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Structural
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Office Building, Washington, DC*
Technical Assignment 3
11/21/05

All renderings courtesy of Dreyfus Property Group and KRJDA

Executive Summary

The office building analyzed is located in downtown Washington, DC. It is a reinforced concrete flat-slab system with drop panels. The tenant floor slabs are post-tensioned, but this was not considered in this report. The controlling lateral loads on the building came primarily from the seismic analysis. 1.2 Dead + 1.0 Earthquake + 1.0 Live + 0.2 Snow was therefore found to be the controlling load combination in most cases. The structure performed brilliantly in storey drift and building drift calculations. Overturning and torsion were determined to not be issues with the building design. No member strength issues were evident during the hand calculations and computer modeling. The designed structure adequately dissipates all lateral forces safely and efficiently to the foundation system.

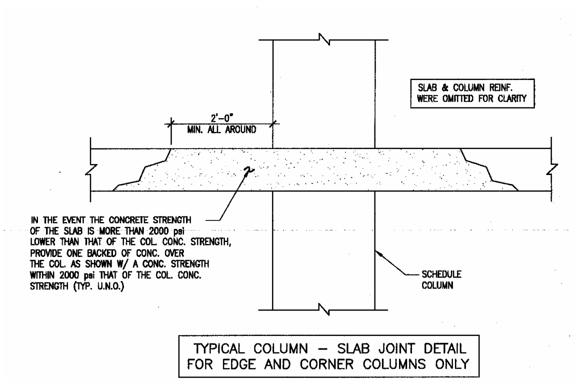
*Building location withheld at owners request

Introduction

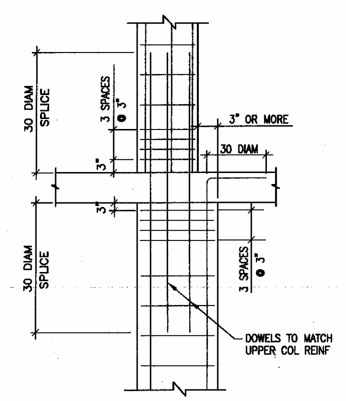
The building being analyzed is located in downtown Washington, DC. This is an urban setting, which is reflected in several factors in lateral load determination. Its primary use is for standard office tenants, and just meets DC height restrictions at 12 stories and 128 feet. Gravity loads are resisted by a two-way flat slab with drop panels poured around the majority of columns. The slab contains post-tensioned tendons for additional stiffness and an enhanced stress profile.

Lateral System

Concrete is the material of choice in the construction of this building. As stated, the structural system is a two-way flat-slab with drop panels and post-tensioning. The columns, drops, and slabs are all poured integrally and further joined by steel reinforcement. Typical framing details are reproduced below to prove the moment carrying ability of column-slab joints. Because the slab-column connection is properly reinforced and poured monolithically, the system can be considered a rigid moment frame. The lateral loads are transferred from the glass curtain wall to the slab edge. The slab acts as a rigid diaphragm and uniformly distributes the lateral loads to the columns along each frame section.

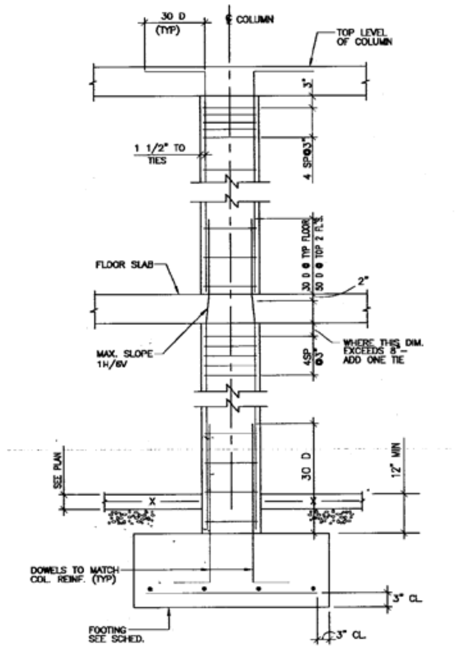


DETAIL 4-S.202



TYP CHANGE IN COLUMN SIZE

DETAIL 1-S.202

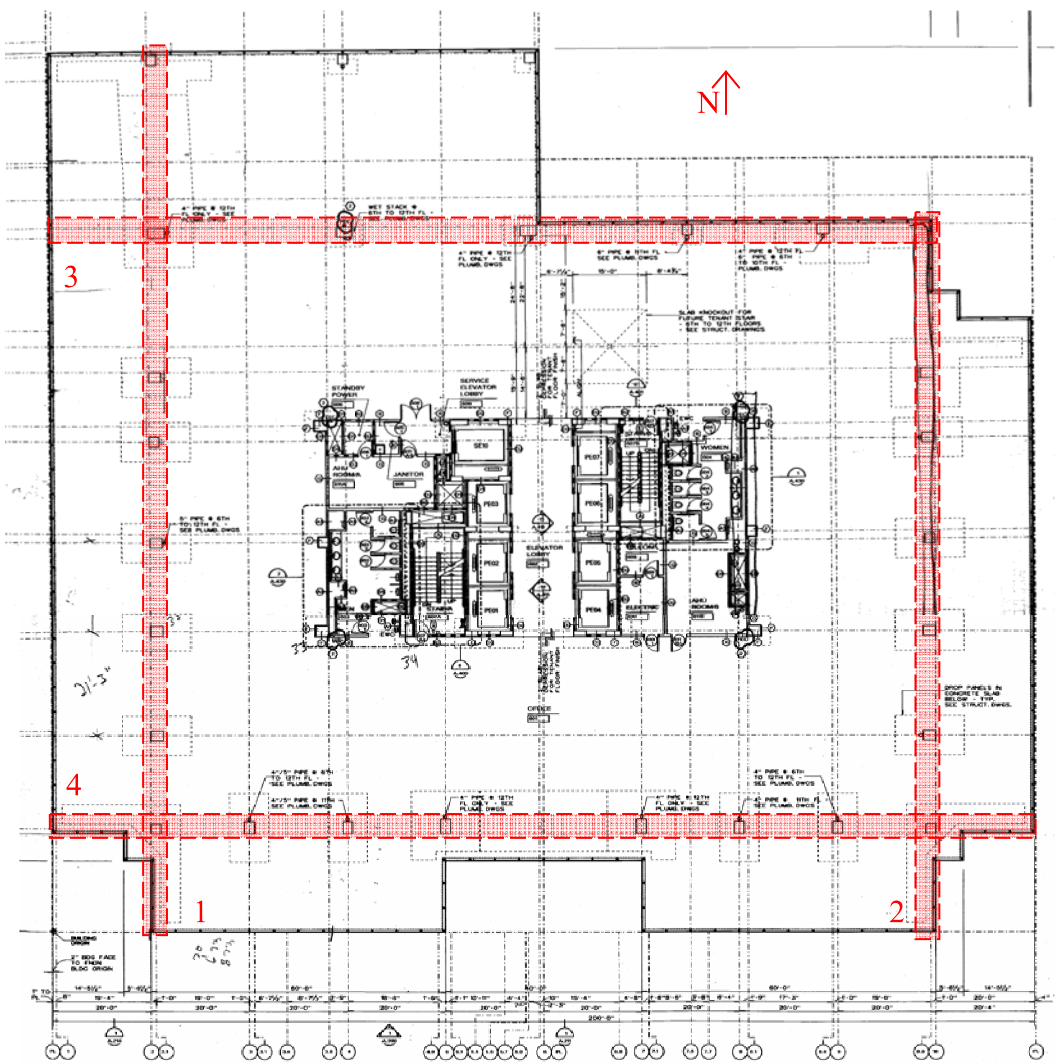


TYP COL, FTG & SPLICE

DETAIL 2-S.201

There are no marked shear walls on the plan. Typically elevator and stair cores are designed as shear walls, and possibly exterior CIP or masonry walls. In this design there are no exterior walls, but there are obviously stair and elevator cores. They are constructed of 8" concrete block, and filled to meet 1 hr or 2 hr fire ratings. These walls are not considered as shear walls. Furthermore, the walls do not fully enclose the cores, and are not connected to each other. There are columns located at the corners of each stair and elevator core. Because they do not from a closed section, these regions cannot provide torsional resistance either. All lateral load (direct and torsional) resistance comes from the rigid moment frame of the building.

Due to the eclectic column lines in the plan, only a few moment frames will be considered to resist the lateral forces. There are two major frames in each direction. All are comprised of the outmost columns except the north frame in the E-W resisting system. The frames were chosen where the larger columns were aligned for more than 3 spans. An overview of the primary moment frame locations is shown below. Frames 1 & 2 resist in the N-S direction, and frames 3 & 4 resist in the E-W direction.



Lateral Loads

Possible Load Combinations per ASCE 7-02

1.2D+1.6L
1.2D+1.6L+0.5(Lr or S)
1.2D+1.6(Lr or S)+(L or 0.8W)
1.2D+1.6W+L+0.5(Lr or S)
1.2D+1.0E+L+0.2S
0.9D+1.6W
0.9D+1.0E

Load cases play a much bigger role when lateral forces are introduced. Typically 1.2 Dead + 1.6 Live controls gravity design. Depending on wind and earthquake effects, new load combinations can provide a worst-case design scenario. 0.9 Dead + 1.6 Wind is a notable combination where high wind loads are expected. The lateral loading on this building is almost exclusively controlled by the equivalent seismic effects. That makes 0.9 Dead + 1.0 Earthquake an important combination. After brief inspection, the case that looks to most likely control design will be 1.2 Dead + 1.0 Earthquake + 1.0 Live load + 0.2 Snow load. This will be checked in all analysis conditions.

Wind load

Velocity = 90 mph

Exposure class B

Importance factor (II) = 1.0

Effective Building Size; B=200' L=200' h=128.56'

Kzt = 1.0, Kd = 0.85

Gust Factor = 0.822

Storey	Kz	qz (psf)	qh (psf)
Roof	1.098	19.35291	19.35291
12	1.070	18.85939	19.35291
11	1.040	18.33062	19.35291
10	1.010	17.80186	19.35291
9	0.978	17.23784	19.35291
8	0.942	16.60332	19.35291
7	0.898	15.82779	19.35291
6	0.850	14.98176	19.35291
5	0.800	14.10048	19.35291
4	0.736	12.97244	19.35291
3	0.652	11.49189	19.35291
2	0.570	10.04659	19.35291
Base	0.570	10.04659	19.35291

Design method 2 was used because this building does not meet the criteria for simplified design. Also the building dimensions are essentially equal in both directions, and therefore results apply to both N-S and E-W directions. The surface area in N-S and E-W are the same, but the lateral resisting frames are different, so lateral load effects will be checked in each direction.

Seismic loads

Sds = 0.143

Sd1 = 0.0713

Seismic use group 1

Seismic design category B

Site class C

Cd = 2.5

Response modification; R = 3

Cs = 0.021 (both N-S and E-W)

System over-strength factor = 3

The structure was considered to be ordinary reinforced concrete moment frames when using the equivalent lateral force procedure. Response modifications, site class, seismic use group, and accelerations were checked in conjunction with ASCE 7-02 and design values used by the building engineers.

Storey	Wx (kips)	hx (ft)	Wxhx	Cwx
12	6848	128.59	5652404	0.159913
11	6848	116.9	4954437	0.140166
10	6848	105.21	4282719	0.121163
9	6848	93.52	3639020	0.102952
8	6848	81.83	3025461	0.085594
7	6848	70.14	2444645	0.069162
6	6848	58.45	1899861	0.053749
5	6848	46.76	1395442	0.039479
4	6848	35.07	937439.5	0.026521
3	6848	23.38	535105.5	0.015139
2	6848	11.69	205195.1	0.005805

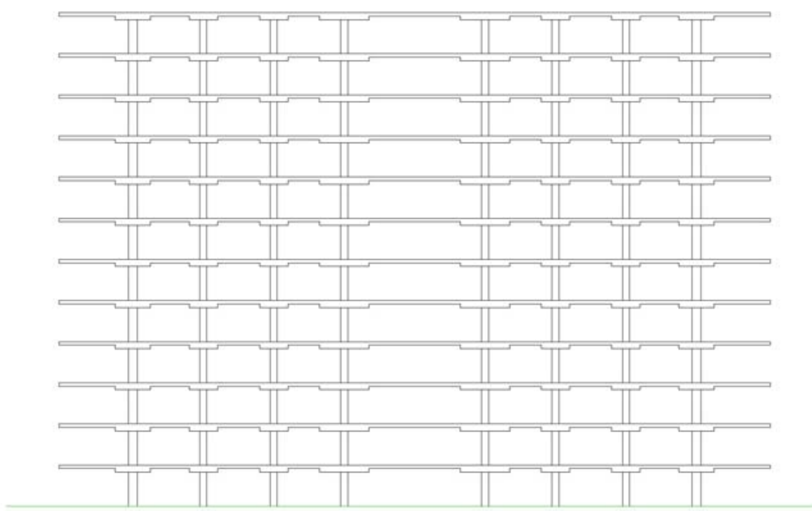
Lateral load summary

	Shear				Storey Force			
	E-W		N-S		E-W		N-S	
Storey	Wind	Seismic	Wind	Seismic	Wind	Seismic	Wind	Seismic
12	21.46	292.99	21.46	292.99	21.46	292.99	21.46	292.99
11	63.64	552.77	63.64	552.77	42.17	259.78	42.17	259.78
10	105.00	780.47	105.00	780.47	41.36	227.70	41.36	227.70
9	145.54	977.29	145.54	977.29	40.55	196.83	40.55	196.83
8	185.22	1144.54	185.22	1144.54	39.68	167.24	39.68	167.24
7	223.92	1283.58	223.92	1283.58	38.70	139.05	38.70	139.05
6	261.44	1395.94	261.44	1395.94	37.51	112.35	37.51	112.35
5	297.65	1483.25	297.65	1483.25	36.21	87.31	36.21	87.31
4	332.50	1547.38	332.50	1547.38	34.86	64.13	34.86	64.13
3	365.63	1590.47	365.63	1590.47	33.12	43.08	33.12	43.08
2	396.47	1615.06	396.47	1615.06	30.85	24.59	30.85	24.59
Base	425.10	1624.49	425.10	1624.49	28.63	9.43	28.63	9.43
Base Moment	33882.76		33882.76		425.10	1624.49	425.10	1624.49

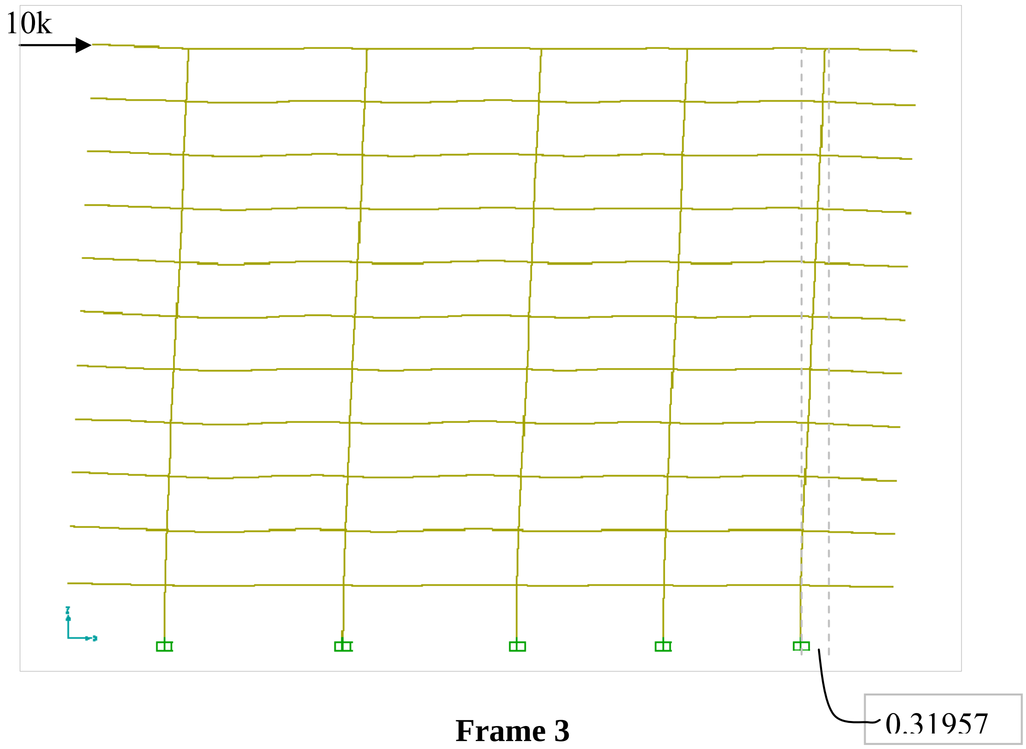
*all values in K and Ft-k

Distribution/Analysis of Lateral Loads

As was determined earlier, 4 main frames will be considered for load resistance. In each direction there are 2 frames that will take the brunt of the lateral loads. This will simplify both lateral load dissipation and torsion calculations. In addition a full ETABS model will be constructed to account for the horizontal load carrying capacity of the entire structure. A section through the front moment frame (#4 in above figure) is shown below.



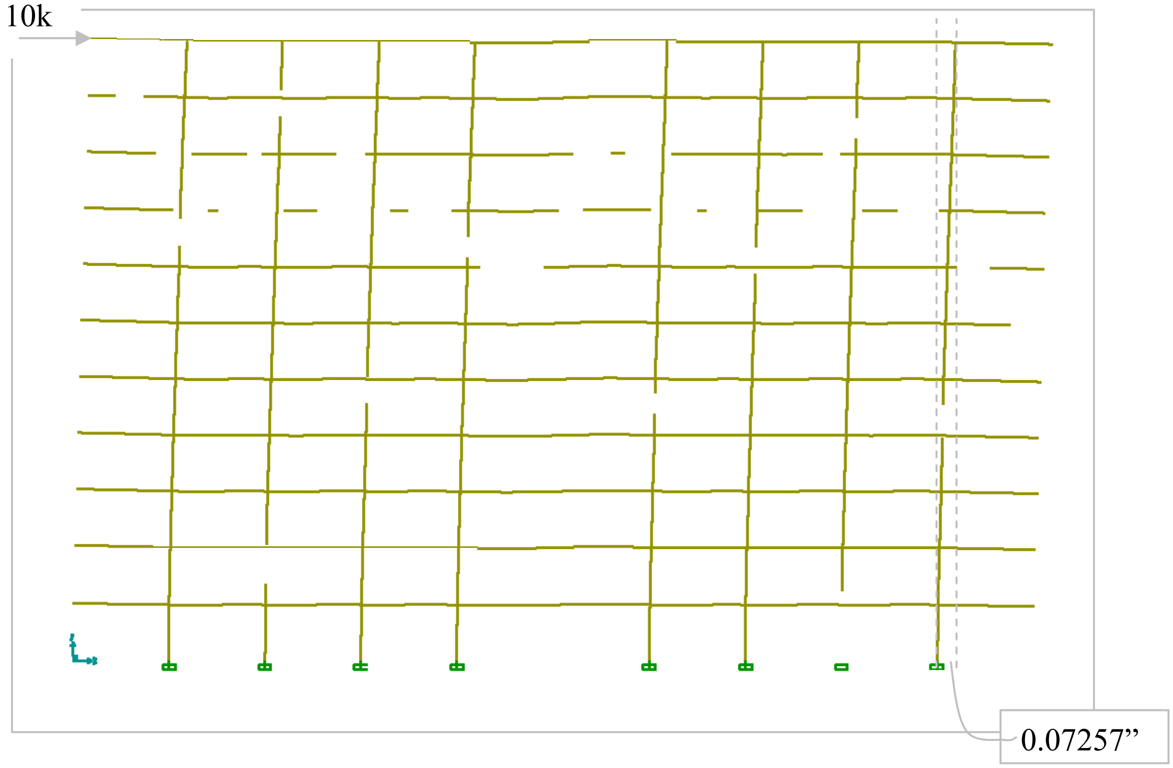
Frames 3 and 4 were modeled individually using SAP2000. These two frames share the lateral load in the east west direction. The models were exposed to a 10k load applied on the uppermost storey. The frames were also modeled with full gravity and lateral loads to determine interior member forces and the stress distribution path. Using SAP2000, the total frame deflection under the 10k unit load was calculated. This in turn is used to determine frame stiffness using the equation ki=P/Δ. The deflected frames are shown next.



Frame 3

Frame 3, along column line F.7 experienced a 10k load. The resulting deflection was 0.31957 inches in the direction of the applied load. This results in a total frame stiffness of 10k/0.31957" = 31.292. Individual storey drifts were calculated based on full gravity and lateral loads and are summarized in the following table.

Frame 3 Storey Drifts				
Storey	storey ht (ft)	Drift (ft)	L/x ratio	meets L/240?
12	11.69	0.0015	7793	yes
11	11.69	0.0036	3247	yes
10	11.69	0.0086	1359	yes
9	11.69	0.0054	2164	yes
8	11.69	0.0096	1217	yes
7	11.69	0.0177	660	yes
6	11.69	0.0241	485	yes
5	11.69	0.0298	392	yes
4	11.69	0.031	377	yes
3	11.69	0.0271	431	yes
2	11.69	0.0098	1192	yes

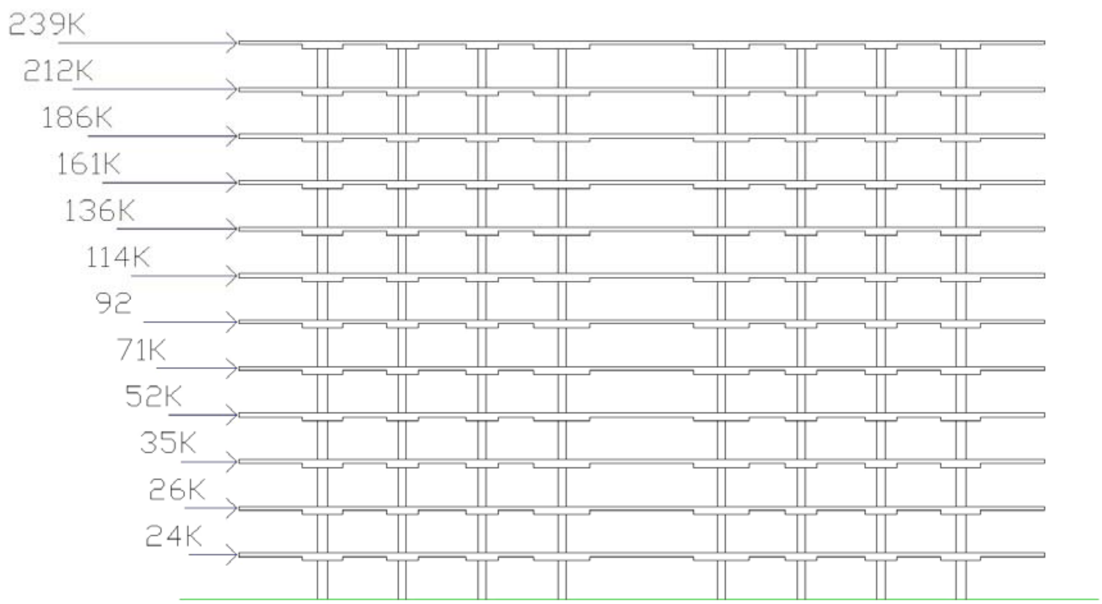


Frame 4

Frame 4 accounts for all columns at column line A.1. This frame is longer, and incorporates 8 columns instead of just 5, thus making it much stiffer than frame 3. The resulting displacement from the 10k unit load was 0.07257". This makes for a stiffness of 10k/0.07257"=137.797. The storey drifts were also calculated, and are shown below.

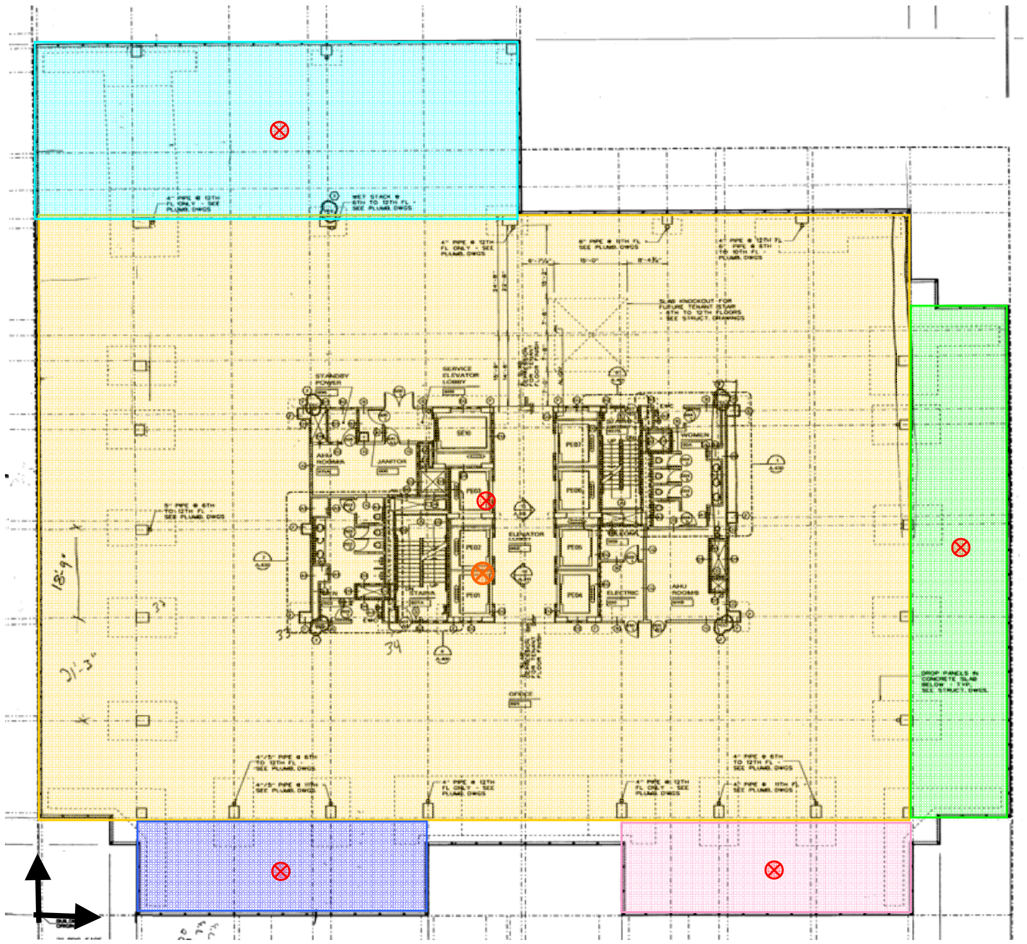
Frame 4 Storey Drifts				
Storey	storey ht (ft)	Drift (ft)	L/x ratio	meets L/240?
12	11.69	0.0048	2435	yes
11	11.69	0.0016	7306	yes
10	11.69	0.0002	58450	yes
9	11.69	0.0001	116900	yes
8	11.69	0.0002	58450	yes
7	11.69	0.0004	29225	yes
6	11.69	0.0056	2087	yes
5	11.69	0.0058	2015	yes
4	11.69	0.0002	58450	yes
3	11.69	0.0006	19483	yes
2	11.69	0.001	11690	yes

The relative stiffness of each moment frame dictates how the lateral loads will be distributed to each frame. The total stiffness (of the frames considered) in the E-W direction is 31.292 + 137.797 = 169.098. The percent distributed to frame 3 is (31.292/169.098)x100 = 18.5%. The amount that goes to the stiffer frame, frame 4, is (137.797/169.098)x100 = 81.5%. Between frame 3 and 4, 100% of the lateral load is resisted, and within storey drift serviceability limitations. The proportioned loads to frame 4 are shown below in elevation. A similar procedure is required for the frames in the N-S direction. This investigation is omitted in this instance due to the time required to create further models.



Torsion

The main lateral resisting frames are located at some distance from the centroid of the building. This results in an eccentricity of the applied lateral loads and creates torsion about that centroid. The torsional effects created will induce additional shear forces into the moment frames. This is more of a problem when you have shear walls that only resist shear in one direction. Moment frames are capable of resisting in all directions and therefore mitigate the effect of torsion better. The net moment due to torsion on the building will be very small because of the geometry. The primary moment frames are located on the perimeter of the building, roughly equidistant from the center of the building's mass. This resulting net moment is very small and can be counteracted by an correspondingly small couple. A small couple will add negligible shear to the moment frames. The calculation of the center of mass/ center of geometry is shown below, to provide visual evidence of the stated case.



Center of mass/Geometric center (all dimensions in feet)

Ybar = Σ y(area) / Σ area

Ybar = 164(32x100)+82(125x180)+72.5(105x20)+10(20x60)+10(20x60) / 30200

Ybar = 70.40 ft

Xbar = Σ x(area) / Σ area

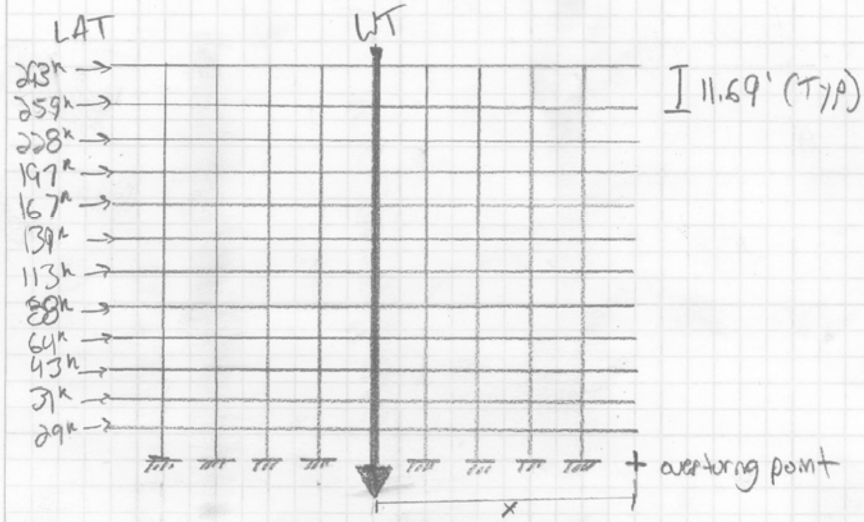
Xbar = 50(32x100)+90(125x180)+190(105x20)+50(20x60)+150(20x60) / 30200

Xbar = 89.27 ft

Overturning

Lateral loads create moments about the outside toe of the building. This moment can create an overturning risk if not properly resisted. Typically this moment is resisted by the weight of the building itself, but in some cases can also be resisted by friction on deep foundation systems and soil bearing pressure above foundations. Overturning is more of a problem for light, tall buildings with a smaller footprint. This building is wider than it is tall, and made entirely of reinforced concrete. The building has a great deal of dead load to resist overturning. The moment due to lateral loads was found to be 58 times smaller than the resisting moment due to structure weight. This allots a very comfortable margin of safety, making overturning a non-issue.

OVERTURNING/UPLIFT



Building wt = 76,332 Kips (from seismic spreadsheet)

Lateral overturning moment

$$243(128.59) + 259(116.9) + 228(105.21) + 197(93.52) + 167(81.83) + 139(70.14) + 88(58.45) + 64(46.76) + 43(35.07) + 31(23.38) + 29(11.69)$$

$M_{Lat} = 144492.57$ ft-k

resisting moment $x = 110'$ from centroid

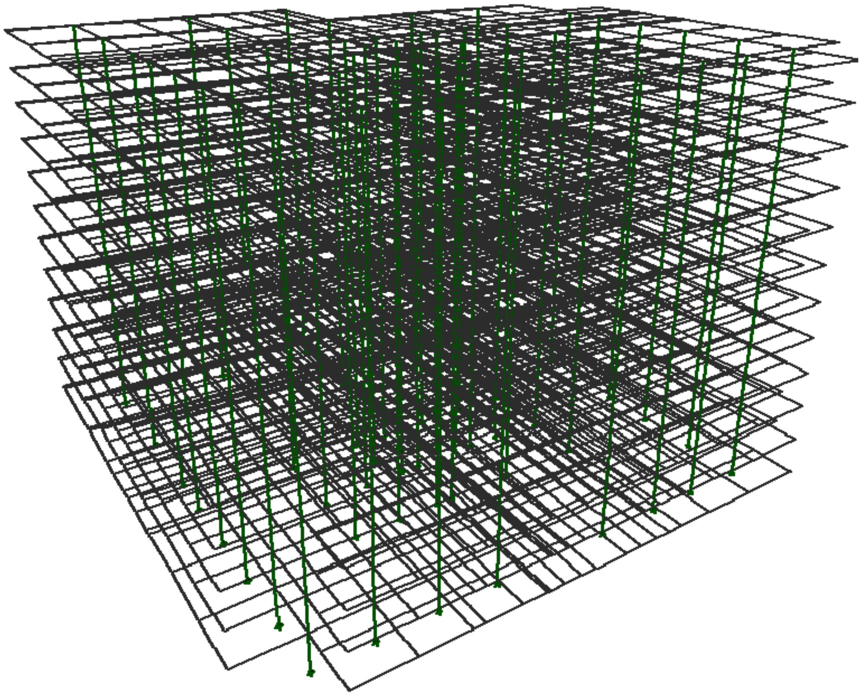
$$76332k\,(110)$$

$M_{res} = 8396520$ ft-k

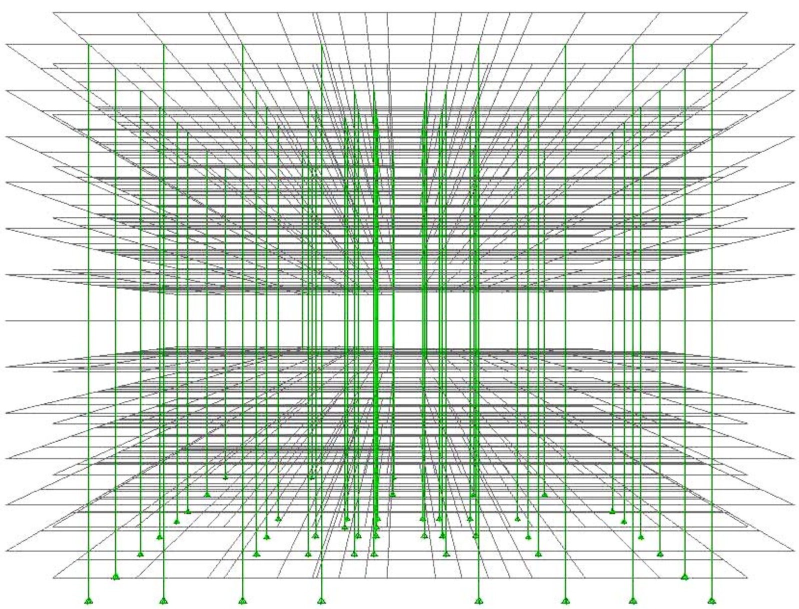
8396520 ft-k $\gg 144492$ ft-k

No overturning risk

A full ETABS model was created in order to accurately distribute lateral loads to each member of the frame. Because of the large number of individual components able to resist lateral effects, this was very necessary. The model aided in the analysis and design check of the structure. Another benefit of creating an ETABS model was the automatic application of wind and earthquake loading. The storey forces and shears were determined by hand earlier and this served as a good way check accuracy of assumptions made. The hand calculated numbers were also entered into ETABS and applied to the model, and compared to the automatically calculated lateral loads. The resulting forces from ETABS were surprisingly close the hand calculated values. ETABS calculates its wind loads based on ASCE 7-98 and earthquake loads by IBC 2000. The method used in hand calculation was ASCE 7-02 for both wind and earthquake. SAP2000 uses ASCE 7-02 for wind analysis and IBC 2003 for earthquake. The planar frames were created in SAP2000 and the full 3D structure was created in ETABS 8. The resulting model is shown below. For design simplicity, the model takes a few liberties with the actual design. Drop panels are omitted, so slab bending and column punching shear is not applicable. Columns were modeled as accurately as possible, taking care to orient them correctly. Similar columns were assumed to be identical. Some columns received an increase in f'c where the majority of other columns were dissimilar, to enable a timely creation of the model.



North-West Isometric



Looking North

Base Reactions

As expected, the 1.2 Dead + 1.0 Seismic + 1.0 Live + 0.2 Snow was the controlling case for base reactions. The base reactions at each of the foundation points are listed below. Due to the considerable amount of data, it is unreasonable to create a model showing all the base reactions. (table continues onto next page).

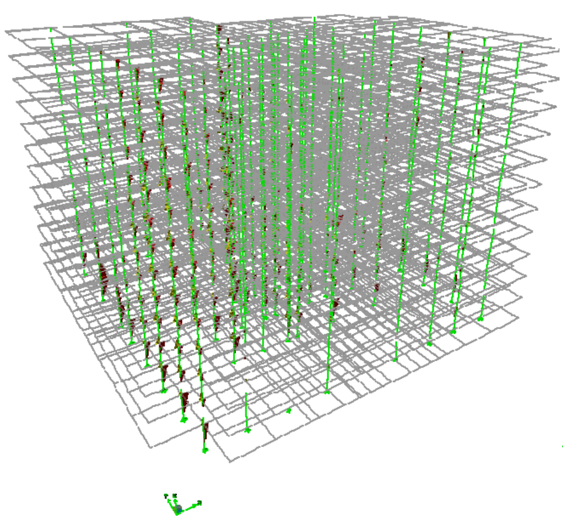
Story	Point	Load	FX	FY	FZ
BASE	1	12D1Q1L2S	-82.12	-6.08	1794.48
BASE	2	12D1Q1L2S	-77.43	-12.62	3256.46
BASE	3	12D1Q1L2S	-87.76	-6.63	3635.71
BASE	4	12D1Q1L2S	-70.51	-13.89	6077.04
BASE	5	12D1Q1L2S	-111.39	-23.26	5726.25
BASE	6	12D1Q1L2S	-84.21	-24.4	3280.28
BASE	7	12D1Q1L2S	-82.4	-38.36	2881.42
BASE	8	12D1Q1L2S	-48.05	-17.24	2960.56
BASE	9	12D1Q1L2S	-112.16	10.84	2618.53
BASE	10	12D1Q1L2S	-78.3	0.95	3496.23
BASE	11	12D1Q1L2S	-110.73	8.74	2919.04
BASE	12	12D1Q1L2S	-143.29	0.34	2922.92
BASE	15	12D1Q1L2S	-98.93	-2.04	756.15
BASE	16	12D1Q1L2S	-92.2	-2.42	2204.13
BASE	18	12D1Q1L2S	-138.87	-3.33	2902.37
BASE	19	12D1Q1L2S	-120.19	-12.13	3747.47
BASE	22	12D1Q1L2S	-78.91	-0.4	3736.36
BASE	24	12D1Q1L2S	-90.35	14.33	2788.88
BASE	25	12D1Q1L2S	-120.83	15.68	1983.27
BASE	26	12D1Q1L2S	-92.5	9.5	2961.19
BASE	27	12D1Q1L2S	-121.88	30.09	4132.88
BASE	28	12D1Q1L2S	-193.63	18.92	6541.44
BASE	29	12D1Q1L2S	-87.12	3.6	2855.74
BASE	30	12D1Q1L2S	-59.34	-28.84	4100.82
BASE	32	12D1Q1L2S	-84.73	44.92	4889.86
BASE	33	12D1Q1L2S	-288.69	-15.47	5983.57
BASE	34	12D1Q1L2S	-63.74	-3.71	1760.24
BASE	35	12D1Q1L2S	-57.13	-13.83	2430.76
BASE	36	12D1Q1L2S	-81.89	-13.16	2631.23
BASE	37	12D1Q1L2S	-101.02	15.22	3710.29
BASE	38	12D1Q1L2S	-67.56	-3.07	3433.62
BASE	39	12D1Q1L2S	-73.59	8.94	3568.32
BASE	40	12D1Q1L2S	-101.52	5.88	2771.55

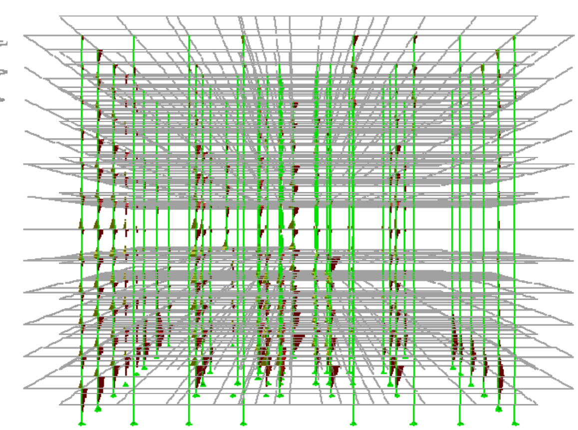
BASE	41	12D1Q1L2S	-77.42	10.99	3830.61
BASE	42	12D1Q1L2S	-22.55	8.58	1832.54
BASE	43	12D1Q1L2S	-22.63	0.82	1115.79
BASE	44	12D1Q1L2S	-68.13	1.32	1343.53
BASE	45	12D1Q1L2S	-66.65	1.82	658.92
BASE	46	12D1Q1L2S	-88.85	4.89	990.37
BASE	47	12D1Q1L2S	-87.93	5.19	1920.73
BASE	49	12D1Q1L2S	-65.42	1.85	204.25
BASE	50	12D1Q1L2S	-75	0.63	1174.66
BASE	51	12D1Q1L2S	-70.91	1.22	948.32
BASE	52	12D1Q1L2S	-66.09	0.77	196.4
BASE	53	12D1Q1L2S	-123.27	11.63	2370.71
BASE	54	12D1Q1L2S	-77.37	6.44	1897.72
BASE	58	12D1Q1L2S	-28.91	-2.12	1439.43
BASE	59	12D1Q1L2S	-21.29	-1.11	681.07

The footing reactions are very reasonable and can easily be resisted by the spread base bearing on allowable soil pressure of 12000 psf.

Column Check

The column stresses were determined from the ETABS analysis output. The model showing member stresses is shown below.





Using the member end forces that ETABS has provided me, I checked the column for bending and axial forces using the PCA interaction diagram. The column reinforcement was found to be (12) #8 bars. The actual column is reinforced with (12) #11 bars. This could be to an oversight in my loading or strength calculations.

COLUMN CHECK

STOREY 1
30"x30" COLUMN
$f'_c = 10$ ksi

$A_T = \frac{(20'\times20')+(20'\times20')}{2}$

$A_T = 800$ sq ft

$K_{LL} = 4$ $A_T = 800(4) = 3200$ sq ft

LL Reduction

$L = L_o(.25 + \frac{15}{\sqrt{A_T}})$

$L = .51 L_o$

$P_u = 606^k$
$.960^k$ 2.14^k

$h = 11.69'$

$M_x = 11.2$ ft k $M_y = 25.0$ ft-k
$P_u = 557^k$

As req'd = 9.48 in² use (12) #8 equally distributed

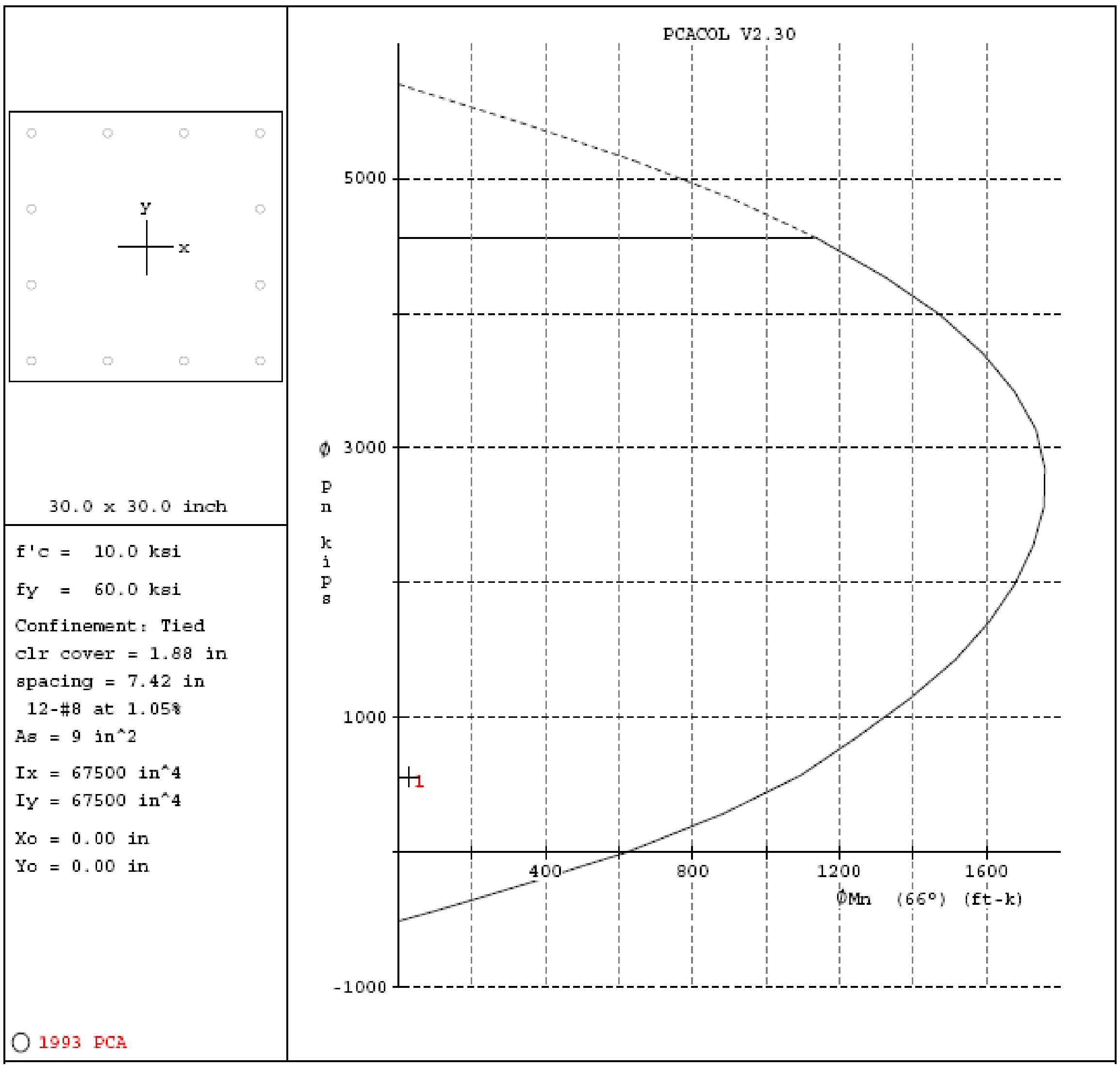
Plot on interaction diagram

LOAD
$1.2D + 1.0Q + 1.0L + 0.2S$
$1.2(300) + 1.0(140) + 1.0(100) + .2(3$
$360 + 140 + 100 + 6$

$P_u = 606$ k

$P_u = 360 + 140 + .51(100) +$

$P_u = 557$ k


PCACOL V2.30
30.0 x 30.0 inch
f'c = 10.0 ksi
fy = 60.0 ksi
Confinement: Tied
clr cover = 1.88 in
spacing = 7.42 in
12-#8 at 1.05%
As = 9 in^2
Ix = 67500 in^4
Iy = 67500 in^4
Xo = 0.00 in
Yo = 0.00 in
1993 PCA
y
x
φ Pn kips
5000
3000
1000
-1000
400
800
1200
1600
φMn (66°) (ft-k)
1

Conclusion

The designed lateral load resisting system for this office building in downtown DC seems to be adequate for the loads. Lateral loads were attributed to the effects of wind and seismic base accelerations. Several load combinations of the aforementioned loads were considered in ultimate strength design. Service load deflections were checked and easily passed L/240 storey and L/400 total deflection limitations. Overturning and torsion were not found to be problems with this building design. All members seemed to be adequate to carry applied loads, and the full ETABS model confirmed this.