third technical report

an analysis of alternate lateral systems for the new york times building

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IPD/BIM Structural Thesis

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EXECUTIVE SUMMARY

The purpose of the third technical report is to investigate an alternate lateral system as a replacement for the existing steel braced frame system of the New York Times Building. Bracing sizes were not available for the existing system, so three alternate systems were studied by the BIM Thesis structural students to assess their applicability as replacements:

1. Steel braced frames with outriggers and moment frames
2. Concrete shear wall core – Benjamin Barben
3. Concrete shear wall core with outrigger truss system – Andres Perez

Each analysis set forth with the goal of meeting a maximum period of 6.75 seconds, that of the existing system, as well as all required code limitations. The designs of these systems are still somewhat schematic and approximate; more consideration will have to be given to optimization and coordination with other trades. However, these designs provide the framework for future proposals and BIM alternatives.

This report will detail the first alternate system listed above, a steel frame system with concentric chevron braces, moment frames, and an outrigger and belt truss at the 36th floor (see Figure 1 on page 4). To simplify the analysis, the New York Times Building was divided into four vertical segments of 13 floors each to standardize the member sizes and masses within the segments. ETABS software was utilized for preliminary design and analysis of the system; member capacities, building and story drifts, and torsional and overturning effects were then checked with hand calculations. A pro-con analysis was also completed for the system in order to display the merits and drawbacks of the design. Through this analysis, the alternate bracing layout was found to be a very feasible alternative to the original system in terms of structural performance; the period of the building was lowered from the original 6.25 seconds in the North-South direction and 6.75 seconds in the East-West direction to a maximum period of 5.26 seconds in the North-South direction, and the controlling drift from wind was just under the H/450 limit given by Thornton Tomasetti. Additionally, it provides a penthouse level at the 52nd floor that was previously occupied by a two-story outrigger system and a mechanical room. This space would be attractive to the owner, as it could bring in more profit through the rental price. However, a problem with this new lateral system is that it changes the architectural aesthetic, mechanical layout, and planning of spaces. These effects can be accounted for in the future and will provide an interesting challenge, as the building will be undergoing many changes through the BIM Thesis. The design of this system was performed using the 13th Edition AISC Manual, ASCE 7-05, and ETABS software.

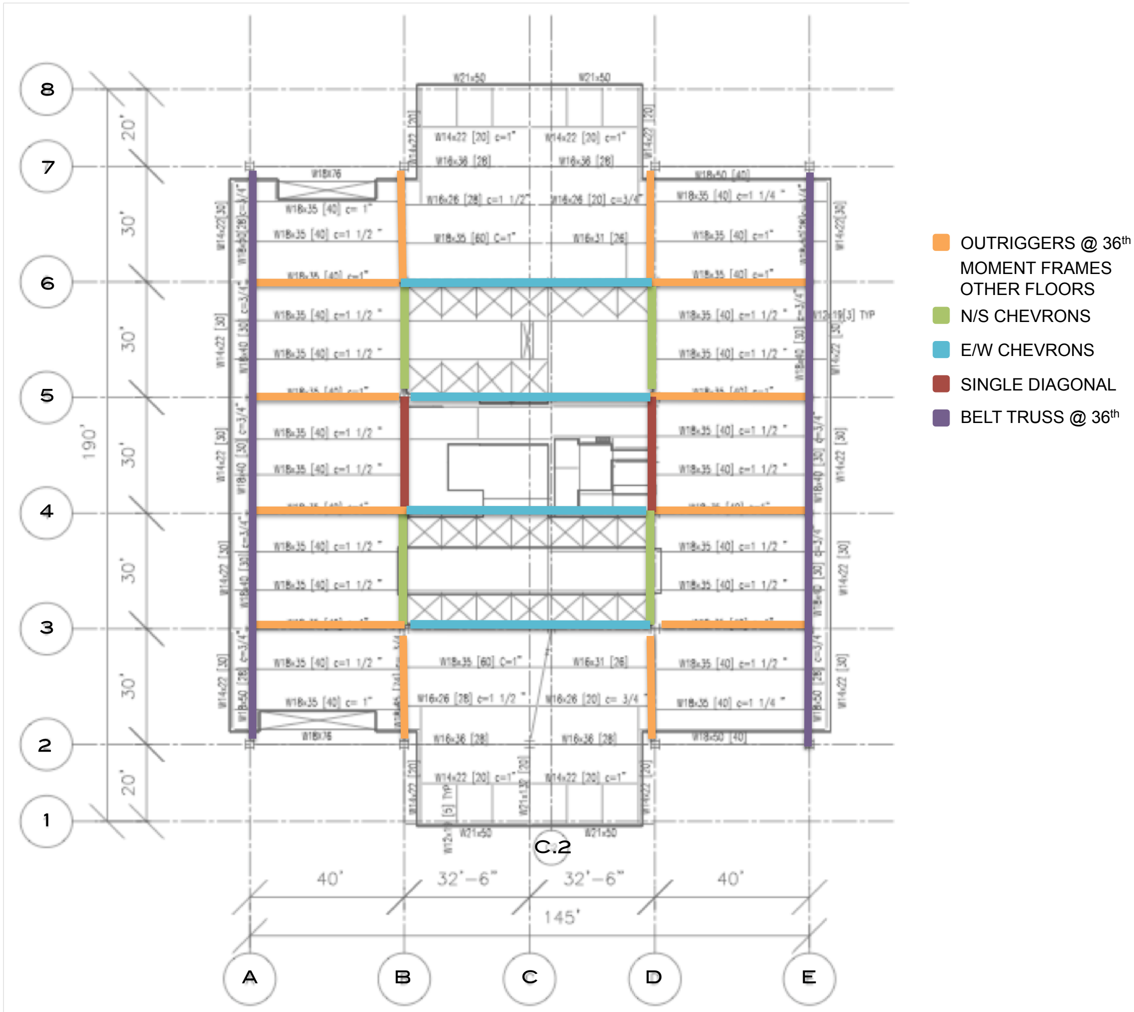


Figure 1: Bracing Layout of Steel Alternative

The two other alternate lateral systems also proved to be viable alternatives to the existing system. The second alternate system, a concrete shear wall core, met the design base requirements except for a slight increase in the period at a maximum of 7.709 seconds under seismic forces. This is an alternative that may have been considered today, because of a change in construction trade sequencing rules in New York City. However, Thornton Tomasetti ruled it out as an option because of increased schedule time. The system would change the architectural vision of transparency, as the walls in lower levels reach 2'-6" thick, decreasing the width of the corridors. In addition, the concrete would impact foundations with an increased system mass and would concentrate overturning at the central core. More detailed information can be viewed in Benjamin Barben's third technical report.

The third alternate lateral system, a concrete shear wall and outrigger system, was investigated by Andres Perez to determine the impact of outriggers on the thickness of the concrete core. It was found through analysis that the core could be decreased a maximum of 14 inches, from 2'-6" to 1'-4", with the addition of outrigger trusses on the 28th story and 51st story. Compressive strengths of concrete were also lowered from a maximum of 12000 psi in the purely concrete system to 10000 psi for this system. Drifts due to wind were affected significantly in the East-West direction, increasing from 10.8 inches to 16.9 inches, but not much in the North-South direction. In addition, the periods in both directions due to wind and seismic were slightly lower than those of the concrete core-only system. For a more detailed discussion, please view Andres Perez's third technical report.

INTRODUCTION

The 52-story New York Times Headquarters Building is located on Eighth Avenue between 41st and 42nd Streets. Home to the New York Times newsroom, 26 floors of Times administrative offices, and several law firms, it was intended to be a flagship building promoting sustainability, lightness, and transparency. The architectural façade reflects the ever-changing environment surrounding the building, an appropriate acknowledgement of the heart of New York City. Thornton Tomasetti worked closely with architect Renzo Piano to create a building that displayed not only transparency in

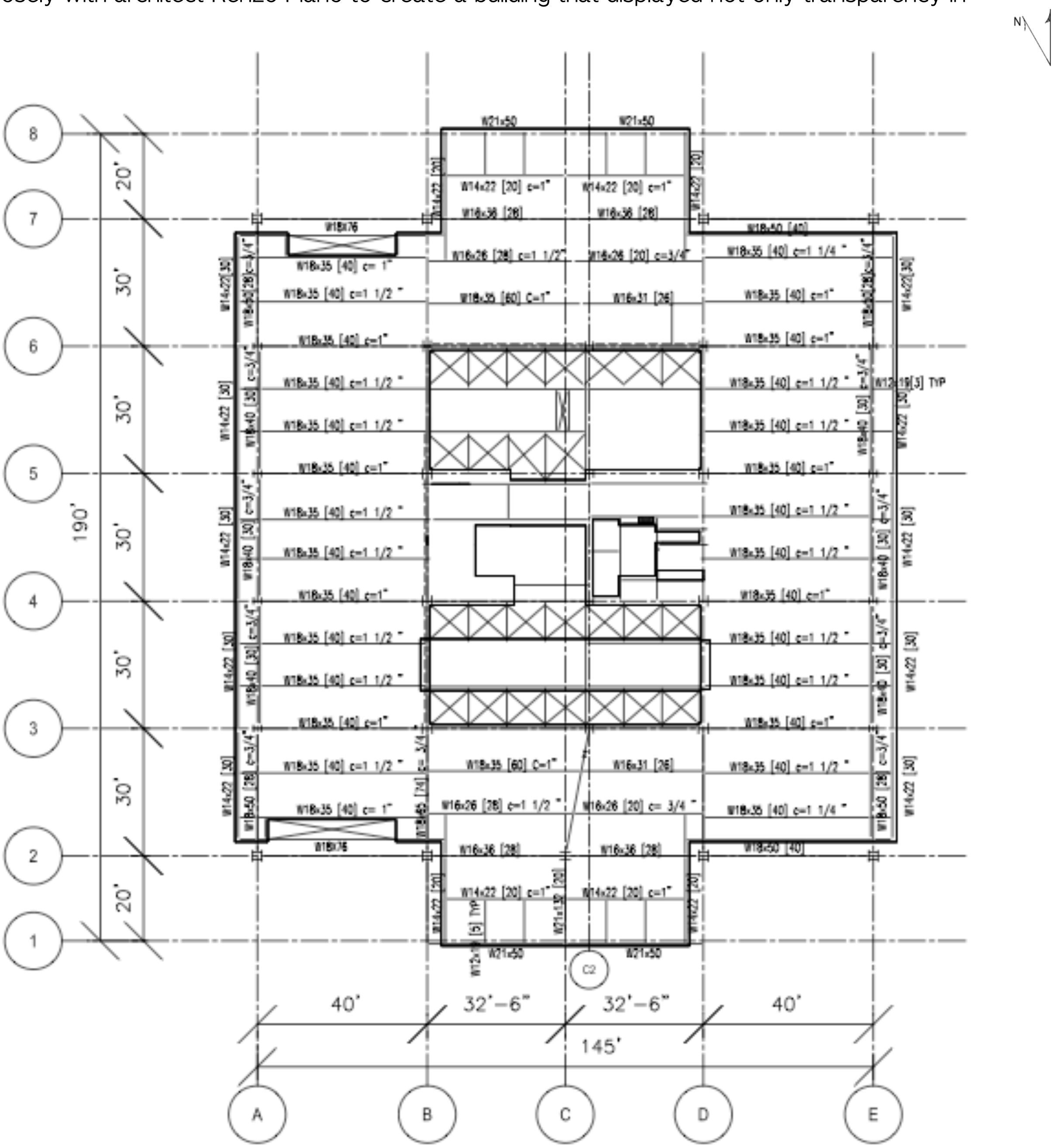


Figure 2: Typical tower framing plan

the media, but also structural transparency. For this reason, exterior columns, X-bracing, and beams were shifted outside of the façade, and the visual appearance of these elements and connections was given special attention.

The office floors are intended to be open plans, with minimal disturbance from columns and other structural elements. For this reason, two-story outriggers were used at mechanical levels (floors 28 and 51) to engage exterior columns in the lateral system and increase stiffness. Story heights average approximately 13'-9", and floor-to-ceiling heights are approximately 10'-9" due to the 16" allowance for an under-floor air distribution system and 20" structural depth.

In the alternate system analysis, the outriggers at the 51^{st} floor were removed and the 28^{th} floor outriggers were shifted to the 36^{th} floor for more optimized drift control. A belt truss was designed for controlling differential deflection due to the outriggers on the 36^{th} floor. The exterior X-braces were also removed from the structural analysis; the new design was conceived to optimize the sizes of the interior braces and eliminate the need for the expensive exterior braces, while creating a profit-increasing penthouse level. Additionally, beams already playing a role in the gravity system were utilized for added stiffness in moment frames on either end of the braced frames. The location of all of these elements can be seen in Figure 1 on page 4.

CODES AND REFERENCES

Original Design Codes:

National Model Code:
- 1968 Building Code of the City of New York

Structural Standards:
- ASCE 7-98, Minimum Design Loads for Buildings and other Structures

Structural Design Codes:
- AISC – LRFD, Steel Construction Manual, 2nd Edition, American Institute of Steel Construction, 1998
- National Building Code of Canada, 1995
- Uniform Building Code, 1997
- ACI 318-95, Manual of Concrete Practice, American Concrete Institute

Design Deflection Criteria:

Lateral Deflections:
- Total building sway deflection for 10-year wind loading is limited to H/450.

Thermal Deflections:
- The shortening and elongating effects due to temperature changes are designed to L/300.
- At this point in time additional gravity and lateral deflections were not disclosed.

Thesis Design Codes:

National Model Code:
- 2006 International Building Code

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

Structural Design Codes:
- AISC, Steel Construction Manual 13th Edition
- ACI 318-08, Building Code Requirements for Structural Concrete
- PCI Design Handbook, Precast and Prestressed Concrete, 6th Edition via Nitterhouse Concrete Products

MATERIAL STRENGTHS

Structural Steel:

Wide Flange Shapes……………………………………………ASTM A572 or A992, Grade 50
Built-Up Sections……………………………………………ASTM A572, Grade 50 & Grade 42
HSS Shapes……………………………………………………………ASTM A500 Grade B
Diagonal & X-Braced Rod………………………………………………ASTM A572, Grade 65
Connection Plates……………………………………………………………………ASTM A36

Concrete:

Caissons………………………………………………………………………………f'_c = 6000 psi
Spread Footings……………………………………………………………………f'_c = 6000 psi
Slabs on Deck (normal weight concrete) U.N.O……………………………f'_c = 4000 psi
Concrete Shear Walls…………………………………………………f'_c = 4000-12000 psi

Metal Decking:

3" Composite Deck………………………………………………………………F_y = 40 ksi

At this point in time, the designer did not disclose shear stud, weld, bolt, and reinforcement strengths.

STRUCTURAL SYSTEM

Foundation

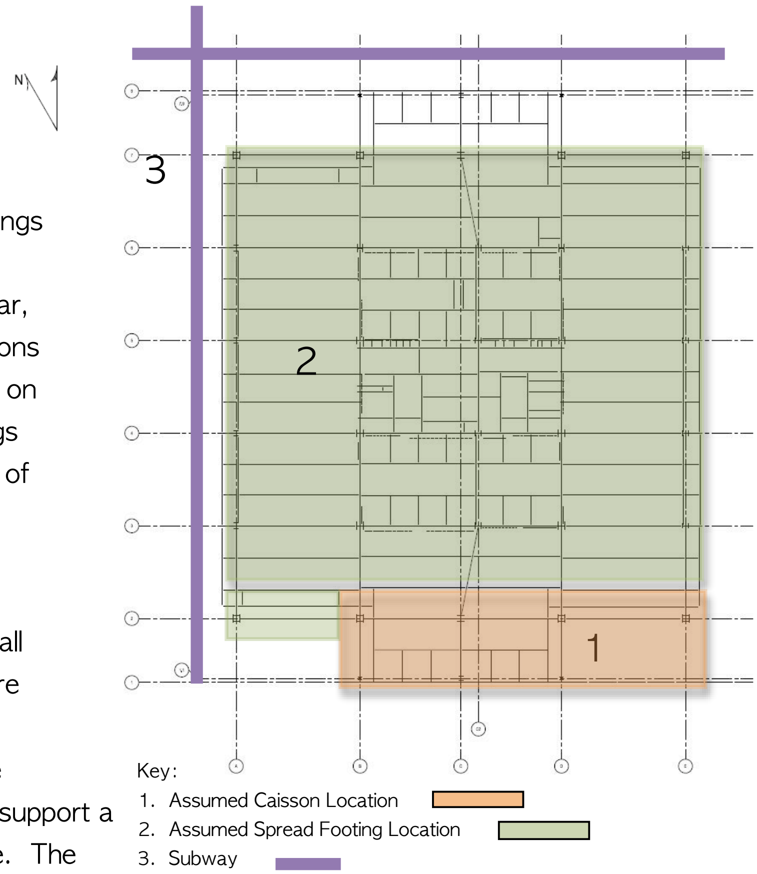


Figure 3: Foundation locations

The foundation of the New York Times Headquarters 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 20 tons per square foot rock; in this area, indicated on Figure 3 in green, 6,000-psi spread footings were used under each column (dimensions of footings not disclosed by the design team). However, at the southeast corner of the tower, the rock only has 8 tons per square foot capacity. At the seven columns that fall within this area, indicated in orange on Figure 3, 24-inch diameter concrete-filled steel caissons were used to transfer loads to the rock below. Each caisson was designed to support a load of 2,400 kips with 6,000 psi concrete. The structural engineers did not disclose the depth of the caissons; it is only known that they extend until they reach rock with a bearing capacity of 20 tons per square foot or greater.

The New York City Subway passes below Eighth Avenue to the west and 41st Street to the north of the New York Times Building. However, this is not a major site restriction since the transit system is not directly beneath the structure.

Floor System

The floor system is a steel composite system with a typical bay size of 30'-0"x 40'-0", with 2½" normal weight concrete on 3" metal deck. Typical beam sizes are W18x35 with a 10'-0" typical spacing, bearing on W18x40 girders. The girders frame into the various built-up columns, box columns along the exterior and built-up non-box columns in the core. Framing of the core consists of W14 and W16 shapes for beams, which bear on W33 girders.

In the New York Times spaces, the structural steel is 16 inches below the finished floor to accommodate the under-floor air distribution plenum. Because the façade is transparent and office spaces are visible from the exterior, the architect wanted members passing through to the outside to line up with the perceived floors. To align the girder with the office floor level and not the level of the structure, engineers created a "dog leg" at the end of the girders on these floors. Figure 4 depicts the

Figure 4: 'Dog-leg' beam connection

dog leg during construction; an aluminum spandrel was used to mask the location of the girder, as shown in Figure 5. The top of steel of the girder is at the bottom of the spandrel in the figure, and the spandrel covers up the plenum.

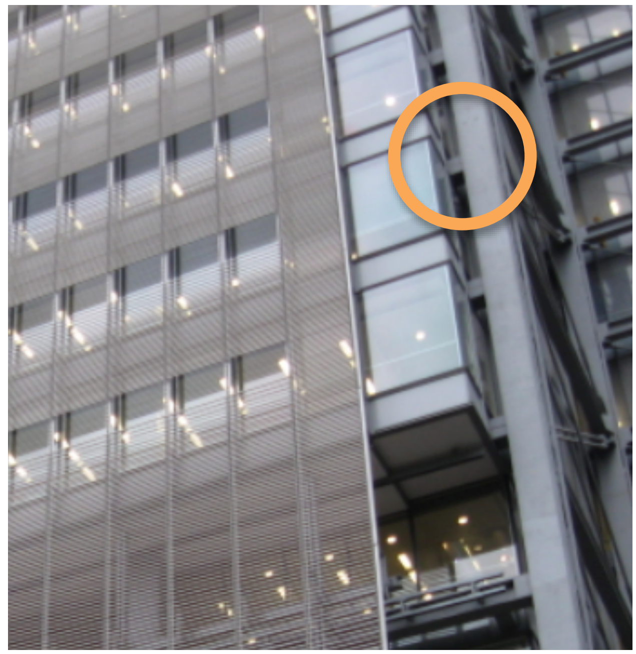


Figure 5: 'Dog-leg' beam connection

Columns

The 30" by 30" box columns (Figure 6), exposed at the exterior corners of the tower, as seen in Figure 4, consist of two 30-inch wide flange plates and two web plates inset three inches from the exterior of the column on either side. Each web plate decreases in thickness from 7 inches at the bottom of the building to adjust to the loads at each level. The flange plates decrease thickness from 4 inches to conform to the "lightness" of the architecture with an increase in elevation. Although the yield strength of the plates also varies with tower height, the strength was assumed to be a uniform 50 ksi for calculations. Interior columns are a combination of built-up sections and rolled shapes. Column locations stay consistent throughout the height of the building, spaced with the grid at 30 feet in one direction and 40 feet in the other. Every column is engaged in the lateral system via connections to bracing and outriggers; this system is described in more detail in the lateral system section.

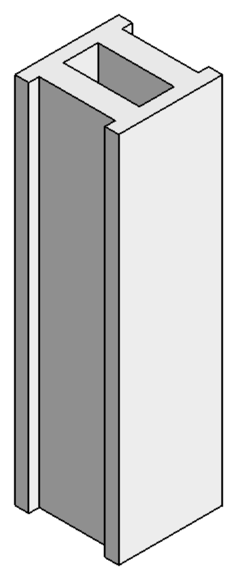


Figure 6: Box Column

EXISTING LATERAL SYSTEM

The main lateral load resisting system for the tower of the New York Times Building consists of a centralized steel braced frame core with outriggers without belt trusses on the two mechanical floors (Levels 28 and 51) to engage the exterior columns. The structural core consists of single diagonal bracing in the North-South direction between grids 4 and 5, concentric chevron bracing in both the North-South and East-West directions, and eccentric chevron bracing in the North-South direction between grids 5 and 6 (Figure 7 & 8, page 13). These braced frames surround the elevator shafts, MEP shafts, and stairwells. At this time, the member sizes of the braces have not been disclosed. The core configuration remains consistent from the ground level to the 27th floor as shown in Figure 7 on the next page. But above the 28th floor, some elevators were no longer required due to capacity. In order to optimize the rentable space on the upper levels of the tower, the number of bracing lines in the North-South direction was reduced from two to one (Figure 8, page 13). Please refer to Figures 10 and 11 on page 14 to view the typical core bracing elevations.

The outriggers on the mechanical floors consist of single diagonal braces extending from the core bracing to the exterior columns at grids 3, 4, 5, and 6 on either side of the core (seen in Figure 9, page 13). The outrigger system was designed to increase the efficiency and redundancy of the tower by engaging the perimeter columns in the lateral system.

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 drift and deflection criteria, the structural engineers utilized the double story steel rod X-braces instead of increasing the member sizes of the main lateral force resisting system. These X-brace locations can be seen in Figures 7-9 below on page 13, as well as in the photo on the cover page. The high strength steel rods transition from 2.5″ to 4″ in diameter and were prestressed to 210 kips, according to Thornton Tomasetti. This induced tensile load prevents the need for large compression members that would not conform to the architectural vision of the exterior.

Although the X-braces reduced 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 that 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 H/450 with a 10 year return period and a building acceleration of less than 0.025g for non-hurricane winds.

According to information obtained from the structural engineer, the podium of the New York Times Building was designed with a separate lateral system. Though the owner did not disclose information about the podium, it is known that the lateral system is comprised of concrete shear walls. The podium was not considered in this analysis.

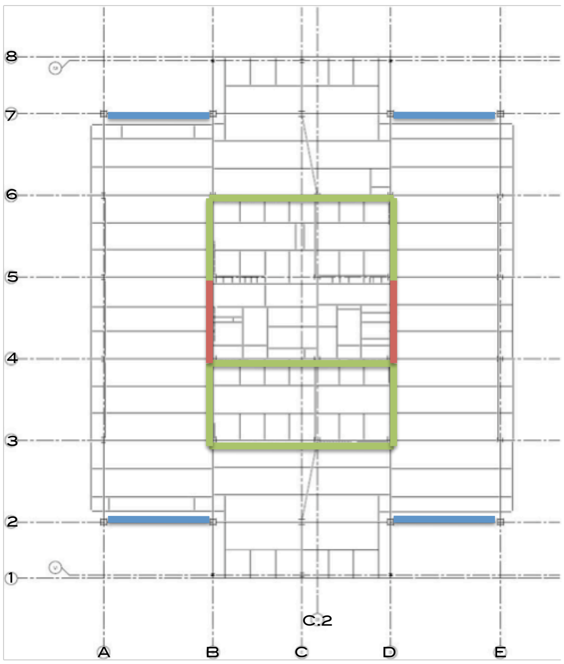


Figure 7: Typical Lateral System (Floors 1-27)

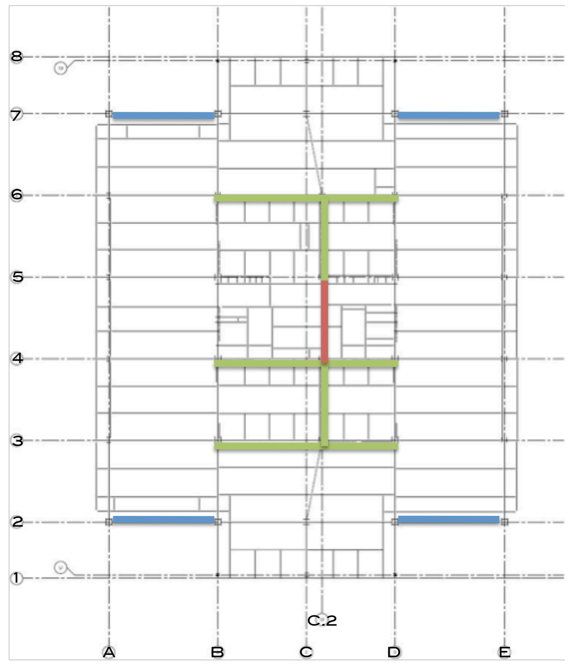


Figure 8: Typical Lateral System (Floors 29-50)

KEY:

- EXTERIOR X-BRACES
- CONCENTRIC & ECCENTRIC CHEVRONS
- SINGLE DIAGONAL BRACES
- OUTRIGGER

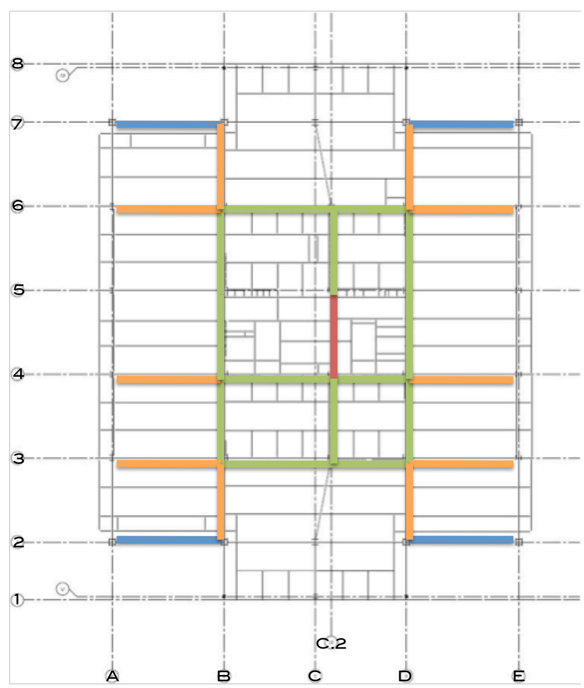


Figure 9: Mechanical Levels 28 & 51

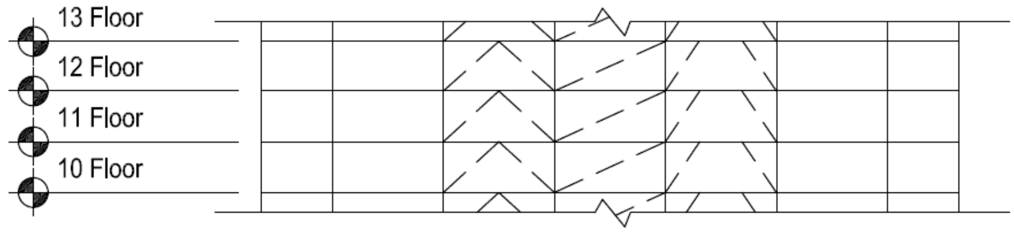


Figure 10: Typical Core N-S Bracing Elevation

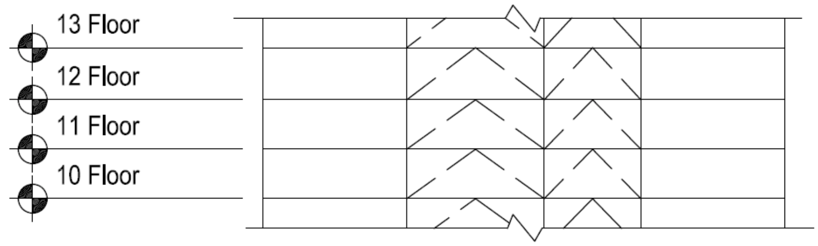


Figure 11: Typical Core E-W Bracing Elevation

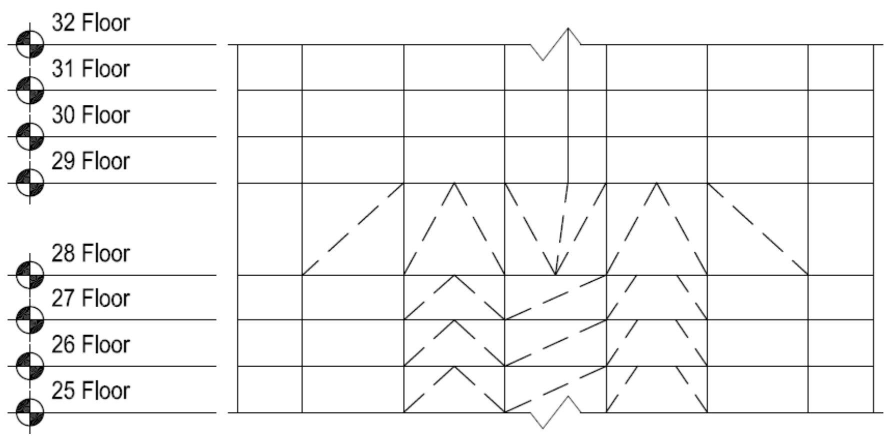


Figure 12: Typical E-W Outrigger Elevation

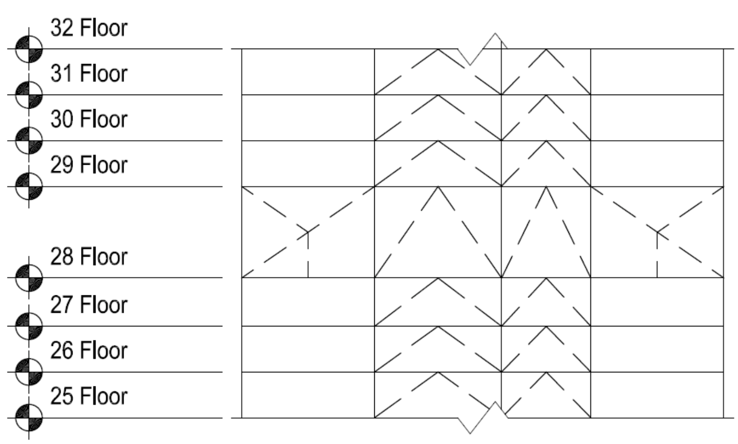


Figure 13: Typical N-S Outrigger Elevation

FIRST ALTERNATE SYSTEM: STEEL BRACED CORE WITH OUTRIGGERS

Design Process

The first alternative steel system considered was a moment frame system that employed two bracing lines in each direction: one interior and one exterior. However, through a preliminary analysis using ETABS, it was found that this system would not feasibly meet the desired period and drift limitations imposed by the existing system. Members would have to be very large, which would impose on the architecture as well as the ceiling heights of the spaces. For this reason, an adjusted alternative to the existing braced frame system was studied.

Loads

Gravity loads were based off of those found in the first technical report; however, the seismic weight was updated to include the change in weight of the braces with an increase in height of the building. Weights were determined by taking an average value at each of four vertical building segments, 14 stories each, and assigned to each story accordingly. These loads can be found in Table 1 below and as the seismic weights in Table 6 on page 17 below.

Gravity Loads for Analysis	
Dead Loads	*93-103 psf*
2.5″ slab on 3″ metal deck	53 psf
Ceiling	5 psf
MEP systems	20 psf
Bracing & gravity structure	15-25 psf
Façade Load	*25 psf*
Live Loads	*70 psf*
Open office	50 psf
Partitions	20 psf

Table 1: Gravity Loads

The lateral loads applied to the structure at each level are resisted by moment frames and steel braced frames connected to a rigid diaphragm that distributes the loads to each bay. The columns then carry these vertically down to the foundations, where the loads are dissipated by the soil or carried by the rock below.

Seismic calculations were updated after the first technical report based on the actual period of 6.75 seconds and the difference in floor self-weight, as shown in Tables 5 and 6 on pages 16 and 17. These updates led to a very slight decrease in the lateral seismic forces applied to the structure at the center of mass; these forces are tabulated below in Table 7. When analyzed, the combination 1.2D + 1.0E + 1.0L was used, since it would likely have the greatest combined impact on the structure in terms of seismic lateral forces and gravity loads. However, the forces due to wind were much greater than the seismic forces, and seismic forces were not assumed to control.

Soil Classification			
NYCBC:	2-65 (medium hard rock) 4-65 (soft rock)		
ASCE 7-05:	seismic design category C		conservative estimate
	Occ. Cat. III	*T 11.5-1*	
	Importance factor= 1.25		

Table 2: Seismic Design Category

Spectral Response Acceleration					
(using USGS Ground Motion Parameter Calculator)					
latitude: 40.756192	F_a= 1.2				
longitude: -73.990130	F_v= 1.7				
SDC 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*	use SDC C		

Table 3: Seismic Design Category Verification

Period of Building		
$T_a <= 0.8T_s =$	0.2199	
T_s	0.2749	S_{D1}/S_{DS}
$T_a= C_t*h_n^x =$	2.854	
C_t	0.02	*T 12.2.1.B*
x	0.75	*T 11.5-1*
h	745.7	

Table 4: Seismic Design Period

Seismic Base Shear			
$V = C_s * W$	1831.3 k	*12.8-1*	
C_s= min{	0.1119	$S_{DS}/(R/I)$	
	0.00634	$S_{D1}/(C_u*T*R/I)$	
	>	min 0.01	use C_s = 0.01
R	3.25	*T 12.2.1.B*	
I	1.25	*T 11.5-1*	
C_u	1.7	*T 12.12-1*	
T_{calc}	6.75 s	> C_u*T_a = 4.93 s	use C_u*T_a in C_s

Table 5: Seismic Design Base Shear

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	123	25	17639	12325839	25.20	25.2	7826106348
2	96625	123	25	10828	12155578	15.47	40.7	2.0103E+10
3	96625	123	25	10828	12155578	15.47	56.1	3.8304E+10
4	96625	123	25	10026	12135526	14.32	70.5	6.0245E+10
5	21550	123	25	9625	2891275	13.75	84.2	2.0502E+10
6	21550	123	25	9625	2891275	13.75	98.0	2.7744E+10
7	21550	123	25	9625	2891275	13.75	111.7	3.608E+10
8	21550	123	25	9625	2891275	13.75	125.5	4.5508E+10
9	21550	123	25	9625	2891275	13.75	139.2	5.603E+10
10	21550	123	25	9625	2891275	13.75	153.0	6.7645E+10
11	21550	123	25	9625	2891275	13.75	166.7	8.0353E+10
12	21550	123	25	9625	2891275	13.75	180.5	9.4155E+10
13	21550	123	25	10442	2911692	14.92	195.4	1.1114E+11
14	21550	123	25	8808	2870858	12.58	208.0	1.2416E+11
15	21550	121	25	9625	2837400	13.75	221.7	1.3947E+11
16	21550	121	25	9625	2837400	13.75	235.5	1.5731E+11
17	21550	121	25	9625	2837400	13.75	249.2	1.7622E+11
18	21550	121	25	9625	2837400	13.75	263.0	1.962E+11
19	21550	121	25	9625	2837400	13.75	276.7	2.1725E+11
20	21550	121	25	9625	2837400	13.75	290.5	2.3938E+11
21	21550	121	25	9625	2837400	13.75	304.2	2.6258E+11
22	21550	121	25	9625	2837400	13.75	318.0	2.8685E+11
23	21550	121	25	9625	2837400	13.75	331.7	3.122E+11
24	21550	121	25	9625	2837400	13.75	345.5	3.3862E+11
25	21550	121	25	9625	2837400	13.75	359.2	3.6611E+11
26	21550	121	25	9625	2837400	13.75	373.0	3.9468E+11
27	21550	121	25	9625	2837400	13.75	386.7	4.2431E+11
28	21550	118	25	9625	2783525	13.75	400.5	4.4639E+11
29	21550	118	25	9625	2783525	13.75	414.2	4.7757E+11
30	21550	118	25	9625	2783525	13.75	428.0	5.098E+11
31	21550	118	25	9625	2783525	13.75	441.7	5.4308E+11
32	21550	118	25	9625	2783525	13.75	455.5	5.7742E+11
33	21550	118	25	9625	2783525	13.75	469.2	6.1281E+11
34	21550	118	25	9625	2783525	13.75	483.0	6.4925E+11
35	21550	118	25	9275	2774775	13.25	496.2	6.8321E+11
36	21550	118	25	20183	3047483	28.83	525.0	8.401E+11
37	21550	118	25	9625	2783525	13.75	538.8	8.0805E+11
38	21550	118	25	9625	2783525	13.75	552.5	8.4982E+11
39	21550	118	25	9625	2783525	13.75	566.3	8.9264E+11
40	21550	118	25	9625	2783525	13.75	580.0	9.3651E+11
41	21550	113	25	9625	2675775	13.75	593.8	9.4345E+11
42	21550	113	25	9625	2675775	13.75	607.5	9.8765E+11
43	21550	113	25	9625	2675775	13.75	621.3	1.0329E+12
44	21550	113	25	9625	2675775	13.75	635.0	1.0791E+12
45	21550	113	25	9625	2675775	13.75	648.8	1.1263E+12
46	21550	113	25	9625	2675775	13.75	662.5	1.1746E+12
47	21550	113	25	9625	2675775	13.75	676.3	1.2238E+12
48	21550	113	25	9625	2675775	13.75	690.0	1.2741E+12
49	21550	113	25	9625	2675775	13.75	703.8	1.3254E+12
50	21550	113	25	10063	2686713	14.38	718.2	1.3857E+12
51	21550	113	25	9625	2675775	13.75	731.9	1.4334E+12
52	21550	113	25	9625	2675775	13.75	745.7	1.4878E+12
				ΣW	**183133**	**k**	$\Sigma w_i*h_i^k$	**2.7602E+13**

Table 6: Seismic Weights

Lateral Seismic Force	
k=	2.0
C_{vx}	F_x (kips)
0.0003	0.5192
0.0007	1.334
0.0014	2.541
0.0022	3.997
0.0007	1.360
0.0010	1.841
0.0013	2.394
0.0016	3.019
0.0020	3.717
0.0025	4.488
0.0029	5.331
0.0034	6.247
0.0040	7.374
0.0045	8.237
0.0051	9.254
0.0057	10.437
0.0064	11.692
0.0071	13.017
0.0079	14.414
0.0087	15.882
0.0095	17.422
0.0104	19.032
0.0113	20.714
0.0123	22.467
0.0133	24.291
0.0143	26.186
0.0154	28.153
0.0162	29.617
0.0173	31.686
0.0185	33.824
0.0197	36.033
0.0209	38.311
0.0222	40.659
0.0235	43.077
0.0248	45.330
0.0304	55.739
0.0293	53.613
0.0308	56.384
0.0323	59.225
0.0339	62.136
0.0342	62.596
0.0358	65.529
0.0374	68.528
0.0391	71.595
0.0408	74.729
0.0426	77.930
0.0443	81.198
0.0462	84.534
0.0480	87.936
0.0502	91.939
0.0519	95.105
0.0539	98.712
V= ΣF_x	**1831.3**

Table 7: Seismic Forces

Wind loads as outlined in the first technical report were used with a few slight updates to the overturning moments. Design variables for wind calculations according to ASCE 7-05 Section 6.5.4 are listed in the table below; a more detailed description of the calculations can be found in the first technical report. Cases 1 and 3 from ASCE were considered in this analysis; cases 2 and 4, which include accidental torsion, must be looked at in the future for a comprehensive design. For now, torsion was not included in calculations based on the symmetry of the building in both directions and the equal stiffness contribution of all braced frames in the East-West direction and the North-South direction. The values for the wind forces and overturning moments can be found in Table 9 on page 19 below. The Case 1 wind condition yielded higher drifts in both directions; these are explained in more detail under the analysis results below.

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
a	7.0	---	ASCE 7-05 6.5.6.6
z_g	1200	---	ASCE 7-05 6.5.6.6

Table 8: Wind Load Parameters

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.

Table 9: Wind Loads

Simplifying Assumptions

The basic goal for design was to meet the criteria presented by the existing system, at a minimum, while eliminating the need for the exterior X-braces and 51st story outriggers. These criteria included a maximum total building displacement of H/450 and a maximum period in each direction of 6.75 seconds. To begin the design, the building was segmented into 4 groups of 14 floors each. Within each group, member sizes and masses were standardized for simplification in the iterative design process; this way, the impact of different sizes and members could be seen more easily. Somewhat arbitrary bracing sizes were chosen in the beginning; these were later optimized based on member forces from the ETABS output, as shown in Table 10 below. Beams and columns were preliminarily assigned sizes based on the plans of the existing system, and beam sizes were later changed to limit drift and periods of vibrations. The column sizes originally modeled in ETABS were not changed after analysis; these can be seen in Table 10 below.

Size Standardization	
Bracing	
Floors 1-13	W14x283
Floors 14-27	W14x176
Floors 28-40	HSS16x16x1/2
Floors 41-52	HSS12x12x3/8
Beams	
All beams W30x116	
Box Columns	*all 30"x30", thicknesses vary:*
Floors 1-13	7" flange, 4" web
Floors 14-27	6" flange, 3.5" web
Floors 28-40	5" flange, 3" web
Floors 41-52	4" flange, 2.5" web

Table 10: Size Standardizations

ETABS Model & Iterations

The next step in the design process was to model the lateral system in ETABS. Since many elements contribute to the overall lateral resistance of the structure, these were modeled in steps to avoid errors. First, the concentric chevron braces and moment frames were input and analyzed, in the same layout as the original system. Floors were assumed to be perfectly rigid, and given a mass based on their floor level (floor weights are included in Table 6 on page 17, above). The beams and columns in the core on either side of the chevron braces were assumed to contribute as moment frames for added stiffness; these connections were modeled as perfectly rigid. A basement level with concrete walls was modeled to more accurately account for the shear at the first floor. One difference between this system and the original is that, above the 28th floor, the second North-South brace was not dropped out in anticipation of future changes in the office plans, and an additional brace was added in each direction. In the future, it will be necessary to consider dropping out braces from the core. Please see page 23 for typical bracing elevations in both the East-West (Figure 16) and North-South (Figure 17) directions, and Figure 15 for a typical plan view of the system.

After this initial analysis, beams were changed from the W18s and W24s of the original design to W30x116s on all levels. This beam size has a larger depth than those used originally, and the effect of this depth on other systems will have to be considered further. In addition, the beam sizes should be optimized further based on their participation in the moment frames and the braced frames.

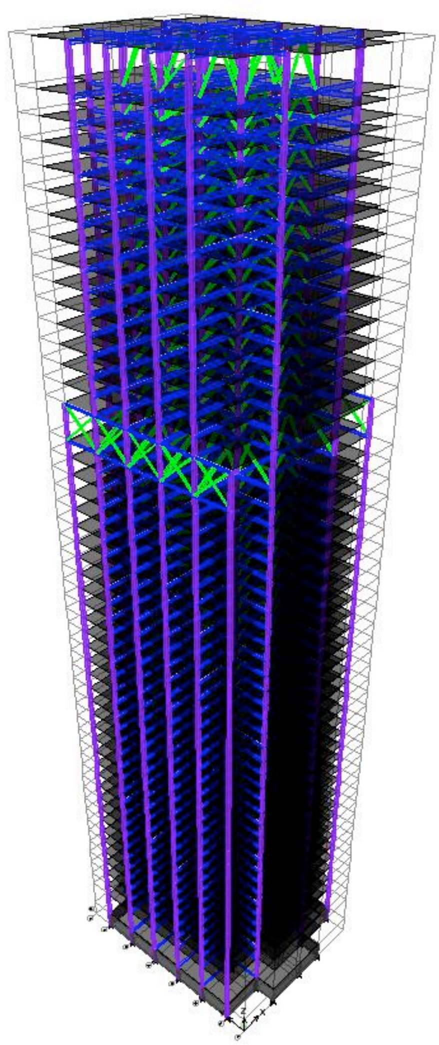


Figure 14: ETABS 3D Lateral Model

To attempt to lower the drift to an acceptable level, outriggers were added to the model at the 28th floor at the locations shown on page 13 in Figure 9. This system's periods and drifts were compared against the existing system's numbers, and they were found to be over the prescribed limits outlined above. Since the 28th floor location did not appear to be optimal for design considerations, the 36th floor location was optimized through iterations in ETABS. Other locations considered were floors 32 and 40; each of these locations yielded periods and drifts that were higher than the original values. These iterative values for period and drift are available upon request.

Even with the optimized outrigger location, the system was still not meeting the drift standards set forth for the design. To counter this drift, a belt truss was added at the 36th floor in the East-West direction along grids A and E to provide additional stiffness at that level. The truss was designed in SAP2000; diagonal member sizes were set as HSS14x14x3/8 based on the deflection of the truss under a point load matched to the vertical deflection of the column below the 36th floor. It can be viewed in Figure 31 of Appendix B. The axial forces in the members are checked below in Figure 24 of Appendix A.

After all contributing elements were modeled together in ETABS, periods were determined to be 5.17 seconds in the East-West direction, 5.26 seconds in the

North-South direction, and 3.92 seconds in the torsional direction. These periods were all well below the 6.75 second maximum; however, some member sizes are likely larger than necessary to meet the initial requirements, and further optimization would bring the period closer to this "limit." The final layout of the lateral system can be seen below in plan in Figure 15 and in elevation in Figures 16 and 17 on page 23. Final member sizes are tabulated in Table 10 on page 20.

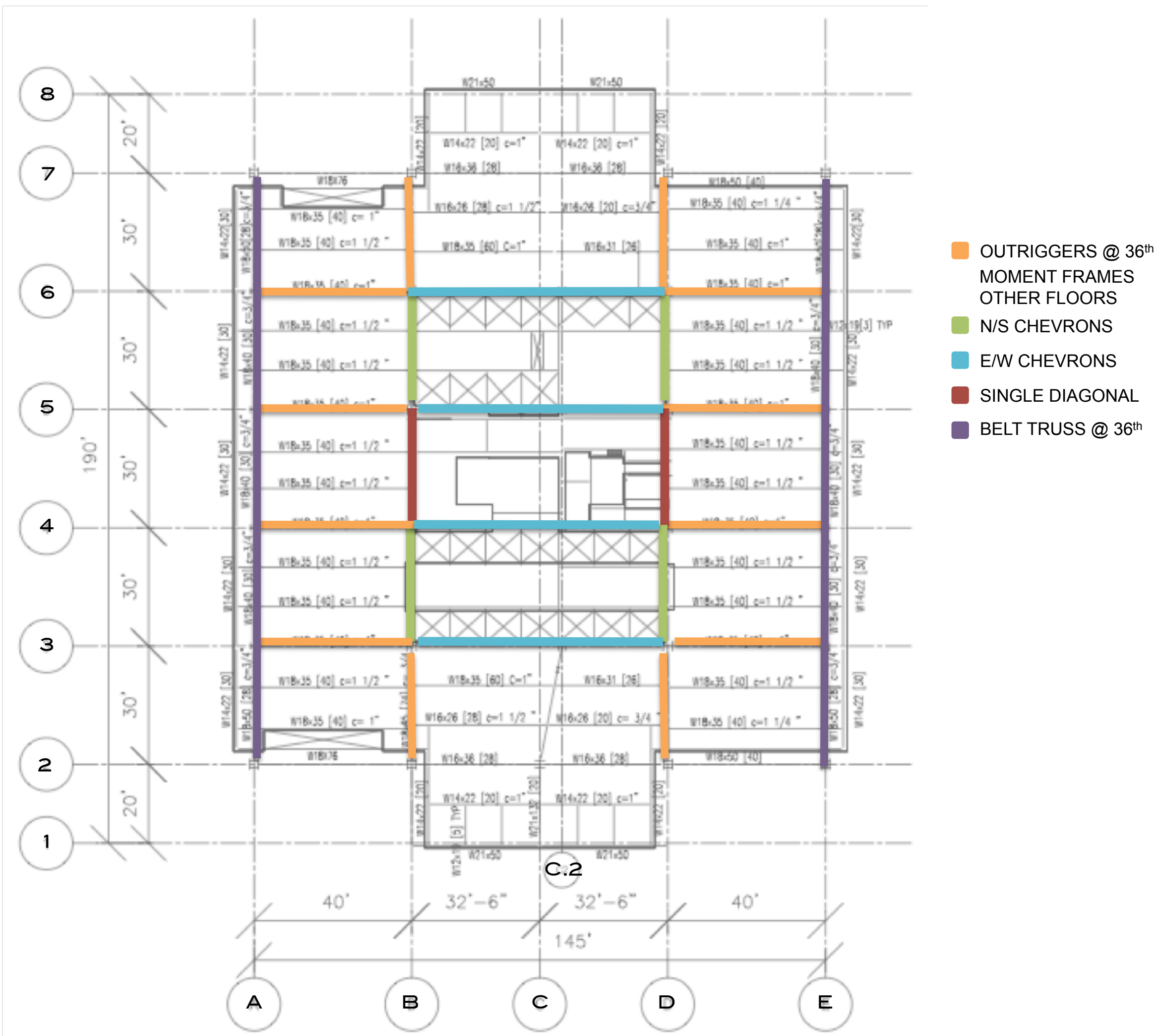


Figure 15: Lateral System Layout Plan

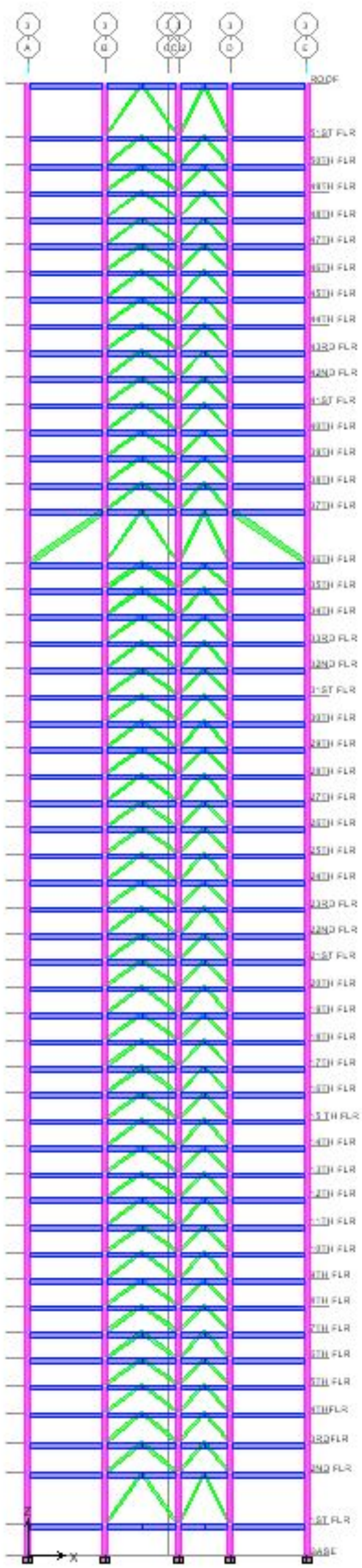


Figure 16: East-West Bracing Elevation

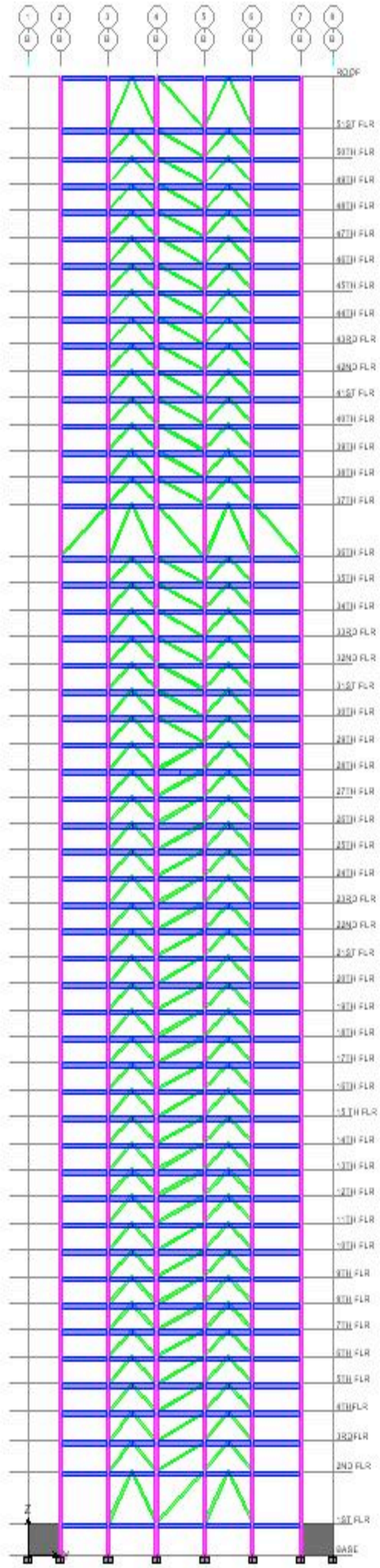


Figure 17: North-South Bracing Elevation

Analysis Results

After the initial design of the alternate system met the design requirements of a period less than 6.75 seconds and drifts less than H/450, critical members and sections within the system were checked for strength and serviceability requirements. Building periods were 5.17 seconds in the East-West direction, 5.26 seconds in the North-South direction, and 3.92 seconds in the torsional direction. The Case 1 wind load produced the highest building drift in both the North-South and East-West directions; these drifts were 16.7 inches and 19.8 inches, respectively. The drift in the East-West direction was just under the calculated H/450 limiting value of 19.9 inches, as shown by calculations in Figure 21 of Appendix A. ASCE 7-05 commentary permits the wind forces to be reduced using the combination 1.0D + 0.5L + 0.7W; for simplification in the model and checks, the wind values times the 0.7 factor were used without inclusion of dead and live loads. The Case 3 wind loading condition yielded drifts of 10.5 inches in the North-South direction and 13.8 inches in the East-West direction. With further investigation into this system, all load combinations must be factored into the design and analysis of each member.

Seismic story drifts were checked against the maximum requirements of ASCE 7-05; the allowable maximum values of 2.47 inches for the typical 13'-9" story height and 4.96 inches for the double-height outrigger level were only met at one of the four locations checked. Levels 14 and 37 were checked for both North-South and East-West directions, as they are representative of the typical story height and mechanical level height, respectively. All drifts at the locations checked were determined to be acceptable. More detailed drift calculations can be viewed in Figures 21 and 22 of Appendix A. In addition, the overall building drift was still well under the H/450 serviceability limit. The drifts at each level must be checked if this system is to be utilized in the BIM Proposal.

The strengths of certain critical members were also checked against allowable values from AISC and hand calculations. First, box column axial and moment capacities were checked at each of four representative floors, 1, 14, 28, and 41, using the controlling load combination of 1.2D + 0.5L + 1.6W from the ETABS model. This combination was used simply because it had the largest amplification factors applied to the wind and dead loads. Factored dead and live loads were not included into the axial load values from ETABS; only the self-weights of the lateral members and floor diaphragms played a role in the axial loads. In addition, k was assumed as 1.0 for this analysis. This factor along with additional loads and combinations will need to be included in a future analysis. At the first floor, the columns reach up to 53% of their total capacity carrying only 1.6W and unfactored self-weight. At the 14th floor, the controlling column axial force from ETABS is only 23% of the total capacity of the column. At the 28th and 41st floors, columns carry 12% and 4% of their total capacity from the 1.6W and self-weight, respectively. All columns were found to have sufficient axial capacities for wind and self-weight only, based on the material properties calculated for the box columns. These calculations can be found in Figures 23 and 24 of Appendix A.

The flexural capacity of each column was checked separately from axial, also using the 1.6W plus self-weight values from ETABS. Interactions between axial compression and flexure will clearly have a significant impact on the adequacy of the column capacity; this needs to be investigated in more detail as well. However, based purely on flexural capacity, all columns were found to be well within the total allowable flexural strength. The column checked at the 35th floor, at grid intersection 3-A, carries a large moment due to the outrigger above; 1.6W only accounted for 22% of the total flexural capacity at this level. Other moment demands on upper floors were much lower, and were not considered critical enough to be checked. At the first floor, there is also a large moment due to the wind loads on the structure. This moment, again considering 1.6W, only accounted for 19% of the total flexural capacity of the column. For detailed calculations on flexural capacities, as well as design assumptions, please view the calculations in Figures 23 through 25 in Appendix A.

The shear from ETABS in the columns at levels 1 and 35 were also checked using 1.6W and the unfactored self-weight and were found to be significantly lower than the shear capacity of these sections. This is not surprising given the large web area of the box columns. At the first floor, the actual shear from the model was found to be 191 kips, while the capacity of the column is 3840 kips. At the 35th floor, the model shear was found to be 303 kips, as compared to a much larger shear capacity of 3240 kips. The shear in the columns will also be affected by the addition of factored dead and live gravity loads; however, it is expected that combined axial and flexure will still govern the design of these members. For the shear calculations, please view Figure 26 in Appendix A.

Beam members within the moment frames were also checked for shear and flexural strengths at the 14th and 41st levels in each direction. All beams were preliminarily sized as W30x116s; the load combination again was solely 1.6W plus unfactored self-weight. Beams all met the total loads obtained from ETABS, with flexure controlling the design: at the 41st level, beams carried around 65% of their total flexural capacity. While this is acceptable with 1.6W, the addition of factored gravity loads will most likely prove that this member is under-designed. At the 14th level, beams are almost at their total capacity of 1420 foot-kips (with 1390 foot-kips carried in the North-South direction and 1124 foot-kips carried in the East-West direction) and will definitely need to be redesigned to accommodate the additional gravity loads of the structure. It is recognized that the factored dead and live loads will have a large impact on these members, as they also participate in the gravity system of the structure. This must be taken into consideration in the future. These checks can be found in Figure 27 of Appendix A.

Concentric chevron braces were also checked for axial capacity, using 1.6W plus self-weight, at four critical levels. At floors 1, 14, 28, and 41, the controlling axial forces were all found to be in the North-South direction. However, only the braces at floors 1, 28, and 41 had sufficient axial capacity.

These capacities were very close to the axial forces obtained from the ETABS model, suggesting that the braces are somewhat optimized as modeled. However, at the 14th floor, with a ϕP_n of 1200 kips, the W14x176 brace did not meet the required capacity of 1478 kips. For a more detailed view of the axial capacity calculations, please see Figure 28 of Appendix A.

It was also necessary to check the outriggers for capacity, as these members are very critical to the performance of the structure. The W36x247 was checked for the same load combination as all of the other members, 1.6W plus self-weight, at gridline 3 in the East-West direction and gridline B in the North-South direction. The combined flexural and axial capacity was checked assuming the single-diagonal outriggers are braced in the center, as they are in the original lateral system. The interaction equation for the East-West outrigger yielded a value of 1.53, which is significantly greater than 1.0. This indicates that the outrigger must be significantly upsized to carry the capacity afforded to it in ETABS. For the North-South direction, the interaction equation yielded only 0.486, which is significantly less than 1.0, indicating that this outrigger could potentially be decreased in size. For the interaction calculations, please see Figure 29 of Appendix A.

At this time, torsion was not included in the design or analysis of this system. This is simply because the relative stiffnesses of the braces in the East-West direction are all equal, and they are arranged symmetrically around the center of mass. Likewise, the relative stiffnesses of the braces at each level in the North-South direction are equal, and these braces are also arranged symmetrically about the center. Of course, it is recognized that accidental and inherent torsion will play some role in the addition of loads to the structure, and these must be analyzed in detail in the future.

Overturning calculations were performed for both directions of the building, as shown in Figure 30 of Appendix A. The weight of the building proved to be more than sufficient to prevent overturning of the structure. However, this moment would undoubtedly have an effect on foundations; the owner has not currently disclosed detailed information on foundation sizes or capacities, but the foundations will have to be checked upon further analysis.

This is by no means the most economical, optimized system. It simply provides a reference point for future consideration of an alternate steel system. Some additional elements left out of this initial analysis were: the inclusion of P-Delta effects in ETABS and hand calculations, other load combinations that could possibly control, and optimization of lateral members. These must all be included in a future in-depth analysis of the lateral system.

SECOND ALTERNATE SYSTEM: CONCRETE SHEAR WALL CORE

Summary Results

A concrete shear wall lateral system was not originally considered for the design because of restrictions on construction sequencing between steel and concrete trades in New York City. However, these restrictions have since been lifted, allowing a concrete core to compete with a steel braced-frame system. The main goals of meeting drift criteria of H/450 and a maximum period of 6.75 seconds were set for this design as well. However, replacing the steel braced-frame system is a concrete shear wall core with layout shown in Figure 18 on page 28 below; this plan stays consistent throughout the height of the structure. Thicknesses and compressive strengths of the walls, shown in red on Figure 18, vary with height of the building, as shown in Table 11 to the right. Returns running in the North-South direction are 2'-6" thick for the entire height of the building. Coupling beams, indicated in Figure 18 in yellow, are 3'-0" deep and 2'-6" wide. This system was also analyzed in ETABS and represents a preliminary design for the core. The merits of a concrete system are found within its material properties; concrete provides a greater stiffness than steel and, as shown in Benjamin Barben's third technical report, eliminates the need for any additional outriggers. However, one drawback is the mass of the walls; the architectural vision of transparency would have to be redefined with the use of a concrete core. This vision would include a more visible structural system in replace of a visibly transparent building, as the walls in lower levels reach 2'-6" thick and would impact the width of the corridors. In addition, the concrete would impact foundations with an increased system mass. For a detailed analysis of this system, please see Benjamin Barben's Third Technical Report.

Concrete Shear Wall Core Data	
Thicknesses	
Floor 1 - 20	2'-6"
Floor 21 - 40	2'-0"
Floor 41 - 52	1'-6"
Compressive Strengths	
Floor 1 - 10	12,000 psi
Floor 11 - 30	10,000 psi
Floor 31 - 52	8,000 psi
Periods of Vibrations	
Seismic	
East-West	7.709 s
North-South	6.893 s
Torsional	3.690 s
Wind	
East-West	6.528 s
North-South	5.926 s
Torsional	3.265 s
Building Drifts	
Seismic	
East-West	5.44"
North-South	7.45"
Wind	
East-West	10.76"
North-South	16.76"

Table 11: Shear Wall Core Data

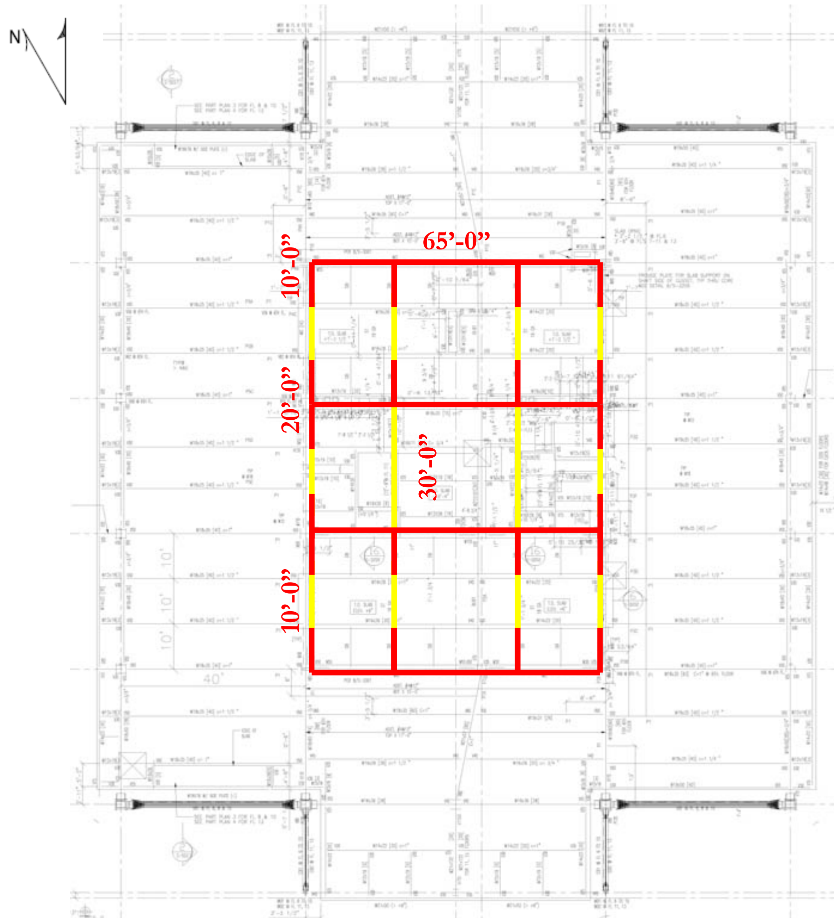


Figure 18: Concrete Shear Wall Core Layout

THIRD ALTERNATE SYSTEM: CONCRETE SHEAR WALL CORE WITH OUTRIGGERS

Summary Results

In an attempt to limit the thickness of concrete core shear walls, this third alternate system was devised. An outrigger truss system was added at the 28th and 51st stories, where the outriggers are located in the original lateral system (indicated in yellow in Figure 20), to control the drift and downsize the core member sizes. To find preliminary member sizes for the truss, a SAP2000 model was created to attempt to match the deflection of the column below to that of the proposed truss. This method yielded the design shown below in Figure 19, which contributes additional stiffness to the outrigger level. The total thickness of the walls was decreased almost by half: a decrease of 14 inches at the bottom to 4 inches at the top. Coupling beams, indicated in green in Figure 20, were designed as 18"x42" and stay consistent in size throughout the height of the building. However, their compressive strengths change with those of the walls and returns, as shown in Table 12 below. Data concerning periods and drifts are listed below in Table 12. Again, this design is not finalized, and was intended to provide a rough estimate of the benefits of adding outriggers to the system. Please view Andres Perez's third technical report for a detailed analysis and discussion of this system.

Concrete Core with Outrigger Data	
Thicknesses	
Floor 1 - 30	1'-4"
Floor 31 - 52	1'-2"
Compressive Strengths	
Floor 1 - 30	10,000 psi
Floor 31 - 40	8,000 psi
Floor 41 - 50	6,000 psi
Floor 51 - 52	8,000 psi
Periods of Vibrations	
Seismic	
East-West	6.23 s
North-South	6.97 s
Torsional	4.88 s
Wind	
East-West	5.69 s
North-South	6.44 s
Torsional	4.57 s
Building Drifts	
Seismic	
East-West	8.16"
North-South	8.97"
Wind	
East-West	16.86"
North-South	16.12"

Table 12: Shear Wall Core with Outrigger Data

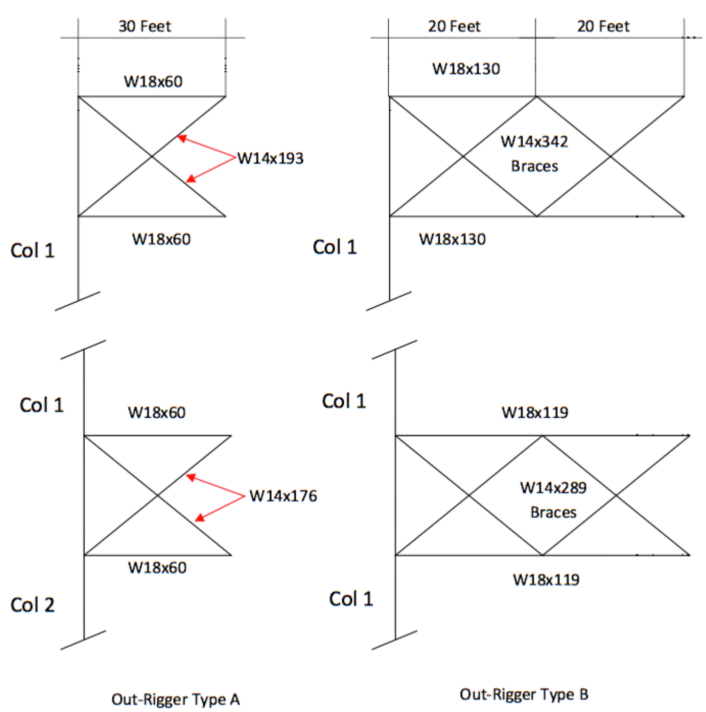


Figure 19: Outrigger Details

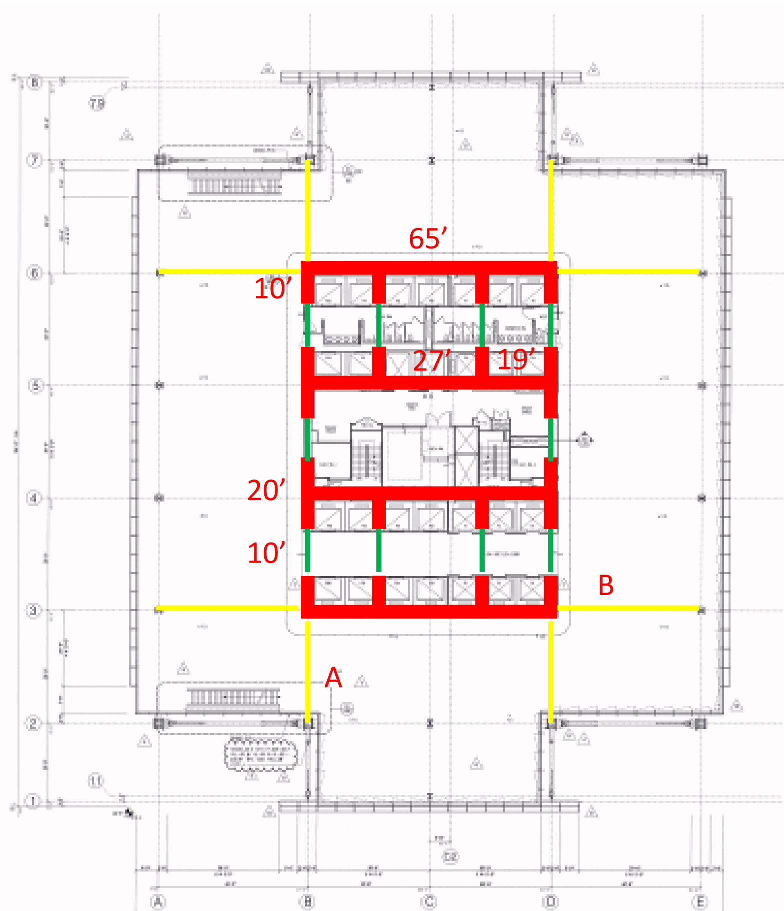


Figure 20: Concrete Shear Wall Core with Outrigger Layout

TEAM 2 BIM PRE-PROPOSAL

The ultimate product at the end of this semester is a finalized group proposal outlining the general and specific goals for further research, study, and collaboration next semester. One main goal of Team 2 is to incorporate a dedicated cogeneration plant to produce more electricity on-site, using a byproduct of building heat, and reduce the building's reliance on the grid. It is recognized that this system will require a higher start-up cost, therefore another main goal became reducing costs elsewhere to accommodate this system and maintain a similar overall cost. Some option-specific ideas for reducing costs include the following:

- Utilizing rolled W-shapes for lateral and gravity members as opposed to built-up sections.
- Replacing the expensive and under-performing façade with a more efficient system.
- Using bus ducts instead of conduit in the electrical distribution system.
- Value engineering systems and components of the building where possible.
- Replacing the under-floor air distribution system with an optimized ducted system.
- Installing a demand-controlled ventilation system.
- Redesigning core of building for better functionality.

While this list displays the façade as a secondary concern to the cogeneration plant, it is actually one of the main focal points for future change. The current façade shading system, consisting of horizontal cylindrical tubes, only accounts for a 1% reduction in overall building energy use (please reference the BIM Lighting/Electrical technical reports). This system could surely be optimized to further reduce the heat gain in the building, possibly by installation of an automatic louvered system that adjusts to the position of the sun.

A few important findings were brought up through the lateral investigation in this third technical report. It is possible to remove the outriggers on the 51st floor; however, more outriggers and bracing lines were added to increase the stiffness to meet the acceptable periods and depths. The removal of these extra bracing lines on upper floors should be investigated. In addition, using rolled shapes instead of the built-up box columns will lead to a large decrease in total stiffness of the structure due to the lower cross-sectional area and moments of inertia in comparison to the box sections. This will cause a significant increase in the drift of the structure, and the drift may not be able to be controlled by one floor of outriggers. An alternative to eliminating the outriggers on the 51st floor may become moving those outriggers to a lower floor.

Another essential element in the combined proposal is the methodology behind how BIM will be utilized to advance and assist the goals highlighted above. As of now, Team 2 would like to use BIM to coordinate trades through clash detection and to create a 4D-scheduling model. Continued meetings to address the BIM Execution Plan will help consolidate and focus these uses for the proposal.

CONCLUSIONS

The main goals of this analysis were to create an alternative lateral system that eliminated the need for outriggers at the 51st floor and exterior X-braces, which were used in the original system to control drift. These goals were met on a preliminary level based on the steel frame system with concentric chevron bracing, moment frames, and an outrigger and belt truss on the 36th floor, which was modeled in ETABS. This system and the floor location of the outrigger were not completely optimized but provides a basis of comparison to the original system, proving that it is a feasible alternative to be looked at in more detail in the future.

Certain criteria were checked to prove that this system is a viable option for future study as a replacement for the original lateral system of the New York Times Building. The initial drift limit of H/450 and period requirement of less than 6.75 seconds were met; however, it is evident that a more in-depth design will need to be done in order to optimize the members and overall system. For example, hand calculations revealed some weak areas in need of further optimization, including the columns at the first floor, the outriggers in the East/West direction, and the limited factored loads considered in the system. In addition, the loads at the outrigger level will affect the belt truss, and it will also need to be redesigned. P-Delta effects, effective lengths, and torsional effects on loading conditions are also very important inclusions for the future success of this design.

One issue with this new lateral system is that it changes the architectural aesthetic of the structure. Since the exterior X-braces are no longer needed in this updated design, they may be removed from even an architectural context contributing to the building's theme of transparency. This system also has a large impact on the mechanical spaces, due to the shifting outrigger and belt truss locations. The mechanical student within BIM Team 2, Peter Clarke, advises that this is not exactly a problem; the placement of the mechanical floors seems to be controlled more by the location of the outriggers than the necessity of mechanical units at these particular levels. The 28th floor mechanical space serves floors above and below it, and it may actually be worth reorganizing the mechanical feeds in a more methodical pattern. Finally, the inclusion of an additional core brace in both the North-South and East-West directions will impact the planning of the interior core. This may not be a bad thing, as the core has the potential for better functional performance with rearrangement.

Overall, this system could work well with the BIM Proposal, as it could bring in more profit with the creation of a penthouse level. It could also be combined with the idea of using rolled W shapes to save costs; however, other members would likely need to be upsized due to the lower stiffness of the W shapes, and a cost analysis would need to be performed.

The two concrete shear wall systems proved to be feasible alternatives to the original; these are also potential solutions for future consideration. Now that New York City lifted requirements preventing concrete trades from working above steel trades, these systems can compete on a level playing field with a steel frame system. Care would need to be taken to ensure proper coordination between trades; mechanical openings and construction sequencing are both areas to be discussed with other options prior to the final thesis proposal. The benefits of a concrete core-only system are that no space is required for the diagonal outriggers within the interior of the mechanical floors and that the system is innately stiffer than the steel braced frame core. However, at the bottom levels, the shear walls consume 14 more inches of space in the core than the concrete shear wall system with outriggers. This system's performance is comparable to the concrete core-only system, but it also engages the perimeter columns to increase stiffness without thickening the walls.

APPENDIX A:

STRENGTH AND SERVICEABILITY CHECKS

* CHECK DRIFTS:

• WIND: H/450 (DESIGN DRIFT LIMIT OBTAINED FROM THORNTON TOMASETTI)

H = 745.5' TO TOP OF SLAB OF ROOF

$$\frac{745.5 \cdot 12}{450} = 19.9''$$

* NORTH-SOUTH DIRECTION:

DRIFT FROM ETABS = 16.7" ≤ 19.9" OK

* EAST-WEST DIRECTION:

DRIFT FROM ETABS = 19.8" ≤ 19.9" OK

• SEISMIC:

+ TOTAL DRIFT:

* N-S DIRECTION: 9.22" FROM ETABS
* E-W DIRECTION: 9.57" FROM ETABS

BOTH < 19.9" OK

+ STORY DRIFT: SPOT CHECKS:

* N-S DIRECTION:

14TH FLR: $\Delta_{14} = 1.484'' - 1.326'' = 0.158''$

$\Delta_a = 0.015 h_{sx}$ T.12.12-1 Occ. Cat. III

$= 0.015(\frac{165''}{12})$

$= 0.206'' < 0.385''$ NG

$$\Delta_{14} = \frac{0.158(3.25)}{1.25}\left(\frac{4.93}{5.26}\right) = 0.385''$$

$T_{wb} = 4.93s$ $T_b = 5.26s$

37TH FLR: $\Delta_{37} = 6.114 - 5.930 = 0.184''$

$$\Delta_{37} = \frac{0.184(3.25)}{1.25} * \frac{4.93}{5.26} = 0.448''$$

$\Delta_a = 0.015(330/12) = 0.413'' < 0.448''$ NG

Figure 21: Drift Checks

* STORY DRIFT: SPOT CHECKS:

* E-W DIRECTION:

14TH FLR: $\Delta_{14} = 1.461 - 1.302 = 0.159''$

$$\Delta_{14} = \frac{0.159(3.25)}{1.25} * \frac{4.93}{5.17} = 0.394''$$

$C_d = 3.25$
$I = 1.25$
$T_{Vb} = 4.93$
$T_b = 5.17$

$\Delta_a = 0.015(13.8') = 0.206'' < 0.394''$ NG

37TH FLR: $\Delta_{37} = 5.952 - 5.813 = 0.139''$

$$\Delta_{37} = \frac{0.139(3.25)}{1.25} * \frac{4.93}{5.17} = 0.345''$$

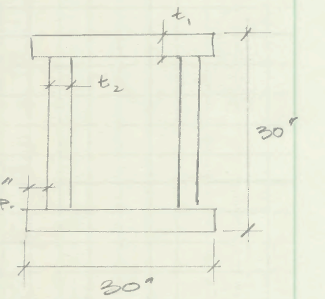
$\Delta_a = 0.015(27.5') = 0.413'' > 0.345''$ OK

Figure 22: Story Drift Checks

BOX COLUMN PROPERTIES: $E = 29000$ ksi, $F_y = 50$ ksi

* MOMENT OF INERTIA:

$$I = \frac{1}{12}bh^3 - \frac{1}{12}b_1h_1^3 - 2Ad^2$$



* FLANGE & WEB THICKNESSES VARY

FLR. 1–13: $t_1 = 7"$, $t_2 = 4"$

$$I = \frac{1}{12}(30)(30^3) - \frac{1}{12}(16)(16)^3 - 2(3)(16)(13.5)^2 = 44543 \text{ in}^4$$

FLR. 14–27: $t_1 = 6"$, $t_2 = 3.5"$

$$I = \frac{1}{12}(30)(30)^3 - \frac{1}{12}(17)(18)^3 - 2(3)(18)(13.5)^2 = 39555 \text{ in}^4$$

FLR. 28–40: $t_1 = 5"$, $t_2 = 3"$

$$I = \frac{1}{12}(30)^4 - \frac{1}{12}(18)(20)^3 - 2(3)(20)(13.5)^2 = 33630 \text{ in}^4$$

FLR. 41–R: $t_1 = 4"$, $t_2 = 2.5"$

$$I = \frac{1}{12}(30)^4 - \frac{1}{12}(19)(22)^3 - 2(3)(22)(13.5)^2 = 26584 \text{ in}^4$$

$A_1 = 548 \text{ in}^2$
$A_2 = 486 \text{ in}^2$
$A_3 = 420 \text{ in}^2$
$A_4 = 350 \text{ in}^2$

* KL/r VALUES:

$\frac{KL}{r}$ calculated directly for simplification

$r_{1\text{-}13} = \sqrt{\frac{I}{A}} = 9.02"$ $\quad r_{28\text{-}40} = 8.95"$

$r_{14\text{-}27} = 9.02"$ $\quad r_{41\text{-}R} = 8.72"$

* PLASTIC SECTION MODULUS:

$Z_x = 2b \cdot t \cdot y$ (for 2 flanges; excludes "web")

$Z_{x\,1\text{-}13} = 2(30)(7)(11.5) = 4830 \text{ in}^3$

$Z_{x\,14\text{-}27} = 2(30)(6)(12) = 4320 \text{ in}^3$

$Z_{x\,28\text{-}40} = 2(30)(5)(12.5) = 3750 \text{ in}^3$

$Z_{x\,41\text{-}R} = 2(30)(4)(13) = 3120 \text{ in}^3$

Figure 23: Box Column Properties

* WEB SHEAR COEFFICIENT:

· $\frac{h}{t_w}_{1-13} = \frac{16}{8} = 2$, $k_v = 5$

$< 1.1\sqrt{5(29000)/50} = 59.2$

∴ use $C_v = 1.0$ (assume h/tw values don't vary greatly from above)

* CHECK COLUMN AXIAL CAPACITY:

$\phi P_n = 0.9\, F_{cr} A_g$ (use $k = 1.0$)

FLR. 1: $F_{cr} = 0.658^{F_y/F_e}(F_y)$

$= 0.658^{50/631.7}(50) = 48.4$ ksi

E3-4: $F_e = \frac{\pi^2 E}{(KL/r)^2} = \frac{\pi^2(29000)}{(192/9.02)^2} = 631.7$ ksi

E3(a): $\frac{KL}{r} \leq 4.71\sqrt{\frac{E}{F_y}} = 113.4$

$\frac{KL}{r} = \frac{1.0(192")}{9.02} = 21.3 < 113.4$ ∴ use eq. E3-2 for Fcr

$\phi P_n = 0.9(48.4)(548) = 23856$ k

· controlling $P_{u\,1st} = 12760$ k $< 23856^k$ OK
E/W direction

FLR. 14: $F_{cr} = 0.658^{50/855.3}(50) = 48.8$ ksi

$F_e = \frac{\pi^2(29000)}{(165/9.02)^2} = 855.3$ ksi

$\frac{KL}{r} = 18.3 < 113.4$ ∴ eq. E3-2

$\phi P_n = 0.9(48.8)(486) = 21341$ k

· controlling $P_{u\,14th} = 4907$ k < 21341 k OK
N/S direction

Figure 124: Column Axial Check

FLR. 28: $F_{cr} = 0.658^{50/842.1}(50) = 48.8$ ksi

$F_e = \dfrac{\pi^2(29000)}{(165/8.95)^2} = 842.1$ ksi

$\phi P_n = 0.9(48.8)(420) = 18436$k

∴ controlling $P_{u\,28th} = 2256$k < 18436k OK

FLR. 41: $F_{cr} = 0.658^{50/799.4}(50) = 48.7$ ksi

$F_e = \dfrac{\pi^2(29000)}{(165/8.72)^2} = 799.4$ ksi

$\phi P_n = 0.9(48.7)(350) = 15341$ k

∴ controlling $P_{u\,41st} = 654$ k < 15341k OK

* all axial loads available upon request; these loads from 1.6W only!

* CHECK COLUMN FLEXURAL CAPACITY:

$\phi M_n = \phi M_p = F_y \cdot Z_x$ * assume all sections compact for simplification

FLR. 1: $\phi M_p = (50)(4830) = 241500$ ''k
$= 20125$ 'k

∴ controlling $M_{u\,1st} = 3811$ 'k < 20125'k OK
E/W direction

FLR. 35: $\phi M_p = (50)(3750)/12 = 15625$ 'k

∴ controlling $M_{u\,35th} = 3481$'k < 15625'k OK
E/W direction

* FLR. 35 & 36 HAVE HIGHER MOMENTS DUE TO OUTRIGGER; 35th FLR. MOMENT IS GREATER & THEREFORE WAS CHECKED

Figure 25: Column Flexural Check

* CHECK COLUMN SHEAR CAPACITY:

$\phi V_n = 0.6 F_y A_w C_v \quad (\phi = 1.0)$

FLR. 1: $\phi V_n = 0.6(50)(8)(16)(1.0) = 3840^k$

∙ controlling $V_{u\,1st} = 191.3^k \ll 3840^k$ OK

FLR. 35: $\phi V_n = 0.6(50)(6)(18)(1.0) = 3240^k$

∙ controlling $V_{u\,35th} = 302.7^k \ll 3240^k$ OK

* INTERACTIONS BETWEEN SHEAR, AXIAL, & MOMENT WERE NOT CHECKED, BUT SHOULD BE CONSIDERED IN THE FUTURE. ALL CONTROLLING FORCES & MOMENTS ARE DUE TO 1.6W ONLY FOR DESIGN PURPOSES.

Figure 26: Column Shear Check

CHECK BEAMS IN MOMENT FRAMES:

* NORTH/SOUTH DIRECTION:

+ STORY 14, FRAME B: W30x116

$L = 30'$ $M_{uw} = 1390'^k$ $V_{uw} = 92.5^k$ > FOR 1.6W ONLY

(Assume fully braced, typical all bms.)

From AISC T. 3-2: $\phi M_p = 1420'^k > 1390'^k$ OK

$\phi V_n = 509^k >> 92.5^k$ OK

→ beam may need to be upsized to accomodate dead + live loads

+ STORY 41, FRAME D: W30x116

$L = 30'$ $M_{uw} = 989'^k$ $V_{uw} = 65.8^k$

T. 3-2: $\phi M_p = 1420'^k > 989'^k$ OK

$\phi V_n = 509^k >> 65.8^k$ OK

* EAST/WEST DIRECTION:

+ STORY 14, FRAME 3: W30x116

$L = 40'$ $M_{uw} = 1124'^k$ $V_{uw} = 56.1^k$

T.3-2: $\phi M_p = 1420'^k > 1124'^k$ OK

$\phi V_n = 509^k >> 56.1^k$ OK

+ STORY 41, FRAME 5: W30x116

$L = 40'$, $M_{uw} = 892'^k$ $V_{uw} = 44.6^k$

T.3-2: $\phi M_p = 1420'^k > 892'^k$ OK

$\phi V_n = 509^k >> 44.6^k$ OK

Figure 27: Moment Frame Beam Check

* CHECK BRACING CAPACITY:

FLR. 1: W14×257

, controlling $P_{u\,NIS}$ = 1724k, L = 408" = 34'

from T.4-1 AISC: ϕP_n = 1860 k > 1724k OK

FLR. 14: W14×176

, controlling $P_{u\,NIS}$ = 1478k, L = 32'

from T.4-1 AISC: ϕP_n = 1200 k < 1478k NG

→ brace will need to be enlarged @ this level

FLR. 28: HSS16×16×1/2

controlling $P_{u\,NIS}$ = 845k, L = 32'

from T.4-4 AISC: ϕP_n = 915k > 845 k OK

FLR. 41: HSS12×12×3/8

controlling $P_{u\,NIS}$ = 422k, L = 32'

from T.4-4 AISC: ϕP_n = 424k > 422k OK

* ALL CONTROLLING FORCES FROM 1.6W ONLY.

Figure 28: Bracing Capacity Check

* CHECK OUTRIGGER CAPACITY:

W36×247: $M_u = 1178\ 'k$, $P_u = 2962\ k$ — E/W direction

$L_b = 20'$ (assumed braced in the middle) & $C_b = 1.0$

interaction equation: $P_r/P_c \geq 0.2$ ∴ H1-1a

$$\frac{0.414(2962) + 0.255(1178)}{1000} \leq 1.0$$

$1.53 > 1.0$ NG!

∴ OUTRIGGER MUST BE UPSIZED SIGNIFICANTLY!

W36×247: $M_u = 78\ 'k$, $P_u = 1125\ k$ — N/S direction

$L_b = 20'$ (assumed $C_b = 1.0$ & braced @ mid point)

interaction equation: $P_r/P_c \geq 0.2$ ∴ H1-1a

$$\frac{0.414(1125) + 0.255(78)}{1000} \leq 1.0$$

$0.486 < 1.0$ OK

∴ OUTRIGGER CAN BE DOWNSIZED IN N/S DIRECTION!

* SHEAR IN OUTRIGGERS IS NEGLIGIBLE AS COMPARED TO AXIAL & BENDING.

Figure 29: Outrigger Capacity Check

CHECK OVERTURNING:

W = 183133 k (from seismic calcs.)

• $M_{E/W} = 3922512'^{k}$ $\quad d_{E/W} = 145'$

$\frac{W}{2} > \frac{M}{d}$ TO RESIST O.M.

$\frac{183133}{2} > \frac{3922512}{145}$

$91567^{k} \gg 27052^{k}$ OK IN E/W DIR.

• $M_{N/S} = 3185465'^{k}$ $\quad d_{N/S} = 190'$

$\frac{183133}{2} > \frac{3185465}{190}$

$91567^{k} \gg 16766^{k}$ OK IN N/S DIR.

SO, NO OVERTURNING WILL OCCUR.

Figure 30: Overturning Check

APPENDIX B:

ADDITIONAL FIGURES

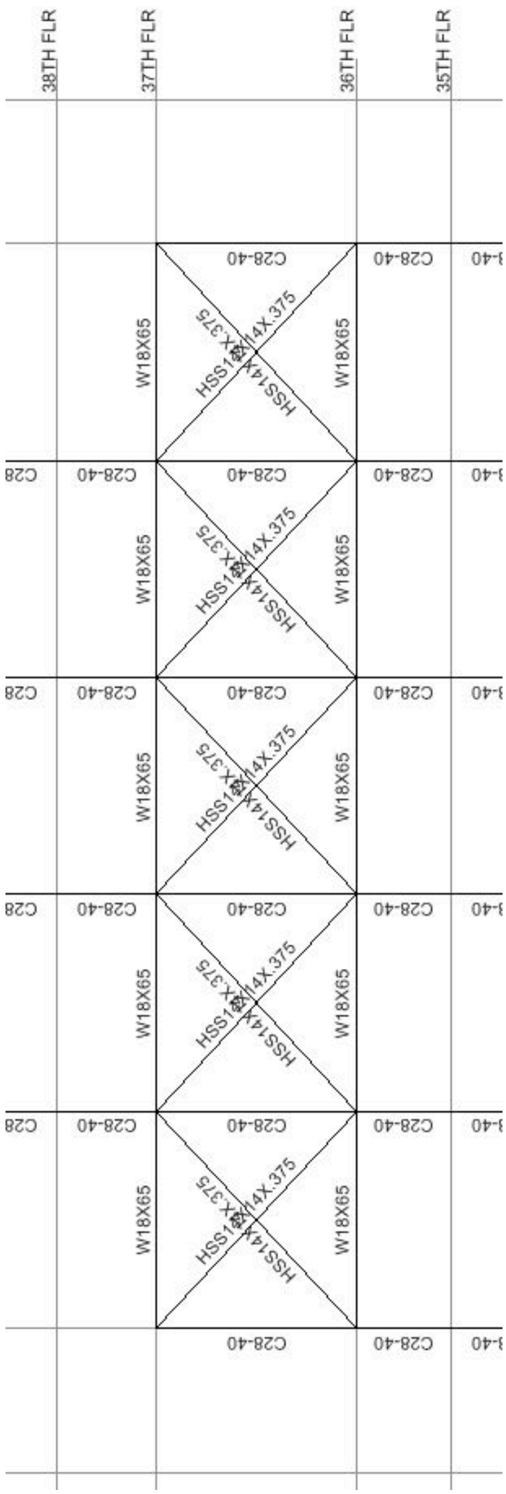


Figure 31: Belt Truss Design