The New York Times Building
New York, NY

IPD/BIM Thesis
Technical Report #3

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Executive Summary

In the third technical report of the New York Times Building, three alternatives to the existing lateral force resisting system were investigated and designed in a preliminary manner. Each one of these designs was developed by each structural student participating in the alternative IPD/BIM Thesis in order to compare the feasibility of three different alternatives to the existing lateral system. The three systems which were investigated are as follows:

- Modified Braced Frame Core w/ outriggers at the 36th floor
- Pure Concrete Shear Wall Core
- Concrete Core w/ outriggers at the 28th and 51st floors

The alternative that was investigated in this report was the concrete core with outriggers system. A modified braced frame core and pure concrete shear wall core were investigated in the technical reports of Erika Bonfanti and Benjamin Barben respectively. Each of the alternatives systems were designed to fall within 10% of the existing period of vibration, 6.75s – 6.25s. Also, an overall building deflection due to wind of H/450, that of the existing structure, was not exceeded by any of the three alternative systems.

The design of the concrete shear wall with outriggers alternative resulted in four 65' long walls in the East/West direction and sixteen 18" returns in the North/South direction. Also, the thickness of the 65' long shear walls decreases from 16" to 14" on the 30th Level. The concrete compressive strength changes from 10,000 psi to 8,000 psi at Level 30, from 8,000 psi to 6,000 psi at Level 40, and then from 6,000 psi back to 8,000 psi at Level 50. This alternative system also utilized W14 braces and W18 beams in the design of the outriggers.

After the three alternative designs were completed, they were presented to the other members of Team 3 in order to determine their feasibility. The modified braced frame system was found to be infeasible because of the design would lead to a single mechanical floor on the outrigger level. A single mechanical floor on the 36th Level would not facilitate the required floors with heating, ventilating, and cooling in an energy efficient manner.

Because the layouts of the two concrete systems are very similar, their feasibility was discussed by the team simultaneously. Though an attempt was made by both designs to conform to the architectural layout of the existing core, it was determined that they do infringe upon the architecture on the First Floor where shear walls were required to be placed into the central corridor of the lobby. Also, the core layouts do not allow for the increase in rentable space provided by the existing lateral system in the Forest City Ratner portion of the tower. Therefore, it was concluded that if a concrete core alternate is to be optimized in the future, an architectural redesign of the core would need to be conducted.

When comparing the two concrete shear wall alternatives, the design which utilized outriggers required smaller shear wall sections than that of the pure concrete core. Therefore, a concrete solution which engages the perimeter columns into the lateral system was found to be the best alternative to the existing lateral force resisting system.

Introduction

The New York Times Headquarters Building (NYTB) is home to the New York Times newsroom and offices, as well as several law firms, whose offices are leased through Forest City Ratner. In collaboration with FXFOWLE Architects, the intent of the Renzo Piano Workshop was to introduce a flagship structure which promoted sustainability, lightness, and transparency. The architectural façade reflects the ever-changing environment surrounding the building, an appropriate acknowledgment of the heart of New York City.

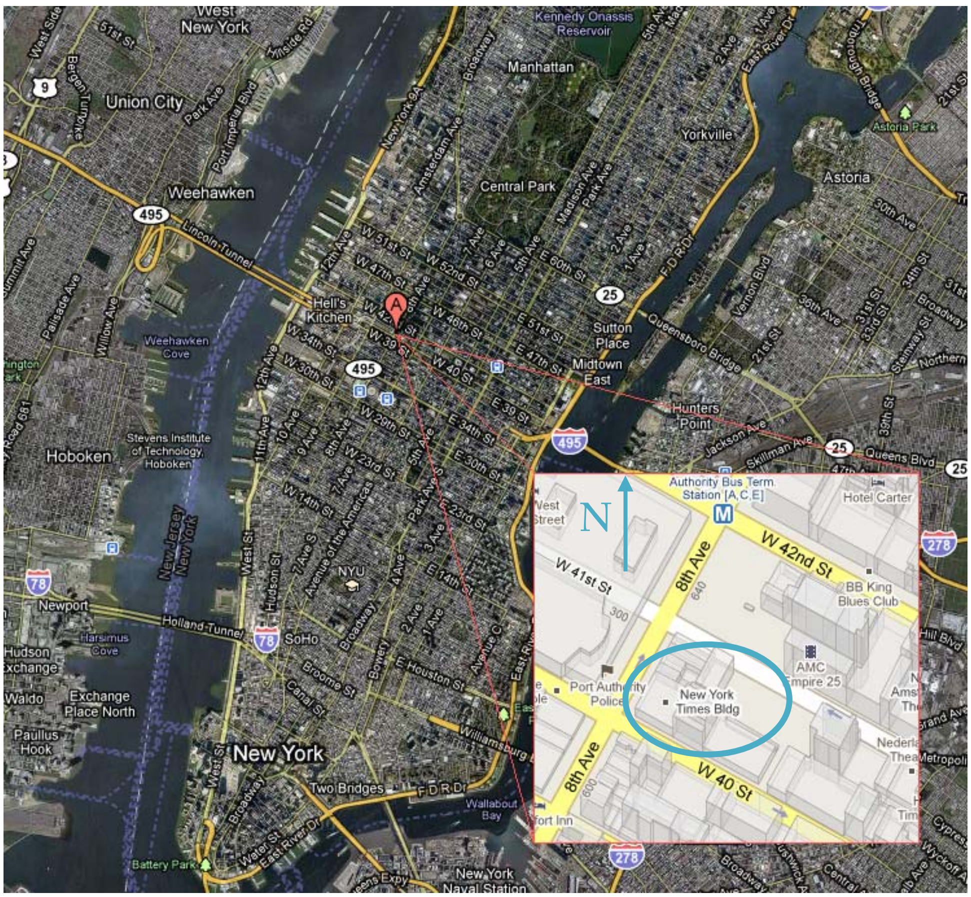


Figure 1: New York Times Building Location (Google Maps)

The 52 story, 1,500,000 square foot building rises 744 feet above Eighth Avenue between 40^{th} and 41^{st} Street creating a 200' x 400' footprint. The tower's 300 foot mast allows for the structure to top out at 1048 feet above ground level. The New York Times occupies the entire five-story podium of the structure, and the first 27 levels in the tower. The additional levels are the office spaces leased through Forest City Ratner. Story heights average approximately 13 feet 9 inches in the tower, lending a great view to the open office plans. At the mechanical floors on levels 28 and 51, however, the floor height is approximately 27 feet to accommodate equipment and steel outriggers which link the perimeter columns to the braced framed core.

The remainder of this report investigates alternatives to the existing tower's lateral force-resisting system. One different preliminary design was developed by each structural student participating in the alternative IPD/BIM Thesis in order to compare the feasibility of three different alternatives to the existing lateral system. The three preliminary designs are as follows:

- Modified Braced Frame Core w/ outriggers at the 36^{th} floor
- Pure Concrete Shear Wall Core
- Concrete Core w/ outriggers at the 28^{th} and 51^{st} floors

The analysis found in this report pertains to the preliminary design of the concrete core with outriggers. Hand calculations, as well as, computer analysis software (ETABS and SAP) were both utilized to perform this preliminary design. For the more detailed analyses on the modified braced frame and the pure concrete core, please refer to the Technical Report 3 of Erika Bonfanti and Benjamin Barben respectively.

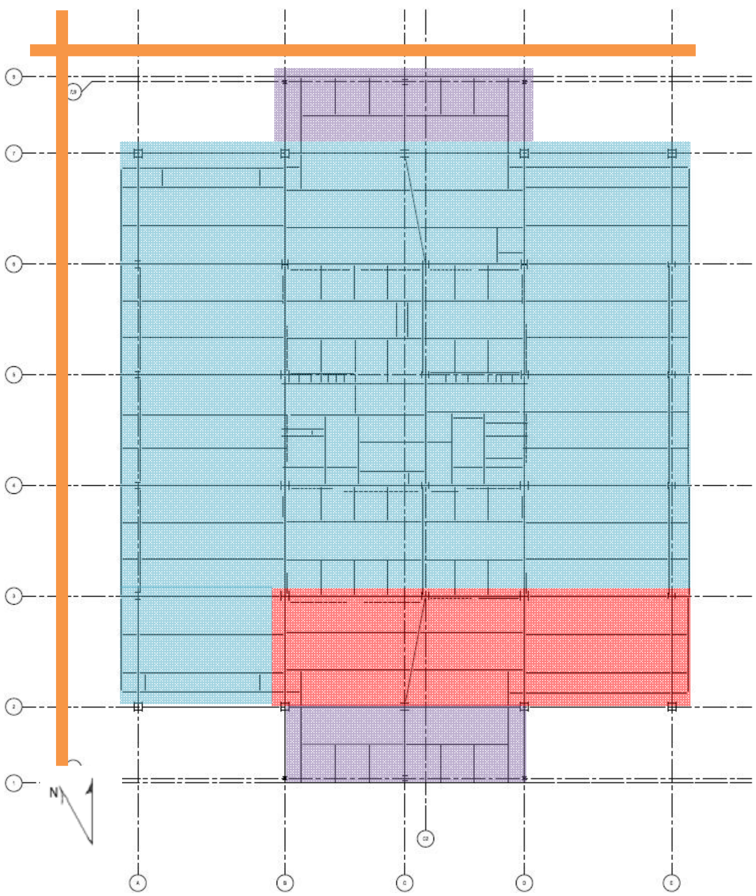
Existing Structural System Description

Foundation

The foundation of the NYTB combines typical spread footings with caissons to achieve its maximum axial capacity. Below the building's 16-foot cellar, the tower and podium mostly bear on Medium/Hard rock with a bearing capacity of 80 ksf., Class 2-65 per the New York City Building Code. However, a core sample taken just before finalizing the site investigation report indicated that rock at the southeast corner of the tower only had a 16 ksf bearing capacity, Class 4-65. At the seven columns that fall within this area, indicated in red on Figure 2, 24-inch diameter concrete-filled steel caissons were used to replace the original foundation designs. Each caisson was designed to support a load of 2,400 kips with 6,000 psi concrete.

Under the other 22 columns (indicated on Figure 2 in teal), spread footings with a concrete compressive strength of 6,000 psi are used to support the loads. The areas depicted in purple represent the two cantilevered sections of the tower. The columns which fall in these areas do not directly transfer load to the ground which removes the need for footings at these locations.

The New York City Subway does pass the north and eastern sides of the New York Times Building. However, this is not a major site restriction since the transit system passes below Eighth Avenue and 41st Street and not directly beneath the structure. But, vibration effects on the foundation and building structure may have had an impact on the design.



Key:
- Assumed Caisson Location
- Assumed Spread Footing Location
- Cantilevered Area
- Subway

Figure 2: Foundation Locations

Columns

The 30" by 30" box columns (Figure 3) at the exterior notches of the tower consist of two 30 inch long flange plates and two web plates inset 3 inches from the exterior of the column on either side. Each web plate decreases in thickness from 7 inches as the column extends up the structure to account for the reduction in axial loads. Each flange plate decreases from 4 inches in thickness to relate to the architectural vision of the tower. Interior columns are a combination of built-up sections and rolled shapes. Column locations stay consistent throughout the height of the building, and every column is engaged in the lateral system. Refer to Figure 4 to view the column locations. Note that the unfilled boxes denote columns in the cantilevered areas which do not extend to the ground.

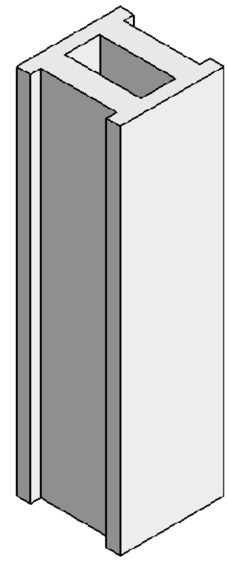


Figure 3: Box Column as Modeled in Revit Structure

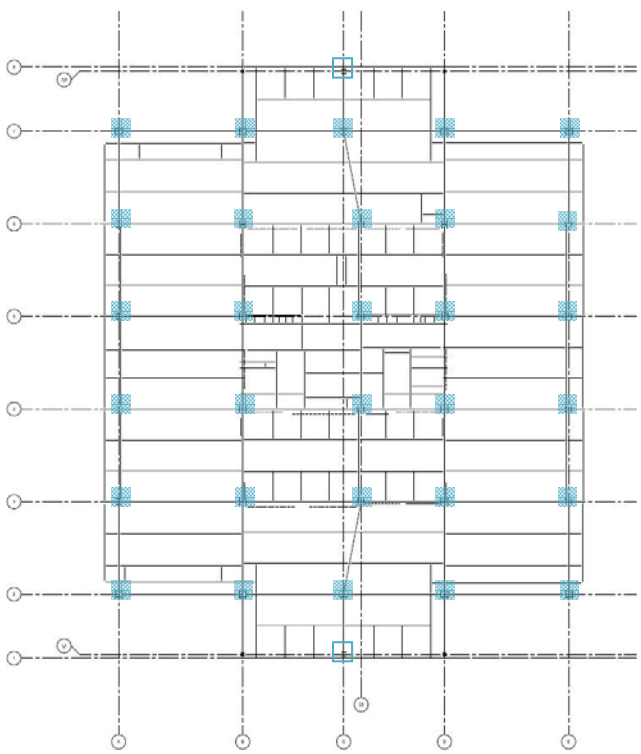


Figure 4: Tower Column Locations

Vierendeel Frame

A Vierendeel frame was used by Thornton Tomasetti as a combined solution at the 20 foot cantilever sections of the tower. Renzo Piano did not want columns obstructing the glass storefronts at the ground level, so these sections were cantilevered from the main structure. As a unique way to control deflections in the middle beams of the cantilevered section, the ladder-like moment frame engages all floors throughout the entire height of the tower. It connects to 28^{th} and 52^{nd} floor outriggers through the use of diagonal braces which effectively transfer loads from the frame to the core of the tower. Refer to Figure 9 on page 10 to view the brace location.

Existing Floor System

The existing floor structure of the NYTB is comprised of a composite steel beam system . The typical bay size is 30'-0"x 40'-0" with 2 ½" normal weight concrete and 3" metal deck, typically spanning 10'-0" from W12x19 to W18x35 infill beams. These infill beams frame into W18x40 girders which in turn, transfer the floor loads to the various build-up columns throughout the structure. The rectangular bays are configured into a cruciform shape around the perimeter of the core. This composite system was selected to reduce the self weight of the structural system which greatly affects member sizes in high rise buildings. By reducing member sizes, the structural system was able to conform to "transparency" desired by the architectural design. Refer to Appendix A to view the typical floor framing plan.

Existing Lateral System

The main lateral load resisting system for the tower of the NYTB consists of a centralized steel braced frame core with outriggers on the two mechanical floors (Levels 28 and 51). The structural core consists of a combination of concentric and eccentric bracing which surrounds elevator shafts, MEP shafts, and stair wells. At this time, the member sizes of these braces have yet to be disclosed. The core configuration remains consistent from the ground level to the 27th floor as shown in Figure 5. But above the 28th floor, the low rise elevators were no longer required. In order to optimize the rentable space on the upper levels of the tower, the number of bracing lines in the North/South direction were reduced from two to one (Figure 6). Refer to Figures 7 and 8 to view the typical core bracing configurations.

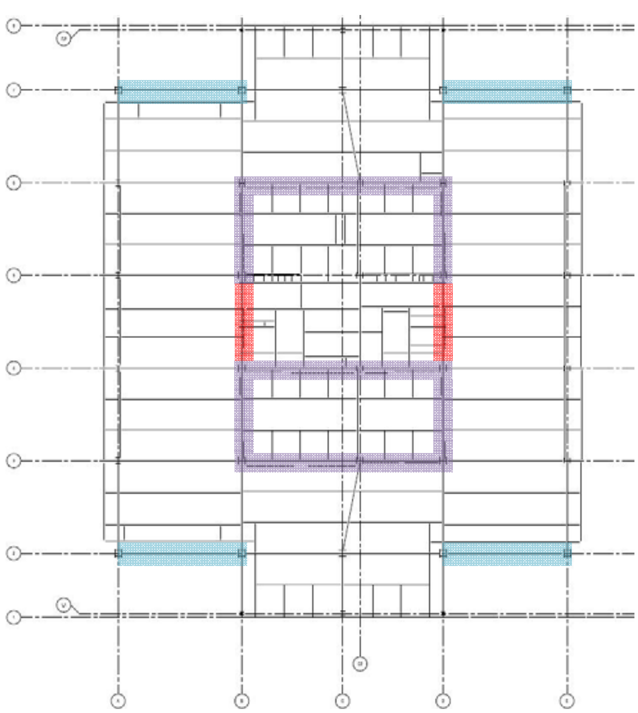


Figure 5: Typical Lateral System (Floors 29-50)

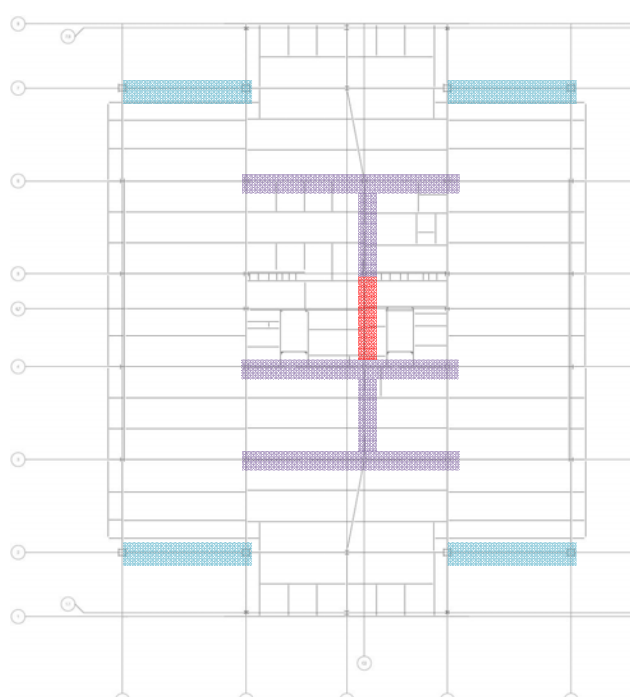


Figure 6: Typical Lateral System (Floors 1-27)

Key:
- Single Diagonal Bracing
- Pre-Tensioned Steel Rod X-Bracing
- Chevron & Eccentric Bracing

The outriggers on the mechanical floors consist of chevron braces (Figure 10) and single diagonal braces. The outrigger system was designed to increase the stiffness of the tower by engaging the perimeter columns into the lateral system. Refer to page 10 to view the framing plans and bracing elevations of the outrigger system.

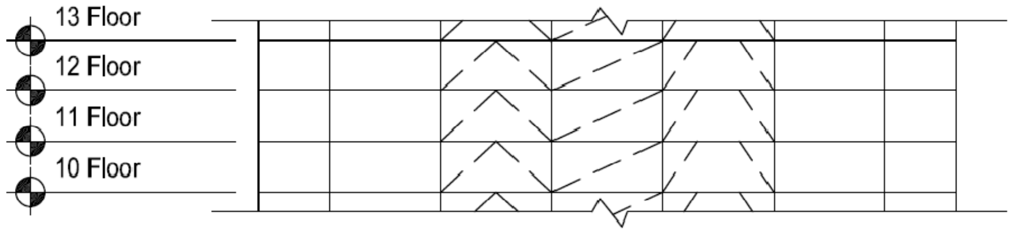


Figure 7: Typical Core N/S Core Bracing Elevation

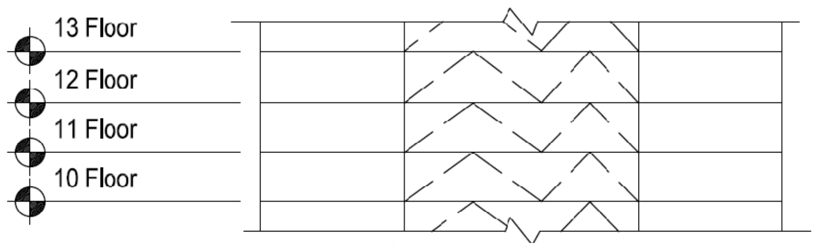


Figure 8: Typical Core E/W Core Bracing Elevation

During the design of the tower, the engineers at Thornton Tomasetti sized the members of the main lateral force resisting system merely for strength. In order to increase stiffness and meet wind deflection criterion, the structural engineers utilized the double story steel rod X-braces (original to Renzo Piano's exterior design) instead of increasing the member sizes of the main lateral force resisting system. These X-braces can be located on Figures 5 and 6 on the previous page. The steel rods transition from 2.5" to 4" in diameter and were prestressed to 210 kips. This induced tensile load prevents the need for large compression members which would not conform to the architectural vision of the exterior.

Although the X-braces did reduce the need for an overall member size increase, the lateral system still did not completely conform to the deflection criterion. Therefore, some of the 30” by 30” base columns were designed as built-up solid sections which reduced the building drift caused by the building overturning moment. After combining these solid base columns and the X-braces with the main lateral force resisting system, the calculated deflection of the tower due to wind was L/450 with a 10 year return period and a building acceleration of less than 0.025g for non-hurricane winds.

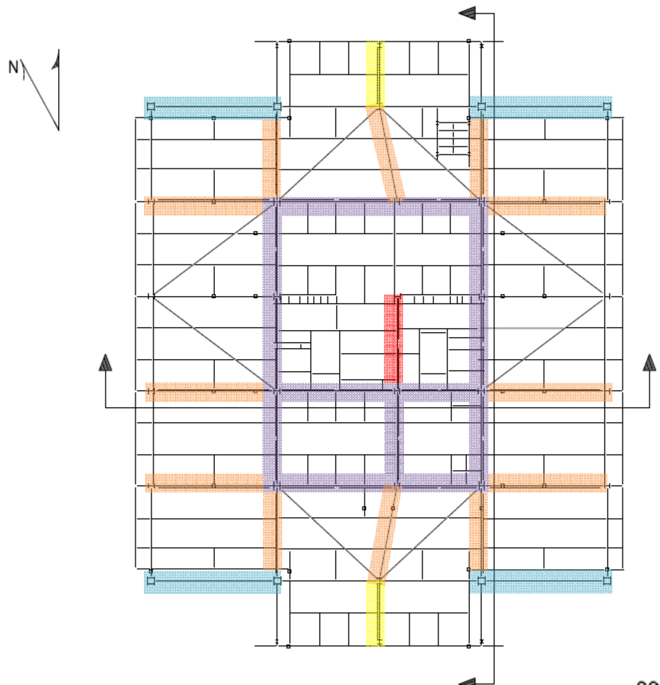


Figure 9: Mechanical Floor Framing Plan (Floors 28 & 51)

Key:
- Single Diagonal Bracing
- Pre-Tensioned Steel Rod X-Bracing
- Chevron & Open Knee Bracing
- Outrigger Bracing
- Single Diagonal Brace at Cantilever

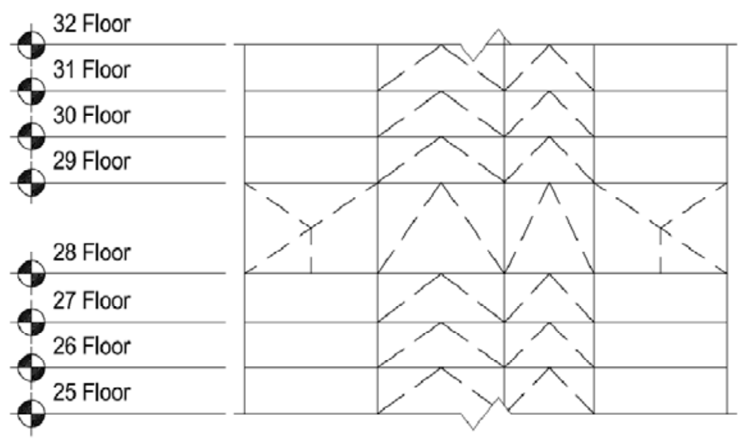


Figure 10: Typical E/W Outrigger Section (28th Floor)

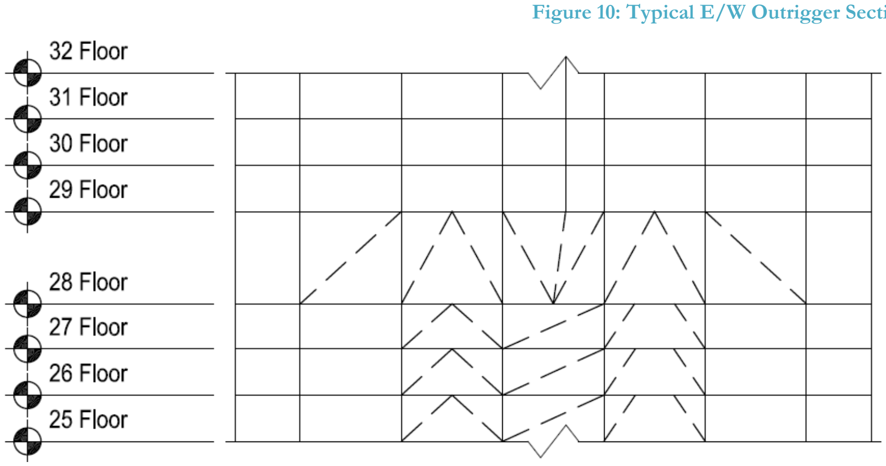


Figure 11: Typical N/S Outrigger Section (28th Floor)

Design Parameters

When investigating the design of alternative lateral force resisting system of the New York Times Building, several parameters were put into place in order to yield comparable results between each alternative as well as to the existing lateral system. Due to the flexible nature of high rise structures, the period of vibration was the first criterion put into place. According for information obtained from the structural design engineer, the period of vibration of the NYTB ranges from 6.75s – 6.25s with the North/South being the more flexible direction. The goal of the three preliminary alternative designs was to maintain a period of vibration within 10% of the existing structure, making the target period of vibration 7.425s – 5.625s.

In addition to period of vibration, the three preliminary alternatives were required to meet a target building deflection due to wind of H/450 which was achieved by the existing design. Story drifts due to wind and seismic were determined and compared to the allowable story drift listed in the drift criterion section. Also, strength requirements per code could be utilized for each alternative to result in a reasonable design. However, strength was not an overall parameter for these preliminary designs. A more in depth strength analysis must be considered if one of these alternative designs is to be optimized.

Design Codes and References

2006 International Building Code

AISC – LRFD, Steel Construction Manual 13th edition, American Institute of Steel Construction

ACI 318 – 08, Building Code Requirements for Structural Concrete, American Concrete Institute

ASCE 7-05, Minimum Design Loads for Buildings and other Structures

Nilson, A. H., Darwin, D., Dolan, C. W., (2004) "Design of Concrete Structures, Thirteenth Edition," McGraw-Hill, New York, NY, 2004.

PCI Design Handbook: Precast and Prestressed Concrete, (1992). "Section 3.7 Shear Wall Buildings", 4th ed.

LRFD Design Load Combinations (ASCE 7-05)

1.4 (D+F)
1.2 (D+F+T) + 1.6 (L+H) + 0.5 (Lr or S or R)
1.2 D + 1.6 (Lr or S or R) + (L or .8W)
1.2 D + 1.6 W + L + .5 (Lr or S or R)
1.2 D + 1.0 E + L + .2S
.9 D + 1.6 W +1.6 H
.9 D + 1.0 E + 1.6 H

D= dead load | Lr= roof live load | W= wind load
E= earthquake load | L= live load | T= self-straining force
R= rain load | S= snow load | F= load due to fluids
H= load due to lateral earth pressure, ground water pressure, or pressure of bulk materials

Note: The controlling load combinations for lateral loads are denoted in bold.

Drift Criterion

Wind:

Load combination for short-term effects: D + 0.5 L + 0.7 W (ASCE 7-05, CC.1.2)
Lateral Deflection Range: H/600 to H/400 (ASCE 7-05, CC.1.2)
Existing Design: H/450 (Thornton Tomasetti)

Seismic (ASCE 7-05):

TABLE 12.12-1 ALLOWABLE STORY DRIFT, Δ_a [a,b]

Structure	Occupancy Category		
	I or II	III	IV
Structures, other than masonry shear wall structures, 4 stories or less with interior walls, partitions, ceilings and exterior wall systems that have been designed to accommodate the story drifts.	$0.025h_{sx}$ [c]	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures [d]	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
All other structures	$0.020h_{sx}$	$\boxed{0.015h_{sx}}$	$0.010h_{sx}$

Note: Occupancy Category taken as Type III because the occupant load for the NYTB is greater than 5000 persons (2006 IBC, Table 1604.5).

Stiffness Modification

When designing reinforced building systems, a reduction in stiffness due to cracking associated with the concrete shear walls must be taken into account. The concrete sections designed in this report assumed 50% of the stiffness values were based on gross section properties. However, the code allows for a 1.4 modifier to be applied when designing for lateral loads resulting from wind.(ACI 318 sections 8.8 & 10.10.4)

Design Loads

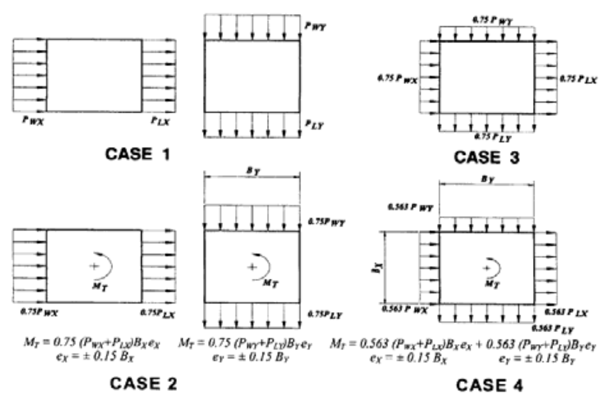
Gravity Loads

The following table is a summary of the typical gravity loads used for this and/or the existing design of the New York Times Building. Other than the live load, the gravity loads were only used to calculate the building mass per story which is required to calculate a structure's period of vibration. Due to the inherent mass of a shear wall core, the shear walls were initially assumed to be 24" thick in order to result in a more accurate period of vibration.

Gravity Loading	Typical Floor	Mechanical Floor	Roof
Curtain Wall	25 psf	25 psf	25 psf
Floor Dead	93 psf	110 psf	100 psf
24" Shear Walls	300 psf	300 psf	---
Live	50 psf +20 psf (Partitions)	150 psf	30 psf + Drift

Please note that at this point in the preliminary design of the alternative lateral force resisting system, the gravity system design was unknown. Therefore, gravity loads were not applied in this design because the amount of load transferred to the shear walls was unable to be determined. The effects due to gravity on the lateral system must be considered once the gravity load paths have been determined.

Wind Loads



The wind pressures used in this for the design for the alternative lateral systems were calculated using Method 2 from ASCE 7-05.Refer to Appendix C to view this calculation. For this preliminary design, the stiffnesses of each shear wall were initially unknown making the load applied due to torsion indeterminable. Therefore, only the Case 1 wind loading was used to perform this preliminary lateral design. The center of rigidity was initially unknown as well. However, once the concrete shear wall core layout was performed, it was determined that the center of rigidity would be at the center of mass due to symmetry. Using this assumption, the applied loads due to each case were determined. A summary of these results can be found on pages 16 - 18. The validity of this assumption was determined after the preliminary design was performed. Also, an investigation on the effects due to Case 2 wind was conducted using ETABS in order to determine if the torsional effects from that loading condition will control the design of any shear walls within the core, refer to page 26. Case 3 and 4 Wind will also need to be considered if this alternative lateral system is to be optimized.

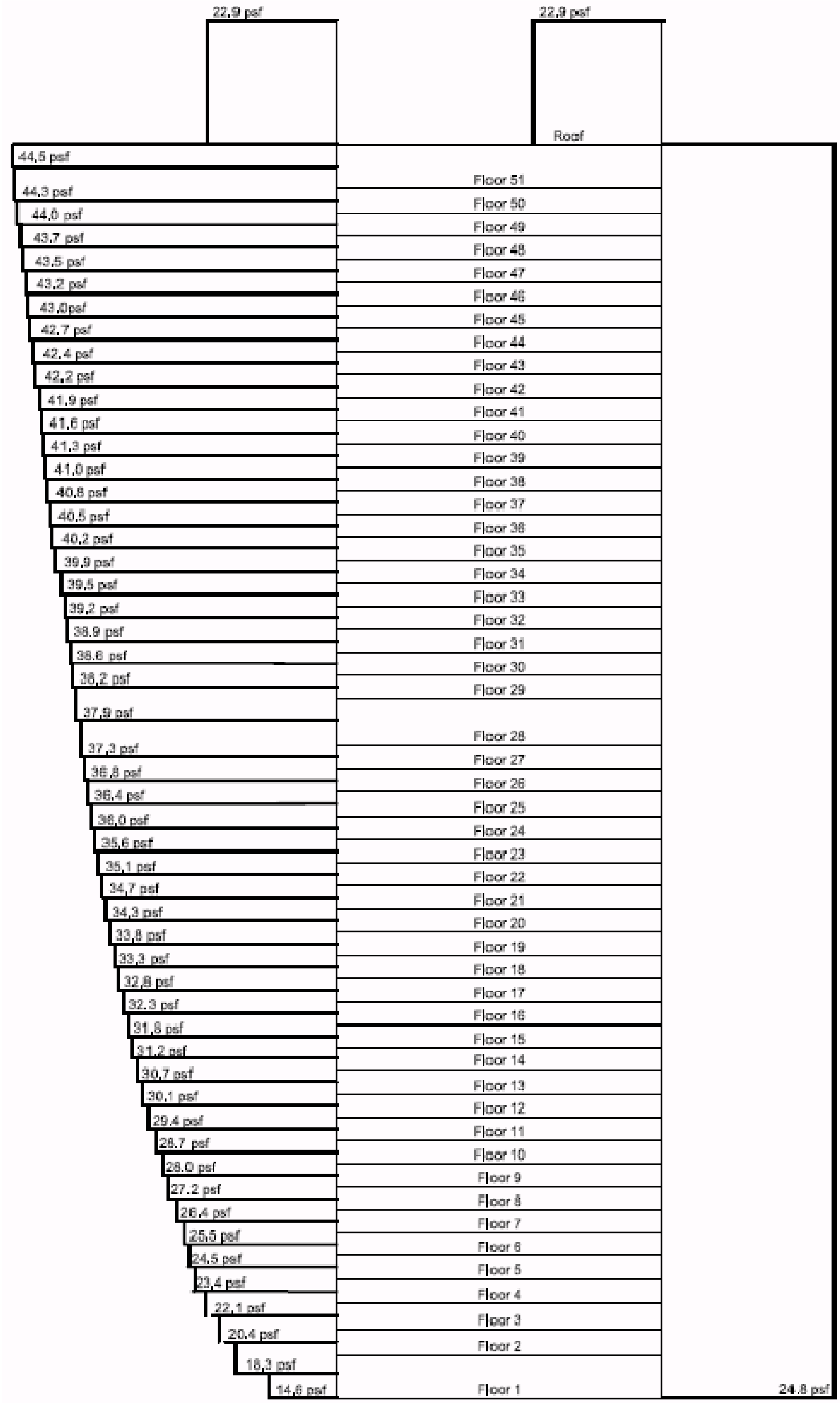


Figure 12: North/South Wind Pressure Diagram

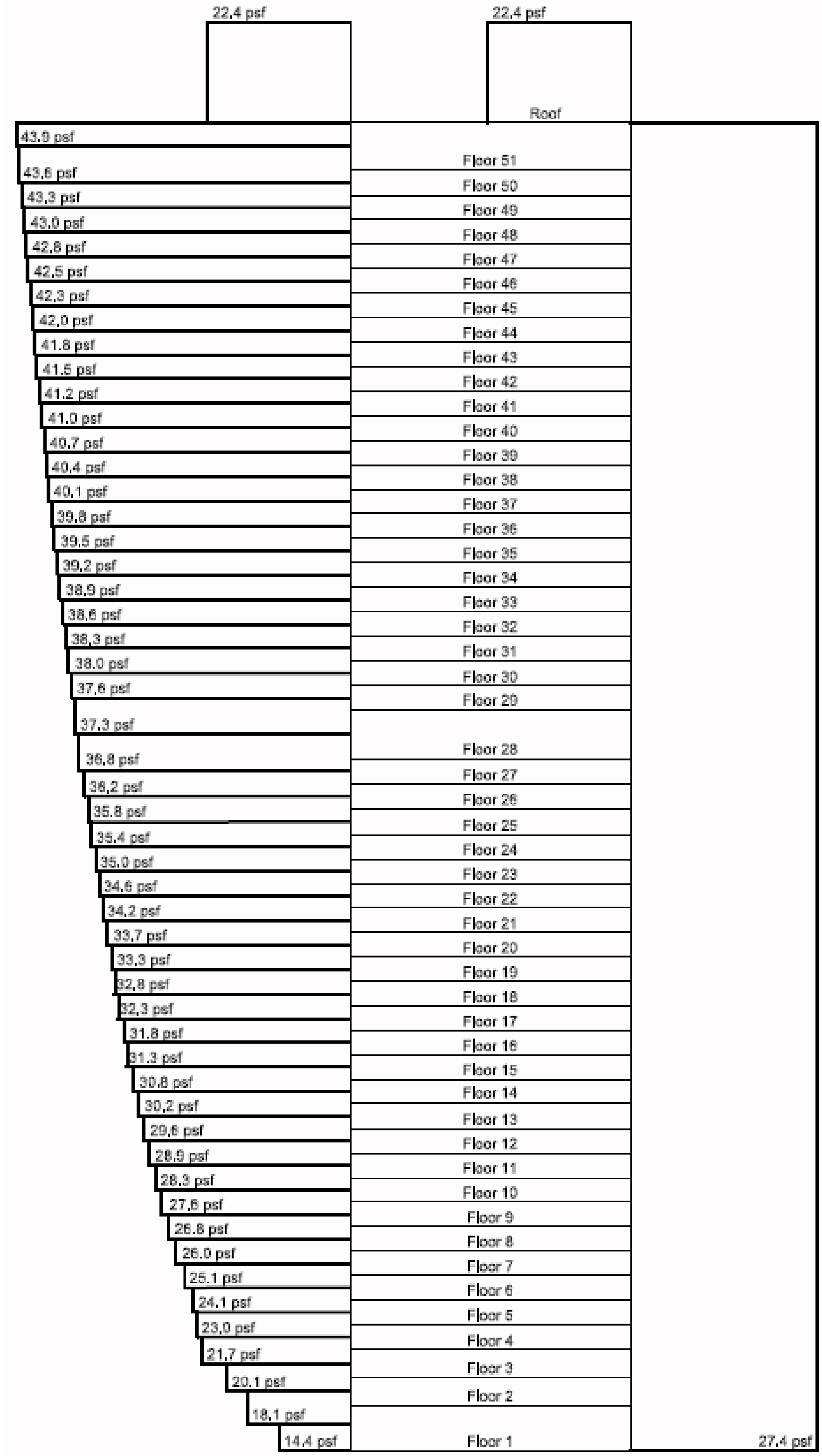


Figure 13: East/West Wind Pressure Diagram

Calculated Wind Forces on Tower (Using Method 2, ASCE 7-05)							
Level	Height Above Ground (ft)	Load (kips)		Shear (kips)		Moment (ft-kips)	
		E/W	N/S	E/W	N/S	E/W	N/S
2	25.66	181	125	9155	7313	3802748	3090052
3	41.13	143	110	9012	7203	3612177	2938076
4	56.59	142	110	8870	7094	3471668	2825801
5	70.92	137	106	8733	6987	3338442	2719288
6	86.00	137	106	8596	6881	3209059	2615791
7	98.42	140	109	8456	6772	3089835	2520375
8	112.17	142	111	8313	6662	2978339	2431095
9	125.92	145	112	8169	6550	2863055	2338734
10	139.67	147	114	8022	6436	2749743	2247905
11	153.42	149	116	7873	6320	2638433	2158633
12	167.17	150	117	7723	6203	2529151	2070938
13	180.92	159	124	7565	6079	2421925	1984843
14	195.83	154	120	7411	5960	2312408	1896856
15	208.42	149	116	7262	5844	2209361	1814018
16	222.17	157	122	7106	5721	2112805	1736347
17	235.92	158	123	6948	5598	2014024	1656837
18	249.67	159	124	6788	5474	1917406	1579015
19	263.42	161	126	6628	5348	1822969	1502898
20	277.17	162	127	6466	5221	1730733	1428499
21	290.92	163	128	6303	5094	1640714	1355834
22	304.67	164	129	6138	4965	1552930	1284917
23	318.42	165	129	5973	4836	1467397	1215760
24	332.17	167	130	5807	4705	1384130	1148377
25	345.92	168	131	5639	4574	1303145	1082780
26	359.67	169	132	5470	4442	1224457	1018982
27	373.42	175	137	5296	4305	1148081	956995
28	388.00	262	205	5034	4100	1071859	895063
29	415.50	259	203	4775	3897	964032	807299
30	429.25	173	136	4601	3761	861993	724137
31	443.00	174	137	4427	3624	797532	671492
32	456.75	175	138	4252	3486	735462	620723
33	470.50	176	138	4076	3348	675796	571841
34	484.25	177	139	3899	3209	618546	524855
35	498.00	178	140	3721	3069	563723	479775
36	511.75	179	140	3542	2929	511338	436609
37	525.50	179	141	3363	2788	461403	395369
38	539.25	180	142	3183	2647	413929	356061
39	553.00	181	142	3002	2504	368927	318696
40	566.75	182	143	2820	2362	326407	283282
41	580.50	182	143	2638	2218	286379	249828
42	594.25	183	144	2455	2074	248854	218341
43	608.00	184	145	2271	1930	213841	188831
44	621.75	185	145	2086	1784	181352	161304
45	635.50	185	146	1901	1639	151395	135771
46	649.25	186	146	1715	1492	123980	112237
47	663.00	187	147	1529	1345	99116	90711
48	676.75	187	147	1342	1198	76813	71201
49	690.50	188	148	1154	1050	57080	53714
50	704.25	193	152	961	898	39926	38257
51	718.67	284	224	676	674	25071	24564
Roof	745.50	676	674	0	0	0	0
Screen *	802 & 819	491	528	---	---	---	---
Total		9336	7438	9336	7438	3922512	3185465

* Loads from the screens are superimposed on to the Roof level.

Load Case 1						
Level	E/W			N/S		
	P (kips)	e (ft)	M_t (kip-ft)	P (kips)	e (ft)	M_t (kip-ft)
2	181.35	0	0	124.64	0	0
3	142.66	0	0	109.95	0	0
4	141.97	0	0	109.66	0	0
5	137.24	0	0	106.18	0	0
6	137.36	0	0	106.41	0	0
7	139.98	0	0	108.56	0	0
8	142.37	0	0	110.53	0	0
9	144.57	0	0	112.33	0	0
10	146.61	0	0	114.01	0	0
11	148.51	0	0	115.58	0	0
12	150.31	0	0	117.05	0	0
13	158.52	0	0	123.53	0	0
14	153.68	0	0	119.82	0	0
15	148.56	0	0	115.89	0	0
16	156.61	0	0	122.23	0	0
17	158.01	0	0	123.38	0	0
18	159.36	0	0	124.49	0	0
19	160.65	0	0	125.56	0	0
20	161.90	0	0	126.58	0	0
21	163.11	0	0	127.58	0	0
22	164.28	0	0	128.54	0	0
23	165.41	0	0	129.47	0	0
24	166.51	0	0	130.37	0	0
25	167.58	0	0	131.25	0	0
26	168.61	0	0	132.10	0	0
27	174.80	0	0	136.99	0	0
28	261.91	0	0	205.34	0	0
29	258.84	0	0	203.02	0	0
30	173.48	0	0	136.10	0	0
31	174.37	0	0	136.83	0	0
32	175.25	0	0	137.55	0	0
33	176.10	0	0	138.26	0	0
34	176.94	0	0	138.95	0	0
35	177.76	0	0	139.62	0	0
36	178.57	0	0	140.29	0	0
37	179.36	0	0	140.94	0	0
38	180.14	0	0	141.57	0	0
39	180.90	0	0	142.20	0	0
40	181.65	0	0	142.82	0	0
41	182.39	0	0	143.42	0	0
42	183.11	0	0	144.02	0	0
43	183.83	0	0	144.61	0	0
44	184.53	0	0	145.18	0	0
45	185.22	0	0	145.75	0	0
46	185.90	0	0	146.31	0	0
47	186.57	0	0	146.86	0	0
48	187.23	0	0	147.41	0	0
49	187.88	0	0	147.94	0	0
50	193.11	0	0	152.08	0	0
51	284.23	0	0	223.89	0	0
Roof	676.30	0	0	674.18	0	0

Load Case 2						
Level	E/W			N/S		
	P (kips)	+/- e (ft)	M_t (kip-ft)	P (kips)	+/- e (ft)	M_t (kip-ft)
2	136.01	29.1	3957.895	93.48	23.55	2201.3785
3	107.00	29.1	3113.593	82.46	23.55	1942.0258
4	106.48	29.1	3098.57	82.24	23.55	1936.8247
5	102.93	29.1	2995.323	79.63	23.55	1875.3259
6	103.02	29.1	2997.91	79.81	23.55	1879.4436
7	104.99	29.1	3055.112	81.42	23.55	1917.4996
8	106.78	29.1	3107.2	82.89	23.55	1952.1538
9	108.42	29.1	3155.151	84.25	23.55	1984.0557
10	109.95	29.1	3199.676	85.51	23.55	2013.6779
11	111.39	29.1	3241.309	86.68	23.55	2041.3761
12	112.73	29.1	3280.464	87.79	23.55	2067.4254
13	118.89	29.1	3459.799	92.65	23.55	2181.8555
14	115.26	29.1	3354.034	89.87	23.55	2116.3713
15	111.42	29.1	3242.377	86.92	23.55	2046.9671
16	117.46	29.1	3417.977	91.67	23.55	2158.9122
17	118.51	29.1	3448.586	92.54	23.55	2179.276
18	119.52	29.1	3477.98	93.37	23.55	2198.8319
19	120.49	29.1	3506.269	94.17	23.55	2217.6528
20	121.43	29.1	3533.549	94.94	23.55	2235.8017
21	122.33	29.1	3559.901	95.68	23.55	2253.3335
22	123.21	29.1	3585.397	96.40	23.55	2270.2962
23	124.06	29.1	3610.102	97.10	23.55	2286.7323
24	124.88	29.1	3634.072	97.78	23.55	2302.6793
25	125.68	29.1	3657.357	98.44	23.55	2318.1709
26	126.46	29.1	3680.003	99.08	23.55	2333.237
27	131.10	29.1	3814.912	102.74	23.55	2419.505
28	196.43	29.1	5716.21	154.01	23.55	3626.9037
29	194.13	29.1	5649.257	152.27	23.55	3585.8461
30	130.11	29.1	3786.129	102.07	23.55	2403.8419
31	130.78	29.1	3805.642	102.63	23.55	2416.8239
32	131.43	29.1	3824.734	103.16	23.55	2429.5256
33	132.08	29.1	3843.425	103.69	23.55	2441.961
34	132.71	29.1	3861.736	104.21	23.55	2454.1431
35	133.32	29.1	3879.684	104.72	23.55	2466.0839
36	133.93	29.1	3897.287	105.21	23.55	2477.7947
37	134.52	29.1	3914.559	105.70	23.55	2489.2858
38	135.10	29.1	3931.515	106.18	23.55	2500.5667
39	135.68	29.1	3948.169	106.65	23.55	2511.6466
40	136.24	29.1	3964.534	107.11	23.55	2522.5338
41	136.79	29.1	3980.62	107.57	23.55	2533.2361
42	137.33	29.1	3996.44	108.02	23.55	2543.761
43	137.87	29.1	4012.004	108.46	23.55	2554.1153
44	138.40	29.1	4027.32	108.89	23.55	2564.3055
45	138.91	29.1	4042.4	109.31	23.55	2574.3376
46	139.42	29.1	4057.25	109.73	23.55	2584.2176
47	139.93	29.1	4071.88	110.15	23.55	2593.9508
48	140.42	29.1	4086.297	110.55	23.55	2603.5422
49	140.91	29.1	4100.508	110.96	23.55	2612.9969
50	144.83	29.1	4214.612	114.06	23.55	2686.1205
51	213.17	29.1	6203.267	167.92	23.55	3954.4265
Roof	507.22	29.1	14760.18	505.63	23.55	11907.645

Load Case 3							
Level	E/W			N/S			Total
	P (kips)	+/- e (ft)	M_t (kip-ft)	P (kips)	+/- e (ft)	M_t (kip-ft)	M_t (kip-ft)
2	136.01	0	0	93.48	0	0	0
3	107.00	0	0	82.46	0	0	0
4	106.48	0	0	82.24	0	0	0
5	102.93	0	0	79.63	0	0	0
6	103.02	0	0	79.81	0	0	0
7	104.99	0	0	81.42	0	0	0
8	106.78	0	0	82.89	0	0	0
9	108.42	0	0	84.25	0	0	0
10	109.95	0	0	85.51	0	0	0
11	111.39	0	0	86.68	0	0	0
12	112.73	0	0	87.79	0	0	0
13	118.89	0	0	92.65	0	0	0
14	115.26	0	0	89.87	0	0	0
15	111.42	0	0	86.92	0	0	0
16	117.46	0	0	91.67	0	0	0
17	118.51	0	0	92.54	0	0	0
18	119.52	0	0	93.37	0	0	0
19	120.49	0	0	94.17	0	0	0
20	121.43	0	0	94.94	0	0	0
21	122.33	0	0	95.68	0	0	0
22	123.21	0	0	96.40	0	0	0
23	124.06	0	0	97.10	0	0	0
24	124.88	0	0	97.78	0	0	0
25	125.68	0	0	98.44	0	0	0
26	126.46	0	0	99.08	0	0	0
27	131.10	0	0	102.74	0	0	0
28	196.43	0	0	154.01	0	0	0
29	194.13	0	0	152.27	0	0	0
30	130.11	0	0	102.07	0	0	0
31	130.78	0	0	102.63	0	0	0
32	131.43	0	0	103.16	0	0	0
33	132.08	0	0	103.69	0	0	0
34	132.71	0	0	104.21	0	0	0
35	133.32	0	0	104.72	0	0	0
36	133.93	0	0	105.21	0	0	0
37	134.52	0	0	105.70	0	0	0
38	135.10	0	0	106.18	0	0	0
39	135.68	0	0	106.65	0	0	0
40	136.24	0	0	107.11	0	0	0
41	136.79	0	0	107.57	0	0	0
42	137.33	0	0	108.02	0	0	0
43	137.87	0	0	108.46	0	0	0
44	138.40	0	0	108.89	0	0	0
45	138.91	0	0	109.31	0	0	0
46	139.42	0	0	109.73	0	0	0
47	139.93	0	0	110.15	0	0	0
48	140.42	0	0	110.55	0	0	0
49	140.91	0	0	110.96	0	0	0
50	144.83	0	0	114.06	0	0	0
51	213.17	0	0	167.92	0	0	0
Roof	507.22	0	0	505.63	0	0	0

Load Case 4							
Level	E/W			N/S			Total
	P (kips)	+/- e (ft)	M_t (kip-ft)	P (kips)	+/- e (ft)	M_t (kip-ft)	M_t (kip-ft)
2	102.10	29.1	2971.06	70.17	23.55	1652.501	4623.561
3	80.32	29.1	2337.271	61.90	23.55	1457.814	3795.085
4	79.93	29.1	2325.993	61.74	23.55	1453.91	3779.903
5	77.27	29.1	2248.489	59.78	23.55	1407.745	3656.234
6	77.33	29.1	2250.431	59.91	23.55	1410.836	3661.267
7	78.81	29.1	2293.37	61.12	23.55	1439.403	3732.773
8	80.15	29.1	2332.471	62.23	23.55	1465.417	3797.888
9	81.39	29.1	2368.467	63.24	23.55	1489.364	3857.831
10	82.54	29.1	2401.89	64.19	23.55	1511.601	3913.491
11	83.61	29.1	2433.143	65.07	23.55	1532.393	3965.536
12	84.62	29.1	2462.535	65.90	23.55	1551.947	4014.482
13	89.25	29.1	2597.156	69.55	23.55	1637.846	4235.002
14	86.52	29.1	2517.761	67.46	23.55	1588.689	4106.451
15	83.64	29.1	2433.944	65.25	23.55	1536.59	3970.534
16	88.17	29.1	2565.761	68.82	23.55	1620.623	4186.385
17	88.96	29.1	2588.738	69.47	23.55	1635.91	4224.648
18	89.72	29.1	2610.804	70.09	23.55	1650.59	4261.393
19	90.45	29.1	2632.04	70.69	23.55	1664.718	4296.758
20	91.15	29.1	2652.517	71.27	23.55	1678.342	4330.859
21	91.83	29.1	2672.299	71.83	23.55	1691.502	4363.801
22	92.49	29.1	2691.438	72.37	23.55	1704.236	4395.674
23	93.13	29.1	2709.983	72.89	23.55	1716.574	4426.557
24	93.74	29.1	2727.977	73.40	23.55	1728.545	4456.521
25	94.35	29.1	2745.456	73.89	23.55	1740.174	4485.63
26	94.93	29.1	2762.456	74.37	23.55	1751.483	4513.939
27	98.41	29.1	2863.728	77.12	23.55	1816.242	4679.969
28	147.46	29.1	4290.969	115.61	23.55	2722.596	7013.564
29	145.73	29.1	4240.709	114.30	23.55	2691.775	6932.484
30	97.67	29.1	2842.121	76.62	23.55	1804.484	4646.605
31	98.17	29.1	2856.769	77.04	23.55	1814.229	4670.998
32	98.66	29.1	2871.1	77.44	23.55	1823.764	4694.864
33	99.15	29.1	2885.131	77.84	23.55	1833.099	4718.23
34	99.62	29.1	2898.877	78.23	23.55	1842.243	4741.12
35	100.08	29.1	2912.35	78.61	23.55	1851.207	4763.557
36	100.53	29.1	2925.563	78.98	23.55	1859.998	4785.561
37	100.98	29.1	2938.529	79.35	23.55	1868.624	4807.153
38	101.42	29.1	2951.257	79.71	23.55	1877.092	4828.349
39	101.85	29.1	2963.759	80.06	23.55	1885.409	4849.168
40	102.27	29.1	2976.043	80.41	23.55	1893.582	4869.625
41	102.68	29.1	2988.119	80.75	23.55	1901.616	4889.735
42	103.09	29.1	2999.994	81.08	23.55	1909.517	4909.511
43	103.49	29.1	3011.677	81.41	23.55	1917.289	4928.967
44	103.89	29.1	3023.175	81.74	23.55	1924.939	4948.114
45	104.28	29.1	3034.495	82.06	23.55	1932.469	4966.964
46	104.66	29.1	3045.642	82.37	23.55	1939.886	4985.528
47	105.04	29.1	3056.625	82.68	23.55	1947.192	5003.817
48	105.41	29.1	3067.447	82.99	23.55	1954.392	5021.839
49	105.78	29.1	3078.115	83.29	23.55	1961.49	5039.604
50	108.72	29.1	3163.769	85.62	23.55	2016.381	5180.15
51	160.02	29.1	4656.586	126.05	23.55	2968.456	7625.042
Roof	380.76	29.1	11079.97	379.56	23.55	8938.672	20018.65

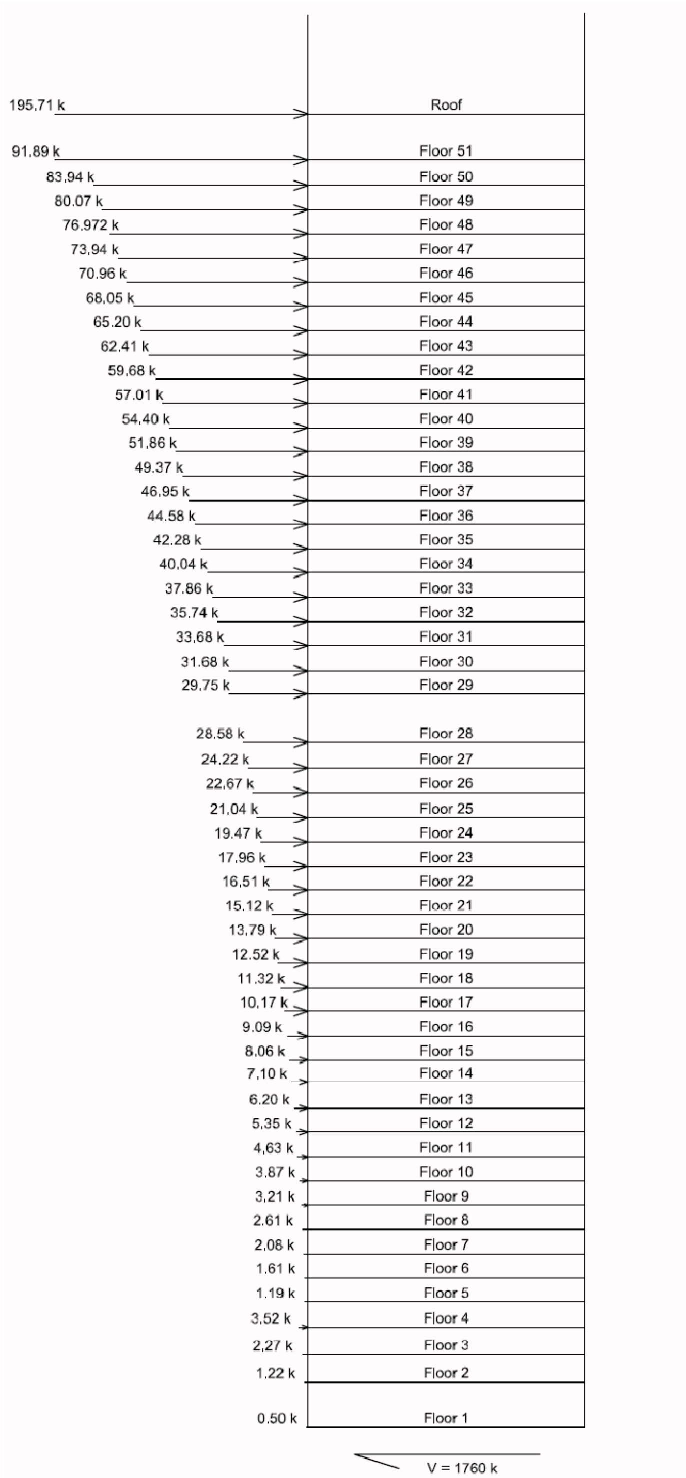


Figure 14: Seismic Equivalent Lateral Force Diagram

Seismic Loads

The seismic loads utilized for this preliminary design were calculated for the existing structure in the Technical Report #1 according to the Equivalent Lateral Force Method found in ASCE 7-05. Please note that the period of vibration of the existing structure, 6.75 seconds, was used in the calculation of this report. The weight of the existing building was also used for the calculation of the seismic base shear. If the alternative design is to be optimized, the actual period and weight of the alternative design will have to be used to recalculate the seismic base shear. The diagram to the left provides a summary of the applied seismic loads for this preliminary design. Refer to Appendix D to view the seismic load calculations.

Alternative Lateral System Design (Concrete Core w/ Outriggers)

The alternative to the existing lateral force-resisting system of the New York Times Building designed in this report was a concrete shear wall system with steel outriggers at the 28th and 51st levels. The design resulted in a core layout with four 65'shear walls in the East/West direction as well as twelve 10' returns and four 20' shear walls in the North/South direction. This layout was intended to minimize the impact to the existing architecture by constraining the shear walls to the elevator shafts. Please note that shear walls 2, 3, 14 and 15 had to be extended away from the elevator shafts in order to stiffen the structure in the North/South direction. In order to result in a realistic design, the thickness and f'c of the shear walls change throughout the height of the building, refer to the table to the right.

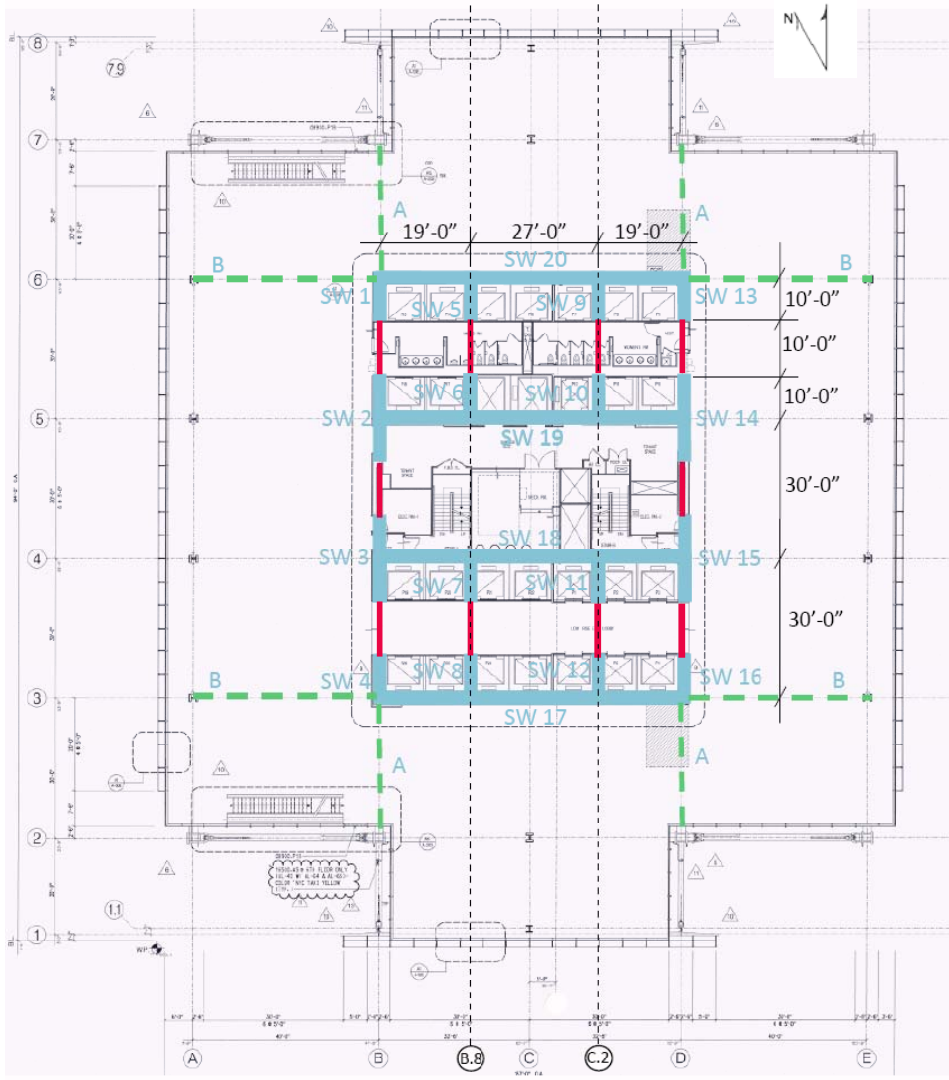


Figure 15: Concrete Core w/ Outriggers Layout

Four outriggers in each direction, depicted on the plan in green, were added to both mechanical floors in order to reduce the concrete section from that of a pure concrete core. To view the outrigger sizes and configurations please refer to the preliminary outrigger discussion on page 23 .In addition to the outriggers, ten 18"x42" concrete coupling beams, depicted in red, were added at each level in order to prevent an overly flexible structure in the North/South direction. Please note

Wall	Level Range	f'c	t (in)	l (in)
SW 1, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13 & 16	Base - 15	10000	18	120
	15 - 30	10000	18	120
	30 - 40	8000	18	120
	40 - 50	6000	18	120
	50 - 52	8000	18	120
SW 2, 3, 14 & 15	Base - 15	10000	18	240
	15 - 30	10000	18	240
	30 - 40	8000	18	240
	40 - 50	6000	18	240
	50 - 52	8000	18	240
SW 17, 18, 19 & 20	Base - 15	10000	16	65
	15 - 30	10000	16	65
	30 - 40	8000	14	65
	40 - 50	6000	14	65
	50 - 52	8000	14	65

Period of Vibration			
Load	Mode	Direction	T (sec)
Wind	1	N/S	6.44
	2	E/W	5.69
	3	Tors.	4.57
Seismic	1	N/S	6.97
	2	E/W	6.23
	3	Tors.	4.88

Building Drift		
Load	Direction	Dist (in)
Wind (Case 1)	N/S	16.119
	E/W	16.856
Seismic	N/S	8.974
	E/W	8.162

that these beams were sized based upon the existing core floor plenum, an average of 4 feet, and the return wall thicknesses. The coupling beams strength was not considered in this preliminary design. However, the strength of the coupling beams must be considered if this alternative system is to be investigated further. A summary of the resulting period of vibration and building drifts due to the preliminary design loading are reported in the tables on the previous page. To view elevations of this design, refer to Appendix B.

Design Assumptions

Several simplifying assumptions were made for the preliminary design of the concrete shear wall core with outriggers. First off, the center of mass, pressure, and rigidity of the structure were assumed to align with the center of geometry due to the symmetry associated to the core configuration. Also, the shear walls were assumed to be continuous throughout their entire height. However, mechanical penetrations and door openings have a negative effect on the strength of shear walls and will have to be considered for a more optimized design. It was also assumed that core configuration was uniform throughout the entire building height. This will result in impacts on the architectural layout of the core on most floors above the 28th Level. This impact must be investigated further if the design is to be optimized.

Initial Sizing of Shear Walls

As mentioned previously, structural analysis/design software was utilized for the preliminary design of the alternative lateral systems. However, rough strength and deflection calculations were conducted in order to determine the lower level shear wall thicknesses to be used for the initial model. After comparing the both factored and un-factored lateral loads, it was assumed that the loading due Case 1 wind would control over the seismic loading for both strength and serviceability. Therefore, Case 1 wind was used for these rough calculations. Also the shear walls were assumed to have a uniform f'c of 12,000 psi. Please note that these calculations do not take into account the effects due to the outriggers.

Shear

Required thickness due to shear was the first calculation to be performed. All walls in each direction were assumed to carry the shear loading equally. The strength equation utilized was:

$$V_u \leq \phi 4(f'c)^{0.5} A_{cw}$$

The resulting required thicknesses were 15" for the 65' walls in the East/West direction and the 18" for the walls in the North/South direction. Refer to Appendix E to view this calculation.

Overall Wind Drift

The limitation of H/450 for wind drift was the next parameter utilized to roughly calculate the required wall thicknesses. The allowable wind deflection (19.88" for the New York Times Building) was back figured to determine a total building moment of inertia about the North/South axis. The moment of inertia due to the sixteen returns with the thickness of 18", determined from the rough required wall thickness for shear, were then subtracted from the total building moment of inertia to obtain the required moment of inertia needed for the 65' long walls. After finding this moment of inertia, the thickness of the 65' long walls required to meet the allowable drift could be determined.

In order to conduct this calculation, several assumptions needed to be made. First of all, due to the height of the structure, 745.5 feet, deflection would be controlled by flexural deformations; shear deflections could be considered to be negligible. The moment of inertia and the elastic modulus were also assumed to be uniform thorough out the height of the NYTB. Also, effects from the outriggers were negated for this initial size calculation. Lastly, the wind loads were assumed to be applied at the center of geometry which would align with the centroid of the core section. Based on these assumptions, the structure could be treated as a simple cantilever with several point loads though out its length. The following equation was then utilized to perform the calculation of overall total moment of inertia:

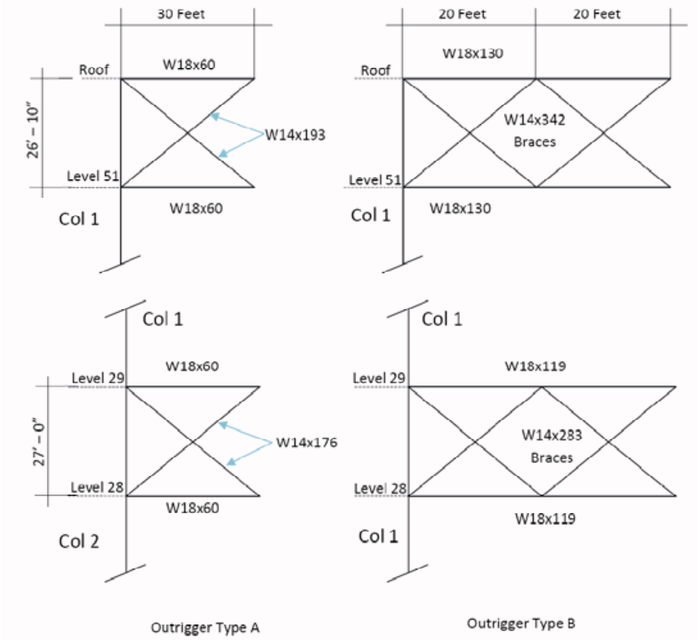
$$I_{Total} = \frac{\Sigma[0.7P_i h_i^2(3H-h_i)]}{[(6)(1.4)(0.5)E(H/450)]}$$

This equation considers the D + 0.5 L + 0.7 W load combination. However, the gravity loads were not considered for this calculation. Also, stiffness modifiers were applied in order to account for a cracked concrete section. Please note that this relationship could only be used about the North/South axis for loads applied in the East/West direction. This relationship could not be considered for loads in the North/South direction because the coupling beams cannot treated as part of a solid section. This calculation resulted in a rough thickness of 17" for each 65' wall. Refer to Appendix E for a more detailed calculation.

Moment Capacity

A shear wall flexural strength check was a third calculation conducted before a structural modeling program was utilized. As with the rough drift calculation, the moment of inertia of the concrete core about the North/South axis was utilized to determine a rough relative stiffness of each of the 65' walls. As stated previously, the height of the New York Times Building causes the building deflection to be dominated by flexural deformations resulting in the deflection to be proportional to the moment of inertia. Because stiffness and deflection are proportional, it can be correlated that the stiffness of the shear walls in the East/West direction are proportional to their moment of inertia about the North/South axis. Therefore, relative stiffness of each shear wall in the East/West direction could be roughly calculated by determining the percentage of the moment of inertia accounted for each shear wall individually. After relative stiffnesses were calculated, they were multiplied by the factored overturning moment due to Case 1 wind in order to determine a rough flexural loading required to be carried by the 65' walls. After performing a flexural design check on the 65' foot walls due to this loading, it was determined that a 17" could be designed to carry the required loading. To review this initial flexural capacity calculation, refer to Appendix E. Please note that as with the total building drift, this calculation could not be utilized for loads for wind running in North/South direction because the coupling beams cannot be treated as part of a solid section.

Outrigger Design



Col	Dim	tf	tw
1	30"x30"	4"	4"
2	30"x30"	4"	4.75"

Figure 16: Unit Load Application (SAP)

A two-dimensional frame analysis in SAP 2000 was performed in order to size the outriggers. Before the analysis could be performed, some assumed member sizes were utilized as a base. First, the columns used were the same 30"x30" dimension as the existing columns. Flange and web thicknesses were of similar thickness to the box columns of the existing columns as well. The beams were of the same 18" depth as those used in the existing structure. Also, all members assumed a yield strength of 50 ksi. Using these size parameters, the outrigger configurations pictured above, as well as a 388' column, base to 28th floor, and a 358' column, 28th to roof, were modeled in SAP. In order for the outriggers to be considered to work efficiently, the outriggers and their respective columns should have equal stiffness. To achieve this, unit loads were applied to the columns and outriggers as shown in Figure 16. For the stiffnesses to be the same, the axial deformation on the columns must be equal to the vertical displacement of the outriggers. Element sizes were then modified for each outrigger configuration until the resulting displacements were essentially equal. The final members sizes used for this preliminary design are pictured above. Please note that the outriggers for this design were not sized for strength. If this alternative to the lateral system is to be optimized, strength must be considered in the design.

Displacement due to Unit Load			
Upper OR Type A		Upper OR Type B	
Col	0.000328	Col	0.000328
OR	0.000325	OR	0.000328
Lower OR Type A		Lower OR Type B	
Col	0.000358	Col	0.000386
OR	0.000356	OR	0.000383

ETABS Model

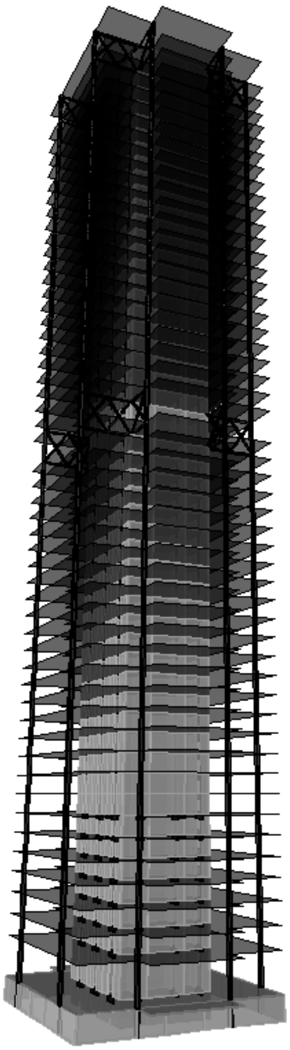


Figure 17: ETABS Model

Once the initial sizes of the shear walls and outriggers were determined through the implementation of rough hand calculations and a 2-D frame analysis, a three dimensional structural model could then be produced using ETABS. The outriggers were modeled based upon the results found though the SAP analysis. All returns were initially modeled with an 18" thickness while the 65' long walls were modeled with a 17" thickness. However, it was known that concrete core with a uniform concrete compressive strength of 12,000 psi throughout its entire height would be an irrational design. Therefore, the compressive strength was lowered to 10,000 psi at level 15, then to 8,000 psi at level 30, and finally to 6,000 psi at level 40. It was assumed that the outriggers would cause more load to be transferred back into the core at the upper levels. Therefore, the concrete compressive strength was increased back up to 8,000 psi at level 50 and remained so until the core reached the roof. In addition to the lateral system, a 20" perimeter basement wall with 4,000 psi concrete was modeled in order to replicate a realistic building response at the base.

After utilizing the assumption of a rigid diaphragm for all floors, the following six load cases were applied to the center of pressure or center of mass correspondingly:

1.6 W (E/W Direction)
1.6 W (N/S Direction)
0.7W (E/W Direction)
0.7W (N/S Direction)
1.0E (E/W Direction)
1.0E (N/S Direction)

Once a working model was developed, an iterative process went underway to modify the model until the design fell within 10% of the target period of vibration, 6.75s – 6.25s, as well as complying with the allowable building drifts due to Case 1 wind and seismic loadings.

Results

Once the alternative design was determined to meet the set criterion of this preliminary design, an investigation was performed to determine if the shear walls were capable in meeting the required shear and flexural strengths. The following page reports the ETABS output of the shear walls at the Base Level, Level 15, 28, 29, 30 , 40,50, and 51 due to Case 1 wind and seismic. Though observation, it could be determined that as assumed, Case 1 wind controlled over seismic. Spot checks were preformed for the loadings boxed in red. Other than Shear Wall 19 at Level 28, all walls were found to meet the required strength. A more in depth strength design will have to be conducted if this system is to be optimized. To view the spot check calculations, refer to Appendix F.

Final Shear Wall Results From ETABS

Direction	Shear Wall	Base				Level 15				Level 28				Level 29			
		Wind (Case 1)		Seismic		Wind (Case 1)		Seismic		Wind (Case 1)		Seismic		Wind (Case 1)		Seismic	
		Shear (k)	Moment (k-ft)	Shear (k)	Moment (k-ft)	Shear (k)	Moment (k-ft)	Shear (k)	Moment (k-ft)	Shear (k)	Moment (k-ft)	Shear (k)	Moment (k-ft)	Shear (k)	Moment (k-ft)	Shear (k)	Moment (k-ft)
N/S	1	499.54	7084.22	-9.48	16.75	344.53	1984.56	2.97	56.10	-175.30	434.61	72.39	789.21	361.75	5133.77	58.94	874.72
	2	1363.58	43494.24	6.68	196.84	1256.94	8939.46	30.90	215.67	-719.03	-10986.38	9.63	124.27	712.68	14942.90	36.09	293.35
	3	1363.58	43494.24	13.03	304.29	1256.94	8939.46	32.90	229.52	-719.03	-10986.39	4.23	50.90	712.68	14942.90	23.63	213.10
	4	499.54	7084.22	26.95	91.94	344.53	1984.56	29.83	103.56	-175.30	434.60	-72.14	-796.61	361.76	5133.76	-35.35	-792.25
	5	443.80	6271.85	-3.11	-4.05	338.44	1877.56	1.15	18.99	-264.51	-2093.58	18.80	53.13	251.82	2403.62	20.14	59.76
	6	502.12	6414.86	5.68	29.57	396.41	2015.65	11.01	40.83	-333.14	-2193.78	-3.36	12.06	201.66	2341.83	0.74	36.14
	7	502.12	6414.86	-2.83	4.65	396.41	2015.65	1.56	22.22	-333.14	-2193.79	6.96	18.01	201.66	2341.82	11.27	28.99
	8	443.80	6271.85	9.31	48.83	338.44	1877.56	12.28	45.66	-264.51	-2093.58	-17.32	-27.18	251.82	2403.64	-9.80	2.92
	9	443.80	6271.85	3.11	4.05	338.44	1877.56	-1.15	-18.99	-264.51	-2093.64	-18.81	-53.08	251.81	2403.58	-20.10	-59.53
	10	502.12	6414.86	-5.68	-29.57	396.41	2015.65	-11.01	-40.83	-333.14	-2193.79	3.35	-12.08	201.67	2341.88	-0.72	-36.05
	11	502.12	6414.86	2.83	-4.65	396.41	2015.65	-1.56	-22.22	-333.14	-2193.79	-6.97	-18.02	201.67	2341.87	-11.24	-28.89
	12	443.80	6271.85	-9.31	-48.83	338.44	1877.56	-12.28	-45.66	-264.51	-2093.58	17.32	27.16	251.83	2403.69	9.82	-2.82
	13	499.54	7084.22	9.48	-16.75	344.53	1984.56	-2.97	-56.09	-175.31	434.18	-72.40	-788.78	362.06	5142.79	-59.16	-883.53
	14	1363.58	43494.24	-6.68	-196.84	1256.94	8939.47	-30.90	-215.67	-719.05	-10986.37	-9.65	-124.29	712.73	14943.37	-36.04	-292.76
	15	1363.58	43494.24	-13.03	-304.29	1256.94	8939.47	-32.90	-229.52	-719.04	-10986.39	-4.25	-50.90	712.73	14943.44	-23.58	-212.60
	16	499.54	7084.22	-26.95	-91.94	344.53	1984.56	-29.83	-103.56	-175.31	434.60	72.13	796.59	361.79	5133.90	35.37	792.35
E/W	17	3557.01	375615.02	99.86	3997.04	2400.89	109302.05	77.16	275.40	-2058.03	-635.46	4.68	-230.82	4061.98	122065.80	65.55	507.75
	18	3792.94	363789.40	28.27	1392.89	3405.55	111327.58	1.71	71.31	-7596.84	-31120.06	-32.46	-280.54	-394.78	38146.49	-9.54	51.04
	19	3792.94	363789.46	-28.27	-1392.84	3405.54	111327.83	-1.71	-71.26	-7596.99	-31115.81	32.45	280.74	-394.02	38153.99	9.59	-50.67
	20	3557.01	375615.21	-99.86	-3996.98	2400.88	109302.69	-77.16	-275.35	-2060.85	-566.02	-4.74	231.98	4059.44	122245.24	-65.60	-504.25

Direction	Shear Wall	Level 30				Level 40				Level 50				Level 51			
		Wind (Case 1)		Seismic		Wind (Case 1)		Seismic		Wind (Case 1)		Seismic		Wind (Case 1)		Seismic	
		Shear (k)	Moment (k-ft)	Shear (k)	Moment (k-ft)	Shear (k)	Moment (k-ft)	Shear (k)	Moment (k-ft)	Shear (k)	Moment (k-ft)	Shear (k)	Moment (k-ft)	Shear (k)	Moment (k-ft)	Shear (k)	Moment (k-ft)
N/S	1	188.81	1524.43	-18.94	-67.97	135.58	611.59	1.35	30.11	68.57	-771.80	19.14	89.11	-79.66	-392.94	48.76	487.79
	2	783.20	9449.69	28.66	223.10	513.26	2392.83	18.45	117.09	177.85	-6580.79	17.50	113.36	-397.78	-9425.77	12.17	141.23
	3	783.20	9449.69	20.43	144.17	513.26	2392.83	19.66	125.65	177.85	-6580.79	8.17	48.76	-397.78	-9425.77	4.85	55.06
	4	188.82	1524.43	45.89	199.95	135.58	611.58	17.95	62.12	68.57	-771.80	-8.63	-28.59	-79.66	-392.94	-42.40	-431.42
	5	222.21	1545.07	8.49	45.16	133.38	593.12	0.59	12.42	66.08	-814.14	10.38	34.04	-127.09	-1634.74	15.43	45.00
	6	245.58	1598.52	4.13	25.56	158.67	637.03	6.46	21.71	35.32	-874.62	-2.27	9.91	-181.79	-1725.31	-2.24	16.26
	7	245.58	1598.51	5.47	22.71	158.67	637.02	1.05	14.89	35.32	-874.61	7.73	16.56	-181.79	-1725.31	5.85	9.50
	8	222.21	1545.07	0.71	1.57	133.38	593.12	7.24	24.50	66.08	-814.13	-5.97	-9.64	-127.09	-1634.74	-12.72	-20.86
	9	222.22	1545.10	-8.53	-45.28	133.38	593.13	-0.59	-12.43	66.08	-814.14	-10.38	-34.03	-127.09	-1634.75	-15.42	-44.99
	10	245.58	1598.52	-4.13	-25.61	158.67	637.03	-6.46	-21.72	35.32	-874.62	2.27	-9.91	-181.79	-1725.32	2.24	-16.25
	11	245.58	1598.51	-5.48	-22.75	158.67	637.03	-1.05	-14.89	35.32	-874.62	-7.73	-16.56	-181.79	-1725.32	-5.85	-9.49
	12	222.21	1545.07	-0.72	-1.60	133.38	593.12	-7.24	-24.50	66.08	-814.14	5.97	9.64	-127.09	-1634.74	12.72	20.87
	13	188.81	1524.46	19.05	68.83	135.58	611.59	-1.35	-30.12	68.57	-771.81	-19.14	-89.10	-79.67	-392.99	-48.76	-487.74
	14	783.21	9449.65	-28.69	-223.35	513.26	2392.83	-18.45	-117.10	177.85	-6580.82	-17.50	-113.34	-397.79	-9425.87	-12.16	-141.18
	15	783.20	9449.62	-20.44	-144.32	513.26	2392.83	-19.66	-125.65	177.85	-6580.82	-8.17	-48.75	-397.79	-9425.87	-4.85	-55.03
	16	188.82	1524.45	-45.89	-199.98	135.58	611.59	-17.95	-62.12	68.57	-771.80	8.63	28.59	-79.66	-392.95	42.40	431.43
E/W	17	2397.87	67752.55	68.60	437.68	966.34	-12007.88	43.52	-321.74	1858.21	-65036.40	5.08	-306.57	-650.35	-34928.12	1.77	-190.85
	18	1132.52	38756.51	-0.78	57.28	1285.08	-11010.85	-0.22	-130.89	-1147.04	-38083.74	-11.71	-121.37	-4163.19	-35822.38	-10.05	-74.26
	19	1132.76	38756.27	0.80	-57.12	1285.05	-11012.72	0.22	130.82	-1147.07	-38085.04	11.71	121.31	-4163.32	-35823.77	10.04	74.19
	20	2397.71	67738.27	-68.62	-437.92	966.31	-12013.23	-43.52	321.67	1858.18	-65040.77	-5.08	306.49	-650.70	-34931.24	-1.78	190.79

Drift and Deflection

As mentioned, one of the overall parameters for the alternatives to the existing lateral system was for the structure to achieve the same H/450 wind drift as the existing New York Times Building. Story drifts, at several levels of interest, due to both wind and seismic were also checked for h/450 and code compliance respectively. After reviewing the ETABS output, all drift were found to comply with their corresponding limitations. Please note that the D + 0.5 L + 0.7 W load combination was applied for wind drift while no load modification was implemented for seismic drift. Also, stiffness modifiers were applied as mentioned previously.

Building Drift From Wind (Case 1)			
Direction	Dist (in)	H/450 (in)	Compliance ?
N/S	16.119	19.88	ok
E/W	16.856	19.88	ok

Story Drift Check									
Level	h_{sx} (ft)	Seismic				Wind			
		$0.015*h_{sx}$ (in)	Story Drift from ETABS (in)		Compliance ?	h/450 (in)	Story Drift from ETABS (in)		Compliance ?
			E/W	N/S			E/W	N/S	
2	25.66	0.3848	0.00964	0.00059	ok	0.0570	0.00625	0.01568	ok
15	12.58	0.1888	0.00986	0.00119	ok	0.0280	0.02326	0.02639	ok
28	14.58	0.2188	0.00117	0.00096	ok	0.0324	0.02539	0.01970	ok
29	27.50	0.4125	0.00107	0.00080	ok	0.0611	0.02318	0.01684	ok
30	13.75	0.2063	0.00097	0.00096	ok	0.0306	0.02549	0.01909	ok
40	13.75	0.2063	0.00107	0.00119	ok	0.0306	0.02687	0.02189	ok
50	13.75	0.2063	0.00100	0.00077	ok	0.0306	0.02393	0.01530	ok
51	14.42	0.2162	0.00027	0.00067	ok	0.0320	0.02360	0.01372	ok

Wind Case 2

As stated, Wind Case 1 was used to perform the preliminary design of this alternative lateral system. However, once the design was completed to a reasonable point due for the scope of this analysis, an investigation was performed in ETABS to examine the effects due to Wind Case 2. Upon reviewing the ETABS output, it was determined that the torsional effects from the Case 2 loading would control the design for several of the shear walls throughout the height of the structure. If this alternative to the existing lateral system is to be further optimized, the effects due to Case 2 wind load will have to be taken into account. To view the shear wall loadings from the ETABS output, refer to Appendix G.

Model Verification

Relative Stiffness Comparison

Hand Calculations

Relative Stiffness About N/S Axis

Wall		SW 17 or 20 w/ Returns			SW 18 or 19 w/ Returns		
		Interior Returns	Exterior Returns	SW 17 or 20	Interior Returns	Exterior Returns	SW 18 or 19
b (in)		104	104	16	104	224	16
h (in)		18	18	780	18	18	780
A (in^2)	b*h	1872	1872	12480	1872	4032	12480
I_i (in^4)	$bh^3/12$	50544	50544	632736000	50544	108864	632736000
d (in)		390	162	0	390	162	0
N		2	2	1	2	2	1
I (in^4)	$\Sigma(I_i + Ad^2)$	5.6956.E+08	9.8359.E+07	6.3274.E+08	5.6956.E+08	2.1185.E+08	6.3274.E+08
Total I (in^4)				1.3007.E+09			1.4141.E+09
					Overall I	Σ(Total I)	5.4296.E+09
					% = I / Σ(Total I)		
					Relative Stiffness (%)	SW17 w/ R	0.2395
						SW18 w/ R	0.2605
						SW19 w/ R	0.2605
						SW20 w/ R	0.2395

Relative Stiffness About E/W Axis

Wall		120" Returns	240" Returns
b (in)		18	18
h (in)		120	240
A (in^2)	b*h	2160	4320
I_i (in^4)	$bh^3/12$	2592000	20736000
N		12	4
Total I (in^4)		31104000	82944000
Overall I		Σ(Total I)	1.1405.E+08
% = I / Σ(Total I)			
Relative Stiffness (%)		SW1	0.0227
		SW2	0.1818
		SW3	0.1818
		SW4	0.0227
		SW5	0.0227
		SW6	0.0227
		SW7	0.0227
		SW8	0.0227
		SW9	0.0227
		SW10	0.0227
		SW11	0.0227
		SW12	0.0227
		SW13	0.0227
		SW14	0.1818
		SW15	0.1818
		SW16	0.0227

In order to determine the validity of the ETABS model, a relative stiffness comparison between hand calculations and the ETABS output was performed. As stated previously, the height of the NYTB causes flexural deformations to control the lateral deflection over shear deformations. Based on this fact, stiffness can then be considered to be proportional to the moment of inertia. Therefore, the moment of inertia was taken about the North/South axis to determine the relative stiffness of the shear walls in the East/West direction. This hand calculation was also performed for each shear wall individually about the East/West axis. However, a calculation about this axis was assumed to be inaccurate because it would not take into account any effects from the coupling beams.

Level 1

	Wall	Shear from ETABS	Relative Stiffness (%)
Stiffness from N/S Loading	SW 1	42.22	0.0422
	SW 2	114.91	0.1149
	SW 3	114.91	0.1149
	SW 4	42.22	0.0422
	SW 5	36.41	0.0364
	SW 6	41.79	0.0418
	SW 7	41.79	0.0418
	SW 8	36.41	0.0364
	SW 9	36.41	0.0364
	SW 10	41.79	0.0418
	SW 11	41.79	0.0418
	SW 12	36.41	0.0364
	SW 13	42.22	0.0422
	SW 14	114.91	0.1149
	SW 15	114.91	0.1149
	SW 16	42.22	0.0422
	Total V_y	1000	
Stiffness from E/W Loading	SW 17	242.3	0.2423
	SW 18	246.84	0.2468
	SW 19	246.84	0.2468
	SW 20	242.3	0.2423
	Total V_x	1000	

In order to find the relative stiffness in ETABS, a 1000 k load was placed in both the North/South and East/West directions. The relative stiffness was then calculated at Level 1 by calculating the percent total shear carried by each wall. After comparing these relative stiffnesses to the hand calculated relative stiffnesses from the moment of inertia about the North/South axis, it was determined that because the relative stiffnesses were fairly close to each other, the model could be considered to be accurate. The comparison between the relative stiffnesses of the walls in the North/South direction also confirmed the assumption that the hand calculated relative stiffnesses for that direction would be inaccurate. The comparison can be viewed in the tables above and to the right.

Center of Rigidity

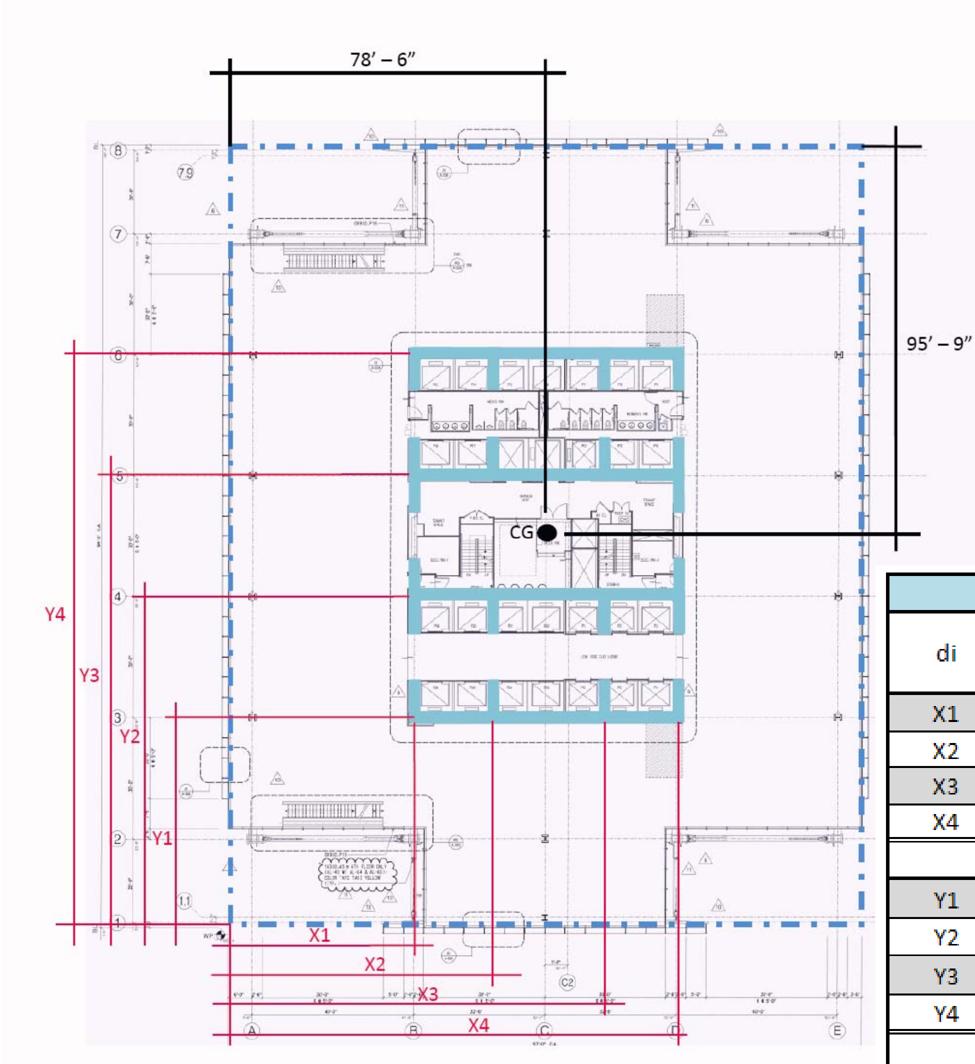


Figure 18: Center of Rigidity (di)

Center of Rigidity				
di	Distance (ft)	Shear Walls of Influence	ki/Ki	ki*di/Ki
X1	46	1, 2, 3, 4	0.33385	15.35712
X2	65	5, 6, 7, 8	0.16615	10.79973
X3	92	9, 10, 11, 12	0.16615	15.28577
X4	111	13, 14, 15, 16	0.33385	37.05739
		X (ft)	Σki*d/ΣKi	78.5
Y1	50.75	20	0.2477	12.56974
Y2	80.75	19	0.2523	20.37487
Y3	110.75	18	0.2523	27.94448
Y4	140.75	17	0.2477	34.8609
		Y (ft)	Σki*d/ΣKi	95.75

As stated previously, the center of rigidity of this alternative lateral system was assumed to align with the center of geometry, CG, and the center of mass due to the symmetry of the tower. Using the first floor relative stiffnesses calculated from the ETABS output, a hand calculation was performed using the relationship:

$$COR = \Sigma ki*di/\Sigma Ki$$

This investigation verified that the initial assumption was valid. Refer to the figure and table above to view this calculation. The center of rigidity and center of mass reported in the ETABS output also coincided with this hand calculation and the initial assumption.

Concrete Shear Wall Core Design Summary

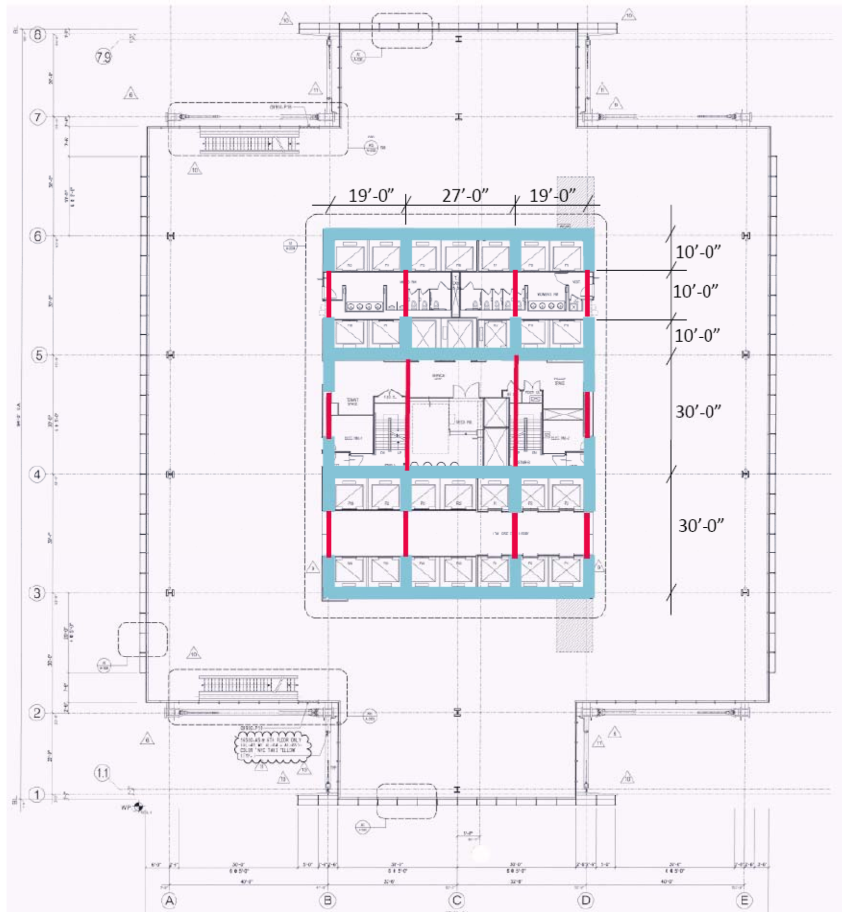


Figure 19: Pure Concrete Core Layout

A second alternative to the existing lateral system of the New York Times building was a sole concrete shear wall core system as pictured in the Figure 19. As with the concrete core and outrigger system, the core was configured to coincide with the existing architectural layout as much as possible. In the North/South direction, the core is comprised of twelve 10'-0" returns and four 20'-0" returns. The North/South direction is also tied together with ten 10'-0" and two 30'-0" 30"x36" coupling beams. The coupling beam dimensions, the returns sizes, and layout depicted above remain constant throughout the entire height of the building. Conversely, the compressive strength and wall thickness for the 65'-0" long walls in the East/West direction are modified at several heights throughout the structure. The alternative system utilizes 12,000 psi concrete from the basement to the tenth floor, 10,000 psi concrete from the eleventh to the thirtieth floor, and 8,000 psi concrete from the thirty-first to the roof. The 65' long shear walls begins at the basement with a 2'-6" thickness. At the twenty-first story, the thickness is reduced to 2'-0" and modified a final time at the forty-first level to a thickness of 1'-6". The periods of vibration due to seismic for this alternative were found to be 7.709s in the East/West direction, 6.893s in the North/South direction, and 3.265s in the torisonal direction. The overall lateral displacements due to the seismic loading were 5.44" in the East/West direction and 7.45" in the North/South direction. The periods of vibration due to wind were found to be 6.528s in the East/West direction, 5.926s in the North/South direction, and 3.265 second in the torisonal direction. The overall lateral displacements due to a Case 1 wind loading were 16.75" in the North/South direction, and 10.76" in the East/West direction. In order to review the preliminary design of this alternative to the existing lateral system, refer to the Technical Report 3 authored by Benjamin Barben.

Modified Braced Frame Core Design Summary

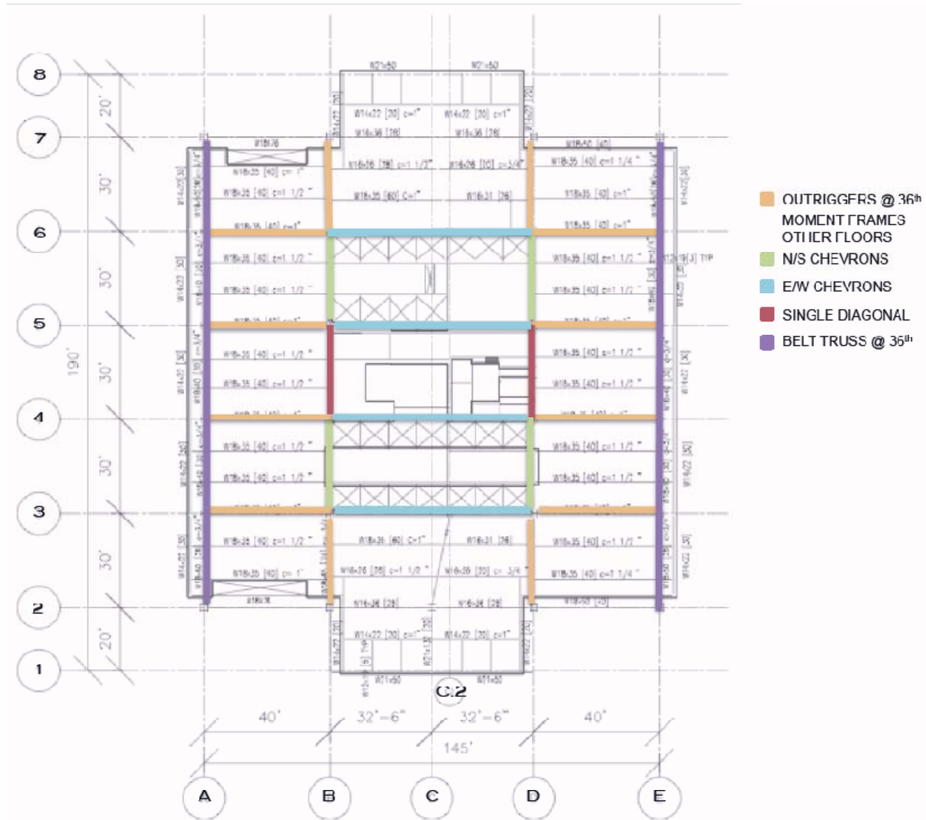


Figure 20: Modified Braced Frame Core Layout

The third alternative lateral system investigated for the New York Times Building was a modified version of the existing lateral system. .As with the original design, this option utilizes a steel braced frame core with outriggers. However, instead of placing outriggers on the 28th and 51st mechanical floors, the alternative system was designed with a single level of W36x247outriggers on the 36th floor with two belt trusses on the East and West edges of the level, depicted on Figure 20 in purple.

The core configuration of this alternative lateral consists mostly of chevron braces. However, single diagonal braces, shown in red, were utilized were the chevron braces would not conform to the existing architectural layout of the core. Though the core configuration remains uniform throughout, member sizes did change with the height of the building. W14x283 braces were used from the base to the thirteenth floor, while W14x136 braces were used form the fourteenth to the twenty-seventh. The braces were changed again to HSS 16x16x 1/2 at the twenty-eighth floor and a final time to HSS 12x12x 3/8 at the forty-first floor. The column sizes of this alternative design were changed at these three levels as well. At the base of the structure, both flanges of the 30"x30" box columns had a thickness of 7 inches and both webs had a thickness of 4 inches. The flange thickness decreases by an inch at each column change while the web thickness decrease by half an inch. Moment frames were added to all levels, except the 36th floor, in order to increase the stiffness of the structure. The resulting period of vibrations for this alternative design were 5.26s in the North/South direction, 5.17s in the East/West direction, and 3.92s in the torsional direction. The overall building drift due to Case 1 wind was 16.7" in the North/South direction, and 19.8" in the East/West direction. In order to review the preliminary design of this alternative to the existing lateral system, refer to the Technical Report 3 authored by Erika Bonfanti.

IPD/BIM Team Comparison

Once the three preliminary alternatives to the existing lateral system of the New York Times Building were completed, they were brought before the other members of the IPD/BIM Team 3 to determine their feasibility for future optimization. The first concern was with the modified braced frame core alternative. It was determined that if the outriggers were to be placed on the 36th Level, the floor would not be able to be used as optimal rentable space for Forest City Ratner. Therefore, the only possible use for the level would be a mechanical floor. This presents an issue because a single mechanical floor would not be capable of distributing heating and cooling to the required locations in an energy efficient manner. Due to this fact, Team 3 found that the modified braced frame core with outriggers on the 36th floor would be an unfeasible design and should not be investigated further.

The main concerns presented by the two concrete alternatives were very similar. The group found that both alternatives would require an architectural redesign of the existing core configuration in order to optimize a concrete solution and provide an equal amount of functional space in the core. Concerns about duct work not being able to pass thought the elevator lobbies due to the depth of the coupling beams were also expressed. One of the major concerns with both alternative designs was that the four returns which extend away in to the central corridor on the entrance level, would greatly infringe the architectural vision of transparency. This architectural issue can be seen in Figure 21 where the area in blue represents one of the returns which would negatively influence the architecture of the central corridor. If either of the concrete design alternatives is to be optimized, these architectural impacts on the New York Times Building must be considered. Also after comparing the shear wall thicknesses of the two concrete alternatives, the team determined that a concrete core alone would be less economical than that of a concrete core with outriggers. This is due to the fact that the alternative design with outriggers resulted in the use of much smaller shear walls with a lower concrete compressive strength.

Figure 21: Lobby Central Corridor

After a team review of the alternative lateral systems was performed, Team 3 agreed that if the lateral system of the New York Times Building was to be redesigned, a concrete solution which engaged the perimeter columns into the lateral system would be the best alternative to the existing steel braced frame core with outriggers.

Conclusion

For the third technical report on the structural system of the New York Times Building, three alternatives to the existing lateral force-resisting system were investigated and designed in a preliminary manner. One different design was developed by each structural student participating in the alternative IPD/BIM Thesis in order to compare the feasibility of three different alternatives to the existing lateral system. The three preliminary designs were:

- Modified Braced Frame Core w/ outriggers at the 36th floor
- Pure Concrete Shear Wall Core
- Concrete Core w/ outriggers at the 28th and 51st floors

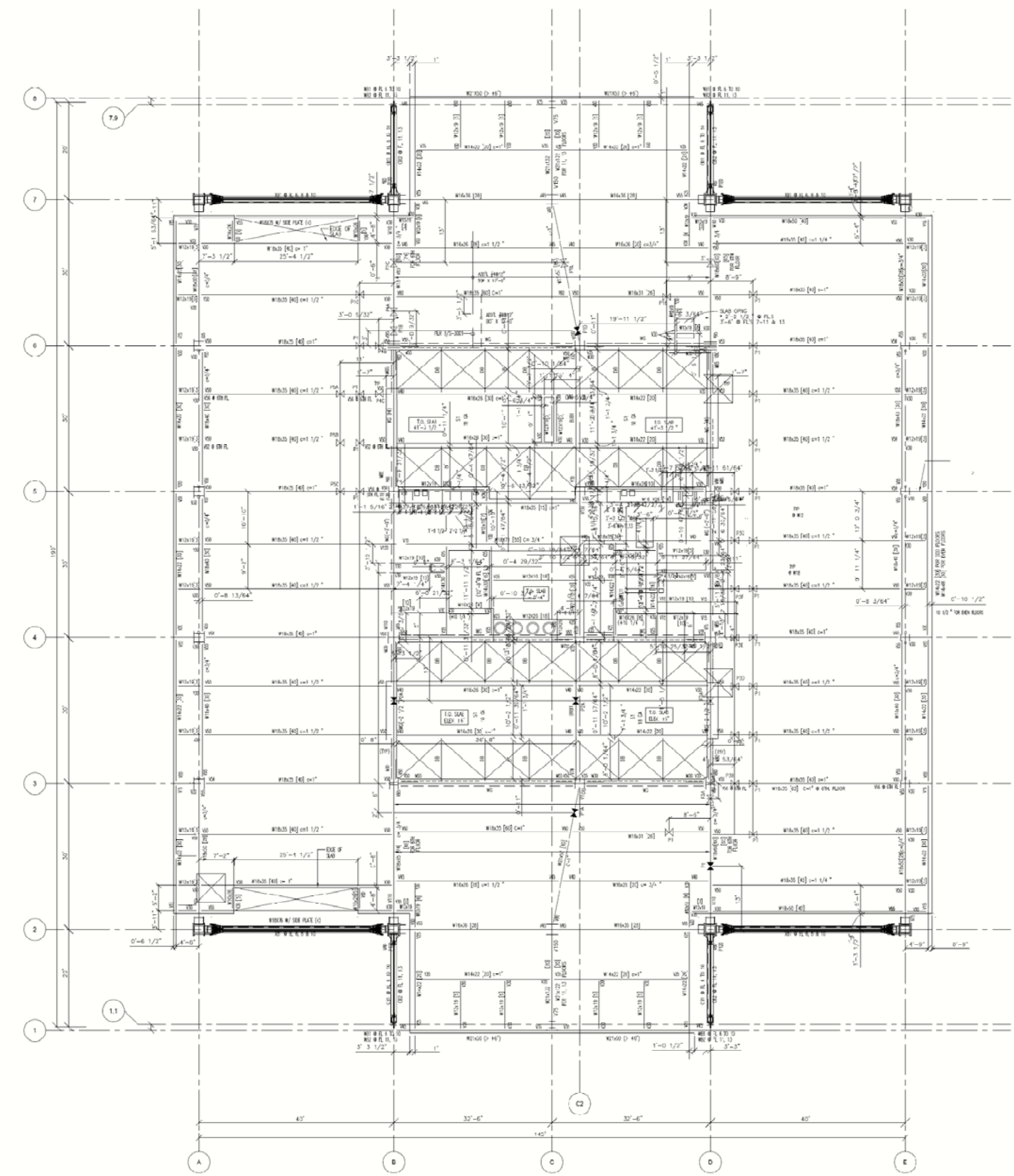
The alternative that was investigated in this report was the concrete core with outriggers system. The modified braced frame core and pure concrete shear wall core were investigated in the technical reports of Erika Bonfanti and Benjamin Barben respectively.

All the alternative systems were designed to be within 10% of the existing structure's period of vibration, 6.75s-6.25s. Also, the preliminary designs did not exceed the overall building wind drift of H/450 of the existing New York Times Building as well as seismic story drift criterion found in ASCE 7-05.

After each of the alternative lateral system designs were completed, they were brought before the IPD/BIM Team 3 in order to discuss the feasibility of optimizing any of the three preliminary designs. Team 3 first had concerns with the modified braced frame core. The team felt that the only possible use of the 36th Floor, based on the configuration of the alternative design, was that of a mechanical floor. The modified braced frame core was then determined not to be a feasible alternative because a mechanical floor on the 36th Level would not facilitate the required floors with heating, ventilating, and cooling in an energy efficient manner.

Concerns were also expressed with the designs of the two concrete solutions. The current concrete core configurations do not conform to the architectural layout of the existing core. They both currently infringe upon the architectural vision of transparency on the lobby floor by placing returns into the main central corridor. Also, their core configurations do not provide Forest City Ratner with the same amount of open rentable space as that of the existing lateral system. Therefore, it was determined that if a concrete core system was to be designed in place of the existing lateral system, an architectural redesign of the core configuration must be conducted as well. Also, the only main difference between two concrete core systems was that the concrete core with outrigger system required less concrete section to meet the same design parameters. Therefore, a concrete solution which engages the perimeter columns into the lateral system was found to be the best alternative to the existing steel lateral force resisting system.

Appendix A – Typical Framing Plan



Appendix B – Alternative Design Elevations (Concrete Core w/ Outriggers)

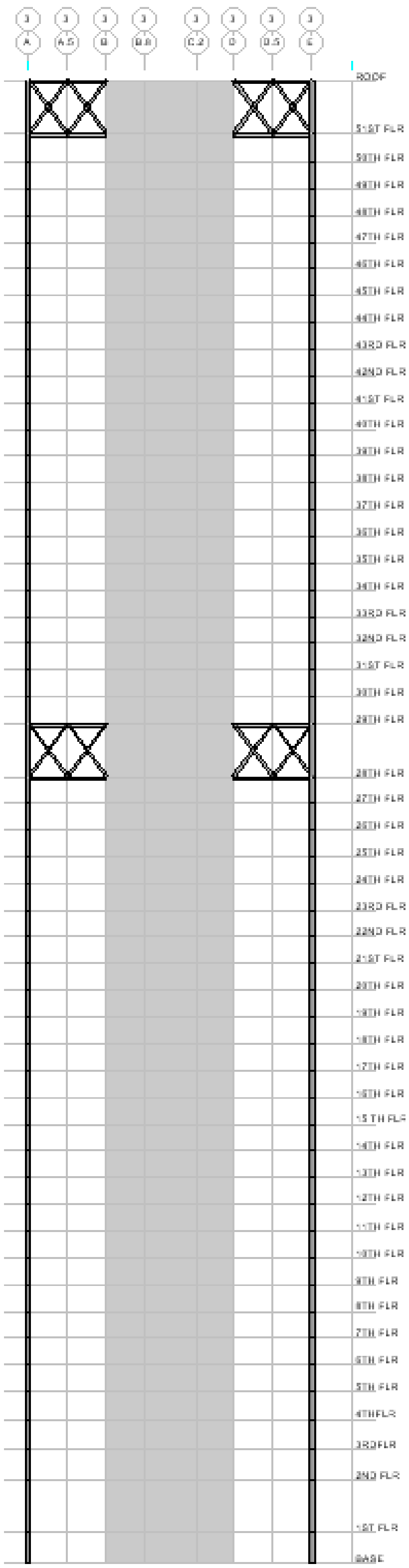


Figure 22: Alternative Lateral System Elevation (Grids 3 & 6)

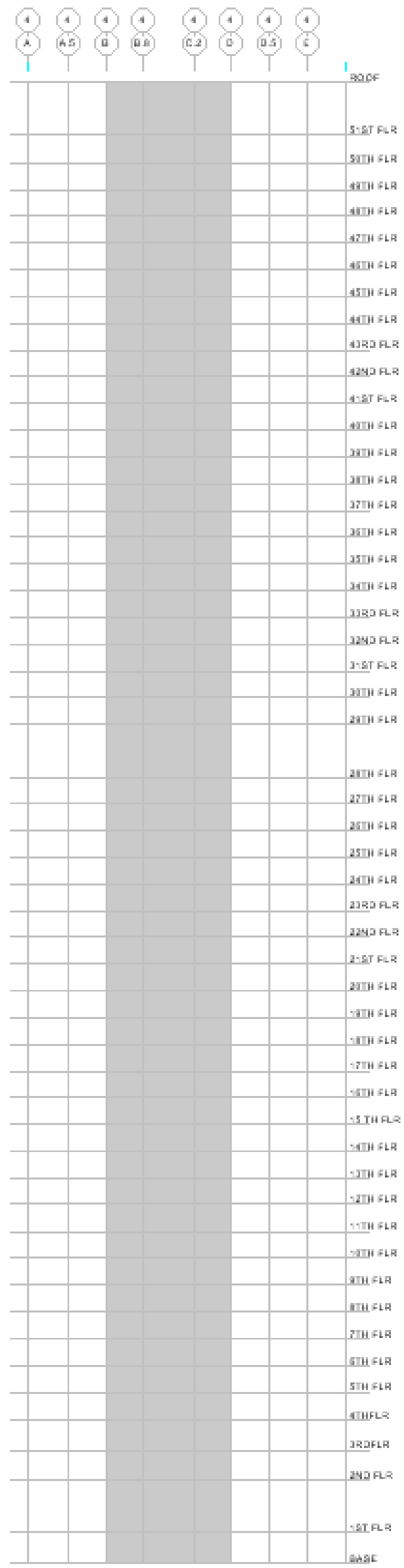


Figure 23: Alternative Lateral System Elevation (Grids 4 & 5)

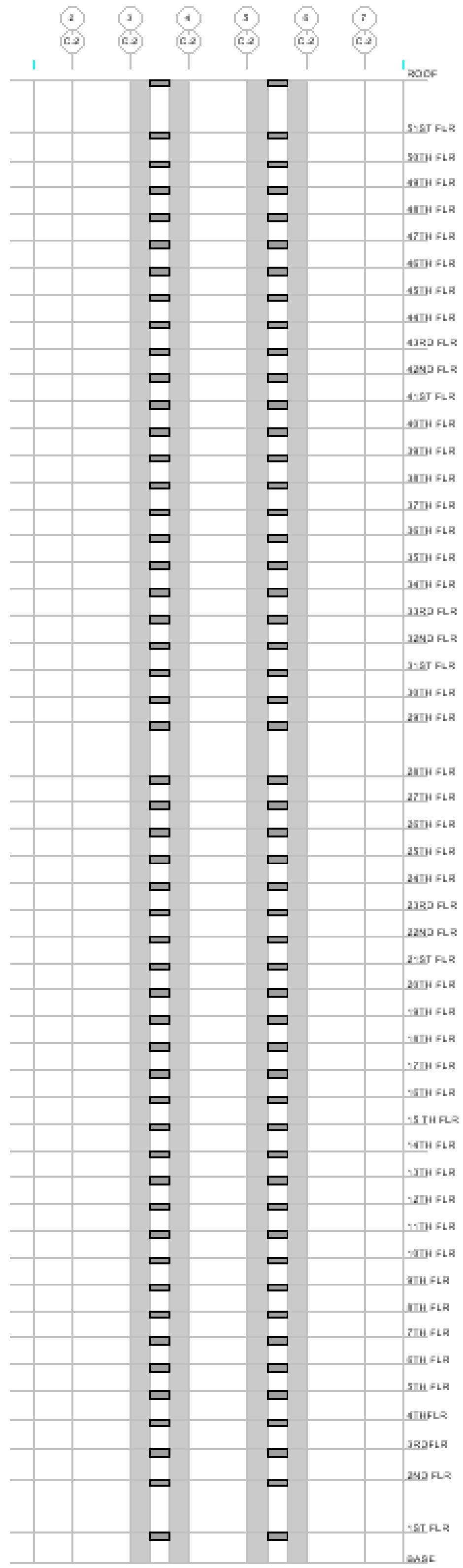


Figure 24: Alternative Lateral System Elevation (Grids B.8 & C.2)

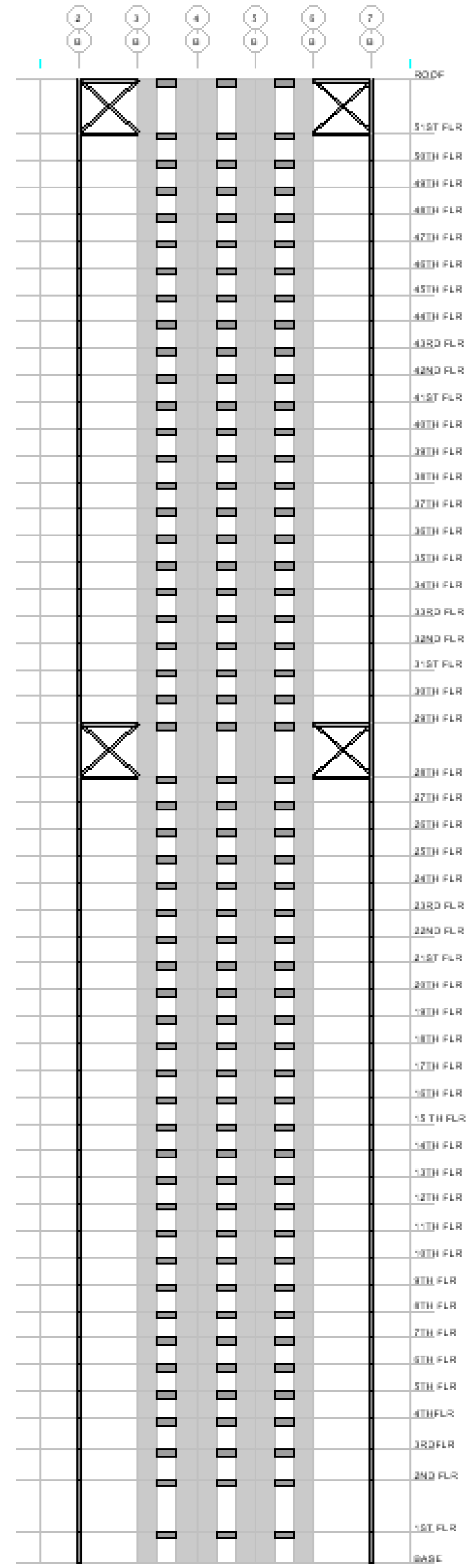


Figure 25: Alternative Lateral System Elevation (Grids B & D)

Appendix C – Wind Load Calculation

Calculated Wind Pressures in East/West Direction of Tower (Using Method 2, ASCE 7-05)							
	Height (z)	K_z[a]	q_z & q_h (psf) {$.00256K_zK_{zt}K_dV^2I$}	External Pressure (psf) {qGC_p}	Internal Pressure (psf) {q_hGC_{pi}}	Net Pressure p (psf)	
						+ (GC_{pi})	- (GC_{pi})
Windward	15.0	0.57	17.40	14.4	9.6	4.8	23.9
	33.4	0.72	21.87	18.1	9.6	8.5	27.6
	48.9	0.81	24.39	20.1	9.6	10.6	29.7
	63.8	0.87	26.31	21.7	9.6	12.2	31.3
	77.8	0.92	27.85	23.0	9.6	13.4	32.6
	86.0*	0.95	28.66	23.7	9.6	14.1	33.2
	91.5	0.96	29.18	24.1	9.6	14.5	33.6
	105.3	1.00	30.37	25.1	9.6	15.5	34.6
	119.0	1.04	31.45	26.0	9.6	16.4	35.5
	132.8	1.07	32.45	26.8	9.6	17.2	36.3
	146.5	1.10	33.37	27.6	9.6	18.0	37.1
	160.3	1.13	34.24	28.3	9.6	18.7	37.8
	174.0	1.16	35.06	28.9	9.6	19.4	38.5
	188.4	1.18	35.86	29.6	9.6	20.0	39.2
	202.1	1.21	36.59	30.2	9.6	20.6	39.8
	215.3	1.23	37.25	30.8	9.6	21.2	40.3
	229.0	1.25	37.92	31.3	9.6	21.7	40.9
	242.8	1.27	38.55	31.8	9.6	22.3	41.4
	256.5	1.29	39.17	32.3	9.6	22.8	41.9
	270.3	1.31	39.75	32.8	9.6	23.3	42.4
	284.0	1.33	40.32	33.3	9.6	23.7	42.8
	297.8	1.35	40.87	33.7	9.6	24.2	43.3
	311.5	1.37	41.40	34.2	9.6	24.6	43.7
	325.3	1.38	41.91	34.6	9.6	25.0	44.2
	339.0	1.40	42.41	35.0	9.6	25.5	44.6
	352.8	1.42	42.90	35.4	9.6	25.9	45.0
	366.5	1.43	43.37	35.8	9.6	26.2	45.4
	380.7	1.45	43.84	36.2	9.6	26.6	45.8
	401.8	1.47	44.52	36.8	9.6	27.2	46.3
	422.4	1.49	45.16	37.3	9.6	27.7	46.8
	436.1	1.51	45.58	37.6	9.6	28.1	47.2
	449.9	1.52	45.98	38.0	9.6	28.4	47.5
	463.6	1.53	46.38	38.3	9.6	28.7	47.9
	477.4	1.54	46.77	38.6	9.6	29.0	48.2
	491.1	1.56	47.15	38.9	9.6	29.4	48.5
	504.9	1.57	47.52	39.2	9.6	29.7	48.8
	518.6	1.58	47.89	39.5	9.6	30.0	49.1
	532.4	1.59	48.25	39.8	9.6	30.3	49.4
	546.1	1.61	48.60	40.1	9.6	30.6	49.7
	559.9	1.62	48.95	40.4	9.6	30.8	50.0
	573.6	1.63	49.29	40.7	9.6	31.1	50.3
	587.4	1.64	49.62	41.0	9.6	31.4	50.5
	601.1	1.65	49.95	41.2	9.6	31.7	50.8
	614.9	1.66	50.28	41.5	9.6	31.9	51.1
	628.6	1.67	50.60	41.8	9.6	32.2	51.3
	642.4	1.68	50.91	42.0	9.6	32.5	51.6
	656.1	1.69	51.22	42.3	9.6	32.7	51.8
	669.9	1.70	51.52	42.5	9.6	33.0	52.1
	683.6	1.71	51.82	42.8	9.6	33.2	52.3
	697.4	1.72	52.12	43.0	9.6	33.5	52.6
	711.5	1.73	52.42	43.3	9.6	33.7	52.8
	732.1	1.75	52.85	43.6	9.6	34.1	53.2
	745.5**	1.75	53.12	43.9	9.6	34.3	53.4
	802***	1.79	54.24	22.4	9.6	12.8	32.0
Leeward	All	---	53.12	-27.4	9.6	-37.0	-17.8
Side	All	---	53.12	-38.4	9.6	-47.9	-28.8
Roof	745.5	---	53.12	-57.0	9.6	-66.6	-47.4

* Top of Podium

** Finish Floor Elevation of Roof

*** Top of Screen Elevation (0.5 multiplier is applied to account for the ability for wind to pass through the screen.)

[a] $K_z = 2.01(15/z_g)2/\alpha$ {$z_g < 15$ft} -or- $K_z = 2.01(z/z_g)2/\alpha$ {$15 \text{ ft} < z < z_g$} [T 6-2, ASCE 7-05]

Calculated Wind Pressures in North/South Direction of Tower(Using Method 2, ASCE 7-05)							
	Height (z)	K_z*	q_z & q_h (psf) {.00256$K_zK_{zt}K_dV^2I$}	External Pressure (psf) {qGC_p}	Internal Pressure (psf) {q_hGC_{pi}}	Net Pressure p (psf)	
						+ (GC_{pi})	- (GC_{pi})
Windward	15.0	0.57	17.40	14.6	9.6	5.0	24.2
	33.4	0.72	21.87	18.3	9.6	8.8	27.9
	48.9	0.81	24.39	20.4	9.6	10.9	30.0
	63.8	0.87	26.31	22.1	9.6	12.5	31.6
	77.8	0.92	27.85	23.4	9.6	13.8	32.9
	86.0*	0.95	28.66	24.0	9.6	14.5	33.6
	91.5	0.96	29.18	24.5	9.6	14.9	34.0
	105.3	1.00	30.37	25.5	9.6	15.9	35.0
	119.0	1.04	31.45	26.4	9.6	16.8	35.9
	132.8	1.07	32.45	27.2	9.6	17.6	36.8
	146.5	1.10	33.37	28.0	9.6	18.4	37.5
	160.3	1.13	34.24	28.7	9.6	19.2	38.3
	174.0	1.16	35.06	29.4	9.6	19.8	39.0
	188.4	1.18	35.86	30.1	9.6	20.5	39.6
	202.1	1.21	36.59	30.7	9.6	21.1	40.2
	215.3	1.23	37.25	31.2	9.6	21.7	40.8
	229.0	1.25	37.92	31.8	9.6	22.2	41.4
	242.8	1.27	38.55	32.3	9.6	22.8	41.9
	256.5	1.29	39.17	32.8	9.6	23.3	42.4
	270.3	1.31	39.75	33.3	9.6	23.8	42.9
	284.0	1.33	40.32	33.8	9.6	24.3	43.4
	297.8	1.35	40.87	34.3	9.6	24.7	43.8
	311.5	1.37	41.40	34.7	9.6	25.2	44.3
	325.3	1.38	41.91	35.1	9.6	25.6	44.7
	339.0	1.40	42.41	35.6	9.6	26.0	45.1
	352.8	1.42	42.90	36.0	9.6	26.4	45.5
	366.5	1.43	43.37	36.4	9.6	26.8	45.9
	380.7	1.45	43.84	36.8	9.6	27.2	46.3
	401.8	1.47	44.52	37.3	9.6	27.8	46.9
	422.4	1.49	45.16	37.9	9.6	28.3	47.4
	436.1	1.51	45.58	38.2	9.6	28.7	47.8
	449.9	1.52	45.98	38.6	9.6	29.0	48.1
	463.6	1.53	46.38	38.9	9.6	29.3	48.5
	477.4	1.54	46.77	39.2	9.6	29.7	48.8
	491.1	1.56	47.15	39.5	9.6	30.0	49.1
	504.9	1.57	47.52	39.9	9.6	30.3	49.4
	518.6	1.58	47.89	40.2	9.6	30.6	49.7
	532.4	1.59	48.25	40.5	9.6	30.9	50.0
	546.1	1.61	48.60	40.8	9.6	31.2	50.3
	559.9	1.62	48.95	41.0	9.6	31.5	50.6
	573.6	1.63	49.29	41.3	9.6	31.8	50.9
	587.4	1.64	49.62	41.6	9.6	32.1	51.2
	601.1	1.65	49.95	41.9	9.6	32.3	51.5
	614.9	1.66	50.28	42.2	9.6	32.6	51.7
	628.6	1.67	50.60	42.4	9.6	32.9	52.0
	642.4	1.68	50.91	42.7	9.6	33.1	52.3
	656.1	1.69	51.22	43.0	9.6	33.4	52.5
	669.9	1.70	51.52	43.2	9.6	33.6	52.8
	683.6	1.71	51.82	43.5	9.6	33.9	53.0
	697.4	1.72	52.12	43.7	9.6	34.1	53.3
	711.5	1.73	52.42	44.0	9.6	34.4	53.5
	732.1	1.75	52.85	44.3	9.6	34.8	53.9
	745.5**	1.75	53.12	44.5	9.6	35.0	54.1
	819***	1.80	54.57	22.9	9.6	13.3	32.4
Leeward	All	---	53.12	-24.8	9.6	-34.4	-15.3
Side	All	---	53.12	-38.4	9.6	-47.9	-28.8
Roof	745.5	---	53.12	-57.9	9.6	-67.5	-48.3

* Top of Podium

** Finish Floor Elevation of Roof

*** Top of Screen Elevation (0.5 multiplier is applied to account for the ability for wind to pass through the screen.)

* $K_z = 2.01(15/z_g)2/\alpha$ {z_g < 15ft} -or- $K_z = 2.01(z/z_g)2/\alpha$ {15 ft < z < z_g} [T 6-2, ASCE 7-05]

Method 2 Wind Load Design Variables			
Variable	**Value**	**Unit**	**Reference**
V	110	miles/hr	ASCE 7 05 6.5.4
K_d	0.85	---	ASCE 7-05 6.5.4.4
Occupancy Cat.	III	---	IBC Table 1604.5
I	1.15	---	ASCE 7-05 6.5.5
Surf. Rough. Cat.	B	---	ASCE 7-05 6.5.2
Exp. Cat.	B	---	ASCE 7-05 6.5.6
K_{zt}	1	---	ASCE 7-05 6.5.7
α	7.0		ASCE 7 05 6.5.6.6
z_g	1200	---	ASCE 7-05 6.5.6.6

Gust Factor {Tower}						
Variable	**Equation**	**Direction**		**Unit**	**Reference (ASCE 7)**	**Comments**
		E/W	**N/S**			
$n_1\ (f_{n1})$	150/h	0.20121	0.20121	---	C6.5.8	Flexible Structure
gQ = gv	---	3.4	3.4	---	6.5.8.2	
g_r	$(2LN(3600n_1))^{1/2}+(0.577/(2LN(3600n_1))^{1/2}$	3.7881	3.7881	---	6.5.8.2	
h	---	745.5	745.5	ft		
z bar	.6h	447.3	447.3	ft		
z_{min}	---	30	30	ft	Table 6-2	z bar $\geq z_{min}$ (ok)
c	---	0.3	0.3	---	Table 6-2	
I_z	$c(33/z)^{1/6}$	0.1943	0.1943	---	6.5.8.1	
ℓ	---	320	320	ft	Table 6-2	
ε	---	0.3333	0.3333	---	Table 6-2	
L_z	$\ell(z/33)^{\varepsilon}$	762.98	762.98	ft	6.5.8.1	
B	---	194.00	157.00	ft		
L	---	157.00	194.00	ft		
Q	$(1/(1+0.63((B+h)/L_z)^{0.63})^{1/2}$	0.76288	0.76690	---	6.5.8.1	
V	---	110	110	miles/hr	6.5.4	
b bar	---	0.45	0.45	---	Table 6-2	
α bar	---	0.25	0.25	---	Table 6-2	
V_z	$b(z/33)^{\alpha}V(88/60)$	139.3022	139.3022	ft/s	6.5.8.2	
N_1	n_1L_z/V_z	1.1020	1.1020	---	6.5.8.2	
R_n	$7.47N_1/(1+10.3N_1)^{5/3}$	0.12474	0.12474	---	6.5.8.2	
$\eta\ (R_h)$	$4.6n_1h/V_z$	4.9533	4.9533	---	6.5.8.2	
R_h	$1/\eta - (1/2\eta^2)(1-e^{-2\eta})$	0.18151	0.18151	---	6.5.8.2	
$\eta\ (R_B)$	$4.6n_1B/V_z$	1.2890	1.0431	---	6.5.8.2	
R_B	$1/\eta - (1/2\eta^2)(1-e^{-2\eta})$	0.49772	0.55619	---	6.5.8.2	
$\eta\ (R_L)$	$15.4n_1L/V_z$	3.4923	4.3153	---	6.5.8.2	
R_L	$1/\eta - (1/2\eta^2)(1-e^{-2\eta})$	0.24539	0.20489	---	6.5.8.2	
β	---	0.01	0.01	---	C6.5.8	
R	$((1/\beta)(R_nR_hR_B(.53+0.47R_L)))^{1/2}$	0.852786	0.888092	---	6.5.8.2	
G_f	$0.925(1+1.7I_z(g_Q{}^2Q^2+g_R{}^2R^2)^{1/2})/(1+1.7g_vI_z)$	1.032	1.048	---	6.5.8.2	

E/W Wind Direction (Tower) {h/L >1.0 & q < 10}			
L/B	**Wall Pressure Coeff. (Cp)**		
	Windward	**Leeward**	**Side**
0.809	0.8	-0.5	-0.7
h/L	**Roof Pressure Coeff. (Cp)**		
	Roof Area (ft²)	**Reduction**	**Cp**
4.748	27400	0.8	-1.040

Internal Pressure	
GC_{pi}	0.18
	-0.18

[F 6-5, ASCE 7-05]

N/S Wind Direction (Tower) {h/L >1.0 & θ < 10}			
L/B	**Wall Pressure Coeff. (Cp)**		
	Windward	**Leeward**	**Side**
1.236	0.8	-0.453	-0.7
h/L	**Roof Pressure Coeff. (Cp)**		
	Roof Area (ft²)	**Reduction**	**Cp**
3.843	27400	0.8	-1.040

Internal Pressure	
GC_{pi}	0.18
	-0.18

[F 6-5, ASCE 7-05]

Appendix D – Seismic Load Calculation

Soil Classification

NYCBC:	2-65 (medium hard rock)	recommended by geotechnical report
	4-65 (soft rock)	in areas of lower bearing capacity
ASCE 7-05:	seismic design category C	conservative estimate
	Occ. Cat. III *T 11.5-1*	
	Importance factor= 1.25	

Spectral Response Acceleration

(using USGS Ground Motion Parameter Calculator)

latitude: 40.756192 F_a= 1.2
longitude: -73.990130 F_v= 1.7

site class C					
T=0.2s			*T=1.0s*		
S_{MS}	0.436	g	S_{M1}	0.119	g
S_{DS}	0.291	g	S_{D1}	0.08	g

ASCE 7-05:	S_{DS} -> SDC B	*T 11.6-1*	
	S_{D1} -> SDC B	*T 11.6-2*	therefore, use site class C

Period of Building

$T_a <= 0.8T_s$ =	0.2199	
T_s	0.2749	S_{D1}/S_{DS}
$T_a = C_t * h_n^{\ x}$ =	2.991	
C_t	0.02	*T 12.2.1.B*
x	0.75	*T 11.5-1*
h	793.8	
T(Existing)	6.75	*Thornton Tomasetti*

Seismic Base Shear

$V = C_s * W$	1759.8 k	*12.8-1*			
$C_s = min\{$	0.1119	$S_{DS}/(R/I)$			
	0.0046	$S_{D1}/(T_a * R/I)$	Cu	1.7	0.0027
	>= 0.01	use 0.01 for C_s			
R	3.25	*T 12.2.1.B*			
I	1.25	*T 11.5-1*			

Tower Weight by Floor

		w_i (psf)						
floor	*area (sf)*	*floor*	*façade*	*wall area (sf)*	W_i (#)	h_x (ft)	h_i (ft)	$w_i*h_i^k$
1	96625	113	25	18893	11390943	26.9896	27.0	8.298E+09
2	96625	113	25	10828	11189329	15.4688	42.5	2.017E+10
3	96625	113	25	10828	11189329	15.4688	57.9	3.755E+10
4	96625	113	25	10026	11169276	14.3229	72.3	5.83E+10
5	21550	113	25	9625	2675775	13.75	86.0	1.979E+10
6	21550	113	25	9625	2675775	13.75	99.8	2.662E+10
7	21550	113	25	9625	2675775	13.75	113.5	3.447E+10
8	21550	113	25	9625	2675775	13.75	127.3	4.333E+10
9	21550	113	25	9625	2675775	13.75	141.0	5.32E+10
10	21550	113	25	9625	2675775	13.75	154.8	6.408E+10
11	21550	113	25	9975	2684525	14.25	169.0	7.667E+10
12	21550	113	25	9275	2667025	13.25	182.3	8.859E+10
13	21550	113	25	9625	2675775	13.75	196.0	1.028E+11
14	21550	113	25	9625	2675775	13.75	209.8	1.177E+11
15	21550	113	25	9625	2675775	13.75	223.5	1.337E+11
16	21550	113	25	9625	2675775	13.75	237.3	1.506E+11
17	21550	113	25	9625	2675775	13.75	251.0	1.686E+11
18	21550	113	25	9625	2675775	13.75	264.8	1.876E+11
19	21550	113	25	9625	2675775	13.75	278.5	2.075E+11
20	21550	113	25	9625	2675775	13.75	292.3	2.285E+11
21	21550	113	25	9625	2675775	13.75	306.0	2.505E+11
22	21550	113	25	9625	2675775	13.75	319.8	2.736E+11
23	21550	113	25	9625	2675775	13.75	333.5	2.976E+11
24	21550	113	25	9625	2675775	13.75	347.3	3.227E+11
25	21550	113	25	9625	2675775	13.75	361.0	3.487E+11
26	21550	113	25	9625	2675775	13.75	374.8	3.758E+11
27	21550	113	25	9275	2667025	13.25	388.0	4.015E+11
28	21550	105	25	19250	2744000	27.5	415.5	4.737E+11
29	21550	113	25	9625	2675775	13.75	429.3	4.93E+11
30	21550	113	25	9625	2675775	13.75	443.0	5.251E+11
31	21550	113	25	9625	2675775	13.75	456.8	5.582E+11
32	21550	113	25	9625	2675775	13.75	470.5	5.923E+11
33	21550	113	25	9625	2675775	13.75	484.3	6.275E+11
34	21550	113	25	9625	2675775	13.75	498.0	6.636E+11
35	21550	113	25	9625	2675775	13.75	511.8	7.008E+11
36	21550	113	25	9625	2675775	13.75	525.5	7.389E+11
37	21550	113	25	9625	2675775	13.75	539.3	7.781E+11
38	21550	113	25	9625	2675775	13.75	553.0	8.183E+11
39	21550	113	25	9625	2675775	13.75	566.8	8.595E+11
40	21550	113	25	9625	2675775	13.75	580.5	9.017E+11
41	21550	113	25	9625	2675775	13.75	594.3	9.449E+11
42	21550	113	25	9625	2675775	13.75	608.0	9.891E+11
43	21550	113	25	9625	2675775	13.75	621.8	1.034E+12
44	21550	113	25	9625	2675775	13.75	635.5	1.081E+12
45	21550	113	25	9625	2675775	13.75	649.3	1.128E+12
46	21550	113	25	9625	2675775	13.75	663.0	1.176E+12
47	21550	113	25	9625	2675775	13.75	676.8	1.225E+12
48	21550	113	25	9625	2675775	13.75	690.5	1.276E+12
49	21550	113	25	9625	2675775	13.75	704.3	1.327E+12
50	21550	113	25	10267	2691816.7	14.6667	718.9	1.391E+12
51	21550	105	25	18958	2736708.3	27.0833	746.0	1.523E+12
52	21550	200	25	33491	5147266.5	47.8438	793.8	3.244E+12
				ΣW	**175984.02**	**k**	**$\Sigma w_i*h_i^k$**	**2.917E+13**

Lateral Seismic Force

k=	2.0	(T > 2.5s)
C_{vx}	F_x	
0.0003	0.5006	
0.0007	1.217	
0.0013	2.265	$C_{vx} = w_x * h_x^k / (\Sigma w_i * h_i^k)$
0.0020	3.518	
0.0007	1.1940	$F_x = C_{vx} * V$
0.0009	1.606	
0.0012	2.080	
0.0015	2.614	
0.0018	3.2096	
0.0022	3.866	
0.0026	4.626	
0.0030	5.345	
0.0035	6.2018	
0.0040	7.102	
0.0046	8.064	
0.0052	9.087	
0.0058	10.1708	
0.0064	11.316	
0.0071	12.522	
0.0078	13.788	
0.0086	15.1164	
0.0094	16.505	
0.0102	17.956	
0.0111	19.467	
0.0120	21.0388	
0.0129	22.672	
0.0138	24.224	
0.0162	28.581	
0.0169	29.7459	
0.0180	31.682	
0.0191	33.679	
0.0203	35.738	
0.0215	37.8570	
0.0228	40.037	
0.0240	42.279	
0.0253	44.581	
0.0267	46.9447	
0.0281	49.369	
0.0295	51.855	
0.0309	54.402	
0.0324	57.0092	
0.0339	59.678	
0.0355	62.408	
0.0370	65.199	
0.0387	68.0504	
0.0403	70.963	
0.0420	73.937	
0.0437	76.972	
0.0455	80.0682	
0.0477	83.938	
0.0522	91.889	
0.1112	195.706	
V= ΣF_x (k)	**1759.8**	

Appendix E – Initial Rough Hand Calculations

INITIAL SHEAR WALL thickness – SHEAR

BASE SHEARS (CASE 1 WIND)

E/W → 9336 k × 1.6 = 14938 k

N/S → 7438 k × 1.6 = 11901 k

USING $V_u \le \phi\, 4 \sqrt{f'_c}\, A_{cw}$

ASSUME f'_c = 12000 psi

N/S

$(11901)(1000) = (0.75)(4)\sqrt{12000}\, A_{cw}$

A_{cw} = 36213.6 in²

LENGHT OF ALL RETURNS $4(240'') + 8(120'') = 1920''$

$t_{req} = A_{cw}/L = 18.86''$ → SAY 18''

ASSUME REIN.
CAN TAKE
ADDITION SHEAR.

E/W

$(14938)(1000) = (0.75)(4)\sqrt{12000}\, A_{cw}$ $A_{cw} = 45454.9''$

LENGHT OF (4) 65' WALLS $4(780'') = 3120''$

$t_{req} = 14.56''$ → SAY 15'' MIN.

E/W Delta Multiplier			
Level	Height (h_i)	Unfactored E/W Wind Load (Case 1)	E/W Delta Multiplier
2	25.66	181.35	8520782
3	41.13	142.66	17102248
4	56.59	141.97	32004191
5	70.92	137.24	48259858
6	86.00	137.36	70538416
7	98.42	139.98	93596541
8	112.17	142.37	122854282
9	125.92	144.57	156192418
10	139.67	146.61	193609333
11	153.42	148.51	235094783
12	167.17	150.31	280631143
13	180.92	158.52	344360985
14	195.83	153.68	388314453
15	208.42	148.56	422556642
16	222.17	156.61	502723360
17	235.92	158.01	568048873
18	249.67	159.36	637206845
19	263.42	160.65	710146607
20	277.17	161.90	786814283
21	290.92	163.11	867153016
22	304.67	164.28	951103167
23	318.42	165.41	1038602489
24	332.17	166.51	1129586285
25	345.92	167.58	1223987544
26	359.67	168.61	1321737069
27	373.42	174.80	1466138626
28	388.00	261.91	2353216605
29	415.50	258.84	2627326811
30	429.25	173.48	1865112458
31	443.00	174.37	1981561555
32	456.75	175.25	2100816535
33	470.50	176.10	2222793917
34	484.25	176.94	2347408758
35	498.00	177.76	2474574707
36	511.75	178.57	2604204052
37	525.50	179.36	2736207767
38	539.25	180.14	2870495551
39	553.00	180.90	3006975872
40	566.75	181.65	3145556002
41	580.50	182.39	3286142052
42	594.25	183.11	3428639006
43	608.00	183.83	3572950752
44	621.75	184.53	3718980111
45	635.50	185.22	3866628864
46	649.25	185.90	4015797782
47	663.00	186.57	4166386647
48	676.75	187.23	4318294279
49	690.50	187.88	4471418557
50	704.25	193.11	4738181942
51	718.67	284.23	7193998662
Roof	745.50	676.30	18094030948
Σmult. = $\Sigma[0.7P_ih_i^2(3H-h_i)]/(6E)$			1.10825E+11
I_{Total} (in^4) = Σmult./(0.7*H/450)			**7078955283**

H/450 (in)	19.877

Returns			
	Equation	inner	outer
L(in)		120	120
t (in)		18	18
A (in^2)	L*t	2160	2160
I_i (in^4)	L*t^3/12	58320	58320
d (in)		162	390
I_g (in^4)	I_i + A*d^2	56745360	328594320
N		8	12
Ir (in^4)	ΣI_g	453962880	3943131840
Ir (total)		4397094720	

65' walls		
I_{Req}	I (tota)-Ir	2681860563
L (in)		780
N		4
t (in)	I_{Req}*12/(L^3*N)	**17.0**

Relative Stiffness About N/S Axis				
Wall		Interior Returns	Exterior Returns	65' wall
b (in)		104	104	17
h (in)		18	18	780
A (in^2)	b*h	1872	1872	13260
I_i (in^4)	$bh^3/12$	50544	50544	672282000
d (in)		390	162	0
N		8	12	4
I (in^4)	$\Sigma(I_i + Ad^2)$	2.2783.E+09	5.9015.E+08	2.6891.E+09
ΣI (in^4)				5.5575.E+09
RS for (1) 65' SW = I_{SW} / ΣI (%)				12.0968

Shear Wall Design Check For Flexure						
Variable	Equation	Value				
Factored Total Moment (Case 1 Wind)(TFM)	1.6*3922512	6276020				
Mu (ft-k)	RS*TFM	759195.6				
t (in)		17				
lw (in)		780				
d (in)	0.8*lw	624				
f'c (psi)		12000				
β_1		0.65				
fy (ksi)		60				
ϕ		0.9				
Assumed jd	0.9*d	561.6				
As From assumed jd	Mu/(ϕfy*jd)	300.4098	tension length (in)		415.948	
a	As*fy/(.85*f'c*t)	103.948	Max # of Spaces (Assume 4.5" Spacing)		92.4329	say 92
jd (in)	d-(a/2)	572.026				
As (in^2)	Mu/(ϕfy*jd)	294.9344	Abar Req'd w/ 3 Bars (in^2)		1.068603	
Reasonable Reinf.		Yes				
As (used)	(3) #10	350.52				
a	As*fy/(.85*f'c*t)	121.2872				
c	a/β_1	186.5957				
dt	lw-3	777				
εt	εu*(dt-c)/c	0.009492	>	0.005	OK	

Appendix F – Shear Wall Spot Checks

Spot Check For Flexure (SW 20 @ Base Level)						
Variable	Equation	Value				
Mu (ft-k)		375615.2				
t (in)		16				
lw (in)		780				
d (in)	0.8*lw	624				
f'c (psi)		10000				
β_1		0.65				
fy (ksi)		60				
ϕ		0.9				
Assumed jd	0.9*d	561.6				
As From assumed jd	Mu/(ϕfy*jd)	148.629	tension length (in)	377.5716		
a	As*fy/(.85*f'c*t)	65.57162	Max # of Spaces	83.9048	say	84
jd (in)	d-(a/2)	591.2142	(Assume 4.5" Spacing)			
As (in^2)	Mu/(ϕfy*jd)	141.1841	Abar Req'd w/ 2 Bars (in^2)	0.840382		
Reasonable Reinf.		Yes				
As (used)	(2) #9	168				
a	As*fy/(.85*f'c*t)	74.11765				
c	a/β_1	114.0271				
dt	lw-3	777				
εt	εu*(dt-c)/c	0.017443	>	0.005	OK	

Spot Check For Flexure (SW 1@ Base Level)						
Variable	Equation	Value				
Mu (ft-k)		7084.22				
t (in)		18				
lw (in)		120				
d (in)	0.8*lw	96				
f'c (psi)		10000				
β_1		0.65				
fy (ksi)		60				
ϕ		0.9				
Assumed jd	0.9*d	86.4				
As From assumed jd	Mu/(ϕfy*jd)	18.22073045	tension length (in)	55.14538		
a	As*fy/(.85*f'c*t)	7.145384491	Max # of Spaces	12.2545	say	12
jd (in)	d-(a/2)	92.42730775	(Assume 4.5" Spacing)			
As (in^2)	Mu/(ϕfy*jd)	17.03253237	Abar Req'd w/ 2 Bars (in^2)	0.709689		
Reasonable Reinf.		Yes				
As (used)	(2) #9	24				
a	As*fy/(.85*f'c*t)	9.411764706				
c	a/β_1	14.47963801				
dt	lw-3	117				
εt	εu*(dt-c)/c	0.021240938	>	0.005	OK	

Spot Check For Flexure (SW 3 @ Base Level)						
Variable	Equation	Value				
Mu (ft-k)		43494.24				
t (in)		18				
lw (in)		240				
d (in)	0.8*lw	192				
f'c (psi)		10000				
β_1		0.65				
fy (ksi)		60				
ϕ		0.9				
Assumed jd	0.9*d	172.8				
As From assumed jd	Mu/(ϕfy*jd)	55.93395062	tension length (in)	117.9349		
a	As*fy/(.85*f'c*t)	21.9348826	Max # of Spaces	26.2078	say	26
jd (in)	d-(a/2)	181.0325587	(Assume 4.5" Spacing)			
As (in^2)	Mu/(ϕfy*jd)	53.39032236	Abar Req'd w/ 2 Bars (in^2)	1.026737		
Reasonable Reinf.		Yes				
As (used)	(2) #10	66.04				
a	As*fy/(.85*f'c*t)	25.89803922				
c	a/β_1	39.84313725				
dt	lw-3	237				
εt	εu*(dt-c)/c	0.01484498	>	0.005	OK	

Spot Check For Flexure (SW 2 @ Level 30)						
Variable	Equation	Value				
Mu (ft-k)		9449.69				
t (in)		18				
lw (in)		240				
d (in)	0.8*lw	192				
f'c (psi)		8000				
β_1		0.65				
fy (ksi)		60				
ϕ		0.9				
Assumed jd	0.9*d	172.8				
As From assumed jd	Mu/(ϕfy*jd)	12.15237912	tension length (in)	101.957		
a	As*fy/(.85*f'c*t)	5.957048586	Max # of Spaces	22.6571	say	23
jd (in)	d-(a/2)	189.0214757	(Assume 4.5" Spacing)			
As (in^2)	Mu/(ϕfy*jd)	11.10948427	Abar Req'd w/ 2 Bars (in^2)	0.241511		
Reasonable Reinf.		Yes				
As (used)	(2) #5	16.1				
a	As*fy/(.85*f'c*t)	7.892156863				
c	a/β_1	12.14177979				
dt	lw-3	237				
εt	εu*(dt-c)/c	0.055558137	>	0.005	OK	

Spot Check For Flexure (SW 17 @ Level 40)						
Variable	Equation	Value				
Mu (ft-k)		12007.88				
t (in)		14				
lw (in)		780				
d (in)	0.8*lw	624				
f'c (psi)		6000				
β_1		0.7				
fy (ksi)		60				
ϕ		0.9				
Assumed jd	0.9*d	561.6				
As From assumed jd	Mu/(ϕfy*jd)	4.751456	tension length (in)	315.9928		
a	As*fy/(.85*f'c*t)	3.99282	Max # of Spaces	70.2206	say	70
jd (in)	d-(a/2)	622.0036	(Assume 4.5" Spacing)			
As (in^2)	Mu/(ϕfy*jd)	4.290036	Abar Req'd w/ 2 Bars (in^2)	0.030643		
Reasonable Reinf.		Yes				
As (used)	(2) #5	49				
a	As*fy/(.85*f'c*t)	41.17647				
c	a/β_1	58.82353				
dt	lw-3	777				
εt	εu*(dt-c)/c	0.036627	>	0.005	OK	

Spot Check For Flexure (SW 11 @ Level 40)						
Variable	Equation	Value				
Mu (ft-k)		637.03				
t (in)		18				
lw (in)		120				
d (in)	0.8*lw	96				
f'c (psi)		6000				
β_1		0.7				
fy (ksi)		60				
ϕ		0.9				
Assumed jd	0.9*d	86.4				
As From assumed jd	Mu/(ϕfy*jd)	1.638451646	tension length (in)	49.07088		
a	As*fy/(.85*f'c*t)	1.070883429	Max # of Spaces	10.9046	say	11
jd (in)	d-(a/2)	95.46455829	(Assume 4.5" Spacing)			
As (in^2)	Mu/(ϕfy*jd)	1.482877256	Abar Req'd w/ 2 Bars (in^2)	0.067404		
Reasonable Reinf.		Yes				
As (used)	(2) #5	7.7				
a	As*fy/(.85*f'c*t)	5.032679739				
c	a/β_1	7.189542484				
dt	lw-3	117				
εt	εu*(dt-c)/c	0.045820909	>	0.005	OK	

Spot Check For Flexure (SW 3 @ Level 40)						
Variable	Equation	Value				
Mu (ft-k)		2393.83				
t (in)		18				
lw (in)		240				
d (in)	0.8*lw	192				
f'c (psi)		6000				
β_1		0.7				
fy (ksi)		60				
ϕ		0.9				
Assumed jd	0.9*d	172.8				
As From assumed jd	Mu/(ϕfy*jd)	3.078485082	tension length (in)	98.01208		
a	As*fy/(.85*f'c*t)	2.012081753	Max # of Spaces (Assume 4.5" Spacing)	21.7805	say	22
jd (in)	d-(a/2)	190.9939591				
As (in^2)	Mu/(ϕfy*jd)	2.785230615	Abar Req'd w/ 2 Bars (in^2)	0.063301		
Reasonable Reinf.		Yes				
As (used)	(2) #5	15.4				
a	As*fy/(.85*f'c*t)	10.06535948				
c	a/β_1	14.37908497				
dt	lw-3	237				
εt	εu*(dt-c)/c	0.046446818	>	0.005	OK	

Spot Check For Flexure (SW 17 @ Level 50)						
Variable	Equation	Value				
Mu (ft-k)		65036.4				
t (in)		14				
lw (in)		780				
d (in)	0.8*lw	624				
f'c (psi)		8000				
β_1		0.65				
fy (ksi)		60				
ϕ		0.9				
Assumed jd	0.9*d	561.6				
As From assumed jd	Mu/(ϕfy*jd)	25.73457	tension length (in)	328.2193		
a	As*fy/(.85*f'c*t)	16.21927	Max # of Spaces (Assume 4.5" Spacing)	72.9376	say	73
jd (in)	d-(a/2)	615.8904				
As (in^2)	Mu/(ϕfy*jd)	23.46608	Abar Req'd w/ 2 Bars (in^2)	0.160727		
Reasonable Reinf.		Yes				
As (used)	(2) #5	51.1				
a	As*fy/(.85*f'c*t)	32.20588				
c	a/β_1	49.54751				
dt	lw-3	777				
εt	εu*(dt-c)/c	0.044046	>	0.005	OK	

Spot Check For Flexure (SW 10 @ Level 51)						
Variable	Equation	Value				
Mu (ft-k)		1725.32				
t (in)		18				
lw (in)		120				
d (in)	0.8*lw	96				
f'c (psi)		8000				
β_1		0.65				
fy (ksi)		60				
ϕ		0.9				
Assumed jd	0.9*d	86.4				
As From assumed jd	Mu/(ϕfy*jd)	4.43755144	tension length (in)	50.17527		
a	As*fy/(.85*f'c*t)	2.175270314	Max # of Spaces (Assume 4.5" Spacing)	11.1501	say	11
jd (in)	d-(a/2)	94.91236484				
As (in^2)	Mu/(ϕfy*jd)	4.039562654	Abar Req'd w/ 2 Bars (in^2)	0.183616		
Reasonable Reinf.		Yes				
As (used)	(2) #9	22				
a	As*fy/(.85*f'c*t)	10.78431373				
c	a/β_1	16.59125189				
dt	lw-3	117				
εt	εu*(dt-c)/c	0.018155727	>	0.005	OK	

Spot Check For Shear (SW 19 @ Level 28)			
Variable	Equation	Value	
Vu (k)		7597	
t (in)		16	
lw (in)		780	
hw (in)		175	
d	0.8*lw	624	
f'c (psi)		10000	
fy (ksi)		60	
phi		0.75	
Max phi Vn	phi*10*(f'c)^0.5 *t*d	7488	NG

Spot Check For Shear (SW 7 @ Base Level)					
Variable	Equation	Value			
Vu (k)		502.12			
t (in)		18			
lw (in)		120			
hw (in)		308			
d	0.8*lw	96			
f'c (psi)		10000			
fy (ksi)		60			
phi		0.75			
Max phi Vn	phi*10*(f'c)^0.5 *t*d	1296	OK		
Vc (k)	3.3*(f'c)^0.5 *t*d	570.24			
Phi Vc (k)		427.68			
Provision	Design w/ 11.10.9				
Req'd Vs	(Vu/phi)-Vc	99.25333			
Req'd Av/s	Vs/(fy*d)	0.017231			
s for (2) #5		35.98066	Use	10	
rhot	Av/(s*t)	0.003444	>	0.0025	OK
rhol	0.0025+0.5*(2.5-(h/l))*(rhot-0.0025)	0.002469			
s for (2) #5		9.785563			

Spot Check For Shear (SW 2 @ Base Level)					
Variable	Equation	Value			
Vu (k)		1363.58			
t (in)		18			
lw (in)		240			
hw (in)		308			
d	0.8*lw	192			
f'c (psi)		10000			
fy (ksi)		60			
phi		0.75			
Max phi Vn	phi*10*(f'c)^0.5 *t*d	2592	OK		
Vc (k)	3.3*(f'c)^0.5 *t*d	1140.48			
Phi Vc (k)		855.36			
Provision	Design w/ 11.10.9				
Req'd Vs	(Vu/phi)-Vc	677.6267			
Req'd Av/s	Vs/(fy*d)	0.058822			
s for (2) #5		10.54032	Use	10	
rhot	Av/(s*t)	0.003444	>	0.0025	OK
rhol	0.0025+0.5*(2.5-(h/l))*(rhot-0.0025)	0.003075			
s for (2) #5		7.856742			

Spot Check For Shear (SW 7 @ Level 30)					
Variable	Equation	Value			
Vu (k)		245.58			
t (in)		18			
lw (in)		120			
hw (in)		165			
d	0.8*lw	96			
f'c (psi)		8000			
fy (ksi)		60			
phi		0.75			
Max phi Vn	phi*10*(f'c)^0.5 *t*d	1159.178	OK		
Vc (k)	3.3*(f'c)^0.5 *t*d	510.0382			
Phi Vc (k)		382.5286			
Provision	Design w/ 11.10.9				
Req'd Vs	(Vu/phi)-Vc	----			
Req'd Av/s	Vs/(fy*d)	----			
s for (2) #5		----	Use	10	
rhot	Av/(s*t)	0.003444	>	0.0025	OK
rhol	0.0025+0.5*(2.5-(h/l))*(rhot-0.0025)	0.003031			
s for (2) #5		14.87535			

Spot Check For Shear (SW 3 @ Level 30)					
Variable	Equation	Value			
Vu (k)		783.2			
t (in)		18			
lw (in)		240			
hw (in)		165			
d	0.8*lw	192			
f'c (psi)		8000			
fy (ksi)		60			
phi		0.75			
Max phi Vn	phi*10*(f'c)^0.5 *t*d	2318.355	OK		
Vc (k)	3.3*(f'c)^0.5 *t*d	1020.076			
Phi Vc (k)		765.0572			
Provision	Design w/ 11.10.9				
Req'd Vs	(Vu/phi)-Vc	24.19034			
Req'd Av/s	Vs/(fy*d)	0.0021			
s for (2) #5		295.2583	Use	10	
rhot	Av/(s*t)	0.003444	>	0.0025	OK
rhol	0.0025+0.5*(2.5-(h/l))*(rhot-0.0025)	0.003356			
s for (2) #5		13.4363			

Spot Check For Shear (SW 19 @ Level 40)					
Variable	Equation	Value			
Vu (k)		1285			
t (in)		16			
lw (in)		780			
hw (in)		165			
d	0.8*lw	624			
f'c (psi)		6000			
fy (ksi)		60			
phi		0.75			
Max phi Vn	phi*10*(f'c)^0.5 *t*d	5800.18	OK		
Vc (k)	3.3*(f'c)^0.5 *t*d	2552.079			
Phi Vc (k)		1914.059			
Provision	Design w/ 11.10.9				
Req'd Vs	(Vu/phi)-Vc	----			
Req'd Av/s	Vs/(fy*d)	----			
s for (2) #5		----	Use	10	
rhot	Av/(s*t)	0.003875	>	0.0025	OK
rhol	0.0025+0.5*(2.5-(h/l))*(rhot-0.0025)	0.004073			
s for (2) #5		11.06982			

Spot Check For Shear (SW 10 @ Level 40)			
Variable	Equation	Value	
Vu (k)		158.67	
t (in)		18	
lw (in)		120	
hw (in)		165	
d	0.8*lw	96	
f'c (psi)		6000	
fy (ksi)		60	
phi		0.75	
Max phi Vn	phi*10*(f'c)^0.5 *t*d	1003.877	OK
Vc (k)	3.3*(f'c)^0.5 *t*d	441.706	
Phi Vc (k)		331.2795	
Provision	Design w/ min shear rein.		

Spot Check For Shear (SW 2 @ Level 40)					
Variable	Equation	Value			
Vu (k)		513.26			
t (in)		18			
lw (in)		240			
hw (in)		165			
d	0.8*lw	192			
f'c (psi)		6000			
fy (ksi)		60			
phi		0.75			
Max phi Vn	phi*10*(f'c)^0.5 *t*d	2007.755	OK		
Vc (k)	3.3*(f'c)^0.5 *t*d	883.412			
Phi Vc (k)		662.559			
Provision	Design w/ 11.10.9				
Req'd Vs	(Vu/phi)-Vc	----			
Req'd Av/s	Vs/(fy*d)	----			
s for (2) #5		----	Use	10	
rhot	Av/(s*t)	0.003444	>	0.0025	OK
rhol	0.0025+0.5*(2.5-(h/l))*(rhot-0.0025)	0.003356			
s for (2) #5		13.4363			

Spot Check For Shear (SW 19 @ Level 51)					
Variable	Equation	Value			
Vu (k)		4163.32			
t (in)		14			
lw (in)		780			
hw (in)		192			
d	0.8*lw	624			
f'c (psi)		8000			
fy (ksi)		60			
phi		0.75			
Max phi Vn	phi*10*(f'c)^0.5 *t*d	5860.287	OK		
Vc (k)	3.3*(f'c)^0.5 *t*d	2578.526			
Phi Vc (k)		1933.895			
Provision	Design w/ 11.10.9				
Req'd Vs	(Vu/phi)-Vc	2972.567			
Req'd Av/s	Vs/(fy*d)	0.079395			
s for (2) #5		7.809008	Use	10	
rhot	Av/(s*t)	0.004429	>	0.0025	OK
rhol	0.0025+0.5*(2.5-(h/l))*(rhot-0.0025)	0.004673			
s for (2) #5		8.291694			

Appendix G – ETABS Output for Case 2 Wind

Shear Wall Results From ETABS for Case 2 Wind																	
Wind Direction	Shear Wall	Base				Level 15				Level 28				Level 29			
		e=+0.15B		e=-0.15B		e=+0.15B		e=-0.15B		e=+0.15B		e=-0.15B		e=+0.15B		e=-0.15B	
		Shear (k)	Moment (k-ft)	Shear (k)	Moment (k-ft)	Shear (k)	Moment (k-ft)	Shear (k)	Moment (k-ft)	Shear (k)	Moment (k-ft)	Shear (k)	Moment (k-ft)	Shear (k)	Moment (k-ft)	Shear (k)	Moment (k-ft)
	1	55.91	816.07	-292.54	-1373.973	184.95	1102.244	-327.51	-1367.379	305.78	3190.06	293.75	3232.694	346.87	3964.782	27.26	2780.876
	2	171.3	4736.925	-207.63	-5350.702	486.12	3261.843	-497.53	-3341.48	113.4	1283.648	-89.57	-958.686	424.11	3392.44	-374.46	-3067.234
	3	207.63	5350.733	-171.3	-4736.895	497.54	3341.523	-486.12	-3261.802	89.43	959.009	-113.54	-1283.318	374.97	3072.335	-423.6	-3387.333
	4	292.54	1373.978	-55.91	-816.065	327.51	1367.39	-184.95	-1102.232	-293.82	-3232.768	-305.84	-3190.112	-26.98	-2779.762	-346.62	-3963.651
	5	21.94	288.702	-97.04	-585.726	75.19	429.25	-134.33	-571.149	77.19	327.182	51.48	-34.872	127.22	539.776	-12.62	-304.616
	6	51.88	401.996	-1.58	-271.291	120.58	532.188	-71.32	-437.635	11.78	204.959	-39.54	-210.941	62.79	460.435	-97.97	-422.388
	7	1.58	271.295	-51.88	-401.993	71.32	437.642	-120.58	-532.181	39.5	210.902	-11.82	-204.992	98.12	423.133	-62.64	-459.679
	8	97.04	585.729	-21.94	-288.699	134.33	571.155	-75.19	-429.244	-51.51	34.866	-77.23	-327.191	12.8	305.48	-127.04	-538.94
	9	-21.94	-288.7	97.04	585.728	-75.19	-429.247	134.33	571.154	-77.21	-326.937	-51.51	35.217	-127.04	-538.848	12.87	305.927
	10	-51.88	-401.993	1.58	271.295	-120.58	-532.184	71.32	437.641	-11.81	-204.973	39.51	210.921	-62.71	-460.047	98.08	422.935
	11	-1.58	-271.292	51.88	401.996	-71.32	-437.64	120.58	532.184	-39.53	-210.907	11.79	204.985	-98.03	-422.695	62.77	460.298
	12	-97.04	-585.726	21.94	288.703	-134.33	-571.153	75.19	429.247	51.49	-34.87	77.19	327.186	-12.71	-305.045	127.17	539.553
	13	-55.91	-816.067	292.54	1373.977	-184.95	-1102.239	327.51	1367.386	-305.82	-3188.462	-293.81	-3230.437	-347.84	-4000.028	-28.63	-2830.657
	14	-171.3	-4736.905	207.63	5350.731	-486.12	-3261.832	497.53	3341.497	-113.46	-1283.459	89.49	958.953	-423.9	-3390.108	374.74	3070.527
	15	-207.63	-5350.71	171.3	4736.928	-497.54	-3341.514	486.12	3261.814	-89.49	-958.7	113.45	1283.754	-374.75	-3070.255	423.9	3390.27
	16	-292.54	-1373.974	55.91	816.07	-327.51	-1367.389	184.95	1102.235	293.8	3232.756	305.81	3190.095	27.06	2780.19	346.73	3964.256
	17	80.53	184359.514	5386.09	399391.859	194.85	85580.136	3518.49	88106.072	-1708.09	12993.234	-1509	-9498.202	2146.7	97043.425	4225.57	98712.594
	18	2130.32	245328.112	3692.64	320208.047	2540.68	88430.479	2725.98	88336.307	-5400.46	-14732.445	-6493.42	-29513.849	-118.35	33849.68	-492.49	30318.447
	19	3692.64	320208.676	2130.32	245327.588	2725.97	88337.086	2540.68	88430.097	-6493.62	-29509.07	-5400.49	-14730.535	-491.56	30326.852	-118.07	33853.104
	20	5386.09	399392.582	80.53	184359.078	3518.49	88107.149	194.85	85580.064	-1511.64	-9434.507	-1709.91	13039.044	4223.22	98879.109	2145.05	97159.857
		Level 30				Level 40				Level 50				Level 51			
E/W	Shear Wall	e=+0.15B		e=-0.15B		e=+0.15B		e=-0.15B		e=+0.15B		e=-0.15B		e=+0.15B		e=-0.15B	
		Shear (k)	Moment (k-ft)	Shear (k)	Moment (k-ft)	Shear (k)	Moment (k-ft)	Shear (k)	Moment (k-ft)	Shear (k)	Moment (k-ft)	Shear (k)	Moment (k-ft)	Shear (k)	Moment (k-ft)	Shear (k)	Moment (k-ft)
	1	54.28	395.5	-296.91	-1331.427	83.96	502.259	-146.47	-621.362	123.73	620.666	-19.96	-212.627	243.13	2305.703	144.32	1456.162
	2	348.72	2522.161	-314.49	-2199.952	231.73	1434.89	-236.04	-1465.918	188.33	1263.132	-149.51	-996.597	137.85	1608.302	-105.72	-1232.37
	3	314.51	2199.207	-348.69	-2522.9	236.05	1465.88	-231.72	-1434.923	149.52	996.477	-188.33	-1263.254	105.7	1232.051	-137.87	-1608.623
	4	296.98	1331.437	-54.22	-395.538	146.48	621.373	-83.96	-502.244	19.96	212.619	-123.73	-620.676	-144.33	-1456.141	-243.14	-2305.685
	5	81.14	407.717	-40.38	-218.285	34.15	203.276	-59.34	-248.381	65	259.46	4.78	-78.742	74.2	299.303	32.99	-49.847
	6	61.57	328.042	-67.84	-320.795	56.1	239.003	-36.46	-214.466	16.98	173.382	-54.29	-186.382	12.57	201.013	-37.96	-166.01
	7	67.82	320.601	-61.59	-328.229	36.46	214.468	-56.1	-238.998	54.29	186.384	-16.97	-173.383	37.96	166.009	-12.57	-201.018
	8	40.36	218.109	-81.16	-407.892	59.34	248.38	-34.14	-203.273	-4.78	78.743	-65	-259.461	-32.99	49.845	-74.2	-299.306
	9	-81.29	-408.1	40.17	217.744	-34.15	-203.29	59.34	248.361	-64.99	-259.436	-4.78	78.775	-74.19	-299.256	-32.98	49.914
	10	-61.6	-328.255	67.79	320.493	-56.1	-239.018	36.45	214.444	-16.97	-173.367	54.29	186.403	-12.57	-200.974	37.96	166.065
	11	-67.84	-320.778	61.55	327.98	-36.46	-214.465	56.1	239.001	-54.29	-186.383	16.97	173.385	-37.96	-165.986	12.58	201.05
	12	-40.39	-218.288	81.12	407.639	-59.34	-248.378	34.15	203.276	4.78	-78.741	64.99	259.464	33	-49.822	74.21	299.339
	13	-53.85	-392.174	297.51	1336.124	-83.97	-502.276	146.47	621.338	-123.72	-620.633	19.97	212.675	-243.12	-2305.471	-144.3	-1455.835
	14	-348.81	-2523.265	314.36	2198.392	-231.73	-1434.939	236.04	1465.849	-188.33	-1263.079	149.51	996.672	-137.84	-1608.147	105.73	1232.588
	15	-314.57	-2199.947	348.61	2521.855	-236.05	-1465.903	231.72	1434.891	-149.52	-996.448	188.32	1263.296	-105.69	-1231.928	137.88	1608.797
	16	-297	-1331.627	54.19	395.27	-146.47	-621.371	83.96	502.246	-19.96	-212.616	123.72	620.681	144.34	1456.182	243.15	2305.743
	17	1292.54	69459.628	3450.81	70767.061	160.82	-975.367	1403.43	-15222.152	1405.74	-47975.176	1674.42	-55840.642	-587.9	-25212.448	-385.05	-30365.087
	18	607.19	35777.515	508.17	32927.454	1079.48	-4460.798	1003.42	-10200.914	-697.97	-28397.864	-1051.41	-31832.328	-3172.93	-27538.553	-3455.37	-29483.211
	19	508.73	32930.634	607.48	35778.617	1003.4	-10202.967	1079.45	-4461.695	-1051.44	-31833.858	-697.97	-28398.372	-3455.54	-29484.803	-3172.98	-27539.158
	20	3450.61	70758.269	1292.59	69453.881	1403.41	-15227.002	160.8	-978.958	1674.41	-55844.822	1405.71	-47977.898	-385.35	-30368.064	-588.15	-25214.403

Shear Wall Results From ETABS for Case 2 Wind																	
Wind Direction	Shear Wall	Base				Level 15				Level 28				Level 29			
		e=+0.15B		e=-0.15B		e=+0.15B		e=-0.15B		e=+0.15B		e=-0.15B		e=+0.15B		e=-0.15B	
		Shear (k)	Moment (k-ft)	Shear (k)	Moment (k-ft)	Shear (k)	Moment (k-ft)	Shear (k)	Moment (k-ft)	Shear (k)	Moment (k-ft)	Shear (k)	Moment (k-ft)	Shear (k)	Moment (k-ft)	Shear (k)	Moment (k-ft)
	1	272.96	4773.592	499.11	6198.151	99.94	745.216	436.45	2369.269	-142.71	367.031	-132.77	356.105	180.2	3677.967	393.95	4477.43
	2	930.58	30411.439	1176.76	36978.89	656.23	4879.989	1302.53	9234.28	-634.14	-9305.835	-496.74	-7758.12	299.11	9763.596	831.96	14085.779
	3	930.58	30411.439	1176.76	36978.891	656.23	4879.989	1302.53	9234.281	-634.14	-9305.836	-496.74	-7758.126	299.11	9763.595	831.96	14085.774
	4	272.96	4773.592	499.11	6198.151	99.94	745.216	436.45	2369.27	-142.71	367.027	-132.78	356.086	180.2	3677.965	393.97	4477.416
	5	304.04	4570.747	381.3	5139.775	194.65	1143.305	332.23	1801.348	-217.02	-1757.384	-199.29	-1511.768	153	1631.891	246.49	2196.767
	6	370.48	4747.905	405.24	5186.223	245.56	1261.76	371.67	1899.778	-279.51	-1853.19	-244.97	-1572.414	106.27	1571.389	213.56	2161.305
	7	370.48	4747.905	405.24	5186.223	245.56	1261.76	371.67	1899.778	-279.51	-1853.191	-244.97	-1572.419	106.27	1571.388	213.55	2161.296
	8	304.04	4570.748	381.3	5139.775	194.65	1143.306	332.23	1801.349	-217.02	-1757.384	-199.29	-1511.765	153	1631.896	246.49	2196.79
	9	381.3	5139.775	304.04	4570.747	332.23	1801.349	194.65	1143.306	-199.29	-1511.783	-217.02	-1757.467	246.51	2196.863	152.98	1631.732
	10	405.24	5186.224	370.48	4747.904	371.67	1899.779	245.56	1261.76	-244.98	-1572.422	-279.51	-1853.194	213.57	2161.4	106.26	1571.378
	11	405.24	5186.224	370.48	4747.904	371.67	1899.779	245.56	1261.76	-244.97	-1572.422	-279.51	-1853.192	213.57	2161.399	106.26	1571.37
	12	381.3	5139.775	304.04	4570.747	332.23	1801.349	194.65	1143.306	-199.29	-1511.767	-217.02	-1757.385	246.51	2196.891	153	1631.877
	13	499.11	6198.151	272.96	4773.592	436.45	2369.271	99.94	745.216	-132.78	355.98	-142.71	366.467	394.07	4479.88	180.58	3690.111
	14	1176.76	36978.895	930.58	30411.438	1302.53	9234.287	656.23	4879.992	-496.76	-7758.081	-634.14	-9305.848	832.03	14086.48	299.13	9763.656
	15	1176.76	36978.894	930.58	30411.436	1302.53	9234.287	656.23	4879.993	-496.76	-7758.088	-634.14	-9305.882	832.03	14086.498	299.12	9763.746
	16	499.11	6198.151	272.96	4773.591	436.45	2369.271	99.94	745.217	-132.78	356.088	-142.71	367.032	394	4477.585	180.21	3678.015
	17	1719.99	70120.407	-1719.99	-70119.816	1089.12	1155.427	-1089.12	-1154.78	74.62	-6924.613	-75.39	6942.536	692.92	1012.887	-693.6	-964.682
	18	505.85	24420.828	-505.85	-24420.255	59.49	83.102	-59.49	-82.5	-359.91	-4702.081	359.8	4704.045	-123.71	-1003.484	124.13	1006.913
	19	-505.85	-24420.648	505.85	24421.204	-59.49	-82.921	59.49	83.456	359.88	4702.529	-359.94	-4701.684	123.81	1004.262	-123.59	-1002.813
	20	-1719.99	-70120.226	1719.99	70120.769	-1089.12	-1155.247	1089.12	1155.726	-74.74	6927.406	74.72	-6927.814	-693.03	-1005.066	693.02	1005.906
		Level 30				Level 40				Level 50				Level 51			
N/S	Shear Wall	e=+0.15B		e=-0.15B		e=+0.15B		e=-0.15B		e=+0.15B		e=-0.15B		e=+0.15B		e=-0.15B	
		Shear (k)	Moment (k-ft)	Shear (k)	Moment (k-ft)	Shear (k)	Moment (k-ft)	Shear (k)	Moment (k-ft)	Shear (k)	Moment (k-ft)	Shear (k)	Moment (k-ft)	Shear (k)	Moment (k-ft)	Shear (k)	Moment (k-ft)
	1	1.49	673.277	236.24	1830.765	32.63	137.888	190.36	908.932	12.49	-878.836	114.08	-296.517	-93.61	-553.922	-26.4	13.827
	2	435.68	8636.927	879.72	11819.818	261.46	1115.858	581.01	3124.922	47.62	-5988.101	284.9	-4414.421	-395.73	-8512.946	-227.75	-6570.426
	3	435.68	8636.926	879.72	11819.813	261.46	1115.857	581.01	3124.918	47.62	-5988.1	284.9	-4414.419	-395.73	-8512.946	-227.75	-6570.424
	4	1.49	673.285	236.25	1830.805	32.63	137.888	190.36	908.929	12.49	-878.836	114.08	-296.516	-93.61	-553.922	-26.4	13.83
	5	172.5	1411.701	253.92	1832.274	77.72	352.173	141.7	662.378	39.9	-742.074	82.37	-505.727	-113.41	-1420.018	-85.12	-1179.969
	6	155.22	1354.721	241.88	1790.486	98.85	388.115	162.04	699.092	11.75	-797.361	61.58	-546.699	-161.07	-1499.482	-126.25	-1246.912
	7	155.22	1354.72	241.88	1790.481	98.85	388.115	162.04	699.089	11.75	-797.36	61.58	-546.696	-161.07	-1499.482	-126.25	-1246.909
	8	172.5	1411.7	253.92	1832.273	77.72	352.172	141.7	662.375	39.9	-742.073	82.37	-505.725	-113.41	-1420.018	-85.12	-1179.967
	9	253.93	1832.269	172.55	1411.801	141.7	662.377	77.72	352.176	82.37	-505.727	39.9	-742.081	-85.12	-1179.971	-113.41	-1420.034
	10	241.88	1790.465	155.22	1354.759	162.04	699.092	98.85	388.12	61.58	-546.698	11.75	-797.364	-126.25	-1246.913	-161.07	-1499.494
	11	241.88	1790.463	155.22	1354.751	162.04	699.091	98.85	388.116	61.58	-546.698	11.75	-797.362	-126.25	-1246.912	-161.07	-1499.491
	12	253.92	1832.256	172.51	1411.732	141.7	662.377	77.72	352.174	82.37	-505.726	39.9	-742.075	-85.12	-1179.97	-113.41	-1420.027
	13	236.23	1830.572	1.36	672.17	190.36	908.933	32.63	137.894	114.08	-296.52	12.49	-878.848	-26.41	13.817	-93.62	-553.996
	14	879.73	11819.762	435.71	8637.174	581.01	3124.917	261.46	1115.866	284.9	-4414.444	47.62	-5988.138	-227.75	-6570.489	-395.73	-8513.051
	15	879.73	11819.739	435.7	8637.055	581.01	3124.916	261.46	1115.861	284.9	-4414.443	47.62	-5988.132	-227.75	-6570.487	-395.73	-8513.042
	16	236.25	1830.791	1.51	673.323	190.36	908.933	32.63	137.891	114.08	-296.518	12.49	-878.839	-26.41	13.829	-93.61	-553.934
	17	720.04	845.092	-720.17	-847.799	428.38	-4364.006	-428.39	4362.683	119.68	-2750.215	-119.67	2748.924	80.62	-1758.33	-80.69	1757.417
	18	-32.89	-788.063	33.1	789.439	-24.03	-1782.208	24.02	1781.396	-116.37	-1159.866	116.35	1159.195	-98.01	-680.731	97.93	680.064
	19	32.94	788.388	-32.88	-787.729	24.03	1782.028	-24.02	-1782.347	116.36	1159.706	-116.37	-1160.036	97.99	680.577	-98.03	-680.876
	20	-720.08	-845.567	719.97	845.293	-428.38	4363.809	428.39	-4363.719	-119.67	2749.987	119.69	-2750.124	-80.63	1758.171	80.65	-1758.258