CEE 465

Final Submittal

LondonHouse

The University of Illinois at Urbana-Champaign
CEE 465 – Design of Structural Systems

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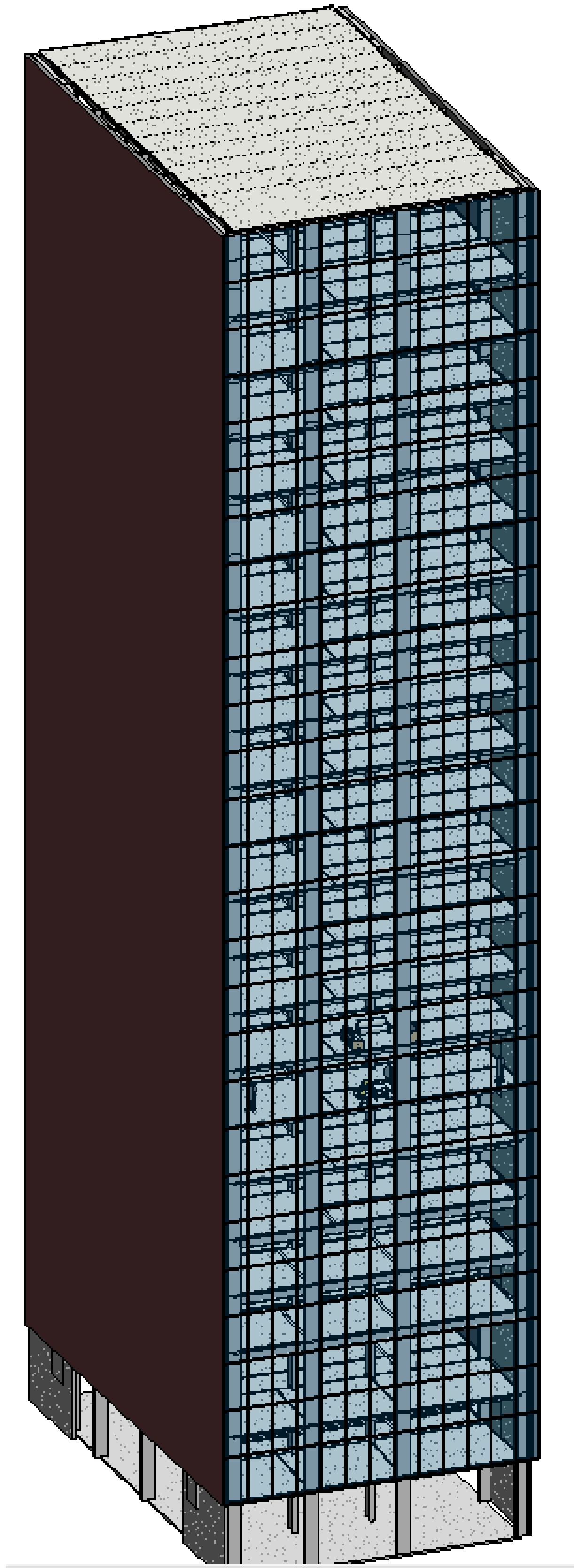
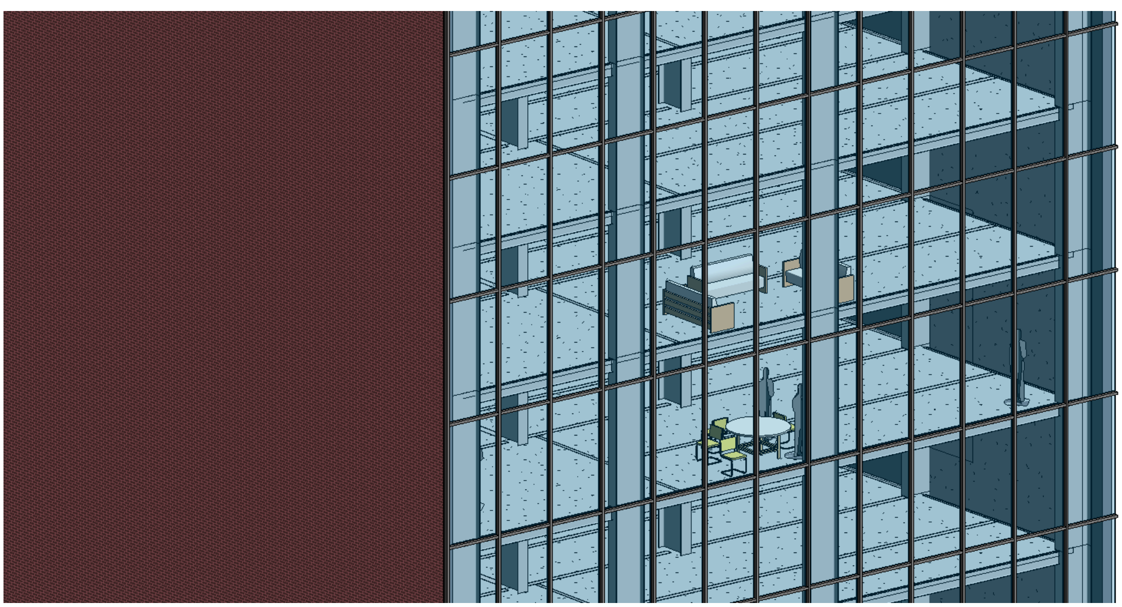


Design Criteria

Following the comments from the 4th submittal, all errors in lateral loading, materials, seismic loads and etc. as well as several unclear data have been corrected with more detailed information. In addition, information relating to foundation design and special transfer girder system has been updated into the design criteria.

For lateral loading resisting system, the new design contains 4 shear walls with 12" thickness and length ranging from 20 to 10 ft depending on the height above the concourse (lowest) level. Each shear wall is located on each corner of the building. In regards to the concrete density, the concrete being used for structural walls has a density of 165 pcf instead of the typical 145 pcf concrete used for slabs and other purpose. Further information relating to the grades of steel used for welded wire fabric, welds, and anchor rods have been added in the final submittal. Another project consideration included is that the area where the drift snow loads are applied is specified to be near the parapet walls. This resulted in a revision of the diagram of the snow drift loads. The lateral displacement values were based on the calculations of wind loads and the design of the lateral resisting system, and thus was compared to the suggested limit of h/500 and H/500. In the Load Cases and Combinations section, the load case U=0.9D+1.0W was added to the newest version.



Gravity Load Resisting System

The gravity load system was designed based on the design criteria in order to meet the corresponding limits for safety and serviceability. For the gravity load resisting system, a composite metal deck system with a 2VLI20 metal deck was selected. To satisfy the 2 hr. fire rating requirement, 4.5 inch of normal weight concrete was used for the slab on the metal deck. Seeing that deflection was governing the selection of the beam and girders, this portion of the gravity load system was designed for partial composite action.

Figure 1 shows the floor framing plan for the 6th to 22nd floor. All floors support a combination of hotel room loads of 40 psf and conference rooms of 100 psf. All of the floors above the 6th floor are identical to figure 1, with the exception of the length of the shear wall, which will be discussed in detail in the lateral resisting systems section.

To correct the shear stud design on the previous submittal, the interior girder on the 6th floor layout now contains 32 total studs for partially composite action. Previously, 30 studs were placed at equal spacing along the girder. It was found that for this degree of compositeness, 15 studs were required on each side of the beam from the column faces to only 1/3rd the length of the girder because there was no shear demand in the middle third of the girder. According to design codes, however, minimum amount of shear studs are required even in regions with no shear demand. The maximum spacing for shear studs is defined as the lower of the two following values: 36 inches or 8*the thickness of the concrete and the metal deck. As a result, two shear studs were placed at a spacing of 36 inches. Similarly on the 5th floor layout, for example the exterior girder W21 x 44 has 24 studs along the length with 11 on both the first third and last third of the length, whereas the center only has two studs spaced 36 inches apart.

Furthermore, the floor layout now includes the number of shear studs used for each beam or girder as well as the camber value in inches, if needed.

In regards to the ground floor to the 4th floors, one of the framing systems was designed under the presumption that all the other floors have the same layout. The ground floor does have a small discrepancy where instead of having a ballroom/conference room with a live load of 100 psf, it has a restaurant in the same area. According to ASCE 7-10, restaurants have a live load of 100 psf as well, so the assumption that all floors will have the same layout holds true.

Redesign–Chosen Layout

6th to 22nd Floor Design; Beams east to west

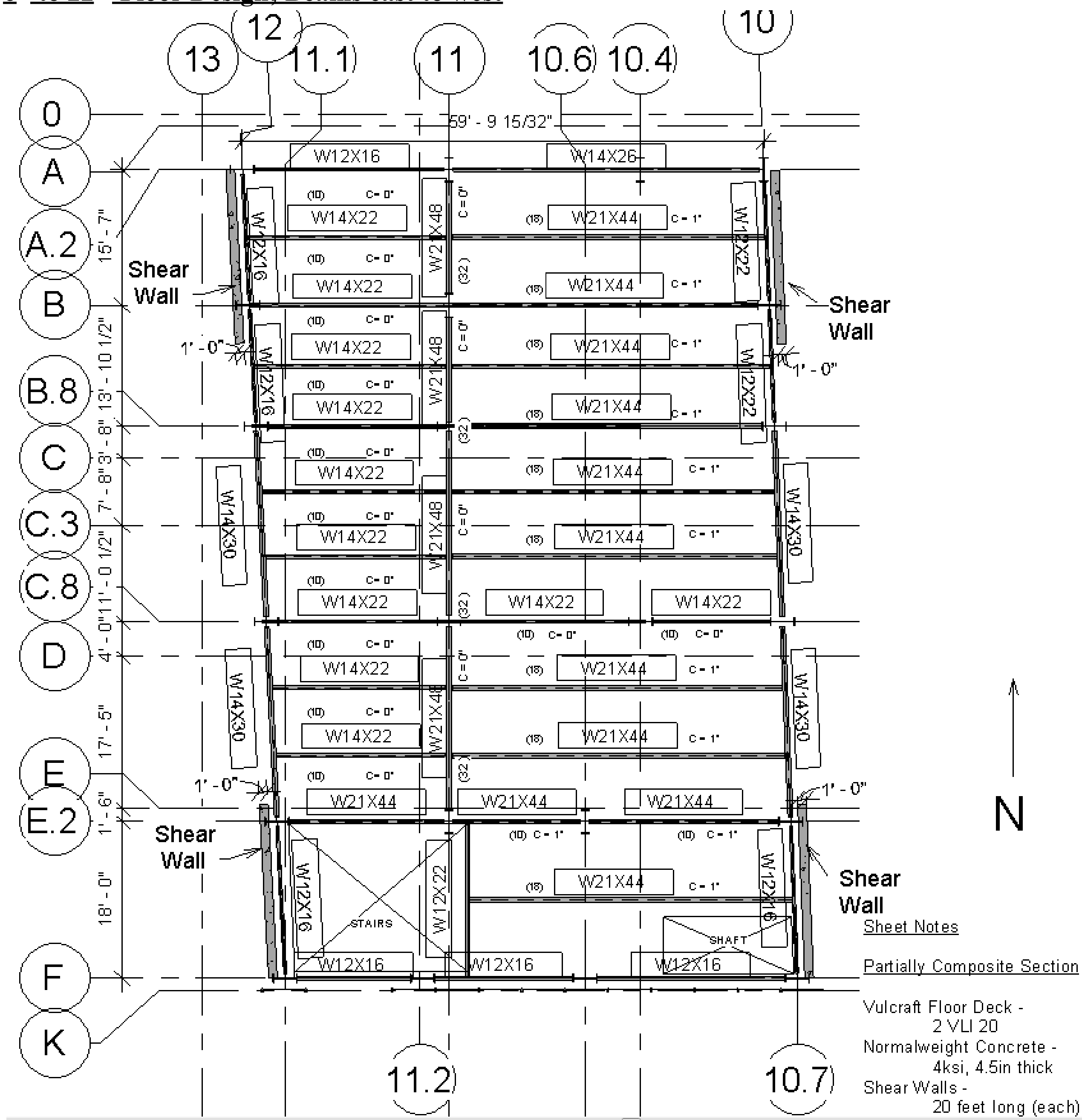


Figure 1. 6th to 22nd Floor Layout

6th – 22nd Floor Beams & Girders

Table 1. Beams and Girder Design Result

Properties	West Interior Beam	East Interior Beam	Interior Girder
Orientation	E-W	E-W	N-S
Section	W14x22	W21x44	W21x48
Length (ft)	20	38.9	23
Steel deck	2 VLI 20	2 VLI 20	2 VLI 20
Design moment (k-ft)	108	414	471
Moment capacity (φM_n) (k-ft)	194	555	687
Degree of compositeness	26.50%	29.77%	48.80%
Deflection (in)	0.668	1.352	1.126
Deflection limit (in)	1	1.945	1.148
Diagonal deflection (in)	1.513	2.197	---
Diagonal deflection limit (in)	1.52	2.257	---

Redesign – Chosen Layout

Ground, 2^{nd}, 3^{rd}, & 4^{th} Floor Design

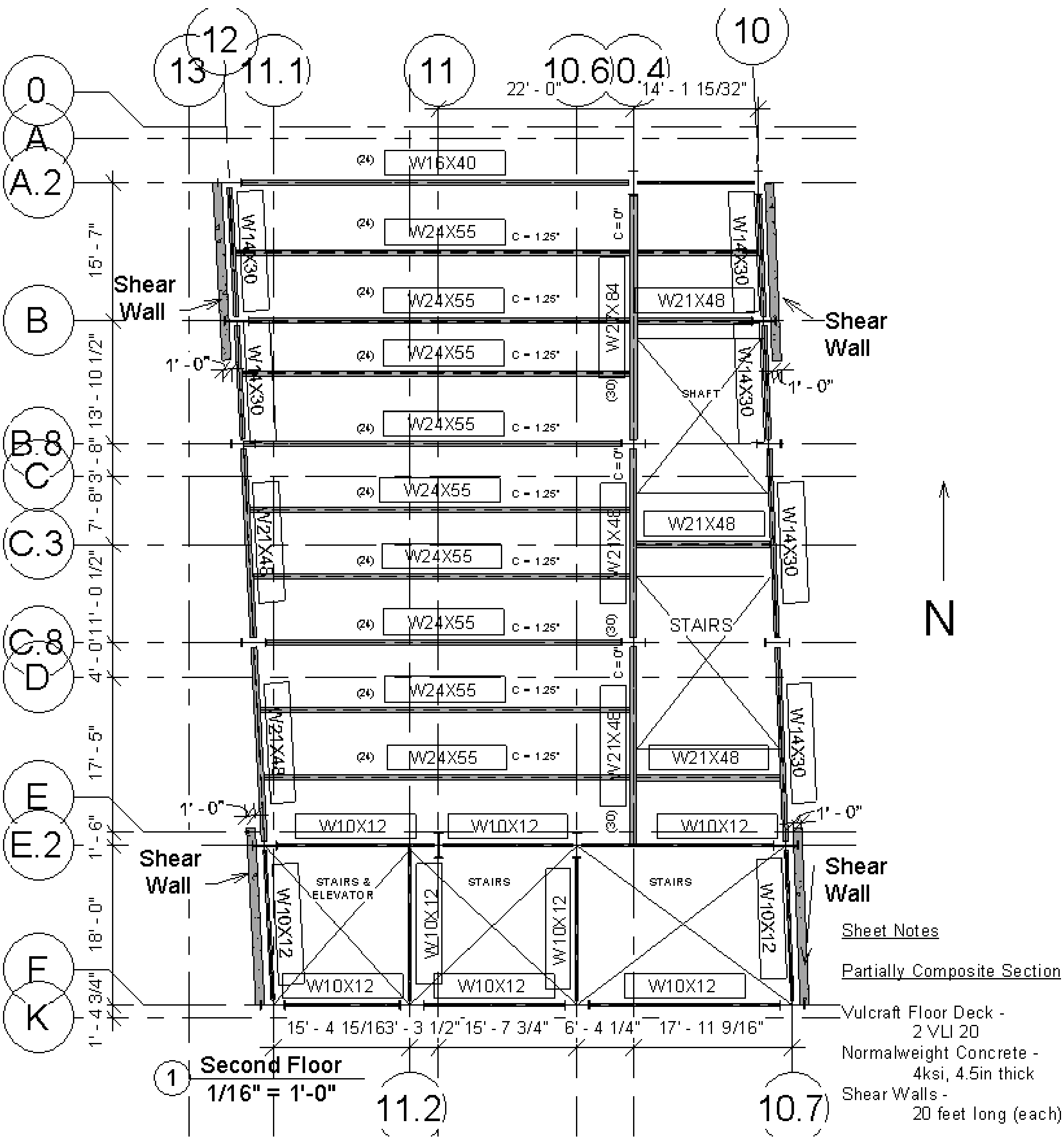


Figure 2. Ground, 2nd, 3rd, & 4th Floor Layouts

Ground, 2nd,3rd & 4th Floor Beams (East to West)

Table 3. Beam & Girder Design Result

Properties	Interior Beam	Interior Girder

Orientation	E-W	N-S
Section	W24x55	W27x84
Length (ft)	45.67	29.5
Steel deck	2 VLI 20	2 VLI 20
Design moment (k-ft)	580	905
Moment capacity (φM_n) (k-ft)	723	1292
Degree of compositeness	31.85%	26.11%
Deflection (in)	1.752	0.944
Deflection limit (in)	2.283	1.475
Diagonal deflection (in)	2.696	

Steel Materials Conclusion

Table 5. Final Design Criteria

Floors	Ground, 2nd, 3rd, & 4th	6th to 22nd
Beam Orientation	East to West	East to West
Beam Design	W 24 x 55	W 21 x 44
Beam Deflection	1.752 in	1.35 in
Beam Deflection Limit	2.283 in	1.94 in
Girder Orientation	North to South	North to South
Girder Design	W 27 x 84	W 21 x 48
Girder Deflection	0.944 in	1.126 in
Girder Deflection Limit	1.475 in	1.148 in
Total Deflection	2.70 in	2.20 in
Total Deflection Limit	2.72 in	2.26 in
Main concrete Thickness	4.5 in	4.5 in
Steel Deck	2 VLI 20	2 VLI 20
Shear Studs Along Beam	24	18
Beam Composite Degree	31.85 %	29.8 %

Special Feature: Transfer Girders

There are three columns along grid line 11 carrying the cumulative loads from the 5th floor to the top floor that terminate on the 5th floor. Because these cumulative loads are not transferred directly to the foundation, the loads must follow a path to different columns that could transfer these loads to the foundation. As a result, we have come up with a floor framing suggested layout for where the loads from the columns will travel through a truss system and into the columns that stem into the foundation. Therefore, the trusses on the 5th floor will be much heavier sections than those on the 6th - 22nd floors.

However, this layout may cause certain serviceability issues. Since all the columns terminate on floor 5, the transfer girder in the suggested layout would have to be one really long girder that spans 75 ft in the north-south direction. Additionally, this long girder would have to carry 3 large point loads along its span. There may be a big enough section that could sustain these loads, but there will most likely be extremely large deflections. Seeing as to how complicated the design of the transfer girder could be, we have suggested an additional layout.

This suggested layout removes the load transfer truss/girder oriented in the North-South direction, forcing the beams to span from the west end of the building to the interior girders or columns near the east side of the building. In this case, the “transfer trusses” are oriented in the East-West direction and are connected to the columns of the LondonHouse addition that extend down to the foundation. There are 3 total Pratt truss systems that each carry one-point load. A Pratt truss system was selected since its diagonal members are quite effective in resisting large point loads located at the center of the span. Each of the 45’ - 8” trusses span from a column on the west end to an interior column near the east end of the building. Since all the columns that support the trusses are apparent in the architectural drawings on the concourse floor, ground floor, and second floor, it is assumed they extend from the foundation and terminate on the floor of the fifth storey (ceiling of the fourth storey). Consequently, the transfer truss can be simply connected to it, allowing the transfer girder length to be short enough to possibly satisfy serviceability constraints.

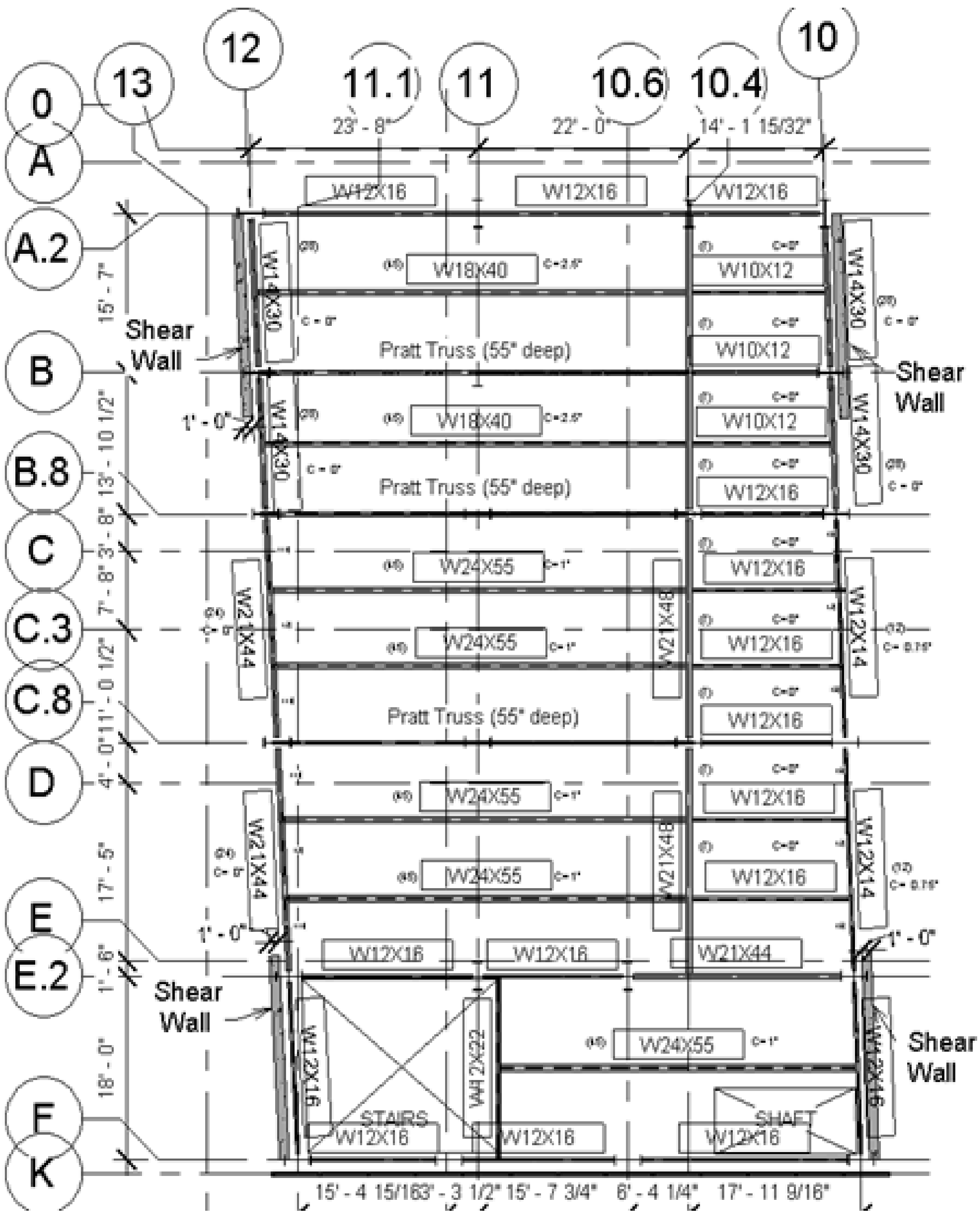


Figure 3. Fifth Floor Framing Layout with Transfer Truss

Due to architectural purposes, the depth of the framing system is limited to 60 inches. In order to meet this constraint, the truss system was designed to have a maximum depth of 55 inches. Making each section of the truss 55 inches long in the horizontal direction allows for 10 sections to fit within the 45’-8” span. The general framing concept of the truss is indicated in the figure below.

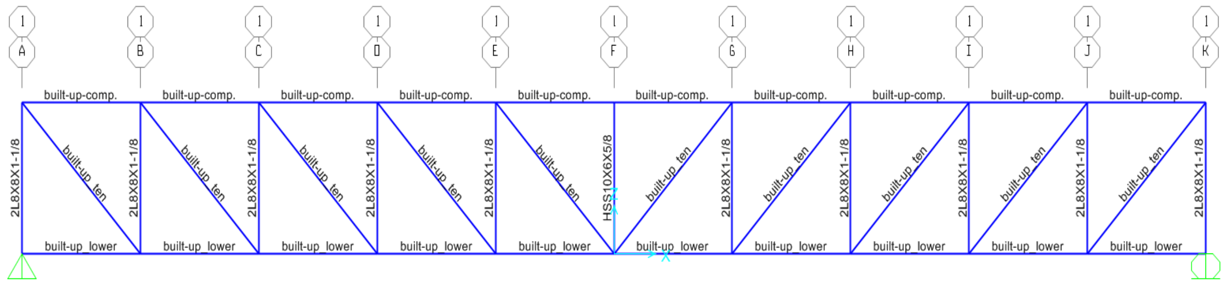


Figure 4. Pratt Truss Frame Sections

Because the large point load acts at the center of the pratt truss, the top horizontal members and all the vertical members undergo axial compression. On the other hand, the bottom horizontal members and the diagonal members all experience axial tension. The immensely large point load transferring through the truss and into the columns on either side causes the axial forces in the compression and tension members to be far too great for any double angle section to withstand. To resists these axial loads, built-up double angle plated sections are used for the top horizontal, bottom horizontal, and diagonal members. Details of a sample built-up section is shown in the Figure 5. The only members that can be designed using only a 2L8x8x1-⅛ section are the vertical members, with the exception of the vertical member right in the center. The vertical member in the center carries a load of 1929 kips. In order to reach this load demand and stay within the 6-inch width available on the face of a single angle section, 4 rectangular HSS10x6x⅝ sections were utilized, where 2 rectangular HSS sections were placed on either side of the double angle section. Details of this portion of the pratt truss are indicated in Figure 6. Sample calculations indicating the axial load capacities of each member is included on the next page.

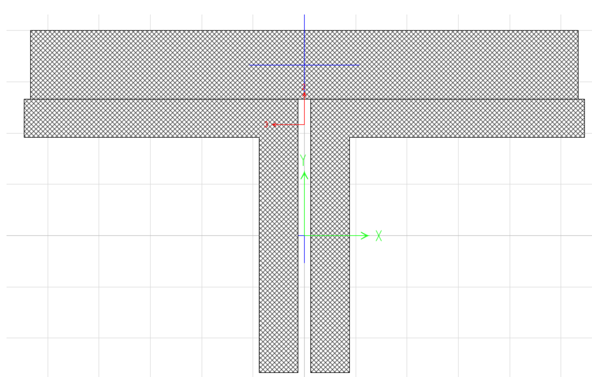


Figure 5. Built-up Bottom Horizontal Member

Wind Design

The design wind loads were calculated for each of the two primary orthogonal directions for the main lateral force resisting system. For the wind load analysis, it was assumed there exists a 4ft parapet above the highest elevation. At the advice of TGRWA's David Nickell, the wind load due to this parapet was included in the top elevation. The design wind loads were determined in accordance with the ASCE 7-10 design code. As per ASCE 7-10, the basic parameters used are tabulated below.

Table 6. Wind Load Parameters

Basic Parameters			
Name	Symbol	Unit	Value
Basic Wind Speed	V	mph	115
Wind Directionality factor	k_d	-	0.85
Exposure category	B		
	α	-	7
	z_g	ft	1200
	z_{min}*	ft	30
Topographical factor	k_{zt}	-	1
Gust effect factor	G	-	0.85
Enclosure classification	Enclosed Building		
Internal pressure coefficient	Gc_{pi}	-	0.18
			-0.18

These basic parameters, used in conjunction with the story tributary areas and other design criteria were used. See the following five pages for the wind load analysis procedure in the North-South and East-West directions, a diagram of the wind pressures by story height and elevation, and plots of the cumulative shear and cumulative overturning moment per building orientation. The cumulative shear and cumulative overturning moment plots show similar behavior that increase as the height of the building decreases. It should be noted that the overturning moment at the highest elevation is zero because the shear of the parapet is included into that elevation. In reality, a small, negative overturning moment will exist because the force exerted is above the highest elevation.

Table 7. East West Wind Loadings by Elevation

Floor	Elevation	Height Above Ground	Story Load Trib Length	Story Load Trib Area			External Pressure Coefficients			Velocity Pressures			Design Wind Pressure		
				w- 97.75			Windward	Leeward	Side	Windward	Leeward	Side	Windward	Leeward	Si
					k_z	k_h	c_p	c_p	c_p	q_z	q_h	q_h	p	p	p
Unit	ft	ft	ft	ft^2	-	-	-	-	-	psf	psf	psf	psf	psf	ps
Parapet-Assume 4'	303	284.25	2	195.5	1.3319	1.33	0.8	-0.5	-0.7	38.33027	38.175	38.18	26.06458	-16.22453	-22.
22	299	280.25	12.75	1246.3125	1.3266	1.33	0.8	-0.5	-0.7	38.17537	38.175	38.18	25.95925	-16.22453	-22.
21	281.5	262.75	15.125	1478.4688	1.3024	1.33	0.8	-0.5	-0.7	37.47852	38.175	38.18	25.4854	-16.22453	-22.
20	268.75	250	12.75	1246.3125	1.284	1.33	0.8	-0.5	-0.7	36.94965	38.175	38.18	25.12576	-16.22453	-22.
19	256	237.25	12.5	1221.875	1.2649	1.33	0.8	-0.5	-0.7	36.40113	38.175	38.18	24.75277	-16.22453	-22.
18	243.75	225	12.75	1246.3125	1.2459	1.33	0.8	-0.5	-0.7	35.85393	38.175	38.18	24.38067	-16.22453	-22.
17	230.5	211.75	12.5	1221.875	1.2245	1.33	0.8	-0.5	-0.7	35.23754	38.175	38.18	23.96152	-16.22453	-22.
16	218.75	200	11.75	1148.5625	1.2047	1.33	0.8	-0.5	-0.7	34.66743	38.175	38.18	23.57386	-16.22453	-22.
15	207	188.25	11.75	1148.5625	1.184	1.33	0.8	-0.5	-0.7	34.07288	38.175	38.18	23.16956	-16.22453	-22.
14	195.25	176.5	11.75	1148.5625	1.1624	1.33	0.8	-0.5	-0.7	33.45119	38.175	38.18	22.74681	-16.22453	-22.
13	183.5	164.75	11.75	1148.5625	1.1397	1.33	0.8	-0.5	-0.7	32.7992	38.175	38.18	22.30346	-16.22453	-22.
12	171.75	153	11.75	1148.5625	1.1159	1.33	0.8	-0.5	-0.7	32.11309	38.175	38.18	21.8369	-16.22453	-22.
11	160	141.25	11.75	1148.5625	1.0907	1.33	0.8	-0.5	-0.7	31.38824	38.175	38.18	21.344	-16.22453	-22.
10	148.25	129.5	11.75	1148.5625	1.064	1.33	0.8	-0.5	-0.7	30.61894	38.175	38.18	20.82088	-16.22453	-22.
9	136.5	117.75	11.75	1148.5625	1.0355	1.33	0.8	-0.5	-0.7	29.79804	38.175	38.18	20.26267	-16.22453	-22.
8	124.75	106	11.75	1148.5625	1.0048	1.33	0.8	-0.5	-0.7	28.91634	38.175	38.18	19.66311	-16.22453	-22.
7	113	94.25	11.75	1148.5625	0.9717	1.33	0.8	-0.5	-0.7	27.96179	38.175	38.18	19.01402	-16.22453	-22.
6	101.25	82.5	11.75	1148.5625	0.9354	1.33	0.8	-0.5	-0.7	26.918	38.175	38.18	18.30424	-16.22453	-22.
5	89.5	70.75	11.75	1148.5625	0.8952	1.33	0.8	-0.5	-0.7	25.7619	38.175	38.18	17.51809	-16.22453	-22.
4	77.75	59	11.75	1148.5625	0.8499	1.33	0.8	-0.5	-0.7	24.4592	38.175	38.18	16.63226	-16.22453	-22.
3	66	47.25	14.625	1429.5938	0.7977	1.33	0.8	-0.5	-0.7	22.95541	38.175	38.18	15.60968	-16.22453	-22.
2	48.5	29.75	15.5	1515.125	0.6989	1.33	0.8	-0.5	-0.7	20.11318	38.175	38.18	13.67697	-16.22453	-22.
Ground	35	16.25	14.875	1454.0313	0.588	1.33	0.8	-0.5	-0.7	16.92165	38.175	38.18	11.50672	-16.22453	-22.
	33.75	15	8.125	794.21875	0.5747	1.33	0.8	-0.5	0.3	16.53905	38.175	38.18	11.24656	-16.22453	9.7
Concourse	18.75	0	7.5	733.125	0.5747	1.33	0.8	-0.5	-0.7	16.53905	38.175	38.18	11.24656	-16.22453	-22.

East West Wind Loadings
kz = 2.01(height above ground/1,200)^(2/7)
Cp = .8, -0.5, -.7 (from Fig 27.4-1 - 27.4-7) L/B = 0.599
qz = .00256*kz*kzt*kd*V^2
P = (qz-qh)*Story Trib Area

Table 8. North South Wind Loadings by Elevation

Floor	Elevation	Height Above Ground	Story Load Trib Length	Story Load Trib Area			External Pressure Coefficients			Velocity Pressures			
				w=			Windward	Leeward	Side	Windward	Leeward	Side	Wi
				58.609375	k_z	k_h	c_p	c_p	c_p	q_z	q_h	q_h	
Unit	ft	ft	ft	ft^2	-	-	-	-	-	psf	psf	psf	
Parapet-Assume 4'	303	284.25	2	117.21875	1.332	1.3266	0.8	-0.3667	-0.7	38.330266	38.1754	38.18	26.(
22	299	280.25	10.75	630.0507813	1.327	1.3266	0.8	-0.3667	-0.7	38.175375	38.1754	38.18	25.9
21	281.5	262.75	15.125	886.4667969	1.302	1.3266	0.8	-0.3667	-0.7	37.478524	38.1754	38.18	25.4
20	268.75	250	12.75	747.2695313	1.284	1.3266	0.8	-0.3667	-0.7	36.949645	38.1754	38.18	25.1
19	256	237.25	12.5	732.6171875	1.265	1.3266	0.8	-0.3667	-0.7	36.401134	38.1754	38.18	24.7
18	243.75	225	12.75	747.2695313	1.246	1.3266	0.8	-0.3667	-0.7	35.853925	38.1754	38.18	24.3
17	230.5	211.75	12.5	732.6171875	1.224	1.3266	0.8	-0.3667	-0.7	35.237536	38.1754	38.18	23.9
16	218.75	200	11.75	688.6601563	1.205	1.3266	0.8	-0.3667	-0.7	34.667434	38.1754	38.18	23.5
15	207	188.25	11.75	688.6601563	1.184	1.3266	0.8	-0.3667	-0.7	34.07288	38.1754	38.18	23.1
14	195.25	176.5	11.75	688.6601563	1.162	1.3266	0.8	-0.3667	-0.7	33.451194	38.1754	38.18	22.7
13	183.5	164.75	11.75	688.6601563	1.14	1.3266	0.8	-0.3667	-0.7	32.7992	38.1754	38.18	22.3
12	171.75	153	11.75	688.6601563	1.116	1.3266	0.8	-0.3667	-0.7	32.113091	38.1754	38.18	21.8
11	160	141.25	11.75	688.6601563	1.091	1.3266	0.8	-0.3667	-0.7	31.388241	38.1754	38.18	21.3
10	148.25	129.5	11.75	688.6601563	1.064	1.3266	0.8	-0.3667	-0.7	30.618944	38.1754	38.18	20.8
9	136.5	117.75	11.75	688.6601563	1.035	1.3266	0.8	-0.3667	-0.7	29.798039	38.1754	38.18	20.2
8	124.75	106	11.75	688.6601563	1.005	1.3266	0.8	-0.3667	-0.7	28.916344	38.1754	38.18	19.6
7	113	94.25	11.75	688.6601563	0.972	1.3266	0.8	-0.3667	-0.7	27.961789	38.1754	38.18	19.0
6	101.25	82.5	11.75	688.6601563	0.935	1.3266	0.8	-0.3667	-0.7	26.918003	38.1754	38.18	18.3
5	89.5	70.75	11.75	688.6601563	0.895	1.3266	0.8	-0.3667	-0.7	25.761896	38.1754	38.18	17.
4	77.75	59	11.75	688.6601563	0.85	1.3266	0.8	-0.3667	-0.7	24.459202	38.1754	38.18	16.6
3	66	47.25	14.625	857.1621094	0.798	1.3266	0.8	-0.3667	-0.7	22.955411	38.1754	38.18	15.
2	48.5	29.75	15.5	908.4453125	0.699	1.3266	0.8	-0.3667	-0.7	20.113184	38.1754	38.18	13.6
Ground	35	16.25	7.375	432.2441406	0.588	1.3266	0.8	-0.3667	-0.7	16.921648	38.1754	38.18	11.5
-	33.75	15	8.125	476.2011719	0.575	1.3266	0.8	-0.3667	0.3	16.539053	38.1754	38.18	11.2
Concourse	18.75	0	8.125	476.2011719	0.575	1.3266	0.8	-0.3667	-0.7	16.539053	38.1754	38.18	11.2

North South Wind Loadings
kz = 2.01(height above ground/1,200)^(2/7)
Cp = .8, -0.3667 , -.7 (from Fig 27.4-1 - 27.4-7) L/B = 1.66 -> use this value for linea
qz = .00256*kz*kzt*kd*V^2
P = (qz-qh)*Story Trib Area

Figure 8. Windward Design Pressures (psf); West Wall

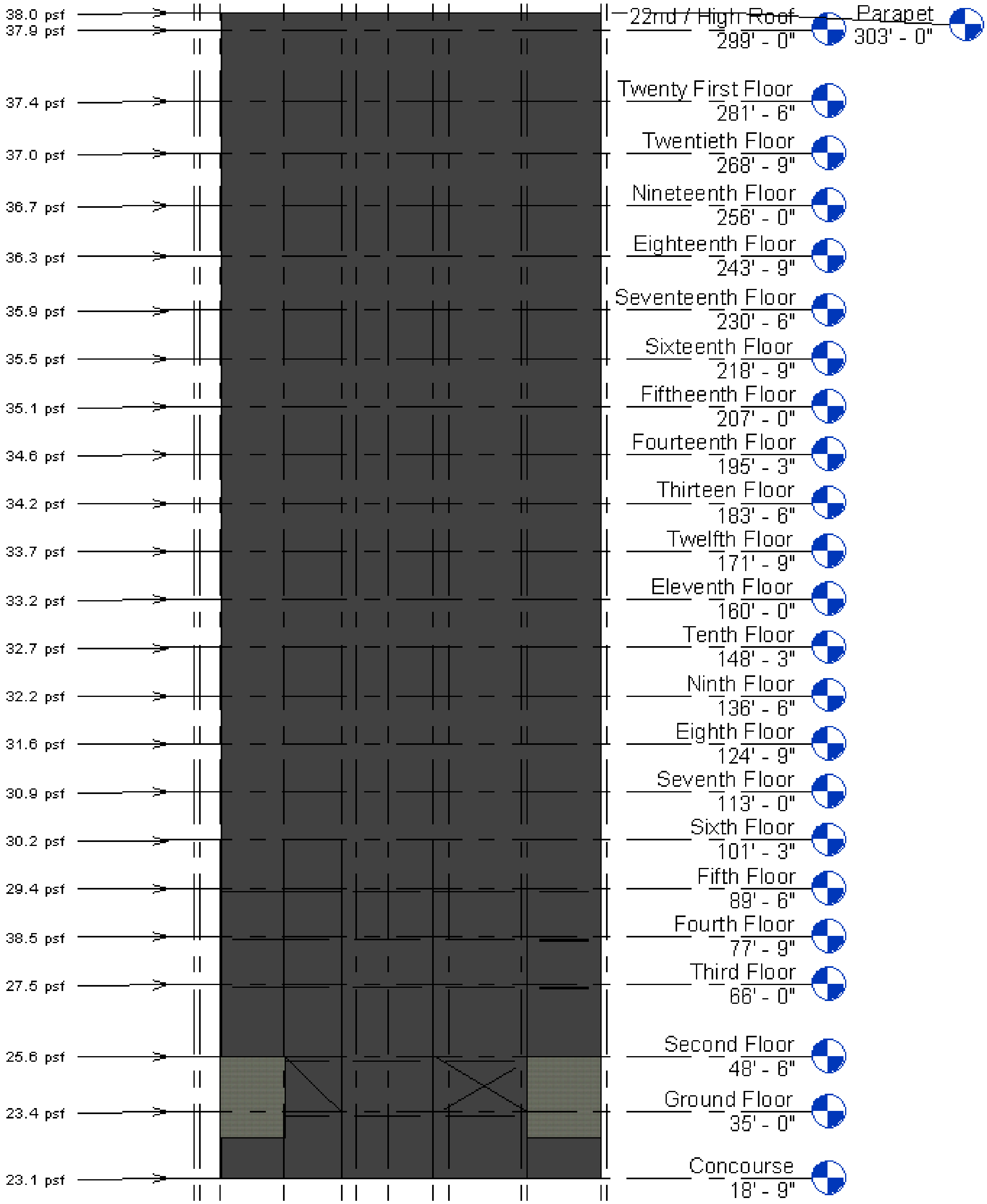
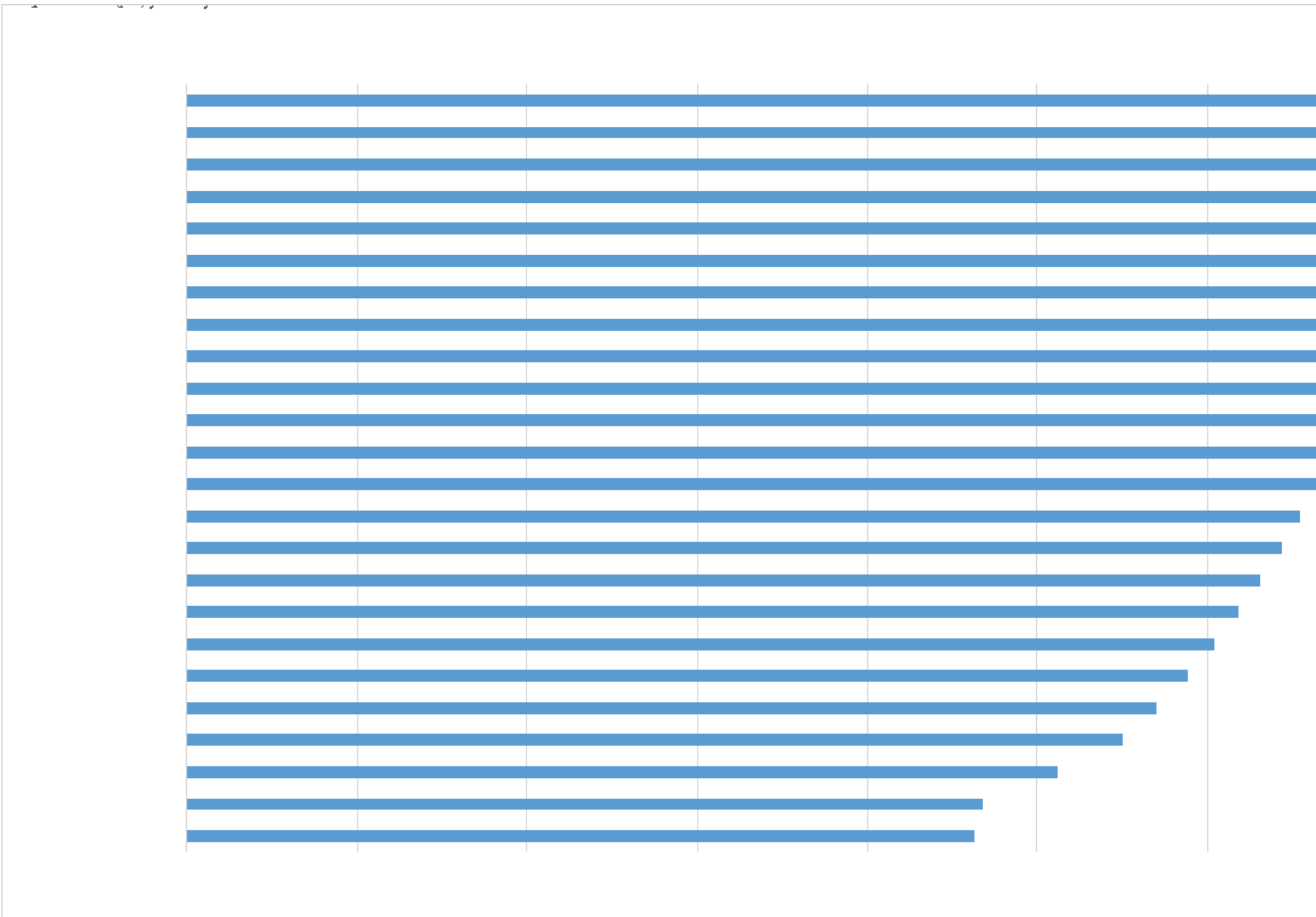


Figure 9. Wind Pressure Loadings by Story (psf)



Seismic Design

In order to design for the shear walls, both the wind loads and the earthquake loads needed to be taken into account. After some preliminary design for the lateral system due to the wind loads, it was discovered that the moment capacity at the base of the wall is what governs. Both the shear and deflection limits were satisfied with our first wall design. However, now the earthquake loads must be taken into account, and the larger of either the wind overturning moment or the earthquake overturning moment needs to be designed for.

The following tables illustrate how the calculations for earthquakes were carried out. All values and constants have been taken from ASCE 7-10 as well as the USGS website.

The following table shows the weight of each floor of the as well as the total floor of the structure. The fifth floor was taken as the sample section because it has the largest total dead loads, every other floor has taken the same weight thus making our design conservative. Calculations are attached after this section.

Typica lFloor Weight(kips)	628.2
Total Building-W (kips)	14,616

Table 9. Floor and Total Weight of Structure

Next, with the help of ASCE 7-10 and the USGS website, the typical risk-adjusted MCE_R values have been found.

Ss (g)	S1 (g)	T_L (s)	Site Class	Imp. Factor (II)	Sms	Sm1	Sds	Sd1	R
0.132	0.061	12	D	1	0.211	0.147	0.141	0.098	4

Table 10. Geographic Values

Next the values for Cs have been calculated using the values of S_{D1}, T_a, R, and importance factors Keeping in mind that Cs cannot be less than both $0.044S_{DS}I_e$ or 0.01. In our case, Cs has been calculated as 0.0177.

Cs (min)	Cs (desigr
0.01	0.017695

Table 11. Seismic Response Coefficient

Following this, the approximate fundamental period of the structure T_a is discovered to be 1.38 seconds. The base seismic shear and the value for k can be found on the following table. K was taken as 1.44 by interpolation between 0.5 sec, and 2.5 sec.

Ta (sec)	hn	k	Ct	x	Seismic Base Shear (V, kips)
1.38	284.25	1.44	0.02	0.75	258.64

Table 12. Fundamental Period and Seismic Base Sheer

Finally, the seismic forces can be calculated knowing the total base shear of the system. The following tables break down the forces per floor. Calculations for all these values, as well as a sample calculation for the 15th floor have been attached following the tables attached.

Floor Level	Story height (ft)	Floor to Floor Height	Story Weight (kips)	wi * hi^k	Cvx	Fx (kips)
Parapet-Assume 4'	284.25	4	167	569,541	0.028	7.15
22	280.25	17.5	628	2,101,816	0.102	26.40
21	262.75	12.75	628	1,915,448	0.093	24.06
20	250	12.75	628	1,783,046	0.087	22.39
19	237.25	12.25	628	1,653,583	0.080	20.77
18	225	13.25	628	1,532,046	0.074	19.24
17	211.75	11.75	628	1,403,830	0.068	17.63
16	200	11.75	628	1,293,040	0.063	16.24
15	188.25	11.75	628	1,185,079	0.058	14.88
14	176.5	11.75	628	1,080,044	0.052	13.56
13	164.75	11.75	628	978,042	0.047	12.28
12	153	11.75	628	879,194	0.043	11.04
11	141.25	11.75	628	783,632	0.038	9.84
10	129.5	11.75	628	691,508	0.034	8.69
9	117.75	11.75	628	602,994	0.029	7.57
8	106	11.75	628	518,286	0.025	6.51
7	94.25	11.75	628	437,617	0.021	5.50
6	82.5	11.75	628	361,262	0.018	4.54
5	70.75	11.75	628	289,558	0.014	3.64
4	59	11.75	628	222,923	0.011	2.80
3	47.25	17.5	628	161,908	0.008	2.03
2	29.75	13.5	628	83,167	0.004	1.04
Ground	16.25	16.25	628	34,814	0.002	0.44
Concourse	15	0	628	31,024	0.002	0.39
	Sum		14,616	20,593,402	1	258.64

Table 13. Earthquake Load Calculations (1)

Floor Level	Story height (ft)	Floor to Floor Height (ft)	Cumulative Shear (kips)	Story O.T. (k-ft)	Cummulative O.T. (k-ft)
Parapet- Assume 4'	284.25	4	7.15	0	-
22	280.25	17.5	33.55	28.61	28.61
21	262.75	12.75	57.61	587.14	615.76
20	250	12.75	80.00	734.51	1,350.26
19	237.25	12.25	100.77	1,020.03	2,370.29
18	225	13.25	120.01	1,234.44	3,604.74
17	211.75	11.75	137.64	1,590.17	5,194.90
16	200	11.75	153.88	1,617.32	6,812.22
15	188.25	11.75	168.77	1,808.14	8,620.36
14	176.5	11.75	182.33	1,983.03	10,603.39
13	164.75	11.75	194.62	2,142.41	12,745.80
12	153	11.75	205.66	2,286.75	15,032.55
11	141.25	11.75	215.50	2,416.50	17,449.05
10	129.5	11.75	224.19	2,532.14	19,981.19
9	117.75	11.75	231.76	2,634.19	22,615.38
8	106	11.75	238.27	2,723.18	25,338.55
7	94.25	11.75	243.77	2,799.66	28,138.21
6	82.5	11.75	248.30	2,864.24	31,002.46
5	70.75	11.75	251.94	2,917.56	33,920.01
4	59	11.75	254.74	2,960.29	36,880.30
3	47.25	17.5	256.77	2,993.19	39,873.48
2	29.75	13.5	257.82	4,493.52	44,367.01
Ground	16.25	16.25	258.25	3,480.53	47,847.54
Concourse	15	0	258.64	4,196.64	52,044.17

Table 14. Earthquake Load Calculations (2)

Recall that the governing factor in the design of the shear walls is the overturning moment. With that we can compare between the wind load overturning moment and the earthquake overturning moment. The wind load had 82,012 k-ft at the base of the structure, which is significantly larger than the 52,044 k-ft that a design for earthquake overturning calls for. Therefore, wind loading governs and the design for the shear walls will account for this.

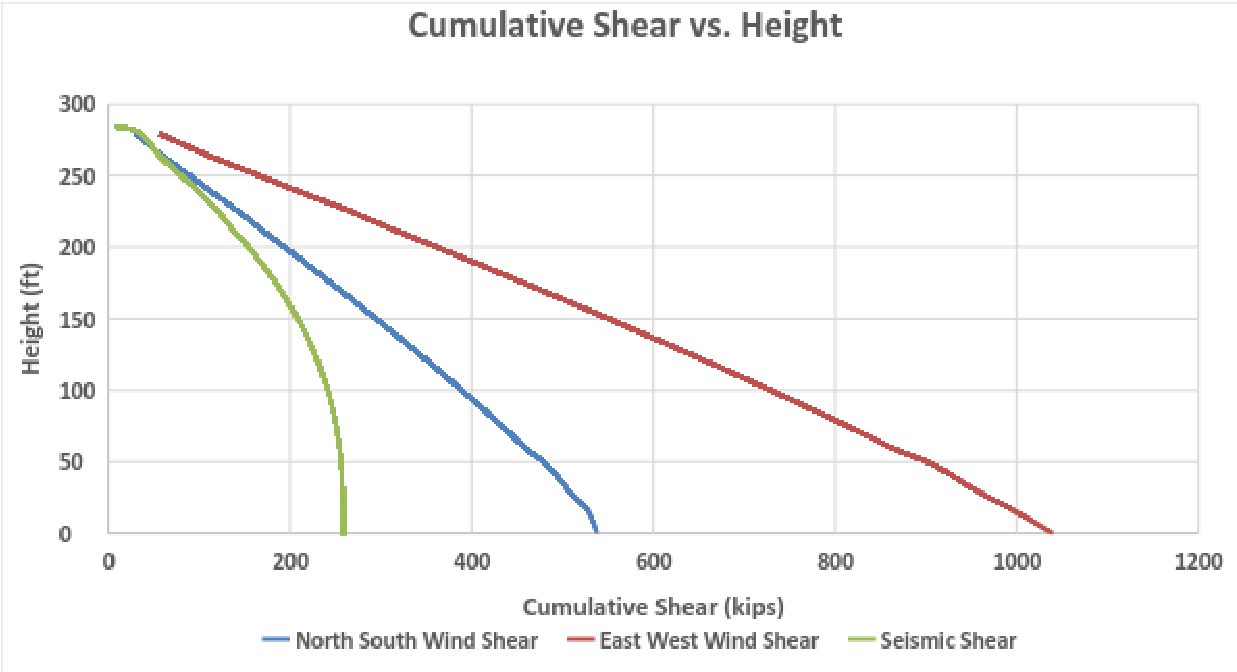


Figure 10. Cumulative Shear vs. Height

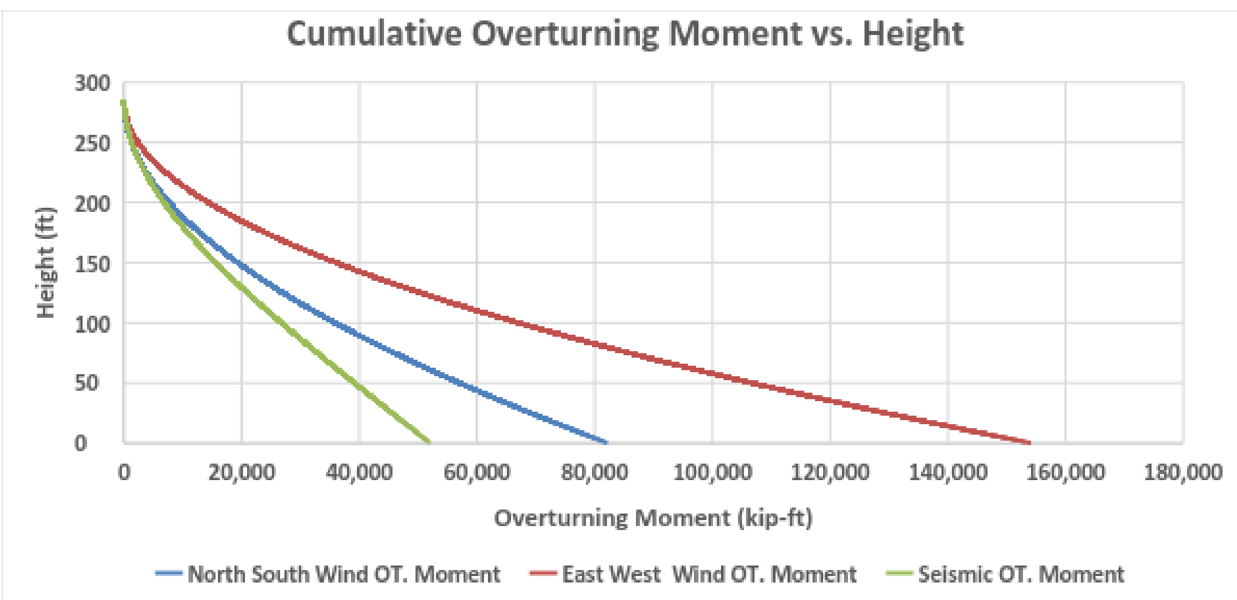


Figure 11. Cumulative Overturning Moment vs. Height

Lateral Load Resisting System

After receiving further instruction during lecture regarding the design for wind loadings, our group has proceeded in the design of structural shear wall. After a preliminary check of our first proposed system where we would have four shear walls with a wall width over 60 feet with in the A.2 – B and E.2 – F gridlines, our team noticed that this system would provide too much shear strength.

Our team decided to check the deflection of the total system to see if that would govern. Again, with over 60 feet of wall width the total deflection was way below the limit state of h/500. However, upon further analysis, we discovered that what ends up governing the design of the shear wall is in fact the overturning moment capacity of the wall.

The new lateral system specifications are as follows: They are still within the same grid lines as before, except the two walls between A.2 – B and the two walls between gridlines E.2 – F have been increased in length from 18 feet each, to now 20 feet each from the concourse to the 6th floor. Because the full strength of a twenty-foot-long wall is not required, the design called for a reduction in the length of the wall. This is why between floors 7 to 14, the wall length is reduced from 20 feet to 15feet, for all four walls. The final sections spanning from the 15th floor to the 22nd floor has also been reduced by 5feet, for a length of 10 feet.

This change in length of the walls effectively changes the moment of inertia of the wall, therefore the drift calculations need to take into account the variability of the length with respect to the height of the building. Our drift calculations for the shear walls have been attached.

Lastly, because our project is an addition to an already existing building. The existing buildings acts as lateral supports in the east - west direction. Thus, we did not have to design for the east - west lateral system. We have included the wind pressures and forces on our attached wind design, and although the wind is acting on a larger surface area in the east west layout, we were not expected to design for this wind load.

Torsional wind load effects

Since the shear walls are symmetric, it is assumed that the torsional effects of the wind loads are negligible. However, a quick check was performed to make sure that the used wind load case governed, and not the torsion. This was performed by using the following equation:

$$Overturning\ Moment\ =\ .75(Wind\ Pressure)\ *\ Width\ *\ Height\ +.75(Wind\ Pressure)\ *\ Wight\ *\ H$$

The resulting cumulative overturning moment at the base of the building due to torsion is 56,000 k-ft, which is smaller than 82,000 k-ft. This means that our symmetric assumption was correct and that torsion is not the governing load case. See the table below for calculations.

Table 15. Torsional Wind Load Effects

Case 1				Case 2			
Total Windload	Cumm. Shear	Story Overturn	Cumm. Overturn	75% Windload	75% Cumm Shear	Story Overturn	Cumm Overturn
kips	*kips*	*k-ft*	*k-ft*	*kips*	*kips*	*k-ft*	*k-ft*
23.85226	28.302	-8.9	-8.9	17.8892	21.227	2680.71	2680.71
33.13952	61.442	495.289	486.389	24.8546	46.081	3711.21	6391.92
27.66704	89.109	783.382	1269.77	20.7503	66.832	3075.68	9467.6
26.8513	115.96	1136.14	2405.91	20.1385	86.97	2967.31	12434.9
27.11026	143.07	1420.51	3826.42	20.3327	107.3	2980.33	15415.2
26.27162	169.34	1895.68	5722.1	19.7037	127.01	2869.71	18284.9
24.42835	193.77	1989.77	7711.87	18.3213	145.33	2646.58	20931.5
24.14992	217.92	2276.8	9988.67	18.1124	163.44	2600.6	23532.1
23.85879	241.78	2560.56	12549.2	17.8941	181.33	2553.36	26085.5
23.55347	265.33	2840.9	15390.1	17.6651	199	2504.7	28590.2
23.23218	288.56	3117.66	18507.8	17.4241	216.42	2454.42	31044.6
22.89274	311.46	3390.63	21898.4	17.1696	233.59	2402.29	33446.9
22.53248	333.99	3659.62	25558.1	16.8994	250.49	2348.03	35794.9

22.14806	356.14	3924.38	29482.4	16.611	267.1	2291.26	38086.2
21.73517	377.87	4184.62	33667.1	16.3014	283.4	2231.52	40317.7
21.28817	399.16	4440.01	38107.1	15.9661	299.37	2168.19	42485.9
20.79937	419.96	4690.15	42797.2	15.5995	314.97	2100.43	44586.4
20.25798	440.22	4934.54	47731.7	15.1935	330.16	2027.05	46613.4
19.64794	459.87	5172.57	52904.3	14.736	344.9	1946.26	48559.7
23.5789	483.45	5403.43	58307.8	17.6842	362.58	2376.71	50936.4
23.23384	506.68	8460.3	66768	17.4254	380.01	2323.74	53260.1
10.11674	516.8	6840.17	73608.2	7.58755	387.6	834.125	54094.2
11.02167	527.82	645.995	74254.2	8.26625	395.86	929.86	55024.1
11.02167	527.82	8397.94	82006.2	8.26625	404.13	923.247	55947.3

Shear Wall Reinforcement Designs

We designed the shear wall at 3 sections in order to reduce the cost and prevent over designing. We came up with the M_u and P_u values and based on the greatest of those, which are the floors at the bottom, we came up with the design of 4 20 ft walls. Once we obtained this value, we decided to alter the length of walls at higher level floors since it would be overdesigned given the lower M_u and P_u values at those floors. We decided that it will be economical to use 3 different sections for every 8 floors. From concourse to 6th floor we used 20 ft, from 7th to 14th we used 15 ft, and from 15th to 22nd we used 10 ft wall in our design. So this means just like columns, we are going to use splicing in our shear walls. The process of deciding on the length was based on the P-M diagram.

For the rebar, we used 4 inch spacing throughout the walls. At the 20 and 15 ft sections, #9 bars are used and at the top 10 ft section #7 bars are used in the design. Concrete cover is 2 inches throughout the walls so based on that at the 20 ft section, total of 120 bars are used, 60 bars per row. At the 15 ft section, 90 bars are used in the design again with two rows so this means 45 bars per row. And finally at the 10 ft section, we used 60 bars, 30 bars per row. Also we used F_y=60 ksi rebar for our shear walls.

$$\varphi P_{bn} = \varphi * 0.42 f'_c \ell_w h$$

$$\varphi M_{bn} = \varphi * P_{bn} * 0.32 \ell_w + \varphi [0.6 A_{se} + 0.15 A_{ss}] f_y (\ell_w / 2 - d')$$

$$A_{st} = A_{se} + A_{ss}$$

$$\varphi = 0.65$$

[57] This is the equation we used.

Table 16 Foot Shear Wall Properties

phi	coeff	f'c(ksi)	lw(in)	h(in)	phi Pbn(kips)
0.65	0.42	4	240	12	3144.96

Table 17. Capacity of the 20' Wall

phi Pbn (k)	0.32*lw	phi As (in^2)	fy(ksi)	(lw/2)-d (in)	phi Mbn(kft)
3144.96	76.8	12.87	60	118	27721.044

Table18. 15' Shear Wall Properties

phi	coeff	f'c(ksi)	lw(in)	h(in)	phi Pbn(kips)
0.65	0.42	4	180	12	2358.72

Table 19. Capacity of the 15' Wall

phi Pbn (k)	0.32*lw	phi As (in^2)	fy(ksi)	(lw/2)-d (in)	phi Mbn(kft)
2358.72	57.6	9.945	60	88	15697.656

Table 20. 10' Shear Wall Properties

phi	coeff	f'c(ksi)	lw(in)	h(in)	phi Pbn(kips)
0.65	0.42	4	120	12	1572.48

Table 21 Capacity of the 10' Wall

phi Pbn (k)	0.32*lw	phi As (in^2)	fy(ksi)	(lw/2)-d (in)	phi Mbn(kft)
1572.48	38.4	4.212	60	58	6253.416

*All Values above correspond to one wall

And here is a sample calculation for the Area of Steel that is in the formula. This is the one used for the bottom 8 floors. 2 rows of bars are used so there are a total of 120 #9 bars.

0.6Ase	0.15Ass	phi As (in^2)
2.4	17.4	12.87

Stories	h(in)	d(in)	f'c^0.5(ksi	V_c(kips)
22-19	12	96	0.063246	145.718
18-15	12	96	0.063246	145.718
14-11	12	144	0.063246	218.577
10-7	12	144	0.063246	218.577
6-3	12	192	0.063246	291.436
2-concours	12	192	0.063246	291.436

The table to the top displays the V_c values obtained for every 4 floors. It is calculated based on the formula 2*f'c^0.5*h*d. h is

Table 22. Shear Strength of 4ksi Concrete

Stories	A_v(in^:	Fy(ksi)	d(in)	s(in)	V_s
22-19	0.4418	60	96	14	181.8
18-15	0.4418	60	96	14	181.8
14-11	0.4418	60	144	14	272.6
10-7	0.4418	60	144	14	272.6
6-3	0.4418	60	192	14	363.5
2-concours	0.4418	60	192	14	363.5

The table shows the V_s values obtained for every 4 floors. Same amount of horizontal reinforcement is used since it is very close to the minimum amount with #6 bars. D decreases because as we go up, our group decided to decrease the length of the shear wall.

Table 23. Shear Strength of Steel

Stories	V_u(kips	phi V_c(kip	phi V_s(kips	phi V_n(kips
22-19	29.0	145.71775	136.322681	282.040436
18-15	54.5	145.71775	136.322681	282.040436
14-11	77.9	218.57663	204.484022	423.060654
10-7	99.8	218.57663	204.484022	423.060654
6-3	120.9	291.43551	272.645362	564.080872
2-concours	132.0	291.43551	272.645362	564.080872

The table to the top is showing the V_u and V_n values obtained for every 4 floors. Phi V_n values for every floor is greater than the V_u at that floor.

Table 24. Nominal Strength vs. Design Criteria

Lateral Displacements

Because of the existence of lateral loads, the structure will undergo lateral displacements. The shear wall system that has been implemented as part of the lateral load resisting system provides a major contribution to controlling the deflections of the building. The lateral loads were calculated based off of a 700 year wind, since this is a serviceability check we used a 50 year wind load reduction, which was 70% of our story wind loads. Based on the recalculated lateral loads and on the selected shear wall design, the structure experiences a maximum drift of 0.306 inches in the North-South direction. This is well under the building's deflection limit of 6.73 inches. Additionally, the story drift values were compared to the story drift limits, which were mostly in the range of 0.28 - 0.42 inches. The story drift limits were also satisfied. The calculated drift values are extremely low compared to their corresponding limits because of the minimum design restrictions enforced on the shear walls. In order to compensate for the overturning moments, the length of the shear wall had to be increased, resulting in a greater cross-sectional area and a greater moment of inertia. The large cross-sectional area limited the shear component of the story drift while the large moment of inertia reduced the flexural component of the story drift.

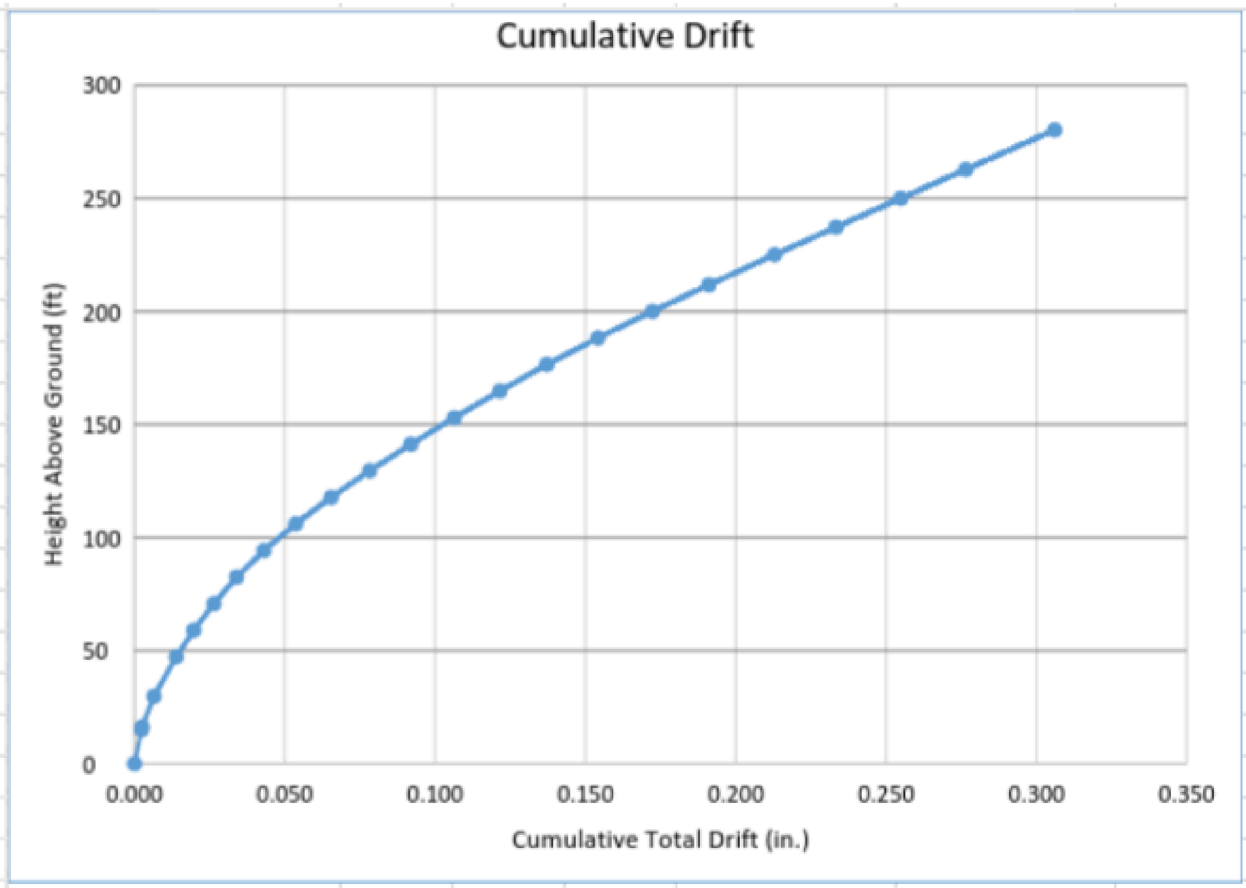


Figure 12. Cumulative Total Drift Above Ground

Floor	Moment of Inertia	Cum. Mid overturning moment	M/EI	Change in slope	Total Slope	Shear Wall Length	Flexural Story drift	Shear Drift	Total Drift/Story	Story Drift Limit	Cum. Total Drift
	I(i)								h/500		
Unit	in^4	k-in	k/in			in.	in	in.	in.	in.	
22	2.48E+09	0	0	0	1.38E-04	10	2.89E-02	0.0006	0.029497	0.42	**0.3056**
21	2.48E+09	2079	1.86E-09	2.85E-07	1.38E-04	10	2.11E-02	0.0005	0.0215628	0.306	0.2761
20	2.48E+09	7448	6.67E-09	1.02E-06	1.37E-04	10	2.10E-02	0.0004	0.0214378	0.306	0.2545
19	2.48E+09	15509	1.39E-08	2.04E-06	1.36E-04	10	2.01E-02	0.0004	0.0204354	0.294	0.2331
18	2.48E+09	26245	2.35E-08	3.74E-06	1.34E-04	10	2.14E-02	0.0004	0.0217829	0.318	0.2126
17	2.48E+09	40172	3.60E-08	5.07E-06	1.31E-04	10	1.84E-02	0.0004	0.0187783	0.282	0.1909
16	2.48E+09	56490	5.06E-08	7.14E-06	1.26E-04	10	1.77E-02	0.0003	0.0180376	0.282	0.1721
15	2.48E+09	74408	6.67E-08	9.40E-06	1.18E-04	10	1.67E-02	0.0003	0.0170277	0.282	0.154
14	2.48E+09	94724	2.51E-08	3.55E-06	1.09E-04	15	1.54E-02	0.0002	0.0155895	0.282	0.137
13	2.48E+09	117409	3.12E-08	4.39E-06	1.06E-04	15	1.49E-02	0.0002	0.0150869	0.282	0.1214
12	2.48E+09	142433	3.78E-08	5.33E-06	1.01E-04	15	1.43E-02	0.0002	0.0144644	0.282	0.1063
11	2.48E+09	169767	4.51E-08	6.35E-06	9.58E-05	15	1.35E-02	0.0002	0.0137097	0.282	0.0919
10	2.48E+09	199377	5.29E-08	7.46E-06	8.94E-05	15	1.26E-02	0.0002	0.0128106	0.282	0.0782
9	2.48E+09	231229	6.14E-08	8.65E-06	8.19E-05	15	1.16E-02	0.0002	0.011755	0.282	0.0654
8	2.48E+09	265285	7.04E-08	9.93E-06	7.33E-05	15	1.03E-02	0.0002	0.010531	0.282	0.0536
7	2.48E+09	301507	8.00E-08	1.13E-05	6.34E-05	15	8.93E-03	0.0002	0.0091271	0.282	0.0431
6	2.48E+09	339853	3.81E-08	5.37E-06	5.21E-05	20	7.34E-03	0.0001	0.0074841	0.282	0.0339
5	2.48E+09	380275	4.26E-08	6.00E-06	4.67E-05	20	6.59E-03	0.0001	0.0067238	0.282	0.0265
4	2.48E+09	422724	4.73E-08	6.67E-06	4.07E-05	20	5.74E-03	0.0001	0.005873	0.282	0.0197
3	2.48E+09	467142	5.23E-08	1.10E-05	3.40E-05	20	7.15E-03	0.0002	0.0073856	0.42	0.0139
2	2.48E+09	525368	5.88E-08	9.53E-06	2.30E-05	20	3.73E-03	0.0002	0.0039152	0.324	0.0065
Ground	2.48E+09	589628	6.60E-08	9.90E-07	1.35E-05	20	2.03E-04	7E-06	0.00021	0.03	0.0026
-	2.48E+09	621070	6.95E-08	1.25E-05	1.25E-05	20	2.25E-03	1E-04	0.0023497	0.36	0.0024
Concourse	0	656340	0	0	0	20	0	0	0	0	0
						Total	0.299	0.0059		Δ limit	6.726
						Total drift (in)		**0.306**			

Table 25. Drift Calculations

Estimate of Material Quantities

With the help of Revit, a building information model, the quantities of each of the structural elements was able to be calculated. The following tables break down the elements between the framing system, columns, shear walls, and metal deck and floor slab.

Because the sixth though twenty-second floors are all identical, the schedule for the beams and girders is identical and is as follows:

<Structural Framing Schedule>

A	B	C	D	E
Reference Level	Family and Type	Count	W	Length
Sixth Floor				
Sixth Floor	W-Wide Flange: W12X16	8	16	
Sixth Floor	W-Wide Flange: W12X22	3	22	
Sixth Floor	W-Wide Flange: W14X22	11	22	
Sixth Floor	W-Wide Flange: W14X26	1	26	36' - 1 15/32"
Sixth Floor	W-Wide Flange: W14X30	4	30	
Sixth Floor	W-Wide Flange: W21X44	12	44	
Sixth Floor	W-Wide Flange: W21X48	4	48	

With this schedule finding the total weight of these floors became trivial. The following table illustrates the calculations. The total weight of the floors was done so by multiplying the values of the first table by the amount of floors with the layout, in this case 17.

Sixth Floor Framing Weight			
W	Count	Total Length (ft)	Weight (kips)
16	8	148.2	2.4
22	3	47.5	1.0
22	11	233.5	5.1
26	1	36.1	0.9
30	4	90.7	2.7
44	12	396.7	17.5
48	4	71.6	3.4
Total		1024.2	33.1

Table 26. Sixth Floor Framing Weight

6 to 22nd Total Weight			
W (lbs/ft)	Count	Total Length (ft)	Weight (kips)
16	136	2,518.8	40.3
22	51	806.8	17.7
22	187	3,968.9	87.3
26	17	613.7	16.0
30	68	1,541.9	46.3
44	204	6,743.6	296.7
48	68	1,217.2	58.4
Total		16,193.7	504.3

Table 27. Total Weight of 6^{th} to 22^{nd} Floors

The same was done for the ground, second, third, and fourth floors since these are all identical. With the exception that the final table was calculated by multiplying by four floors.

<Structural Framing Schedule>			
A	B	C	D
Reference Level	Family and Type	Count	W
Second Floor			
Second Floor	W-Wide Flange: W10X12	11	12
Second Floor	W-Wide Flange: W14X30	6	30
Second Floor	W-Wide Flange: W16X40	1	40
Second Floor	W-Wide Flange: W21X48	7	48
Second Floor	W-Wide Flange: W24X55	10	55
Second Floor	W-Wide Flange: W27X84	1	84

Second Floor Framing Weight			
W	Count	Total Length (ft)	Weight (kips)
12	11	203.4	2.4
30	6	104.3	3.1
40	1	45.7	1.8
48	7	138.1	6.6
55	10	407.6	22.4
84	1	29.5	2.5
Total	36	928.6	38.9

Table 28. Second Floor Framing Weight

Ground, 2,3,4th Floor Total Weight			
W (lbs/ft)	Count	Total Length (ft)	Weight (kips)
12	44	813.7	9.8
30	24	417.3	12.5
40	4	182.7	7.3
48	28	552.5	26.5
55	40	1,630.4	89.7
84	4	117.8	9.9
Total		3,714.3	155.7

Table 29. Total Weight of Ground to Fourth Floors

Fifth Floor Framing Weight			
W	Count	Total L(	Weight (kips)
12	3	44.2	0.5
14	2	45.3	0.6
16	16	271.0	4.3
22	1	18.0	0.4
30	4	58.9	1.8
40	2	89.8	3.6
44	6	200.0	8.8
48	3	75.3	3.6
55	5	208.8	11.5
Pratt Truss x 3			62.3
Total	42	1011.2	97.4

Table 30. Total Weight of Fifth Floor

Columns							
Location	W (lbs/ft)	Span (ft)	Weight (kips)	Base Plate Volume (in^3)	Unit weight of A992 steel (pci)	Weight of Base Plate (kips)	Total Weight (kips)
11-C.8	53	68.5	3.6	2340	0.3	0.7	118.0
	120	70.5	8.5				
	230	70.5	16.2				
10.4-C.8	99	127.3	12.6	4320		1.2	
	211	82.3	17.4				
	342	70.8	24.2				
12-C.8	33	127.3	4.2	168		0.0	
	45	82.3	3.7				
	68	70.8	4.8				
10-F	53	127.3	6.7	402.8		0.1	
	79	82.3	6.5				
	106	70.8	7.5				

Table 31. Total Weight of Designed Columns

Reinforcement Bars of One Shear Wall					
Type	Count	Weight (lbs/ft)	Length (ft)	Total Weight (lbs)	Weight of all 4 Walls (kips)
#9	210	3.4	188.25	67218	268.872
#7	60	2.044	92	11282.88	45.13152
#6	240	1.5	45	5415	21.66
Total	510	6.944	325.25	83915.88	335.66352

Table 32. Total Weight of Reinforcing Bars in Shear Wall

Total Weight of Steel (kips)	(lbs/sf)
1,211.03	15.14

Table 33. Total Weight of Steel in the LondonHouse

Concrete			
Slabs			
Floor Level	Thickness (in)	Metal Deck Thickness(in)	Volume(cy)
Roof	4.5	2	112
2 to 22	4.5	2	112
Ground	4.5	2	112
Concourse	6	0	103
Total			2,669

Table 34. Concrete Slab Quantity Estimate

Concrete			
Shear Walls			
Total Height of Walls (ft)	Length of Walls(ft)	Thickness of Walls (in)	Volume, subtracting Area Steel(cy)
284.25	20	12	599

Table 35. Concrete Shears Walls Quantity Estimate

Foundations
Total Volume (cy)
186.66

Table 36. Concrete Foundations Quantity Estimate

Total Volume of Concrete (cy)	cy/sf
3454.404753	0.04318

Table 37. Total Quantity of Concrete in the LondonHouse

Design Considerations

Columns and base plates:

- Connections are not fully designed
- Special requirements for large flange thickness are not considered

Foundations:

- Reinforcements for concrete need to be designed
- Lateral resistance of foundation need to be designed