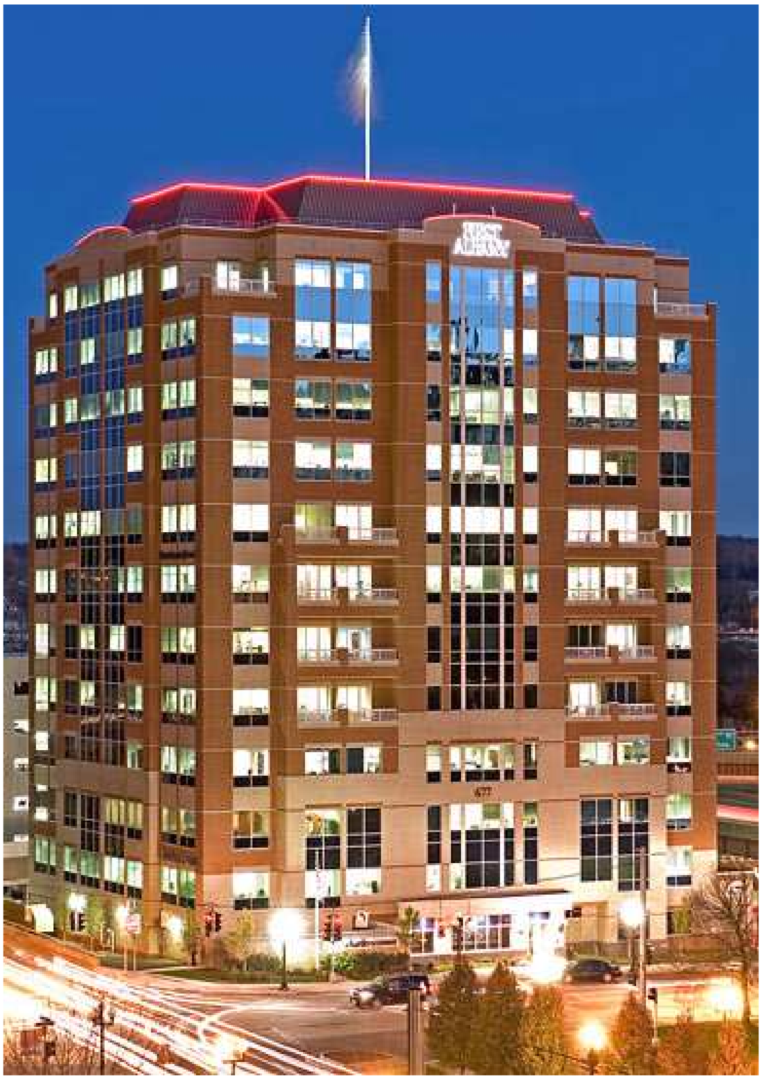
The First Albany Building

677 Broadway
Albany, NY



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Section 1 - EXECUTIVE SUMMARY

The First Albany Building is a 12 story, 180,000 square feet structure designed for mixed-use office space and condominiums. The building's footprint is approximately 115' x 137'. It is located in downtown Albany, NY.

The foundation is a concrete slab on grade over a network of reinforced concrete grade-beams and pile caps. The first floor is at grade and the building has no basement. H-piles were driven to practical refusal to fully support the building. Gravity loads are resisted by a reinforced concrete slab supported by a grid of simply supported steel beams and girders. Partial composite beam and composite deck design was incorporated in to the building. The main lateral force resisting system is comprised of steel braced frames. There are five braced frames, two in the East – West direction and three in the North – South Direction, all located in the core of the building. The braced frames each act as a vertical, cantilevered truss.

Loads determined from ASCE 7-05 in Technical Report 1 are refined and used to analyze the lateral force resisting system. The relative stiffness of each braced frame in the building is determined and utilized to distribute direct and torsional shear forces appropriately.

Each frame has been individually modeled and analyzed (2D) using structural analysis software (ETABS). It is found that total horizontal deflection of each frame to be acceptable (< L/400), however story drift ratios exceed industry standards (0.0025 or 0.25%). Story drift ratios for upper floors approach 0.00275. This is likely caused by the engineer using different methods to calculate lateral loads, as the calculated drifts are not drastically higher than permitted values. The structure is checked for stability and strength, and is found that pile capacities are sufficient to prevent overturning and uplift. Bracing members at levels 1, 7, and 12/ROOF are checked for strength and it is determined that they have sufficient strength capacities. Lastly, one of the braced frames is checked using hand calculations to verify that the assumptions made in the computer model are correct. In addition to modeling each frame individually, a 3D model has been created and analyzed using ETABS. Results from the 3D model coincide with the results from the individual 2D models.

Section 2 - INTRODUCTION

This report breaks down and analyzes the lateral load resisting system of the building. Lateral frames are analyzed separately in two dimensions and then concurrently using computer analysis software (ETABS). Loads are calculated and distributed accordingly and then the structure is checked against permitted drifts and strength requirements.

Section	Topic
3	Required Load Cases
4	Gravity Loads
5	Wind Loads
6	Seismic Loads
7	Lateral Analysis
8	Conclusions

Building Information:

The First Albany Building is a 12 story, 180,000 square feet structure designed for mixed-use office space and condominiums. The building's footprint is approximately 115' x 137'. It is located along the Hudson River in downtown Albany, NY.

The foundation is comprised of a 6" thick concrete slab on grade over a network of reinforced concrete grade-beams and pile caps. The first floor is at grade and the building has no basement. H-piles were driven to practical refusal to fully support the building. Pile capacities are 120 tons, tested and verified on site during installation.

Gravity loads are resisted by a 4.5" reinforced composite concrete deck supported by a grid of simply supported beams and girders. Partial composite beam design was also incorporated in to the building's structural system. Bays are typically 25'x25' with some variations. Sizes of floor members generally range between W12x14 and W18x60 shapes with a determined number of shear stud connectors on each member. Column lines transfer loads directly to the ground through pile caps and to the piles themselves. The piles were carefully laid out as to not cause eccentric forces in any one group of piles.

Wind and seismic loads are resisted by sets of concentrically braced frames around the core of the building. Two frames are oriented in the East – West direction and three narrower frames are oriented in the North – South direction. Bracing patterns include "K", inverted "K", and standard diagonal. The braced frames each act as a vertical, cantilevered truss.

Figure 2.1 – Framing Layout

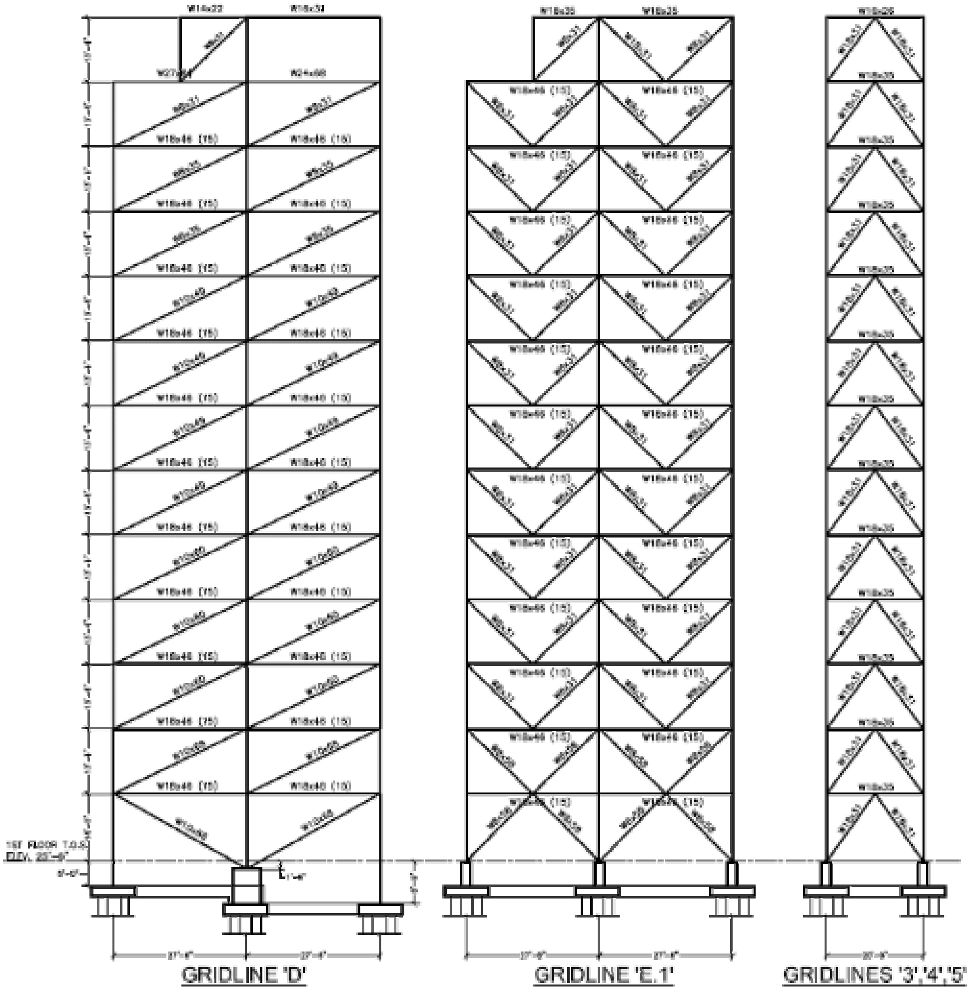


Figure 2.2 – Braced Frame Elevations

Section 3 - LOAD CASES

The First Albany Building was designed based on the New York State Building Code, and the allowable stress design method was used by the engineer. In this report loads are determined from ASCE 7-05, and the strength design method is used.

- Case #1: 1.4D
- Case #2: 1.2D + 1.6L + 0.5S
- Case #3: 1.2D + 1.6S + 0.8W
- Case #4: 1.2D + 1.6W + 1.0L + 0.5S
- Case #5: 1.2D + 1.0E + 1.0L + 0.2S => (1.2 + 0.2SDS)D + ρQ_E + L + 0.2S
- Case #6: 0.9D + 1.6W + 1.6H
- Case #7: 0.9D + 1.0E + 1.6H => (0.9 − 0.2SDS)D + ρQ_E + 1.6H

(ASCE 7-05 2.3.2 & 12.4)

For this report, the braced frames are checked for lateral forces using cases #4 & #5 where wind and seismic loading controls, respectively. Using the factored wind and seismic loads, it is found that base shear and moment from wind loading controls the design in both the North-South and East-West directions for strength and drift. Therefore, case #4 is used to check the foundations for uplift, and overturning.

Section 4 - APPLICABLE BUILDING CODES & GRAVITY LOADS

New York State Building Code 2002
New York State Energy Conservation Code
"Manual of Steel Construction" AISC ASD 9th Ed.
"Building Code Requirements for Structural Concrete" ACI 318-02

Gravity Live Loads

	Loading Used	Current Required Loading	
Office Space (2-8) Partition Allowance	50 psf +20 psf	50 psf +15	(ASCE 7-05, Table 4.1)
Office Space (9-12) +Computer Use Access Flooring	100 psf +15 psf	100 psf	(ASCE 7-05 Table 4.1)
Office Space +File Storage	125 psf	125 psf	(ASCE 7-05 Table 4.1)
Stairways	100 psf	100 psf	(ASCE 7-05 Table 4.1)
Roof Snow Load	65 psf	65 psf	(NYS Bldg Code)
Balconies	100 psf	100 psf	(ASCE 7-05 Table 4.1)
Roof	20 psf	20 psf	(ASCE 7-05 Table 4.1)
Restaurants	100 psf	100 psf	(ASCE 7-05 Table 4.1)

Table 4.1

Dead Loads

Loading Breakdown	
MEP	15 psf
Structural Steel (Columns Only)	4 psf
Structural Steel (All Other)	10 psf
Lightweight Concrete Slab	34 psf
Deck	2 psf
Finishes	5 psf
Misc	10 psf
Total	80 psf

Table 4.2

Live Load Reductions

For structural members supporting 1 floor; RF > 0.5
For structural members supporting 2 or more floors; RF > 0.4

Section 5 – DESIGN WIND LOADS as per ASCE 7-05

Wind loads were analyzed using section 6 of ASCE 7-05. Appendix A contains a detailed analysis of wind loads using the equations and factors set forth in ASCE. These factors are dependent on building location and characteristics as well as experimental data.

Design Criteria

Height	h			172’
Dimensions				137’x115’
Wind directionality factor	Kd	6.5.4		0.85
Importance Factor	I	6.5.5		1.0
Wind Exposure Category		6.5.6		B
Basic Wind Speed	V			90 MPH
Topographic Factor	Kzt	6.5.7		1.0
Gust Factor	Gf	6.5.8		0.85
External Pressure Coeff.	Cpf	6.5.11.2	Windward	0.8
			Leeward	-0.5
			Sides	-0.7

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

h	Kz	Kzt	Kd	V	I	qz	Gf	Cp	Pressure (psf)	
0-15	0.57	1.00	0.85	90.00	1.00	10.05	0.85	0.80	Windward	6.83
20	0.62	1.00	0.85	90.00	1.00	10.93	0.85	0.80		7.43
25	0.66	1.00	0.85	90.00	1.00	11.63	0.85	0.80		7.91
30	0.70	1.00	0.85	90.00	1.00	12.34	0.85	0.80		8.39
40	0.76	1.00	0.85	90.00	1.00	13.40	0.85	0.80		9.11
50	0.81	1.00	0.85	90.00	1.00	14.28	0.85	0.80		9.71
60	0.85	1.00	0.85	90.00	1.00	14.98	0.85	0.80		10.19
70	0.89	1.00	0.85	90.00	1.00	15.69	0.85	0.80		10.67
80	0.93	1.00	0.85	90.00	1.00	16.39	0.85	0.80		11.15
90	0.96	1.00	0.85	90.00	1.00	16.92	0.85	0.80		11.51
100	0.99	1.00	0.85	90.00	1.00	17.45	0.85	0.80		11.87
120	1.04	1.00	0.85	90.00	1.00	18.33	0.85	0.80		12.46
140	1.09	1.00	0.85	90.00	1.00	19.21	0.85	0.80		13.06
160	1.13	1.00	0.85	90.00	1.00	19.92	0.85	0.80		13.54
180	1.17	1.00	0.85	90.00	1.00	20.62	0.85	0.80		14.02
180	1.17	1.00	0.85	90.00	1.00	20.62	0.85	-0.50	Leeward	-8.76
180	1.17	1.00	0.85	90.00	1.00	20.62	0.85	-0.70	Sides	-12.27

Table 5.1 – Wind Pressures

Through a generalized analysis of the buildings fundamental period set forth in ASCE 7-05 the building was found to behave as a rigid structure. (*See the seismic loads section for the building period calculation*)

Section 6 – DESIGN SEISMIC LOADS as per ASCE 7-05

Seismic loads were found using the applicable sections of ASCE 7-05; Equivalent Lateral Force procedure (12.8). All factors and accelerations were found using the tables and equations contained in ASCE. All dead loads used are based on ASCE 7-05 and are listed in the gravity loads section of this report.

Design Criteria

Site Class	D	
Occupancy Category	II	
Importance Factor	1.0	
Seismic Design Category	B	
Response Modification Factor (R)	5	Table 12.2-1
Period (Ta)	1.57	Eq. 12.8-7
Ss	0.229	*1
S_1	0.069	*1
SDS	0.28	*2
SD1	0.12	*2
T_L	6	Figure 22-15
Cs	0.015	Eq. 12.8-2,3,4,5
Base Shear (V)	277.3 (K)	1.5% of weight

*1 - From USGS website - earthquake.usgs.gov/research/hazmaps/design
*2 – Based on Proshake Analysis performed by Dente Engineering, Nov. 4, 2003

Level	area ft²	weight (psf)	wx (k)	hf (ft)	hx (ft)	wx(hx)^k	Fx (K)	Vx (K)	Mx (FT-K)
Pent	2715	100	271.5	10.00	172.00	46698.0	7.9	7.9	1354.6
12	13913	100	1641.3	14.67	162.00	265890.6	44.8	52.7	7264.5
11	14888	100	1488.8	13.33	147.33	219349.9	37.0	89.7	5450.4
10	14888	100	1488.8	13.33	134.00	199499.2	33.6	123.4	4508.5
9	14888	100	1488.8	13.33	120.67	179648.5	30.3	153.7	3655.9
8	14888	100	1488.8	13.33	107.33	159797.9	26.9	180.6	2892.6
7	15172	100	1517.2	13.33	94.00	142616.8	24.1	204.7	2260.9
6	15172	100	1517.2	13.33	80.67	122387.5	20.6	225.3	1665.0
5	15172	100	1517.2	13.33	67.33	102158.1	17.2	242.5	1160.1
4	15172	100	1517.2	13.33	54.00	81928.8	13.8	256.3	746.1
3	15172	100	1517.2	13.33	40.67	61699.5	10.4	266.7	423.2
2	15172	100	1517.2	13.33	27.33	41470.1	7.0	273.7	191.2
1	15172	100	1517.2	14.67	14.00	21240.8	3.6	277.3	50.2
	Total 182384		Total 18488.4	Cs 0.015		Total 1644385.7	Total 277.3		Total 31623.2

Table 6.1 – Seismic Loading

Section 7 – LATERAL ANALYSIS:

7.1 Load Distribution

The lateral analysis utilizes the wind and seismic loads calculated (revised from Technical Report 1) to determine drift and strength requirements. ETABS was used to analyze each braced frame individually (2D models and then all simultaneously (3D model). Using a 100k force at the top of the each frame, the relative stiffness for each frame is found. Lateral and torsional forces are then distributed appropriately.

	E-W Direction		N-S Direction		
Frame	D	E1	3	4	5
Load	100	100	100	100	100
Displacement	2.028	1.561	7.453	7.453	7.453
K=P / Δ	49.310	64.061	13.417	13.417	13.417
Relative K (in same direction)	0.435	0.565	0.333	0.333	0.333
Relative K (all frames)	0.321	0.417	0.087	0.087	0.087
d	25.43	19.57	27.50	0.00	27.50
ki*di	8.16	8.16	2.40	0.00	2.40
kd²	207.571	159.755	66.051	0.000	66.051
ki*di/ Σkd²	0.01634	0.01634	0.00481	0.00000	0.00481

Table 7.1.1 – Lateral Force Distribution

7.2 - 2D Analysis

Story loads due to wind are calculated from the pressures on the building faces. Direct forces and torsional forces from eccentricities are considered when determining the final loads. Four cases that combine direct and torsional loading are to be considered when determining maximum loads (ASCE 7-05 Figure 6.9, shown in appendix B)

Wind	Frame D	Frame E.1	Frame 3,4,5
Level	Max (K)	Max (K)	Max (K)
Roof/12	37.28	43.57	18.04
11	25.90	30.27	12.72
10	23.06	26.95	11.33
9	22.76	26.59	11.18
8	22.41	26.19	11.01
7	21.74	25.41	10.68
6	21.24	24.82	10.43
5	20.67	24.15	10.15
4	19.94	23.30	9.79
3	19.22	22.46	9.44
2	18.17	21.24	8.93
1	18.40	21.50	9.04

Table 7.2.1 – Wind Loads

Full supporting data and calculations can be found in Appendix B.

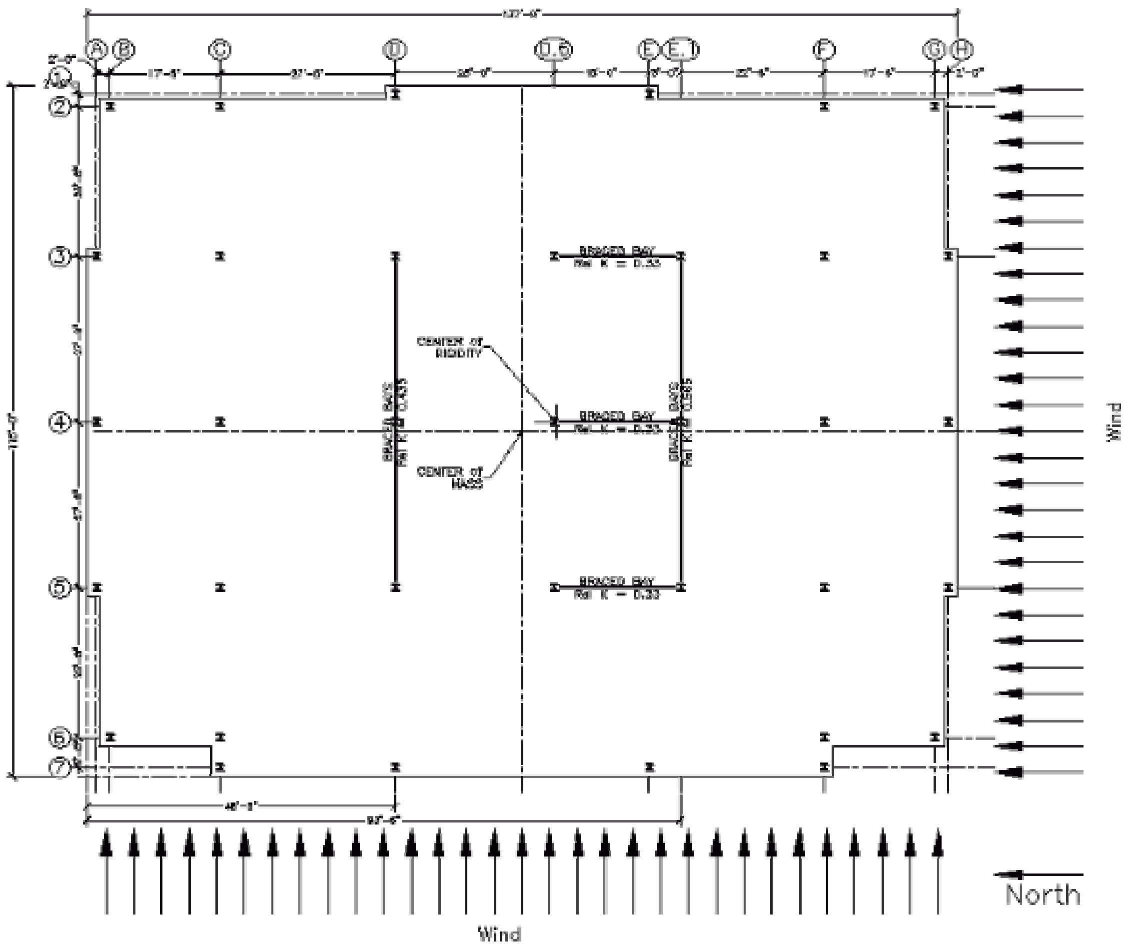


Figure 7.1.1 - Wind direction, Frame locations, and Centers of rigidity, pressure, mass

Wind Drift	Story Height	Frame D		Frame E1		Frame 3,4,5		Permitted 1 / 400
		Total	Story	Total	Story	Total	Story	
Pent	144	2.2292	0.0008	2.3813	0.0009	4.3403	0.00220	0.0025
Roof/12	176	2.1130	0.0011	2.2460	0.0012	4.0241	0.00262	0.0025
11	160	1.9249	0.0012	2.0319	0.0013	3.5636	0.00265	0.0025
10	160	1.7381	0.0013	1.8277	0.0013	3.1398	0.00265	0.0025
9	160	1.5380	0.0012	1.6171	0.0013	2.7161	0.00259	0.0025
8	160	1.3492	0.0012	1.4026	0.0013	2.3013	0.00252	0.0025
7	160	1.1572	0.0012	1.1883	0.0013	1.8988	0.00238	0.0025
6	160	0.9626	0.0012	0.9746	0.0013	1.5175	0.00222	0.0025
5	160	0.7690	0.0011	0.7655	0.0013	1.1626	0.00199	0.0025
4	160	0.5962	0.0010	0.5626	0.0012	0.8446	0.00174	0.0025
3	160	0.4309	0.0010	0.3699	0.0011	0.5669	0.00142	0.0025
2	160	0.2753	0.0008	0.1909	0.0007	0.3395	0.00116	0.0025
1	168	0.1404	0.0008	0.0824	0.0005	0.1535	0.00091	0.0025

Table 7.2.2 – Drifts from Wind (Total & Story Ratios)

Story loads due to seismic activity are calculated using the Equivalent Lateral Force Procedure (ASCE 7-05 12.8). Direct forces and torsional forces from a 5% eccentricity are considered when determining the final loads.

Seismic	Frame D	Frame E.1	Frame 3,4,5
Level	Max (K)	Max (K)	Max (K)
Penthouse	4.31	5.11	2.84
Roof/12	24.53	29.10	16.19
11	20.23	24.01	13.35
10	18.40	21.83	12.15
9	16.57	19.66	10.94
8	14.74	17.49	9.73
7	13.15	15.61	8.68
6	11.29	13.39	7.45
5	9.42	11.18	6.22
4	7.56	8.97	4.99
3	5.69	6.75	3.76
2	3.83	4.54	2.52
1	1.96	2.32	1.29

Table 7.2.3 – Seismic Loads

Seismic	Story	Frame D		Frame E1		Frame 3,4,5		Permitted
Drift	Height	Total	Story	Total	Story	Total	Story	1 / 400
Pent	144	1.5847	0.0007	1.7158	0.0007	4.3316	0.00230	0.0025
Roof/12	176	1.4788	0.0008	1.6088	0.0009	4.0003	0.00274	0.0025
11	160	1.3395	0.0009	1.4484	0.0010	3.5184	0.00276	0.0025
10	160	1.2010	0.0009	1.2950	0.0010	3.0768	0.00275	0.0025
9	160	1.0517	0.0009	1.1366	0.0010	2.6364	0.00267	0.0025
8	160	0.9122	0.0009	0.9758	0.0010	2.2086	0.00257	0.0025
7	160	0.7717	0.0009	0.8166	0.0010	1.7970	0.00241	0.0025
6	160	0.6313	0.0009	0.6598	0.0009	1.4120	0.00221	0.0025
5	160	0.4943	0.0007	0.5090	0.0009	1.0590	0.00194	0.0025
4	160	0.3750	0.0007	0.3660	0.0008	0.7493	0.00165	0.0025
3	160	0.2644	0.0006	0.2342	0.0007	0.4859	0.00129	0.0025
2	160	0.1643	0.0005	0.1162	0.0004	0.2788	0.00100	0.0025
1	168	0.0817	0.0005	0.0470	0.0003	0.1185	0.00071	0.0025

Table 7.2.4 – Seismic Drifts (Total & Story Ratios)

7.3 - 3D Analysis

Drift values obtained from a full 3D model are nearly identical. Wind cases 1 through 4 and seismic loads with accidental torsion were considered. Levels 7 – PHROOF had maximum drifts due to seismic loads while the lower levels maximums were from wind.

Drifts	N-S	E-W	Permitted 1 / 400
PHROOF	0.00273	0.00097	0.0025
ROOF	0.00274	0.00183	0.0025
11	0.00278	0.00194	0.0025
10	0.00280	0.00202	0.0025
9	0.00273	0.00187	0.0025
8	0.00262	0.00186	0.0025
7	0.00250	0.00184	0.0025
6	0.00232	0.00177	0.0025
5	0.00211	0.00155	0.0025
4	0.00191	0.00149	0.0025
3	0.00163	0.00140	0.0025
2	0.00135	0.00120	0.0025
1	0.00097	0.00091	0.0025

Table 7.3.1 – Story Drift Ratios (3D Model results)

Story Drifts in the upper floors exceed the limitation of L/400 (0.25%). This is likely caused by the engineer using different methods to calculate lateral loads, as the calculated drifts are not drastically higher than permitted values.

7.4 - Strength Check

Using the appropriate load combinations, ultimate loads were found and compared to nominal strengths. All members checked were found to have adequate strength.

Level	Brace Location	Pu (k)	Size	Length (ft)	ΦPn (k)
1	D	191.9	w10x68	30.86	221.01
	E1	161.2	w8x58	20.10	292.82
	3,4,5	180.9	w8x31	17.75	182.02
7	D	139.4	w10x49	30.56	156.05
	E1	148.1	w8x31	19.15	159.19
	3,4,5	100.8	w8x31	16.67	200.34
12/Roof	D	49.2	w8x31	30.86	60.09
	E1	55.8	w8x31	20.10	144.35
	3,4,5	25.7	w8x31	16.67	200.34

Table 7.4.1 – Strength Check

Braces in the upper floor may appear to be oversized, but their sizes are dictated by drift limitations. Brace strength calculations can be found in Appendix D.

7.5 - Overturning Check

Maximum overturning moments (Mu) are determined from wind loads. Resistance is calculated from pile capacities (240 k per) and distances (moment arm). The base of each frame column is supported by 5 piles. Resistance to overturning is sufficient to maintain stability.

	Mu Ft-k	R (k)	D (ft)	Mr Ft-k
D	41400	1200	55	66000
E1	48382	1200	55	66000
3,4,5	20269	1200	20	24000

Table 7.5.1 – Overturning & Resisting Moments

7.6 – Computer Model Verification by Hand Calculation

To verify that the computer models and assumptions made were correct, the total deflection of frame 4 was checked by hand using virtual work. Calculated total deflection was 4.36 inches. If beams were assumed to be a rigid diaphragm, deflection was 4.17 inches. Hand calculations are within 4% of computer generated results. The computer models and assumptions made for them are correct. The slight variations are from columns not having pin-pin connections at every floor. Hand calculated deflections assumed every member in the frame to be pin-pin connected. Not being pin-pin connected at every floor produced a stiffer model, thereby decreasing deflections. See Appendix E for supporting data and calculations.

Virtual Work: (external work) = (internal work)

$$\Sigma P_i D_i = \Sigma \frac{F_v * F_d * L}{AE}$$

Pi = external force
Di = displacement
Fv = member axial force due to virtual load
Fd = member axial force due to real load
L = member length
A = member cross sectional area
E = modulus of elasticity

Section 8 – CONCLUSIONS

Using the appropriate load combinations, ultimate loads were found and compared to nominal strengths. All members checked were found to have adequate strength. Braces in the upper floor may appear to be oversized, but their sizes are dictated by drift limitations. Brace strength calculations can be found in Appendix D.

To verify that the computer models and assumptions made were correct, the total deflection of frame 4 was checked by hand using virtual work. Calculated total deflection was 4.36 inches. If beams were assumed to be a rigid diaphragm, deflection was 4.17 inches. Computer generated results are within 4% of Hand calculations and a accepted as correct.

Each frame has been individually modeled and analyzed (2D) using structural analysis software (ETABS). It is found that total horizontal deflection of each frame to be acceptable (< L/400), however story drift ratios exceed industry standards (0.0025 or 0.25%). Story drift ratios for upper floors approach 0.00275. This is likely caused by the engineer using different methods to calculate lateral loads, as the calculated drifts are not drastically higher than permitted values. The structure is checked for stability and strength, and is found that pile capacities are sufficient to prevent overturning and uplift. Bracing members at levels 1, 7, and 12/ROOF are checked for strength and it is determined that they have sufficient strength capacities. Lastly, one of the braced frames is checked using hand calculations (virtual work) to verify that the assumptions made in the computer model are correct. In addition to modeling each frame individually, a 3D model has been created and analyzed using ETABS. Results from the 3D model coincide with the results from the individual 2D models.

APPENDIX A – MATERIAL SPECIFICATIONS

Structural Steel –

Miscellaneous shapes, plates, bars	–	ASTM A36, Fy = 36 ksi
Structural Shapes, W8 and larger	–	ASTM A572, Grade 50, Fy = 50 ksi
Hollow Structural Shapes (HSS)	–	A500, Grade B, Fy = 46 ksi (square and rect.)
	–	ASTM A53, Type E or S, Fy = 35 ksi (round shapes)
Anchor Bolts	–	ASTM A307
	–	ASTM A449 (at braced bays)

Cast-in-place Concrete –

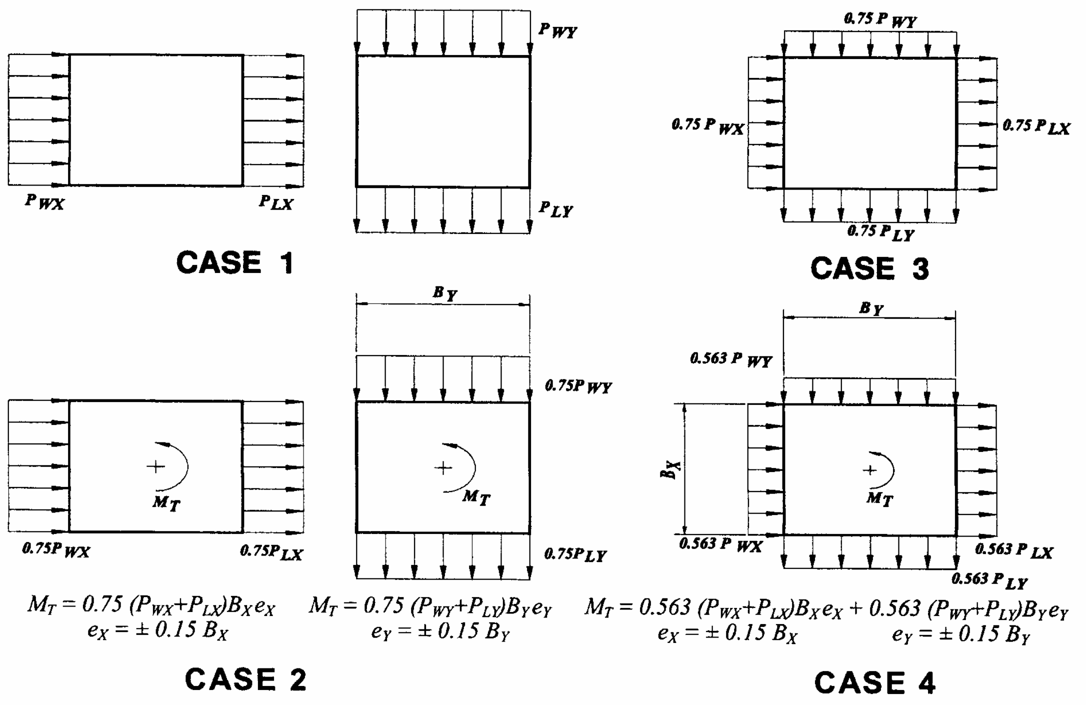
Slab on Grade	–	3500 psi (28 day compressive strength)
Supported Floor Slabs	–	4000 psi, lightweight (115 pcf)
Grade Beams, Pile Caps, Walls	–	4000 psi
Foundation Piers	–	6000 psi
Reinforcing bars	–	ASTM A615, Grade 60, deformed
Welded Reinforcing bars	–	ASTM A706, Grade 60
Welded Wire Fabric	–	ASTM A185 (Sheet type only)

Steel Deck –

Roof Deck	–	1 ½” x 22 Gage Type B Rib Deck
Floor Deck	–	2” x 22 Gage Composite Floor Deck

APPENDIX B – WIND LOADING CALCULATIONS

Main Wind Force Resisting System– Method 2		All Heights
Figure 6-9	Design Wind Load Cases	



Case 1. Full design wind pressure acting on the projected area perpendicular to each principal axis of the structure, considered separately along each principal axis.

Case 2. Three quarters of the design wind pressure acting on the projected area perpendicular to each principal axis of the structure in conjunction with a torsional moment as shown, considered separately for each principal axis.

Case 3. Wind loading as defined in Case 1, but considered to act simultaneously at 75% of the specified value.

Case 4. Wind loading as defined in Case 2, but considered to act simultaneously at 75% of the specified value.

Notes:

1. Design wind pressures for windward and leeward faces shall be determined in accordance with the provisions of 6.5.12.2.1 and 6.5.12.2.3 as applicable for building of all heights.
2. Diagrams show plan views of building.
3. Notation:
   - P_{WX}, P_{WY}: Windward face design pressure acting in the x, y principal axis, respectively.
   - P_{LX}, P_{LY}: Leeward face design pressure acting in the x, y principal axis, respectively.
   - e (e_X, e_Y) : Eccentricity for the x, y principal axis of the structure, respectively.
   - M_T: Torsional moment per unit height acting about a vertical axis of the building.

*Note:
A screen wall attached to the roof (but not to the Penthouse) adds wind load to the roof level and shields the Penthouse from wind pressures. Area is calculated accordingly.

6.1.4.1 Main Wind-Force Resisting System. The wind load to be used in the design of the MWFRS for an enclosed or partially enclosed building or other structure shall not be less than 10 lb/ft2

Wind Forces / Level	Level Ht ft	X Width ft	Y Width ft	X Area ft²	Y Area ft²	Wind-ward Press. lb/ft²	Lee-ward Press. lb/ft²	Total Pressure lb/ft²	Pwx Force K	Pwy Force K
Scr.Wall	15.00	116.0	94.0	1740.00	1410.00	14.02	8.76	22.78		
Roof /12*	8.00	137.0	115.0	1096.00	920.00	13.92	8.76	22.68	64.5	53.0
11	14.67	137.0	115.0	2009.33	1686.67	13.54	8.76	22.30	44.8	37.6
10	13.33	137.0	115.0	1826.67	1533.33	13.08	8.76	21.84	39.9	33.5
9	13.33	137.0	115.0	1826.67	1533.33	12.79	8.76	21.55	39.4	33.0
8	13.33	137.0	115.0	1826.67	1533.33	12.46	8.76	21.22	38.8	32.5
7	13.33	137.0	115.0	1826.67	1533.33	11.83	8.76	20.59	37.6	31.6
6	13.33	137.0	115.0	1826.67	1533.33	11.35	8.76	20.11	36.7	30.8
5	13.33	137.0	115.0	1826.67	1533.33	10.81	8.76	19.57	35.7	30.0
4	13.33	137.0	115.0	1826.67	1533.33	10.12	8.76	18.88	34.5	28.9
3	13.33	137.0	115.0	1826.67	1533.33	9.44	8.76	18.20	33.2	27.9
2	13.33	137.0	115.0	1826.67	1533.33	8.45	8.76	17.21	31.4	26.4
1	14.67	137.0	115.0	2009.33	1686.67	7.08	8.76	15.84	31.8	26.7
	Total 172.33								Total 468.4	Total 392.0

X Total Shear	Y Total Shear	X Over Turn	Y Over Turn	X Frame D F-direct K	Y Frame D F-direct K	X Frame E.1 F-direct K	Y Frame E.1 F-direct K	X Frame 3,4,5 F-direct K	Y Frame 3,4,5 F-direct K
64.5	53.0	10147.1	8336.4	28.1	0.0	36.4	0.0	0	17.7
109.3	90.6	6691.3	5616.8	19.5	0.0	25.3	0.0	0	12.5
149.2	124.1	5372.4	4509.7	17.4	0.0	22.5	0.0	0	11.2
188.6	157.1	4776.2	4009.3	17.1	0.0	22.2	0.0	0	11.0
227.3	189.7	4186.3	3514.0	16.9	0.0	21.9	0.0	0	10.8
264.9	221.2	3560.5	2988.8	16.4	0.0	21.3	0.0	0	10.5
301.7	252.1	2987.7	2507.9	16.0	0.0	20.8	0.0	0	10.3
337.4	282.1	2430.9	2040.5	15.6	0.0	20.2	0.0	0	10.0
371.9	311.0	1885.3	1582.6	15.0	0.0	19.5	0.0	0	9.6
405.1	338.9	1374.1	1153.5	14.5	0.0	18.8	0.0	0	9.3
436.6	365.3	880.2	738.9	13.7	0.0	17.8	0.0	0	8.8
468.4	392.0	466.8	391.8	13.8	0.0	18.0	0.0	0	8.9
		Total 44759.0	Total 37390.2	Total 203.8		Total 264.7			Total 130.7

Case 1 : Pw & Actual Ecc										
Ecc x ft	Pwx*ex	Frame D,E1 ki*di/ Σkd²	F-tors K	Frame D Ft+Fd K	Frame E.1 Ft+Fd K	Ecc y ft	Pwy*ey	Frame 3,4,5 ki*di/ Σkd²	F-tors K	Frame 3,4,5 Ft+Fd K
5.42	349.9	0.01634	5.72	33.77	40.73	1.5	79.5	0.00481	0.38	18.04
5.42	243.1	0.01634	3.97	23.46	28.30	1.5	38.9	0.00481	0.19	12.72
5.42	216.4	0.01634	3.54	20.89	25.19	1.5	34.6	0.00481	0.17	11.33
5.42	213.5	0.01634	3.49	20.61	24.86	1.5	34.1	0.00481	0.16	11.18
5.42	210.3	0.01634	3.44	20.30	24.48	1.5	33.6	0.00481	0.16	11.01
5.42	204.0	0.01634	3.33	19.69	23.75	1.5	32.6	0.00481	0.16	10.68
5.42	199.3	0.01634	3.26	19.24	23.20	1.5	31.9	0.00481	0.15	10.43
5.42	193.9	0.01634	3.17	18.72	22.57	1.5	31.0	0.00481	0.15	10.15
5.42	187.1	0.01634	3.06	18.06	21.78	1.5	29.9	0.00481	0.14	9.79
5.42	180.3	0.01634	2.95	17.41	20.99	1.5	28.8	0.00481	0.14	9.44
5.42	170.5	0.01634	2.79	16.46	19.85	1.5	27.3	0.00481	0.13	8.93
5.42	172.7	0.01634	2.82	16.67	20.10	1.5	27.6	0.00481	0.13	9.04
Case 2 : 0.75 Pw & Ecc = 0.15*B										
Ecc x ft	Pwx*ex	Frame D,E1 ki*di/ Σkd²	F-tors K	Frame D Ft+ 0.75Fd K	Frame E.1 Ft+ 0.75Fd K	Ecc y ft	Pwy*ey	Frame 3,4,5 ki*di/ Σkd²	F-tors K	Frame 3,4,5 Ft+ 0.75Fd K
20.55	1325.4	0.01634	21.66	37.28	43.57	17.25	914.0	0.00481	4.40	16.54
20.55	920.8	0.01634	15.05	25.90	30.27	17.25	648.8	0.00481	3.12	11.74
20.55	819.8	0.01634	13.40	23.06	26.95	17.25	577.7	0.00481	2.78	10.46
20.55	808.9	0.01634	13.22	22.76	26.59	17.25	570.0	0.00481	2.74	10.32
20.55	796.6	0.01634	13.02	22.41	26.19	17.25	561.3	0.00481	2.70	10.16
20.55	772.9	0.01634	12.63	21.74	25.41	17.25	544.6	0.00481	2.62	9.86
20.55	754.9	0.01634	12.33	21.24	24.82	17.25	531.9	0.00481	2.56	9.63
20.55	734.6	0.01634	12.00	20.67	24.15	17.25	517.6	0.00481	2.49	9.37
20.55	708.7	0.01634	11.58	19.94	23.30	17.25	499.4	0.00481	2.40	9.04
20.55	683.2	0.01634	11.16	19.22	22.46	17.25	481.4	0.00481	2.32	8.71
20.55	646.0	0.01634	10.56	18.17	21.24	17.25	455.2	0.00481	2.19	8.24
20.55	654.1	0.01634	10.69	18.40	21.50	17.25	460.9	0.00481	2.22	8.34
Case 3 : 0.75Pwx, 0.75 Pwy, Actual Ecc										
Ecc x ft	Pwx*ex	Frame D,E1 ki*di/ Σkd²	F-tors K	Frame D Ft+ 0.75Fd K	Frame E.1 Ft+ 0.75Fd K	Exx y ft	Pwy*ey	Frame 3,4,5 ki*di/ Σkd²	F-tors K	Frame 3,4,5 Ft+ 0.75Fd K
5.42	349.9	0.01634	7.02	28.06	34.34	1.50	79.5	0.00481	2.07	15.31
5.42	243.1	0.01634	4.89	19.51	23.88	1.50	56.4	0.00481	1.44	10.84
5.42	216.4	0.01634	4.36	17.37	21.26	1.50	50.2	0.00481	1.28	9.65
5.42	213.5	0.01634	4.30	17.14	20.98	1.50	49.6	0.00481	1.27	9.53
5.42	210.3	0.01634	4.23	16.88	20.66	1.50	48.8	0.00481	1.25	9.38
5.42	204.0	0.01634	4.11	16.38	20.05	1.50	47.4	0.00481	1.21	9.10
5.42	199.3	0.01634	4.01	16.00	19.58	1.50	46.3	0.00481	1.18	8.89
5.42	193.9	0.01634	3.90	15.57	19.05	1.50	45.0	0.00481	1.15	8.65

5.42	187.1	0.01634	3.77	15.02	18.38	1.50	43.4	0.00481	1.11	8.35
5.42	180.3	0.01634	3.63	14.48	17.72	1.50	41.9	0.00481	1.07	8.05
5.42	170.5	0.01634	3.43	13.69	16.75	1.50	39.6	0.00481	1.01	7.61
5.42	172.7	0.01634	3.48	13.86	16.96	1.50	40.1	0.00481	1.02	7.70
Case 4 : 0.563Pwx, 0.563 Pwy, Ecc = 0.15*B										
Ecc x ft	Pwx*ex	Frame D,E1 ki*di/ Σkd²	F-tors K	Frame D Ft+ 0.75Fd K	Frame E.1 Ft+ 0.563Fd K	Exx y ft	Pwy*ey	Frame 3,4,5 ki*di/ Σkd²	F-tors K	Frame 3,4,5 Ft+ 0.563Fd K
20.55	1325.4	0.01634	36.59	36.40	41.12	17.25	914.0	0.00481	10.77	16.01
20.55	920.8	0.01634	25.65	25.41	28.69	17.25	648.8	0.00481	7.55	11.31
20.55	819.8	0.01634	22.84	22.63	25.55	17.25	577.7	0.00481	6.72	10.07
20.55	808.9	0.01634	22.53	22.33	25.21	17.25	570.0	0.00481	6.63	9.94
20.55	796.6	0.01634	22.19	21.98	24.82	17.25	561.3	0.00481	6.53	9.78
20.55	772.9	0.01634	21.53	21.33	24.08	17.25	544.6	0.00481	6.34	9.49
20.55	754.9	0.01634	21.03	20.83	23.52	17.25	531.9	0.00481	6.19	9.27
20.55	734.6	0.01634	20.46	20.27	22.89	17.25	517.6	0.00481	6.02	9.02
20.55	708.7	0.01634	19.74	19.56	22.08	17.25	499.4	0.00481	5.81	8.70
20.55	683.2	0.01634	19.03	18.86	21.29	17.25	481.4	0.00481	5.60	8.39
20.55	646.0	0.01634	17.99	17.83	20.13	17.25	455.2	0.00481	5.30	7.93
20.55	654.1	0.01634	18.22	18.05	20.38	17.25	460.9	0.00481	5.36	8.03

Maximums					
Frame D Max K	over turning	Frame E.1 Max K	over turning	Frame 3,4,5 Max K	over turning
37.28	5866.0	43.57	6855.3	18.04	2838.9
25.90	3868.2	30.27	4520.6	12.72	1900.2
23.06	3105.8	26.95	3629.6	11.33	1525.6
22.76	2761.1	26.59	3226.8	11.18	1356.3
22.41	2420.0	26.19	2828.2	11.01	1188.8
21.74	2058.3	25.41	2405.4	10.68	1011.1
21.24	1727.2	24.82	2018.5	10.43	848.4
20.67	1405.3	24.15	1642.3	10.15	690.3
19.94	1089.9	23.30	1273.7	9.79	535.4
19.22	794.4	22.46	928.4	9.44	390.2
18.17	508.9	21.24	594.7	8.93	250.0
18.40	269.9	21.50	315.4	9.04	132.6
Total 270.8	Total 25874.8	Total 316.5	Total 30238.8	Total 132.7	Total 12667.9

APPENDIX C – SEISMIC LOADING CALCULATIONS

Site Class – D (Firm Soils)

V_s = 600 to 1200 ft/s   N = 15 to 50 S_u = 1000 to 2000 psf

$S_1 = 0.069$ earthquake.usgs.gov/research/hazmaps/design
$S_s = 0.229$

SDS = 0.28   Based on Proshake Analysis Performed by Dente Engineering, Nov. 4, 2003
SD1 = 0.12

Occupancy Category	- II	
Importance Factor	- 1.0	
Seismic Design Cat.	- B	
Response Mod. Factor	- 5	(Table 12.2-1)
T_L	- 6	(Figure 22-15)

$T_a = C_t * h_n^{(x)} = 0.02\,(172)^{(0.75)} = 0.95$

$C_t = 0.02$ (Table 12.8-2)
$x = 0.75$
$h = 172$

$$C_u = \min \begin{cases} \dfrac{SDS}{(R / I)} = 0.056 \\ \dfrac{SD1}{T\,(R / I)} = 0.015 \\ \dfrac{SD1\,(T_L)}{T^2\,(R / I)} = 0.058 \end{cases}$$

Weight:

w = 100 psf & 250 k (Roof Mech. Equip. Load)
A_{TOTAL} = 182384 ft²
W_{TOTAL} = 18488.4 k

Base Shear:

$V = C_s * W_{TOTAL} = 0.015 * 18488.4 = 277.3$ k
k = 1

$F_x = [\, w_x h_x^{(k)} / \Sigma w_i h_i^{(k)} \,]\, V$

APPENDIX D – BRACE STRENGTH CALCULATIONS

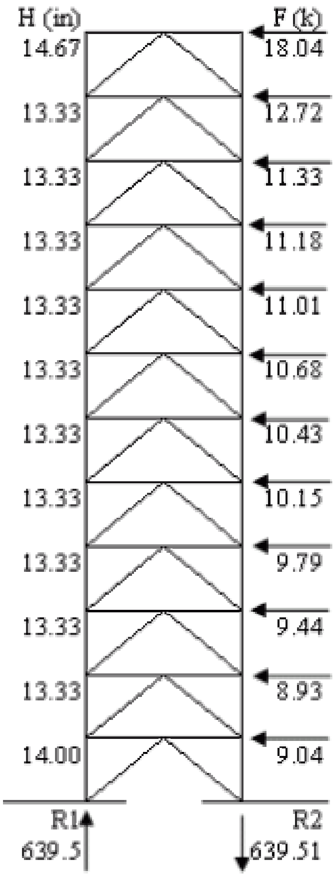
Axial Capacity Worksheet

$\phi = 0.90$
$\phi Pn = \phi(Ag)(Fcr)$ (E2-1)
$Fy = 50$ ksi, $E = 29000$ ksi
For $\lambda c < 1.5$ $Fcr = (0.658^{(\lambda c^2)})Fy$ (E2-2)
For $\lambda c > 1.5$ $Fcr = (0.877/\lambda c^2)Fy$ (E2-3)
$\lambda c = (Kl / r\pi)\sqrt{(Fy / E)}$ (E2-4)

K	L	Shape	Ag	r	λc	Eq.	Fcr	ϕPn	Shape	Kl/r <200	Comments
	ft		in²	in			ksi	K			
1.00	31.17	w8x31	9.12	2.02	2.45	E2-3	7.32	60.09	w8x31	185.16	weak axis buckling
1.00	30.56	w8x31	9.12	2.02	2.40	E2-3	7.62	62.51	w8x31	181.54	weak axis buckling
1.00	20.11	w8x31	9.12	2.02	1.58	E2-3	17.59	144.35	w8x31	119.47	weak axis buckling
1.00	19.15	w8x31	9.12	2.02	1.50	E2-2	19.39	159.19	w8x31	113.76	weak axis buckling
1.00	17.75	w8x31	9.12	2.02	1.39	E2-2	22.18	182.02	w8x31	105.45	weak axis buckling
1.00	16.67	w8x31	9.12	2.02	1.31	E2-2	24.41	200.34	w8x31	99.03	weak axis buckling
1.00	30.56	w8x35	10.30	2.03	2.39	E2-3	7.69	71.30	w8x35	180.65	weak axis buckling
1.00	20.10	w8x58	17.10	2.10	1.52	E2-3	19.03	292.82	w8x58	114.86	weak axis buckling
1.00	30.56	w10x49	14.40	2.54	1.91	E2-3	12.04	156.05	w10x49	144.38	weak axis buckling
1.00	30.56	w10x60	17.60	2.57	1.89	E2-3	12.33	195.26	w10x60	142.69	weak axis buckling
1.00	30.56	w10x68	20.00	2.59	1.87	E2-3	12.52	225.36	w10x68	141.59	weak axis buckling
1.00	30.86	w10x68	20.00	2.59	1.89	E2-3	12.28	221.01	w10x68	142.98	weak axis buckling

APPENDIX E – DEFLECTION CALCULATION USING VIRTUAL WORK

Member		Length in	Section	Area in^2	F_D k	F_V 1k	$\frac{(F_D)(F_V)(L)}{AE}$
1	Ca	180.00	14x211	62.0	539.41	7.39	0.3991
1	Cb	180.00	14x211	62.0	-539.41	-7.39	0.3991
1	Ba	216.33	8x31	9.1	120.31	0.91	0.0896
1	Bb	216.33	8x31	9.1	-120.31	-0.91	0.0897
1	Ma	120.00	18x35	10.3	-71.22	-0.50	0.0143
1	Mb	120.00	18x35	10.3	62.25	0.50	0.0125
2	Ca	160.00	14x211	62.0	457.21	6.73	0.2738
2	Cb	160.00	14x211	62.0	-457.21	-6.73	0.2738
2	Ba	200.00	8x31	9.1	102.75	0.83	0.0645
2	Bb	200.00	8x31	9.1	-102.75	-0.83	0.0646
2	Ma	120.00	18x35	10.3	-66.15	-0.50	0.0133
2	Mb	120.00	18x35	10.3	57.16	0.50	0.0115
3	Ca	160.00	14x145	42.7	380.59	6.06	0.2980
3	Cb	160.00	14x145	42.7	-380.59	-6.06	0.2980
3	Ba	200.00	8x31	9.1	95.77	0.83	0.0601
3	Bb	200.00	8x31	9.1	-95.77	-0.83	0.0602
3	Ma	120.00	18x35	10.3	-62.18	-0.50	0.0125
3	Mb	120.00	18x35	10.3	52.75	0.50	0.0106
4	Ca	160.00	14x145	42.7	310.56	5.40	0.2167
4	Cb	160.00	14x145	42.7	-310.56	-5.40	0.2167
4	Ba	200.00	8x31	9.1	87.54	0.83	0.0549
4	Bb	200.00	8x31	9.1	-87.54	-0.83	0.0551
4	Ma	120.00	18x35	10.3	-57.42	-0.50	0.0115
4	Mb	120.00	18x35	10.3	47.63	0.50	0.0096
5	Ca	160.00	14x120	35.3	246.87	4.73	0.1825
5	Cb	160.00	14x120	35.3	-246.87	-4.73	0.1825
5	Ba	200.00	8x31	9.1	79.61	0.83	0.0500
5	Bb	200.00	8x31	9.1	-79.61	-0.83	0.0501
5	Ma	120.00	18x35	10.3	-52.84	-0.50	0.0106
5	Mb	120.00	18x35	10.3	42.69	0.50	0.0086
6	Ca	160.00	14x120	35.3	190.04	4.06	0.1206
6	Cb	160.00	14x120	35.3	-190.04	-4.06	0.1206
6	Ba	200.00	8x31	9.1	71.04	0.83	0.0446
6	Bb	200.00	8x31	9.1	-71.04	-0.83	0.0447
6	Ma	120.00	18x35	10.3	-47.84	-0.50	0.0096
6	Mb	120.00	18x35	10.3	37.41	0.50	0.0075
7	Ca	160.00	14x99	29.1	140.07	3.40	0.0903
7	Cb	160.00	14x99	29.1	-140.07	-3.40	0.0903
7	Ba	200.00	8x31	9.1	62.47	0.83	0.0392
7	Bb	200.00	8x31	9.1	-62.47	-0.83	0.0393
7	Ma	120.00	18x35	10.3	-42.82	-0.50	0.0086
7	Mb	120.00	18x35	10.3	32.14	0.50	0.0065
8	Ca	160.00	14x99	29.1	97.3	2.73	0.0504
8	Cb	160.00	14x99	29.1	-97.3	-2.73	0.0504
8	Ba	200.00	8x31	9.1	53.46	0.83	0.0336



8	Bb	200.00	8x31	9.1	-53.46	-0.83	0.0336
8	Ma	120.00	18x35	10.3	-37.58	-0.50	0.0075
8	Mb	120.00	18x35	10.3	26.57	0.50	0.0053
9	Ca	160.00	14x68	20.0	61.78	2.07	0.0353
9	Cb	160.00	14x68	20.0	-61.78	-2.07	0.0353
9	Ba	200.00	8x31	9.1	44.39	0.83	0.0279
9	Bb	200.00	8x31	9.1	-44.39	-0.83	0.0279
9	Ma	120.00	18x35	10.3	-32.22	-0.50	0.0065
9	Mb	120.00	18x35	10.3	21.04	0.50	0.0042
10	Ca	160.00	14x68	20.0	33.74	1.40	0.0130
10	Cb	160.00	14x68	20.0	-33.74	-1.40	0.0130
10	Ba	200.00	8x31	9.1	35.06	0.83	0.0220
10	Bb	200.00	8x31	9.1	-35.06	-0.83	0.0221
10	Ma	120.00	18x35	10.3	-26.7	-0.50	0.0054
10	Mb	120.00	18x35	10.3	15.37	0.50	0.0031
11	Ca	160.00	14x43	12.6	13.23	0.73	0.0042
11	Cb	160.00	14x53	15.6	-13.23	-0.73	0.0034
11	Ba	200.00	8x31	9.1	25.63	0.83	0.0161
11	Bb	200.00	8x31	9.1	-25.63	-0.83	0.0161
11	Ma	120.00	18x35	10.3	-21.74	-0.50	0.0044
11	Mb	120.00	18x35	10.3	9.02	0.50	0.0018
12	Ca	176.00	14x43	12.6	0	0.00	0.0000
12	Cb	176.00	14x53	15.6	0	0.00	0.0000
12	Ba	213.12	8x31	9.1	16.02	0.89	0.0115
12	Bb	213.12	8x31	9.1	-16.02	-0.89	0.0115
12	Ma	120.00	18x35	10.3	18.04	1.00	0.0072
12	Mb	120.00	18x35	10.3	0	0.00	0.0000
						Δ =	4.3589
If beams are assumed to be a rigid diaphragm (A = ∞)							
						Δ =	4.1760

APPENDIX F – PICTURES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