North Mountain IMS Medical Office Building

Phoenix, Arizona

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Technical Assignment 3

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AE 481W-Senior Thesis

The Pennsylvania State University

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Executive Summary

North Mountain IMS Medical Office Building utilizes a complete precast concrete framing system. An overview of the entire system can be found in the Typical Framing Plans and Details section. However, this report will focus on analysis and design of the lateral load resisting system. Lateral loads are supported mostly by exterior shear walls; interior shear walls also resist lateral loads.

The lateral load resisting system is explored and analyzed using the PCI Design Handbook, hand calculations, and an ETABS computer model. Both wind and seismic loads are compared resulting with seismic loading controlling lateral design. Hand calculations include determining and distributing lateral forces. Since the precast concrete double tee floor framing provides a rigid diaphragm, lateral loads are distributed based on relative stiffness. The center of rigidity is not the center of lateral load, so torsional effects are also considered within this report. Drift analysis, wall overturning calculations, and a strength check of a shear wall are also included within this report.

The shear wall forces calculated within this report are compared to values listed on the design drawings, resulting in a few discrepancies. Further investigation into the assumptions made for the hand calculations is needed to close these discrepancies to acceptable tolerances.

Codes and Standards

Note: The Senior Thesis project requires the use of the most current codes and standards, those referenced for calculations in this report are listed at the end of this section.

Building Codes:

1. International Building Code (IBC), 2003 edition
2. International Energy Code (IECC), 2003 edition with 2004 supplements
3. National Electric Code (NEC), 2005 edition with Phoenix amendments
4. International Mechanical Code (IMC), 2003 edition with Phoenix amendments
5. Arizona State Plumbing Code, with 2003 supplements
6. Uniform Fire Code (UFC), 1997 edition with Phoenix amendments

Structural Codes:

1. American Concrete Institute (ACI-318), 2002 edition
2. Precast Concrete Institute (PCI), 6th edition

Building Design Loads:

1. American Society of Civil Engineers (ASCE-7), 2002 edition

Thesis Project Codes and Loads:

1. IBC 2005
2. ASCE-7, 2005
3. ACI-318 2005
4. AISC 2005
5. PCI, 6th edition

Loading Analysis

Design Loads:

Live Loads:

- Roof Live Load……………………………………………..20 psf
- Floor Live Load……………………………………………80 psf
- Stair Live Load……………………………………………100 psf
- Partition Live Load…………………………………………20 psf

Dead Loads:

- Superimposed Roof Dead Load……………………………15 psf
- Superimposed Floor Dead Load……………………………15 psf

Wind Load:

- Total Wind Force (North-South Direction)…………………218 kips
- Total Wind Force (East-West Direction)…………….…….285 kips

Seismic Load:

- Design Base Shear………………...……………………..1627 kips

Gravity Loads:

The floor live loads for North Mountain are typical office loads. The second, third, and forth floors all feature an open floor plan with no set dimensions for walls or corridors. Because of the open floor plan, the floor live load is 80 psf. By code, corridor loading above the first floor is 80 psf. However, 50 psf is the minimum recommended live load for office space above the first floor. For design, the corridor value was used as the live load over the entire floor; it is much easier to assume a uniform load over the entire floor compared to breaking the loads down between office and corridors. Also, a partition live load of 20 psf is used over the entire floor.

The floor dead load only accounts for 15 psf of superimposed load which includes mechanical, electrical, and pluming equipment. The nature of precast concrete structures makes it very simple to calculate the actual weight of the structure; a dead load in pounds per square foot is not needed because each piece of precast is detailed and the exact weight calculated. Tabulated structure weights can be found on pages 12-15 in the Appendix.

In Phoenix, there is no snow load. However, a roof live load is still required. This live load accounts for potential ponding of rain water and construction loads.

Wind Load:

Wind Load Factors:
Basic Wind Speed, V = 90 mph
Importance Factor, I = 1.15
Occupancy Category, IV
Exposure Category, B
Topographic Factor, Kzt = 1.0
Gust Factor, G = 0.803 (E-W) 0.814 (N-S)
Exposure Classification, Enclosed
Internal Pressure Coefficient, GCpi = +-0.18
External Pressure Coefficient, Cp = 0.8 (Windward) -0.5 (Leeward) -0.7 (Side)

Wind load was not expected to control the lateral design due to the overall dimensions of North Mountain. The building is fairly short and it is not located in a high wind zone. Also, there are no abnormal site features, such as hills or valleys, which would increase the wind speed. Wind load calculations are based on ASCE 7 Method 1. The resulting calculations gave a base shear value of 218 kips in the North-South direction and 285 kips in the East-West direction. Complete wind load calculations are provided in the Appendix on page 16.

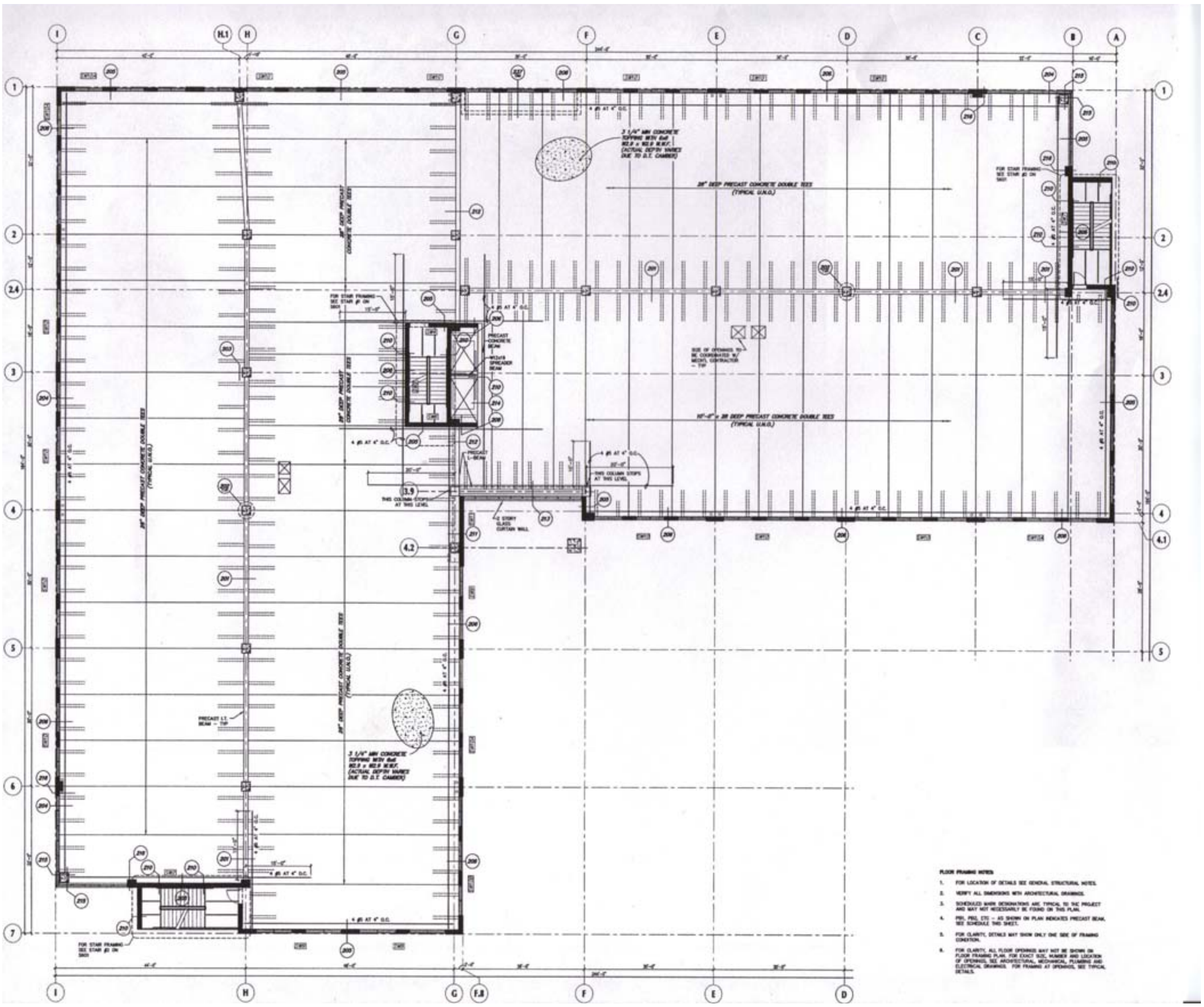
Seismic Load:

Seismic Load Factors:
Seismic Response Coefficient, Cs = 0.0769
Total Dead Load, W = 21,153 kips
Spectral Response Accelerations, Ss = 0.256, S_1 = 0.075
Site Classification, C
Response Accelerations, Sms = 0.307, Sm_1 = 0.128
5% Damped Design Spectral Response Accelerations, Sds = 0.205, Sd_1 = 0.085
Approximate Fundamental Period, Ta = 0.409 s

Seismic loading controls the lateral design. The design base shear for seismic loads is actually over five times higher than the shear load due to wind. The precast structure is very heavy, which is the main cause for such a high seismic load. The calculated design base shear is 1627 kips. Complete seismic load calculations can be found on page 19 in the Appendix.

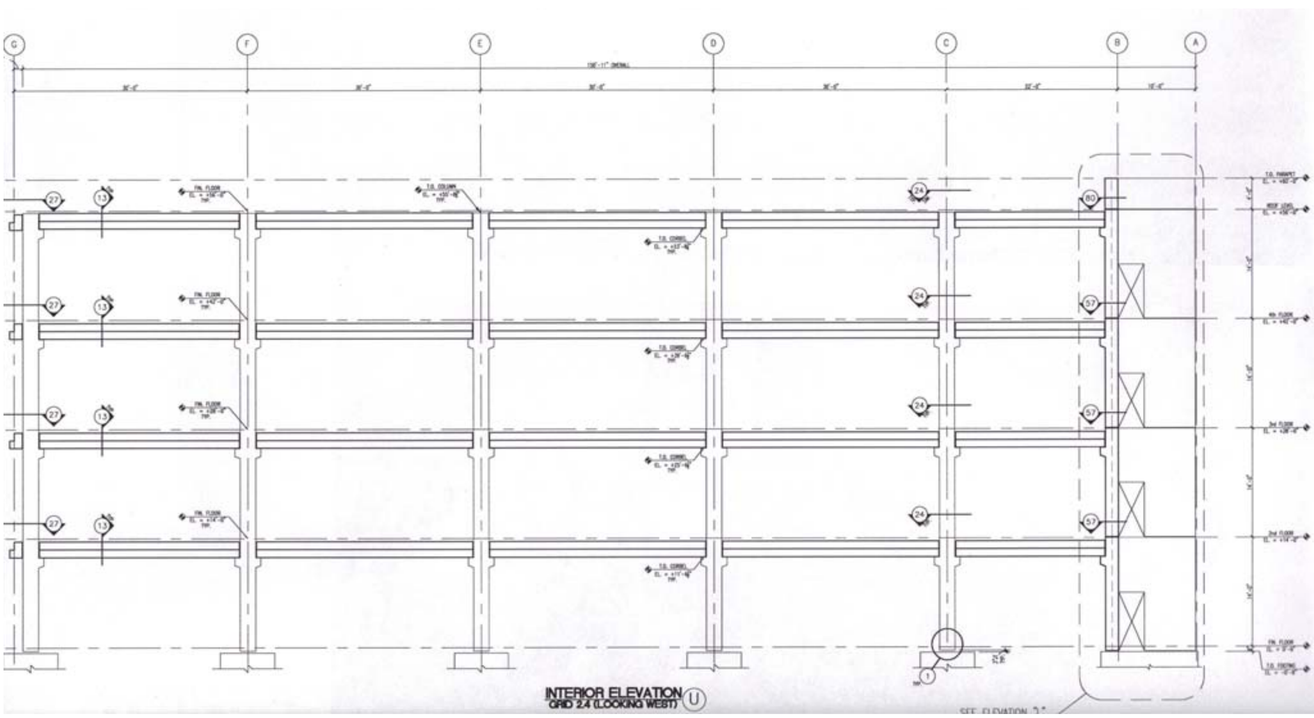
Typical Framing Plans and Details

North Mountain IMS Office Building floor framing consists of 24” deep, 10’ wide double tees with a minimum of 3-1/4” concrete topping. The tees are normal-weight concrete and have a 28-day compressive strength of 6,000 psi. The minimum prestress release strength is 4,200 psi. The prestressing strand is 7 wire, ½” diameter 270 ksi low relaxation strand. Each strand is pulled to 72.5% capacity, which results in a 30 kip force. The strand is held down at one point in the middle of the tee. Depressed strand provides greater flexural strength while reducing the stresses in the concrete during prestress release. Typical spans are 44’, 48’, and 54’. A typical floor plan is shown below.



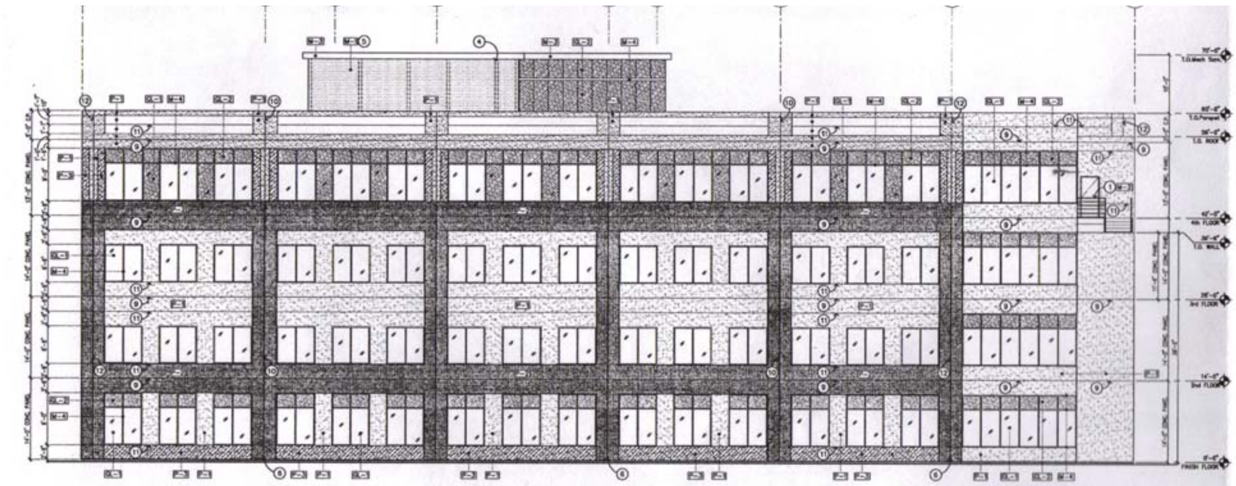
The 24" deep double tees are supported on the interior by 24" deep by 32" wide inverted tee girders. 28-day strength is 7,500 psi and minimum release strength is 3,750 psi. Typical inverted tee girders use 22 ½" diameter stand for tensile reinforcement. Span length for a 30' bay is 28' due to the columns on each end. Dapped ends on the double tees allow the top of the tee to line flush with the top of the girder. The topping is then poured over the tee and the girder at the same time, interlocking them. This construction technique is known as emulation. Emulation design creates construction that is either monolithic at critical joints, or provides connections that act as if they are monolithic at those locations. This is a great way to connect precast pieces in high seismic zones.

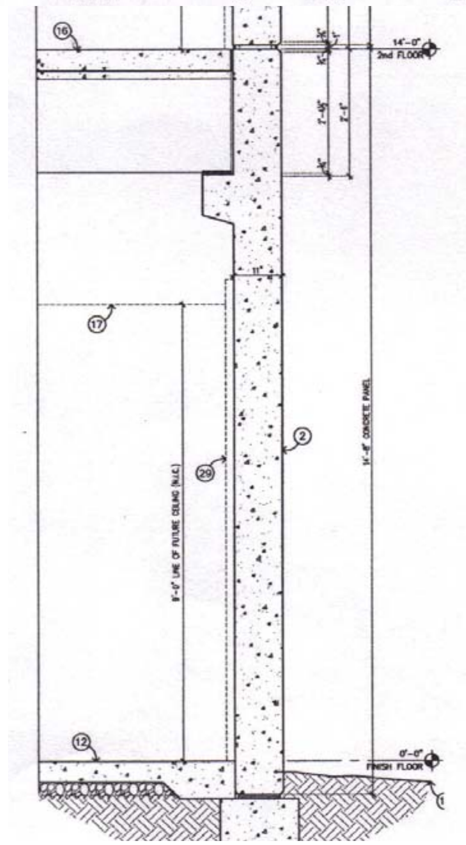
Interior spans of inverted tee girders bear on 24" x 24" columns. Concrete strength is 6,000 psi. There is no need for prestressing strand in columns, because there is no large tensile zone. Any tension in the columns is addressed with traditional reinforcing bars. These columns are 56' tall and arrive on site in one piece. These columns showcase precast concrete's advantages over other structural systems. The columns only need one connection, to the foundation. This ease of construction makes North Mountain's erection duration much shorter compared to other systems. However, long lead times may be an issue due to cure time and storage at the precast fabrication plant. A typical interior elevation is shown below to demonstrate the bearing conditions for inverted tee girders and columns.



The exterior walls for North Mountain IMS Office Building fulfill many different structural requirements. First, and most importantly, they provide the building enclosure. Second, they support gravity load from double tees. Third, the walls are detailed to provide a pleasant architectural aesthetic. Last, but also extremely important, they resist the lateral forces due to wind and earthquakes. These walls give the structure its rigidity and structural integrity. Without shear walls, a moment-resisting frame system would have to be used. This structure utilizes interior and exterior shear walls. The interior shear walls are located in the center of the building around the elevator shaft and a stair tower. Shear wall design will be discussed at length in the Lateral Load Resisting System section of this report.

Below is an exterior elevation. It is easy to notice the different textures applied to the exterior of these walls. These finishes are applied when the panels are cast, which makes for no further work when they arrive on site. Also, the exterior wall sections depict the bearing condition for double tees.

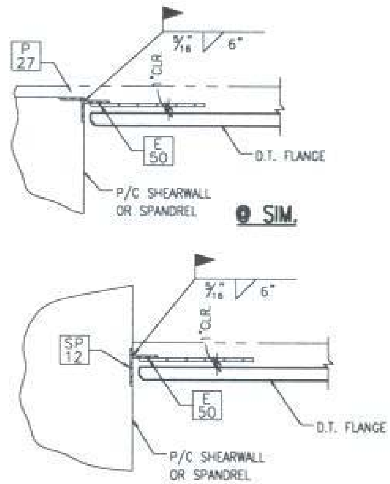




Lateral Load Resisting System

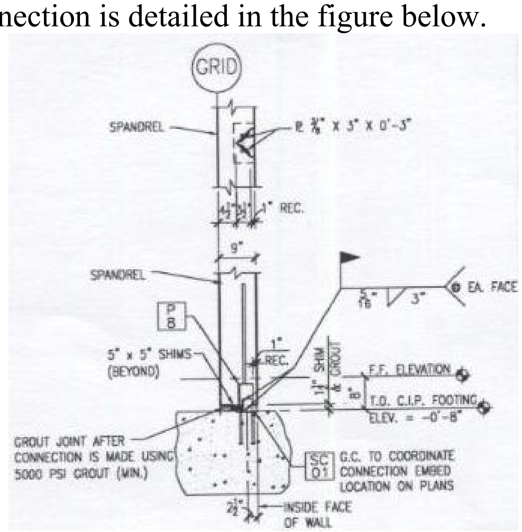
As stated previously, North Mountain IMS Office Building uses a lateral load resisting system comprised entirely of shear walls. These shear walls resist code required earthquake load as well as provide the building envelope. The walls also support double tees and inverted tee girders. This added weight is useful because it increases the panel's resistance to uplift and overturning.

The lateral load collected in the diaphragm is transferred to the shear wall through a series of connections. The figure below illustrates a typical connection along the edge of a double tee. A similar connection is used above each stem at the end of a double tee.

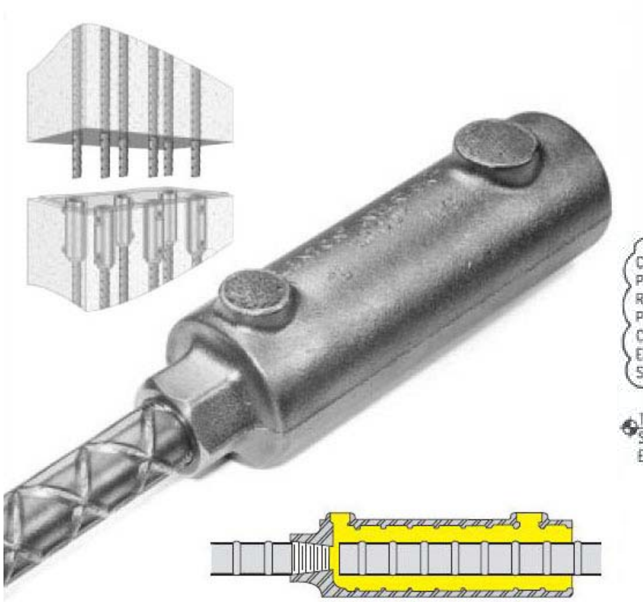


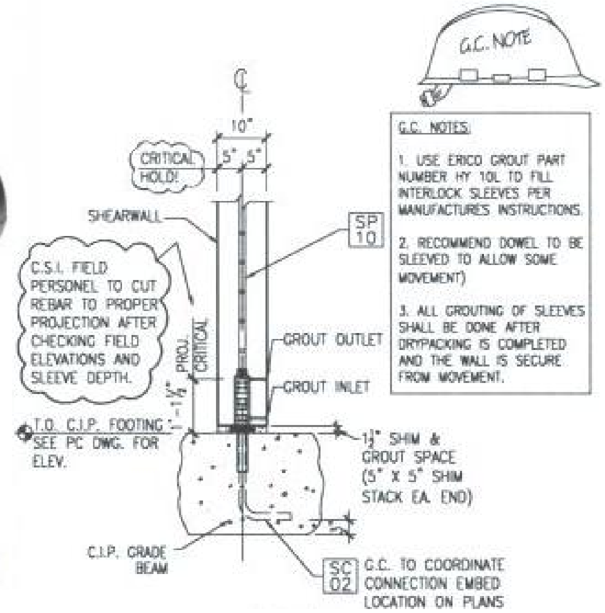
Spaced at equal distances, a plate is embedded into the shear walls. Careful coordination and detailing ensure that these elements are in the correct location. A small plate with rebar attached, is welded to the plate in the shear wall. This plate is suspended until the concrete topping is poured. Both the weld and the embedded bars must be sized correctly for the given shear in each connection. The entire connection is then covered by the concrete topping, keeping the welded plates out of view. The scope of this report does not necessitate calculations to verify strength of the above connection. However, these connections are an extremely important part of the later load resisting system.

Connecting each wall panel to the foundation requires embedded plates in both the foundation and the wall. The type of connection used is referred to as a "v" connection; because the plate in the panel looks like a "v". The connection is detailed in the figure below.



There is another connection used to secure the shear walls to the foundation. Lenton manufactures an interlocking rebar system which is utilized to resist the tensile forces caused by overturning. Details of this system are shown below.





When the panels arrive on site, they are placed over dowels protruding out of the foundation. The Lenton connectors slide over the dowels and is then pumped with grout. When the grout cures, the connection provides a continuous link from the top shear wall to foundation. This detail is also used when connecting panel to panel.

Load Distribution

To determine the seismic load distribution, each exterior wall must be considered either a shear wall or ordinary wall. Walls with large openings, such as doors, were not selected as shear walls. The ordinary walls can be neglected from the lateral load resisting system because they will not be attached to the diaphragm. Also, in the event of an earthquake, it is assumed that these walls will yield and load will be redistributed to other shear walls. Referring to the shear wall layout diagram on page 22 in the Appendix, there are 14 shear walls to resist seismic forces in the x-direction and 9 shear walls in the y-direction.

Rigid floor diaphragms distribute lateral forces to individual shear walls by stiffness; flexible floor diaphragms distribute lateral forces by tributary area. So, the first condition effecting load distribution that must be analyzed is the diaphragm. The behavior of the diaphragm as either flexible or rigid depends on many factors including span, aspect ratio and connections. Based on these factors and following PCI standard practice, it is reasonable to assume that the floor diaphragm is rigid. Most precast concrete structures are assumed to be rigid diaphragms.

Since the assumption of a rigid diaphragm has been made, stiffness calculations for each shear wall must be performed. Stiffness was determined for each wall using the equation k=12*E*I/z^3, where E is the modulus of elasticity of the concrete, I is the moment of inertia, and z is the wall height. A spread sheet calculating stiffness is provided in the Appendix on page 25. Since each shear wall has window openings, ledges, and other geometrical abnormalities, finding an equivalent thickness for each wall was necessary to simplify load distribution. Also, the panels vary thickness on each level. The equivalent thickness was calculated by adding the volume of all precast panels in a vertical wall and then dividing by the area the wall occupies in elevation. The openings were included in the area. It should be recognized that the perforated exterior shear walls will behave more like individual frames than solid concrete shear walls. This may be investigated further in later reports to determine a more accurate lateral load distribution. Equivalent thickness calculations can be found on page 23 in the Appendix.

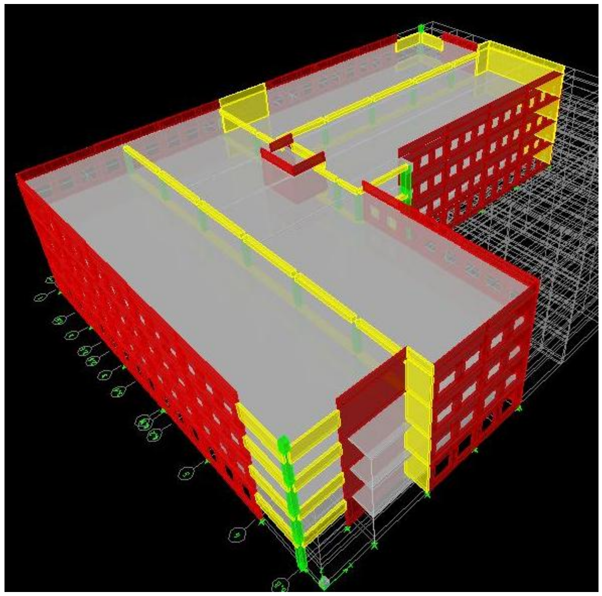
Everything is now prepared to calculate the forces to each shear wall. The equation used to distribute forces is F=k/Σk, which is the individual wall stiffness divided by the sum of all wall stiffness's in the same direction. The direct shear distribution for each wall is on page 26 in the Appendix. These values are direct shear only, meaning that building torsion effects have not been included. Building torsion will be discussed in the next section of this report.

Building Torsion

Building torsion is caused by an eccentricity of the lateral load and the center of rigidity. The PCI Design Handbook prescribes that actual and accidental torsion be accounted for. Accidental torsion only needs to be considered if the eccentricity in a given direction is less than 5% of the plan dimension in the same direction. Eccentricities in both plan directions are greater than 5%, therefore only actual torsion will be considered. The shear wall layout on page 22 in the Appendix shows the location of both the center of mass and the center of rigidity. Based on these calculations, the center of rigidity produces an eccentricity in the x-direction of 35'-6" and 27'-0" in the y-direction.

The equation used to calculate the additional shear produced by the torsional moment in each shear wall, was $V_i = V*e*d_i*k_i/J$. V is the base shear, e is the eccentricity, d_i is the distance of the individual shear wall to the center of rigidity, k_i is the wall stiffness and J is the torsional moment of inertia. J is calculated by the equation $\Sigma k_i d_i^2$. The calculation for J is on page 29 and the torsional shear calculations are on page 30 in the Appendix.

Drift



Drift calculations were determined by ETABS. The computer program calculated a maximum drift of 0.21” in the y-direction and 0.15” in the x-direction. Industry standards restrict a building to drift no more than H/400. This would allow the top story to deflect a maximum of 1.68”. Clearly, drift in each direction does not control the design of the shear walls.

Conclusions

The shear wall comparison summary is shown below. There are many discrepancies between the two sets of data. Further investigation is required to determine the factors accounting for the differences.

Tech 3 Results					
Shear Wall	Lateral Load to Wall (Ultimate)				
	2nd floor	3rd floor	4th floor	Roof	Total
B_{x1}	24.2	37.4	62.8	61.2	158.6
F_{x2}	2.0	3.2	5.4	5.3	15.9
F_{x3}	2.0	3.1	5.3	5.2	15.6
E_{x4}	12.8	20.5	34.4	33.5	101.2
B_{x5}	21.2	32.6	54.6	53.3	161.7
A_{x6}	25.1	40.1	67.4	65.7	198.3
C_{x7}	32.8	52.4	88.1	85.9	259.2
D_{x8}	16.2	25.8	43.5	42.3	127.8
A_{y1}	41.6	66.3	111.6	108.5	328.0
B_{y2}	33.6	53.8	90.5	88.1	266.0
E_{y3}	5.7	9.1	15.4	15.0	45.2
D_{y4}	9.0	14.3	24.0	23.5	70.8
A_{y5}	30.8	49.0	82.7	80.3	249.9
C_{y6}	48.9	78.1	131.4	128.0	386.4

Design Documents					
Shear Wall	Lateral Load to Wall (Ultimate)				
	2nd floor	3rd floor	4th floor	Roof	Total
SW10	0.0	22.0	35.0	57.0	114.0
SW4	53.0	1.0	8.0	4.0	66.0
SW6	82.0	22.0	25.0	5.0	134.0
SW10	0.0	22.0	35.0	57.0	114.0
SW10	0.0	22.0	35.0	57.0	114.0
SW10A	33.0	27.0	42.0	79.0	181.0
SW2	65.0	91.0	113.0	71.0	340.0
SW9	5.0	13.0	21.0	42.0	81.0
SW10A	33.0	27.0	42.0	79.0	181.0
SW10	0.0	22.0	35.0	57.0	114.0
SW3	103.0	64.0	88.0	15.0	270.0
SW9	5.0	13.0	21.0	42.0	81.0
SW10B	12.0	20.0	40.0	100.0	172.0
SW7	70.0	94.0	120.0	94.0	378.0

The design professional modeled the building in ETABS, and used the program to determine the lateral load distribution. The shear walls in ETABS were modeled more accurately than the hand calculations presented in this report. Since the walls have large perforations, each shear wall will behave as an individual frame and therefore have a different stiffness and deflection when compared to a solid concrete wall. Assumptions for hand calculations will have to be reevaluated, so that shear values for both sets of calculations close to acceptable tolerances.

Appendix

Wind Load Calculation

MICHAEL HOPPLE
TECH ASSIGNMENT 1

WIND LOAD

MAIN WIND-FORCE RESISTING SYSTEM: METHOD 2
(ASCE 7-05, SECTION 6.5)

1. BASIC WIND SPEED $V = 90$ mph (FIGURE 6-1)

2. IMPORTANCE FACTOR $I = 1.15$ (TABLE 6-1)

OCCUPANCY CATEGORY IV (HEALTH CARE FACILITY HAVING SURGERY OR EMERGENCY TREATMENT)
(TABLE 1-1)

3. EXPOSURE CATEGORY B (SECTION 6.5.6.3)

4. TOPOGRAPHIC FACTOR $K_{zt} = 1.0$

5. GUST EFFECT FACTOR $G = 0.803$ (EAST-WEST), $G = 0.814$ (NORTH-SOUTH)

(ASSUMES RIGID STRUCTURE)

$$G = 0.925\left(\frac{1+1.7g_Q I_{\bar z} Q}{1+1.7g_v I_{\bar z}}\right) = 0.925\left(\frac{1+1.7(3.4)(0.299)(0.791)}{1+1.7(3.4)(0.299)}\right) = 0.803\ (E\text{-}W) = 0.814\ (N\text{-}S)$$

$$I_{\bar z} = c\left(\frac{33}{\bar z}\right)^{1/6} = 0.3\left(\frac{33}{33.6}\right)^{1/6} = 0.299$$

$\bar z = 0.6h = 0.6(56') = 33.6' > z_{min} = 30'$

$c = 0.30$ (TABLE 6-2)

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar z}}\right)^{0.63}}} = \sqrt{\frac{1}{1+0.63\left(\frac{242+56}{321.9}\right)^{0.63}}} = 0.791\ (E\text{-}W) = 0.810\ (N\text{-}S)$$

$B = 242'$, $h = 56'$

$$L_{\bar z} = \ell\left(\frac{\bar z}{33}\right)^{\bar\epsilon} = 320\left(\frac{33.6}{33}\right)^{1/3} = 321.9$$

NOTE: SUBSTITUDE $B = 184$ FOR NORTH-SOUTH DIRECTION

MICHAEL HOPPLE
TECH ASSIGNMENT 1

WIND LOAD (CONT.)

6. ENCLOSURE CLASSIFICATION
 • ENCLOSED BUILDING (SECTION 6.2)

7. INTERNAL PRESSURE COEFFICIENT $GC_{pi} = \pm 0.18$ (FIGURE 6-5)

8. EXTERNAL PRESSURE COEFFICIENTS
 FOR MWFRS
 C_p: WINDWARD WALL → $C_p = 0.8$
 LEEWARD WALL → $C_p = -0.5$
 SIDE WALL → $C_p = -0.7$

9. VELOCITY PRESSURE q_z

$q_z = 0.00256 K_z \cdot K_{zt} \cdot K_d \cdot V^2 I$ (6-15); K_z FROM 6.5.6.6, $K_d = 0.85$ (6.5.4.4)
$K_{zt} = 1.0$, $V = 90$, $I = 1.15$

HEIGHT	K_z	q_z
0-30'	0.70	14.19
30'-40'	0.76	15.40
40'-50'	0.81	16.42
50'-60'	0.85	17.23
60'-70'	0.89	18.04

MICHAEL HOPPLE
TECH ASSIGNMENT 1

WIND LOAD (CONT.)

10. DESIGN WIND LOAD

$$P = q_h\left[G \cdot C_{pf} - G \cdot C_{pi}\right] \quad (6\text{-}18) \; ; \; q_h = q_z \, , \; G = 0.803 \, (E\text{-}W)$$

$G = 0.814$ (N-S)

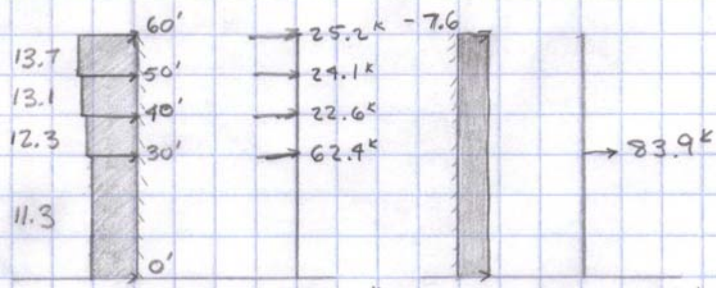
$C_{pf} = 0.8$ WINDWARD

-0.5 LEEWARD

$C_{pi} = \pm 0.18$

FOR N-S WINWARD

HEIGHT	P (psf)
0-30'	11.3
30-40'	12.3
40-50'	13.1
50-60'	13.7
60-70'	14.4

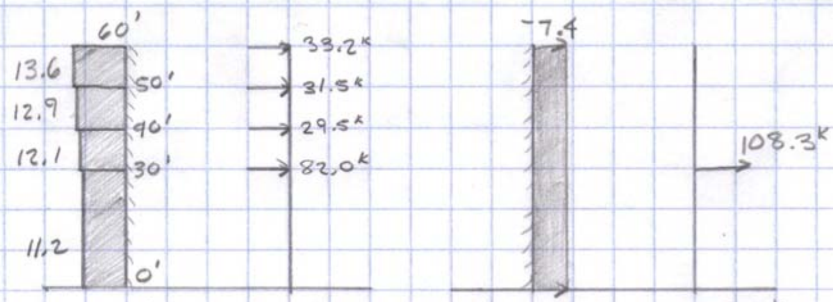


WINDWARD FORCE = 134.3k LEEWARD FORCE = 83.9k

TOTAL FORCE = 134.3+83.9 = 218.2k (N-S)

FOR E-W WINDWARD

HEIGHT	P (psf)
0-30'	11.2
30-40'	12.1
40-50'	12.9
50-60'	13.6
60-70'	14.2



WINDWARD FORCE = 176.2 LEEWARD FORCE = 108.3k

TOTAL FORCE = 176.2+108.3 = 284.5k (E-W)

Seismic Load Calculation

MICHAEL HOPPLE
TECH ASSIGNMENT 1

SEISMIC BASE SHEAR (EQUIVALENT LATERAL FORCE METHOD)

TOTAL DEAD LOAD OF STRUCTURE

Superimposed Dead Load:	1771	Kips
Partition Load:	1770	Kips
Double Tee Weight:	5561	Kips
Topping Weight:	5904	Kips
Inverted Tee Girder Weight:	730	Kips
L-Beam Weight:	143	Kips
Column Weight:	615	Kips
Exterior Wall Panel Weight:	4142	Kips
Inertior Wall Panel Weight:	517	Kips
Total Structure Weight:	21153	Kips

TO DETERMINE C_S:

1. SPECTRAL RESPONSE ACCELERATION:
   $S_S = 0.256 \quad S_1 = 0.075$

2. SITE CLASSIFICATION:
   SITE CLASS C

3. RESPONSE ACCELERATIONS:
   $S_{MS} = F_a S_S \quad S_{M1} = F_v S_1$
   $S_{MS} = 1.2(0.256) \quad S_{M1} = 1.7(0.075)$
   $S_{MS} = 0.307 \quad S_{M1} = 0.128$

4. 5%-DAMPED DESIGN SPECTRAL RESPONSE ACCELERATIONS:
   $S_{DS} = 2/3\, S_{MS} \quad S_{D1} = 2/3\, S_{M1}$
   $S_{DS} = 2/3(0.307) \quad S_{D1} = 2/3(0.128)$
   $S_{DS} = 0.205 \quad S_{D1} = 0.085$

SEISMIC BASE SHEAR (CONT.)

5. APPROXIMATE FUNDAMENTAL PERIOD:

$T_a = C_t \cdot h_n^x$ $\quad C_t = 0.02$

$T_a = 0.02(56)^{(0.75)}$ $\quad h_n = 56'$

$T_a = 0.409s$ $\quad x = 0.75$

6. DETERMINING C_s FROM LESSER OF TWO EQN'S:

$C_s = \frac{S_{DS}}{(R/I)}$ OR $C_s = \frac{S_{D1}}{(R/I)T}$ $\quad R = 4.0$, $I = 1.5$

$C_s = \frac{0.205}{(4/1.5)}$ $\quad C_s = \frac{0.085}{(4/1.5)(0.409)}$

$\boxed{C_s = 0.0769}$ $\quad C_s = 0.0779$

BUT C_s CAN NOT BE LESS THAN

$C_s = 0.044 S_{DS} I$

$C_s = 0.044(0.205)(1.5)$

$C_s = 0.0135 < 0.0769 \therefore$ O.K.

DESIGN BASE SHEAR:

$V = C_s \cdot W$

$V = 0.0769(21153^k)$

$\boxed{V = 1627^k}$

Story Forces

MICHAEL HOPPLE
TECH ASSIGNMENT 1

SEISMIC LOAD DISTRIBUTION

LATERAL FORCE AT EACH LEVEL:

$$F_x = C_{vx} \cdot V \quad \text{WHERE} \quad C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} \qquad k = 1.0 \text{ FOR } T < 0.5s$$

DEAD LOAD BY LEVEL:
LEVEL 2 = 6213^k
LEVEL 3 = 5041^k
LEVEL 4 = 5689^k
ROOF = 4210^k

$$\sum_{i=1}^{n} w_i h_i^k = 6213(14.5) + 5041(28.5) + 5689(42.5) + 4210(56) = 711300$$

LEVEL 2 $C_{v2} = \frac{6213(14.5)}{711300} = 0.127$

$F_2 = 0.127(1627) = 206^k$

LEVEL 3 $C_{v3} = \frac{5041(28.5)}{711300} = 0.202$

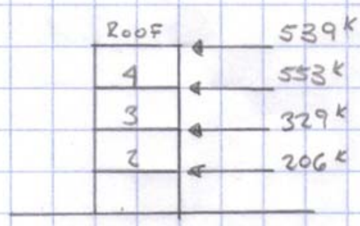
$F_3 = 0.202(1627) = 329^k$

LEVEL 4 $C_{v4} = \frac{5689(42.5)}{711300} = 0.340$

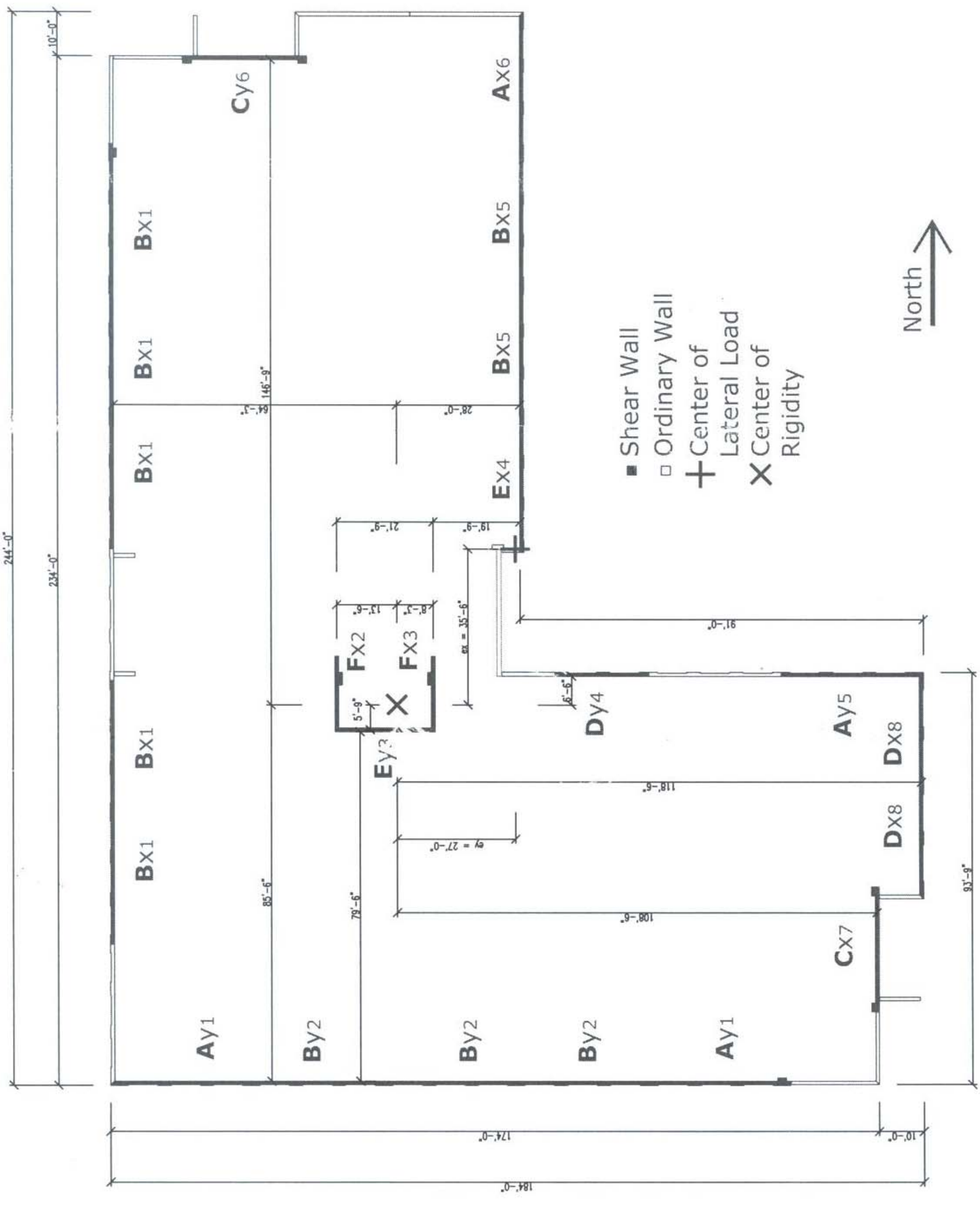
$F_4 = 0.34(1627) = 553^k$

ROOF LEVEL $C_{vR} = \frac{4210(56)}{711300} = 0.331$

$F_R = 0.331(1627) = 539^k$



Shear Wall Layout



Equivalent Thickness

East - West Walls					North - South Walls			

East - West Walls

Shear Wall: **Ay**
Panel Length: W = 32 ft.

Tier	Height (ft.)	Volume ($yd.^3$)
Parapet	4.0	3.31
Roof	14.0	12.29
4	14.0	11.90
3	14.0	10.13
2	14.0	10.60

Total Volume	48.23	
	1302	$ft.^3$
Total Wall Area	1920	$ft.^2$
Equivalent Thickness	0.678	ft.
	8.139	in.

Shear Wall: **By**
Panel Length: W = 30 ft.

Tier	Height (ft.)	Volume ($yd.^3$)
Parapet	4.0	3.12
Roof	14.0	11.09
4	14.0	11.10
3	14.0	9.36
2	14.0	9.79

Total Volume	44.46	
	1200	$ft.^3$
Total Wall Area	1800	$ft.^2$
Equivalent Thickness	0.667	ft.
	8.003	in.

Shear Wall: **Cy**
Panel Length: W = 28.33 ft.

Tier	Height (ft.)	Volume ($yd.^3$)
Parapet	4.0	3.67
Roof	14.0	14.00
4	14.0	14.00
3	14.0	14.00
2	14.0	14.60

Total Volume	60.27	
	1627	$ft.^3$
Total Wall Area	1700	$ft.^2$
Equivalent Thickness	0.957	ft.
	11.49	in.

North - South Walls

Shear Wall: **Ax**
Panel Length: W = 32 ft.

Tier	Height (ft.)	Volume ($yd.^3$)
Parapet	4.0	3.31
Roof	14.0	12.29
4	14.0	11.90
3	14.0	10.13
2	14.0	10.60

Total Volume	48.23	
	1302	$ft.^3$
Total Wall Area	1920	$ft.^2$
Equivalent Thickness	0.678	ft.
	8.139	in.

Shear Wall: **Bx**
Panel Length: W = 30 ft.

Tier	Height (ft.)	Volume ($yd.^3$)
Parapet	4.0	3.12
Roof	14.0	11.09
4	14.0	11.10
3	14.0	9.36
2	14.0	9.79

Total Volume	44.46	
	1200	$ft.^3$
Total Wall Area	1800	$ft.^2$
Equivalent Thickness	0.667	ft.
	8.003	in.

Shear Wall: **Cx**
Panel Length: W = 28.33 ft.

Tier	Height (ft.)	Volume ($yd.^3$)
Parapet	4.0	3.67
Roof	14.0	14.00
4	14.0	14.00
3	14.0	14.00
2	14.0	14.60

Total Volume	60.27	
	1627	$ft.^3$
Total Wall Area	1700	$ft.^2$
Equivalent Thickness	0.957	ft.
	11.49	in.

Shear Wall: **Dy**
Panel Length: W = 21 ft.

Tier	Height (ft.)	Volume ($yd.^3$)
Parapet	4.0	2.22
Roof	14.0	8.14
4	14.0	8.01
3	14.0	6.80
2	14.0	7.39

Total Volume	32.56	
	879.1	$ft.^3$
Total Wall Area	1260	$ft.^2$
Equivalent Thickness	0.698	ft.
	8.373	in.

Shear Wall: **Dx**
Panel Length: W = 25.67 ft.

Tier	Height (ft.)	Volume ($yd.^3$)
Parapet	4.0	2.69
Roof	14.0	9.18
4	14.0	8.94
3	14.0	7.50
2	14.0	6.85

Total Volume	35.16	
	949.3	$ft.^3$
Total Wall Area	1540	$ft.^2$
Equivalent Thickness	0.616	ft.
	7.396	in.

Shear Wall: **Ey**
Panel Length: W = 20.75 ft.

Tier	Height (ft.)	Volume ($yd.^3$)
Parapet	13.0	8.28
Roof	14.0	9.38
4	14.0	9.38
3	14.0	9.38
2	14.0	9.78

Total Volume	46.20	
	1247	$ft.^3$
Total Wall Area	1432	$ft.^2$
Equivalent Thickness	0.871	ft.
	10.45	in.

Shear Wall: **Ex**
Panel Length: W = 30.75 ft.

Tier	Height (ft.)	Volume ($yd.^3$)
Parapet	4.0	3.22
Roof	14.0	11.09
4	14.0	11.51
3	14.0	9.73
2	14.0	10.16

Total Volume	45.71	
	1234	$ft.^3$
Total Wall Area	1845	$ft.^2$
Equivalent Thickness	0.669	ft.
	8.027	in.

Shear Wall: **Fx**
Panel Length: W = 16 ft.

Tier	Height (ft.)	Volume ($yd.^3$)
Parapet	13.0	3.45
Roof	14.0	8.53
4	14.0	7.78
3	14.0	7.78
2	14.0	7.85

Total Volume	35.39	
	955.5	$ft.^3$
Total Wall Area	1104	$ft.^2$
Equivalent Thickness	0.866	ft.
	10.39	in.

Shear Wall Stiffness

North - South Walls

Shear Wall	# of Walls	Length (in.)	Equivalent Thickness (in.)	Height (in.)	Moment of Inertia (in.^{4})	Modulous of Elasticity (ksi)	Stiffness (k/in.)
		h	b	z	I	E	k
Ax	1	384	8.136	672	38390465	4286	6507
Bx	7	360	8.004	672	31119552	4286	5274
Cx	1	340	11.496	672	37653232	4286	6382
Dx	2	308	7.392	672	17998357	4286	3050
Ex	1	308	8.028	672	19546917	4286	3313
Fx	2	192	10.392	828	6129451	4286	555
						Total =	60332

East - West Walls

Shear Wall	# of Walls	Length (in.)	Equivalent Thickness (in.)	Height (in.)	Moment of Inertia (in.^{4})	Modulous of Elasticity (ksi)	Stiffness (k/in.)
		h	b	z	I	E	k
Ay	3	384	8.136	672	38390465	4286	6507
By	3	360	8.004	672	31119552	4286	5274
Cy	1	340	11.496	672	37653232	4286	6382
Dy	1	252	8.376	672	11170100	4286	1893
Ey	1	249	10.452	828	13446715	4286	1218
						Total =	44835

Lateral Load Distribution

North - South Walls

Shear Wall: **A_x**
Rigidity: 6507
Total Rigidity: 60332

Level	Seismic Base Shear	Shear Distribution for Each Level
R	539	58.1
4	553	59.6
3	329	35.5
2	206	22.2
	Total	175.5

Shear Wall: **B_x**
Rigidity: 5274
Total Rigidity: 60332

Level	Seismic Base Shear	Shear Distribution for Each Level
R	539	47.1
4	553	48.3
3	329	28.8
2	206	18.0
	Total	142.2

Shear Wall: **C_x**
Rigidity: 6382
Total Rigidity: 60332

Level	Seismic Base Shear	Shear Distribution for Each Level
R	539	57.0
4	553	58.5
3	329	34.8
2	206	21.8
	Total	172.1

East - West Walls

Shear Wall: **A_y**
Rigidity: 6507
Total Rigidity: 44835.0

Level	Seismic Base Shear	Shear Distribution for Each Level
R	539	78.2
4	553	80.3
3	329	47.7
2	206	29.9
	Total	236.1

Shear Wall: **B_y**
Rigidity: 5274
Total Rigidity: 44835.0

Level	Seismic Base Shear	Shear Distribution for Each Level
R	539	63.4
4	553	65.1
3	329	38.7
2	206	24.2
	Total	191.4

Shear Wall: **C_y**
Rigidity: 6382
Total Rigidity: 44835.0

Level	Seismic Base Shear	Shear Distribution for Each Level
R	539	76.7
4	553	78.7
3	329	46.8
2	206	29.3
	Total	231.6

Shear Wall: **D_x**
Total Rigidity: 60332

Level	Seismic Base Shear	Shear Distribution for Each Level
R	539	27.2
4	553	28.0
3	329	16.6
2	206	10.4
	Total	82.3

Shear Wall: **D_y**
Total Rigidity: 44835.0

Level	Seismic Base Shear	Shear Distribution for Each Level
R	539	22.8
4	553	23.3
3	329	13.9
2	206	8.7
	Total	68.7

Shear Wall: **E_x**
Rigidity: 3313
Total Rigidity: 60332

Level	Seismic Base Shear	Shear Distribution for Each Level
R	539	29.6
4	553	30.4
3	329	18.1
2	206	11.3
	Total	89.3

Shear Wall: **E_y**
Rigidity: 1218
Total Rigidity: 44835.0

Level	Seismic Base Shear	Shear Distribution for Each Level
R	539	14.6
4	553	15.0
3	329	8.9
2	206	5.6
	Total	44.2

Shear Wall: **F_x**
Rigidity: 555
Total Rigidity: 60332

Level	Seismic Base Shear	Shear Distribution for Each Level
R	539	5.0
4	553	5.1
3	329	3.0
2	206	1.9
	Total	15.0

Center of Rigidity

North - South Walls

Shear Wall	# of Walls	Distance (in.)	Stiffness (k/in.)	Σk*d
		d	k	
A_{x6}	1	1086	9507	10324602
B_{x5}	2	1086	5274	11455128
B_{x1}	5	2208	5274	58224960
C_{x7}	1	120	6382	765840
D_{x8}	2	0	3050	0
Ex_4	1	1086	3313	3597918
F_{x2}	1	1590	555	882450
F_{x3}	1	1329	555	737595
		Total =	60332	85988493

Center of Rigidity	Y =	1425.3	in.
	Y =	118.8	ft.

East - West Walls

Shear Wall	# of Walls	Distance (in.)	Stiffness (k/in.)	Σk*d
		d	k	
A_{y1}	2	0	6507	0
B_{y2}	3	0	5274	0
E_{y3}	1	954	1218	1161972
D_{y4}	1	1125	1893	2129625
A_{y5}	1	1125	6507	7320375
C_{y6}	1	2808	6382	17920656
		Total =	27781	28532628

Center of Rigidity	X =	1027.1	in.
	X =	85.6	ft.

Torsional Moment of Inertia

North - South Walls

Shear Wall	# of Walls	Stiffness (kip/in.) k	Dist. From Wall to C.O.R. (ft.) d	d^2	$\Sigma k*d^2$
A_{x6}	1	9507	28	784.0	7453488
B_{x5}	2	5274	28	784.0	8269632
B_{x1}	5	5274	64.25	4128.1	108857008
C_{x7}	1	6382	108.5	11772.3	75130500
D_{x8}	2	3050	118.5	14042.3	85657725
Ex_4	1	3313	28	784.0	2597392
F_{x2}	1	555	13.5	182.3	101149
F_{x3}	1	555	8.25	68.1	37775
	Torsional Moment of Inertia (kip-ft.2/in.):			J =	288104668

East - Wast Walls

Shear Wall	# of Walls	Stiffness (kip/in.) k	Dist. From Wall to C.O.R. (ft.) d	d^2	$\Sigma k*d^2$
A_{y1}	2	6507	85.6	7327.4	95358263
B_{y2}	3	5274	85.6	7327.4	115933490
E_{y3}	1	1218	5.75	33.1	40270
D_{y4}	1	1893	6.5	42.3	79979
A_{y5}	1	6507	6.5	42.3	274921
C_{y6}	1	6382	146.75	21535.6	137439960
	Torsional Moment of Inertia (kip-ft.2/in.):			J =	349126883

Torsional Shear

North - South Walls

Shear Wall:	**Bx_1**	
Eccentricity:	27	ft.
Dist. To C.O.R.	64.3	ft.
Stiffness:	5274	(kip/in.)
Torsional Moment of Inertia	349126883	(kip-ft.^{2}/in.)
Level	Seismic Base Shear (kips)	Torsional Shear for Each Level (kips)
R	539	14.1
4	553	14.5
3	329	8.6
2	206	5.4

Shear Wall:	**F_{x2}**	
Eccentricity:	27	ft.
Dist. To C.O.R.	13.5	ft.
Stiffness:	555	(kip/in.)
Torsional Moment of Inertia	349126883	(kip-ft.^{2}/in.)
Level	Seismic Base Shear (kips)	Torsional Shear for Each Level (kips)
R	539	0.3
4	553	0.3
3	329	0.2
2	206	0.1

Shear Wall:	**F_{x3}**	
Eccentricity:	27	ft.
Dist. To C.O.R.	8.3	ft.
Stiffness:	555	(kip/in.)
Torsional Moment of Inertia	349126883	(kip-ft.^{2}/in.)
Level	Seismic Base Shear (kips)	Torsional Shear for Each Level (kips)
R	539	0.2
4	553	0.2
3	329	0.1
2	206	0.1

East - West Walls

Shear Wall:	**A_{y1}**	
Eccentricity:	35.5	ft.
Dist. To C.O.R.	85.5	ft.
Stiffness:	6507	(kip/in.)
Torsional Moment of Inertia	349126883	(kip-ft.^{2}/in.)
Level	Seismic Base Shear (kips)	Torsional Shear for Each Level (kips)
R	539	30.5
4	553	31.3
3	329	18.6
2	206	11.7

Shear Wall:	**B_{y2}**	
Eccentricity:	35.5	ft.
Dist. To C.O.R.	85.5	ft.
Stiffness:	5274	(kip/in.)
Torsional Moment of Inertia	349126883	(kip-ft.^{2}/in.)
Level	Seismic Base Shear (kips)	Torsional Shear for Each Level (kips)
R	539	24.7
4	553	25.4
3	329	15.1
2	206	9.4

Shear Wall:	**E_{y3}**	
Eccentricity:	35.5	ft.
Dist. To C.O.R.	5.8	ft.
Stiffness:	1218	(kip/in.)
Torsional Moment of Inertia	349126883	(kip-ft.^{2}/in.)
Level	Seismic Base Shear (kips)	Torsional Shear for Each Level (kips)
R	539	0.4
4	553	0.4
3	329	0.2
2	206	0.1

Shear Wall:	**E_{x4}**	
Dist. To C.O.R.	28.0	ft.
Stiffness:	3313	(kip/in.)
Torsional Moment of Inertia	349126883	(kip-ft.^{2}/in.)

Level	Seismic Base Shear (kips)	Torsional Shear for Each Level (kips)
R	539	3.9
4	553	4.0
3	329	2.4
2	206	1.5

Shear Wall:	**B_{x5}**	
Eccentricity:	27	ft.
Dist. To C.O.R.	28.0	ft.
Stiffness:	5274	(kip/in.)
Torsional Moment of Inertia	349126883	(kip-ft.^{2}/in.)

Level	Seismic Base Shear (kips)	Torsional Shear for Each Level (kips)
R	539	6.2
4	553	6.3
3	329	3.8
2	206	2.4

Shear Wall:	**A_{x6}**	
Eccentricity:	27	ft.
Dist. To C.O.R.	28.0	ft.
Stiffness:	6507	(kip/in.)
Torsional Moment of Inertia	349126883	(kip-ft.^{2}/in.)

Level	Seismic Base Shear (kips)	Torsional Shear for Each Level (kips)
R	539	7.6
4	553	7.8
3	329	4.6
2	206	2.9

Shear Wall:	**D_{y4}**	ft.
Dist. To C.O.R.	6.5	ft.
Stiffness:	1893	(kip/in.)
Torsional Moment of Inertia	349126883	(kip-ft.^{2}/in.)

Level	Seismic Base Shear (kips)	Torsional Shear for Each Level (kips)
R	539	0.7
4	553	0.7
3	329	0.4
2	206	0.3

Shear Wall:	**A_{y5}**	
Eccentricity:	35.5	ft.
Dist. To C.O.R.	6.5	ft.
Stiffness:	6507	(kip/in.)
Torsional Moment of Inertia	349126883	(kip-ft.^{2}/in.)

Level	Seismic Base Shear (kips)	Torsional Shear for Each Level (kips)
R	539	2.3
4	553	2.4
3	329	1.4
2	206	0.9

Shear Wall:	**C_{y6}**	
Eccentricity:	35.5	ft.
Dist. To C.O.R.	146.8	ft.
Stiffness:	6382	(kip/in.)
Torsional Moment of Inertia	349126883	(kip-ft.^{2}/in.)

Level	Seismic Base Shear (kips)	Torsional Shear for Each Level (kips)
R	539	51.3
4	553	52.7
3	329	31.3
2	206	19.6

Shear Wall:	**C_{x7}**	
Eccentricity:	27	ft.
Dist. To C.O.R.	108.5	ft.
Stiffness:	6382	(kip/in.)
Torsional Moment of Inertia	349126883	(kip-ft.^{2}/in.)
Level	Seismic Base Shear (kips)	Torsional Shear for Each Level (kips)
R	539	28.9
4	553	29.6
3	329	17.6
2	206	11.0

Shear Wall:	**D_{x8}**	
Eccentricity:	27	ft.
Dist. To C.O.R.	118.5	ft.
Stiffness:	3050	(kip/in.)
Torsional Moment of Inertia	349126883	(kip-ft.^{2}/in.)
Level	Seismic Base Shear (kips)	Torsional Shear for Each Level (kips)
R	539	15.1
4	553	15.5
3	329	9.2
2	206	5.8

Shear Wall Schedule

North - South Walls

Shear Wall: **B_{x1}**

Level	Direct Shear	Torsional Shear	Total Shear
Roof	47.1	14.1	61.2
4th floor	48.3	14.5	62.8
3rd floor	28.8	8.6	37.4
2nd floor	18.8	5.4	24.2
Total	143.0	42.6	185.6

Shear Wall: **F_{x2}**

Level	Direct Shear	Torsional Shear	Total Shear
Roof	5.0	0.3	5.3
4th floor	5.1	0.3	5.4
3rd floor	3.0	0.2	3.2
2nd floor	1.9	0.1	2.0
Total	15.0	0.9	15.9

Shear Wall: **F_{x3}**

Level	Direct Shear	Torsional Shear	Total Shear
Roof	5.0	0.2	5.2
4th floor	5.1	0.2	5.3
3rd floor	3.0	0.1	3.1
2nd floor	1.9	0.1	2.0
Total	15.0	0.6	15.6

Shear Wall: **E_{x4}**

Level	Direct Shear	Torsional Shear	Total Shear
Roof	29.6	3.9	33.5
4th floor	30.4	4.0	34.4
3rd floor	18.1	2.4	20.5
2nd floor	11.3	1.5	12.8
Total	89.4	11.8	101.2

East - West Walls

Shear Wall: **A_{y1}**

Level	Direct Shear	Torsional Shear	Total Shear
Roof	78.0	30.5	108.5
4th floor	80.3	31.3	111.6
3rd floor	47.7	18.6	66.3
2nd floor	29.9	11.7	41.6
Total	235.9	92.1	328.0

Shear Wall: **B_{y2}**

Level	Direct Shear	Torsional Shear	Total Shear
Roof	63.4	24.7	88.1
4th floor	65.1	25.4	90.5
3rd floor	38.7	15.1	53.8
2nd floor	24.2	9.4	33.6
Total	191.4	74.6	266.0

Shear Wall: **E_{y3}**

Level	Direct Shear	Torsional Shear	Total Shear
Roof	14.6	0.4	15.0
4th floor	15.0	0.4	15.4
3rd floor	8.9	0.2	9.1
2nd floor	5.6	0.1	5.7
Total	44.1	1.1	45.2

Shear Wall: **D_{y4}**

Level	Direct Shear	Torsional Shear	Total Shear
Roof	22.8	0.7	23.5
4th floor	23.3	0.7	24.0
3rd floor	13.9	0.4	14.3
2nd floor	8.7	0.3	9.0
Total	68.7	2.1	70.8

Shear Wall:	B_{x5}		
Level	Direct Shear	Torsional Shear	Total Shear
Roof	47.1	6.2	53.3
4th floor	48.3	6.3	54.6
3rd floor	28.8	3.8	32.6
2nd floor	18.8	2.4	21.2
Total	143.0	18.7	161.7

Shear Wall:	A_{y5}		
Level	Direct Shear	Torsional Shear	Total Shear
Roof	78.0	2.3	80.3
4th floor	80.3	2.4	82.7
3rd floor	47.7	1.4	49.1
2nd floor	29.9	0.9	30.8
Total	235.9	7.0	242.9

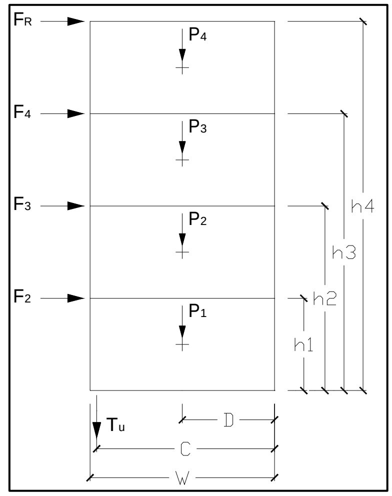
Shear Wall:	A_{x6}		
Level	Direct Shear	Torsional Shear	Total Shear
Roof	58.1	7.6	65.7
4th floor	59.6	7.8	67.4
3rd floor	35.5	4.6	40.1
2nd floor	22.2	2.9	25.1
Total	175.4	22.9	198.3

Shear Wall:	C_{y6}		
Level	Direct Shear	Torsional Shear	Total Shear
Roof	76.7	51.3	128.0
4th floor	78.7	52.7	131.4
3rd floor	46.8	31.3	78.1
2nd floor	29.3	19.6	48.9
Total	231.5	154.9	386.4

Shear Wall:	C_{x7}		
Level	Direct Shear	Torsional Shear	Total Shear
Roof	57.0	28.9	85.9
4th floor	58.5	29.6	88.1
3rd floor	34.8	17.6	52.4
2nd floor	21.8	11.0	32.8
Total	172.1	87.1	259.2

Shear Wall:	D_{x8}		
Level	Direct Shear	Torsional Shear	Total Shear
Roof	27.2	15.1	42.3
4th floor	28.0	15.5	43.5
3rd floor	16.6	9.2	25.8
2nd floor	10.4	5.8	16.2
Total	82.2	45.6	127.8

Shear Wall Overturning



$M_o = 1.0*\Sigma(F*h)$

$M_r = 0.859*\Sigma(P*D)$

$T_u = (M_o - M_r) / C$

$A_s = T_u / 60$

Shear Wall: **Bx1**

Panel Width:	W =	30	ft.	
Panel Thickness:	t =	8.003	in.	
Moment Arm:	D =	15	ft.	D = W / 2
Coupled Moment Arm:	C =	28	ft.	C = W - 2'

Tier	Height h (ft.)	Seismic F (kips)	Dead P (kips)	Overturning Mo (ft-k)	Resisting Mr (ft-k)	Uplift Tu (kips)	Required Steel As,req (in.^{2})	Number of # 9 Bars	Provided Steel As,provd (in.^{2})
Roof	56.0	61.2	117.5	3427.2	1514.0	68.3	1.14	2	2.00
4	42.0	62.8	117.5	6064.8	3028.0	108.5	1.81	2	2.00
3	28.0	37.4	117.5	7112.0	4542.0	91.8	1.53	2	2.00
2	14.0	24.2	117.5	7450.8	6056.0	49.8	0.83	1	1.00

Shear Wall: **Fx2**

Panel Width:	W =	16	ft.	
Panel Thickness:	t =	10.39	in.	
Moment Arm:	D =	8	ft.	D = W / 2
Coupled Moment Arm:	C =	14	ft.	C = W - 2'

Tier	Height h (ft.)	Seismic F (kips)	Dead P (kips)	Overturning Mo (ft-k)	Resisting Mr (ft-k)	Uplift Tu (kips)	Required Steel As,req (in.^{2})	Number of # 9 Bars	Provided Steel As,provd (in.^{2})
Roof	56.0	5.3	63.4	296.8	435.7	-9.9	N/A	N/A	N/A
4	42.0	5.4	63.4	523.6	871.4	-24.8	N/A	N/A	N/A
3	28.0	3.2	63.4	613.2	1307.1	-49.6	N/A	N/A	N/A
2	14.0	2.0	63.4	641.2	1742.7	-78.7	N/A	N/A	N/A

Shear Wall: **Fx3**

Panel Width:	W =	16	ft.	
Panel Thickness:	t =	10.39	in.	
Moment Arm:	D =	8	ft.	D = W / 2
Coupled Moment Arm:	C =	14	ft.	C = W - 2'

Tier	Height h (ft.)	Seismic F (kips)	Dead P (kips)	Overturning Mo (ft-k)	Resisting Mr (ft-k)	Uplift Tu (kips)	Required Steel As,req (in.^{2})	Number of # 9 Bars	Provided Steel As,provd (in.^{2})
Roof	56.0	5.2	49.6	291.2	340.9	-3.5	N/A	N/A	N/A
4	42.0	5.3	49.6	513.8	681.7	-12.0	N/A	N/A	N/A
3	28.0	3.1	49.6	600.6	1022.6	-30.1	N/A	N/A	N/A

2	14.0	2.0	49.6	628.6	1363.4	-52.5	N/A	N/A	N/A

Shear Wall: **Ex4**
Panel Width: W = 30.75 ft.
Panel Thickness: t = 8.027 in.
Moment Arm: D = 15.375 ft. D = W / 2
Coupled Moment Arm: C = 28.75 ft. C = W - 2'

Tier	Height h (ft.)	Seismic F (kips)	Dead P (kips)	Overturning Mo (ft-k)	Resisting Mr (ft-k)	Uplift Tu (kips)	Required Steel As,req (in.^{2})	Number of # 9 Bars	Provided Steel As,provd (in.^{2})
Roof	56.0	33.5	131.2	1876.0	1732.8	5.0	0.08	1	1.00
4	42.0	34.4	131.2	3320.8	3465.5	-5.0	N/A	N/A	N/A
3	28.0	20.5	131.2	3894.8	5198.3	-45.3	N/A	N/A	N/A
2	14.0	12.8	131.2	4074.0	6931.1	-99.4	N/A	N/A	N/A

Shear Wall: **Bx5**
Panel Width: W = 30 ft.
Panel Thickness: t = 8.003 in.
Moment Arm: D = 15 ft. D = W / 2
Coupled Moment Arm: C = 28 ft. C = W - 2'

Tier	Height h (ft.)	Seismic F (kips)	Dead P (kips)	Overturning Mo (ft-k)	Resisting Mr (ft-k)	Uplift Tu (kips)	Required Steel As,req (in.^{2})	Number of # 9 Bars	Provided Steel As,provd (in.^{2})
Roof	56.0	53.3	127.8	2984.8	1646.7	47.8	0.80	1	1.00
4	42.0	54.6	127.8	5278.0	3293.4	70.9	1.18	2	2.00
3	28.0	32.6	127.8	6190.8	4940.1	44.7	0.74	1	1.00
2	14.0	21.2	127.8	6487.6	6586.8	-3.5	N/A	N/A	N/A

Shear Wall: **Ax6**
Panel Width: W = 32 ft.
Panel Thickness: t = 8.139 in.
Moment Arm: D = 16 ft. D = W / 2
Coupled Moment Arm: C = 30 ft. C = W - 2'

Tier	Height h (ft.)	Seismic F (kips)	Dead P (kips)	Overturning Mo (ft-k)	Resisting Mr (ft-k)	Uplift Tu (kips)	Required Steel As,req (in.^{2})	Number of # 9 Bars	Provided Steel As,provd (in.^{2})
Roof	56.0	65.7	28.7	3679.2	394.5	109.5	1.82	2	2.00
4	42.0	67.4	28.7	6510.0	788.9	190.7	3.18	4	4.00
3	28.0	40.1	28.7	7632.8	1183.4	215.0	3.58	4	4.00
2	14.0	25.1	28.7	7984.2	1577.8	213.5	3.56	4	4.00

Shear Wall: **Cx7**

Panel Width: W = 28.33 ft.

Panel Thickness: t = 11.49 in.

Moment Arm: D = 14.165 ft. D = W / 2

Coupled Moment Arm: C = 26.33 ft. C = W - 2'

Tier	Height h (ft.)	Seismic F (kips)	Dead P (kips)	Overturning Mo (ft-k)	Resisting Mr (ft-k)	Uplift Tu (kips)	Required Steel As,req (in.^{2})	Number of # 9 Bars	Provided Steel As,provd (in.^{2})
Roof	56.0	63.2	116.2	3539.2	1413.9	80.7	1.35	2	2.00
4	42.0	64.9	116.2	6265.0	2827.8	130.5	2.18	3	3.00
3	28.0	38.6	116.2	7345.8	4241.7	117.9	1.96	2	2.00
2	14.0	24.2	116.2	7684.6	5655.6	77.1	1.28	2	2.00

Shear Wall: **Dx8**

Panel Width: W = 25.67 ft.

Panel Thickness: t = 7.396 in.

Moment Arm: D = 12.835 ft. D = W / 2

Coupled Moment Arm: C = 23.67 ft. C = W - 2'

Tier	Height h (ft.)	Seismic F (kips)	Dead P (kips)	Overturning Mo (ft-k)	Resisting Mr (ft-k)	Uplift Tu (kips)	Required Steel As,req (in.^{2})	Number of # 9 Bars	Provided Steel As,provd (in.^{2})
Roof	56.0	42.3	0.0	2368.8	0.0	100.1	1.67	2	2.00
4	42.0	43.5	0.0	4195.8	0.0	177.3	2.95	3	3.00
3	28.0	25.8	0.0	4918.2	0.0	207.8	3.46	4	4.00
2	14.0	16.2	0.0	5145.0	0.0	217.4	3.62	4	4.00

Shear Wall: **Ay1**

Panel Width: W = 32 ft.

Panel Thickness: t = 8.139 in.

Moment Arm: D = 16 ft. D = W / 2

Coupled Moment Arm: C = 30 ft. C = W - 2'

Tier	Height h (ft.)	Seismic F (kips)	Dead P (kips)	Overturning Mo (ft-k)	Resisting Mr (ft-k)	Uplift Tu (kips)	Required Steel As,req (in.^{2})	Number of # 9 Bars	Provided Steel As,provd (in.^{2})
Roof	56.0	108.5	129.7	6076.0	1782.6	143.1	2.39	3	3.00
4	42.0	111.6	129.7	10763.2	3565.2	239.9	4.00	4	4.00
3	28.0	66.3	129.7	12619.6	5347.8	242.4	4.04	5	5.00
2	14.0	41.6	129.7	13202.0	7130.4	202.4	3.37	4	4.00

Shear Wall: **By2**
Panel Width: W = 30 ft.
Panel Thickness: t = 8.003 in.
Moment Arm: D = 15 ft. D = W / 2
Coupled Moment Arm: C = 28 ft. C = W - 2'

Tier	Height h (ft.)	Seismic F (kips)	Dead P (kips)	Overturning Mo (ft-k)	Resisting Mr (ft-k)	Uplift Tu (kips)	Required Steel As,req (in.^{2})	Number of # 9 Bars	Provided Steel As,provd (in.^{2})
Roof	56.0	88.1	120.9	4933.6	1557.8	120.6	2.01	3	3.00
4	42.0	90.5	120.9	8734.6	3115.6	200.7	3.34	4	4.00
3	28.0	53.8	120.9	10241.0	4673.4	198.8	3.31	4	4.00
2	14.0	33.6	120.9	10711.4	6231.2	160.0	2.67	3	3.00

Shear Wall: **Ey3**
Panel Width: W = 20.75 ft.
Panel Thickness: t = 10.45 in.
Moment Arm: D = 10.375 ft. D = W / 2
Coupled Moment Arm: C = 18.75 ft. C = W - 2'

Tier	Height h (ft.)	Seismic F (kips)	Dead P (kips)	Overturning Mo (ft-k)	Resisting Mr (ft-k)	Uplift Tu (kips)	Required Steel As,req (in.^{2})	Number of # 9 Bars	Provided Steel As,provd (in.^{2})
Roof	56.0	15.0	82.6	840.0	736.1	5.5	0.09	1	1.00
4	42.0	15.4	82.6	1486.8	1472.3	0.8	0.01	1	1.00
3	28.0	9.1	82.6	1741.6	2208.4	-24.9	N/A	N/A	N/A
2	14.0	5.7	82.6	1821.4	2944.6	-59.9	N/A	N/A	N/A

Shear Wall: **Dy4**
Panel Width: W = 21 ft.
Panel Thickness: t = 8.373 in.
Moment Arm: D = 10.5 ft. D = W / 2
Coupled Moment Arm: C = 19 ft. C = W - 2'

Tier	Height h (ft.)	Seismic F (kips)	Dead P (kips)	Overturning Mo (ft-k)	Resisting Mr (ft-k)	Uplift Tu (kips)	Required Steel As,req (in.^{2})	Number of # 9 Bars	Provided Steel As,provd (in.^{2})
Roof	56.0	23.5	91.0	1316.0	820.8	26.1	0.43	1	1.00
4	42.0	24.0	91.0	2324.0	1641.5	35.9	0.60	1	1.00
3	28.0	14.3	91.0	2724.4	2462.3	13.8	0.23	1	1.00
2	14.0	9.0	91.0	2850.4	3283.1	-22.8	N/A	N/A	N/A

Shear Wall: **Ay5**

Panel Width:	W =	32	ft.	
Panel Thickness:	t =	8.139	in.	
Moment Arm:	D =	16	ft.	D = W / 2
Coupled Moment Arm:	C =	30	ft.	C = W - 2'

Tier	Height h (ft.)	Seismic F (kips)	Dead P (kips)	Overturning Mo (ft-k)	Resisting Mr (ft-k)	Uplift Tu (kips)	Required Steel As,req (in.^{2})	Number of # 9 Bars	Provided Steel As,provd (in.^{2})
Roof	56.0	80.3	137.1	4496.8	1884.3	87.1	1.45	2	2.00
4	42.0	82.7	137.1	7970.2	3768.6	140.1	2.33	3	3.00
3	28.0	49.1	137.1	9345.0	5652.9	123.1	2.05	3	3.00
2	14.0	30.8	137.1	9776.2	7537.2	74.6	1.24	2	2.00

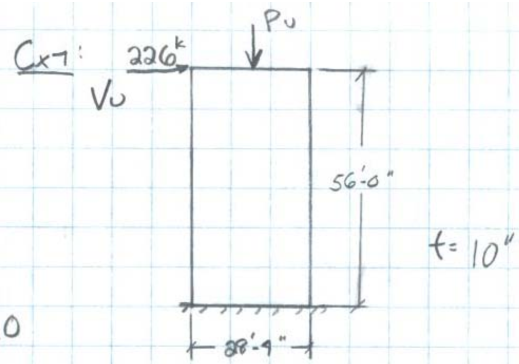
Shear Wall: **Cy6**

Panel Width:	W =	28.33	ft.	
Panel Thickness:	t =	11.49	in.	
Moment Arm:	D =	14.165	ft.	D = W / 2
Coupled Moment Arm:	C =	26.33	ft.	C = W - 2'

Tier	Height h (ft.)	Seismic F (kips)	Dead P (kips)	Overturning Mo (ft-k)	Resisting Mr (ft-k)	Uplift Tu (kips)	Required Steel As,req (in.^{2})	Number of # 9 Bars	Provided Steel As,provd (in.^{2})
Roof	56.0	128.0	121.7	7168.0	1480.8	216.0	3.60	4	4.00
4	42.0	131.4	121.7	12686.8	2961.6	369.4	6.16	7	7.00
3	28.0	78.1	121.7	14873.6	4442.4	396.2	6.60	7	7.00
2	14.0	48.9	121.7	15558.2	5923.3	365.9	6.10	7	7.00

Shear Wall Spot Check

SHEAR WALL SPOT CHECK:



SHEAR CAPACITY:

$$\phi V_n = \phi A_{cv}\left(\alpha\sqrt{f'_c} + \rho_t \cdot f_y\right)$$

$$\frac{h_w}{l_w} = \frac{56}{28.33} = 1.98 \cong 2.0 \quad \therefore \alpha = 2.0$$

$A_{cv} = (28.33' \times 12)(11.5'')$
$A_{cv} = 3910 \text{ in}^2$

$$\rho_t = \frac{1(0.20 \text{in}^2)}{10'' \times 12''} = 0.00167$$

#4's @ 1'-0" o.c.
$f'_c = 6000 \text{psi}$
$P_D = 116.2(4) = 464.8^k$
$P_L = 132(3) + 8.8 = 404.8^k$
$P_u = 1.2(464.8) + 1.6(404.8)$
$P_u = 1205^k$

$\phi V_n = 0.6(3910)\left(2\sqrt{6000} + 0.00167(60000)\right)$
$\phi V_n = 599^k$

$$\phi V_n = 599^k > V_u = 226^k$$

CHECK BOUNDARY ELEMENT:
NEEDED IF EXTREME FIBER = $0.2f'_c$ IN COMPRESSION

$$\text{STRESS} = f = \frac{P_u}{A} + \frac{M_u \cdot y}{I}$$

$M_u = P \cdot h_w = 226^k \cdot 56' = 12656'^k$
$y = 14.165' = 170''$
$I = \frac{bh^3}{12} = \frac{(10'')(340'')^3}{12} = 3.28 \times 10^7 \text{ in}^4$

$$f = \frac{1205}{3910} + \frac{12656(12)(170)}{3.28 \times 10^7}$$

$f = 0.308 \text{ksi} + 0.787 \text{ksi}$

$f = 1.095 \text{ ksi} = 1095 \text{psi} \qquad 0.2f'_c = 0.2(6000) = 1200 \text{psi}$

$f_c = 1095 \text{psi} < 0.2f'_c = 1200 \text{psi}$

$\therefore$ NO BOUNDARY ELEMENT NEEDED