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Lexington II, Washington D.C.
Technical Report #1
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Executive Summary:

Structural Concepts/ Existing Conditions Report is an initial investigation into the structural system of 12-story Lexington II residential tower at Market Square North in Washington D.C. For this report, a detailed look will be taken at the structural system of Lexington II, its primary features, design code, and loadings. This report will also check several members for structural integrity.

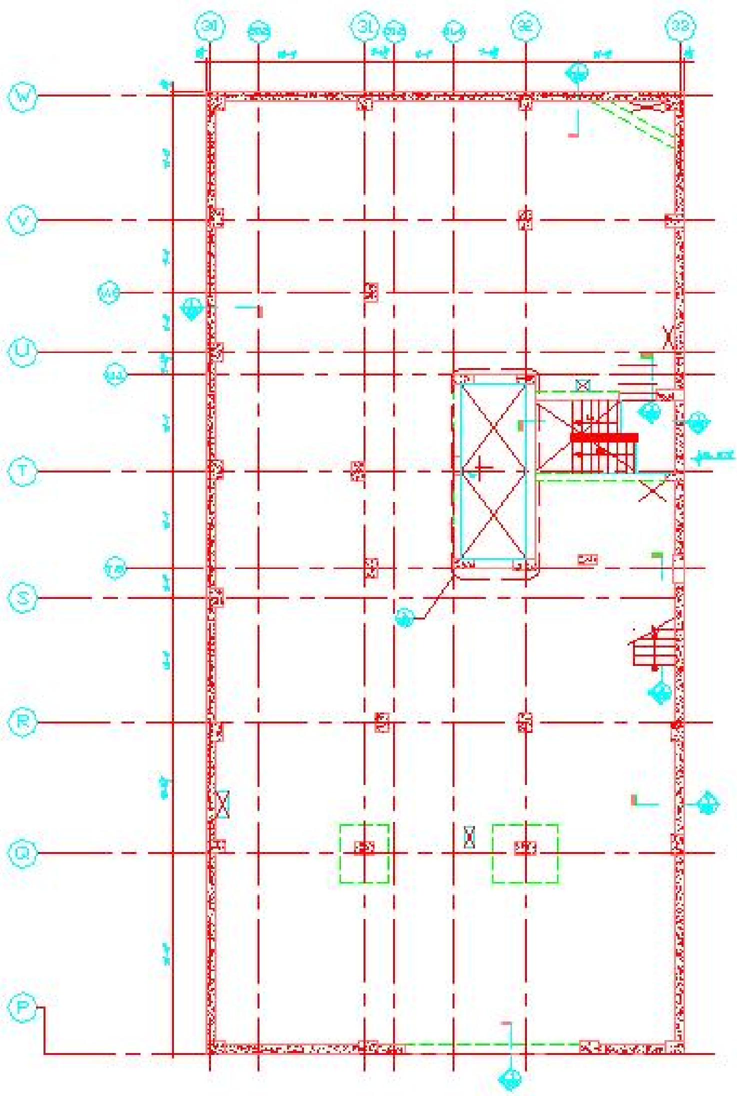
The structural system for Lexington II is cast in place concrete. The foundation is a MAT foundation resting on original soil. All floor systems throughout Lexington II are 2-way slabs which support the required gravity load. A grid of concrete columns is used to transfer the gravity loads from the floor slabs to the foundation. A typical bay size would be 13.5' by 16.6', however many columns are offset from this grid. A core of several small shear walls is located around the elevator shafts. These shear walls counteract the lateral loads on the building.

When the Lexington was designed for completion in 2002, the primary building code used was the 1996 edition of the BOCA code. For my report, loading was recalculated using a more recent code, ASCE7-02. The dead load, live load, roof live load, snow load, wind load, and seismic loads were found. The live load I calculated was very comparable to the one used in the original design of the building. I was unable to compare the other loadings to the original design load due to lack of information about the original design load.

The final part of my investigation was to spot check several structural members for gravity and lateral loading. My results were similar to the original design of the structure and often proved why there is additional top rebar added to most of the structural drawings. My spot checks also proved that the shear walls do not carry the complete lateral load, but instead the columns spaced throughout the building are strong enough to help support the building against the lateral load.

Structural System:

Lexington II is a 12-story residential tower located as part of the Market Square North complex in the historical Penn Quarter of Washington D.C. The structural system is composed of 2-way slab systems resting on columns. The columns are then sitting on a MAT foundation. All concrete is normal weight with a compressive strength at 28 days of 4000psi. The MAT foundation is made of 5000 psi concrete. The general layout of Lexington II is 3 bays by 7 bays with an average bay size of 13.4’ by 16.6’. Several columns are offset changing the bay sizes slightly. All of the structural elements in Lexington II are cast in place concrete with 60psi reinforcing steel.



At the southern end of the building is additional structural support in the form of drop panels, shear caps, and edge beams along some of the bays.

The lateral load is carried by small shear walls which are located at the core of the building surrounding the elevator shaft. The shear walls span the entire height of the building.

Foundation:

The foundation of Lexington II is a 3'-6" thick MAT foundation which is reinforced with deformed #8 bars located every 9" o.c. The MAT foundation is also reinforced with #11 top bars in some locations and designed in a 2-way slab formation. Below the MAT foundation is a 3" subgrade working MAT. The foundation rests on original soil and structural fill with a compressive strength of 8000psf. Along the southern wall of Lexington II, the foundation rests on HP 14 x 89 piles every five feet on center with one inch cap plates. The piles are in place because the pre-existing building to the south of Lexington II (which Lexington II abuts) is a story lower. Rather then undermining the existing building's foundation, piles were installed as an alternative to providing control fills stepped up to the new foundation level (which is more costly).

The below grade walls are also reinforced concrete, which is 14" thick from level P1 until the concourse level where they are reduced to 12" until ground level is reached. Both the concrete walls and the MAT foundation have a compressive strength of 5000 psi. The reinforcing steel in both the MAT foundation and the below grade walls is ASTM A615, grade 60.

Floor System:

The levels of Lexington II are 2-way slab systems of concrete with a compressive strength of 4000psi. The floors of the 3 level substructure are 10" thick while the superstructure has floors that are 8" thick. The exceptions to this are 5" drop panels around the southern columns of the concourse level, and a 10" slab at the south end of the ground floor. The drop panels are bending drops which are in place to provide for the greater flexural and shear load caused by an increased live load on the concourse level. Similarly, the 10" thick slab localized to the south end of the ground floor is a loading area which will have an additional weight of trucks needing to access the retail portions of the building.

The majority of the bays have 2-way flat-plate slabs with no edge beams. However, edge beams can be found on the lower levels where the live load is increased. Edge beams are also in place along the east exterior bays.

The 2-way slab is reinforced with continuous bottom mat of #4 bars 12 inches on center. Theses bars are ASTM A216, grade 60. In addition to the #4 @ 12" mat, there is top reinforcing in some locations. Typically the top reinforcing are #4 or #5 bars, and are often found by columns and shafts cut into the slab where a stronger moment would be found.

Columns:

Supporting the 2-way slab is a grid of columns. All of the columns throughout Lexington II are 5000psi compressive strength concrete with ASTM A615 grade 60 reinforcement. Columns range in size from 14" x 14" columns reinforced by 4 #9 bars to 42" x 14" columns reinforced with 18 #11 bars. As expected, the larger columns are in the lower stories of the building which carry the building's entire weight.

Columns are located approximately every 16.2' in the west to east direction, and every 13.5' north to south. Several columns throughout the building are offset from the grid pattern. These offset columns coincide with vertical shafts (such as elevators and trash chutes) which run the length of the building and where extra support for the two-way slab is needed.

Lateral System:

The lateral loads which affect the building are counteracted by shear walls located around the elevator shafts of the building. All shear walls in Lexington II are 12" reinforced concrete with a compressive strength of 4000psi. The reinforcement in the shear walls are #4 bars placed every 12" on center.

Material Strengths:

Concrete:

MAT foundation……………………….5000psi
Columns……………………………...5000psi
Basement Walls……………………...5000psi
2-way floor slabs…………………….4000psi
Shear walls…………………………..4000psi
Beams………………………………..4000psi

Reinforcing steel:

Bar reinforcing………….ASTM A-615, grade 60……………...60psi
Welded Wire Mesh……..ASTM A-185

Existing Code:

Lexington II was designed following the **1996 edition of the BOCA** code.

Other design codes used while designing Lexington II include:

ACI 318-95	Reinforced Concrete
AISC- 9th Ed.	Structural Steel (design, fabrication, and erection)
AWS D1.1-98	Structural Welding
NDS, 1991	Wood Construction
ACI 530-95/ ASCE 5-96	Masonry

Loading:

Dead Load: (Appendix)

- Substructure Slab (10")…………….125psf
- Superstructure Slab (8")……………100psf
- Mechanical/ Lighting…………………5psf
- Finishes……………………………...15psf
- Partitions……………………………included in live load, see below

Live Load:

Lexington II was designed following the loading as prescribed by the 1996 edition of the BOCA code. The engineers assumed the following live loads:

- Roof……………………………….. 30psf
- Ground, L1, and P1 level stairs……100psf
- Mechanical Rooms………………...150psf
- Lobbies…………………………….100psf
- Concourse level……………………225psf
- Residential Levels…………………..60psf + 20psf (for partitions)

For my report, I will be using a more recent code, ASCE7-02. Live loads obtained from ASCE 7-02 are comparable with those used in the building's original design

- Roof………………………………...20psf (see calculations below)
- Public Levels/ Stairs………………100psf (ASCE7-02)
- Mechanical………………………..150psf (Common assumption)
- Lobbies……………………………100psf (ASCE&-02)
- Residential Levels…………………40psf + 20psf (for partitions)

Roof Live Load: (Appendix)

R_1 = .992
R_2 =1

L_r=20*(.992) (.1)
= 20psf

Snow Load: (Appendix)

C_e = .9	(Table 7.2, B-urban, partially exposed)
C_t = 1	(Table 7.3)
I = 1	(Table 7.4)
P_g = 25psf	(Fig. 7-1)

P_f= .7*(.9)(1)(1)(25) = 15.75psf

Wind Load:

Basic Wind Speed	V= 90mhp	(Fig. 6-1)
Wind Direction const.	K_d =.85	(Table 6-4)
Importance Factor	I = 1	(Table 6-1)
Topical Factor	K_{zt} =1	(assume not on a hill)
Wind Exposure	K_z = varies by height (Table 6-3)	
Velocity Pressure	q_z = 17.6256 K_z	
Gust Factor	G = .86	(Appendix)
External Pressure Coef.	C_p= .8 windward	C_p= -.5 leeward

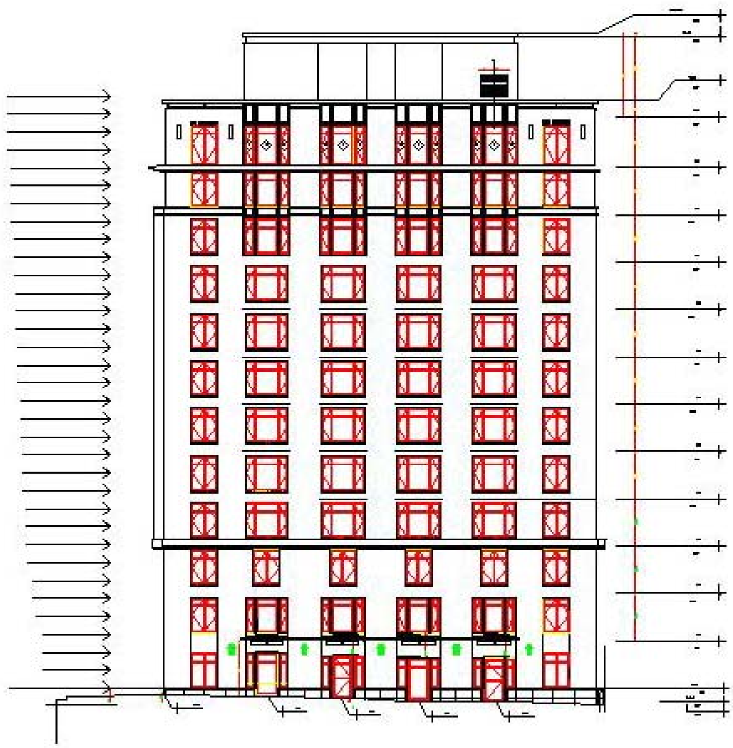
Design Pressures:

N/S

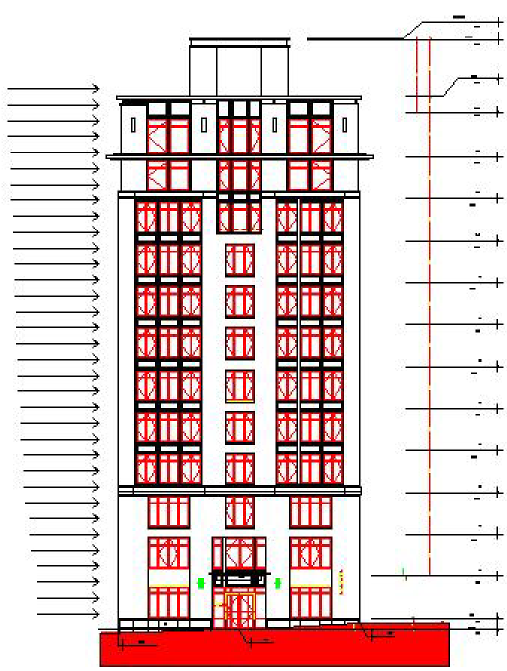
Height	Kz	qz	P(windward)	P(leeward)	Pnet (Psf)
0-15	0.57	10.04659	6.9120553	-8.342	15.25406
20	0.62	10.92787	7.51837594	-8.342	15.86038
25	0.66	11.6329	8.00343245	-8.342	16.34543
30	0.7	12.33792	8.48848896	-8.342	16.83049
40	0.76	13.39546	9.21607373	-8.342	17.55807
50	0.81	14.27674	9.82239437	-8.342	18.16439
60	0.85	14.98176	10.3074509	-8.342	18.64945
70	0.89	15.68678	10.7925074	-8.342	19.13451
80	0.93	16.39181	11.2775639	-8.342	19.61956
90	0.96	16.92058	11.6413563	-8.342	19.98336
100	0.99	17.44934	12.0051487	-8.342	20.34715
120	1.04	18.33062	12.6114693	-8.342	20.95347
140	1.09	19.2119	13.21779	-8.342	21.55979
160	1.13	19.91693	13.7028465	-8.342	22.04485

E/W

Height	Kz	qz	P(windward)	P(leeward)	Pnet
0-15	0.57	10.04659	10.0707	-7.29246	17.36316
20	0.62	10.92787	10.9541	-7.29246	18.24656
25	0.66	11.6329	11.66081	-7.29246	18.95327
30	0.7	12.33792	12.36753	-7.29246	19.65999
40	0.76	13.39546	13.42761	-7.29246	20.72007
50	0.81	14.27674	14.311	-7.29246	21.60346
60	0.85	14.98176	15.01772	-7.29246	22.31018
70	0.89	15.68678	15.72443	-7.29246	23.01689
80	0.93	16.39181	16.43115	-7.29246	23.72361
90	0.96	16.92058	16.96119	-7.29246	24.25365
100	0.99	17.44934	17.49122	-7.29246	24.78368
120	1.04	18.33062	18.37462	-7.29246	25.66708
140	1.09	19.2119	19.25801	-7.29246	26.55047
160	1.13	19.91693	19.96473	-7.29246	27.25719



Wind loads applied to the North-South Direction



Wind loads applied to the West-East Direction

Seismic Loading:

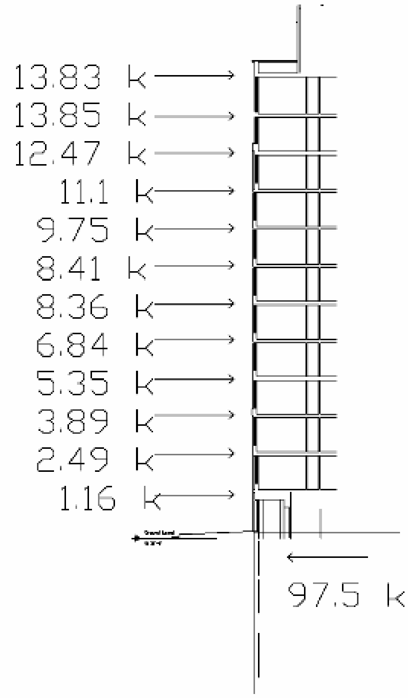
Seismic Use Group	I	(Table 9.1.3)
Importance Factor	I=1	(Table 9.1.4)
Max Ground Motions	S_s= .187	(Fig. 9.4.1.1a)
	S_1=.063	(Fig. 9.4.1.1b)
Site Class	C	(Sec. 9.4.2.4)
Site Class Factors	F_a=1	(Table 9.4.1.2.4a)
	F_v=1.3	(Table 9.4.1.2.4b)

S_{ms}=.187 S_{mi}=.0819
S_{ds}=.125 S_{d1}=.0546

Seismic Design Category	A	(Table 9.4.21a and b)
Response Mod. Factor	R=5	(Table 9.5.2.2)
	W_o=2.5	(Table 9.5.2.2)
	C_d=4.5	(Table 9.5.2.2)
Seismic Response Coef.	C_t=.02	(Table 9.5.5.3.2)
	X= .75	(Table 9.5.5.3.2)

Shear Vx= 195

Floor	Height	Floor Load	exterior wall	Total load (kips)	wxhx^k	Cvx	Fx (kips)	Vx (kips)	Mx (ft kips)
roof	114	373870	42750	416.62	77358	0.14	13.83		1577
12	104.5	373870	85500	459.37	77490	0.14	13.85	13.83	1448
11	95	373870	85500	459.37	69758	0.13	12.47	27.68	1185
10	85.5	373870	85500	459.37	62104	0.11	11.10	40.15	949
9	76	373870	85500	459.37	54538	0.10	9.75	51.26	741
8	66.5	373870	85500	459.37	47069	0.09	8.41	61.01	560
7	57	455492.5	85500	540.99	46765	0.09	8.36	69.42	477
6	47.5	455492.5	85500	540.99	38246	0.07	6.84	77.78	325
5	38	455492.5	85500	540.99	29901	0.05	5.35	84.62	203
4	28.5	455492.5	85500	540.99	21771	0.04	3.89	89.97	111
3	19	455492.5	85500	540.99	13921	0.03	2.49	93.86	47
2	9.5	455492.5	85500	540.99	6481	0.01	1.16	96.35	11
1		455492.5	85500	540.99	0	0.00	0.00	97.51	0
					545402				
			Total Load (kips)	6500.42				Mt (kips)	7633



Spot Checks:

For my spot checks, I have chosen to analysis the existing design for both a 2-way slab carrying gravity loads, and a shear wall carrying lateral loads.

Gravity Check:

2-way Slab:

In order to check the building system for gravity loads, I first decided to check a 2-way slab bay. In truth, there is no typical interior bay to my building; all bays are either affected by an offset column or mechanical shaft running through the building. In order to simultaneously check the majority of the building bays for gravity loads, I assumed a typical bay size of 13.4' x 16.6'. This size would be the average size of a bay, had columns been located on the column lines and not offset.

I used the direct design method to check the 2-way slab system. My 'typical' bay met all of the code requirements and therefore I was able to use ACI equation 13-3 for the static moment, rather then moment distribution. That is to say that there are 3 or more bays in each direction and that the column offsets are never greater then 10% of the span length. It also means that the length to width ratio of a bay and the length to length ratio of

continuous bays meet certain requirements. Even with the offset columns in my building, all of the criteria were met.

My gravity load check proved that an 8" slab meets the minimum thickness as set by the ACI code. I also showed that minimum steel area and spacing as set forth by the ACI code are met in the original design of #4 bars 12" on center. I then did a moment check to make sure the loads could be sustained. During this check I found that the #4 at 12" on center met all moments on the slab except for the interior moment on the column strip. To meet this moment I had to increase my bar size to #5. The original design accounts for this by adding top reinforcement in the area local to all of the column locations. This follows my calculations exactly. I did this check for both an interior and exterior bay.

Interior Column (6^{th} and Concourse levels):

My second gravity load check was to ensure that the columns could support the loads applied to them. I calculated the loads and tributary area acting on each column of a typical bay. I then solved for the max axial load which could be applied to a 5000psi column at the dimensions designed could support. I checked both a column on the 6^{th} floor, and a column on the concourse level which has a higher live load. Both columns were very over designed, however this is due to the fact that in reality there would not be a pure axial load on a column.

Lateral Load:

Shear Wall:

In my building, I assumed that the shear walls carry the entire lateral load; therefore my spot check was for the shear wall bearing capacity versus the critical lateral loads applied to my building.

To determine critical lateral loads on my building, I compared wind loading and seismic point loads for each floor level and chose the more critical case. In both directions, N/S and E/W, the critical case varied by floor between wind loading and seismic loading. Once the critical cases were found, I summed the shears and moments to find the base shear and overturning moment caused by the critical loading case.

To spot check the shear walls, I followed ACI 11.10 Special Provisions for Wall. Using this method, I solved the shear strength of the concrete provided by the shear walls (V_c). V_c was determined assuming the shear wall is in compression due to the gravity loads. I then subtracted V_c from the factored ultimate shear load to determine the portion of the ultimate shear load which must be carried by the reinforcing steel (V_s) in the shear wall

(ACI eq. 11-2). The area of shear reinforcement was checked with the area needed to achieve V_s.

My results show that the present shear wall in the north/south direction has enough strength to counteract the critical lateral loading which might be applied to it. However, I found that the shear walls running in the east/west direction needed greater reinforcement then the present design uses. This leads me to believe that the shear in the east –west direction is not solely carried by the shear walls. Some of the lateral loads must be carried by either the columns or the columns in some areas where beams are also used. Another option is that the shear walls of the entire building work as a complete entity. That is to say that because the shear walls in the e/w direction are connected to the shear wall in the n/s direction they work together as a tube resist torsion.

Column (6th floor):

My last check was that a column on the 6th floor could withstand the combined axial and lateral loads which may be applied to it. The axial load was previously determined in the gravity loading spot check. To find the moment created by the lateral loads, I first had to distribute out the critical lateral point load for the story. This load was then multiplied by the tributary area acting on the row of columns. Once this force was know, the portal method was employed to distribute the force to each column of the row. The forces acting on the columns are multiplied by the height of ½ of the story height in order to determine the moment acting on each column. I then checked the combined moment and axial load on a typical 6th floor column with a Concrete Column Strength Interaction Diagram. The column was found to be sufficient.

Appendix

Dead loads

slab = 150pcf (49' × 100') = 735 lbs/ft
or 150pcf (8"/12) = 100psf (superstructure)
150pcf (10"/12) = 125psf (substructure)

floor	area
P1, conc, L1, ground	49·100
2-7	(49·100)-(21.4·16.125)
8-12	(49·76.3)

Mechanical/Electrical: 5psf (common assumption)

Finishes: 15psf (Assumption, luxuary finishes

Dead Loads are the weight of concrete (pcf) times the thickness of the slab to determine weight in psf. This weight is then multiplied by the tributary area.

An additional 5psf for mechanical equipment and 15psf for finishes were also added.

Roof live load:

ASCE 7 section 4.6

For a flat roof $L_r = 20R_1R_2$

R_1 : dependent on A_T

Area of a typical bay = 16.2'·13'

$A_T = 208 \text{ ft}^2$

$R_1 = 1.2 - .001 A_T$

$= 1.2 - .001(208)$

$= .992$

R_2 : dependent on F

F is slope of roof

For a flat roof, F = 0

$R_2 = 1$

$L_r = 20R_1R_2$

$= 20(.992)(1)$

≈ 20 psf

Roof load was found using ASCE7-2, section 4.6
A flat roof was used.

Snow load:
ASCE 7 section 7

Snow load on a flat roof
$p_f = .7 C_e C_t I p_g$

C_e (Table 7.2)
B – Urban areas
Partially Exposed
$C_e = .9$

C_t (Table 7.3)
$C_t = 1.0$

I (Table 7.4)
Category II
$I = 1.0$

$p_g = 25 psf$

$p_f = .7 C_e C_t I p_g$
$= .7(.9)(1)(1)(25)$
$= 15.75 psf$

Snow loads were found in accordance with ASCE7-02, section 7

Excel, what is group?

Wind Load

Basic Wind speed (V) = 90 mph (Fig. 6-1 ASCE-7)
Wind directionality (K_d) = .85 (Table 6-4)
Importance factors (I) = 1 (Table 6-1)
6.5.6 Exposure Category ( ) = B
Surface Roughness B : Urban
Exposure Category : B
Exposure for Wind Resisting system

External pressure coefficients (C_p) (Figure 6-6)
Windward $C_p = .8$
Leeward $C_p = -.5$
$L/B = 50/100 = .5$

Topical factor (K_{zt}) = 1 (Assume building is not on a hill)
$K_{zt} = (1+K_1K_2K_3)^2$

Gust Factor (G) = .78
$G = .925\left(\frac{1+1.7g_Q I_z Q}{1+1.7g_v I_z}\right)$
$I_z = c(33/\bar{z})^{1/6}$
$\bar{z} = .6h = .6(150) = 90 \leftarrow z_{min} = 30$ controls
$c = .3$
$I_z = .3(33/90)^{1/6} = .2538$
$Q = \sqrt{1/\left(1+.63\left(\frac{B+h}{L_z}\right)^{.63}\right)}$
B = 100 ft / 50 ft NS
h = 150 ft
$L_z = l(\bar{z}/33)^{\bar{\epsilon}} = 320\left(\frac{90}{33}\right)^{1/3} = 447$
$Q = \sqrt{1/\left(1+.63\left(\frac{100+150}{447}\right)^{.63}\right)} = .83$
$G = .925\left(\frac{1+1.7(3.4)(.25)(.83)}{1+1.7(3.4)(.25)}\right) = .83$

Wind loads found in accordance with ASCE7-02, section 6
All parameters were found as defined in code.
Gust factor calculations were preformed and then checked on an excel spreadsheet, the excel spreadsheet's value of G= .86 was taken to be more accurate.

Wind Load (2)

enclosure classification: Enclosed
internal pressure coefficient (GC_{pi}): $\pm.18$ $-.18$ (Fig 6-5)
external pressure coefficients

Velocity pressure (q): 2.64

$q_z = .00256(K_z K_{zt} K_d V^2 I) =$

$K_d = .85$

$K_z = 2.01(z/z_g) = 2.01 \frac{90}{1200} = .15$

$K_{zt} = 1$

$q_z = .00256(1 \cdot .15 \cdot .85 \cdot 90^2 \cdot 1) = 2.64$

q_z is for height z
q_h is for roof height

Design Wind Pressures

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

$q = q_z = 2.64$

$q_i = q_h$

All equations are taken from ASCE7-02 and then used in an excel spreadsheet.

Wind

Basic wind speed - 90mph
Importance - 1
Wind Exposure = B
Wind design (windward) =
" " (leeward) =

G - .8 or .86 (from spreadsheet)

Cp: Windward → Cp = .8
leeward → Cp = -.5

Kz Exposure B Kz = 2.01 (z/1200) → Eq. Table 6-3
windward
0-15
15-20
20-25
25-30

Final parameter values used in Excel spreadsheet.

Results from wind loading spreadsheet are on page 7 of the report for both N/S and E/W directions.

70	93	1.20
76	96	1.28
81	99	1.35
85	1.04	1.41
89	1.09	1.47
	1.13	1.52
	1.17	1.56

Exp. B
Speed ~~120~~ 90
Importance 1
K_{zt} 1
K_d .85
$q_z = .00256\, K_{zt} K_z K_d V^2 \cdot 1$ q_h is roof q_z
$= 17.6256 K_z$
q_h = Biggest q_z (roof)
C_p = .8 windward
= -.5 leeward (N/S)   -.3 (E/W)

Design Pressure $P = qGC_p$

$P = q_z GC_p - q_h (GC_{pi})$
windward

$= q_h GC_p - q_z (GC_p)$ leeward

$P = qGCp$

Final parameter values used in Excel spreadsheet.

Results from wind loading spreadsheet are on page 7 of the report for both N/S and E/W directions.

Seismic Loading:

Seismic use group = I (table 9.1.3)
Occupancy Category = II (table 1-)
Occupancy Importance Factor $I = 1$ (Table 9.1.4)
Max ground motions $S_S = 18.7\%$ (Fig 4.1.1)
$S_1 = 6.3\%$ (Fig 4.1.1)
Site class = C (9.4.24)
Site class Factors $F_a = 1$ (Table 9.4.1.24a)
$F_v = 1.3$ (Table 9.4.1.24b)

$S_{MS} = F_a \cdot S_S = 18.7\%$
$S_{M1} = F_v \cdot S_1 = 8.19\%$

$S_{DS} = \frac{2}{3} S_{MS} = 12.5\%$
$S_{D1} = \frac{2}{3} S_{M1} = 5.46\%$

Seismic Design Category = A (Table 9.4.21a+b)

Responce Mod. Factor $R_{N-S} = 5$ (Table 9.5.2.2)
(ord. rein. concrete shearwall) $R_{E-W} = 5$
Building Framesystem $\Omega_{0\,N-S} = 2\frac{1}{2}$ $\Omega_{0\,E-W} = 2\frac{1}{2}$
$C_{d\,N-S} = 4\frac{1}{2}$ $C_{d\,E-W} = 4\frac{1}{2}$

$C_t = .02$ $x = .75$ (Table 9.5.5.3.2)

Seismic loading done according to ASCE7-02, section 9.
All parameters are defined as according to the code.

$V = C_s W$ (Eq 9.5.5.2-1)

$C_s = S_{DS}/(R/I) = .025$ (Eq 9.5.5.2.1-1)

$C_{s_{MAX}} = S_{D1}/T(R/I) = .015$ (Eq 9.5.5.2.1-2)

$C_{s_{MIN}} = .044 S_{DS} I = .0055$ (Eq 9.5.5.2.1-3)

$V = .015 W$

$w_x h_x^k$

$k = 1 + (T_{N-S} - .5)/(2.5 - .5) = 1.103$ (Eq 9.5.3.4-2)

Vertical Distribution (Eq. 5.5.4-1)

$F_x = C_{vx} V$

$C_{vx} = \frac{w_x h_x^k}{\Sigma w_i h_i^k}$ (Eq. 9.5.5.4-2)

Overturning Moment

$M_x = \Sigma F_i (h_i - h_x)$ (Eq. 9.5.5.6)

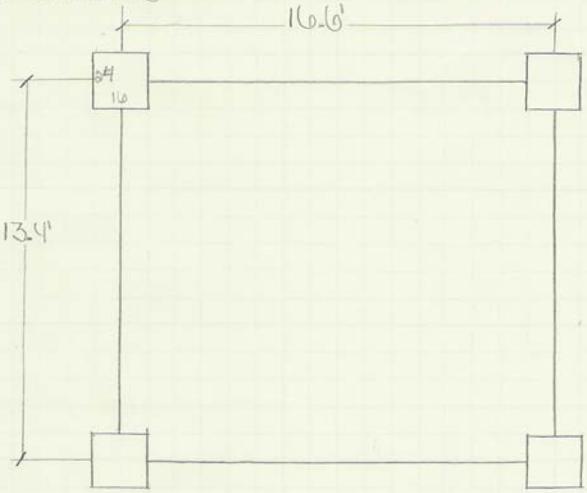
All seismic parameters and equations, as taken from ASCE7-02, were used in an Excel Spreadsheet to solve for the seismic load.

Spreadsheet results for both the N/S and E/W directions can be found on page 9 of this report.

Spot-check

Gravity Loads:

Typical Bay
4-6 floor
interior bay
average size 13.4' by 16.6'
24 x 16 columns



Factor Loads

Dead load: slab = 100 psf
MEP = 5 psf
finishes = 15 psf
Total Dead = 120 psf

Live load:
Residential level = 60 psf (same as real)
elevators/lobbies = 80 psf
Assume typical bay is not lobby or elevator
Total live = 60 psf

Total Load

$w_u = 1.2D_L + 1.6L_L$
$= 1.2(120) + 1.6(60)$
$= 240$ psf

Typical Bay diagram.
Factored Gravity Loads.

1

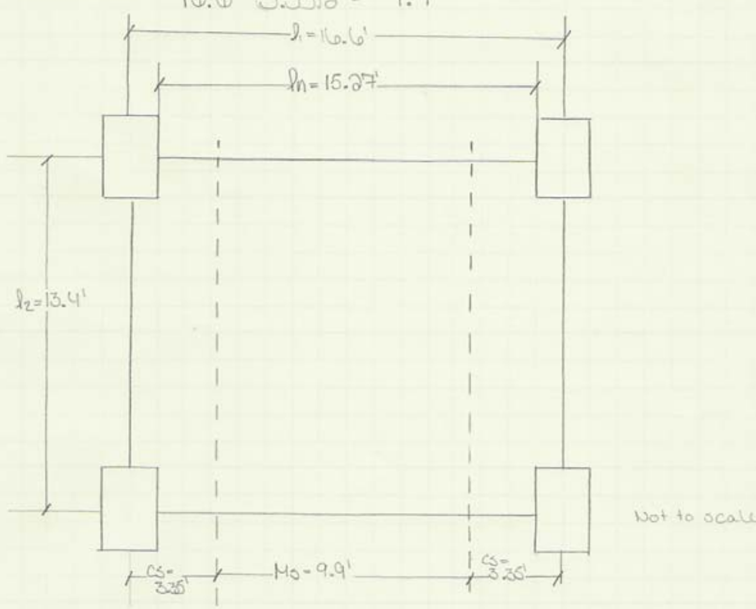
Interior bay short span

Determine Bay Parameters:

$\ell_n = 16.6' - 8'' - 8'' = 15.27'$ (ACI 13.0)
$\ell_1 = 16.6'$
$\ell_2 = 13.4'$

Column Strip (ACI 13.2.1)
lesser or $.25\,\ell_1 = .25(16.6) = 4.15'$
$.25\,\ell_2 = .25(13.4) = 3.35'$ ← controls

Middle Strip
$16.6 - (3.35)2 = 9.9'$



Geometries for interior bay in short span direction.

Slab Design — Short span

Limitations (ACI 13.6.1)

i) Minimum of 3 continous spans
ii) Rectangular; ratio < 2
iii) Successive spans not differ by 1/3
iv) offset columns max of 10% span
v) uniform loads

All conditions are met ∴ Direct Design Method can be used.

Moment (ACI 13.6.2.2)

$$M_o = \frac{w_u l_2 l_n^2}{8}$$

$$= \frac{240psf \cdot 13.4' \cdot 15.27'^2}{8}$$

$$= 93735.5 \text{ lb}\cdot\text{ft}$$
$$\approx 94 \text{ kip ft}$$

Neg. + Positive factored Moments (ACI 13.6.3)

			Trib width	Moment / trib width
Interior Neg. (65%) 61.1'k	Column strip 75% = 46'k		6.7'	6.8 'k/ft
	Middle strip 25% - 15.3'k		10'	1.53 'k/ft
Midspan Pos. (35%) 32.9'k	Column strip 60% = 20'k		6.7'	3 'k/ft
	Middle strip 30% = 10'k		10'	1 'k/ft

3

All limitations are met allowing for Direct Design Method to be used. ACI 13.6.2.2

Factored moments for short span interior bay as described in ACI 13.6.3

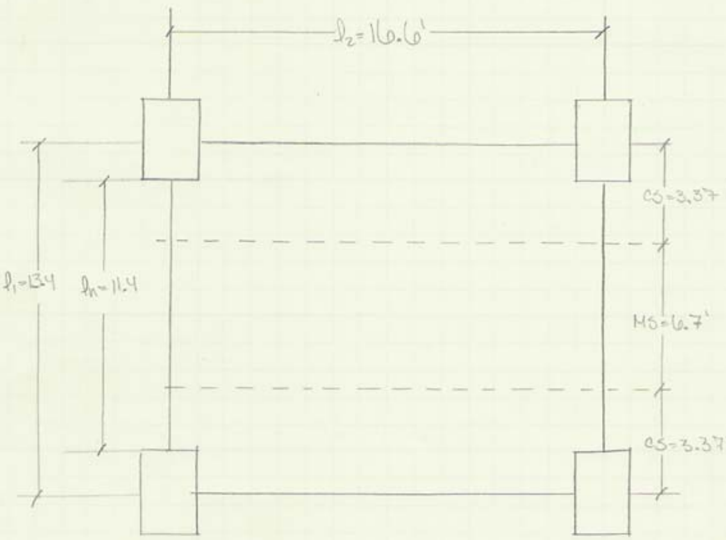
long span

Determine Bay Parameters

$\ell_n = 13.4' - 12'' - 12'' = 11.4'$
$\ell_1 = 13.4'$
$\ell_2 = 16.6'$

Column strip
.25 (shortspan) = 3.35'

Middle strip
13.4 - 2(3.35) = 6.7'



Geometries for a typical bay in the long span direction. ACI 13.0 and ACI 13.2.1

long span

All direct design method limitations (ACI 13.6.1) are met, there for the direct design moment can be used

$$M_o = \frac{w_u l_2 l_n^2}{8}$$
$$= \frac{240psf \cdot 16.6 \cdot 11.4^2}{8}$$
$$= 64720 \text{ lb}\cdot\text{ft}$$
$$= 65 \text{ kip}\cdot\text{ft}$$

Factored Moment

			Trib width	Moment/ft
Interior Neg. 65% 42.25^{1k}	Column Strip 75%	= 31.68	6.7'	4.7 ^{1k}/ft
	Middle Strip 25%	= 10.56	6.7'	1.6^{1k}/ft
Midspan pos 35% 22.75^{1k}	Column strip 60%	= 13.65	6.7'	2^{1k}/ft
	Middle Strip 30%	= 6.825	6.7'	1.018

Check Min span thickness
$t_{min} = l_n/33 = 11.4/33 = .345'$
$= 4.145''$

Min. Reinforcement Requirments
$A_{s\,min} = .0018 A_g$ (7.12)
$A_g = (8'' \text{slab} \cdot 12''/1) = 96 \text{ in}^2$
$A_{s\,min} = .0018(96) = .1728 \text{ in}^2/\text{ft}$

$S_{max} = 2t = 16''$ @ critical sections (ACI 13.3.2)
$S_{max} = 5t = 40''$ or $18''$ (ACI 7.12.2.2)

$S_{max} = 16''$
$A_{s\,min} = .1728 \text{ in}^2/\text{ft}$

5

Factored moments for interior bay in the long span direction. ACI 13.6

Minimum reinforcing requirement and spacing. ACI 7.12, 13.3.2, and 7.12.2.2

Required Steel

$A_s = .1728 \text{ in}^2/\text{ft}$
$S_{max} = 16''$

$\therefore$ Try #4 @ 12" on center

$A_s = .2 \text{ in}^2$ $\quad A/\text{ft} = .2 \text{ in}^2/\text{ft}$

Find d

$d = t - \text{cover} - \text{stirrup} - d_b/2$
$= 8 - 3/4'' - 1/2'' - \frac{4/8}{2}$
$= 6.5''$

$a = \dfrac{A_s f_y}{.85 f'_c b}$

$= \dfrac{.2 \cdot 60}{.85 \cdot 4 \cdot 12/1}$

$= .29 \text{ in}''$

$\Phi M_n = \Phi A_s f_y (d - a/2)$
$= .9(.2)(60)(6.5 - .29/2)$
$= 68.634 \text{ ''k/ft}$
$= 5.719 \text{ 'k/ft}$

$\Phi M_n > M_u$
$5.719 > ?$

#4 @ 12" oc is sufficent reinforcement for
long span: Interior Column Strip
Interior Middle Strip
Midspan Column Strip
Midspan Middle Strip
short span: Interior Middle Strip
Midspan Column Strip
Midspan Middle Strip

$\therefore$ only additional reinforcement is need is short span Interior column strip

Check required steel area with actual steel area.
Determine if $\Phi M_n > M_u$

6

Reinforcement Design
@ Short Span Interior Column Strip

Try #5 @ 12" oc
$A_s = .31\ in^2$ $\qquad A/ft = .31\ in^2/ft$

Find d
$d = 8 - 3/4" - 1/2" - \frac{5/8}{2}$
$= 6.4375"$

$a = \frac{.31(60)}{.85(4)(12/1)}$
$= .455\ in$

$\phi M_n = .31(60)(6.44 - .46/2)$
$= 115.54\ \phi$
$= 9.62\ \phi$
$= 8.66\ ^{1k}/_{ft}$

$\phi M_n > M_u$
$8.66 > 6.8$ ✓

∴ Use a number 5 bar every 12" oc for the short span interior column strip

7

Design steel for short span interior column strips because the previously found moment did not check.

Typical Bay
4-6 floor (residental)
Exterior Bay

Bay is same size as interior bay
same static Mo as interior bay

Short Span
M_o = 94 kip-ft

Long Span
M_o = 65 kip-ft

Factored Moments

Short Span

		Tribwidth	Moment / Tribwidth
Interior neg 65% 61.1^K	Column Strip 100% = 61.1^K	6.7	9.119^K/ft
	Middle Strip 0% = 0^K	10	0^K/ft
Midspan pos 35% 32.9^K	Column Strip 60% = 20^K	6.7	3^K/ft
	Middle Strip 30% = 10^K	10	1^K/ft

B_t = taken to equal zero (Masonry w/o torsional resistance

Long Span

		Tribwidth	Moment / Tribwidth
Interior neg 65% 42.25^K	Column Strip 100% = 42.25^K	6.7'	6.305^K/ft
	Middle Strip 0% = 0^K	6.7'	0^K/ft
Midspan pos 35% 22.75^K	Column Strip 60% = 13.65^K	6.7'	2^K/ft
	Middle Strip 30% = 6.825^K	6.7'	1.018^K/ft

Try #4 oc 12" → same calcs. as interior bay
ϕM_n = 5.7 ^K/ft
∴ works for all but Interior Column strip (both spans)

Try #5 oc 12" → same calcs. as interior bay
ϕM_n = 8.66^K/f
∴ works for Long span interior column strip

Factor moments for exterior bays in both the long and short span directions.
Assume steel sizes and check moments.

8

Reinforcement Design
@ Short Span (Exterior Bay) Interior Column Strip

Try #5 @ 6" o.c.
$A_s = .31 \text{ in}^2$ $\quad$ $^{n}/_{ft} = .62 \text{ in}^2/\text{ft}$

Find d
$d = 8 - 3/4" - 1/2" - 5/8/2$
$= 6.4375"$

$a = \frac{.62(60)}{.85 \cdot 4 \cdot 12/1}$
$= .91$

$\phi M_n = (.62)(60)(6.4375 - .91/2) \cdot .9$
$= 201\ ^{"k}/_{ft}$
$= 16.7\ ^{'k}/_{ft}$

$\therefore$ This work

Conclusion:

My Entire slab has been designed for #4 @ 12" oc. #4 @ 12" on center is commonly concidered the least partical amount of reinforcement.

My spot checks have proved that #4 @ 12 oc works in all loctions, except by the Interior Column Strip. By viewing my drawings, it can be seen that most columns are marked with additional reinforcement in the slab around them. This additional reinforcements raised the [illegible] moment the slab can support to a good one.

Conclusions

9

Spot check

Gravity loads on columns

Tributary Floor Area for a column on a typical interior bay.

$A_T = (13.4')(16.6') = 222\ ft^2$

Loading

Dead load: $D_L = 120$ psf
Live load: $L_L = 60$ psf for residential areas
Roof live: $R_L = 20$ psf
Snow load: $S = 15.75$ psf

Total load

$W_U = 1.2D + 1.6L + .5(L_r \text{ or } S)$ ← controls
$= 1.2(120) + 1.6(60) + .5(20)$
$= 250$

$W_U = 1.2D + 1.6(L_r \text{ or } S) + L$
$= 1.2(120) + 1.6(20) + 60$
$= 236$

Floor 6

Dead: $1.2(222ft^2)\cdot(120psf)\cdot 7 stories$
$= 224$ kips
Live: $1.6((60)(7 stories)(222)$
$= 150$ kips
Roof: $20\cdot 222\cdot .5$
$= 2.2$ kips

Total $= 376.2$ kips

Floor Concourse

D: $1.2(222)(120)$ 13 stories
$= 416$ kips
L: $1.6\cdot(222)(60)\cdot 12 + 1.6(100)(222)$
$= 291$ kips
R_L: $20\cdot 222\cdot .5$
$= 2.2$ kips

Total $= 637.2$ kips

①

Gravity loads on interior 6th floor and concourse level columns.
Determine tributary area and loading.

Check columns for gravity loads,
Assume no laterial load or bending

Typical 6th floor column
24"x16"

Typical concourse column
16"x24"

$f = P/A$

$5000\,psi = P/(24 \times 16)$

$P = 1{,}920{,}000$ lbs
$= 1920$ kips

$P > 376.2$ ✓

$P > 637.2$ ✓

∴ Both of my column spot checks work for pure axial loading caused by gravity loads

Accounting for steel

$n = E_s/E_c$
$= 60000/5000$
$= 12$

$P = f_c(A_g + (n-1)A_{st})$

4 #9
$A_s = 4\ in^2$

$P = 5000(192 + (12-1)4)$
$= 1180$ kips ✓

12 #11
$A_s = 18.72\ in^2$

$P = 5000(192 + (12-1)18.72)$
$= 1989.6$ kips ✓

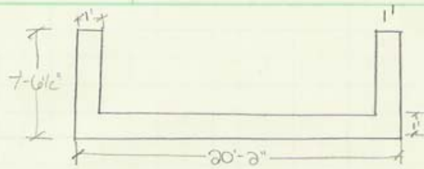
(2)

16. 24, 5. 4

Determine maximum load that column can support and compare to load that will be applied to column under full design loading.

Spot check

Shear walls



11.10.2 $\phi V_n \geq V_u$
$V_n = V_c + V_s$

11.10.3 V_n cannot be larger then
$V_n = 10\sqrt{f'_c}\,hd$

N-S	E-W (2 walls)	
$h = 1'$ $l_w = 20'\text{-}2" = 20.17'$ $d = .8l_w = 16.136'$	$h = 1'$ $l_w = 7.54'$ $d = .8l_w = 6.03'$	(11.10.4)
limit $V_n = 10\sqrt{4000}(1)(16.136)$ $= 10205.3\ \text{lb/in}^2\cdot\text{ft}^2$ $= 1470$ kips	$V_n = 10\sqrt{4000}(1)(6.03)$ $= 3813.7\ \text{lb/in}^2\ \text{ft}^2$ $= 550$ kips	(11.10.3)
$V_{cc} = 2\sqrt{f'_c}\,hd$ $= 2\sqrt{4000}(1)(16.136)$ $= 294$ kips	$V_{cc} = 2\sqrt{f'_c}\,hd$ $= 2\sqrt{4000}(1)(6.03)$ $= 110$ kips	(11.10.5)
V_{ct}		

Assume only axial load is compression
∴ only V_{cc} is needed
(11.10.8)

$\phi V_c/2 = (.75)294/2$
$= 110.6$ kips

$\phi V_c/2 = (.75)110/2$
$= 41.25$

$V_u < \phi V_c/2$
$120 \not< 110.6$

$V_u < \phi V_c/2$
$228 \not< 41.25$

∴ Shear Reinforcement is needed

$\phi = .75$ (9.3)
(11.10.8)

①

Lateral loading spot check on shear walls.
Assuming that shear wall is in compression due to gravity loads, calculations of the nominal shear limit and shear strength in the concrete will determine if reinforcement is need. ACI 11.10

Design of Shear Wall Reinforcement

N-S

$V_s = \frac{A_v f_y d}{s_2}$

$V_s = V_u - \phi V_c/2$
$= 120 - 110.6$
$= 9.4$ kips

$9.4 = \frac{A_v \cdot 60 \cdot 16.136'}{1 ft}$
$A_v = .009\ in^2/ft$

check $A_v/A_g > .0025$
$A_v/(1 \cdot 20.17) = .0025$
$A_v = .050$

$A_v = .05\ in^2/ft$ controls

$s_2 max$ $l_w/5 = 20.17/5 = 48.4 in$
$3h = 3/1 = 36 in^2$
$18 in = 18 in \leftarrow$

E-W

$V_s = \frac{A_v f_y d}{s_2}$ (11.10.9.1)

$V_s = V_u - \phi V_c/2$ (11.1.1)
$= 228 - (41.25 \cdot 2)$
$= 145.5$ kips

$V_u = V_s + V_c$
↳ my own equation

$145.5 = \frac{A_v \cdot 60 \cdot 6.03}{12"/1'}$
$A_v = .40\ in^2/ft$

$A_v/A_g > .0025$ (11.10.9.2)
$A_v/(1 \cdot 2 \cdot 6.03) > .0025$
$A_v = .030$

$A_v = .4\ in^2/ft$ controls

$l_w/5 = 6.03 \cdot 2/5 = 29 in^2$ (11.10.9.3)
$3h = 3/1 = 36 in^2$.
$18 in \leftarrow$ controls

Try #4 @ 12" oc
$A_s = .2 in^2$ $A/ft = .2\ in^2/ft$

N-S

$V_s = \frac{A_v f_y d}{s_2}$
$= \frac{(.2)(60)(16.136' \cdot 12)}{12}$
$= 193.632$ kips

$V_n = V_c + V_s$
$= 193 + 294$
$= 487$ kips

$\phi V_n \geq V_u$
$.75(487) \geq 120$
$365.25 > 120$ ✓

E-W

$V_s = \frac{A_v f_y d}{s_2}$
$= \frac{(.2)(60)(2 \cdot 6.03)12}{12}$
$= 144.72$

$V_n = 144 + 110$
$= 254$ kips

$\phi V_n \geq V_u$
$.75(254) \geq 228$
$190.5 \ngeq 228$

Find shear strength provided by reinforcing steel in the horizontal direction. Check that shear strength in concrete and shear strength in steel are greater then required shear on the wall.

2

For E-W direction
Try #4 @ 6" oc.

$A_s = .4\ in^2/ft$
↳ Meets calculated placement

$$V_s = \frac{(.4)(60)(2 \cdot 6.03)}{1} = 290 \text{ kips}$$

$$V_n = V_s + V_c = 290 + 110 = 400 \text{ kips}$$

$$\phi V_n \geq V_u$$
$$(.75)400 \geq 288$$
$$300 \geq 288$$ This works!

∴ For N-S direction, use #4@12" on center for horizonal section this is minumin pratical reinforcement.

For E-S direction use #4@6" on center for horizonal section This differs from given dusign. Therefore I believe the columns + some beams will take part of the lateral force. The other option is for the concrete shear walls to act as a tube and increase strength

Add additional reinforce to E/W shear walls.

3

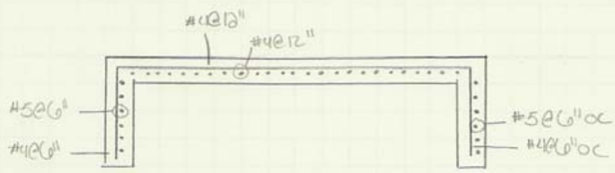
Check Vertical Reinforcement

$\rho_{v\,min} = .0025 + .5(2.5 - h_w/l_w)(\rho_n - .0025)$

$\rho = .0025 + .5(2.5 - 1/20.17)(.2 - .0025)$
$= .003112$
$\rho_n = .2 in^2/ft$

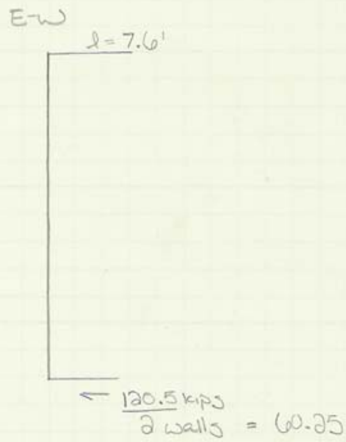
$\rho = .0025 + .5(2.5 - 1/6.03)(.4 - .0025)$
$= .4664$

∴ #4 @ 6" does not ρ_{min}
Try #5 @ 6" o.c.



Ratios for vertical steel in shear walls (ACI eq 11-32) were also checked.
Final design of shear wall with reinforcement.

Shear Wall



Mu= 7763 k-ft

Mu= 13,968 k-ft / 2walls = 6984 k-ft

Wall thickness estimate
h_{wmin}=
use 12" already given design

Wall design Flexure / Shear

N-S
Vu= 239 kips
Mu= 7763 k-ft

E-W
Vu= 60.25 kips
Mu= 6984 k-ft

check ϕV_{nmax}
ϕV_{nmax} = .75 (1470)
= 1103 > Vu

ϕV_{nmax} = .75 (275)
= 206.2 > Vu ok!

compute shear strength of concrete
Nu → compression

ϕV_c = .75 (294) = 221 < 239
∴ Reinforcement is needed

ϕV_c = .75 · 55 = 41.25 < 60.25

Lateral loading spot check for column on the 6th floor. Ultimate shear and moment acting on each floor was found using an excel spreadsheet.

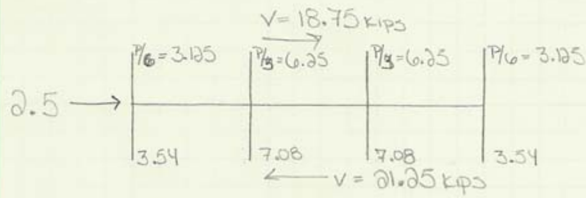
4

Determination of Moment on Columns due to lateral loads

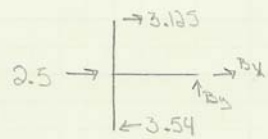
6th floor

story h = 8.8
h/2 = 4.2

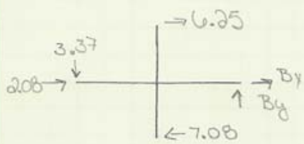
d = 16.6/2 = 8.3



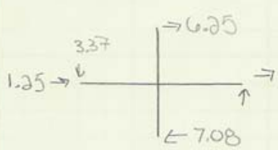
19 kips / 100ft = .19 k/ft
Tribwidth = 13.4'
P = .19 · 13.4' = 2.5 kips



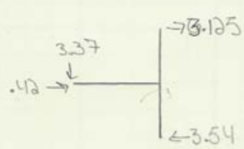
Bx = −2.085
ΣM: 0 = (3.125·4.2) − (By 8.3) + (3.54·4.2)
By = 3.37



Bx = 1.25
M: 0 = ((6.25+7.08)4.2 − (3.37·8.3) − (By 8.3)
By = 3.37



Bx = .42
M: 0 = ((6.25+7.08)4.2 − (3.37·8.3) − (By 8.3)
By = 3.37



✓

Portal method analysis to determine the lateral forces acting on each column.

③

Moment on an Interior Column
6th floor

$M = 6.25\text{ kips} \cdot 4.2'$
$= 26.25 \text{ ft-kips.}$

I need chart 16.24.5.4
instead, checked 16.16.5.6

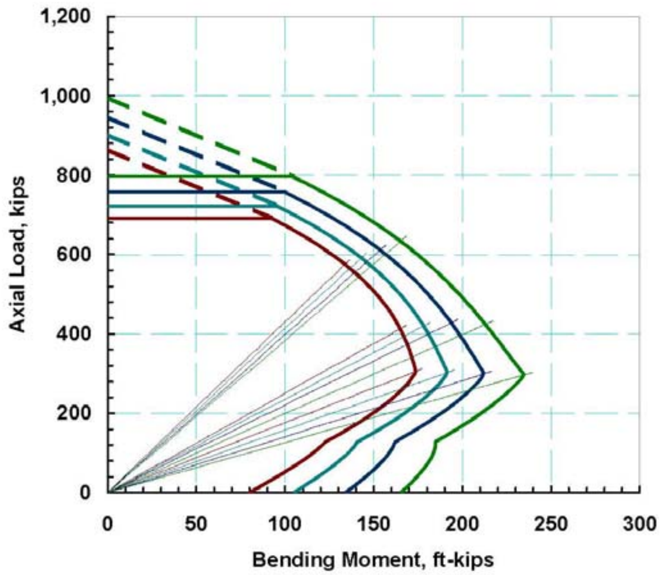
Using the following chart, My column was well w/in the acceptible area

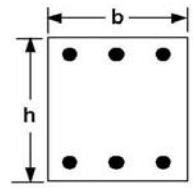
Concrete Column Strength Interaction Diagram

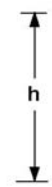
Final moment on 6^{th} column is calculated. The combined moment and axial load were checked on a column interaction chart. Column interaction chart is on next page.

STRUNET
CONCRETE DESIGN AIDS

Concrete Colum Strength Interaction Diagram: 16.16.5.6

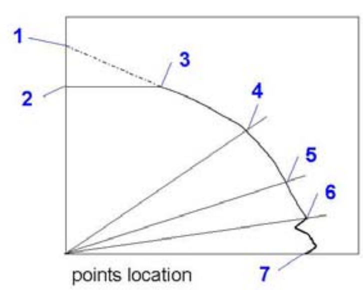






Conncrete Strength, f'_c		**5 ksi**
Steel Yield Strength, f_y		**60 ksi**
Clear Cover to Ties		**1.5 in**
Column Dimensions		
	b	**16 in**
	h	**16 in**

Ties	#3		#3		#3		#3	
Bars	**6#6**		**6#7**		**6#8**		**6#9**	
P. No.	**ϕP**	**ϕM**	**ϕP**	**ϕM**	**ϕP**	**ϕM**	**ϕP**	**ϕM**
1	865	0	902	0	947	0	996	0
2	692	0	722	0	757	0	797	0
3	692	92	722	96	757	100	797	104
4	575	134	591	143	611	154	633	165
5	413	165	420	178	428	194	424	213
6	306	174	303	191	300	212	297	235
7	0	80	0	105	0	134	0	165



strunet.com
concrete column interaction diagrams