303 Third Street

Cambridge, Massachusetts

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AE 481W – Thesis

Advisor: Dr. Ali Memari

Technical Assignment 3

December 3, 2007

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EXECUTIVE SUMMARY

The purpose of this report is to determine through analytical methods the response of the lateral force resisting system implemented for the design of the 86.5 foot tall residential facility at 303 Third Street, Cambridge, MA under seismic and wind loads.

EXISTING LATERAL SYSTEM

This is a dual lateral system comprised of exterior moment frames and concentrically braced steel frames in both the N-S and E-W directions and moment frames in the E-W direction. These frames consist of wide flange columns, wide flange beams at each story and two HSS (hollow structural section) diagonal braces between each story and may include moment connections depending on the frame type.

GRAVITY SYSTEM

The existing gravity system is a 3-1/4” lightweight concrete slab on a 3” deep 16 gage metal composite deck and WWF 6x6 W2.1xW2.1 reinforcing. Supporting the slab are W12x16 composite steel beams which span 18’-1” N-S in a typical bay. The beams frame into composite steel girders on the interior which are typically W14x82 spanning E-W.

CONCLUSIONS

In order to determine 303 Third Street’s lateral resisting system response to seismic and wind loads a model of Tower 333’s lateral system was created in ETABS. Only the braced frames and perimeter moment frames were modeled and connected with a rigid diaphragm on each floor. Then the model was loaded with seismic and wind forces calculated using spreadsheets in accordance with ASCE-7 ’05 and analyzed under the different load combinations required by ASCE-7. Using this ETABS model, in conjunction with hand calculated spot checks I was able to confirm that the assumptions made prior to the design of 303 Third Street were correct. These assumptions include drift limitations of L/400 for wind and ASCE-7 ’05 section 12.8.6 allowable drift for seismic. An examination of the drift results reveals that the dual system as originally designed is well balanced and subject to only minimal building torsions.

EXISTING STRUCTURAL SYSTEM

FOUNDATION:

The slab on grade concrete is normal weight (145 pcf dry unit weight) and has a minimum 28-day strength of 3500 psi. The 5” slab on grade is reinforced with 6x6 W2.9xW2.9 welded wire fabric. Column loads are supported by square spread footings (f’c = 4000 psi) ranging from 5’-6” to 14’-0”. The spread footing bear directly on the undisturbed, natural outwash sand, marine clay, or marine sand deposits proportioned utilizing a maximum bearing pressure of 2.5 tons per square-foot. The foundation also contains a few internal and external piers (f’c = 4000 psi) for supporting larger loads. The foundation bears on belled caissons with a typical depth of 20’. The caissons bear on 3 TSF bearing material. A groundwater cut-off at the perimeter is maintained as well as underdraining of the lowest level slab to avoid hydrostatic uplift forces acting on the lowest level slab. The continuous perimeter wall footings are founded at least 12 inches below the surface of the relatively impervious marine clay deposit to provide a groundwater cut-off. The surface of the bedrock deposit was observed to vary from 66.3 to 90 feet below the existing ground surface.

FLOOR SYSTEM:

The sublevel floor system P1 consists of a 4 ½” normal weight concrete (f’c = 5000 psi) slab on a 3” deep 18 gage composite metal floor deck reinforced with #5 rebar at 12”parallel to the deck and #4 rebar at 12” temp for a total slab thickness of 7 ½”. The slab is supported by steel beams with typical sizes ranging from W12 to W18. Wide flange beams typically span 25’ with 8’ spacing. Composite action is created by ¾” diameter shear studs with 5½” length. Girders are also wide flanges sized up to W24 with cambers over 1”. The typical floor system throughout the rest of the building is 3 ¼” light weight concrete slab on a 3” deep 16 gage composite metal floor deck reinforced with 6x6 W2.1xW2.1 welded wire fabric. This slab is supported by steel beams with typical sizes ranging from W12 to W14. Wide flange beams typically span 18-26’ with 12’-6” spacing.

COLUMNS:

The columns are ASTM A992 Grade 50 wide flange steel shapes laid out in a mostly rectangular grid. The columns act as the primary gravity resistance members. The columns that are attached as braced and moment frames are also the main lateral resistant force members. The braces between columns are ASTM A 500 Grade B HSS shapes ranging in size from 7x5x1/2” to 9x7x5/8”. The largest column is a W14x159 and the smallest is a W12x53 on the ground floor. The maximum unbraced length is 15’ which is the floor to floor height of the ground floor. Column splices occur every 20’ – 25’ at 4’-0” above the floor.

LATERAL FRAMING:

There is a dual lateral system implemented consisting of concentrically braced steel frames in both the N-S and E-W directions and moment frames in the E-W direction. These frames consist of wide flange columns, wide flange beams at each story and two HSS (hollow structural section) diagonal braces between each story and may include moment connections depending on the frame type.

CODES

DESIGN CODES:

Building Code:

Massachusetts State Building Code – 6th Edition

Reinforced Concrete:

American Concrete Institute (ACI) 318 – 1995 Edition

Reinforced Masonry:

American Concrete Institute (ACI) 530 – 2005 Edition

Structural Steel:

American Institute of Steel Construction (AISC)
Load and Resistance Factor Design Specification for Structural Steel Buildings – Latest Edition

Metal Decking:

American Iron and Steel Institute (AISI)
Specification for the Design of Cold Formed Structural Members

Building Design Loads:

Massachusetts State Building Code – 6th Edition

THESIS SUBSTITUTED CODES:

American Society of Civil Engineering (ASCE)
Minimum Design Loads for Buildings and Other Structures – ASCE 7-05

American Institute of Steel Construction (AISC)
Steel Construction Manual – 13th Edition – 2005

The International Building Code – 2006

LOADS

DEAD LOADS:

Metal Deck + Light Weight Concrete		30 PSF Vulcraft Catalog
Steel Beams		AISC Values

Superimposed Dead Loads:

Mechanical, Electrical, Sprinkler		20 PSF
Ceiling Finishes		5 PSF
Floor Finishes		5 PSF

LIVE LOADS:	**Design Value**	**ASCE 7 Ch 4**
Floor Live Loads:		
Corridors above 1st floor	80 PSF	100 PSF
First floor lobbies, public areas and corridors	100 PSF	100 PSF
Assembly rooms	100 PSF	100 PSF
Residential	40 PSF + Partition	40 PSF
Retail	100 PSF	100 PSF
Exercise room	100 PSF	100 PSF
Slab on grade	100 PSF	N/A
Storage (light)	125 PSF	125 PSF
Loading dock slab on deck	250 PSF	250 PSF
Framed exterior at ground	100 PSF + Soil	N/A
Fire pump room	150 PSF	N/A
Stairs	100 PSF	100 PSF
Mechanical areas	150 PSF	N/A
Elevator machine room	150 PSF	N/A
Transformer vault	250 PSF	N/A
Parking levels and ramps	50 PSF	40 PSF
Roof Live Loads:		
Roof Live Loads		20 PSF min
Basic Uniform Snow Load (Pf)	30 PSF	

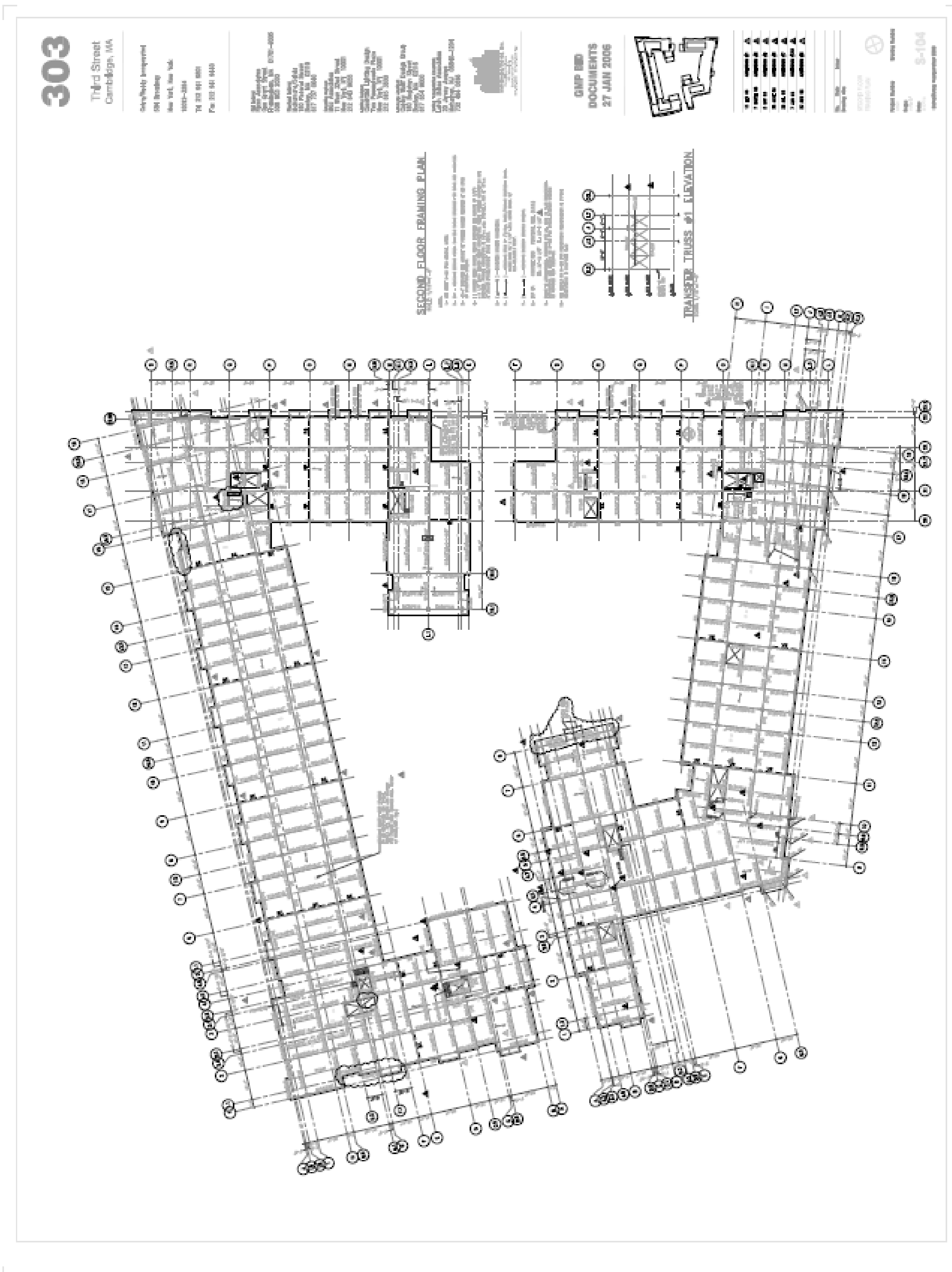

Figure 1: Existing Typical Framing Plan

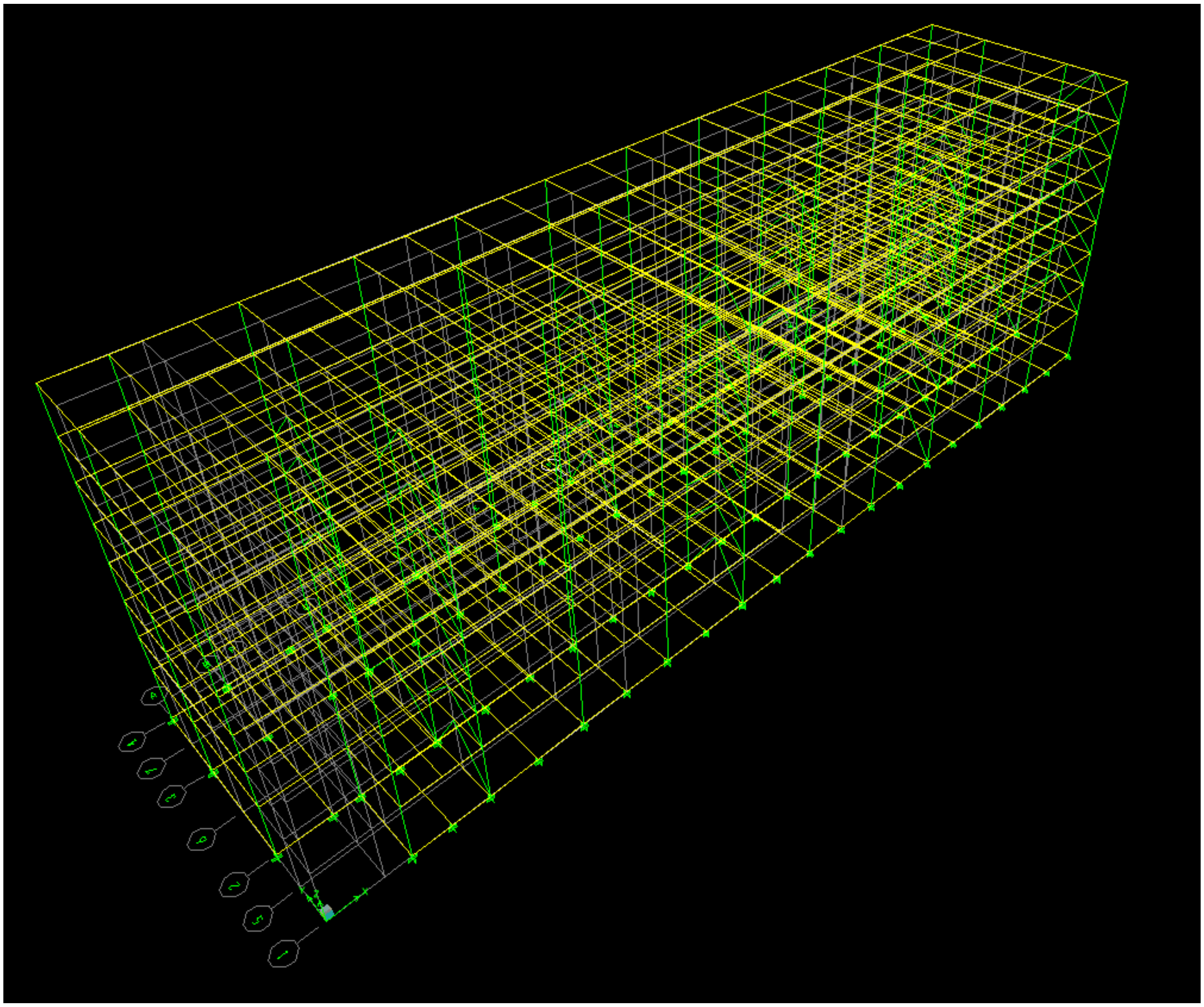

Figure 2: Existing Braced Frame and Moment Frame Lateral System

LATERAL LOADS

WIND FORCES

It was determined in Technical Report 1 that the lateral wind load was not the governing factor in the lateral system design. Therefore, the effect of wind loading on 303 Third Street was not analyzed with the ETABS model. Below is a summary of the computer wind loads and the resulting overturning moment calculations for both the North and South building.

North Building: N-S Wind Direction								
Floor	Height	Trib Height	Windward	Leeward	Total	Story Force	Total Shear	Overturning Moment
Ground	0	0	0	0	0	0	1116.71	54264.77
2	15	12.5	16.37	-15.29	31.66	163.46	1116.71	2451.84
3	25	10	16.37	-15.29	31.66	130.75	953.25	3268.76
4	35	10	18.16	-15.29	33.45	138.14	822.50	4835.07
5	45	10	19.05	-15.29	34.34	141.84	684.36	6382.89
6	55	10	19.77	-15.29	35.06	144.80	542.51	7963.99
7	65	10	20.48	-15.29	35.78	147.76	397.71	9604.24
8	75	10.75	21.20	-15.29	36.49	162.02	249.96	12151.43
Roof	86.5	5.75	21.74	-15.29	37.03	87.94	87.94	7606.54

North Building: E-S Wind Direction								
Floor	Height	Trib Height	Windward	Leeward	Total	Story Force	Total Shear	Overturning Moment
Ground	0	0	0	0	0	0	463.99	22630.33
2	15	12.5	16.37	-10.95	27.32	67.12	463.99	1006.76
3	25	10	16.37	-10.95	27.32	53.69	396.87	1342.17
4	35	10	18.16	-10.95	29.11	57.21	343.19	2002.19
5	45	10	19.05	-10.95	30.00	58.96	285.98	2653.41
6	55	10	19.77	-10.95	30.72	60.37	227.02	3320.46
7	65	10	20.48	-10.95	31.44	61.78	166.64	4015.66
8	75	10.75	21.20	-10.95	32.15	67.93	104.87	5094.44
Roof	86.5	5.75	21.74	-10.95	32.69	36.94	36.94	3195.25

South Building: N-S Wind Direction								
Floor	Height	Trib Height	Windward	Leeward	Total	Story Force	Total Shear	Overturning Moment
Ground	0	0	0	0	0	0	1116.71	54264.77
2	15	12.5	16.37	-15.29	31.66	163.46	1116.71	2451.84
3	25	10	16.37	-15.29	31.66	130.75	953.25	3268.76
4	35	10	18.16	-15.29	33.45	138.14	822.50	4835.07
5	45	10	19.05	-15.29	34.34	141.84	684.36	6382.89
6	55	10	19.77	-15.29	35.06	144.80	542.51	7963.99
7	65	10	20.48	-15.29	35.78	147.76	397.71	9604.24
8	75	10.75	21.20	-15.29	36.49	162.02	249.96	12151.43
Roof	86.5	5.75	21.74	-15.29	37.03	87.94	87.94	7606.54

South Building: E-S Wind Direction								
Floor	Height	Trib Height	Windward	Leeward	Total	Story Force	Total Shear	Overturning Moment
Ground	0	0	0	0	0	0	462.99	22582.95
2	15	12.5	16.37	-10.89	27.26	66.96	462.99	1004.38
3	25	10	16.37	-10.89	27.25	53.56	396.03	1339.00
4	35	10	18.16	-10.89	29.04	57.08	342.47	1997.75
5	45	10	19.05	-10.89	29.94	58.84	285.39	2647.71
6	55	10	19.77	-10.89	30.66	60.25	226.55	3313.49
7	65	10	20.48	-10.89	31.37	61.65	166.31	4007.43
8	75	10.75	21.20	-10.89	32.09	67.79	104.66	5084.22
Roof	86.5	5.75	21.74	-10.89	32.63	36.87	36.87	3188.95

Floor	HT (ft)	Kz	qz
1	15	0.57	15.007872
2	25	0.66	17.377536
3	35	0.76	20.010496
4	45	0.81	21.326976
5	55	0.85	22.38016
6	65	0.89	23.433344
7	75	0.93	24.486528
8	86.5	0.96	25.276416

North Building			South Building	
N-S Direction			N-S Direction	
L	195.625		L	196.52
B	413		B	413
h	86.5		h	86.5
Windward Cp	0.8		Windward Cp	0.8
Leeward Cp	-0.5		Leeward Cp	-0.5
E-W Direction			E-W Direction	
L	413		L	413
B	195.625		B	196.52
h	86.5		h	86.5
Windward Cp	0.8		Windward Cp	0.8
Leeward Cp	-0.298		Leeward Cp	-0.295

Figure 3: Lateral Forces Due to Wind

SEISMIC LOADS

The controlling forces in the lateral direction for 303 Third Street were determined to be the seismic forces. These forces were obtained through ASCE-7 ’05 Chapter 12 and are displayed in Figure 4. Using the shear forces per floor (in kips) from Figure 4 and inputting that data into the ETABS model acting at the center of gravity of each floor, it was found that the maximum displacement was 7.91 inches in the North-South direction. A summary of this info follows:

Story	Displacement (inches)
Roof	7.90643
7	6.13584
6	4.67811
5	3.38113
4	2.27969
3	1.39092
2	0.7196
1	0.26668
Base	0

When inspecting overturning moment the seismic forces produce a total moment of 115,375.2 FT-K, taking the total weight of the structure to the seismic base which is 1,333.82 K and multiplying by the moment arm from the center of gravity, to the edge of the core, 28 feet the resulting resistive moment is approximately 275,365 ft-kips. This value is about 1.5 times that of the overturning moment which implies there are no uplift forces due to overturning moment.

Equivalent Lateral Force Method					
Braced Frame Cu = 0.0547					
V	1792.624				
Moment Frame Cu = 0.0407					
V	1333.817				

North Building					
Braced Frame - k = 1.197			v	1792.624	
Floor	wx	hx	wx*hx^1.197	Cvx	Fx
Roof	3641.325	86.5	758346.56	0.23	311.64
8	3641.325	75	639304.52	0.20	262.72
7	3641.325	65	538662.49	0.17	221.36
6	3641.325	55	441035.53	0.14	181.24
5	3641.325	45	346860.48	0.11	142.54
4	3641.325	35	256749.08	0.08	105.51
3	3641.325	25	171630.17	0.05	70.53
2	3641.325	15	93119.51	0.03	38.27
Ground	3641.325	0	0	0	0
Sum			3245708.34		
Moment Frame - k = 1.31			v	1333.817	
Floor	wx	hx	wx*hx^1.31	Cvx	Fx
Roof	3641.325	86.5	1255308.71	0.39	515.87
8	3641.325	75	1041333.16	0.32	427.93
7	3641.325	65	863328.23	0.27	354.78
6	3641.325	55	693640.70	0.21	285.05
5	3641.325	45	533295.40	0.16	219.16
4	3641.325	35	383697.16	0.12	157.68
3	3641.325	25	246922.61	0.08	101.47
2	3641.325	15	126455.87	0.04	51.97
Ground	3641.325	0	0	0	0
Sum			5143981.83		

Figure 4: Lateral Forces Due to Wind in North Building

South Building					
Braced Frame - k = 1.197			v	1792.624	
Floor	wx	hx	wx*hx^1.197	Cvx	Fx
Roof	2887.541	86.5	601362.55	0.19	247.13
8	2887.541	75	506963.20	0.16	208.34
7	2887.541	65	427154.89	0.13	175.54
6	2887.541	55	349737.53	0.11	143.72
5	2887.541	45	275057.49	0.08	113.03
4	2887.541	35	203599.89	0.06	83.67
3	2887.541	25	136101.31	0.04	55.93
2	2887.541	15	73843.00	0.02	30.35
Ground	2887.541	0	0	0	0
Sum			2573819.86		
Moment Frame - k = 1.31			v	1333.817	
Floor	wx	hx	wx*hx^1.31	Cvx	Fx
Roof	2887.541	86.5	995449.43	0.31	409.08
8	2887.541	75	825768.58	0.25	339.35
7	2887.541	65	684612.14	0.21	281.34
6	2887.541	55	550051.33	0.17	226.04
5	2887.541	45	422898.84	0.13	173.79
4	2887.541	35	304268.67	0.09	125.04
3	2887.541	25	195807.58	0.06	80.47
2	2887.541	15	100278.46	0.03	41.21
Ground	2887.541	0	0	0	0
Sum			4079135.03		

Figure 4: Lateral Forces Due to Wind in South Building

LOAD CASES

The following load cases as obtained from chapter 2 of ASCE-7 ’05 were used in the analysis of 303 Third Street in ETABS.

1) 1.4(D + F)
2) 1.2(D + F + T) +1.6(L + H) + 0.5(L_r or S or R)
3) 1.2D + 1.6(L_r or S or R) + (L or 0.8W)
4) 1.2D + 1.6W + L + 0.5(L_r or S or R)
5) 1.2D + 1.0E + L + 0.2S
6) 0.9D + 1.6W + 1.6H
7) 0.9D + 1.0E + 1.6H
8.) 1.0D + 1.0L + 1.0E

LOAD DISTRIBUTION

Through hand calculations following ASCE-7 ’05 Chapter 12 the story shears for the seismic forces were computed. These values were then inserted into ETABS at each floor’s center of gravity in the Tower 333 model. After analysis, the results for shear in each moment frame at the seismic base level were obtained and compared to the total shear at the seismic base level. The shear forces in the N-S direction were approximately 173 kips. Compared to the total seismic base shear of 1334 kips, that results in 12.9% of the total base shear calculated from ASCE-7 ’05 to be taken by the perimeter moment frames. This 13% accumulation of base shear in the perimeter moment frames confirms the assumption that the frames were designed for at least 25% of the total base shear as required by code; in reality they are only taking roughly 12-13% of the total shear.

It is believed that the perimeter frames also play a significant role in minimizing torsion. This aspect of the structure will be analyzed in more detail as part of future investigations.

ETABS ANALYSIS

When using ETABS the model was simplified for 303 Third Street down to the lateral system consisting of the perimeter moment frames and the concentrically braced frames for a section of the North building only (see Figure 5). This was due to the off-axis grid structure of the other sections, which would have added difficulty to the modeling process and increase the chances of making an error. To allow all the frames to act as one system the members were connected to a rigid diaphragm at each floor. This analysis technique permits a more direct analysis and interpretation of the results and easier application of the loads. The seismic loads were then applied at each floor manually using the story forces calculated for the wind and seismic analysis (see Figures 3 and 4 for force results.) These forces were calculated in accordance with ASCE-7 ’05 chapters 6 and 12.

When viewing the animated results and story displacement values, it is clear that the wind and seismic forces develop little torsion. Acceptable maximum drift values that are below those mandated by ASCE-7 ’05, were also calculated using ETABS. The expected levels of shear in the moment frames and braced frames as assumed were confirmed. This, in addition to the fact that the forces were user-defined from hand calculations rather than calculated by ETABS helps to confirm that the ETABS model has run properly and has developed justifiable results.

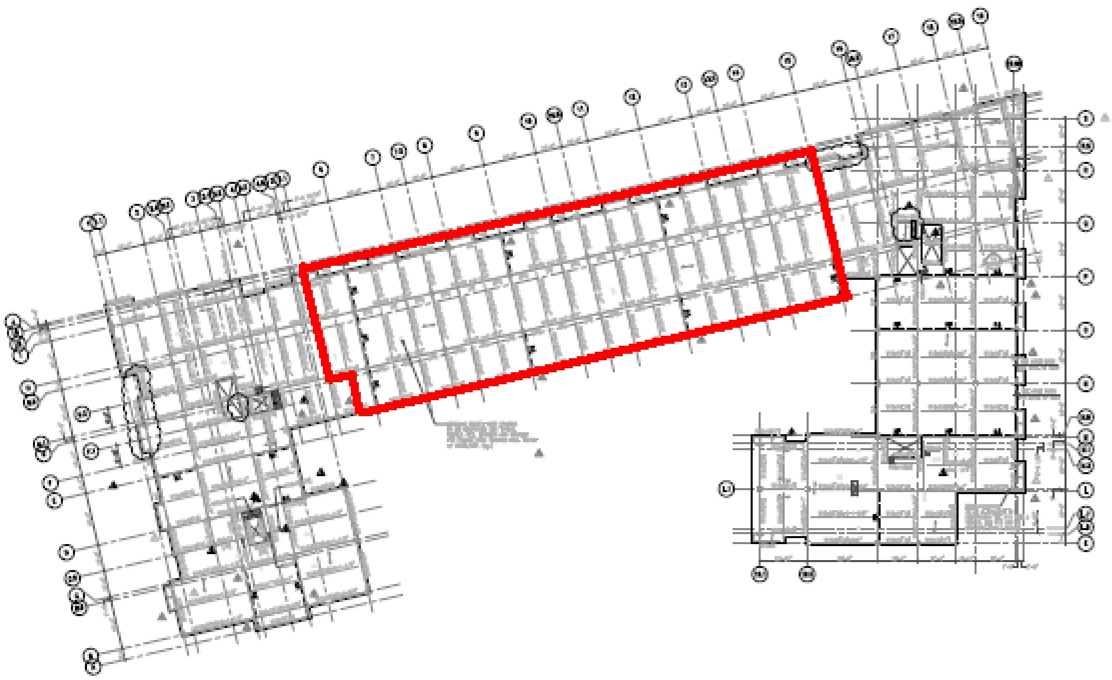

Figure 5: Section of North Building Modeled Using ETABS

CONCLUSIONS

By virtue of a thorough analysis utilizing the North Building of 303 Third Street structure modeled in ETABS as well as hand calculations, the response characteristics of 303 Third Street's lateral force resisting system to seismic loads was successfully determined. Modeling only the braced frames and perimeter moment frames for ease of analysis and connecting them with a rigid diaphragm on each floor, it was possible to apply the lateral loads as calculated by ASCE-7 '05 to 303 Third Street and run a analysis of the lateral systems.

I have concluded through the 303 Third Street ETABS model as well as confirmation of spot checks that the assumptions the engineer of record made in designing 303 Third Street were confirmed. These assumptions include drift limitations of L/400 for wind and ASCE-7 '05 section 12.8.6 allowable drift for seismic. Based on the relative base shear distributions, it was determined that the moment frames resist 13% of the lateral load in the N-S direction. An examination of the drift results reveals that the dual system as originally designed is well balanced and subject to only minimal building torsions. Based on this analysis, it is reasonable to conclude that the loads and force distributions used as part of this educational study are in agreement with the original values used by the designers.

APPENDIX

PENN STATE UNIVERSITY

CLASS:
DATE:
ASSIGNMENT:

PAGE: ___ of ___

WIND CALC

SINCE $T_a < 1$, $f > 1\,Hz$ ∴ RIGID

$I_w = 1.0$ FOR OCCUPANCY CAT 2

BASIC WIND SPEED: $V = 110$ MPH

DIRECTIONALITY FACTOR: $K_d = 0.85$ - ONLY USED W/LOAD COMBINATIONS

EXPOSURE CATEGORY: B

VELOCITY PRESSURE EXPOSURE COEFFICIENT: CASE 2 SINCE > 60' height

$K_h =$

floor	HT	K_z
1	15	0.57
2	25	0.66
3	35	0.76
4	45	0.81
5	55	0.85
6	65	0.89
7	75	0.93
8	86.5	0.96

CONSERV

TOPOGRAPHIC FACTOR: $K_{zt} = 1.0$ ASSUMED

GUST FACTOR: $G = 0.85$ FOR RIGID STRUCTURE (CONSERVATIVE)

RESONANT RESPONSE FACTOR: 1.0 (CONSERVATIVE)

MEAN ROOF HT = 86.5'

ENCLOSURE CLASSIFICATION: FULLY ENCLOSED

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

INTERNAL PRESSURE COEFFICIENTS: $+0.55, -0.55 = GC_{pi}$

REDUCTION FACTOR FOR LARGE VOLUME BLDGS: $R_i = 1.0$ 6.5.11.1.1

EXTERNAL PRESSURE COEFFICIENTS p.48-49 of ASCE

North Building: N-S Wind Direction								
Floor	Height	Trib Height	Windward	Leeward	Total	Story Force	Total Shear	Overturning Moment
Ground	0	0	0	0	0	0	1116.71	54264.77
2	15	12.5	16.37	-15.29	31.66	163.46	1116.71	2451.84
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8	75	10.75	21.20	-10.95	32.15	67.93	104.87	5094.44
Roof	86.5	5.75	21.74	-10.95	32.69	36.94	36.94	3195.25

South Building: N-S Wind Direction								
Floor	Height	Trib Height	Windward	Leeward	Total	Story Force	Total Shear	Overturning Moment
Ground	0	0	0	0	0	0	1116.71	54264.77
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3	25	10	16.37	-10.89	27.25	53.56	396.03	1339.00
4	35	10	18.16	-10.89	29.04	57.08	342.47	1997.75
5	45	10	19.05	-10.89	29.94	58.84	285.39	2647.71
6	55	10	19.77	-10.89	30.66	60.25	226.55	3313.49
7	65	10	20.48	-10.89	31.37	61.65	166.31	4007.43
8	75	10.75	21.20	-10.89	32.09	67.79	104.66	5084.22
Roof	86.5	5.75	21.74	-10.89	32.63	36.87	36.87	3188.95

Floor	HT (ft)	Kz	qz
1	15	0.57	15.007872
2	25	0.66	17.377536
3	35	0.76	20.010496
4	45	0.81	21.326976
5	55	0.85	22.38016
6	65	0.89	23.433344
7	75	0.93	24.486528
8	86.5	0.96	25.276416

North Building			South Building	
N-S Direction			N-S Direction	
L	195.625		L	196.52
B	413		B	413
h	86.5		h	86.5
Windward Cp	0.8		Windward Cp	0.8
Leeward Cp	-0.5		Leeward Cp	-0.5
E-W Direction			E-W Direction	
L	413		L	413
B	195.625		B	196.52
h	86.5		h	86.5
Windward Cp	0.8		Windward Cp	0.8
Leeward Cp	-0.298		Leeward Cp	-0.295

SEISMIC

Index Force Analysis						
North Building						
Floor	Area	Weight	Fx	Wall Area	Wall Wt	Wall Fx
Ground	45364	3629.12	36.2912	813.6667	12.205	0.12205
2	45364	3629.12	36.2912	813.6667	12.205	0.12205
3	45364	3629.12	36.2912	813.6667	12.205	0.12205
4	45364	3629.12	36.2912	813.6667	12.205	0.12205
5	45364	3629.12	36.2912	813.6667	12.205	0.12205
6	45364	3629.12	36.2912	813.6667	12.205	0.12205
7	45364	3629.12	36.2912	813.6667	12.205	0.12205
8	45364	3629.12	36.2912	813.6667	12.205	0.12205
Roof	45364	3629.12	36.2912	813.6667	12.205	0.12205
sum	408276	32662.08	326.6208	7323	109.845	1.09845
		Total Base Shear		327.7193		
		Overturning Moment		28347.72		
South Building						
Floor	Area	Weight	Fx	Wall Area	Wall Wt	Wall Fx
Ground	35940.07	2875.206	28.752056	822.3333	12.335	0.12335
2	35940.07	2875.206	28.752056	822.3333	12.335	0.12335
3	35940.07	2875.206	28.752056	822.3333	12.335	0.12335
4	35940.07	2875.206	28.752056	822.3333	12.335	0.12335
5	35940.07	2875.206	28.752056	822.3333	12.335	0.12335
6	35940.07	2875.206	28.752056	822.3333	12.335	0.12335
7	35940.07	2875.206	28.752056	822.3333	12.335	0.12335
8	35940.07	2875.206	28.752056	822.3333	12.335	0.12335
Roof	35940.07	2875.206	28.752056	822.3333	12.335	0.12335
sum	323460.6	25876.85	258.768504	7401	111.015	1.11015
		Total Base Shear		259.8787		
		Overturning Moment		22479.5		

Equivalent Lateral Force Method					
Braced Frame Cu = 0.0547					
V	1792.624				
Moment Frame Cu = 0.0407					
V	1333.817				

North Building					
Braced Frame - k = 1.197			v	1792.624	
Floor	wx	hx	wx*hx^1.197	Cvx	Fx
Roof	3641.325	86.5	758346.56	0.23	311.64
8	3641.325	75	639304.52	0.20	262.72
7	3641.325	65	538662.49	0.17	221.36
6	3641.325	55	441035.53	0.14	181.24
5	3641.325	45	346860.48	0.11	142.54
4	3641.325	35	256749.08	0.08	105.51
3	3641.325	25	171630.17	0.05	70.53
2	3641.325	15	93119.51	0.03	38.27
Ground	3641.325	0	0	0	0
Sum			3245708.34		
Moment Frame - k = 1.31			v	1333.817	
Floor	wx	hx	wx*hx^1.31	Cvx	Fx
Roof	3641.325	86.5	1255308.71	0.39	515.87
8	3641.325	75	1041333.16	0.32	427.93
7	3641.325	65	863328.23	0.27	354.78
6	3641.325	55	693640.70	0.21	285.05
5	3641.325	45	533295.40	0.16	219.16
4	3641.325	35	383697.16	0.12	157.68
3	3641.325	25	246922.61	0.08	101.47
2	3641.325	15	126455.87	0.04	51.97
Ground	3641.325	0	0	0	0
Sum			5143981.83		

South Building					
Braced Frame - k = 1.197			v	1792.624	
Floor	wx	hx	wx*hx^1.197	Cvx	Fx
Roof	2887.541	86.5	601362.55	0.19	247.13
8	2887.541	75	506963.20	0.16	208.34
7	2887.541	65	427154.89	0.13	175.54
6	2887.541	55	349737.53	0.11	143.72
5	2887.541	45	275057.49	0.08	113.03
4	2887.541	35	203599.89	0.06	83.67
3	2887.541	25	136101.31	0.04	55.93
2	2887.541	15	73843.00	0.02	30.35
Ground	2887.541	0	0	0	0
Sum			2573819.86		
Moment Frame - k = 1.31			v	1333.817	
Floor	wx	hx	wx*hx^1.31	Cvx	Fx
Roof	2887.541	86.5	995449.43	0.31	409.08
8	2887.541	75	825768.58	0.25	339.35
7	2887.541	65	684612.14	0.21	281.34
6	2887.541	55	550051.33	0.17	226.04
5	2887.541	45	422898.84	0.13	173.79
4	2887.541	35	304268.67	0.09	125.04
3	2887.541	25	195807.58	0.06	80.47
2	2887.541	15	100278.46	0.03	41.21
Ground	2887.541	0	0	0	0
Sum			4079135.03		

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SEISMIC WEIGHT

DEAD LOAD

30 PSF CONC SLAB (VULCRAFT CATALOG)
20 PSF MECH, ELEC, SPRINKLERS
10 PSF FLOOR + CEILING FINISHES
15 PSF STEEL MEMBERS + DECK – CONSERVATIVE
5 PSF MISC
$\boxed{80 \text{ PSF}}$

STORAGE AREAS

0.25 × 125 = 31.25 PSF

PARTITION LOADS 20 PSF

ASSUME 80 PSF ROOF LOAD → CONSERVATIVE

AREAS:

NORTH BUILDING: 11064.5 SF (WEST END)
18897.62 SF (MIDDLE SECTION)
15401.92 SF (EAST END)
$\boxed{45364 \text{ SF}}$ TOTAL

SOUTH BUILDING: 10994.27 SF (WEST END)
12833.85 SF (MIDDLE SECTION)
12111.95 SF (EAST END)
$\boxed{35940.07 \text{ SF}}$ TOTAL

WALLS

WALLS ARE CURTAIN WALLS: 15 PSF

PERIMETER

NORTH BLDG: WEST = 154' + 49' + 69' = 272'
SOUTH = 88' + 124' = 212'
NORTH = 425' + 56.5' = 481.5'
EAST = 180' + 50' + 18' + 7' = 255'
~1220.5'

SOUTH BLDG: NORTH = 173' + 66' + 170' = 409'
EAST = 57.5' + 25.5' + 190.5' = 273.5'
WEST = 125' + 136' = 261'
SOUTH = 290' = 290'
1233.5'

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SEISMIC DESIGN

OCCUPANCY CATEGORY: 2
IMPORTANCE FACTOR: 1.0
SITE CLASS: E ← ASSUMED S_3 UNDER MASS BUILDING CODE ASSUMED E TO BE CONSERVATIVE

303 THIRD ST CAMBRIDGE, MA
LAT: 42.365°   LONG: -71.083°

$S_s = 0.28$ $F_a = 2.404$
$S_1 = 0.068$ $F_v = 3.5$

$S_{MS} = F_a S_s = 2.404(0.28) = 0.673$

$S_{M1} = F_v S_1 = 3.5(0.068) = 0.238$

$S_{DS} = 2/3\, S_{MS} = \frac{2}{3}(0.673) = 0.449$

$S_{D1} = 2/3\, S_{M1} = \frac{2}{3}(0.238) = 0.159$

NOTE: STRUCTURAL ENGINEER USES MASS STATE BUILDING CODE BUT I AM USING ASCE 7-05 AND IBC 2006 FOR ANALYSIS

RESPONSE MODIFICATION FACTORS

ORDINARY STEEL CONCENTRICALLY BRACED FRAME: $R = 3.25$
ORDINARY STEEL MOMENT FRAME: $R = 3.5$

DETERMINATION OF C_s FOR BRACED FRAME

$h_n = 85.5'$

$S_{D1} = 0.159 > 0.15 \rightarrow C_u = 1.59$ FROM INTERPOLATION

$T = C_u T_a = 1.59(0.02(85.5)^{0.75}) = 0.894\ s$

$C_s \le S_{D1} / ((R/I) \cdot T) = \frac{0.159}{(3.25/1) \cdot 0.894} = 0.0547$ ← GOVERNS

$C_s \ge 0.044 S_{DS} I = 0.044(0.449)(1) = 0.01976$

$\boxed{C_s = 0.0547}$

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DETERMINE OF C_s FOR MOMENT FRAME

$T_a = 0.702\ s$

$T = C_u T_a = 1.159\ (0.02\ (85.5)^{0.8}) = 1.117\ s$

$C_s \leq \frac{S_{D1}}{(R/I)T} = \frac{0.159}{(3.5/1)1.117} = 0.0407 \leftarrow$ GOVERNS

$C_s \geq 0.044\ S_{DS} I = 0.01976$

$\boxed{C_s = 0.0407}$

INTERPOLATION FOR K

MOMENT FRAME: $K = 1 + (1.117 - 0.5)\left(\frac{2-1}{2.5-0.5}\right) = 1.3085 \sim 1.31$

BRACED FRAME: $K = 1 + (0.894 - 0.5)\left(\frac{2-1}{2.5-0.5}\right) = 1.197$

SPOT CHECK

SEISMIC STORY DRIFT

DEFLECTION @ LEVEL X

$$\delta_x = \frac{C_d \delta}{I} \qquad I = 1.0, \quad C_d = 5.0 \rightarrow \text{BRACED FRAMES}$$

@ ROOF

SIESMIC IN Y DIRECTION

$\delta_{xe} = 7.906 \rightarrow$ FROM ETABS

$$\delta_x = \frac{5(7.906)}{1.0} = 39.53''$$

STORY LIMIT DRIFT

$\max\Delta = 0.025\, h_{sx}$

$h_{sx} = 86.5'(12'') = 1038''$

$\max\Delta = 0.025\,(1038) = 25.95''$

$\delta_x = 39.53'' > 25.95''$

THE SLIGHT DIFFERENCE IS DUE TO THE HIGH SEISMIC SHEAR VALUES WHICH WERE CALCULATED USING THE ENTIRE SEISMIC WEIGHT OF THE NORTH BUILDING, RATHER THAN JUST THE SEGMENT ANALYSED IN ETABS. IN REALITY, THE DRIFT WOULD BE BELOW THIS max Δ CASE.

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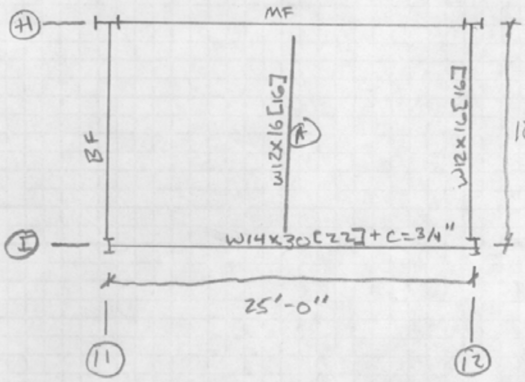
TYPICAL SLAB - 7TH FLOOR   SOUTH BLDG



3 1/4" LT WT CONC
3" METAL DECK

LOAD FOR TYP FLOOR:

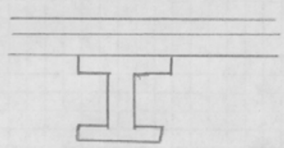
50 PSF SLAB
30 PSF SDL
100 PSF LIVE LOAD - CONS.
180 PSF

1.2D+1.6L = 256 PSF (12.5')
+1.2(16) ← STEEL WT
3219.2 PLF
BEAM A

MAX SHEAR = $\frac{PL}{2} = \frac{3.22(18+\frac{1}{12})}{2} = 29.1$ K

MID SPAN MOMENT = $\frac{wl^2}{8} = \frac{3.22(18+\frac{1}{12})^2}{8} = 131.6$ FT-K

TRY LOCATION TFL WITH $Y2 = 2"$ ($\phi M_n = 141$ FT-K, $\Sigma Q_N = 236^K$)



b_{eff} < 1/4 SPAN = $\frac{1}{4}$(18'-1") = 54.25"
SPACING = 150"

$Y2 = 6.25 - 1.28 = 4.97$ → CLOSE TO 5"
✓ok

$T_s = A_s F_y = 4.71(50\text{ KSI}) = 235.5^K$
$\Sigma Q_N = 236^K$

$CC = 236^K = 0.85 f_c' b_{eff}\, a$

$a = \frac{236}{0.85(4)(54.25)} = 1.28"$

@ $Y2 = 5"$, TFL, $\phi M_n = 141$ FT-K

141 FT-K > 131.6 FT-K

∴ COMPOSITE BEAM IS PROPERLY SIZED

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COLUMN 6-F - 7TH FLOOR W14X68

LOADS:

DL: 50 PSF (SLAB) + 20 (MEP) + 5 (BEAMS) = 75 PSF

LL: 40 PSF (RESIDENTIAL) + 20 (PARTITIONS) = 60 PSF

EXTERIOR COLUMN $A_T = (9+12) \times \left(\frac{15'-11''}{2}\right) = 167.125$ SF $\overset{\text{ONE STORY ABOVE}}{(2)} = 334.25$

LL RED

$$L = L_0\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right) = 60\left(0.25 + \frac{15}{\sqrt{2(334.25)}}\right) = 49.81 \text{ PSF}$$

LOAD COMB

$1.2D + 1.6L = 1.2(75) + 1.6(60) = 186$ PSF

$P_{FL} = 186(334.25) = 62170.5 = 62.2^K = P_U$

$KL = 10'$ FROM AISC TABLE 4-1

ϕP_n FOR W14X68 $= 755^K \gg 62.2^K$

THIS COLUMN MUST HAVE BEEN DESIGNED FOR FLEXURE OR AS A BRACING ELEMENT AS IT IS PART OF AN EXTERIOR MOMENT FRAME. THE MEMBER CAN CERTAINLY WITHSTAND THE GRAVITY LOAD OF THE STRUCTURE.

STAAD SPOT CHECK OF BRACED FRAME

Job Information

	Engineer	Checked	Approved
Name:	Tufts		
Date:	04-Oct-07		

Structure Type	PLANE FRAME

Number of Nodes	34	Highest Node	34
Number of Elements	35	Highest Beam	51

Number of Basic Load Cases	1
Number of Combination Load Cases	0

Included in this printout are data for:

All	The Whole Structure

Included in this printout are results for load cases:

Type	L/C	Name
Primary	1	LOAD CASE 1

Nodes

Node	X (ft)	Y (ft)	Z (ft)
1	0.000	0.000	0.000
2	0.000	15.000	0.000
3	0.000	25.000	0.000
4	0.000	35.000	0.000
5	0.000	45.000	0.000
6	0.000	55.000	0.000
7	0.000	65.000	0.000
8	0.000	75.000	0.000
9	0.000	86.500	0.000
10	18.083	0.000	0.000
11	18.083	15.000	0.000
12	18.083	25.000	0.000
13	18.083	35.000	0.000
14	18.083	45.000	0.000
15	18.083	55.000	0.000
16	18.083	65.000	0.000
17	18.083	75.000	0.000
18	18.083	86.500	0.000
19	9.042	86.500	0.000
20	9.042	75.000	0.000
21	9.042	65.000	0.000
22	9.042	55.000	0.000

Nodes Cont...

Node	X (ft)	Y (ft)	Z (ft)
23	9.042	45.000	0.000
24	9.042	35.000	0.000
25	9.042	25.000	0.000
26	9.042	15.000	0.000
27	0.000	29.000	0.000
28	0.000	49.000	0.000
29	0.000	69.000	0.000
30	0.000	6.500	0.000
31	18.083	6.500	0.000
32	18.083	29.000	0.000
33	18.083	49.000	0.000
34	18.083	69.000	0.000

Beams

Beam	Node A	Node B	Length (ft)	Property	β (degrees)
17	1	10	18.083	1	0
18	9	18	18.083	1	0
19	2	11	18.083	1	0
20	3	12	18.083	1	0
21	4	13	18.083	1	0
22	5	14	18.083	1	0
23	6	15	18.083	1	0
24	7	16	18.083	1	0
25	8	17	18.083	1	0
26	8	19	14.629	6	0
27	17	19	14.628	6	0
28	1	26	17.514	2	0
29	10	26	17.514	2	0
30	2	25	13.482	2	0
31	11	25	13.481	2	0
32	3	24	13.482	2	0
33	12	24	13.481	2	0
34	4	23	13.482	3	0
35	13	23	13.481	3	0
36	14	22	13.481	4	0
37	5	22	13.482	4	0
38	6	21	13.482	4	0
39	15	21	13.481	4	0
40	7	20	13.482	5	0
41	16	20	13.481	5	0
42	1	30	6.500	12	0
43	27	28	20.000	9	0

Beams Cont...

Beam	Node A	Node B	Length (ft)	Property	β (degrees)
44	28	29	20.000	8	0
45	29	9	17.500	7	0
46	10	31	6.500	11	0
47	31	32	22.500	11	0
48	32	33	20.000	9	0
49	33	34	20.000	8	0
50	34	18	17.500	7	0
51	30	27	22.500	10	0

Section Properties

Prop	Section	Area (in^2)	I_{yy} (mil^4)	I_{zz} (mil^4)	J (mil^4)	Material
1	W12X30	8.790	20.3E12	238E12	437E 9	STEEL
2	HSST9X5X0.5	11.600	45.2E12	115E12	107E12	STEEL
3	HSST7X5X0.625	11.700	40.6E12	69.4E12	86.3E12	STEEL
4	HSST7X5X0.5	9.740	35.6E12	60.6E12	73.8E12	STEEL
5	HSST6X4X0.5	7.880	17.8E12	34E12	39.3E12	STEEL
6	HSST6X4X0.375	6.180	14.9E12	28.3E12	31.9E12	STEEL
7	W14X61	17.900	107E12	640E12	2.01E12	STEEL
8	W14X68	20.000	121E12	723E12	2.8E12	STEEL
9	W14X90	26.500	362E12	999E12	3.82E12	STEEL
10	W14X120	35.300	495E12	1.38E15	8.99E12	STEEL
11	W14X132	38.800	548E12	1.53E15	11.9E12	STEEL
12	W14X145	42.700	677E12	1.71E15	14.7E12	STEEL

Supports

Node	X (kip/in)	Y (kip/in)	Z (kip/in)	rX (kip˙ft/deg)	rY (kip˙ft/deg)	rZ (kip˙ft/deg)
1	Fixed	Fixed	Fixed	Fixed	Fixed	Fixed
10	Fixed	Fixed	Fixed	Fixed	Fixed	Fixed

Basic Load Cases

Number	Name
1	LOAD CASE 1

Node Displacements

Node	L/C	X (in)	Y (in)	Z (in)	Resultant (in)	rX (rad)	rY (rad)	rZ (rad)
1	1:LOAD CASE	0.000	0.000	0.000	0.000	0.000	0.000	0.000
2	1:LOAD CASE	-34E 6	30.4E 6	0.000	45.7E 6	0.000	0.000	-282E 3
3	1:LOAD CASE	-62.7E 6	56.2E 6	0.000	84.2E 6	0.000	0.000	-519E 3
4	1:LOAD CASE	-154E 6	139E 6	0.000	208E 6	0.000	0.000	-1.28E 6
5	1:LOAD CASE	-239E 6	215E 6	0.000	322E 6	0.000	0.000	-1.98E 6
6	1:LOAD CASE	-304E 6	274E 6	0.000	409E 6	0.000	0.000	-2.52E 6
7	1:LOAD CASE	-659E 6	594E 6	0.000	887E 6	0.000	0.000	-5.48E 6
8	1:LOAD CASE	-1.35E 9	1.06E 9	0.000	1.71E 9	0.000	0.000	-9.74E 6
9	1:LOAD CASE	-637.328	-0.802	0.000	637.328	0.000	0.000	0.320
10	1:LOAD CASE	0.000	0.000	0.000	0.000	0.000	0.000	0.000
11	1:LOAD CASE	-34E 6	-30.7E 6	0.000	45.8E 6	0.000	0.000	-282E 3
12	1:LOAD CASE	-62.7E 6	-56.5E 6	0.000	84.4E 6	0.000	0.000	-519E 3
13	1:LOAD CASE	-154E 6	-139E 6	0.000	208E 6	0.000	0.000	-1.28E 6
14	1:LOAD CASE	-239E 6	-215E 6	0.000	322E 6	0.000	0.000	-1.98E 6
15	1:LOAD CASE	-304E 6	-274E 6	0.000	409E 6	0.000	0.000	-2.52E 6
16	1:LOAD CASE	-659E 6	-594E 6	0.000	888E 6	0.000	0.000	-5.48E 6
17	1:LOAD CASE	-1.35E 9	-1.06E 9	0.000	1.71E 9	0.000	0.000	-9.74E 6
18	1:LOAD CASE	-637.460	0.745	0.000	637.460	0.000	0.000	0.331
19	1:LOAD CASE	-3.36E 6	-258E 3	0.000	3.37E 6	0.000	0.000	-9.74E 6
20	1:LOAD CASE	-1.74E 6	-209E 3	0.000	1.75E 6	0.000	0.000	-5.48E 6
21	1:LOAD CASE	-1.15E 6	-169E 3	0.000	1.16E 6	0.000	0.000	-2.52E 6
22	1:LOAD CASE	-904E 3	-169E 3	0.000	920E 3	0.000	0.000	-1.98E 6
23	1:LOAD CASE	-557E 3	-141E 3	0.000	575E 3	0.000	0.000	-1.28E 6
24	1:LOAD CASE	-375E 3	-142E 3	0.000	401E 3	0.000	0.000	-519E 3
25	1:LOAD CASE	-203E 3	-142E 3	0.000	248E 3	0.000	0.000	-282E 3
26	1:LOAD CASE	0.000	0.000	0.000	0.000	0.000	0.000	0.000
27	1:LOAD CASE	-117.679	-0.180	0.000	117.679	0.000	0.000	0.623
28	1:LOAD CASE	-304.067	-0.352	0.000	304.068	0.000	0.000	0.874
29	1:LOAD CASE	-513.331	-0.580	0.000	513.331	0.000	0.000	0.794
30	1:LOAD CASE	-6.171	-0.035	0.000	6.171	0.000	0.000	0.152
31	1:LOAD CASE	-7.015	0.036	0.000	7.015	0.000	0.000	0.173
32	1:LOAD CASE	-118.632	0.159	0.000	118.633	0.000	0.000	0.608
33	1:LOAD CASE	-302.519	0.323	0.000	302.519	0.000	0.000	0.868
34	1:LOAD CASE	-511.733	0.537	0.000	511.733	0.000	0.000	0.799

Node Displacement Summary

	Node	L/C	X (in)	Y (in)	Z (in)	Resultant (in)	rX (rad)	rY (rad)	rZ (rad)
Max X	1	1:LOAD CASE	**0.000**	0.000	0.000	0.000	0.000	0.000	0.000
Min X	8	1:LOAD CASE	**-1.35E 9**	1.06E 9	0.000	1.71E 9	0.000	0.000	-9.74E 6
Max Y	8	1:LOAD CASE	-1.35E 9	**1.06E 9**	0.000	1.71E 9	0.000	0.000	-9.74E 6
Min Y	17	1:LOAD CASE	-1.35E 9	**-1.06E 9**	0.000	1.71E 9	0.000	0.000	-9.74E 6
Max Z	1	1:LOAD CASE	0.000	0.000	**0.000**	0.000	0.000	0.000	0.000
Min Z	1	1:LOAD CASE	0.000	0.000	**0.000**	0.000	0.000	0.000	0.000
Max rX	1	1:LOAD CASE	0.000	0.000	0.000	0.000	**0.000**	0.000	0.000
Min rX	1	1:LOAD CASE	0.000	0.000	0.000	0.000	**0.000**	0.000	0.000
Max rY	1	1:LOAD CASE	0.000	0.000	0.000	0.000	0.000	**0.000**	0.000
Min rY	1	1:LOAD CASE	0.000	0.000	0.000	0.000	0.000	**0.000**	0.000
Max rZ	28	1:LOAD CASE	-304.067	-0.352	0.000	304.068	0.000	0.000	**0.874**
Min rZ	8	1:LOAD CASE	-1.35E 9	1.06E 9	0.000	1.71E 9	0.000	0.000	**-9.74E 6**
Max Rst	17	1:LOAD CASE	-1.35E 9	-1.06E 9	0.000	**1.71E 9**	0.000	0.000	-9.74E 6

Beam Displacement Detail Summary

Displacements shown in italic indicate the presence of an offset

	Beam	L/C	d (ft)	X (in)	Y (in)	Z (in)	Resultant (in)
Max X	17	1:LOAD CASE	12.658	**0.000**	-0.085	0.000	0.085
Min X	25	1:LOAD CASE	0.000	**-1.35E 9**	1.06E 9	0.000	1.71E 9
Max Y	25	1:LOAD CASE	0.000	-1.35E 9	**1.06E 9**	0.000	1.71E 9
Min Y	25	1:LOAD CASE	18.083	-1.35E 9	**-1.06E 9**	0.000	1.71E 9
Max Z	17	1:LOAD CASE	0.000	0.000	0.000	**0.000**	0.000
Min Z	17	1:LOAD CASE	0.000	0.000	0.000	**0.000**	0.000
Max Rst	25	1:LOAD CASE	18.083	-1.35E 9	-1.06E 9	0.000	**1.71E 9**

Reaction Summary

			Horizontal	Vertical	Horizontal	Moment		
	Node	L/C	FX (kip)	FY (kip)	FZ (kip)	MX (kip`ft)	MY (kip`ft)	MZ (kip`ft)
Max FX	10	1:LOAD CASE	**156.484**	-501.605	0.000	0.000	0.000	-8.77E 3
Min FX	1	1:LOAD CASE	**155.156**	566.704	0.000	0.000	0.000	-8.52E 3
Max FY	1	1:LOAD CASE	155.156	**566.704**	0.000	0.000	0.000	-8.52E 3
Min FY	10	1:LOAD CASE	156.484	**-501.605**	0.000	0.000	0.000	-8.77E 3
Max FZ	1	1:LOAD CASE	155.156	566.704	**0.000**	0.000	0.000	-8.52E 3
Min FZ	1	1:LOAD CASE	155.156	566.704	**0.000**	0.000	0.000	-8.52E 3
Max MX	1	1:LOAD CASE	155.156	566.704	0.000	**0.000**	0.000	-8.52E 3
Min MX	1	1:LOAD CASE	155.156	566.704	0.000	**0.000**	0.000	-8.52E 3
Max MY	1	1:LOAD CASE	155.156	566.704	0.000	0.000	**0.000**	-8.52E 3
Min MY	1	1:LOAD CASE	155.156	566.704	0.000	0.000	**0.000**	-8.52E 3
Max MZ	1	1:LOAD CASE	155.156	566.704	0.000	0.000	0.000	**-8.52E 3**
Min MZ	10	1:LOAD CASE	156.484	-501.605	0.000	0.000	0.000	**-8.77E 3**

Beam Maximum Moments

Distances to maxima are given from beam end A.

Beam	Node A	Length (ft)	L/C		d (ft)	Max My (kip`ft)	d (ft)	Max Mz (kip`ft)
17	1	18.083	1:LOAD CASE	Max -ve	0.000	0.000	0.000	49.051
				Max +ve	0.000	0.000	9.042	-24.525
18	9	18.083	1:LOAD CASE	Max -ve	0.000	0.000	0.000	4.85E 3
				Max +ve	0.000	0.000	18.083	-4.81E 3
19	2	18.083	1:LOAD CASE	Max -ve	0.000	0.000	18.083	26.453
				Max +ve	0.000	0.000	7.535	-62.417
20	3	18.083	1:LOAD CASE	Max -ve	0.000	0.000	18.083	135.658
				Max +ve	0.000	0.000	4.521	-37.541
21	4	18.083	1:LOAD CASE	Max -ve	0.000	0.000	18.083	10.163
				Max +ve	0.000	0.000	6.028	-117.740
22	5	18.083	1:LOAD CASE	Max -ve	0.000	0.000	18.083	281.373
				Max +ve	0.000	0.000	0.000	-82.379
23	6	18.083	1:LOAD CASE	Max -ve	0.000	0.000	18.083	48.206
				Max +ve	0.000	0.000	4.521	-124.047
24	7	18.083	1:LOAD CASE	Max -ve	0.000	0.000	18.083	42.928
				Max +ve	0.000	0.000	3.014	-171.444
25	8	18.083	1:LOAD CASE	Max -ve	0.000	0.000	18.083	1.1E 3
				Max +ve	0.000	0.000	0.000	-464.903
26	8	14.629	1:LOAD CASE	Max -ve	0.000	0.000	0.000	464.903
				Max +ve	0.000	0.000	14.629	-1.16E 3
27	17	14.628	1:LOAD CASE	Max -ve	0.000	0.000	14.628	1.87E 3
				Max +ve	0.000	0.000	0.000	-1.1E 3
28	1	17.514	1:LOAD CASE	Max -ve	0.000	0.000	0.000	0.000
				Max +ve	0.000	0.000	0.000	0.000
29	10	17.514	1:LOAD CASE	Max -ve	0.000	0.000	0.000	0.000
				Max +ve	0.000	0.000	0.000	0.000
30	2	13.482	1:LOAD CASE	Max -ve	0.000	0.000	0.000	3.272
				Max +ve	0.000	0.000	13.482	-204.025
31	11	13.481	1:LOAD CASE	Max -ve	0.000	0.000	13.481	178.659
				Max +ve	0.000	0.000	0.000	-26.453
32	3	13.482	1:LOAD CASE	Max -ve	0.000	0.000	0.000	21.697
				Max +ve	0.000	0.000	13.482	-290.520
33	12	13.481	1:LOAD CASE	Max -ve	0.000	0.000	13.481	414.769
				Max +ve	0.000	0.000	0.000	-135.658
34	4	13.482	1:LOAD CASE	Max -ve	0.000	0.000	0.000	83.593
				Max +ve	0.000	0.000	13.482	-597.789
35	13	13.481	1:LOAD CASE	Max -ve	0.000	0.000	13.481	457.295
				Max +ve	0.000	0.000	0.000	-10.163
36	14	13.481	1:LOAD CASE	Max -ve	0.000	0.000	13.481	829.429
				Max +ve	0.000	0.000	0.000	-281.373
37	5	13.482	1:LOAD CASE	Max -ve	0.000	0.000	0.000	82.379
				Max +ve	0.000	0.000	13.482	-595.958
38	6	13.482	1:LOAD CASE	Max -ve	0.000	0.000	0.000	107.891
				Max +ve	0.000	0.000	13.482	-962.977

Beam Maximum Moments Cont...

Beam	Node A	Length (ft)	L/C		d (ft)	Max My (kip`ft)	d (ft)	Max Mz (kip`ft)
39	15	13.481	1:LOAD CASE	Max -ve	0.000	0.000	13.481	849.418
				Max +ve	0.000	0.000	0.000	-48.206
40	7	13.482	1:LOAD CASE	Max -ve	0.000	0.000	0.000	165.269
				Max +ve	0.000	0.000	13.482	-1.19E 3
41	16	13.481	1:LOAD CASE	Max -ve	0.000	0.000	13.481	1.02E 3
				Max +ve	0.000	0.000	0.000	-42.928
42	1	6.500	1:LOAD CASE	Max -ve	0.000	0.000		
				Max +ve	0.000	0.000	0.000	-8.57E 3
43	27	20.000	1:LOAD CASE	Max -ve	0.000	0.000		
				Max +ve	0.000	0.000	0.000	-4.07E 3
44	28	20.000	1:LOAD CASE	Max -ve	0.000	0.000	20.000	2.13E 3
				Max +ve	0.000	0.000	0.000	-969.753
45	29	17.500	1:LOAD CASE	Max -ve	0.000	0.000	17.500	4.85E 3
				Max +ve	0.000	0.000		
46	10	6.500	1:LOAD CASE	Max -ve	0.000	0.000		
				Max +ve	0.000	0.000	0.000	-8.73E 3
47	31	22.500	1:LOAD CASE	Max -ve	0.000	0.000		
				Max +ve	0.000	0.000	0.000	-7.71E 3
48	32	20.000	1:LOAD CASE	Max -ve	0.000	0.000		
				Max +ve	0.000	0.000	0.000	-4.19E 3
49	33	20.000	1:LOAD CASE	Max -ve	0.000	0.000	20.000	2.07E 3
				Max +ve	0.000	0.000	0.000	-1.06E 3
50	34	17.500	1:LOAD CASE	Max -ve	0.000	0.000	17.500	4.81E 3
				Max +ve	0.000	0.000		
51	30	22.500	1:LOAD CASE	Max -ve	0.000	0.000		
				Max +ve	0.000	0.000	0.000	-7.56E 3