360 State Street

New Haven, Connecticut

LATERAL SYSTEM ANALYSIS & CONFIRMATION DESIGN

STRUCTURAL TECHNICAL REPORT III

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Executive Summary

360 State Street is a new landmark for New Haven, Connecticut. It consists of street level retail, four stories of parking, and five-hundred rentable apartment units. The design of the building couples sustainable resources and tactics with location and architectural allure. Overall, the building reaches 32 stories and makes a statement about the convenience and romance of an urban lifestyle.

The intent of this report however; is to analyze the lateral systems of 360 State Street and confirm the sufficiency of design. The upper twenty-stories of the building were used for this investigation to maintain simplicity. Frame X, composed of X-braces and a moment frame, and Frame Y, composed of staggered steel trusses, were examined and modeled in RAM Structural Systems. A two-dimensional analysis was completed with both hand calculations and computer generated results to verify findings. The analysis included an inspection of potential torsional problems and lateral movements that may exceed service requirements. In addition, two load combinations were used to check story drift and overturning moments. The analysis yielded a better understanding of how the frames distribute loads across the structure as well as their impact on the foundations.

To summary, the building was found to have no rotational movement because the centers of mass and rigidity were found to be equal. Therefore, purely translational movements became the focus of the analysis. 1.6*Wind* and 1.0*Earthquake* were the primary loads used in the two-dimensional analysis of Frames X and Y. It was found that Frame X dominated the design in terms of passing all service requirements. Frame Y exceeded the allowable story drift for wind and an overturning moment caused by seismic loads was greater than that of the dead loads' resisting moment. Although it is evident the computer generated results were more accurate than the hand calculations, a distinct pattern was established to understand the distribution of loads across each frame. It is concluded that the lateral systems are tolerably designed and a more substantial investigation is recommended for more accurate results.

Introduction

360 State Street is an innovative building project developed by Becker + Becker Associates. Located in downtown New Haven, Connecticut, the building is situated on the corner of Chapel and State Street just two blocks east of the historic town green. As the newest addition to the city's skyline, the project consists of thirty-two stories of retail, parking, and residential living space. Architecturally, it features a landscaped garden terrace and a façade composed of precast panels, ornamentation, and glazing. As a whole, the building's location and design encourages a sustainable lifestyle and creates an attractive urban environment.

Figure 1 : Rendering of 360 State Street

The following report is an investigation of 360 State Street's lateral systems. A two-dimensional analysis using RAM Structural Systems will focus on the primary frames to determine the sufficiency of the building's design. In order to maintain simplicity, only the upper twenty-six stories will be examined. A comparison of hand calculated and computer generated results will verify the performance of the lateral systems by checking torsion, story drifts, and overturning moments. Each calculation will use loads determined by a governing combination outlined in ASCE 7 – 05. Furthermore, the report includes a discussion of the load paths and the lateral systems' impact on the foundations.

Overall Structural Systems

The building's gravity systems are composed of two materials—concrete and steel—that separate the structure into two distinct sections. The foundation consists of pressure injected footings and a mat slab that varies in thickness. The first six stories, appropriately dubbed the 'concrete base', are framed with reinforced concrete columns and beams. Additionally, two unique concrete floor systems are present on each level. The slabs are mostly cast-in-place with varying thicknesses however; a third of the floor plan includes a post-tensioned slab. The base consists mainly of an open-air parking garage but masonry walls are present throughout the street level retail and lobby area.

The remaining twenty-six stories of the building compose the residential tower; this portion of the building is roughly one-third of the area of the original footprint. The tower is framed with mostly steel W-shapes. Spandrel beams run along the perimeter and staggered steel trusses clear-span across the shorter dimension of the building. Columns are only present along the exterior and around the elevator core. A singular floor system is also present throughout the residential tower. Hollow core precast planks span between the staggered trusses and are topped with a unique floor finish. The main roof construction also consists of planks however; the terrace level is composed of a composite metal deck and concrete slab.

Lateral Systems

The base of 360 State Street is laterally supported by four unique shear walls. Each is composed of 8,000 psi strength concrete and is heavily reinforced with bar sizes ranging from #5's to #11's in both vertical and horizontal directions. One shear wall encloses the elevator core and another encloses a stairwell. The remaining two have three sides in order to leave room for parking. In general, the shear walls begin below grade and top off at the sixth floor.

The residential tower has two distinct frames that compose the upper level lateral systems. In the short dimension of the building, staggered steel trusses combine to create a unique frame. The trusses are composed of W-shapes and hollow structural sections. Each spans 64' – 0" between the exterior columns and alternate between levels. Altogether there are nine frames that begin on the sixth floor and terminate at the roof level.

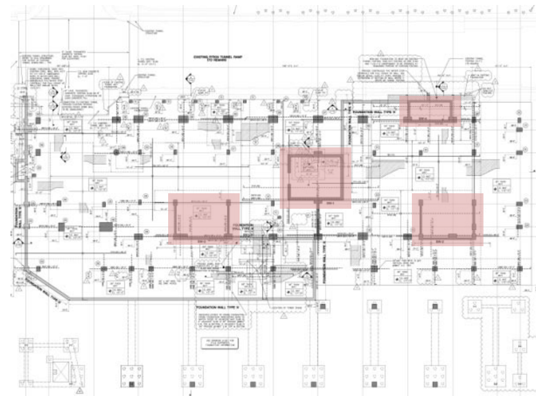
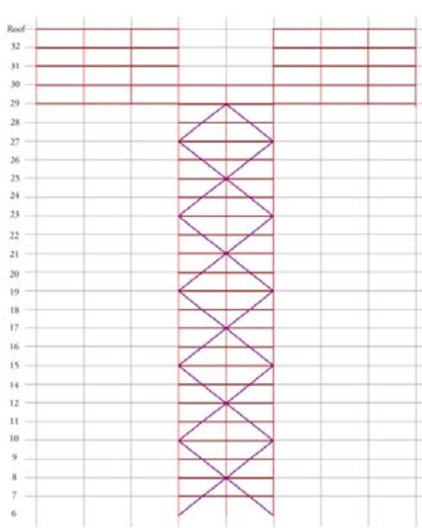


Figure 2: Foundation Plan w/ Shear Walls Highlighted

The second frame present in the tower is a system of X-braces on the north and south exterior of the building. The braces span between three columns and are composed of 14 x 14 and 10 x 10 hollow structural sections. Each diagonal member is roughly two stories in height and intersects the intermediate spandrel beam. The X-braces are included in a moment frame composed of the spandrel beams and columns on the level above and including the twenty-ninth floor.

In the following sections, a two-dimensional analysis will focus on the two steel frames in the residential tower. The concrete shear walls will be ignored in this report however; future investigations into the building's lateral systems may include these elements. Furthermore, the concrete base of the building will be considered as a separate entity due to the geometry of the building. By simply removing the top portion of the building, the sixth story can now be considered as the ground level. The X-braces and moment frames will be identified as the *X-Frame* and the staggered steel truss frame will be identified as the *Y-Frame*. This nomenclature is based on the x-y coordinate system which will be heavily present in this report. Furthermore the investigation will evaluate the sufficiency of the lateral systems' design according to ASCE 7 – 05. For information regarding the original design criteria, please refer to Appendix B.



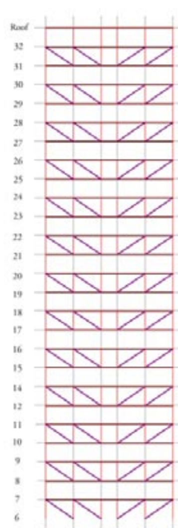


Figure 3: Frame X – X-braces and moment frame (right)
Figure 4: Frame Y – Staggered steel trusses (left)

Loads

To begin the analysis, design loads were established before any other calculations were initiated. The majority of the data present was determined previously in *Structural Concepts & Existing Conditions: Structural Technical Report I (Duk, S.).* This section includes a summary of the un-factored design loads. More information and sample calculations can also be found in Appendix B, C and D.

The live and dead loads have been outlined by the Engineer of Record on the structural drawings. Wind and seismic design criteria was also taken from the drawings to determine the following information. Both wind and seismic loads were calculated by following an example provided by David A. Fanella in his publication *Structural Load Determination Under 2006 IBC & ASCE/SEI 7 – 05.* The following tables and figures summarize the design loads used in the analysis.

Level	Load Type	Dead Load (psf)	Super-Imposed Dead Load (psf)	Live Load (psf)
7th to 31st	Residential	61	20	40
	Public	61	20	100
Roof	Mechanical	61	20	40

Table 1: Dead & Live Load Schedule

Wind Pressures		
Height above ground level, z (ft)	K_z	q_z (psf)
338.58	1.40	36.9
300	1.35	35.5
250	1.28	33.7
200	1.20	31.6
180	1.17	30.8
160	1.13	29.8
140	1.09	28.7
120	1.04	27.4
100	0.99	26.1
90	0.96	25.3
80	0.93	24.5
70	0.89	23.4
60	0.85	22.4
50	0.81	21.3
40	0.76	20.0
30	0.70	18.4
25	0.66	17.4
20	0.62	16.3
15	0.57	15.0
Leeward (all)	-	36.9

Table 2: Wind Load Schedule based on ASCE 7 – 05

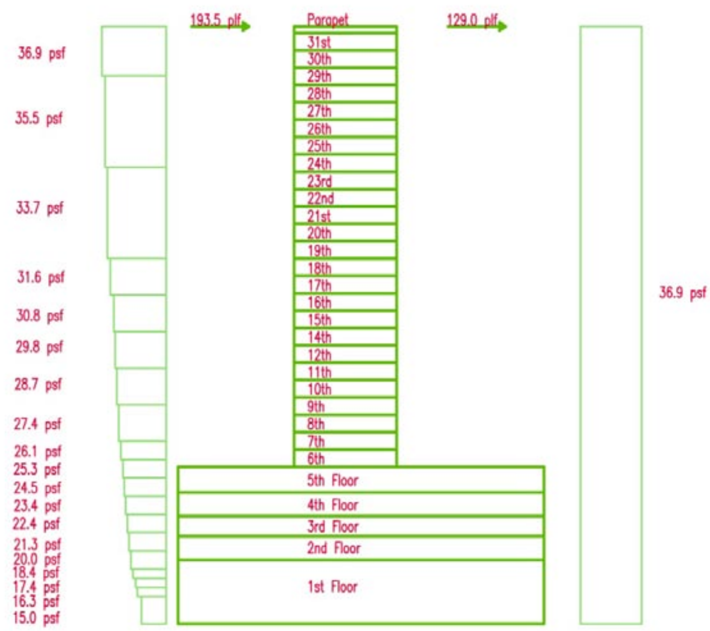


Figure 5: Diagram of Service Wind Pressures

Note: Although the sixth floor is considered the ground level for simplification, the wind pressures will be established according to the actual height of each floor for accuracy. The wind pressures are equal in both the North-South and East-West direction.

Seismic Design Criteria			
I_e = 1.0	T_s = 0.499	S_{ms} = 0.455	x = 0.75
S_s = 0.290	R = 3	S_{m1}= 0.204	H_n = 326.9
S_1 = 0.085	T_a = 1.54	S_{ds} = 0.303	C_s = 0.0133
Soil Class D	Ct = 0.02	S_{d1} = 0.136	K = 1.52
Category C	F_a = 1.568	F_v = 2.4	**V = 1250 k**

Table 3: Seismic Design Criteria used for Calculations

Note: The story seismic forces illustrated are equal in both North-South and East-West directions. In the analysis, the forces will be taken according to the actual height of the story although the sixth floor will be considered the ground level.

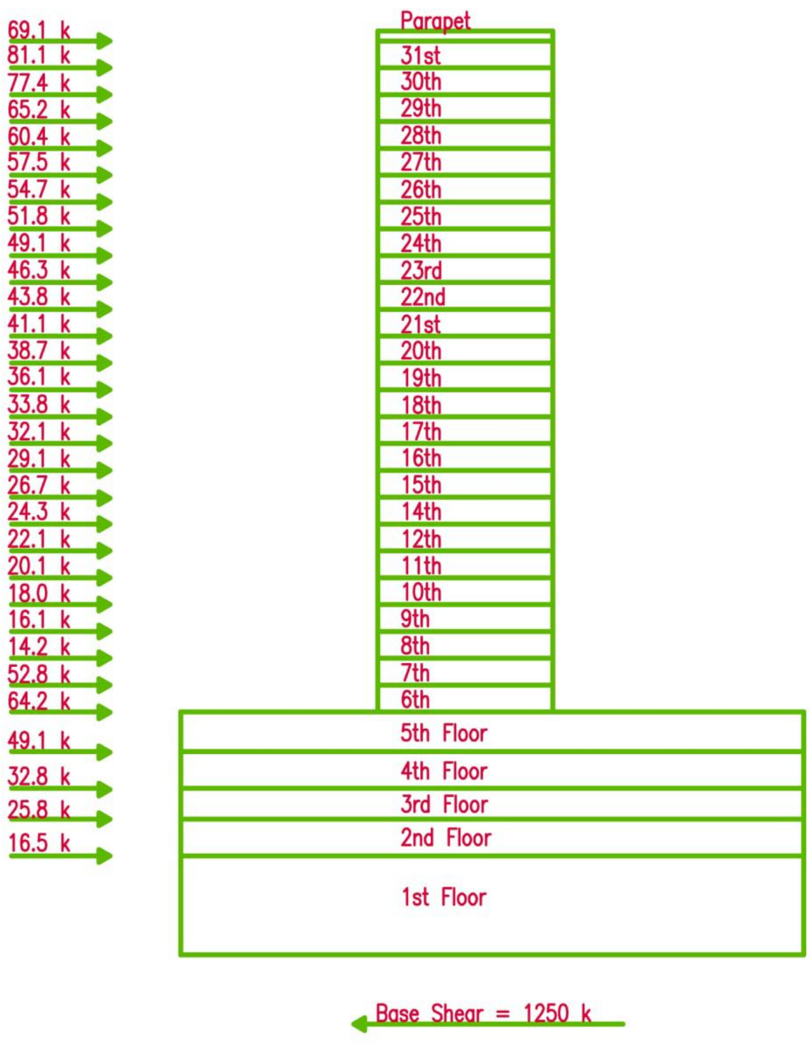


Figure 6: Diagram of Seismic Forces

Load Combinations

For the lateral systems to be appropriately designed, a governing force must be determined using ASCE 7 – 05 load combinations outlined in Chapter 2. Since horizontal forces are the focus of this report, several load types can be eliminated through inspection. Each section of calculations will discuss what loads were applied and how they were determined. For the purpose of this analysis, all calculations will be conducted using LRFD. The following summarizes the prescribed combinations.

Basic Load Combinations			
	All load types included.	Available load types.	Lateral load types only.
i	$1.4(D + F)$	$1.4(D)$	-
ii	$1.2(D + F + T) + 1.6(L + H) + 0.5(Lr$ or S or $R)$	$1.2(D) + 1.6(L) + 0.5(Lr$ or S or $R)$	-
iii	$1.2D + 1.6(Lr$ or S or $R) + (L$ or $0.8W)$	$1.2D + 1.6(Lr$ or S or $R) + (L$ or $0.8W)$	$0.8W$
iv	$1.2D + 1.6W + L + 0.5(Lr$ or S or $R)$	$1.2D + 1.6W + L + 0.5(Lr$ or S or $R)$	$1.6W$
v	$1.2D + 1.0E + L + 0.2S$	$1.2D + 1.0E + L + 0.2S$	$1.0E$
vi	$0.9D + 1.6W + 1.6H$	$0.9D + 1.6W$	$1.6W$
vii	$0.9D + 1.0E + 1.6H$	$0.9D + 1.0E$	$1.0E$

D = dead load; E = earthquake load; F = load due to fluids w/ defined pressures; H = load due to lateral earth pressure, or ground water pressure; L = live load; Lr = roof live load; R = rain load; S = snow load; T = self-straining force; W = wind load

Table 4: Summary of Load Combinations from ASCE 7 – 05

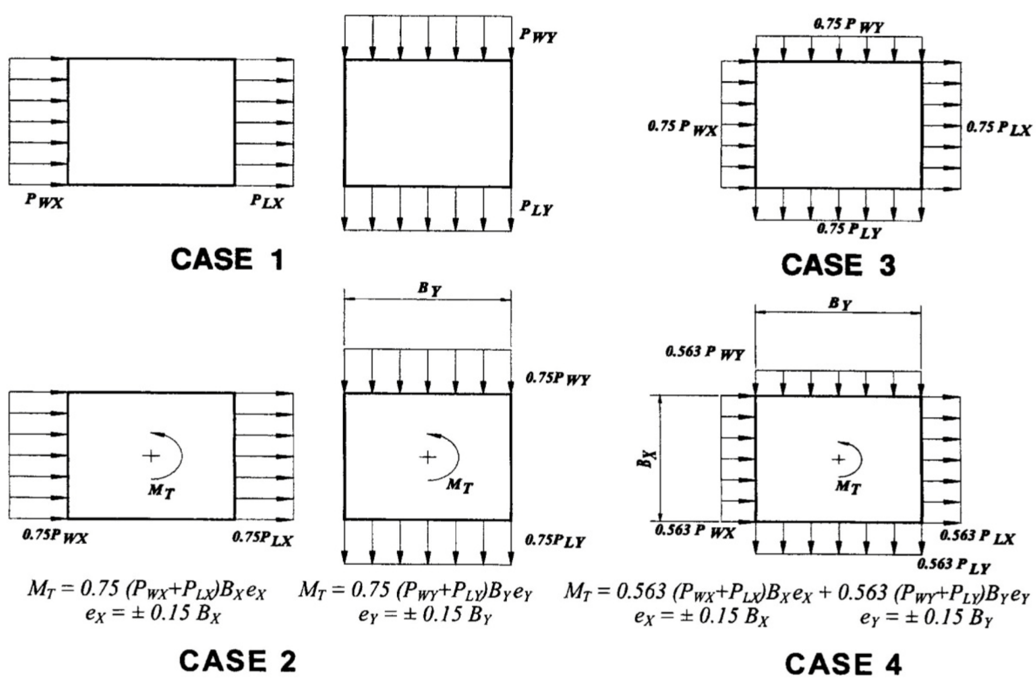


Figure 7: Wind Load Cases from ASCE 7 – 05

Analysis

As previously determined, two frames within 360 State Street will be the focus of this investigation. The behavior of Frames X and Y will be examined using a two-dimensional analysis. Hand calculations and computer generated results will be compared to establish the validity of the findings. All the frames will be examined as a combine system to determine if there are any torsional issues within the structure. Then a variety of horizontal loads will be applied to the two individual frames and story drifts will be calculated to determine if they are within service requirements. A discussion will be included regarding the load paths as well as the presences of overturning moments and their impact on the foundations. Lastly, strength checks will be conducted on a few critical members within each frame. Overall, this analysis should provide enough information to conclude the lateral systems of 360 State Street have been sufficiently designed.

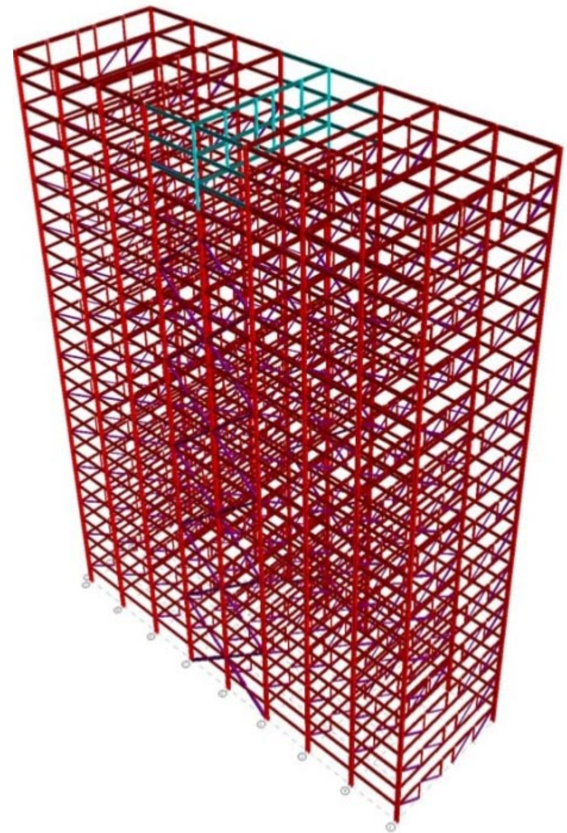


Figure 8: Rendering of Residential Tower's structure

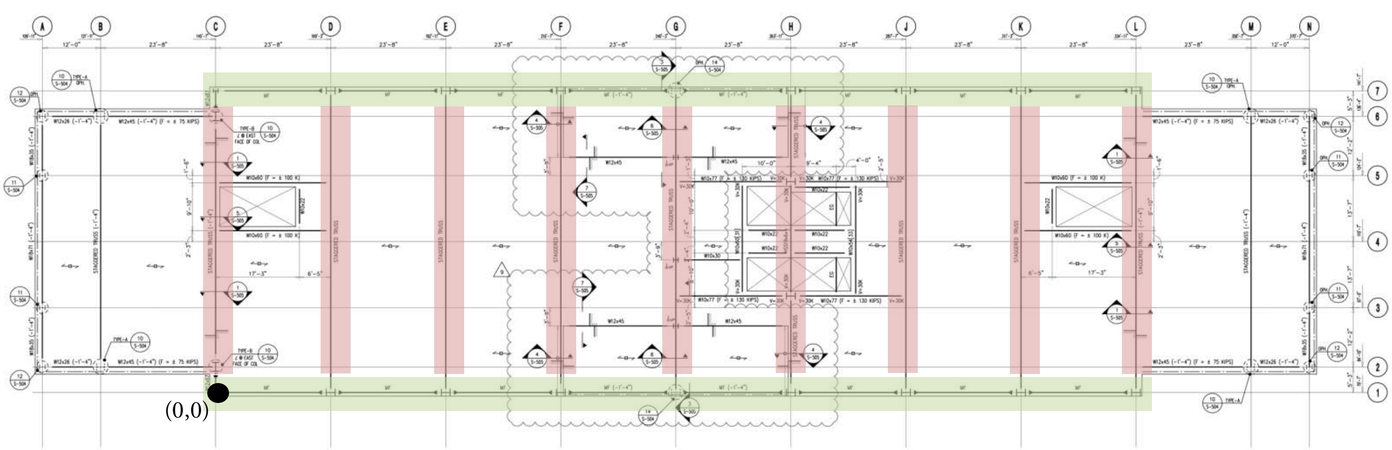


Figure 9: Residential Tower Floor Plan with shaded frames

In Figure 9, the frames shaded in green represent the X frames located on the North and South exteriors of the building. The frames shaded in red represent the Y frames spaced evenly across the long dimension. This figure also illustrates the origin of the x-y coordinate plane that will be continually referenced.

Torsion

Lateral systems are designed to resist loads along a singular axis however; if insufficiently designed, a building can experience torsion caused by horizontal forces. This includes a combination of translational and rotational displacements. The objective is to design a system where the center of rigidity is as close as possible to the center of mass. The intent is to eliminate the presence of an eccentricity caused by an applied force. Generally loads follow rigidity and it is the responsibility of the lateral systems to provide enough stiffness to evenly distribute the forces throughout a frame.

Frame Stiffnesses						
	X Frame			Y Frame		
Floor	δ (in)	Stiffness, k	% of k	δ (in)	Stiffness, k	% of k
32	22.162	45.1	50.0	7.578	132.0	11.1
31	20.084	49.8	50.0	6.831	146.4	11.1
30	17.812	56.1	50.0	6.024	166.0	11.1
29	16.079	62.2	50.0	5.411	184.8	11.1
28	14.525	68.8	50.0	4.888	204.6	11.1
27	13.499	74.1	50.0	4.548	219.9	11.1
26	12.611	79.3	50.0	4.260	234.7	11.1
25	11.689	85.6	50.0	3.961	252.5	11.1
24	10.763	92.9	50.0	3.660	273.2	11.1
23	9.875	101.3	50.0	3.369	296.8	11.1
22	9.000	111.1	50.0	3.082	324.5	11.1
21	8.119	123.2	50.0	2.791	358.3	11.1
20	7.267	137.6	50.0	2.509	398.6	11.1
19	6.474	154.5	50.0	2.245	445.5	11.1
18	5.701	175.4	50.0	1.986	503.4	11.1
17	4.918	203.3	50.0	1.723	580.5	11.1
16	4.194	238.4	50.0	1.478	676.6	11.1
15	3.584	279.0	50.0	1.271	787.1	11.1
14	3.021	331.0	50.0	1.078	927.9	11.1
12	2.447	408.6	50.0	0.879	1137.5	11.1
11	1.917	521.6	50.0	0.694	1441.3	11.1
10	1.477	676.9	50.0	0.538	1859.1	11.1
9	1.074	931.0	50.0	0.393	2543.9	11.1
8	0.668	1497.9	50.0	0.246	4068.3	11.1
7	0.318	3142.7	50.0	0.118	8503.4	11.1
6	0.086	11614.4	50.0	0.032	31446.5	11.1

Table 5: Summary of Stiffness Calculations

Using a two-dimensional model built in RAM Structural Systems, stiffness factors were determined by applying a 1,000 kip force laterally to the top story of each frame. The output provided by the computer program included story displacements that were used to compute the stiffness of each frame. By calculating the percent of stiffness, it was found that if a force is applied in the y-direction, each Y frame will carry 11.1% of the load. Likewise, if a force is applied in the x-direction, each X frame will carry 50% of the load. Using this information, the direct forces in each frame were found to account for any translational movement.

With the principle of superposition, the rotational movement can be found separately. In order to establish the eccentricity of the forces, the stiffness factors were used to calculate the center of rigidity of each floor and then compared it to the center of mass. (See Appendix E.) Both were found to be equal with a center at (94.8', 32'). From this information, no eccentricities were found within the structure. With zero rotational movement, indirect forces cannot act on the frames and additional load combinations can be eliminated from further investigation. Moreover, it can be concluded that 360 State Street does not have any apparent torsional issues. The remaining translational movements will be evaluated in the next section.

Lateral Movement

Lateral or translational movement in a building is a quantifiable measurement known as drift. This can be defined as the displacement a structure undergoes at the height of an applied load. Drifts can also be calculated for each level in order to understand how a force is distributed throughout a frame. These displacements are typically limited by code requirements in order to maintain serviceability. The following section discusses the two-dimensional analysis of frames Y and X with a variety of applied loads.

Frame Y is composed of trusses that clear span between two exterior columns and are staggered between the levels. Two different loads were used to determine the drift of each story. Since lateral loads are the focus of this investigation, all vertical loads were disregarded. The primary combinations evaluated for Frame Y were 1.6*Wind* and 1.0*Earthquake*. In addition, *Calculations of Wind Drift in Staggered-Truss Buildings*, a publication written by R.E. Leffler was used as a reference for the hand calculations.

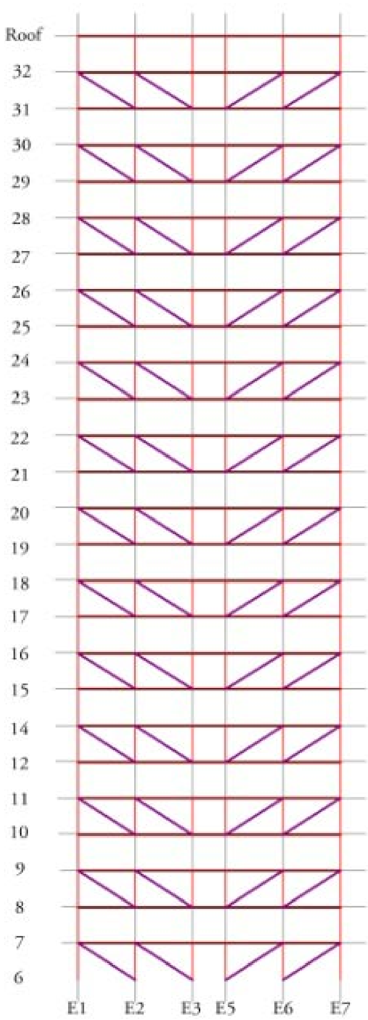


Figure 10: Frame Y

According to Leffler, interior frames have higher drift values and the floor slabs spanning between the frames distribute their shear force to the adjacent trusses. Therefore, Frame Y is modeled after the frame along the interior grid line E. In addition, the loads were assumed to accumulate as the levels decrease in the hand calculations. Essentially, the loads were taken from two bays instead of one. Furthermore, a universal displacement value was developed within a single truss using the virtual-work method. In order to simplify calculations, slab displacements and changing column lengths were excluded.

Comparing the computer generated results with the hand calculations for wind, the numbers vary in magnitude however; they have a similar pattern. The drift decreases as the story height increases. The simplification of the calculations may have caused this discrepancy. The same process developed for wind calculation was used for seismic loads. The fact that this calculation was developed for uniform pressures can explain the wide variation in the seismic drift. The RAM model more accurately illustrates the increased drift on levels that do not have a truss. A more substantial analysis is recommended for accurate drift measurements in staggered steel trusses.

The analysis of Frame Y has however, lead to a better understanding of the load distribution in a system of staggered steel trusses. The lateral loads applied translate into shear forces that move to an adjacent frame and then return again but at a lower level. This pattern continues down through the building until the loads can be transmitted into the foundation. Although the hand calculations for both wind and seismic loads vary significantly from the computer generated results, the idea of this load path is reinforced by the magnitudes of the story drifts. (See Table 6.) As the building height increases, the drift decreases; that is to say, as the upper level loads increase, the story drift decreases. The impact this load path has on the foundations may have governed its size. A thick mat slab could have been chosen over a typical foundation slab in order to distribute the heavy columns loads more evenly. In addition, the design of the large columns in the concrete base may have been governed by the staggered truss's ability to distribute loads.

Y - Frame Story Drifts (in)							
Floor	1.0E RAM	1.0E Hand Calc	1.0E % difference (RAM vs. Hand Calc)	Seismic $\Delta_{allowable}$	1.6W RAM	1.6W Hand Calc	1.6W % difference (RAM vs. Hand Calc)
32	0.695	0.026	*96.1*	8.2	0.131	0.024	*82.0*
31	0.506	0.059	*88.3*	8.0	0.092	0.047	*48.6*
30	1.283	0.085	*93.0*	7.7	0.222	0.070	*68.4*
29	0.898	0.116	*87.1*	7.5	0.159	0.093	*41.7*
28	2.270	0.132	*93.8*	7.3	0.405	0.115	*71.5*
27	1.182	0.163	*86.2*	7.0	0.217	0.138	*36.4*
26	2.508	0.173	*92.7*	6.8	0.468	0.160	*65.8*
25	1.396	0.205	*85.3*	6.6	0.266	0.183	*31.3*
24	2.785	0.211	*91.9*	6.3	0.542	0.204	*62.4*
23	1.585	0.243	*84.7*	6.1	0.313	0.225	*28.1*
22	2.927	0.245	*91.1*	5.9	0.592	0.247	*58.3*
21	1.754	0.276	*84.2*	5.6	0.360	0.268	*25.4*
20	2.991	0.274	*90.3*	5.4	0.629	0.290	*53.9*
19	1.806	0.306	*83.1*	5.2	0.686	0.310	*54.7*
18	3.091	0.300	*89.7*	4.9	0.677	0.330	*51.2*
17	1.793	0.332	*81.5*	4.7	0.399	0.350	*12.3*
16	3.104	0.323	*88.9*	4.5	0.709	0.369	*48.0*
15	1.867	0.354	*81.0*	4.2	0.433	0.388	*10.5*
14	3.092	0.342	*88.2*	4.0	0.737	0.407	*44.8*
12	1.895	0.372	*80.3*	3.8	0.459	0.425	*7.5*
11	2.989	0.358	*87.3*	3.5	0.744	0.444	*40.4*
10	1.931	0.387	*79.9*	3.3	0.489	0.461	*5.8*
9	2.866	0.371	*86.3*	3.0	0.744	0.478	*35.7*
8	1.832	0.399	*78.2*	2.8	0.479	0.495	*-3.3*
7	2.163	0.396	*80.6*	2.6	0.565	0.511	*9.5*
6	0.433	0.420	*-2.9*	2.3	0.111	-	-
Σ	51.642	6.868	-	Passes	11.628	7.032	$\Delta_{allowable}$=7.614"
					Fail	Pass	

Table 6: Summary of Story Drifts

Frame X is composed of columns, spandrel beams, and X-braces that span between three column lines. Additionally, the columns and beams on the upper stories form a moment frame. The same two load combinations from the previous analysis of Frame Y were also used to calculate story drift. The method of virtual-work was applied to gain an understanding of the loads' distribution. Again computer generated results were compared with hand calculated numbers to find that one source is more accurate than the other. The magnitudes of the story drifts vary significantly in the wind loading however; the seismic results are fairly similar. In both cases, the drift is significantly increased on the upper four stories where the moment frame is located. It can be presumed that the X-braces provide more lateral resistance.

The analysis also reveals a slight pattern in the distribution of loads. Beginning at the twenty-eighth floor, the drift alternates values as the story height decreases. This can be explained by the orientation of the diagonal members. As the load is sent diagonally downwards, a portion of that load is distributed into that level thus increasing its drift. Thus, every other level experiences an increase in force as the load approaches the foundation. The effect of this load distribution may explain the location of two shear walls in the concrete base. Both are positioned on either sides of the frame and are assumed to assist in the distribution of loads.

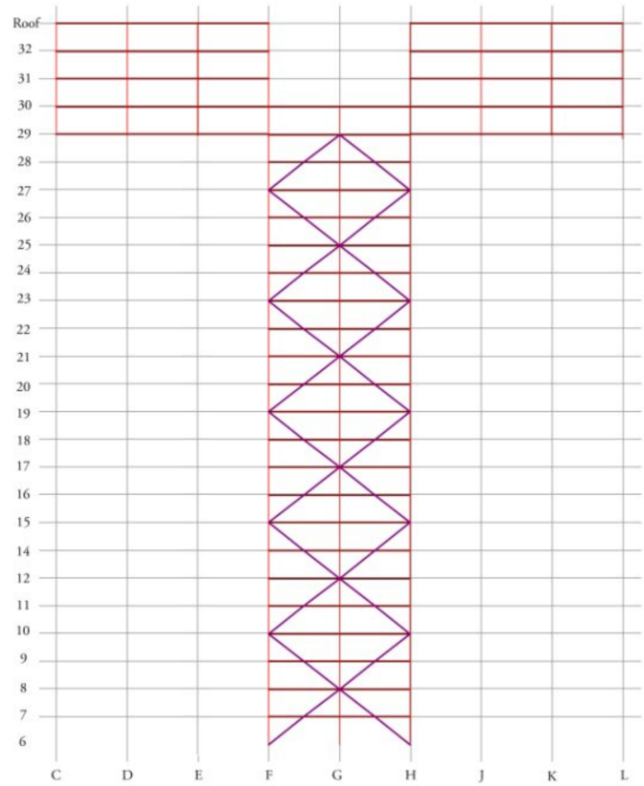


Figure 11: Frame X

The basis of an X-brace's design entails that each diagonal member should be capable of withstanding forces applied cyclically or from the opposite direction. The X-braces additionally experience tension and compression as a load is applied. This could also explain the alternating drift values as one member may be contributing to the drift instead of resisting the force. Further investigation is recommended to fully understand the distribution of loads and to calculate more accurate story drifts.

X - Frame Story Drifts (in)							
Floor	1.0E RAM	1.0E Hand Calc	1.0E % difference (RAM vs. Hand Calc)	Seismic $\Delta_{allowable}$	1.6W RAM	1.6W Hand Calc	1.6W % difference (RAM vs. Hand Calc)
32	1.152	1.153	-0.1	8.2	0.196	0.059	69.8
31	1.265	1.354	-7.0	8.0	0.515	0.057	88.9
30	1.257	1.293	-2.9	7.7	0.215	0.057	73.5
29	1.197	1.089	9.1	7.5	0.212	0.054	74.4
28	0.655	0.526	19.8	7.3	0.128	0.028	78.0
27	0.497	0.500	-0.6	7.0	0.106	0.028	73.4
26	0.551	0.475	13.7	6.8	0.113	0.028	75.1
25	0.617	0.450	27.0	6.6	0.127	0.028	77.8
24	0.575	0.427	25.8	6.3	0.119	0.028	76.4
23	0.603	0.403	33.1	6.1	0.126	0.026	79.1
22	0.607	0.380	37.3	5.9	0.127	0.026	79.1
21	0.612	0.357	41.6	5.6	0.130	0.026	79.7
20	0.569	0.336	40.9	5.4	0.122	0.026	78.3
19	0.585	0.314	46.2	5.2	0.127	0.026	79.2
18	0.599	0.294	51.0	4.9	0.130	0.026	80.2
17	0.572	0.279	51.1	4.7	0.127	0.026	79.7
16	0.475	0.253	46.8	4.5	0.107	0.025	76.7
15	0.457	0.233	49.1	4.2	0.106	0.025	76.5
14	0.473	0.211	55.3	4.0	0.110	0.024	78.1
12	0.447	0.192	57.0	3.8	0.107	0.024	77.6
11	0.367	0.174	52.5	3.5	0.091	0.023	74.7
10	0.354	0.157	55.7	3.3	0.090	0.023	74.5
9	0.365	0.140	61.6	3.0	0.094	0.022	76.7
8	0.330	0.124	62.5	2.8	0.087	0.022	75.0
7	0.227	0.459	-101.9	2.6	0.063	0.021	66.6
6	0.100	0.558	-457.0	2.3	0.027	0.020	23.8
∑	15.506	12.131	-	Passes	3.504	0.781	$\Delta_{allowable}$=7.614”
					Pass	Pass	

Table 7: Summary of Story Drifts

Conclusion

Within this report, the lateral systems of 360 State Street were examined to establish the sufficiency of design. Two frames in the residential tower were investigated in a two-dimensional analysis. Hand calculations were completed and compared to the output of a computer generated model in RAM Structural Systems. Previously determined loads were applied to the frames according to a variety of load combinations outlined in ASCE 7 – 05 and load paths were determined through the structure.

The analysis began with an investigation of potential torsional movements in the lateral systems. A unit load was applied to each frame to establish stiffness factors. Once the properties of the frames were known, the centers of mass and rigidity were calculated. If the centers were not equal, the difference would be an eccentricity that would cause a rotational moment and subsequent indirect forces. In the case of 360 State Street, the center of rigidity was found to be equal to the center of mass and no rotational movement is present. This find eliminated the consideration of other load combinations and narrowed the analysis down to an investigation of the translational forces.

The two-dimensional analysis extended into the calculation of story drifts in the frames dubbed Y and X. Frame Y consisted of a series of staggered steel trusses and Frame X contained X-braces and a moment frame. The load combinations applied were 1.6*Wind* and 1.0*Earthquake*. Hand calculations as well as computer generated results were developed however; it was found that the computer generated results were more accurate. The analysis of translational forces developed an understanding of the load distribution within each frame and its impact on the foundations. All in all, it is recommended that a more substantial analysis be undertaken to more accurately calculate drift and understand the distribution of loads.

Using the 1.6*Wind* and 1.0*Earthquake* load combinations, it can be summarized that the Y-Frame fails the allowable drift displacement for wind and fails in resisting an overturning moment cause by seismic loading. The X-Frame passes all the service requirements however; the overturning moment due to wind closely approached the magnitude of the resisting moment. Altogether, it can be concluded that the X-Frame controlled the design of the lateral systems and not enough attention was placed on the Y-Frame. Earthquake loads appeared to have the most dramatic impact on both frames however; not enough attention was given to this loading type. Overall, the two frames were tolerably designed for lateral loads but a three-dimensional investigation is recommended to understand how the frames work together as one system.

These conclusions may be different than those established by the Engineer of Record. Discrepancies in the hand calculations and the accuracy of the RAM model may have caused the variation in results and opinions.

Appendix A – Building Floor Plans & Elevations

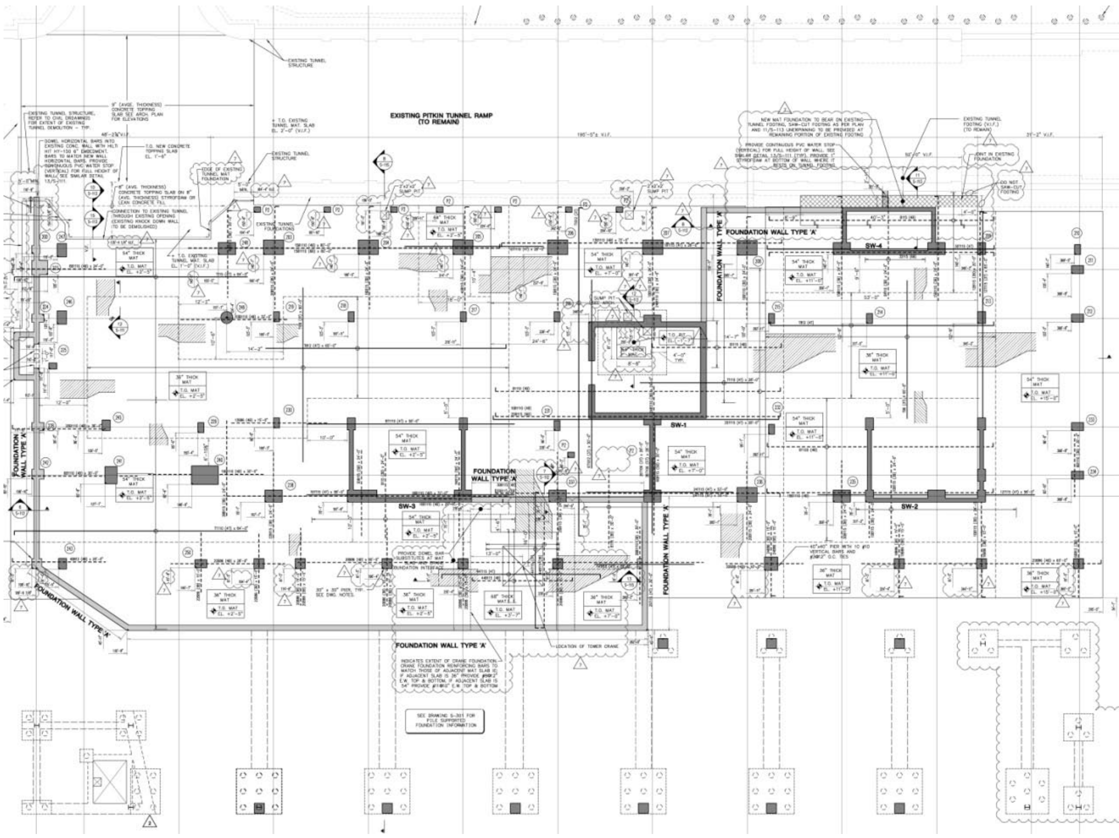


Figure A.1 Foundation Plan, Shear Walls are Shaded

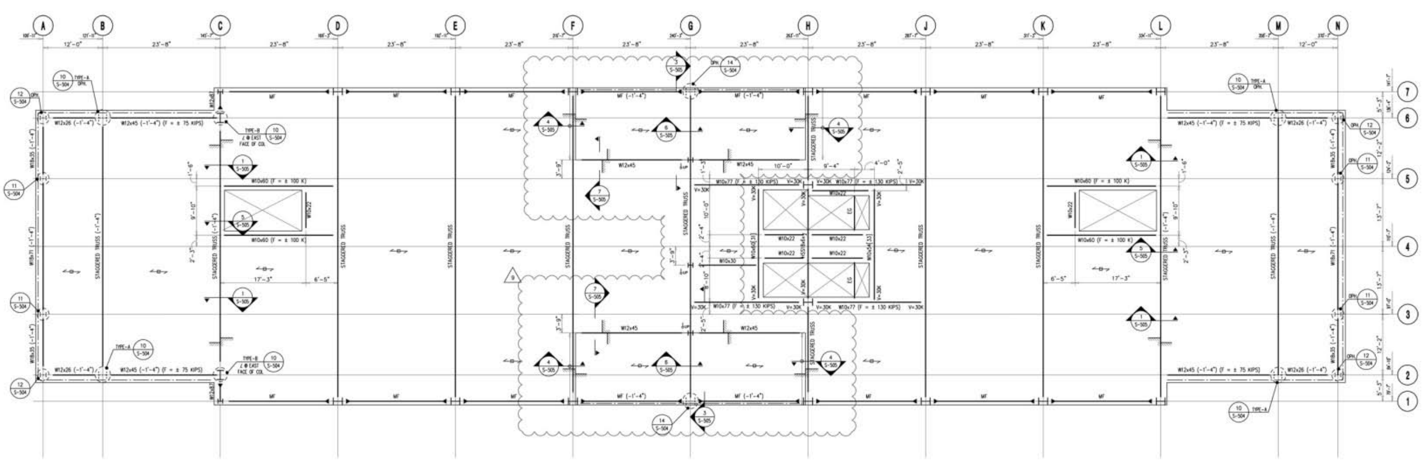


Figure A.2 Typical Floor Plan for Residential Tower

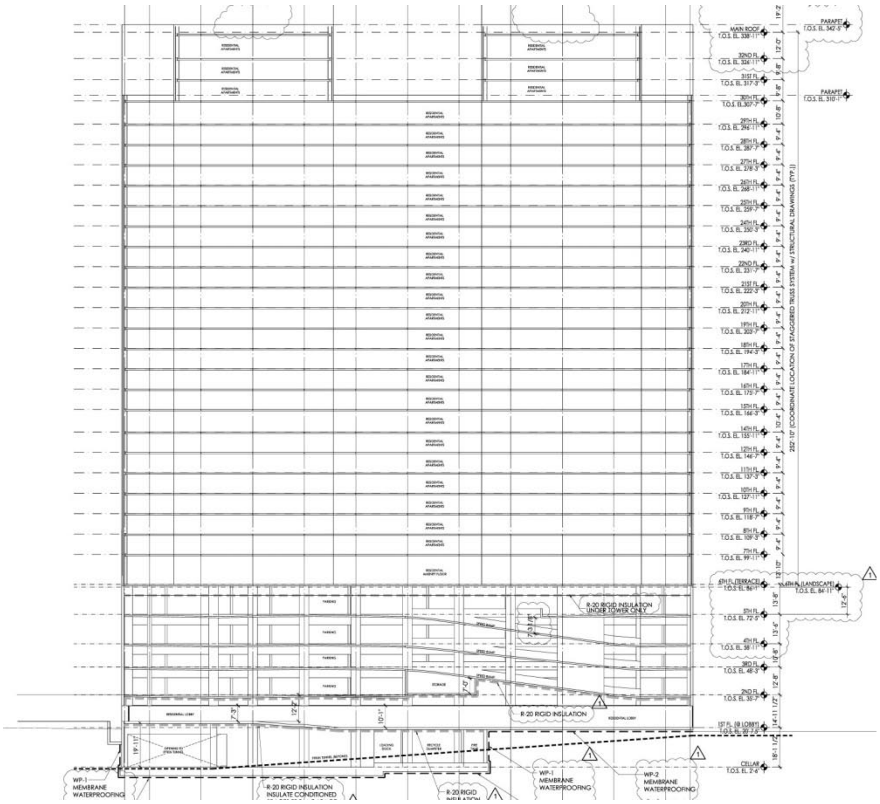


Figure A.3 North/South Building Elevation

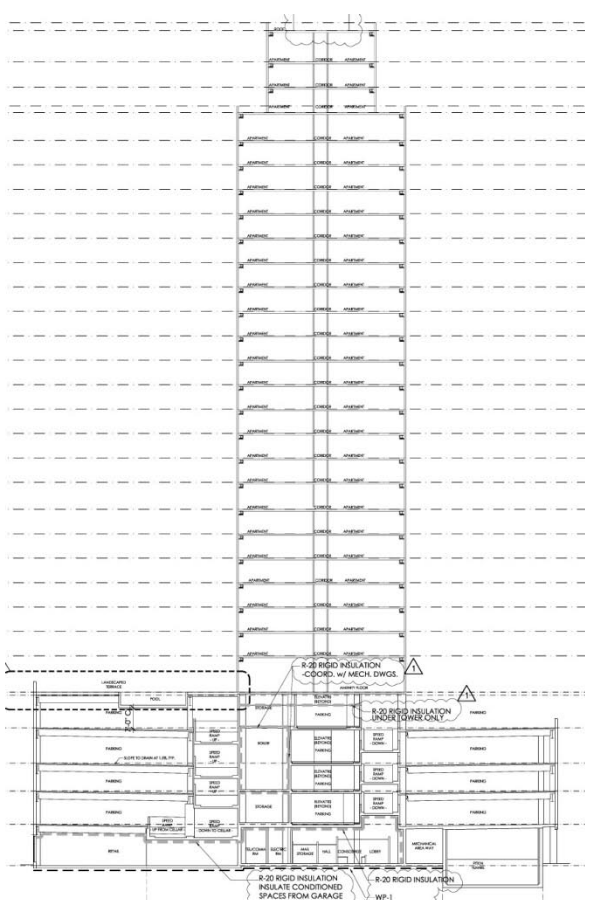


Figure A.4 East/West Building Elevation

Appendix B – Design Criteria

The following data is provided to illustrate the general design criteria used for 360 State Street.

Codes & Design Standards

Applied to Original Design
2005 Connecticut State Building Code consisting of the 2003 International Building Code as modified by the 2005 Connecticut Supplement∵
American Institute of Steel Construction *Specification for Structural Steel Buildings – Allowable Stress Design and Plastic Design* 01 June 1989 (AISC)∵
American Concrete Institute *Building Code Requirements for Structural Concrete* ACI 318-02 (ACI)∵
American Concrete Institute *Building Code Requirements for Masonry Structures* ACI 530-99 (ACI 530)
American Iron and Steel Institute *Specification for the Design of Cold-Formed Steel Structural Members* 1996 (AISI)

∵Substituted for Analysis
American Society for Civil Engineers *Minimum Design Loads for Buildings and Other Structures* ASCE-7-05
American Institute of Steel Construction *Steel Construction Manual, Thirteenth Edition* April 2007 (AISC)
American Concrete Institute *Building Code Requirements for Structural Concrete and Commentary* ACI 318-08 (ACI)

Table 1: Codes & Standards used for Original & Analyzed Design
Note: Thesis Design Analysis was conducted using Load and Resistance Factor Design (LRFD).

Material Strength Requirements

Material	Strength Requirement
Structural Steel: All Rolled Shapes Connection Materials	 ASTM A572 (A992), Grade 50 ASTM A36
Metal Deck	ASTM A611 or A653 w/ ASTM A653 G60 Galv.
Cast-In-Place Concrete: Foundations Slabs-On-Grade Formed Slabs Columns and Walls	 4 ksi NWC 4 ksi NWC 5 ksi NWC 8 ksi NWC (Foundation to 6th Floor)
Reinforcement	ASTM A615, Grade 60 Except all #11 Bars are Grade 75
Light Gage Framing	ASTM A653, Grade 50

Table B.1: Material Strength Requirements as per drawing S001

Deflection Criteria

Construction[1]	Live	Snow or Wind[f]	D + L[g]
Roof Members[e]:			
Supporting Plaster Ceiling	ℓ/360	ℓ/360	ℓ/240
Supporting Non-Plaster Ceiling	ℓ/240	ℓ/240	ℓ/180
Not Supporting Ceiling	ℓ/180	ℓ/180	ℓ/120
Floor Members	ℓ/360	-	ℓ/240
Exterior Walls and Interior Partitions:			
With Brittle Finishes	-	ℓ/240	-
With Flexible Finishes	-	ℓ/120	-

Table B.2: Deflection Limitations outlined by IBC 2003

e. The above deflections do not ensure against ponding.
f. The wind is permitted to be taken as 0.7 times the "component and cladding" loads for the purpose of determining deflection limits herein.
g. For steel structural members, the dead load shall be taken as zero.

Dead & Live Loads

Level	Load Type	Design Dead Load (psf)	Design Super-Imposed Dead Load (psf)	Design Live Load (psf)	Live Load per ASCE 7 - 05(psf)
Foundation	Loading Dock	Varies on Mat Slab Thickness	40	100	-
Grade	Public	150	40	100	100
2nd to 5th	Parking	125	22	40	40
6th Terrace	Amenities	150	25	100	100
	Terrace Typ.	200	160	100	100
	Terrace Planters	200	400	100	100
	Large Tree Planters	250	620	100	-
7th	Residential	61	20	40	40
	Public	61	20	100	100
8th to 31st	Residential	61	20	40	40
	Public	61	20	100	100
Mechanical/Roof	Mechanical	61	20	40	-

Table B.3: Dead & Live Load Schedule

Note: According to Section 1606.1 in the International Building Code 2003, dead loads considered for design shall be the actual weight of materials and construction.

Occupancy/Function	psf	Occupancy/Function	psf
Corridor	100	Public Space	100
Storage (Light)	125	Lobby	100
Office	50	Terrace (Private, Public)	60, 100
Residential	40	Parking (Passenger Cars)	40

Table B.4: Additional Uniformly Distributed Live Loads from ASCE 7 Table 4-1

[1] Table 1604.3 Deflection Limits, 2003 International Building Code Portion of the 2005 Connecticut State Building Code

Appendix C – Wind Load Calculations

Wind Load Design Criteria			
Basic Wind Speed (3 s Gust)	V = 110 mph	K_d = 0.85	G = 0.85
Wind Importance Factor	I_w = 1.0	K_{zt} = 1.0	
Wind Exposure	B	Design Category	II
Internal Pressure Coefficient (Enclosed Building)	GC_{pi} = + 0.18 windward = - 0.18 leeward	Combined Net Pressure Coefficient	GC_{pn} = + 1.5 windward parapet = - 1.0 leeward parapet
p_p = 55.29 psf windward parapet -36.86 leeward parapet	F_p = 193.5 plf windward parapet -129.0 plf leeward parapet		

Table C.1 Wind Design Criteria According to ACSE 7 – 05

Sample Calculation

The wind calculations were determined according to Chapter 6 in ASCE 7 – 05. Additionally, an example problem from David A. Fanella's *Structural Load Determinations Under 2006 IBC and ASCE/SEI 7-05* was used as a reference. Table B.1 can be referenced for specific design criteria for 360 State Street.

$q_z = 0.00256K_zK_{zt}K_dV^2I$ (eq 6-15)

K_z was interpolated for each floor height (Table 6-3)

$q_{z\text{ (25th floor)}} = (0.00256)*(1.29)*(1.0)*(0.85)*(110^2)*(1.0) = 33.97$ psf

qGC_p = External Pressure where C_p = 0.8 windward

$qGC_{p\text{ (25th floor)}} = (33.97\text{ psf})*(0.85)*(0.8) = 23.10$ psf

q_hGC_{pi} = Internal Pressure

$q_hGC_{pi\text{ (25th floor)}} = (33.97\text{ psf})*(-0.18) = -6.54$ psf

Net Pressure p was determined by the summation of the external and internal pressures.

Force (k) = (Floor height)*(Length of building)*(External pressure)/1000

$F_{(25th\text{ floor})} = (9.34\text{ ft})*(276\text{ ft})*(23.10\text{ psf}) = 59.5$ k E/W

Shear (k) = Force of current floor + Force of above floor

$S_{(25th\text{ floor})} = 59.5\text{ k} + 465.8\text{ k} = 525.3$ k E/W

The frame analysis excludes internal pressures and suction/uplift pressures.

Location	Floor	Height Above Ground, z (ft)	Floor Height (ft)	K_z	q_z (psf)	Forces (k)		Shear (k)	
						E/W	N/S	E/W	N/S
Windward	Roof Parapet	338.583	3.5	1.4	36.86	-	-	-	-
	32	326.917	12	1.38	36.34	81.8	20.8	81.8	20.8
	31	317.25	9.67	1.37	36.07	65.5	16.6	147.3	37.4
	30	307.583	9.67	1.36	35.81	65.0	16.5	212.3	53.9
	29	296.917	10.67	1.345	35.41	70.9	18.0	283.2	71.9
	28	287.583	9.34	1.33	35.02	61.4	15.6	344.6	87.5
	27	278.25	9.34	1.32	34.76	60.9	15.5	405.5	103
	26	268.917	9.34	1.306	34.39	60.3	15.3	465.8	118.3
	25	259.583	9.34	1.29	33.97	59.5	15.1	525.3	133.4
	24	250.25	9.34	1.28	33.70	59.1	15.0	584.4	148.4
	23	240.917	9.34	1.265	33.31	58.4	14.8	642.8	163.2
	22	231.583	9.34	1.25	32.91	57.7	14.6	700.5	177.8
	21	222.25	9.34	1.235	32.52	57.0	14.5	757.5	192.3
	20	212.917	9.34	1.22	32.12	56.3	14.3	813.8	206.6
	19	203.583	9.34	1.205	31.73	55.6	14.1	869.4	220.7
	18	194.25	9.34	1.19	31.33	54.9	13.9	924.3	234.6
	17	187.917	9.34	1.18	31.07	54.5	13.8	978.8	248.4
	16	175.583	9.34	1.16	30.54	53.5	13.6	1032.3	262
	15	166.25	9.34	1.14	30.02	52.6	13.3	1084.9	275.3
	14	155.917	10.34	1.12	29.49	57.2	14.5	1142.2	289.8
	12	146.583	9.34	1.103	29.04	50.9	12.9	1193.1	302.7
	11	137.25	9.34	1.083	28.52	50.0	12.7	1243.1	315.4
	10	127.917	9.34	1.059	27.88	48.9	12.4	1291.9	327.8
	9	118.583	9.34	1.036	27.28	47.8	12.1	1339.8	339.9
	8	109.25	9.34	1.013	26.67	46.8	11.9	1386.5	351.8
	7	99.917	9.34	0.99	26.07	45.7	11.6	1432.2	363.4
	6	86.03	10.83	0.948	24.96	50.7	12.9	1482.9	376.3
	5	72.417	13.67	0.899	23.67	60.7	44.0	1543.7	420.3
	4	58.917	13.5	0.846	22.28	56.4	40.9	1600.1	461.2
	3	48.25	10.67	0.805	21.20	42.4	30.8	1642.5	492
	2	35.583	12.67	0.726	19.12	45.5	32.9	1688.0	524.9
	1	2.5	14.125	0.7	18.43	48.9	35.4	1736.9	560.3
Leeward	All				36.30				

Table C.2: Summary of Wind Pressures & Forces.

Appendix D – Seismic Load Calculations

Wind Load Design Criteria			
Basic Wind Speed (3 s Gust)	V = 110 mph	K_d = 0.85	G = 0.85
Wind Importance Factor	I_w = 1.0	K_{zt} = 1.0	
Wind Exposure	B	Design Category	II
Internal Pressure Coefficient (Enclosed Building)	GC_{pi} = + 0.18 windward = - 0.18 leeward	Combined Net Pressure Coefficient	GC_{pn} = + 1.5 windward parapet = - 1.0 leeward parapet
p_p = 55.29 psf windward parapet -36.86 leeward parapet	F_p = 193.5 plf windward parapet -129.0 plf leeward parapet		

Table D.1 Seismic Design Criteria According to ACSE 7 – 05

The seismic calculations were determined according to Chapter 12 in ASCE 7 – 05. Additionally, an example problem from David A. Fanella's *Structural Load Determinations Under 2006 IBC and ASCE/SEI 7-05* was used as a reference. Table B.1 can be referenced for specific design criteria for 360 State Street.

Level	Story Weight w_x (kips)	Height h_x (ft)	$w_x h_x^k$	Lateral Force F_x (kips)	Story Shear V_x (kips)
32	1588	326.92	10538905	69.1	69.1
31	1951	317.25	12370514	81.1	150.1
30	1953	307.58	11814218	77.4	227.5
29	1735	296.92	9947290	65.2	292.7
28	1689	287.58	9224646	60.4	353.2
27	1689	278.25	8773464	57.5	410.7
26	1691	268.92	8339948	54.7	465.3
25	1691	259.58	7903938	51.8	517.1
24	1694	250.25	7489314	49.1	566.2
23	1694	240.92	7068901	46.3	612.5
22	1699	231.58	6676480	43.8	656.3
21	1699	222.25	6271810	41.1	697.4
20	1707	212.92	5903550	38.7	736.1
19	1707	203.58	5514684	36.1	772.2
18	1713	194.25	5153071	33.8	806.0
17	1713	187.92	4899884	32.1	838.1
16	1719	175.58	4434955	29.1	867.1
15	1719	166.25	4081630	26.7	893.9
14	1722	155.92	3708781	24.3	918.2
12	1722	146.58	3376605	22.1	940.3
11	1725	137.25	3060609	20.1	960.4
10	1725	127.92	2749920	18.0	978.4
9	1730	118.58	2457877	16.1	994.5
8	1730	109.25	2169935	14.2	1008.7
7	7354	99.92	8053330	52.8	1061.5
6	11229	86.03	9795102	64.2	1125.7
5	11171	72.42	7499800	49.1	1174.8
4	10208	58.92	5008485	32.8	1207.7
3	10889	48.25	3943700	25.8	1233.5
2	11048	35.58	2518676	16.5	1250.0
			190750023		
V =	1250 k				

Table D.2: Summary of Seismic Forces & Shears

Appendix E – Torsion Calculations

Center of Rigidity								
Floor	k_{iy}	x_i (ft)	$k_{iy}x_i$	k_{ix}	y_i (ft)	$k_{ix}y_i$	X	Y
32	132.0	853.2	112589	45.1	64	2888	94.8	32.0
31	146.4	853.2	124901	49.8	64	3187	94.8	32.0
30	166.0	853.2	141633	56.1	64	3593	94.8	32.0
29	184.8	853.2	157679	62.2	64	3980	94.8	32.0
28	204.6	853.2	174550	68.8	64	4406	94.8	32.0
27	219.9	853.2	187599	74.1	64	4741	94.8	32.0
26	234.7	853.2	200282	79.3	64	5075	94.8	32.0
25	252.5	853.2	215400	85.6	64	5475	94.8	32.0
24	273.2	853.2	233134	92.9	64	5946	94.8	32.0
23	296.8	853.2	253228	101.3	64	6481	94.8	32.0
22	324.5	853.2	276833	111.1	64	7111	94.8	32.0
21	358.3	853.2	305686	123.2	64	7883	94.8	32.0
20	398.6	853.2	340083	137.6	64	8807	94.8	32.0
19	445.5	853.2	380061	154.5	64	9886	94.8	32.0
18	503.4	853.2	429542	175.4	64	11227	94.8	32.0
17	580.5	853.2	495240	203.3	64	13013	94.8	32.0
16	676.6	853.2	577306	238.4	64	15259	94.8	32.0
15	787.1	853.2	671547	279.0	64	17855	94.8	32.0
14	927.9	853.2	791686	331.0	64	21182	94.8	32.0
12	1137.5	853.2	970538	408.6	64	26153	94.8	32.0
11	1441.3	853.2	1229749	521.6	64	33380	94.8	32.0
10	1859.1	853.2	1586168	676.9	64	43322	94.8	32.0
9	2543.9	853.2	2170440	931.0	64	59585	94.8	32.0
8	4068.3	853.2	3471115	1497.9	64	95866	94.8	32.0
7	8503.4	853.2	7255102	3142.7	64	201131	94.8	32.0
6	31446.5	853.2	26830189	11614.4	64	743322	94.8	32.0
∑	523020	-	49582280	42524	-	1360757	-	-
			Overall	**X = 94.8’**	**Y = 32.0’**			

Table E.1: Center of Rigidity Summary

Mass Matrix											
	Y-Frames									X-Frames	
Frame	C	D	E	F	G	H	J	K	L	1	6
Floor	0	23.7	47.4	71.1	94.8	118.5	142.2	165.9	189.6	0.0	64.0
32		12.7		12.7		12.7		12.7		56.8	56.8
31	12.7		12.7		12.7		12.7		12.7	55.7	55.7
30		13.0		13.0		13.0		13.0		57.4	57.4
29	13.0		13.0		13.0		13.0		13.0	56.2	56.2
28		13.0		13.0		13.0		13.0		12.3	12.3
27	13.5		13.5		13.5		13.5		13.5	12.1	12.1
26		13.5		13.5		13.5		13.5		13.6	13.6
25	14.1		14.1		14.1		14.1		14.1	13.4	13.4
24		14.1		14.1		14.1		14.1		15.2	15.2
23	14.4		14.4		14.4		14.4		14.4	14.0	14.0
22		14.4		14.4		14.4		14.4		16.0	16.0
21	15.3		15.3		15.3		15.3		15.3	15.8	15.8
20		15.3		15.3		15.3		15.3		18.2	18.2
19	15.8		15.8		15.8		15.8		15.8	16.7	16.7
18		5.3		5.3		5.3		5.3		19.4	19.4
17	5.8		5.8		5.8		5.8		5.8	17.8	17.8
16		5.8		5.8		5.8		5.8		20.7	20.7
15	6.4		6.4		6.4		6.4		6.4	19.0	19.0
14		6.4		6.4		6.4		6.4		22.2	22.2
12	7.5		7.5		7.5		7.5		7.5	21.1	21.1
11		7.5		7.5		7.5		7.5		25.3	25.3
10	8.6		8.6		8.6		8.6		8.6	23.7	23.7
9		8.6		8.6		8.6		8.6		28.0	28.0
8	9.4		9.4		9.4		9.4		9.4	25.4	25.4
7		9.4		9.4		9.4		9.4		30.1	30.1
6	9.4		9.4		9.4		9.4		9.4	25.4	25.4

Table E.2: Summary of Masses per Frame

Note: The Mass Matrix organizes the weight of each frame according to the story it is located. The unit of measurement is in kips.

Center of Mass						
Floor	$\sum m_x$	$\sum m_y$	$\sum m_i x_i$	$\sum m_i y_i$	X	Y
32	50.9	113.7	4826.1	3637.2	94.8	32.0
31	63.6	111.4	6032.6	3565.1	94.8	32.0
30	51.8	114.8	4911.6	3673.3	94.8	32.0
29	64.8	112.3	6139.5	3593.9	94.8	32.0
28	51.8	24.6	4911.6	787.8	94.8	32.0
27	67.3	24.2	6380.1	773.3	94.8	32.0
26	53.8	27.2	5104.1	869.0	94.8	32.0
25	70.5	26.7	6683.1	855.2	94.8	32.0
24	56.4	30.4	5346.5	971.3	94.8	32.0
23	72.2	28.1	6843.5	898.5	94.8	32.0
22	57.8	32.0	5474.8	1025.4	94.8	32.0
21	76.5	31.5	7253.4	1009.2	94.8	32.0
20	61.2	36.4	5802.7	1163.8	94.8	32.0
19	79.0	33.5	7485.1	1071.7	94.8	32.0
18	21.3	38.8	2017.5	1242.0	94.8	32.0
17	29.2	35.6	2771.4	1139.1	94.8	32.0
16	23.4	41.4	2217.1	1326.2	94.8	32.0
15	32.1	37.9	3047.6	1213.7	94.8	32.0
14	25.7	44.4	2438.1	1419.5	94.8	32.0
12	37.4	42.1	3546.7	1348.5	94.8	32.0
11	29.9	50.6	2837.3	1618.6	94.8	32.0
10	42.8	47.4	4054.6	1516.3	94.8	32.0
9	34.2	55.9	3243.7	1790.0	94.8	32.0
8	47.0	50.8	4455.6	1624.6	94.8	32.0
7	37.6	60.2	3564.5	1925.4	94.8	32.0
6	47.0	50.8	4455.6	1624.6	94.8	32.0
		Overall	**X = 94.8'**	**Y = 32.0'**		

Table E.3: Center of Mass Summary

TORSION

SAMPLE CALCULATION: **STIFFNESS**

STORY DRIFT & DISPLACEMENT TAKEN FROM RAM MODEL w/ AN APPLIED UNIT LOAD

$$k = \frac{AE}{L}, \quad \delta = \frac{PL}{AE} \rightarrow A = \frac{PL}{\delta E}$$

$$k = \frac{\left(\frac{PL}{\delta \cancel{E}}\right)\frac{\cancel{E}}{1}}{\frac{L}{1}}\left(\frac{1}{\cancel{L}}\right)\left(\frac{1}{\cancel{L}}\right)$$

$$k = \frac{P}{\delta}$$ WHERE $P = 1000^k$ APPLIED UNIT LOAD

$$\% k = \frac{k_i}{\Sigma k_i} \times 100$$

FRAME X: $\delta_{25} = 11.689''$

$$k = \frac{1000^k}{11.689''}$$

$$\boxed{k = 85.6}$$

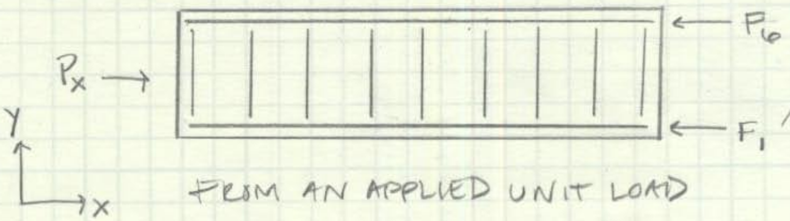
$$\% k = \frac{85.6}{(2 \times 85.6)} \times 100$$

$$\boxed{\% k = 50\%}$$

1/6

$\boxed{\text{TORSION}}$

SAMPLE CALCULATION: $\boxed{\text{DIRECT LOAD}}$



SINCE k IS EQUAL FOR BOTH FRAMES, $P_1 = F_6$

FROM AN APPLIED UNIT LOAD

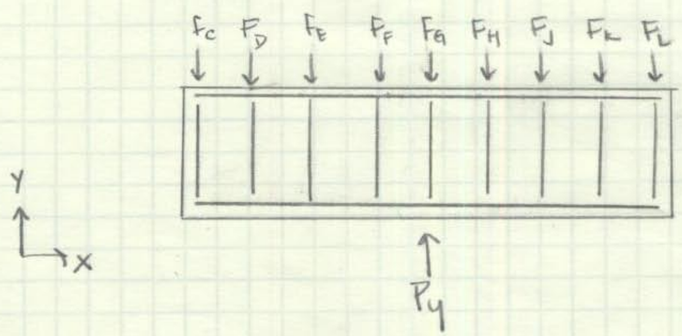
FROM EQUILIBRIUM $\quad F_{ix} = k_{ix}\Delta_x$

$$P_x = \Sigma F_{jx} \rightarrow P_x = \Delta_x \Sigma k_{jx} \rightarrow \Delta_x = \frac{P_x}{\Sigma k_{jy}}$$

$$\hookrightarrow F_{ix} = \frac{k_{ix}}{\Sigma k_{jx}} P_x$$

$$F_{25x_1} = \frac{85.6}{(2 \times 85.6)}(1000^k) = 500^k$$

$$\boxed{F_{x_1} = P_{x_6} = 500^k}$$



SINCE k IS EQUAL FOR ALL FRAMES, ALL FORCES ARE EQUAL

$$F_{iy} = \frac{k_{iy}}{\Sigma k_{jy}} P_y$$

$$F_{25y} = \frac{252.5}{(9 \times 252.5)}(1000^k) = 111.1^k$$

$$\boxed{F_{25y_C} = 111.1^k}$$

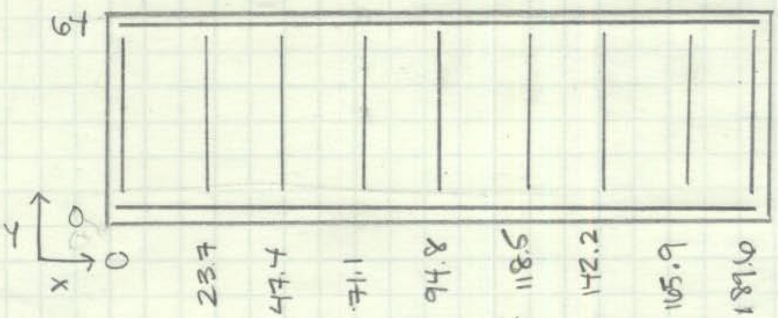
2/6

TORSION

SAMPLE CALCULATION: **CENTER OF RIGIDITY**

$$\bar{x} = \frac{\sum k_{iy} x_i}{\sum k_{iy}} \quad ; \quad \bar{y} = \frac{\sum k_{ix} y_i}{\sum k_{ix}}$$

k_{iy} & k_{ix} VALUES TAKEN FROM TABLE X



$\sum x_i = 853.2'$

$\sum y_i = 64'$

$$\left.\begin{aligned} \sum k_{iy} x_i &= k_{iy}(x_1 + x_2 + \ldots) \\ \sum k_{ix} y_i &= k_{ix}(y_1 + y_2 + \ldots) \end{aligned}\right\}$$

k_{ix} & k_{iy} ARE EQUIVALENT FOR EACH FRAME PER FLOOR

THEREFORE

$$\sum k_{iy} x_i = k_y \sum x_i$$
$$\sum k_{ix} y_i = k_x \sum y_i$$

$k_{25y} = 252$

$k_{25x} = 85.6$

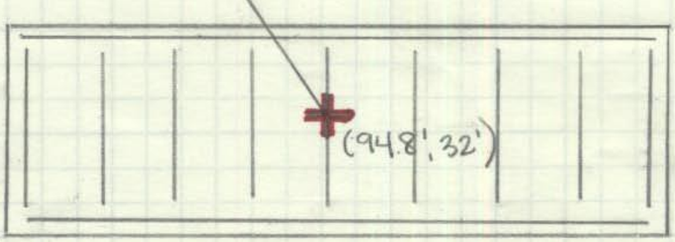
$$\bar{x}_{25} = \frac{252(853.2')}{9(252)}$$

$$\boxed{\bar{x}_{25} = 94.8'}$$

$$\bar{y}_{25} = \frac{85.6(64)}{2(85.6)}$$

$$\boxed{\bar{y}_{25} = 32'}$$

CENTER OF RIGIDITY @25TH FLOOR



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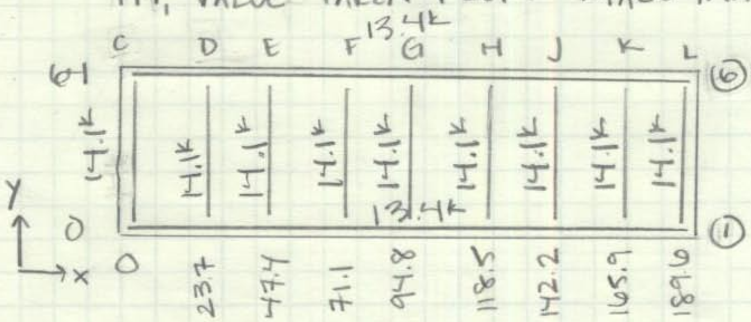
TORSION

SAMPLE CALCULATION: **CENTER OF MASS**

$$\bar{x} = \frac{\Sigma m_i x_i}{\Sigma m_i} \quad ; \quad \bar{y} = \frac{\Sigma m_i y_i}{\Sigma m_i}$$

m_i VALUE TAKEN FROM MASS MATRIX



$\Sigma x_i = 853.2'$

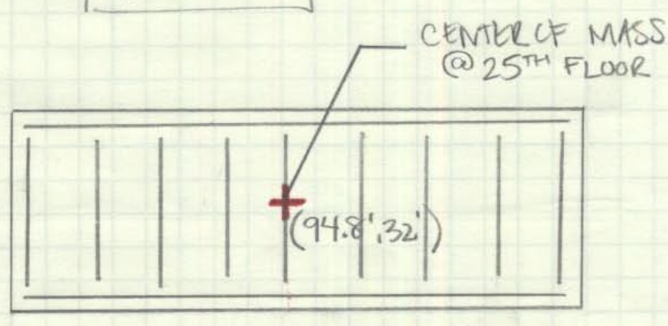
$\Sigma y_i = 64'$

$$\bar{x}_{25} = \frac{(14.1^k)(853.2')}{(9 \times 14.1^k)}$$

$$\boxed{\bar{x}_{25} = 94.8'}$$

$$\bar{y}_{25} = \frac{(13.4^k)(64')}{(2 \times 13.4^k)}$$

$$\boxed{\bar{y}_{25} = 32'}$$



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TORSION

SAMPLE CALCULATION: INDIRECT FORCE or TORSION

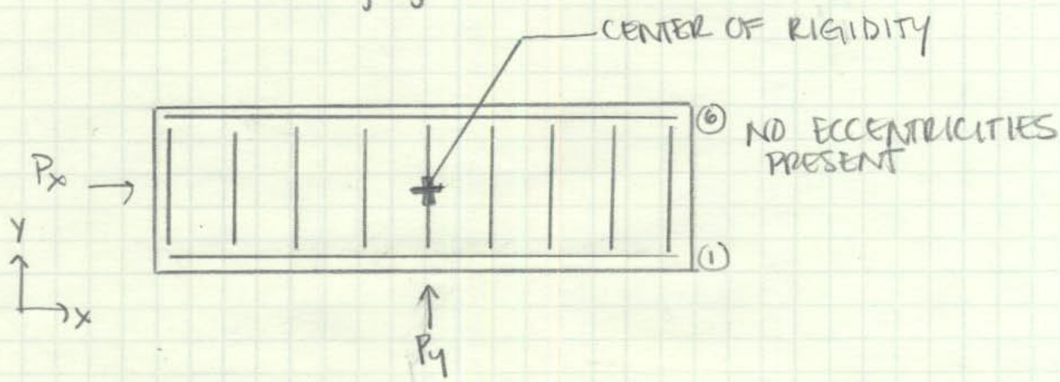
FROM EQUILIBRIUM $\Sigma M = 0$

$$M_{px} = P_x e_y = \Sigma F_{jt} d_j$$

$$= \Sigma k_j d_j^2 \theta_t$$

$$\theta_t = \frac{P_x e_y}{\Sigma k_j d_j^2}$$

$$F_{it} = \frac{k_i d_i P_x e_y}{\Sigma k_j d_j^2}$$



$$F_{25tx_1} = \frac{(85.6)(32')(500^k)(0)}{(2)(85.6)(32')^2}$$

$$\boxed{F_{25tx} = 0}$$

IT CAN BE ASSUMED THAT

$F_{25ty} = 0$ AS WELL

THE ANGLE OF TWIST θ_t CAN ALSO BE ASSUMED TO EQUAL ZERO SINCE THERE ARE NO ECCENTRICITIES PRESENT.

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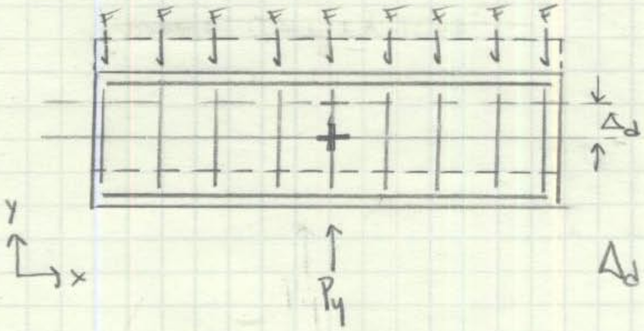
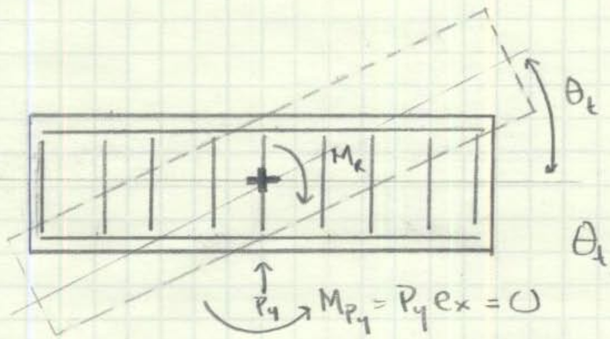
TORSION - SUMMARY



TRANSLATION
DIRECT FORCE

Δ_d = BUILDING TRANSLATION



ROTATION
INDIRECT FORCE

θ_t = BUILDING TWIST

NO ROTATION IS PRESENT THEREFORE THERE IS NO TORSIONAL CONCERNS HOWEVER TRANSLATIONAL DISPLACEMENTS EXIST WITH THE APPLICATION OF LATERAL FORCES.

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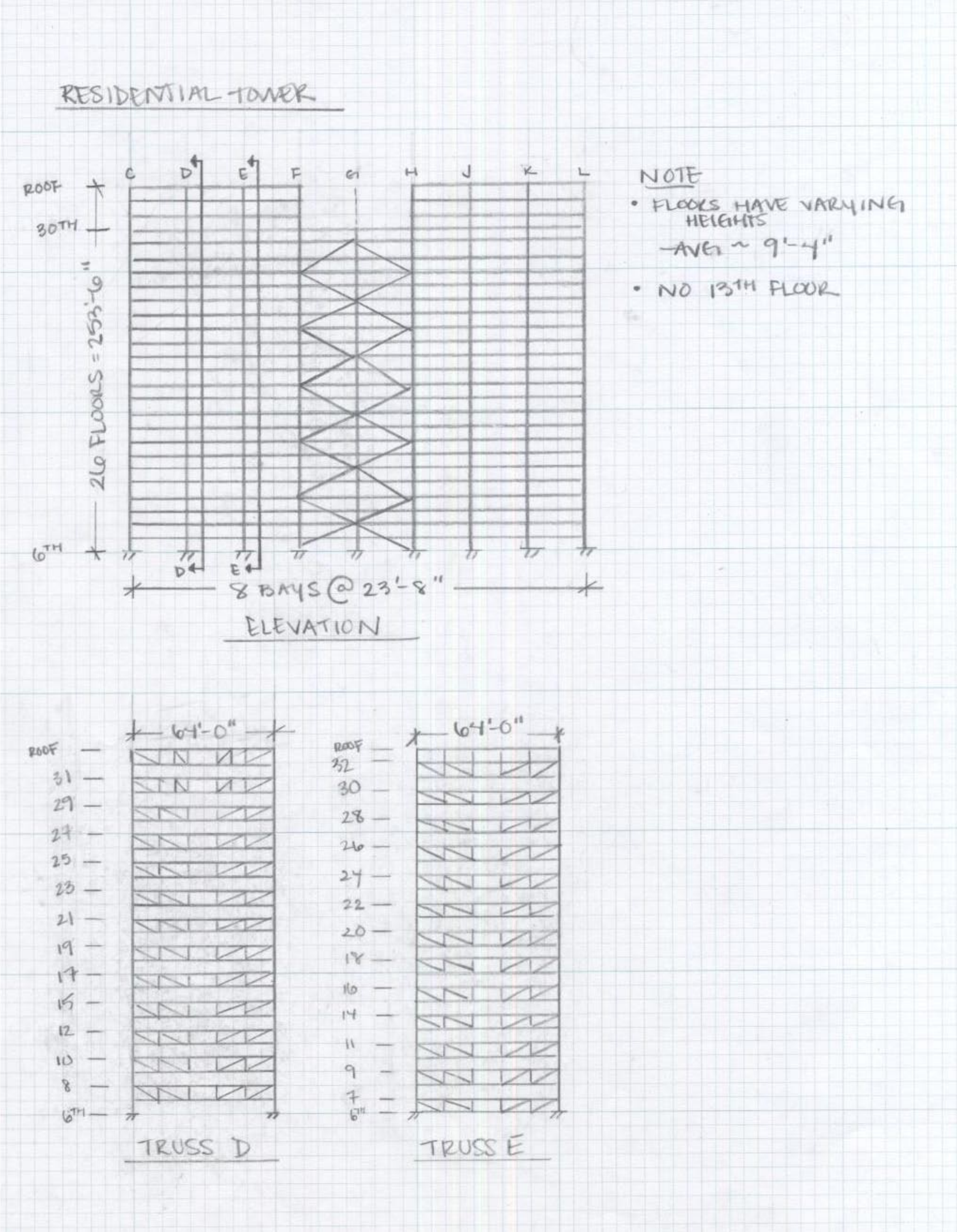
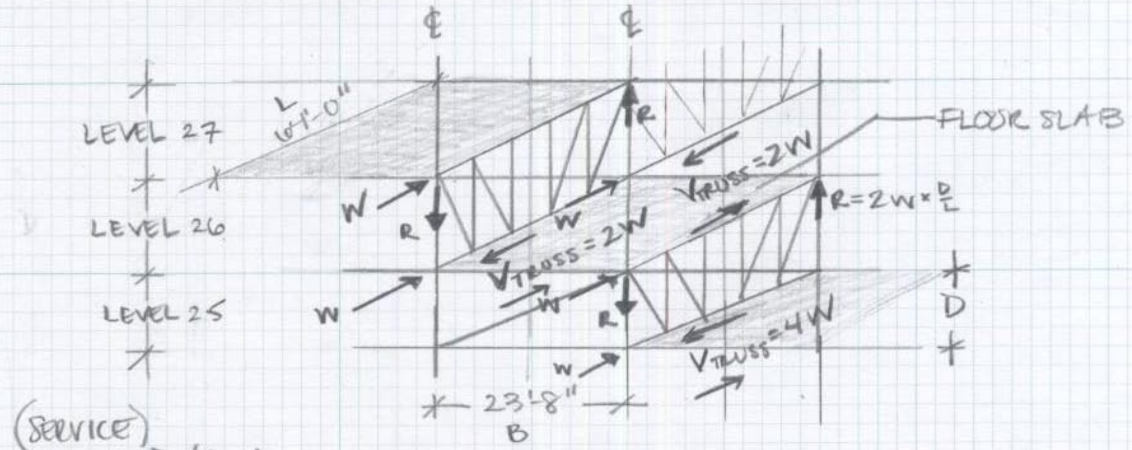
Appendix F – Lateral Movement Calculations



WIND DISTRIBUTION THROUGH STAGGERED TRUSS



(SERVICE)
FACTORED, 1.6W

WIND LOAD	Level	INCREMENTAL WIND LOAD, W_i	CUMMULATIVE WIND LOAD, ΣW TRUSS D	E
	ROOF	6.6k		
33'-7" (36.9psf) 59.0psf	32	13.4k	6.6k	
	31	13.4k		20k
	30	13.4k	33.4k	
	29	12.9k		46.8k
50' (35.5psf) 56.8psf	28	12.6k	59.7k	
	27	12.6k		72.3k
	26	12.6k	84.9k	
	25	12.6k		97.5k
50' (33.7psf) 53.9psf	24	12.3k	110.1k	
	23	12.0k		122.4k
	22	12.0k	134.4k	
	21	12.0k		146.4k
	20	12.0k	158.4k	
50.6 psf (31.6psf)	19	11.6k		170.4k
	18	11.2k	182.0k	
49.3 psf (30.8psf)	17	11.1k		193.2k
	16	11.0k	204.3k	
47.7psf (29.8psf)	15	10.8k		215.3k
	14	10.6k	226.1k	
6@20' (28.7psf) 45.9psf	12	10.4k		236.7k
	11	10.2k	247.1k	
(27.4psf) 43.8psf	10	9.9k		257.3k
	9	9.6k	267.2k	
(26.1psf) 41.8psf	8	9.4k		276.8k
	7	9.3k	286.2k	
	6	4.7k	295.5k*	295.5k

253'-6"

WIND LOAD CONTINUED

* 6TH STORY WIND LOAD IS RESISTED BY CONCRETE BASE
VALUE WILL BE EXCLUDED FROM TRUSS SHEAR TOTALS

NOTE
- WIND PRESSURES ARE TAKEN FROM TABLE X
- AVERAGE FLOOR HEIGHT USED FOR SIMPLIFICATION

SAMPLE CALC : $W_i = (\text{FLOOR HEIGHT})(\text{BAY WIDTH})(\text{WIND PRESSURE})$

$$W_{25th} = (9'\text{-}4'')(23'\text{-}8'')(56.8\ \text{psf})/1000 = 12.6^k$$

SUMMARY

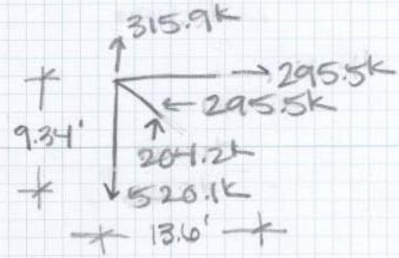
TOTAL TRUSS SHEAR	D	E
1 BAY	1715.8^k	2150.6^k
2 BAYS	3431.6^k	4301.2^k

OVERTURNING

$R = 2W\,(D/L)$ [EQ 1, R.E. LEFFLER]

WHERE R = VERTICAL COLUMN LOAD

	TRUSS D	TRUSS E
USING 1 BAY 7TH STORY	$1715.8^k\left(\frac{9.34'}{64'}\right)$*	$2150.6^k\left(\frac{9.34'}{64'}\right)$
	$R = 252.0^k$	$R = 315.9^k$
6TH STORY	NO CHANGE	
	$R = 252.0^k$	$R = 520.1^k$



DEAD LOADS (SEE TABLE X)

$DL_{RF} = 80\,psf \rightarrow 0.08\,ksf(23.7')(64')(1/2) = 60.7^k$

$DL = 80\,psf \rightarrow 0.08\,ksf(23.7')(64')(1/2) = 60.7^k$

DL IN 6TH STORY COLUMN

1 ROOF @ $60.7^k = 60.7^k$

25 FLOORS @ $60.7^k = 1517.5^k$

$\Sigma = 1578.2^k$

OVERTURNING CHECK

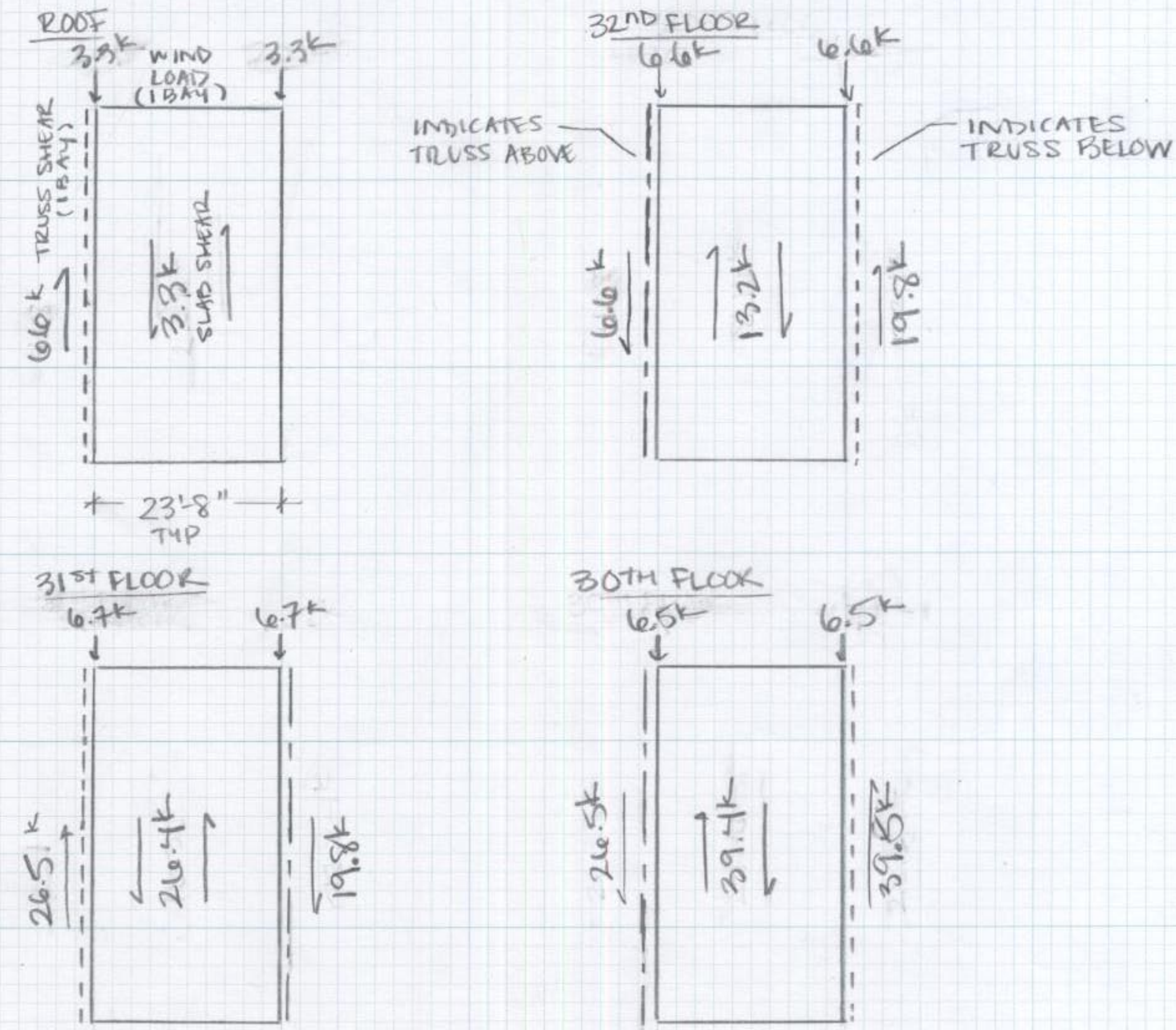
TRUSS D $\frac{DL}{WL} = \frac{1578^k}{252^k} = 6.3$ ok

TRUSS E $\frac{DL}{WL} = \frac{1578^k}{520.1^k} = 3.0$ ok

NOTE * BY MULTIPLYING TOTAL SHEAR IN TRUSS BY DEPTH TO SPAN RATIO SIMPLIFIES EQ 1 BY R.E. LEFFLER

WIND LOAD DISTRIBUTION

TYPICAL WIND LOADING FOR AN INDIVIDUAL BAY ASSUMING WIND IS EQUALLY DISTRIBUTED TO EACH COLUMN.



WIND LOAD AND SHEAR DISTRIBUTION

FOLLOWING THE PATTERN ESTABLISHED ON THE PREVIOUS PAGE, THE TABULATED TRUSS AND FLOOR WIND LOADS ARE BELOW

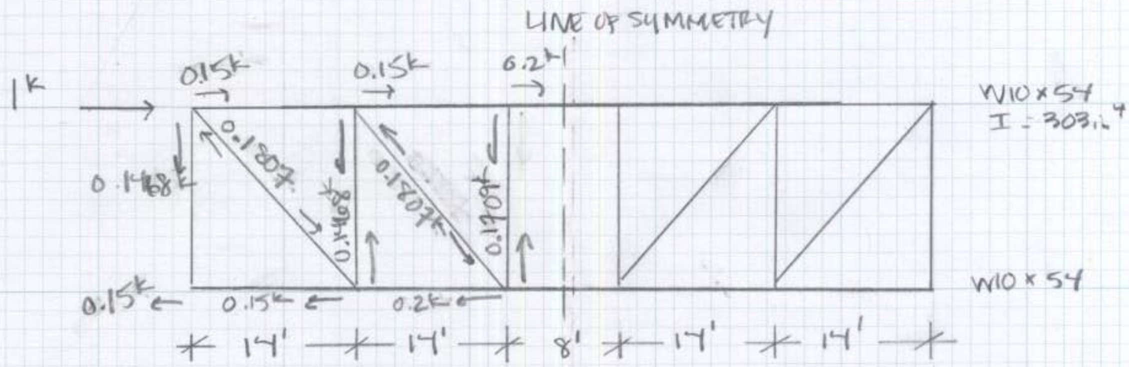
		TRUSS D			TRUSS E	
FLOOR	STORY	WL @ FLOOR	TRUSS* W. SHEAR	FLOOR SLAB W. SHEAR	WL @ FLOOR	TRUSS* W. SHEAR
ROOF	32	3.3k		3.3k	3.3k	
32	31	6.7k	6.6k	13.2k	6.7k	
31	30	6.7k		26.4k	6.7k	20k
30	29	6.5k	33.4k	39.4k	6.5k	
29	28	6.3k		52.1k	6.3k	46.8k
28	27		59.7k	64.7k		
27	26			77.3k		72.3k
26	25		84.9k	89.9k		
25	24	6.3k		102.5k	6.3k	97.5k
24	23	6.0k	110.1k	114.5k	6.0k	
23	22			126.5k		122.4k
22	21		134.4k	138.5k		
21	20			150.5k		146.4k
20	19	6.0k	158.4k	162.5k	6.0k	
19	18	5.8k		174.1k	5.8k	170.4k
18	17	5.6k	182.0k	185.3k	5.6k	
17	16	5.5k		196.3k	5.5k	193.2k
16	15	5.4k	204.3k	207.1k	5.4k	
15	14	5.3k		217.7k	5.3k	215.3k
14	12	5.3k	226.1k	228.3k	5.3k	
12	11	5.2k		238.7k	5.2k	236.7k
11	10	5.1k	247.1k	248.9k	5.1k	
10	9	4.9k		258.7k	4.9k	257.3k
9	8	4.8k	267.2k	268.3k	4.8k	
8	7	4.7k	286.2k	277.7k	4.7k	276.8k
7	6	4.6k		286.9k	4.6k	
6	–	2.3k	295.5°	2.3k°	2.3k	295.5k

* TRUSS WIND SHEAR FOR 1 BAY. TOTAL TRUSS SHEAR INCLUDES WIND LOAD FROM ADJACENT BAY

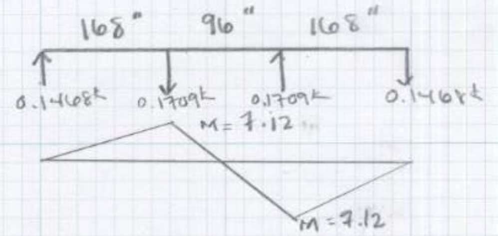
° VALUES EXCLUDED BECAUSE THEY ARE TRANSFERRED IN CONCRETE BASE AT 6TH FLOOR

TRUSS DISPLACEMENT – WIND LOAD RELATIONSHIP

VIRTUAL WORK



$$\Delta_{BT} = \sum \int \frac{M_i m_i dx}{EI_i} \rightarrow \frac{MmL}{3EI}$$



$$\Delta_{BT} = 2\left[\frac{(7.12)^2(168+48)}{3(2.9\times10^4)\,I}\right] = \frac{0.2517}{I}$$

$$= 0.2517\left[\frac{1}{303} + \frac{1}{303}\right] = 0.001666 \text{ in}$$

MEMBER	F	u	L	A	$\frac{FuL}{A}$
DIAGONAL	0.1807	0.1807	202.5	10.4	0.6357
↓	0.1807	0.1807	202.5	10.4	↓
COLUMN	0.1468	0.1469	112.8	9.71	0.2503
↓	0.1468	0.1469	112.8	9.71	↓

$\sum \frac{FuL}{A}$ FOR FULL TRUSS = 3.544

$$\Delta_{AT} = 3.544/29000 = 1.222\times10^{-4}$$

$$\Delta_{TOTAL\ FULL\ TRUSS} = \Delta_{BT} + \Delta_{AT} = 0.0017822 \text{ in/kip}$$

$$\Delta_{TOTAL\ NO\ TRUSS} = 0.001677 \text{ in/kip}$$

STRUCTURAL DRIFT FOR TRUSS E (FRAME Y)

TRUSS DISPLACEMENT = TRUSS SHEAR × 0.0017822 in/kip

NOTE: THIS CALCULATION EXCLUDES DRIFT DUE TO FLOOR SLABS

STORY	SHEAR (k)	DRIFT/STORY (in)
32	13.2k	0.024
31	26.4k	0.047
30	39.4k	0.070
29	52.1k	0.093
28	64.7k	0.115
27	77.3k	0.138
26	89.9k	0.160
25	102.5k	0.183
24	114.5k	0.204
23	126.5k	0.225
22	138.5k	0.247
21	150.5k	0.268
20	162.5k	0.290
19	174.1k	0.310
18	185.3k	0.330
17	196.3k	0.350
16	207.1k	0.369
15	217.7k	0.388
14	228.3k	0.407
12	238.7k	0.425
11	248.9k	0.444
10	258.7k	0.461
9	268.3k	0.478
8	277.7k	0.495
7	286.9k	0.511
6	—	—

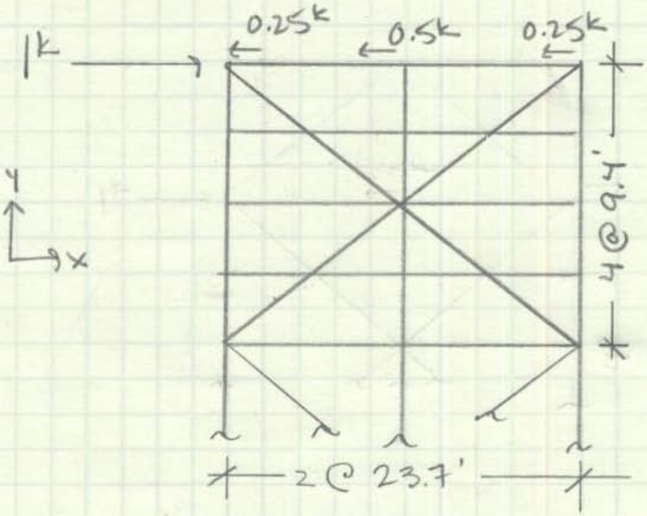
SEISMIC LOAD DISTRIBUTION THROUGH STAGGERED TRUSS

USING THE SAME PRINCIPLES THAT WERE APPLIED FOR WIND FORCES

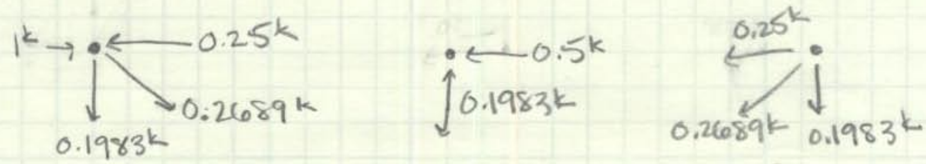
SEISMIC LOADS		SHEAR (k)	DRIFT/STORY (in)
	ROOF		
69.1k	32	15.3	0.027
81.1k	31	33.4	0.059
77.4k	30	50.6	0.090
65.2k	29	65.1	0.116
60.4k	28	78.5	0.140
57.5k	27	91.3	0.163
54.7k	26	103.4	0.184
51.8k	25	114.9	0.205
49.1k	24	125.8	0.224
46.3k	23	136.1	0.243
41.1k	22	145.8	0.260
38.7k	21	155.0	0.276
36.1k	20	163.6	0.292
33.8k	19	171.6	0.306
32.1k	18	179.1	0.319
29.1k	17	186.2	0.332
26.7k	16	192.7	0.343
24.3k	15	198.6	0.354
22.1k	14	204.0	0.364
20.1k	12	209.0	0.374
18.0k	11	213.4	0.380
16.1k	10	217.4	0.387
14.2k	9	221.0	0.394
52.8k	8	224.2	0.399
64.2k	7	235.9	0.420
	6	—	—

X-FRAME

VIRTUAL WORK



FRAME CONSIDERED SYMMETRICAL ABOUT Y-AXIS



MEMBER	F	u	L	A	$\frac{FuL}{A}$
DIAG 1	0.2689^k	0.2689	306"	17.2	1.2864
2	0.2689^k	0.2689	306"	17.2	↓
COL 1	0.1983^k	0.1983	113"	68.5	0.0649
2	0.1983^k	0.1983	113"	68.5	↓
3	0.1983^k	0.1983	113"	68.5	↓
BEAM 1	0.25^k	0.25	284"	15.6	1.1378
2	0.25	0.25	284"	15.6	↓

$$\Sigma \frac{FuL}{A} = 5.0431$$

$$\Delta_{TOTAL} = 5.0431/29000 = 0.0001739 \text{ in/kip}$$

$$\Delta_{TOTAL_{29 \,\&\, ABOVE}} = 9.6865/29000 = 0.000334 \text{ in/kip}$$

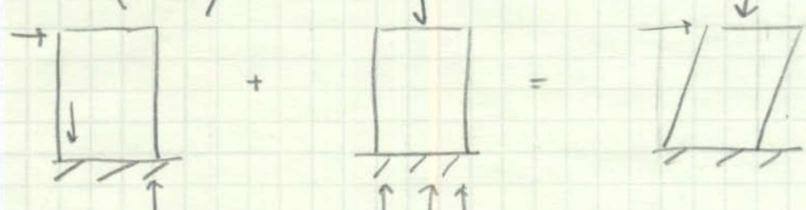
MULTIPLY Δ_{TOTAL} BY FORCES APPLIED TO EACH FRAME

1/1

Appendix G – Overturning Moment

OVERTURNING MOMENT   LATERAL ONLY-FRAME

LOAD (E or W)   DEADLOAD

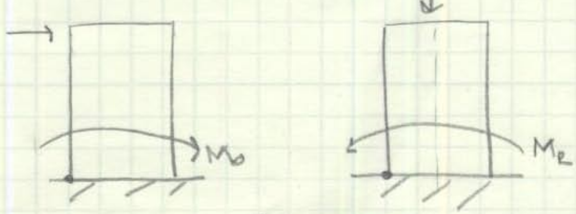


OVERTURNING < RESISTING

BUILDING HEIGHT = 253.8'
x = 189.6'
y = 64'

WIND

x = 145.9k
y = 651.3k

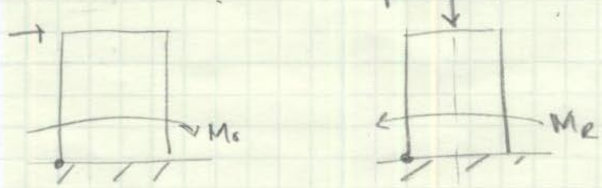


$\boxed{X}$ $M_o = 89.5^k\left(\frac{253.8'}{2}\right)$ < $\boxed{X}$ $M_R = 145.9^k(94.8')$ OKAY
= 11,358 k ft   = 13,831 k ft

$\boxed{Y}$ $M_o = 121^k\left(\frac{253.8}{2}\right)$ < $\boxed{Y}$ $M_R = 651.3^k(32')$ OKAY
= 15,355 k ft   = 20,842 k ft

SEISMIC

x = 145.9k
y = 651.3k



$\boxed{X}$ $M_o = (69.4^k)(126.9')$ < $\boxed{X}$ $M_R = 13,831$ k ft   OKAY
= 8,807 k ft

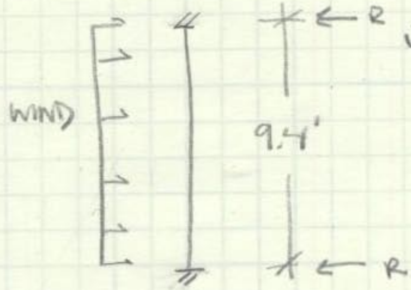
$\boxed{Y}$ $M_o = 312.5^k(126.9')$ > $\boxed{Y}$ $M_R = 20,842$ k ft   NOT OKAY
= 39,656 k ft

1/1

Appendix H – Strength Checks

$\boxed{\text{STRENGTH CHECK}}$ W14×193

COLUMN IN Y-FRAME @ 25TH FLOOR



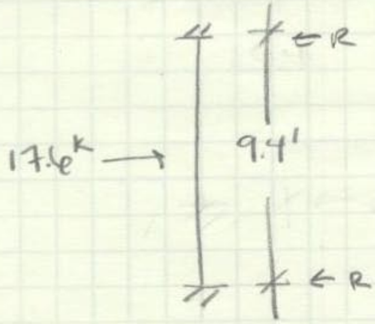
$$w_{WIND} = 1.6(35.5psf)(23.7') = 1346plf$$

$$R = \frac{wl}{2} = \frac{1346plf(9.4')}{2 \times 1000} = 6.3^k$$

$$M_u = \frac{wl^2}{24} = \frac{(1346plf)(9.4')^2}{24 \times 1000} = 4.95 kft$$

$$\phi M_n = 1330^{kft} > M_u = 4.95 kft \rightarrow OK$$

$$\Delta_{CENTER} = \frac{wl^4}{384EI} = \frac{(1346plf)(9.4^4)(1728)}{384(29000000)(2400 in^4)} = 0.000679''$$



$P_{1.0E} = 17.6^k$ AT CENTER

$$R = \frac{P}{2} = \frac{17.6^k}{2} = 8.8^k$$

$$M_u = \frac{Pl}{8} = \frac{17.6^k(9.4')}{8} = 20.7 kft$$

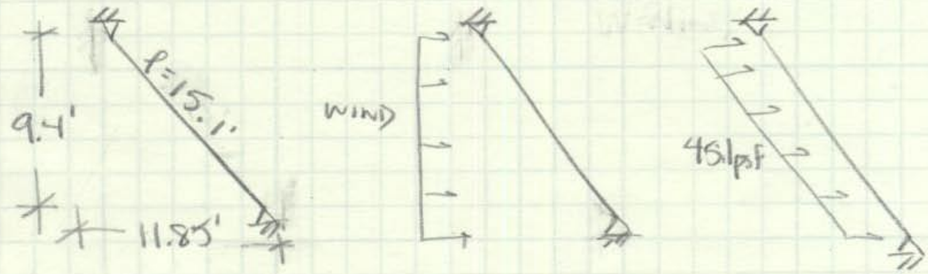
$$\phi M_n = 1330^{kft} > M_u = 20.7 kft$$

$$\Delta_{CENTER} = \frac{Pl^3}{192EI} = \frac{17.6^k(9.4^3)(1728)}{192(29 \times 10^6)(2400)} = 1.89 \times 10^{-6} in$$

1/2

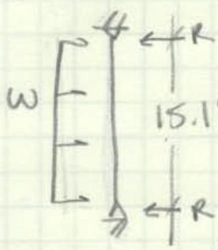
STRENGTH CHECK HSS 10 × 10 × 1/2

DIAGONAL IN X-FRAME @ 25TH FLOOR



$$W_{IND} = 1.6(35.5\text{psf})\left(\frac{9.4'}{11.85'}\right) = 45.1\text{psf}$$

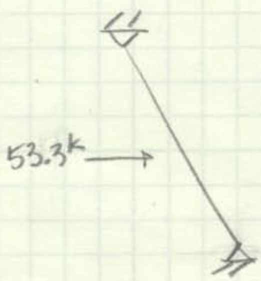
$$w = 45.1\text{psf}(32') = 1443\text{ plf}$$



$$R = \frac{w\ell}{2} = \frac{1443\text{plf}(15.1')}{2 \times 1000} = 10.9\text{k}$$

$$M_u = \frac{w\ell^2}{8} = \frac{1443\text{plf}(15.1')^2}{8 \times 1000} = 41.1\text{ kft}$$

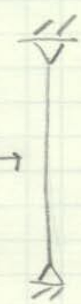
$$\phi M_n = 210^{kft} > M_u = 41.1\text{ kft} \rightarrow \text{ok}$$



$$P_{1.0E} = 53.3^k$$

$$P = 53.3^k\left(\frac{9.4'}{11.85'}\right) = 42.3^k$$

P →



$$R = \frac{P}{2} = \frac{42.3^k}{2} = 21.2k$$

$$M_u = \frac{P\ell}{4} = \frac{42.3^k(15.1')}{4} = 159.7\text{ kft}$$

$$\phi M_n = 210^{kft} > M_u = 159.7\text{ kft} \rightarrow \text{ok}$$

2/2