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Structural Option

Pennsylvania Judicial Center

Harrisburg, PA

Technical Report #1

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Executive Summary

The purpose of this report is to research the methods used to design the structural system for the Pennsylvania Judicial Center in Harrisburg, PA. This report will explain the loads used based on industry standards and codes. The primary codes that I used in my analysis are the International Building Code 2003 and ASCE 7-05.

The Pennsylvania Judicial Center has a steel frame with composite floor slabs. The building resists lateral loads using concentrically braced frames between the floor slabs. The frames use stiffness in the plane of the lateral load and act similar to a truss to transfer the loads to the columns, which then transfer the loads to the foundation below.

A seismic analysis was conducted by first using index force analysis to ensure that the at least the order of magnitude of the calculations was appropriate. This helped to provide assurance that the building weight was reasonable. Then, the much more accurate and detailed equivalent lateral force method was used for increased accuracy. The loads were distributed to the floors by the equations given in code, and the base shear was calculated and compared to the given design base shear.

A full wind analysis was also conducted using Method 2 on the overall main wind-force resisting system. The pressures on the wall were distributed as loads on the floor slabs based on tributary areas. Based on the height and surface area of the structure, and its classification in the relatively mild Seismic Zone B, it was expected that the wind force would be the controlling factor in the lateral system design.

The lateral system also received considerable attention in this report. Five typical sections were sketched using two bays each. These sections were analyzed in RAM Advanse as simply supported bays with a unit lateral load acting on the top corner. The computer model showed how the frames deflected, and the maximum deflections of each were used in calculating relative stiffness. These relative stiffnesses were used to calculate the distribution of the seismic and wind loads, by floor, to the frames.

Finally, spot checks were performed on a typical composite beam and an exterior column to check the accuracy of my calculations as well as the cohesion of my work to that of the professional designer's. The composite beam was found to be optimized to code requirements based on the loads used. The column was found to be appropriately designed when considering the interaction between the amounts of compressive strength used and moment capacity used, since a large girder framed into one side, and there was no beam to counter this eccentric force on the other side.

Structural System Overview

Floor system:

The typical floor is supported by a composite system. The concrete is lightweight (110 pcf dry unit weight) and has a minimum 28-day strength of 4000 psi. There is 3½” of concrete above a 3” 18-gage galvanized composite cellular metal deck, which makes a total slab depth of 6½”. Typical reinforcement is welded wire fabric, 6x6-W2.9xW2.9. The slab is supported by steel beams with typical sizes ranging from W16x36 to W24x68. Typical spans run as long as 42 feet, and the widest spacing between beams is typically ten feet. Composite action is created by ¾” diameter shear studs with 5½” length.

Roof system:

The flat roof system is identical to the typical floor system. The sloped monitor roof on the ninth-floor tower has a 3” 20-gage galvanized metal deck. The roof is supported by sloped beams ranging from W8x10 to W12x19, with spans no longer than 25 feet and a 9’ maximum spacing. The monitor above the main atrium features the same deck, but it is supported by bent W30x90 beams spanning 56’ and spaced at ten feet o.c.

Lateral system:

The structure is laterally supported by concentrically braced steel frames in both the N-S and E-W directions. These frames consist of the wide flange columns, wide flange beams at each story and two HSS (hollow structural section) diagonal braces between each story. The geometry of the diagonal members vary, and this has an impact on their relative stiffnesses. This lateral system features no moment connections, and relies on concrete floor and roof slabs to act as rigid diaphragms and distribute the lateral loads accordingly.

Foundation:

The slab on grade concrete is normal-weight (145 pcf dry unit weight) and has a minimum 28-day strength of 5000 psi. The slab on grade is fiber-reinforced at not less than 1.5 lb/yd^3 in some areas and is reinforced with #3 bars @ 18” c/c in the rest of the slab. Typical slab thicknesses are 5” with 6” drainage fill and 8” with 8” drainage fill. Column loads of up to 1,000 kips can be supported using concrete piers with diameter of up to eight feet end bearing on rock. Larger column loads are supported by socketed caissons with diameters up to 4.5 feet with up to 18’ depth. The piers will bear on grey limey shale bedrock with a bearing capacity of 30 ksf. The median core depth to reach bedrock was 9.5 feet, and bedrock depth is relatively uniform throughout the site. The concrete basement foundation walls will be supported by continuous wall footings.

Columns:

The columns are ASTM A992 Grade 50 wide flange steel shapes laid out in a mostly rectangular grid. In this system the columns are acting as the primary gravity resistance members. The columns that are attached as braced frames are also the main lateral resistant force members. The braces between columns are ASTM A 500 Grade B HSS shapes ranging in size from 8x8x1/2” to 12x12x5/8”. The largest column is a W14x550, though most of the columns are on the order of about 300 lb/ft at the ground floor.

Codes

Codes Employed for Original Design:

- DGS, Bureau of Engineering and Architecture Project Procedure Manual
- International Building Code, 2003 Edition
- American Society of Civil Engineering (ASCE)
  - ASCE –7-02, Minimum Design Loads for Buildings and Other Structures
- American Institute of Steel Construction (AISC)
  - Specifications for Structural Steel Buildings – Load and Resistance Factor Design – LRFD 1999
  - Specifications for Steel Hollow Structural Sections – Load and Resistance Factor Design – HSS 2000
  - Seismic Provisions for Structural Steel Buildings – 341-02
- American Concrete Institute (ACI)
  - ACI 318-02, Building Code Requirements for Reinforced Concrete.
- American Concrete Institute (ACI)
  - ACI 530-02, Building Code Requirements for Masonry Structures.
- Steel Joist Institute (SJI)
  - Standard Specifications, Load Tables, and Weight Tables for Steel Joist and Joist Girders"-1994.
- Government Services Administration
  - Progressive Collapse Analysis and Design Guidelines
  - PBS-P100 Chapters 4 and 8

Code Substitutions for Thesis Design:

- American Society of Civil Engineering (ASCE)
  - ASCE –7-05, Minimum Design Loads for Buildings and Other Structures
- American Institute of Steel Construction (AISC)
  - Specifications for Structural Steel Buildings – Unified Design – 2005

Loads

Floor Live Loads:

Load Area	**Building Design Load**	**Minimum Load, ASCE 7-05**
Corridors	125 psf	100 psf, first floor 80 psf, all other floors
Offices	125 psf	50 psf
Courtrooms	60 psf + 20 psf partition	60 psf, if seats are fixed
Lobbies and Stairs	125 psf	100 psf
Storage Rooms	125 psf	125 psf for light storage (warehouse)
Archive Storage Room	250 psf	250 psf for heavy storage (warehouse)
Conference Center	125 psf	100 psf (assembly area)
Library (Stacks)	150 psf	150 psf
Cafeteria	100 psf	100 psf (assembly area)
Mechanical Rooms (fans only)	125 psf	n/a
Mechanical Penthouse	250 psf	n/a
Exterior Plaza	100 psf	100 psf (assembly area)
fire vehicle access area	300 psf	n/a
Parking Garage	100 psf	n/a
Loading Dock	250 psf	n/a

Roof Live Loads:

Item	**Design Value**	**Code Basis**
Roof Live Load	20 psf min	ASCE 7-05
Ground Snow Load (Pg)	30 psf	IBC Figure 1608.2
Flat-roof Snow Load (Pf)	21 psf + drift	IBC Section 1608.3
Snow Exposure Factor (Ce)	1.0	IBC Table 1608.3.1
Snow Importance Factor (I)	1.0	IBC Table 1604.5
Thermal Factor (Cf)	1.0	IBC Table 1608.3.2
Rainwater Ponding Load	30 psf (avg. of 6”)	n/a

Dead Loads:

Item	**Design Value**
Concrete Slab, Typical Floor	50 psf
Superimposed Dead Loads	
Mechanical, Electrical, Sprinkler	20 psf
Ceiling Finishes	5 psf
Floor Finishes	5 psf
Steel Structure	Varies
Other Dead Loads	Where applicable

Analyses and Conclusions

Seismic

Harrisburg, Pennsylvania is not considered a high-risk area for seismic activity by any means. However, due to an increased emphasis on seismic design in the new codes, seismic considerations must be made for almost every new structure constructed in the United States. I determined that the equivalent lateral force method was appropriate and sufficient for a seismic analysis for this area. The seismic coefficients used in the design were provided in the construction documents. I corroborated these data with what was shown in the ASCE 7-05 code and was in agreement with their numbers. The only discrepancy was that the new code suggests a value of 3.25 for R and C_d, rather than 3. I decided that it would be appropriate to use their coefficients in an attempt to keep my analysis in line with the design professional's as much as possible.

Seismic weight typically includes dead load only, but there are code provisions to include percentages of certain live loads. I accounted for this with a relatively conservative uniform dead load, 100 psf. I also added an exterior wall load of 45 pounds per square foot of wall area. I used index force analysis, the simplest possible seismic analysis, to determine if my assumptions were reasonable. The spreadsheet can be found on page 11 of this report. The result was a base shear of 407k, which is relatively close to the design base shear of 640k, given all the assumptions that were made.

When I attempted to find seismic forces using the equivalent lateral force method, the result was a base shear of 1100k, which was considerably farther away from the design base shear for which I was attempting to recreate. Since I used the same coefficients as the design professional, I only thought of two possibilities for the discrepancy: weight and building period. I ruled out weight since the difference in base shears was so great, it would require a radically different weight to approach the same value. This leaves building period; I used the approximate period equation in the code to obtain a period T of 0.89s but found a provision enabling the period to be increased to up to 1.51s. At this longer period, I calculated the base shear to be 655k, almost exactly equal to the design value. I will investigate their value for the period in order to gain a better understanding of the overall idea that went into this structural design.

Wind

Since seismic is usually not a driving factor in this building's region, it will probably be the wind force that controls the design of the lateral resistance system. Therefore, a relatively rigorous wind calculation would be an essential endeavor. For this report, Method 2 will be used to calculate wind pressures on the main wind-force resisting system. If wind is found to control, a components and cladding analysis could also be useful. However, for the purpose of getting a wind load on the overall building for this report, a MWFRS analysis is sufficient.

The building was designed as a partially enclosed structure, and I agree with this assumption due to large areas of curtain wall and the shape of the entrance which could cause strong, focused wind gusts. Designing the building as partially enclosed will result in higher wind loads, which, in turn, can increase your structural requirements. A redesign of the façade for the thesis to create an enclosed condition could be a design possibility.

The first step in the wind calculations was to determine all of the wind coefficients; this work is shown on pages 15-16. An analysis was conducted in each of the two principal directions. The windward and leeward pressures are the essential values for the overall building system. Roof pressure is relatively unimportant for this building, since the uplift will be easily resisted by the heavy, primarily flat roof slab. Side wall pressures may be important to component design or deflection criteria, but for overall system design, they will not control and can be ignored.

A positive pressure on the windward building face and a negative pressure on the leeward face will both occur in the same direction; therefore, their effects can be considered cumulative when discussing overall building criteria such as base shear. For 90 MPH wind acting on the north or south face, the building experiences a 665k windward force and a 600k leeward force, resulting in a 1265k base shear. The east and west faces, which have a smaller surface area normal to the wind, would experience a 577k windward force and a 448k leeward force, for a total of 1025k.

Seismic vs. Wind

A comparison of the base shear of the two types of lateral forces will show that the wind in the N-S direction will clearly control. In the other direction, the wind force was calculated as 1025k, while my initial seismic calculation yielded a base shear of 1100k. However, it has already been mentioned that wind will usually control in this area, the design professional calculated a much lower base shear, and there are simple alterations that could be made to the seismic calculations to get the load to be lower than the wind. I will make the statement that wind controls the design for this building; however, with the advent of sophisticated computer software such as RAM, it is easy to consider all load cases simultaneously; therefore, attempting to determine which type of loading controls is a moot point.

Lateral System

The primary lateral force resistance is achieved using concentric braced frames. All of the frames in this system safely transfer the forces using the same concepts; however, minor differences in geometry can have a large impact on the frame's stiffness. My analysis of the lateral system including constructing models of frame bays in RAM Advanse to test the deflections of the frames. The inverses of the deflections can show the relative stiffnesses of the frames. This method is, of course, an approximation, but I feel that it is a reasonably accurate assumption for the purposes of this report. There were five typical types of frames, and these frame patterns usually repeated from the ground floor to the roof.

The spreadsheets on pages 22-25 show how the lateral loads were distributed by floor to the frames. For this analysis, loads were distributed based on relative stiffness only. Torsional factors were not considered. These load distribution factors were combined with the vertical distributions of seismic and wind loads to attempt to show how the load was distributed by frame, by floor.

Seismic Analysis

Item	Design Value	Code Basis
Hazard Exposure Group	I	IBC Section 1616.2
Performance Category	B	IBC Table 1616.3
Importance Factor (I)	1.0	IBC Table 1604.5
Spectral Acceleration for Short Periods (Ss)	0.21g	IBC Figure 1615 (1)
Spectral Acceleration for a One Second Period (S_1)	0.064g	IBC Figure 1615 (2)
Damped Design Spectral Response Acceleration at Short Periods (S_{DS})	0.168g	IBC Section 1615.1.3
Damped Design Spectral Response Acceleration at Short Periods (S_{D1})	0.073g	IBC Section 1615.1.3
Seismic Response Coeff. (Cs)	0.013	IBC Section 1617.4
Site Class	C (very dense soil)	IBC Table 1615.1.1
Basic Structural System	Building Frames	IBC Table 1617.6.2
Seismic Resisting System	Concentric Braced Frames	IBC Table 1617.6.2
Response Modification Factor (R)	3.0	IBC Table 1617.6.2
Deflection Modification Factor (Cd)	3.0	IBC Table 1617.6.2
Analysis Procedure Utilized	Equivalent Lateral Force	
Design Base Shear	640k	

Seismic Weight:
- Total Dead Load
- 25% of Live Load for Storage
- Include Partition Loads (20psf)
- Equipment Operating Weight
- 20% of flat roof snow load if $P_F > 30psf$

Dead Load for Typical Floor:
50psf Concrete Slab
30psf Superimposed DL (Mech/Elec/Sprinkler/Finishes)
15psf Steel Structure + Metal Deck
5psf Collateral Loads
$\boxed{100psf\ DL}$

Storage Areas:
Light storage: use $0.25 \times 125psf \approx \boxed{35psf}$
Archive storage: use $0.25 \times 250psf \approx \boxed{65psf}$

Partition Loads:
$\boxed{+20psf}$ in courtrooms

Equipment Operating Weights:
As noted on plan.

Snow Load:
$P_f = 21psf$ + drift, which will occur. The flat roof portions feature the same DL as typical floors, and sloped roofs have a DL of just 30 psf. Therefore, I will make the assumption that the whole roof is flat and has 100 psf DL, which should be a conservative assumption based on comparative areas.

Areas:
East Wing "A": $60' \times 90' = 5400 ft^2$
Tower "B": $(110' \times 200') + 2(5' \times 60') + (5' \times 110') + (9' \times 90') = 24000 ft^2$
West Wing "C": $(110' \times 40') + (155' \times 120') + (87' \times 30') - 2(\frac{1}{2})(43' \times 12') = 25\,000 ft^2$
* Separations between parts are match lines on drawings

Total Area $\approx 54400 ft^2$

# of Floors:
"A": 2 (accommodates future expansion to 5 floors)
"B": 9
"C": 6

Walls:
Curtainwalls weigh 15 lb/sq ft wall area and comprise approx. 1/3 of total wall area. Limestone panels weigh 60 lb/sq ft and comprise approx. 2/3 of wall area. Therefore, I will use an allowance of $(15 \times \frac{1}{3}) + (60 \times \frac{2}{3}) = 45psf$ for walls.

Index Force Analysis
- Very simplified method will be conducted in order to get a general order of magnitude for the calculations.
- For this method, I will just use area x typical DL to get a rough estimate of the weights.
- Building is treated like a cantilever

Floor	Area(ft²)	Weight(k)	F_x=0.01W_x(k)	T/slab	Height Above Ground
ground	54400	5440	54.4	19.0'	0'
2	54400	5440	54.4	37.0'	18'
3	49000	4900	49.0	52.0'	33'
4	49000	4900	49.0	67.0'	48'
5	49000	4900	49.0	82.0'	63'
6	49000	4900	49.0	97.0'	78'
7	24000	2400	24.0	112.0'	93'
8	24000	2400	24.0	128.0'	109'
9	24000	2400	24.0	143.0'	124'
penthouse/roof	24000	2400	24.0	158.25'	139.25'
$\sum_{2}^{roof}$	346400	34640	346.4k		

Wall weights were not included on the above spreadsheet. Typically, I will consider that a floor slab will support 1/2 the wall height of the floor below it and 1/2 of the floor above it.

Perimeter "A" = 210'
Perimeter "B" = 465' @ floor 1, 555' above floor 2, 620' above floor 7
Perimeter "C" = 555'

Floor	Wall Area (ft²)	Weight(k)	Fx=0.01Wx (k)	Height
ground	11685	526	5.3	0
2	19395	873	8.7	18'
3	16650	750	7.5	33'
4	16650	750	7.5	48'
5	16650	750	7.5	63'
6	16650	750	7.5	78'
7	13285	598	6.0	93'
8	9610	433	4.3	109'
9	9300	419	4.2	124'
penthouse/roof	4650	210	2.1	139.25'
$\sum_{2}^{roof}$	134525	6059	60.6k	

Index Force Analysis Results

Calculated Base Shear = 346.4k + 60.6k = 407k
Design Base Shear = 640k

- The base shear calculated is about 2/3 of the design base shear. A lot of simplifications and assumptions were made in this method, some of them unconservative, so this is a reasonable result.

Overturning Moment = $\Sigma F_x h_x$ = 26700 ftk

Equivalent Lateral Force Method:

- Since lateral loads are not resisted by moment frames, calculate period by: $T_a = C_t h_n^x$, where C_t and x are found in ASCE Table 9.5.5.3.2

$$T_a = 0.02(158')^{0.75} = 0.89 < 1 \therefore \text{ rigid system}$$

$$C_s = \frac{S_{DS}}{R/I} \leq \frac{S_{D1}}{T(R/I)}$$

$$C_s = \frac{0.168}{3/1} \leq \frac{0.073}{(0.89 \times 3)}$$

$$C_s = 0.056 \leq \boxed{0.027}$$

$$C_s \geq 0.044\, S_{DS}\, I = 0.007 \checkmark \text{ok}$$

$$V = C_s W = 0.027 \times 40700^k = 1100^k \text{ base shear} > 640^k \text{ base shear}$$

The value that I calculated for base shear is considerably higher than the design base shear used. Since I used the same seismic coefficients in my analysis as the design professional, I believe that there are two possibilities for the discrepancy. One source of difference could be dead loads that I used were more conservative. However, since the difference is almost a factor of 2, it is doubtful that this had a large impact.

More likely, the period that the designer used was higher than mine, which could have a dramatic effect on base shear. The code has a provision listing that $T \leq C_u T_a$, where C_u ranges from 1.4→1.7. Bumping T to $1.7T_a$ changes analysis to a flexible building.

This yields a Cs of 0.016 for a base shear of 655k, which is almost exactly the value the design professional calculated.

Distribution of Loads:

$$C_{vx} = \frac{w_x h_x^k}{\sum w_i h^k}$$

$$F_x = C_{vx} V$$

For $T = T_a = 0.89s$, $k = 1.20$ by interpolation ($V = 1100^k$)
$T = 1.7T_a = 1.51s$, $k = 1.50$ by interpolation ($V = 655^k$)

See spreadsheet for load distribution.

Seismic Load Distribution

Period T = Approximate Period Ta
V = 1100k
k = 1.20

Level	Weight	Story Height h	h^k	Wx*hx^k	Cvx	Fx
2	6320	18	32.09	202789	0.03	34.6
3	6190	33	66.41	411062	0.06	70.2
4	5650	48	104.11	588217	0.09	100.4
5	5650	63	144.28	815187	0.13	139.1
6	5650	78	186.43	1053324	0.16	179.8
7	3000	93	230.24	690718	0.11	117.9
8	2840	109	278.56	791098	0.12	135.0
9	2820	124	325.17	916970	0.14	156.5
penthouse/roof	2610	139.25	373.73	975427	0.15	166.5
Sum	**40730**			**6444792**	**1**	**1100**

Period T = Max Cu*Ta
V = 650k
k = 1.50

Level	Weight	Story Height h	h^k	Wx*hx^k	Cvx	Fx
2	6320	18	76.37	482643	0.02	12.9
3	6190	33	189.57	1173442	0.05	31.3
4	5650	48	332.55	1878929	0.08	50.1
5	5650	63	500.05	2825266	0.12	75.4
6	5650	78	688.88	3892157	0.16	103.9
7	3000	93	896.86	2690579	0.11	71.8
8	2840	109	1137.99	3231901	0.13	86.2
9	2820	124	1380.81	3893872	0.16	103.9
penthouse/roof	2610	139.25	1643.21	4288775	0.18	114.4
Sum	**40730**			**24357563**	**1**	**650**

Wind Analysis

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Design Wind Speed V=90 MPH — IBC 2003 Figure 1609
Wind Importance Factor I=1.0 — Table 1604.5
Wind Exposure Category B — Table 1609.4

Building is Partially Enclosed
→ Internal Pressure Coefficient $GC_{pi} = \pm 0.55$

Exposure Category (K_h/K_z) — ASCE 7-05 Table 6-3

Height Above Ground	C&C	MWFRS
0-15'	0.70	0.57
20'	0.70	0.62
25'	0.70	0.66
30'	0.70	0.70
40	0.76	0.76
50	0.81	0.81
60	0.85	0.85
70	0.89	0.89
80	0.93	0.93
90	0.96	0.96
100	0.99	0.99
120	1.04	1.04
140	1.09	1.09
160	1.13	1.13
180	1.17	1.17

Topographic Factor $K_{zt}=1.0$ — ASCE 7-05 Section 6.5.7
Wind Directionality Factor $K_d=0.85$ — Table 6-4
$T = 0.1 \times \#\text{stories} = 0.9$
$n_1 = \frac{1}{T} = 1.11 \Rightarrow$ assume rigid structure

Gust Factor $G = 0.85$ or $G = 0.925\left(\frac{(1+1.7g_Q I_{\bar{z}} Q)}{1+1.7g_v I_{\bar{z}}}\right)$ — Section 6.5.8.1

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.30\left(\frac{33}{0.6(177')}\right)^{1/6} = 0.25$$

$$g_Q = g_v = 3.4$$

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} = 0.835 \perp \text{short dir}, \; 0.805 \perp \text{long dir}$$

B = 213' in short direction, 380' in long direction
h = 177'

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{10}\right)^{\bar{\epsilon}} = 320\left(\frac{0.6(177')}{10}\right)^{1/3} = 703.4$$

G = 0.835 for short dir, G = 0.818 for long dir

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From Fig. 6-6:

C_p: Windward Wall → $C_p = 0.8$ (Use q_z)
Leeward Wall → $C_p = -0.5$ for $L/B \leq 1$
-0.3 $= 2$ (Use q_h)
-0.2 ≥ 4
Side Wall → $C_p = -0.7$ (use q_h)

Roof (see spread sheet)

$P = qGC_p - q_i(GC_{pi})$
$q = q_z$ for Windward Wall (height dependent)
$q = q_h$ for Leeward wall, side walls, and roofs
$q_i = q_h$ for negative internal pressure evaluation
$q_i = q_z$ for positive internal pressure evaluation
z is highest opening that will effect enclosure status (Assume 90')

$q = 0.00256 K_z \underset{1.0}{K_{zt}} \underset{0.85}{K_d} \underset{90}{V^2} \underset{1.0}{I}$

$q = 17.625 K_z$

$q_h = 17.625(1.17) = 20.62$ psf

Long dimension ~310'
Short dimension ~200'
Tower extends ~150' in the long dimension

Tall Roof 140' above ground
Wing Roof 78' above ground

Surface Area in N-S direction:
$(310' \times 78') + (150' \times 62') = 33500 ft^2$
Surface Area in E-W direction:
$(200' \times 140') = 28000 ft^2$

When wind acts on N-S direction, $L/B < 1$ ∴ LW pressure $= -17.9$ psf
Total LW Force $= 17.9 \times 33500 = 600k$

When wind acts on E-W direction, $L/B = 1.55$ ∴ LW pressure ≈ -16.0 psf
Total LW Force $= 16.0 \times 28000 = 448k$

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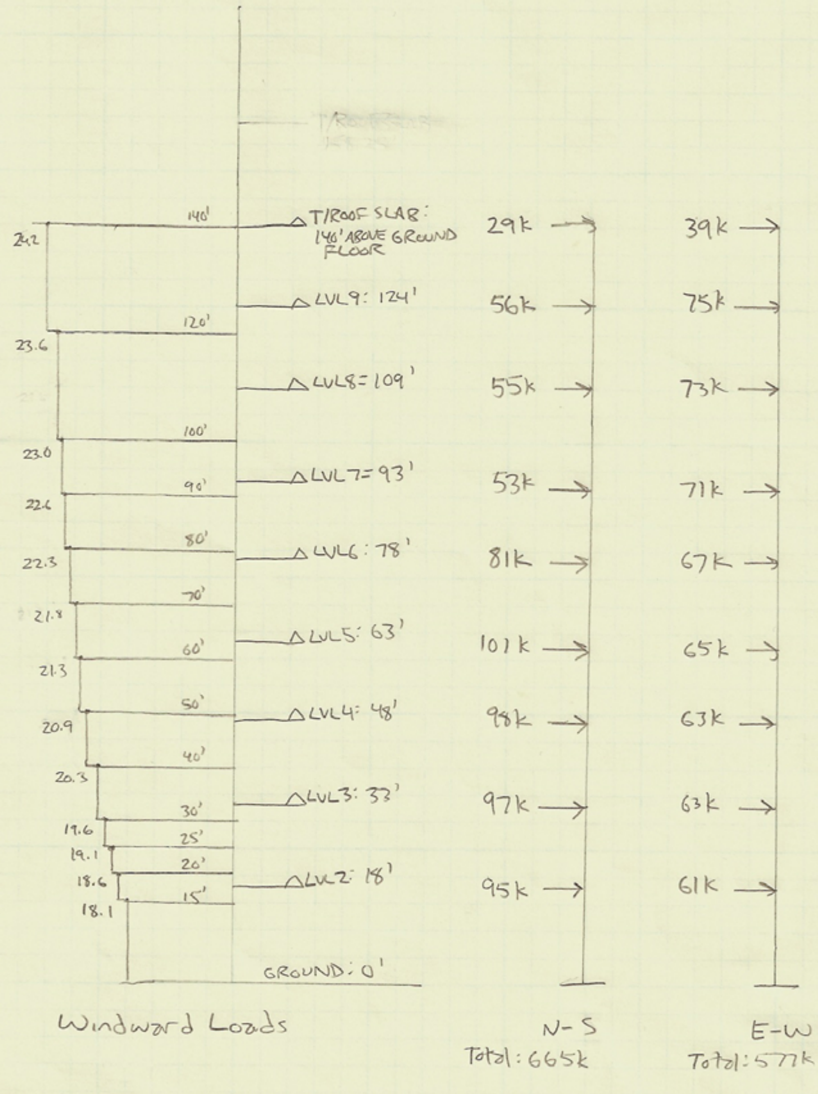


Wind Pressures

Windward Wall Pressures (MWFRS)

Height	Kd	qz	P (short dir)	P (long dir)
0-15'	0.57	10.05	18.1	17.9
20	0.62	10.93	18.6	18.5
25	0.66	11.63	19.1	19.0
30	0.7	12.34	19.6	19.4
40	0.76	13.40	20.3	20.1
50	0.81	14.28	20.9	20.7
60	0.85	14.98	21.3	21.1
70	0.89	15.69	21.8	21.6
80	0.93	16.39	22.3	22.1
90	0.96	16.92	22.6	22.4
100	0.99	17.45	23.0	22.8
120	1.04	18.33	23.6	23.3
140	1.09	19.21	24.2	23.9
160	1.13	19.92	24.6	24.4
180	1.17	20.62	25.1	24.8

Leeward Wall Pressures (MWFRS)

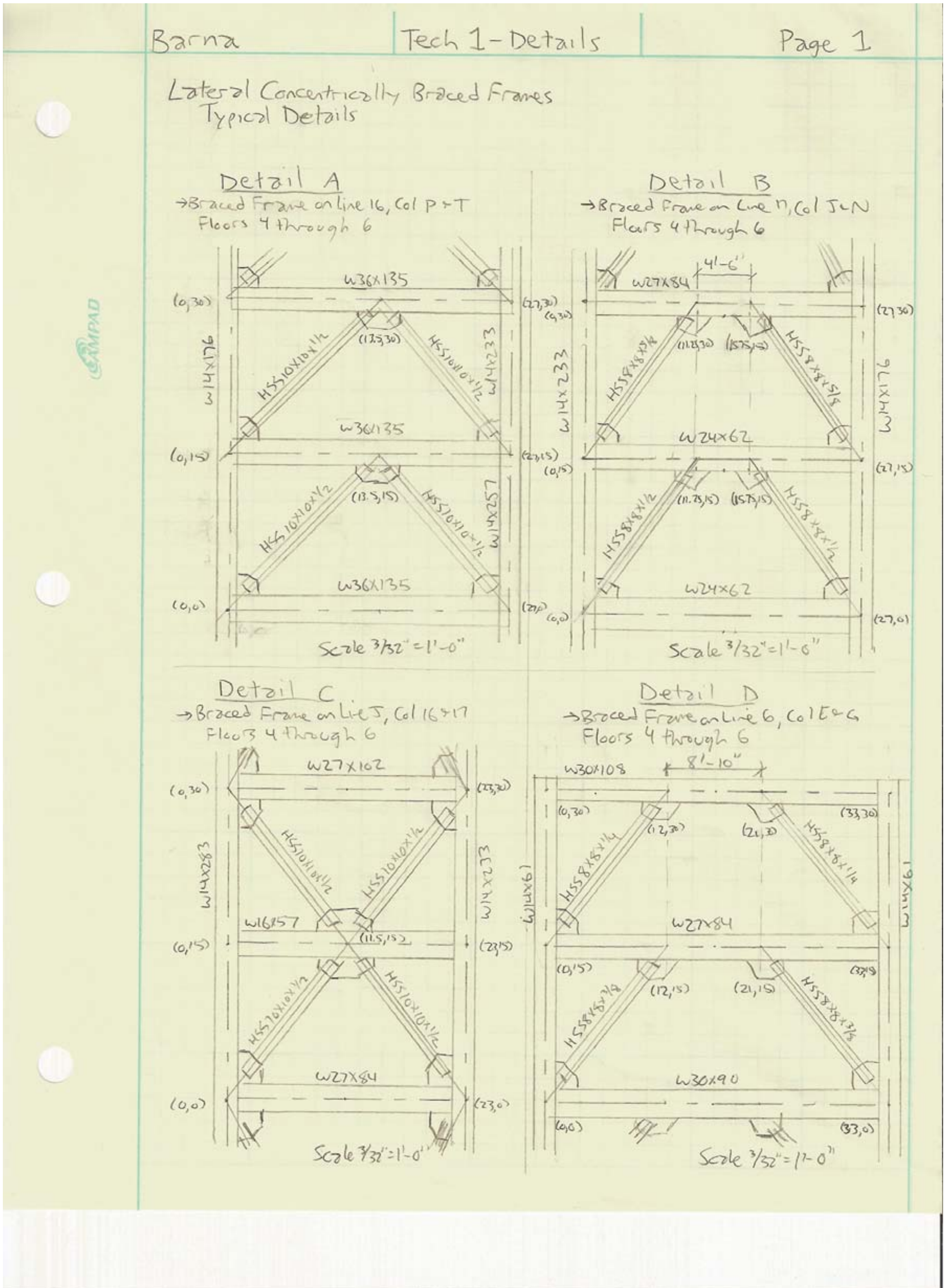
L/B<1	-17.9
L/B=2	-14.5
L/B>4	-12.7

Side Wall Pressure (MWFRS)

P=	-21.4

Long direction: 665k windward + 600k leeward = 1265k
Short direction: 577k windward + 448k leeward = 1025k

Lateral System



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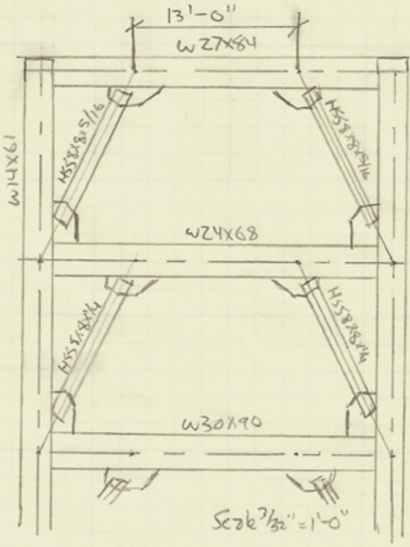
Lateral Concentrically Braced Frames
Typical Details

Detail E

→ Braced Frame on Line E, Col 6 & 7
Floors 4 through 6



Lateral Force Distribution

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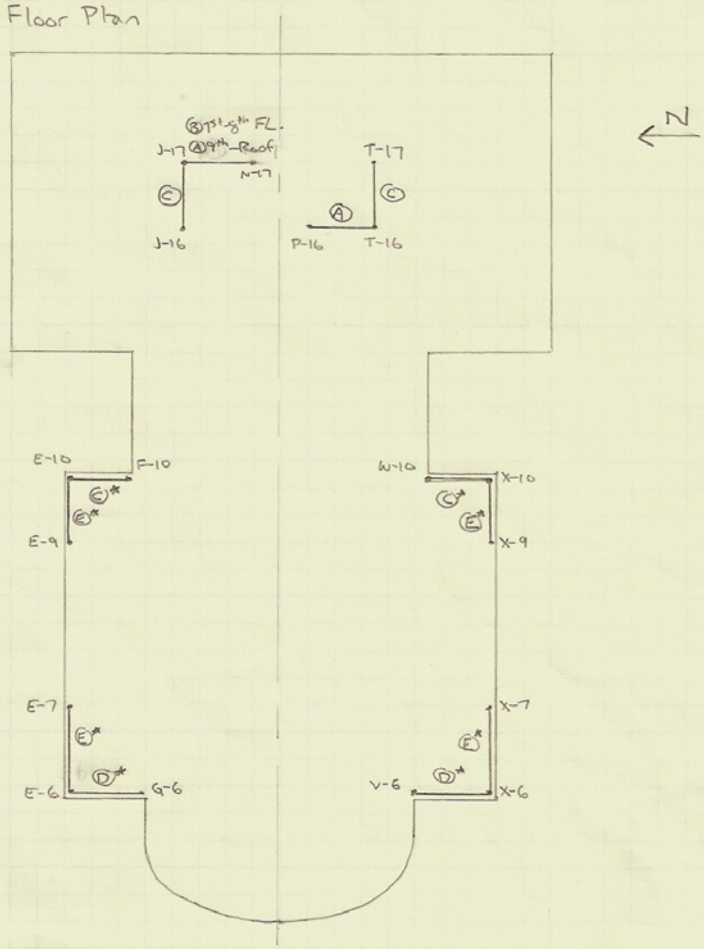
Relative Stiffness

- to be performed using a unit load on 2-stories of each of 4 typical braced framed details. (see details)
- RAM Advanse models using 100k unit loads

	Δ @ Top Right Corner	$1/\Delta$
Detail A	0.129	7.75
Detail B	0.266	3.76
Detail C	0.132	7.58
Detail D	0.436	2.29
Detail E	0.881	1.14

Typical Floor Plan



* Represents that frame stops at floor 6

Note: A braced frame in the east wing only between floors 1-2 to accommodate future expansion was ignored.

Lateral Distribution of Loads

East-West Direction

					Percent of Load Distributed to Frame, by floor						
Frame	**Detail**	**1/Defl**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**penthouse/roof**
J/16-17	C	7.58	38.4%	38.4%	38.4%	38.4%	38.4%	50.0%	50.0%	50.0%	50.0%
T/16-17	C	7.58	38.4%	38.4%	38.4%	38.4%	38.4%	50.0%	50.0%	50.0%	50.0%
E/9-10	E	1.14	5.8%	5.8%	5.8%	5.8%	5.8%	0.0%	0.0%	0.0%	0.0%
X/9-10	E	1.14	5.8%	5.8%	5.8%	5.8%	5.8%	0.0%	0.0%	0.0%	0.0%
E/6-7	E	1.14	5.8%	5.8%	5.8%	5.8%	5.8%	0.0%	0.0%	0.0%	0.0%
X/6-7	E	1.14	5.8%	5.8%	5.8%	5.8%	5.8%	0.0%	0.0%	0.0%	0.0%
		19.72	100%	100%	100%	100%	100%	100%	100%	100%	100%

North-South Direction

					Percent of Load Distributed to Frame, by floor						
Frame	**Detail**	**1/Defl**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**penthouse/roof**
17/J-N	B*	3.76	12.0%	12.0%	12.0%	12.0%	12.0%	32.7%	32.7%	50.0%	50.0%
16/P-T	A	7.75	24.8%	24.8%	24.8%	24.8%	24.8%	67.3%	67.3%	50.0%	50.0%
10/E-F	C	7.58	24.3%	24.3%	24.3%	24.3%	24.3%	0.0%	0.0%	0.0%	0.0%
10/W-X	C	7.58	24.3%	24.3%	24.3%	24.3%	24.3%	0.0%	0.0%	0.0%	0.0%
6/E-G	D	2.29	7.3%	7.3%	7.3%	7.3%	7.3%	0.0%	0.0%	0.0%	0.0%
6/V-X	D	2.29	7.3%	7.3%	7.3%	7.3%	7.3%	0.0%	0.0%	0.0%	0.0%
		31.25	100%	100%	100%	100%	100%	100%	100%	100%	100%

*Detail B for 1st-8th floor, then Detail A up to roof

Seismic Load Distribution on Braced Frames

Period T = Approximate Period
Ta
V = 1100k
k = 1.20

Approximate Load on Each Frame Story, kips

Frame	**Detail**	**1/Defl**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**penthouse/roof**	**Total Load**
J/16-17	C	7.58	13	27	39	53	69	59	68	78	83	489
T/16-17	C	7.58	13	27	39	53	69	59	68	78	83	489
E/9-10	E	1.14	2	4	6	8	10	0	0	0	0	30
X/9-10	E	1.14	2	4	6	8	10	0	0	0	0	30
E/6-7	E	1.14	2	4	6	8	10	0	0	0	0	30
X/6-7	E	1.14	2	4	6	8	10	0	0	0	0	30
		19.72	35	70	100	139	180	118	135	157	166	1100

North-South Direction

Approximate Load on Each Frame Story, kips

Frame	**Detail**	**1/Defl**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**penthouse/roof**	**Total Load**
17/J-N	B*	3.76	4	8	12	17	22	39	44	78	83	307
16/P-T	A	7.75	9	17	25	35	45	79	91	78	83	462
10/E-F	C	7.58	8	17	24	34	44	0	0	0	0	127
10/W-X	C	7.58	8	17	24	34	44	0	0	0	0	127
6/E-G	D	2.29	3	5	7	10	13	0	0	0	0	38
6/V-X	D	2.29	3	5	7	10	13	0	0	0	0	38
		31.25	35	70	100	139	180	118	135	157	166	1100

*Detail B for 1st-8th floor, then Detail A up to roof

Seismic Load Distribution on Braced Frames

Period T = Max Cu*Ta
V = 650k
k = 1.50

Approximate Load on Each Frame Story, kips

Frame	**Detail**	**1/Defl**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**penthouse/roof**	**Total Load**
J/16-17	C	7.58	5	12	19	29	40	36	43	52	57	293
T/16-17	C	7.58	5	12	19	29	40	36	43	52	57	293
E/9-10	E	1.14	1	2	3	4	6	0	0	0	0	16
X/9-10	E	1.14	1	2	3	4	6	0	0	0	0	16
E/6-7	E	1.14	1	2	3	4	6	0	0	0	0	16
X/6-7	E	1.14	1	2	3	4	6	0	0	0	0	16
		19.72	13	31	50	75	104	72	86	104	114	650

North-South Direction

Approximate Load on Each Frame Story, kips

Frame	**Detail**	**1/Defl**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**penthouse/roof**	**Total Load**
17/J-N	B*	3.76	2	4	6	9	12	23	28	52	57	194
16/P-T	A	7.75	3	8	12	19	26	48	58	52	57	283
10/E-F	C	7.58	3	8	12	18	25	0	0	0	0	66
10/W-X	C	7.58	3	8	12	18	25	0	0	0	0	66
6/E-G	D	2.29	1	2	4	6	8	0	0	0	0	20
6/V-X	D	2.29	1	2	4	6	8	0	0	0	0	20
		31.25	13	31	50	75	104	72	86	104	114	650

*Detail B for 1st-8th floor, then Detail A up to roof

Wind Load Distribution on Braced Frames

Windward load only

East-West Direction - Total Load: 577k

Approximate Load on Each Frame Story, kips

Frame	Detail	1/Defl	2	3	4	5	6	7	8	9	penthouse/roof	Total Load
J/16-17	C	7.58	23	24	24	25	26	36	37	38	20	252
T/16-17	C	7.58	23	24	24	25	26	36	37	38	20	252
E/9-10	E	1.14	4	4	4	4	4	0	0	0	0	18
X/9-10	E	1.14	4	4	4	4	4	0	0	0	0	18
E/6-7	E	1.14	4	4	4	4	4	0	0	0	0	18
X/6-7	E	1.14	4	4	4	4	4	0	0	0	0	18
		19.72	61	63	63	65	67	71	73	75	39	577

North-South Direction - Total Load: 665k

Approximate Load on Each Frame Story, kips

Frame	Detail	1/Defl	2	3	4	5	6	7	8	9	penthouse/roof	Total Load
17/J-N	B*	3.76	11	12	12	12	10	17	18	28	15	135
16/P-T	A	7.75	24	24	24	25	20	36	37	28	15	232
10/E-F	C	7.58	23	24	24	24	20	0	0	0	0	114
10/W-X	C	7.58	23	24	24	24	20	0	0	0	0	114
6/E-G	D	2.29	7	7	7	7	6	0	0	0	0	35
6/V-X	D	2.29	7	7	7	7	6	0	0	0	0	35
		31.25	95	97	98	101	81	53	55	56	29	665

*Detail B for 1st-8th floor, then Detail A up to roof

Spot Check – Composite Beam

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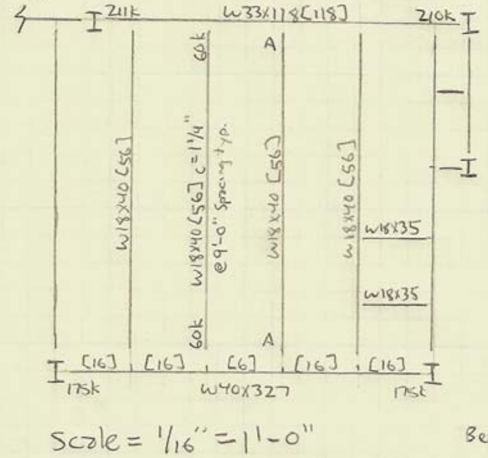
Typical Slab - Fifth Floor - Southwest Bay



Scale = $1/16" = 1'-0"$

Slab Type 'S3':
3½" LW Concrete on 3" metal Deck

Load for typical floor:
50psf concrete slab
30psf Superimposed DL
125psf Live Load (conservative)

205psf Unfactored Load
296 psf LRFD (1.2D+1.6L)

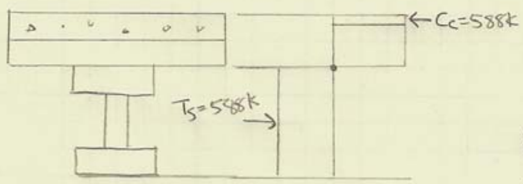
Beam A: [$296 \times 9' = 2664$ plf
$+ 1.2(50plf)$ steel w/studs
2724 plf]

Beam A
Max shear $= \frac{PL}{2} = \frac{(2.724klf)(42')}{2} = 57.2k \approx 60k$ ✓ok

Midspan Moment $= \frac{wl^2}{8} = \frac{(2.724klf)(42ft)^2}{8} = M_u = 601\ ft\text{-}k$

From the AISC Composite Tables, $Y2 \geq 5"$ and PNA must be no lower than location (3)

TRY location TFL with $Y2 = 5"$ ($\phi M_n = 615ft\text{-}k$, $\Sigma Q_n = 588k$)



588k ≈ 590k
→ ASSUME full crosssection in yielding

$Y2 = 6.5" - \frac{a}{2} = 5.7" > 5"$ ✓ok

@$Y2 = 5.5"$, TFL → $\phi M_n = 637 ft\text{-}k$

$b_{eff} = \min\left\{ \frac{1}{4} Span,\ Spacing \right\} = \min\left\{ 126",\ 108" \right\} = 108"$

$M_n = 588k \times (5.7") + 588k \times \left(\frac{12.9"}{2}\right)$
$= 8614\ in\text{-}k = 717 ft\text{-}k$

$\phi M_n = 0.9(717) = 645 ft\text{-}k > 601$ ✓ok

$T_s = A_s F_y = (11.8 in^2)(50ksi) = 590k$

$\Sigma Q_n = 588k$

ϕM_n is between table values for $Y2 = 5.5"$ and $Y2 = 6"$; therefore, calculation is correct.

$C_c = 588k = 0.85 f'_c b_{eff}\, a$

$a = \frac{588k}{0.85(4ksi)(108")} = 1.60"$

Spot Check – Column

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Column Spot Check → C-17 in the tower

Flat roof on 8^{th} Floor
W12x170 < 7^{th} Floor, 6^{th} Floor
W12x190 < 5^{th} Floor, 4^{th} Floor
W12x230 < 3^{rd} Floor, 2^{nd} Floor
W12x336 1^{st} Floor
W12x336 Foundation

Unfactored Axial Load = 1850k

Base Plate 30"x30"x4½"

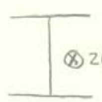
Column is exterior ∴ it supports floor load and wall weight.
Column is not part of a braced frame ∴ it is assumed that it will take gravity loads only

Col D.6-17 bears on beam that frames into C-17 at roof level.
Col D.6-17 has a compression load P_u of 345k. This number is taken by adding shear load on connections on the column. Loads shown on plan.
345k + 30ft x 54plf = 347k

Assume C-17 takes half the D.6-17 load = 174k

See spreadsheet for load accumulation of the column through stories.

Assume an effective length of 15' (story height) for the columns.
This yields a $\phi_c P_n$ of 1790k, which is much higher than the load of 380k that I calculated. However, there is a huge eccentricity associated with this column since a transfer girder frames into one side and there is no beam to balance it. An interaction requirement is necessary.



⊗ 215k + 455k = 670k x (6" off center ASSUME) = 335 ft-k

380/1790 = 0.21 ∴ $\frac{P_c}{P_c} + \frac{8}{9}\left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \leq 1.0$ (Spec. Chapter H1.1)

$$\frac{380k}{1790k} + \frac{8}{9}\frac{335ft\text{-}k\,(12in/ft)}{(0.9)(50ksi)(126in^3)} = 0.84 \leq 1.0$$

$\leftarrow Z_y$

The combined effects of axial compression and sideways moment make the choice of columns appropriate

Column
Loads

Floor	Trib Area (sqft)	DL (psf)	LL (psf)	LL Reduction Factor	Reduced LL	1.2DL + 1.6LL Load Combo (k)	Sum Force (k)*	(unreduced) Load Combo (k)	(unreduced) Sum Force (k)*
8	500	65	60	1.000	60.0	87.0	261.0	87.0	261.0
7	500	100	125	0.585	73.2	118.5	379.5	160.0	421.0
6	500	100	125	0.487	60.9	108.7	488.3	160.0	581.0
5	500	100	125	0.444	55.5	104.4	592.6	160.0	741.0
4	500	100	125	0.418	52.2	101.8	694.4	160.0	901.0
3	500	100	125	0.400	50.0	100.0	794.4	160.0	1061.0
2	500	100	125	0.400	50.0	100.0	894.4	160.0	1221.0
1	500	100	125	0.400	50.0	100.0	994.4	160.0	1381.0

*Includes D.6-17 Column Load transferred to C-17
column by transfer girder