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Æ Structural
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THE hub on Chestnut

PHILADELPHIA, PA

Technical Assignment 1

October 5, 2006

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Philadelphia, PA
October 5, 2006

Executive Summary

Structural Technical Report 1

The contents of this report provide an arrangement of analyses that contributes to my presumption that The HUB on Chestnut is designed as a concrete moment-resisting frame structure. The first investigation was to obtain the contributing lateral loads due to wind and seismic lateral forces. A wind load analysis was performed to locate in which direction the wind would be most critical. The East/West direction, which is perpendicular to the long dimension of the building, is calculated to be most critical. By inspection, the rectangular structural columns are oriented to allow their strong axis, by moment of inertia, to be exposed in this direction which will function better to resist the moment produced by wind forces. The seismic loading analysis has governed as the most critical lateral load over wind. Although Philadelphia is located in an earthquake active zone I chose to apply wind loading during spot-checks.

Another observation from the columns' schedule is that most supports are all uniform in size with minimal changes in reinforcement. This led me to believe that there is a low ratio between steel and concrete in the upper levels. After performing a pure axial spot-check on an interior and exterior column, located on the Level 5, I found that minimal steel is needed and the girth of the column provides axial support. I concluded that the steel provide in the columns are to resist moment. When performing a column calculation with an applied moment, on the same level, I found the column was still oversized. My conclusion in the column design is that the post-tensioning system running through the column lines must exhibit a large factor in determining size and reinforcement.

In slab design, because the columns are spaced almost square, my first assumption was two-way spanning with minimum reinforcing due to post-tensioning. After concluding my column design I revisited this assumption. I declared that the one interior column line and two exterior lines provide support for a one-way slab system and the post-tensioning oriented E/W provides extra support from the exterior panels load. The tendons running N/S are used to help resist moment in the frames as well as supplying strength the floor system.

With the conclusions stated above my first presumptions of design had been altered. Although I did not find exact numerical data to compare with the erected design, my spot-checks had made me modify my predictions on the structural design based on the inspection of working drawing.

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Structural Technical Report 1

Concepts and Existing Conditions

Within this report are a detailed description of the overall structural system and a preliminary analysis of the newly erected structure located in the University City section of Philadelphia, Pennsylvania. The HUB, located at the northeast corner of Chestnut and 40th streets, is a mid-rise, mixed-use structure which began construction in the spring of 2005. The building is predominantly a concrete structure that stands 9-levels with one sub-grade level covering a footprint of approximately 11,000 square-feet. The north/south length of the building extends one-hundred forty-eight feet down 40th Street and the west/east width extends sixty-eight feet along Chestnut Street. The HUB provides the local community with 110 apartment units and 3-levels of retail and mercantile use. Levels three to nine are designed for a residential occupancy, while the sub-grade, first, and second levels are designed primarily for commercial occupancy. The residential space is approximately 68,000 square-feet and 30,000 square-feet are for commercial use. Architectural accents include a balcony level, studio and multi-room living units, and double height commercial ceilings.

The foundation system is comprised of concrete caissons and spread footings. Starting below grade, the superstructure is a system of exterior and interior concrete columns that support a concrete slab throughout each level. The building envelope is a paneled rain-screen system and a EPDM. The commercial space is designed using a thicker two-way slab and rectangular columns. Residential levels use a post-tensioned slab with a mixed use of rectangular and round columns.

In the following pages are more descriptive synopsizes for each of the structural elements of The HUB on Chestnut. Preliminary design concepts, codes, standards, and visual aids will be included throughout this report to enhance concepts and to display the collection of data. An analysis of lateral forces, such as wind and seismic criteria, are also available. Calculations and 'spot checks' were performed on the primary structure to help satisfy the thought process that was initialized by the original designers for this project.

Codes

National Design Code

International Building Code 2003 Edition

Disciplinary Design Code

American Society of Civil Engineers	[ASCE 7-02]
American Concrete Institute	[ACI 318-03]
American Institute of Steel Construction	[AISC - 3rd Edition]
American Society for Testing and Materials	[ASTM – *X]

***   Please see individual structural element sections for material specific code**
**** Construction began in May 2005, Assume up-to-date codes had not be initiated**

Loads

The loads considered in design pertain to any element that produces a force on the structure, such as self weight, arbitrary movement, and construction. Dead loads are classified as any object that is integrated into the structure or permanently attached. Live loads are any contributing factor that exhibits a force over a duration of time, sudden impact or continuous. Other forms of loading include snow, wind, and seismic.
Below are the considered loads that will directly influence the design process of selecting structural members. All live loads are taken from the applicable codes. The International Building Code 2003 was the main documented used in designing The HUB. Many items sited below where found in the IBC. Often, the IBC directs items and guidelines to be referenced in ASCE 7. The dead loads that are listed below have been modified from the original design. A few loads have been added to incorporate some features that may not have been taken into account previously. The collateral loads have been modified and a MEP dead load has been added. MEP has been considered to account for an excessive amount of plumbing due to fire protection and multiple water closets from residences.
*Please see Appendix for the designer's original anticipated loading plan.

Live Load Reduction

$$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right)$$

Roof Live Load Reduction

$$L_r = 20R_1R_2 \text{ where } 12 \le L_r \le 20$$

DEAD/LIVE LOADS

ASCE 7-02 Chapter 6

Dead Loads					
Concrete (Reinforced)					
12"	150 lbs/ft^2		X		
9"	113 lbs/ft^2			X	X
4"	50 lbs/ft^2	X			
Partitions	20 lbs/ft^2			X	
MEP	10 lbs/ft^2	X	X	X	X
Curtain Wall	10 lbs/ft^2				
Collateral					
Mechanical	15 lbs/ft^2	X			X
Commercial	10 lbs/ft^2		X		
Residential	5 lbs/ft^2			X	
Live Loads [ASC 7-02 T4-1]					
Stores (Retail)	100 lbs/ft^2	X			
Assembles (Lobbies)	100 lbs/ft^2		X		
Residental (Private Rooms)	40 lbs/ft^2			X	
Roof	30 lbs/ft^2				X

Slab on Grade *Dead* 75 lbs/ft^2
Live 100 lbs/ft^2

1st - 2nd Levels *Dead* 170 lbs/ft^2
Live 100 lbs/ft^2

3rd - 9th Levels *Dead* 148 lbs/ft^2
Live 40 lbs/ft^2

Roof Level *Dead* 138 lbs/ft^2
Live 30 lbs/ft^2

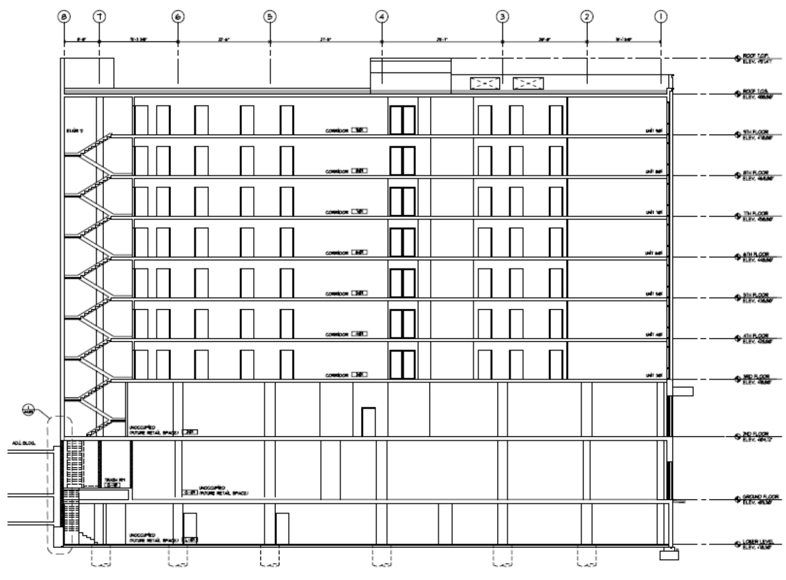
SNOW LOAD

IBC 2003 Edition

P_g	25	FIGURE 1608.2
C_e	1	TABLE 1608.3.1
I_s	1	TABLE 1604.5
C_t	1	TABLE 1608.3.2
$P_f = 0.7C_e C_t I P_g$	18	

Structural System

The overall building structural system, previous stated in the introduction, functions as a moment resisting frame. The reinforced concrete, geometry, and connections of the structure all work in unison to resist the effects of lateral and gravity loading conditions. Before analyzing any data and making a visual inspection, I expected The HUB on Chestnut to resist moment by using an ordinary reinforced concrete frame system. The 9-level structure does not exhibit any shear walls or cross lateral bracing. The connections must withstand these effects of loading. The design of the building displays a sense of geometry and redundancy which allows for direct structural analysis and uniform performance by the structural elements throughout.



LONGITUDINAL SECTION (NORTH-SOUTH)
Scale: 1/8"=1'-0"

Structural Elements

Foundation

The main foundation system is a grid of straight shaft caissons varying in size from 3'-6" to 4'-6" in diameter. All Caissons are constructed using a compressive strength of 3000 *PSI* concrete and bearing on undisturbed rock. The interior and exterior concrete columns are directly supported by caissons. All exterior walls are cast-in-place concrete placed on top of soil capable of supporting a load of 3000 *PSF*. A keyway system is oriented into the footing to resist lateral movement from the surrounding earth. The building footprint is classified as type *D* soil. Masonry walls, which are placed below grade, are constructed of Type N-1, ASTM C90 hollow grouted solid masonry units. All mortar is Type S, ASTM C270 with a minimum compressive strength of 1800 *PSI* after 28 days. Vertical reinforcement members of the masonry units are spaced at 16 inches on center. A 4" concrete slab-on-grade with 4" of crushed stone base and perforated pipe underdrain system is placed at the lowest elevation of the structure. Finished floor elevation is 73.30' above sea-level. Also inlayed, is 6 x 6 welded wire fabric with a 8 mil vapor barrier.

Columns

The main structural supports of the building are designed using three column lines forming six bays along each. Although the bays and column lines are unequally spaced throughout, the typical geometry is 28' x 25'. The columns are placed directly over one another from level to level to provide a stacked effect for transferring loads. At each level the columns are spliced by lapping the protruding rebar from the lower level to the newly formed column above. All columns are constructed of reinforced concrete having a minimum compressive strength of 5000 *PSI* after 28 days. The columns located on the lower levels are sized 30" x 30" while the upper floors (3-9) are sized 20" x 30". All reinforcement uses a #3 bar spaced twelve inches on center with varying rebar ranging from #7 to #10 bar.

Steel

The HUB has a predominantly concrete structure but does incorporate steel into the design. Located within the stairways and the elevator shafts are steel framing systems. A typical frame consists of several shapes. All wide-flanges are Gr 50 ASTM A992/A572, hollow rectangular/square steel Gr 50 ASTM A500 with a yield strength of 46 *KSI*. All other steel members are ASTM A36 UNO. After fabrication, the steel was coated with a rust inhibitive paint and later the steel was to be sprayed with a layer of fibrous fireproofing material.

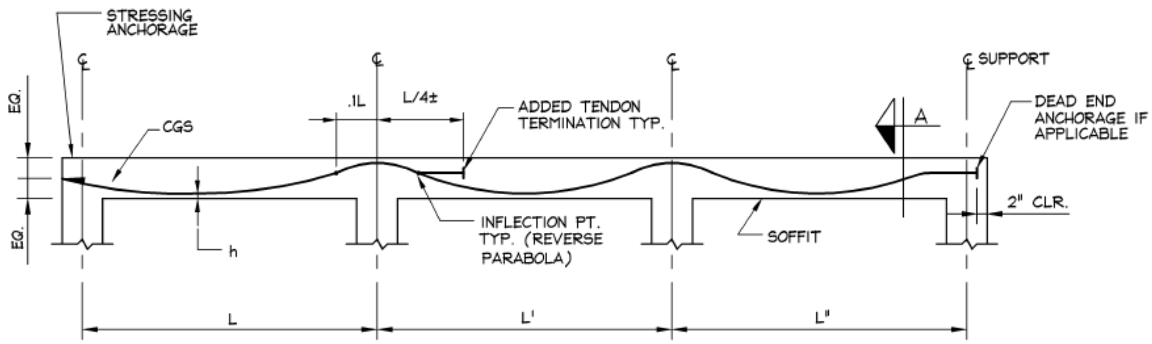
Two-Way Slabs

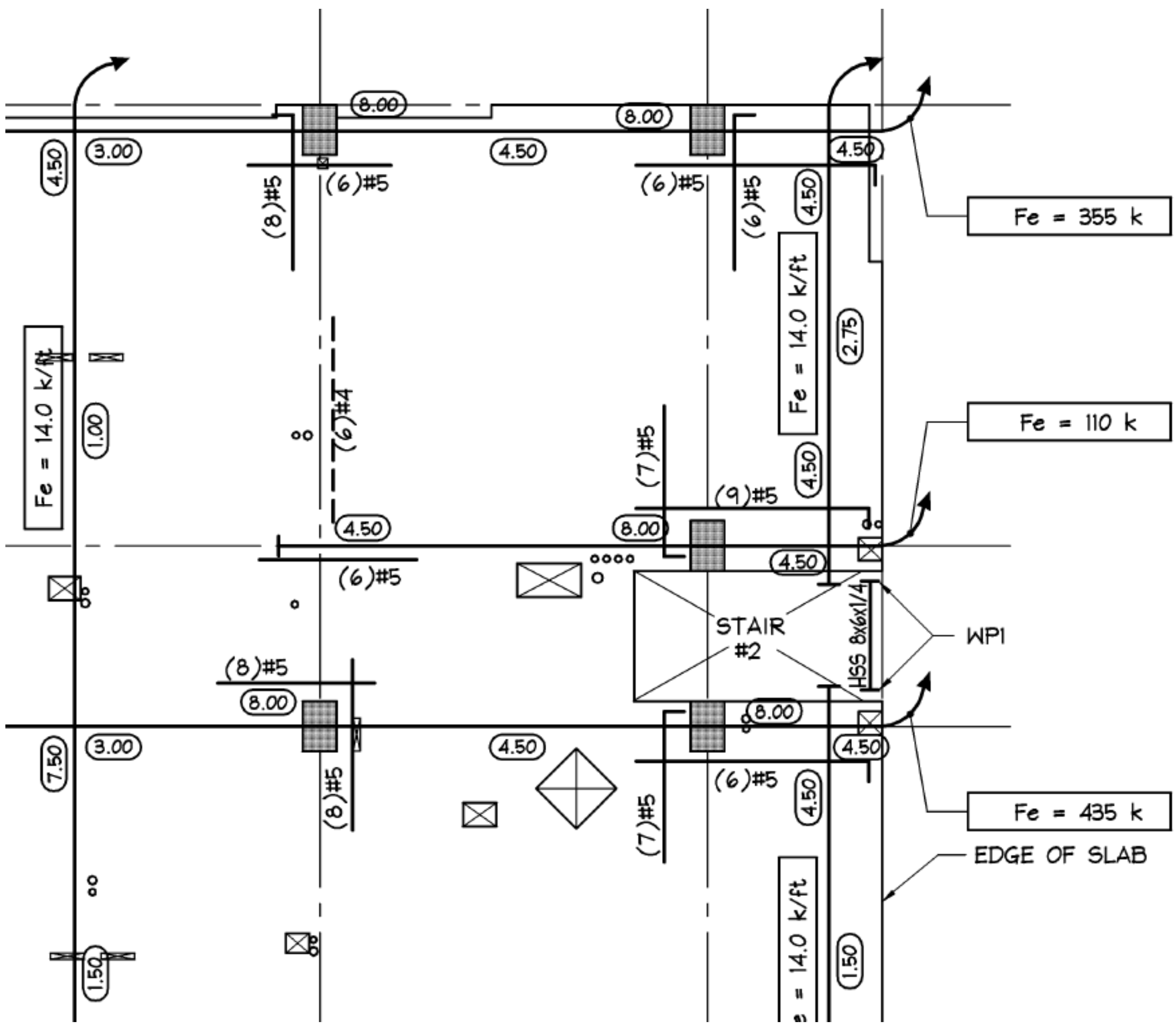
The ground level and second level are assumed to be flat two-way slab systems. These two (2) slabs located in the commercial space are at a depth of 12" compared to the 9" slabs located above in the structure. It is primarily reinforced in two directions using #6 rebars spaced sixteen-inches on center with additional rebar added in regions of needed higher strength. A large elliptical opening is placed on the ground level and the surrounding slab system is high reinforced. The slabs are also highly reinforced around the support columns. No detailing of edge beams or dropped panels are integrated into the floor system.

Post-Tensioned Slabs

All elevated slabs from level three to the roof are strengthened using post-tensioning. The process involves shoring the under layer of the slab, placing the conduits and tendons in accordance with its structural design, and then placing the concrete over the conduit layout. After the concrete has a reached a sustained strength, jacks or rams, are used to pull the tendons allowing the slab to carry the designed load. All tendons are designed to be ½ " Type 270 *KSI*, greased, and manufactured in a plastic sheath. Three main conduits are placed along each of the column lines. The two exterior tendon lines are symmetric in profile and in jacking force while the interior tendon line is ran around the central stair way and detailed with a much higher jacking force. The interior tendon profile also has an additional strand with a lesser post-tensioned force to accommodate the center stairway access.

Below are two schematics of the tendons' typical profile and a plan view of the post-tensioned strands. Notice the parabolic profile of the tendons. This profile can be inverted to replicate the moment diagram of that line of action.



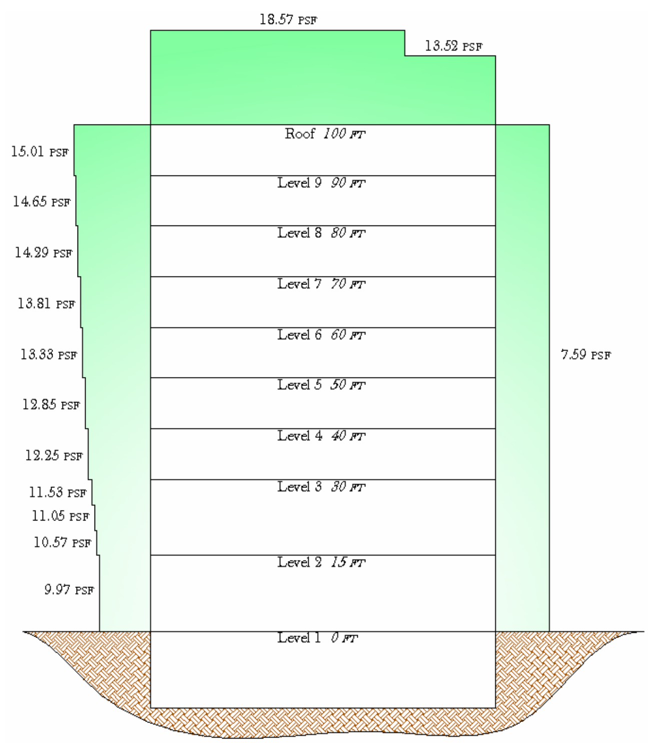


Lateral Load Analysis

The lateral loads performed on The HUB were analyzed by standard practice and the guidelines recognized in the 2003 IBC along with ASCE 7-02. Both load calculations are based on the size, geometry, type, and geologic location of the structure. The wind load analysis is performed on the main wind force-resisting system (MWFRS), which is guided by ASCE 7-02 Chapter 6 and the seismic loading is performed on the structural framing, which is guided by ASCE 7-02 Chapter 8.

Wind

The wind loads on the MWFRS are calculated based the geometry, height, type, and geological location of the building. Philadelphia is not located within any hurricane region of the United States but is subjected to substantially high wind. Although the building's glazing is not blast proof, I believe that the glazing is able to withstand most windborne debris therefore classifying the structure as fully enclosed. In the case of a 'breached' building envelope it is possible to increase the internal pressure by almost three times (±0.18 → ±0.55). The data obtained has proven that the East-to-West wind direction is the most critical orientation because higher pressures are to be exerted on the structure. This conclusion is based on adding both the windward and leeward pressures and observing which produces higher result. Another observation is that when the interior pressure is negative the windward pressure is greatest. Contrary to this assumption, when the interior pressure is positive the leeward pressures are greatest. The calculated results have been summarized in the illustration and tables below.



WIND ANALYSIS

ASCE 7-02 Chapter 6

Location	Philadelphia, PA			
Typography	Homogeneous			
Dimensions	148'	Length	99' - 6"	Height
	68'	Wide		
Framing	Moment Resisting Frame System			
Cladding	Rainscreen Panel Assembly			
Frequency	Rigid Structure	f = 1.11 Hz *[6.2]*		
Enclosure Class	Enclosed			

Velocity Pressure

q_z	$0.00256 K_z K_{zt} K_d V^2 I$
V_3	90
I_w	1.00
K_d	0.85
K_{zt}	1.00

Gust Effect Factor

c	0.3	
l	320	
e	1/3.0	
z_{min}	30	
z	59.7	($0.6h = z_{min}$)
L_z	390	
$g_Q = g_v$	3.4	
Q	0.85	
I_z	0.27	
G	0.84	*0.85*

G = 0.85 ASCE7 6.5.81

Internal Pressure Coefficient

GC_{pi}	± 0.18

External Pressure Coefficients

North/South			East/West		
Wall	*Cp*		*Wall*	*Cp*	
	0.80	Windward		0.80	Windward
	-0.30	Leeward		-0.50	Leeward
	-0.70	Side		-0.70	Side
Roof	-0.95	0 to h/2	*Roof*	-1.04	0 to h/2
	-0.83	h/2 to h		-0.70	> h/2
	-0.57	h to 2h			

Wall Pressures

			Windward				Leeward				MWFRS	
			North/South		East/West		North/South		East/West			
Height (FT)	K_z	q_z	*+0.18*	*-0.18*	*+0.18*	*-0.18*	*+0.18*	*-0.18*	*+0.18*	*-0.18*	*North/South*	*East/West*
0-15	0.57	10.05	3.69	9.97	3.69	9.97	-7.59	-1.31	-10.56	-4.28	11.28	14.25
20	0.62	10.93	4.29	10.57	4.29	10.57	-7.59	-1.31	-10.56	-4.28	11.88	14.85
25	0.66	11.63	4.77	11.05	4.77	11.05	-7.59	-1.31	-10.56	-4.28	12.36	15.33
30	0.70	12.34	5.25	11.53	5.25	11.53	-7.59	-1.31	-10.56	-4.28	12.84	15.81
40	0.76	13.40	5.97	12.25	5.97	12.25	-7.59	-1.31	-10.56	-4.28	13.56	16.52
50	0.81	14.28	6.57	12.85	6.57	12.85	-7.59	-1.31	-10.56	-4.28	14.16	17.12
60	0.85	14.98	7.05	13.33	7.05	13.33	-7.59	-1.31	-10.56	-4.28	14.64	17.60
70	0.89	15.69	7.53	13.81	7.53	13.81	-7.59	-1.31	-10.56	-4.28	15.12	18.08
80	0.93	16.39	8.01	14.29	8.01	14.29	-7.59	-1.31	-10.56	-4.28	15.60	18.56
90	0.96	16.92	8.37	14.65	8.37	14.65	-7.59	-1.31	-10.56	-4.28	15.96	18.92
100	0.99	17.45	8.72	15.01	8.72	15.01	-7.59	-1.31	-10.56	-4.28	16.32	19.28

Roof Pressures

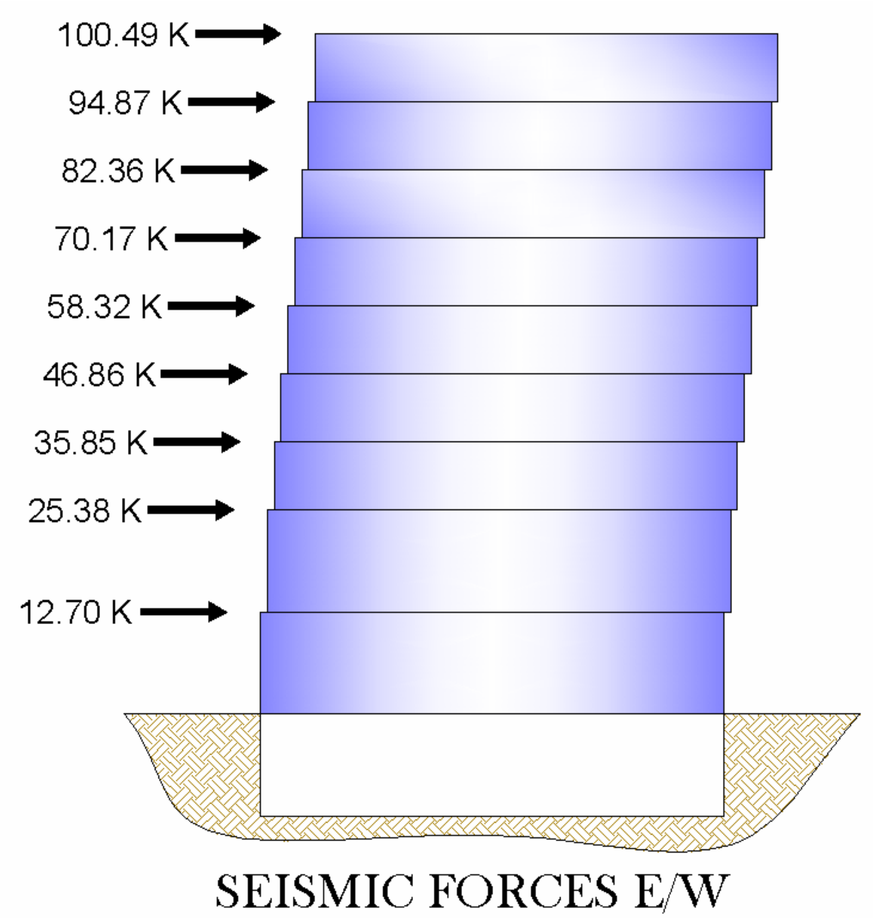
Zone	North/South		East/West	
	+0.18	*-0.18*	*+0.18*	*-0.18*
0 to h/2	-17.23	-10.95	-18.57	-10.95
h/2 to h	-15.45	-9.17	-13.52	-9.17
h to 2h	-11.60	-5.31		

MWRS Forces and Moments

Wind Zone	North/South	East/West	Level	Height (FT)	Floor-to-Floor	Level $A_{TN/S}$	Level $A_{TE/W}$	F_{WIND} (K) North/South	F_{WIND} (K) East/West	M_{WIND} (K-FT) North/South	M_{WIND} (K-FT) East/West
0-15	*11.28*	*14.25*	1	0	0	510	1110	5.75	15.81	0	0
20	*11.88*	*14.85*	2	15	15	1020	2220	11.89	32.47	178.4	487.1
25	*12.36*	*15.33*	3	30	10	850	1850	11.08	29.60	332.3	887.9
30	*12.84*	*15.81*	4	40	10	680	1480	9.42	24.90	376.9	996.0
40	*13.56*	*16.52*	5	50	10	680	1480	9.79	25.70	489.5	1284.9
50	*14.16*	*17.12*	6	60	10	680	1480	10.12	26.41	607.0	1584.5
60	*14.64*	*17.60*	7	70	10	680	1480	10.44	27.12	731.0	1898.2
70	*15.12*	*18.08*	8	80	10	680	1480	10.73	27.74	858.2	2219.1
80	*15.60*	*18.56*	9	90	10	680	1480	10.97	28.27	987.5	2544.4
90	*15.96*	*18.92*	Roof	100	0	340	740	5.55	14.27	554.7	1426.8
100	*16.32*	*19.28*								5115.5	13328.9
			N/S B(FT) 68		*E/W* B(FT) 148				*CONTROLS*		

Seismic

Seismic activity can be catastrophic to a building structure. A lateral load produced by an earthquake causes the structure to absorb a tremendous amount of moment at its connections and distributes forces horizontally as well as vertical. From the data collected, seismic lateral loads are the controlling factor over wind. The calculated seismic forces are the same in both directions. The HUB is not a very heavy structure, in regards to its gravity load, therefore it is less prone to damage from seismic activity. With seismic controlling, the structure is more likely to be a moment framed design. Moment frames are less influenced by lateral loads because there joints are more heavily reinforced than braced frames.



SEISMIC ANALYSIS

ASCE 7-02 Chapter 9

Location	Philadelphia, PA			
Dimensions	148'	*Length*	99' - 6"	*Height*
	68'	*Wide*		
Occupancy Category	II			
Seismic Use Group	II			
Importance Factor	1.00			
Site Classification	D			
Basic Structural System	Moment Resisting Frame System			
Seismic Resisting System	Ordinary Reinforcement Moment Frame			
Frequency	Rigid Structure	f = 1.11 Hz *[6.2]*		

I_E	1	S_{DS}	0.329
S_S	0.32	S_{D1}	0.131
S_1	0.082	R	3
F_a	1.54	C_d	2.5
F_V	2.40	V	527
S_{MS}	0.493	C_s	0.039
S_{M1}	0.197	k	1.2

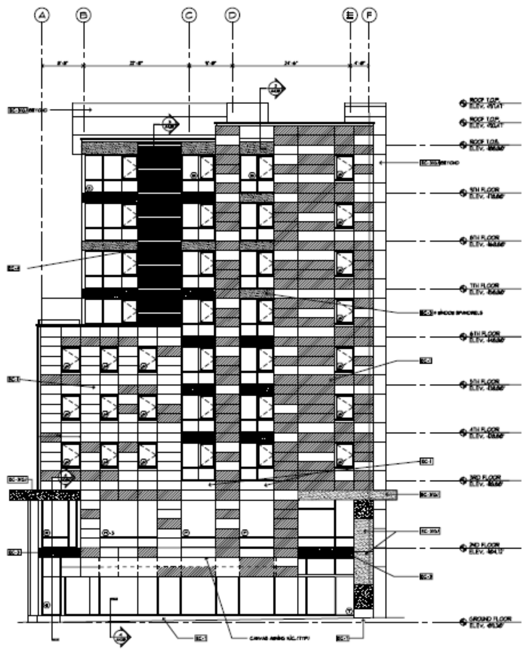
Level	w_x (K)	h_x (FT)	$w_x h^k_x$	C_{vx}	F_x (K)	M_x (FT-K)
Roof	1390	100	349152.2	0.191	100.49	10049.46
9	1489	90	329598.9	0.180	94.87	8538.00
8	1489	80	286155.9	0.156	82.36	6589.01
7	1489	70	243788.0	0.133	70.17	4911.77
6	1489	60	202617.1	0.111	58.32	3499.09
5	1489	50	162801.6	0.089	46.86	2342.91
4	1489	40	124556.6	0.068	35.85	1434.02
3	1489	30	88194.2	0.048	25.38	761.53
2	1711	15	44112.3	0.024	12.70	190.45
1	0	0	0	0	0	0
?	*13524*		*1830977*	*1*	*527*	*38316.24*

Appendix

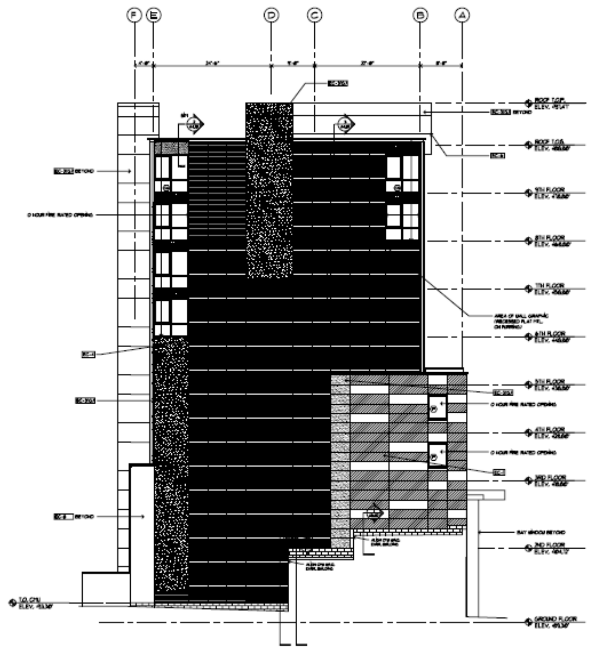
Architectual Schematics

Designer's Reference

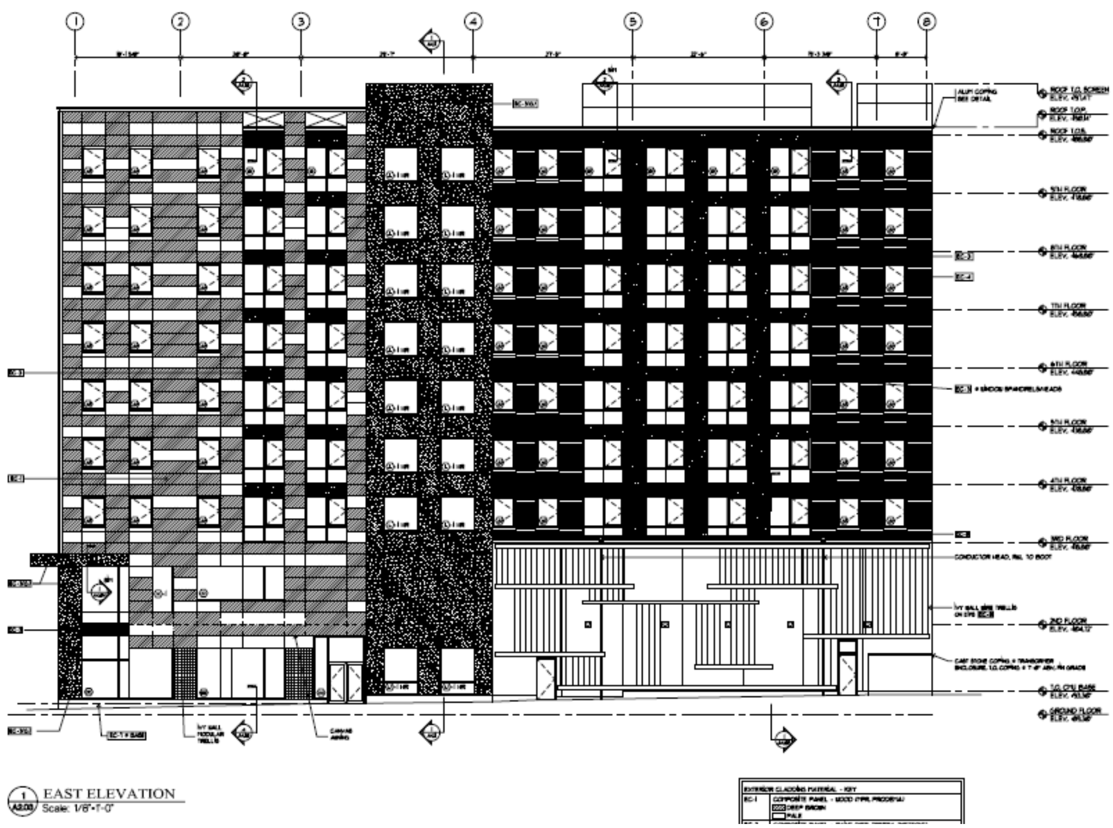
Calculations



2 A2.02 SOUTH ELEVATION
Scale: 1/8"=1'-0"

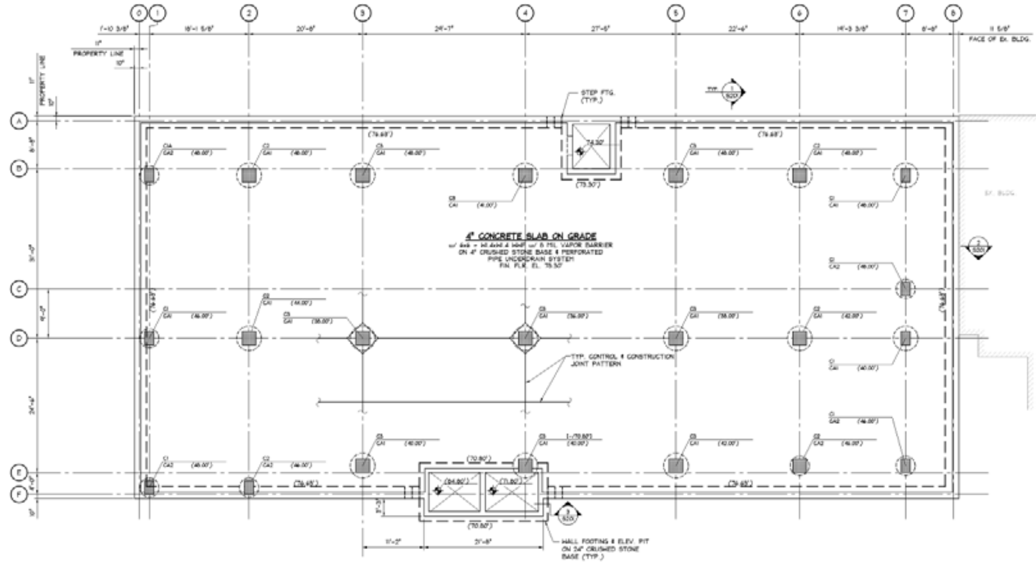


1 A2.02 NORTH ELEVATION
Scale: 1/8"=1'-0"

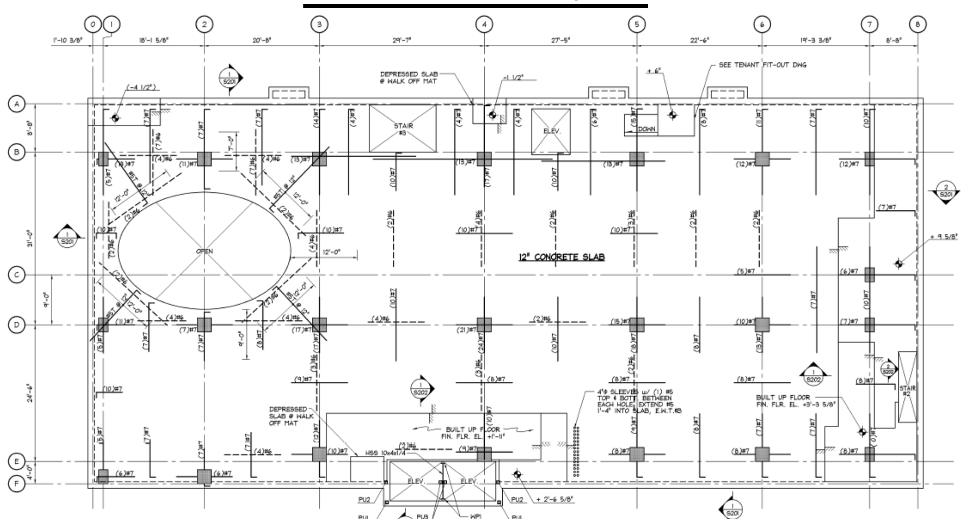


1 A2.03 EAST ELEVATION
Scale: 1/8"=1'-0"

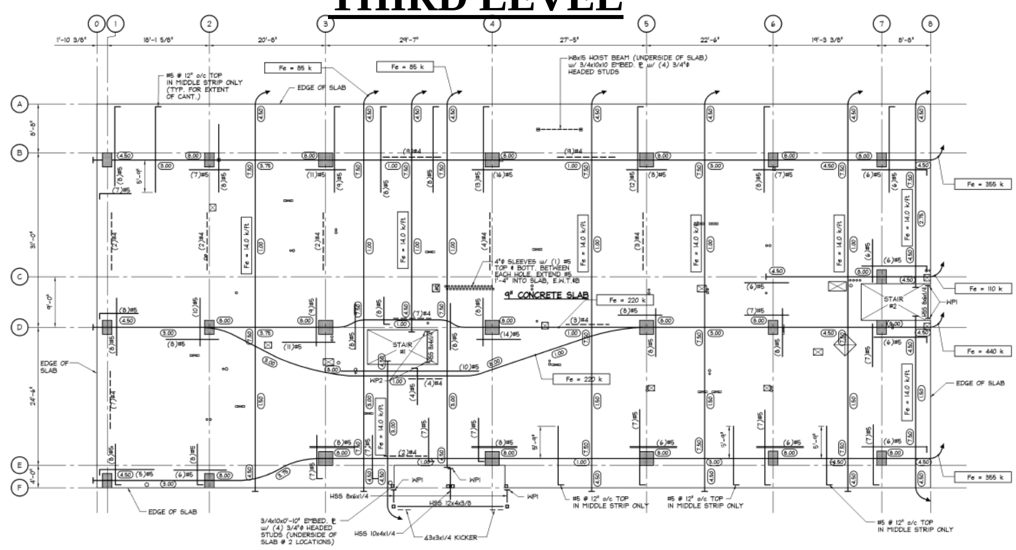
FOUNDATION PLAN

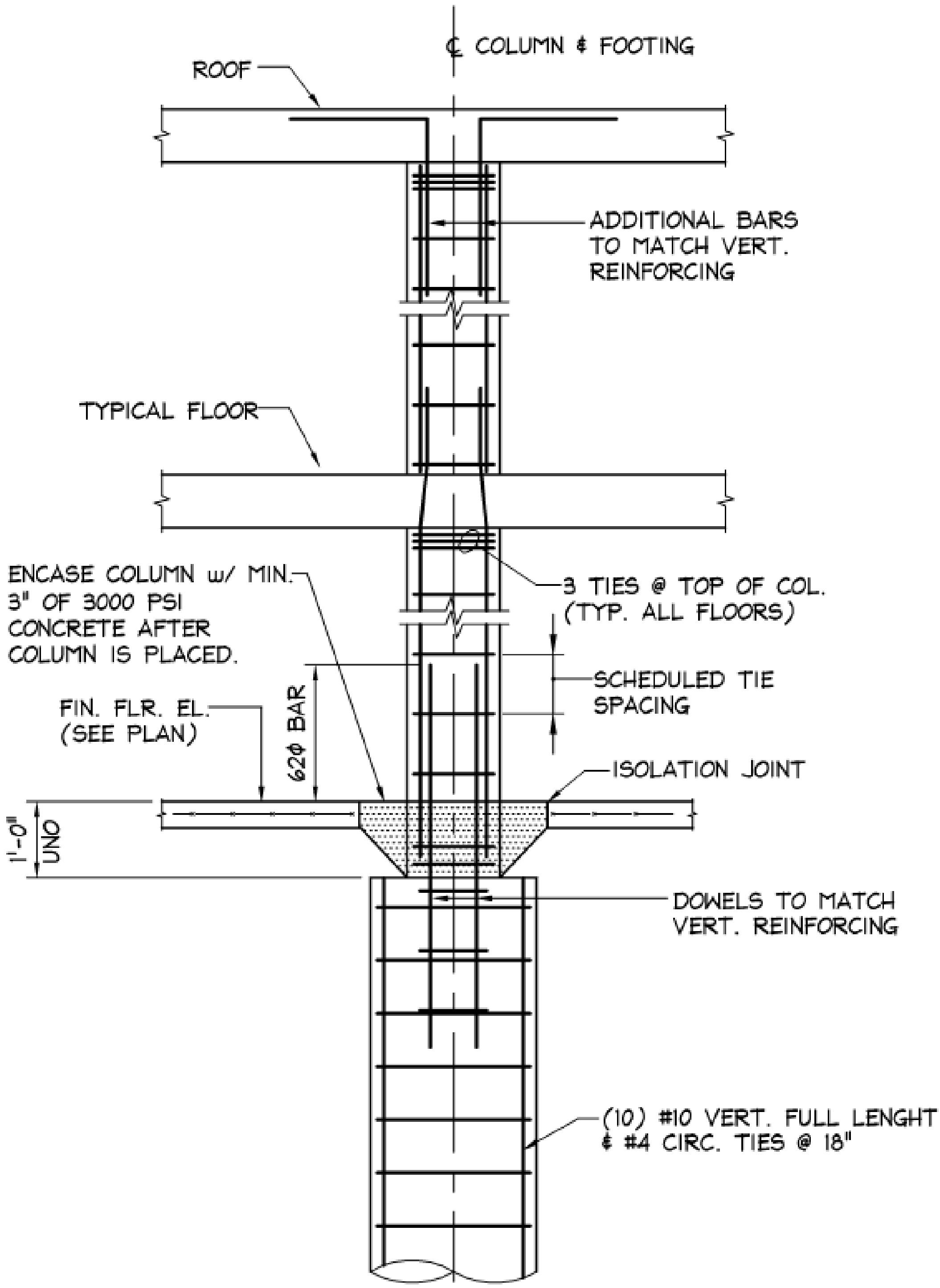


GROUND LEVEL



THIRD LEVEL





TYPICAL CONCRETE COLUMN DETAIL

***Schedule put together by Designer**

SNOW DESIGN LOAD SCHEDULE
INTERNATIONAL BUILDING CODE (2003)

ITEM	SYMBOL	VALUE	REFERENCE
GROUND SNOW LOAD	P_g	25	FIGURE 1608.2
SNOW EXPOSURE FACTOR	C_e	1.0	TABLE 1608.3.1
SNOW LOAD IMPORTANCE FACTOR	I_s	1.0	TABLE 1604.5
THERMAL FACTOR	C_t	1.0	TABLE 1608.3.2
FLAT-ROOF SNOW LOAD	P_f	18	SECTION 1608.3

LATERAL LOAD DESIGN SCHEDULE
INTERNATIONAL BUILDING CODE (2003)

WIND LOAD			
ITEM	SYMBOL	VALUE	REFERENCE
BASIC WIND SPEED (3 SEC. GUST)	V	90	FIGURE 1609
WIND LOAD IMPORTANCE FACTOR	I_w	1.0	TABLE 1604.5
WIND EXPOSURE CATEGORY	-	B	SECTION 1609.4
SEISMIC LOAD			
ITEM	SYMBOL	VALUE	REFERENCE
IMPORTANCE FACTOR	I_E	1.0	TABLE 1604.5
SHORT PERIOD SPECTRAL ACCELERATION	S_{DS}	0.32g	SECTION 1615.1
(1) SECOND PERIOD SPECTRAL ACCELERATION	S_{D1}	0.082g	SECTION 1615.1
SEISMIC USE GROUP	-	II	SECTION 1616.2
SEISMIC DESIGN CATEGORY	-	B	TABLE 1616.3
SITE CLASSIFICATION	S	D	TABLE 1615.1.1
BASIC STRUCTURAL SYSTEM	-	MOMENT RISISTING FRAME SYSTEM	TABLE 1617.6
BASIC SEISMIC-RESISTING SYSTEM	-	ORDINARY REINF. CONC. MOMENT FRAME	TABLE 1617.6
RESPONSE MODIFICATION FACTOR	R	3	TABLE 1617.6
DEFLECTION AMPLIFICATION FACTOR	C_d	2 1/2	TABLE 1617.6
ANALYSIS PROCEDURE	EQUIVALENT LATERAL FORCE PROCEDURE		SECTION 1617.4

DESIGN LOAD SCHEDULE
(ALL LOADS SHOWN ARE IN POUNDS PER SQ. FT.)

COMPONENT \ AREA	SLAB ON GRADE	1st&2nd FLR.	3rd-9th FLR.	ROOF	MECH. PAD	
CONCRETE SLAB	75	150	113	113	113	
ROOF & INSULATION				11	11	
COLLATERAL		10	8	6	6	
PARTITIONS			20			
TOTAL DEAD LOAD	75	160	141	130	130	
TOTAL LIVE LOAD	100	100	40	30	80	
TOTAL LOAD	175	260	181	160	210	

CAISSON CAPACITY (kips)

BOTT. EL. \ CAISSION DIA.	CA1	CA2
41.00'	1260k	650k
43.00'	-	825k
45.00'	1375k	-
47.00'	1480k	890k
49.00'	-	1055k
51.00'	1715k	-
53.00'	1880k	-

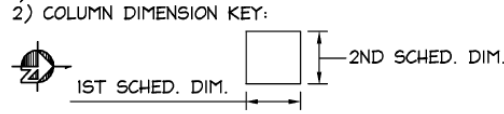
CONCRETE COLUMN SCHEDULE

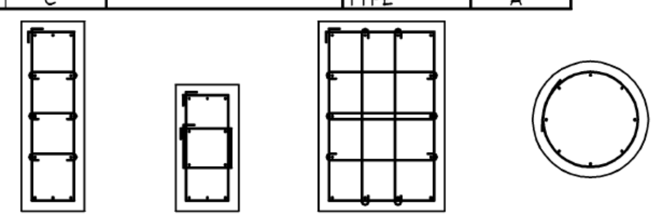
(f'c = 5000 psi)

LEVEL	MARK	C1	C2	C3
4th LEVEL TO ROOF LEVEL	SIZE	20"x30"	20"x30"	20"x30"
	VERT. REINF.	(10) #7	(12) #7	(12) #7
	TIES	#3@12	#3@12	#3@12
	TYPE	A	A	A
3rd LEVEL TO 4th LEVEL	SIZE	20"x30"	20"x30"	20"x30"
	VERT. REINF.	(10) #8	(12) #9	(12) #10
	TIES	#3@12	#3@12	#3@12
	TYPE	A	A	A
2nd LEVEL TO 3rd LEVEL	SIZE	20"x30"	20"x30"	30"x30"
	VERT. REINF.	(12) #8	(12) #10	(14) #10
	TIES	#3@12	#3@12	#3@12
	TYPE	A	C	C
FOUNDATION TO 2nd LEVEL	SIZE	20"x30"	30"x30"	30"x30"
	VERT. REINF.	(12) #8	(14) #10	(14) #10
	TIES	#3@12	#3@12	#3@12
	TYPE	A	C	C

LEVEL	MARK	C1A
6th LEVEL TO ROOF LEVEL	SIZE	20"ϕ
	VERT. REINF.	(8) #6
	TIES	#3@12
	TYPE	D
3rd LEVEL TO 6th LEVEL	SIZE	20"x30"
	VERT. REINF.	(10) #7
	TIES	#3@12
	TYPE	D
2nd LEVEL TO 3rd LEVEL	SIZE	20"x30"
	VERT. REINF.	(12) #8
	TIES	#3@12
	TYPE	A
FOUNDATION TO 2nd LEVEL	SIZE	20"x30"
	VERT. REINF.	(12) #8
	TIES	#3@12
	TYPE	A

NOTES:
1) DIMENSIONS AND TIE SPACING INDICATED IN INCHES.
2) COLUMN DIMENSION KEY:





TYPE A TYPE B TYPE C TYPE D

Seismic Calculations

Occupancy Category	*II*
Seismic Use Group	*II*
Importance Factor	1.00
Site Classification	*D*
Structural System	*Moment Resisting Frame System*
Seismic-Resisting System	*Ordinary Reinforced Concrete Moment Frame*

Philadelphia, PA
9 Levels
100 ft
10' Stories
68' x 148' Building Plan

Philadelphia, PA

$S_s = 32\%$ FIGURE 9.4.1.1(a) $F_a = 1.54$ TABLE 9.4.1.2.4a
$S_1 = 8.2\%$ FIGURE 9.4.1.1(b) $F_v = 2.40$ TABLE 9.4.1.2.4b

$S_{MS} = F_a S_s \rightarrow S_{DS} = ⅔ S_{MS} = 0.329$
$S_{M1} = F_v S_1 \rightarrow S_{D1} = ⅔ S_{M1} = 0.131$

Seismic Use Group *II*

$0.167g \leq S_{DS} < 0.33g \rightarrow$ Seismic Design Category *B* TABLE 9.4.1.2a
$0.067g \leq S_{D1} < 0.133g \rightarrow$ Seismic Design Category *B* TABLE 9.4.1.2b

Moment Resisting Frame Systems

Ordinary Reinforced Concrete Moment Frames $\rightarrow$ $R = 3$ TABLE 9.5.2.2
$W_0 = 3$
$C_d = 2½$

Analytical Procedure

Equivalent Lateral Force Analysis TABLE 9.5.2.5.1

Base Shear

$$C_s = \frac{0.329}{(3/1.00)} = 0.110 > C_s = \frac{0.131}{(1.11)(3/1.00)} = 0.039 > C_s = 0.044(0.329)(1.0) = .015$$

$W \rightarrow W_R = (138\ \text{PSF})(148\ \text{FT})(68\ \text{FT}) = 1390^K$
$W_{3\text{-}9} = (148\ \text{PSF})(148\ \text{FT})(68\ \text{FT}) = 1489^K$
$W_{1\text{-}2} = (170\ \text{PSF})(148\ \text{FT})(68\ \text{FT}) = 1711^K$
$W_T = 1390^K + 7(1489^K) + 1711^K = 13524^K$

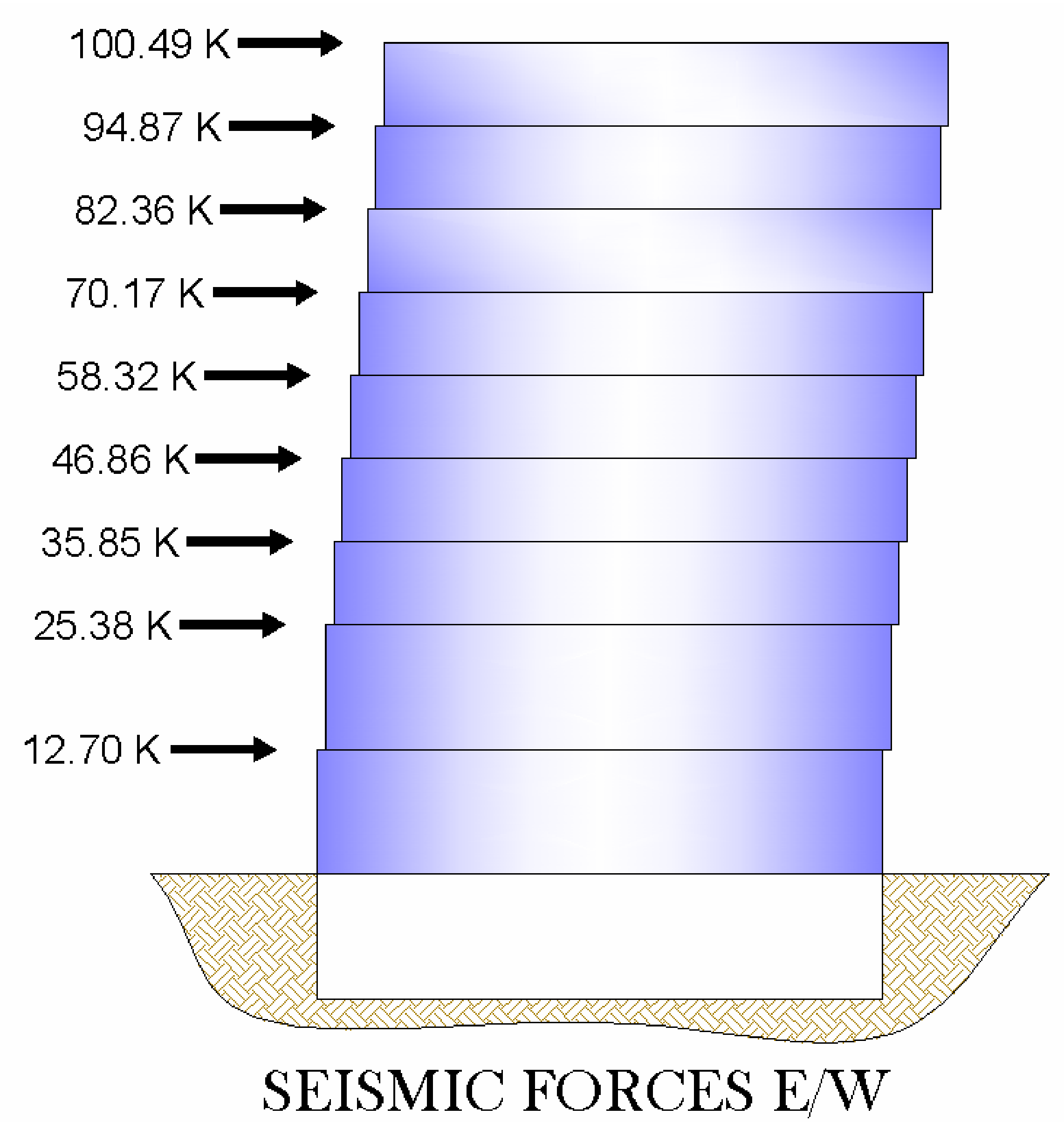
$V = C_s W \rightarrow (0.039)(13524^K) = 527^K$

Vertical Distribution

$F_x = C_{vx}V$ *See Spreadsheet*
$C_{vx} \rightarrow w_x h^k_x / \sum w_i h^k_i \rightarrow k = 1.20 \quad (0.5 < T < 2.5)$

Overturning

$M_x = \sum F_i(h_i - h_x)$ *See Spreadsheet*


100.49 K
94.87 K
82.36 K
70.17 K
58.32 K
46.86 K
35.85 K
25.38 K
12.70 K
SEISMIC FORCES E/W

Wind Calculations

Location	Philadelphia, PA	Typography	Homogeneous
Dimensions	PLAN 148' x 68'	Enclosure Class	Fully Enclosed
	HEIGHT 99'-6"	Framing System	Moment Frame
Occupancy Category	II		
Importance Factor	1.00		
Exposure Category	B		

Building Frequency $T_a = 0.1N \rightarrow \frac{1}{T} = 1.11$ [9.5.5.3.2]

$\frac{1}{T} \geq 1 \rightarrow$ Rigid Structure

*Analytical Procedure

Velocity Pressure

$q_z = 0.00256\, K_z\, K_{zt}\, K_d\, V^2 I$

V_3	90	FIGURE 6-1
I_w	1.00	TABLE 6-1
K_d	0.85	TABLE 6-4
K_{zt}	1.00	

Internal Pressure Coefficient

$GCpi$ ±0.18 FIGURE 6-5

Gust Effect Factor

B	68	
h	99.5	
c	0.3	TABLE 6-2
ℓ	320	TABLE 6-2
$\in$	1/3.0	TABLE 6-2
z_{min}	30	TABLE 6-2
z	59.7	$(0.6h \geq z_{min})$
L_z	390	
$g_Q = g_v$	3.4	
Q	0.854	
I_z	0.272	
G	0.842	→ use 0.85

External Pressure Coefficient

North/South

		C_p
L/B = 148/68	Windward	0.80
= 2.18 → 2.00	Leeward	- 0.30
h/L = 100/148	Side	- 0.70
= 0.68 → ≥ 1.0	0 to h/2	-0.95
	h/2 to h	-0.83
	h to 2h	- 0.57

East/West

		C_p
L/B = 68/148	Windward	0.80
= 0.46 → 0-1	Leeward	- 0.50
h/L = 100/68	Side	- 0.70
= 1.47 → ≥ 1.0	0 to h/2	- 1.04
	> h/2	- 0.70

Area Reduction Factor

(h/2)(148) ≥ 1000→ 0.8(-1.3)

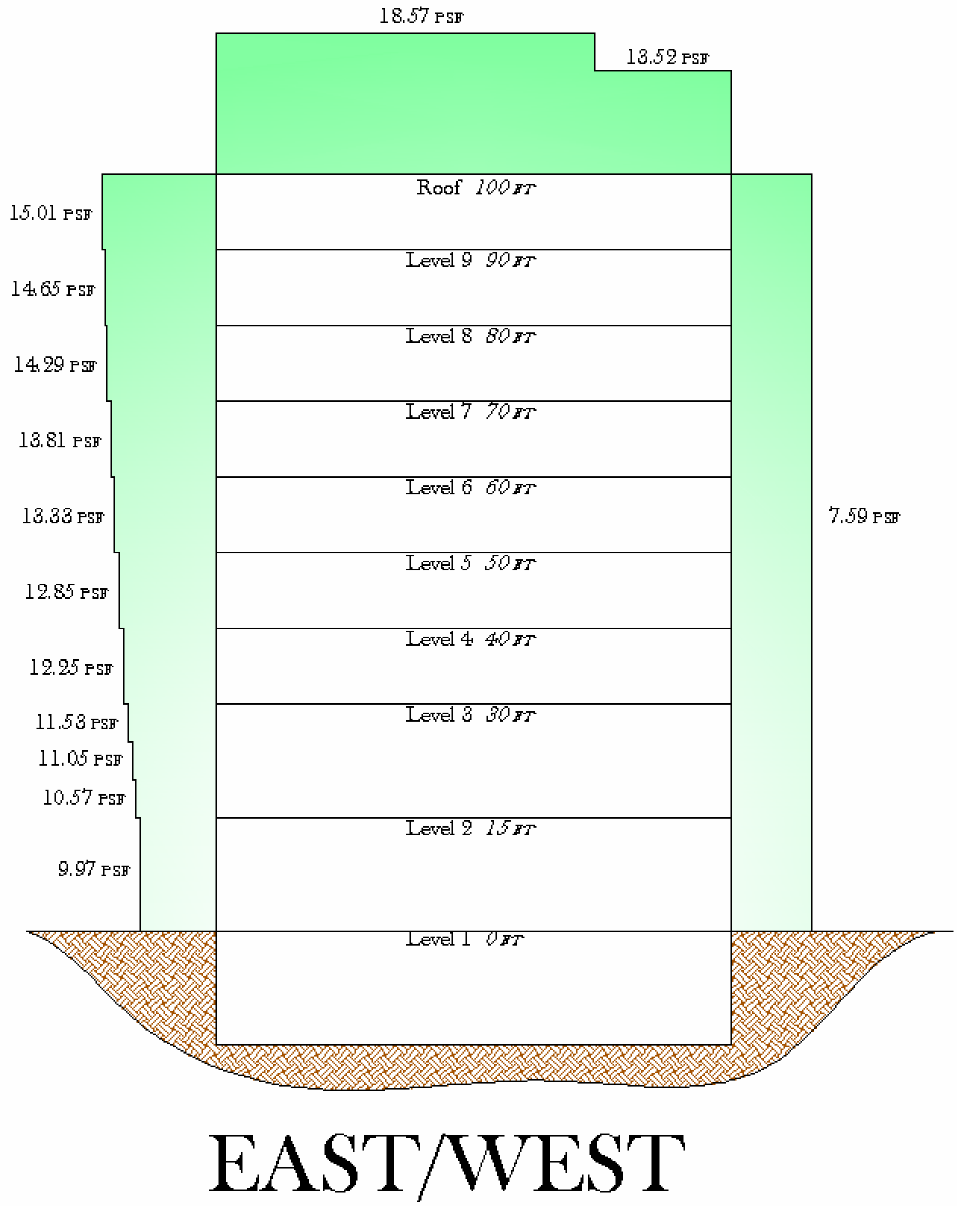
Main Wind Force-Resisting Systems

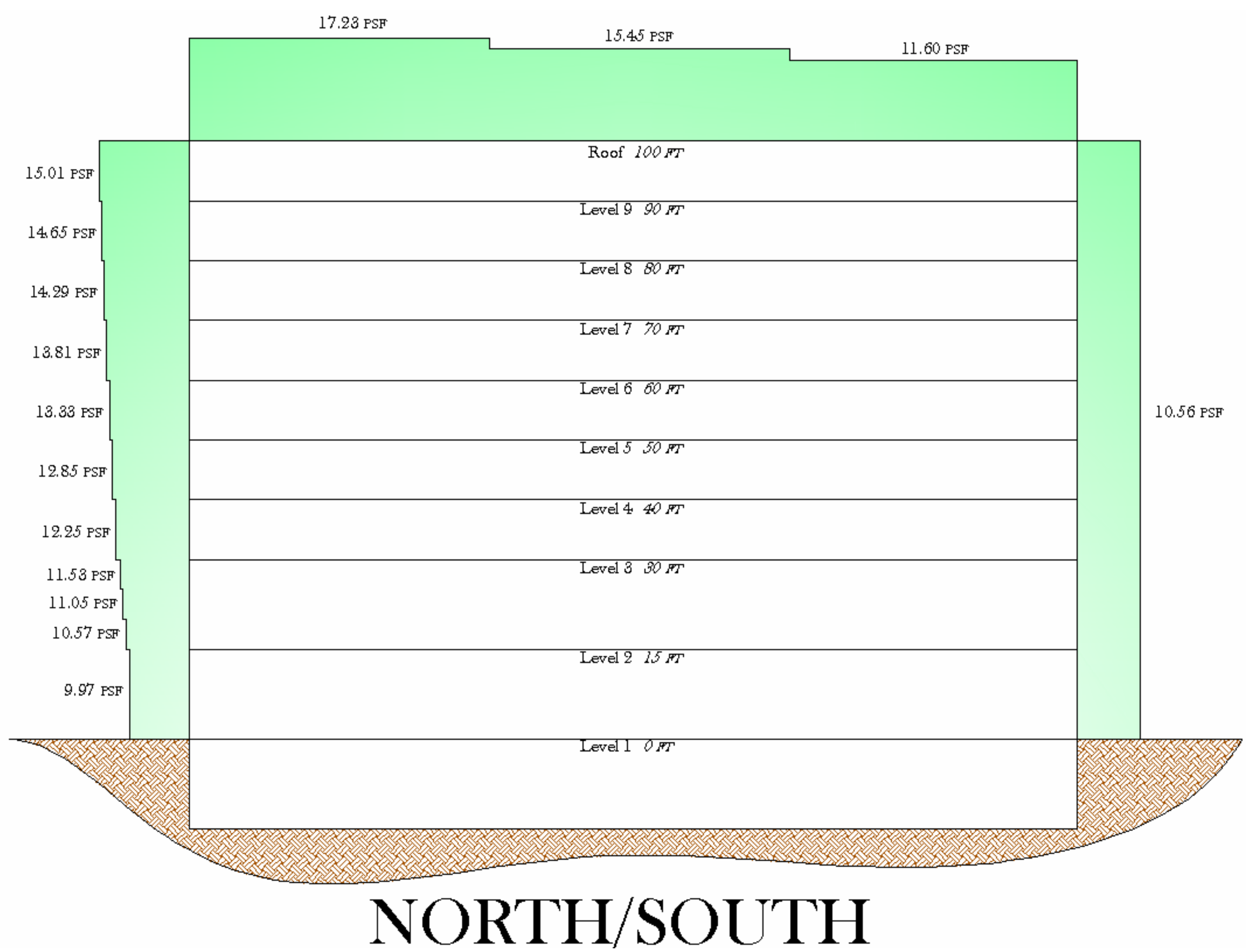
$p = qGC_p - q_i(GC_{pi})$

Windward

$0 < z < h$ $p = q_z GC_p - q_h(GC_{pi})$

$z = h$ $p = q_h GC_p - q_h(GC_{pi})$


18.57 PSF
13.52 PSF
15.01 PSF
14.65 PSF
14.29 PSF
13.81 PSF
13.33 PSF
12.85 PSF
12.25 PSF
11.53 PSF
11.05 PSF
10.57 PSF
9.97 PSF
7.59 PSF
Roof 100 FT
Level 9 90 FT
Level 8 80 FT
Level 7 70 FT
Level 6 60 FT
Level 5 50 FT
Level 4 40 FT
Level 3 30 FT
Level 2 15 FT
Level 1 0 FT
EAST/WEST


17.23 PSF
15.45 PSF
11.60 PSF
15.01 PSF
14.65 PSF
14.29 PSF
13.81 PSF
13.33 PSF
12.85 PSF
12.25 PSF
11.53 PSF
11.05 PSF
10.57 PSF
9.97 PSF
10.56 PSF
Roof 100 FT
Level 9 90 FT
Level 8 80 FT
Level 7 70 FT
Level 6 60 FT
Level 5 50 FT
Level 4 40 FT
Level 3 30 FT
Level 2 15 FT
Level 1 0 FT
NORTH/SOUTH

Summary of Lateral Forces

Wind/Seismic Shear Forces

	Wind		Seismic	
Shear Level	*North/South*	*East/West*	*N/S/E/W*	*Total*
Roof	5.55	14.27	100.49	114.76
9	10.97	28.27	94.87	123.14
8	10.73	27.74	82.36	110.10
7	10.44	27.12	70.17	97.29
6	10.12	26.41	58.32	84.73
5	9.79	25.70	46.86	72.56
4	9.42	24.90	35.85	60.75
3	11.08	29.60	25.38	54.98
2	11.89	32.47	12.70	45.17
1	5.75	15.81	0.00	15.81
Base Shear (K)	95.74	252.28	527.0	779.28
Overturning (FT-K)	5115.50	13328.86	38316.24	51645.10

Level 5 (Interior Column Axial)

20" × 30"

Loads: $D_{LR} = 138\ ^{lb}/_{ft^2}$ $D_{CL} = (150^{lb}/_{ft^3})(10')$
$D_L = 148\ ^{lb}/_{ft^2}$ $(30)(20)(1/144)$
$L_{LR} = 30\ ^{lb}/_{ft^2}$ (4 Levels)
$L_L = 40\ ^{lb}/_{ft^2}$ $= 25^k$

Tributary Area
$b = (24'6")/2 + (31'-0")/2 = 27.75' \rightarrow 28'$
$a = (27'-5")/2 + (22'6")/2 = 25'$

ASCE 7 4.8.1 $L_L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right)$ $K_{LL} = 4$

$= L_o\left(0.25 + 15/\sqrt{(4)(28)(25)}\right)$

ASCE 7 4.9.1 $= 0.53 L_o > 0.40 L_o \therefore OK$

Roof$_L$ Reduction
$L_r = 20R_1R_2$ $12 \le L_r \le 20$
$A_T = 700\ ft^2 \rightarrow R_1 = 0.6$
$F = 1.0 \longrightarrow R_2 = 1.0$
$L_r = 20(0.6)(1.0) = 12 \therefore OK$

$L_R = (20)(0.6)(1.0)(28')(25') = 8400^{lbs}$
$L_L = 0.53(40^{lb}/_{ft^2})(28')(25')(4) = 14840^{lbs}$
$D_R = (138^{lb}/_{ft})(28')(25') = 96600^{lbs}$
$D_L = (4)(148^{lb}/_{ft})(28')(25') = 414400^{lbs}$
$D_C =$

Total Axial Load (Factored)

$P_u = 1.2(96.6^k + 414.4^k + 25^k) + 1.6(8.4^k + 14.8^k)$
$= 680.3^k$

$$P_u = \phi 0.80(0.85 f'_c(A_g - A_{ST}) + f_y A_{ST})$$

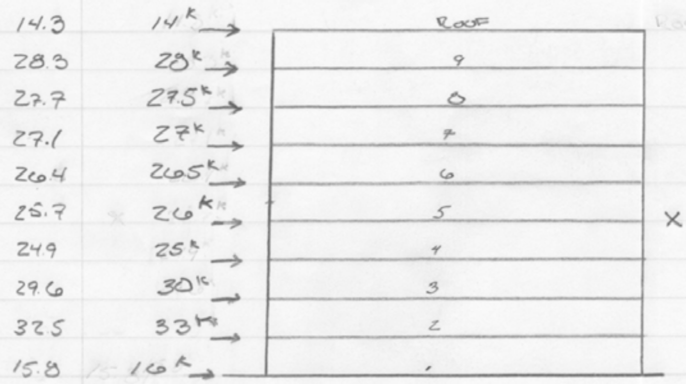
$\phi = 0.70$
$f'_c = 5000$ psi
$A_g = (30'')(20'') = 600\ in^2$
$f_y = 60$ ksi

$$680.3^k = 0.70(0.80)((0.85)(5)(600 - A_{ST}) + (60)A_{ST})$$

A_{ST} = very low amount of steel

*Assume design by engineer to compensate moment in column due to moment frames.

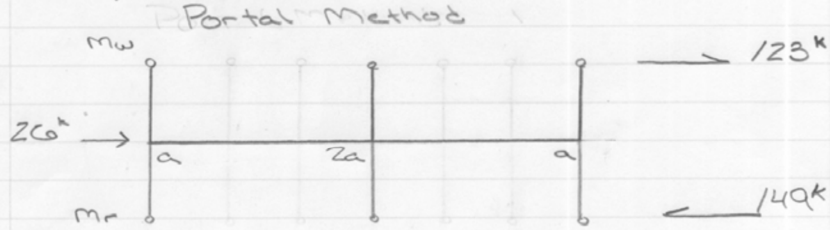
Find Moment
on Interior Column
Level 5



* EAST/WEST Controls Wind

x level of interest

Analyze Level 5
Portal Method



Floors above : $14^k + 28^k + 27.5^k + 27^k + 26.5^k = 123^k$

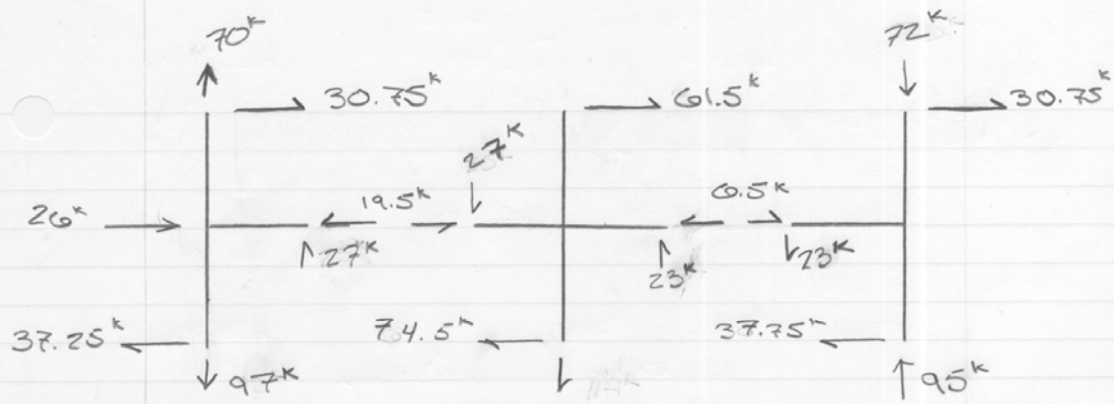
5th + above : $123^k + 26^k = 149^k$

$M_w = (26.5)(10/2) + (27)(10/2 + 10) + (27.5)(10/2 + 20)$
$+ (28)(10/2 + 30) + (14)(10/2 + 40)$
$= 3835'^k$

$M_r = M_w + (26)(10/2) + (123)(10)$
$= 5195'^k$

$123^k = a + 2a + a \longrightarrow a = 30.75^k$

$149^k = a + 2a + a \longrightarrow a = 37.25^k$

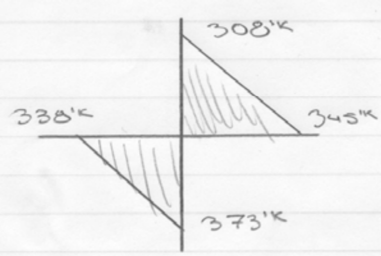


$M_r = 5195'^k / (25' + 30')$
$= 95^k$

$M_w = 3835'^k / (30' + 25')$
$= 70^k$

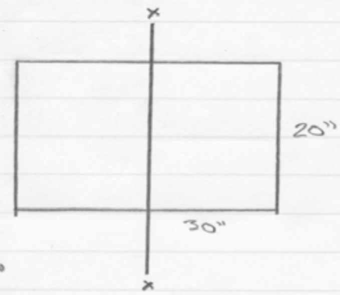
Analyze Center Column

$(27^k)(12.5') = 338'^k$
$(23^k)(15') = 345'^k$
$(61.5^k)(5') = 308'^k$
$(74.5^k)(5') = 373'^k$ → Design Colum M_u



Column Check (moment due to LATERAL Forces)

Level 5



$M_u = 373'^k$
$P_u = 680^k$
(see previous example)

$P_n = 971^k$
$M_n = 533'^k$

$h = 30"$

$P_u e = M_u \longrightarrow e = (12"/ft)(373'^k) / 680^k$
$= 6.58$

$e/h = 6.58/30$
$= 0.220$

$\phi P_n / A_g = \frac{971^k}{(600)} = 1.62$

$\frac{\phi P_n}{A_g} \frac{e}{h} = \frac{(0.70)(971^k)(6.58)}{(30)(20)(30)} = 0.245$

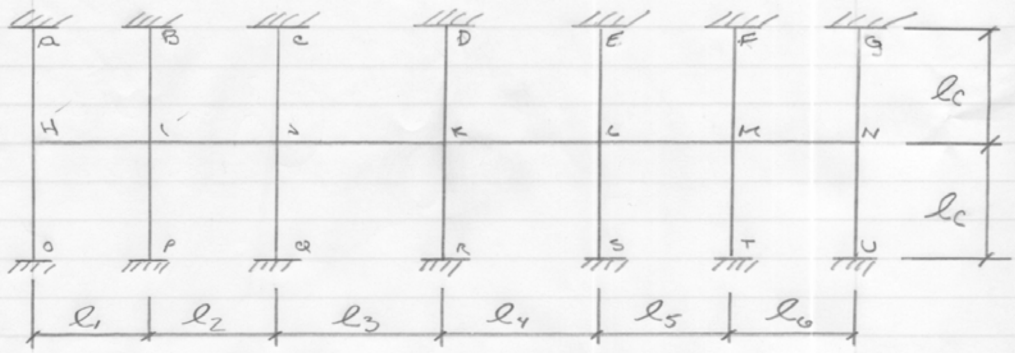
$\gamma h = 25 \text{ in} \longrightarrow \gamma = 25/30 = 0.833$

Interpolate

$\gamma = 0.75 \rightarrow$
$\gamma = 0.90 \rightarrow$ } $\rho_g < 1\%$

* Assume 20" x 30" Column is designed by lower Column in system or Controlled by post-Tensioning

Check One-Way Slab



- Provision of ACI R18.12.1 → analysis should use Equivalent Frame Method

- Geometry → Slab / Colums

  Slab: $l_1 = l_6 = 18.5'$, $l_2 = l_5 = 21.5'$, $l_3 = l_4 = 28.5'$

  $t_{slab} = 9''$

  $f'_c = 5{,}000$ psi

  Colums: All equal $20'' \times 30''$

  $l_c = 10'$ (STORY HT)

  $f'_c = 5{,}000$ psi

- LOADS → As Determined in Report

  D_L = Gravity + Superimposed
  = $148\ ^{lb}/_{ft^2}$

  $L_L = 40\ ^{lb}/_{ft^2}$

- Column Strip / Slab Area = $\left(\frac{24.5'}{2}\right) + \left(\frac{31'}{2}\right)$
  $= 27.75'$
  = USE $28'$

- Assume worst case loading

  All Live + All Dead on All bays

  $w_u = 1.2D_L + 1.6L_L$
  $= 1.2(148^{PSF})(28') + 1.6(40^{PSF})(28')$
  $= 6.765\ ^{K}/_{ft}$

  (ACI 13.7.6.2)

Fixed-End Moments

$FEM_{HI} = -FEM_{IH} = wl^2/12 = (6.8)(18.5)^2/12 = 194^{'-k}$

$FEM_{IJ} = -FEM_{JI} = wl^2/12 = (6.8)(21.5)^2/12 = 262^{'-k}$

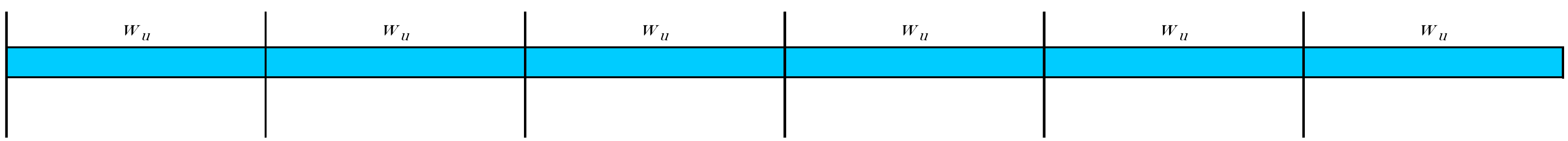
$FEM_{JK} = -FEM_{KJ} = wl^2/12 = (6.8)(28.5)^2/12 = 460^{'-k}$

Symmetry → Mirrors opposite side

Moment Distribution

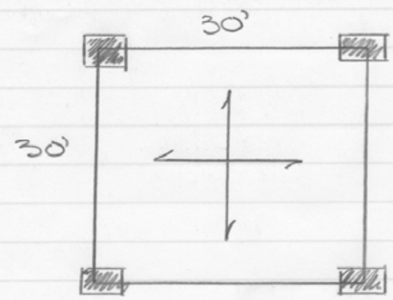
* See spreadsheet

0.212	0.180	0.156	0.166	0.124	0.130	0.130	0.124	0.166	0.156	0.180	0.212
-194	194	-262	262	-460	460	-460	460	-262	262	-194	194
41.13	20.57									-20.57	-41.13
4.27	8.54	7.40	3.70					-3.70	-7.40	-8.57	-4.27
-0.91	-0.46	-16.13	-32.25	-24.09	-12.05	12.05	24.09	32.25	16.13	0.46	0.91
1.50	2.99	2.59	1.30					-1.30	-2.59	-2.99	-1.50
-0.32	-0.16	-0.11	-0.22	-0.16	-0.08	0.08	0.16	0.22	0.11	0.16	0.32
0.07	0.04									-0.04	-0.07
	-0.04	-0.04							0.04	0.04	
-148.26	225.48	-268.29	234.53	-484.25	447.87	-447.87	484.25	-234.53	268.29	-225.51	148.26



Two-way Slab Check

Level 2



SLAB
$t_s = 12"$
$f'_c = 3000$ KSI

#6 @ 16"
BOTH WAYS

$\ell_1 = \ell_2 = 30'$

- LOADS

  Dead : 170 lb/ft² (LOADS From Report)
  Live : 100 lb/ft²

  $w_u = 1.2(170) + 1.6(100)$
  $= 364$ PSF

- ACI R9.5.3.2

  minimum thickness
  w/o drop panel, interior panel
  $f_y = 60$ksi $\longrightarrow \ell_n/33 = (30' - 30"/12")/33$
  $= .8\bar{3} = 10"$

  12" > 10" ∴ OK

- Spacing maximum → $2t_s = 2(12) = 24" > 16"$ ∴ ✓

- Minimum Steel → $0.0018 t_s$ (per ft)

  $= 0.0018(12")(12"/ft)$
  $= 0.259$ in²/ft

  (2) #6 = 0.88 in² → (0.88)(12"/ft)(1/16")
  = 0.66 ∴ OK

  (ACI 7.12.2.1)