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Structural
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Office Building, Washington, DC*
Technical Assignment 1
10/5/05

All renderings courtesy of Dreyfus Property Group and KRJDA

Executive Summary

An in-depth description was done for an office building in downtown Washington, DC. It is 12 stories above grade and sits in an urban environment. The structure was found to be a mix of concrete systems. The below grade floors are 2-way flat plate concrete slabs. There are drop panels on some below grade columns and most columns on the occupied floors. The slabs on the upper floors are post-tensioned with a 20’ exterior cantilever on 3 sides. Lateral load analysis was done to determine storey shears due to wind and seismic loading. A snow load analysis was done on the roof and aided in a spot check of a rectangular column due to gravity loads. On the lower level a 2-way slab span was also spot checked. The building is governed by IBC 2000 with a 2003 Washington DC code supplement. Concrete design complies with ACI code specifications. This gave a valuable insight into the building design but much more thorough analysis will be needed in future investigations.

*Building specifics withheld at owners request

Structure Summary

The general system used in gravity load resisting is a mixture of concrete floors systems and load bearing reinforced concrete columns. The below grade floors are flat plate concrete slabs with drop panels, and above grade Post-tensioned floors slabs were utilized. Lateral wind and seismic forces are resisted primarily by the rigid moment frames created by the concrete superstructure and to a lesser extent by smaller shear walls that make up the elevator core.

Foundation/S.O.G.

The building has 3 levels of below grade parking. The bottom-most level rests on a traditional reinforced slab on grade. The slab is a minimum 4” thick in parking areas and a minimum 5” thick at the ramp. The parking area is designed for 50psf LL. The slab on grade is 4ksi concrete reinforced with 6x6 – 10x10 WWF. There is a 15” deep strip footing around the perimeter that has a cropped toe as per proximity to the property line. The SOG is thickened an additional 6” (from 4” to 10”) where non-load-bearing exist above. At the elevator pit, the slab is thickened to 8” but additionally has a 12” deep strip footing. The load bearing concrete columns terminate into reinforced concrete pier footings. Around the perimeter of the building, at the outermost columns, these piers vary in thickness from 59” up to a full 75” of depth. The remaining interior columns see footings of 40” to 61”.

Below grade structure

Above the foundation and level 3 parking, but still below the finish grade, there is a flat plate concrete slab. This reinforced slab also employs a few drop panels to maintain the slab thicknesses. These drops are either 5-1/2” or 8”. The slab here is 8” thick 5ksi normal weight concrete. Live load is again 50 psf in parking areas. The live load is increased to 125 psf in the interior building/elevator core. This is likely to add

strength for equipment and an increased factor of safety in vital egress zones. The ramp slab is also increased to 14” because of the relative span distances in the area which approach 30’. Columns in this area match columns below and are specified as 10ksi normal weight concrete. LL is dramatically increased in some areas of the uppermost parking level 1 and the lower level floor. The LL varies from 100 psf up to 150 psf. In turn, slab thickness is increased to 9-1/2” until we get above grade. The reason for this is because of the large amount of mechanical equipment that is placed in the lower level on the north side.

<u>Ground floor and above</u>

The reinforced concrete slab on the ground floor varies from 8” to 10” to 12” thick and is 5 ksi normal wt. concrete. Ground floor columns are still 10 ksi n.w. concrete. Above the first floor things get very interesting. There is a 20’ cantilever on the three street facing sides of the building (west, south, and east). The slab is now a post-tensioned concrete system with folds and cast beams to account for the overhanging load. The slab is 12” except above the loading dock where it is bumped up to 14”. All concrete here must meet a 28 day compressive strength of 6000 psi (6 ksi). Also, 12” thick drop panels are designed around 1st floor columns. Column C/C distance is typically 20’ and there is a 20’ cantilever around 3 sides of the building. This span is doubled to approximately 40’ between the outer set of columns and the core set. Design live load is a constant 100 psf for all office and corridor areas, which make up the majority of the upper floors. Floors 6 through 12 are all duplicates of one another. Floors 2 through 5 only differ from those in that they include the 5 story atrium space. All concrete is 6 ksi in the post-tensioned slabs. Slabs are typically 12” thick with 12” drops around major columns. The only noteworthy change is that from floor 3 to 6 the column concrete can be reduced to 8 ksi strength from the 10 ksi used below. On floors 7 – 10, the f’c in the column concrete drops again to 6 ksi. Finally from the 11th floor and up, only 4 ksi concrete is required for columns. The roof slab is very similar to the other PT slabs. It is 12” 6 ksi n.w.c. There are special considerations for (3) 16,000 lb

emergency generators and (2) elevator hoists weighing 12,500 lbs each. Most columns end here but a few continue up to support the penthouse. The penthouse roof will be an 8” reinforced concrete slab and will support more equipment such as (3) 23,000 lb cooling towers. There is also an additional 30 psf superimposed dead load due to a green roof.

Codes

General

2003 Supplements – The District of Columbia Construction Codes
2000 ICC – International Building Code
2000 ICC – International Plumbing Code
2000 ICC – International Mechanical Code
2000 ICC – International Fire Code
2000 ICC – International Property Maintenance Code
2000 ICC – Fuel Gas Code
2000 ICC – Energy Conservation Code
1996 NFPA National Electric Code
1998 ICC/ANSI A117.1 – Accessible and usable buildings and facilities
1991 ADA – Accessibility guidelines
1996 ASME – Safety code for elevators and escalators

Concrete

AASHTO M171 – Sheet materials for curing concrete
AASHTO T237 – Testing epoxy resin adhesive
AASHTO T260 – Test for sampling and testing for total chloride ion in concrete and concrete raw materials.
AASHTO T318 – Measurement of water content of fresh concrete using the microwave oven.
ACI 117 – Standard specifications for tolerance of concrete construction and materials.

ACI 301 – Standard specification for structural concrete.

ACI 302.1R – Guide for concrete floor and slab construction.

ACI 304R – Guide for measuring, mixing, transporting, and placing concrete.

ACI 308 – Standard practice for curing concrete.

ACI 315 – Details and detailing of concrete reinforcement.

ACI 318 – Building code requirements for structural reinforced concrete.

ACI 347R – Guide to framework for concrete.

ACI 360 – Design of slabs on grade.

Various ASTM standards.

AWS D1.1 – Structural Welding Code, Steel.

AWS D1.4 – Structural Welding Code, Reinforcing steel.

AWS D12.1 – Recommended practices for welding reinforcing steel metal inserts and connections in reinforced concrete construction.

CRSI – Manual of standard practice

CRSI – Placing reinforcing bars

PCI – Standard building code for prestressed concrete

PTI – Post-tensioning manual

Wind/Seismic/Snow Design

<u>Snow load</u>

Roof snow load (Pg) = 25 psf - DC

Snow exposure factor (Ce) = 0.9

Importance factor (I) = 1.0

Thermal factor (Ct) = 1.0

Pf = 0.7(0.9)(1.0)(1.0)(25 psf) = 15.75 psf

Pf min = I(20psf) = 20psf

Pf = 20 psf

The design team specified a roof snow load of Pf = 21 psf. This is likely due to the use of a Pg = 30 psf because using the figures in ASCE 7-02 DC is located near 30 psf design area, or due to allowances for drifting.

Wind load

Velocity = 90 mph

Exposure class B

Importance factor (II) = 1.0

Effective Building Size

B=200’ L=200’ h=128.56’

Design method 2 was used because this building does not meet the criteria for simplified design. Also the building is essentially equal in both dimensions, and therefore results apply to both N-S and E-W directions.

Building height data

Storey	Sea Level	Local	Storey Ht
12th	187.81	128.56	11.69
11th	176.13	116.88	11.68
10th	164.44	105.19	11.69
9th	152.75	93.50	11.69
8th	141.06	81.81	11.69
7th	129.38	70.13	11.68
6th	117.69	58.44	11.69
5th	106.00	46.75	11.69
4th	94.31	35.06	11.69
3rd	82.62	23.37	11.69
2nd	70.94	11.69	11.68
1st	59.25	0.00	11.69
LL	46.50	-12.75	12.75
P1	36.00	-23.25	10.50
P2	25.20	-34.05	10.80
P3	16.16	-43.09	9.04

Shear

Shear Location	Value
12th floor	52.21
11th floor	52.21
10th floor	88.46
9th floor	106.12
8th floor	104.55
7th floor	105.50
6th floor	104.56
5th floor	99.88
4th floor	97.23
3rd floor	95.23
2nd floor	165.14
1st floor	176.25
Ground	49.43
Base Shear	1,519.12
Overturning Moment	80,933.22

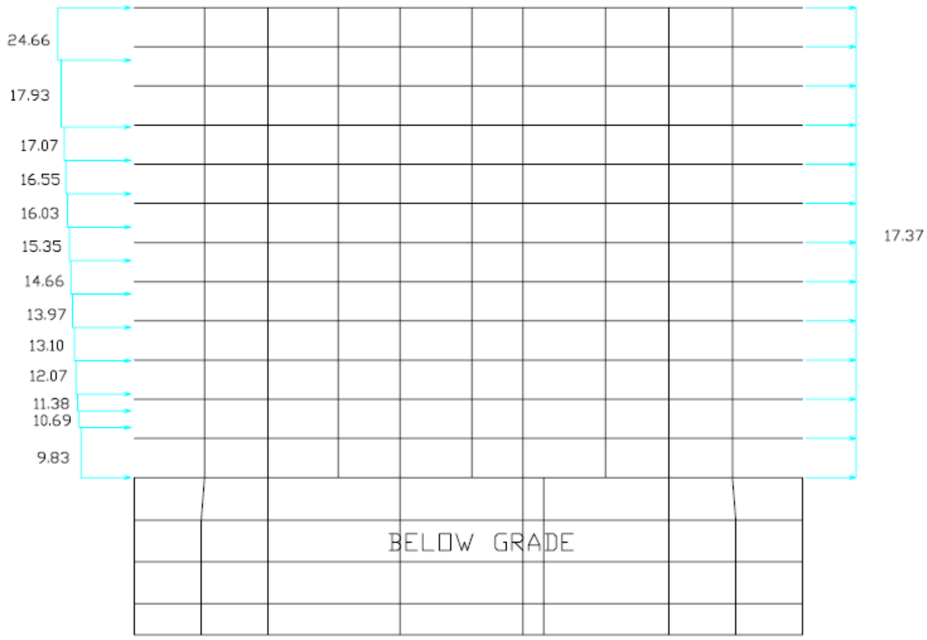
Velocity Pressure

	Windward		Leeward		Max
Z(ft)	N-S	E-W	N-S	E-W	(psf)
0-15	9.83	9.83	-17.37	-17.37	27.19
20	10.69	10.69	-17.37	-17.37	28.06
25	11.38	11.38	-17.37	-17.37	28.75
30	12.07	12.07	-17.37	-17.37	29.44
40	13.10	13.10	-17.37	-17.37	30.47
50	13.97	13.97	-17.37	-17.37	31.33
60	14.66	14.66	-17.37	-17.37	32.02
70	15.35	15.35	-17.37	-17.37	32.71
80	16.03	16.03	-17.37	-17.37	33.40
90	16.55	16.55	-17.37	-17.37	33.92
100	17.07	17.07	-17.37	-17.37	34.44
120	17.93	17.93	-17.37	-17.37	35.30
128.56	24.66	24.66	-17.37	-17.37	42.02

*Negative pressure indicates suction

Pressure Distribution

Storey	Storey h (ft)	Z (ft)	V (k)	M (ft-k)
12	11.69	140.28	176.25	7,323.90
11	11.69	129	165.14	6,713.57
10	11.69	116.90	95.23	10,340.70
9	11.69	105.21	97.23	11,164.59
8	11.69	94	99.88	9,777.82
7	11.69	81.83	104.56	8,632.87
6	11.69	70.14	105.50	7,333.72
5	11.69	58.45	104.55	5,838.28
4	11.69	46.76	106.12	4,546.65
3	11.69	35	88.46	3,339.78
2	11.69	23.38	52.21	3,860.96
1	11.69	11.69	52.21	2,060.37
Σ			1,519.12	80,933.22



E-W WIND DISTRIBUTION

Seismic design

Sds = 0.143

Sd1 = 0.0713

Seismic use group 1

Seismic design category B

Site class C

Cd = 2.5

R = 3

System over-strength factor = 3

Ordinary reinforced concrete moment frames

Equivalent lateral force procedure

Summary						
Level	w_x	h_x	$w_x h_x^k$	C_{vx}	F_x (kips)	M_x (ft-kips)
12	1,116.64	140.28	3,441,766	0.08511	4.2	12,707
11	2,940.28	128.59	7,867,747	0.19456	207.1	26,627
10	2,940.28	116.90	6,738,873	0.16664	177.4	20,734
9	2,940.28	105.21	5,678,470	0.14042	149.5	15,724
8	2,940.28	93.52	4,689,305	0.11596	123.4	11,542
7	2,940.28	81.83	3,774,606	0.09334	99.3	8,129
6	2,940.28	70.14	2,938,211	0.07266	77.3	5,424
5	2,940.28	58.45	2,184,809	0.05403	57.5	3,361
4	2,940.28	46.76	1,520,319	0.03759	40.0	1,871
3	2,940.28	35.07	952,597	0.02356	25.1	879
2	2,940.28	23.38	492,902	0.01219	13.0	303
1	2,940.28	11.69	159,804	0.00395	4.2	49
Σ	46,099.72		40,439,410	1	1,064.3	107,351

Fig. 1. Southwest Corner Below Grade Column Placement

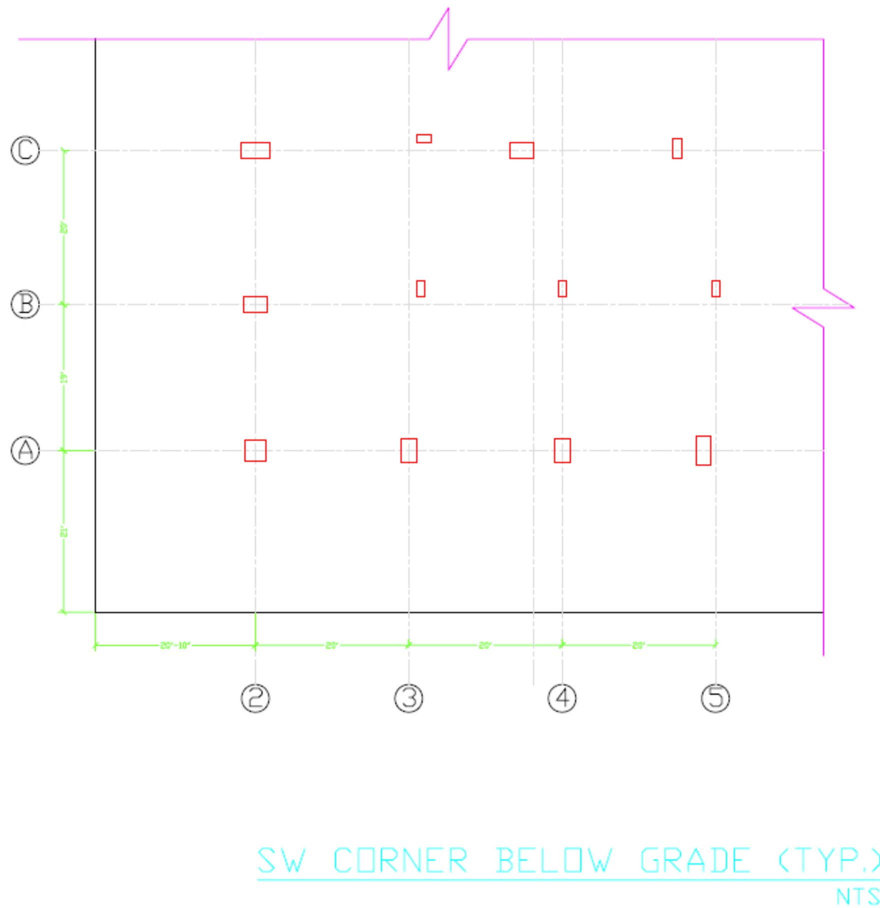
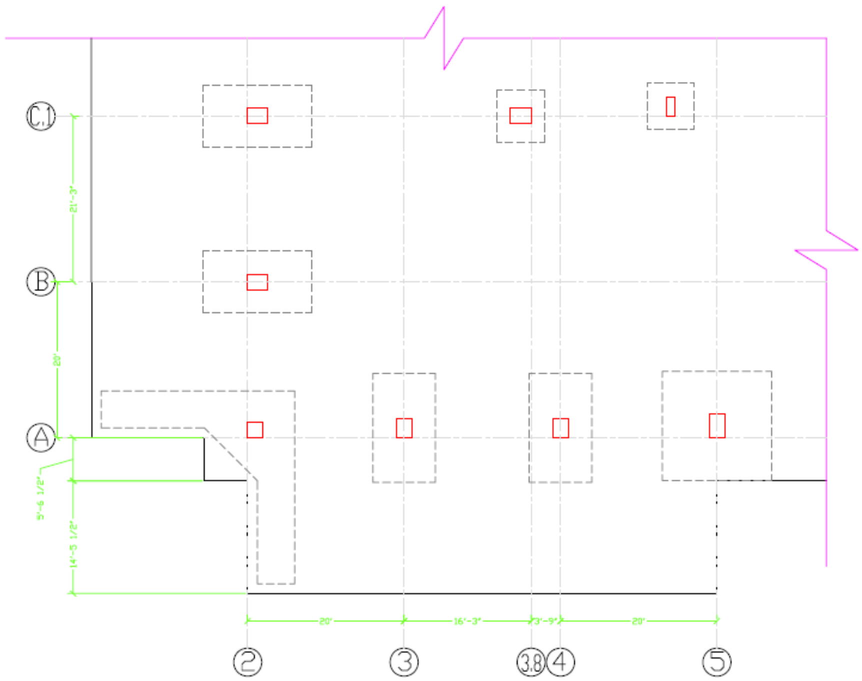


Fig 2. Southwest Corner Column and Drop Panel Placement

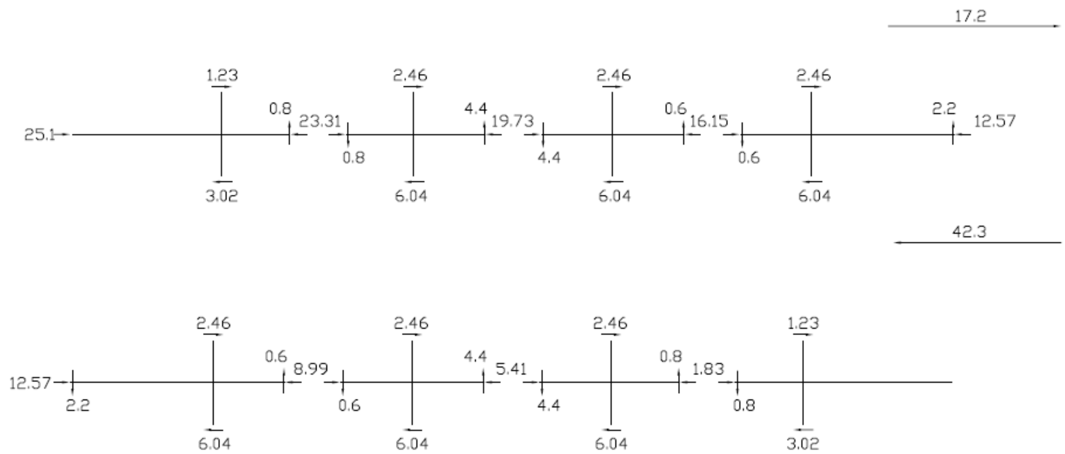


SW CORNER FLOORS 6-12 (TYP.)
NTS

This is as close to a typical "bay" that exists in this office building. The columns are spaced roughly at 20' intervals along a 20' setback from the façade. There is a 40' span along column line A after column A-5. In parking levels there are additional infill columns to help dissipate the added load from the heavy concrete structure.

Lateral Analysis by Portal Frame

The seismic storey shear was analyzed on a typical office floor.



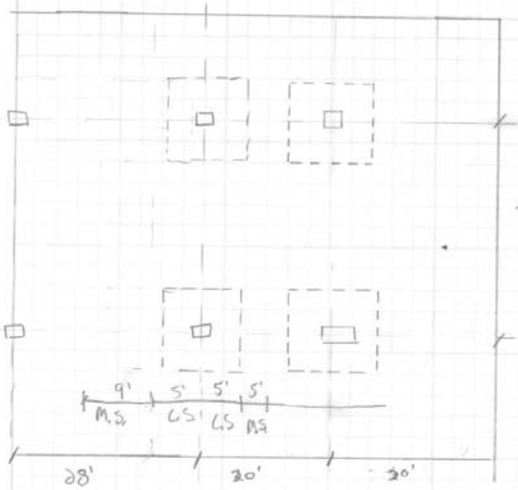
PORTAL FRAME ANALYSIS AT 10TH FLOOR
SPLIT AT CENTER BAY ALL LOADS IN KIPS

Spot Checks

Ideally one would analyze the slab and post tension system at the floor where the portal frame analysis was done. I do not yet have this design knowledge so the post tension will have to be postponed.

A 2-way reinforced concrete slab was analyzed on the lower level. This area is to be used as office space. A similar slab thickness and drop depths were determined to the ones in the plans. The reinforcing was a bit heavier than the designed bars. This could be due to an over-estimated superimposed dead load and structure self weight. The dimensions of the drops were not as large as those designed. This may be because of another area where the drops were required to be a larger size and were kept the same for the sake of uniformity.

LOWER LEVEL 2-WAY SLAB W/ DROP PANEL



$f'_c = 5$ ksi
$f_y = 60$ ksi
LL = 80psf + 20psf
= 100psf
SDL = 5psf MEP
20psf FINISHES
= 25psf

DROP SIZE = $\frac{1}{6} \ell_n$
USE 5x5 drops.

30' : 20' ≤ 2:1

3 OR MORE SPANS
UNIFORMLY DISTRIBUTED GRAVITY LOAD
- USE DIRECT DESIGN METHOD FOR 2-WAY SLAB

MIN SLAB thk PER ACI 9.5.3 = $\ell_n/36$ = 6.75"
USE 8" SLAB W/ 8" DROPS (MIN DROP = $\frac{1}{4}$h = 2" < 8" ok)
SELF WT = 150pcf × 1' (AVG DEPTH) × 24' (TRIB WIDTH) = 3.6 KLF
$W_u = 1.2(.6 + 3.6) + 1.6(2.4) = 8.88$ Kip/Ft

MIN S+T REINFORCEMENT (1 FT STRIP)

$A_s = 0.0018\,A_g = (8")(12"/ft)(.0018) = .1728\ in^2$

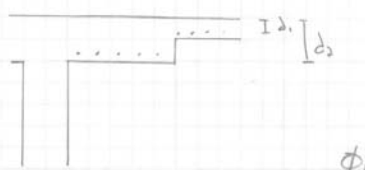
USE MIN #4 @ 12" T+B $A_s = .2 > .1728$ ok

STATIC MOMENT

$M_o = \dfrac{(888\ k/ft)(24')(29')^2}{8} = 22404\ ft\text{-}k$!

	TOTAL M_u	TOTAL WIDTH	MOMENT / FT
INTERIOR SUPPORT (-) 65%	C.S. 75% 10921	10 ft	1092.1
14562	M.S. 25% 3640	14 ft	260
MIDSPAN (+) 35%	C.S. 60% 4704	10 ft	470
7841	M.S. 40% 3136	14 ft	224

TRY #8 @ 12"



$d_1 = 8" - .75" - 1" - 1/2" = 5.75"$

$d_2 = 16" - .75" - 1" - 1/2" = 13.75"$

$a = \dfrac{(.79)(60)}{.85(5)12} = .93$

$\phi M_n = .9(.79)(60)\left(d_{1,2} - \dfrac{.93}{2}\right)$

$\phi M_{n_1} = 225.4$ ft-k/ft

$\phi M_{n_2} = 566.7$ ft-k/ft

USE #8 @ 12" O.C. For MIDSPAN MS.
USE #8 @ 8" O.C. For SUPPORT M.S.
USE #8 @ 12" O.C. For MIDSPAN C.S.
USE #8 @ 6" O.C. For Support C.S.

Also analyzed were gravity loads on an upper story column. The column size determined was slightly smaller than the actual column size, but contained slightly more reinforcing.

GRAVITY LOAD DESIGN ON COLUMN #4 @ 10th FLOOR

COLUMN HEIGHT = $11.69' - 1' = 10.69'$

TRIBUTARY AREA = $32.21' \times 38.12' = 1227.84\ ft^2$

$LL = 100\ psf\ (32.21' \times 38.12') = 122.7\ k/floor$

$DL = 150\ pcf \times (32.21' \times 38.12' \times 1') = 184.1\ k/floor$

2 typ floors, 1 roof load

Add'l Roof DL = $20\ psf\ (32.21' \times 38.12') = 24.5\ k$

Total column load = $1.2(184.1(3) + 24.5) + 1.6(122.7(2))$

$= \underline{1084.8\ k}$

TRY 24"×24" w (4) #8 E.F.

$A_s = .79(4) = 3.16\ in^2$

$.79(2) = 1.58\ in^2$

$P_o = .85 f'_c (bh - \Sigma A_{si}) + \Sigma A_s f_{si}$

$\frac{\phi P_n}{A_g} = \frac{1084/.75}{24^2} = 2.5$ plot on ACI interaction diagram

✓ OK, use 24"×24" column w/ (12) #8