North Mountain IMS Medical Office Building

Phoenix, Arizona

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Technical Assignment 1

October 5th, 2007

AE 481W-Senior Thesis

The Pennsylvania State University

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Table Of Contents

Executive Summary

Technical Assignment 1 is an existing conditions report of North Mountain IMS Medical Office Building. This report will explore the basic structural concepts that led to the design of this complete precast concrete building. Framing plans and lateral load-resisting systems are analyzed to better understand the design concepts of the overall structure. Calculations of seismic and wind loading are included in the Appendix.

North Mountain IMS Office Building is a 123,400 square feet precast concrete office building located in Phoenix, Arizona. This $10 million design-build project started construction in June of 2007 and is expected to be completed February 2008. It features a state-of-the-art outpatient diagnostic imaging center and ambulatory surgery center on the ground floor. The remaining floors feature over 92,000 square feet of open, rentable office space. The total building height is 60 feet, with a mechanical parapet wall that reaches 70 feet above ground level.

North Mountain's structural system consists exclusively of precast concrete products. Precast, pre-stressed concrete double tees provide the floor and roof. These double tees bear on exterior walls and interior pre-stressed inverted tee girders. The girders then bear on columns which carry gravity load down to the foundation. The columns are attached to circular concrete caissons, which are drilled to a max depth of 30 feet. The lateral load resisting system consists entirely of concrete shear walls.

A flexural spot-check calculation for a typical 48' span double tee was conducted to check the size of the double tee. A lateral analysis was also completed. Due to the weight of the structure and the higher seismic loads in the Phoenix region, earthquake loads exceed those of wind loads. Calculations of the spot-check and lateral loads are located in the Appendix.

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. PCI, 6th edition

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.
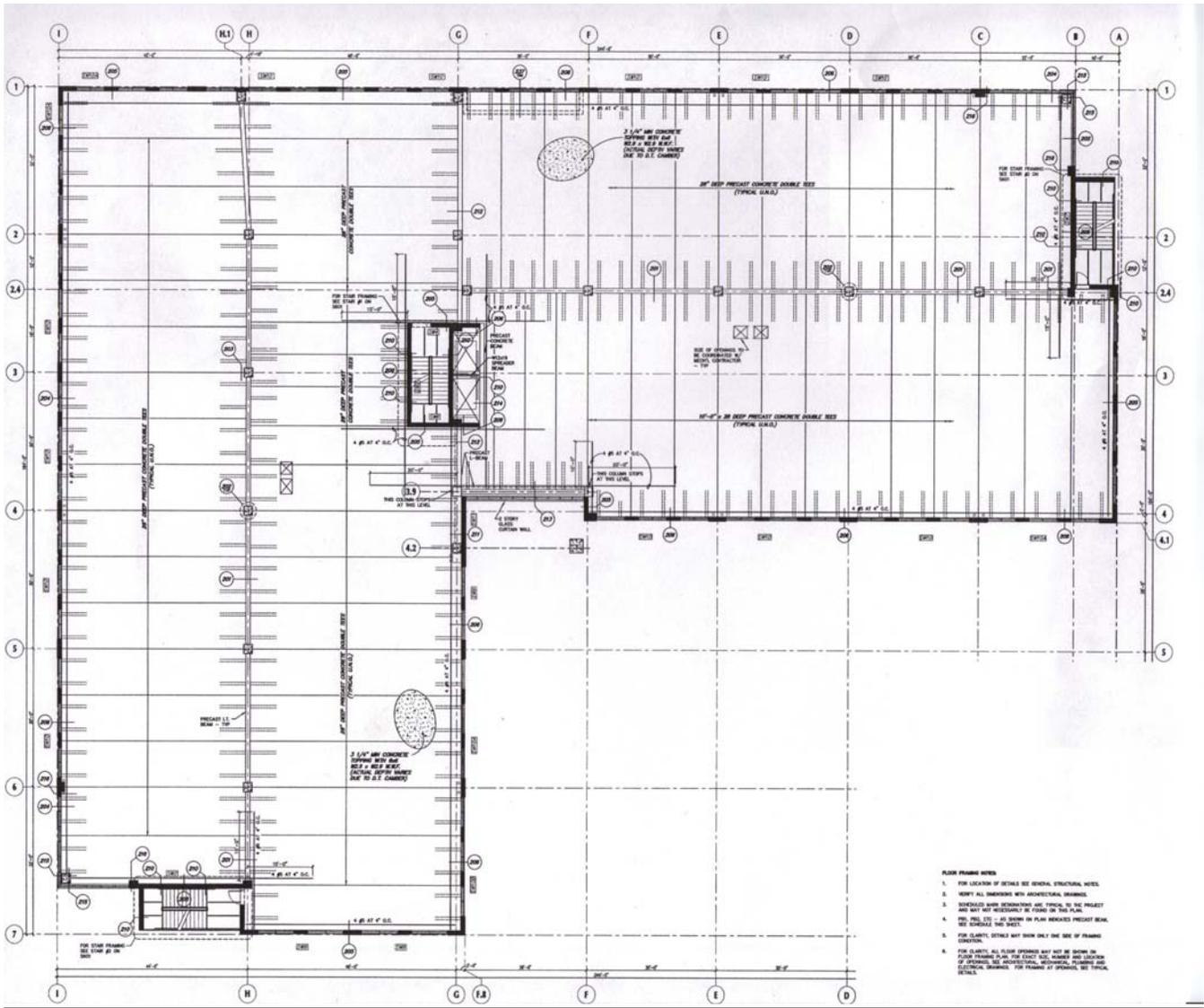
The 24" deep double tees bear on 24" deep by 32" wide inverted tee girders. 28-day strength is 7,500 psi and minimum release strength is 3,750 psi. Typical inverted tee girders use 22 ½" diameter stand for tensile reinforcement. Span length for a 30' bay is 28' due to the columns on each end. Dapped 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 are a great showcase of 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.

INTERIOR ELEVATION
GRID 2.4 (LOOKING WEST)
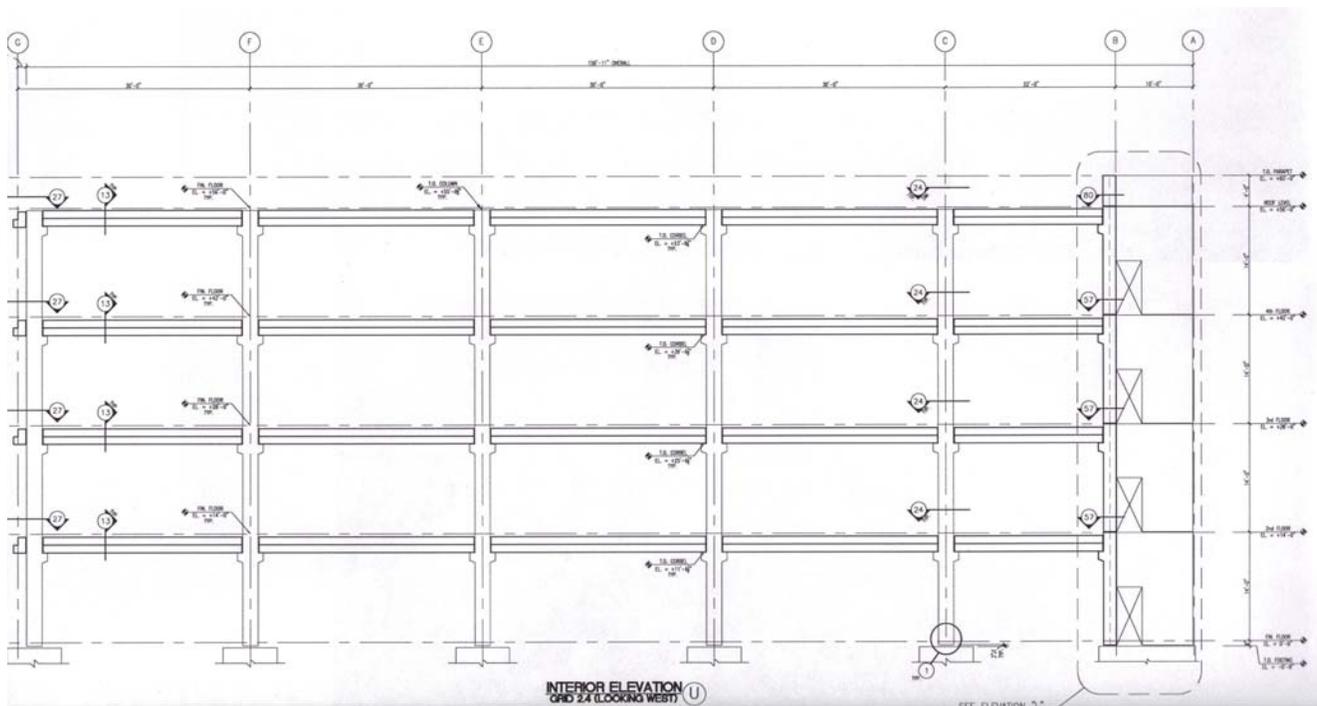
The exterior walls for North Mountain IMS Office Building fulfill many different structural requirements. First, and most importantly, they provide the building enclosure. Second, they support gravity load from double tees. Third, the walls are detailed to provide a pleasant architectural aesthetic. 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.

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 showcase the bearing condition for double tees.
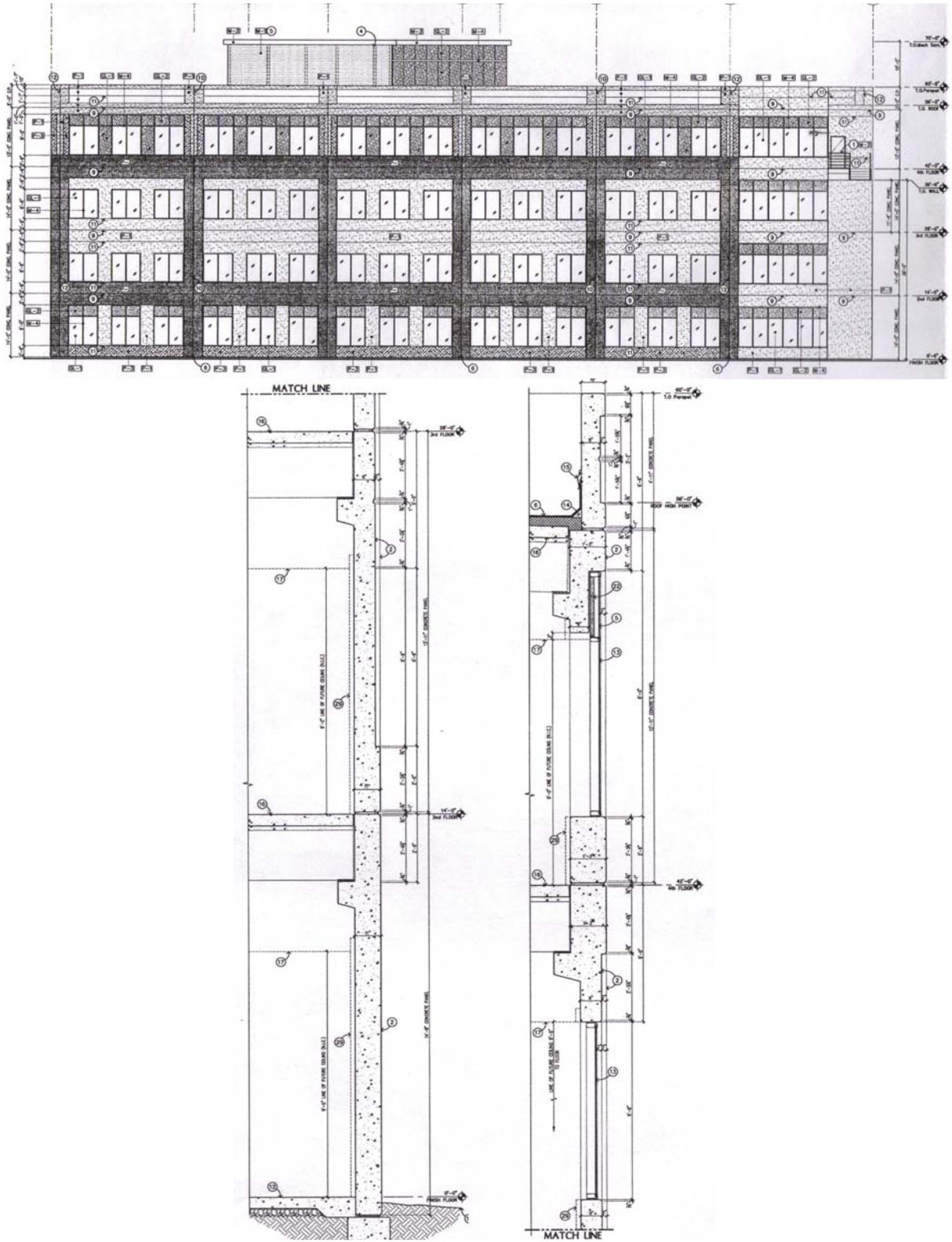
Load Summary

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

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. This value was used as the live load over the entire floor. In design, 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.

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. The complete wind load calculation is provided in the Appendix on pages 19-21. For this report, Method 2 from ASCE-7 2005 was used to calculate wind pressures on the main wind-force resisting system. The resulting calculations gave a base shear value of 218 kips in the North-South direction and 285 kips in the East-West direction.

Seismic loading was expected to control the lateral design, and this turned out to be the case. The design base shear for North Mountain 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. A significant component of that weight is the topping for the double tees. The precast double tees have a 2" flange with a minimum topping of 3-1/4". I used a topping dimension of 4" for each floor and the roof, giving a total slab depth of 6". Also, the exterior walls are concrete which is much heavier than

typical curtain wall systems. The calculated design base shear is 1627 kips, which is very similar to the value the design professional used.

All concrete on the project is normal-weight. Using light-weight concrete would considerably reduce the earthquake load on the building. Complete seismic calculations can be found on pages 16-18 in the Appendix.

Evaluation of Structural System

Precast concrete as a material and building technique can provide structures that are very economical and efficient. North Mountain IMS Office Building is well balanced between function, economics, and efficiency. However, obtaining a balanced building project is no simple task. Precast design encompasses many different disciplines which add to the complexity of a project. There is the architect, engineer, fabricator, and erector each with their own unique objectives. Economical and efficient precast structures effectively combine all party's objectives. Communication and good project management is extremely important; all parties involved must work closely together.

The precast fabricator is a large component because they can only make pieces in certain sizes and shapes. The architect and design engineer must first consider the capabilities of the fabricator when considering a precast structure. This is only one example why early communication is an essential part of the design process.

Repetition and similarity work best for precast; money is saved because there is less time associated with detailing and fabrication. North Mountain utilizes this principle very well. Each floor is almost identical with similar double tee and girder spans.

North Mountain exclusively uses normal-weight concrete. Due to large structure weight, the seismic load for this building is more than five times than the wind load. It would be a reasonable exercise to analyze the structure using a light-weight concrete, since reducing the weight is the only way to reduce the seismic load. Besides using light-weight concrete, another way to reduce weight would be to eliminate the exterior shear walls. These concrete walls account for roughly 20% of the total structure weight. Investigations for future reports may include different envelope configurations that could reduce structure weight. However, eliminating exterior shear walls would change the lateral load resisting system. This new lateral system would most likely be moment-resisting frames. This type of system would require more columns in the interior of the building. This system may have been avoided to keep the upper floors as open as possible. A cost analysis is required determine the most economical design solution.

Precast concrete presents an interesting thesis project; there are numerous design elements that must be considered during design. Changing just one of these elements could result in a completely different building.

Double Tee Spot-check

To spot check a 48' span double tee, the PCI Design Handbook 6th edition was used. The calculation checks the concrete stresses at release and at service loads in flexure only. Other design requirements for double tees include transverse bending of the flange shear capacity. These requirements, although important for structural performance, are beyond the scope of this report. Such requirements are based on the fabricator's standards and are more a detailing issue than a engineering issue. Refer to pages 22-24 in the Appendix for calculations.

Upon review of the calculations, two areas of concern arose. At release, the compressive stress in the bottom fibers is higher than code limits. Since the stress is over by less than 5%, this error could be accounted for in the estimated section properties. Section properties used by the design professional were unavailable for the hand calculation. Also, the tensile stress at 0.4L under service load was calculated much higher than allowable by code. This could be the result of using a span greater than the actual span. A span of 48' does not account for the ledges on the inverted tee girder or the thickness of the girder itself. A more accurate span of the double tee is 47'. Once again, a difference in section properties may have had role in the discrepancies in stresses.

Appendix

Structure Weight Calculation

Exterior Wall Panels

Elevation	Level	Number of Panels	Thickness (in.)	Length (ft.)	Height (ft.)	Opening Size (ft.²)	Number of Openings	Panel Vol. (ft.³)	Panel Wt. (kips)	Total Wt. (kips)
South	4	3	15	30	14	46	3	387.0	58.1	174.2
	4	2	15	32	14	46	3	422.0	63.3	126.6
	3	3	9	30	14	43	3	186.0	27.9	83.7
	3	2	9	32	14	43	3	207.0	31.1	62.1
	2	3	11	30	14	43	3	256.0	38.4	115.2
	2	2	11	32	14	43	3	281.7	42.3	84.5
	1	3	11	30	14.5	46	3	260.8	39.1	117.3
	1	2	11	32	14.5	46	3	287.3	43.1	86.2
West	4	5	15	30	14	46	3	387.0	58.1	290.3
	4	2	15	32	14	46	3	422.0	63.3	126.6
	3	5	9	30	14	43	3	186.0	27.9	139.5
	3	2	9	32	14	43	3	207.0	31.1	62.1
	2	5	11	30	14	43	3	256.0	38.4	192.0
	2	2	11	32	14	43	3	281.7	42.3	84.5
	1	5	11	30	14.5	46	3	260.8	39.1	195.6
	1	2	11	32	14.5	46	3	287.3	43.1	86.2
North	4	2	15	25.75	14	58	2	334.6	50.2	100.4
	4	1	15	28.3	14	0	0	495.3	74.3	74.3
	3	2	9	25.75	14	55	2	160.4	24.1	48.1
	3	1	9	28.3	14	0	0	297.2	44.6	44.6
	2	2	11	25.75	14	55	2	220.5	33.1	66.1
	2	1	11	28.3	14	0	0	363.2	54.5	54.5
	1	1	11	25.75	14.5	188	1	154.3	23.1	23.1
	1	1	11	25.75	14.5	101	1	241.3	36.2	36.2
	1	1	11	28.3	14.5	0	0	376.2	56.4	56.4
East	4	1	15	31	14	46	3	404.5	60.7	60.7
	4	2	15	30	14	46	3	387.0	58.1	116.1
	4	1	15	32	14	46	3	422.0	63.3	63.3
	4	2	15	25.7	14	58	2	333.8	50.1	100.1
	4	1	15	28.3	14	0	0	495.3	74.3	74.3
	3	1	9	31	14	43	3	196.5	29.5	29.5
	3	2	9	30	14	43	3	186.0	27.9	55.8
	3	1	9	32	14	43	3	207.0	31.1	31.1
	3	2	9	25.7	14	55	2	159.9	24.0	48.0
	3	1	9	28.3	14	0	0	297.2	44.6	44.6
	2	1	11	31	14	43	3	268.8	40.3	40.3
	2	2	11	30	14	43	3	256.0	38.4	76.8
	2	1	11	32	14	43	3	281.7	42.3	42.3
	2	2	11	25.7	14	55	2	219.8	33.0	65.9
	2	1	11	28.3	14	0	0	363.2	54.5	54.5
	1	1	11	31	14.5	46	3	274.0	41.1	41.1
	1	2	11	30	14.5	46	3	260.8	39.1	78.2
	1	1	11	32	14.5	46.0	3.0	287.3	43.1	43.1
	1	2	11	25.7	14.5	55.0	2.0	231.6	34.7	69.5
	1	1	11	28.3	14.5	0.0	0.0	376.2	56.4	56.4
Parapet	Roof	1	10	841	4	0	0	2803	421	421

Total Exterior Wall Weight:	4142	kips	

Interior Wall Panels

Level	Number of Panels	Thickness (in.)	Length (ft.)	Height (ft.)	Opening Size (ft.^{2})	Number of Openings	Panel Vol. (ft.^{3})	Panel Wt. (kips)	Total Wt. (kips)
1	1	10	10.5	14.5	0	0	126.9	19.0	19.0
1	2	10	20.75	14.5	0	0	250.7	37.6	75.2
1	1	10	10.5	14.5	35	1	91.9	13.8	13.8
2	1	10	11	14	0	0	122.5	18.4	18.4
2	2	10	21	14	0	0	242.1	36.3	72.6
2	1	10	11	14	35	1	87.5	13.1	13.1
3	1	10	11	14	0	0	122.5	18.4	18.4
3	2	10	21	14	0	0	242.1	36.3	72.6
3	1	10	11	14	35	1	87.5	13.1	13.1
4	1	10	11	14	0	0	122.5	18.4	18.4
4	2	10	21	14	0	0	242.1	36.3	72.6
4	1	10	11	14	35	1	87.5	13.1	13.1
Roof	1	10	11	13	0	0	113.8	17.1	17.1
Roof	2	10	21	13	0	0	224.8	33.7	67.4
Roof	1	10	11	13	35	1	78.8	11.8	11.8

Total Interior Wall Weight:	517	kips

Superimposed Dead Load

Level	Dead Load (psf)	Floor Area (ft.^{2})	Superimposed Load (kips)
2	15	29289	439
3	15	29656	445
4	15	29536	443
Roof	15	29600	444

Total Topping Weight =	1771	Kips

Partition Load

Level	Dead Load (psf)	Floor Area (ft.^{2})	Partition Load (kips)
2	20	29289	586
3	20	29656	593
4	20	29536	591

Total Topping Weight =	1770	Kips

Double Tee

Level	Length (ft.)	Width (ft.)	Single Tee Weight (plf)	Tee Weight (kips)	Number of Tees on Level	Total Weight (kips)
2	48	10	470	22.6	28	631.7
2	44	10	470	20.7	31	641.1
2	44	8	418	18.4	3	55.2
2	37	8	418	15.5	3	46.4
3	54	8	418	22.6	3	67.7
3	48	10	470	22.6	28	631.7
3	44	10	470	20.7	31	641.1
3	44	8	418	18.4	3	55.2
4	54	8	418	22.6	3	67.7
4	48	10	470	22.6	28	631.7
4	44	10	470	20.7	31	641.1
4	44	8	418	18.4	3	55.2
Roof	54	8	418	22.6	3	67.7
Roof	48	10	470	22.6	28	631.7
Roof	44	10	470	20.7	31	641.1
Roof	44	8	418	18.4	3	55.2

Total Double Tee Weight =	5561	Kips

Topping

Level	Thickness (in.)	Floor Area (ft.^{2})	Topping Weight (kips)
2	4	29289	1464
3	4	29656	1483
4	4	29536	1477
Roof	4	29600	1480

Total Topping Weight =	5904	Kips

Inverted Tee Girder

Level	Length (ft.)	Width (in.)	Height (in.)	Girder Weight (plf)	Single Girder Weight (kips)	Number of Tees on Level	Total Weight (kips)
2	20	32	24	625	13	2	25.0
2	28	32	24	625	18	9	157.5
3	20	32	24	625	13	2	25.0
3	28	32	24	625	18	9	157.5
4	20	32	24	625	13	2	25.0
4	28	32	24	625	18	9	157.5
Roof	20	32	24	625	13	2	25.0
Roof	28	32	24	625	18	9	157.5

Total Inverted Tee Girder Weight =	730	Kips

L-Beam

Level	Length (ft.)	Width (in.)	Height (in.)	Girder Beam (plf)	Single Beam Weight (kips)	Number of Beams on Level	Total Weight (kips)
2	28	19	24	388	11	2	21.7
2	18	19	24	388	7	2	14.0
3	28	19	24	388	11	2	21.7
3	18	19	24	388	7	2	14.0
4	28	19	24	388	11	2	21.7
4	18	19	24	388	7	2	14.0
Roof	28	19	24	388	11	2	21.7
Roof	18	19	24	388	7	2	14.0

Total Inverted Tee Girder Weight =	143	Kips

Column

Depth (in.)	Width (in.)	Height (in.)	Column Weight (plf)	Single Column Weight (kips)	Number of Columns	Total Weight (kips)
24	24	56	600	34	17	571.2
24	24	42	600	25	1	25.2
36	36	14	1350	19	1	18.9

Total Column Weight =	615	Kips

Seismic Base Shear Calculation

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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:

$$S_s = 0.256 \qquad S_1 = 0.075$$

2. SITE CLASSIFICATION:

SITE CLASS C

3. RESPONSE ACCELERATIONS:

$$S_{MS} = F_a S_s \qquad S_{M1} = F_v S_1$$
$$S_{MS} = 1.2(0.256) \qquad S_{M1} = 1.7(0.075)$$
$$S_{MS} = 0.307 \qquad S_{M1} = 0.128$$

4. 5% - DAMPED DESIGN SPECTRAL RESPONSE ACCELERATIONS:

$$S_{DS} = \tfrac{2}{3} S_{MS} \qquad S_{D1} = \tfrac{2}{3} S_{M1}$$
$$S_{DS} = \tfrac{2}{3}(0.307) \qquad S_{D1} = \tfrac{2}{3}(0.128)$$
$$S_{DS} = 0.205 \qquad S_{D1} = 0.085$$

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SEISMIC BASE SHEAR (CONT.)

5. APPROXIMATE FUNDAMENTAL PERIOD:

$$T_a = C_t \cdot h_n^x$$
$$T_a = 0.02(56)^{(0.75)}$$
$$T_a = 0.409 s$$

$C_t = 0.02$
$h_n = 56'$
$x = 0.75$

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

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

$R = 4.0$
$I = 1.5$

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

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

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:

$$V = C_s \cdot W$$
$$V = 0.0769(21153^k)$$

$$\boxed{V = 1627^k}$$

Seismic Load Distribution

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SEISMIC LOAD DISTRIBUTION

LATERAL FORCE AT EACH LEVEL:

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

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 $\quad C_{v2} = \dfrac{6213(14.5)}{711300} = 0.127$

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

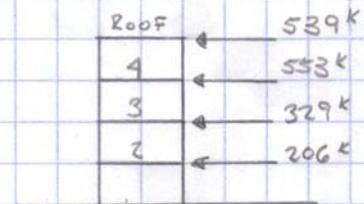LEVEL 3 $\quad C_{v3} = \dfrac{5041(28.5)}{711300} = 0.202$

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

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

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

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

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

ROOF	539^k
4	553^k
3	329^k
2	206^k

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
(Table 1-1)   Surgery or Emergency Treatment)

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)

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

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

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

$$c = 0.30 \qquad (\text{Table 6-2})$$

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

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

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

Note: Substitude $B = 184$ For North-South Direction

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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

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WIND LOAD (CONT.)

10. DESIGN WIND LOAD

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

$$G = 0.819 \ (N\text{-}S)$$

$$C_{pf} = 0.8 \ \text{WINDWARD}$$
$$-0.5 \ \text{LEEWARD}$$

$$C_{pi} = \pm 0.18$$

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

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
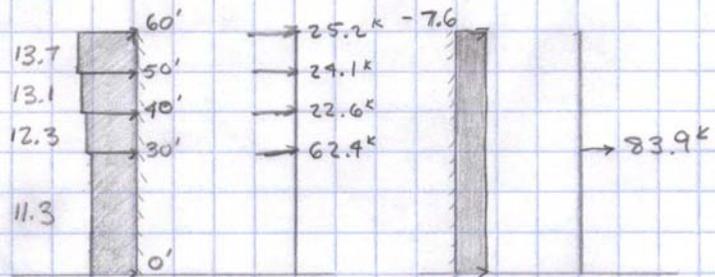
WINDWARD FORCE = 134.3ᵏ   LEEWARD FORCE = 83.9ᵏ

TOTAL FORCE = 134.3 + 83.9 = 218.2ᵏ (N-S)
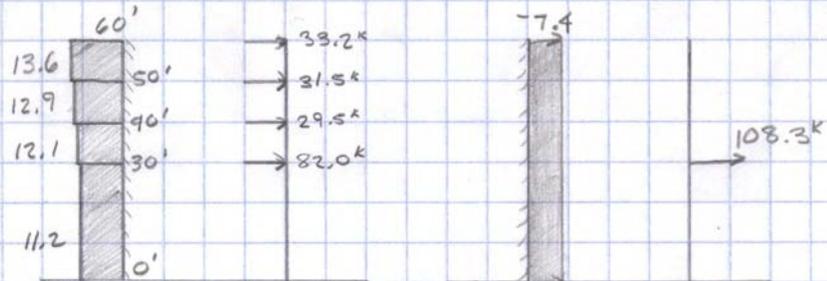
WINDWARD FORCE = 176.2   LEEWARD FORCE = 108.3ᵏ

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

Double Tee Spot-check Calculation

Michael Hopple
Tech Report 1

Double Tee Spot-Check (PCI 6th Edition)

Use Standard 10'-0" x 24" Double Tee Section Properties

Note: Section Properties for Standard Tee may be different from Manufacture's.

Span: 48'-0"

Live Loads:

Partitions:	20 psf	= 200 plf
Corridors:	80 psf	= 800 plf
Total:	100 psf	= 1000 plf

Dead Loads:

Self-Weight	47 psf	= 470 plf
Superimposed	15 psf	= 150 plf
Topping	50 psf	= 500 plf
	112 psf	= 1120 plf

Concrete Strength

$f'_{ci} = 4,200$ psi , $f'_c = 6,000$ psi

Prestressing Strand:

(12) ½" Ø 270K Low-Relaxation Strand

$A_{ps} = 12 \times 0.153 = 1.836 \text{ in}^2$

Section Properties:

$A = 449 \text{ in}^2$

$I = 22,470 \text{ in}^4$

$y_b = 17.77 \text{ in.}$

$y_t = 6.23 \text{ in.}$

$S_b = 1264 \text{ in}^3$

$S_t = 3607 \text{ in}^3$

$W_t = 468 \text{ plf} = 47 \text{ psf}$

Eccentricity, Single-Point Depression:

END SPAN

$$\bar{y}_{\text{strand}} = \frac{2\frac{1}{4} + 4 + 6 + 8 + 14 + 16}{6} = 8.4"$$
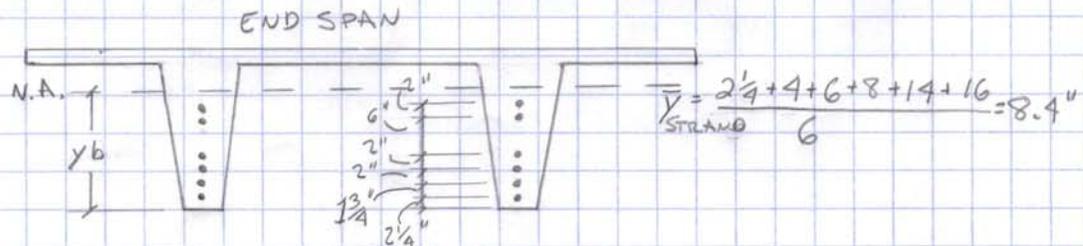
$e_e = y_b - \bar{y}$

$= 17.77 - 8.4$

$= 9.37"$

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TECH REPORT 1

DOUBLE TEE SPOT-CHECK (CONT.)

@ MID SPAN, $\bar{y}_{STRAND} = \dfrac{2\frac{1}{4} + 2\frac{3}{4} + 3\frac{1}{4} + 3\frac{3}{4} + 4\frac{1}{4} + 4\frac{3}{4}}{6} = 3.5''$

$e_c = y_b - \bar{y}$

$e_c = 17.77 - 3.5$

$e_c = 14.27''$

$e' = 19.27 - 9.37'' = 4.9''$

$e @ 0.4\ell \left(\dfrac{0.8\ell}{2} \right) = 9.37 + 0.8 (4.9'') = 13.29''$

PRESTRESS FORCE:

$P_i = 0.725 A_{ps} f_{pu}$

$P_i = 0.725 (1.836)(270)$

$P_i = 359$ KIPS

$P_o = 0.9 (359) = 323$ KIPS (ASSUMES 10% INITIAL PRESTRESS LOSS)

$P = 0.8 (359) = 287$ KIPS (ASSUMES 20% TOTAL PRESTRESS LOSS)

SERVICE LOAD MOMENTS AT MIDSPAN:

$M_d = \dfrac{w\ell^2}{8} = \dfrac{0.470 (48)^2 (12)}{8} = 1624$ K-in (SELF-WEIGHT)

$M_{sd} = \dfrac{w\ell^2}{8} = \dfrac{0.150 (48)^2 (12)}{8} = 518$ K-in $\left(\begin{array}{c} \text{SUPERIMPOSED} \\ \text{DEAD LOAD} \end{array} \right)$

$M_\ell = \dfrac{w\ell^2}{8} = \dfrac{1.0 (48)^2 (12)}{8} = 3456$ K-in (LIVE LOAD)

$M_{TOP} = \dfrac{w\ell^2}{8} = \dfrac{0.5 (48)^2 (12)}{8} = 1728$ K-in (TOPPING)

SERVICE LOADS AND MOMENTS AT 0.4ℓ:

$M_d = 1624 (0.96) = 1559$ K-in

$M_{sd} = 518 (0.96) = 497$ K-in

$M_\ell = 3456 (0.96) = 3318$ K-in

$M_{TOP} = 1728 (0.96) = 1659$ K-in

MICHAEL HOPPLE
TECH REPORT 1

DOUBLE TEE SPOT-CHECK (CONT.)

SERVICE LOAD STRESSES:

LOAD	TRANSFER Pt. @ RELEASE $P = P_o$		MIDSPAN @ RELEASE $P = P_o$		0.4ℓ @ SERVICE LOAD $P = P$		
	f_b	f_t	f_b	f_t	f_b	$f_t(a)$	$f_t(b)$
P/A	+719	+719	+719	+719	+639	+639	+639
P·e/S	+2394	-839	+3647	-1278	+3018	-1057	-1057
M_d/S	-190	+43	-1285	+450	-1233	+432	+432
M_{sd}/S	—	—	—	—	-393	+138	+138
M_{TOP}/S	—	—	—	—	+1313	+460	+460
M_ℓ/S	—	—	—	—	-2625	—	+920
STRESSES	+2923	-77	+3081	-109	-1907	+612	1532
LIMITING STRESSES	$0.70 f'_{ci}$	$7.5\sqrt{f'_{ci}}$	$0.70 f'_{ci}$	$7.5\sqrt{f'_{ci}}$	$12\sqrt{f'_c}$	$0.45 f'_c$	$0.6 f'_c$
	+2940	-486	+2940	-486	-930	2700	3600
	O.K.	O.K.	NOT O.K.	O.K.	NOT O.K.	O.K.	O.K.

(+) INDICATES COMPRESSION
(-) INDICATES TENSION