Force	Total Factored Lateral Force to the Level (k)	Factored Lateral Force to Frame #4 (k)	Factored Lateral Force to Frame #5 (k)
Roof	Seismic	68.25	34.13	34.13
12	Seismic	83.90	41.95	41.95
11	Seismic	69.98	34.99	34.99
10	Seismic	72.68	36.34	36.34
9	Seismic	63.19	31.60	31.60
8	Seismic	53.96	26.98	26.98
7	Seismic	45.03	22.52	22.52
6	Seismic	41.94	20.97	20.97
5	Wind	38.78	19.39	19.39
4	Wind	37.07	18.54	18.54
3	Wind	35.06	17.53	17.53
2	Wind	37.64	18.82	18.82

Lateral Load Distribution Diagrams for Frames #4 and #5



East / West Lateral Forces

Frames #1, #2, and #3 will take the lateral loads in the east / west direction. The controlling lateral force for the east / west direction is wind for all of the levels. For the purposes of this report, the following are a list of assumptions used to distribute the east / west lateral forces to these three braced fames.

- All elevator shafts and stair openings are considered negligible and neglected
- The building is assumed rectangular in plan
- Braced frames are of equal stiffness – all three braced frames are of similar steel shapes and sizes, and of similar height and width
- The Distribution by Rigidity method was used even though the building is more than 7 stories
- A more detailed an accurate lateral force distribution procedure will be addressed in Technical Reports 2 and 3

*The Distribution by Rigidity calculations can be found in the Appendix.

The following table illustrates the lateral force distribution to Frames #1, #2 and #3.

Level	Controlling Lateral Force	Total Factored Lateral Force to the Level (k)	Factored Lateral Force to Frame #1 (k)	Factored Lateral Force to Frame #2 (k)	Factored Lateral Force to Frame #3 (k)
Roof	Wind	93.72	31.24	31.57	32.58
12	Wind	128.64	42.88	43.34	44.72
11	Wind	74.59	24.86	25.13	25.93
10	Wind	83.57	27.86	28.16	29.05
9	Wind	82.05	27.35	27.64	28.52
8	Wind	80.14	26.71	27.00	27.86
7	Wind	77.98	25.99	26.27	27.11
6	Wind	87.89	29.30	29.61	30.55
5	Wind	107.92	35.97	36.36	37.52
4	Wind	82.13	27.38	27.67	28.55
3	Wind	78.43	26.14	26.42	27.26
2	Wind	85.79	28.60	28.90	29.82

The following table summarizes the results of the Distribution by Rigidity lateral distribution procedure.

Level	Lateral Force (k)	Mt (ft-k)	$F_{1,dir}$ (k)	$F_{2,dir}$ (k)	$F_{3,dir}$ (k)	$F_{1,tor}$ (k)	$F_{2,tor}$ (k)	$F_{3,tor}$ (k)	$F_{total,1}$ (k)	$F_{total,2}$ (k)	$F_{total,3}$ (k)
Roof	93.72	140.58	31.24	31.24	31.24	-1.67	0.33	1.34	31.24	31.57	32.58
12	128.64	192.96	42.88	42.88	42.88	-2.30	0.46	1.84	42.88	43.34	44.72
11	74.59	111.89	24.86	24.86	24.86	-1.33	0.27	1.07	24.86	25.13	25.93
10	83.57	125.36	27.86	27.86	27.86	-1.49	0.30	1.19	27.86	28.16	29.05
9	82.05	123.08	27.35	27.35	27.35	-1.47	0.29	1.17	27.35	27.64	28.52
8	80.14	120.21	26.71	26.71	26.71	-1.43	0.29	1.14	26.71	27.00	27.86
7	77.98	116.97	25.99	25.99	25.99	-1.39	0.28	1.11	25.99	26.27	27.11
6	87.89	131.84	29.30	29.30	29.30	-1.57	0.31	1.26	29.30	29.61	30.55
5	107.92	161.88	35.97	35.97	35.97	-1.93	0.39	1.54	35.97	36.36	37.52
4	82.13	123.20	27.38	27.38	27.38	-1.47	0.29	1.17	27.38	27.67	28.55
3	78.43	117.65	26.14	26.14	26.14	-1.40	0.28	1.12	26.14	26.42	27.26
2	85.79	128.69	28.60	28.60	28.60	-1.53	0.31	1.23	28.60	28.90	29.82

Center of Mass (111.5', 59.5')
Center of Rigidity (150', 0')

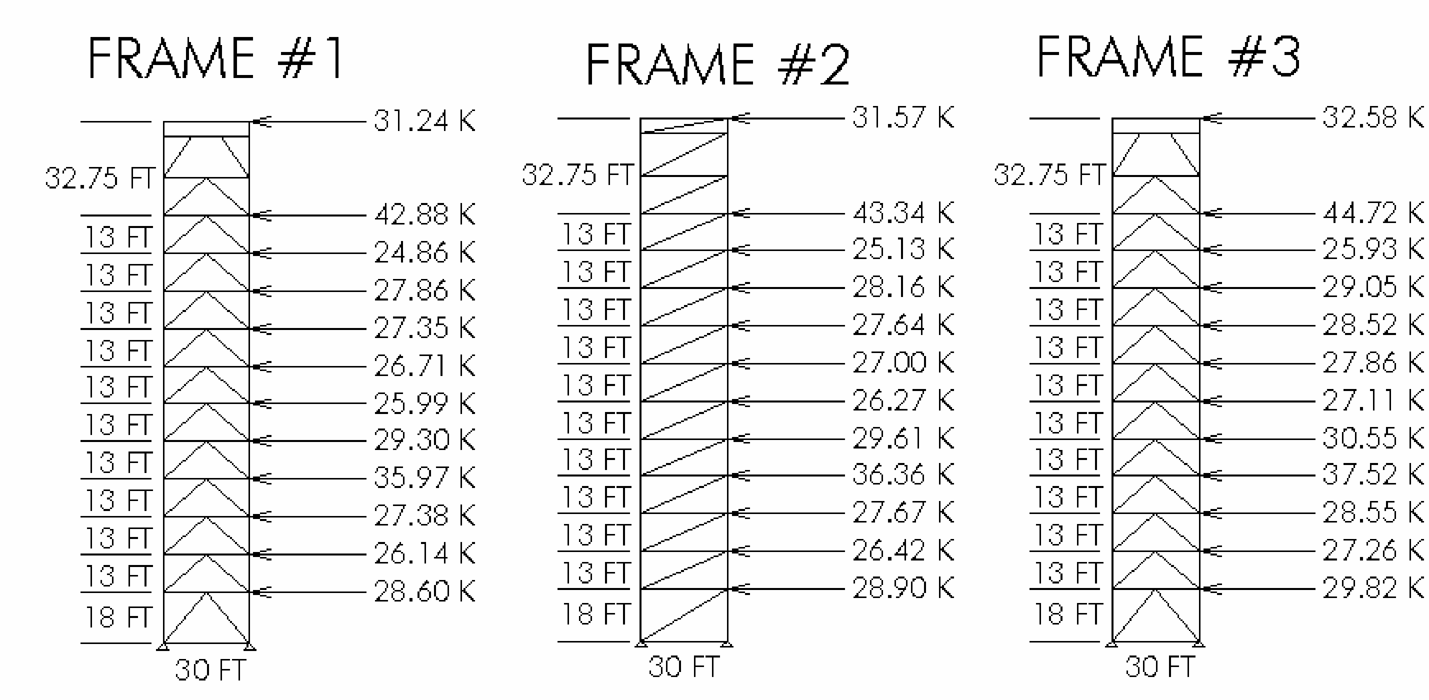
e_x	1.5	ft
e_y	59.5	ft

k_1	1
k_2	1
k_c	1
Σk	3

d_1	-50	ft
d_2	10	ft
d_3	40	ft

J	4200	ft^3

Lateral Load Distribution Diagrams for Frames #1, #2, and #3



Structural Elements that will Eventually Need to be Addressed or Designed

There are several structural elements not discussed in this report that will eventually need to be addressed. These structural elements are listed below. Technical Reports 2 and 3 will address and/or cover the design of these structural elements.

- The design of the parking garage walls and the foundation system to account for soil conditions
- The design of the cantilevered roof brow and sunken mechanical penthouse
- The design and detailing of the exterior walls, wall systems, and connections taking into account gravity and lateral loading
- Adjustment of the lateral force distribution to the braced frames based on more detailed lateral force distribution analysis
- Adjustment of wind loading based on a more detailed wind analysis
- Adjustment of seismic loading based on a more detailed seismic analysis
- The design of the lower level canopy and 2nd floor roof terrace
- The design / design check of the sheeting and shoring system around the perimeter of the parking garage walls

Spot Checks Performed and their Results

Since the purpose of this report is to analyze the existing structural design, spot checks of different members and elements were checked throughout different areas of the building. All of these detailed calculations are included in the Appendix. This section will summarize and explain the conclusions of the spot checked elements.

The following is a list of members and elements that were spot checked:

- Composite beam design for a steel beam and concrete slab system from a typical bay
- Plaza slab for minimum thickness requirements, minimum reinforcement requirements and required moment capacities to carry design loads
- Lateral frame element from braced frame #4

Composite Beam Design

A composite beam design for a typical bay was checked. A W18 x 50 steel beam and typical floor slab (3 ¼" light weight concrete on 3" deck) composite design was checked and compared the design moment due to the dead loads, self weight and live loads. The dead loads include the slab self weight, concrete ponding, miscellaneous dead loads (mechanical ducts, ceiling, plumbing, etc.), and the self weight of the steel beam. The live load is 100 PSF because it is office space. The live load was able to be reduced to 74.72 PSF according to ASCE 7-02 Section 4.8.1. The controlling load combination was 1.2D + 1.6L.

The composite beam was determined to be fully composite with the plastic neutral axis in the concrete slab, and therefore the steel controlled the design. The moment capacity of the composite beam was determined to be 658.2 FT-K and it needs to carry a moment of 546.1 FT-K. Therefore, the composite beam design was okay and a bit on the conservative side.

This composite beam design (6 ¼" slab with W18 x 50's) is typical throughout the building. There are several possible reasons for the approximately 100 FT-K discrepancy between design moment and composite beam moment capacity. They are listed below:

- The live load reduction may not have been taken into account during the composite beam design
- The slab thickness, deck size, and steel beam size may be controlled by another structural analysis in which the resultant sizes yield a greater moment capacity than actually needed
- The assumed miscellaneous dead load design value of 15 PSF may have been unconservative

Plaza Slab Design

An interior bay of the plaza slab was analyzed. The plaza slab design incorporates a 12" slab with 10' x 10' x 12" drop panels. The dead loads for the plaza slab include the self weight of the slab, the self weight of the panels distributed over the bay, and a miscellaneous dead load for lighting, electrical, etc. The live load for the plaza slab is 350 PSF to accommodate a fire-truck loading since the plaza includes emergency vehicle access. The live load was reduced to 263 PSF as per ASCE 7-02 Section 4.8.1.

The 12" specified slab is greater than the minimum thickness of 9" which was calculated using Table 9.5(c) of the ACI 318-02 code. The column strip and middle strip design moments were calculated using the Direct Design Method of ACI 318-02 Section 13.6. The specified reinforcement for the interior support and midspan column and middle strips was checked to see if it was adequate to carry the loads and to see if it met minimum steel area requirements. The specified reinforcement was determined to be inadequate for the midspan column strip, but was adequate for the interior support middle and column strips and the midspan middle strip.

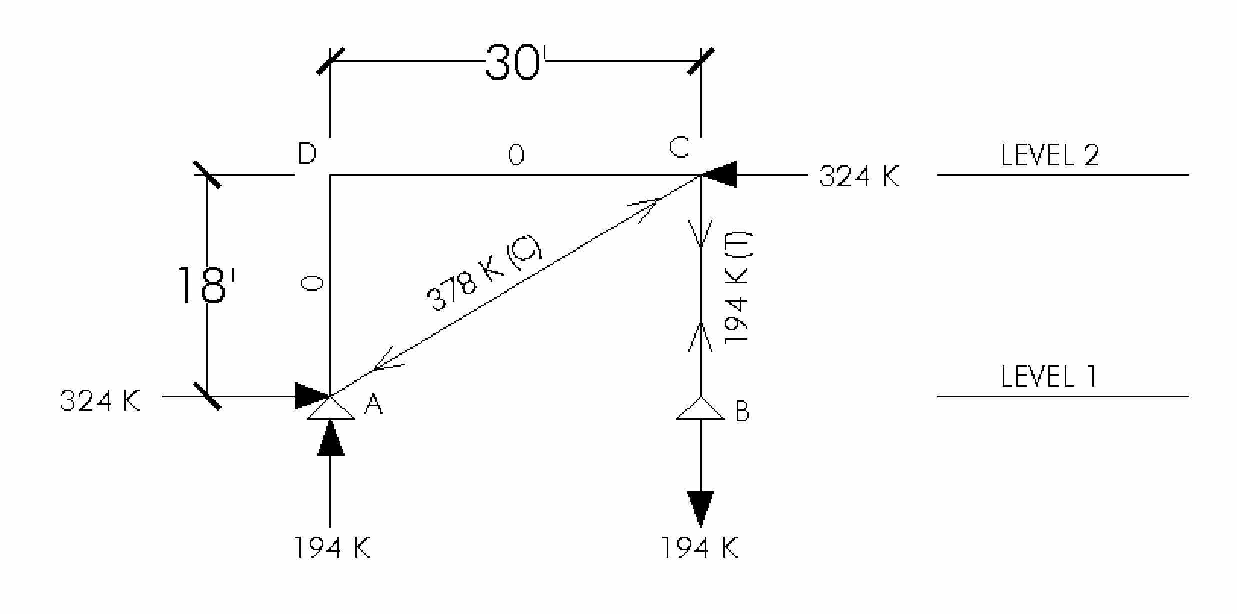
The possible reasons why the midspan reinforcement was determined to be inadequate are listed below:

- The assumed miscellaneous dead load value of 10 PSF may be higher than that actually considered in the design
- The Direct Design Method may not have been used to determine the design moments; and alternative approach to determining the design moments may have yielded lower design moments

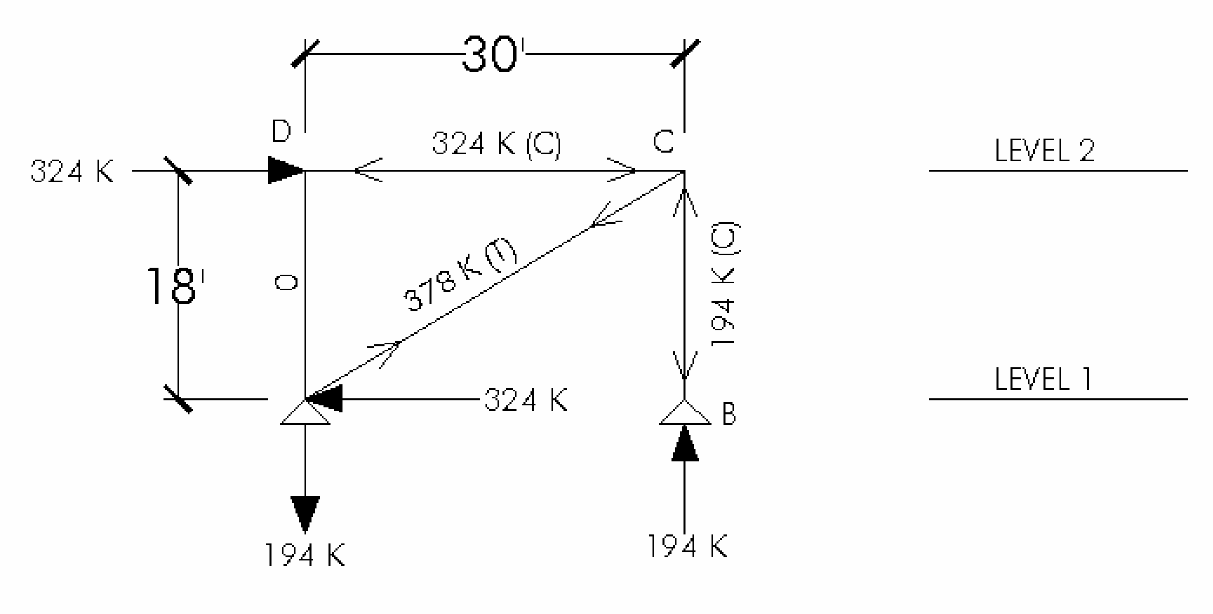
Lateral Member in a Braced Frame

The bottom diagonal bracing member for Frame #4 was checked for the base shear force due to the worst case lateral loadings in the north / south direction. The factored base shear is 324 K.

Wind from the North



Wind from the South



The force in the diagonal member is 378 k in either tension or compression depending on which way the lateral force is acting. Since the steel is specified to be 50 ksi, the required area of steel can be calculated, and a member size selected. Since all of the braced frames are using HSS 10 x 10, 8 x 8 or 12 x 12 members, with HSS 10 x 10's being the most common, an HSS 10 x 10 member was selected and compared to the actual designed member.

$$A_{steel,req'd} = \frac{378k}{50ksi} = 7.56in^2$$

From Table 1-11 from the AISC's Manual of Steel Construction, an HSS 10 x 10 x ¼ was selected, with an area of 8.96 in^2, which is greater than 7.56 in^2, and therefore should be okay.

The actual member size is an HSS 10 x 10 x 5/8 with an area of steel of 21 in^2. In reviewing the compression and tensile forces listed with the actual member in the braced frame elevation in the structural drawings, the diagonal member has calculated factored forces of 378 k (C) and 295 k (T). My calculated values were equal to or greater than the forces designed for. Since a thickness of ¼" meets the stress requirements, a 5/8" was probably chosen based off of other structural calculations and is considered a more conservative section. The member may also have had to meet minimum thickness requirements because it is a critical member in the braced frame.

Foundation System and its Impact on the Superstructure Design and Analysis

The foundation design and analysis is out of the scope of this report, but this section will describe how the foundation system impacts the superstructure design and analysis. The foundation system was previously described in detail in the *Detail Description of the Structure* section.

When the lateral pressures and forces were being determined, the building was assumed to be fixed into the ground at the first level for simplification purposes. In reality, there is a foundation system and three levels of a concrete parking garage below grade as well as a sheeting and shoring system around the perimeter of the building. Considering the actual foundations and sheeting and shoring systems in the lateral loading calculations may have resulted in different lateral loadings.

The foundations and the surrounding soils and soil pressure are a part of the lateral force resisting system and as a result, the applied lateral forces will cause overturning moments and stresses in the below-grade structure and the foundations systems. These imposed moments and stresses in the foundation systems need to be considered for a complete and accurate structural analysis. Technical Reports 2 and 3 will take the actual foundation systems and below grade structures into consideration and a more accurate and detailed lateral loading analysis will be performed.

Conclusion

After completing a preliminary investigation and analysis of the structure, it has been determined that the existing structural systems, design loads, and member sizes are in the ballpark of the calculated design loads and member sizes selected as a result of the preliminary structural analysis done in this report. In the case of the composite beam design, the moment capacity of the existing design significantly exceeded the calculated design load. Therefore, in this case, the existing design was determined to be conservative or some other analysis may have controlled the design, which resulted in the conservative design. In the case of the Plaza slab, it was determined to be under-reinforced for the midspan column strip, but not significantly. The slab reinforcement was adequate for the interior support column and middle strips and the midspan middle strip. In the case of the lateral element diagonal member, the calculated loads were equal to or greater than the loads listed in the structural plans, but the existing design of the lateral member was significantly more conservative than required by the analysis done in this report.

Technical Reports 2 and 3 will go into further detail and analysis of all of the loads on the structure, including loads out of the scope of this report, and a more accurate lateral distribution will be performed taking into account actual building and lateral framing characteristics and the foundation systems. Once these more in-depth investigations and analyses of the structural system are completed for Technical Reports 2 and 3, it is anticipated that the existing design should better coincide with the design resulting from the more detailed analyses.

Appendix

Wind Loads

Assumptions

- Assumed fixed at ground level even though there is a 3-level parking garage below grade
- Building shape, in plan and elevation, was assumed rectangular with the dimensions being 222.5' in the North / South direction and 119' in the East / West direction and a height of 180.75', which is the tallest height measurement for the building. See framing plans and elevations for actual building shape and dimensions.

  NOTE: These assumed building shapes and dimensions were used to calculate the pressure profiles along the height of the building for a conservative approach. When the actual forces to each floor were calculated, actual building dimensions and shapes were used.

- The wind load calculation procedures were taken from ASCE 7-02, Chapter 6. Method 2: Analytical Procedure (Sec. 6.5) was used for this building.

Building Information

- N-S direction – Steel Braced Frames
- E-W direction – Steel Braced Frames
- Location: Arlington, VA
- Exposure B
- Building Use: Office (Primary), Retail (1st Level), Parking (Below Grade)

Velocity Pressure

○ K_{zt} = 1.0	(Fig. 6-4)	area is flat
○ K_d = 0.85	(Table 6-4)	Building MWFRS
○ V = 90 mph	(Fig. 6-1)	
○ Use Group II	(Table 1-1)	
○ I = 1.0	(Table 6-1)	

From Table 6-3 (Exposure B, Case 2)

z (ft)	Kz
0-15	0.57
20	0.62
25	0.66
30	0.70
40	0.76
50	0.81
60	0.85
70	0.89
80	0.93
90	0.96
100	0.99
120	1.04
140	1.09
160	1.13
180	1.17
200	1.20

$$q_z = 0.00256 K_{zt} K_d V^2 I K_z$$

$$q_z = 0.00256(1.0)(0.85)(90)^2 (1.0) K_z$$
$q_z = 17.63 K_z$ PSF

$q_h = 17.63(1.17^*)$ *linear interpolation
$q_h = 20.65$ PSF

External Pressure Coefficients (Fig. 6-6)

Windward Wall: Cp = 0.8
Leeward Wall:
N-S: L/B = 222.5'/119' = 1.87 Cp = -0.326* *linear interpolation
E-W: L/B = 119'/222.5' = 0.53 Cp = -0.5

Gust Factor (N-S Direction)

N-S Direction: B = 119', L = 222.5'

Estimate Frequency (C_t = 0.02, x = 0.75 – Table 9.5.5.3.2)

$$f = \frac{1}{C_t h_n^x} = \frac{1}{0.02(180.75)^{0.75}} = 1.01 Hz > 1.0 \therefore Rigid$$ (Inverse of Eq. 9.5.5.3.2-1)

G = 0.85 or

Calculate G

From Table 6-2 (Exposure B)

$$\bar{z}_{\min} = 30\,ft$$
$$c = 0.3$$
$$l = 320\,ft$$
$$\bar{\varepsilon} = 1/3$$

$$g_Q = 3.4$$
$$g_V = 3.4$$
(6.5.8.1)

$$\bar{z} = 0.6h = 0.6(180.75) = 108.45' > 30' \therefore \bar{z} = 108.45'$$ (6.5.8.1)

$$L_z = l(\bar{z}/33)^{\bar{\varepsilon}} = 320(108.45/33)^{1/3} = 475.76$$ (Eq. 6-7)

$$I_z = c(33/\bar{z})^{1/6} = 0.3(33/108.45)^{1/6} = 0.246$$ (Eq. 6-5)

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_z}\right)^{0.63}}} = \sqrt{\frac{1}{1 + 0.63\left(\frac{119 + 180.75}{475.76}\right)^{0.63}}} = 0.82$$ (Eq. 6-6)

$$G = 0.925\left(\frac{1 + 1.7 g_Q I_z Q}{1 + 1.7 g_V I_z}\right) = 0.925\left(\frac{1 + 1.7(3.4)(0.246)(0.82)}{1 + 1.7(3.4)(0.246)}\right) = 0.83$$ (Eq. 6-4)

Since 0.83 < 0.85, use G=0.83

Gust Factor (E-W Direction)

E-W Direction: B = 222.5', L = 119'

Estimate Frequency (C_t = 0.02, x = 0.75 – Table 9.5.5.3.2)

$$f = \frac{1}{C_t h_n^x} = \frac{1}{0.02(180.75)^{0.75}} = 1.01 Hz > 1.0 \therefore Rigid$$ (Inverse of Eq. 9.5.5.3.2-1)

G = 0.85 or

Calculate G

From Table 6-2 (Exposure B)

$$\bar{z}_{\min} = 30\,ft$$
$$c = 0.3$$
$$l = 320\,ft$$
$$\bar{\varepsilon} = 1/3$$

$$g_Q = 3.4$$
$$g_V = 3.4$$
(6.5.8.1)

$$\bar{z} = 0.6h = 0.6(180.75) = 108.45' > 30' \therefore \bar{z} = 108.45'$$ (6.5.8.1)

$$L_z = l(\bar{z}/33)^{\bar{\varepsilon}} = 320(108.45/33)^{1/3} = 475.76$$ (Eq. 6-7)

$$I_z = c(33/\bar{z})^{1/6} = 0.3(33/108.45)^{1/6} = 0.246$$ (Eq. 6-5)

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_z}\right)^{0.63}}} = \sqrt{\frac{1}{1 + 0.63\left(\frac{222.5 + 180.75}{475.76}\right)^{0.63}}} = 0.799$$ (Eq. 6-6)

$$G = 0.925\left(\frac{1 + 1.7 g_Q I_z Q}{1 + 1.7 g_V I_z}\right) = 0.925\left(\frac{1 + 1.7(3.4)(0.246)(0.799)}{1 + 1.7(3.4)(0.246)}\right) = 0.82$$ (Eq. 6-4)

Since 0.82 < 0.85, use G=0.82

N-S Windward Pressure

$$P_{wz} = q_z C_p G = q_z 0.8(0.83) = 0.664 q_z \text{ PSF}$$

N-W Leeward Pressure

$$P_{lh} = q_h C_p G = 20.65(-0.326)(0.83) = -5.59 \text{ PSF}$$

E-W Windward Pressure

$$P_{wz} = q_z C_p G = q_z 0.8(0.82) = 0.656 q_z \text{ PSF}$$

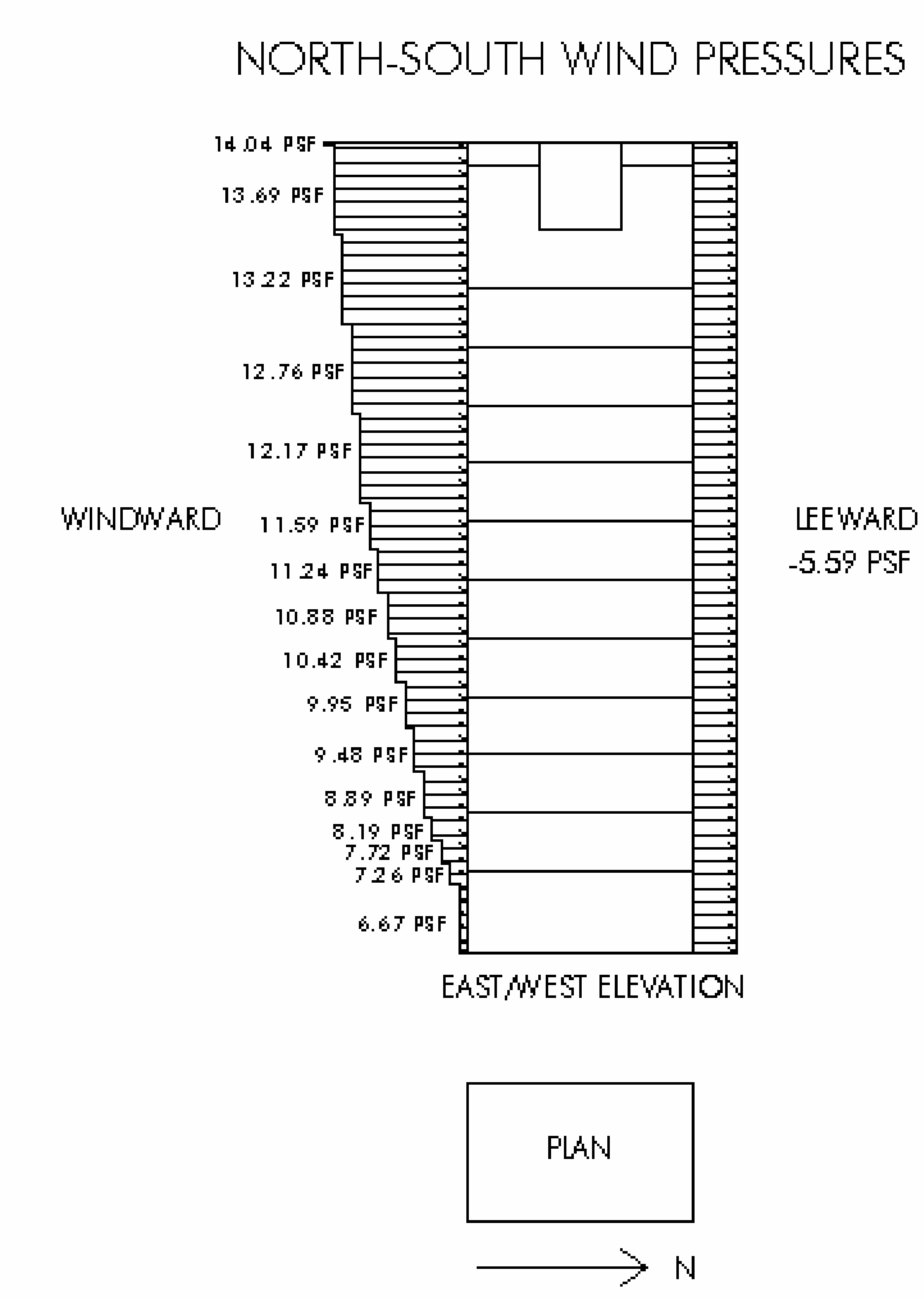
E-W Leeward Pressure

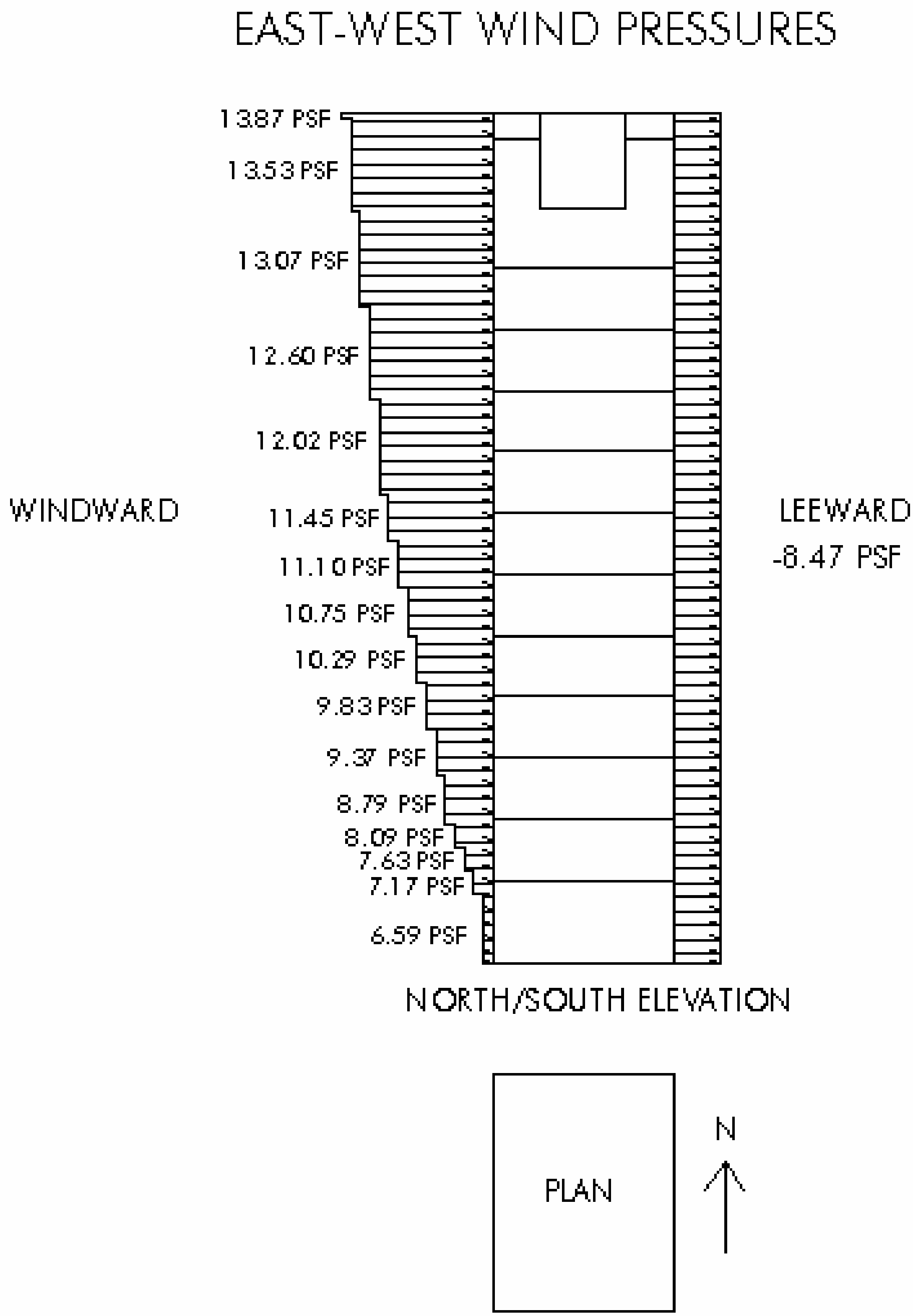
$$P_{lh} = q_h C_p G = 20.65(-0.5)(0.82) = -8.47 \text{ PSF}$$

Total Pressures

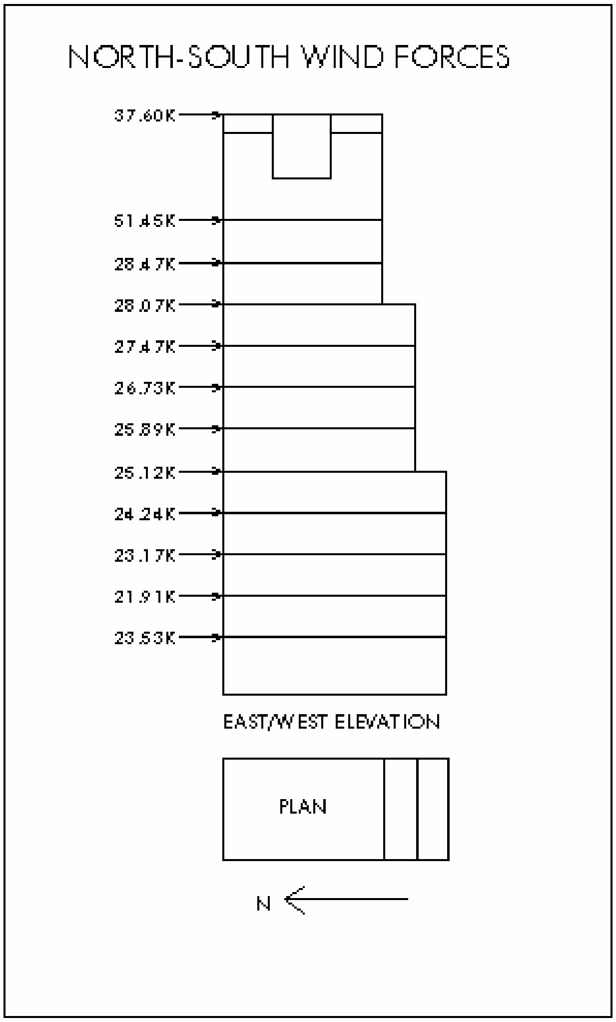
z	Kz	qz	N-S Windward Pressure (PSF)	E-W Windward Pressure (PSF)	N-S Leeward Pressure (PSF)	E-W Leeward Pressure (PSF)	P_{total} (N-S) (PSF)	P_{total} (E-W) (PSF)
0-15	0.57	10.05	6.67	6.59	-5.59	-8.47	12.26	15.06
20	0.62	10.93	7.26	7.17	-5.59	-8.47	12.85	15.64
25	0.66	11.63	7.72	7.63	-5.59	-8.47	13.31	16.10
30	0.70	12.34	8.19	8.09	-5.59	-8.47	13.78	16.56
40	0.76	13.40	8.89	8.79	-5.59	-8.47	14.48	17.26
50	0.81	14.28	9.48	9.37	-5.59	-8.47	15.07	17.84
60	0.85	14.98	9.95	9.83	-5.59	-8.47	15.54	18.30
70	0.89	15.69	10.42	10.29	-5.59	-8.47	16.01	18.76
80	0.93	16.39	10.88	10.75	-5.59	-8.47	16.47	19.22
90	0.96	16.92	11.24	11.10	-5.59	-8.47	16.83	19.57
100	0.99	17.45	11.59	11.45	-5.59	-8.47	17.18	19.92
120	1.04	18.33	12.17	12.02	-5.59	-8.47	17.76	20.49
140	1.09	19.21	12.76	12.60	-5.59	-8.47	18.35	21.07
160	1.13	19.92	13.22	13.07	-5.59	-8.47	18.81	21.54
180	1.17	20.62	13.69	13.53	-5.59	-8.47	19.28	22.00
200	1.20	21.15	14.04	13.87	-5.59	-8.47	19.63	22.34

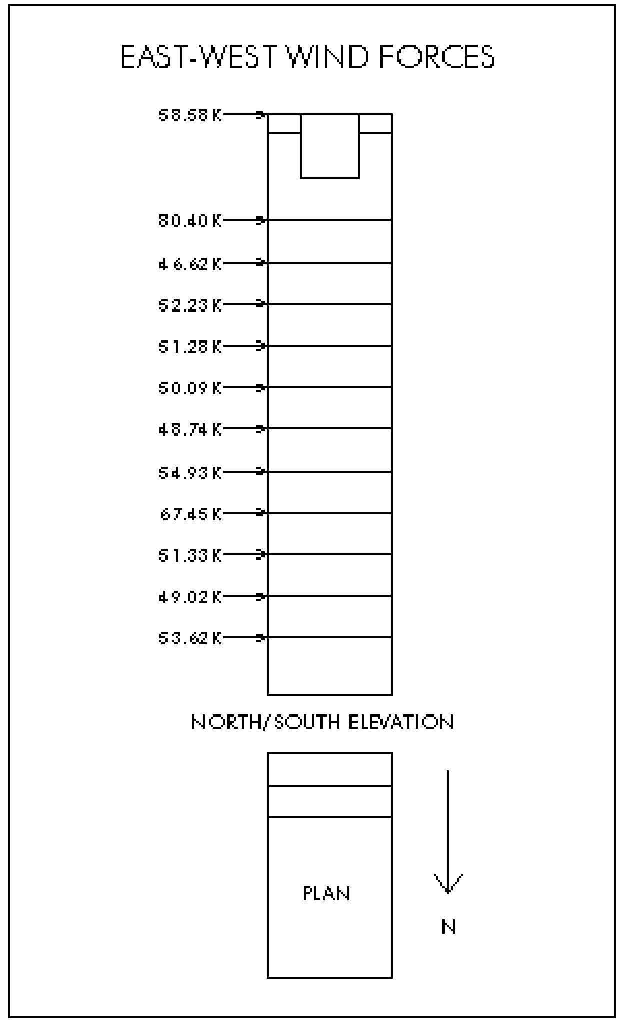
Wind Pressure Diagrams




EAST-WEST WIND PRESSURES
13.87 PSF
13.53 PSF
13.07 PSF
12.60 PSF
12.02 PSF
WINDWARD
11.45 PSF
11.10 PSF
10.75 PSF
10.29 PSF
9.83 PSF
9.37 PSF
8.79 PSF
8.09 PSF
7.63 PSF
7.17 PSF
6.59 PSF
LEEWARD
-8.47 PSF
NORTH/SOUTH ELEVATION
PLAN
N

Wind Force Diagrams





Seismic Loads

Assumptions

- ASCE 7-02, Chapter 9 was used to calculate the seismic loads for this building.

Building Information

- N-S Direction: Steel Braced Frames
- E-W Direction: Steel Braced Frames
- Location: Arlington, VA
- Building Use: Office (Primary), Retail (1st Level), Parking (Below Grade)

Seismic Design Category

Occupancy Category - II	(Table 1-1)
Seismic Use Group: 1	(Table 9.1.3)
Site Class C:	(Structural Notes)
Acceleration from Maps:	
$S_S = 0.190$	(Fig. 9.4.1.1a)
$S_1 = 0.070$	(Fig. 9.4.1.1b)
Adjust for Site Class:	
$Fa = 1.2$	(Table 9.4.1.2.4a)
$F_V = 1.7$	(Table 9.4.1.2.4b)

$S_{ms} = F_a S_s = 1.2(0.19) = 0.228$ (Eq. 9.4.1.2.4-1)
$S_{m1} = F_v S_1 = 1.7(0.07) = 0.119$ (Eq. 9.4.1.2.4-2)

Design Spectral Response Acceleration Parameters

$S_{DS} = 2/3\ S_{ms} = 2/3(0.228) = 0.152$ (Eq. 9.4.1.2.5-1)
$S_{D1} = 2/3\ S_{m1} = 2/3(0.119) = 0.0793$ (Eq. 9.4.1.2.5-2)

Seismic Design Category

(Table 9.4.2.1a)
S.D.C. based on short period response acceleration = S.D.C.-A

(Table 9.4.2.1b)
S.D.C. based on 1-sec. period response acceleration = **S.D.C.-B**

***S.D.C.-B is worst case**

NOTE: Building does not meet any plan or vertical irregularities as specified in Tables 1616.5.1.1 or 1616.5.1.2 of the IBC 2000, therefore it is still S.D.C.-B.

Equivalent Lateral Force Procedure can be used.

Seismic Base Shear (V=C_SW)

R = 3 (Table 9.5.2.2)
I = 1.0 (Table 9.1.4)
$T = C_t h_n^x$ (Eq. 9.5.5.3.2-1)
N-S: $T = C_t h_n^x = 0.02(180.75)^{0.75} = 0.986$ (Table 9.5.5.3.2)
E-W: $T = C_t h_n^x = 0.02(180.75)^{0.75} = 0.986$ (Table 9.5.5.3.2)

$$C_s = \frac{S_{DS}}{R/I} = \frac{0.152}{3/1} = 0.050667$$

$$C_{S,\max}(N-S) = \frac{S_{D1}}{T\left(\frac{R}{I}\right)} = \frac{0.0793}{0.986\left(\frac{3}{1}\right)} = 0.02681$$ *Controls

$$C_{S,\max}(E-W) = \frac{S_{D1}}{T\left(\frac{R}{I}\right)} = \frac{0.0793}{0.986\left(\frac{3}{1}\right)} = 0.02681$$ *Controls

$$C_{S,\min} = 0.044IS_{DS} = 0.044(1.0)(0.152) = 0.006688 < 0.02681 \therefore OK$$

Dead Loads

Roof Dead Load		
	Metal Deck	5 PSF
	Insulation	3 PSF
	Misc. DL	10 PSF
	Roofing	20 PSF
		38 PSF
Snow Load (need to include 20% Snow Load)		30 PSF (See Snow Load Calculations)
Typical Floor Load		
	3 ¼" lt. wt. slab on 3" metal deck	46 PSF
	Ponding of Concrete	10 PSF
	Misc. DL	15 PSF
	mech. ducts, plumbing, sprinklers, ceiling, etc.	71 PSF
Exterior Wall Loads		
	Glass Curtain Wall (N façade)	15 PSF
	Precast/Windows (S,E,W facades)	20 PSF

$$w_{roof} = 1026k$$
$$w_{11} = 1617k$$
$$w_{10} = 1512k$$
$$w_{9-6} = 1781k$$
$$w_{5-2} = 2050k$$
$$w_{1} = 2083k$$

$$W = w_{roof} + w_{11} + w_{10} + 4w_{9-6} + 4w_{5-2} + w_{1}$$
$$W = 1026k + 1617k = 1512k + 4(1781k) + 4(2050k) + 2083k$$
$$W = 21{,}562k$$

$V_{N\text{-}S}$ = 0.02681(21,562k) = 578.1k
$V_{E\text{-}W}$ = 0.02681(21,562k) = 578.1k

$$F_x = C_{vx}V$$
$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$

$$k(N-S)=1+\frac{0.986-0.5}{2}=1.243^{*}$$ *linear interpolation

$$k(E-W)=1+\frac{0.986-0.5}{2}=1.243^{*}$$ *linear interpolation

Seismic Base Shear and Overturning Moment

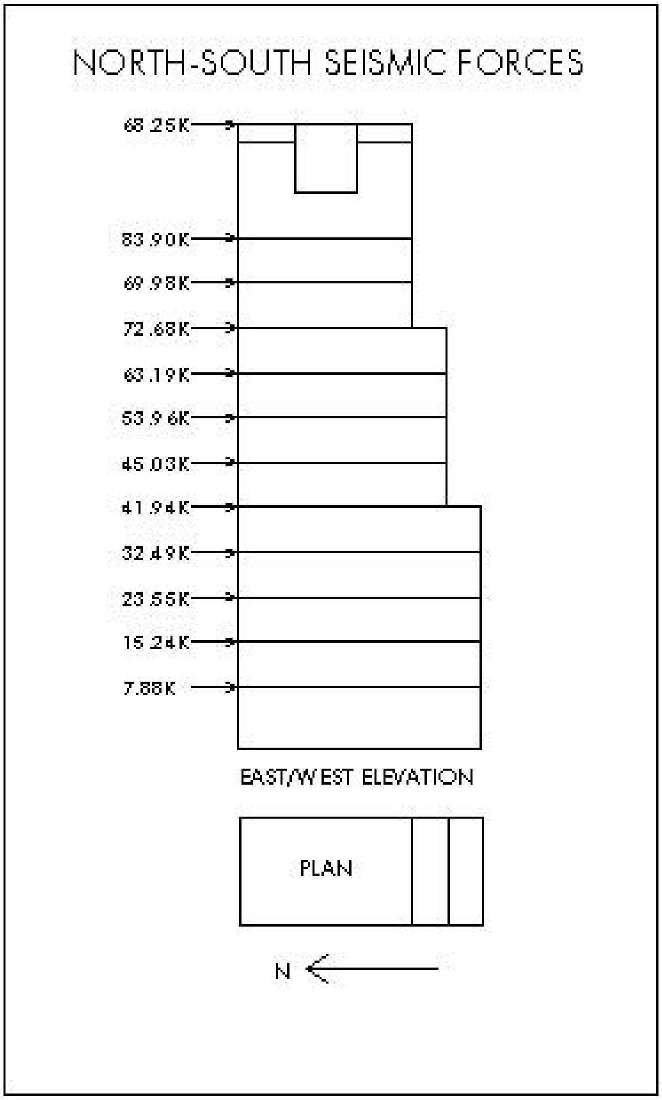
Level	w_x	h_x	$w_x h_x^{1.243}$	$w_x h_x^{1.243}$	C_{vx} (N-S)	C_{vx} (E-W)	F_x (N-S)	F_x (E-W)
12 (roof)	1026	180.75	655686	655686	0.118	0.118	68.25	68.25
11	1617	148	806019	806019	0.145	0.145	83.90	83.90
10	1512	135	672290	672290	0.121	0.121	69.98	69.98
9	1781	122	698247	698247	0.126	0.126	72.68	72.68
8	1781	109	606995	606995	0.109	0.109	63.19	63.19
7	1781	96	518355	518355	0.093	0.093	53.96	53.96
6	1781	83	432592	432592	0.078	0.078	45.03	45.03
5	2050	70	402912	402912	0.073	0.073	41.94	41.94
4	2050	57	312109	312109	0.056	0.056	32.49	32.49
3	2050	44	226238	226238	0.041	0.041	23.55	23.55
2	2050	31	146392	146392	0.026	0.026	15.24	15.24
1	2083	18	75682	75682	0.014	0.014	7.88	7.88
			5553516	5553516	1.000	1.000	578.10	578.10

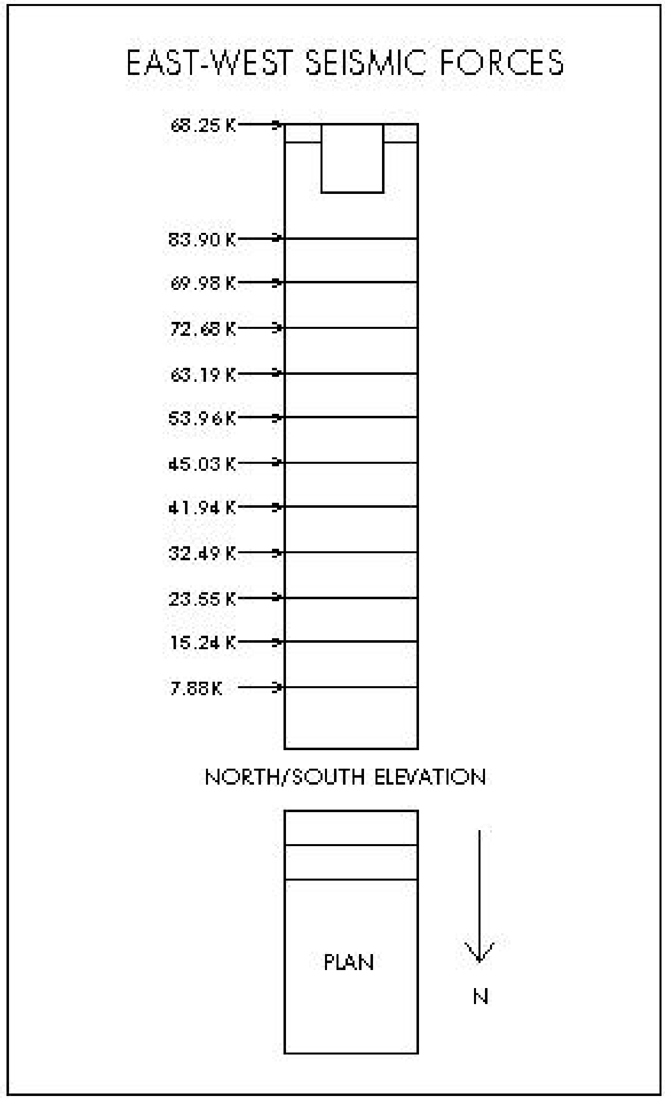
k (N-S)	1.243	
k (E-W)	1.243	
V (N-S)	578.1	k
V (E-W)	578.1	k

Base Shear		
N-S	578.10	k
E-W	578.10	k

Overturning Moment		
Overturning Moment (N-S)	65313.0733	ft-k
Overturning Moment (E-W)	65313.0733	ft-k

Seismic Force Diagrams





Snow Load

Assumptions

- ASCE 7-02, Chapter 7 was used to calculate the snow loads for this building.

Building Information

- Location: Arlington, VA
- Max. Roof Slope = 4.55% or 2.62° *

*Since the maximum roof slope is less than 5°, then ASCE 7-02, Chapter 7, Section 7-3 can be used.

$p_f = 0.7 C_e C_t I p_g$ (Eq. 7-1)

$C_e = 0.9$	Surface roughness B (6.5.6.2) Fully exposed (Table 7-2)
$C_t = 1.0$	(Table 7-3)
$I = 1.0$	Category II (Table 7-4)
$p_g = 25$ PSF	(Fig. 7-1)

$p_f = 0.7(0.9)(1.0)(1.0)(25PSF) = 15.75PSF$

$p_{f,\min} = 20PSF \cdot I$ p_g>20 PSF (Sec. 7-3)

$p_{f,\min} = 20PSF \cdot 1 = 20PSF$ 20 PSF > 15.75 PSF, therefore use 20 PSF

NOTE: Structural Notes specify a snow load value of 30 PSF.

Roof Live Load

Assumptions

- ASCE 7-02, Chapter 4 was used to check the minimum roof live load.

$L_r = 20R_1R_2$ (Eq. 4-2)

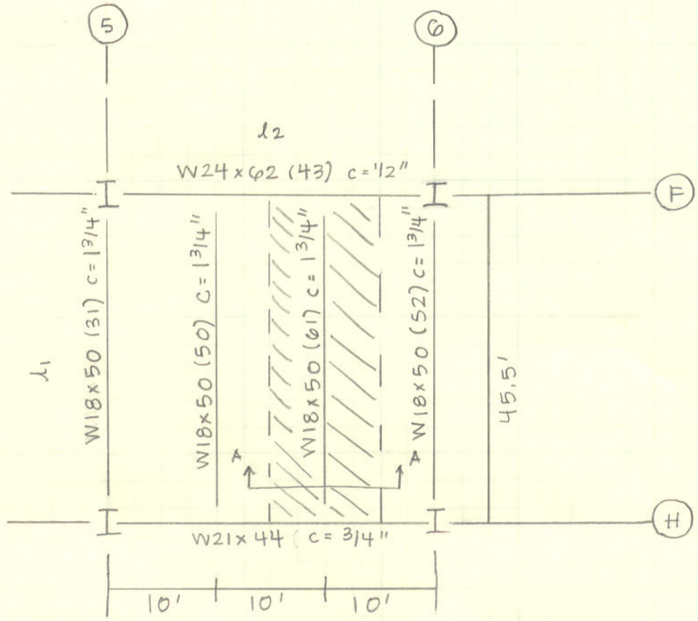
R2 = 1	F<4 (4.9.1)	Max. roof slope = 2.61°
R1= 0.6	A_t > 600 SF (4.9.1)	At(roof col.) = 30’[(46’+30’)/2] At(roof col.) = 1140 SF > 600 SF

$L_r = 20(1)(0.6)$
L_r = 12 PSF < 20 PSF and 30 PSF*
*Therefore snow load controls with a roof live load of 30 PSF.

Composite Beam Design Check

1

Composite Beam



Typical Bay (7th/8th Floor)

$f'_c = 3$ ksi

$f_y = 50$ ksi

Slab: 3 1/4" lt. wt. concrete on 3" metal deck = 6 1/4"

Shear studs - 3/4"

Loads:

Dead Loads

Slab	46 PSF
concrete ponding	10 PSF
Misc. DL	15 PSF
self wt. W18x50	50 PLF

Live Loads

Office	100 PSF* reduces to 74.72 PSF

* Check for live load reduction

ASCE 7-02 Sec. 4.8.1

$L_o = 100$ PSF
$A_T = 10'(45.5') = 455\ FT^2$
$K_{LL} = 2$ (int. beam) (Table 4-2)

$$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL} A_T}}\right) \quad \text{(Eq. 4-1)}$$

$$L = 100\ PSF\left(0.25 + \frac{15}{\sqrt{2(455)}}\right)$$

$L = 74.72$ PSF $> 0.5(100$ PSF$) = 50$ PSF $\therefore$ OK

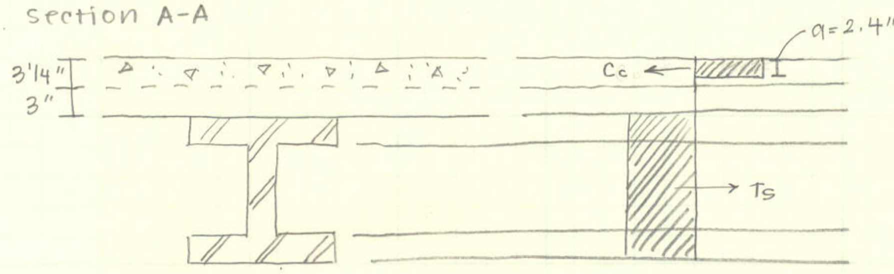
b_{eff} = spacing = 10'(12"/FT) = 120" * smaller ∴ controls
b_{eff} = $l_1/4$ = 45.5'/4 (12"/FT) = 136.5"

b_{eff} = 120"

W18x50 Properties:

$A = 14.7\ in^2$
$d = 18"$

Section A-A



Shear Studs

3/4" ϕ shear studs
$f'c = 3$ ksi
lt. wt. concrete

$Q_n = 17.7K$ (Table 5-13 AISC LRFD manual)

$\Sigma Q_n = 17.7K(61 \text{ studs})$
$= 1079.7K$

$C_c = A_s(0.85)f'c = 120''(3.25'')(0.85)(3\text{ksi}) = 994.5K$

$T_s = A_s f_y = 14.7 \text{ in}^2 (50\text{ksi}) = 735K$

P.N.A. is in concrete – steel controls

$\Sigma Q_n < C_c$ $\therefore$ fully composite
$< T_s$

$a = \dfrac{735K}{0.85(3\text{ksi})(120'')}$
$a = 2.4''$

$M_p = 735K(18''/2) + 530K(6.25'' - 2.4''/2)$
$M_p = 9291.5$ in·K
$M_p = 774.3$ ft·K

$\phi M_n = 0.85(774.3 \text{ FT·K})$
$\boxed{\phi M_n = 658.2 \text{ FT·K}}$

4

Factored Moment, Mu

DL:	71 PSF (10')	= 710 PLF
Self wt:		= 50 PLF
LL:	74.72 (10')	= 747.2 PLF

$W_u = 1.2(710 + 50) + 1.6(747.2)$
$W_u = 2107.5$ PLF
$W_u = 2.11$ KLF

$M_u = \frac{W_u l^2}{8} = \frac{2.11 \text{ KLF} (45.5')^2}{8}$

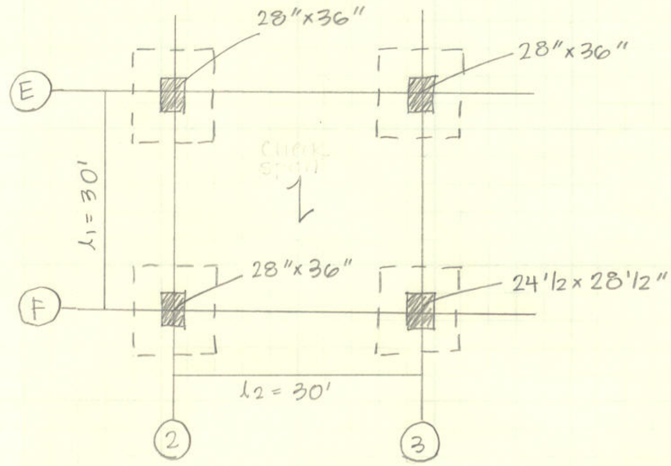
$\boxed{M_u = 546.1 \text{ FT}\cdot\text{K}}$

$\phi M_n = 658.2$ FT > 546.1 FT·K $= M_u$ ∴ composite beam design OK

Plaza Slab Check

1

Plaza slab



check slab in E-W direction of interior panel

12" slab

drop panels 10'x10'x12"

$f'_c = 4$ ksi

$f_y = 60$ ksi

$\ell_1 = 30'$

$\ell_2 = 30'$

$\ell_n = 30' - \frac{36"}{12} = 27'$

Check min. thickness:

ACI Table 9.5(c)

$f_y = 60$ ksi
w/ panels
int. panel $\Rightarrow t_{min} = \frac{\ell_n}{36}$

$\frac{\ell_n}{36} = \frac{27'}{36} \cdot 12"/\text{FT} = 9" < 12"$ ∴ slab meets minimum thickness requirements

DL:

Self: 1 FT (150 PCF) = 150 PSF
SDL: = 10 PSF
Panel self wt. distributed over bay = 17 PSF
177 PSF

LL:

Plaza = 350 PSF (Fire truck loading)

ASCE 7-02 LL Reduction (Sec. 4.8.1)

L_o = 350 PSF
A_T = 30'(30') = 900 FT^2
K_{LL} = 1 (slab)

$L = 350\ PSF\left(0.25 + \frac{15}{\sqrt{900}}\right)$

L = 263 PSF > 0.5(350) = 177 PSF ∴ OK

Direct Design Method

$W_u = 1.2(177) + 1.6(263)$
W_u = 633 PSF

$M_o = \frac{W_u l_2 l_n^2}{8}$ (ACI 13-3)

$M_o = \frac{633\ PSF(30')(27')^2}{8}$

M_o = 1731 K·FT

		Mu(FT·K)	width	Mu/FT
Int. supp. (−) 65% Mo = 1125 K	CS (75%)	844	15'	56.3
	MS (25%)	282	15'	18.8
Midspan (+) 35% Mo = 606 K	CS (60%)	364	15'	24.3
	MS (40%)	243	15'	16.2

CS width = $l/4 = \frac{30'}{4}$ = 7.5'

$A_{s,min}$ = 0.0018(12")(12"/FT) = 0.26 in^2/FT

Int. Support Col. Strip

Specified Reinforcement = 26 #8

$\frac{26}{15} = 1.73$ bars/FT

$A_{\#8} = 0.79\ in^2$

$A_s = 0.79(1.73)$
$= 1.37 > 0.26 \therefore OK$

$a = \frac{A_s f_y}{0.85 f'_c b}$

$a = \frac{1.73(0.79in^2)(60KSi)}{0.85(4)(12")}$

$a = 2.01"$

← includes drop panel thickness

$d = 24" - 0.75" - 1" - 1"/2$
(cover) (d #8)

$d = 21.75"$

$\phi M_n = 0.9 A_s f_y (d - a/2)$

$\phi M_n = 0.9(1.73)(0.79)(60)(21.75 - 2.01/2)$

$\phi M_n = 1531$

$\phi M_n = 127.6$ FT·K/FT $> 56.3 \therefore$ 26 #8 bars OK for int. supp. CS

Int. Support MS

Specified Reinforcement: 20 #7

$\frac{20}{15'} = 1.33$ bars/FT

$A_{\#7} = 0.6\ in^2$

$A_s = 1.33(0.6) = 0.8 > 0.26 \therefore OK$

$a = \frac{1.33(0.6)(60)}{0.85(4)(12)}$

$a = 1.18"$

$d = 12 - 0.75 - 7/8 - 7/16$
(cover)

$d = 9.94"$

$\phi M_n = 0.9(1.33)(0.6)(60)(9.94 - 1.18/2)$

$\phi M_n = 403$ in·K/FT

$\phi M_n = 33.6$ FT·K/FT $> 18.8 \therefore$ OK 20 #7 bars OK for int. supp. MS

Midspan CS & MS

Specified Reinforcement $\#6 @ 12''$

$a = \frac{0.44(60)}{0.85(4)(12)}$ $A_{\#6} = 0.44 > 0.26$ $\therefore$ OK

$a = 0.65''$

$d = 12'' - 0.75 - 0.75 - 0.75/2$
(cover) (d #6) (d#6/2)

$d = 10.125''$

$\phi Mn = 0.9(0.44)(60)(10.125 - 0.65/2)$

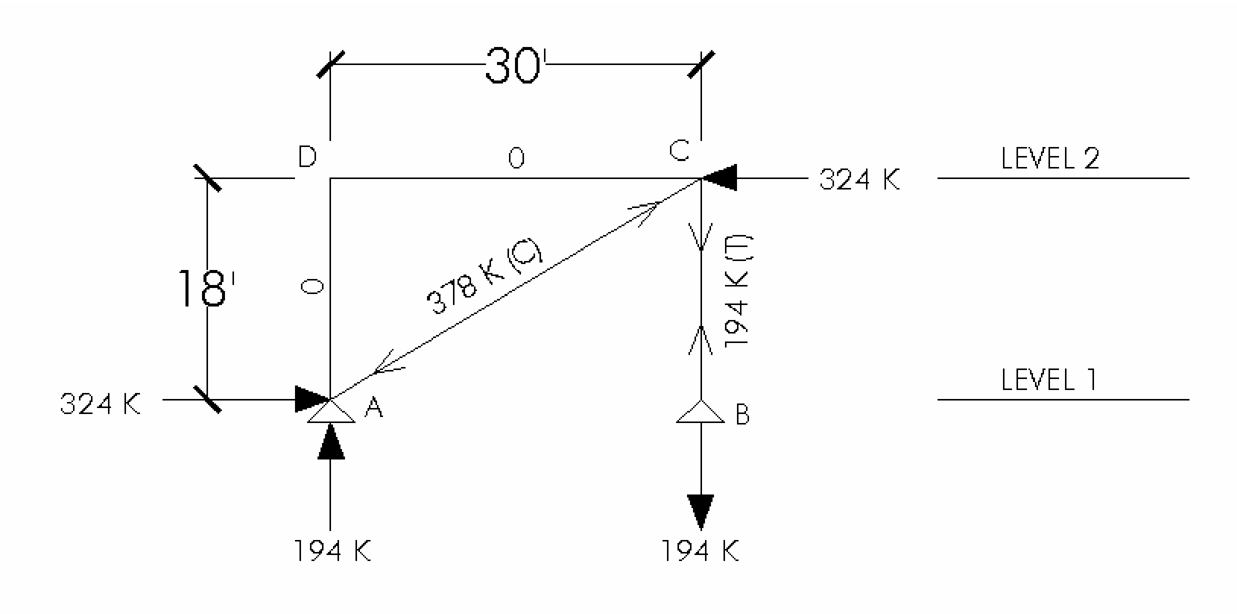
$\phi Mn = 19.4$ FT·K/FT < 24.30 $\therefore$ NOT OK for CS

> 16.20 $\therefore$ OK for MS

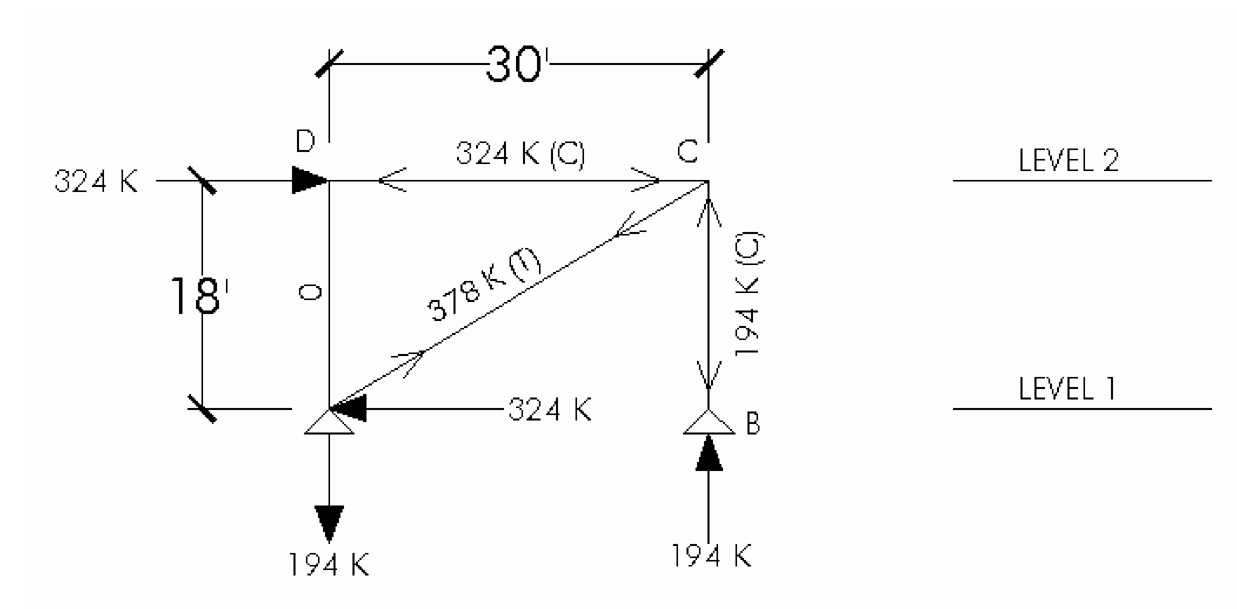
Lateral Member Check

The bottom diagonal bracing member for Frame #4 was checked for the base shear due to the worst case lateral loadings. The factored base shear is 324 K.

Wind from the North



Wind from the South



The force in the diagonal member is 378 k in either tension or compression depending on which way the lateral force is acting. Since the steel is specified to be 50 ksi, the required area of steel can be calculated, and a member selected. Since all of the braced frames are using HSS 10 x 10, 8 x 8 or 12 x 12 members, with HSS 10 x 10's being used most often, an HSS 10 x 10 member was selected and compared to the actual designed member.

$$A_{steel,req'd} = \frac{378k}{50ksi} = 7.56in^2$$

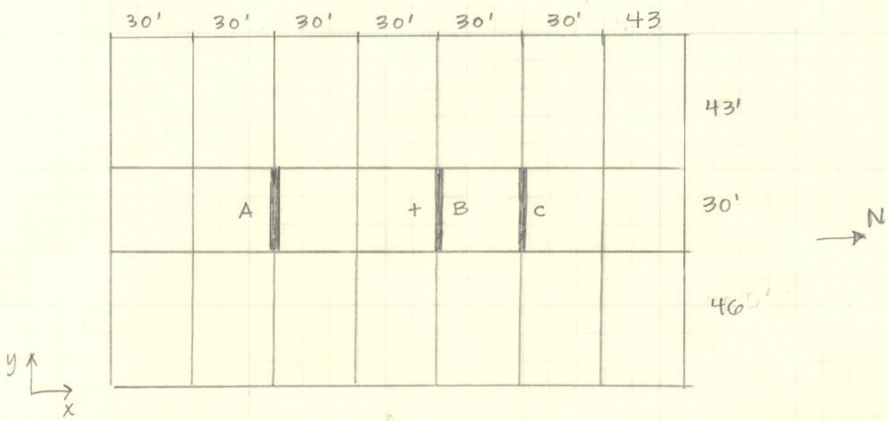
From Table 1-11 from the AISC's Manual of Steel Construction, an HSS 10 x 10 x ¼ was selected, with an area of 8.96 in^2, which is greater than 7.56 in^2, and therefore should be OK.

The actual member size is an HSS 10 x 10 x 5/8 with an area of steel of 21 in^2. In reviewing the compression and tensile forces listed with the actual member in the braced frame elevation, the diagonal member is to have calculated factored forces of 378 k (C) and 295 k (T). My calculated values were equal to or greater than the forces designed for. Since a thickness of ¼" meets the stress requirements, a 5/8" was probably chosen based off of other structural calculations and is considered a more conservative section. The member may also have had to meet minimum thickness requirements because it is a critical member in the braced frame.

Lateral Force Distribution – Distribution by Rigidity

1

Lateral Force Distribution (E-W loading)



Assumptions:

- Elevator shafts and stair openings are negligible
- Building is assumed rectangular in plan
- Shear walls are approximately equal stiffness
- Lateral Load Distribution Procedure - Distribution by Rigidity used even though there are more than 7 stories

Step 1:

CM coordinates: (111.5, 59.5)

Step 2:

$\Delta_{cant} = \dfrac{Pl^3}{3EI}$ $\quad$ EI = constant

$\Delta_A = \dfrac{Pl^3}{3EI}$ $\quad K_A = \dfrac{1}{\Delta} = 1$

$\Delta_B = \dfrac{Pl^3}{3EI}$ $\quad K_B = \dfrac{1}{\Delta} = 1$

$\Delta_C = \dfrac{Pl^3}{3EI}$ $\quad K_C = \dfrac{1}{\Delta} = 1$

Step 3:

$X_A = 60$
$X_B = 120$
$X_C = 150$

$X_{CR} = \dfrac{\Sigma K_i X_i}{\Sigma K_i} = \dfrac{1(60) + 1(120) + 1(150)}{3} = 110$

$Y_{CR} = 0$

Step 4: Determine eccentricities

$e_x = 111.5 - 110 = 1.5$
$e_y = 59.5 - 0 = 59.5$

Step 5: Determine torsional moment

$M_t = 93.72K (1.5)$
$M_t = 140.5$ FT·K

Step 6: origin at CR

$d_A = -50$
$d_B = +10$
$d_C = +40$

Step 7:

$J = \Sigma(d_i K_i)$

$J = (-50)^2(1) + (10)^2(1) + (40)^2(1)$
$J = 4200$ FT^3

Step 8: N/A

Step 9: $F_{A,dir} = \frac{1}{3}(93.72K) = 31.24K$

$F_{B,dir} = \frac{1}{3}(93.72K) = 31.24K$

$F_{C,dir} = \frac{1}{3}(93.72K) = 31.24K$

Step 10:

$$F_{i,tor} = \frac{K_i d_i M_t}{J}$$

$$F_{A,tor} = \frac{1(-50)(140.5)}{4200}$$
$$= -1.67\ K$$

$$F_{B,tor} = \frac{1(10)(140.5)}{4200}$$
$$= 0.33\ K$$

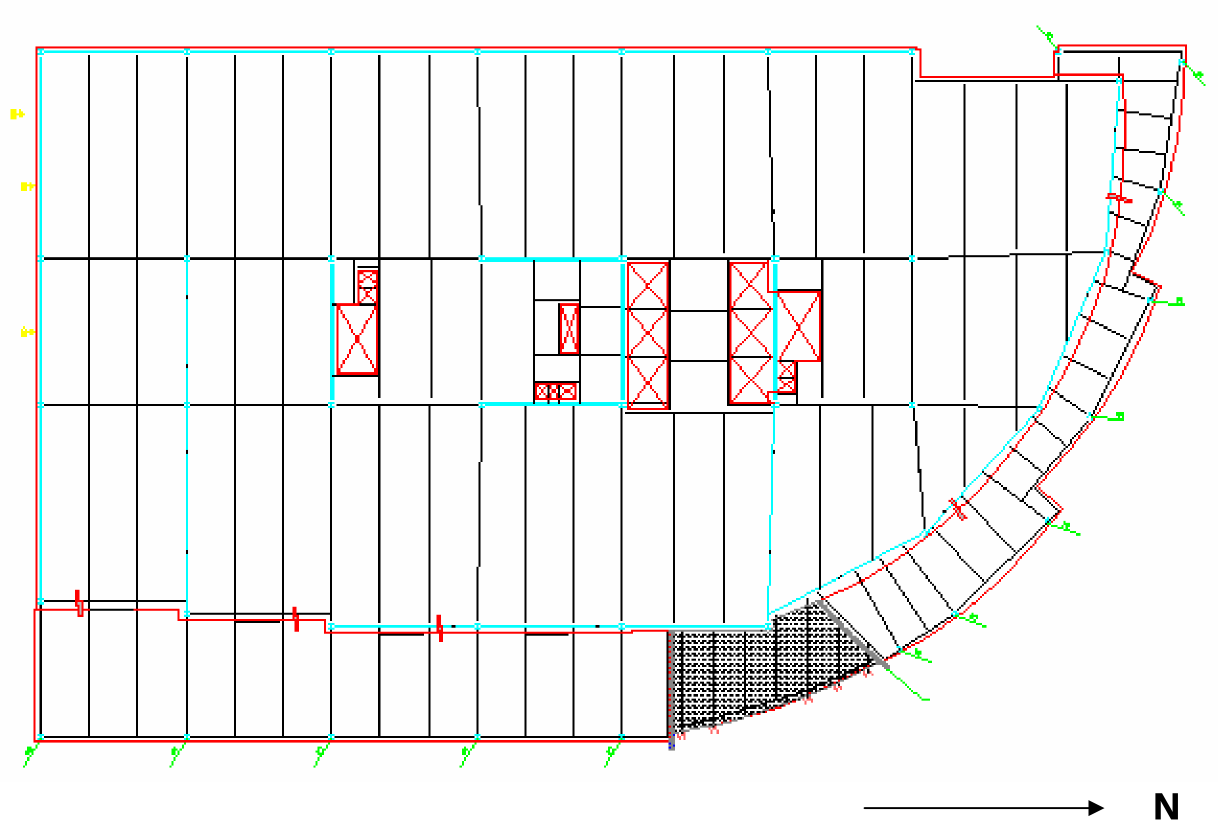
$$F_{C,tor} = \frac{1(40)(140.5)}{4200}$$
$$= 1.33\ K$$

Step 11:

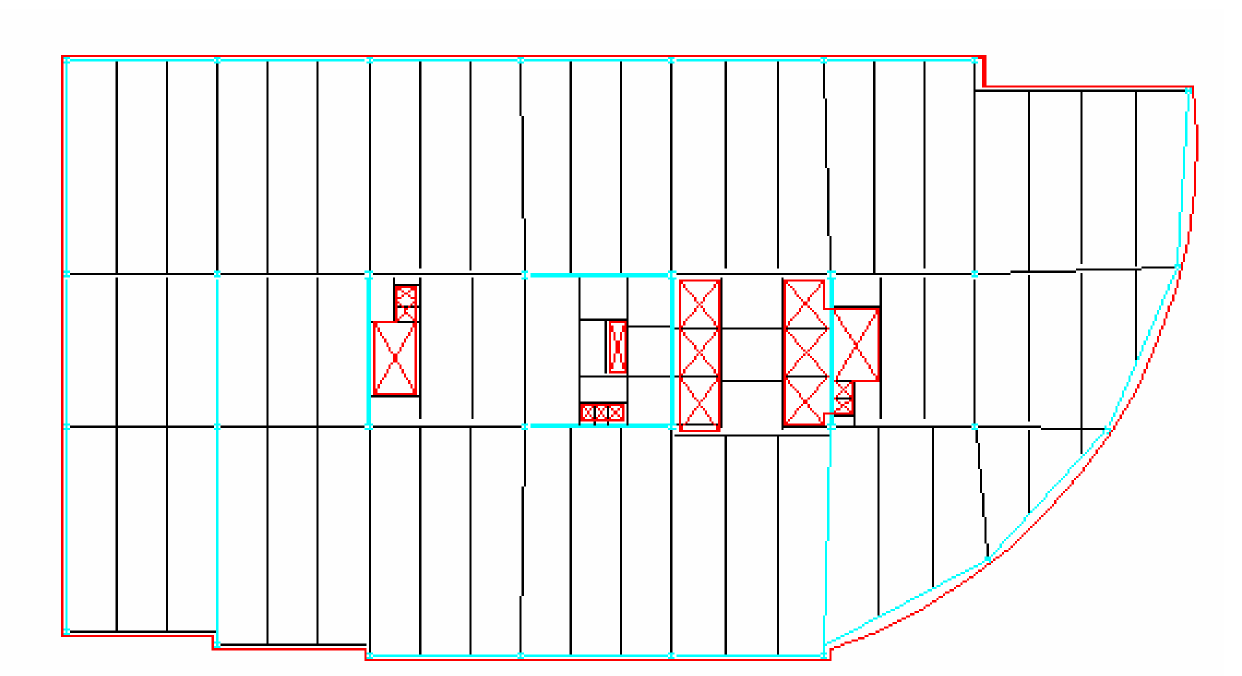
ignore if (-)

	Fdir	Ftor	FTOT
A	31.24	-1.67	31.24 K
B	31.24	0.33	31.57 K
C	31.24	1.33	32.57 K

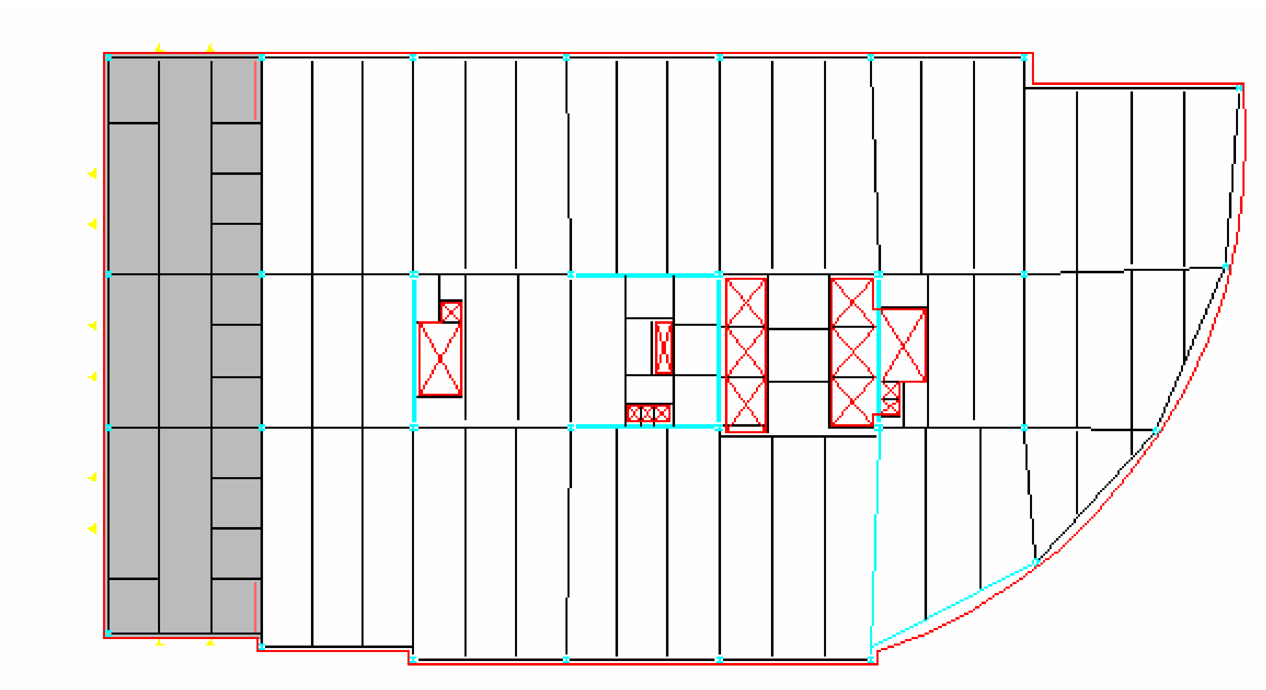
2nd Floor Faming Plan



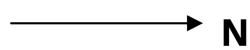
3rd – 5th Floor Framing Plan



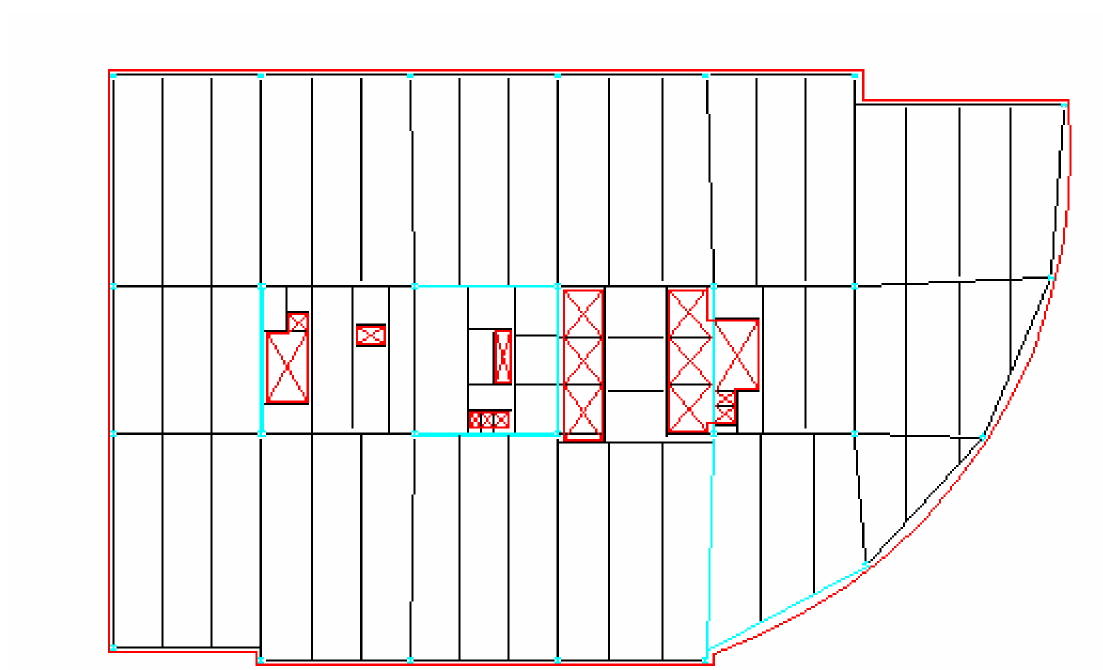
6th Floor Framing Plan



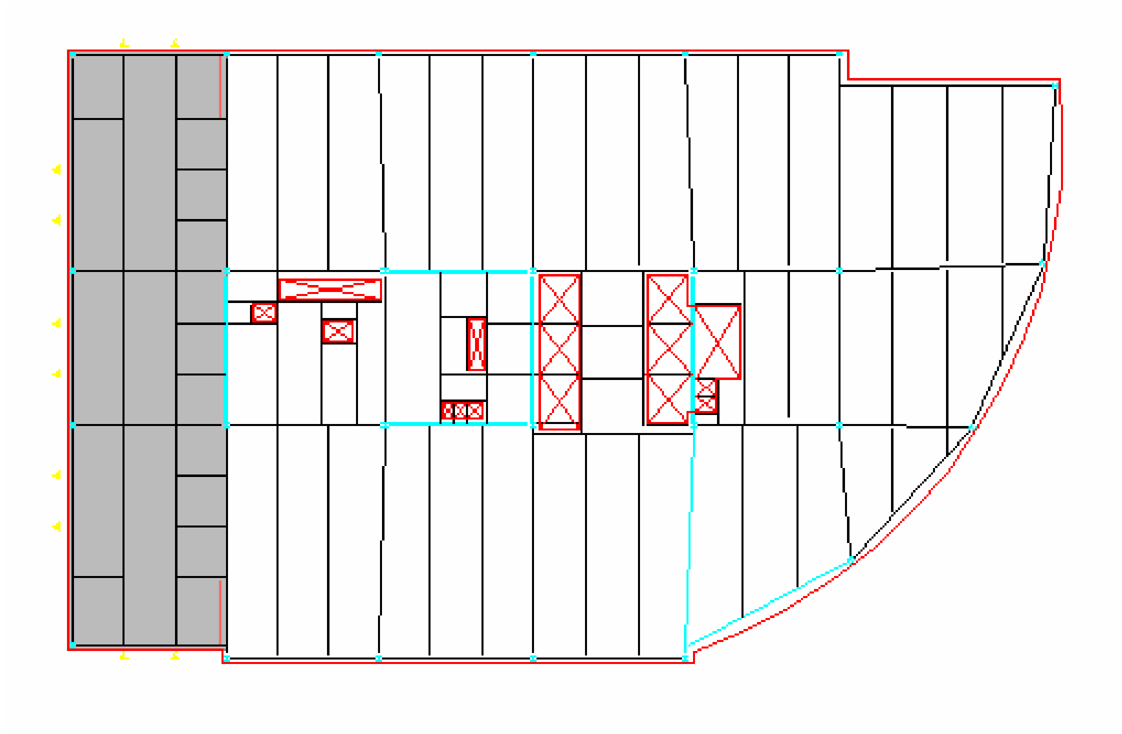
Note: Shaded area is roof construction



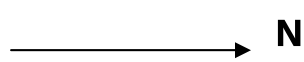
7-9th Floor Framing Plan



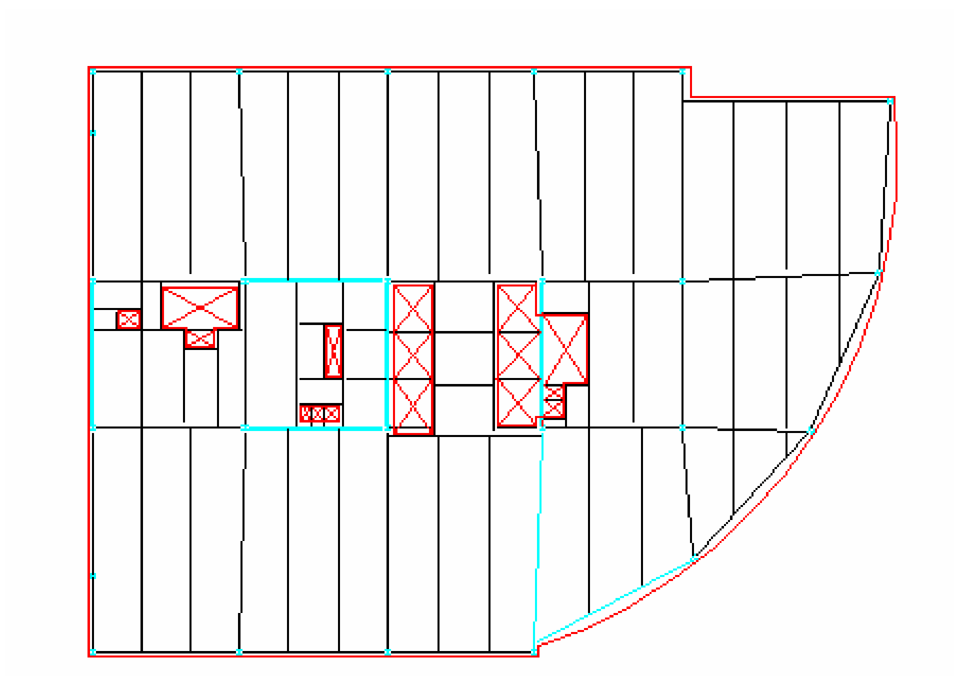
10th Floor Framing Plan



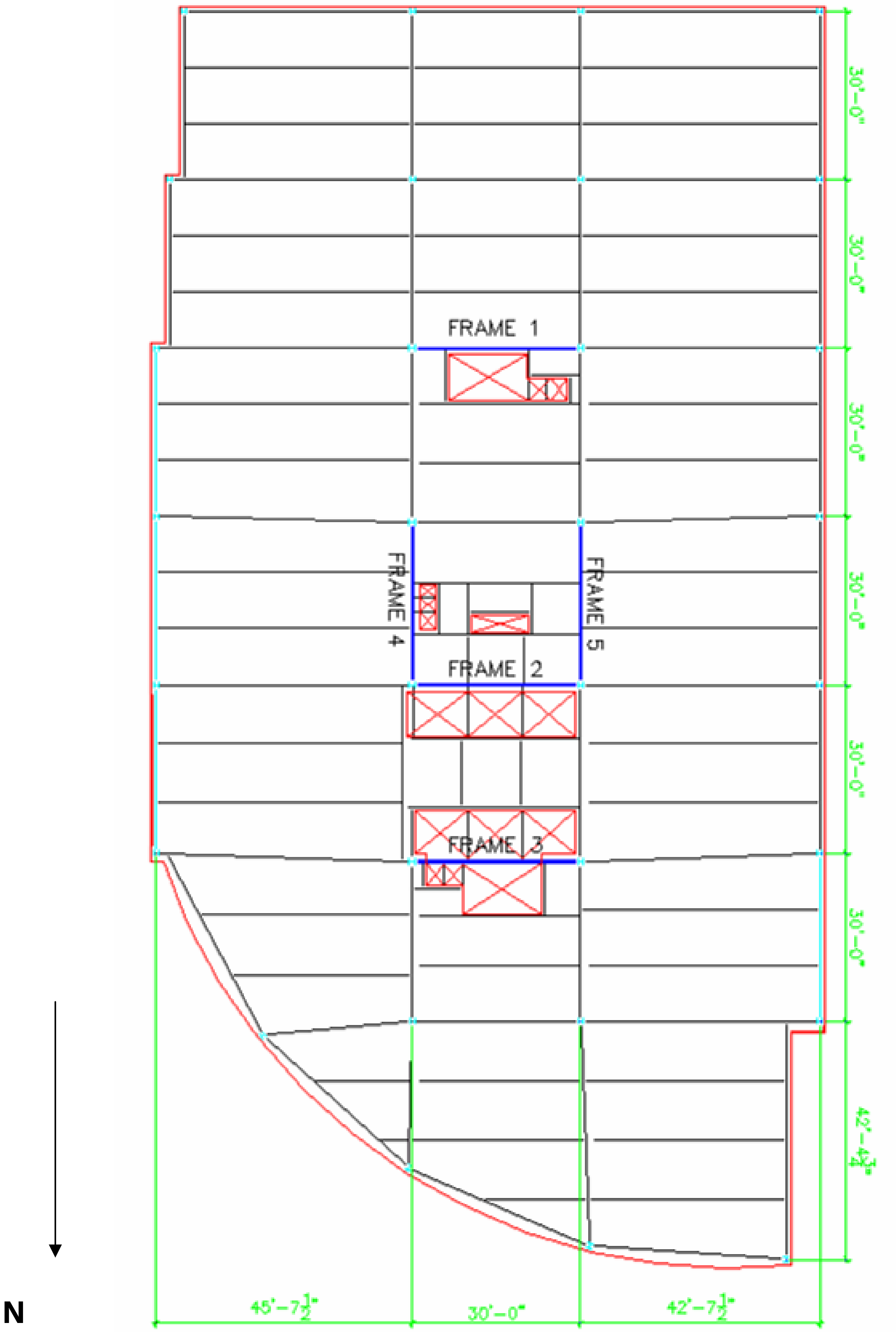
Note: Shaded area is roof construction



11th and 12th Floor Framing Plan



Enlarged Typical Framing Plan with Dimensions



Concrete Column and Wall Layout for the Parking Levels Below Grade