The Regent

950 N. Glebe Road
Arlington, VA

Architect: Cooper Carry Architects

Structural Technical Report 3
Lateral System Analysis and Confirmation Design

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Option: Structural
Date: November 21, 2005
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Executive Summary

The Regent is a 12-story office building located at 950 North Glebe Road in Arlington, VA. There is retail space on the first floor and a 3-level concrete parking garage below grade. The Regent is designed to a maximum allowable height of 176'.

The gravity framing system for the tower consists of a steel superstructure. The flooring system includes a 6 ¼" slab on metal deck. Shear studs provide the composite action between the slab on deck and the composite steel beams. Typical bays are 30' x 30' and 43'-46' x 30'.

The lateral system consists of five braced frames centrally located around the core of the building. There are two braced frames that resist the north / south lateral forces and three braced frames that resist the east / west lateral forces.

This report will focus on the lateral system analysis and confirmation of design. The lateral loads considered for this report are wind and seismic forces. Based on the load combinations of ASCE7-02, wind is the controlling lateral force in the east / west direction and in the north / south direction, seismic controls from the Roof level down to and including Level 6, and wind controls from Level 5 to Level 2.

The controlling lateral loads are distributed to the five braced frames based on the Lateral Load Distribution Procedure – Distribution by Rigidity. This method of lateral load distribution takes into account the relative stiffness of each braced frame and any torsional effects due to the braced frame configuration and the changing center of mass for each floor up through the building.

After the lateral loads were appropriately distributed to each braced frame, computer models were produced using ETABS in order to help analyze the lateral framing system through the calculation of member design checks, drifts, story drifts, and axial member forces. There were two types of computer models produced. The first series of computer models, referred to as the Single Frame models, analyzes each frame individually where as the second computer model, referred to as the Whole Building model, includes all five braced frames connected to rigid floor diaphragms.

Throughout this report, the results of the computer analyses, hand calculations, and the existing design and design loads are compared in order fully understand and analyze the lateral force resisting system and to confirm the lateral framing system design.

This report includes lateral member design checks, including a detailed study of critical diagonal bracing members, a check of building drift and story drift in comparison to industry standards L/400 and L/360, and a check of the building's resistance to the overturning moments induced by the lateral loads.

The bottom diagonal bracing members for Frames #2 and #3 were checked for strength. In comparing the results of all the analyses, it was determined that the existing designed

members should be adequate for strength, however the computer analyses for the diagonal member check of Frame #2, found that the diagonal member was not adequate for strength. Since the calculated loads were similar in magnitude across all of the analyses, it was determined that the computer models may not be an exact representation of the lateral framing system. The models need to be reviewed further in order to figure out why the results show they are not correctly designed for strength even though the loads match the other analyses which prove that under the applied loads, the diagonal member is adequate.

All of the other frames were checked for strength in both computer models. It was determined that most of the members met the strength requirements when analyzed as a single frame, however there were more members that did not meet the strength requirements when analyzed as part of the whole lateral force resisting system. The model of the whole lateral force resisting system more closely represents the actual building design and actual configuration. Since several of the members were not meeting the design strengths, this could be an indication of several concerns:

1. The loads applied are similar to the loads applied for the existing design, but the model is not an accurate representation of the existing design
2. The loads applied are more conservative than those assumed for the existing design which are resulting in a lot of the members not meeting the design strength check
3. The members may not be conservatively designed

The initial conclusion is that the computer models are not an exact representation of the lateral force resisting system as it was designed and the models will be corrected or reviewed further in order to make sure that they are accurately representing the existing lateral force resisting system.

The system was then checked for drift and story drift according to the industry standards of L/400 and L/360. The results of the Whole Building model analysis show that the top of the building displaces approximately 7" in the north / south direction and approximately 4" in the east / west direction. According to industry standards, the top of the building is allowed to drift a total of 5.28" to meet L/400 deflection limits and 5.87" to meet L/360 deflection limits. In the north / south direction the building exceeds both of the deflection limits by over 1". In the east / west direction, the displacement of the top of the building meets both of the deflection limits by under and 1". Therefore, according to the results of the Whole Building model analysis, the building drift is okay in the east / west direction, but does not meet the industry standard deflection limits in the north / south direction for the entire building displacement.

The average story drift for the Whole Building model in the north / south direction is approximately 0.6" per story. The average story drift in the east / west direction is approximately 0.35". For the 13' high stories, the L/400 and L/360 deflection limits are 0.39" and 0.43", respectively. For the 18' high story, the L/400 and L/360 deflection limits are 0.54" and 0.6", respectively. The story displacements in the north / south direction are exceeded by approximately 0.2” per story. In the east / west direction, the average story drift meets the L/400 and L/360 story drift limitations.

Since the deflection limits are not met in the north / south direction, this is an indication that the calculated applied lateral loads in the north / south are higher than the actual lateral loads designed for, the computer model is not accurately representing the lateral force resisting system, or L/300 was an acceptable deflection limit for the design in this direction.

The results of the Single Frame model drift and story drift calculations concluded that if each frame is analyzed separately for frame displacements, all of the frames fail to meet the story drift limitations of L/400 and L/360 and only Frame #4 meets L/360 building deflection limit, while the other four frames do not meet any of the industry standards for total building drift.

The overturning moments for The Regent were calculated based off of the controlling lateral force distributed to each braced frame. The moments due to the self weight of the building were much greater than the overturning moments in all cases. Therefore, the building is able to resist the overturning moments induced by the lateral forces.

In conclusion, the lateral system and confirmation of design analyses performed for this report concluded that the building, as analyzed, does not meet all strength, drift, and story drift requirements. This is an indication that the critical load path for distribution of the designed structure does not match the analyses performed in this report. Further research and analyses will determine where and why the critical load paths do not match up. The computer models will be revised to more accurately represent the existing designed structure in order to be able to determine if the designed lateral system is adequate for the calculated loads. It is also a possibility that the calculated lateral loads used in all of the computer models and hand calculations are more conservative than those used in the actual design of the lateral load resisting system or that the lateral loads were not distributed properly among all of the braced frames. Further research and analysis will determine the accuracy of the computer models, the accuracy of the calculated applied lateral loads, and the accuracy of the distribution of the lateral loads to each lateral load resisting element. The results of all methods of analysis need to coincide so that it can be assumed that the critical load paths of the existing system match the critical loads paths developed through this series of technical reports.

Introduction

The Regent is located at 950 North Glebe Road in Arlington, Virginia. The building is a 12-story spec office building with retail space on the first level. There is also a 3-story parking garage below grade. The building is designed to a maximum allowable height of 176 feet.

Gravity Framing System Description

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 are spaced at 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". Headed shear studs, ¾" in diameter and 5" in length, allow for composite action between the slab on deck and the supporting beams.

There is an elevator core running up the center of the building and through the center of each floor. The roof deck construction is 3" x 22 gage, deep rib, type N, painted roof deck.

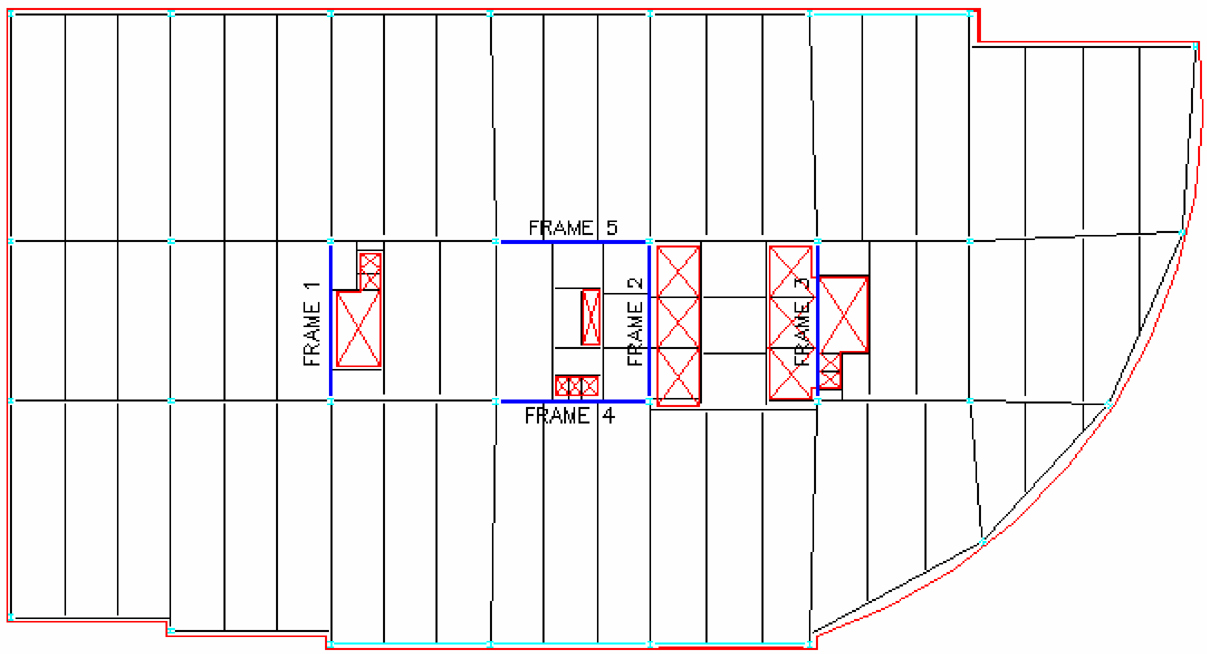
Lateral System Description

The lateral load resisting system for The Regent consists of five braced frames at the core of the building. There are two braced frames, Frame #4 and Frame #5, that span along the building's north / south axis, and three braced frames, Frame #1, Frame #2, and Frame #3, that span along the building's east / west axis. Frame #1, Frame #3, and Frame #5 have chevron style bracing and Frame #2 and Frame #4 have single diagonal bracing. 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 columns in the braced frames 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.

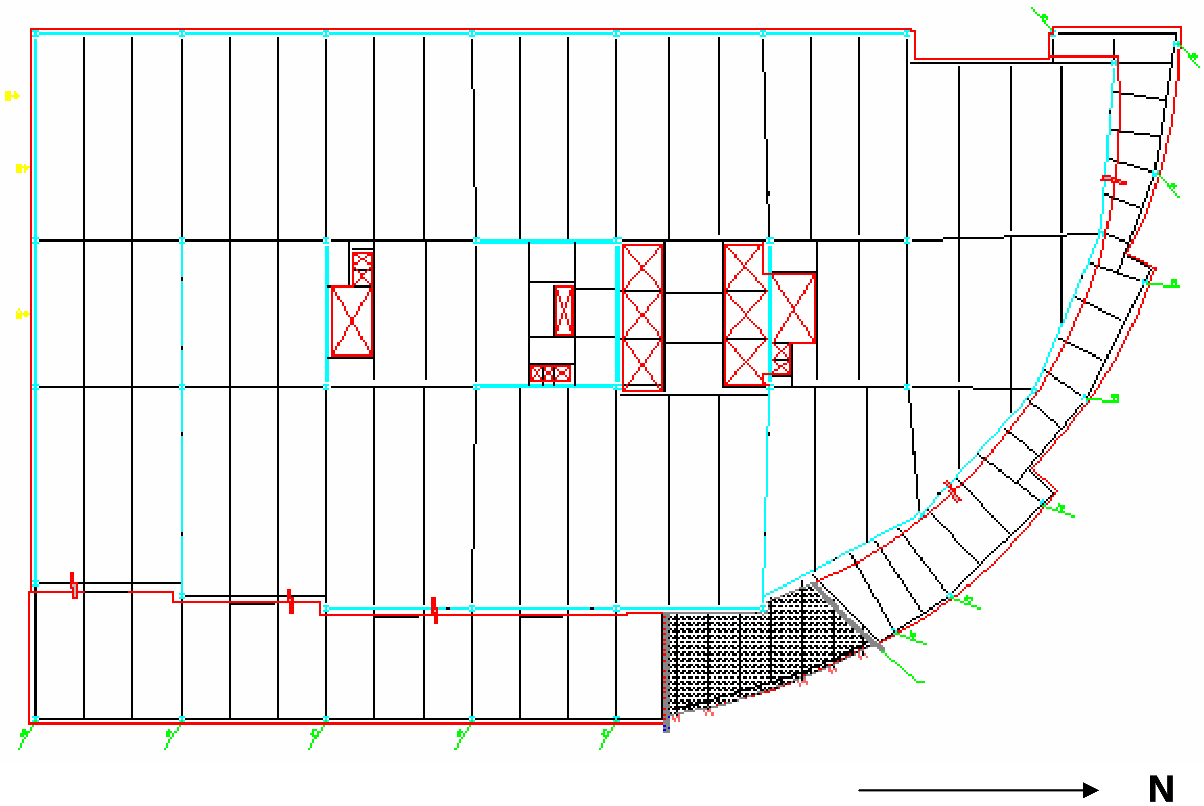
Braced Frame Location Plan

N

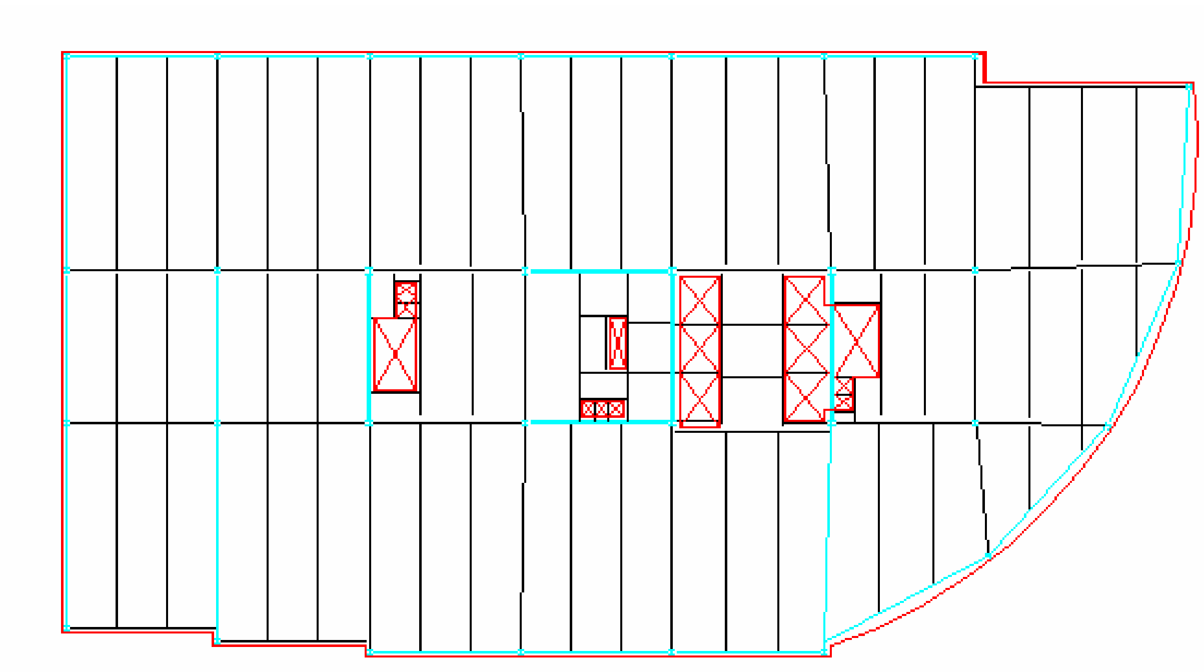


Typical Framing Plans and Elevations

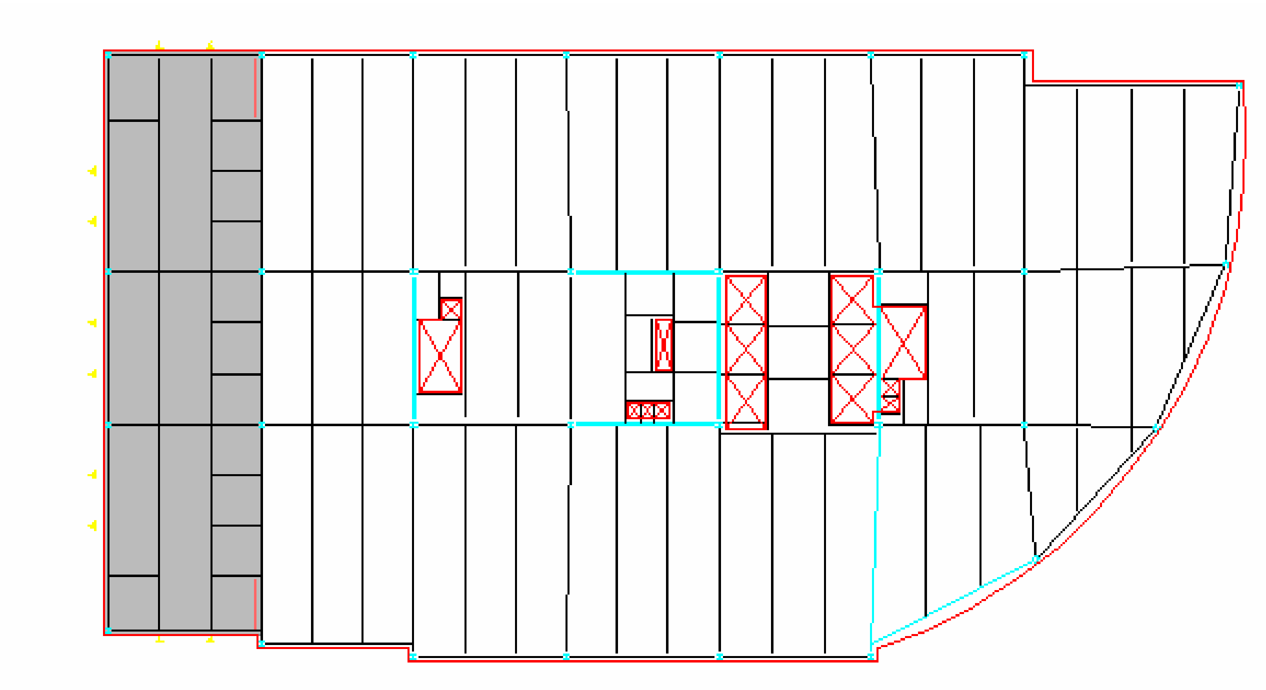
2nd Floor Faming Plan



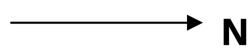
3rd – 5th Floor Framing Plan



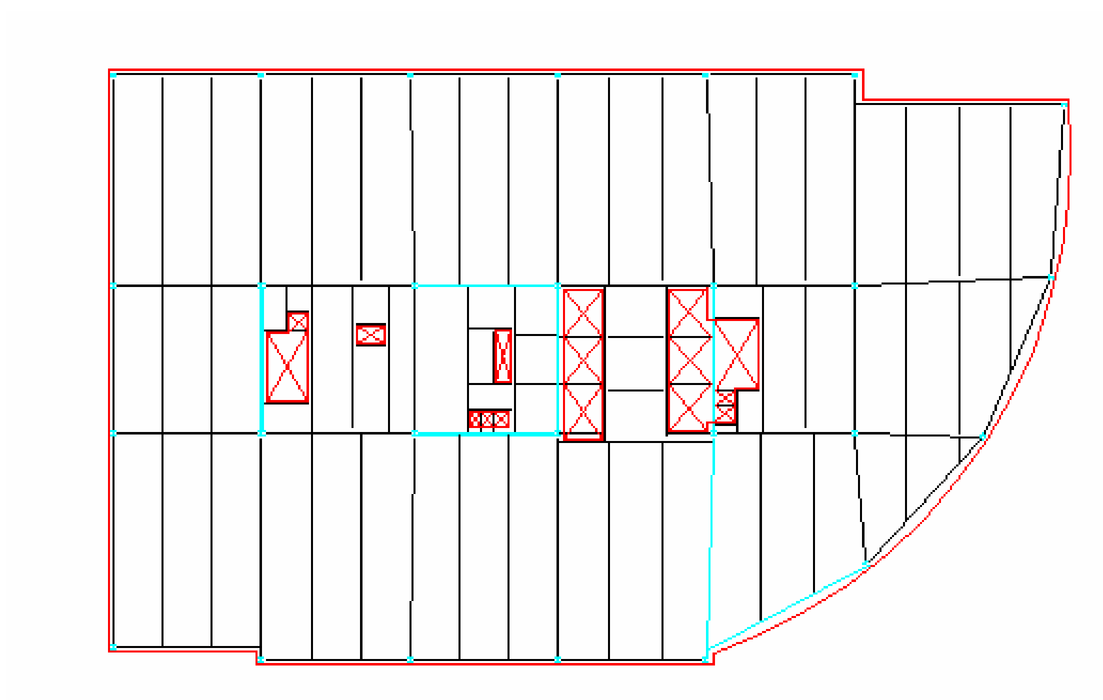
6th Floor Framing Plan



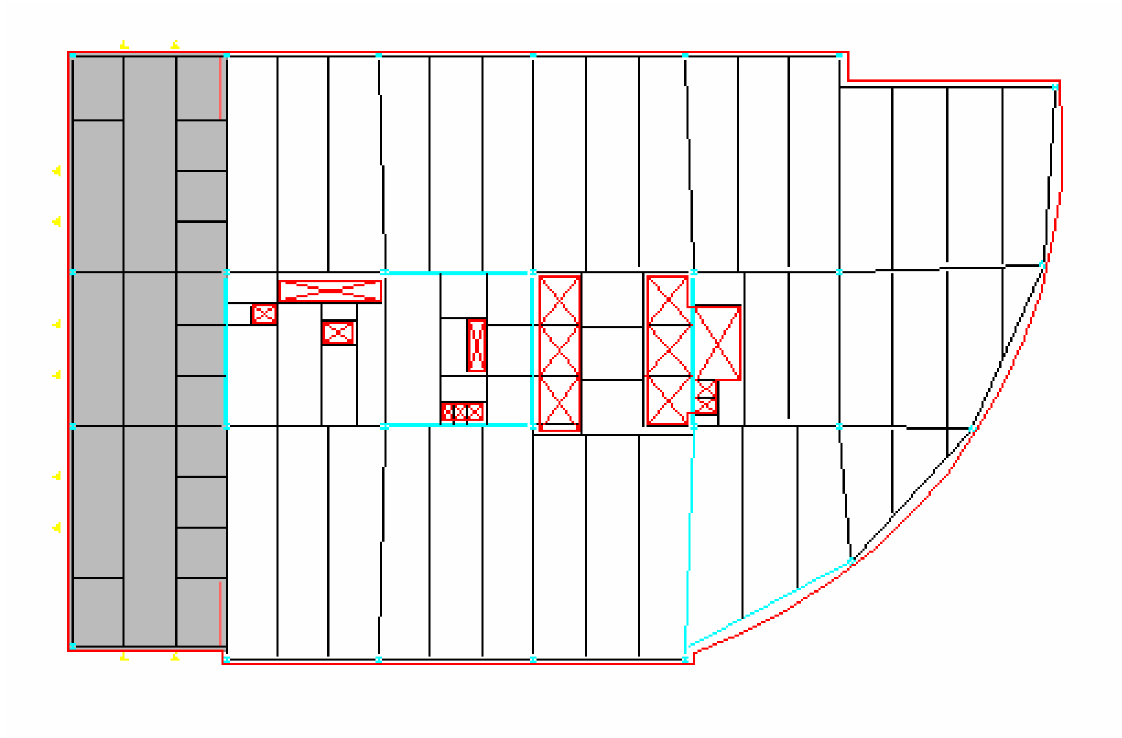
Note: Shaded area is roof construction



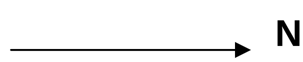
7-9th Floor Framing Plan



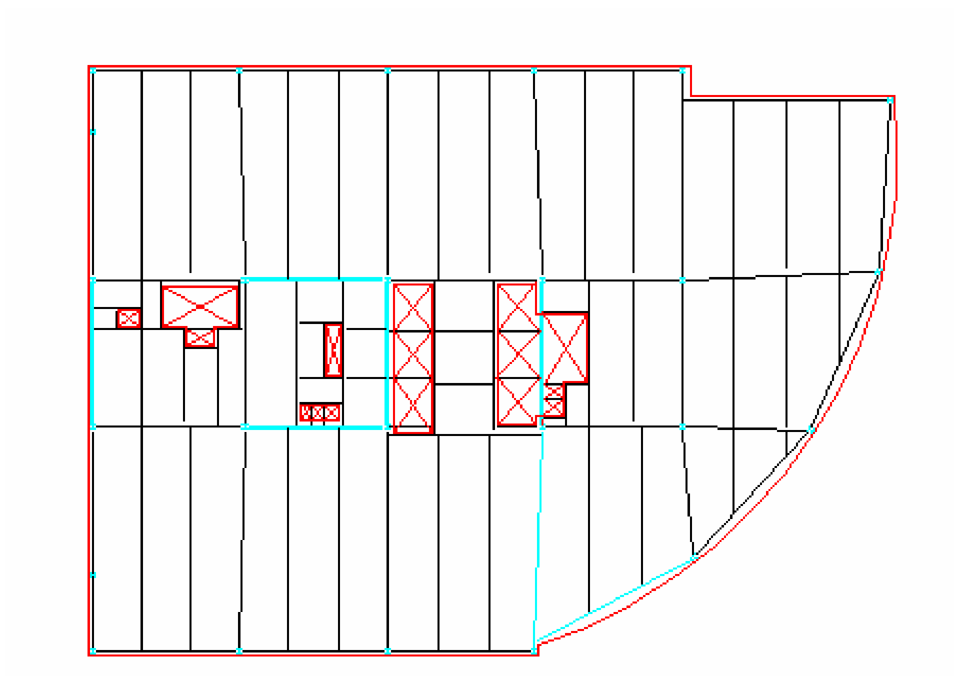
10th Floor Framing Plan



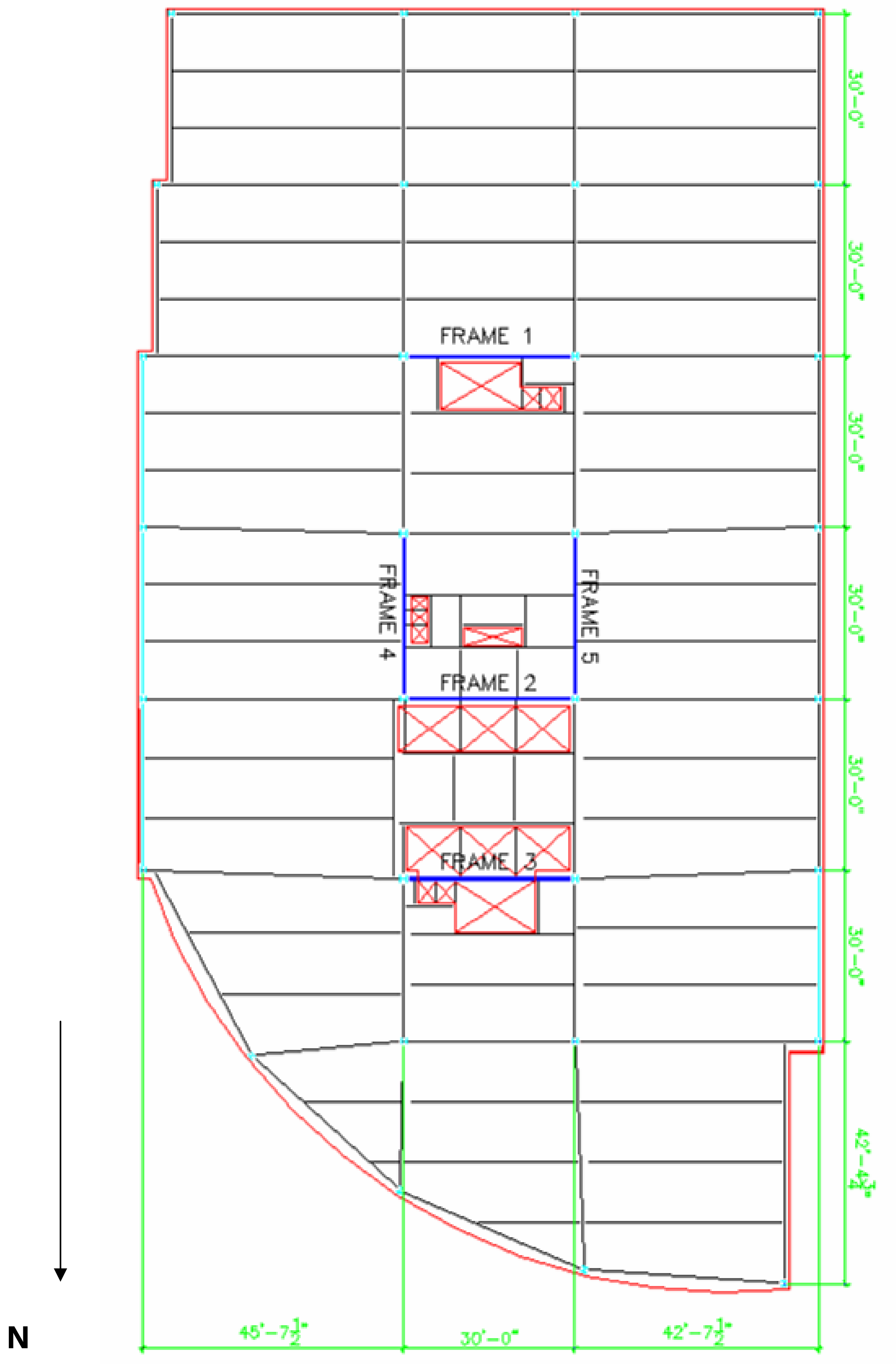
Note: Shaded area is roof construction



11th and 12th Floor Framing Plan



Enlarged Typical Framing Plan with Dimensions



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

Contents of Report

The focus of this report is on lateral system analysis and confirmation of design. This report covers the following:

- Loads
- Load Cases and Controlling Lateral Forces
- Lateral Load Distribution
- Lateral Member Checks
- Braced Frame Member Design Checks
- Drift and Story Drift Checks
- Overturning Moments
- Conclusions
- Appendix

Loads

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*

*NOTE: The slab on metal deck will be unshored during construction.

- **Live Loads** (IBC 2000, Table 1607.1)
  - **Corridors** — **100 PSF**
  - **Stairs** — **100 PSF**
  - **Mechanical Spaces** — **150 PSF**
  - **Offices** — **100 PSF***

    ***Includes 20 PSF Partition Load**
    - 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
  - **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 Load** — 30 PSF

- **Construction Live Load** (unreducible) — 20 PSF

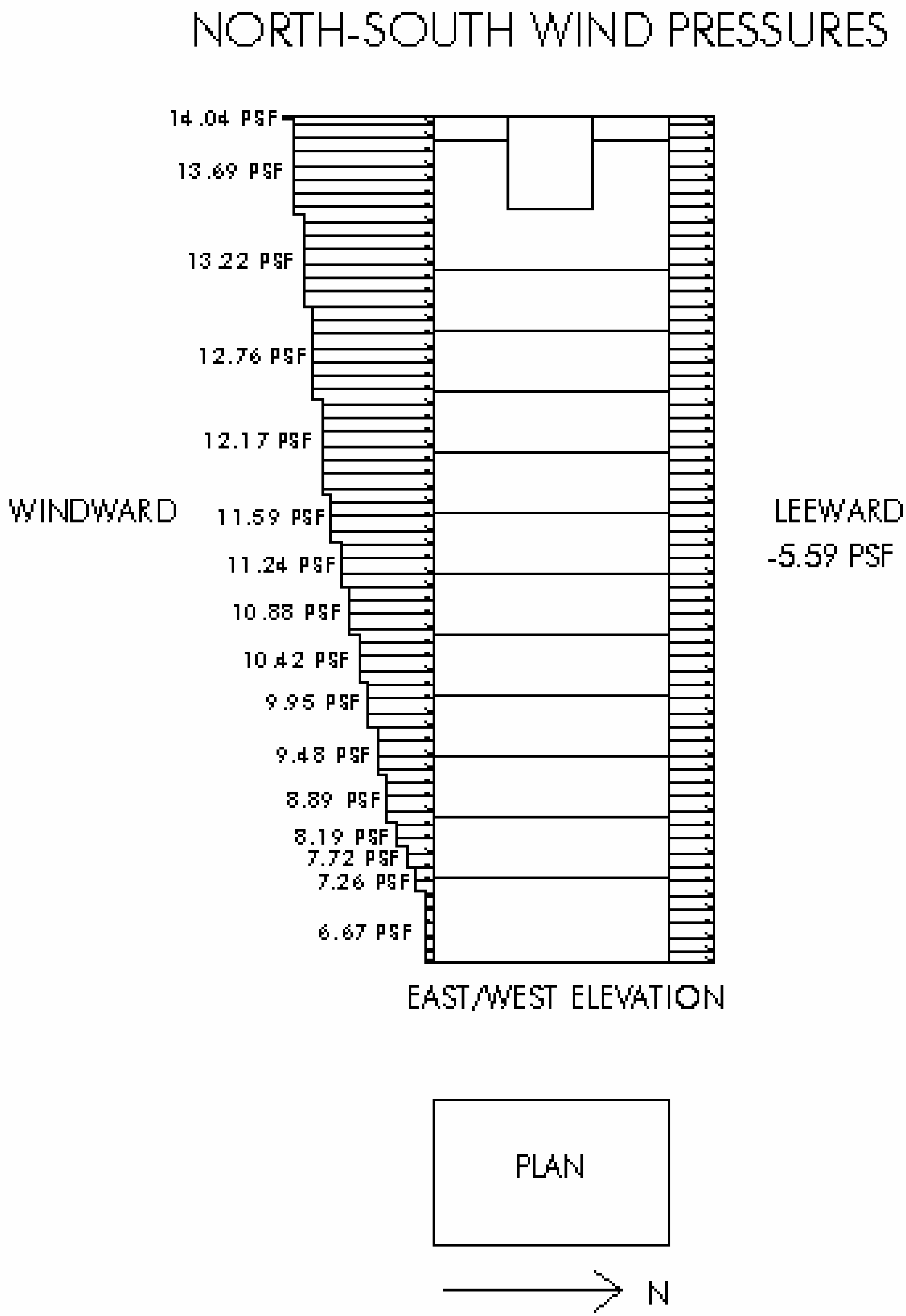
- **Roof Live Load** (as calculated per ASCE 7-02) — 12 PSF

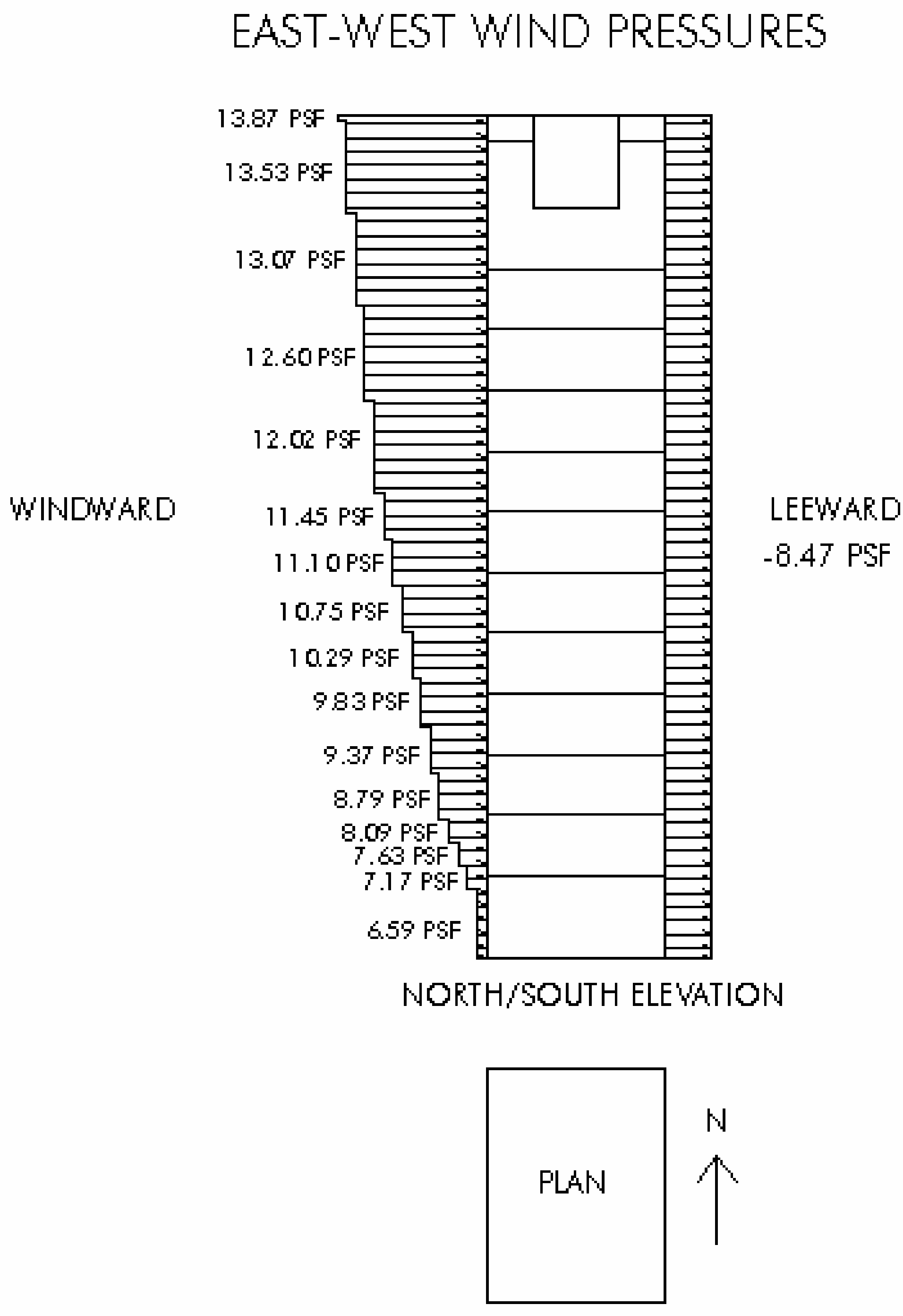
Lateral Loads

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

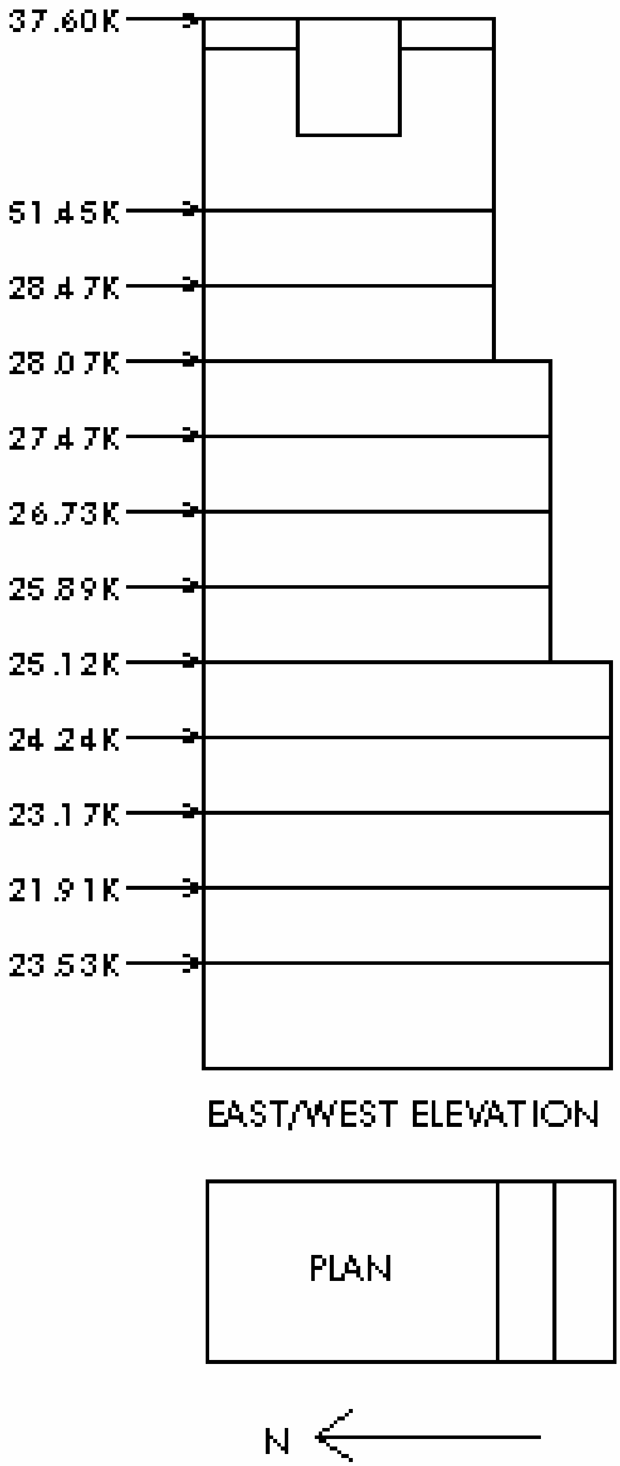
Wind Pressures

z	Kz	qz	N-S Windward Pressure (PSF)	E-W Windward Pressure (PSF)	N-S Leeward Pressure (PSF)	E-W Leeward Pressure (PSF)	Ptotal (N-S) (PSF)	Ptotal (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

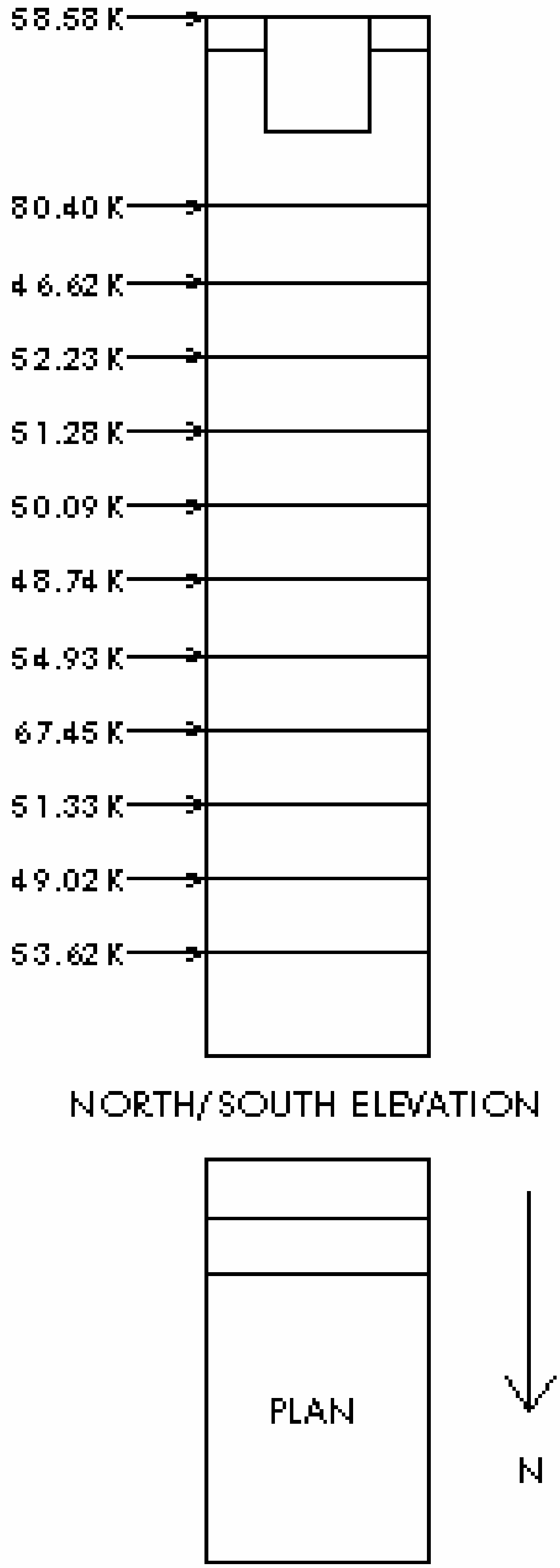

NORTH-SOUTH WIND PRESSURES
14.04 PSF
13.69 PSF
13.22 PSF
12.76 PSF
12.17 PSF
WINDWARD
11.59 PSF
11.24 PSF
10.88 PSF
10.42 PSF
9.95 PSF
9.48 PSF
8.89 PSF
8.19 PSF
7.72 PSF
7.26 PSF
6.67 PSF
LEEWARD
-5.59 PSF
EAST/WEST ELEVATION
PLAN
N


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

NORTH-SOUTH WIND FORCES



EAST-WEST WIND FORCES



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

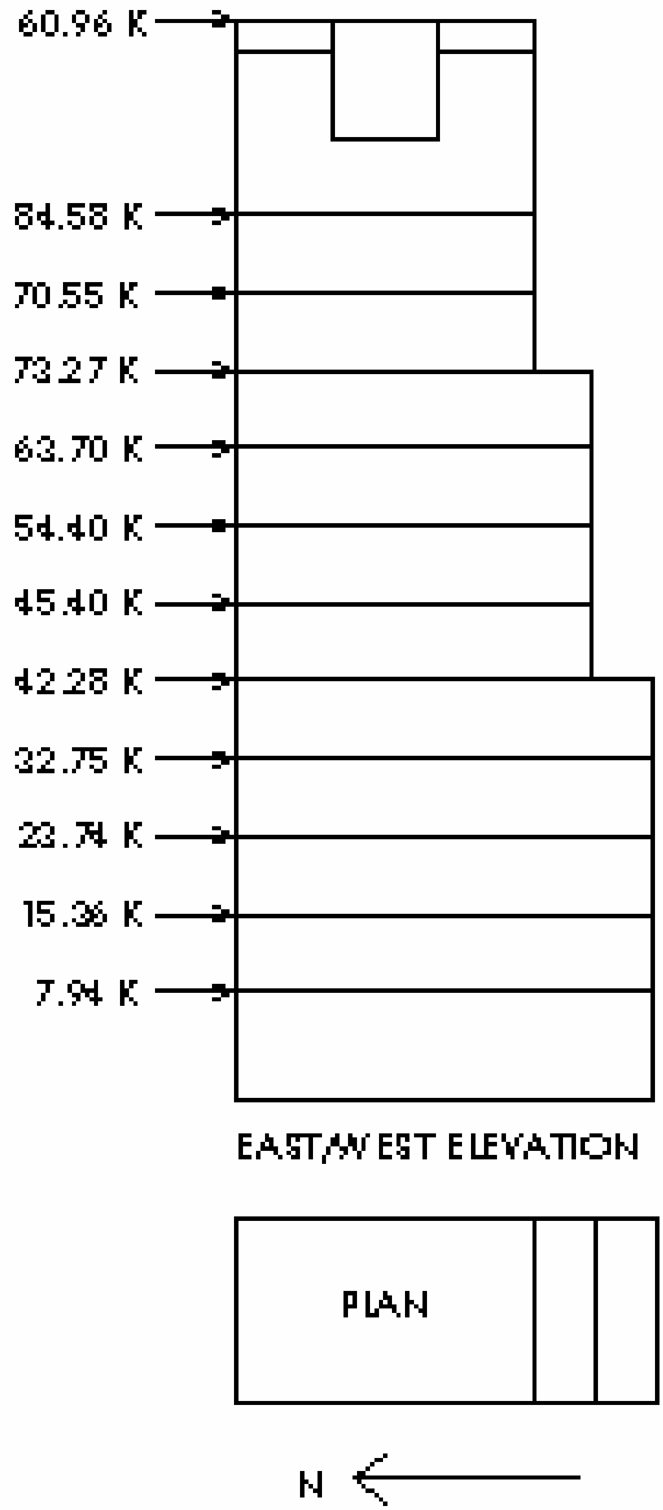
Level	w_x	h_x	$w_x h_x^{1.243}$	$w_x h_x^{1.243}$	C_{vx} (N-S)	C_{vx} (E-W)	F_x (N-S)	F_x (E-W)
Roof	909	180.75	580915	580915	0.106	0.106	60.96	60.96
12	1617	148	806019	806019	0.147	0.147	84.58	84.58
11	1512	135	672290	672290	0.123	0.123	70.55	70.55
10	1781	122	698247	698247	0.127	0.127	73.27	73.27
9	1781	109	606995	606995	0.111	0.111	63.70	63.70
8	1781	96	518355	518355	0.095	0.095	54.40	54.40
7	1781	83	432592	432592	0.079	0.079	45.40	45.40
6	2050	70	402912	402912	0.074	0.074	42.28	42.28
5	2050	57	312109	312109	0.057	0.057	32.75	32.75
4	2050	44	226238	226238	0.041	0.041	23.74	23.74
3	2050	31	146392	146392	0.027	0.027	15.36	15.36
2	2083	18	75682	75682	0.014	0.014	7.94	7.94
			5478745	5478745	1.000	1.000	574.94	574.94

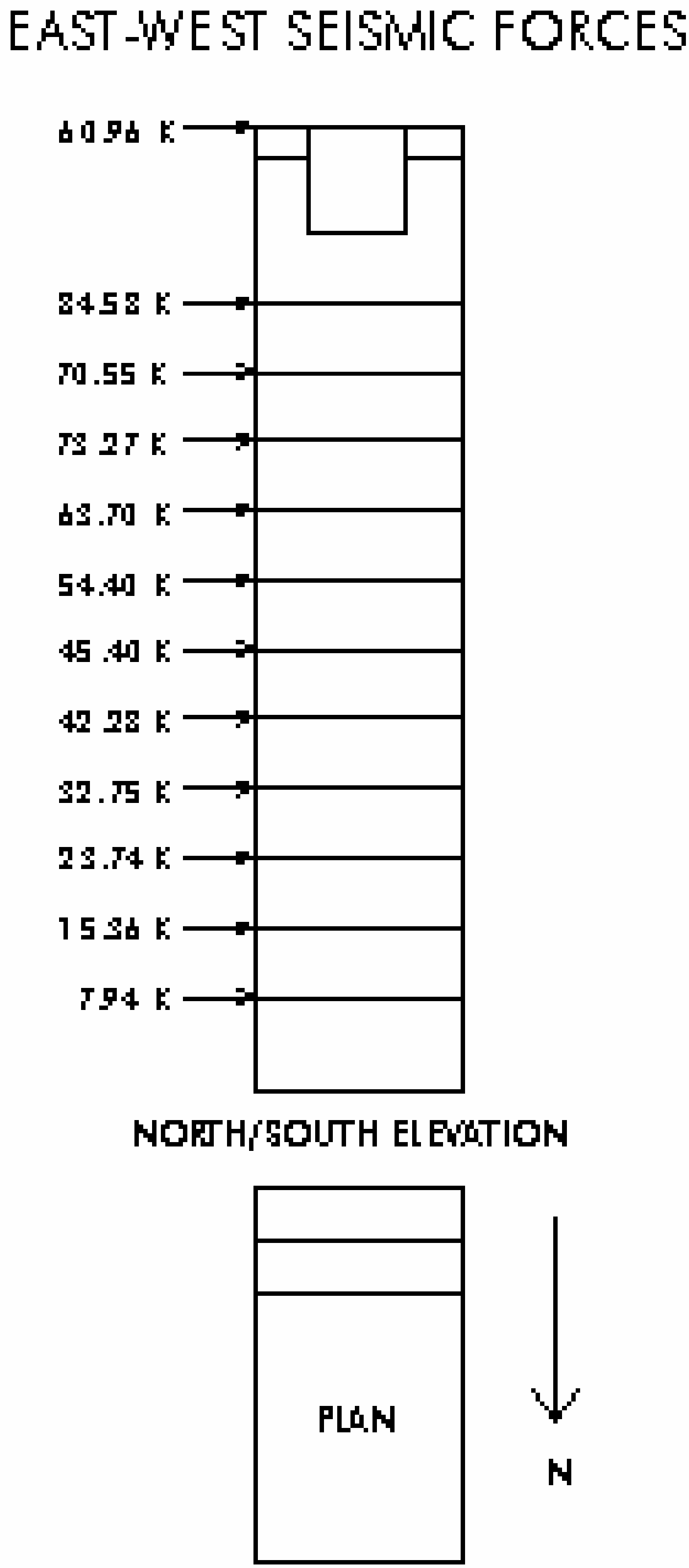
k (N-S)	1.243	
k (E-W)	1.243	
V (N-S)	574.94	k
V (E-W)	574.94	k

Base Shear		
N-S	574.94	k
E-W	574.94	k

Overturning Moment		
Overturning Moment (N-S)	64424.2942	ft-k
Overturning Moment (E-W)	64424.2942	ft-k

NORTH-SOUTH SEISMIC FORCES




EAST-WEST SEISMIC FORCES
60.96 K
84.58 K
70.55 K
73.27 K
63.70 K
54.40 K
45.40 K
42.28 K
32.75 K
23.74 K
15.36 K
7.94 K
NORTH/SOUTH ELEVATION
PLAN
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Load Cases and Controlling Lateral Forces

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)	1.6W (E-W)	1.0E (N-S/E-W)
Roof	60.16	**93.72**	**60.96**
12	82.32	**128.64**	**84.58**
11	45.55	**74.59**	**70.55**
10	44.91	**83.57**	**73.27**
9	43.95	**82.05**	**63.70**
8	42.77	**80.14**	**54.40**
7	41.42	**77.98**	**45.40**
6	40.19	**87.89**	**42.28**
5	**38.78**	**107.92**	32.75
4	**37.07**	**82.13**	23.74
3	**35.06**	**78.43**	15.36
2	**37.64**	**85.79**	7.94

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.

Lateral Load Distribution

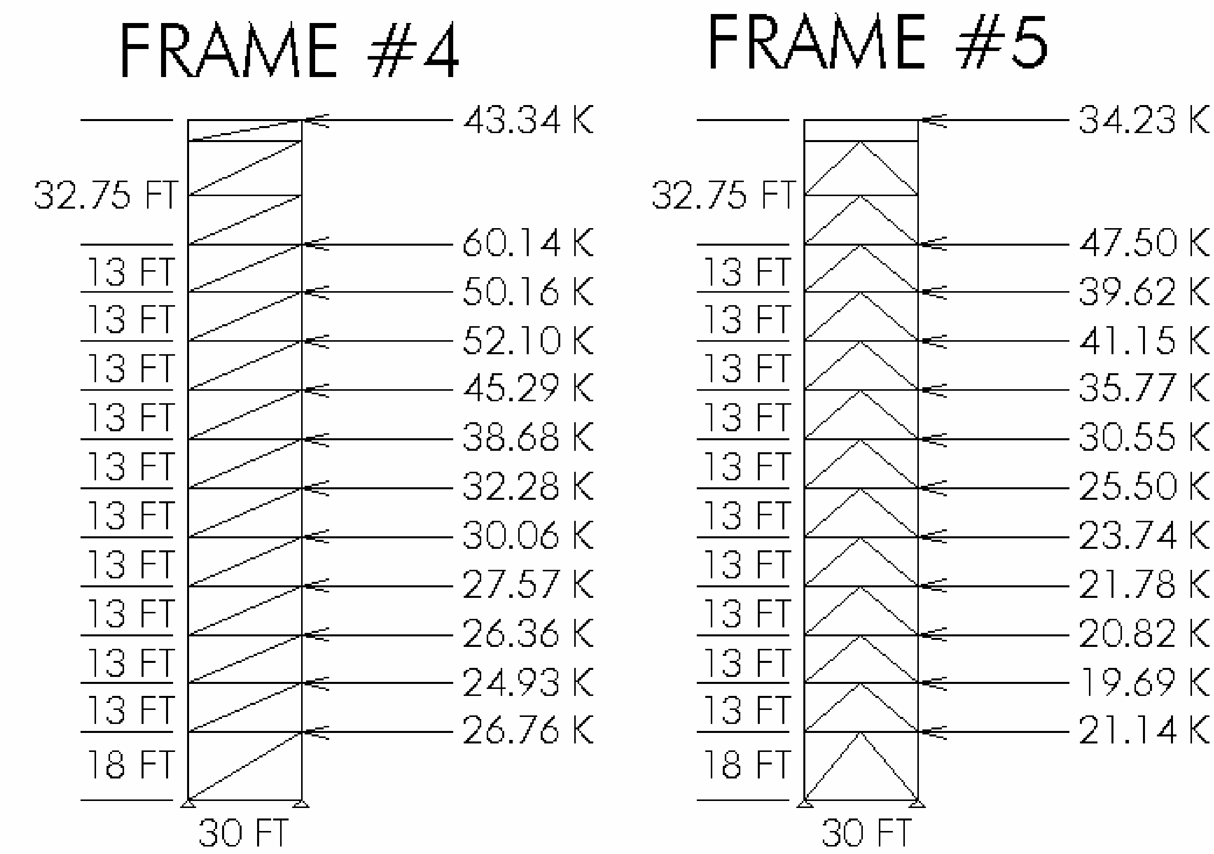
North / South Lateral Forces

When there are lateral forces acting in the north / south direction, Frames #4 and #5 will take the lateral loads. In the north / south direction, seismic is the controlling lateral force from the roof level down to and including the 6^{th} level. For the 5^{th} through 2^{nd} levels, the controlling lateral force is wind. The Lateral Load Distribution Procedure: Distribution by Rigidity was used to determine the distribution of the controlling lateral forces to Frames #4 and #5. This procedure takes into account the relative stiffness of the braced frames and any torsional effects due to the braced frame configuration and the changing center of mass for each floor. The Lateral Load Distribution: Distribution by Rigidity calculations for Frames #4 and #5 can be found in the Appendix. Since Frames #4 and #5 are approximately equal distances from the center of rigidity and the center of mass, a 5% accidental torsion was included based off of an eccentricity of 5% of the building length as a conservative approach. The forces distributed to frames #4 and #5 take into account the relative stiffness of each braced frame and the additional lateral forces due to the accidental torsion.

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

Level	Controlling Lateral Force	Total Direct Factored Lateral Force to the Level (k)	Factored Lateral Force to Frame #4 (k)	Factored Lateral Force to Frame #5 (k)
Roof	Seismic	60.96	43.34	34.23
12	Seismic	84.58	60.14	47.50
11	Seismic	70.55	50.16	39.62
10	Seismic	73.27	52.10	41.15
9	Seismic	63.70	45.29	35.77
8	Seismic	54.40	38.68	30.55
7	Seismic	45.40	32.28	25.50
6	Seismic	42.28	30.06	23.74
5	Wind	38.78	27.57	21.78
4	Wind	37.07	26.36	20.82
3	Wind	35.06	24.93	19.69
2	Wind	37.64	26.76	21.14

Lateral Load Distribution Diagrams for Frames #4 and #5



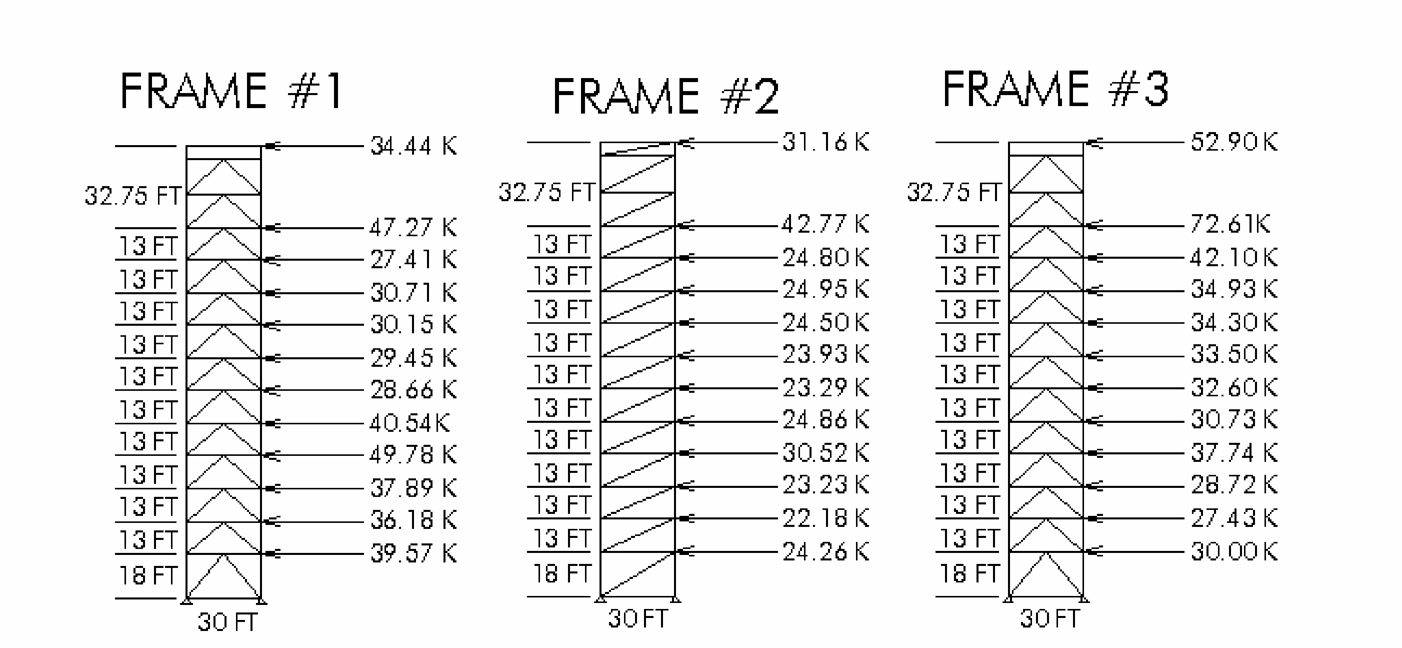
East / West Lateral Forces

When there are lateral forces acting in the east / west direction, Frames #1, #2, and #3 will take the lateral loads. The controlling lateral force for the east / west direction is wind for all of the levels. The Lateral Force Distribution Procedure: Distribution by Rigidity was used to determine the distribution of the controlling factored east / west lateral forces to Frames #1, #2, and #3. This procedure takes into account the relative stiffness of each braced frame and any torsional effects due to the braced frame configuration and the changing center of mass for each floor. The Lateral Load Distribution: Distribution by Rigidity calculations for Frames #1, #2, and #3 can be found in the Appendix.

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

Level	**Controlling Lateral Force**	**Total Direct 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	34.44	31.16	52.90
12	Wind	128.64	47.27	42.77	72.61
11	Wind	74.59	27.41	24.80	42.10
10	Wind	83.57	30.71	24.95	34.93
9	Wind	82.05	30.15	24.50	34.30
8	Wind	80.14	29.45	23.93	33.50
7	Wind	77.98	28.66	23.29	32.60
6	Wind	87.89	40.54	24.86	30.73
5	Wind	107.92	49.78	30.52	37.74
4	Wind	82.13	37.89	23.23	28.72
3	Wind	78.43	36.18	22.18	27.43
2	Wind	85.79	39.57	24.26	30.00

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



Computer Models and Analyses

Two sets of computer models and analyses were done for this report using ETABS 8. Each model is described in this section.

Single Frame Models

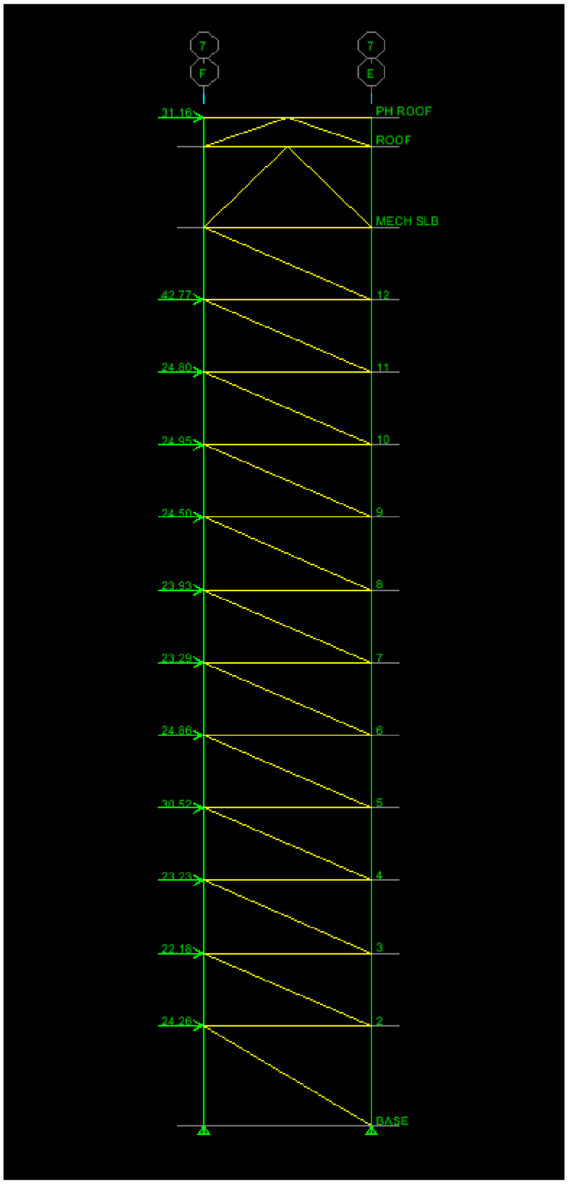
The first set of computer models involves each braced frame analyzed individually and separately from the other braced frames. Each model contains one braced frame and its calculated lateral loads based off of the Lateral Distribution Procedure: Distribution by Rigidity. *Refer to the Lateral Load Distribution section for the calculated loads applied to each braced frame.* This set of computer models will be referred to as the Single Frame models. Each braced frame was constructed as specified in the structural drawings with the corresponding dimensions and actual designed framing members.

In order to get the relative stiffness for each braced frame a 100k force was added horizontally to the top of each braced frame in the plane of the braced frame. The deflections were used to calculate the relative stiffness for each braced frame so that the distribution of lateral forces to each braced frame could be accurately computed.

Once the relative stiffness for each braced frame was calculated, the factored, controlling lateral loads that were distributed and applied to each braced frame. Each braced frame was analyzed individually and separately in order to find the story displacements, axial forces, and to see if the designed members were adequate to carry the calculated forces.

The Single Frame models only take into account the lateral loads applied to the braced frame in the plane of the braced frame individually. The results of this first set of computer models will be used to compare displacements, axial forces, and member design checks with hand calculations and other computer models.

Example of a Single Frame Model – Frame #2

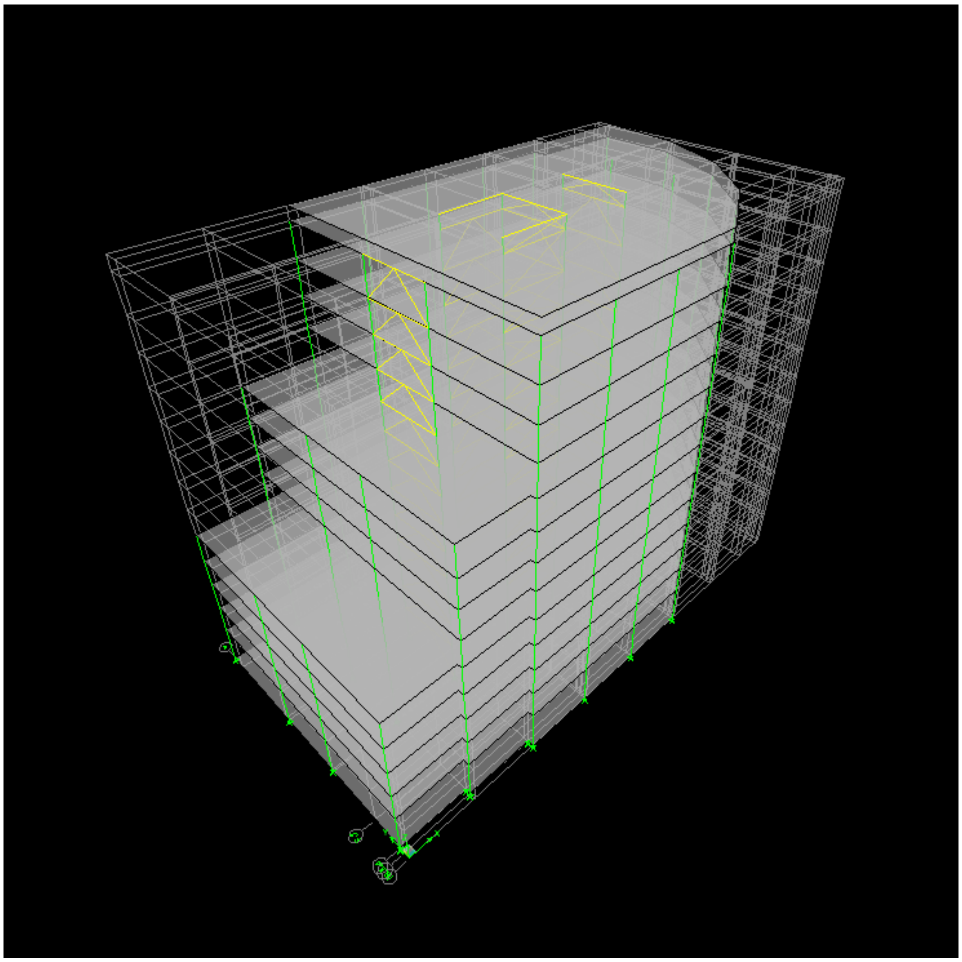


Whole Building Model

The second computer model is a model of all of the braced frames in the building connected by a rigid diaphragm, representing the slab on metal deck, at each level. This model will be referred to as the Whole Building model. The actual designed members and dimensions were used to construct the model. The controlling lateral loads that were calculated in the *Loads Cases* section, were applied to the center of mass of each rigid diaphragm for both the north / south and east / west directions. This model, which includes all of the braced frames and a rigid diaphragm, was constructed and analyzed in order to find the displacements of the braced frames, axial forces in the braced frames, and to see if the designed members were adequate to carry the loads applied.

Throughout this report, results of the computer models, hand calculations, and information provided in the structural drawings will be compared in order to have a complete analysis of the lateral force resisting system of The Regent.

Whole Building Model in ETABS

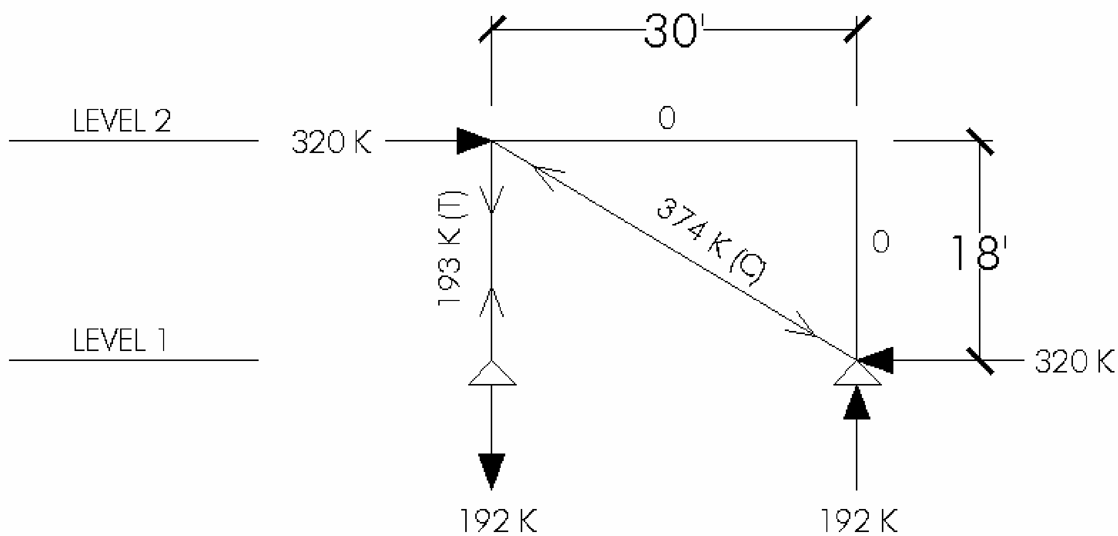


Lateral Members Checks

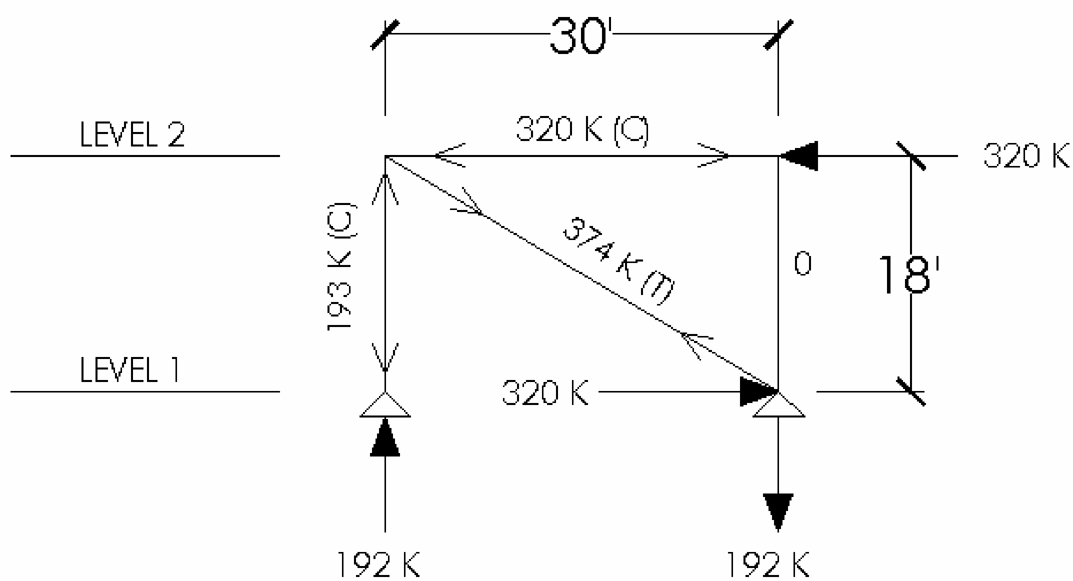
Lateral Member in Braced Frame #2

The bottom diagonal bracing member for Frame #2 was checked for the base shear force due to the worst case lateral loading in the east / west direction. The factored base shear is 320 k.

Wind from the East



Wind from the West



The force in the diagonal member is 373.7 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 an HSS 12 x 12 member was used for the actual design, an HSS 12 x 12 member was selected and compared to the actual designed member and the computer analysis member design check.

$$T_u = \phi F_y A_g$$

$$A_{steel,req} = \frac{T_u}{\phi F_y}$$

$$A_{steel,req} = \frac{373.7k}{0.9(50ksi)}$$

$$A_{steel,req} = 8.30in^2$$

From Table 1-11 from the AISC's Manual of Steel Construction, an HSS 12 x 12 x ¼ was selected, with an area of 10.8 in^2, which is greater than 8.30 in^2, and therefore should be okay.

The actual member size is an HSS 12 x 12 x ½ with an area of steel of 20.9 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 373 k (C) and 488 k (T). The calculated values were equal to or less than the forces designed for. Either the member is conservatively designed or the calculated lateral forces are unconservative. Since a thickness of 1/4" meets the axial stress requirements, a 1/2" member was probably chosen based off of other structural calculations and is considered to be a more conservative section. The member may also have had to meet minimum thickness requirements because it is a critical member in a braced frame.

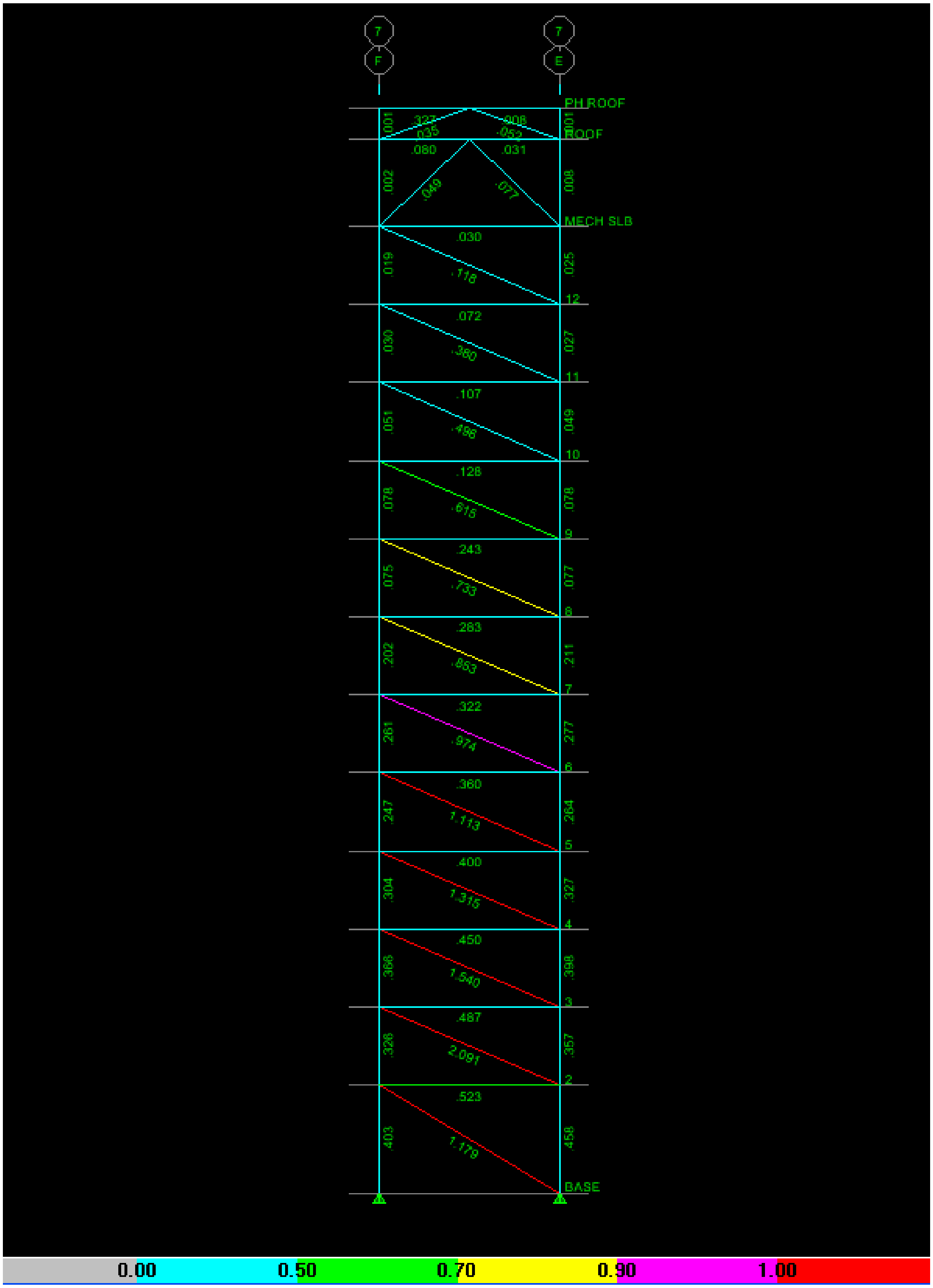
HSS 12 x 12 x ½ - Capacity

$$T_u = \phi F_y A_g$$

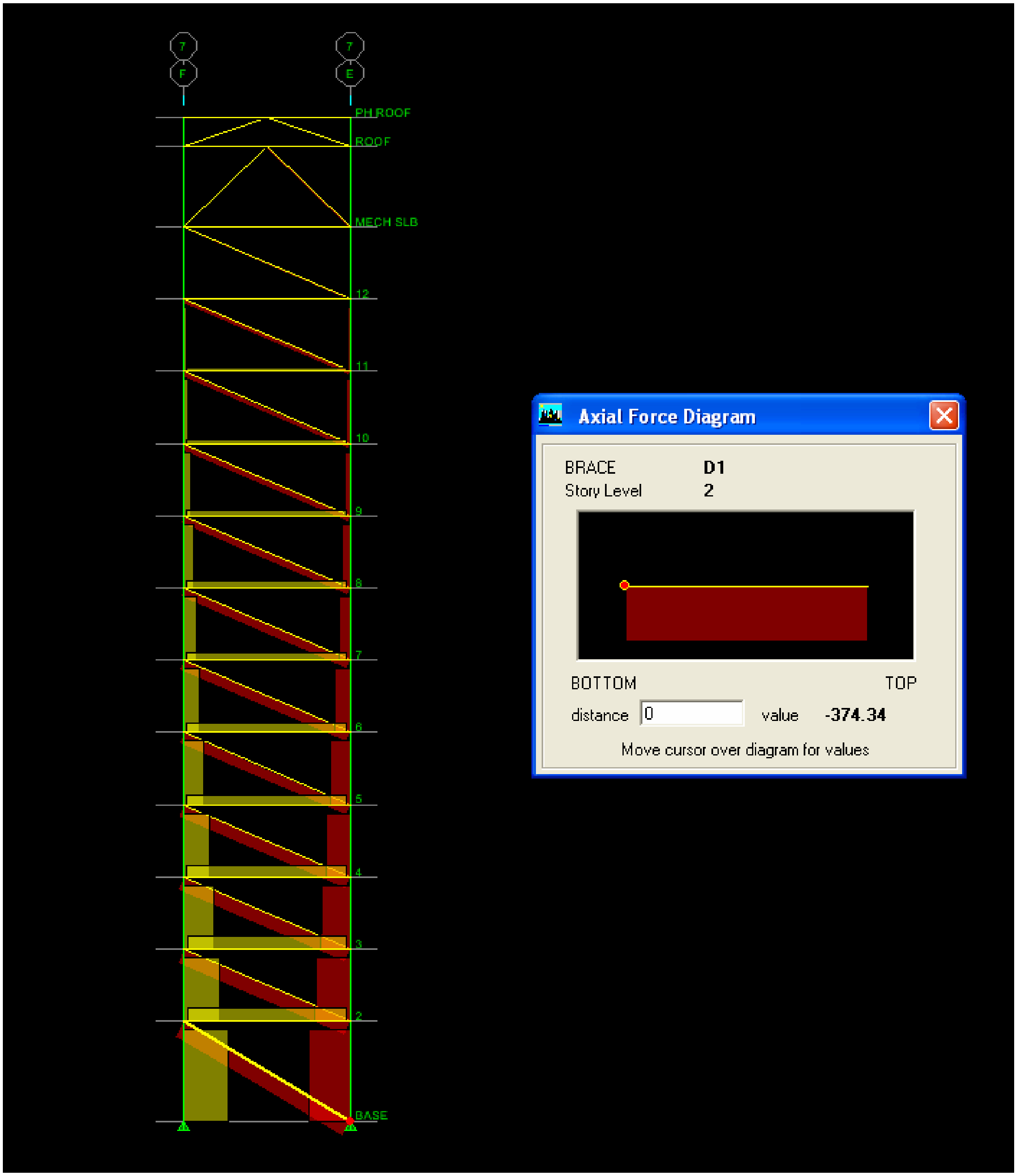
$$T_u = 0.9(50ksi)(20.9in^2)$$

$$T_u = 940.5k$$

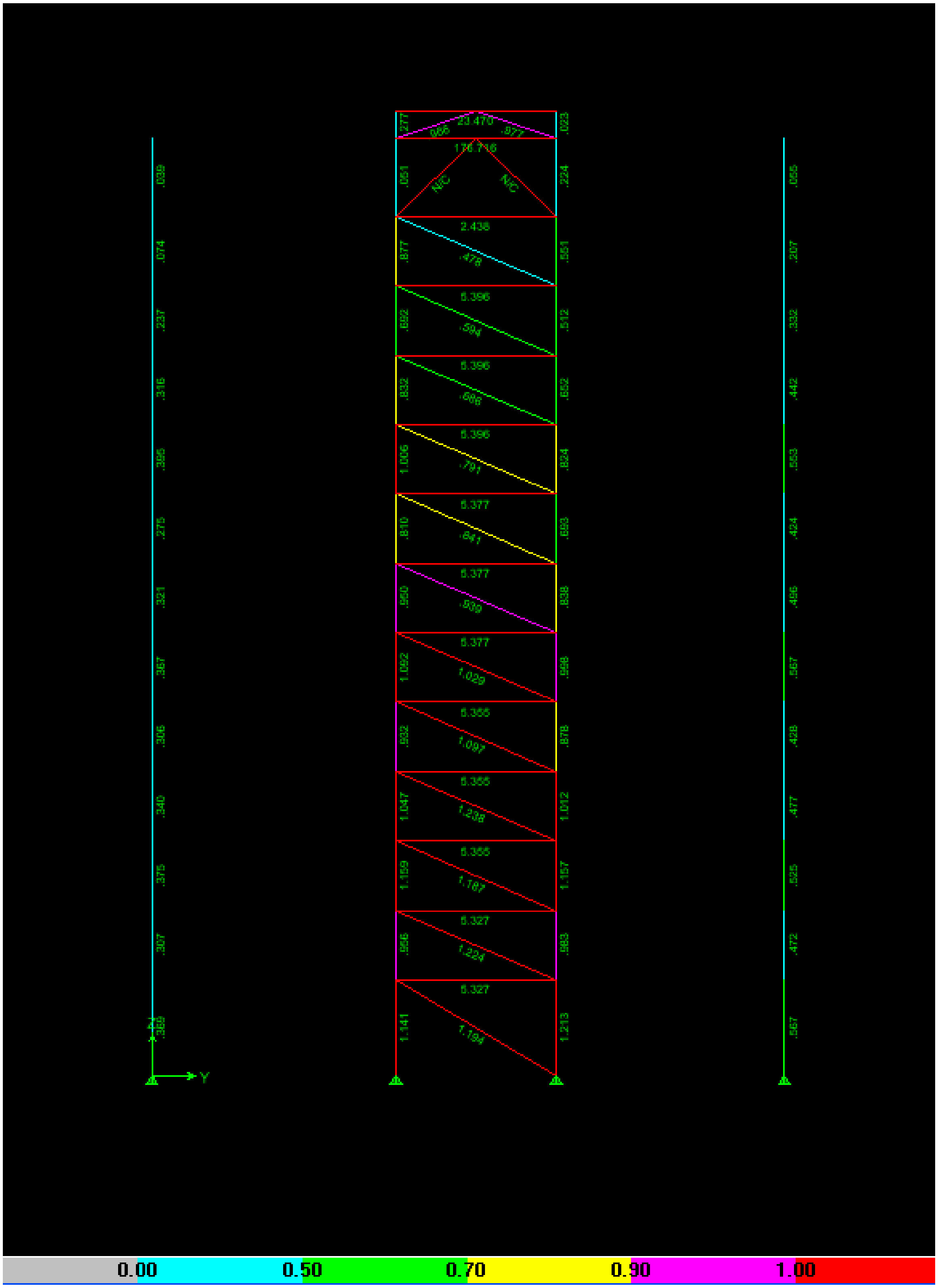
Computer Analysis – Single Frame – Frame #2 - Member Design Check



Computer Analysis – Single Frame – Frame #2 - Axial Member Forces

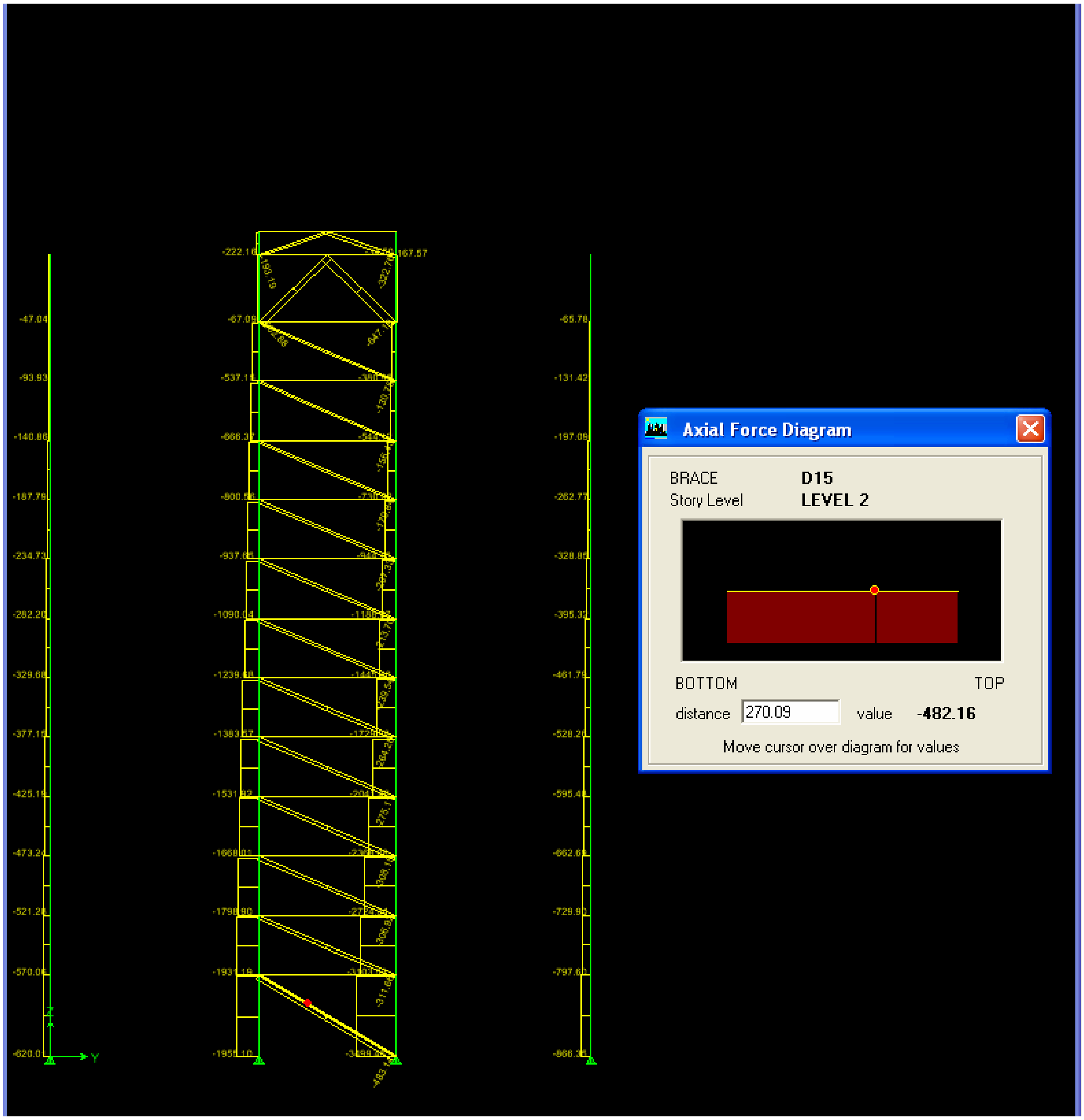


Computer Analysis – Whole Building – Frame #2 - Member Design Check



0.00	0.50	0.70	0.90	1.00

Computer Analysis – Whole Building – Frame #2 - Axial Member Forces



Summary of Results of Lateral Member in Braced Frame #2 Analysis

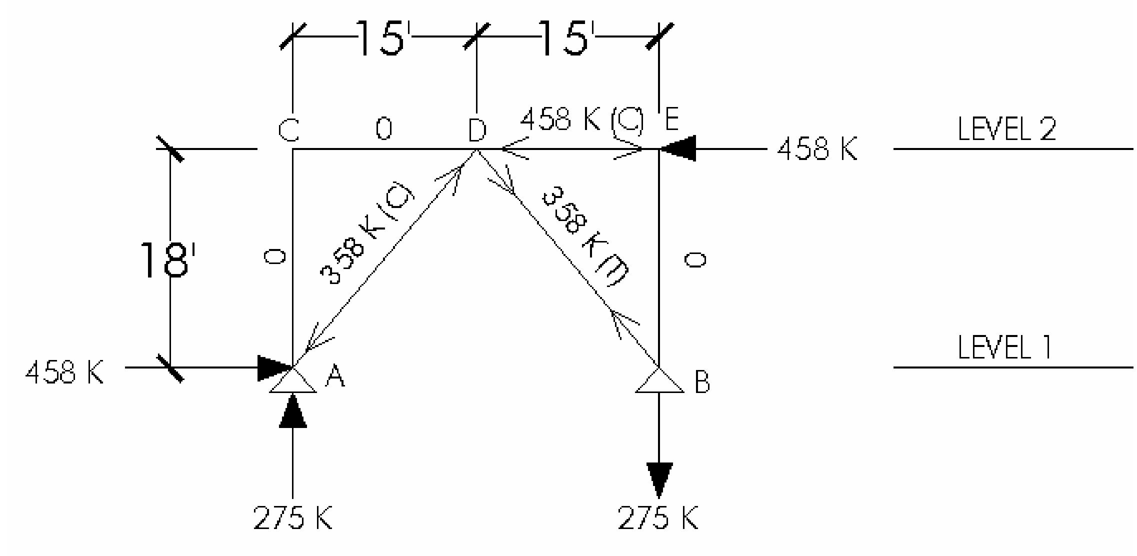
	Single Frame	Whole Building	Existing Design	Hand Calculations
Axial Force	-374.34 k	-482.16 k	373 k (C) 488 k (T)	374 k (C) 374 k (T)
HSS 12 x 12 x ½ OK?	No	No	Yes	Yes

According to the existing design and the hand calculations, this HSS 12 x 12 x ½ should be adequate for this design. The Single Frame analysis shows that there is 374 k of axial force in the member which is the same axial force that the hand calculations and the existing design show. Also, the Whole Building analysis resulted in an axial force of 482 k which is similar to the existing design force of 488 k. An HSS 12 x 12 x ½ can carry and axial load of 940.5 k > than 373.3 k and 482.16 k. Since the calculated loads were similar in magnitude across all of the analyses, it was determined that the computer models may not be an exact representation of the lateral framing system. The models need to be reviewed further in order to figure out why the results show they are not correctly designed for strength even though the loads match the other analyses which prove that under the applied loads, the diagonal member is adequate.

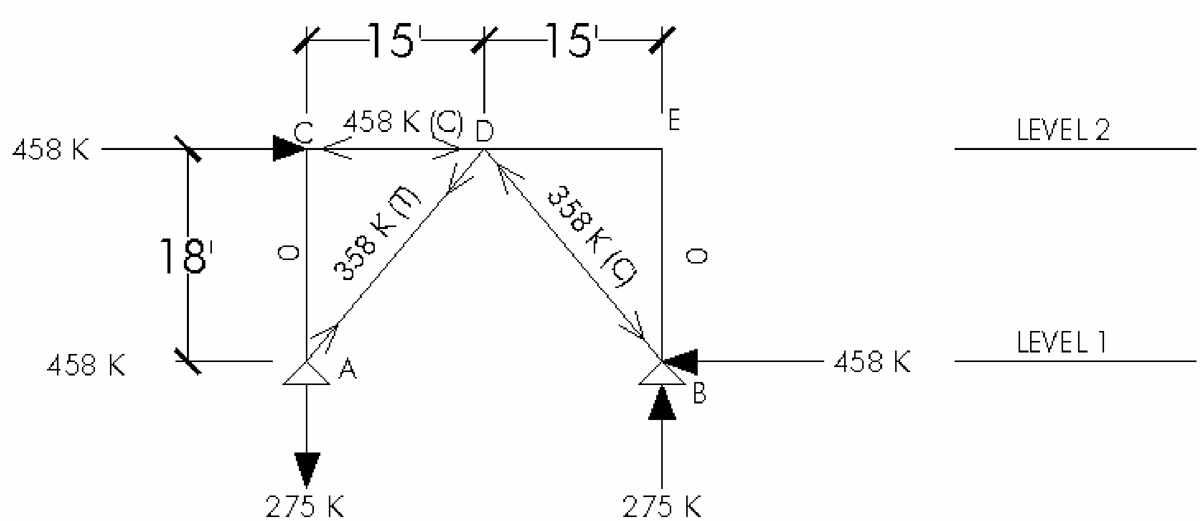
Lateral Member in Braced Frame #3

The bottom diagonal bracing members for Frame #3 were checked for the base shear force due to the worst case lateral loadings in the east / west direction. The factored base shear is 458 k.

Wind from the West



Wind from the East



The forces in the diagonal members are 358 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 being the most common, an HSS 10 x 10 member was selected and compared to the actual designed member and the computer analysis.

$$T_u = \phi F_y A_g$$

$$A_{steel,req} = \frac{T_u}{\phi F_y}$$

$$A_{steel,req} = \frac{358k}{0.9(50ksi)}$$

$$A_{steel,req} = 7.96in^2$$

From Table 1-11 from the AISC's Manual of Steel Construction, and HSS 10 x 10 x ¼ was selected, with an area of 8.96 in^2, which is greater than 7.96 in^2, and therefore should be okay.

The actual member size is an HSS 10 x 10 x ½ which has an area of steel of 17.2 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 members have calculated factored forces of 414 k (T) and 391 k (C). The hand calculated values were less than the forces designed for. The actual forces designed for may be higher than the calculated forces because either the calculated lateral forces are unconservative or the actual forces designed for were the result of a further analysis that resulted in higher lateral loads. The member may also have had to meet minimum thickness requirements because it is a critical member in a braced frame.

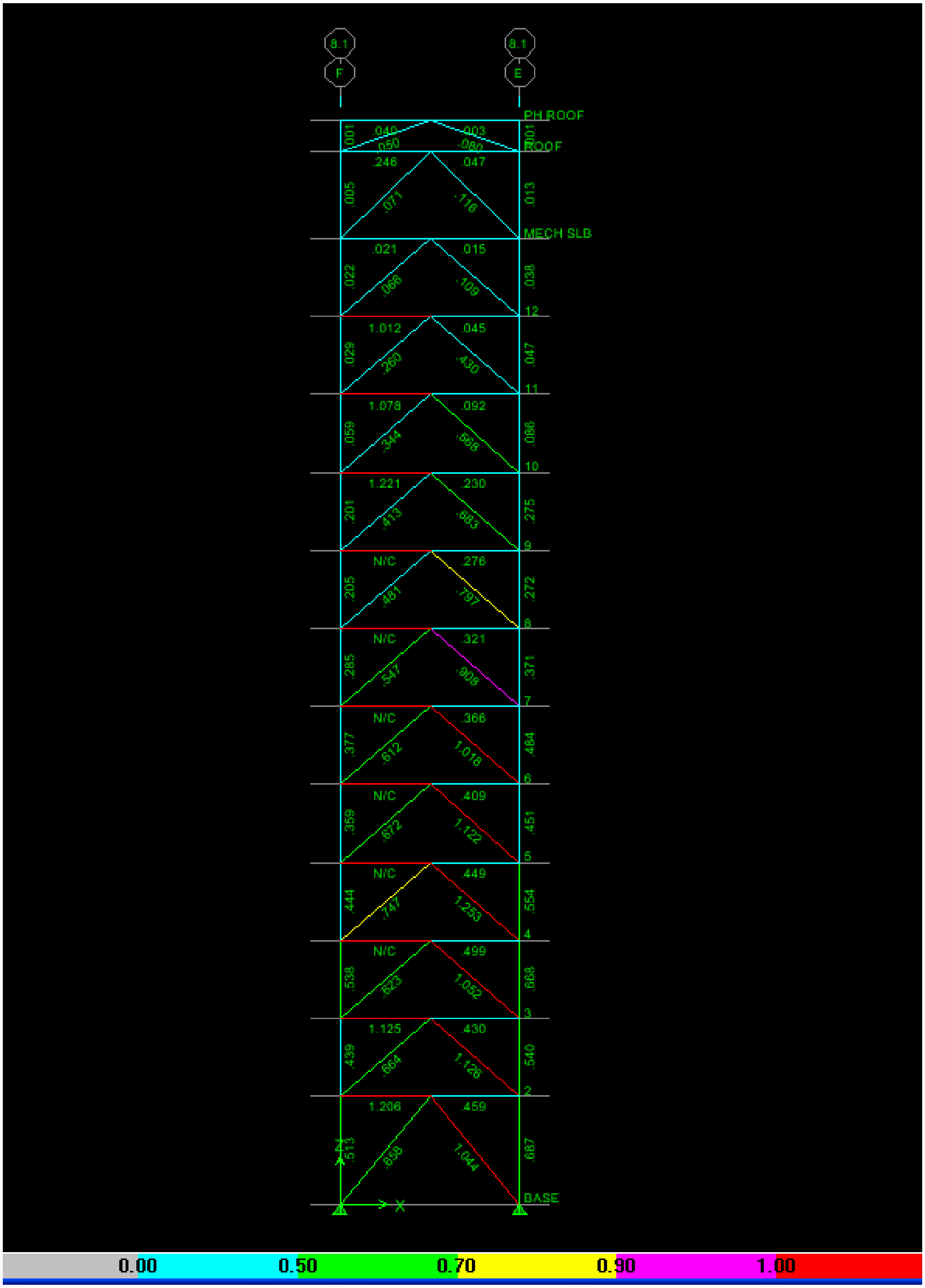
HSS 10 x 10 x ½ - Capacity

$$T_u = \phi F_y A_g$$

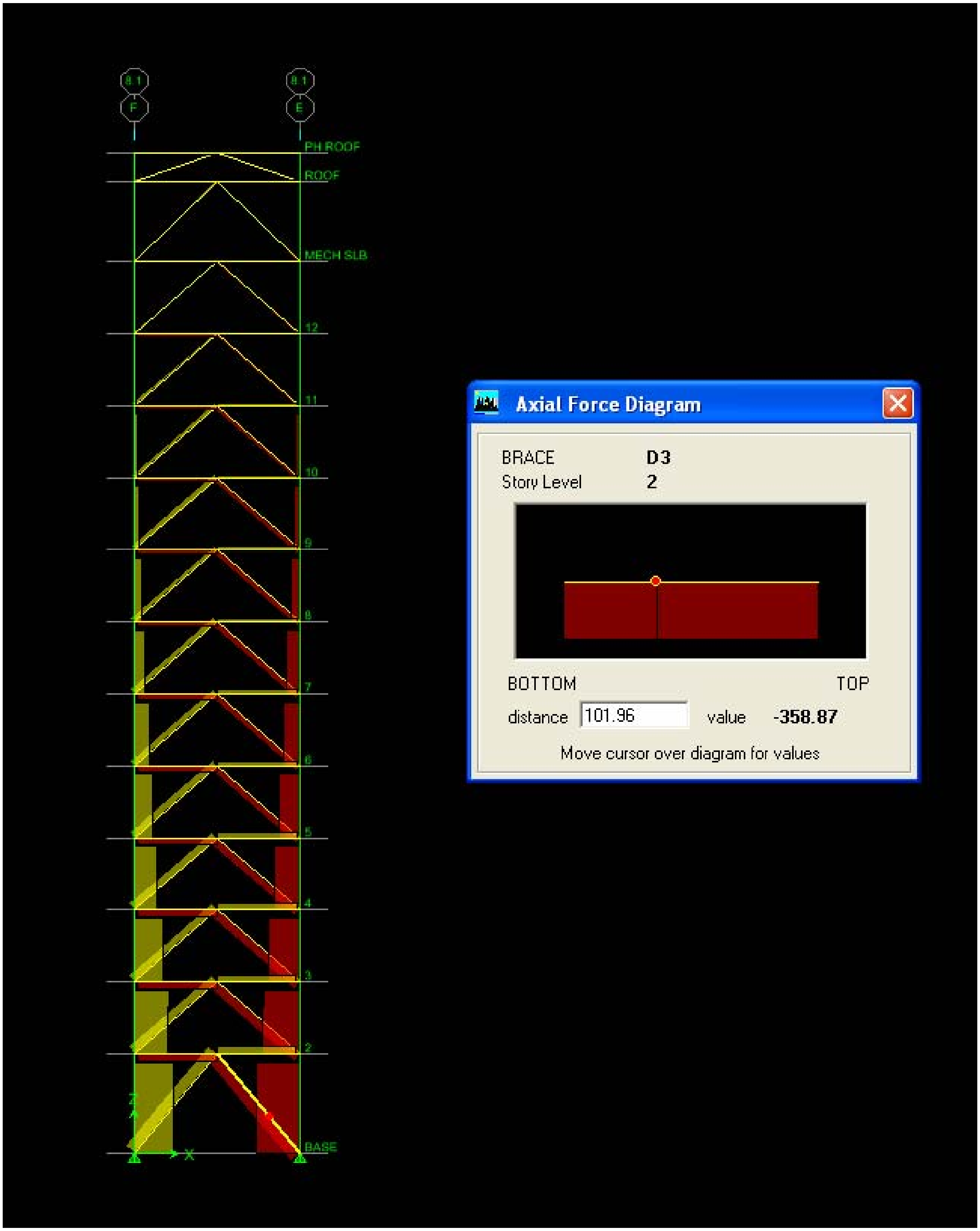
$$T_u = 0.9(50ksi)(17.2in^2)$$

$$T_u = 774k$$

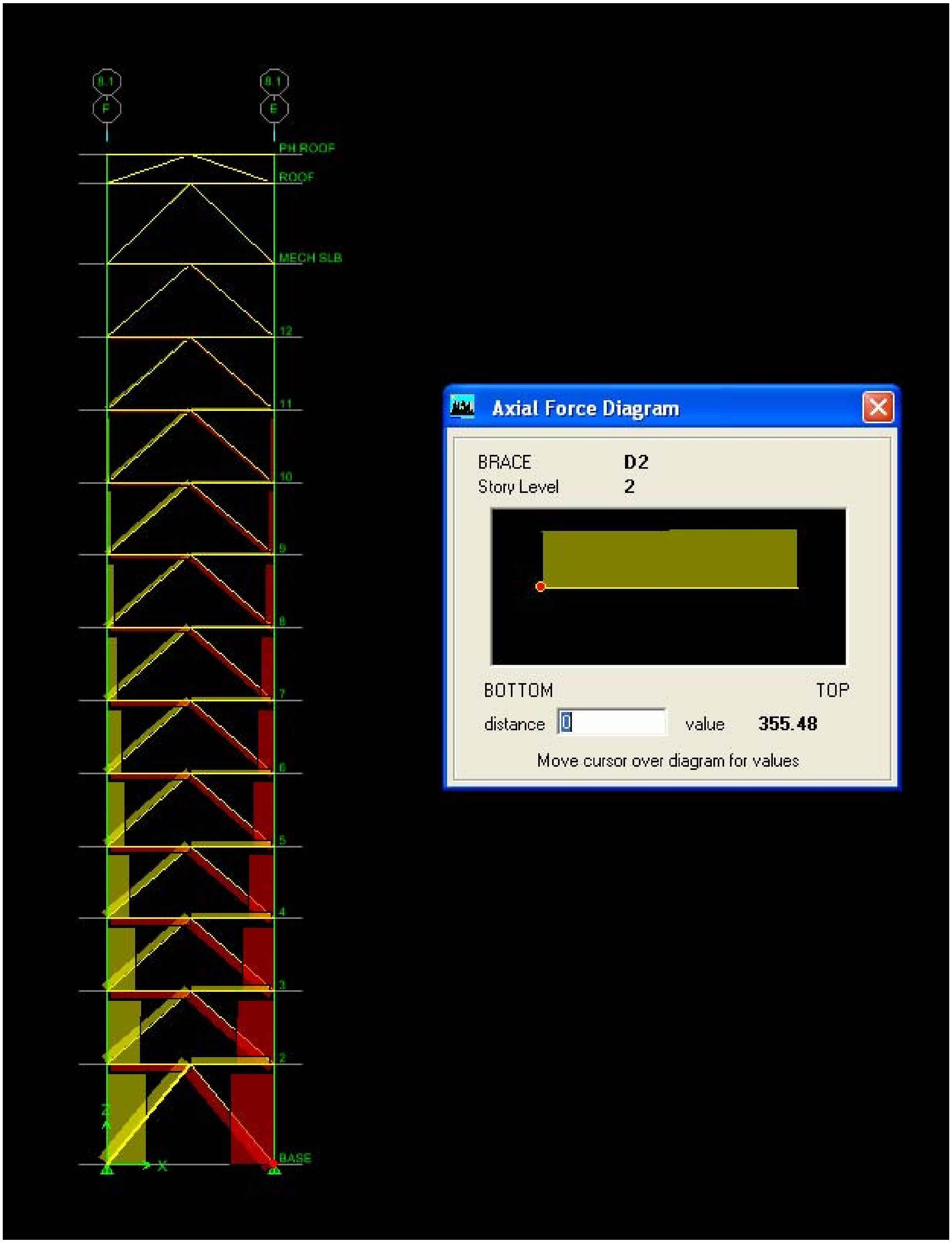
Computer Analysis – Single Frame – Frame #3 - Member Design Check



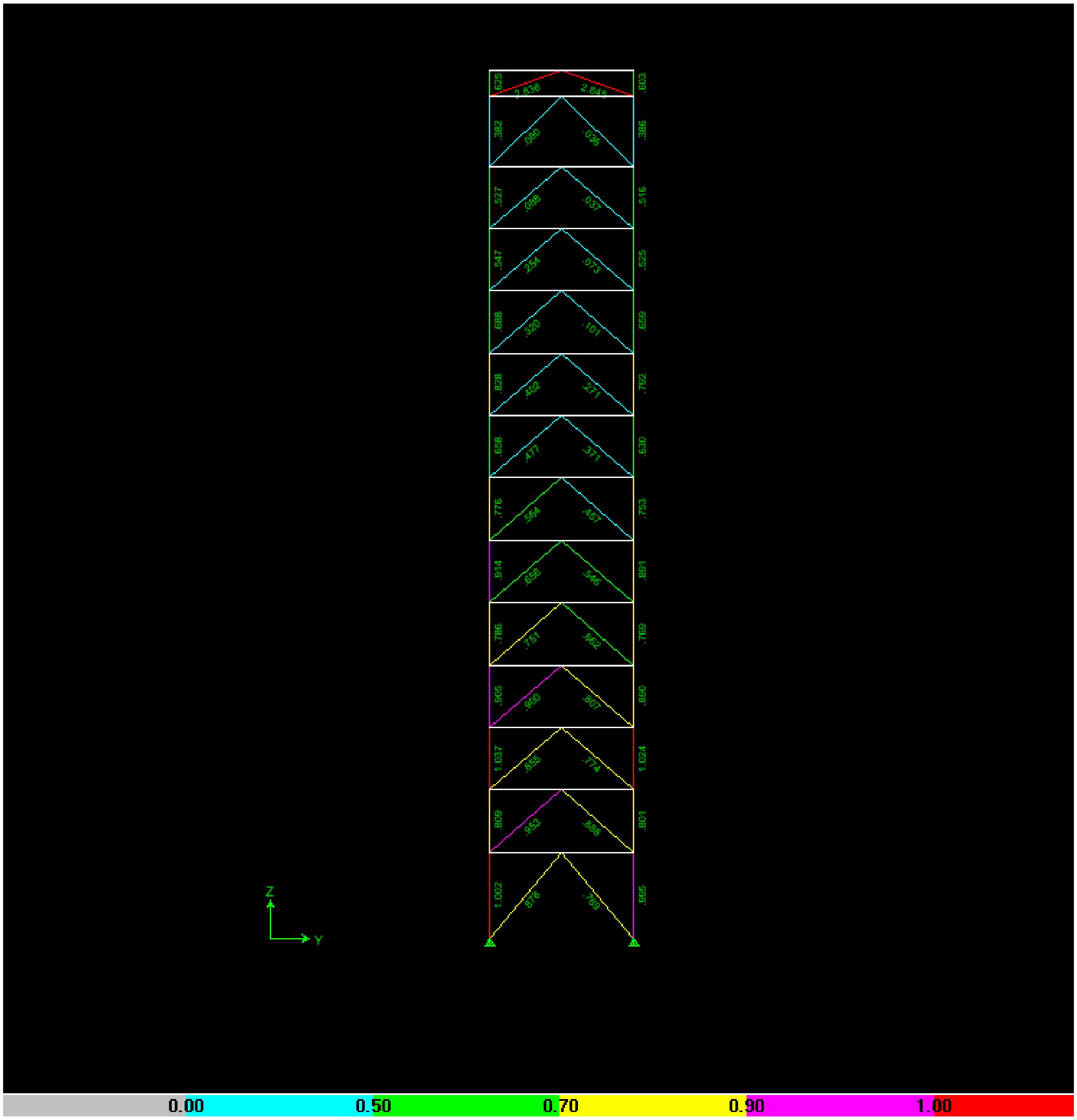
Computer Analysis – Whole Building – Frame #3 - Axial Member Force – Right Brace



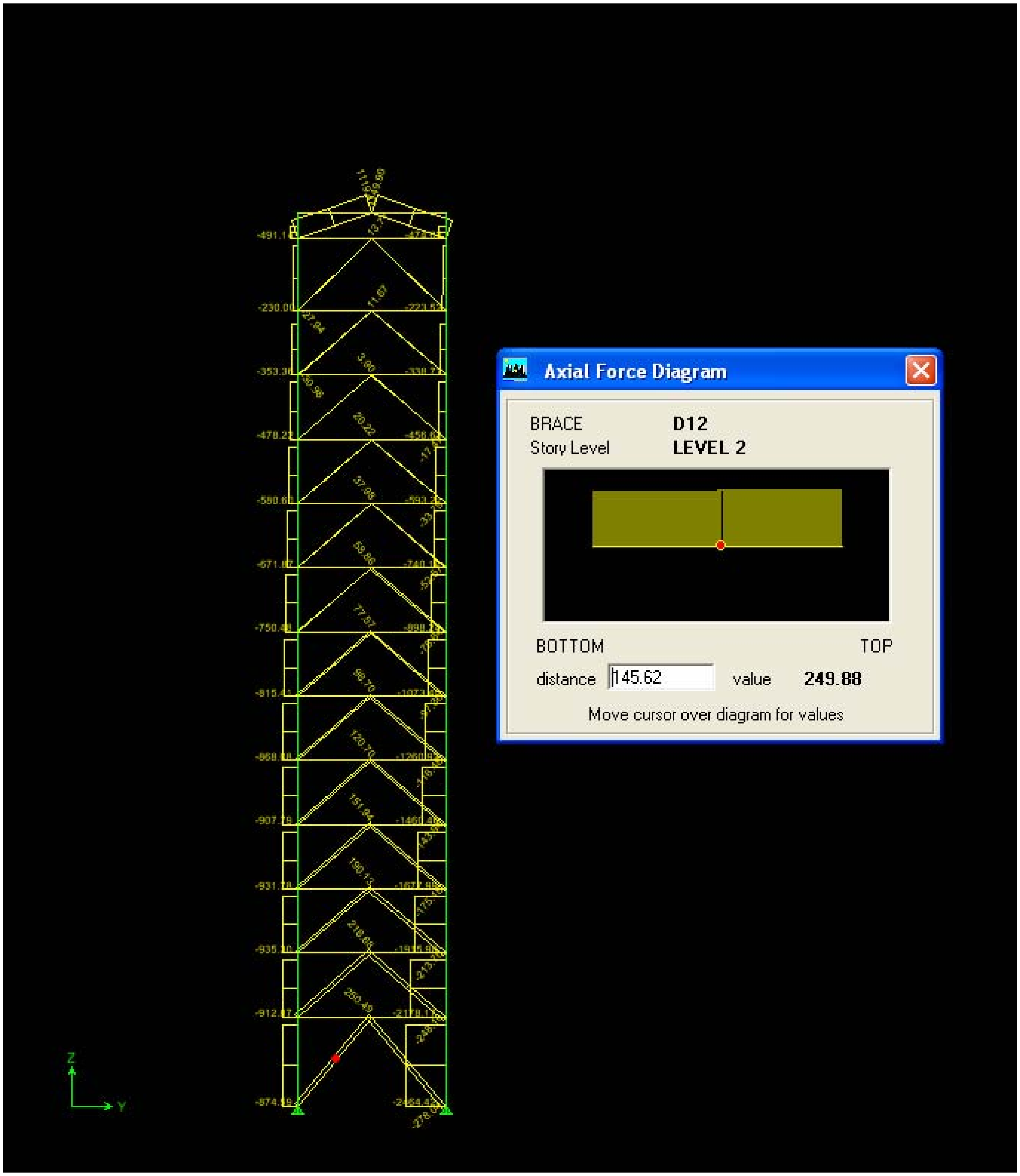
Computer Analysis – Single Frame – Frame #3 - Axial Member Force – Left Brace



Computer Analysis – Whole Building – Frame #3 - Member Design Check



Computer Analysis – Whole Building – Frame #3 - Axial Member Force – Left Brace



Computer Analysis – Whole Building – Frame #3 - Axial Member Force – Right Brace

Summary of Results of Lateral Members in Braced Frame #3 Analysis

	Single Frame	Whole Building	Existing Design	Hand Calculations
Axial Force (Left Brace)	255.48 k	249.88 k	414 k (T) 391 k (C)	358 k (T) 358 k (C)
Axial Force (Right Brace)	-358.87 k	-277.58 k	414 k (T) 391 k (C)	358 k (T) 358 k (C)
HSS 10 x 10 x ½ OK for Left Brace?	Yes	Yes	Yes	Yes
HSS 10 x 10 x ½ OK for Right Brace?	No	Yes	Yes	Yes

The results across all analyses show that an HSS 10 x 10 x ½ is an adequate section for the bottom left brace of Frame #3. All except the Single Frame analysis agree that an HSS 10 x 10 x ½ is also an adequate section for the right brace. Upon further inspection of the Single Frame member design check results, the member is at 1.044 which is very close to 1.0, which means it is very close to being an adequate member for the right brace. An HSS 10 x 10 x ½ has an axial capacity of 774 k which is greater than any axial force value reported from any of the analysis results. In conclusion, an HSS 10 x 10 x ½ is an adequate section for both the left and right bottom brace of Frame #3.

Braced Frame Member Design Checks

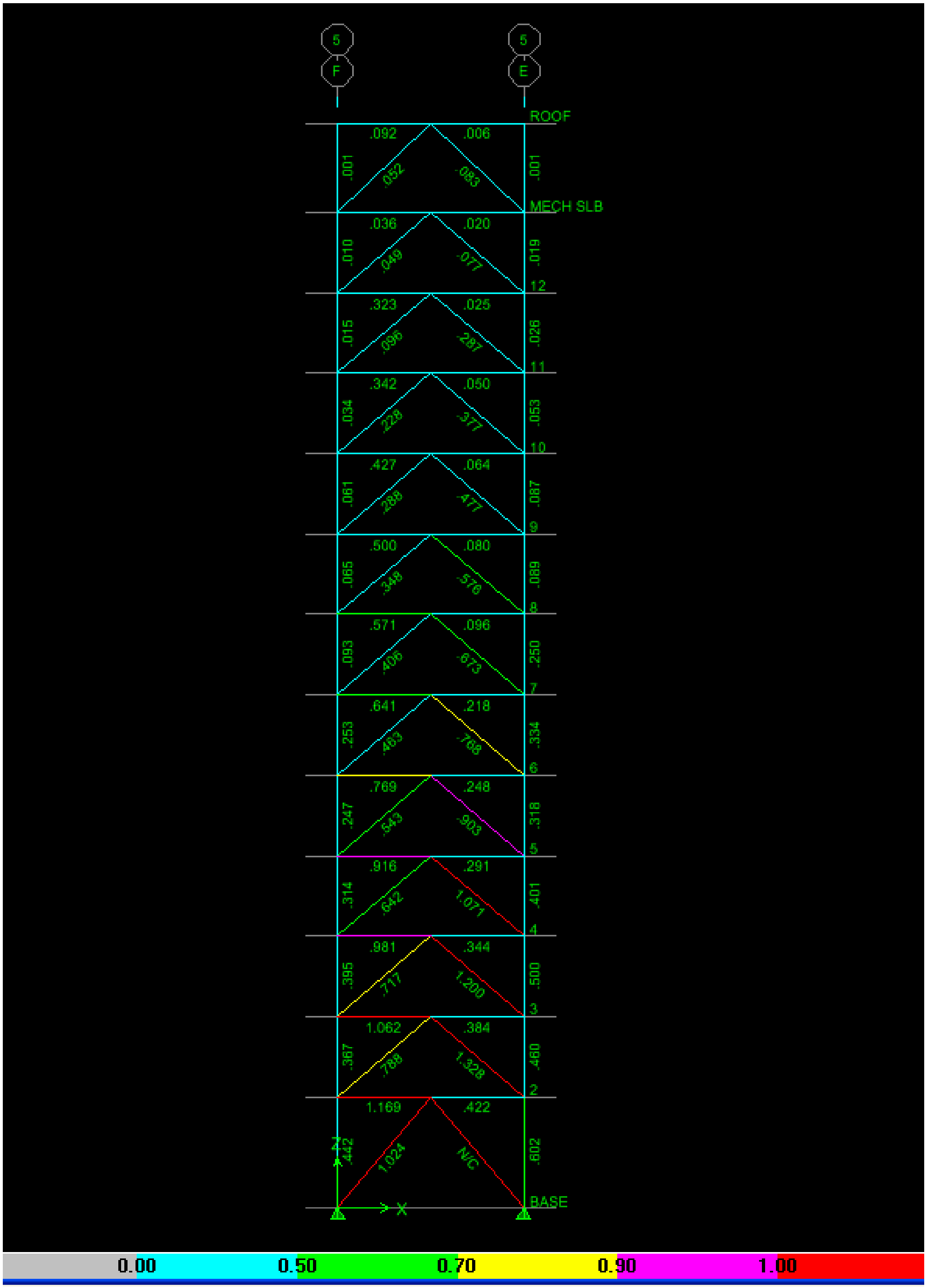
This section includes the results of the member design checks done in both the Single Frame computer models and the Whole Building computer model. The members in red are members that are not adequate to carry the loads applied.

The results of the Single Frame member design checks show that most of the braced frame members are able to carry the calculated lateral loads. There are a few red members in each braced frame including Frames #3 and #4, which have the most members not meeting the design check. However, for the results of the Single Frame analyses, the majority of the braced frame members are adequate to carry the applied calculated lateral loads, which could imply that the calculated applied loads must be similar to the actual design loads.

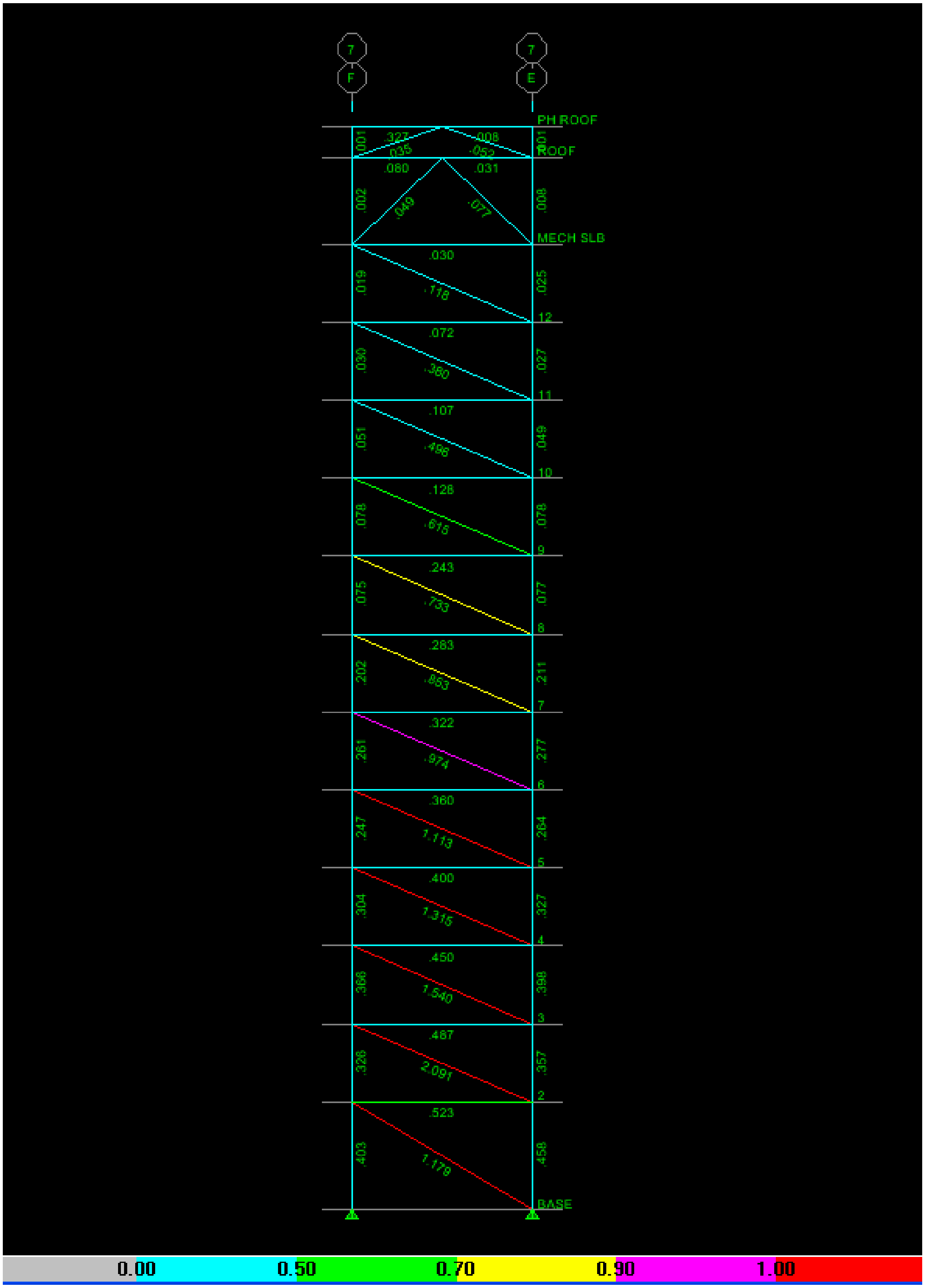
The results of the Whole Building member design checks show that there are more members that are not adequate to carry the applied calculated lateral loads in comparison to the Single Frame analysis. The Whole Building model more closely represents how the lateral system works together with the rigid floor diaphragm in order to resist the lateral loads. Although torsion to the lateral system was accounted for in the lateral distribution of the loads to each braced frame, there may still be some additional torsional effects that were not accounted for in the lateral resisting system, which would induce more load and stresses into the lateral braced frame members. Since there are several braced frame members that are not adequate to carry the applied lateral loads in this model, it can be concluded that the applied calculated lateral forces are too conservative, the model is not an accurate representation of the lateral load resisting system, or the members are not adequate to carry the applied lateral loads.

The initial conclusion is that the computer models are not accurately representing the lateral force resisting system as it was designed and the models will be corrected or reviewed further in order to make sure that they are accurately representing the existing lateral force resisting system.

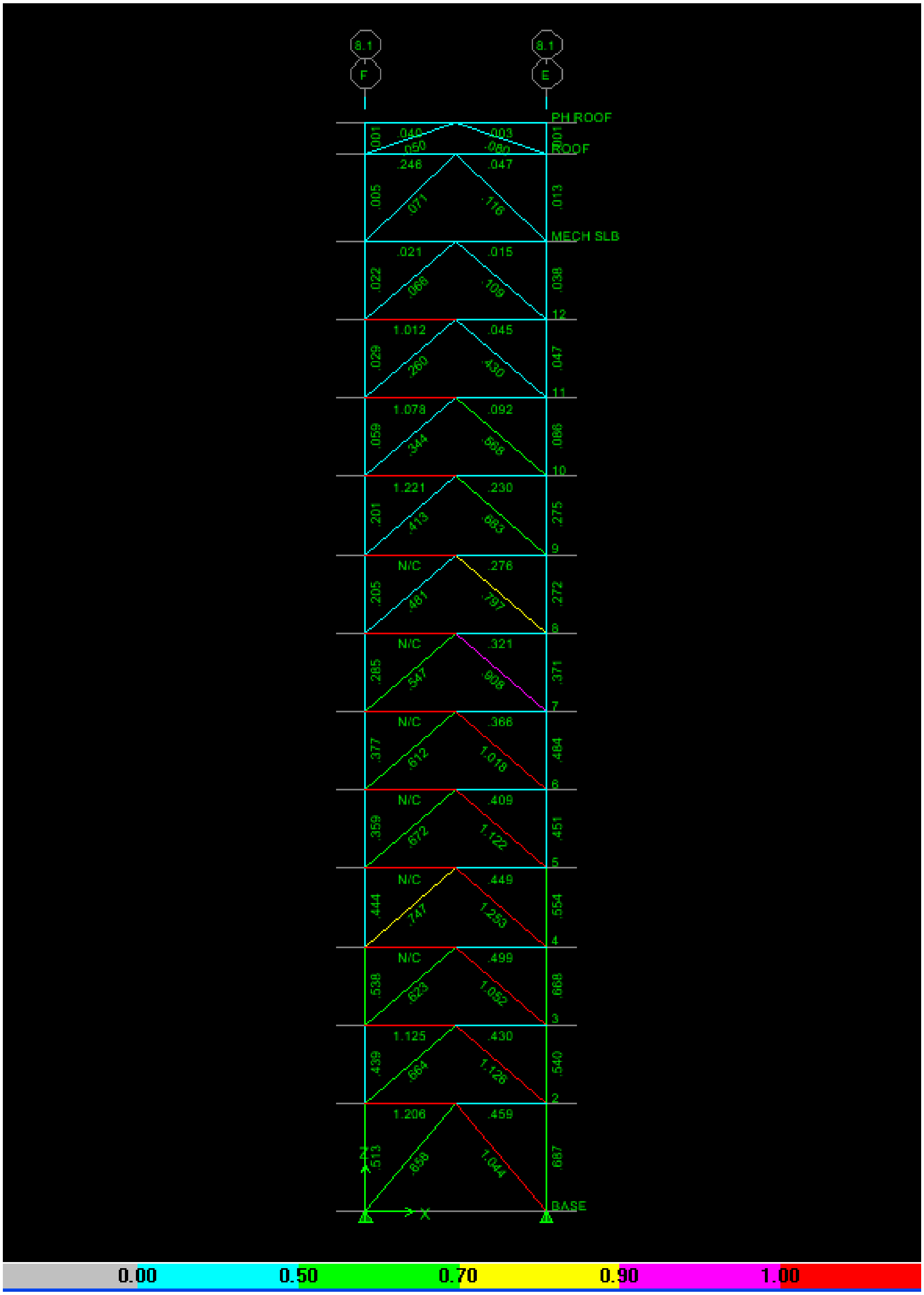
Computer Analysis – Single Frame - Frame #1 – Member Design Check



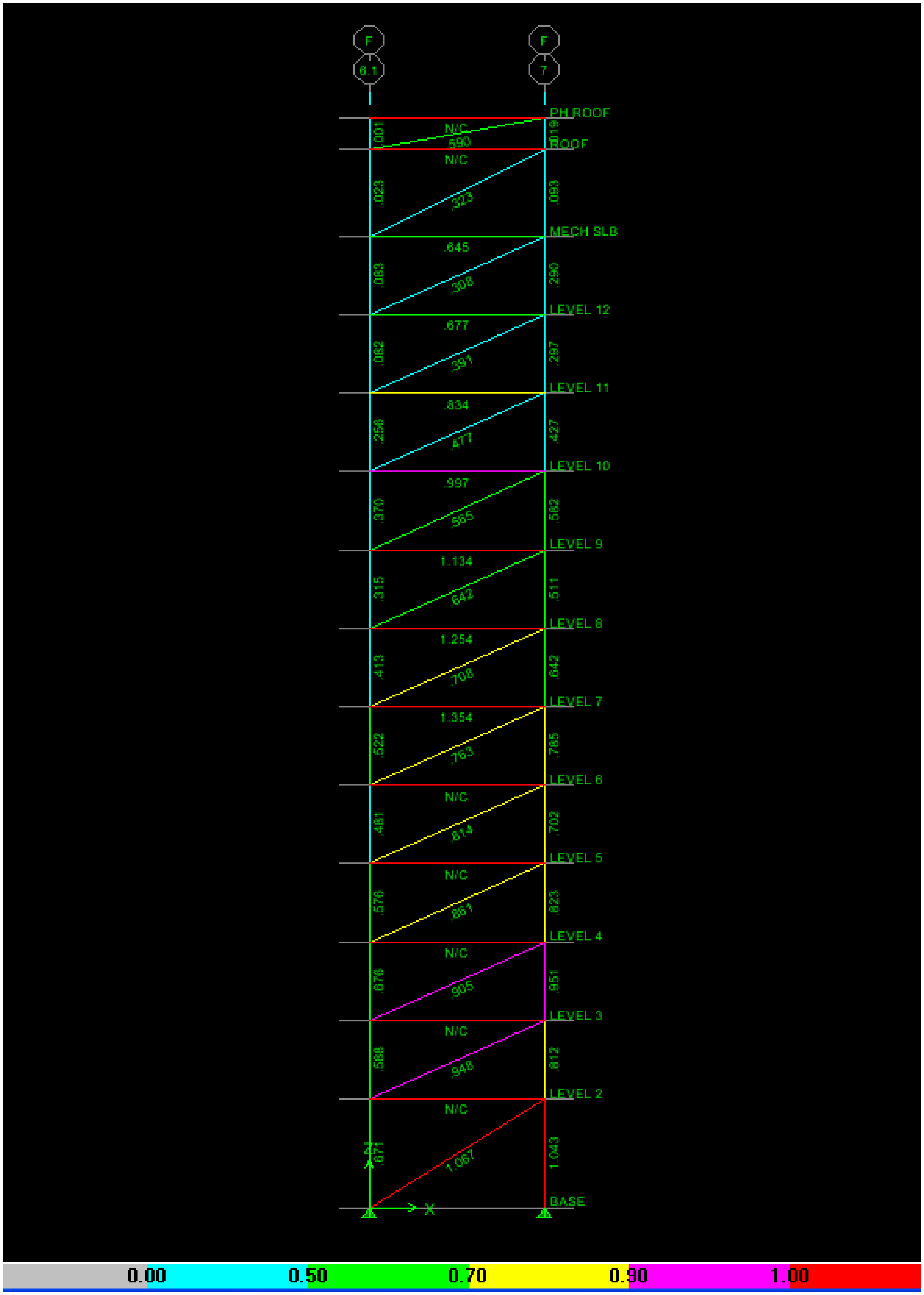
Computer Analysis – Single Frame - Frame #2 – Member Design Check



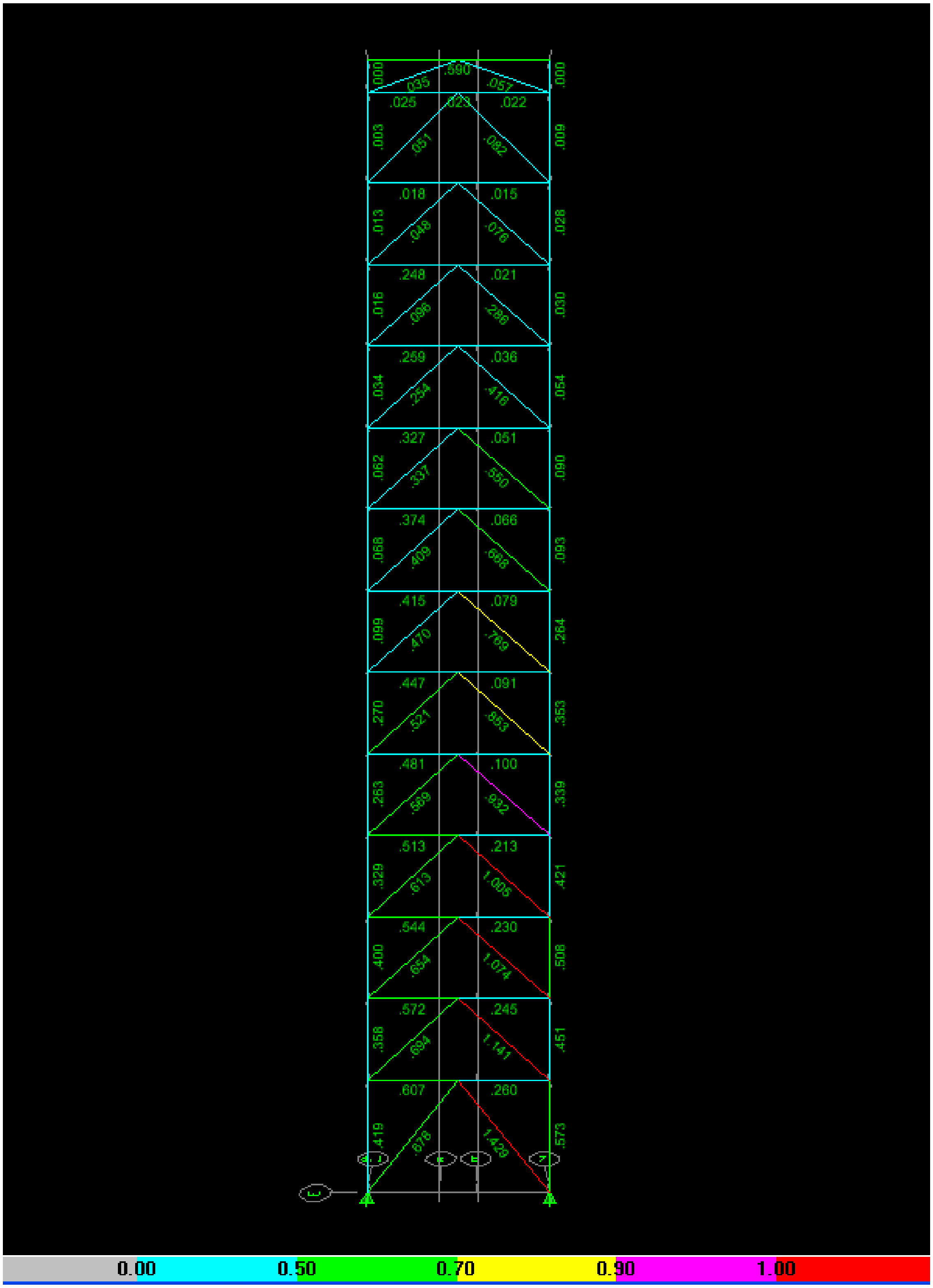
Computer Analysis – Single Frame - Frame #3 – Member Design Check



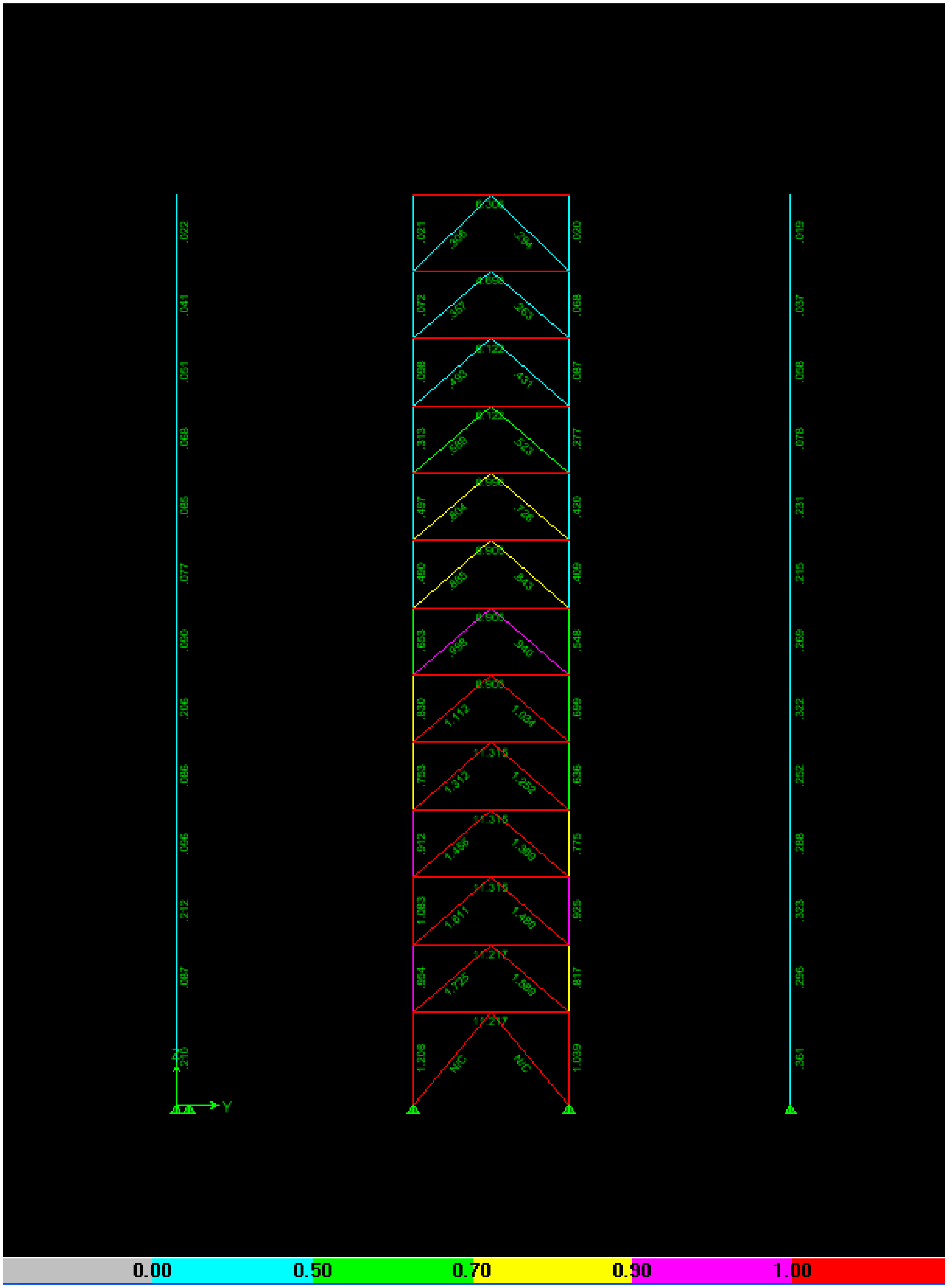
Computer Analysis – Single Frame – Frame #4 – Member Design Check



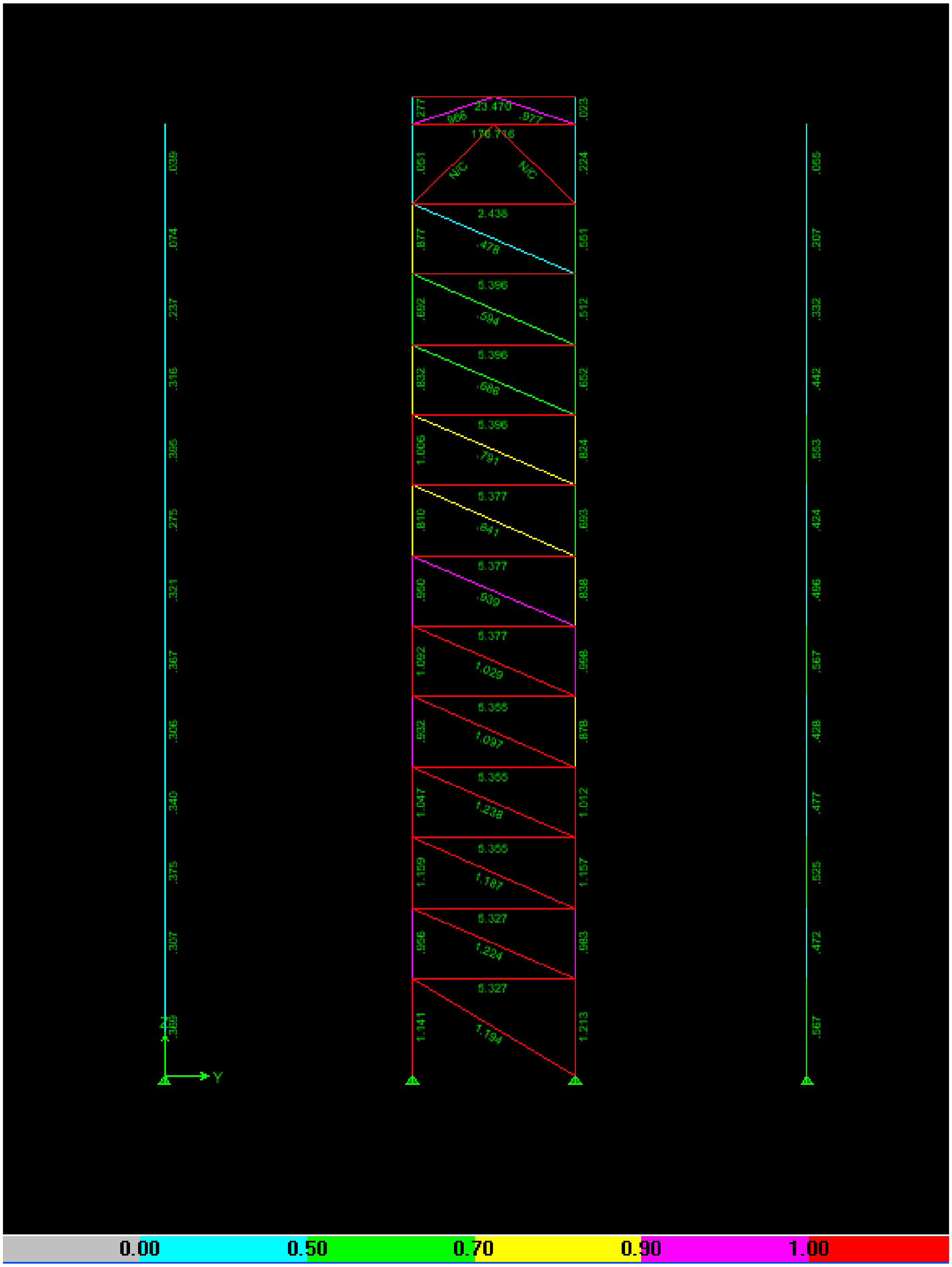
Computer Analysis – Single Frame – Frame #5 – Member Design Check



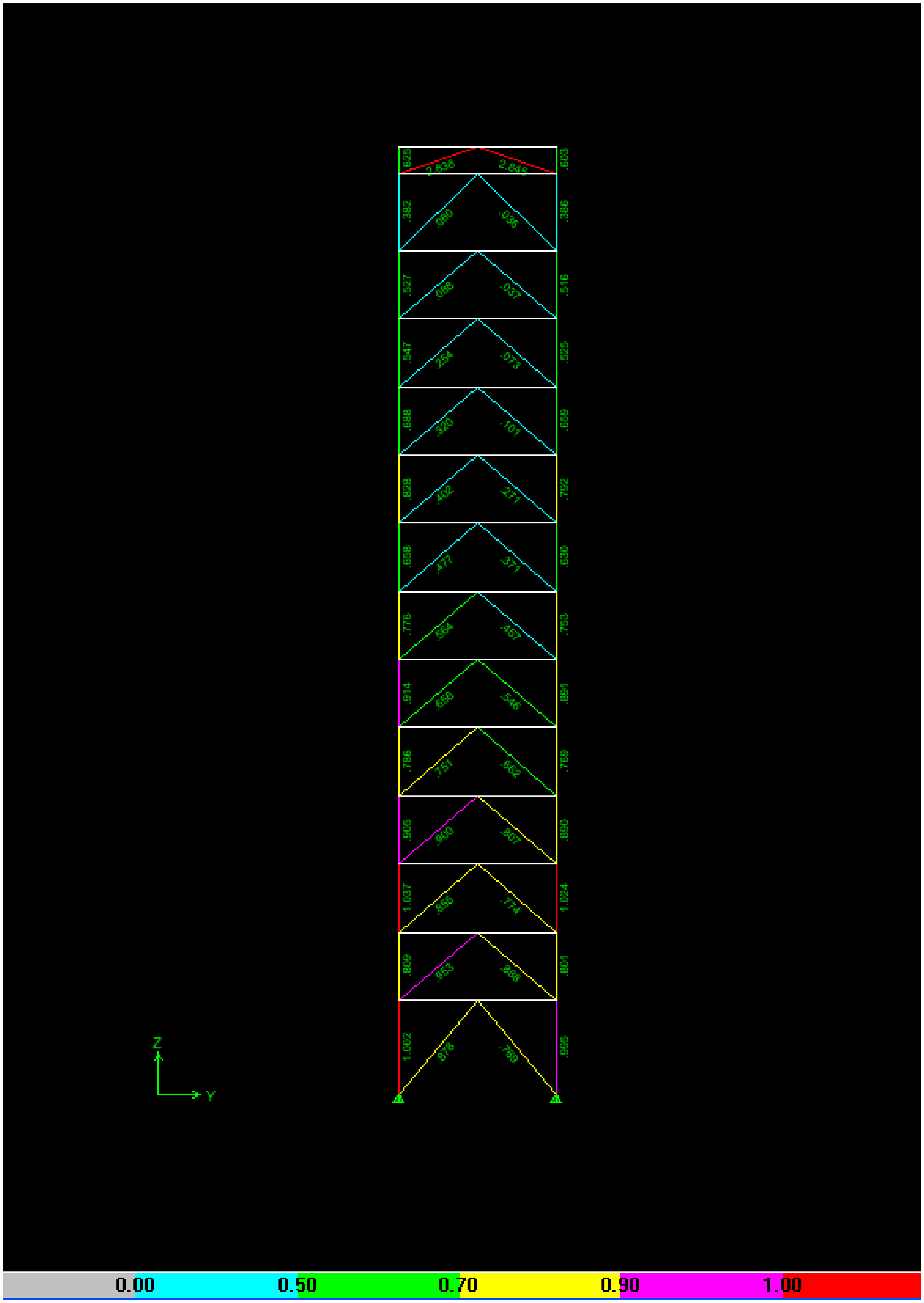
Computer Analysis – Whole Building – Frame #1 – Member Design Check



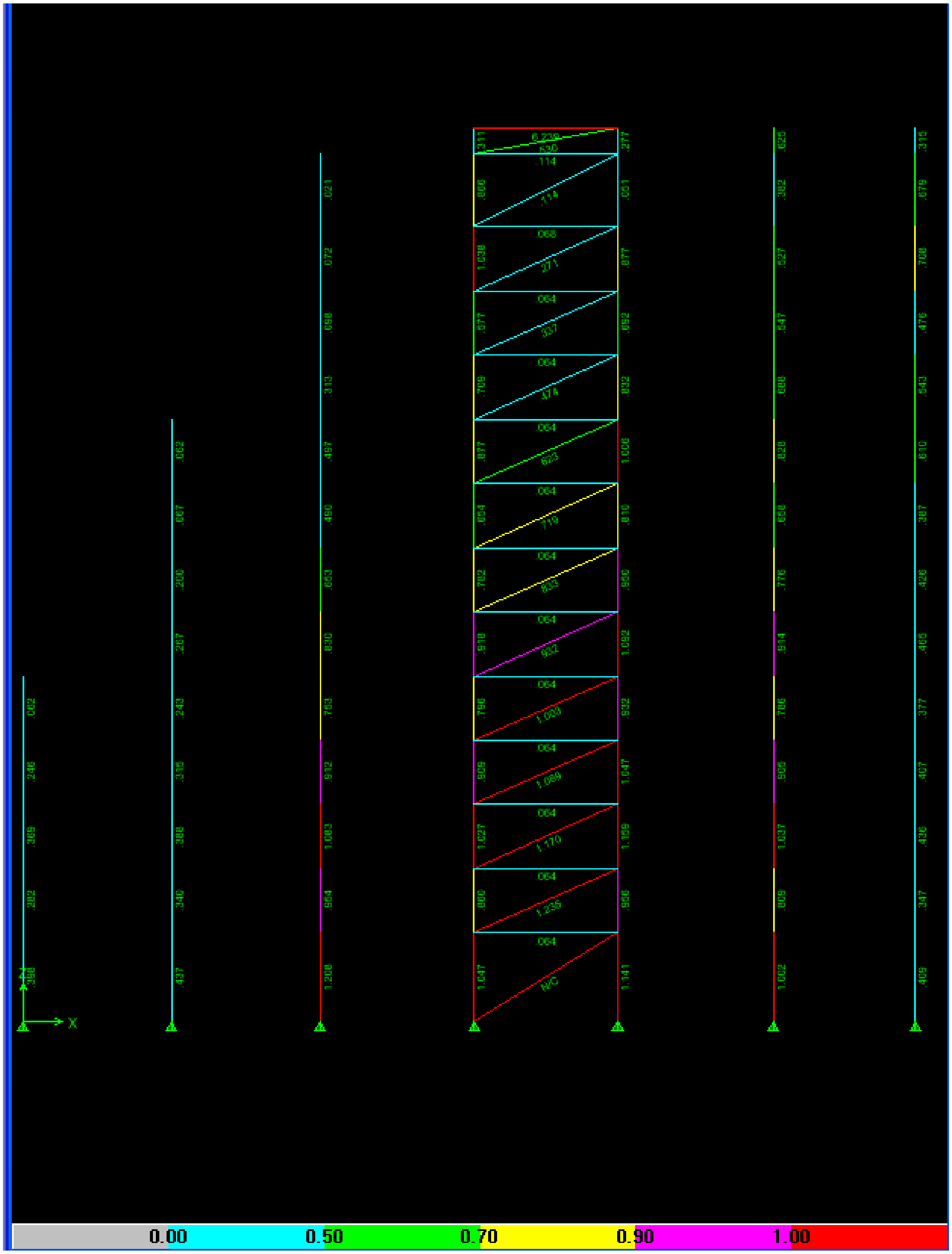
Computer Analysis – Whole Building – Frame #2 – Member Design Check



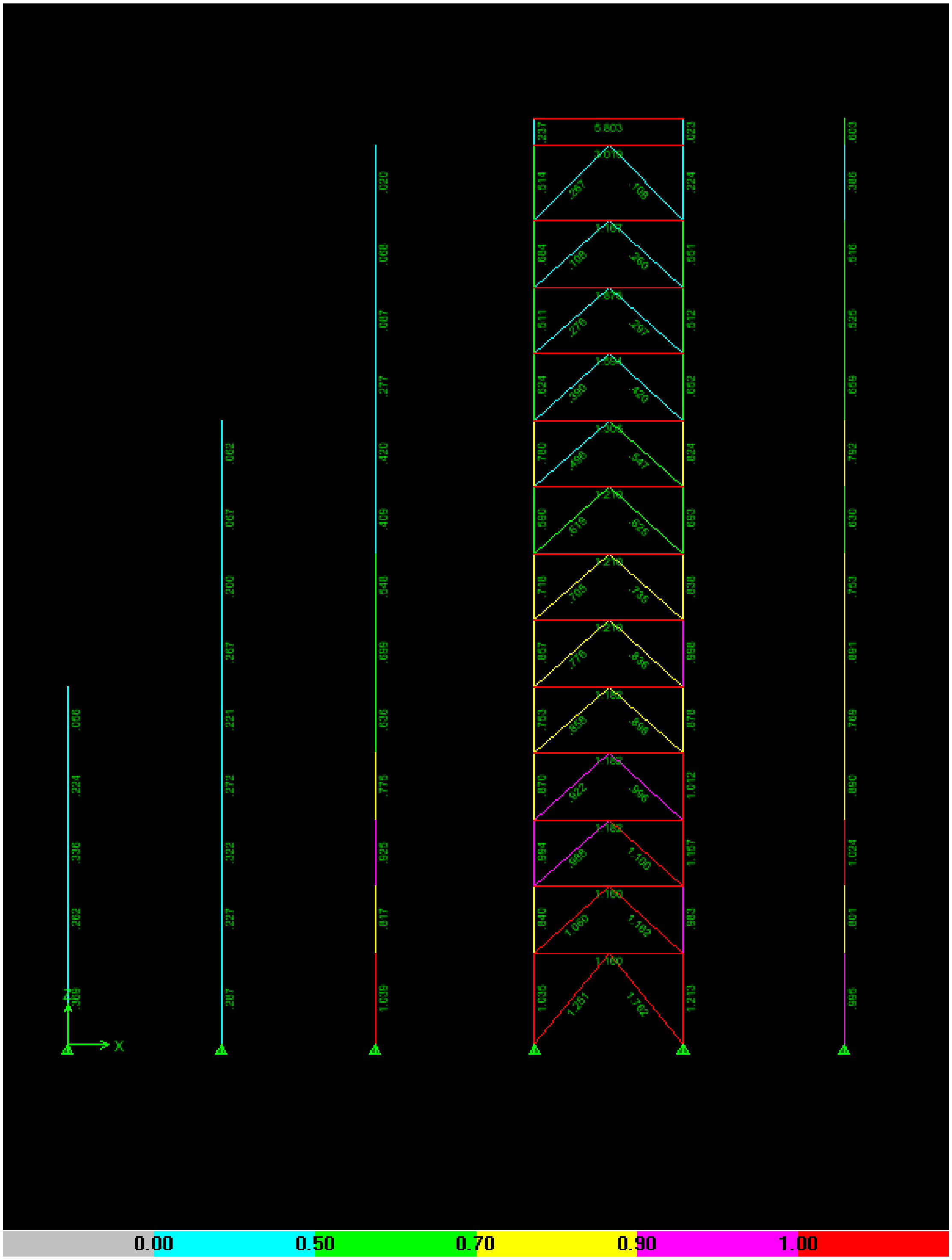
Computer Analysis – Whole Building – Frame #3 – Member Design Check



Computer Analysis – Whole Building – Frame #4 – Member Design Check



Computer Analysis – Whole Building – Frame #5 – Member Design Check



Drift and Story Drift Checks

The allowable drift calculations are based off of the industry standards of L/400 and L/360.

Allowable Drift Calculations

Entire Building (L/400)

$$Drift_{Allowable} = \frac{L}{400}$$

$$Drift_{Allowable} = \frac{176'(12''/\,ft)}{400}$$

$$Drift_{Allowable} = 5.28''$$

Entire Building (L/360)

$$Drift_{Allowable} = \frac{L}{360}$$

$$Drift_{Allowable} = \frac{176'(12''/\,ft)}{360}$$

$$Drift_{Allowable} = 5.87''$$

Allowable Drifts for Each Floor Height

	L/400 (in)	L/360 (in)
13’	0.39	0.43
18’	0.54	0.60

Braced Frame Story Displacements and Drifts

Computer Analysis – Whole Building

Computer Analysis – Whole Building – Frame #1 – Lateral Displacements and Drifts (in)

STORY	DISP-X	DISP-Y	DRIFT-X	DRIFT-Y
TO PH RF	0.000000	0.000000	0.000000	0.000000
TO ROOF	7.031006	4.641440	0.003577	0.002066
TOS M PH	6.401937	4.278112	0.003715	0.001988
LEVEL 12	5.822472	3.967930	0.003856	0.002232
LEVEL 11	5.220874	3.619809	0.003956	0.002316
LEVEL 10	4.603771	3.258589	0.004001	0.002380
LEVEL 9	3.979593	2.887269	0.003924	0.002482
LEVEL 8	3.367408	2.500133	0.003822	0.002483
LEVEL 7	2.771131	2.112725	0.003620	0.002424
LEVEL 6	2.206487	1.734580	0.003372	0.002419
LEVEL 5	1.680491	1.357284	0.003089	0.002318
LEVEL 4	1.198597	0.995606	0.002762	0.002132
LEVEL 3	0.767761	0.662971	0.002397	0.001978
LEVEL 2	0.393906	0.354470	0.001824	0.001641

Computer Analysis – Whole Building – Frame #2 – Lateral Displacements and Drifts (in)

STORY	DISP-X	DISP-Y	DRIFT-X	DRIFT-Y
TO PH RF	7.285309	4.164485	0.004093	0.001757
TO ROOF	7.031006	4.055329	0.003577	0.001719
TOS M PH	6.401937	3.752915	0.003715	0.001696
LEVEL 12	5.822472	3.488319	0.003856	0.001926
LEVEL 11	5.220874	3.187856	0.003956	0.002022
LEVEL 10	4.603771	2.872428	0.004001	0.002099
LEVEL 9	3.979593	2.544937	0.003924	0.002200
LEVEL 8	3.367408	2.201733	0.003822	0.002218
LEVEL 7	2.771131	1.855778	0.003620	0.002182
LEVEL 6	2.206487	1.515383	0.003372	0.002189
LEVEL 5	1.680491	1.173953	0.003089	0.002150
LEVEL 4	1.198597	0.838510	0.002762	0.001866
LEVEL 3	0.767761	0.547479	0.002397	0.001751
LEVEL 2	0.393906	0.274251	0.001824	0.001270

Computer Analysis – Whole Building – Frame #3 – Lateral Displacements and Drifts (in)

STORY	DISP-X	DISP-Y	DRIFT-X	DRIFT-Y
TO PH RF	7.285309	3.841129	0.004093	0.001505
TO ROOF	7.031006	3.747622	0.003577	0.001538
TOS M PH	6.401937	3.477186	0.003715	0.001543
LEVEL 12	5.822472	3.236524	0.003856	0.001766
LEVEL 11	5.220874	2.961081	0.003956	0.001868
LEVEL 10	4.603771	2.669694	0.004001	0.001952
LEVEL 9	3.979593	2.365212	0.003924	0.002052
LEVEL 8	3.367408	2.045073	0.003822	0.002078
LEVEL 7	2.771131	1.720881	0.003620	0.002055
LEVEL 6	2.206487	1.400305	0.003372	0.002068
LEVEL 5	1.680491	1.077703	0.003089	0.002062
LEVEL 4	1.198597	0.756035	0.002762	0.001726
LEVEL 3	0.767761	0.486845	0.002397	0.001633
LEVEL 2	0.393906	0.232136	0.001824	0.001075

Computer Analysis – Whole Building – Frame #4 – Lateral Displacements and Drifts (in)

STORY	DISP-X	DISP-Y	DRIFT-X	DRIFT-Y
TO PH RF	6.977351	4.164485	0.003854	0.001757
TO ROOF	6.737951	4.055329	0.003404	0.001719
TOS M PH	6.139338	3.752915	0.003568	0.001696
LEVEL 12	5.582667	3.488319	0.003704	0.001926
LEVEL 11	5.004898	3.187856	0.003809	0.002022
LEVEL 10	4.410691	2.872428	0.003861	0.002099
LEVEL 9	3.808427	2.544937	0.003783	0.002200
LEVEL 8	3.218208	2.201733	0.003689	0.002218
LEVEL 7	2.642657	1.855778	0.003499	0.002182
LEVEL 6	2.096888	1.515383	0.003257	0.002189
LEVEL 5	1.588825	1.173953	0.003005	0.002150
LEVEL 4	1.120049	0.838510	0.002628	0.001866
LEVEL 3	0.710015	0.547479	0.002283	0.001751
LEVEL 2	0.353796	0.274251	0.001638	0.001270

Computer Analysis – Whole Building – Frame #5 – Lateral Displacements and Drifts (in)

STORY	DISP-X	DISP-Y	DRIFT-X	DRIFT-Y
TO PH RF	7.285309	4.164485	0.004093	0.001757
TO ROOF	7.031006	4.055329	0.003577	0.001719
TOS M PH	6.401937	3.752915	0.003715	0.001696
LEVEL 12	5.822472	3.488319	0.003856	0.001926
LEVEL 11	5.220874	3.187856	0.003956	0.002022
LEVEL 10	4.603771	2.872428	0.004001	0.002099
LEVEL 9	3.979593	2.544937	0.003924	0.002200
LEVEL 8	3.367408	2.201733	0.003822	0.002218
LEVEL 7	2.771131	1.855778	0.003620	0.002182
LEVEL 6	2.206487	1.515383	0.003372	0.002189
LEVEL 5	1.680491	1.173953	0.003089	0.002150
LEVEL 4	1.198597	0.838510	0.002762	0.001866
LEVEL 3	0.767761	0.547479	0.002397	0.001751
LEVEL 2	0.393906	0.274251	0.001824	0.001270

The results of the Whole Building model analysis show that the top of the building displaces approximately 7" (≈ L/300) in the north / south direction and approximately 4" (≈ L/528) in the east / west direction. According to industry standards, the top of the building is allowed to drift a total of 5.28" to meet L/400 limits and 5.87" to meet L/360 deflection limits. In the north / south direction the building exceeds both of the deflection limits by over 1". In the east / west direction, the top of the building meets both deflection limits by under and 1". Therefore, according to the results of the Whole Building model analysis, the building drift is okay in the east / west direction, but does not meet the industry standard limits in the north / south direction for the entire building displacement.

The average story drift for the Whole Building model in the north / south direction is approximately 0.6" per story. The average story drift in the east / west direction is approximately 0.35". For the 13' high stories, the L/400 and L/360 deflection limits are 0.39 in and 0.43 in, respectively. For the 18' high story, the L/400 and L/360 deflection limits are 0.54 in and 0.6 in, respectively. The story displacements in the north / south direction are exceeded by approximately 0.2" per story. In the east / west direction, the average story drift meets the L/400 and L/360 story drift limitations.

Since the deflection limits are not met in the north / south direction, this is an indication that the calculated applied lateral loads in the north / south are higher than the actual lateral loads designed for, the computer model is not accurately representing the lateral force resisting system, or L/300 was an acceptable deflection limit for the design in this direction.

Computer Analysis – Single Frame

Computer Analysis – Single Frame – Frame #1 – Lateral Displacements

Story	Displacement (in)
PH Roof	
Roof	6.60
Mech Slab	6.07
12	5.62
11	5.11
10	4.60
9	4.08
8	3.55
7	3.03
6	2.52
5	2.02
4	1.51
3	1.04
2	0.59

Computer Analysis – Single Frame – Frame #2 – Lateral Displacements

Story	Displacement (in)
PH Roof	7.16
Roof	6.97
Mech Slab	6.50
12	6.05
11	5.55
10	5.02
9	4.46
8	3.89
7	3.30
6	2.71
5	2.12
4	1.52
3	0.93
2	0.36

Computer Analysis – Single Frame – Frame #3 – Lateral Displacements

Story	Displacement (in)
PH Roof	8.44
Roof	8.17
Mech Slab	7.45
12	6.85
11	6.14
10	5.42
9	4.71
8	3.99
7	3.28
6	2.61
5	1.96
4	1.34
3	0.84
2	0.42

Computer Analysis – Whole Building – Frame #4 – Lateral Displacements

Story	Displacement (in)
PH Roof	5.49
Roof	5.12
Mech Slab	4.46
12	3.90
11	3.38
10	2.88
9	2.42
8	1.99
7	1.60
6	1.24
5	0.92
4	0.64
3	0.40
2	0.19

Computer Analysis – Whole Building – Frame #5 – Lateral Displacements

Story	Displacement (in)
PH Roof	6.64
Roof	6.43
Mech Slab	5.89
12	5.41
11	4.89
10	4.35
9	3.80
8	3.24
7	2.69
6	2.16
5	1.65
4	1.18
3	0.76
2	0.38

The displacement results for each frame are summarized below.

Frame	Average Story Displacement (in)	Maximum Total Displacement (in)
1	0.5	6.60
2	0.5	7.16
3	0.7	8.44
4	0.5	5.49
5	0.5	6.64

It can be concluded that if each frame is analyzed separately for frame displacements, all of the frames fail to meet the story drift limitations of L/400 and L/360 and only Frame #4 meets L/360 maximum building deflection limit. The braced frames will never be drifting independently of each other as represented in this set of models so the results of this analysis will neglected as a credible drift analysis check.

Hand-Calculated Story Drifts

The story drifts for Column Line 7 – Level 11 and Column Line 7 – Level 4 were calculated in order to check the results of the computer analyses. These story drift calculations can be found in the Appendix. The results of the calculations are summarized below.

	Story Drift (in)	Allowable Story Drift (L/400) (in)	Allowable Story Drift (L/360) (in)	
Column Line 7 Level 11	2.44	0.39	0.43	NOT OK
Column Line 7 Level 4	6.58	0.39	0.43	NOT OK

The hand-calculated story drift values do not seem reasonable compared to the computer analysis results and the allowable story drift limits. It can be concluded that the computer analysis results seem more reasonable than the hand-calculated story drifts and that there may be an error in the hand-calculated story drifts of which need to be revised or reviewed further.

Overturning Moments

The overturning moments are based off of the distributed factored lateral forces to each braced frame. The following chart summarizes the lateral loads and the overturning moments for the north / south and east / west directions.

		North / South			East / West			
Level	h (ft)	Frame #4 Forces (k)	Frame #5 Forces (k)	∑Forces N/S	Frame #1 Forces (k)	Frame #2 Forces (k)	Frame #3 Forces (k)	∑Forces E/W
Roof	180.75	43.34	34.23	77.57	34.44	31.16	52.90	118.50
12	148	60.14	47.50	107.64	47.27	42.77	72.61	162.65
11	135	50.16	39.62	89.78	27.41	24.80	42.10	94.31
10	122	52.10	41.15	93.25	30.71	24.95	34.93	90.59
9	109	45.29	35.77	81.06	30.15	24.50	34.30	88.95
8	96	38.68	30.55	69.23	29.45	23.93	33.50	86.88
7	83	32.28	25.50	57.78	28.66	23.29	32.60	84.55
6	70	30.06	23.74	53.80	40.54	24.86	30.73	96.13
5	57	27.57	21.78	49.35	49.78	30.52	37.74	118.04
4	44	26.36	20.82	47.18	37.89	23.23	28.72	89.84
3	31	24.93	19.69	44.62	36.18	22.18	27.43	85.79
2	18	26.76	21.14	47.90	39.57	24.26	30.00	93.83
	Overturning Moment (N/S)			84625.948	ft-k			
	Overturning Moment (E/W)			116087.36	ft-k			

Additional Overturning Moment calculations can be found in the Appendix.

Lateral Force Direction	Moment due to Self Weight (ft-k)	Overturning Moment (ft-k)		Overturning Moment / Moment due to Self Weight
North	2067769	84626	OK	0.04
South	2523769	84626	OK	0.03
East / West	1159890	116087	OK	0.1

The moments due to the self weight of the building were much greater than the overturning moments in all cases. Therefore, the building is able to resist the overturning moments induced by the lateral forces.

Overturning Moments and Their Impact on the Foundations

Since the overturning moments were significantly smaller than the moments due to self weight, the overturning moments will not have a significant effect on the design of the foundation systems, but the small overturning moment effects still need to be considered in the foundation design.

Conclusions

The bottom diagonal bracing members for Frames #2 and #3 were checked for strength. In comparing the results of all the analyses, it was determined that the existing designed members should be adequate for strength, however the computer analyses for the diagonal member check of Frame #2, found that the diagonal member was not adequate for strength. Since the calculated loads were similar in magnitude across all of the analyses, it was determined that the computer models may not be an exact representation of the lateral framing system. The models need to be reviewed further in order to figure out why the results show they are not correctly designed for strength even though the loads match the other analyses which prove that under the applied loads, the diagonal member is adequate.

All of the other frames were checked for strength in both computer models. It was determined that most of the members met the strength requirements when analyzed as a single frame, however there were more members that did not meet the strength requirements when analyzed as part of the whole lateral force resisting system. The model of the whole lateral force resisting system more closely represents the actual building design and actual configuration. Since several of the members were not meeting the design strengths, this could be an indication of several concerns:

4. The loads applied are similar to the loads applied for the existing design, but the model is not an accurate representation of the existing design
5. The loads applied are more conservative than those assumed for the existing design which are resulting in a lot of the members not meeting the design strength check
6. The members may not be conservatively designed.

The initial conclusion is that the computer models are not an exact representation of the lateral force resisting system as it was designed and the models will be corrected or reviewed further in order to make sure that they are accurately representing the existing lateral force resisting system.

The system was then checked for drift and story drift according to the industry standards of L/400 and L/360. The results of the Whole Building model analysis show that the top of the building displaces approximately 7" in the north / south direction and approximately 4" in the east / west direction. According to industry standards, the top of the building is allowed to drift a total of 5.28" to meet L/400 deflection limits and 5.87" to meet L/360 deflection limits. In the north / south direction the building exceeds both of the deflection limits by over 1". In the east / west direction, the displacement of the top of the building meets both of the deflection limits by under and 1". Therefore, according to the results of the Whole Building model analysis, the building drift is okay in the east / west direction, but does not meet the industry standard deflection limits in the north / south direction for the entire building displacement.

The average story drift for the Whole Building model in the north / south direction is approximately 0.6” per story. The average story drift in the east / west direction is approximately 0.35”. For the 13’ high stories, the L/400 and L/360 deflection limits are 0.39” and 0.43”, respectively. For the 18’ high story, the L/400 and L/360 deflection limits are 0.54” and 0.6”, respectively. The story displacements in the north / south direction are exceeded by approximately 0.2” per story. In the east / west direction, the average story drift meets the L/400 and L/360 story drift limitations.

Since the deflection limits are not met in the north / south direction, this is an indication that the calculated applied lateral loads in the north / south are higher than the actual lateral loads designed for, the computer model is not accurately representing the lateral force resisting system, or L/300 was an acceptable deflection limit for the design in this direction.

The results of the Single Frame model drift and story drift calculations concluded that if each frame is analyzed separately for frame displacements, all of the frames fail to meet the story drift limitations of L/400 and L/360 and only Frame #4 meets L/360 building deflection limit, while the other four frames do not meet any of the industry standards for total building drift.

The overturning moments for The Regent were calculated based off of the controlling lateral force distributed to each braced frame. The moments due to the self weight of the building were much greater than the overturning moments in all cases. Therefore, the building is able to resist the overturning moments induced by the lateral forces.

In conclusion, the lateral system and confirmation of design analyses performed for this report concluded that the building, as analyzed, does not meet all strength, drift, and story drift requirements. This is an indication that the critical load path for distribution of the designed structure does not match the analyses performed in this report. Further research and analyses will determine where and why the critical load paths do not match up. The computer models will be revised to more accurately represent the existing designed structure in order to be able to determine if the designed lateral system is adequate for the calculated loads. It is also a possibility that the calculated lateral loads used in all of the computer models and hand calculations are more conservative than those used in the actual design of the lateral load resisting system or that the lateral loads were not distributed properly among all of the braced frames. Further research and analysis will determine the accuracy of the computer models, the accuracy of the calculated applied lateral loads, and the accuracy of the distribution of the lateral loads to each lateral load resisting element. The results of all methods of analysis need to coincide so that it can be assumed that the critical load paths of the existing system match the critical loads paths developed through this series of technical reports.

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

Internal Pressure Coefficients (6.5.11.1)

GC_{pi} = +0.18
= -0.18
$q_i = q_h$ = 20.65 PSF ($q_i = q_h$ for windward and leeward walls of enclosed buildings)

Internal Pressure = $qiGC_{pi} = \pm 20.65 PSF(0.18) = \pm 3.72 PSF$

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)

$f = \dfrac{1}{C_t h_n^x} = \dfrac{1}{0.02(180.75)^{0.75}} = 1.01Hz > 1.0 \therefore Rigid$ (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)

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

$G = 0.925\left(\dfrac{1+1.7g_Q I_z Q}{1+1.7g_V I_z}\right) = 0.925\left(\dfrac{1+1.7(3.4)(0.246)(0.82)}{1+1.7(3.4)(0.246)}\right) = 0.83$ (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)

$f = \frac{1}{C_t h_n^x} = \frac{1}{0.02(180.75)^{0.75}} = 1.01Hz > 1.0 \therefore Rigid$ (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)

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

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

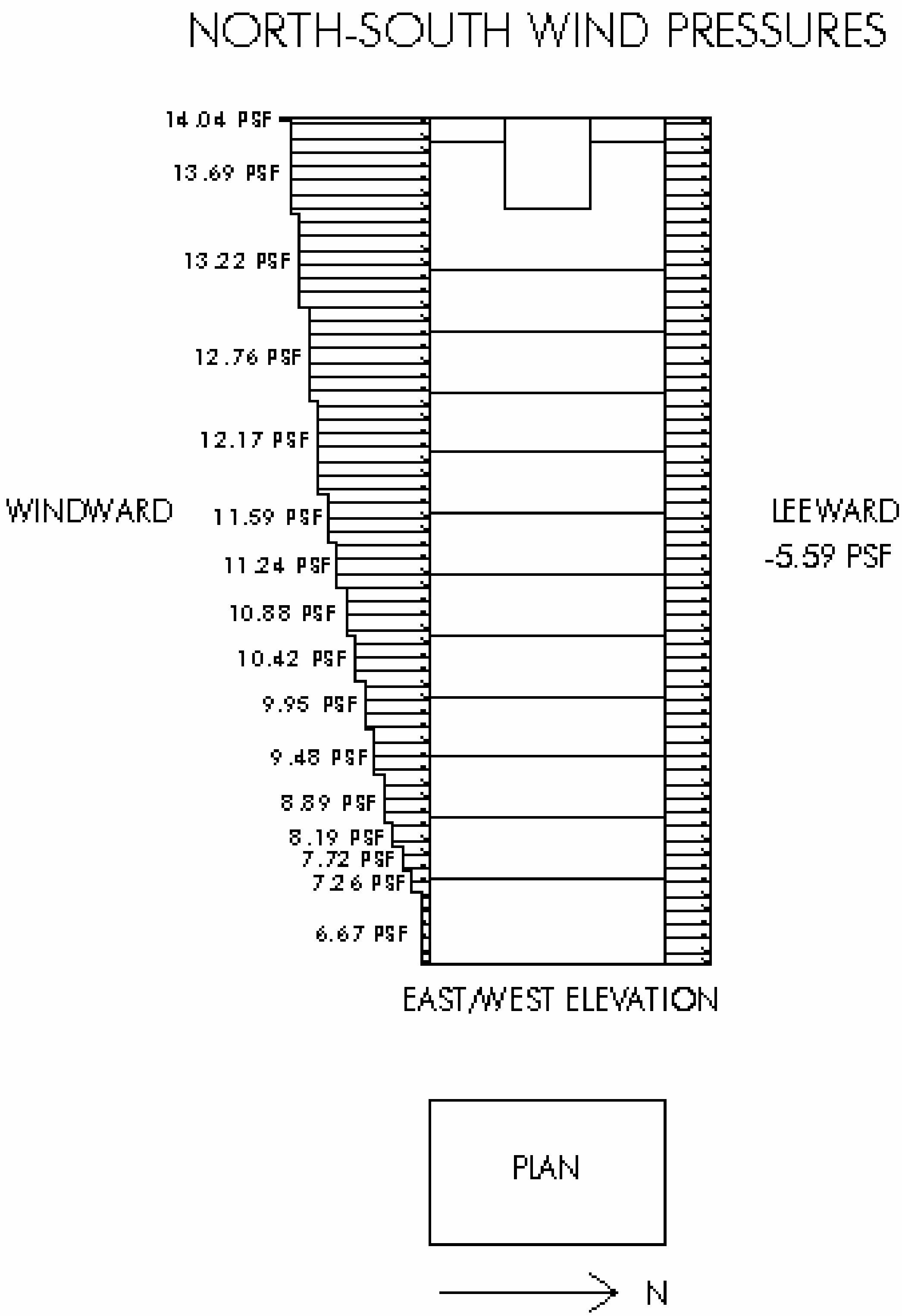
E-W Leeward Pressure

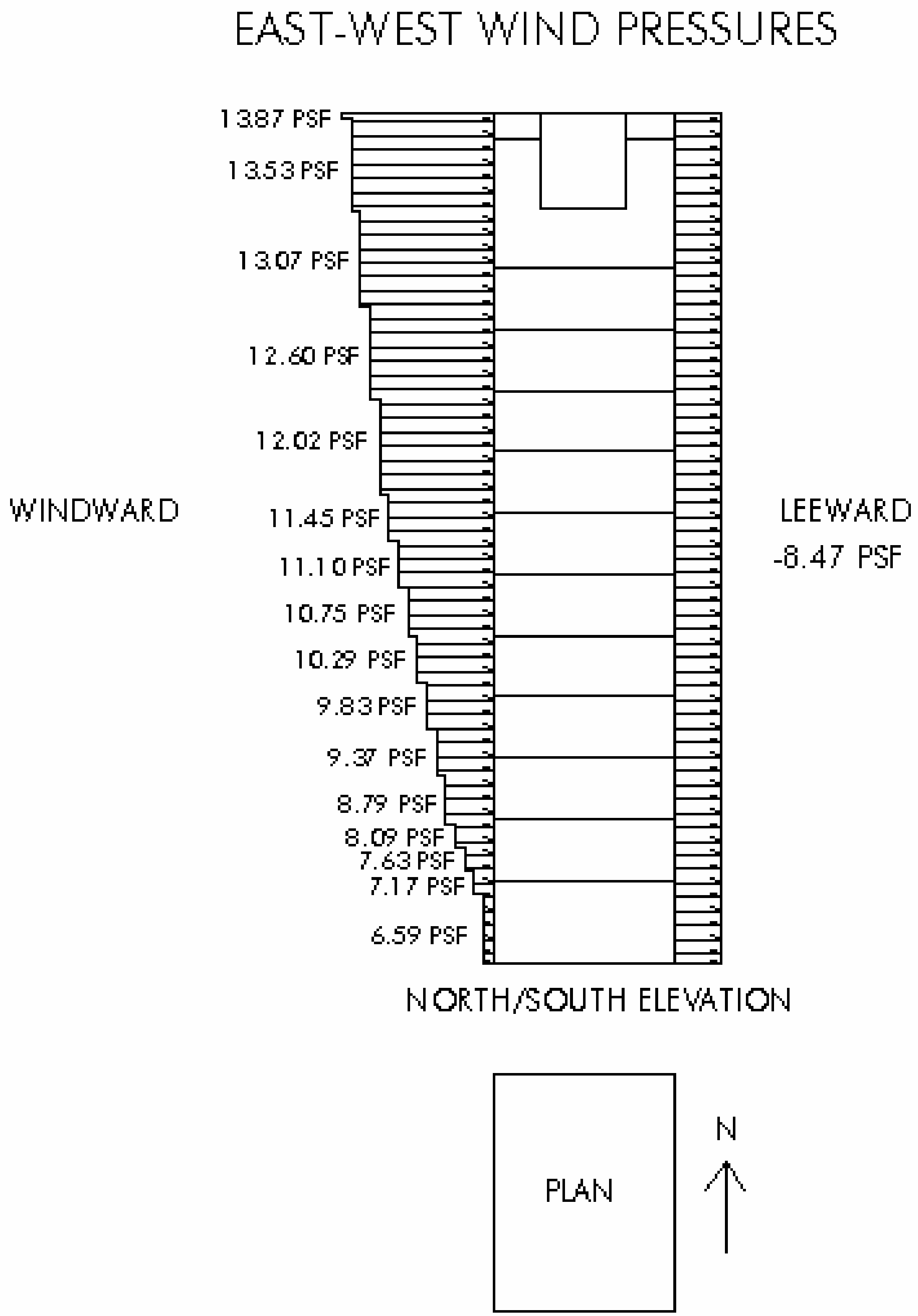
$$P_{lh} = q_h C_p G = 20.65(-0.5)(0.82) = -8.47 \text{ PSF}$$

Total Pressures

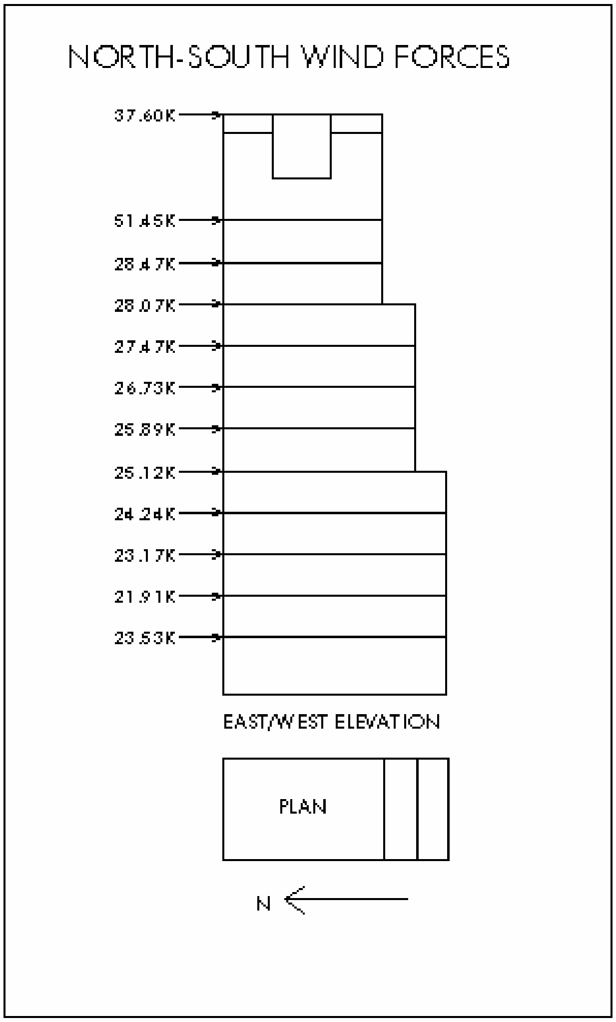
z	Kz	qz	N-S Windward Pressure (PSF)	E-W Windward Pressure (PSF)	N-S Leeward Pressure (PSF)	E-W Leeward Pressure (PSF)	P_{total} (N-S) (PSF)	P_{total} (E-W) (PSF)
0-15	0.57	10.05	6.67	6.59	-5.59	-8.47	12.26	15.06
20	0.62	10.93	7.26	7.17	-5.59	-8.47	12.85	15.64
25	0.66	11.63	7.72	7.63	-5.59	-8.47	13.31	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

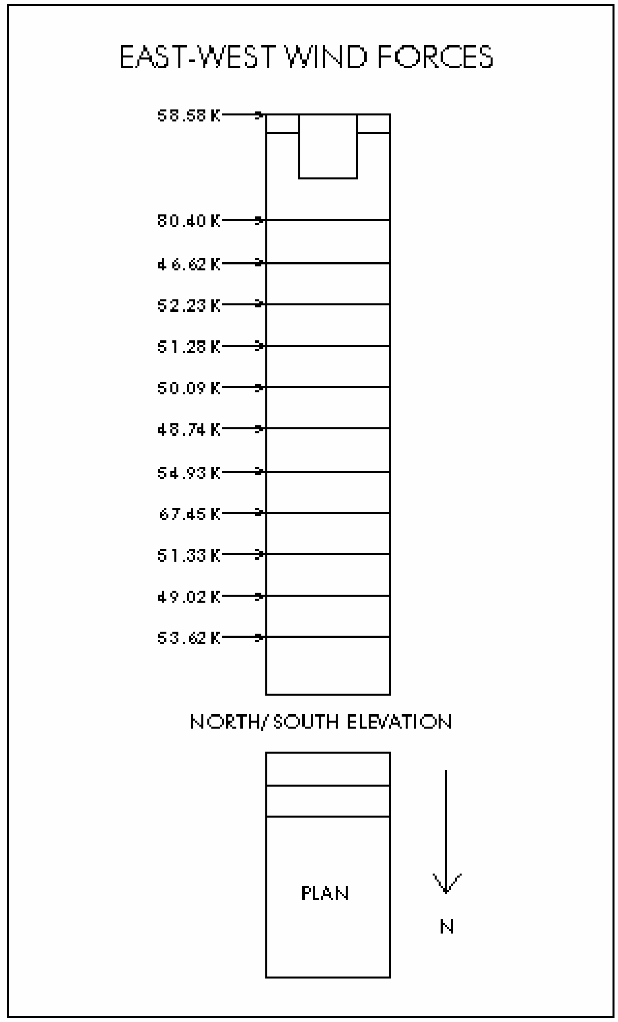
Wind Pressure Diagrams




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 (1^{st} 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:	
$F_a = 1.2$	(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*   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$_S$W)

R = 3 (Table 9.5.2.2) Structural steel systems not specifically detailed for seismic resistance.

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

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

Dead Loads

Roof Dead Load

Metal Deck	5 PSF
Insulation	3 PSF
Misc. DL	10 PSF
Roofing	20 PSF
	38 PSF

Snow Load 30 PSF (See Snow Load Calculations)

NOTE: Since Snow Load is not greater than 30 PSF, 20% of the Snow Load does not need to be considered in the weight 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

$$w_{roof} = 909k$$
$$w_{11} = 1617k$$
$$w_{10} = 1512k$$
$$w_{9-6} = 1781k$$
$$w_{5-2} = 2050k$$
$$w_{1} = 2083k$$

$$W = w_{roof} + w_{11} + w_{10} + 4w_{9-6} + 4w_{5-2} + w_{1}$$
$$W = 909k + 1617k = 1512k + 4(1781k) + 4(2050k) + 2083k$$
$$W = 21{,}445k$$

$V_{N\text{-}S}$ = 0.02681(21,445k) = 574.94k
$V_{E\text{-}W}$ = 0.02681(21,445k) = 574.94k

$$F_x = C_{vx} V$$

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

$$k(N-S) = 1 + \frac{0.986 - 0.5}{2} = 1.243^*$$ *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)	909	180.75	580915	580915	0.106	0.106	60.96	60.96
11	1617	148	806019	806019	0.147	0.147	84.58	84.58
10	1512	135	672290	672290	0.123	0.123	70.55	70.55
9	1781	122	698247	698247	0.127	0.127	73.27	73.27
8	1781	109	606995	606995	0.111	0.111	63.70	63.70
7	1781	96	518355	518355	0.095	0.095	54.40	54.40
6	1781	83	432592	432592	0.079	0.079	45.40	45.40
5	2050	70	402912	402912	0.074	0.074	42.28	42.28
4	2050	57	312109	312109	0.057	0.057	32.75	32.75
3	2050	44	226238	226238	0.041	0.041	23.74	23.74
2	2050	31	146392	146392	0.027	0.027	15.36	15.36
1	2083	18	75682	75682	0.014	0.014	7.94	7.94
			5478745	5478745	1.000	1.000	574.94	574.94

k (N-S)	1.243	
k (E-W)	1.243	
V (N-S)	574.94	k
V (E-W)	574.94	k

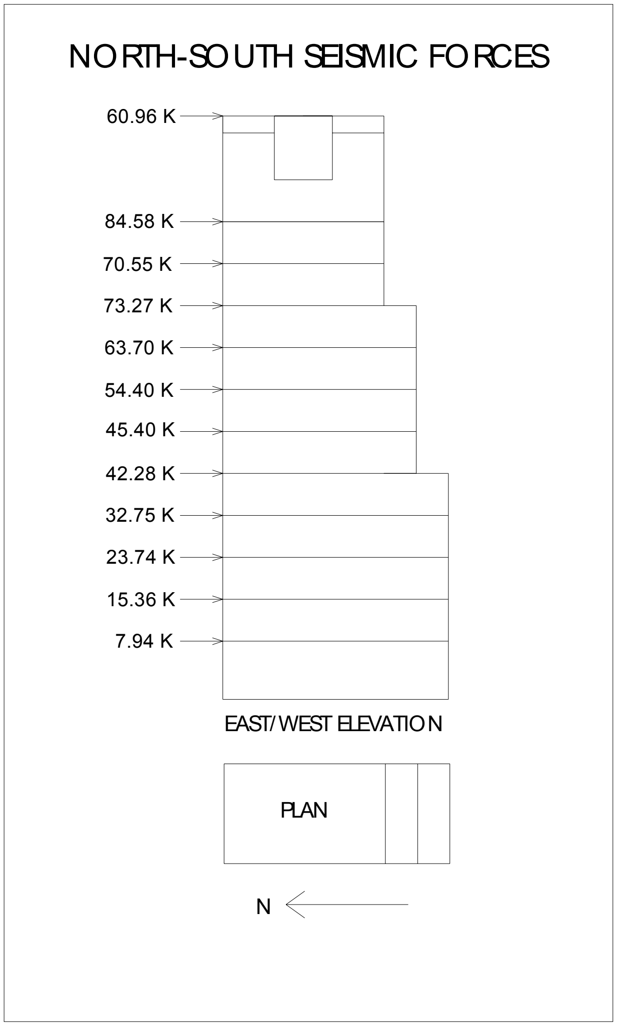
Base Shear

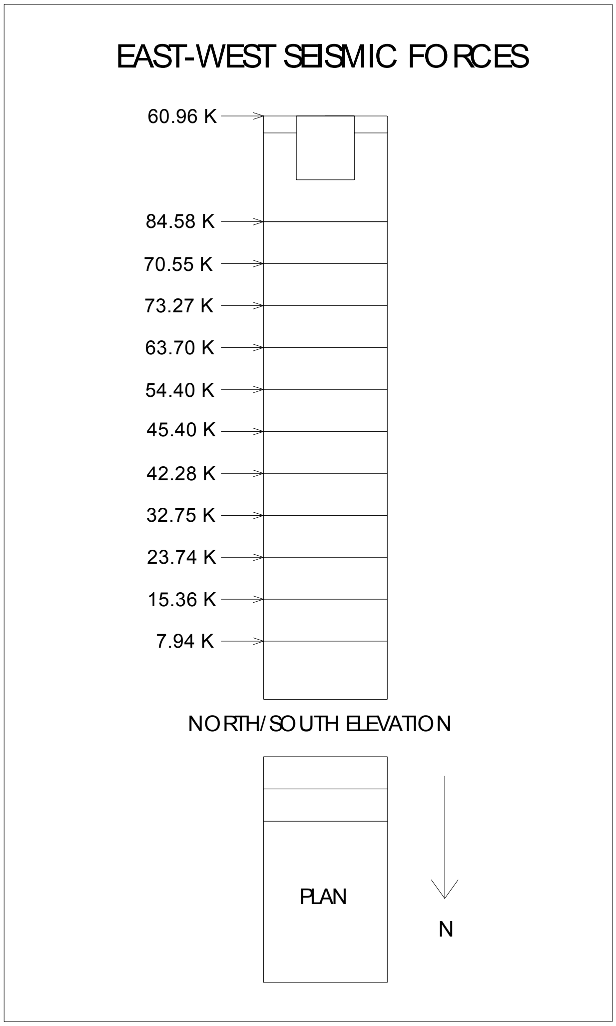
N-S	574.94	k
E-W	574.94	k

Overturning Moment

Overturning Moment (N-S)	64424.2942	ft-k
Overturning Moment (E-W)	64424.2942	ft-k

Seismic Force Diagrams





Lateral Force Distribution: Distribution by Rigidity

Each braced frame was analyzed in ETABS with a virtual 100k force applied to the top chord of each braced frame. The deflections were then recorded and are included in the following table. The relative stiffnesses were then calculated

Braced Frame Displacements and Relative Stiffnesses

	Displacement (in)	Relative K
Frame 1	4.033	1.341k
Frame 2	5.239	1.032k
Frame 3	4.241	1.276k
Frame 4	5.485	1.000k
Frame 5	4.212	1.281k

Center of Rigidity

$$\overline{x_{cr}} = \frac{1.341k(60') + 1.032k(120') + 1.276k(150')}{(1.341k + 1.032k + 1.276k)}$$

$$\overline{x_{cr}} = 108'$$

$$\overline{y_{cr}} = \frac{k(46') + 1.281k(76')}{(k + 1.281k)}$$

$$\overline{y_{cr}} = 63'$$

Center of Mass

Levels 2-6	(100,63)
Levels 7-10	(115,64)
Levels 11-Roof	(130,65)

Eccentricities

$e_y \approx 0$ (For Frames #4 and #5, an accidental torsional eccentricity was included, e_y = 0.05*222.5’ = 11.13’ for a conservative approach)

$e_{x,2\text{-}6}$ = -8’
$e_{x,7\text{-}10}$ = 7’
$e_{x,11\text{-}Roof}$ = 22’

$$J = \sum (k_i d_i^2)$$

$$F_{x,dir} = \frac{k_i}{\sum k_i} P_x$$

$$F_{y,dir} = \frac{k_i}{\sum k_i} P_y$$

$$F_{tor} = \frac{k_i d_i}{J} M_t$$

$$F_{tot} = F_{dir} + F_{tor}$$

NOTE: If $F_{tor} < 0$, then it was neglected and it was not added to F_{dir}.

Calculation of M_t for Frames #4 and #5

Level	P_x (k)	e_x (ft)	M_t (ft-k)
Roof	60.96	11.13	678
12	84.58	11.13	941
11	70.55	11.13	785
10	73.27	11.13	815
9	63.70	11.13	709
8	54.40	11.13	605
7	45.40	11.13	505
6	42.28	11.13	470
5	38.78	11.13	431
4	37.07	11.13	412
3	35.06	11.13	390
2	37.64	11.13	419

Calculation of M_t for Frames #1, #2, and #3

Level	P_y (k)	e_x (ft)	M_t (ft-k)
Roof	93.72	22	2062
12	128.64	22	2830
11	74.59	22	1641
10	83.57	7	585
9	82.05	7	574
8	80.14	7	561
7	77.98	7	546
6	87.89	-8	-703
5	107.92	-8	-863
4	82.13	-8	-657
3	78.43	-8	-627
2	85.79	-8	-686

Lateral Force Distribution: Distribution by Rigidity for Frames #4 and #5

k_4	1	k
k_5	1.281	k
Σk	2.281	k
L	222.5	ft

Level	**F_x (k)**	**$F_{4,dir}$ (k)**	**$F_{5,dir}$ (k)**	**0.05L (ft)**	**M_t (ft-k)**	**$F_{4,tor}$ (k)**	**$F_{5,tor}$ (k)**	**$F_{4,tot}$ (k)**	**$F_{5,tot}$ (k)**
Roof	60.96	26.73	34.23	11.13	678.18	16.62	-27.28	43.34	34.23
12	84.58	37.08	47.50	11.13	940.95	23.06	-37.85	60.14	47.50
11	70.55	30.93	39.62	11.13	784.87	19.23	-31.57	50.16	39.62
10	73.27	32.12	41.15	11.13	815.13	19.97	-32.79	52.10	41.15
9	63.70	27.93	35.77	11.13	708.66	17.37	-28.50	45.29	35.77
8	54.40	23.85	30.55	11.13	605.20	14.83	-24.34	38.68	30.55
7	45.40	19.90	25.50	11.13	505.08	12.38	-20.32	32.28	25.50
6	42.28	18.54	23.74	11.13	470.37	11.53	-18.92	30.06	23.74
5	38.78	17.00	21.78	11.13	431.43	10.57	-17.35	27.57	21.78
4	37.07	16.25	20.82	11.13	412.40	10.11	-16.59	26.36	20.82
3	35.06	15.37	19.69	11.13	390.04	9.56	-15.69	24.93	19.69
2	37.64	16.50	21.14	11.13	418.75	10.26	-16.84	26.76	21.14

d_4	13.15	ft
d_5	-16.85	ft

J	537	ft^3

Lateral Force Distribution: Distribution by Rigidity for Frames #1, #2, and #3

k_1	1.341	k
k_2	1.032	k
k_3	1.276	k
Σk	3.649	k

d_1	-48	ft
d_2	12	ft
d_3	42	ft

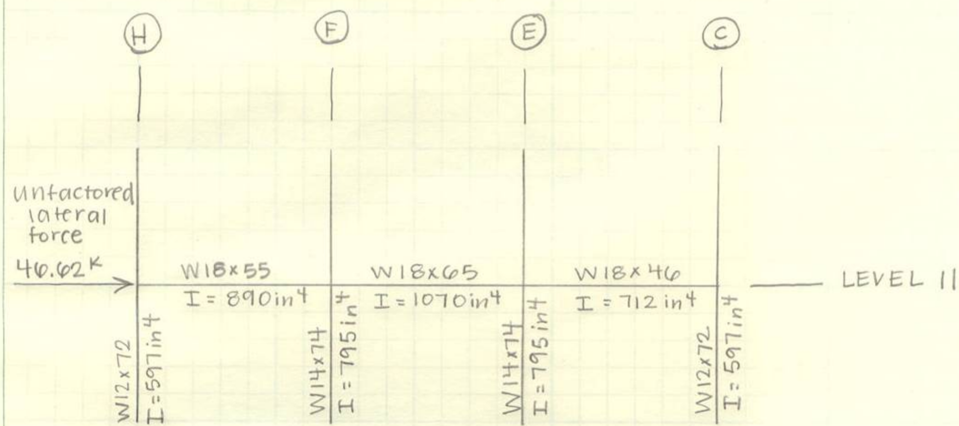
J	5489	ft^3

Level	M_t (ft-k)	F_y (k)	$F_{1,tor}$ (k)	$F_{2,tor}$ (k)	$F_{3,tor}$ (k)	$F_{1,dir}$ (k)	$F_{2,dir}$ (k)	$F_{3,dir}$ (k)	$F_{1,tot}$ (k)	$F_{2,tot}$ (k)	$F_{3,tot}$ (k)
Roof	2062	93.72	-24.18	4.65	20.13	34.44	26.51	32.77	34.44	31.16	52.90
12	2830	128.64	-33.19	6.38	27.63	47.27	36.38	44.98	47.27	42.77	72.61
11	1641	74.59	-19.24	3.70	16.02	27.41	21.10	26.08	27.41	24.80	42.10
10	585	83.57	-6.86	1.32	5.71	30.71	23.64	29.22	30.71	24.95	34.93
9	574	82.05	-6.73	1.30	5.60	30.15	23.21	28.69	30.15	24.50	34.30
8	561	80.14	-6.58	1.27	5.48	29.45	22.66	28.02	29.45	23.93	33.50
7	546	77.98	-6.40	1.23	5.33	28.66	22.05	27.27	28.66	23.29	32.60
6	-703	87.89	8.24	-1.59	-6.86	32.30	24.86	30.73	40.54	24.86	30.73
5	-863	107.92	10.12	-1.95	-8.43	39.66	30.52	37.74	49.78	30.52	37.74
4	-657	82.13	7.70	-1.48	-6.41	30.18	23.23	28.72	37.89	23.23	28.72
3	-627	78.43	7.35	-1.41	-6.12	28.82	22.18	27.43	36.18	22.18	27.43
2	-686	85.79	8.04	-1.55	-6.70	31.53	24.26	30.00	39.57	24.26	30.00

Story Drift Calculations (Level 11 and Level 4)

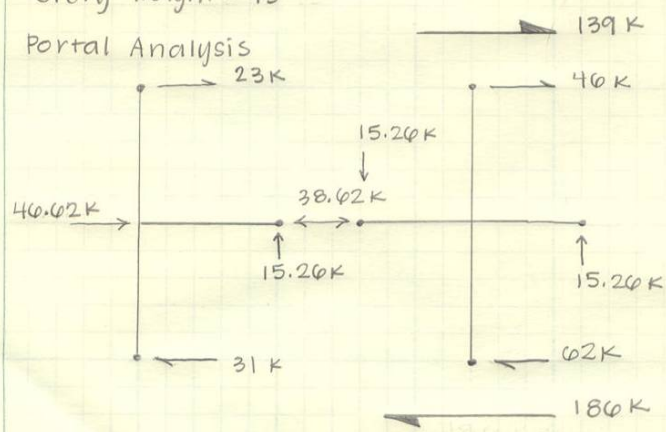
1

Check Story Drift - Column Line 7 - E/W Lateral Forces
Level 11



Story Height = 13'

Portal Analysis



Real Work

$$W_{ext} = \frac{23\Delta}{2} + \frac{46\Delta}{2} + \frac{46\Delta}{2} + \frac{23\Delta}{2} + \frac{46.62(\Delta/2)}{2}$$

$$= 80.66\,\Delta$$

$$W_{int,col} = \frac{1}{2}\left[\int \frac{(23x)^2dx}{E(597)} + \int \frac{(46x)^2dx}{E(795)}\right]2$$

$$+ \frac{1}{2}\left[\int \frac{(31x)^2dx}{E(597)} + \int \frac{(62x)^2dx}{E(795)}\right]2$$

$$= \frac{9.99x^3}{3E}\Big|_0^{6.5}$$

$$= \frac{914.5}{E} \quad \left(\frac{K^2 ft^3}{in^4}\right)$$

$$W_{int,girders} = \frac{1}{2}\left[\int \frac{(15.26x)^2dx}{E(890)} + \int \frac{(15.26x)^2dx}{E(1070)} + \int \frac{(15.26x)^2dx}{E(712)}\right]2$$

$$= \frac{0.262x^3}{3E}\Big|_0^{23} + \frac{0.218x^3}{3E}\Big|_0^{15} + \frac{0.327x^3}{3E}\Big|_0^{21.5}$$

$$= \frac{1063}{E} + \frac{245}{E} + \frac{1083}{E}$$

$$= \frac{2391}{E} \quad \left(\frac{K^2 ft^3}{in^4}\right)$$

$W_{ext} = W_{int}$

$$80.66\Delta = \frac{914.5}{E} + \frac{2391}{E} = \frac{3305.5}{E}$$

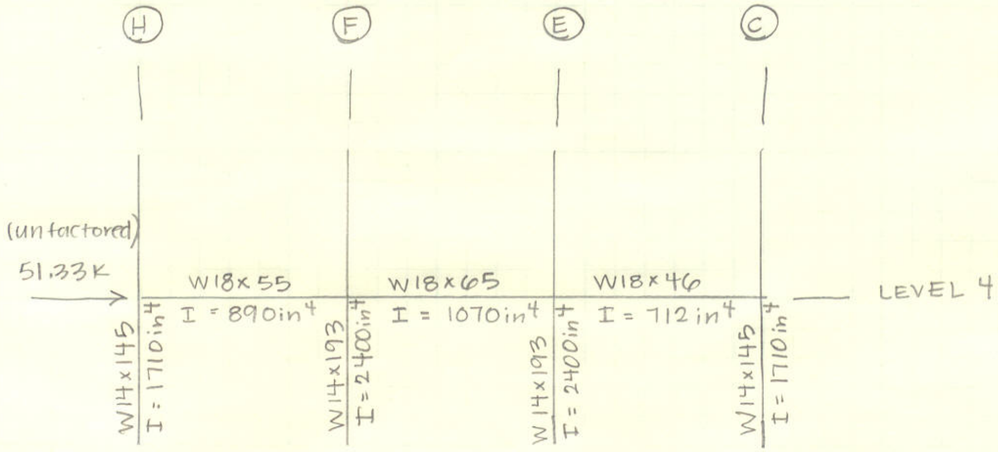
$$80.66\Delta = \frac{3305.5}{E}(12^3)$$

$$\Delta = 2.44''$$

$$\Delta_{all} = \frac{L}{400} = \frac{13'(12''/ft)}{400} = 0.39''$$

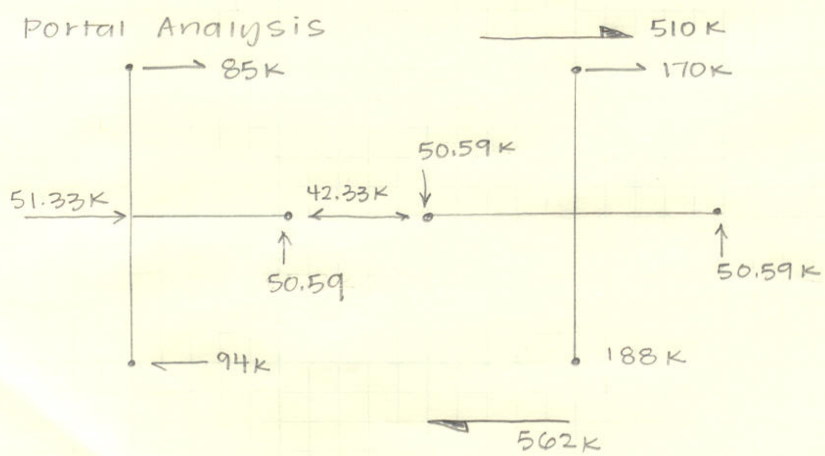
$\Delta = 2.44'' > 0.39'' = \Delta_{all}$ ∴ NOT OK

Check Story Drift - column Line 7 - E/W Lateral Forces
Level 4



Story Height = 13′

Portal Analysis



$$W_{ext} = \frac{85\Delta}{2} + \frac{170\Delta}{2} + \frac{170\Delta}{2} + \frac{85\Delta}{2} + \frac{51.33(\Delta/2)}{2}$$

$$= 267.8\Delta$$

$$W_{int,col} = \frac{1}{2}\left[\int \frac{(85x)^2dx}{E(1710)} + \int \frac{(170x)^2dx}{E(2400)}\right]2$$

$$+ \frac{1}{2}\left[\int \frac{(94x)^2dx}{E(1710)} + \int \frac{(188x)^2dx}{E(2400)}\right]2$$

$$= \frac{36.16x^3}{3E}\Big|_0^{6.5}$$

$$= \frac{3310}{E} \quad \frac{K^2ft^3}{in^4}$$

$$W_{int,girders} = \frac{1}{2}\left[\int \frac{(50.59x)^2dx}{E(890)} + \int \frac{(50.59x)^2dx}{E(1070)} + \int \frac{(50.59x)^2dx}{E(712)}\right]2$$

$$= \frac{2.88x^3}{3E}\Big|_0^{23} + \frac{2.39x^3}{3E}\Big|_0^{15} + \frac{3.59x^3}{3E}\Big|_0^{21.5}$$

$$= \frac{11680}{E} + \frac{2689}{E} + \frac{11893}{E}$$

$$= \frac{26262}{E} \quad \frac{K^2ft^3}{in^4}$$

$W_{ext} = W_{int}$

$$267.8\Delta = \frac{3310}{E} + \frac{26262}{E} = \frac{29572}{E}$$

$$267.8\Delta = \frac{29572}{E}(12^3)$$

$$\Delta = 6.58''$$

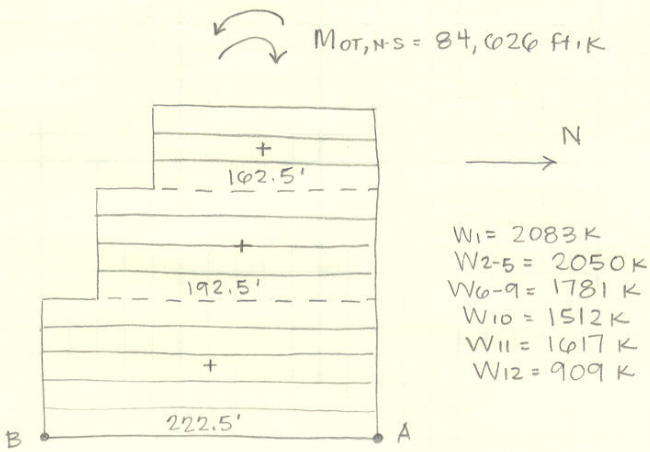
$$\Delta_{all} = \frac{L}{400} = \frac{13'(12''/ft)}{400} = 0.39''$$

$$\Delta = 6.58'' > 0.39'' = \Delta_{all} \quad \therefore \text{ NOT OK}$$

Overturning Moment Calculations

1

Check overturing Moment (N-S direction)



Northern Lateral Forces

$$\Sigma M_A = 0 = 2083(111) + 4(2050)(111') + 4(1781)(96') + 1512(81) + 1617(81) + 909(81) - 84,626$$

$$2067769 \text{ Ft·K} > 84,626 \text{ ft·K} \therefore \text{OK}$$

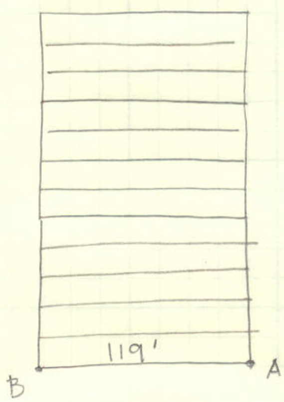
Southern Lateral Forces

$$\Sigma M_B = 2083(111') + 4(2050)(111') + 4(1781)(96+30) + (1512+1617+909)(81+60) - 84626$$

$$2523769 \text{ ft·K} > 84,626 \text{ ft·K} \therefore \text{OK}$$

Check overturning Moment (E-W direction)

$M_{OT, E-W} = 116087 \text{ ft}\cdot\text{K}$



$W_1 = 2083 \text{ K}$
$W_{2-5} = 2050 \text{ K}$
$W_{6-9} = 1781 \text{ K}$
$W_{10} = 1512 \text{ K}$
$W_{11} = 1617 \text{ K}$
$W_{12} = 909 \text{ K}$

22-141 50 SHEETS
22-142 100 SHEETS
22-144 200 SHEETS
AMPAD

East / west Lateral Forces

$$\Sigma M_A = \Sigma M_B = 0 = 2083(59.5') + 4(2050)(59.5') + 4(1781)(59.5') + (1512 + 1617 + 909)(59.5') - 116087$$

$$1{,}159{,}890 \text{ ft}\cdot\text{K} > 116087 \text{ ft}\cdot\text{K} \therefore \text{OK}$$