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Structural Tech Report #3

Lateral System Analysis and Confirmation Design

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1. Executive Summary

The lateral system at Fordham Place consists of concentrically braced chevron frames. In the North – South direction, there are four bents in which contain chevron frames as opposed to three in the East-West direction. However, the frames are placed so that their center of rigidity is very close to the geometric center of the building, eliminating almost all shear caused by eccentricity. Frames in which the bracing is located range from 22’ to 28’ wide and 12’ to 15’ high. All chevron bracing members are HSS 12 x 12 x 1/5” and are considered to adequately resist all lateral loads. The assumption was made to analyze these members to resist only lateral loads, no gravity loads. This is a safe assumption because as the gravity loads try to get transferred into the bracing members, they will be redistributed into the girder.

Bracing members were spot checked to assure they did not deflect too much and were strong enough resist lateral loads. This was done by modeling each frame in SAP 2000 and applying a 1 Kip unit force on the frame to determine the stiffness of each frame. Relative stiffness’ were then found so that distribution of lateral forces could be done based on the stiffness of each frame. All load cases were considered, however since only lateral loads were being transferred into the bracing members, the controlling term was 1.6W. Once the proportion of the lateral force was determined for the selected frame to spot check, the frame was analyzed just as a simple truss. The Manual of Steel Construction, LRFD 3rd Edition, was used to determine the axial capacity of a HSS 12 x 12 x 1/5. This value was then compared to the critical force in the bracing members and was determined to be adequate. Drift of the same frame was determined using SAP2000, and was determined to be approximately 95% of the allowable drift accepted within the industry. Because the actual drift is so close to the acceptable drift, it is obvious the lateral resisting system at Fordham Place was design with the controlling factor being wind. Usually a building of 15 stories will not be controlled by drift, but due to architectural features limiting the building to only three locations where braced frames can be placed; it minimized the ability for the building to reduce drift. As an engineer, it is our job to find a viable design regardless of architectural restraints, and it was done very well with Fordham Place.

2. Building Description

2.1. General

As you look around Bronx, NY you will notice a distinct similarity between most buildings; this being they are shorter older buildings, most less than 6 stories. Once Fordham Place is erected, it will tower over the city of Bronx, rising 15 stories above ground level. As you go up the building, its size decreases as the building steps in at the 6th floor and then again at the 15th floor. The 15 story office tower will connect into an existing 5 story brick and limestone building which will have retail space up to the second floor and a sports club on the third. The office tower base will be clad in GFRC or cast stone to match the limestone base of the existing building. The tower itself will be a panelized brick veneer system to compliment the existing building. The Tower design includes modern references to the classical detailing of the existing building (such as the cornices, cast iron mullions etc.) The floor elevations of the new building will match the existing and there will be an expansion joint separating the new and old.

2.2. Load Combinations

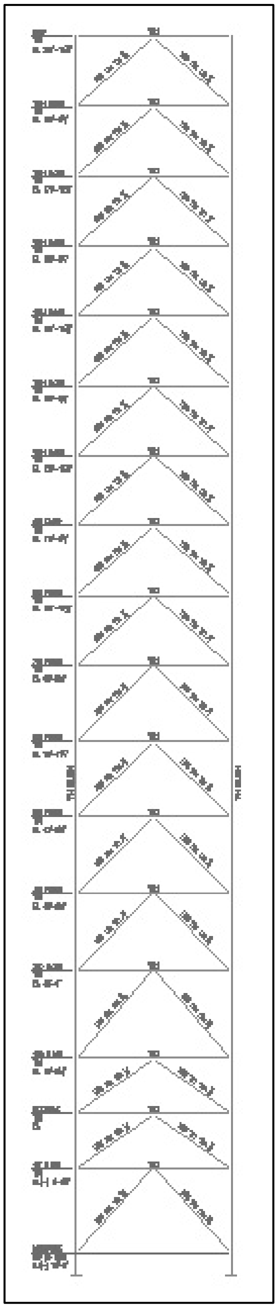
Of the two lateral loads considered in the design of Fordham Place, those being wind and seismic, wind was the controlling design. These loads were designed in accordance with the New York City Building Code. The following is a list of load cases considered based on ASCE 7-02.

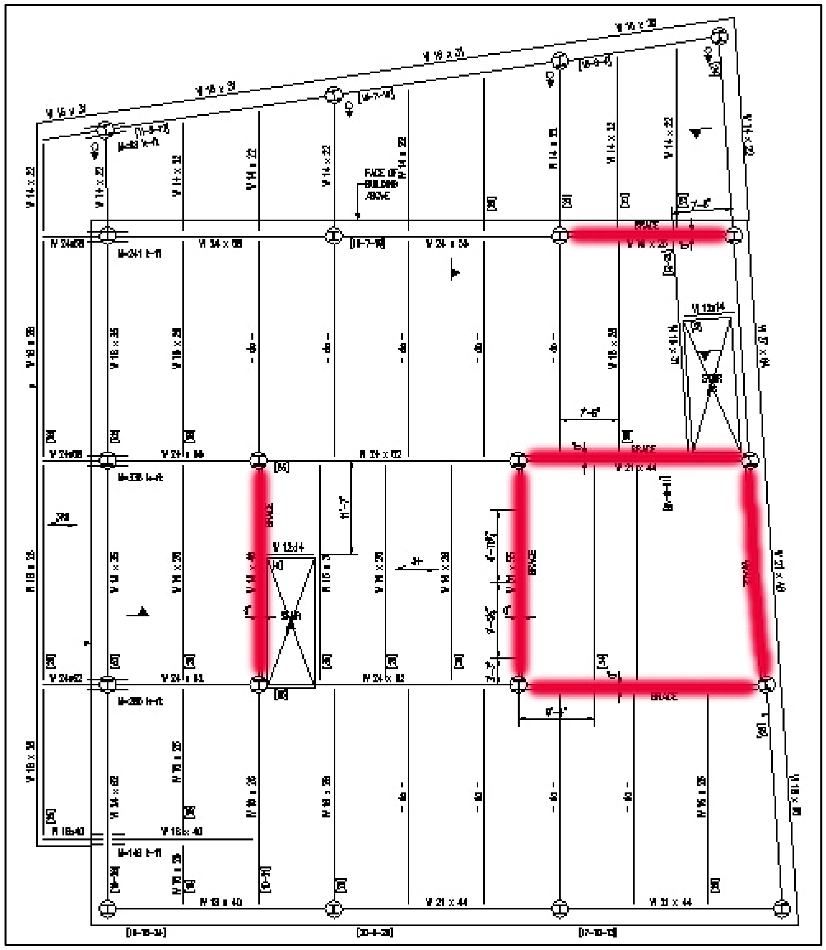
Case 1: 1.4D
Case 2: 1.2D + 1.6L + 0.5S
Case 3: 1.2D + 1.6S + 0.8W
Case 4: 1.2D + 1.6W + 0.5L + 0.5S
Case 5: 1.2D + 1.0E + 0.5L + 0.2S
Case 6: 0.9D +1.6W
Case 7: 0.9D + 1.0E

Since the lateral resisting system at Fordham Place is designed to resist lateral loads only and not gravity loads, load case #4 and #6 controlled. Load case #4 ensures that the foundations are designed for a large enough downward force, whereas load case #6 ensures that wind will not overturn the structure. (uplift on the foundations) A factor of 1.6W will be used to check lateral resisting members only. However load case #4 would be used to check gravity resisting members, including foundations.

2.3. Lateral System

The lateral force resisting system at Fordham Place is very simple and effective. In the East – West direction, it is composed of three bents that contain concentrically braced chevron frames. In the North – South direction, there are four bents in which contain chevron frames. All frames, except two, start at the concourse level, 15ft below grade, and extend up the building to the roof level. The one frame that does not extend to the roof is located on the on the exterior of the building, and terminates where the building envelope steps in at the third floor. The other frame that does not extend to the roof stops at the 13th floor. This is done because if it extended to roof height, large torsional forces would be induced into the building due to eccentric lateral loads. Terminating the frame at the 13th floor, another place where the building envelope steps in, allows for the centroid of the frames to be very close to the centroid of the building, in essence eliminating any considerable torsional effects. The chevron bracing members are primarily HSS 12 x 12 x ½, and are designed to resist lateral loads only. Locations of the braced frames are strategically placed in order to reduce the amount of torsional moment induced due to eccentric lateral loading. This is accomplished by placing the frames in such a way that the centroid of the frames stiffness's is located on the same line of action as the lateral force. This must be considered in both the North – South and East – West directions. Another consideration when designing and determining the location of the braced frames is all frames must be placed in an area compatible with all architectural features. This is the major reason moment connections, a much more expensive lateral resisting system, were considered. The current design is a very simple and effective design, because moment connections were avoided, all architectural features were considered, and erection of the bracing members is easy. The Pictures on the following page show the type of bracing and where the braced frames are located (highlighted in red).

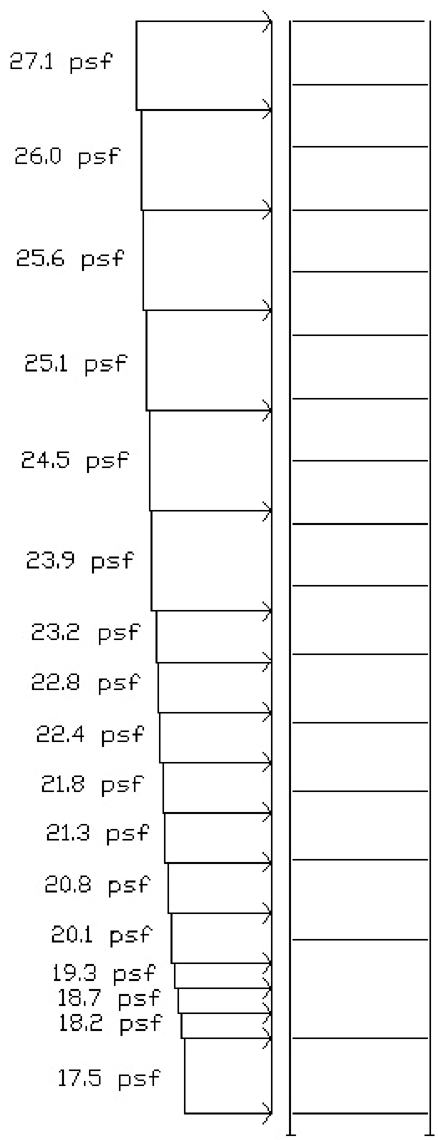




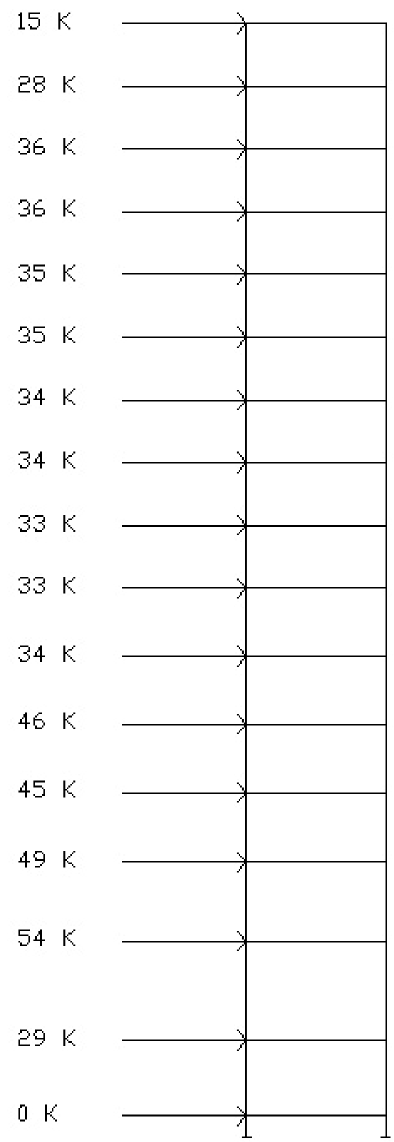
3.Lateral Design

3.1. Calculation of Lateral Loads

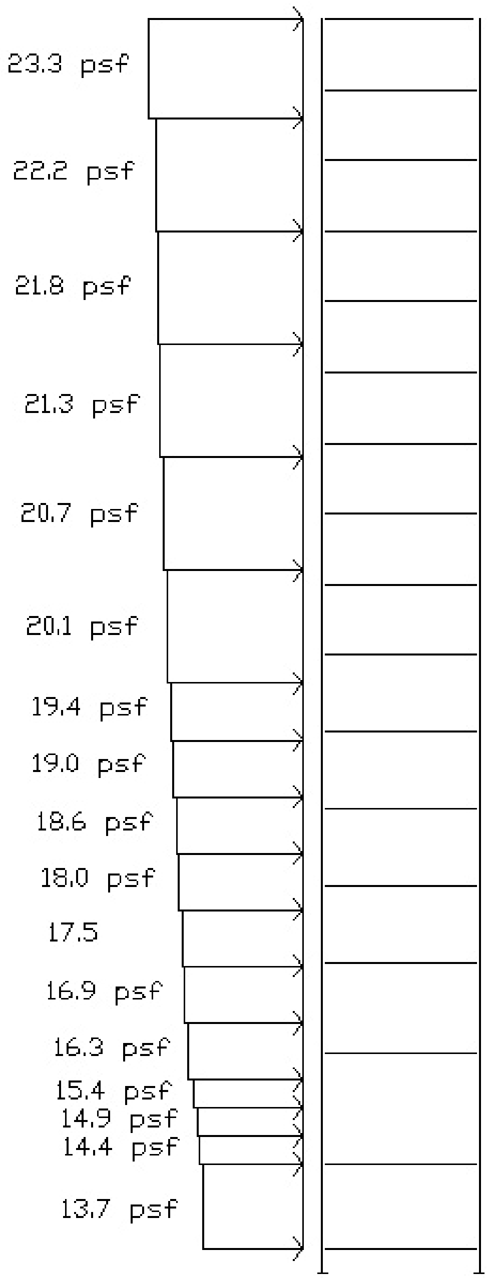
The main lateral force resisting system was designed using loads calculated by ASCE-02. Both wind and seismic loads were calculated in Technical Report #1, Structural Concepts and Structural Existing Conditions Report. As expected, wind force was the controlling design. The following is a diagram showing the uniformly distributed wind loads and wind story forces in both the North-South and East-West directions. See appendix for seismic forces and detailed wind and seismic calculations.



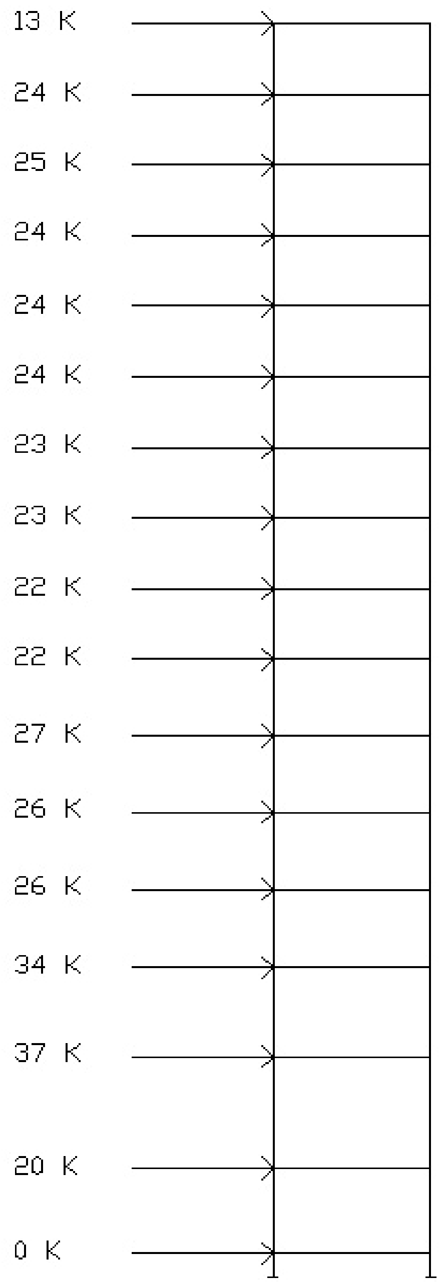
Wind Net Pressures (N-S)



Wind Story Forces (N-S)



Wind Net Pressures (E-W)



Wind Story Forces (E-W)

3.2. Stiffness Calculations

Lateral story forces are distributed throughout the braced frames based on the relative stiffness of each frame. The stiffness of each frame was determined by modeling each set of frames in SAP2000. By placing a unit 1Kip force at the top of the desired frame, and recording deflection at that same floor, stiffness's of each frame at a particular floor were able to be determined by the following equation:

$$F = k\,x \rightarrow k = F/x$$

Where,

k – Stiffness (K/in)
F – Force (Kip)
x – Deflection (in)

Stiffness' for frames at 14th and 15th Floors			
Frame	Deflection (in)	Force (K)	Stiffness (K/in)
C.1	0.0327	1	30.6
D.2	0.037	1	27.0
10	0.0394	1	25.4
12.7	0.0322	1	31.1
15	0.0331	1	30.2

Stiffness' for frames at 3rd - 15th Floors			
Frame	Deflection (in)	Force (K)	Stiffness (K/in)
C.1	0.023	1	43.5
D.2	0.0259	1	38.6
E.4	0.0474	1	21.1
10	0.0263	1	38.0
12.7	0.0223	1	44.8
15	0.0233	1	42.9

Stiffness' for frames at concourse - 2nd Floors			
Frame	Deflection (in)	Force (K)	Stiffness (K/in)
C.1	0.0022	1	454.5
D.2	0.0025	1	400.0
E.4	0.0023	1	434.8
10	0.0022	1	454.5
12.7	0.002	1	500.0
15	0.0024	1	416.7

Picture: Chart showing calculation of stiffness' of each frame

3.3. Relative Stiffness Calculation

Relative stiffness's of each frame at that floor were determined by the following equation:

$$q_i = k_i / \sum k$$

where,

q_i = relative stiffness of desired frame
k_i = stiffness of desired frame
k = stiffness of frame (general)

Relative stiffness's of the frames in the north-south direction are independent of the frames in the east-west direction, and were determined separately.

There are three levels throughout the building where at least one of the frames changes geometry with respect to the other frames. Hence, this is why I determined different stiffness's for three different floors of the building. The stiffness's were determined for the 2nd, 13th, and 15th floors. Relative stiffness's of the floors below each of those calculated is assumed to be the same since the geometry of the frames are the same throughout those floors. For example, relative stiffness's for frames on floors 3 through 12 are the same as that of the 13th floor.

3.4. Lateral Load Distribution

The controlling wind pressures were multiplied by a particular floor's tributary height and the buildings width to determine the wind forces on each floor. These forces are distributed into the braced frames based on relative stiffness of the frames at that particular floor. Two types of forces are transferred into the frames; direct shear forces and torsional forces induced due to eccentricity of the lateral load. The following chart shows both the direct and eccentric shear forces applied to each frame at the 15th floor. Notice there is very little eccentric shear force which is the result of excellent placement of the frames. Calculation of the forces we done using the following equations:

<u>Direct Shear</u>

$V_i = (k_i / \sum k) * P_u$

<u>Eccentric Shear</u>

$V_i = (k_i\, x / \sum kx^2) * M_u$

North-South Wind on 15th Floor			
Pu=	38.4	(COR)x =	40.7
Eccentricity=	0.9	(COR)y =	41.8

DIRECT SHEAR COMPONENT

Frame	Stiffness, k (K/in)	Relative Stiffness, q	Story Force,P (K)	Direct Shear (K)
C.1	30.6	0.53125	38.4	20.4
D.2	27	0.46875	38.4	18
10	0	0	38.4	0
12.7	0	0	0	0
15	0	0	0	0

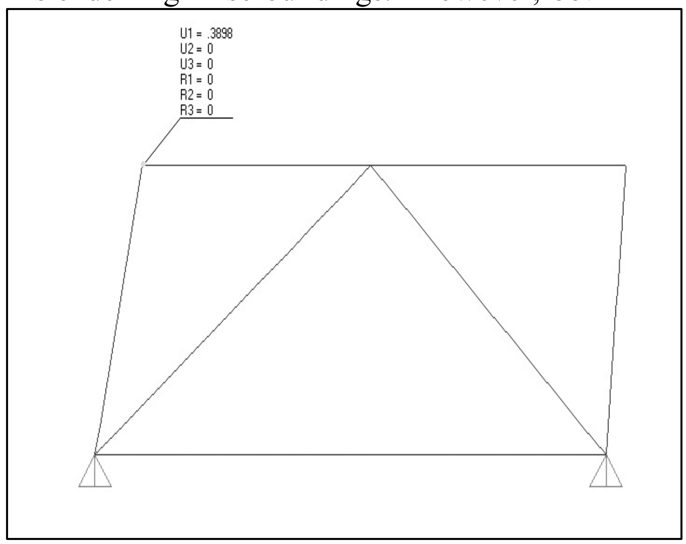
ECCENTRIC SHEAR COMPONENT

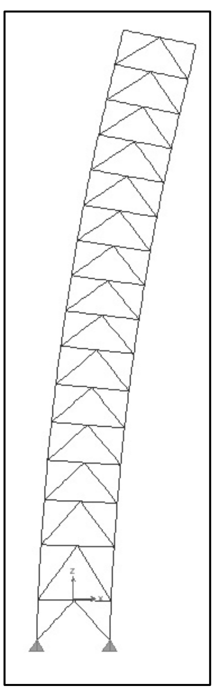
Frame	Stiffness, k (K/in)	Wall Location (ft)	Eccentricity, x (Ft)	kx^2 (K-ft)
C.1	30.6	27.8	13.0	5131.6965
D.2	27	55.5	-14.8	5914.08
10	25.4	0.0	41.8	44273.7875
12.7	31.1	32.7	9.1	2566.003638
15	30.2	64.8	-23.1	16091.72991
			$\Sigma(kx^2)$=	62931.52105

Frame	$kx/\Sigma(kx^2)$	Torsional Moment, M (K-ft)	Torsional Shear (K)	Total Shear (K)
C.1	0.006296844	34.56	0.217618945	20.18238106
D.2	-0.006349759	34.56	-0.219447675	18.21944768
10	0.016850856	34.56	0.582365584	-0.582365584
12.7	0.004488907	34.56	0.155136623	-0.155136623
15	-0.011077369	34.56	-0.382833861	0.382833861

3.5. Deflection

For a 15 story building, like Fordham Place, it is doubtful that story drift or total building drift would be a controlling design. This type of serviceability failures are found more often in slender high rise buildings. However, both story drift and total building drift were compared to H/400. This was done by modeling a single story 2-d chevron frame in SAP2000. The largest wind load and frame C.1 were selected. Frame C.1 was selected because it has a small stiffness and the largest force, which in turn will result in the largest deflection. The load applied to the frame was the correction proportion of direct and eccentric shears based on relative stiffness. The following picture is frame C.1 modeled in SAP2000 with the deflection visible.





H/400
=13.5ft * 12in/ft / 400
=0.405in

With a single story drift of 0.3898in, it can be concluded that both the single story drift and total building drift are within the H/400 deflection criterion. This can be done because if the critical frame, the one with the largest deflection, passes H/400 deflection criteria, then each of the other frames will deflect less than it and therefore the entire building with not deflect more than H/400. The picture to the left shows how the entire building will deflect. As expected, due to braced frames resisting lateral load as opposed to moment frames, the building deflection is non-uniform throughout its length.

3.6. Overturning

The major concern with overturning is the foundation system pulling out of the ground. Although there is some tensile capacity with the current system at Fordham Place, deep piles, they are designed to accept no uplift. With this said, a way to assure that the building will not overturn is to find the uplift force at an exterior column caused from the overturning moment and check that it is lighter than half the weight of the building. The following calculations will determine the total uplift force.

<u>Overturning Moment</u>

$$(Mo)u = 1.6 * [(15*217) + (28*204.5) + (36*192) + (36*179.5) + (35*167) + (35*154.5) + (34*142) + (34*129.5) + (33*117) + (33*104.5) + (34*91) + (46*77.5) + (45*64) + (49*50) + (54*34) + (29*14.5)]$$

$$(Mo)u = 103029.6 \text{ K-ft}$$

<u>Axial uplift Force in Exterior Column</u>

$$Pu = 103029.6 \text{ K-ft} / (4 * 27.75\text{ft}) = 928.2 \text{ K}$$

<u>Approximate Weight of Building</u>

If we consider self weight (60psf) only, a conservative approach, and it is larger than 928.2 K, then it is safe to say the building will not overturn.

Weight = W = Total Weight / 2
= [60psf (83.25ft)(111ft)*10floors] + [60psf (183ft)(83.25ft)*6 floors]/2
= 5514.5 K > 928.2K

4. Spot Check

The member I chose to spot check was the bracing member located at the concourse floor. This member was selected because it will have to transfer the largest lateral load since it will have to resist the wind pressure from the whole building. This raises the question; why are all lateral resisting members at the top of the building the same size as those at the bottom? This was done to avoid the troubles of having different bracing member sizes. Such potential problems are:

1. Extra cost for member / connection design
2. Possibly placing member in wrong frame
3. Time to assure members are in correct location
4. Availability of different member sizes
5. Price to order different member sizes

HSS 12 x 12 x ½
Lb = 20ft
$\Phi P_n = 838k > P_u = 224k$

For Calculation of Pu, See Appendix 6.1 Spot Check Calculation

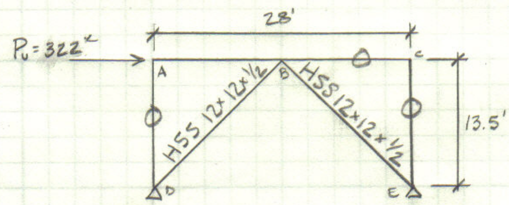
The extra capacity found in these members can be attributed to this design being controlled by drift. From 3.5 Deflections section, where the deflection of this same bay was analyzed using SAP2000 and found to be 0.3898in, or 96% of the allowable drift of H / 400.

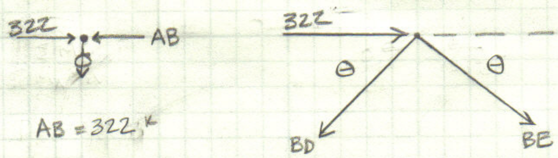
5. Conclusion

After performing the design and analysis calculations for the lateral system at Fordham Place, it is apparent the lateral system designed is adequate. All load combinations were considered. However, since the assumption of lateral resisting members would only resist lateral loads, load case #4 and #6 controlled because of the 1.6W term. It was determined that the lateral members were more than strong enough to carry required loads, but the controlling failure mode was drift. Usually a building of 15 stories will not be controlled by drift, but due to architectural features limiting the building to only three locations where braced frames can be placed, it minimized the ability for the building to reduce drift. Fordham Place was heavy enough to fully overcome the overturning moment induced by the wind loading. The system appeared to not only be adequate but also very efficient.

6. Appendix

6.1 Spot Check Calculations





$AB = 322^k$

$\theta = \tan^{-1}(13.5/14)$

$\theta = 43.9°$

$\Sigma F_y = 0$

$-BD = BE$

$\Sigma F_x = 0$

$322 - BD \cos 43.9 + BE \cos 43.9 = 0$

$322 - 2BD \cos 43.9 = 0$

$BD = 224^k \qquad BE = -224^k$

$L_b = \sqrt{(13.5)^2 + (14)^2}$

$L_b = 19.4$ USE 20 ft

TABLE 4-6 LRFD MANUAL

$\phi P_n = 685^k \geq P_u = 224^k$

6.2. Wind Loads

North – South Direction

Height	K_z	q_h	q_v	$p_{leeward}$	$p_{windward}$	p_{net}
0-15	0.57	25.4592	12.4032	-9.8527104	7.680061	17.53277184
15-20	0.62	25.4592	13.4912	-9.8527104	8.353751	18.20646144
20-25	0.66	25.4592	14.3616	-9.8527104	8.892703	18.74541312
25-30	0.7	25.4592	15.232	-9.8527104	9.431654	19.2843648
30-40	0.76	25.4592	16.5376	-9.8527104	10.24008	20.09279232
40-50	0.81	25.4592	17.6256	-9.8527104	10.91377	20.76648192
50-60	0.85	25.4592	18.496	-9.8527104	11.45272	21.3054336
60-70	0.89	25.4592	19.3664	-9.8527104	11.99167	21.84438528
70-80	0.93	25.4592	20.2368	-9.8527104	12.53063	22.38333696
80-90	0.96	25.4592	20.8896	-9.8527104	12.93484	22.78755072
90-100	0.99	25.4592	21.5424	-9.8527104	13.33905	23.19176448
100-120	1.04	25.4592	22.6304	-9.8527104	14.01274	23.86545408
120-140	1.09	25.4592	23.7184	-9.8527104	14.68643	24.53914368
140-160	1.13	25.4592	24.5888	-9.8527104	15.22538	25.07809536
160-180	1.17	25.4592	25.4592	-9.8527104	15.76434	25.61704704
180-200	1.2	25.4592	26.112	-9.8527104	16.16855	26.0212608
200-250	1.28	25.4592	27.8528	-9.8527104	17.24645	27.09916416

Level	height range (ft)	Tributary Height (ft)	Tributary Width (ft)	Area Ave. Wind Pressure(psf)	F_x (k)
B		0.00	164	0.0	0
1	0-10	10.00	164	17.5	29
2	10-28	18.00	164	18.3	54
3	28-43	15.00	164	20.1	49
4	43-56.5	13.50	158	21.0	45
5	56.5-70	13.50	158	21.7	46
6	70-83.5	13.50	112	22.5	34
7	83.5-96.5	13.00	112	23.0	33
8	96.5-109	12.50	112	23.7	33
9	109-121.5	12.50	112	23.9	34
10	121.5-134	12.50	112	24.5	34
11	134-146.5	12.50	112	24.8	35
12	146.5-159	12.50	112	25.1	35
13	159-171.5	12.50	112	25.6	36
14	171.5-184	12.50	112	25.7	36
15	184-196.5	12.50	86	26.0	28
roof	196.5-203	6.50	86	26.5	15
	Σ =	203	Σ =	370	

East-West Direction

Height	K_z	q_h	q_v	$p_{leeward}$	$p_{windward}$	p_{net}
0-15	0.57	25.4592	12.4032	-5.94981504	7.729674	13.67948928
15-20	0.62	25.4592	13.4912	-5.94981504	8.407716	14.35753088
20-25	0.66	25.4592	14.3616	-5.94981504	8.950149	14.89996416
25-30	0.7	25.4592	15.232	-5.94981504	9.492582	15.44239744
30-40	0.76	25.4592	16.5376	-5.94981504	10.30623	16.25604736
40-50	0.81	25.4592	17.6256	-5.94981504	10.98427	16.93408896
50-60	0.85	25.4592	18.496	-5.94981504	11.52671	17.47652224
60-70	0.89	25.4592	19.3664	-5.94981504	12.06914	18.01895552
70-80	0.93	25.4592	20.2368	-5.94981504	12.61157	18.5613888
80-90	0.96	25.4592	20.8896	-5.94981504	13.0184	18.96821376
90-100	0.99	25.4592	21.5424	-5.94981504	13.42522	19.37503872
100-120	1.04	25.4592	22.6304	-5.94981504	14.10327	20.05308032
120-140	1.09	25.4592	23.7184	-5.94981504	14.78131	20.73112192
140-160	1.13	25.4592	24.5888	-5.94981504	15.32374	21.2735552
160-180	1.17	25.4592	25.4592	-5.94981504	15.86617	21.81598848
180-200	1.2	25.4592	26.112	-5.94981504	16.273	22.22281344
200-250	1.28	25.4592	27.8528	-5.94981504	17.35786	23.30768

Level	height range (ft)	Tributary Height (ft)	Tributary Width (ft)	Area Ave. Wind Pressure(psf)	F_x (k)
B		0.00	112	0.0	0
1	0-10	10.00	112	17.5	20
2	10-28	18.00	112	18.3	37
3	28-43	15.00	112	20.1	34
4	43-56.5	13.50	90	21.0	26
5	56.5-70	13.50	90	21.7	26
6	70-83.5	13.50	90	22.5	27
7	83.5-96.5	13.00	90	19.2	22
8	96.5-109	12.50	90	19.9	22
9	109-121.5	12.50	90	20.1	23
10	121.5-134	12.50	90	20.7	23
11	134-146.5	12.50	90	21.0	24
12	146.5-159	12.50	90	21.3	24
13	159-171.5	12.50	90	21.8	24
14	171.5-184	12.50	90	21.9	25
15	184-196.5	12.50	88	22.2	24
roof	196.5-203	6.50	88	22.7	13
	$\Sigma =$	203	$\Sigma =$	332	

6.3. Seismic Loads

Assumptions:
Occupancy Category I (Table 1-1)
Seismic Use Group I (Table 9.1.3)
Importance Factor = 1.0 (Table 9.1.4)
Site Class D (Table 9.4.1.2)
Steel Concentrically Braced Frames

Ss = 0.43 (Figure 9.4.1.1a)
S1 = 0.095 (Figure 9.4.1.1b)

Sms = 0.626
Sm1 = 0.228

Sds = 0.417
Sd1 = 0.152

T = 1.725
Cs = 0.022

Seismic Design Category B

Effective Seismic Weight of Structure (9.5.3)

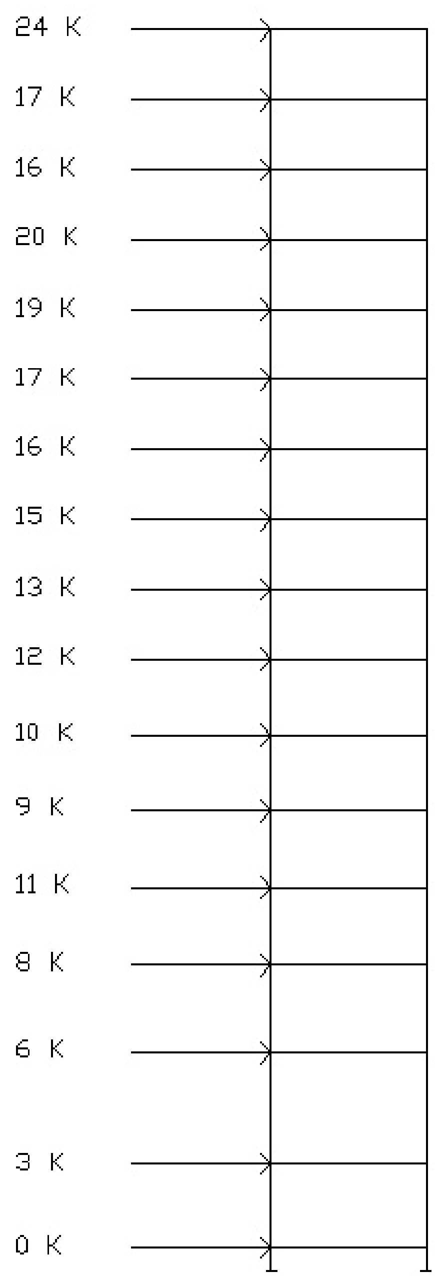
W_{TOTAL}= 9921 k

Seismic Base Shear (9.5.5.2)

$V = C_s W$

V = 218 k

Level	w_x (k)	h_x	$w_x h_x^k$	C_{vx}	F_x (k)
B	0	0	0	0	0
1	910	14.5	13195	0.012221	3
2	871	34.25	29831.75	0.027629	6
3	840	50	42000	0.038899	8
4	840	63.75	53550	0.049596	11
5	569	77.5	44097.5	0.040841	9
6	569	91	51779	0.047956	10
7	554	104.5	57893	0.053618	12
8	561	117	65637	0.06079	13
9	561	129.5	72649.5	0.067285	15
10	561	142	79662	0.07378	16
11	561	154.5	86674.5	0.080274	17
12	561	167	93687	0.086769	19
13	561	179.5	100699.5	0.093264	20
14	423	192	81216	0.075219	16
15	423	204.5	86503.5	0.080116	17
roof	556	217	120652	0.111743	24
Σ=	9921	Σ=	1079727		



Seismic Story Forces (E-W)