ARIC HEFFELFINGER
FORDHAM PLACE
BRONX, NY
STRUCTURAL OPTION
ADVISOR - DR. HANAGAN

Structural Tech Report #1

Structural Concepts / Structural Existing Conditions Report

Table of Contents

1. Executive Summary

Introduction

This report analyzes the structural procedures used to design Fordham Place in Bronx, NY.

Building Description

Fordham Place is a 15 story office / retail building that is located at 400 East Fordham Road, Bronx, NY. The 174060 sq. ft tower is going to tie into an existing 6 story SEARS building. In the new tower, structural engineers used modern design, taking advantage of composite action using steel beams with a 6 ¼ " concrete slab. The slab will be supported by 3" composite floor deck with 3" headed shear studs within the slab. Steel columns are used to transfer load to foundation, where it will be supported by a number of 150 ton piles. The main lateral resisting system is made up of steel concentrically loaded chevron braced frames.

Structural Design Code

Building Code of New York City, 2003, ASCE 7-02

Loads / Spot Checking Summary

When comparing my calculated gravity, wind, and seismic loads to those of the designer, some values will differ. Gravity loads compared very well with those of the designer while my lateral loads appear to differ by a significant amount. When doing both gravity and lateral spot checking of members, I compared capacity of members to the ultimate moments and forces. For both gravity and lateral spot checking my capacities were slightly larger than the ultimate design values. I believe the indifference in the gravity values can be attributed to designing with ASD as opposed to LRFD. Lateral indifferences can a cause of my conservative assumption of each of 3 braced frames will receive 40% of the lateral load, equally a total of 120%. It can also be attributed to the design differences between New York City Building Code and ASCE 7-02, the code that I used for my loading values.

2. Building Description

2.1. General

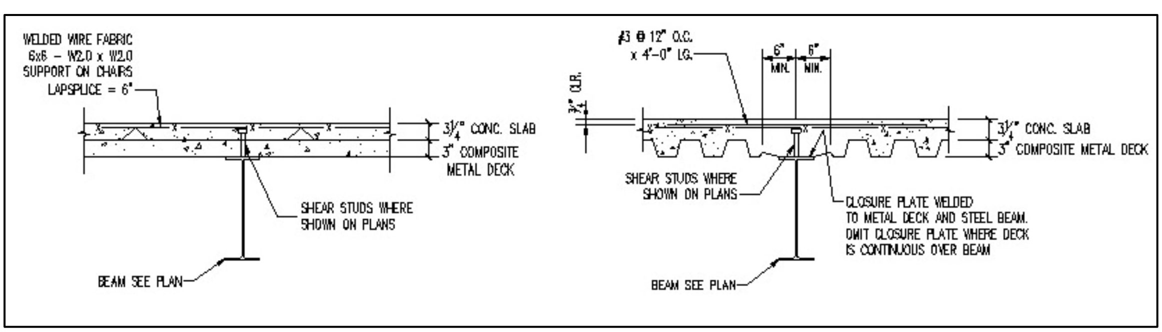
As you look around Bronx, NY you will notice a distinct similarity between most buildings; this being they are shorter older buildings, most less than 6 stories. Once Fordham Place is erected, it will tower over the city of Bronx, rising 15 stories above ground level. As you go up the building, its size 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 Structural System

2.2.1. Superstructure

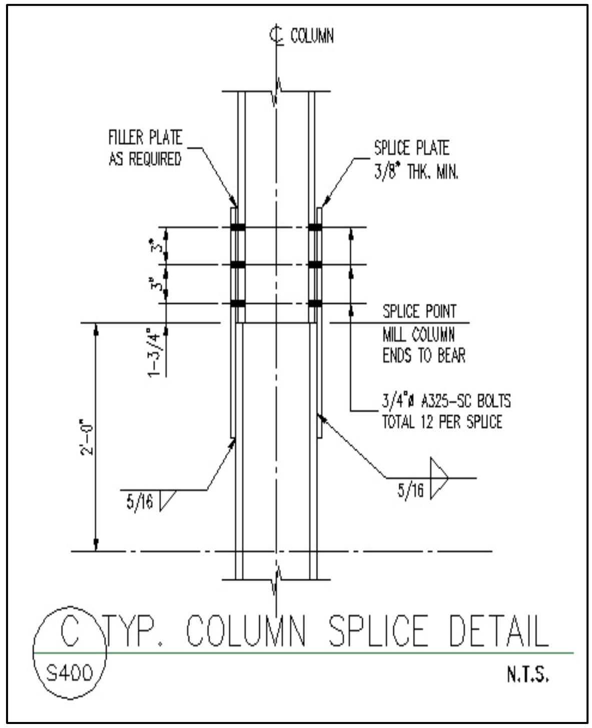
Floor System

The floor system of Fordham Place consists of structural steel W sections that support metal deck and concrete slab. The W shape beams and girders are A992 grade 50 and support a light weight concrete (115pcf) slab of 6.25 in. The concrete's compressive strength is f'_c = 3000psi for all floors. Reinforcing of concrete is done with high strength billet deformed steel bars with fy = 60,000psi as a minimum. All floor deck is 20 gage 3" deep galvanized composite deck and is continuous over 2 spans at the joints of the deck. All shear studs are headed studs of grade 1015 or 1020 cold finish carbon steel. Studs, at a maximum are spaced every 12".



Columns

Columns consist of rolled structural W14 shapes grade 50. However there are a few W10x39's that extend from the 14th floor to the roof at selected areas. Columns extend from the concourse floor to just above the second floor, extending 3 floors or 36'. From the second floor up to the roof, columns are spliced at every two floors or 27'. Column Splices consist of 2 – 3/8" plates applied to the flanges of the columns being spliced. The plates are then connected to the bottom column with a 5/16" fillet weld all around the plate. The top column is then connected to the splice plate with 12 - 3/4" Ø A325 S.C. bolts.



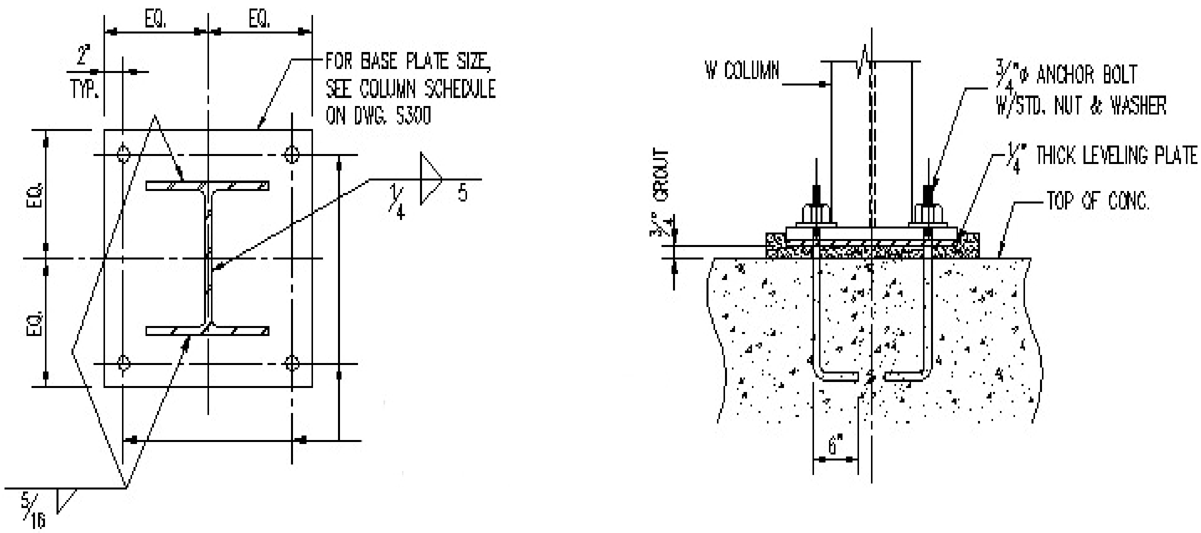
Roof

The roof consists of rolled structural steel W shapes supporting roof deck and a lightweight concrete slab. Structural steel members are grade 50 W16 shapes and typically span approximately 27' with spacing of 9'. Roof deck is 20 gage, 3" deep galvanized wide rib type NI and is continuous over 2 spans at the joints of the deck. The roof deck will span from beam to beam, 9ft., and the short direction of a typical roof bay. The roof deck will be connected to the structural steel with 5/8" puddle weld in a 12-6-12 in pattern. Compressive strength of concrete on the roof is f'c = 3500psi at a minimum. The top of the concrete slab is 3 ¼"above top of slab, totaling to a 6 ¼"concrete slab.

2.2.2 Substructure

Foundations

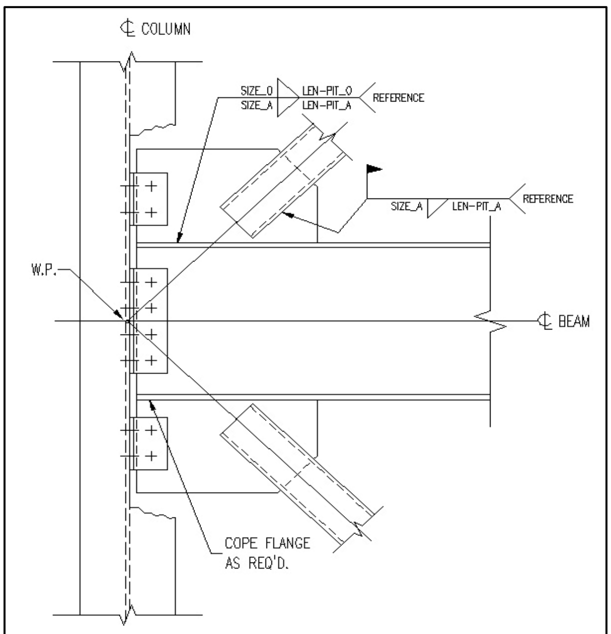
The foundation system of Fordham Place is composed of 150 ton steel piles that extend approximately 45 – 50ft deep into bedrock. The piles are A992 grade 50 rolled W shapes and are capped with concrete caps that have a compressive strength of f'c = 3000psi. The pile caps will range in size depending on the number of piles it needs to contain, which is dependent on the load a given column transfers. The number of piles per pile cap ranges from 4 (PC-4) to 13 (PC-13). Load is transferred from the columns to the pile caps via A36 1/4" steel base plates. The base plate is welded to the column using a 5/16" fillet weld on the exterior of the flanges and a ¼" fillet weld on the web and interior of the flanges. The base plate is connected to the pile cap with 4 - 3/4" Ø anchor bolts extending 12 inches into the pile cap before turning 180 degrees and extending 6 more inches. Flush with the pile cap will be a slab on grade with a compressive strength f'c = 4000psi.



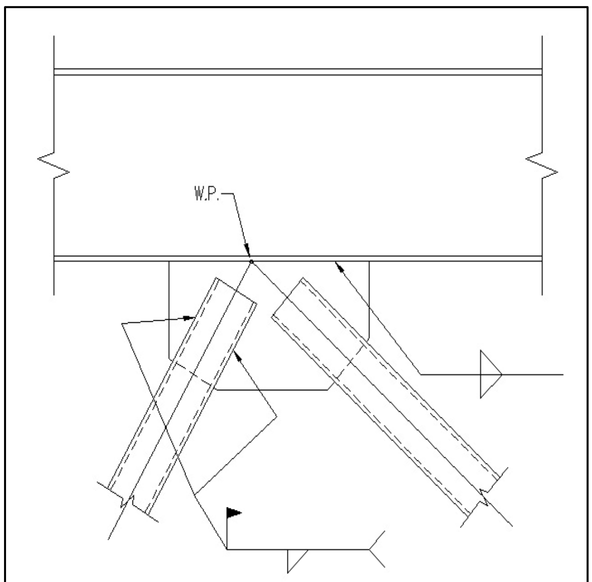
Base Plate Details

2.2.3 Connections

Throughout Fordham Place, there are many different connections, of which I have already talked about two; base plates and column splices. Other connections to consider are shear, moment, bracing connections to both columns and beams. Typical shear connections consist of double angles with the required number of A325 3/4"Ø S.C. bolts. Moment connections will be the same as a typical shear connection but will also have the top and bottom flanges of the beam welded with a 5/16" full penetration field weld. Bracing connections from the braced frames will be to beams and columns at different elevations of the building (See pictures below). Bracing to a column connections will compose of a gusset plate being welded to the underside of a beam and bolted to the column. Bracing members will be bolted to the gusset plate. Bracing to beam connections will occur at the midspan of the beam and will consist of a gusset plate welded to the underside of the beam. Bracing members will then be bolted to the gusset plate.



BRACING CONN. TO COL.



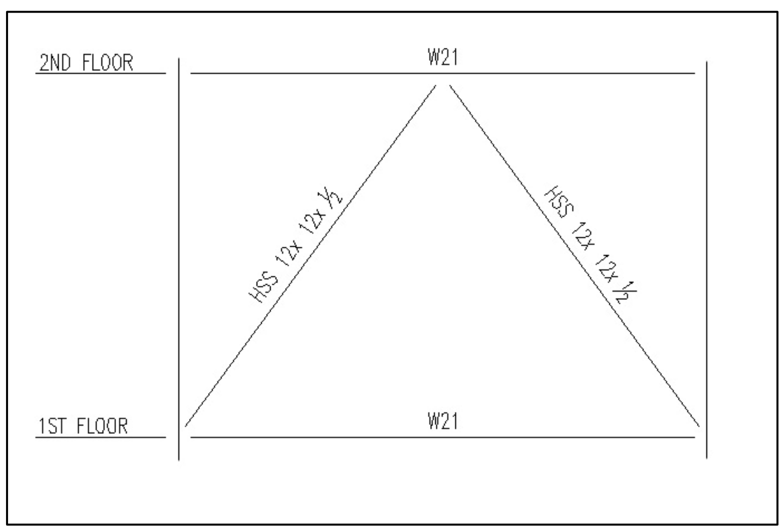
BRACING CONN. TO BEAM

2.2.4 Enclosure

The building enclosure at Fordham Place consists of many different types. For the existing building, you will notice an older light brown brick wall with granite piers running the height of the building to interrupt the brick. At the base there currently is steel covering windows. But soon, when Fordham Place is finished with construction, it will return to display windows for retail stores. Playing off the older style building the existing structure brings, the new tower will match the light brown brick in the façade. The façade will also have sunlight gleaming off the many blue tinted glass panes. Finally, on the lower 2 floors facing Fordham Road, the building will have a glass façade enclosing a two story lobby area.

2.2.5 Lateral System

The lateral system is composed of moment connections and braced frames. Moment connections are mostly located along the plane in which the existing building and new tower are connected. This is done so that each building can act independent of each other. The braced frames are “K” type braces utilizing A500 grade B HSS12x12x1/2” structural steel members. They are located in six different bents, all of which are centrally located near the core of the building and extend from the concourse floor to the roof. The bracing is located near the core of the building in order to avoid inducing any internal torsion. As discussed in the connections part of this report, there is bracing connections to beams and columns. On each side of the bent, a bracing member will be framed from the bottom corner of the bent (column connection) to the midspan of the upper beam (beam connection). See picture to right.



Picture: Typical Chevron Bracing

3. Structural Design Criteria

3.1 Structural Design Code

The 2003 Building Code of New York City

3.2 Structural Design Specifications and Standards

Structural Concrete Design – American Concrete Institute, Building Code Requirements for Structural Concrete, ACI 318-02

Structural Steel Design – American Institute of Steel Construction, Steel Construction Manual, Allowable Stress Design Ninth Addition

Welding - American Welding Society, Structural Welding Code - Reinforcing Steel, AWS D1.4-79

Steel Deck - Design Manual for Floor Decks and Roof Decks, SDI

Masonry – American Concrete Institute, Specifications for masonry Structures, ACI 530.1

3.3 Project Material Strength

Concrete (28 day minimum compressive strength)

Footings: 3000psi
Slab on Grade: 4000psi
Piers: 4000psi
Footings: 4000psi
Steel Deck Slabs (lightweight): 3500psi

Lightweight Concrete: 115pcf
Normal weight Concrete: 145pcf

Steel Reinforcement

Reinforcing Bars – ASTM A615 or A706 Grade 60 (Fy = 60,000psi min)
Welded Wire Fabric – ASTM 185

Metal Deck

Roof Deck: ASTM A653, Grade 33
Floor Deck: ASTM A661, Grade C, D or E.

Structural Steel members

Columns, Beams, Girders: ASTM A992 or ASTM A572, Grade 50.
Structural Steel Plates and miscellaneous steel: ASTM A36
Cold-Formed Steel Tubing: ASTM A500, Grade B.
Structural Steel Pipe: ASTM A53 or A500, Type E or S, Grade B.

Connectors

Headed shear stud: ASTM A108, Grade 1015 or 1020
Anchor Rods: ASTM F1554 Grade 36,
Bolts: ASTM A325

Welding

All Welds: AWS E70XX Electrodes, minimum tensile strength = 70,000psi

Masonry

Concrete Masonry Units: ASTM C90, f'c = 3750psi
Grout: ASTM C476 f'c = 2500psi

3.4 Design Gravity Loads (ASCE 7-02)

Load Type	Existing Retail	Stairs	New Building Retail	Existing Building Community Areas
Dead Load	122	50	60	122
Superimposed Dead Load	20	-	30	20
Live Load	100	100	100/75	50
Truck Load	-	-	250	-

Load Type	New Building Community Areas	Existing Roof	New Roof	Penthouse
Dead Load	60	117	60	20
Superimposed Dead Load	30	10	20	60
Live Load	80	30	30	30
Truck Load	-	-	-	-

Table: Designer's gravity loads.

*Note: See PDF on next page for my gravity loads

GRAVITY

SNOW

TERRAIN CAT "B"
FULLY EXPOSED
$C_e = 0.9$
$C_t = 1.0$
$I = 1.0$
$p_g = 30\ psf$
FLAT ROOF

$p_f = 0.7 C_e C_t I p_g$
$= 0.7\,(0.9)(1.0)(1.0)(30\,psf)$

$p_f = 18.9\ psf$

LIVE

LOBBY = 100 PSF
CORRIDOORS = 100 PSF (80 PSF ABOVE FIRST FLOOR)
OFFICES = 50 PSF (USED 60PSF BASED ON CLIENTS REQUEST)
RETAIL =
FIRST FLOOR = 100 PSF
UPPER FLOORS = 75 PSF

ROOF LIVE LOAD

$L_r = 20 R_1 R_2$ $\qquad R_2 = 1$ FLAT ROOF

$A_T = 258\ ft^2$

$R_1 = 1.2 - 0.001(258)$
$R_1 = 0.94$

$L_r = 20(0.94)(1.0)$

$L_r = 18.8\ PSF$

DEAD LOAD

FOUND FOR EACH INDIVIDUAL CASE

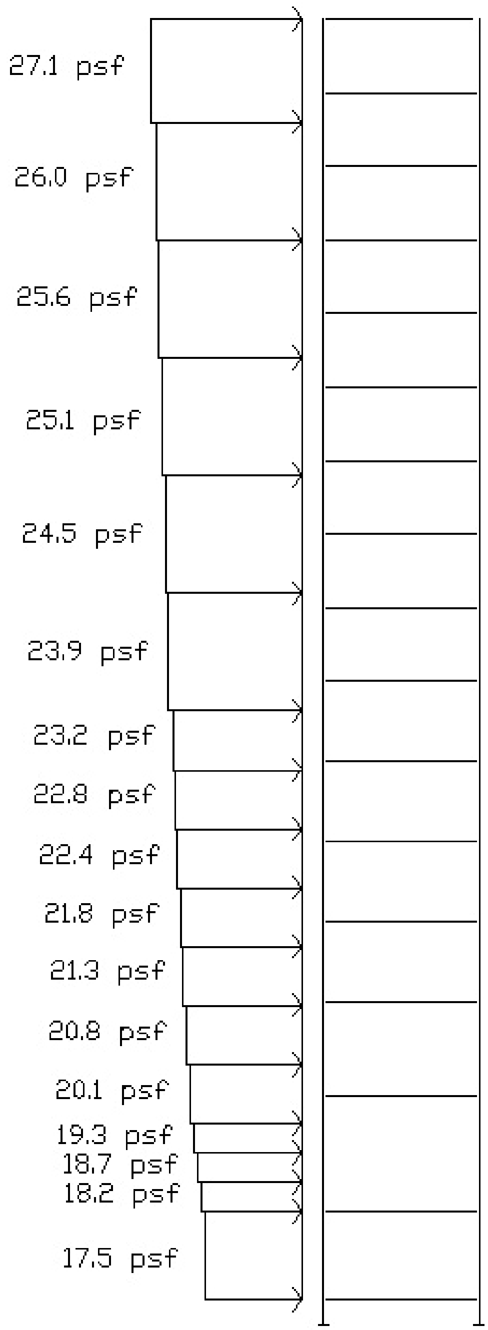
3.5 Wind Loads

North – South Direction

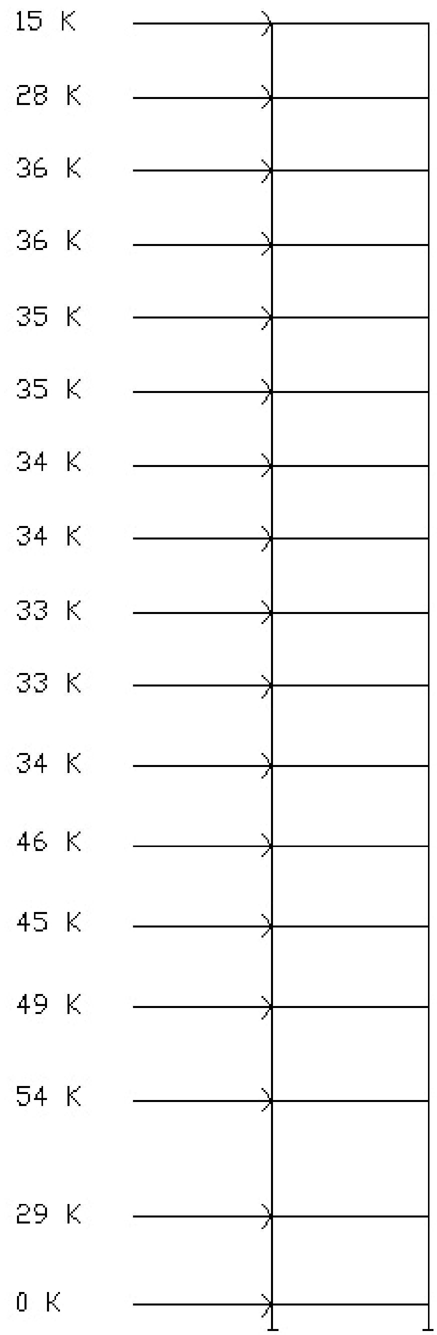
Note: For complete calculations, see Appendix

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	



Wind Net Pressures (N-S)

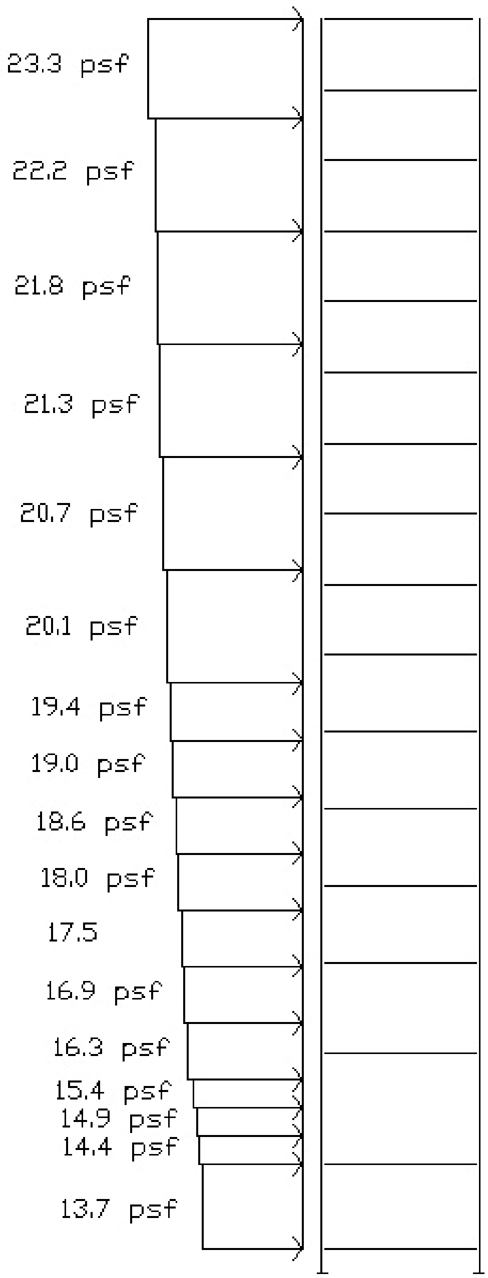


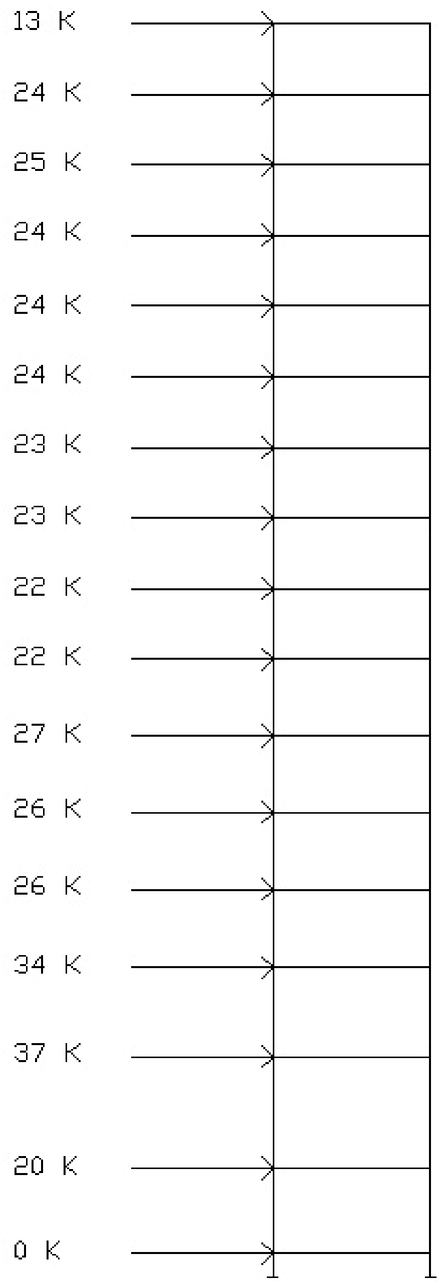
Wind Story Forces (N-S)

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
	Σ =	203	Σ =	332	





Wind Net Pressures (E-W) Wind Story Forces (E-W)

3.6 Seismic Loads

*Note: For complete calculations see Appendix

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)

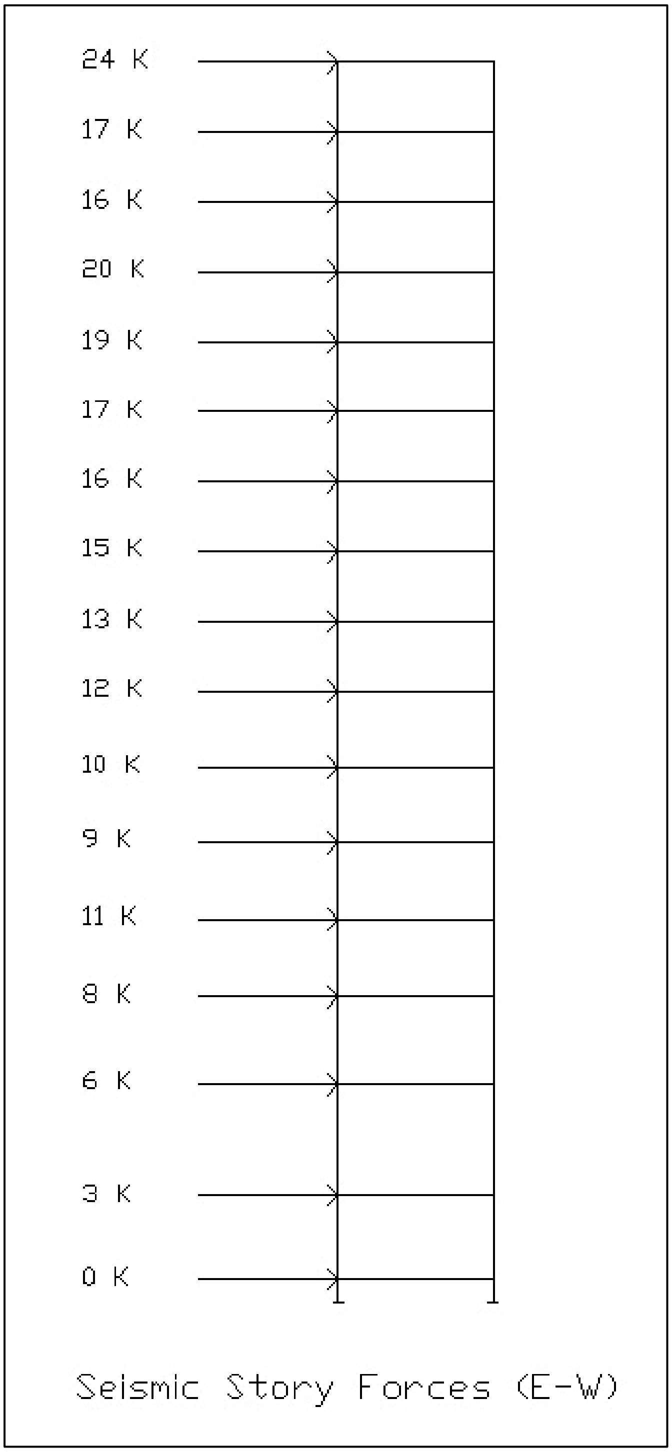
W_{TOTAL} = 9921 k

Seismic Base Shear (9.5.5.2)

$V = C_s W$

V = 218 k

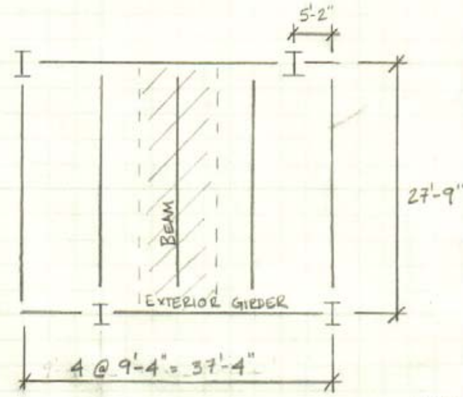
Level	w_x (k)	h_x	$w_x h_x^k$	C_{vx}	F_x (k)
B	0	0	0	0	0
1	910	14.5	13195	0.012221	3
2	871	34.25	29831.75	0.027629	6
3	840	50	42000	0.038899	8
4	840	63.75	53550	0.049596	11
5	569	77.5	44097.5	0.040841	9
6	569	91	51779	0.047956	10
7	554	104.5	57893	0.053618	12
8	561	117	65637	0.06079	13
9	561	129.5	72649.5	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		


24 K
17 K
16 K
20 K
19 K
17 K
16 K
15 K
13 K
12 K
10 K
9 K
11 K
8 K
6 K
3 K
0 K
Seismic Story Forces (E-W)

3.7 Gravity Load Spot Check

GRAVITY LOAD SPOT CHECK

* THERE IS NO "TYPICAL" BAY SO I RANDOMLY SELECTED A BAY ON THE 9TH FLOOR



CONCRETE
3" - 20 GA COMP FLOOR DECK
+ 3¼" L.W. CONCRETE SLAB
TOTAL SLAB THK = 6¼"
$f'_c = 3500$ PSI
$W_c = 115$ PCF

STEEL
A992 $f_y = 50$ KSI

BEAM

SELF = 115 PCF (3¼" + 1.5")(9'-4")/12 + 26 PLF (W16x26)

= 451 plf

SUPERIMPOSED DEAD LOAD = 30 PSF (9'-4") = 279 plf

TOTAL DL = 451 plf + 279 plf = 730 plf

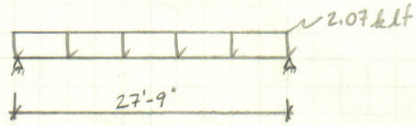
LIVE LOAD = 60 PSF$_{\text{OFFICE}}$ + 20 PSF$_{\text{PARTITION}}$ = 80 PSF (9'-4") = 747 plf

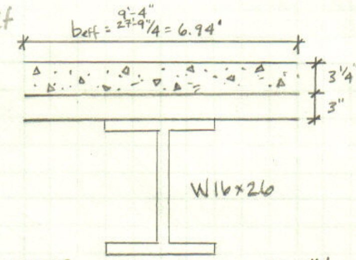
$w_u = 1.2D + 1.6L$
$= 1.2(730) + 1.6(747)$

$w_u = 2.07$ klf

GRAVITY LOAD
SPOT CHECK CONT'

BEAM CONT'





$M_u = \frac{w_u l^2}{8} = \frac{(2.07\,klf)(27.75)^2}{8}$

$M_u = 199^{ft\text{-}k}$

28 SHEAR STUDS   ASSUME 17.7^k/STUD

$\Sigma Q_n = 14(17.7)$
$= 247.8^k$

$0.85 f'_c ba = \Sigma Q_n$

$a = \frac{247.8}{0.85(3.5)(6.94)(12)}$

$a = 1.00$ IN

$T = A_s f_y = (50)(7.68\,in^2)$
$= 384^k$

$C_c = 0.85 f'_c ba = 0.85(3.5)(6.94')(12)(3.25)$
$= 805^k$

PNA IN CONC.

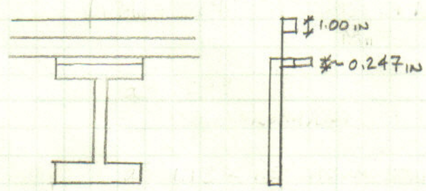
$A_s f_y = 247^k$

$A_s = 247.8^k/50$

$A_s = 4.956\,in^2$

$\frac{7.68 - 4.95}{2} = (A_s)_{comp} = 1.362\,in^2$

$1.362/b_{f} = 1.362/5.5 = 0.247$ IN   ALL IN FLANGE OF STEEL.



$M_n = 0.85 f'_c ba(6.25 - a/2) + A_s f_y(d/2) - (0.247)(5.5)(f_y)(0.247/2)2$
$= 0.85(3.5)(83.3)(1.0)(6.25-0.5) + (384^k)(15.7/2) - (0.247)(5.5)(50)(0.247/2)(2)$

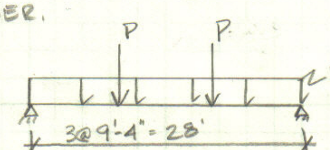
$M_n = 368^{k\text{-}ft}$

$\phi M_n = 0.9(368) = 331.7^{k\text{-}ft}$ $M_u = 199^{k\text{-}ft}$

I ATTRIBUTE THE DIFFERENCE IN ϕM_n AND M_u TO THE DIFFERENCE IN DESIGNING WITH ASD TO LRFD. IN ASD, COMPOSITE DESIGN IS TAKEN AS 0.76 F_y, WHERE AS IN LRFD, COMPOSITE DESIGN IS AT FULL ULTIMATE CAPACITY. ASD DESIGN WOULD YIELD A MUCH STRONGER MEMBER (MORE CONSERVATIVE) THAN LRFD, HENCE THE MOMENT CAPACITY, ϕM_n, OF THE CURRENT SYSTEM DESIGNED WITH ASD, AND ANALYZED WITH LRFD.

GRAVITY LOAD
SPOT CHECK CONT'

EXTERIOR GIRDER.



$P = \frac{w_u l}{2}$

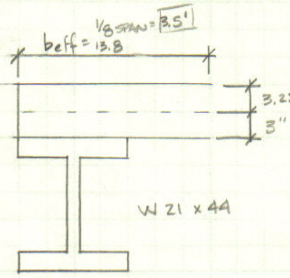
$= \frac{(2.07\,klf)(27.75')}{2}$

$P = 28.7^k$

$M_U = Pa + w_{wall} l^2/8$

$= (28.7^k)(9.\overline{3}') + (0.5)(28^2)/8$

$M_U = 317^{\,k\text{-}ft}$



48 - 3" SHEAR STUDS (17.7^k/STUD)

$\Sigma Q_n = 24(17.7^k) = 425^k$

$C_c = 0.85(3.5)(42")(3.25+1.5)$
$= 594^k$

$T_s = (13\,in^2)(50^{ksi})$
$= 650^k$

$a = \frac{\Sigma Q_n}{0.85 f'_c b} = \frac{425^k}{(0.85)(3.5)(42")}$

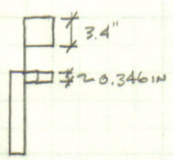
$a = 3.4\,in$

$A_s f_y = 425^k$
$A_s = 8.5\,in^2$

$(A_s)_c = \frac{13 - 8.5}{2} = 2.25\,in^2$

$\frac{(A_s)_c}{b_f} = 2.25/6.5 = 0.346\,in \leq h_f = 0.45$

$\therefore$ ALL COMP. IN FLANGE



$M_n = 0.85 f'_c ba(6.25 - a/2) + A_s f_y(d/2) - (0.346)(6.5)(f_y)(0.346/2)(2)$

$= 0.85(3.5)(42)(3.4)(6.25 - 3.4/2) + (50)(13)(20.7/2) - (0.346)(6.5)(50)(0.346)$

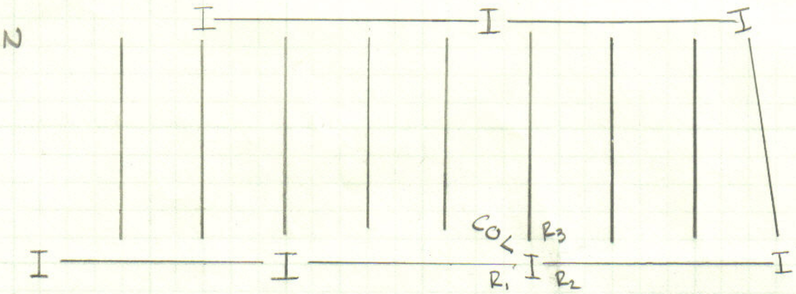
$M_n = 718$ ft-k

$\phi M_n = 0.9(718) = 647$ ft-k $\geq M_u = 317$ ft-k

I ATTRIBUTE THE DIFFERENCE IN M_U AND ϕM_n TO TWO THINGS. FIRST, I MADE THE ASSUMPTION THAT THE EXTERIOR GIRDER WAS SUPPORTING ONLY 1 STORY OF THE BRICK FACADE. WHEN THE NEW YORK CITY BUILDING CODE REQUIRES RELIEVING ANGLES AT LEAST EVERY 3 STORIES. THEREFORE I BELIEVE IT MAY BE DESIGNED FOR 3 STORIES. SECONDLY, SOME OF THE DIFFERENCE MAY BE COMING FROM DESIGNING WITH ASD AS OPPOSED TO LRFD AS WAS THE CASE WITH THE BEAM.

GRAVITY LOAD
SPOT CHECK CONT'

COLUMN



FROM PREVIOUS CALCULATIONS $R_1 = \frac{\omega_{wall}\, l}{2} + P$

$$= \frac{(0.5\ k/l')(27.75')}{2} + 28.7^{k}$$

$$R_1 = 35.6^{k}$$

$R_2 \doteq 35.6^{k}$ - A LITTLE CONSERVATIVE

$R_1 = R_2 = 35.6^{k} \qquad R_3 = 28.7^{k}$

TOTAL REACTION AT COLUMN FROM LOADING ON FLOOR 9

$R_9 = 2R_1 + R_3 = 100.1^{k}$

SINCE SLAB THICKNESS , FRAMING MEMBERS , SUPERIMPOSED DEAD LOADS, AND LIVE LOADS ARE THE SAME THROUGHOUT THE REMAINDER OF THE BUILDING, EACH FLOOR WILL CONTRIBUTE 100.1^{k} TO THAT COLUMN LOCATED BETWEEN THE 8^{TH} & 9^{TH} FLOOR.

$\therefore$

TOTAL AXIAL LOAD ON COLUMN

P_u = 7 FLOOR LOAD + ROOF = $7(100.1^{k}) + 13.7^{k}$

$P_u = 714^{k}$

* ASSUME BRACED FRAME PICKS UP ENTIRE LATERAL LOAD.

$\therefore$ COLUMN IS PURE AXIAL LOAD, NOT A BEAM COLUMN.

ASD DESIGN = W14 x 99 $\quad \phi P_n = 1090^{k} \quad P_u = 714^{k}$

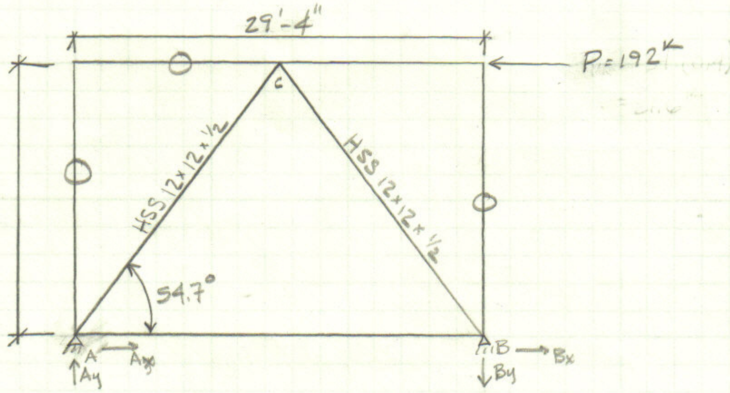
$l_b = 12.5'$

THE DIFFERENCE BETWEEN ϕP_n AND P_u IS ATTRIBUTED TO THE ASSUMPTION THAT THE COLUMN IS PURELY AXIALY LOADED. WITH THE ADDITION OF BENDING MOMENT, THE COLUMN AXIAL CAPACITY, ϕP_n WOULD DECREASE CLOSER TO P_u AS ϕM_n WOULD HAVE TO REACH AT LEAST M_u.

3.8 Lateral Load Spot Check

LATERAL LOAD
SPOT CHECK

CHEVRON BRACING



ASSUMPTION: EACH OF 3 CHEVRON BRACED FRAMES WILL RECEIVE 40% OF LATERAL FORCE. 40% × 3 = 120% MAKING IT A CONSERVATIVE ESTIMATE.

TOTAL SHEAR AT GROUND LEVEL = 576^k

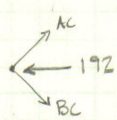
SHEAR PER FRAME = $576^k/3 = 192^k$

$\Sigma M_B = 0$

$A_y(29.\bar{3}) = 192^k(20')$

$A_y = 131^k \uparrow \quad B_y = 131^k \downarrow$

PT C



$\Sigma F_y = 0$

$A_C = BC$

$\Sigma F_x = 0$

$2\,AC\cos(54.7) = 192$

$AC = 166^k$

$P_u = 166^k$

$l_b = 24.8\,ft$

$\phi P_n = 620^k$

THERE IS QUITE A DIFFERENCE BETWEEN ϕP_n AND P_u FOR MY LATERAL LOAD CHECK. THIS COULD BE FOR A NUMBER OF REASONS; 1) CERTAIN SIZE MEMBER NEEDED TO MAKE THE CONNECTION BIG ENOUGH, 2) ME DISTRIBUTING THE LOAD VERY CONSERVATIVELY, 3) DID NOT INCORPERATE GRAVITY LOADS INTO THE COLUMNS.

3.9 Design Considerations not required in Tech 1

OTHER DESIGN CONSIDERATIONS

THE FOLLOWING ARE A LIST OF DESIGN CONSIDERATIONS NOT COVERED IN TECH 1.

1) STORY DRIFT
2) SOIL STRENGTH / SETTLEMENT
3) WIND UPLIFT
4) SNOW DRIFT AGAINST MECHANICAL EQUIPMENT
5) DEFLECTION LIMITATIONS
6) PILE CAPACITIES
7) PILE CAP PUNCHING SHEARS
8) RAIN LOADS
9) LOAD COMBINATIONS INVOLVING GRAVITY AND LATERAL LOADS
10) CONNECTION CHECKS
11) MORE ACCURATE DISTRIBUTION OF LATERAL LOADS
12) SLAB ON GRADE DESIGN
    - SOIL UPLIFT
    - GRAVITY LOADS

4. Appendix

WIND

$K_{zt} = (1 + K_1 K_2 K_3)^2$ NO TOPOGRAPHIC FRACTOR

$K_{zt} = 1.0$

$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6}$
$= (0.3)\left(\frac{33}{110.3}\right)^{1/6}$

$I_{\bar{z}} = 0.795$

TABLE 6-2
$c = 0.3$
$\ell = 320$ ft
$\bar{\epsilon} = 0.\bar{3}$

$h = 183.9$ ft
$B = 164$
$L = 112$
> OPPOSITE FOR E-W

$\bar{z} = 0.6h = 0.6(183.9\text{ft}) = 110.3$
MAX 30 ft

$L_{\bar{z}} = \ell\left(\bar{z}/33\right)^{\bar{\epsilon}}$
$= 320\left(110.3/33\right)^{0.\bar{3}}$
$= 478$

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}}$$

$$= \sqrt{\frac{1}{1 + 0.63\left(\frac{164+183.9}{478}\right)^{0.63}}}$$

$= 0.81$

$$G = 0.925\left(\frac{1 + 1.7 g_Q I_{\bar{z}} Q}{1 + 1.7 g_v I_{\bar{z}}}\right)$$

$g_v = g_Q = 3.4$

$$= 0.925\left(\frac{1 + 1.7(3.4)(0.795)(0.81)}{1 + 1.7(3.4)(0.795)}\right)$$

$G = 0.78$

$q_z = 0.00256\,(K_{zt} K_z K_d V^2 I)$
$= 0.00256\,(100)^2(1.0)(0.85)(1.0)K_z$

$q_z = 21.76 K_z$

$V = 100$ MPH
$K_d = 0.85$
CAT II : $I = 1.0$
$K_z =$ VARIES

$GC_{pi} = \pm 0.18$
WINDWARD $C_p = 0.8$
LEEWARD C_p (N-S) : $L/B = 112/164 = 0.68$ $C_p = -0.5$
LEEWARD C_p (E-W) : $L/B = 164/112 = 1.46$ $C_p = -0.3$

$P = qGC_p - q_i(GC_{pi})$

* SEE SPREADSHEET FOR THE REST OF WIND

SEISMIC

SEISMIC USE GROUP I

$I = 1.0$

SITE CLASS "D"

$S_1 = 9.5 = 0.095$ $\quad$ $S_S = 43 = 0.43$

$F_V = 2.4$ $\quad$ $F_A = 1.456$

$S_{M1} = F_V S_1 = (2.4)(0.095)$ $\quad$ $S_{MS} = F_A S_S = (1.456)(0.43)$

$S_{M1} = 0.228$ $\quad$ $S_{MS} = 0.626$

$S_{D1} = 2/3\, S_{M1} = 2/3\,(0.228)$ $\quad$ $S_{DS} = 2/3\, S_{MS} = 2/3\,(0.626)$

$S_{D1} = 0.152$ $\quad$ $S_{DS} = 0.417$

$W_{roof} = (\underbrace{60\text{ PSF}}_{DL} + \underbrace{18.9\text{ PSF}}_{SNOW})(7045\text{ ft}^2) = 556^k$

$W_{14\text{-}15} = (60\text{ PSF})(7045\text{ ft}^2) = 423^k$

$W_{8\text{-}13} = (60\text{ PSF})(9343\text{ ft}^2) = 561^k$

$W_7 = (60\text{ PSF})(9226\text{ ft}^2) = 554^k$

$W_{5\text{-}6} = (60\text{ PSF})(9483\text{ ft}^2) = 569^k$

$W_{3\text{-}4} = (60\text{ PSF})(13994\text{ ft}^2) = 840^k$

$W_2 = (60\text{ PSF})(14516\text{ ft}^2) = 871^k$

$W_1 = (60\text{ PSF})(15174\text{ ft}^2) = 910^k$

$W_{TOTAL} = 9921^k$

$R = 4.0$ STEEL CONCENTRICALLY BRACED FRAMES

$T = C_t h_n^x = (0.03)(207+15)^{0.75}$

$T = 1.725$ SEC

$C_s = \dfrac{S_{DS}}{R/I} = \dfrac{0.417}{4.0} = 0.104$ BUT NOT MORE THAN

$C_{SMAX} = \dfrac{S_{D1}}{T(R/I)} = \dfrac{0.152}{(1.725)(4)} = 0.022$ AND NOT LESS THAN

$C_{SMIN} = 0.044\, I\, S_{DS} = 0.044(1.0)(0.417) = 0.018$

$C_s = 0.022$

SEISMIC CONT'

$V = C_s W$
$= (0.022)(9921)$

$V =$ BASE SHEAR $= 218^{K}$

$K = 1 + \frac{(1.725 - 0.5)}{2}$

$K = 1.61$

SEE SPREADSHEET FOR THE REST OF SEISMIC