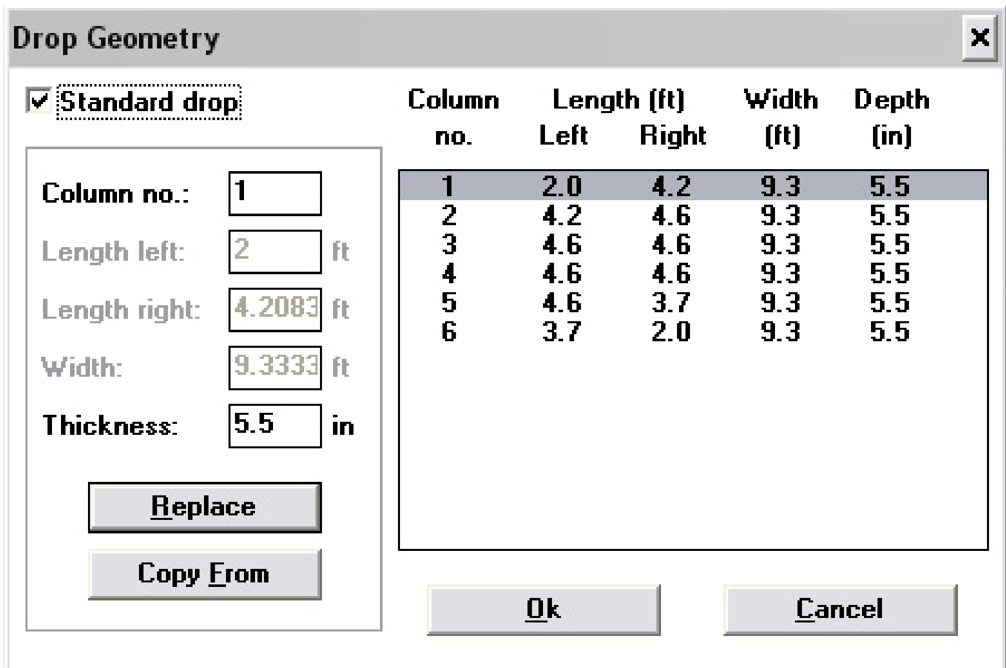
Aric Heffelfinger
Fordham Place
Bronx, NY
Structural Option
Advisor - Dr. Hanagan

Structural Redesign

<u>Two way slab / Drop panels</u>

My redesign of Fordham Place will comprise of a 9” flat slab with 5 ½” drop panels. Materials used for this redesign is normal weight concrete with compressive strength of 4ksi and steel rebar with a yield stress of 60 ksi. Floor slab thickness was determined by ACI 318-02 table 9.5(c) using exterior panels, without edge beams, but with drop panels to get minimum floor slab thickness of $\ell_n/36$. Where ℓ_n = 28’ – 2’ = 26’, and the value $\ell_n/36$ = (26’ x 12”) / 36 = 8.67”. At first I determined the drop projection of ¼ t_{slab} from ACI 318-02 section 13.3.7.2. Where ¼ t_{slab} = 9”/4 = 2.25”. In order to form the drops with 2 x 4’s or 2 x 6’s, drop projection needs to be either 3.5” or 5.5”. Therefore drop projections were 3.5”. However, when analyzed in ADOSS, a 3.5” drop did not provide sufficient shear capacity. I then changed the drop projection to 5.5” and determined the slab had sufficient shear capacity.

In ADOSS, I used the standard drop tool which lets ADOSS determine the width of the panels. (see top left of picture below)



After doing a hand check for the drop widths, I determined that ADOSS calculated drop widths using ACI 13.3.7.1. This section states the minimum drop width shall be 1/6 span from center to center of supports in each direction. Where 1/6 span = 1/6 x 28’ = 4.67’. This can also be seen in the above table. At this point I was able to analyze the floor system in ADOSS. Material properties, slab reinforcement data, geometry, loads, and load factors needed to be input into ADOSS. Flexural reinforcement is located 1.5” from the tension face with a minimum spacing of 6”. #4 bar will be a minimum bar size.

Minimum reinforcement ratio is

$$(A_s)_{min} = 0.0018\ A_g \qquad \text{ACI 7.12.2.1}$$
$$= 0.0018 \times 9” \times 12”$$
$$= 0.19\ in^2/ft$$

Therefore the minimum flexural reinforcement will be #4’s @ 12”.

In order to simplify the design of the slab and columns, there was an assumption that shear walls would resist 100% of the lateral load; leaving the slab and columns to resist only gravity loads. Gravity loads that were considered were dead, live, roof live, and snow. The following is a list of the loads that were used in designing the concrete system.

Superimposed Dead = 30psf

Live = 80psf

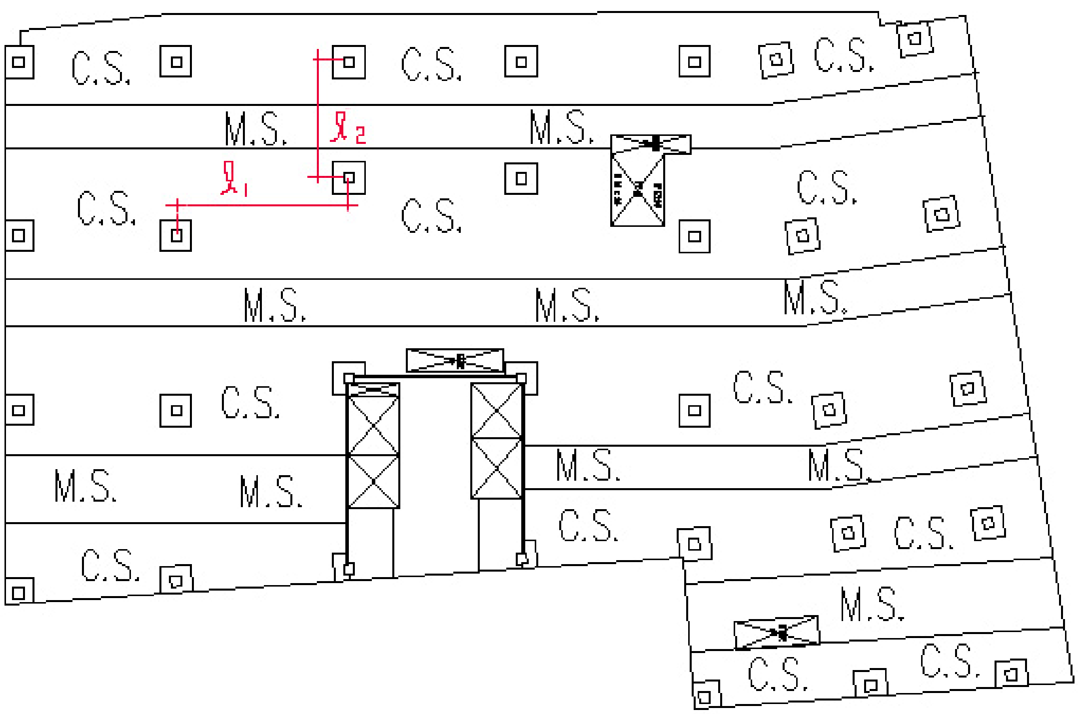
Roof live / snow = 30psf

Live loads were reduced to lesser values based on ASCE 7-02. (See Appendix for complete calculations)

After inputting this information into ADOSS, I was then able to design the system. The following is part of an ADOSS output file showing positive and negative reinforcement. Although ADOSS does design the number and spacing of bars, it was not very uniform throughout the different spans of the slab even though the total amount of steel required was similar. Therefore from the output file, I determined the amount of steel

per foot width and selected bar size and spacing. This was done for both column and middle strips. Having a more uniform steel layout throughout the building reduces the chance of a mistake in the field where a contractor may place the rebar incorrectly.

Because the column locations are staggered in two spans, it was a little difficult determining how I was going to analyze these spans. (See picture below)



Columns were determined using ACI 13.2.1. This section states the column strip shall be the lesser of $0.25\ell_1$ and $0.25\ell_2$. (See picture above for ℓ_1 and ℓ_2) Because of the staggered

columns, I decided to just make the area between those columns a big column strip. From the output file below, you can see the information I took from the output file to determine the area of steel per foot.

$$As = 3.84\text{in}^2 / 12.6\text{ft}$$
$$= 0.304\ \text{in}^2/\text{ft}$$

#5's @ 12"

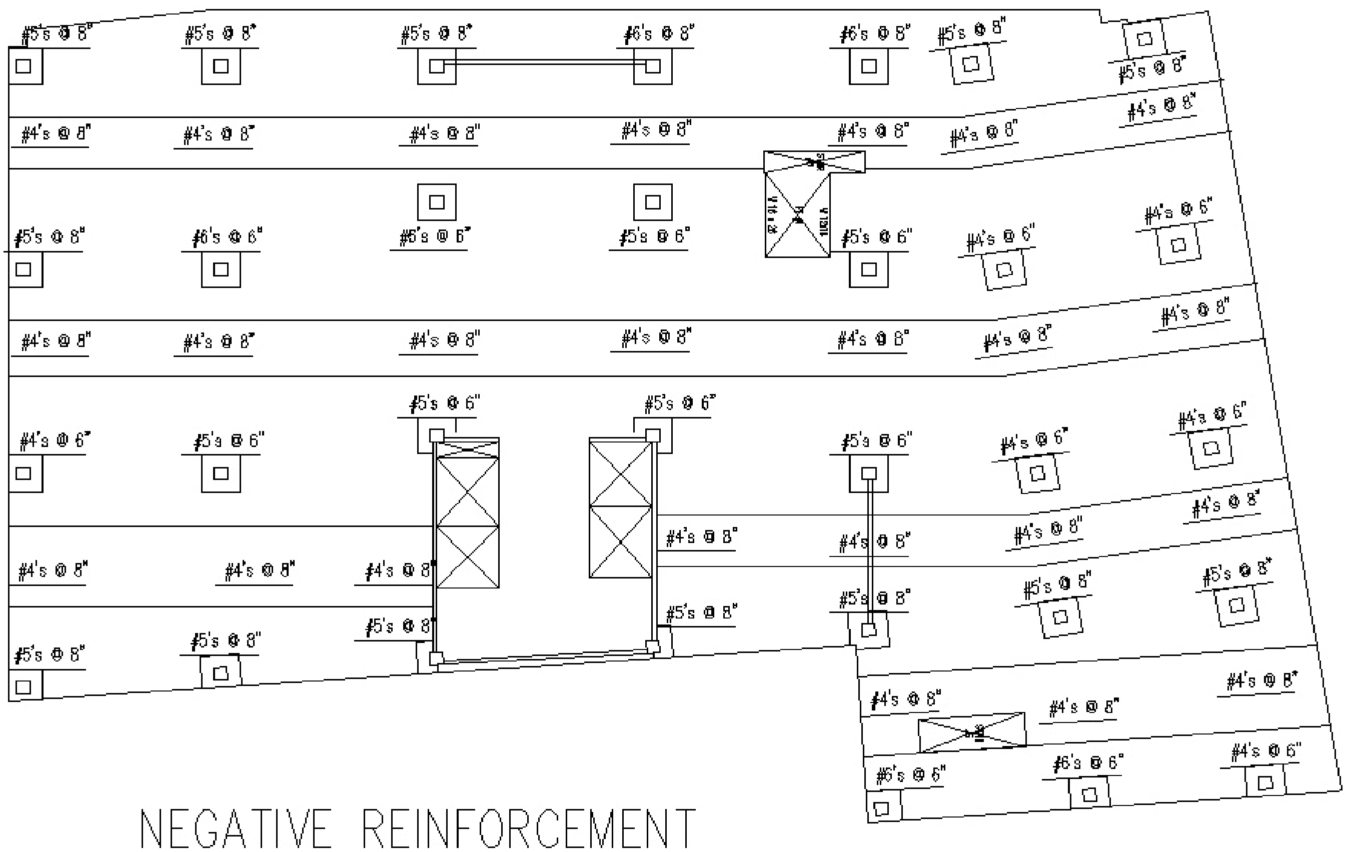
N E G A T I V E   R E I N F O R C E M E N T
****

COLUMN NUMBER	PATT NO.	LOCATION @COL FACE	TOTAL DESIGN (ft-k)	COLUMN STRIP AREA (sq.in)	COLUMN STRIP WIDTH (ft)	MIDDLE STRIP AREA (sq.in)	MIDDLE STRIP WIDTH (ft)
1	4	|| R	231.2	3.84	12.6	3.32	15.4
2	4	L ||	-525.4	6.49	12.6	3.50	15.4
3	4	L ||	-516.4	6.38	13.9	3.45	14.1
4	4	|| R	533.3	6.59	13.9	3.56	14.1
5	4	L ||	-486.3	5.99	11.0	3.67	17.0
6	3	L ||	-146.9	3.48	11.0	3.67	17.0

P O S I T I V E   R E I N F O R C E M E N T
****

SPAN NUMBER	PATT NO.	LOCATION FROM LEFT (ft)	TOTAL DESIGN (ft-k)	COLUMN STRIP AREA (sq.in)	COLUMN STRIP WIDTH (ft)	MIDDLE STRIP AREA (sq.in)	MIDDLE STRIP WIDTH (ft)
2	4	10.7	221.3	3.56	12.6	3.32	15.4
3	2	14.6	213.2	3.42	13.9	3.05	14.1
4	3	13.2	213.8	3.43	13.9	3.05	14.1
5	2	14.6	218.0	3.50	13.9	3.05	14.1
6	4	12.6	163.2	2.61	11.0	3.67	17.0

Once I knew that #5's @ 12" was a good rebar spacing and size, it just needs to be distributed over the entire column strip. The following is an example of a rebar plan in one direction. For complete rebar plans see appendix.



The above rebar plan is showing both long and short bars. Half of the given bars are long bars and half are short bars. Extension of bars was done by ADOSS however it complies with figure 13.3.8 of ACI. This table can be viewed below.

CODE

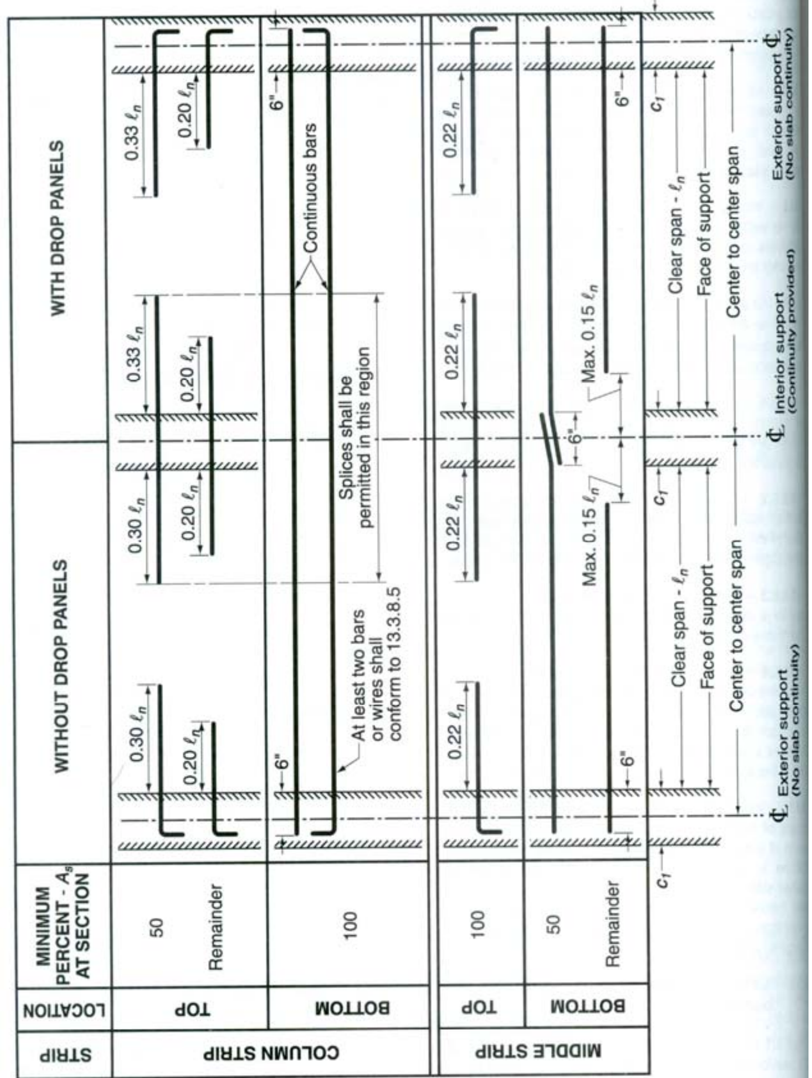


Fig. 13.3.8—Minimum extensions for reinforcement in slabs without beams. (See 12.11.1 for reinforcement extension into supports)

All other spans were analyzed using the same material properties, slab reinforcement data, and loads. The only thing that changed from span to span was its geometry.

Columns

The columns at Fordham Place are 26” x 26” normal weight concrete throughout the entire building. The concrete compressive strength is primarily 4ksi, however there are some 8ksi columns on the bottom 5 floors which support large tributary areas and in turn carry very large axial loads. The columns were designed by taking the unbalanced moment in each direction due to gravity loads and inputting them along with axial loads into PCA Column. Design moments were taken from the ADOSS output file. (see picture below)

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                N E G A T I V E   R E I N F O R C E M E N T
                *******************************************

COLUMN*PATT*LOCATION * TOTAL  * COLUMN STRIP   *  MIDDLE STRIP
NUMBER* NO.*@COL FACE* DESIGN *  AREA  WIDTH   *   AREA  WIDTH
                     * (ft-k) * (sq.in) (ft)   * (sq.in)  (ft)
--------------------------------------------------------------
   1      4      || R    231.2     3.84   12.6       3.32   15.4
   2      4    L ||     -525.4     6.49   12.6       3.50   15.4
   3      4    L ||     -516.4     6.38   13.9       3.45   14.1
   4      4      || R    533.3     6.59   13.9       3.56   14.1
   5      4    L ||     -486.3     5.99   11.0       3.67   17.0
   6      3    L ||     -146.9     3.48   11.0       3.67   17.0


               P O S I T I V E   R E I N F O R C E M E N T
               *******************************************

 SPAN *PATT*LOCATION * TOTAL  * COLUMN STRIP   *  MIDDLE STRIP
NUMBER* NO.*FROM LEFT* DESIGN *  AREA  WIDTH   *   AREA  WIDTH
               (ft)  * (ft-k) * (sq.in) (ft)   * (sq.in)  (ft)
--------------------------------------------------------------
   2      4     10.7      221.3     3.56   12.6       3.32   15.4
   3      2     14.6      213.2     3.42   13.9       3.05   14.1
   4      3     13.2      213.8     3.43   13.9       3.05   14.1
   5      2     14.6      218.0     3.50   13.9       3.05   14.1
   6      4     12.6      163.2     2.61   11.0       3.67   17.0
```

Also, design axial loads were determined using an excel spreadsheet that multiplied tributary area by self weight, superimposed dead load, reduced live load, and roof live load. (See table below)

		roof	15th	14th	13th	12th	11th	10th	9th	8th	7th	6th
D.2 - 10.0	Tributary A	781.3	781.3	781.3	781.3	781.3	781.3	781.3	781.3	781.3	781.3	781.3
		0.51832	0.51832	0.51832	0.51832	0.51832	0.51832	0.51832	0.51832	0.51832	0.51832	0.51832
	LL + SDL (	48.87934	102.3449	102.3449	102.3449	102.3449	102.3449	102.3449	102.3449	102.3449	102.3449	102.3449
	Slab Weig	75	125	125	125	125	125	125	125	125	125	125
	Drop Widtl	7.5	7.5	7.5	7.5	7.5	7.5	7.5	7.5	7.5	7.5	7.5
	Drop Lengt	7.5	7.5	7.5	7.5	7.5	7.5	7.5	7.5	7.5	7.5	7.5
	Drop Deptl	0.458333	0.458333	0.458333	0.458333	0.458333	0.458333	0.458333	0.458333	0.458333	0.458333	0.458333
	Column W	2.166667	2.166667	2.166667	2.166667	2.166667	2.166667	2.166667	2.166667	2.166667	2.166667	2.166667
	Column Le	2.166667	2.166667	2.166667	2.166667	2.166667	2.166667	2.166667	2.166667	2.166667	2.166667	2.166667
	Pu (Kips)	123.7096	336.0698	548.43	760.7902	973.1504	1185.511	1397.871	1610.231	1822.591	2035.796	2249.002
	Mx (K-ft)											
	My (K-ft)											
	Unbraced (	12.5	12.5	12.5	12.5	12.5	12.5	12.5	12.5	12.5	13.5	13.5
	Column Si	16 - #8	16 - #8	16 - #8	16 - #8	16 - #8	16 - #8	16 - #8	16 - #8	20 - #11	20 - #11	20 - #11

The following is a list of other design criteria that was used for the concrete columns at Fordham Place:

- Minimum Reinforcement Ratio = 0.01
- Maximum Reinforcement Ration = 0.08
- Minimum Clear spacing between bars = 1.5”
- Minimum Clear cover = 0.75”
- Minimum bar size = #8
- Maximum Bar Size = #11

Longitudinal reinforcement in columns at a minimum is 12 - #11’s. This is the next smallest reinforcement ratio = 0.014 > 0.01.

Tie reinforcement was designed to conform to ACI 10.16.8.1 through 10.16.8.8. Bar sizes will be #3'and #4's where longitudinal reinforcement bar size is #8's and #11's, respectively. The spacing of ties was determined from the least of the following three criteria from ACI 10.16.8.5:

- $16 \times d_{longitudinal\ bar} = 16(1") = 16"$
- $48 \times d_{tie\ bar} = 48(.375") = 18"$
- $0.5 \times$ column dimension $= 0.5(26) = 13"$

Since the maximum spacing of tie reinforcement was controlled by the column dimension, and the columns are sized the same throughout the entire building, ties throughout the columns will be spaced the same. Furthermore, since the maximum spacing is just 13", tie reinforcement will be spaced at 12" for convenience purposes.

Shear Walls

When a floor system is changed from composite steel beams to an all concrete structure, the original lateral system of braced frames need to be re-evaluated to some kind of concrete system such as shear walls or moment frames. I decided to treat my columns as supporting gravity load only, and therefore the shear walls will be the sole lateral force resisting system. The starting point for designing the lateral system was first to determine the weight and the seismic characteristics of Fordham Place. Then I was able to

compare the new seismic forces to the wind forces determined in Tech 1. The extra weight of the building caused the seismic loads to control the design. The following table shows the seismic characteristics determined in accordance with ASCE 7-02. For building weight see appendix.

Seismic Analysis

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)
Ordinary Reinforced Concrete Shear Walls

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 = 1.07
Cs = 0.03551

Seismic Design Category B

Effective Seismic Weight of Structure (9.5.3)
W_{TOTAL} = 28004 k

Seismic Base Shear (9.5.5.2)
$V = C_sW$
V = 994 k

Level	w_x (k)	h_x	$w_xh_x^k$	C_{vx}	F_x(k)
B	0	0	0	0	0
Mezz.	1068	14.5	15486	0.00492	5
2	2477	34.25	84837.25	0.026953	27
3	2577	50	128850	0.040936	41
4	2286	63.75	145732.5	0.046299	46
5	2286	77.5	177165	0.056285	56
6	1691	91	153881	0.048888	49
7	1691	104.5	176709.5	0.056141	56
8	1691	117	197847	0.062856	63
9	1691	129.5	218984.5	0.069572	69
10	1691	142	240122	0.076287	76
11	1691	154.5	261259.5	0.083002	83
12	1691	167	282397	0.089718	89
13	1691	179.5	303534.5	0.096433	96
14	1691	192	324672	0.103149	103
15	1322	204.5	270349	0.08589	85
roof	764	217	165788	0.052671	52
Σ=	27999	Σ=	3147615		

After determining the story forces located at each floor level, lateral forces were distributed based on stiffness of each shear wall. Since my shear walls are not at the face of the building, the floor slab will have to axially transfer lateral loads on the building to the shear walls. Once the lateral forces reach the shear walls, they will act as point loads on the shear walls. To design the shear walls, I treated the wall as a gigantic cantilever beam with numerous point loads. The following is a diagram of the most severely loaded shear wall showing lateral forces on the wall. However, every shear wall will be designed the same for simplification purposes.

ARIC HEFFELFINGER
FORDHAM PLACE
BRONX, NY
STRUCTURAL OPTION
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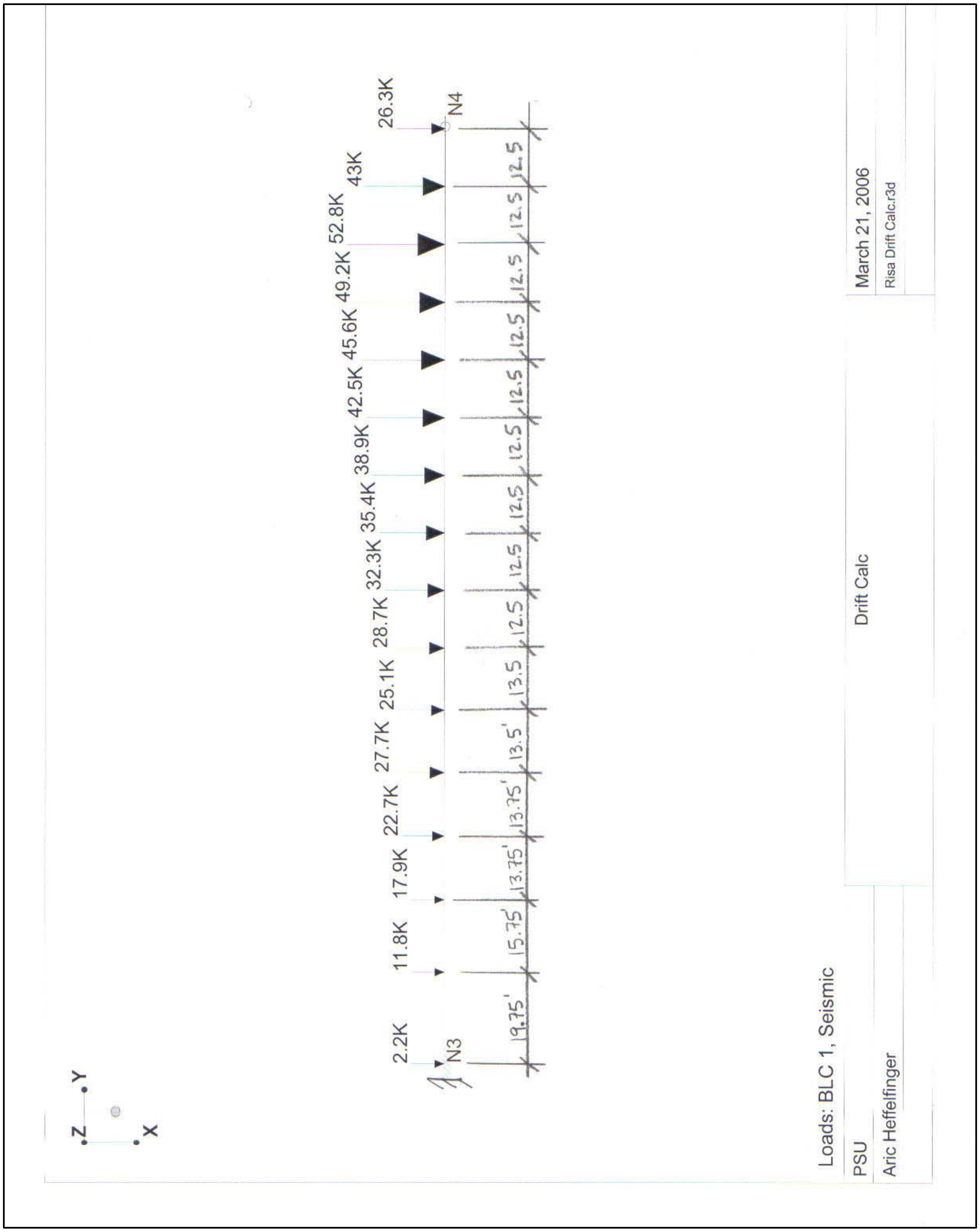
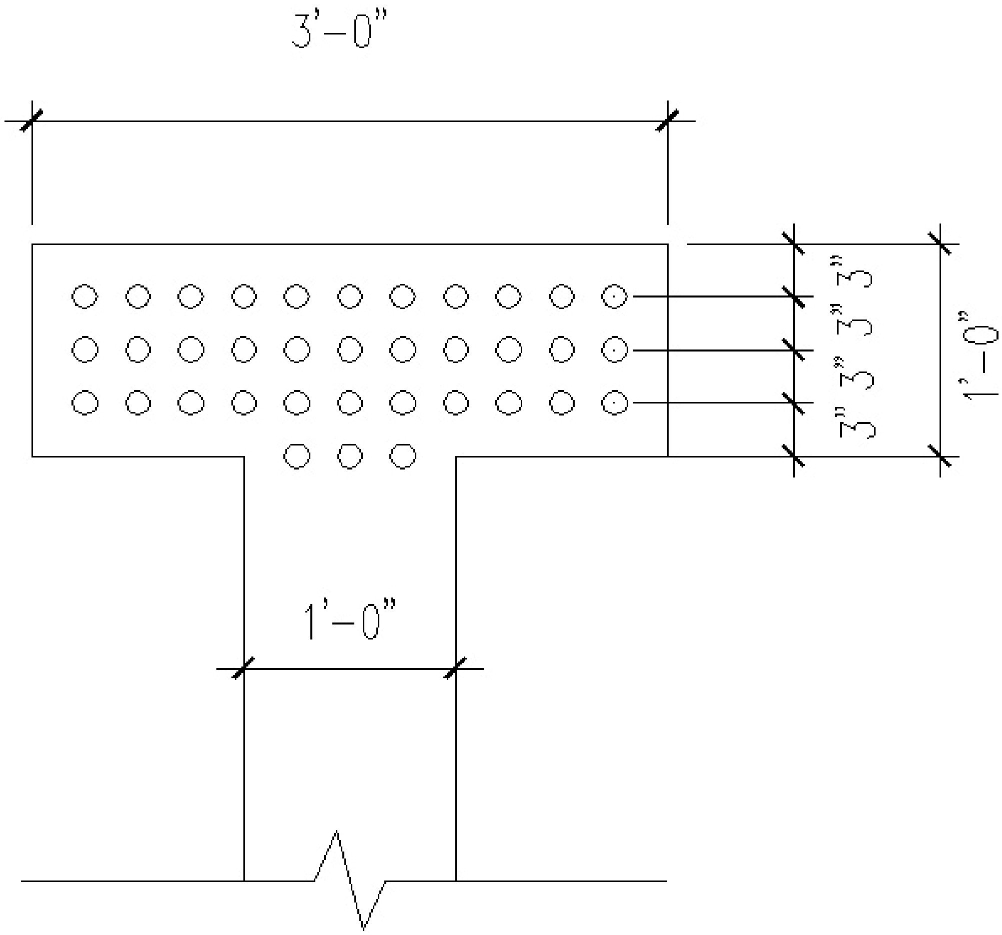


At this point I was able to determine shear and moment diagrams. The max shear was determined to be 502k and was located at the fixed based of the “cantilever beam”. The final design of shear reinforcement in the wall was #5’s at 12” for the first 1/3 of the building height. The second third will contain #5’s at 24”, while the last third will not require shear reinforcement. When I move on to designing the flexural reinforcement, I discovered I would need a lot more steel than I had originally estimated. (As = 53.7in^2) With using a 12” shear wall, it was merely impossible to stuff this steel into the end of the wall with only 1ft width. From here I decided to use a flanged shear wall. The flanged shear wall consisted of the exact same design, but allowed me to fit all the steel in a reasonable configuration. The dimensions of the flanged section are 3ft flange width with a flange thickness of 1ft. There will be 3 rows of 11 - #11’s within the flange while 1 row of 3 - #11’s are just inside the web. See picture below



With 36 - #11's, this gives $A_s = 56.2\text{in}^2 > 53.7\text{in}^2$. See appendix for complete shear and flexural reinforcement calculations. Building drift calculations were determined by taking the most severely loaded shear wall and determining its deflection, and then extrapolating to get the drift of the building corner. This value was then compared to H / 400. To find the drift of the shear wall, I once again treated the shear wall as a cantilever beam, and then used the deflection equation from the Manual of Steel

Construction, Load and Resistance Factor Design, Third Edition, Table 5-17.

$$\Delta = Pb^2(3\ell - b) / 6EI$$

Where, P = Force on beam

b = distance from point load to fixed end

ℓ = length of beam

E = Modulus of Elasticity of concrete

I = Moment of inertia of cross section

Method of superposition was utilized by determining the deflection due to each load and then summing the total up. Calculations of deflections can be seen in the following table.

Load (K)	b (ft)	Δi (in)
8.85	19.75	0.003369325
13.425	35.5	0.016070886
17.025	49.25	0.038282572
20.775	63	0.074557751
18.825	76.5	0.097146053
21.525	90	0.149834326
24.225	102.5	0.213439935
26.55	115	0.287170105
29.175	127.5	0.378046531
31.875	140	0.485021138
34.2	152.5	0.600966102
36.9	165	0.738210665
39.6	177.5	0.890908581
32.25	190	0.807172183
19.725	202.5	0.54399496
	Δtotal =	5.324191114

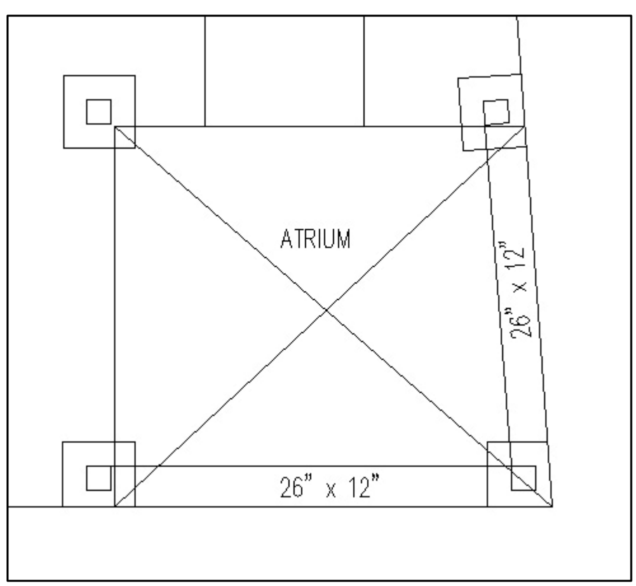
5.32 in < h/400
5.32 in < 6.07 in OK

Special Areas throughout Building

There are a couple different areas throughout the building that required a little extra attention and also a modification to the standard designs. These areas comprise of an atrium space on second floor, a mezzanine floor that resulted in columns with large unbraced lengths, and a large span in the floor slab.

Atrium space on the second floor

The problem with the atrium space is that it is at a corner of the building, which means there is no floor slab to laterally support the columns. To resolve this problem, I designed 26” x 12” beams to span from the corner column both adjacent columns. These beams reduce the unbraced length of the columns and in turn dramatically increase the capacity of the columns. This area can be seen on the following diagram.



Mezzanine floor / columns with large unbraced lengths

There is a mezzanine floor between the ground and second floors that covers only about ¼ of the building footprint. This makes about ¾ of the columns be designed with a large unbraced length. The typical 26" x 26" 4ksi column did not have the capacity to carry required loads with this large unbraced length. However since there were a few columns that carried extremely large axial loads and required 8ksi concrete, this gave me another option to look at. The question was then; would these columns have sufficient capacity using 8 ksi concrete? After running a few of the critical columns in PCA Column with 8ksi concrete, I was able to determine that yes, the 8ksi concrete did provide enough capacity for the given unbraced length.

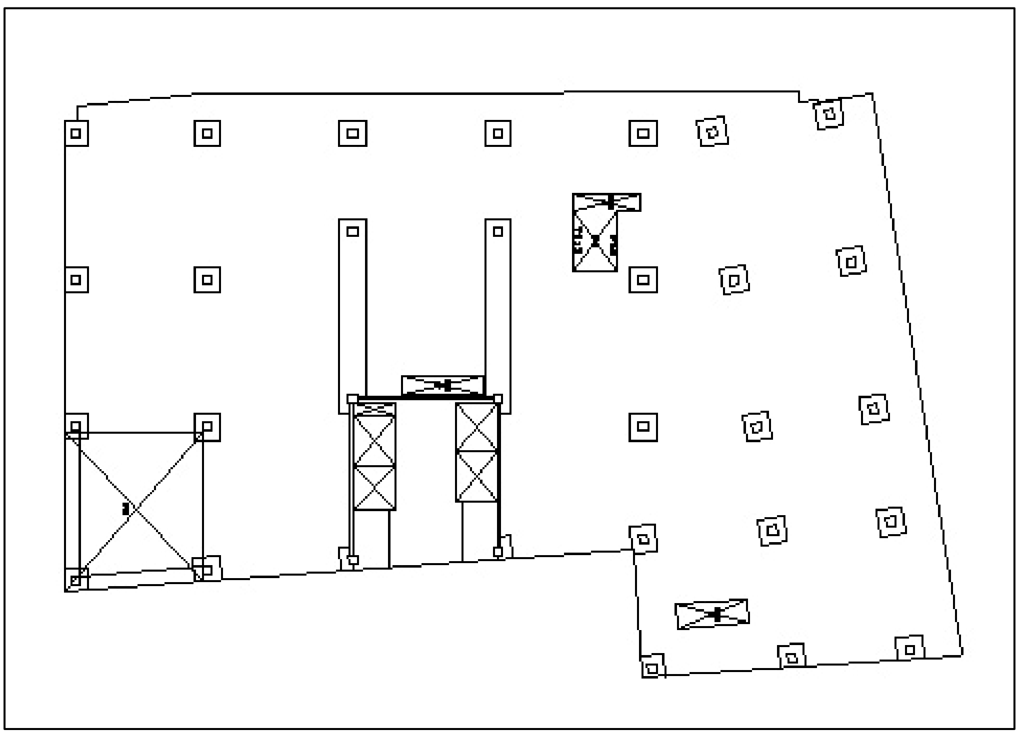
Large span in floor slab

There are two 32' – 2" spans on every floor that are larger than the typical 28' span. I could have just designed the entire building thicker slab that would be sufficient for a 32' span, however once you get over about 30', a two way slab is not very efficient. A common practice when there are one or two larger spans within the building is to use a continuous drop from column to column. This is precisely what I ended up doing. The contractors forming the concrete will just form the drop from one column to the next which will essentially make that part of the slab have a thickness of

$t = 9" + 5.5"$

$= 14.5"$

Reinforcement will be placed at 0.75in from tension face. Although this will require a bit more concrete, it is a far better solution than to just design the entire system based on a typical 32' span. See picture below for specified spans.



Foundations

Final designs of foundations were not completed for the original design, therefore will not be done as a redesign. However is understood that with and increased building weight, there will be a need for larger foundations and in turn be an increase in overall building cost.