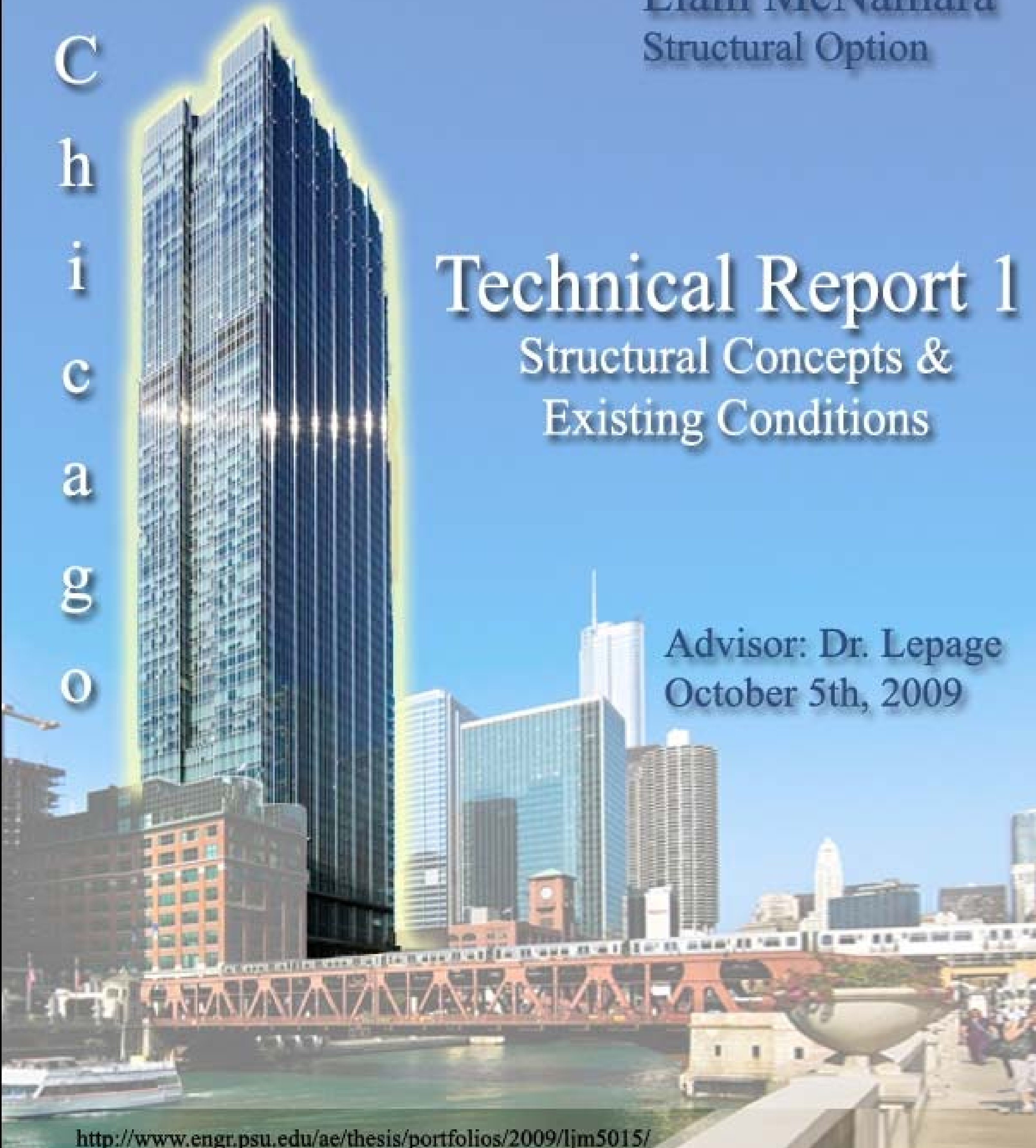
300 North La Salle

Chicago

Liam McNamara
Structural Option

Technical Report 1

Structural Concepts & Existing Conditions



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http://www.engr.psu.edu/ae/thesis/portfolios/2009/ljm5015/

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Executive Summary

The structural concepts and existing conditions report for 300 North La Salle examines and describes the structural system as well as design and loading conditions. This 60-story office high rise building's structure is a concrete bearing wall core with steel outrigger columns. The floor system is comprised of concrete slab over composite decking supported by steel W-shape beams. The lateral loads for the building are carried by the concrete core which also acts as a shear wall core. A "belt" of trusses is located between the 41st and 43rd floors and is used to reduce lateral deflection.

Design wind and seismic lateral forces are determined using ASCE7-05. A comparison of the base shear and overturning moments provides that wind in the North-South direction is the governing lateral load. This base shear is 6748.2 kips, and the overturning moment is 2,846,000 ft-K. These loads are transferred from the shear wall core into the deep foundation consisting of drilled concrete piers and driven steel H-piles. The distribution of these lateral loads will be further investigated in Technical Report 3.

Spot checks were performed on a typical composite deck, beam, and column. These spot checks were based solely on gravity loads and confirmed the strength and size of the members. However, these will have to be analyzed again later examining the strength of these members under lateral and gravity loads.

Introduction

300 North La Salle is a 60-story high rise office building located on the north bank of the Chicago River in Chicago Illinois. It offers 25,000 gsf of rentable, column free floor space per level, with a total square footage of 1.3 million. Construction on the building began in 2006 and was completed in February of 2009 at a cost of $230 million. It is owned and managed by Hines developers and was designed by Pickard Chilton Architects. The primary tenant is Kirkland & Ellis, Chicago's largest law firm, occupying between 24 and 28 floors.

300 North La Salle rises elegantly above the Chicago River with a subtle set back above the 42nd floor. Its "fin-like" steel outriggers and aluminum mullions emphasize verticality. The appearance of structural members on the façade as well as the large open floor plans allude to Mies van der Rohe and the international style he helped make famous in Chicago.

A major focus of the building was to be sustainable achieving a pre-certified LEED Gold rating. 300 North La Salle utilizes high performance glass through its entire façade as well as having a green roof. Another core design aspect for sustainability is its utilization of water from the river to remove heat from its chillers and therefore removes the need for cooling towers and saves energy. The MEP engineering firm Alvine and Associates designed the building to be "approximately 20% under the energy code."

Some other key features are its publicly accessible spaces. Keeping with the trend of revitalizing the river, 300 North La Salle offers direct access to the river's edge from a large waterfront garden and café. On its 1st level it also boasts a restaurant, small bank, and sundry shops.

The structural engineers for the design were Magnussen Klemencic Associates. The superstructure is composed of a bearing concrete core and exterior steel W-shape "outrigger" columns. The typical floor system consists of 3"-12" concrete slabs on 3" composite decking supported by steel W-shape beams & girders. The bearing concrete core wall also acts as a shear wall core to carry lateral forces to the foundation. There is a "belt" of trusses spanning from the 41st to 43rd floors which aide in controlling lateral deflection of the structure and rotation within the shear wall core. The concrete strength of the core varies between 6,000 and 10,000 psi and the wall thicknesses vary between

1’6” and 2’3”. The composite decking is typically 4,000 psi light-weight concrete. The steel members are Fy = 50 Ksi except for select columns on the lower level that are high strength Fy = 65 Ksi steel. The foundation consists of a 5”-12” concrete slab over a combination of drilled concrete piers and driven steel H-piles. The foundation walls are 18” cast in place concrete rising around three sub-grade levels of parking.

The structural concepts and existing conditions report contains an overview of 300 North La Salle’s structure, as well as an overview of design codes and requirements. An analysis of the lateral wind and seismic loads using ASCE7-05 is included as well as spot checks of various structural elements under gravity loads. Images and tables are used throughout the report to illustrate the structure of the building and the forces acting on it. The analyzed data and spot checks are compared to available design data to verify member sizes and design forces.

Structural Systems

Foundations:

The foundation of the building is a combination of poured concrete piers and driven steel H-Piles with a 12" concrete slab sloping away from the core. The foundation slab is 28'-3" below grade and the foundation walls are 18" thick cast-in-place concrete around 3 levels of sub grade parking. The piers are drilled to approximately 72' below grade from top depths of 27'-41' below grade and have a bearing pressure of 40ksf. The piles are driven to refusal in bedrock at approximately 110' below grade and have a design bearing strength of 270 tons.

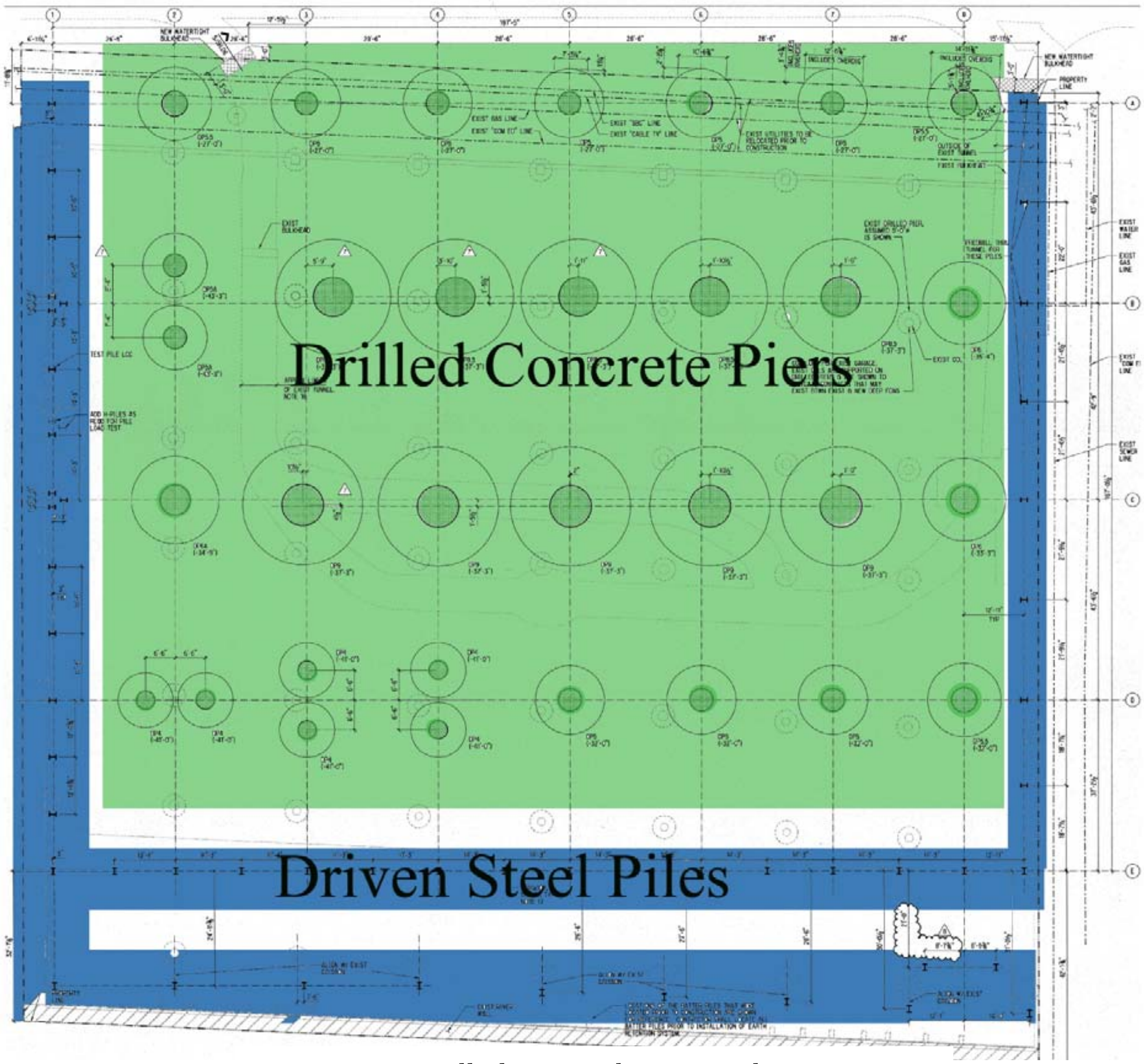


Figure 1 – Drilled Pier and Driven Pile Locations

The building was built on the site of a 1950's parking garage whose foundation included concrete piers and timber piles. In addition to the parking garage there is also an abandoned tunnel below the site at approximately 40' below grade. To deal with these existing situations the structural engineers decided to cut the existing foundation from the parking garage a minimum of 4'0" below the bottom of the foundation slab anywhere that it was conflicting with the new construction. Any timber piles that were in conflict were removed and their holes filled with grout. They also required that the tunnel be filled in with grout of "sufficient strength to meet requirements of the governing authority." For the designed H-piles that would fall above the tunnel, they required that the holes be predrilled through the tunnel before the piles were driven.

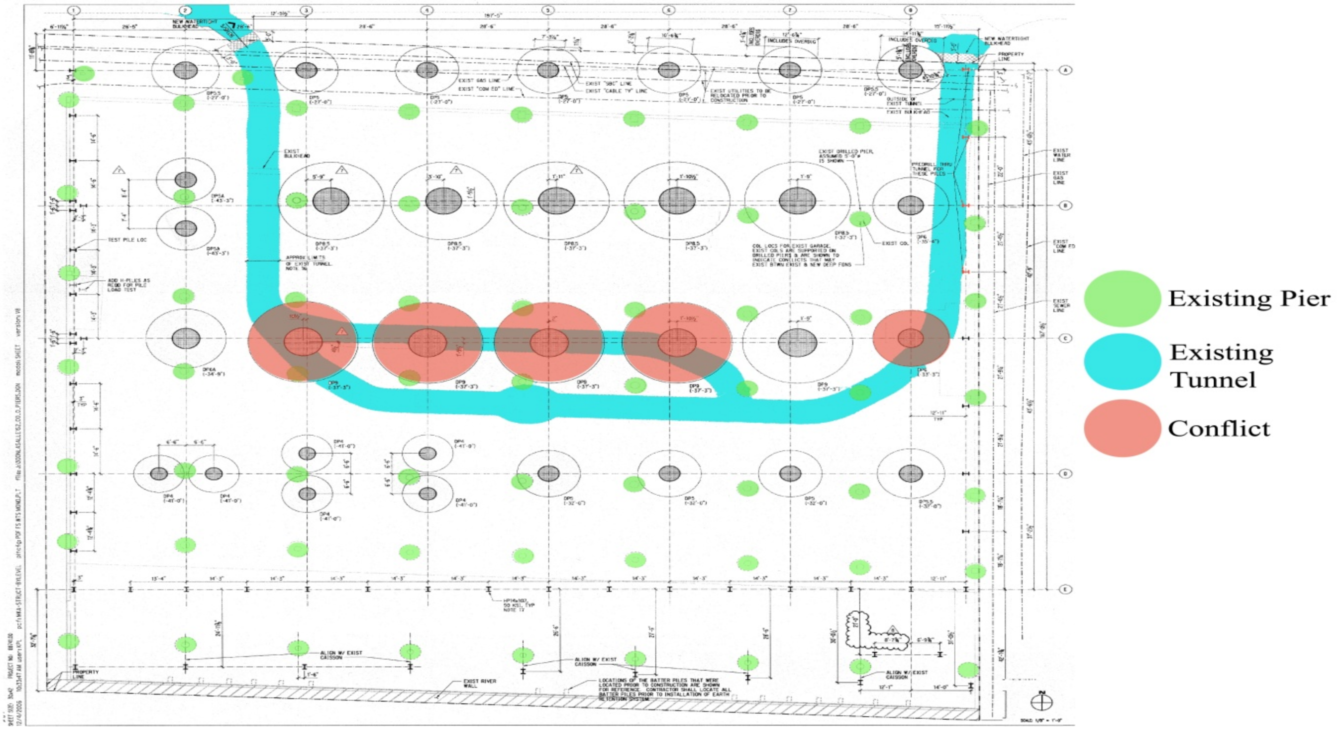


Figure 2 – Existing Sub Grade Conditions & Potential Conflicts

Gravity System:

The main gravity-load is carried to the ground by exterior steel columns and an interior concrete core wall. The floor system on every floor is poured concrete slab over composite decking. While the slab varies from 3" light-weight concrete, on the office floors, to as thick as 8" normal-weight concrete in the mechanical area, the deck is a consistent 3" Type W minimum 20 gage galvanized steel. The composite decking transfers its loads onto 50ksi steel Wide flange beams typically spanning between 42'-9" and 43'-6½". Below the elevator pits and Com Ed rooms on Lower Levels 1-4 the slab changes to a 2-way flat slab between 12" and 14" deep. The thickened two way flat slab is used to more readily carry the large live loads in these areas to the core. The roof system is also a light-weight concrete slab on 3" decking, however the beam size is increased to carry the additional weight from the green roof around the core of the building.

Lateral System:

Wind and seismic forces are resisted by a concrete shear wall core, strengthened by a series of trusses creating a "belt" between the 41st and 43rd floors. The shear wall core is cast-in-place concrete of 6,000; 8,000; and 10,000 psi strength depending on location. The wall reduces in thickness and plan as it rises through the building. The thickness reduces from 2'-3" to 2'-0" and then to 18" on the north and south walls at levels 9 and 43 respectively. The core has four 28'-6" bays running east-west as it rises from Lower Level 4 to Level 42, at Level 43 the core drops its outer two bays and continues through the penthouse with the inner two bays. The shear wall reduction in length corresponds to a 10' reduction in east-west width, and the top of the two story "belt" truss system. The floor and roof diaphragms carry the lateral loads to the shear wall core. The shear walls in the core then transfer the base shear, overturning moment, and rotational forces to the foundation.

The belt truss system is comprised of two braced frames running east-west on the north and south exteriors, and three braced frames spanning north-south to the concrete shear wall on the interior of the building. The truss members are varying sizes of steel Wide flanges. The purpose of this "belt" truss system is to create a couple moment, from the outrigger steel columns in the event of lateral loading. This couple moment is applied on the shear wall core to fight rotation within the core, and therefore reduce the deflection of the building.

"Belt" Trusses and Shear Wall from Level 41- Level 43

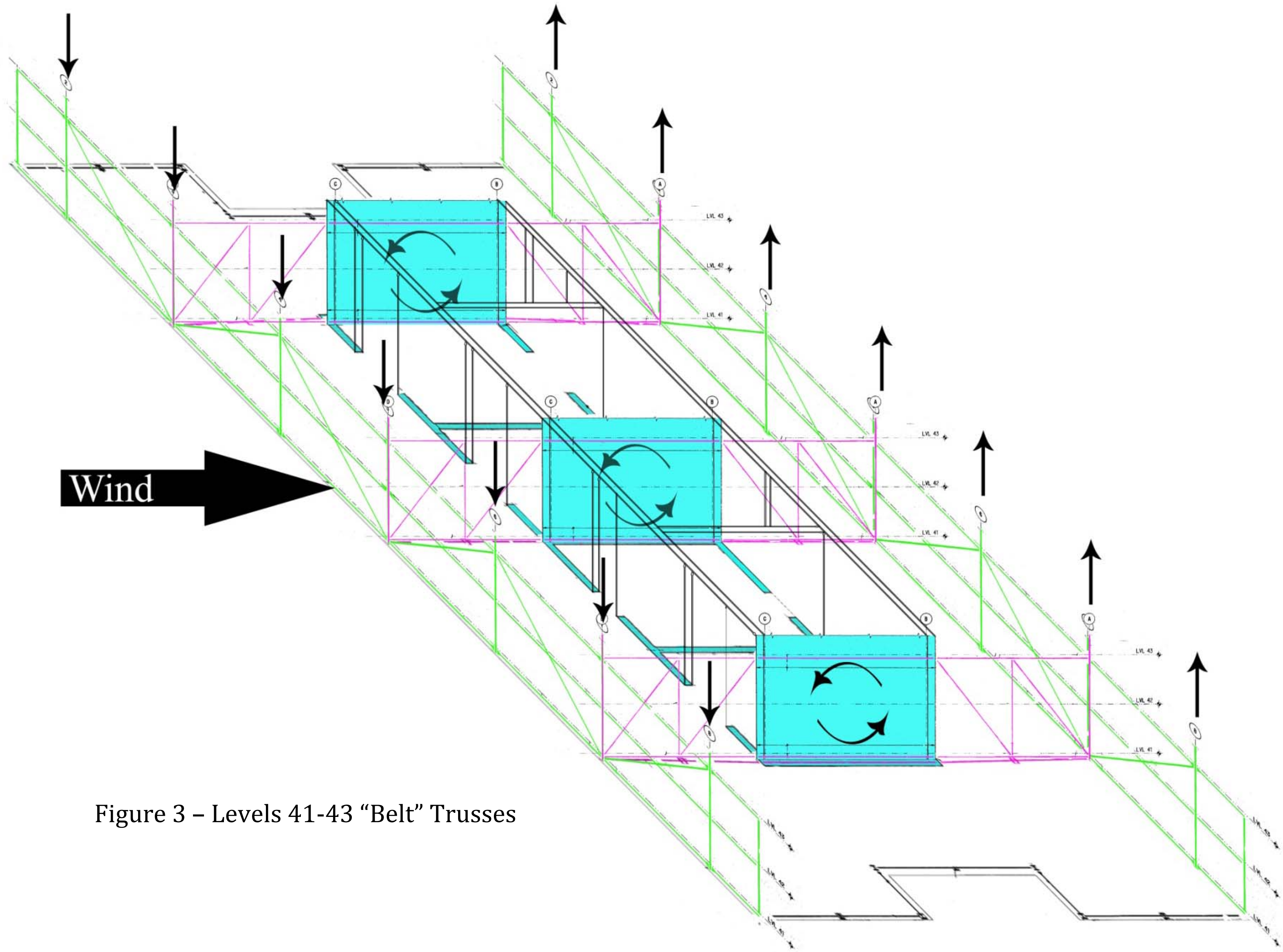


Figure 3 – Levels 41-43 "Belt" Trusses

This is a basic visual representation of how the frames distribute the lateral loading to the primary structure and create a couple moment in the shear wall to resist the wind force and overturning moment.

Structural Materials

Structural Steel:

W-Shapes……………………………………………ASTM A992 or A913, Fy=50 KSI
Angles………………………………………………ASTM A36, Fy=36 KSI
Square of Rectangular
Structural Tube…………………………ASTM A500, Grade B, Fy=36 KSI
Steel Pipe d ≤ 12”………………………………ASTM A53, Type E or S, Grade B, Fy=35 KSI
Material called out on
as (Fy= 65 KSI)…………………………ASTM 913, Fy=65 KSI
All other steel……………………………………ASTM A572, A588, A441, Fy=50 KSI

Metal Decking:

3” Composite Deck……………………………Verco W3 - 20 gage minimum

Welding Electrodes:

E70 XX………………………………………………70 KSI minimal tensile strength

Cast-in-Place Concrete:

Misc. Concrete, Curbs,
Sidewalks……………………………………f’c = 4,000 psi – Normal Weight
Slab on Grade…………………………………f’c = 4,000 psi – Normal Weight
Foundation Walls……………………………f’c = 5,000 psi – Normal Weight
Concrete on Steel Deck……………………f’c = 4,000 psi – Normal Weight
f’c = 4,000 psi – Light Weight
Columns, Reinforced Beams,
and Slabs……………………………………f’c = 5,000 psi – Normal Weight
Shear Walls……………………………………f’c = 6,000 psi – Normal Weight
f’c = 8,000 psi – Normal Weight
f’c = 10,000 psi – Normal Weight
Grade Beams, Elevator Pits,
Caissons, Caps…………………………f’c = 8,000 psi – Normal Weight

Reinforcement:

Reinforcing Bars……………………………………….ASTM A615, Grade 60
Welded Wire Fabric…………………………………..ASTM A185

Masonry:

Hollow Concrete Units………………………………..ASTM C90, $f'c_{min}$ = 1,900 psi

Codes and References

Design Codes:

National Model Code:

Chicago Building Code 2005

Design Codes:

American Concrete Institute (ACI), ACI 530-92, Requirements for Masonry Structures

ACI 318-83, Requirements for Structural Concrete

American Institute of Steel Construction (AISC), LRFD-86," Load and Resistance Factor Design Specification for Steel Buildings"

AISC-2000, "Specification for Structural Joints using ASTM A325 or A490 Bolts"

American Welding Society (AWS), AWS D1.1-2000, "Structural Welding Code- Steel"

AWS D1.3-98, "Structural Welding Code- Sheet Steel"

AWS D1.4-98, "Structural Welding Code-Reinforcing Steel"

AWS A2.4-98, "Symbols for Welding and Nondestructive testing"

American Iron and Steel Institute (AISI), "Specifications for the Design of Cold Formed Steel Structural Members," 1996 with supplement No.1 July 30, 1999

Structural Standards:

American National Standards Institute (ANSI), ANSI A58.1-1982

Thesis Codes:

National Model Code:

2006 International Building Code

Design Codes:

Steel Construction Manual 13th edition, AISC

ACI 318-05, Building Code Requirements for Structural Concrete

Structural Standards:

American Society of Civil Engineers (ASCE), ASCE 7-05, Minimum Design Loads for Buildings and other Structures

Lateral Analysis:

Wind:

The wind loads for the Main Wind-Force Resisting System (MWFRS) within 300 North La Salle are determined using design criteria and data from ASCE 7-05. Primary loads are calculated in the North-South, and East-West directions using Method 2-Analytical Procedure, and referencing "Structural Load Determination Under 2006 IBC and ASCE/SEI 7-05," Flow-charts by David A. Fanella.

The initial step in the procedure is determining whether the building is a flexible or rigid structure based upon its natural frequency. To simplify the calculation of the building's natural frequency, ASCE 7-05 Chapter 12 was referenced to determine the fundamental period. Frequency by definition is the inverse of a period, the inverse of the building's fundamental period provides a natural frequency <1 defining the building as flexible.

The building is simplified as a rectangular box for preliminary evaluation. The penthouse on the roof is protected by the parapet for all but the top 5' feet therefore it was ignored in the calculations. The building's set-backs of 5', on the east and west sides at the 43rd floor, are ignored leading to more conservative values. A reduction in total length will reduce the force felt from the wind because there is less surface area for it to act on. The building receives the largest force from wind in the North-South directions, as these are the longer facades of the building normal to the wind.

The final step in analyzing the wind forces is calculating the base shear and overturning moments applied at the foundation to compare with seismic calculations. Comparison of the base shear and overturning moments between wind and seismic will determine the governing lateral force for design. Reference Appendix B for a complete set of values, tables, and equations used to calculate the design wind pressures and forces.

Note: The irregularities in values on the wind force diagrams come from variations in floor height. A larger floor height results in a larger story force for that level and the one above it due to an increase in tributary area.

East / West Wind Distribution

B=133' 3" L = 199' 6"

East/ West		
Height (ft)	Windward pz (psf)	Leeward ph (psf)
796	47.26	-31.51
786	27.31	-19.19
750	27.02	-19.19
700	26.60	-19.19
650	26.16	-19.19
600	25.70	-19.19
550	25.21	-19.19
500	24.62	-19.19
450	24.13	-19.19
400	23.53	-19.19
350	22.80	-19.19
300	22.07	-19.19
250	21.22	-19.19
200	20.24	-19.19
180	19.88	-19.19
160	19.39	-19.19
140	18.90	-19.19
120	18.30	-19.19
100	17.69	-19.19
90	17.32	-19.19
80	16.96	-19.19
70	16.47	-19.19
60	15.99	-19.19
50	15.50	-19.19
40	14.89	-19.19
30	14.16	-19.19
25	13.68	-19.19
20	13.19	-19.19
15	12.58	-19.19

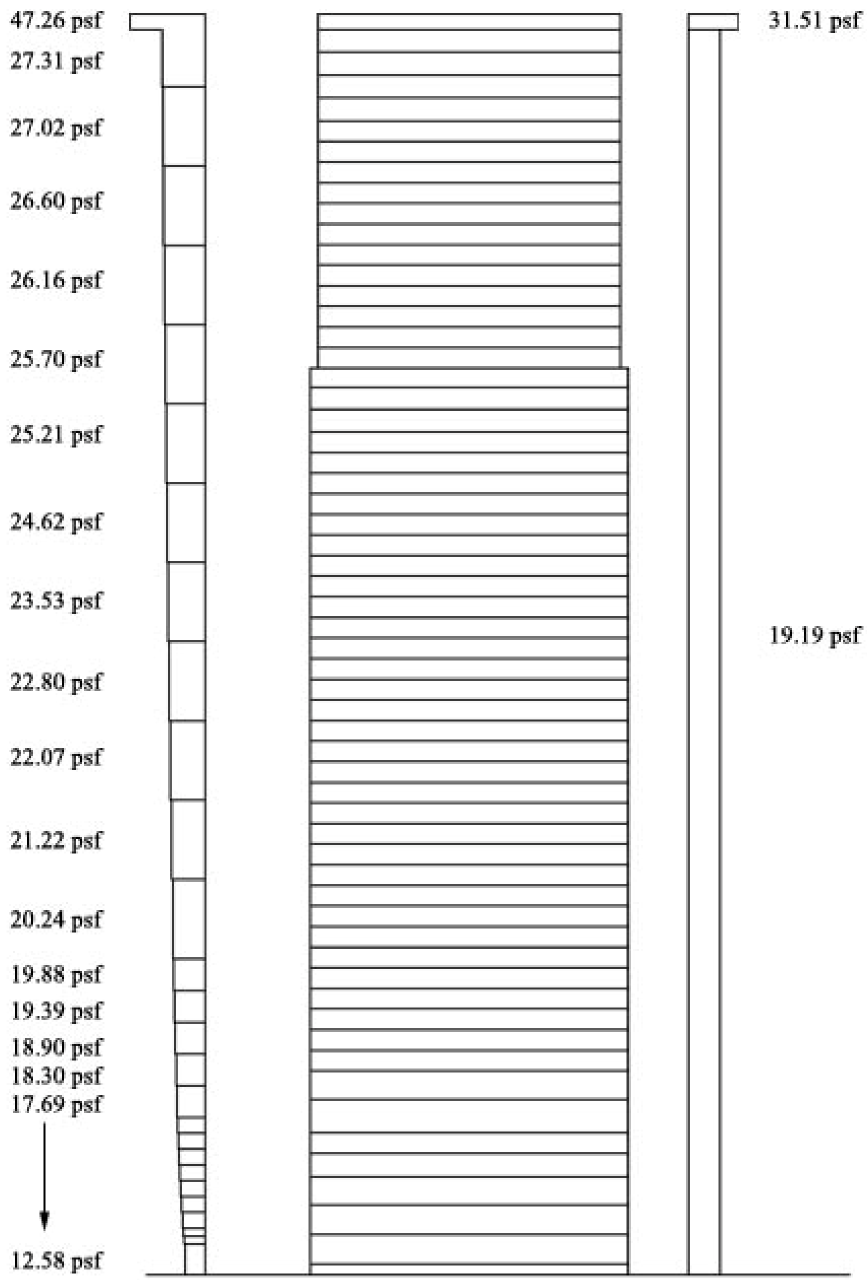


Figure 4 - E/W Wind distribution
*Note: The increased load at the top of the building is due to a 10' curtain wall parapet.

North / South Wind Distribution

B = 199' 6" L = 133' 3"

North/ South		
Height (ft)	Windward pz (psf)	Leeward ph (psf)
796	47.26	-31.51
786	27.76	-19.47
750	27.46	-19.47
700	27.04	-19.47
650	26.59	-19.47
600	26.11	-19.47
550	25.61	-19.47
500	25.01	-19.47
450	24.51	-19.47
400	23.89	-19.47
350	23.15	-19.47
300	22.40	-19.47
250	21.54	-19.47
200	20.54	-19.47
180	20.17	-19.47
160	19.67	-19.47
140	19.18	-19.47
120	18.56	-19.47
100	17.94	-19.47
90	17.56	-19.47
80	17.19	-19.47
70	16.70	-19.47
60	16.20	-19.47
50	15.70	-19.47
40	15.08	-19.47
30	14.34	-19.47
25	13.84	-19.47
20	13.35	-19.47
15	12.72	-19.47

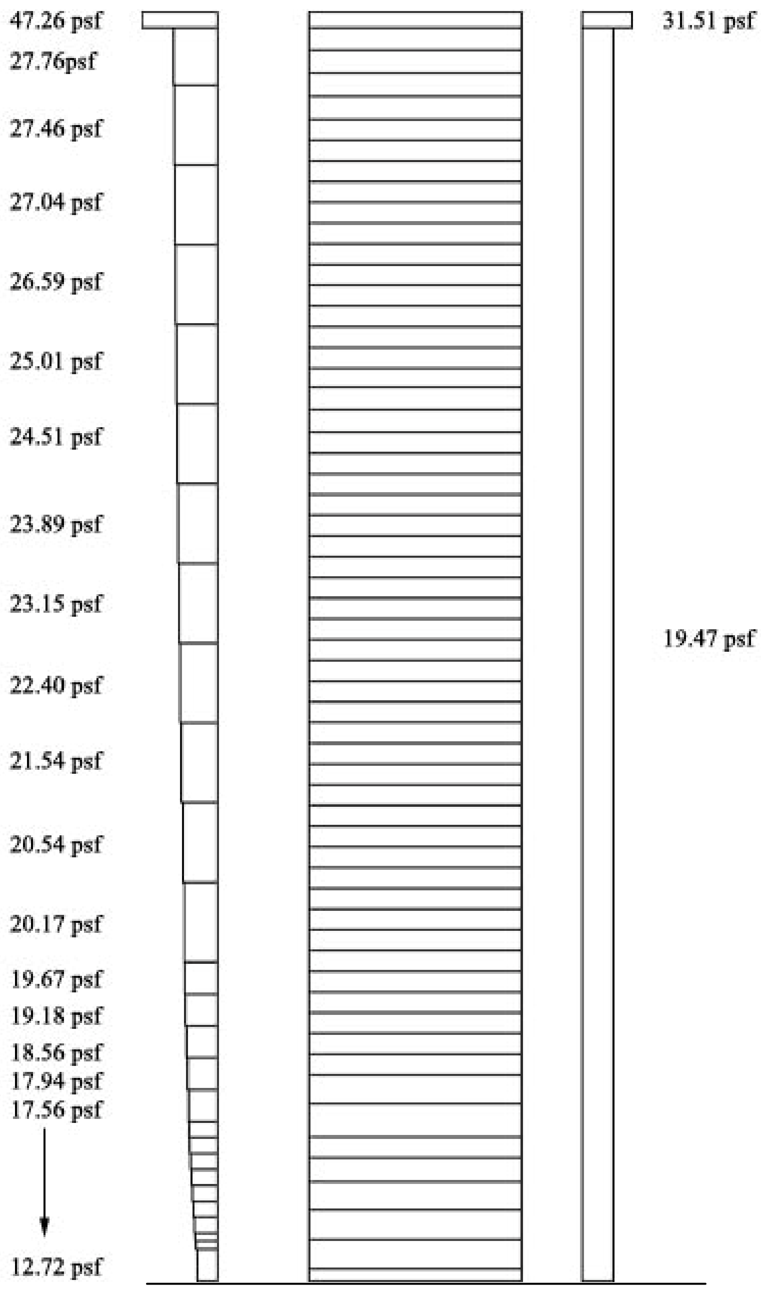


Figure 5- N/S Wind distribution
*Note: The increased load at the top of the building is due to a 10' curtain wall parapet.

East / West Wind Forces			
Story Level	Story Height (ft)	Story Force (Kips)	Moment (k-ft)
Roof	786.00	148.1	116373
58	772.00	87.7	67735
57	757.50	89.3	67629
56	743.00	88.5	65746
55	728.50	83.9	61119
54	715.50	79.3	56754
53	702.50	79.1	55558
52	689.50	78.6	54167
51	676.50	78.6	53146
50	663.50	78.6	52125
49	650.50	78.2	50862
48	637.50	77.8	49570
47	624.50	77.8	48559
46	611.50	77.8	47548
45	598.50	77.2	46223
44	585.50	76.9	45027
43	572.50	73.9	42334
42	560.50	76.9	43105
41	546.50	82.5	45077
40	532.33	79.3	42209
39	519.33	75.9	39409
38	506.33	75.9	38417
37	493.33	75.0	37021
36	480.33	75.0	36045
35	467.33	75.0	35070
34	454.33	74.9	34014
33	441.33	74.0	32654
32	428.33	74.0	31692
31	415.33	74.0	30730
30	402.33	73.6	29605
29	389.33	72.7	28314
28	376.33	72.7	27369
27	363.33	72.7	26423
26	350.33	72.1	25268
25	337.33	71.5	24106
24	324.33	71.5	23177
23	311.33	71.5	22248
22	298.33	70.5	21043
21	285.33	70.0	19970
20	272.33	70.0	19060
19	259.33	70.0	18150
18	246.33	68.7	16915
17	233.33	68.3	15937
16	220.33	68.3	15049
15	207.33	68.3	14161
14	194.33	67.7	13158
13	181.33	67.3	12210
12	168.33	66.8	11249
11	155.33	66.1	10268
10	142.33	65.6	9344
9	129.33	77.4	10013
7	111.33	95.8	10671
6	90.33	82.4	7448
5	77.33	66.8	5163
4	62.33	76.1	4746
2	44.67	82.3	3677
1	26.00	80.9	2104
LL-1	7.5	70.9	532
Total Base Shear (kips)= 4442.2			
Total Overturning Moment (k-ft)= 1873297			

East / West Story Force Diagram

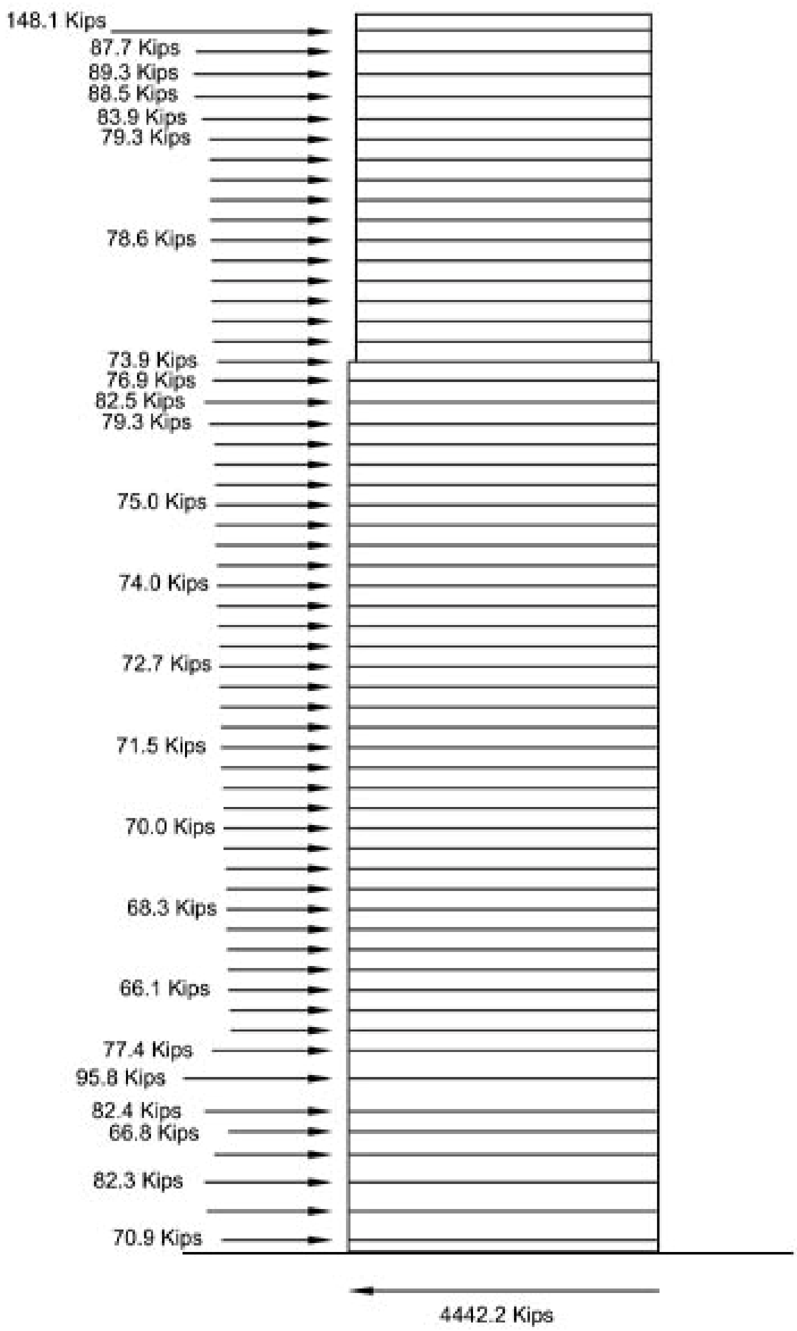


Figure 6 - E/W Wind Story Forces

*Note: Values are left unlabeled for clarity, reference E/W wind force table for values.

North / South Wind Forces			
Story Level	Story Height (ft)	Shear Force (Kips)	Moment (k-ft)
Roof	786.00	222.7	175023
58	772.00	133.4	102992
57	757.50	135.7	102830
56	743.00	134.5	99962
55	728.50	127.6	92927
54	715.50	120.6	86291
53	702.50	120.2	84472
52	689.50	119.4	82354
51	676.50	119.4	80801
50	663.50	119.4	79249
49	650.50	118.9	77327
48	637.50	118.2	75360
47	624.50	118.2	73824
46	611.50	118.2	72287
45	598.50	117.4	70270
44	585.50	116.9	68450
43	572.50	112.4	64356
42	560.50	116.9	65528
41	546.50	125.4	68522
40	532.33	120.5	64161
39	519.33	115.4	59906
38	506.33	115.3	58398
37	493.33	114.1	56272
36	480.33	114.1	54789
35	467.33	114.1	53306
34	454.33	113.8	51701
33	441.33	112.5	49630
32	428.33	112.5	48168
31	415.33	112.5	46706
30	402.33	111.8	44995
29	389.33	110.5	43031
28	376.33	110.5	41594
27	363.33	110.5	40157
26	350.33	109.6	38399
25	337.33	108.6	36632
24	324.33	108.6	35220
23	311.33	108.6	33809
22	298.33	107.2	31975
21	285.33	106.3	30342
20	272.33	106.3	28960
19	259.33	106.3	27577
18	246.33	104.3	25699
17	233.33	103.8	24212
16	220.33	103.8	22863
15	207.33	103.8	21514
14	194.33	102.9	19989
13	181.33	102.3	18548
12	168.33	101.5	17088
11	155.33	100.4	15596
10	142.33	99.7	14192
9	129.33	117.6	15207
7	111.33	145.5	16205
6	90.33	125.2	11309
5	77.33	101.4	7839
4	62.33	115.6	7204
2	44.67	125.0	5581
1	26.00	122.8	3193
LL-1	7.5	107.6	807
Total Base Shear (kips)=	6748.2		
Total Overturning Moment (k-ft)=	2845601		

North / South Story Force Diagram

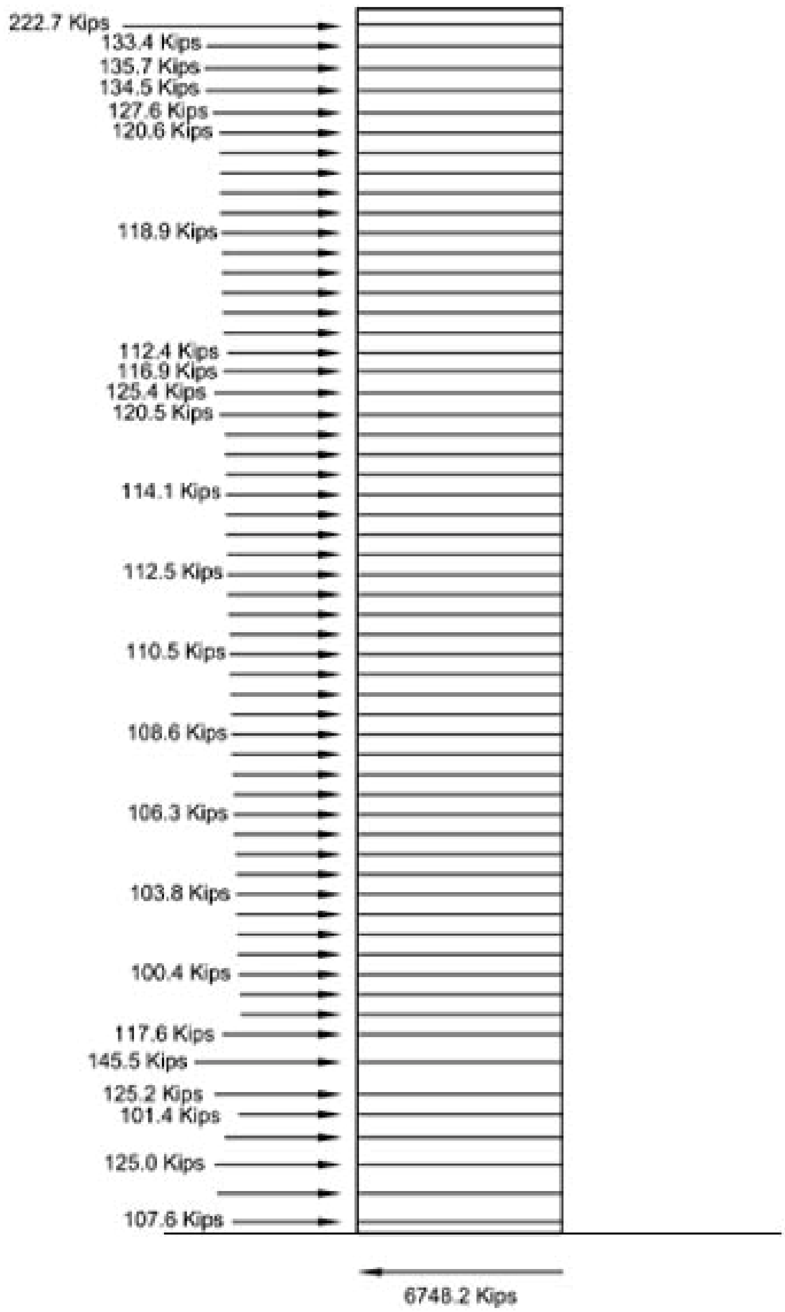


Figure 7 - N/S Wind Story Forces

*Note: Values are left unlabeled for clarity, reference E/W wind force table for values.

Note on Wind loads:

The structural design engineers had a wind tunnel study performed on a scale model of 300 North La Salle. This study could not be obtained for comparison for this stage of the investigation. The study's results will try to be procured for Technical Report 3 in order to have a better comparison of the lateral systems. There is the potential that the values calculated above are more conservative as a damping ratio of 1% was used which assumes the building is steel construction(ASCE7-05: Ref C6-55). This was done to be conservative as the building is actually a combination of steel and concrete construction, and concrete will aide in damping and produce lower magnitude results.

Seismic:

The seismic loads are determined using design criteria and data from ASCE 7-05 Chapters 11 & 12 and referencing Chapters 20&21. Flow-chart 6.8 - "Structural Load Determination Under 2006 IBC and ASCE/SEI 7-05," by David A. Fanella is also referenced. The Equivalent Lateral Force Procedure is the method used to determine the minimal seismic design loads for 300 North La Salle. Due to lack of a Geotechnical report a site soil class D is assumed (ASCE 7-05: 11.4.2) and from this the Seismic Design Category B is determined.

The structural configuration of the building could be interpreted as either a bearing wall system with ordinary reinforced concrete shear walls, or a building frame system with ordinary reinforced concrete shear walls. For calculation purposes it was assumed as a bearing wall system with ordinary reinforced concrete shear walls leading to the more conservative response modification coefficient (R) of 4 (ASCE 7-05 Table 12.2-1).

The building weight (W) is calculated in order to determine the base shear force (V) from the equation V=Cs*W. The building has different floor slab thicknesses and depths depending on use, and has several different size floor plans. Calculation of the floor slab and supporting beam weights is simplified by breaking the levels into several groupings as shown in Appendix C. Calculation of the column weights and curtain wall weight is also shown in Appendix C. There are also superimposed dead loads based on the structural documents' loading diagrams.

The calculation of the shear forces at each floor provides a total base shear and overturning moment to be compared with those from wind. The table below displays the story forces and overturning moments caused by seismic forces. A complete set of values, tables and calculations can be found for reference in Appendix C.

Seismic Calculations

Level	Height (ft)	Wx (k)	wihi^k	Fx (k)	Vx (k)	Moment (k-ft)
Parapet	796	0	0	0	0	0
Roof	786	2877	6,506,974,248	46	46	36457
58	772	2888	6,439,008,796	46	92	35434
57	758	3089	7,226,124,237	52	144	39018
56	743	3090	7,094,991,612	51	194	37577
55	729	3090	6,956,529,461	50	244	36124
54	716	2966	6,293,843,914	45	289	32100
53	703	2966	6,179,490,356	44	333	30944
52	690	2970	6,080,352,743	43	376	29884
51	677	2970	5,965,712,300	43	419	28768
50	664	2975	5,873,942,067	42	461	27781
49	651	2975	5,758,853,526	41	502	26703
48	638	2983	5,673,690,738	40	542	25783
47	625	2983	5,557,991,947	40	582	24742
46	612	2989	5,461,757,249	39	621	23807
45	599	2989	5,345,644,666	38	659	22806
44	586	2994	5,247,381,515	37	696	21900
43	573	4785	13,106,075,479	93	790	53484
42	561	2811	4,428,290,748	32	821	17693
41	547	5247	15,044,029,996	107	928	58605
40	532	4097	8,937,166,696	64	992	33913
39	519	4118	8,806,512,511	63	1055	32601
38	506	3933	7,832,657,098	56	1111	28270
37	493	3933	7,631,555,303	54	1165	26837
36	480	3936	7,443,131,697	53	1218	25485
35	467	3936	7,241,686,772	52	1270	24124
34	454	3942	7,060,809,918	50	1320	22867
33	441	3942	6,858,776,472	49	1369	21577
32	428	3941	6,653,055,677	47	1416	20313
31	415	3941	6,451,134,143	46	1462	19099
30	402	3947	6,269,017,110	45	1507	17979
29	389	3947	6,066,455,663	43	1550	16836
28	376	3953	5,879,740,582	42	1592	15773
27	363	3953	5,676,631,740	40	1633	14702
26	350	3957	5,485,771,012	39	1672	13699
25	337	3930	5,211,116,641	37	1709	12531
24	324	3945	5,047,147,434	36	1745	11669
23	311	3945	4,844,846,560	35	1779	10752
22	298	3944	4,641,199,444	33	1813	9870
21	285	3944	4,438,957,233	32	1844	9028
20	272	3949	4,247,669,834	30	1874	8246
19	259	3949	4,044,904,689	29	1903	7477
18	246	4011	3,963,105,952	28	1932	6959
17	233	4011	3,753,956,923	27	1958	6244
16	220	4016	3,553,820,942	25	1984	5582
15	207	4054	3,407,797,815	24	2008	5036
14	194	4056	3,197,485,928	23	2031	4429
13	181	4056	2,983,588,927	21	2052	3857
12	168	4066	2,782,631,259	20	2072	3339
11	155	4028	2,519,905,837	18	2090	2790
10	142	4031	2,312,382,249	16	2106	2346
9	129	4121	2,196,129,116	16	2122	2025
7	111	4980	2,761,599,649	20	2142	2192
6	90	5145	2,390,907,244	17	2159	1540
5	77	3118	751,589,052	5	2164	414
4	62	7049	3,097,517,890	22	2186	1376
2	45	4571	933,174,983	7	2193	297
1	26	6083	962,186,503	7	2200	178
II-1	8	6861	352,998,782	3	2202	19
Total Base Shear (kips) =					2202	
Total Overturning Moment (ft-K) =						1061879

Note on seismic values:

The story force Fx is noticeably higher on level 41 than any of the other levels. This is due to the large self weight of level 41; it is a mechanical floor with an increased slab size. Level 42 also has a noticeably low Fx this is because the majority of the floor is open where the two story "Belt" trusses pass through it as seen in Appendix A.

Comparison of Design Wind and Seismic Loads:

Total Base Shear:

Wind:

East-West = 4442.2 Kips
North-South= 6748.2 Kips ←----------- Controls

Seismic:

All directions= 2202 Kips

Total Overturning Moment

Wind:

East-West= 1,873,000 ft-K
North-South= 2,846,000 ft-K ←-----------Controls

Seismic:

All directions= 1,062,000 ft-K

Note on Lateral Forces:

It can be seen from the above comparison that wind forces in the North-South direction govern the lateral design. There are multiple reasons that wind forces govern. The first is the location of 300 North La Salle in Chicago, Illinois. Chicago is in a region of low seismic activity and moderate wind speed.

The height of the building is directly related to the wind and seismic shear values. As a building increases in height from the ground (z) it experiences increased wind pressures by the multiplier of $k=2.01(z/z_g)^{2/\alpha}$ (ASCE7-05 Table 6-3) However, as a building's height increases, its fundamental period increases as well. The fundamental period is used as a reduction factor for the Cs value, directly reducing the seismic shear.

Also as can be seen from the prior tabulated values for shear at each level, wind forces are generally the largest at the top of the building reducing as the height decreases, while seismic forces decrease as the building's height increases. Having larger forces farther from the foundation level results in a much greater overturning moment by the definition moment = force (V) * distance (h).

Further evidence of the wind's controlling lateral force is that even its minimum base shear, in the East-West direction, of 4,442.2 Kips is two times the seismic base shear of 2,202 kips.

Spot Checks of Gravity Load:

Spot checks were performed on a typical composite deck under an office live load on level 27, a steel W-shape beam supporting this deck, and an exterior column on level 57 that supports level 58 and the roof. The beam was checked for strength, live load deflection, and deflection during construction loads. The spot checks verified that all 3 components could carry the design load. The beam's initial camber cancelled out nearly all the deflection, without the camber it barely would have met the deflection criteria under construction loads. The calculations and confirmation of the spot checks can be found in Appendix D.

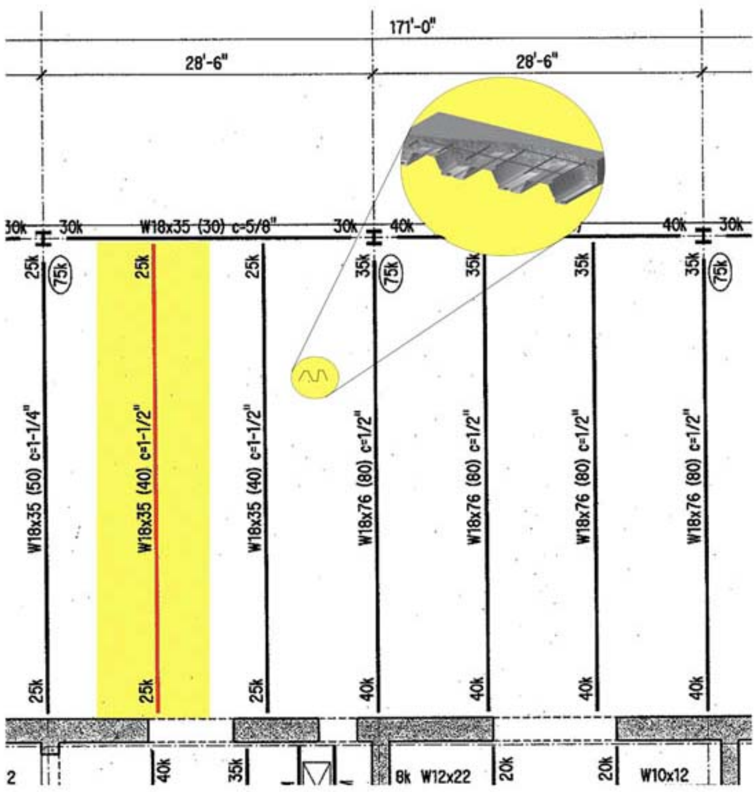


Figure 8 - Level 27 Beam and Deck Spot Check Locations

The yellow highlight over the beam represents the tributary area applying load on the beam. The pullout is a typical "Verco W3 Formlok" from their website (http://www.vercodeck.com/literature/Floor%20Decking%20Catalog-VF3.pdf)

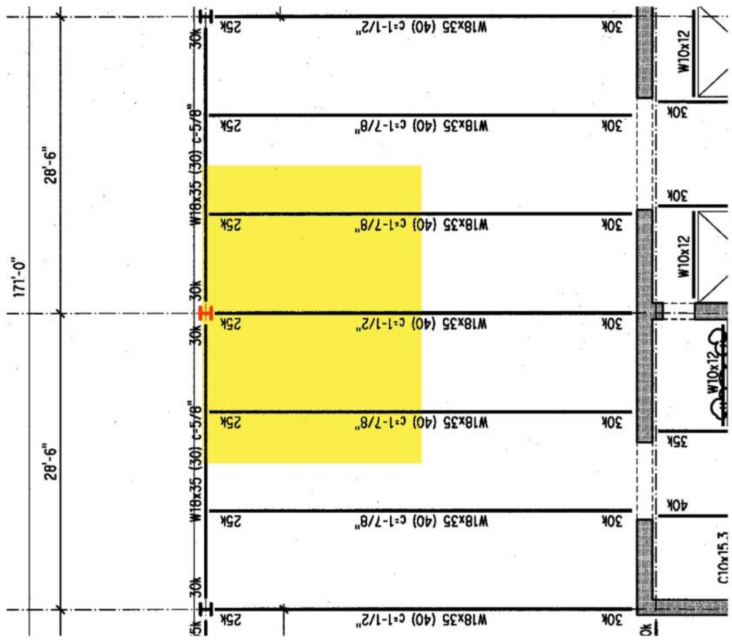


Figure 9 - Level 57 Exterior Column Spot Check Location

The column receives load from the tributary area highlighted on level 58, and the roof. It also receives load from the two stories of curtain wall above it. For simplicity of calculation all loads are assumed to be applied without eccentricity.

Live and Dead Load: Design vs. ASCE 7-05

Floor Live Loads			
Load Description	Load Location	Design Load (psf)	ASCE 7-05 Load
Parking	Lower Levels 2-4	50	40
Storage	LL 3,2,1 Level 4,5 Roof	125*	--
Plaza-General	LL 1, Level 1	100*	--
Lobby	Level 1,9-40, 43-58	100*	100
Office	Levels 9-40, 43-57	50 20 - Partitions	50
Tenant Filing	Levels 9-40	200*	Designed per anticipatory occupancy
Office-Increased Live Load	Levels 43-57	100 20 - Partitions	50
Com Ed	LL 2, Levels 2-58	150*	--
Conference	Levels 6 & 7	100*	--
Data Center	Level 4	200*	--
Central Plant	Lower Level 4	50	--
Mechanical	LL 1-4, Levels 1-58, Roof	125	125
Amenity		100*	--
Green Roof	Roof	40	100
UPS/ Battery	Level 4	350*	--
Terrace	Level 6	100*	100
Elevator Machine	LL 1, Levels 26,42, Roof, Penthouse	150*	300 lb (concentrated load)
Truck Dock	LL 1	250	--
Retail	LL 1, Level 1	100	100
Retail and Built up	Level 1	100	100
Roof	Level 4, Roof, Penthouse, Penthouse Roof (59-61)	40	20
Stairs	All Levels	100	100
Note - * Denotes a non-reducible live load as specified on load diagrams			

It can be noted that the design live loads are provided in a more specific break down than ASCE7-05. Also the elevator machine rooms are applied with 150psf instead of the 300 lb concentrated load called for by ASCE7-05. In general the design loads are more conservative than those required in ASCE7-05. An example of this is the 100psf load MKA calls out as an office with increased live load; ASCE7-05 only has one 50psf standard loading for all office areas. However, this could be a direct comparison to ASCE7-05's corridor demand load of 100psf in order to be conservative since floor plans are not given in detail and corridors could be anywhere within the space.

Superimposed Dead Loads		
Load Description	Load Location	Design Load (psf)
Parking	Lower Levels 2-4	5 - Mech/ Elec
Storage	LL 3,2,1 Level 4,5 Roof	5 - Mech/ Elec
Plaza-General	LL 1, Level 1	5 - Mech/ Elec 75 - Topping
Lobby	Level 1,9-40, 43-58	15 - Mech/Elec/Ceiling
Office	Levels 9-40, 43-57	15 - Mech/Elec/Ceiling
Tenant Filing	Levels 9-40	15 - Mech/Elec/Ceiling
Office-Increased Live Load	Levels 43-57	15 - Mech/Elec/Ceiling
Com Ed	LL 2, Levels 2-58	5 - Mech/ Elec
Conference	Levels 6 & 7	15 - Mech/Elec/Ceiling 40 - Floor Finish
Data Center	Level 4	15 - Mech/Elec/Ceiling
Central Plant	Lower Level 4	Weight of Equipment
Mechanical	LL 1-4, Levels 1-58, Roof	30 - Mech/Elec
Amenity		20 - Mech/Elec
Green Roof	Roof	40 - Green Roof/ Roofing 10 - Mech/ Elec
UPS/ Battery	Level 4	15 - Mech/Elec
Terrace	Level 6	60 - 5" Topping slab 40 - Pavers
Elevator Machine	LL 1, Levels 26,42, Roof, Penthouse	30 - Mech/Elec
Truck Dock	LL 1	15 - Mech/Elec
Retail	LL 1, Level 1	20 - Mech/Elec
Retail and Built up	Level 1	60 - Built up slab 20 - Mech/Elec
Roof	Level 4, Roof, Penthouse, Penthouse Roof (59-61)	10 - Mech/Elec 15 - Roofing
Curtain Wall	All Levels	15 –vertical surface

Conclusion

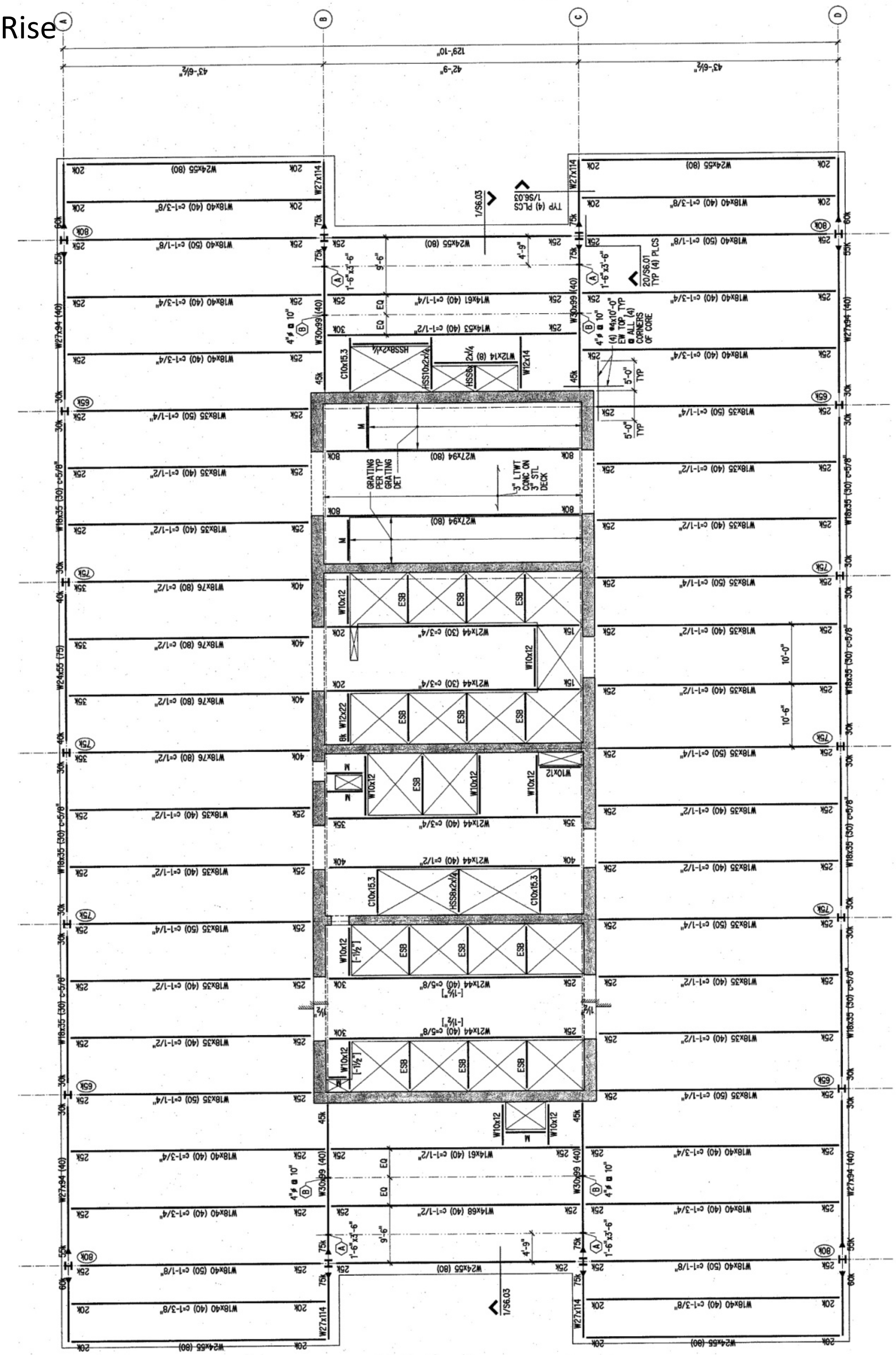
Analysis of the existing structural system and the various gravity spot checks confirm that the structure can adequately carry the applied loads. Examination of lateral wind forces, using ASCE7-05 – Method 2: Analytical Procedure, and lateral seismic forces, using ASCE7-05- Equivalent Lateral Force Procedure determined the controlling lateral force was in the North-South wind direction. The North-South lateral wind force induces a base shear of 6748.2 Kips and an overturning moment of 2,846,000 ft-K into the foundation of the structure. MKA used a wind tunnel for the design wind values, these results have not yet been obtained, but are expected to be more accurate as it is a direct study on a scale model of the building. MKA did not provide any seismic design values and it appears as though they did not inspect them, this could be because 300 North La Salle is in a low seismic area. It could also be due to the height of the building, topping off at 801', the building will receive much larger wind forces as is illustrated in the analysis using ASCE7-05.

Spot checks performed on the composite deck, beam, and column verified that the members sizes were adequate to carry the gravity loads exerted on them. Comparison of construction and serviceability deflections were well under the design deflection criteria. There were no lateral forces analyzed in this comparison. The members will need to be reevaluated under a combination of lateral and gravity loads during Tech Report 3.

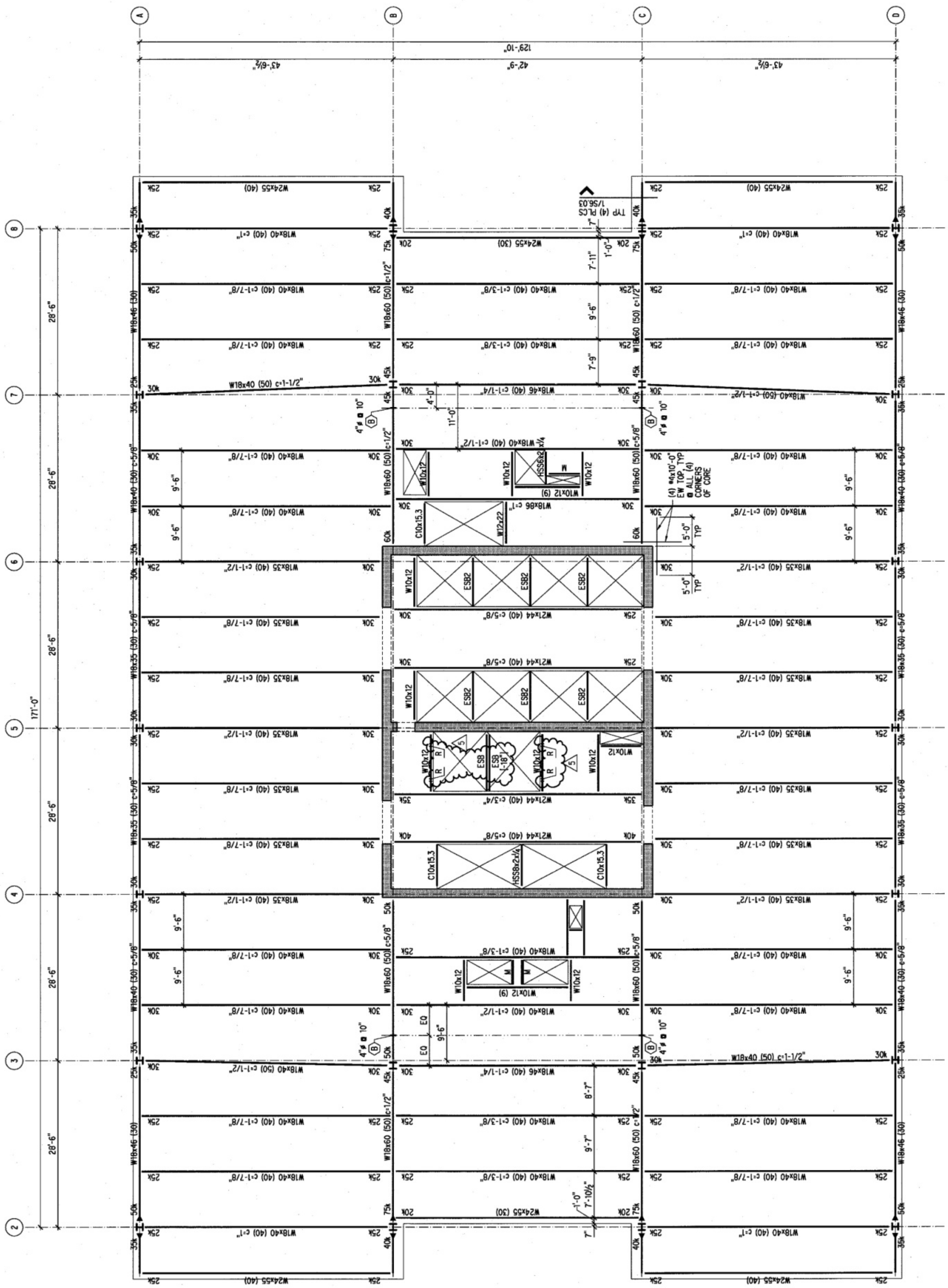
A general comparison of the design live loads called for by MKA shows that they are equal to or more conservative than the load requirements of ASCE7-05. This could be due to load requirements from the Chicago Building Code, or a MKA standard based on a more specified space usage.

Appendix A – Typical Floor Plans

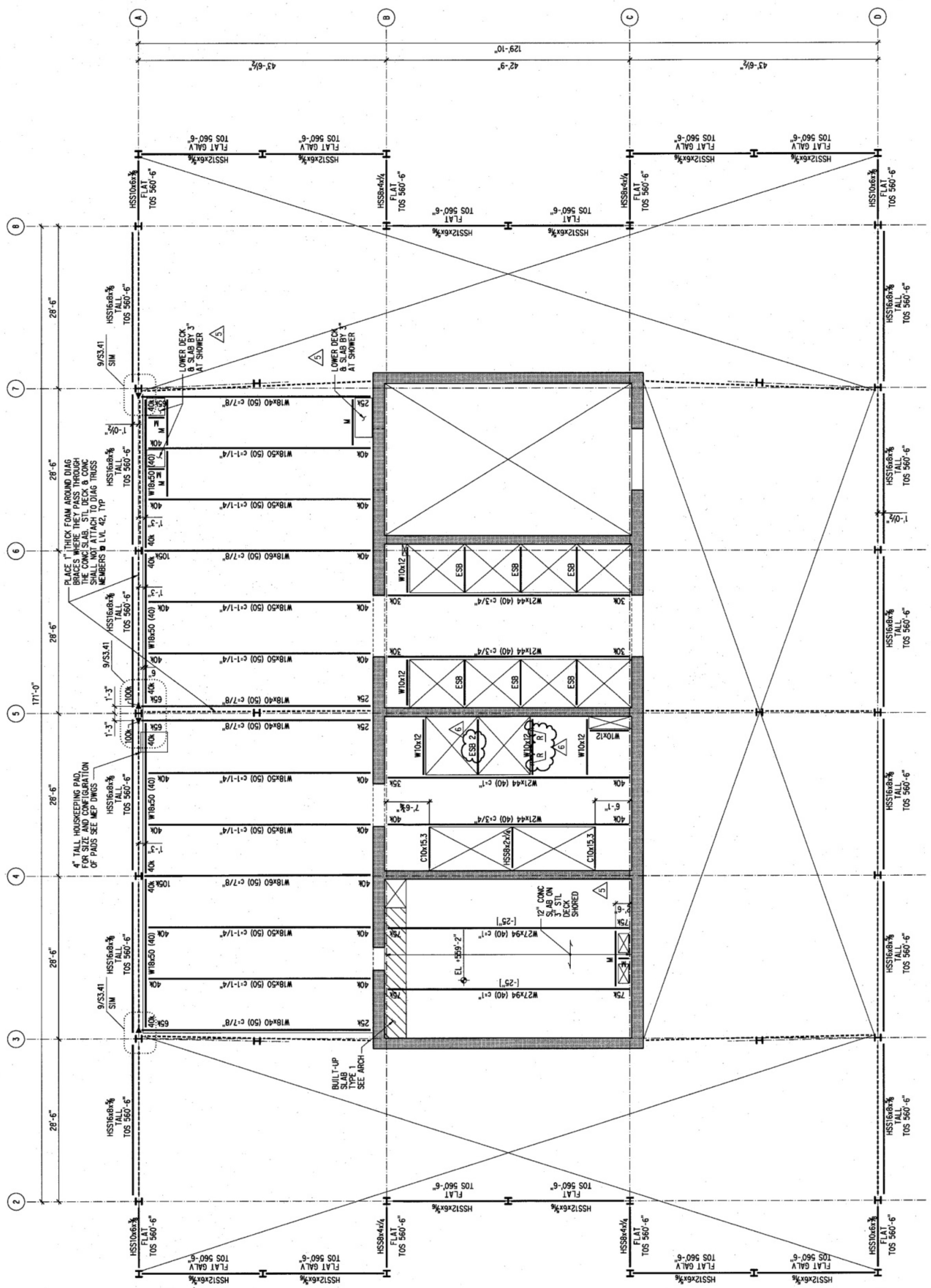
Typical Low/Mid-Rise



Typical High Rise



Level 42 – "Belt Truss" Open Floor



Appendix B : Wind Calculations

Factors and Coefficients		
	North/ South	East/ West
V	90	90
Kd	0.85	0.85
I	1.00	1.00
Exposure	B	B
Kzt	1.00	1.00
Kh	1.78	1.78
α	7.00	7.00
Zg	1200.00	1200.00
z	796.00	796.00
B	199.50	133.25
L	133.25	199.50
h	786.00	786.00
g_Q	3.40	3.40
g_V	3.40	3.40
g_R	3.92	3.92
η_1	0.34	0.34
zbar	471.60	471.60
Izbar	0.19	0.19
Lzbar	776.55	776.55
Q	0.76	0.77
$V_{bar}z$	115.49	115.49
N_1	2.26	2.26
R_n	0.08	0.08
R_h	0.09	0.09
R_B	0.30	0.41
R_L	0.15	0.11
R	0.37	0.42
Gf	0.86	0.88
Windward Cp	0.80	0.80
Leeward Cp	-0.50	-0.50
Parapet Windward GC_{pn}	1.50	1.50
Parapet Leeward GC_{pn}	-1.00	-1.00

Kz and qz Calculations		
Height (ft)	Kz	qz
15	0.57	10.05
20	0.62	10.93
25	0.66	11.63
30	0.70	12.34
40	0.76	13.40
50	0.81	14.28
60	0.85	14.98
70	0.89	15.69
80	0.93	16.39
90	0.96	16.92
100	0.99	17.45
120	1.04	18.33
140	1.09	19.21
160	1.13	19.92
180	1.17	20.62
200	1.20	21.15
250	1.28	22.56
300	1.35	23.79
350	1.41	24.85
400	1.47	25.91
450	1.52	26.79
500	1.56	27.50
550	1.61	28.35
600	1.65	29.06
650	1.69	29.73
700	1.72	30.37
750	1.76	30.98
786	1.78	31.39
796	1.79	31.51
Kh = 1.78		

Wind Calculations

		North / South						East / West					
Story Level	Story Height (ft)	Windward (plf)	Leeward (plf)	Windward (kips)	Leeward (kips)	Story Shear (Kips)	Moment (k-ft)	Windward (plf)	Leeward (plf)	Windward (kips)	Leeward (kips)	Story Shear (Kips)	Moment (k-ft)
LL-1	7.5	210.7	-321.4	28.1	-42.8	4442.2	532	213.1	-326.1	42.5	-65.0	6748.2	807
1	26.00	250.8	-356.6	33.4	-47.5	4371.3	2104	253.8	-361.7	50.6	-72.2	6640.6	3193
2	44.67	269.3	-348.6	35.9	-46.4	4290.4	3677	272.7	-353.6	54.4	-70.6	6517.8	5581
4	62.33	258.0	-313.4	34.4	-41.8	4208.0	4746	261.4	-317.9	52.1	-63.4	6392.9	7204
5	77.33	232.4	-268.6	31.0	-35.8	4131.9	5163	235.6	-272.5	47.0	-54.4	6277.3	7839
6	90.33	292.6	-326.2	39.0	-43.5	4065.2	7448	296.6	-330.9	59.2	-66.0	6175.9	11309
7	111.33	345.1	-374.1	46.0	-49.9	3982.7	10671	350.0	-379.6	69.8	-75.7	6050.7	16205
9	129.33	283.6	-297.4	37.8	-39.6	3886.9	10013	287.6	-301.7	57.4	-60.2	5905.2	15207
10	142.33	243.2	-249.4	32.4	-33.2	3809.4	9344	246.7	-253.1	49.2	-50.5	5787.6	14192
11	155.33	246.7	-249.4	32.9	-33.2	3743.8	10268	250.2	-253.1	49.9	-50.5	5687.9	15596
12	168.33	252.1	-249.4	33.6	-33.2	3677.7	11249	255.8	-253.1	51.0	-50.5	5587.5	17088
13	181.33	255.9	-249.4	34.1	-33.2	3610.9	12210	259.7	-253.1	51.8	-50.5	5486.0	18548
14	194.33	258.7	-249.4	34.5	-33.2	3543.5	13158	262.5	-253.1	52.4	-50.5	5383.7	19989
15	207.33	263.2	-249.4	35.1	-33.2	3475.8	14161	267.1	-253.1	53.3	-50.5	5280.8	21514
16	220.33	263.2	-249.4	35.1	-33.2	3407.5	15049	267.1	-253.1	53.3	-50.5	5177.1	22863
17	233.33	263.2	-249.4	35.1	-33.2	3339.2	15937	267.1	-253.1	53.3	-50.5	5073.3	24212
18	246.33	265.9	-249.4	35.4	-33.2	3270.9	16915	269.9	-253.1	53.8	-50.5	4969.5	25699
19	259.33	275.8	-249.4	36.8	-33.2	3202.2	18150	280.0	-253.1	55.9	-50.5	4865.2	27577
20	272.33	275.8	-249.4	36.8	-33.2	3132.3	19060	280.0	-253.1	55.9	-50.5	4758.9	28960
21	285.33	275.8	-249.4	36.8	-33.2	3062.3	19970	280.0	-253.1	55.9	-50.5	4652.5	30342
22	298.33	279.9	-249.4	37.3	-33.2	2992.3	21043	284.2	-253.1	56.7	-50.5	4546.2	31975
23	311.33	286.9	-249.4	38.2	-33.2	2921.8	22248	291.3	-253.1	58.1	-50.5	4439.0	33809
24	324.33	286.9	-249.4	38.2	-33.2	2850.3	23177	291.3	-253.1	58.1	-50.5	4330.4	35220
25	337.33	286.9	-249.4	38.2	-33.2	2778.8	24106	291.3	-253.1	58.1	-50.5	4221.8	36632
26	350.33	291.8	-249.4	38.9	-33.2	2707.4	25268	296.4	-253.1	59.1	-50.5	4113.2	38399
27	363.33	296.3	-249.4	39.5	-33.2	2635.2	26423	300.9	-253.1	60.0	-50.5	4003.6	40157
28	376.33	296.3	-249.4	39.5	-33.2	2562.5	27369	300.9	-253.1	60.0	-50.5	3893.1	41594
29	389.33	296.3	-249.4	39.5	-33.2	2489.8	28314	300.9	-253.1	60.0	-50.5	3782.6	43031
30	402.33	302.8	-249.4	40.3	-33.2	2417.1	29605	307.5	-253.1	61.4	-50.5	3672.0	44995
31	415.33	305.8	-249.4	40.8	-33.2	2343.5	30730	310.6	-253.1	62.0	-50.5	3560.2	46706
32	428.33	305.8	-249.4	40.8	-33.2	2269.5	31692	310.6	-253.1	62.0	-50.5	3447.8	48168
33	441.33	305.8	-249.4	40.8	-33.2	2195.5	32654	310.6	-253.1	62.0	-50.5	3335.3	49630
34	454.33	312.4	-249.4	41.6	-33.2	2121.5	34014	317.3	-253.1	63.3	-50.5	3222.8	51701
35	467.33	313.7	-249.4	41.8	-33.2	2046.6	35070	318.7	-253.1	63.6	-50.5	3109.0	53306
36	480.33	313.7	-249.4	41.8	-33.2	1971.6	36045	318.7	-253.1	63.6	-50.5	2995.0	54789
37	493.33	313.7	-249.4	41.8	-33.2	1896.6	37021	318.7	-253.1	63.6	-50.5	2880.9	56272
38	506.33	320.0	-249.4	42.6	-33.2	1821.5	38417	325.1	-253.1	64.8	-50.5	2766.9	58398
39	519.33	320.1	-249.4	42.6	-33.2	1745.6	39409	325.1	-253.1	64.9	-50.5	2651.5	59906
40	532.33	334.4	-260.6	44.6	-34.7	1669.8	42209	339.7	-264.4	67.8	-52.8	2536.2	64161
41	546.50	348.8	-270.2	46.5	-36.0	1590.5	45077	354.3	-274.1	70.7	-54.7	2415.6	68522
42	560.50	327.7	-249.4	43.7	-33.2	1508.0	43105	333.0	-253.1	66.4	-50.5	2290.3	65528
43	572.50	315.1	-239.8	42.0	-32.0	1431.1	42334	320.1	-243.3	63.9	-48.5	2173.3	64356
44	585.50	327.7	-249.4	43.7	-33.2	1357.1	45027	333.0	-253.1	66.4	-50.5	2060.9	68450
45	598.50	330.2	-249.4	44.0	-33.2	1280.2	46223	335.5	-253.1	66.9	-50.5	1944.0	70270
46	611.50	334.1	-249.4	44.5	-33.2	1203.0	47548	339.5	-253.1	67.7	-50.5	1826.6	72287
47	624.50	334.1	-249.4	44.5	-33.2	1125.2	48559	339.5	-253.1	67.7	-50.5	1708.4	73824
48	637.50	334.1	-249.4	44.5	-33.2	1047.5	49570	339.5	-253.1	67.7	-50.5	1590.2	75360
49	650.50	337.4	-249.4	45.0	-33.2	969.7	50862	342.8	-253.1	68.4	-50.5	1472.0	77327
50	663.50	340.1	-249.4	45.3	-33.2	891.5	52125	345.6	-253.1	69.0	-50.5	1353.1	79249
51	676.50	340.1	-249.4	45.3	-33.2	813.0	53146	345.6	-253.1	69.0	-50.5	1233.7	80801
52	689.50	340.1	-249.4	45.3	-33.2	734.4	54167	345.6	-253.1	69.0	-50.5	1114.2	82354
53	702.50	344.1	-249.4	45.8	-33.2	655.9	55558	349.7	-253.1	69.8	-50.5	994.8	84472
54	715.50	345.8	-249.4	46.1	-33.2	576.8	56754	351.5	-253.1	70.1	-50.5	874.5	86291
55	728.50	365.8	-263.8	48.7	-35.2	497.5	61119	371.7	-267.7	74.2	-53.4	753.9	92927
56	743.00	385.9	-278.2	51.4	-37.1	413.6	65746	392.1	-282.3	78.2	-56.3	626.4	99962
57	757.50	391.8	-278.2	52.2	-37.1	325.1	67629	398.2	-282.3	79.4	-56.3	491.8	102830
58	772.00	385.0	-273.4	51.3	-36.4	235.8	67735	391.3	-277.4	78.1	-55.3	356.1	102992
Roof	786.00	661.7	-449.4	88.2	-59.9	148.1	116373	664.8	-451.3	132.6	-90.0	222.7	175023

Appendix B : Wind Calculations

Hand Calculations:

Wind Flow Chart

1. Building does not meet requirements of 6.4.4 – Not low-rise
   ∴ Check Req's of 6.5.1 & 6.5.2
2. Yes meets req's of 6.5.1 & 6.5.2
   ∴ Determine wind loads w/ Method 2 (6.5)

Method 2: Find q_z & q_h (Velocity Pressures)

V: 90 mph (Fig 6-1 : ASCE 7-05)

K_d = 0.85 (Table 6-4 : ASCE 7-05)

I = 1.0 (Occupancy Category II)

Exposure category: B – urban (ASCE 6.5.6.3)

3. Conditions of 6.5.7.1 are not met – K_{zt} = 1.0 (topographic factor)
4. Determine K_z & K_h from table 6-3 (6.5.6.6)

   Exposure B / Case 2 (not low-rise)

K_z: α = 7.0 (Table 6-2)
z_g = 1200' (Table 6-2)
z = height = 796'

Gust Factors

North / South
B = 199' 6"
L = 133' 3"
h = 796'

$g_Q = g_v = 3.4$

East / West
B = 133' 3"
L = 198' 6"
h = 796'

$g_Q = g_v = 3.4$

$$g_R = \sqrt{2\ln(3{,}600 n_1)} + \frac{0.577}{\sqrt{2\ln(3{,}600 n_1)}}$$

$$n_1 = \frac{1}{T_a}$$

$T_a = C_t h_n^x$ (From Eqn 12.8-7)

$$n_1 = \frac{1}{(.02)(796)^{.75}} = 0.333 \text{ Hz}$$

Eqn 6-9

$$g_R = \sqrt{2\ln[(3{,}600)(0.333)]} + \frac{0.577}{\sqrt{2\ln[(3600)(0.333)]}} = 3.918$$

$$\bar{z} = 0.6(h) = 0.6(796) = 477.6'$$

$z_{min} = 30'$ from Table 6-2 $< 477.6'$ ✓OK

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} : \quad c = 0.30 \text{ from table 6-2}$$

Eq 6-5: $I_{\bar{z}} = 0.30\left(\frac{33}{477.6}\right)^{1/6} = 0.192$

Eq 6-7: $L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}}$ $\quad \ell = 320, \ \bar{\epsilon} = \frac{1}{3.0}$ (Table 6-2)

$$L_{\bar{z}} = 320'\left(\frac{477.6'}{33}\right)^{(1/3)} = 780.0'$$

Eq 6-6:

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{.63}}}$$

North / South Q:

$$\sqrt{\frac{1}{1 + 0.63\left(\frac{199'6'' + 796'}{780.0'}\right)^{.63}}} = 0.759$$

East / West Q:

$$\sqrt{\frac{1}{1 + 0.63\left(\frac{133'3'' + 796'}{780.0'}\right)^{.63}}} = 0.766$$

Eq 6-14:

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right) \qquad \bar{b} = 0.45, \ \bar{\alpha} = 1/4.0 \text{ from table 6-2}$$

$$\bar{V}_{\bar{z}} = 0.45\left(\frac{477.6'}{33}\right)^{1/4}(90)\left(\frac{88}{60}\right) = 116.0 \text{ mph}$$

Eq 6-12:

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} \qquad N_1 = \frac{0.332\,(780.0')}{116.0} = 2.24$$

Eq 6-11:

$$R_n = \frac{7.47\,N_1}{(1 + 10.3N_1)^{5/3}} = \frac{7.47\,(2.24)}{(1 + 10.3\,(2.24))^{5/3}} = 0.0834$$

$R_h = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1 - e^{-2\eta}\right)$ for $\eta > 0$

$R_h = 1$ for $\eta = 0$

$\eta = 4.6 n_1 h / \bar{V}_{\bar{z}}$ $\quad \eta = 4.6(0.332)(796')/116.0 = 10.51$

$R_h = \frac{1}{10.51} - \frac{1}{2(10.51)^2}\left(1 - e^{-2(10.51)}\right) = 0.0906$

$R_B = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1 - e^{-2\eta}\right)$

$\eta = 4.6 n_1 B / \bar{V}_{\bar{z}}$

North / South R_B:

$\eta = \frac{4.6(.332)(199'6")}{116.0 \text{ mph}} = 2.29$

$R_B = \frac{1}{2.29} - \frac{1}{2(2.29)^2}\left(1 - e^{-2(2.29)}\right)$

$R_B = 0.343$

East/West R_B:

$\eta = \frac{4.6(.332)(133'3")}{116 \text{ mph}} = 1.75$

$R_B = \frac{1}{1.75} - \frac{1}{2(1.75)^2}\left(1 - e^{-2(1.75)}\right)$

$R_B = 0.413$

$R_L = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1 - e^{-2\eta}\right)$

$\eta = 15.4 n_1 L / \bar{V}_{\bar{z}}$

North / South R_L:

$\eta = \frac{15.4(0.332)(133'3")}{116.0} = 5.873$

$R_L = 0.156$

East/West R_L:

$\eta = \frac{15.4(0.332)(199'6")}{116.0} = 8.793$

$R_L = 0.107$

Eq 6.10 $\quad R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)}$

$\beta = 0.01$ (Damping ratio is conservative for steel & concrete combined construction; based on all steel building from ASCE 7-05 commentary Pg 294)

North / South R:

$R = \sqrt{\frac{1}{0.01}(0.0834)(0.0906)(0.343)(.53 + .47(0.156))}$

$R_{N\text{-}S} = 0.395$

Eqn 6.10 cont.

East / West R:

$$R = \sqrt{\frac{1}{0.01}(0.0834)(0.0906)(0.413)(0.53 + .47(0.107))}$$

$$R_{E/W} = 0.426$$

Eqn 6-8 : $G_f = 0.925\left[\dfrac{1 + 1.7 I_{\bar{z}} \sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_{\bar{z}}}\right]$

North / South G_F:

$$G_F = 0.925\left[\frac{1 + 1.7(0.192)\sqrt{(3.4^2)(0.759^2) + (3.918^2)(0.395^2)}}{1 + 1.7(3.4)(0.192)}\right]$$

$$\boxed{G_{F_{N-S}} = 0.869}$$

East / West G_F:

$$G_{F_{E/W}} = 0.925\left[\frac{1 + 1.7(0.192)\sqrt{(3.4^2)(0.766^2) + (3.918^2)(0.135^2)}}{1 + 1.7(3.4)(0.192)}\right]$$

$$\boxed{G_{F_{E-W}} = 0.816}$$

Flow chart 5.7

1) Building is enclosed ✓ yes

3) The Building is not low-rise
4) The building is not rigid

5) q_z & q_h on attached chart

6) Determine C_p for walls & roof from Fig 6-6 or 6-8

From Fig 6-6

North / South

$L/B = \frac{133'3"}{199'6"} = 0.6679$

Windward $C_p = 0.8$ w/ q_z
Leeward $C_p = -0.5$ w/ q_h
Side wall $C_p = -0.7$ w/ q_h

East / West

$L/B = \frac{199'6"}{133'3"} = 1.497$

Windward $C_p = 0.8$,
Leeward = $L/B = 1$ $C_p = -0.5$
$L/B = 2$ $C_p = -0.3$
$\therefore$ $\boxed{L/B = 1.497 \quad C_p = -0.4}$
Sidewall $C_p = -0.7$

Assume flat roof (slope of 8.5" over 32' ≈ 0°)

North / South
$h/L = \frac{796'}{133'3"} = 5.97$

C_p : 0 to $h/2$ = $-1.3, -0.18$

East / West
$h/L = \frac{796'}{199'6"} = 3.99$

C_p : 0 to $h/2$ = $-1.3, -0.18$

7) Determine q_i for walls & roof

From 6.5.12.2.1 : $q_i = q_h$ for enclosed buildings

8) Determine internal pressure coefficients (GC_{pi})
From Fig. 6-5

GC_{pi}: $+0.18$
-0.18

Building = Enclosed (No operable windows, only openings are Doors @ Lower levels)

9) Determine design wind pressures:

Eqn 6-19: $p_z = q_z(G_F C_p) - q_h(GC_{pi})$ - windward walls

$p_h = q_h(G_F C_p) - q_h(GC_{pi})$

Appendix C: Seismic Calculations

Coefficients and References:

Coefficients and References		
Factors	Values	Reference
Longitude/ Latitude	41^o 59' N / 87^o 54' W	
height (ft)	786.000	
S_S	0.162	USGS website
S_1	0.059	USGS website
Site Class	D	ASCE7-05 11.4.2 Site Class
$S_{MS}=F_aS_S$	0.259	ASCE7-05 Eqn 11.4-1
$S_{M1}=F_vS_1$	0.142	ASCE7-05 Eqn 11.4-2
F_a	1.600	ASCE7-05 Table 11.4-1
F_v	2.400	ASCE7-05 Table 11.4-2
$S_{DS}=(2/3)S_{MS}$	0.173	ASCE7-05 Eqn 11.4-3
$S_{D1}=(2/3)S_{M1}$	0.094	ASCE7-05 Eqn 11.4-4
Ta	2.969	ASCE7-05 Eqn 12.8.7
Ts=SD1/SDS	0.546	ASCE7-05 11.6
.8Ts	0.437	
SDC	B	
V=CsW	2202	
Cs= S_{DS}/(R/I)*	0.043	ASCE7-05 12.8
Cs= SDS/(T*R/I)*	0.0064	
R	4.000	ASCE7-05 12.2-1 B.6.
I	1.000	
T_L	12.000	ASCE7-05 Fig 22-15
T=Ta	T<TL	ASCE7-05 12.8.2
W (k)	220212	
*Note: Since lowest C_s is less tha 0.01, 0.01 was used for calculating V		

Weight Calculations:

Approximate Weight of Slabs					
Floor	Area of concrete / floor	Depth of Slab	Type of Conc.	Weight of Conc. (K)	Weight/ Floor
59	22581.3575	4.5	lwc	973.82	973.82
58	20925.75	4.5	lwc	902.42	902.42
High Rise 44-57	21788.25	4.5	lwc	13154.66	939.62
43	22719.625	9.5	nwc	2697.96	2697.96
42	5251.125	4.5	lwc	226.45	226.45
42*	1222.65	13.5	lwc	158.18	158.18
41	20947.5	9.5	nwc	2487.52	2487.52
Mid Rise 26-40	20947.5	4.5	lwc	13550.41	903.36
19-25	20605.5	4.5	lwc	6220.29	888.61
10-18	21417.75	4.5	lwc	8312.76	923.64
9	22570.55	4.5	lwc	973.35	973.35
7	24166.55	4.5	lwc	1042.18	1042.18
6	24166.55	4.5	lwc	1042.18	1042.18
6*	2265.75	4.5	nwc	127.45	127.45
5	14512.175	4.5	lwc	625.84	625.84
4	24166.55	9.5	nwc	2869.78	2869.78
4*	2109	4.5	lwc	90.95	90.95
2	12045	4.5	lwc	519.44	519.44
1	32982.5	4.5	lwc	1422.37	1422.37
LL1	35070	7.5	nwc	3287.81	3287.81
* Assumptions	lwc=115pcf (from spec.)		Total Weight=	60685.82751	

Beam Weights per Floor					
Floors	Beam Weight	# of beams	Beam Length	Weight/floor (kips)	Total Weight (Kips)
High Rise (43-58)	35	14	43.50	21.32	319.73
	35	4	28.50	3.99	59.85
	40	24	43.50	41.76	626.40
	40	5	42.75	8.55	128.25
	40	4	28.50	4.56	68.40
	46	4	28.50	5.24	78.66
	60	8	28.50	13.68	205.20
	55	4	43.50	9.57	143.55
	55	2	42.75	4.70	70.54
	61	2	42.75	5.22	78.23
	53	2	42.75	4.53	67.97
	43	1	42.75	1.84	27.57
	44	4	42.75	7.52	112.86
Sum of High Rise					1987.21
Roof	84	20	43.50	73.08	0.00
	84	12	28.60	28.83	0.00
	76	14	43.50	46.28	0.00
	55	4	43.50	9.57	0.00
	22	4	42.75	3.76	0.00
	22	4	43.50	3.83	0.00
	116	2	42.75	9.92	0.00
	43	24	11.00	11.35	0.00
	44	2	42.75	3.76	0.00
	94	2	42.75	8.04	0.00
	55	4	42.75	9.41	0.00
Sum of Roof					207.83
Mid Rise(25-40)	35	26	43.50	39.59	633.36
	35	8	28.50	7.98	127.68
	40	16	43.50	27.84	445.44
	94	4	28.50	10.72	171.46
	46	4	28.50	5.24	83.90
	55	4	43.50	9.57	153.12
	55	2	42.75	4.70	75.24
	61	3	42.75	7.82	125.17
	53	2	42.75	4.53	72.50
	44	3	28.50	3.76	60.19
	44	6	42.75	11.29	180.58
	114	4	27.00	12.31	110.81
Sum of Mid Rise					2239.45

Low Rise (9-11, 16-24)	35	26	43.50	39.59	475.02
	35	7	28.50	6.98	83.79
	40	16	43.50	27.84	334.08
	94	4	28.50	10.72	128.59
	99	4	28.50	11.29	135.43
	55	1	28.50	1.57	18.81
	55	4	43.50	9.57	114.84
	55	2	42.75	4.70	56.43
	61	2	42.75	5.22	62.59
	53	2	42.75	4.53	54.38
	44	8	42.75	15.05	180.58
	68	1	42.75	2.91	34.88
	53	1	42.75	2.27	27.19
	114	4	27.00	12.31	147.74
Sum of Low Rise					1854.35
Low Rise (12-15)	55	30	43.50	71.78	287.10
	55	9	28.50	14.11	56.43
	40	8	43.50	13.92	55.68
	60	8	43.50	20.88	83.52
	94	4	28.50	10.72	42.86
	99	4	28.50	11.29	45.14
	55	2	42.75	4.70	18.81
	61	3	42.75	7.82	31.29
	53	2	42.75	4.53	18.13
	40	2	42.75	3.42	13.68
	44	8	42.75	15.05	60.19
	48	1	42.75	2.05	8.21
	114	4	27.00	12.31	49.25
Sum of Low Rise					770.30
Conference (6-7)	55	27	43.50	64.60	129.20
	55	4	28.50	6.27	12.54
	217	5	43.50	47.20	94.40
	62	7	43.50	18.88	37.76
	90	4	28.50	10.26	20.52
	62	4	28.50	7.07	14.14
	35	10	28.50	9.98	19.95
	55	16	42.75	37.62	75.24
	50	2	42.75	4.28	8.55
	53	2	42.75	4.53	9.06
	14	25	13.50	4.73	9.45
	114	3	27.00	9.23	18.47
Sum of Conference					449.27

Mechanical (5)	44	16	43.50	30.62	
	57	5	43.50	12.40	
	44	14	42.75	26.33	
	68	2	42.75	5.81	
	50	2	42.75	4.28	
	84	2	28.50	4.79	
	57	2	28.50	3.25	
	68	1	28.50	1.94	
	108	1	28.50	3.08	
	50	1	28.50	1.43	
	26	4	26.00	2.70	
Total					96.63
Mechanical (4)	71	26	43.50	80.30	
	76	5	43.50	16.53	
	55	7	43.50	16.75	
	26	2	43.50	2.26	
	86	5	43.50	18.71	
	55	8	42.75	18.81	
	116	2	42.75	9.92	
	132	1	42.75	5.64	
	68	2	42.75	5.81	
	57	2	42.75	4.87	
	84	1	42.75	3.59	
	35	8	37.21	10.42	
	148	1	28.50	4.22	
	55	4	28.50	6.27	
	94	1	28.50	2.68	
	68	4	28.50	7.75	
	62	5	28.50	8.84	
	235	1	28.50	6.70	
	191	1	28.50	5.44	
	90	1	28.50	2.57	
	99	1	28.50	2.82	
	35	4	28.50	3.99	
Total					244.88

Mezzanine (2)	76	2	37.21	5.66	
	35	4	37.21	5.21	
	68	2	37.21	5.06	
	55	1	43.5	2.39	
	44	4	43.5	7.66	
	40	1	43.5	1.74	
	35	1	43.5	1.52	
	26	1	43.5	1.13	
	44	5	42.75	9.41	
	40	8	42.75	13.68	
	68	1	42.75	2.91	
	55	1	42.75	2.35	
	26	2	21.375	1.11	
	14	2	21.375	0.60	
	35	16	30	16.80	
Total					77.22
Lobby (1)	40	5	21	4.20	
	22	2	21	0.92	
	55	1	21	1.16	
	26	3	21	1.64	
	231	1	43.5	10.05	
	108	16	43.5	75.17	
	247	5	43.5	53.72	
	135	8	43.5	46.98	
	282	1	43.5	12.27	
	26	1	43.5	1.13	
	44	4	43.5	7.66	
	68	2	43.5	5.92	
	118	1	43.5	5.13	
	150	1	43.5	6.53	
	55	2	37.21	4.09	
	35	8	37.21	10.42	
	99	4	37.21	14.74	
	124	2	37.21	9.23	
	132	2	37.21	9.82	
	141	1	37.21	5.25	
	90	1	37.21	3.35	
	55	2	42.75	4.70	
	50	2	42.75	4.28	
	84	1	42.75	3.59	
	44	11	42.75	20.69	
	76	1	42.75	3.25	
	35	7	28.5	6.98	
	55	4	28.5	6.27	
	62	5	28.5	8.84	
	84	3	28.5	7.18	
	76	2	28.5	4.33	
	68	1	28.5	1.94	
	40	1	28.5	1.14	
	108	1	28.5	3.08	
	90	1	28.5	2.57	
	124	1	28.5	3.53	
	116	1	28.5	3.31	
	26	4	28.5	2.96	
	26	1	9.75	0.25	
	19	12	48	10.94	
	26	1	54.33	1.41	
Total					390.60

Lower Level 1	46	17	37.21	29.10	
	50	1	37.21	1.86	
	90	1	37.21	3.35	
	124	1	37.21	4.61	
	57	5	43.5	12.40	
	50	11	43.5	23.93	
	90	2	43.5	7.83	
	71	5	43.5	15.44	
	55	7	43.5	16.75	
	76	2	43.5	6.61	
	57	2	42.75	4.87	
	76	1	42.75	3.25	
	22	2	42.75	1.88	
	22	6	22	2.90	
	35	3	28.5	2.99	
	22	4	16	1.41	
	14	9	16	2.02	
Total					141.20
"Belt" Truss open 42	40	4	43.5	6.96	
	50	8	43.5	17.40	
	60	2	43.5	5.22	
	44	4	42.75	7.52	
	93.1	12	28.5	31.84	* HSS 16x8x5/8
	36.1	4	43.5	6.28	*HSS 12x6x5/16
	36.1	2	42.75	3.09	
	94	2	42.75	8.04	
Total					86.35
Mechanical 41	57	25	43.5	61.99	
	68	8	43.5	23.66	
	55	4	43.5	9.57	
	83	2	43.5	7.22	
	44	3	43.5	5.74	
	55	2	42.75	4.70	
	62	4	42.75	10.60	
	44	4	42.75	7.52	
	40	3	28.5	3.42	
Total					134.43

Shear Wall Weight

Level	North/ South Wall length	North/ South Wall Thckness (ft)	East/ West Wall Length (ft)	Line 3 thickness (ft)	Line 4 thickness (ft)	Line 5 thickness (ft)	Line 6 thickness (ft)	Line 7 thickness (ft)
LL-1 thru 7	118.42	2.25	42.75	2	1.5	1.5	1.5	2
9 thru 42	118.42	2	42.75	2	1.5	1.5	1.5	2
43-60	61.13	1.5	46.75	0	1.5	1.5	1.5	0

Height (ft)	Area of openings/floor	Average Area of Wall	North/ South Reduction due to Openings =(1-(Area openings/ Area Wall))	North/ South Cubic Feet of Concrete	Sum of 5 East/ West Walls in cubic feet
121.83	433.3	1539.416667	0.7185	46648	44271
443.17			0.7185	150829	161036
223.50	185	794.625	0.7672	31443	47019
Sum of Shear Wall Concrete in Cubic Feet			481246		
Weight of Shear Wall (ft^3 Concrete * 150pcf)			72187		

Seismic Calculations

Level	Plf / Floor	Height to next floor	Height (ft)	Wx Columns	Wx Beams	Wx Conc. Slab	Wx Shear wall	Wx SDL	Wx Curtain Wall	Wx (k)	wihi^k	Fx (k)	Vx (k)	Moment (k-ft)
Parapet			796				0							
Roof		10.0	786	0	208	974	842	847	7	2877	6506974248	46	46	36457.01
58	2644	14.0	772	37	132	902	1179	628	9	2888	6439008796	46	92	35433.63
57	1596	14.5	758	23	132	940	1221	763	10	3089	7226124237	52	144	39018.22
56	1704	14.5	743	25	132	940	1221	763	10	3090	7094991612	51	194	37576.82
55	1704	14.5	729	25	132	940	1221	763	10	3090	6956529461	50	244	36124.48
54	2134	13.0	716	28	132	940	1095	763	9	2966	6293843914	45	289	32100.00
53	2134	13.0	703	28	132	940	1095	763	9	2966	6179490356	44	333	30944.14
52	2420	13.0	690	31	132	940	1095	763	9	2970	6080352743	43	376	29884.25
51	2420	13	677	31	132	940	1095	763	9	2970	5965712300	43	419	28767.99
50	2866	13	664	37	132	940	1095	763	9	2975	5873942067	42	461	27781.13
49	2866	13	651	37	132	940	1095	763	9	2975	5758853526	41	502	26703.16
48	3472	13	638	45	132	940	1095	763	9	2983	5673690738	40	542	25782.51
47	3472	13	625	45	132	940	1095	763	9	2983	5557991947	40	582	24741.71
46	3882	13	612	50	132	940	1095	763	9	2989	5461757249	39	621	23807.19
45	3882	13	599	50	132	940	1095	763	9	2989	5345644666	38	659	22805.71
44	4274	13	586	56	132	940	1095	763	9	2994	5247381515	37	696	21900.24
43	4274	13	573	56	132	2698	1095	795	9	4785	13106075479	93	790	53484.45
42	4732	12	561	57	86	385	2081	194	8	2811	4428290748	32	821	17692.58
41	4732	14	547	66	134	2488	1921	628	9	5247	15044029996	107	928	58604.87
40	4602	14	532	65	145	903	2241	733	9	4097	8937166696	64	992	33912.74
39	4602	13	519	60	145	903	2268	733	9	4118	8806512511	63	1055	32600.90
38	4750	13	506	62	145	903	2081	733	9	3933	7832657098	56	1111	28269.95
37	4750	13	493	62	145	903	2081	733	9	3933	7631555303	54	1165	26836.93
36	5008	13	480	65	145	903	2081	733	9	3936	7443131697	53	1218	25484.60
35	5008	13	467	65	145	903	2081	733	9	3936	7241686772	52	1270	24123.81
34	5450	13	454	71	145	903	2081	733	9	3942	7060809918	50	1320	22866.96
33	5450	13	441	71	145	903	2081	733	9	3942	6858776472	49	1369	21577.08
32	5366	13	428	70	145	903	2081	733	9	3941	6653055677	47	1416	20313.39
31	5366	13	415	70	145	903	2081	733	9	3941	6451134143	46	1462	19099.07
30	5846	13	402	76	145	903	2081	733	9	3947	6269017110	45	1507	17978.97
29	5846	13	389	76	145	903	2081	733	9	3947	6066455663	43	1550	16835.89
28	6256	13	376	81	145	903	2081	733	9	3953	5879740582	42	1592	15772.85
27	6256	13	363	81	145	903	2081	733	9	3953	5676631740	40	1633	14701.96
26	6596	13	350	86	145	903	2081	733	9	3957	5485771012	39	1672	13699.30
25	6596	13	337	86	145	889	2081	721	9	3930	5211116641	37	1709	12530.53
24	7000	13	324	91	155	889	2081	721	9	3945	5047147434	36	1745	11668.55
23	7000	13	311	91	155	889	2081	721	9	3945	4844846560	35	1779	10751.90
22	6956	13	298	90	155	889	2081	721	9	3944	4641199444	33	1813	9869.87
21	6956	13	285	90	155	889	2081	721	9	3944	4438957233	32	1844	9028.44
20	7348	13	272	96	155	889	2081	721	9	3949	4247669834	30	1874	8245.76
19	7348	13	259	96	155	889	2081	721	9	3949	4044904689	29	1903	7477.32
18	7212	13	246	94	155	924	2081	750	9	4011	3963105952	28	1932	6958.86
17	7212	13	233	94	155	924	2081	750	9	4011	3753956923	27	1958	6243.75
16	7604	13	220	99	155	924	2081	750	9	4016	3553820942	25	1984	5581.55
15	7604	13	207	99	193	924	2081	750	9	4054	3407797815	24	2008	5036.42
14	7768	13	194	101	193	924	2081	750	9	4056	3197485928	23	2031	4429.30
13	7768	13	181	101	193	924	2081	750	9	4056	2983588927	21	2052	3856.52
12	8496	13	168	110	193	924	2081	750	9	4066	2782631259	20	2072	3338.91
11	8496	13	155	110	155	924	2081	750	9	4028	2519905837	18	2090	2790.16
10	8722	13	142	113	155	924	2081	750	9	4031	2312382249	16	2106	2346.10
9	8722	13	129	113	155	973	2081	790	9	4121	2196129116	16	2122	2024.64
7	11594	18	111	209	225	1042	2164	1329	12	4980	2761599649	20	2142	2191.62
6	11594	21	90	243	225	1170	2164	1329	14	5145	2390907244	17	2159	1539.54
5	11541	13	77	150	97	626	2164	73	9	3118	751589052	5	2164	414.31
4	11541	15	62	173	245	2961	2996	665	10	7049	3097517890	22	2186	1376.30
2	12798	18	45	226	77	519	3495	241	12	4571	933174983	7	2193	297.12
1	12798	19	26	239	391	1422	2164	1855	12	6083	962186503	7	2200	178.32
ll-1	11951	19	8	221	141	3288	2497	701	12	6861	352998782	3	2202	18.87
										Sum of wihi^k=	308931408879			1061879.18

Hand Calculations and Assumptions:

Seismic Flowchart

1) Not a 1 or 2 Family dwelling
2) Not an agricultural storage structure
3) Does not require special consideration per Chapter 15
4) Seismic ground motion values

- Determine S_s & S_1
  - Zipcode : 60654 (Chicago, Cook, IL)
  - $S_s = 0.162$
  - $S_1 = 0.059$

  > USGS website "http://earthquake.usgs.gov/research/hazmaps/design/

5) Is $S_s \leq 0.15$ & $S_1 \leq 0.04$? No

6) Is the structure seismically isolated or does it have damping systems ? NO

7) Determine the site class of the soil in accordance with 11.4.2 & Chapter 20.

11.6 Seismic Design Category

$S_1 < 0.75$ ok

1. $T_a = C_T h^x = (0.02)(786)^{0.75} = 2.97$ (12.8.21)
   $C_T = 0.02$
   $h = 786'$
   $x = 0.75$

$T_S = \frac{S_{D1}}{S_{DS}} = 0.437 < 2.97$ ∴ Not ok

Determine SDC as more severe of 11.6-1 & 11.6-2

SDC = B by both charts due to category II

11.6-1 $0.167 \leq S_{DS} = 0.1728 < 0.33$

11.6-2 $0.067 \leq S_{D1} = 0.0944 < 0.133$

Note: Remember lower level 1 is lower limit around ground.

Assumption: Partition weight added to SDL
∴ all office floors = 35psf superimposed
15psf from Mech / Elec / ceiling
20psf partitions
Floors 6 & 7 have 55psf SDL

9 - 40, 43 - 57 = office

Mechanical area = 41, 42 = 30psf SDL

Level 5 = 5psf SDL from loading diagrams

Level 4 = 50% Mechanical & 50% Data center
30psf 25psf
∴ 27.5psf

Level 2 = Amenity ∴ 20psf SDL

LL 1 = Assume 20psf as average of SDL's for Retail, Truck Dock, Mechanical, & com ED

Level 1 = By inspection: 1/4 (Retail), 1/4 (Retail & Built up),
1/4 (Lobby), 1/4 (Plaza)

$58.25 = \frac{1}{4}(20) + \frac{1}{4}(80) + \frac{1}{4}(45) + \frac{1}{4}(80)$

Curtain wall = 15psf vertical surface

$\approx 2[(199.5) + (133.33)] \times h$

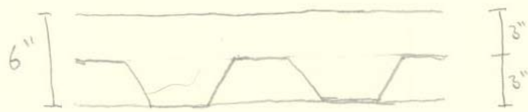
Roof: 50% Green roof, 50% Roof
$\frac{1}{2}(50) + \frac{1}{2}(25) = 37.5$psf

Appendix D: Spot Checks

Typical Slab Spot Check

Typical Slab Spot Check

- Typical Floor = 3" slab of light weight concrete on 3" composite steel deck
  - ★ Based on minimum steel deck = 20 GA
  - ★ Max Dead load deflection = 3/4" or L/190.



Max unsupported length = 9' 6"

Floor DL:

15 psf = Mech / Elec / Ceiling SDL

★ LWC = 115 pcf

Depth = 3" + ½ (Deck = 3") = 4.5"

Conc. PSF = (115 pcf)(4.5"/12"/') = 43.125 psf

Steel Deck = ★ Allowable Decks = Verco Type W or ASC Type W

From Vercodeck.com
W3 Formlock 20 GA Galv.
= 2.3 psf

Total Dead load = 60.425 psf

Total Live load =
based on loading diagram
Most severe load = "G"
increased office live load
LL = 100 psf
LL (Partitions) = 20 psf
LL = 120 psf

$W_u = 1.2D + 1.6L$

$W_u = 1.2(60.425) + 1.6(120)$

$W_u = 264.51$ psf

From: Verco Decking, INC.
Steel Floor Decks Catalog

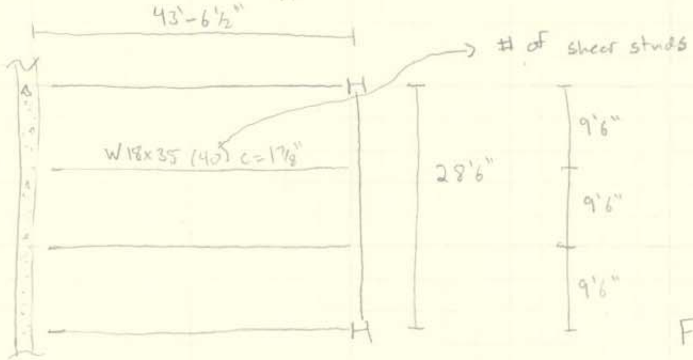
Allowable superimposed loads:

20 GA @ 9'6" span
= 268 psf > 264.51 psf
(too close bump up deck)

18 GA @ 9'6" span
= 338 psf > 264.51 psf ∴ ✓ OK

Typical Beam Spot Check

Beam Spot Check: Typical Floor – Level 27



Floor DL = 45.425
(From typ slab ✓)
SDL from load diag.
= 15 psf
Total DL = 60.425 psf

Floor LL =
50 psf LL
20 psf Partitions
Total LL = 70 psf
* From load diagram

Light weight Concrete
w = 115 pcf
f'_c = 4000 psi

Note: Beam is assumed to be pinned-pinned.

W 18 x 35
F_y = 50 ksi
I_x = 510 in^4
A_s = 10.3 in^2

Live Load Reduction – Reference AISC Steel Construction Manual 13th Ed.

A_I = Influence Area = (9'6" x2)(43'-6½") = 827.3 ft^2 > 400 ft^2 ∴ Reducible

$$LL = L_o\left(0.25 + \frac{15}{\sqrt{A_I}}\right) = 70\,\text{psf}\left(0.25 + \frac{15}{\sqrt{827.3}}\right) = 54.0\ \text{psf}$$

Using LRFD:

Tributary Width = 9'6"

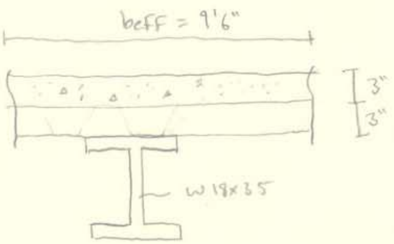
W_u = 1.2D + 1.6L (controlling combination)

W_u = 1.2(60.425) + 1.6(54) = 158.91 psf

w_u = 158.91 psf (9'6")
w_u = 1509.65 plf
+ Self weight of beam = 1.2(35)

$$M_u = \frac{w_u l^2}{8} = \frac{(1{,}551.65\,\text{plf})(43'\text{-}6\tfrac{1}{2}")^2}{8} / 1000 = 366.9\ \text{k-ft}$$

b_{eff} ≤
- Spacing = 9'6" ⟶ controls
- $\frac{\text{span}}{4} = \frac{43'6\tfrac{1}{2}"}{4}$ = 10'11"

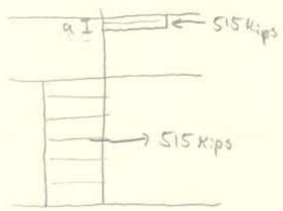


Deck runs perpendicular to beam

$A_{conc} = (3")(9'6" \times \frac{12"}{1'}) = 342 in^2$

$C_c = 0.85 F'_c A_c = 0.85 (4 ksi)(342 in^2) = 1162.8$ Kips

$T_s = A_s F_y = (10.3 in^2)(50 ksi) = 515$ kips



$a = \frac{T_s}{0.85 (F'_c)(beff)} = \frac{515 k}{0.85(4)(9'6 \times 12)} = 1.33"$

$Y_2 = 6" - \frac{a}{2} = 6" - \frac{1.33}{2} = 5.34" \rightsquigarrow$ (5" conservative)

From Steel Manual Table 3-19

@ P.N.A = BFL & Y2 = 5"
$M_n = 435$ k-Ft > 366.9 k-Ft
$\Sigma Q_n = 260$ K

Determine Required # of Shear Studs:

- Drawing notes designate 3/4" "Nelson" or "Tru-Weld" studs @ maximum 2'0" spacing.

Q_n per stud $= 0.5 A_{sc} \sqrt{F'_c E_c} \leq R_g R_p A_{sc} F_u = 0.442(65) = 28.73$ (with $R_g = 1.0$, $R_p = 1.0$)

$E_c = w^{1.5} \sqrt{F'_c} = 115^{1.5} \sqrt{4} = 2466$ ksi
$A_{sc} = \frac{\pi (3/4)^2}{4} = 0.442 in^2$

$Q_n = 0.5 (0.442) \sqrt{4(2466)} = 21.9^k < 28.73^k$ ✓

# of studs $= \frac{260^k}{21.9^k} = 11.87 \approx 12$ studs × 2 sides of beam
= 24 studs < 40 studs ✓ OK

★ Ribs are spaced every 12" o.c. the 40 studs prescribed follows placement of a stud in all but 3 ribs, most likely for constructibility as all beams on the floor call for 40 studs.

Deflection Check

$\frac{lbs}{ft} \times \frac{1}{12}$

Live Load Deflection (ΔL) $= \frac{5 w_L \ell^4}{384 EI} = \frac{(5)(54psf)(9\text{-}6")/12)(43'\text{-}6\tfrac{1}{2}" \times 12"/ft)^3}{384(29{,}000)(I)}$

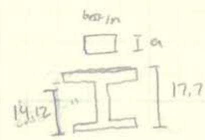
w_L = service Live Load

$I_{TR} = I_o + Ad^2$

$f_c A_c = f_t A_t$

$\frac{f_c A_c}{f_t} = A_t$

$\frac{A_c}{n} = A_t$



$\frac{b_{eff}}{n} = \frac{114"}{11.76} = 9.7"$

$I_T = \frac{bh^3}{12} = \frac{(9.7")(1.33")^3}{12}$

$\frac{E_s}{E_c} = \frac{29{,}000}{2466} = n = 11.76$

$I_T = 1.9 in^4$

$A_T = (9.7)(1.33) = 12.9 in^2$

$A_s = 10.3 in^2$

$\bar{y} = \frac{A_s(\frac{h}{2}) + A_T(h_s + h_c - \frac{a}{2})}{A_s + A_T} = \frac{10.3(17.7/2) + 12.9(13 + 6 - \frac{1.33}{2})}{10.3 + 12.9}$

$\bar{y} = 14.12"$

$I_{tr} = I_{os} + A_s d^2 + I_t + A_c d^2$

$I_{tr} = 510 in^4 + 10.3(14.12 - \frac{17.7}{2})^2 + 1.9 in^4 + 12.9 in^2 (17.7 - 14.12 + 6 - \frac{1.33}{2})^2$

$I_{tr} = 1823.2 in^4$

$\Delta_{LL} = \frac{5(.513^{k}/ft)(43'6\tfrac{1}{2}")^3(1728)}{384(29{,}000)(1823.2)} = 0.018" \ll \frac{L}{360} = 1.45"$ ✓ ok

Deflection During Construction

$W_D = 45.425 psf\,(9'6") = 431.5 plf + 35 plf$ (weight of beam) $= 466.5 plf$

$W_L = 20 psf\,(9'6") = 190 plf$

$W_T = 1.2(466.5 plf) + 1.6(190 plf) = .864^{k}/ft$

$M_u = \frac{(0.864)(43'6\tfrac{1}{2}")^2}{8} = 204'^k < \phi M_p = 249'^k$ (AISC Manual Table 3-6)

$\Delta_{DL} = \frac{5}{384} \frac{(0.4665)(43'\text{-}6\tfrac{1}{2}")^4(1728)}{(29{,}000)(510)} = 2.54" - 1\tfrac{7}{8}"$ camber $= \tfrac{2}{3}" < \tfrac{3}{4}" < 2.9"$ ✓ ok

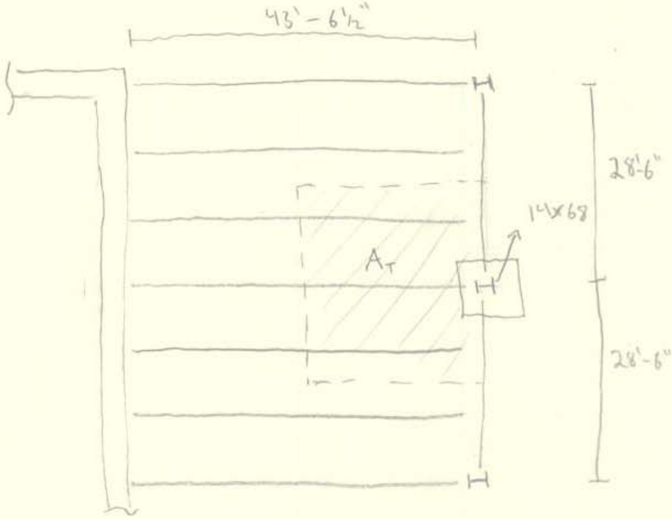
Δ_{DL} Max allowed by design docs $= \frac{L}{180}$ or $\tfrac{3}{4}"$

$L/180 = 2.9"$

★ Beam is adequate to carry loads.

Typical Column Spot Check

Steel Column Spot Check - Level 57 (Supporting 58)



Level 59 weight:
Conc. = same deck as typ slab
∴ DL = 46.425 psF + 25 SDL from Dwgs
71.425 psF × (28'6") × ½(43'6")
= 44.27 K

Steel Beams = 3 × ½ × 43-6½" × 76 plF
= 4.96 K
= 28'6" (84) = 2.4 K

Live Load = 40 psF from DWGs
40 psF (28'6" × ½ (43'-6½"))
= 24.80 K

Curtain Wall =
15 psF vertical surface
= 15 psF (28'6")(14.5' + 14' + 10')
= 16.46 K

Level 58 weight:
Conc. = Same as level 59 but 30 psF SDL
= 47.4 K

Steel Beams:
= 3 × ½ × 43'-6½" × 46 plF (from Dwgs)
= 3.0 K

= 28'6" × 55 plF = 1.6 K

Live Load = Mechanical = 125 psf
(125 psF)(28'6")(½)(43'-6½") = 77.5 K

$P_u = 1.2D + 1.6L$
$= 1.2\,(44.27^k + 4.66^k + 2.4^k + 47.4^k + 16.46^k) + 1.6\,(24.8^k + 77.5^k)$

$= 302.16^k$

$F_y = 50$ ksi
$\ell = 14'\text{-}6"$
$K_x = 1.0$
$r_x/r_y = 2.44$

$K\ell = 14'6"$

From AISC Manual 13th Ed.
Table 4-1

$K\ell$	ϕP_n
14'	637
15'	608

$> 623.5 = \phi P_n > 302.16 = P_u$ ✓ ok

Snow Load Check:

Snow loads:

- Flat roof

$p_f = 0.7 C_e C_t I p_g$

$p_g = 25psf$ From ASCE 07 Figure: 7-1

$C_e = 1.0$ From Table 7-2 Terrain B & Partially exposed

$C_t = 1.0$ From Table 7-3

$I = 1.0$ From Table 7-4 (Category II)

$p_f = 0.7(1.0)(1.0)(1.0)(25) = 17.5psf$

Parapet Curtain wall : Snow drift

7.8 = $h_{d_2} = 0.75 h_d$ (From 7-9)

From figure 7-9 w/ $p_f = 17.5$ & $l_u = 199'6"$
$h_d = 3.75$
$0.75 h_d = 2.8125$

$h_b = p_f / \gamma$

$\gamma = 0.13 p_g + 14$
$\gamma = 0.13(25) + 14 = 17.25$

$h_b = \frac{17.5}{17.25} = 1.01'$

$h_c = 10' - 1.01 = 8.99'$

$\frac{h_c}{h_b} = 8.86 > 0.2 \therefore$ Drift

$p_d = h_d \gamma = 3.75(17.25) = 65psf$

*Note Design Snow Load for flat roof was 25psf > then the 17.5psf calculated. Most likely a reduction factor was not used. A Design Snow Drift Load was not given but referred to ANSI A58.1-1982 Section 7.7 & the CBC, neither of which could be obtained for comparison.