CHRISTIANA HOSPITAL
2010 PROJECT
NEWARK, DE

Final Report
Spring 2007

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Structural Option
Faculty Consultant: Dr. Memari

Christiana Hospital
2010 Project
Newark, DE

Project Team
- Architect
  Wilmot Sanz
- Civil Engineer
  VanDemark & Lynch, Inc.
- MEP Engineer
  RMF Engineering, Inc.
- Structural Engineer
  Cagley & Associates

Architecture
- Brick Veneer
- Glass curtain walls with aluminum frames
- Roofing membrane on tapered insulation

Building Information
- 299,000 square foot addition
- 8 story structurally reinforced concrete hospital
- 2 story structural steel conference wing
- 1 story below grade
- Adds 216 beds
- Creates additional operating rooms, catheterization labs and emergency exam rooms
- Expands Christiana Care's cardiovascular program
- Delivery Method - Design-Bid-Build

Lighting/Electrical
- (2) 35 KV primary feeders
- Primary Voltage – 480/277V
- Secondary Voltage – 208/120V
- Emergency Power – 1500 KVA Generator
- Linear Fluorescent and Halogen Lighting

Mechanical
- 8 AHUs supply air at rates ranging from 22,800 – 32,000 CFM
- Special filters for AHUs supplying clinical areas
- Receives steam and chilled water from outside source

Conference Wing
- Spread Footings
- 3¼" lightweight concrete over 2" metal deck
- 4 concentrically braced frames

Hospital
- 42" thick mat
- 9½" two-way flat slab with 5½" drops around columns
- 12" thick shears walls placed perpendicular to buildings perimeter

Joseph G. Sharkey
Structural
http://www.arche.psu.edu/thesis/eportfolio/2007/portfolios/JGS186/index.htm

Executive Summary

The Christiana Hospital 2010 Project is a $126 million, 360,000 square foot addition to the Christiana Hospital located in Newark, Delaware. The addition is essentially L-Shaped and was designed using both steel, in the conference wing, and reinforced concrete, in the main tower.

My research has looked into an alternative design for the hospital by both dividing the main tower into two separate structures and using a post-tensioned floor system throughout the entire building. These design changes ended up in some cases giving results that were unexpected. In the case of separating the main tower into two independent structures it was assumed that this would allow the shear walls to decrease in size ultimately decreasing both project cost and schedule. The outcome of this result went the opposite way. Instead of reducing the size of the loads on the walls this amplified them to the point where more walls where required.

When comparing the different floor systems it was found that the post-tensioned system proved to be a close competitor. It allowed for a lighter building and a flat slab design that lead to a slightly more economical design in both schedule and cost. While it was cheaper and faster to construct it was determined that these advantages were not great enough to out way the fact that in a hospital there is likely to be many slab penetrations during both construction and throughout the life of the building. These slab penetrations can pose significant and expensive problems when a tendon is hit.

In the end I feel it is safe to say that, given the projects location, layout, and occupancy, this is the best and most efficient solution to this design problem.

Table of Contents

Introduction

The Christiana Hospital 2010 Project is a $126 million, 360,000 square foot addition to the Christiana Hospital located in Newark, Delaware. This addition includes the Bank of America Pavilion and the John H. Ammon Medical Education Center which creates additional operating rooms, catheterization labs, emergency exam rooms, and 216 beds for patients. It will also expand Christiana Care's cardiovascular program and create an education center in partnership with the Delaware Academy of Medicine. Christiana Care is one of the region's largest not-for-profit health care providers, serving Delaware as well as areas of Maryland, Pennsylvania and New Jersey.

For the past eight months I have been researching, analyzing, and redesigning the Christiana Hospital 2010 Project in search of the most efficient and cost effective structural system. The system which I will be comparing to the original structural design is in two parts. My first change to the building will involve making the building more symmetrical for lateral, wind, and seismic loading by sectioning the main tower into two separate structures separated by an expansion joint. This design change will hopefully reduce the torsional effects of lateral load and in turn allow the shear walls to be sized smaller and/or require less total shear walls decreasing the projects schedule and cost.

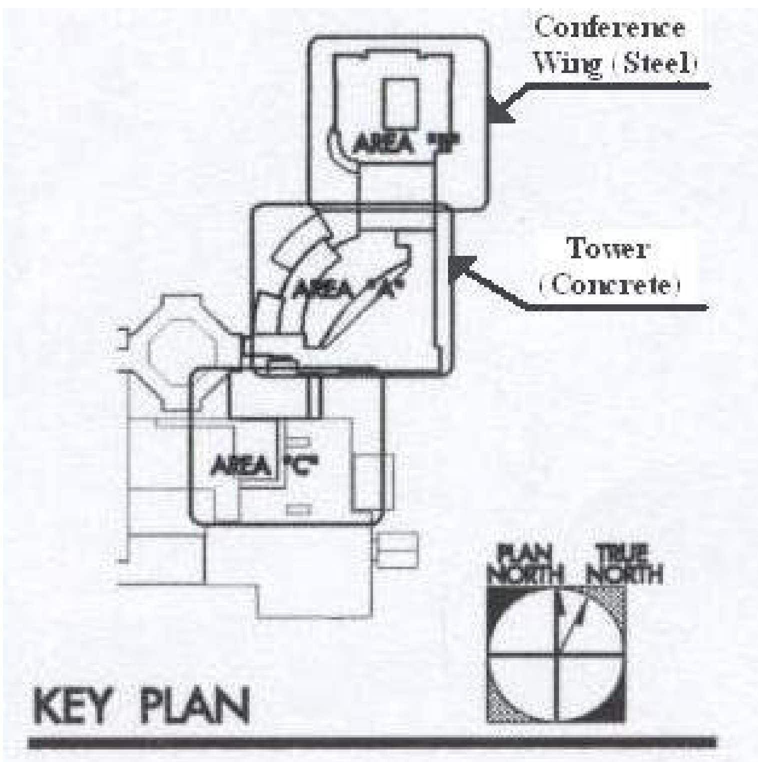
Secondly I will compare the existing structure to a structure using a two-way post-tensioned slab in the main tower and one-way post-tensioned beams and slab in the conference wing. Due to this change in the conference wing I will also make the necessary design changes to the rest of the wing which include reinforced concrete columns and reinforced concrete shear walls. Once all these structural changes have been made I will compare the existing structure with my new design using the criteria of length of schedule, practicality, and final cost.

In addition to these changes I will also do an acoustical breadth. This breadth will look at the design of the major conference room in the conference wing from the perspective of acoustics. I will look into what materials have been used to cover the walls, ceilings, and floors, and using this information will perform sound reverberation and sound transmission loss checks. With my results I will suggest any necessary changes that could be made to improve the room acoustically.

Existing Structure

The Christiana Hospital is mainly composed of structurally reinforced concrete with a stand alone adjacent steel framed conference wing. The concrete portion of the building stands 8 stories with one level underground and a penthouse roof. The structure contains varying spans which are created using a typical 9½ inch thick two-way flat slab with 5½ inch drops or shear caps. This slab transfers load to 24 inch square columns which in turn take the load down to a mat foundation. To prevent rotation and lateral displacement due to wind or seismic loading shear walls are strategically placed perpendicular to the buildings perimeter.

The conference wing is a 3 story structural steel frame with a majority of beams having pinned connections and spanning around 30 feet. In the center of this area is a larger span of over 60 feet. The buildings loads are transferred to the beams using a 3¼ inch, light weight concrete, structural slab over a 2 inch deep by 18 gage galvanized composite metal deck creating a total slab thickness of 5¼ inches. The load in the beams is transferred to steel girders which are attached using a pinned connection to W-shaped columns. These columns continue down to 4000psi concrete spread footings. The wind and seismic loading in this area is distributed using concentrically braced frames.



Foundation:

The building consists of two separate types of foundations. In the concrete tower area the building rests on a 42" thick mat foundation. This mat is reinforced with #9's at 12" o.c. each way, top and bottom, with additional reinforcing added where needed.

In the area of the conference wing, steel columns rest on concrete spread footings. These footings range in size from 4'x4'x 15" deep up to 16'x16'x 48" deep. The allowable soil bearing pressure for this site is 4000psf.

Applications	Concrete Strengths (f'_c)
Footings	4000 psi
Mat Foundation	6000 psi
Grade Beams	4000 psi
Slab-On-Grade	3500 psi

Columns:

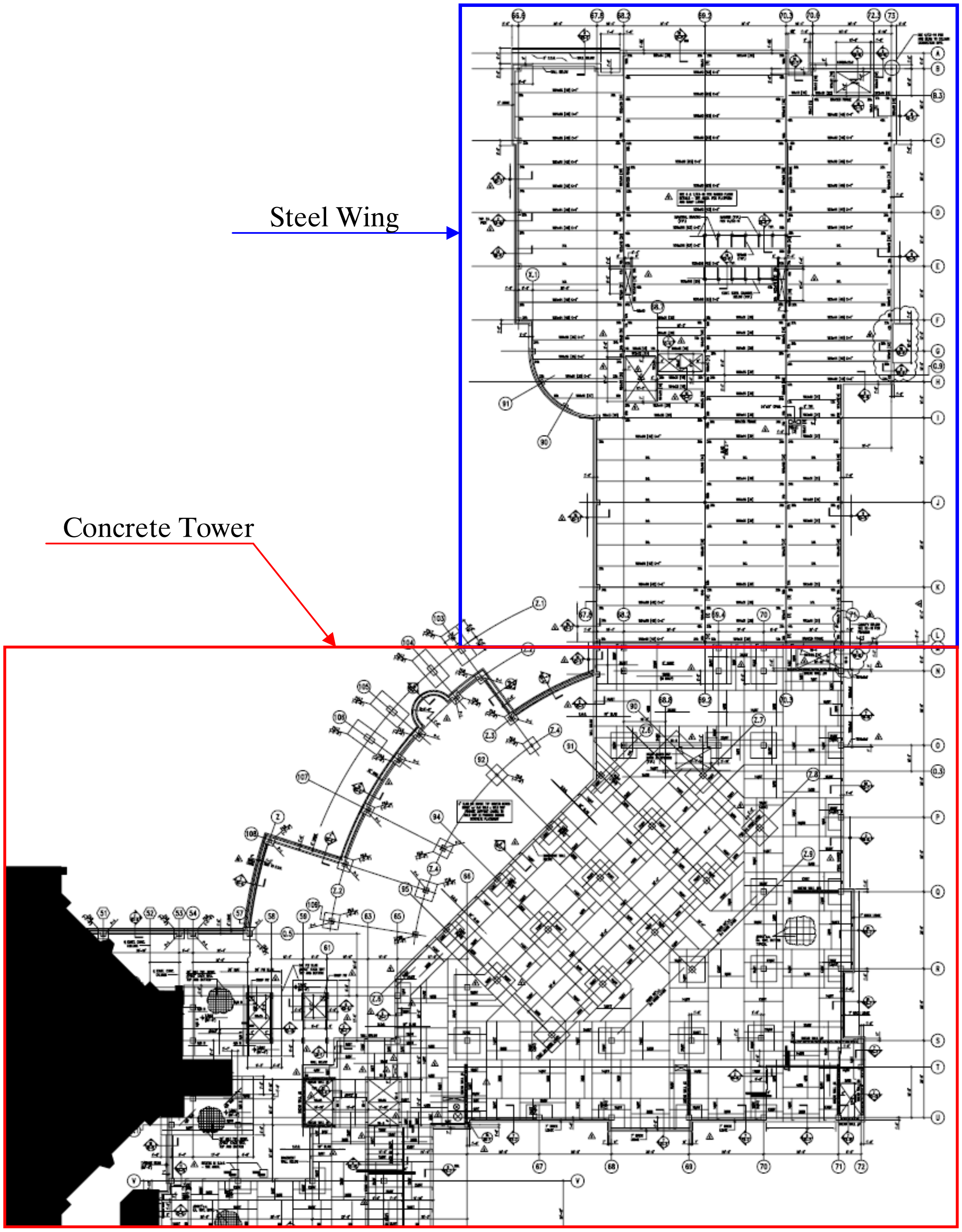
In the tower area a majority of the columns are 24"x24" reinforced concrete columns with only a few occurrences of 12"x24" columns. At the eighth floor nearly all the concrete columns stop and off of them W8 steel columns are posted. The 3 story conference wing is composed of W10 and W12 steel columns.

Applications	Material
Steel Columns	ASTM A992, Grade 50
Concrete Columns (Below Third Floor)	5000 psi
Concrete Columns (Above Third Floor)	4000 psi

Floor System:

Throughout the tower, spans are accomplished using 9½" thick two-way flat slabs with typical 5½" drops or shear caps at each column. Reinforcement for the slabs varies throughout the building.

The conference area uses a completely separate type of floor system. Here steel girders span between columns in one direction while beams, spanning in the opposite direction, frame into the girders. This steel framework works in composite action with the floor slab placed on top. The slab is constructed of 3¼" lightweight concrete over a 2" deep x 18 gage galvanized composite metal deck. The slab is then reinforced with 6x6-W2.1xW2.1 WWF. The bulk of the spans vary anywhere from 20 to 40 feet. Although, running across the middle, is a large 63 foot span made possible using W30x90 beams and the composite action.



Lateral Force Resisting System:

The lateral forces acting on the building are resisted differently in the two areas of the building. In the concrete portion of the building, lateral forces are resisted by reinforced concrete shear walls which run the entire height of the building until they are replaced by concentrically braced frames at the eighth floor (Figure 1). These shear walls are placed in specific areas to also oppose the torsional effect that the lateral loads place on the building due to its L-shape.

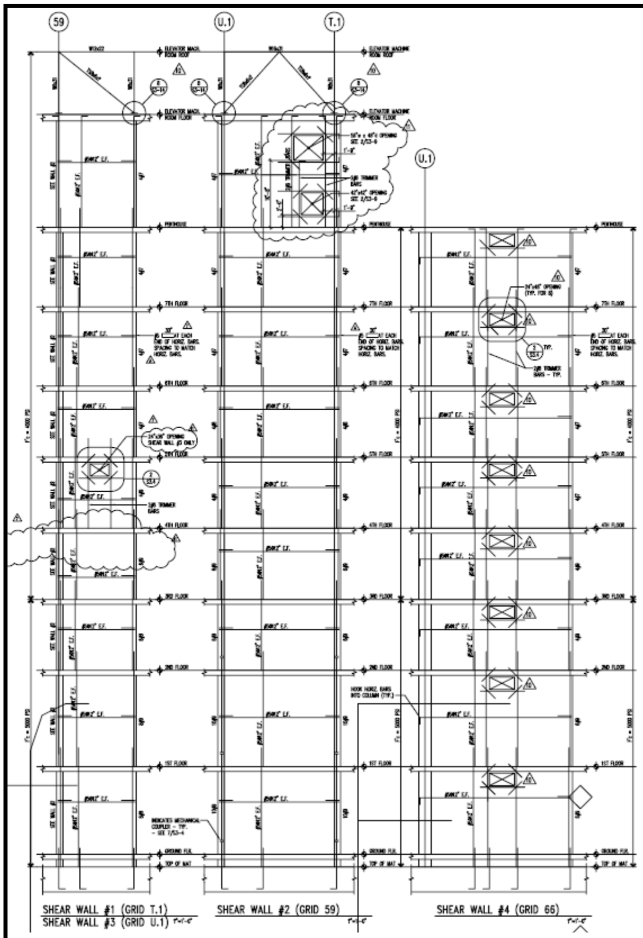


Figure 1 Shear Walls

In the conference wing lateral loads are taken care of with the use of concentrically braced frames (Figure 2). These frames are constructed using rectangular HSS steel. This framing is field welded to gusset plates. These gusset plates are attached in the fabrication shop, by means of a weld, to select beams.

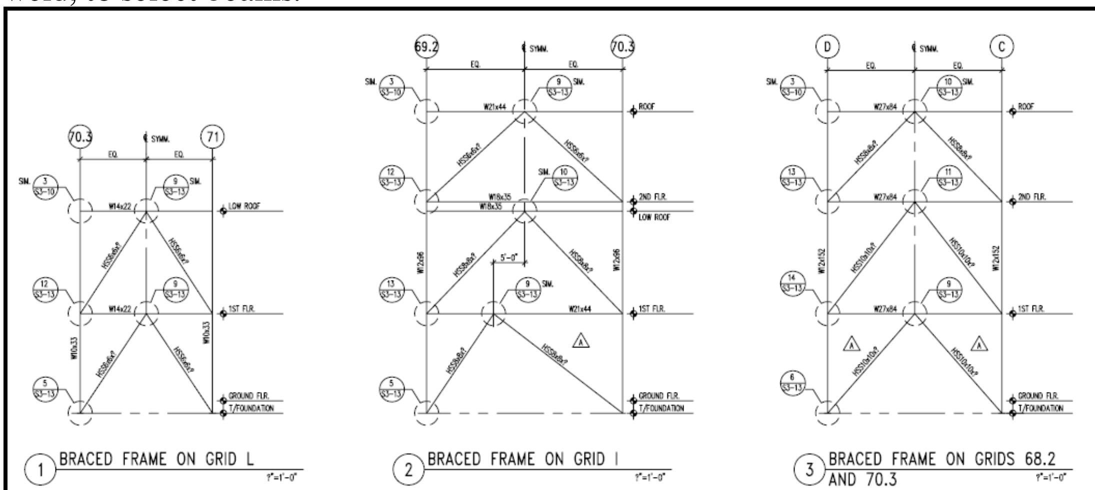


Figure 2 Concentrically Braced Frames

Roof System:

The framing of the roof is done entirely with steel and metal decking. The decking used is a 1½" deep, wide rib, 20 gage galvanized metal deck. On top of the decking is a one hour fire rated roof construction. This consists of a 45 mill fully adhered roofing membrane on tapered insulation on 5/8" exterior gypsum board. The metal decking is also sprayed with a fireproofing at the soffits.

Proposed Structural Design

In my structural design of the Christiana Hospital I have proposed to look at two separate adjustments to the structure. The first involves the lateral system while the second involves the floor system. As previously mentioned, the current lateral system for the main concrete tower of the building is composed of strategically placed shear walls. I feel that these walls have the potential to be reduced in size and/or number by reducing the lateral forces imposed on them. In an attempt to reduce these forces I will create a more symmetrical building by separating the main tower with an expansion joint, along column line 65 (Figure 3), into two separate structures thus decreasing the torsional effects of lateral load on the walls. The purpose for attempting to decrease the number of shear walls and/or their sizes is to reduce the cost of the project.

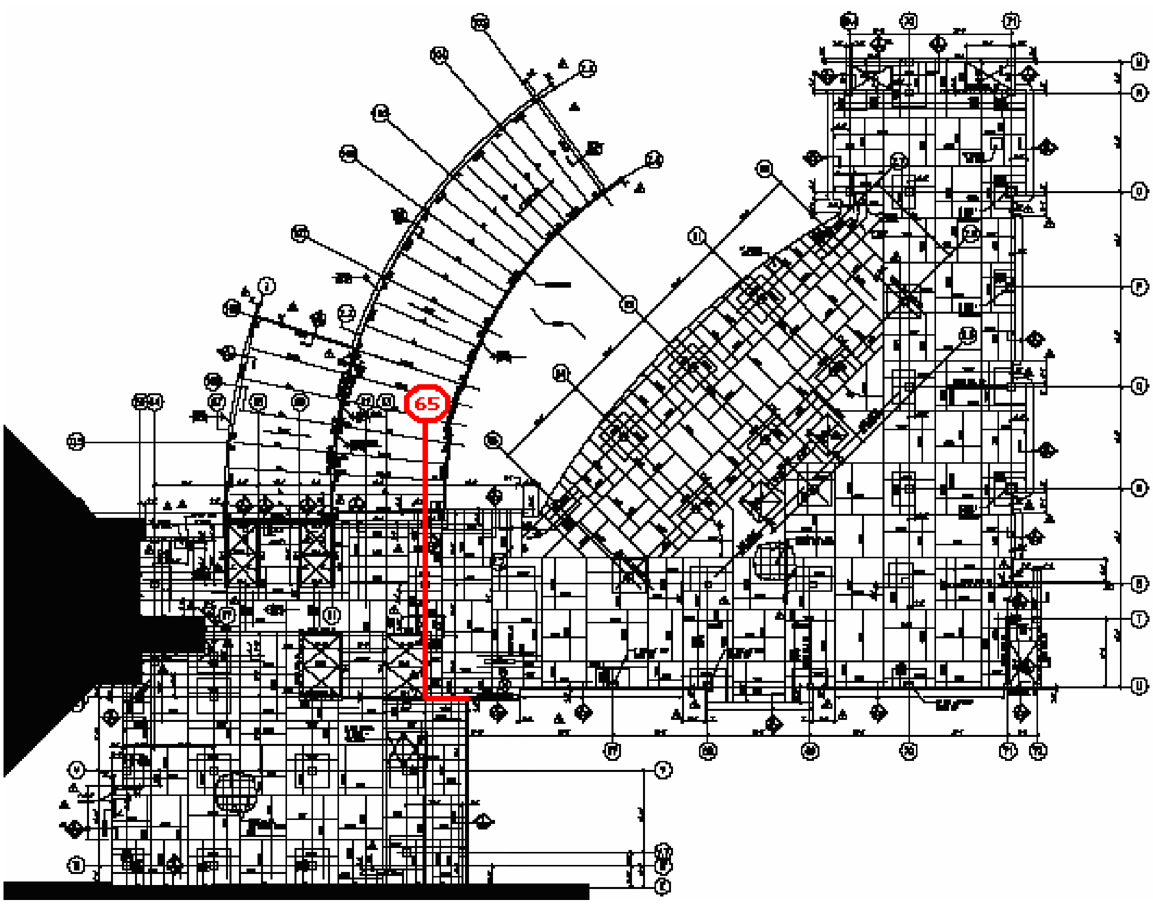


Figure 3 Expansion joint located on grid line 65

My second design change is to change all the floor systems in the structure, including the conference wing, to post-tensioned concrete. Having completed an analysis of alternate floor systems (Refer to Technical Report #2) it is obvious that the only types of floor systems economical enough to be used for the main tower area are two-way systems. Being that the current floor system is a two-way reinforced concrete slab with drop panels at the columns the best alternative to compare with it is a two-way post-tensioned concrete slab. This slab design will hopefully allow for the deletion of the drop panels which can potentially reduce both the project schedule and the project cost by reducing the complexity of the formwork.

To change the conference wing (currently steel) to post-tensioned concrete, a design using one-way post-tensioned slabs and beams has been chosen due to the length of the spans. As a result of this change the columns in the building will also be redesigned as concrete and the lateral force resisting system will be changed from concentrically braced frames to reinforced concrete shear walls. After designing all of the changes mentioned above both a schedule and a cost analysis will be performed comparing the existing design with my proposed design.

Codes & Loading Cases

Codes Used for Original Design

- International Building Code – 2000
- ASCE 7-98, American Society of Civil Engineers – Minimum Design Loads for Buildings and Other Structures
- ACI 318-99, American Concrete Institute – Building Code Requirements for Structural Concrete
- ACI Manual of Concrete Practice – Parts 1 through 5 – 1997
- Manual of Standard Practice – Concrete Reinforcing Steel Institute
- AISC Manual of Steel Construction – Allowable Stress Design, Ninth Ed., 1989
- AISC Manual of Steel Construction – Volume II Connections – ASD Ninth Ed./LRFD First Ed.
- AISC Detailing for Steel Construction
- American Welding Society – Structural Welding Code ANSI/AWS D1.1-96
- Steel Deck Institute – Design Manual for Floor Decks and Roof Decks
- Drift Criterion – h/400

Codes Used for Thesis Design

- International Building Code – 2003
- ACI 318-05, American Concrete Institute – Building Code Requirements for Structural Concrete
- ETABS Model – International Building Code – 2000
- ETABS Model – ASCE 7-98
- AISC Manual of Steel Construction – Load and Resistance Factor Design, Third Ed., 2005
- Drift Criterion – Wind: h/400
  Seismic: 0.01h (ASCE7-02 9.5.2.8)

Load Cases – Obtained using IBC 2003

- $1.4D$
- $1.2D + 1.6L + 0.5(L_r \text{ or } S)$
- $1.2D + 1.6(L_r \text{ or } S) + (f_1L \text{ or } 0.8W)$
- $1.2D + 1.6\, f_1L + 0.5(L_r \text{ or } S)$
- $1.2D + 1.0E + f_1L + f_2S$
- $0.9D + (1.0E \text{ or } 1.6W)$

D = Dead Load
L_r = Roof Live Load
S = Snow Load
W = Wind Load
E = Seismic or Earthquake Loading

L = Live Load
f_1 = 1.0 for live loads in excess of 100 psf and 0.5 for all other loads
f_2 = 0.2

Gravity Loading

Floor Live Loads	
Occupancy or Use	**Uniform Live Load (psf)**
Assembly Space	100
Typical Hospital Floor	60
Corridor	80
Mechanical Rooms	150
Stair	100
Roof	15
Partition	20

Floor Dead Loads	
Occupancy or Use	**Dead Load**
Reinforced Concrete	150 pcf
Steel Members	Varies
Floor Superimposed	15 psf
Roof Superimposed	15 psf

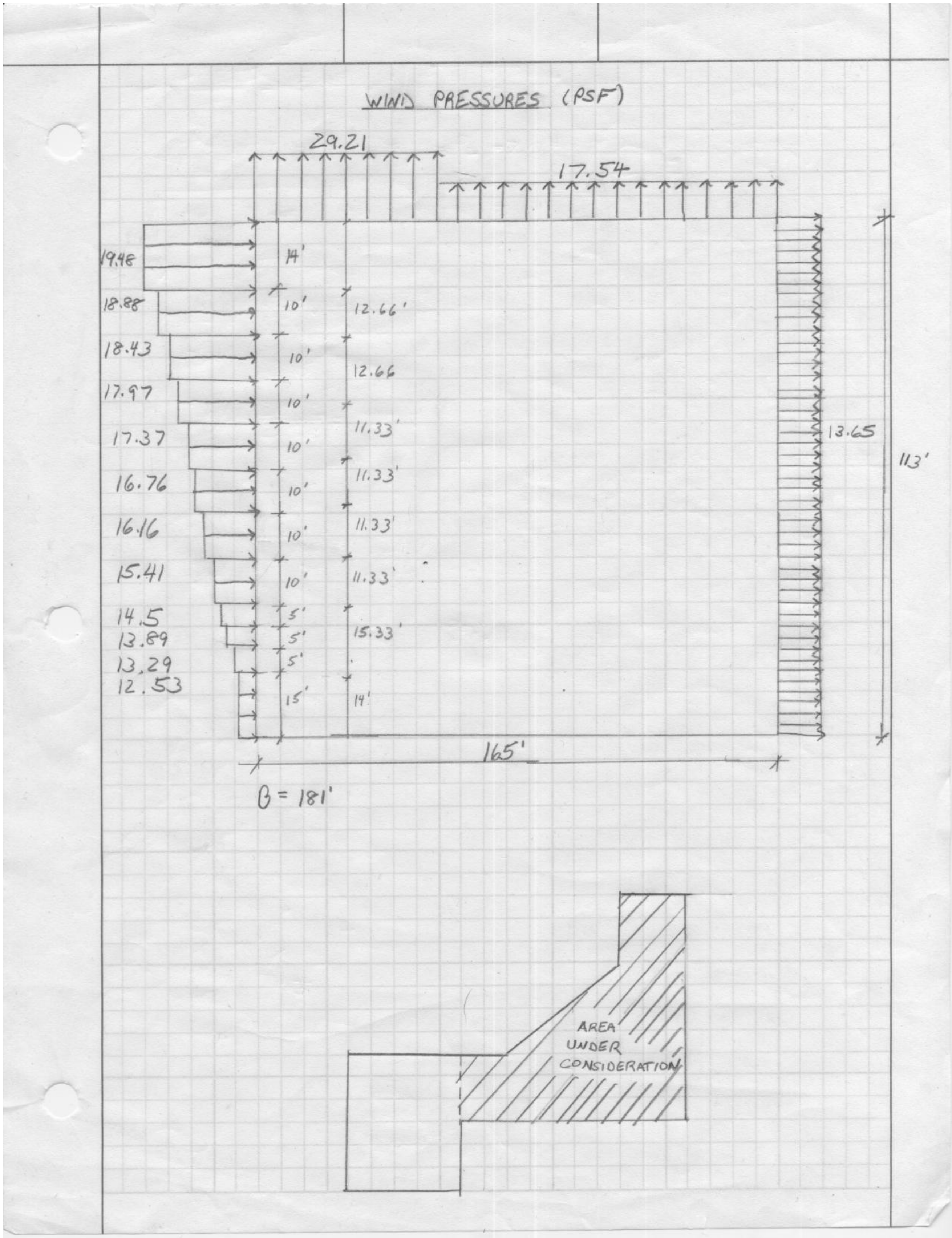
Snow Loading	
Item	**Value**
Ground Snow Load (P_g)	25 psf
Exposure Category	B
Roof Exposure	Partially Exposed
Exposure Factor (C_e)	1.0
Thermal Factor (C_t)	1.0
Occupancy Category	IV
Importance Factor (I_s)	1.2
Flat-Roof Snow Load $P_f = 0.7C_eC_tI_sP_g$	21 psf

Wind Loading

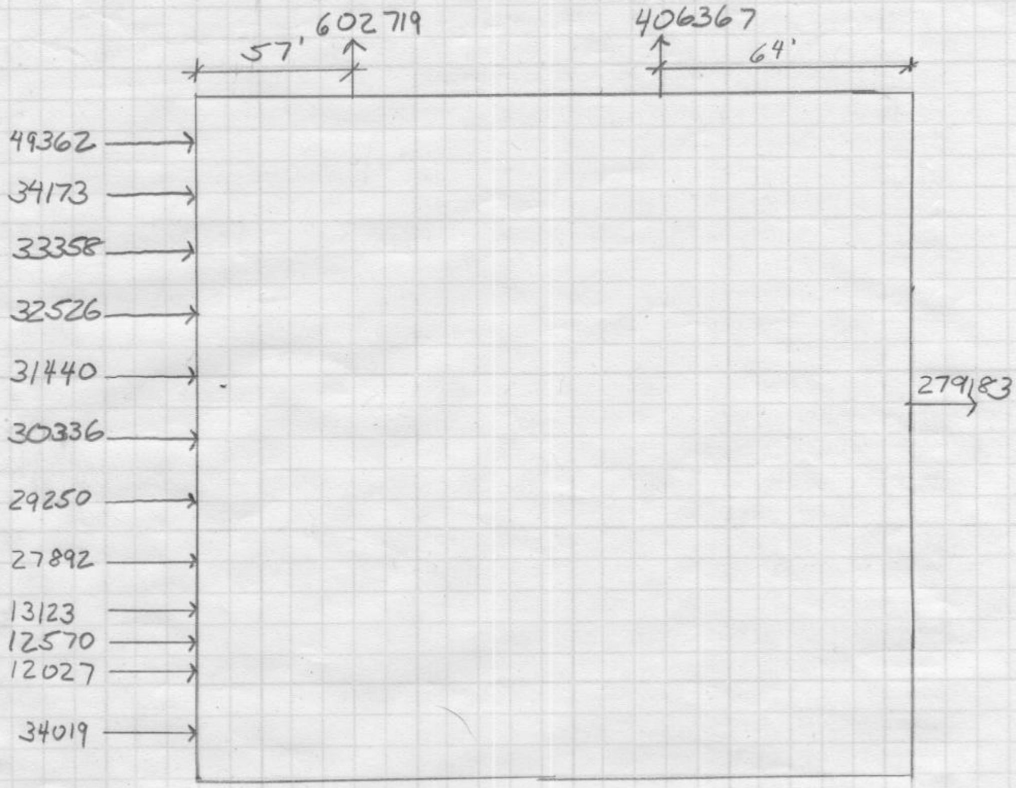
Assumptions: For the wind loading calculations, only one side of the building was calculated. The side chosen was the plan North face of the building. This was done because it is both the longest and tallest side of the building. By doing this the largest wind loads were found. For simplicity these loads will then be applied to all other faces according to their heights. The two separate structures that have been created do to the expansion joint have been both taken into consideration.

Exposure Category	**K_{zt}**	**K_d**	**I**	**V (mph)**	**h (ft)**	**G**	**GC_{pi} (+/-)**
B	1	0.85	1.2	90	114	0.893	0.18

Wind Design Pressures							
			Windward	Leeward	Side Walls	Roof	
						0-57'	>57'
		C_p	0.8	-0.5	-0.7	-1.3	-0.7
h (ft)	K_z	q_z	p (psf)				
0-15	0.57	12.0559	12.53	-13.65	-17.54	-29.21	
20	0.62	13.1134	13.29	-13.65	-17.54	-29.21	
25	0.66	13.9595	13.89	-13.65	-17.54	-29.21	
30	0.7	14.8055	14.5	-13.65	-17.54	-29.21	
40	0.76	16.0745	15.41	-13.65	-17.54	-29.21	
50	0.81	17.1321	16.16	-13.65	-17.54	-29.21	
60	0.85	17.9781	16.76	-13.65	-17.54	-29.21	
70	0.89	18.8241	17.37	-13.65	-17.54		-17.54
80	0.93	19.6702	17.97	-13.65	-17.54		-17.54
90	0.96	20.3047	18.43	-13.65	-17.54		-17.54
100	0.99	20.9392	18.88	-13.65	-17.54		-17.54
114	1.03	21.7852	19.48	-13.65	-17.54		-17.54

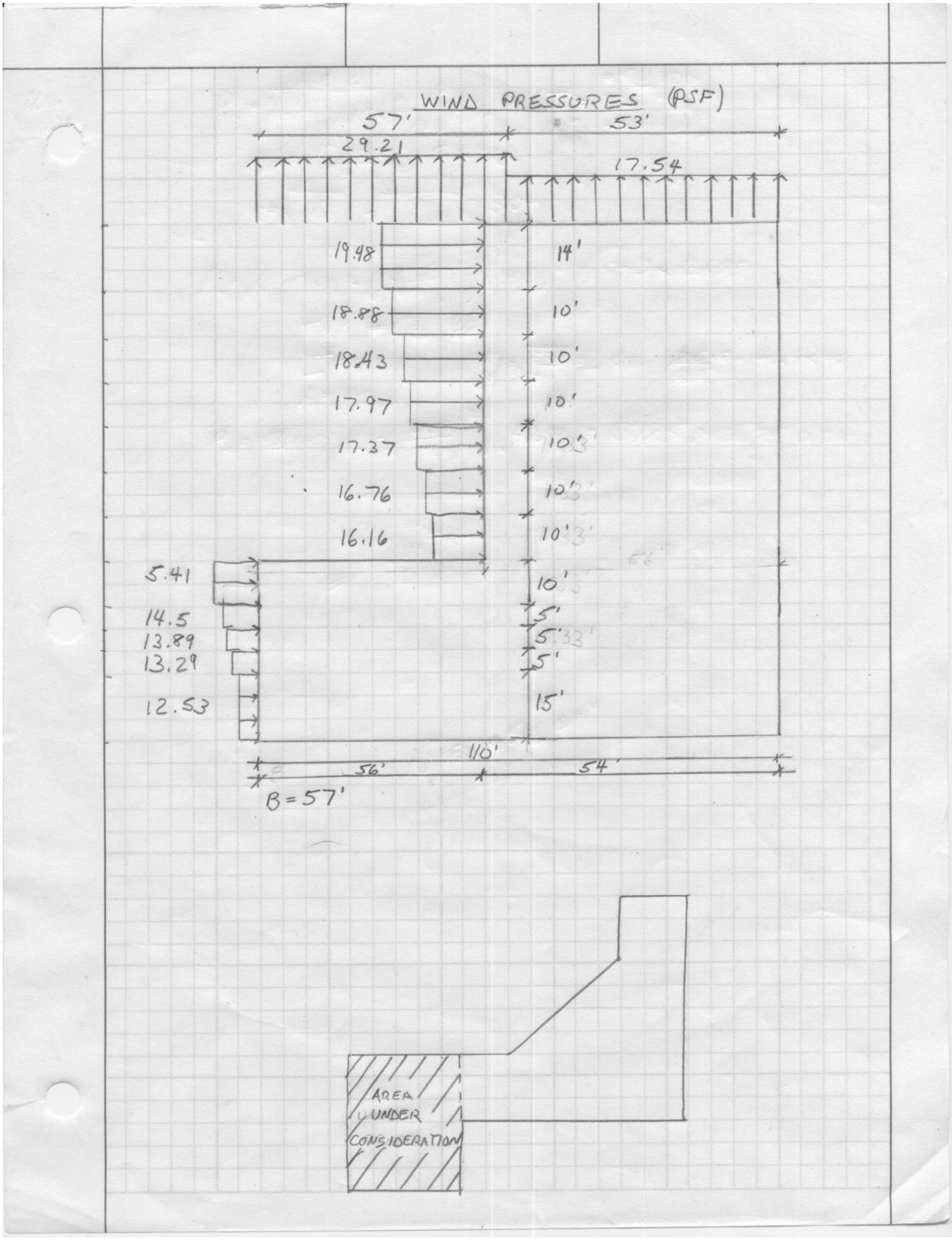

WIND PRESSURES (PSF)
29.21
17.54
19.48
18.88
18.43
17.97
17.37
16.76
16.16
15.41
14.5
13.89
13.29
12.53
14'
10'
10'
10'
10'
10'
10'
10'
5'
5'
5'
15'
12.66'
12.66
11.33'
11.33'
11.33'
11.33'
15.33'
14'
13.65
113'
165'
B = 181'
AREA
UNDER
CONSIDERATION

WIND PRESSURES (lbs)

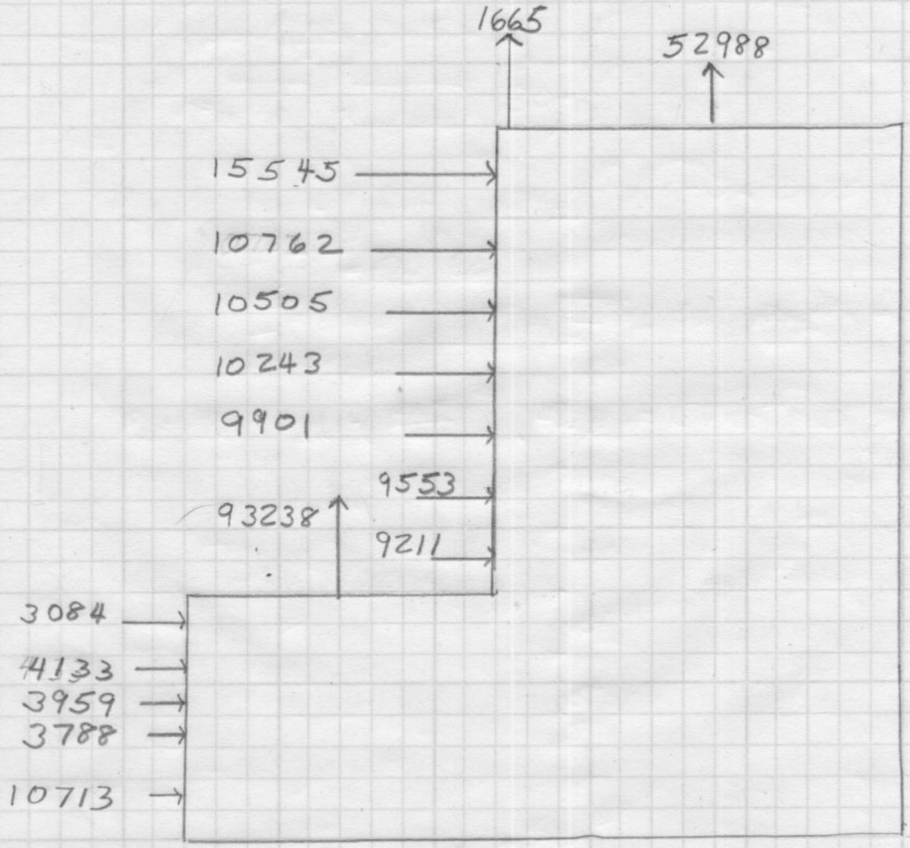


BASE SHEAR (V) = 619259 = 619.26 Kips

OVERTURNING MOMENT (M) = 34(7.5) + 12(17.5) + 12.57(22.5) + 13123(27.5') + 27.9(35') + 29.25(45') + 30.34(55') + 31.44(65') + 32.52(75') + 33.36(85') + 34.17(95') + 49.36(107') + 602.7(108') + 406.37(64') + 279.18(57')
= 130179 ft-kips


WIND PRESSURES (PSF)
57'
53'
29.21
17.54
19.48
14'
18.88
10'
18.43
10'
17.97
10'
17.37
10'
16.76
10'
16.16
10'
5.41
10'
14.5
5'
13.89
5'
13.29
5'
12.53
15'
110'
56'
54'
B=57'
AREA
UNDER
CONSIDERATION

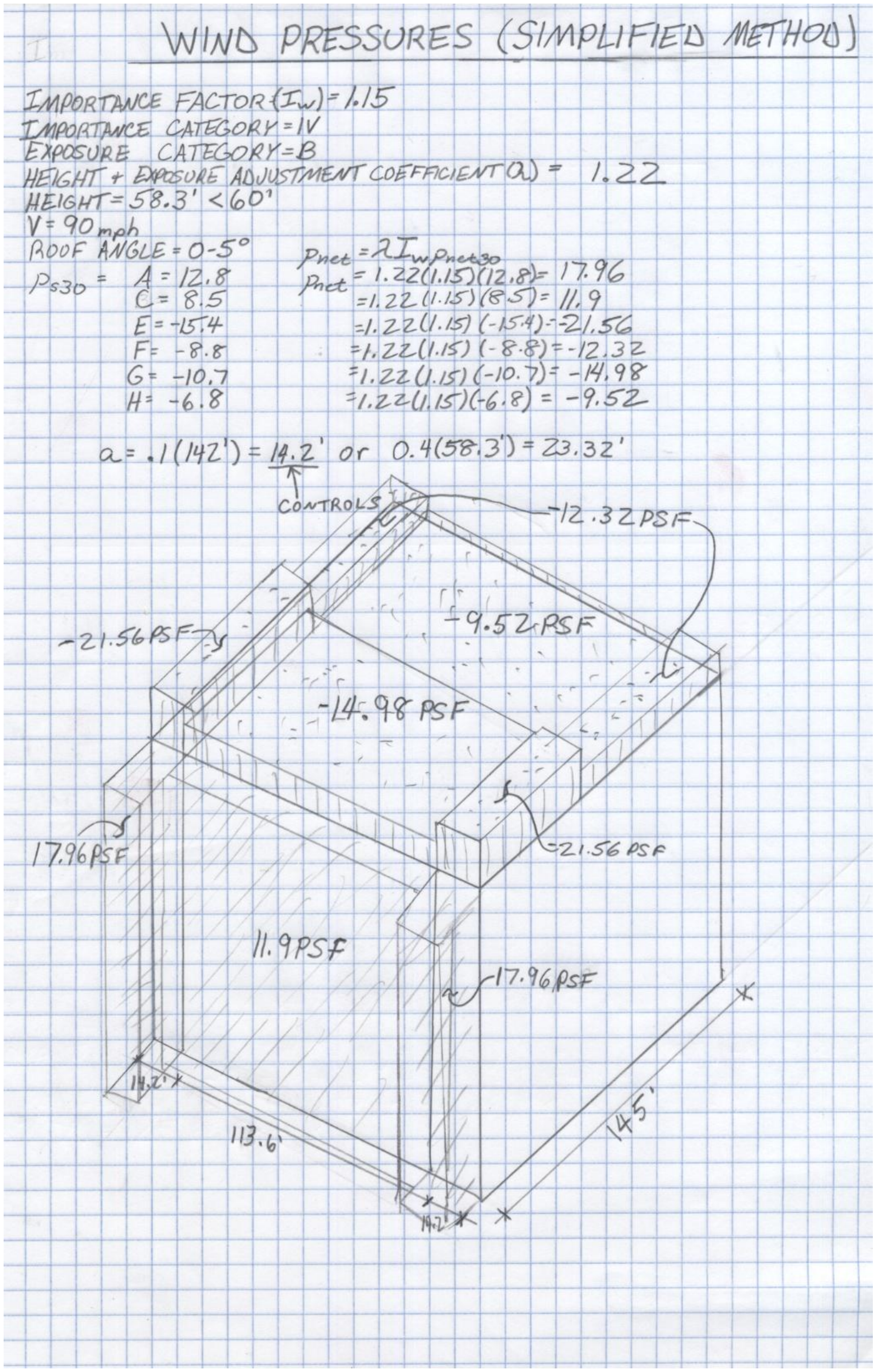
WIND PRESSURES (lbs)



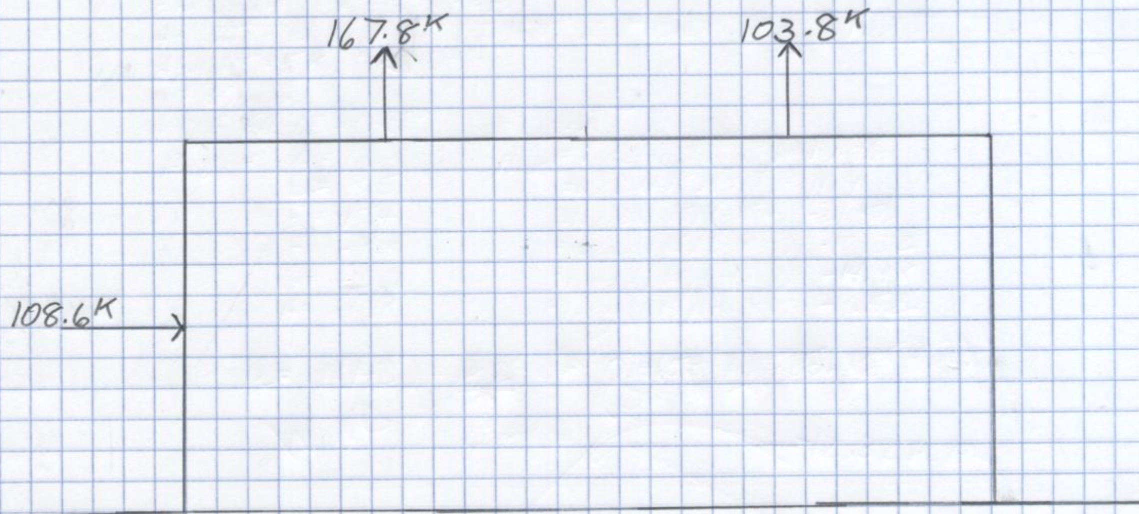
BASE SHEAR (V) = 101.38 KIPS

OVERTURNING MOMENT (M) = 53(26.5) + 16.7(53.5)
+ 15.55(107) + 10.76(95) + 10.51(85) + 10.24(75)
+ 9.9(65) + 9.55(55) + 9.21(45) + 3.08(35)
+ 4.13(27.5) + 3.96(22.5) + 3.79(17.5) + 10.71(7.5)
+ 93.24(82) = 16331 ft-kips

When computing the wind pressures on the shorter conference wing, the simplified method was used. This was done because this portion of the building met the simplified methods criterion and was less than 60 feet tall.



WIND PRESSURES (SIMPLIFIED METHOD) CONT.



$11.9\,PSF(58.3)(113.6') = 78812.3\,lbs = 78.81K$

$2(17.96\,PSF)(58.3')(14.2') = 29736.7\,lbs = 29.74K$

$78.81 + 29.74 = 108.6^K$

$-14.98\,PSF(113.6')\left(\frac{145'}{2}\right) = -123375.3\,lbs = -123.4^K$

$2(-21.56\,PSF)(14.2')\left(\frac{145}{2}\right) = -44392\,lbs = -44.4^K$

$-123.4 - 44.4 = -167.8^K$

$-9.52\,PSF(113.6')\left(\frac{145'}{2}\right) = -78406.7\,lbs = -78.4^K$

$2(-12.32\,PSF)(14.2')\left(\frac{145}{2}\right) = -25366.9\,lbs = -25.4^K$

$-78.4 - 25.4 = -103.8^K$

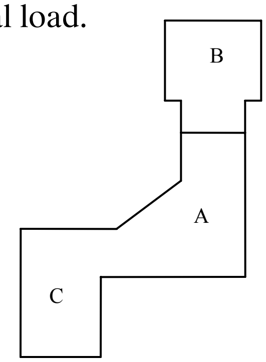
BASE SHEAR $(V) = 108.6^K$

OVERTURNING MOMENT $(M) = 108.6^K\left(\frac{58.3'}{2}\right) + 167.8^K\left(\frac{72.5'}{2} + 72.5'\right) + 103.8^K\left(\frac{72.5}{2}\right)$

$= 25177\,ft\text{-}K$

Seismic Loading

The following are the new seismic loads for the post-tensioned design of the Christiana Hospital. As you can see the loads for the main tower have been decreased do to the lighter floors created from the smaller amount of concrete required for the post-tensioned system and the smaller floor areas created by sectioning the main tower into two structures. On the other hand, the loads for the conference wing have increased greatly due to the inherently heavier concrete design over its original steel design. In all the structures the seismic loading in the controlling lateral load.



Seismic Use Group	Importance Factor	Site Class	S_{MS}	S_{M1}	S_{DS}	S_{D1}
III	1.5	D (Stiff Soil)	0.468	0.192	0.312	0.128

Tower (Area A)

$R = 5$ $C_s = 0.0589$ $k = 1.08$

$C_d = 4.5$ $T = 0.651$

Level	Height (ft)	w_x (k)	$h_x^k w_x$	C_{vx}	F_x (k)	M_x (ft-k)
B	0	0	0	0	0	0
1	14	4397.36	76034.431	0.0248	54.3917	761.484
2	29.33	4186.638	160902.37	0.05248	115.103	3375.96
3	40.66	4400.236	240644.94	0.07849	172.147	6999.49
4	52	4641.76	331105.42	0.10799	236.858	12316.6
5	63.33	4920.478	434255.71	0.14163	310.648	19673.3
6	74.66	5199.196	548114.4	0.17877	392.097	29274
7	87.33	5510.878	688140.84	0.22444	492.266	42989.6
8	100	3582.08	517768.09	0.16887	370.389	37038.9
R	118	400	69134.139	0.02255	49.4556	5835.76
∑		37238.626	3066100.3			

Base Shear: V (kips) = 2193.355071

Overturning Moment: M (ft-kips) = 158265.1089

Tower (Concrete Area C)

$R = 5$ $C_s = 0.0589$ $k = 1.08$

$C_d = 4.5$ $T = 0.651$

Level	Height (ft)	w_x (k)	$h_x^k w_x$	C_{vx}	F_x (k)	M_x (ft-k)
B	0	0	0	0	0	0
1	14	1006	17394.673	0.02743	14.3866	201.412
2	29.33	1902	73098.344	0.11528	60.4573	1773.21
3	40.66	1591	87010.356	0.13722	71.9635	2926.03
4	52	1506	107425.8	0.16941	88.8484	4620.12
5	63.33	647	57100.844	0.09005	47.2263	2990.84
6	74.66	665	70106.239	0.11056	57.9826	4328.98
7	87.33	665	83038.249	0.13095	68.6783	5997.67
8	100	722	104360.75	0.16458	86.3134	8631.34
R	118	200	34567.07	0.05451	28.5893	3373.54
∑		8904	634102.32			

Base Shear: V (kips) = 524.4456
Overturning Moment: M (ft-kips) = 34843.14998

Conference Center (Area B Post-Tensioned)

$R = 5$ $C_s = 0.0384$ $k = 1$

$C_d = 4.5$ $T = 0.271$

Level	Height (ft)	w_x (k)	$h_x^k w_x$	C_{vx}	F_x (k)	M_x (ft-k)
B	0	0	0	0	0	0
1	32	7608	243456	0.33975	268.714	8598.86
2	29.33	7568	221969.44	0.30976	244.999	7185.81
R	46.33	5421	251154.93	0.35049	277.212	12843.2
∑		20597	716580.37			

Base Shear: V (kips) = 790.9248
Overturning Moment: M (ft-kips) = 28627.8952

Shear Wall Design

Main Tower:

As stated earlier the purpose of my lateral design is to attempt to reduce the number or size of shear walls in order to decrease the project's cost and/or schedule. The approach taken to try and achieve this goal was by minimizing the lateral load on the structure by sectioning the tower at column line 65 with an expansion joint. The theory behind this idea was that by creating two independent and more symmetrical structures the center of mass and the center of rigidity would move closer to one another and decrease the forces in the shear walls due to torsional effects.

In my analysis of the shear walls the loads had first been determined on each wall before the structure was separated and then recomputed for the separated structures using ETABS. The results found were actually different than what I had been trying to achieve. Because the controlling lateral force was seismic, the equivalent lateral forces on each floor of the building were a function of the buildings mass. In my design the mass of each floor was lighter due to two separate factors. The first was the lighter post-tensioned slabs which, although were a ½" thicker, required no drop panels at the columns. The second factor was that due to the expansion joint the floor area required to be restrained was less. With the building mass being reduced the equivalent lateral load on the building was also reduced but in the end the load on each individual wall was increased.

This increased load was caused because the eccentricities were actually increased (see Figure 4 below) and, although the equivalent lateral forces were decreased, there were now less shear wall in place to resist the load. The combination of all these factors resulted in larger forces in the shear walls and ultimately forced me to add a total of 7 walls, 3 in Area A and 4 in Area B. The forces in each wall and their resulting deflections can be seen below.

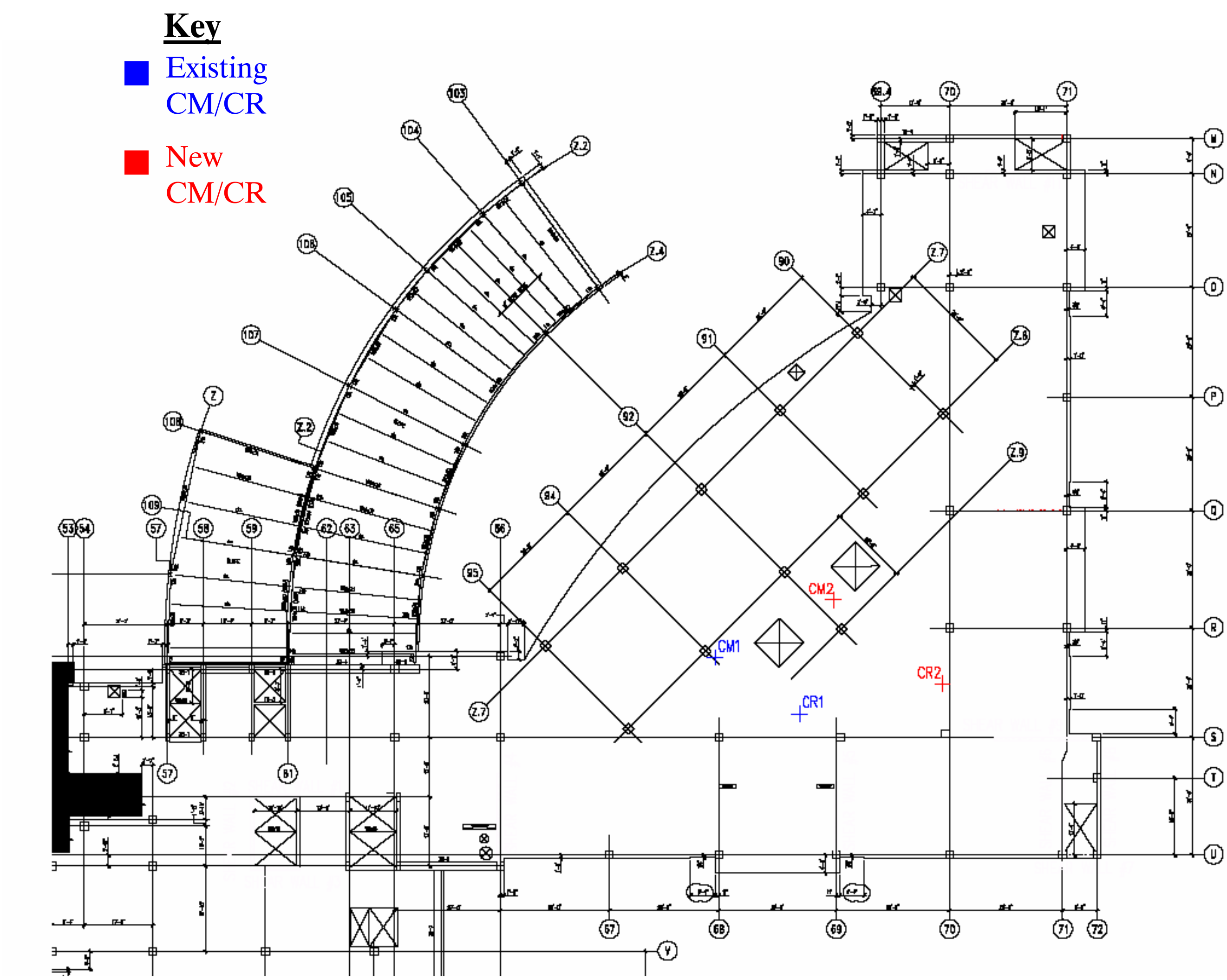


Figure 4 Locations of Center of Mass and Center of Rigidity

Shear Wall Forces					
		Origional Design		My Design (With Expansion Joint)	
Wall #	Story	V (k)	M (ft-k)	V (k)	M (ft-k)
1	ROOF	32.03	426.036	-53.84	-969.052
	EIGHTH STORY	32.03	579.452	81.88	-969.052
	SEVENTH STORY	32.03	763.681	191.1	2488.788
	SIXTH STORY	164.89	2029.899	284.95	5718.17
	FIFTH STORY	169.2	2968.093	367.71	9885.573
	FOURTH STORY	192.92	4094.922	353.38	13890.506
	THIRD STORY	209.03	5305.472	345.6	17807.297
	SECOND STORY	229.41	7317.371	341.66	23046.106
	FIRST FLOOR	295.72	9534.092	363.29	28132.115
2	ROOF	80.04	838.906	28.65	515.65
	EIGHTH STORY	80.04	833.181	114.32	1963.741
	SEVENTH STORY	80.04	986.541	182.79	4279.142
	SIXTH STORY	291.29	2559.436	240.9	7009.398
	FIFTH STORY	332.05	3529.071	287.04	10262.569
	FOURTH STORY	394.53	4696.832	375.93	14523.062
	THIRD STORY	443.48	5939.244	448.64	19607.683
	SECOND STORY	456.38	8122.908	504.21	27338.957
	FIRST FLOOR	404.19	9049.62	537.96	34870.4
3	ROOF	-12.5	149.955	82.33	1481.993
	EIGHTH STORY	30.32	566.247	32.32	1891.401
	SEVENTH STORY	30.32	749.492	-7.32	1891.401
	SIXTH STORY	162.24	2013.333	-45.46	1798.705
	FIFTH STORY	168.29	2966.602	-78.46	1283.441
	FOURTH STORY	196.1	4119.22	12.22	727.069
	THIRD STORY	216.8	5358.93	103.48	1815.459
	SECOND STORY	241.39	7414.164	163.51	4322.631
	FIRST FLOOR	302.44	9566.676	172.55	6738.285
4	ROOF	-47.05	-846.864	20.16	362.82
	EIGHTH FLOOR	100.39	-846.864	148.76	2247.115
	SEVENTH FLOOR	256.34	3671.634	314.8	6234.543
	SIXTH FLOOR	221.44	6181.244	449.81	11332.346
	FIFTH FLOOR	295.62	9531.597	557.19	17647.181
	FOURTH FLOOR	356.81	13575.475	642.06	24923.839
	THIRD FLOOR	405.91	18175.793	726.1	33152.952
	SECOND FLOOR	458.14	25200.607	764.3	44872.184
	FIRST FLOOR	486.43	32010.63	718.19	54926.843
5	ROOF	-8.79	-158.246	4.46	80.22
	EIGHTH FLOOR	75.75	801.254	75.58	1037.542
	SEVENTH FLOOR	167.33	2920.764	170.6	3198.524
	SIXTH FLOOR	193.35	5112.008	243.95	5963.327
	FIFTH FLOOR	248.88	7932.635	301.76	9383.289
	FOURTH FLOOR	286.13	11175.445	345.71	13301.371
	THIRD FLOOR	313.5	14728.498	390.42	17726.146
	SECOND FLOOR	339.56	19935.039	412.73	24054.663
	FIRST FLOOR	407.04	25633.533	481.4	30794.308

6	ROOF	13.3	239.369	12.13	218.251
	EIGHTH FLOOR	112.2	1660.613	97.3	1450.773
	SEVENTH FLOOR	217.51	4415.792	212.57	4143.366
	SIXTH FLOOR	302.9	7848.656	304.1	7589.882
	FIFTH FLOOR	376.38	12114.293	376.55	11857.488
	FOURTH FLOOR	424.45	16924.754	430.46	16735.995
	THIRD FLOOR	460.29	22141.334	482.29	22201.948
	SECOND FLOOR	488.67	29634.324	509.59	30015.634
	FIRST FLOOR	503.58	36684.419	526.69	37389.298
7	ROOF	-2.83	-50.904	3.73	67.085
	EIGHTH FLOOR	2.73	-50.904	9.37	185.749
	SEVENTH FLOOR	18.14	213.446	17.11	402.447
	SIXTH FLOOR	2.93	246.67	20.57	635.54
	FIFTH FLOOR	17.7	447.278	25.38	923.135
	FOURTH FLOOR	13.66	602.084	23.27	1186.852
	THIRD FLOOR	29.4	935.268	48.36	1734.94
	SECOND FLOOR	-0.15	932.906	-7.19	1624.652
	FIRST FLOOR	78.19	2027.534	139.57	3578.653
8	ROOF	16.26	292.687	12.71	228.81
	EIGHTH FLOOR	115.33	1753.525	98.19	1472.518
	SEVENTH FLOOR	220.59	4547.643	214.13	4184.78
	SIXTH FLOOR	313.33	8098.765	306.33	7656.477
	FIFTH FLOOR	388.14	12497.667	379.33	11955.595
	FOURTH FLOOR	436.8	17448.017	433.47	16868.28
	THIRD FLOOR	473.06	22809.379	485.25	22367.787
	SECOND FLOOR	501.06	30492.362	512.54	30226.805
	FIRST FLOOR	511.62	37655.105	526.27	37594.537
9	ROOF	29.95	539.062	299.18	5385.16
	EIGHTH FLOOR	237.76	3055.948	505.52	11788.46
	SEVENTH FLOOR	481.64	9156.766	765.53	21485.169
	SIXTH FLOOR	531.24	15177.47	980.45	32596.951
	FIFTH FLOOR	689.63	22993.273	1150.47	45635.579
	FOURTH FLOOR	807.26	32142.223	1288.06	60233.584
	THIRD FLOOR	888.96	42217.121	1372.2	75785.24
	SECOND FLOOR	948.89	56766.813	1464.05	98234.037
	FIRST FLOOR	746	67210.751	1223.71	115365.98
10	ROOF	-13.01	-234.097	79.44	1429.929
	EIGHTH FLOOR	76.94	740.481	153.15	3369.788
	SEVENTH FLOOR	171.27	2909.94	252.05	6562.412
	SIXTH FLOOR	183.04	4984.444	326.39	10261.535
	FIFTH FLOOR	245.57	7767.551	383.84	14611.769
	FOURTH FLOOR	287.1	11021.329	431.23	19499.002
	THIRD FLOOR	313.58	14575.19	455.51	24661.473
	SECOND FLOOR	348.57	19919.989	500.17	32330.675
	FIRST FLOOR	406.46	25610.367	624.63	41075.477
11	ROOF	-45.86	-825.56	109.93	1978.687
	EIGHTH FLOOR	-49.17	-1448.421	194.61	4443.763
	SEVENTH FLOOR	-51.96	-2106.644	320.23	8500.052
	SIXTH FLOOR	278.7	7210.832	419.6	13255.501
	FIFTH FLOOR	351.55	11195.112	497.97	18899.191
	FOURTH FLOOR	402.67	15758.677	551.97	25154.796
	THIRD FLOOR	439.56	20740.393	590.44	31846.473
	SECOND FLOOR	476.8	28051.298	624.6	41423.601
	FIRST FLOOR	484.67	34836.698	648.1	50496.998

I (Importance Factor) = 1.5 C_d = Amplification Factor

Area A (With Expansion Joint)					
Story	Δx	$\Delta x_{amplified}$	Δy	$\Delta y_{amplified}$	$\Delta_{allowable}$
ROOF	7.0164	21.0492	4.7282	14.1846	14.16
EIGHTH FLOOR	5.584	16.752	3.799	11.397	12
SEVENTH FLOOR	4.5857	13.7571	3.1449	9.4347	10.476
SIXTH FLOOR	3.6107	10.8321	2.4969	7.4907	8.952
FIFTH FLOOR	2.7769	8.3307	1.9342	5.8026	7.596
FOURTH FLOOR	2.0023	6.0069	1.4039	4.2117	6.24
THIRD FLOOR	1.3123	3.9369	0.9255	2.7765	4.884
SECOND FLOOR	0.7362	2.2086	0.5207	1.5621	3.528
FIRST FLOOR	0.1956	0.5868	0.1383	0.4149	1.68

Area C (With Expansion Joint)					
Story	Δx	$\Delta x_{amplified}$	Δy	$\Delta y_{amplified}$	$\Delta_{allowable}$
ROOF	8.0685	24.2055	6.4066	19.2198	14.16
EIGHTH STORY	6.4499	19.3497	5.1385	15.4155	12
SEVENTH STORY	5.318	15.954	4.2479	12.7437	10.476
SIXTH STORY	4.2082	12.6246	3.3709	10.1127	8.952
FIFTH STORY	3.8823	11.6469	2.652	7.956	7.596
FOURTH STORY	2.3585	7.0755	1.9015	5.7045	6.24
THIRD STORY	1.5507	4.6521	1.2574	3.7722	4.884
SECOND STORY	0.8663	2.5989	0.7095	2.1285	3.528
FIRST FLOOR	0.2204	0.6612	0.1875	0.5625	1.68

Original Design					
Story	Δx	$\Delta x_{amplified}$	Δy	$\Delta y_{amplified}$	$\Delta_{allowable}$
ROOF	3.9296	11.7888	3.6717	11.0151	14.16
EIGHTH FLOOR	3.1618	9.4854	3.4625	10.3875	12
SEVENTH FLOOR	2.6179	7.8537	2.863	8.589	10.476
SIXTH FLOOR	2.0778	6.2334	1.9389	5.8167	8.952
FIFTH FLOOR	1.6109	4.8327	1.5019	4.5057	7.596
FOURTH FLOOR	1.1708	3.5124	1.0905	3.2715	6.24
THIRD FLOOR	0.7731	2.3193	0.7195	2.1585	4.884
SECOND FLOOR	0.4364	1.3092	0.4058	1.2174	3.528
FIRST FLOOR	0.1164	0.3492	0.1087	0.3261	1.68

Continuing with my design I placed the shear walls in the areas indicated in Figure 5 below. The locations chosen were decided to be the most effective while not changing the architecture or layout of the building in any way. All the locations of the new shear walls fit within partition walls, stairwells, and elevator shafts. Loads, calculations, and final sizes and reinforcement for these shear walls can be reviewed in Appendix A.

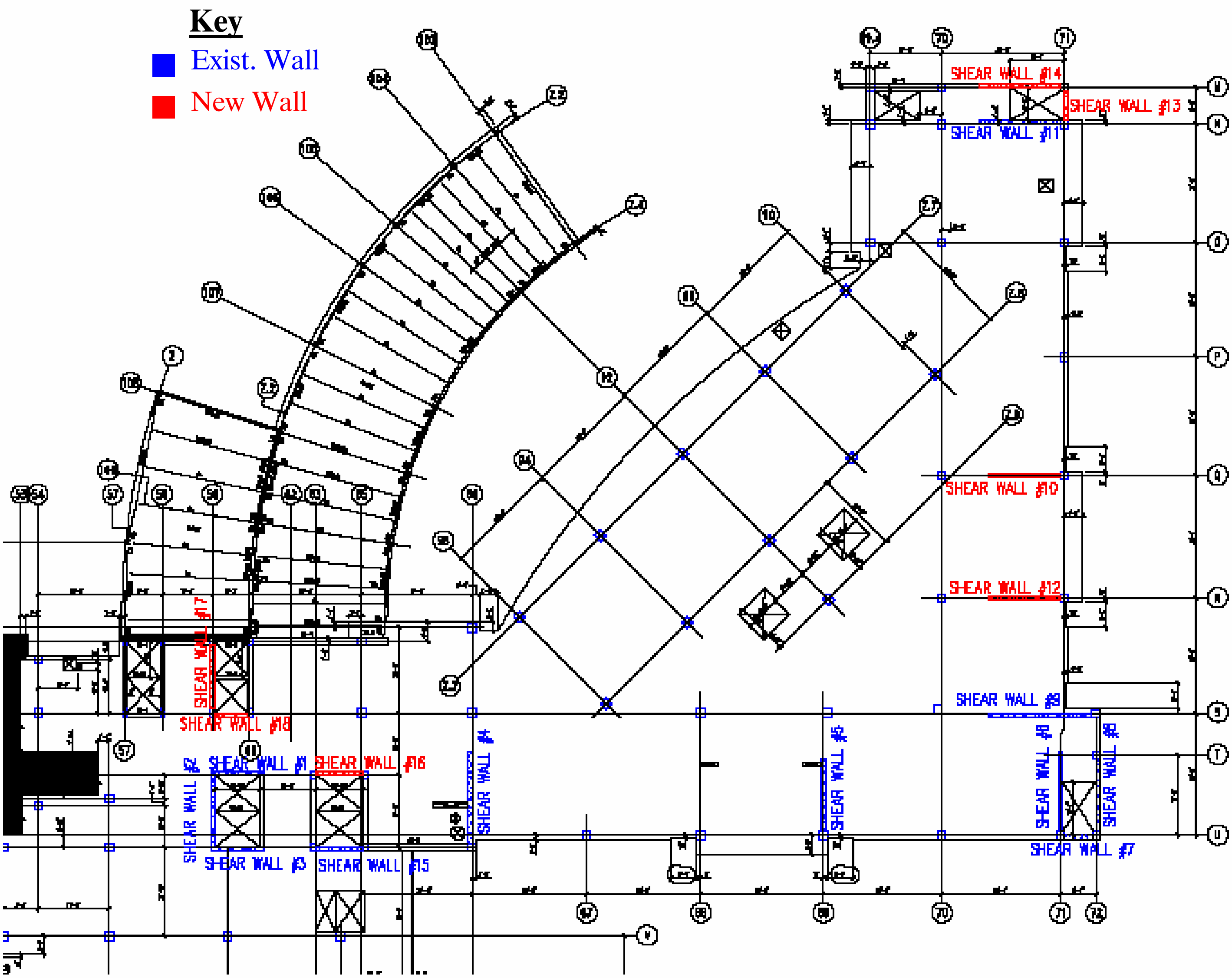


Figure 5 Main Tower Shear Wall Locations

Conference Wing:

Since the conference wing's floor system is being designed using post-tensioned concrete I am replacing all the concentrically braced frames with concrete shear walls. Now that the conference wing is concrete and much heavier than its original steel design the equivalent lateral forces generated from the seismic analysis are much higher. Even though these loads are much higher than the original loads the size of the shear walls is more than enough to restrain the building from lateral movement. As you can see the amplified deflection per ASCE7-02 9.5.2.8 at the top of the building was limited to 0.355" which is much less than the allowable 5.56". To review loads, calculations, and reinforcement for these walls see Appendix A.

Conference Wing Deflections					
Story	Δx	$\Delta x_{amplified}$	Δy	$\Delta y_{amplified}$	$\Delta_{allowable}$
THIRD STORY	0.1185	0.3555	0.0742	0.2226	5.5596
SECOND STORY	0.0689	0.2067	0.0457	0.1371	3.84
FIRST FLOOR	0.0205	0.0615	0.0142	0.0426	1.68
BASE	0	0	0	0	0

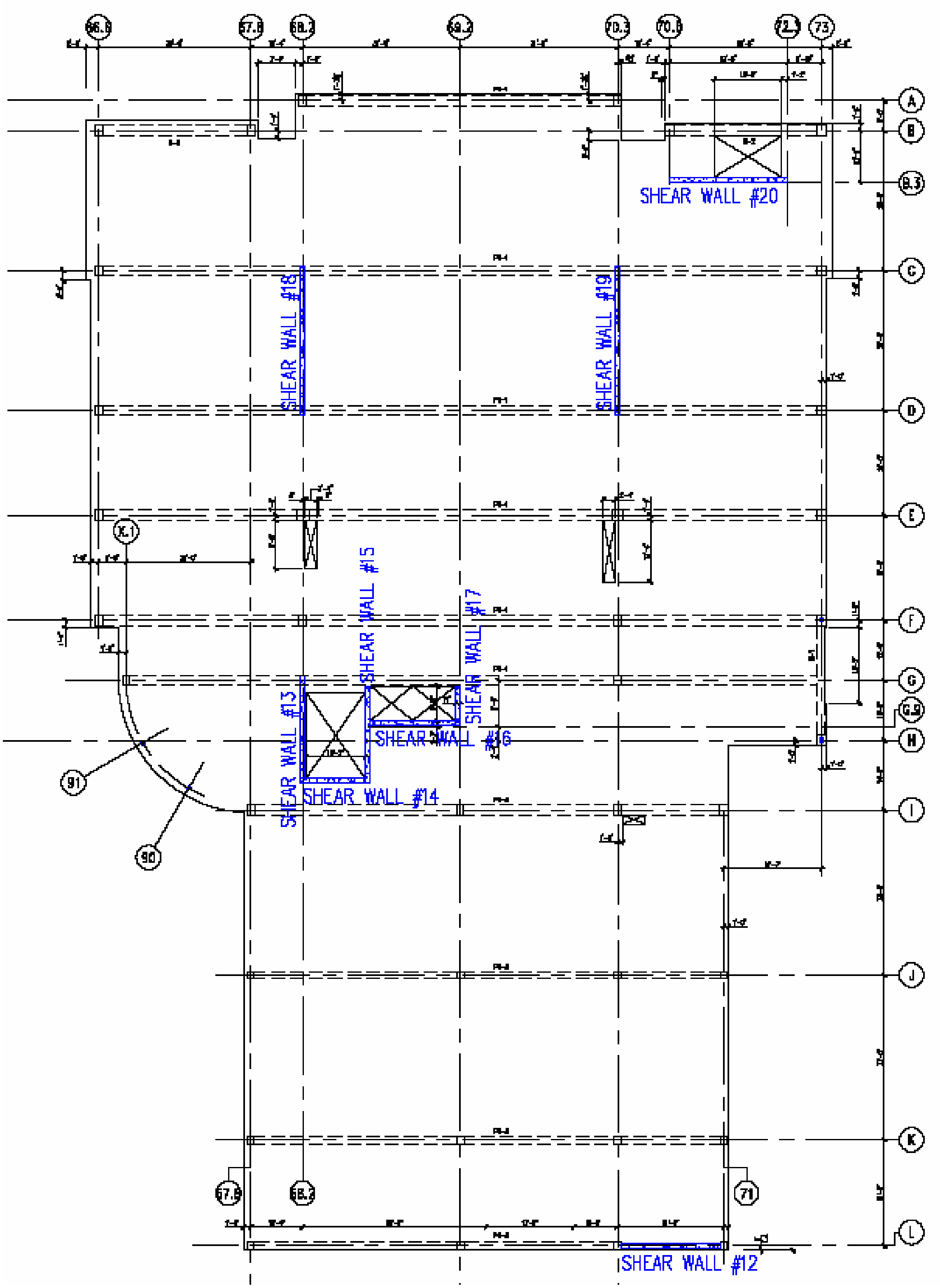


Figure 6 Conference Wing Shear Wall Locations

Post-Tensioned Design

In the designs of all slabs and beams the following equations, code criteria, and material properties were used:

- Tendons – ½” Φ – 270 ksi strands (ASTM A461) – $A_{ps} = 0.153in^2$
- Slab Thickness – 10”
- ACI code provision 18.3.3 - Class U (Uncracked Concrete): $f_t \leq 7.5\sqrt{f'_c}$
- ACI equation 18-5 – Ultimate Tendon Stress
  $f_{su} = f_{se} + (1.0*f'_c)/(100\rho_p) + 10ksi$
- Effective Tendon Stress after losses = $f_{se} = 175$ ksi
- $\rho_p = A_{ps}/bd$
- ACI code provisions for extreme fiber stresses in concrete at transfer:
  (18.4.1a) Compression: $0.6f'_{ci}$
  (18.4.1b) Tension: $3\sqrt{f'_{ci}}$
  (18.4.1c) Tension at end of simply supported member: $6\sqrt{f'_{ci}}$
- ACI equation 11-12 – Punching Shear Capacity
  $Vcw = b'd(3.5\sqrt{f'_c}s+0.3f_{pc})$

Two-Way Slab (Main Tower):

When designing all slabs hand calculations were performed (Appendix B) along with the use of the computer program RAM Concept. When planning tendon layouts the practice of uniformly spacing tendons in one direction and banding tendons in the orthogonal direction centered on the column lines was used.

Slab	f'_c (psi)
First Floor Slab	5000
Typical Slab	4000

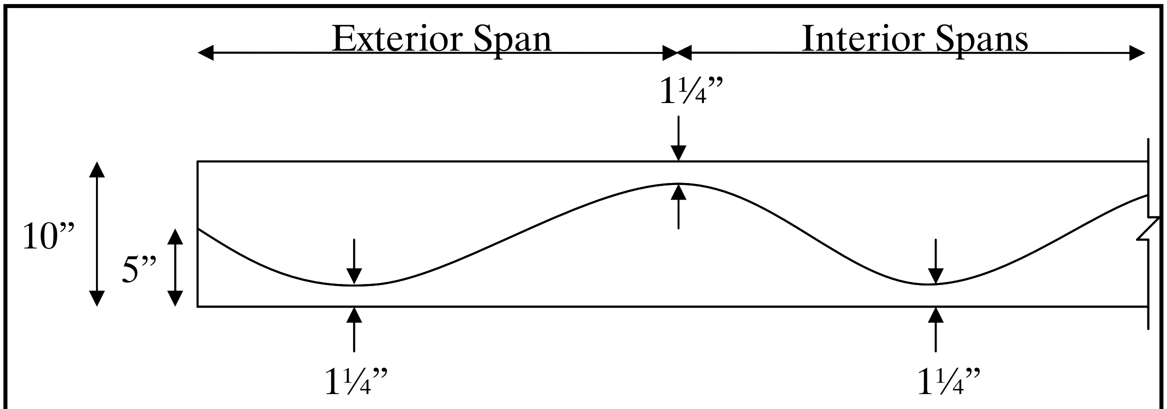


Figure 4 Slab Detail – Tendon Profile

First Floor Slab:
Uniformly Spaced Tendon Plan

The first floor slab was the first to be designed being the most critical having a Live Load = 100psf and a Superimposed Dead Load = 15 psf. The final design required a 10" slab with tendons in bundles of 4 spaced at 6' o.c. Figure 7 below shows the tendon layout for the uniformly spaced tendons in the first floor slab. The separation between the two structures at the 1" expansion has been exaggerated for visual clarity.

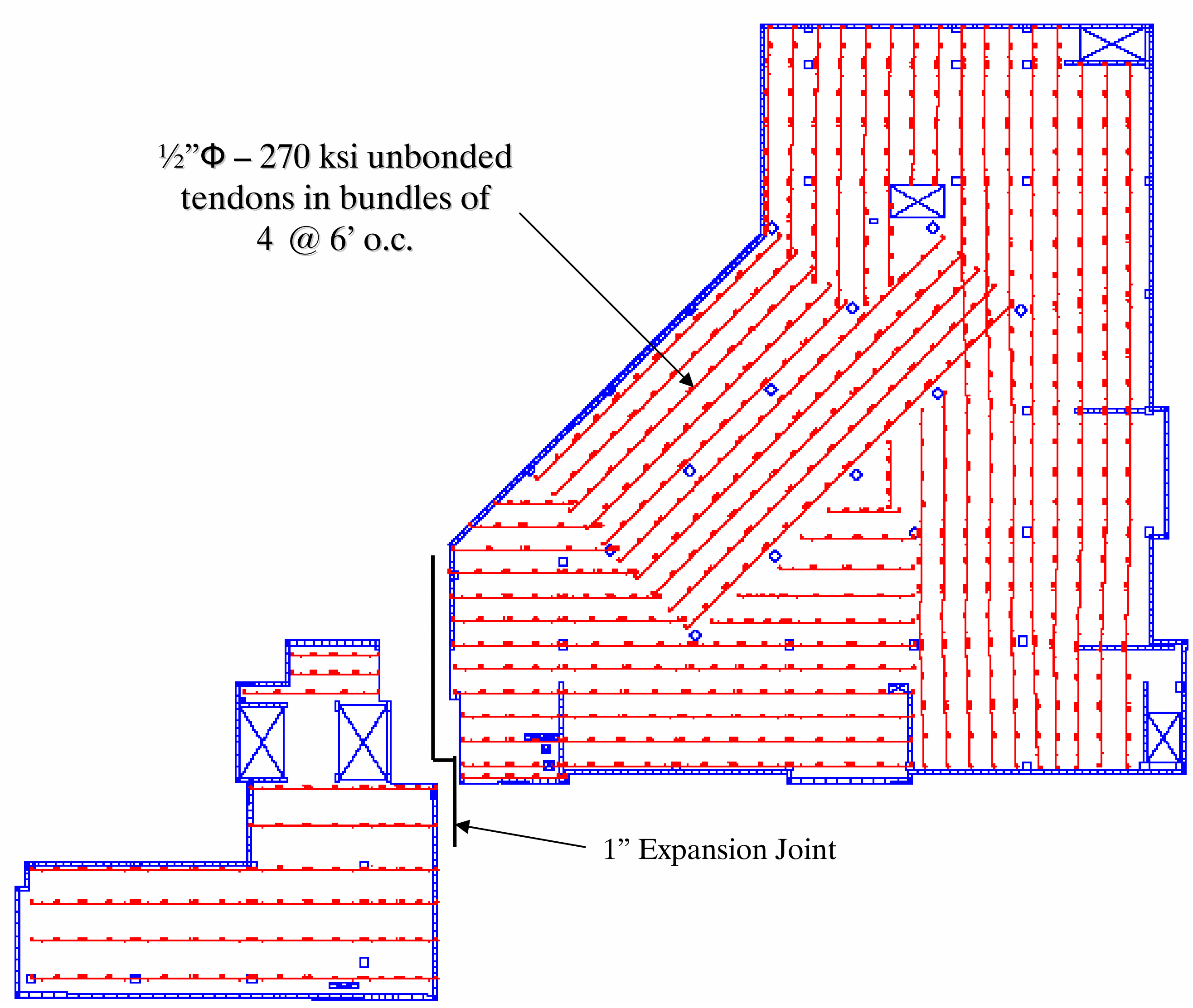


Figure 7 First Floor Uniformly Spaced Tendon Layout

First Floor Slab:
Banded Tendon Plan

Figure 8 below shows the banded tendon layout for the first floor. The amount of tendons banded together varies and is denoted by color. As you can see due to the column layout it was difficult to run tendons in strait paths. Tendons which required an in plane curve of more than 6:1 were stopped in the slab's neutral axis and a new line of tendons was started next to them in the desired direction. The 1" expansion joint between the two separated structures has been exaggerated for clarity.

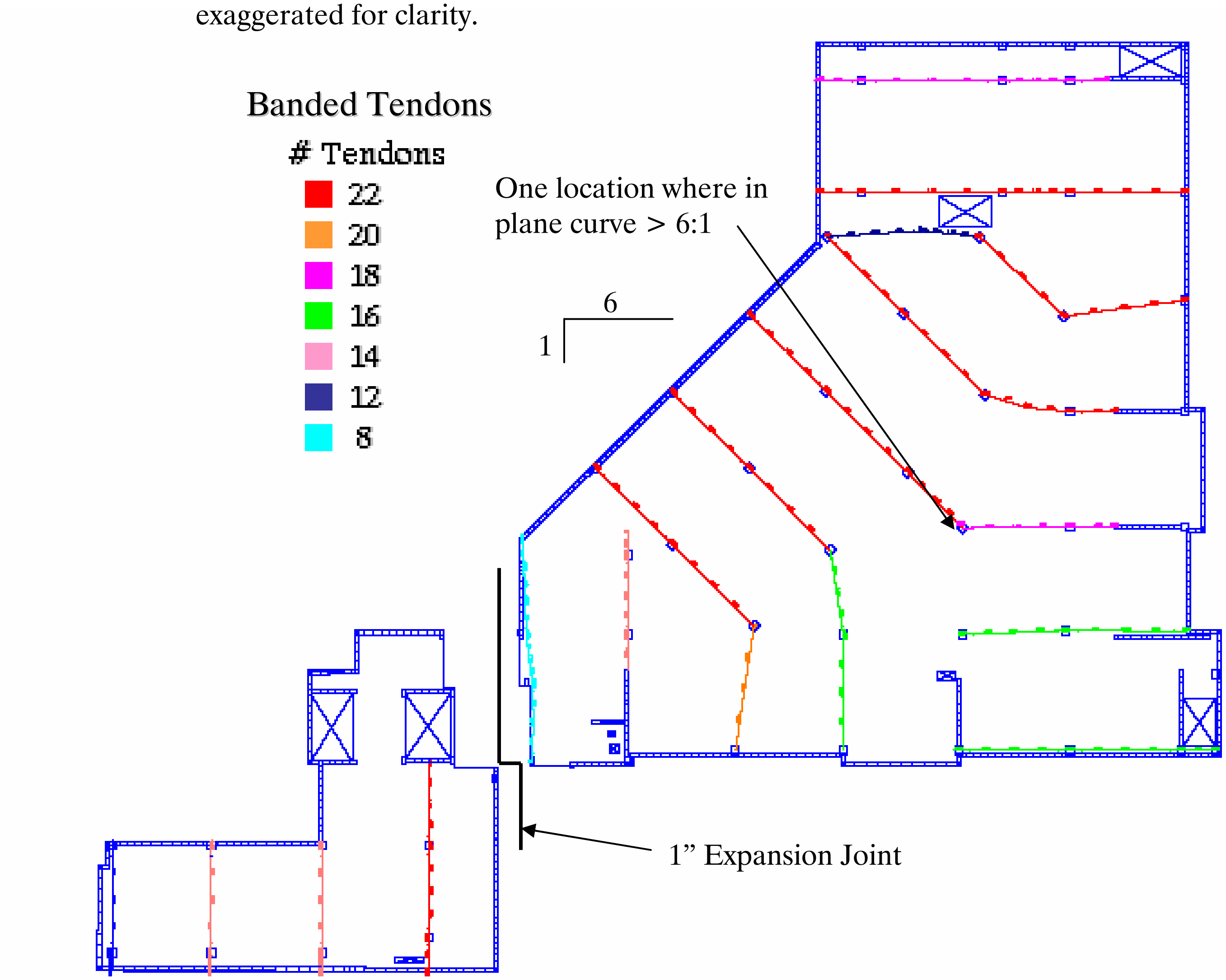


Figure 8 First Floor Banded Tendon Layout

First Floor Slab:
Sustained Service Load Deflection Plan

Figure 9 below shows the sustained service load deflection plan for the first floor. The largest spans in the hospital's floor plan are 30'. Adhering to a deflection criterion of L/360, this gives an allowable deflection (Δ_a) = 30'/360 = 1". In the plan it can be seen that the max sustained service load deflection for this design is only 0.411" (L/876) which is much less than the required and therefore satisfies the deflection criterion.

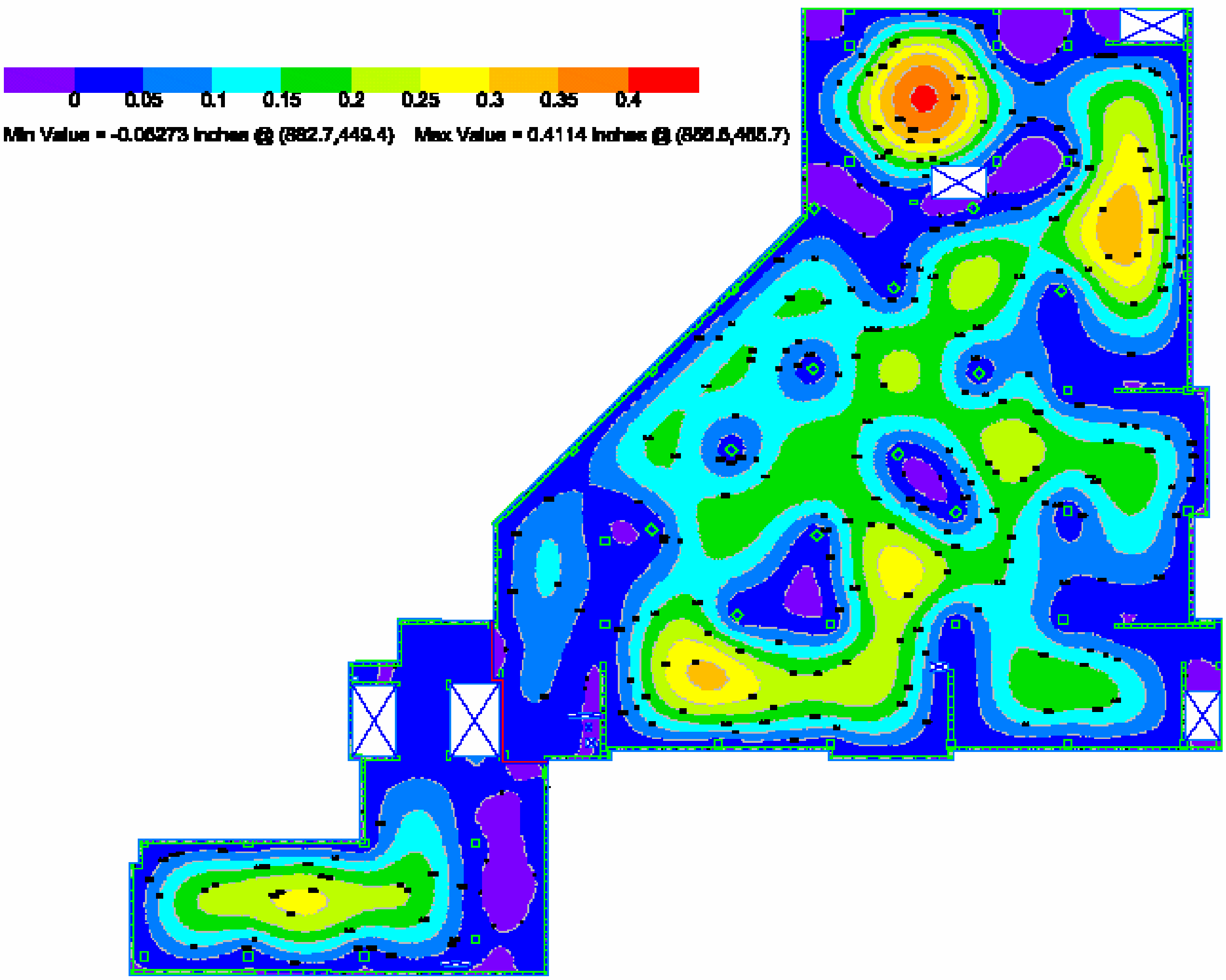


Figure 9 First Floor Sustained Service Load Deflection Plan

Typical Floor Slab (Floors 4 through 7): Uniformly Spaced Tendon Plan

The typical floor slab was the second slab to be designed. This floor carries a Live Load = 80psf and a Superimposed Dead Load = 15psf. The final design required a 10" slab with tendons in bundles of 3 spaced at 3¾' o.c. More tendons where required per foot of slab width than the first floor due to the fact that a lower concrete strength of 4000psi was used for the typical floors. Figure 10 below shows the uniformly spaced tendon layout for the typical floors 4 through 7. The 1" expansion joint has again been exaggerated for visual clarity.

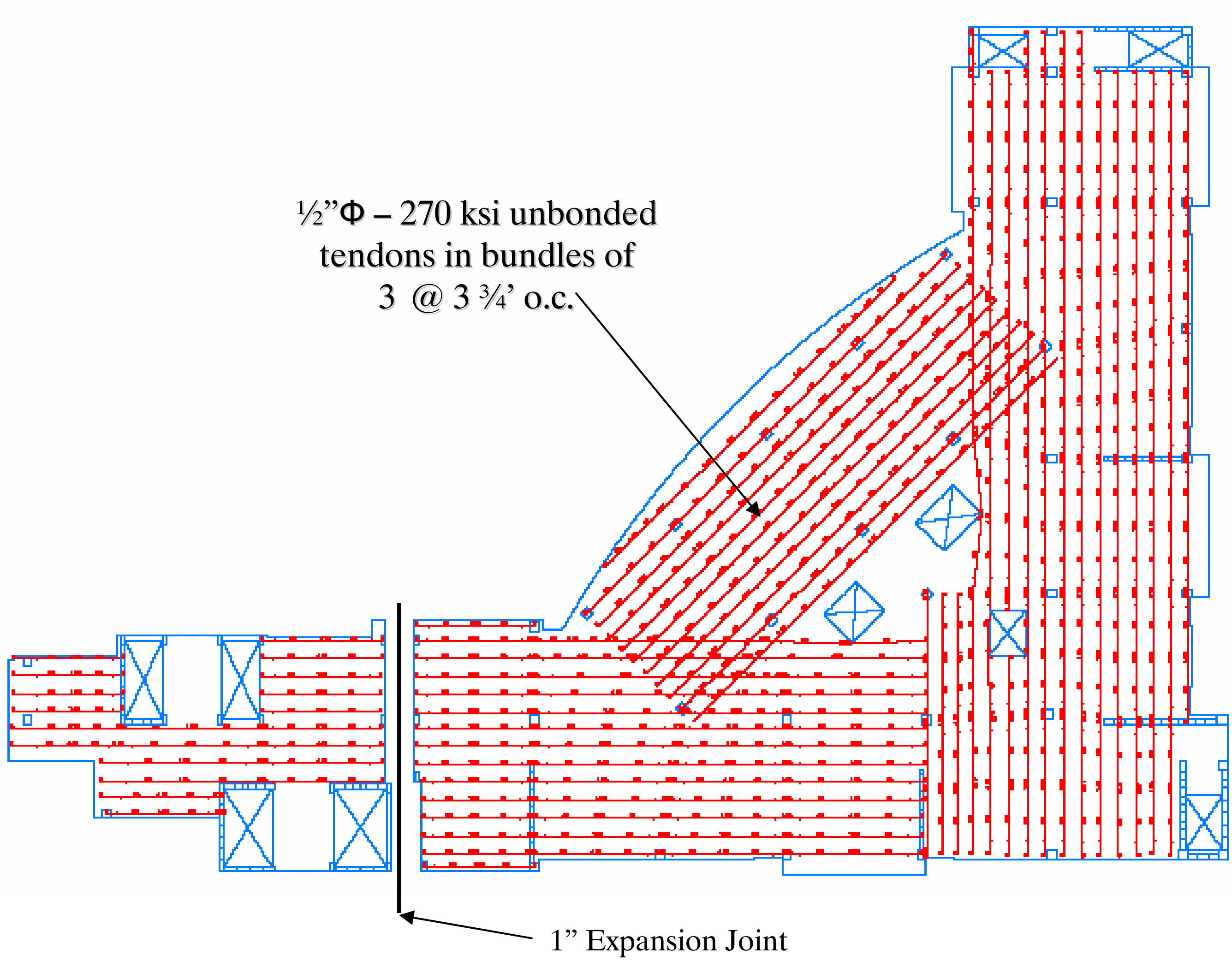


Figure 10 Typical Floor Uniformly Spaced Tendon Layout

Typical Floor Slab (Floors 4 through 7): Banded Tendon Plan

Figure 11 below shows the banded tendon layout for the typical floors 4 through 7. The amount of tendons banded together varies and is denoted by color. As you can see due to the column layout it was difficult to run tendons in strait paths. Tendons which required an in plane curve of more than 6:1 were stopped in the slab's neutral axis and a new line of tendons was started next to them in the desired direction. The 1" expansion joint between the two separated structures has been exaggerated for clarity.

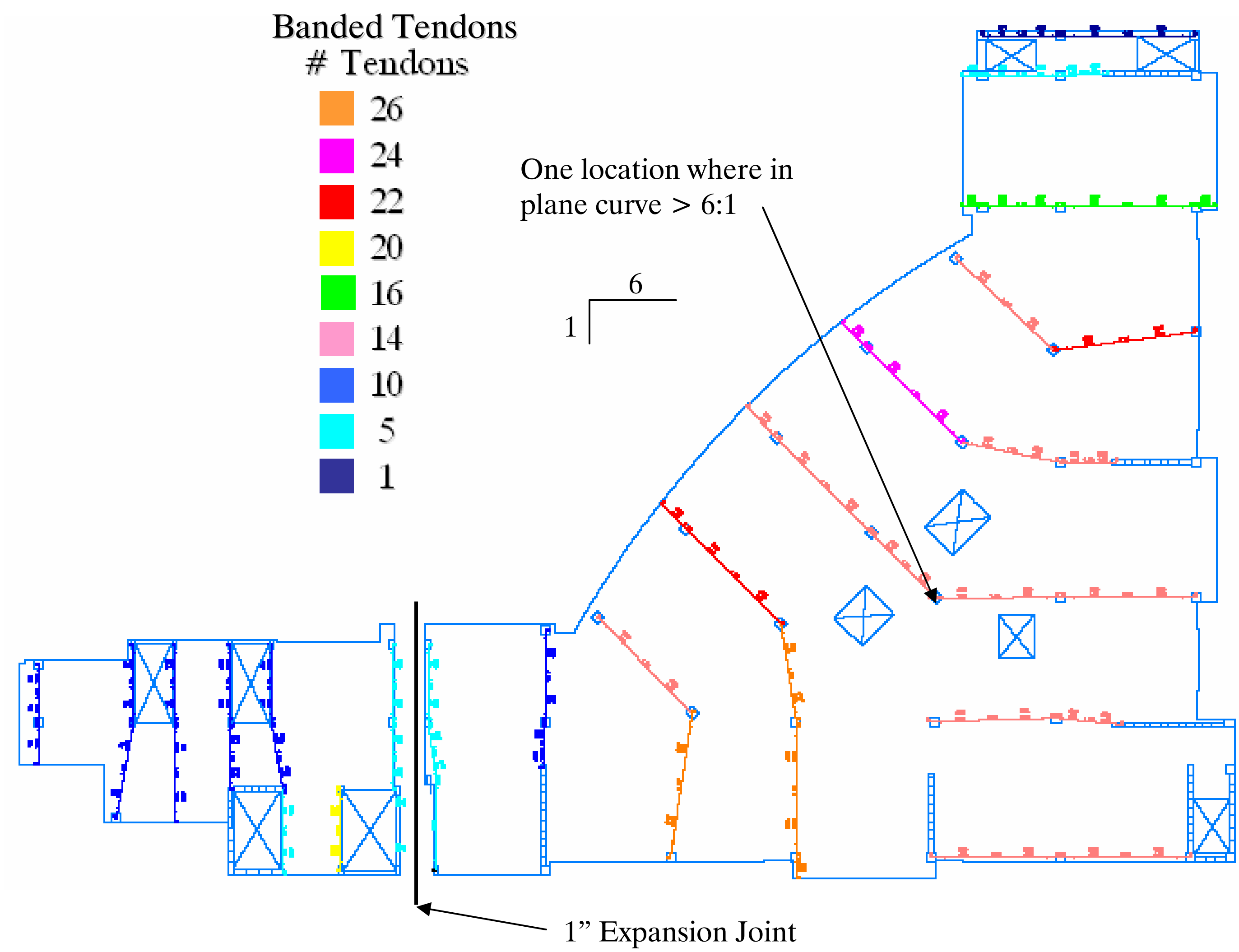


Figure 11 Typical Floor Banded Tendon Layout

Typical Floor Slab:
Sustained Service Load Deflection Plan

Figure 12 below shows the sustained service load deflection plan for the typical floors (floors 4 through 7). The largest spans in the hospital's floor plan are 30'. Adhering to a deflection criterion of L/360, this gives an allowable deflection (Δ_a) = 30'/360 = 1". In the plan it can be seen that the max sustained service load deflection for this design is only 0.355" (L/1014) which is much less than the required and therefore satisfies the deflection criterion.

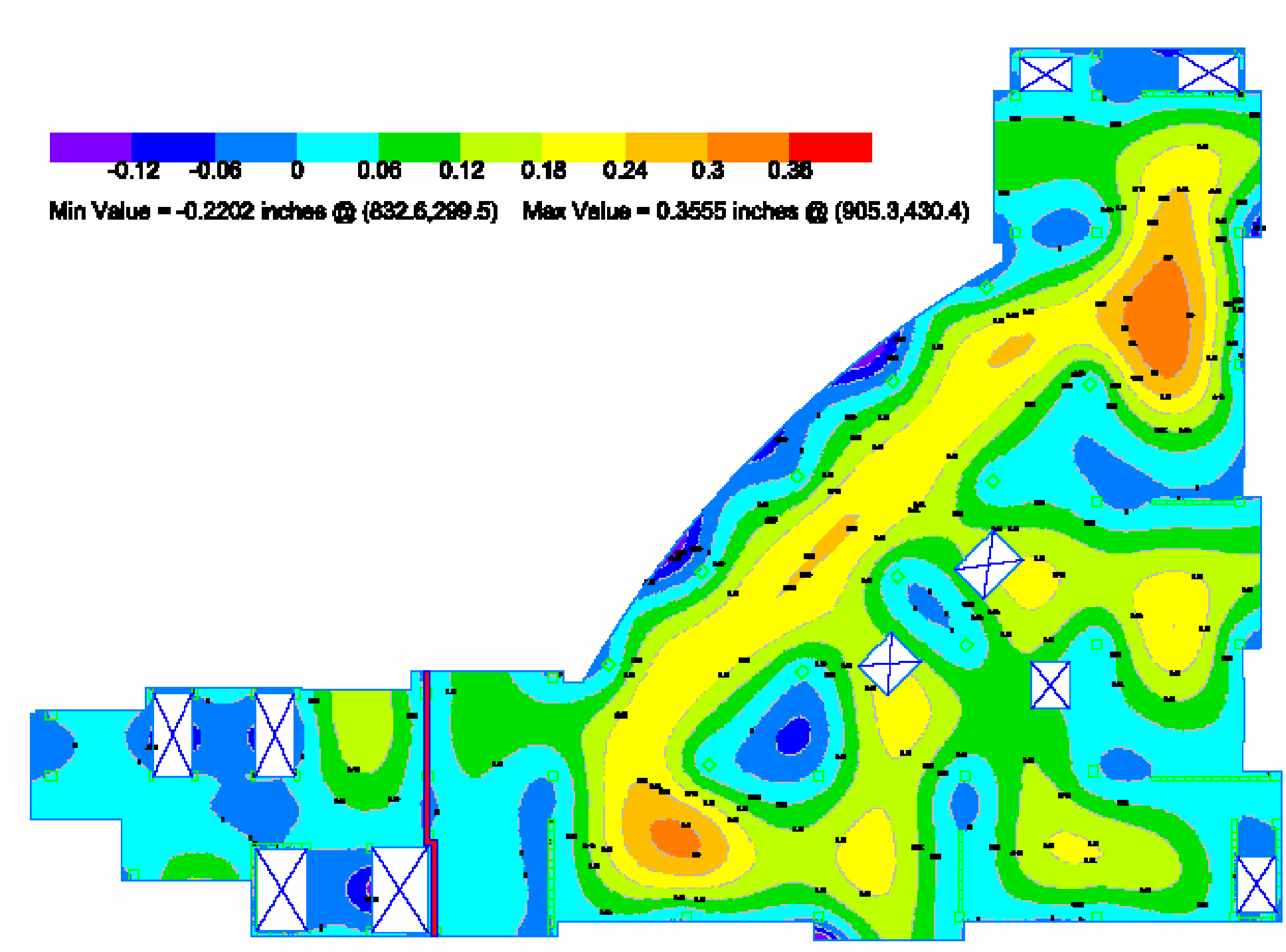


Figure 12 Typical Floor Sustained Service Load Deflection Plan

One-Way Slab and Beams (Conference Wing):

Within the conference wing there are a total of two elevated slabs. With both floors being dimensionally the same they were designed the same for ease of construction. The first floor's design loads were used for the design being the largest loads this area will see. These loads are a Live Load = 100psf and a Superimposed Dead Load = 15psf. The design required a 15" one-way slab with a concrete strength of 5000psi and post-tensioned strands placed in groups of 3 at 4½' o.c. Two separate post-tensioned beam designs and two reinforced concrete beam designs were also needed for this area. The post-tensioned beams dimensionally are 18"x42" and 24"x42". Their designs can be seen in the table below and their calculations in Appendix B. The reinforced concrete beams were designed using PCA Beam. Deflections for this area were not considered to be an issue because the slab and beams were designed as Class U (Uncracked Concrete: ACI 18.3.3).

Concrete Beam Schedule

Mark	Size		Reinforcement		Stirrups		P-T				
							# Strands	Center of Gravity (in)			
	Width	Depth	Top	Bottom	Size	Spacing		c.g.1	c.g.2	c.g.3	c.g.4
PB-1	24	42	4#8	6#9	#4	1@3, 7@5, R@12	30	10.5	4	4	7.25
PB-2	18	42	3#9	6#9	#4	1@3, R@10	16	9.8	2.5	2.5	6.25
B-1	16	36	4#7	4#7	#4	1@3, R@12	-	-	-	-	-
B-2	24	42	8#6	8#6	#4	1@3, R@12	-	-	-	-	-

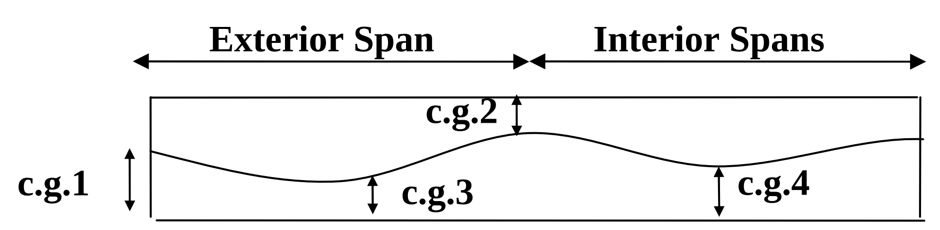


Figure 13 Slab Detail – Tendon Profile

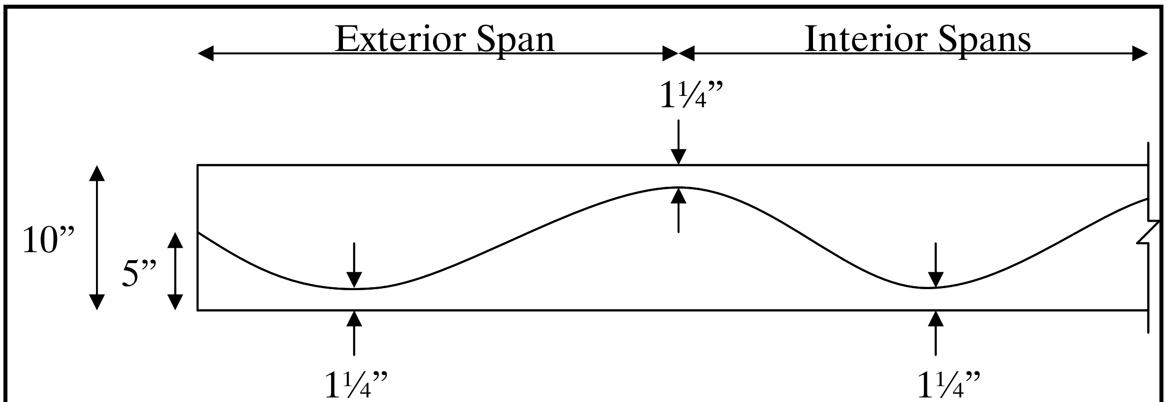


Figure 14 Slab Detail – Tendon Profile

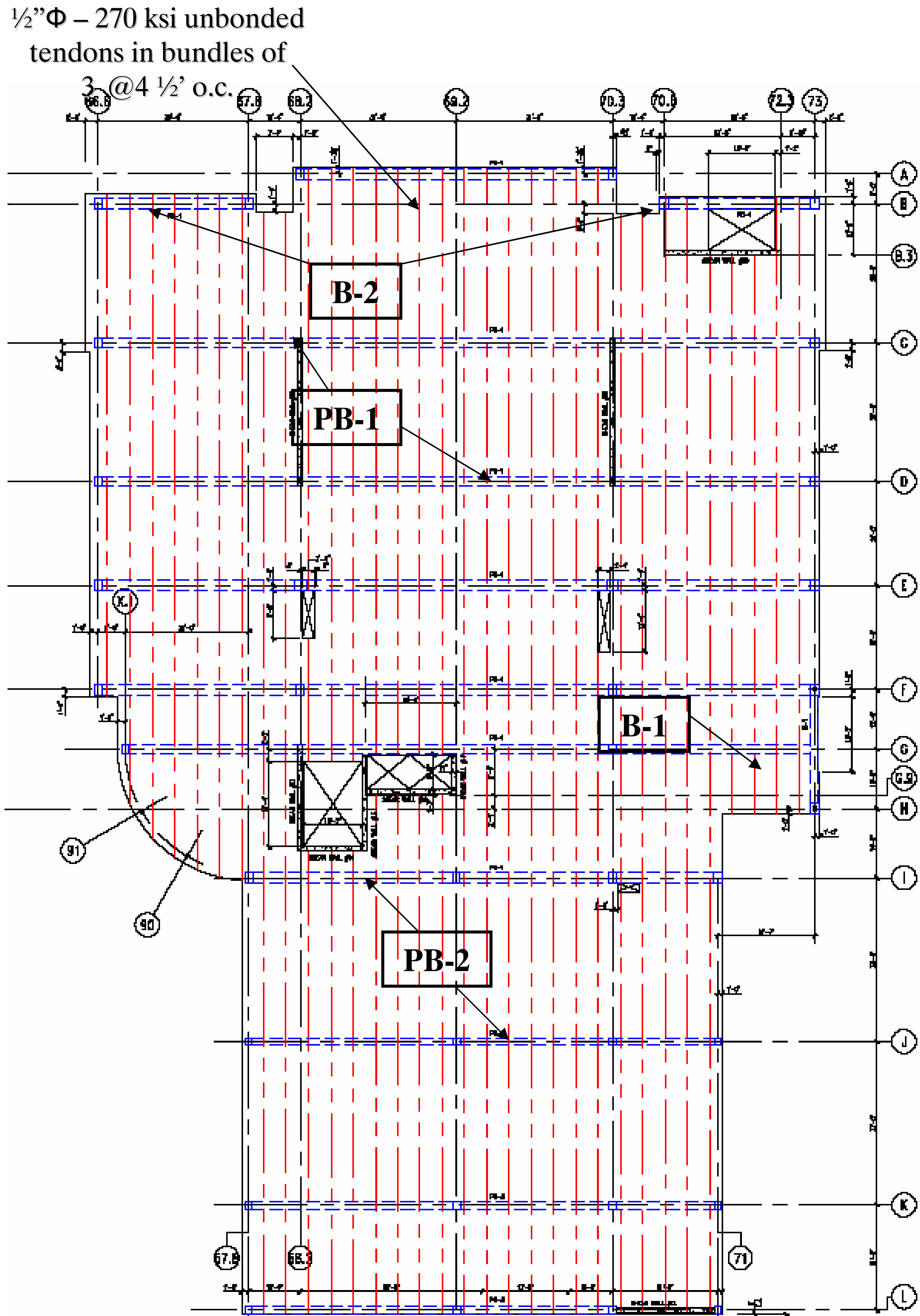


Figure 15 Conference Wing Uniformly Spaced Tendon and Beam Layout

Column Design

With the expansion joint being put in place along column line 65 an additional 4 columns were required to support the edges of the slab. These additional columns were all 12”x24” and their placement can be seen in Figure 16 below. All other columns were also redesigned due to the changes in the floor systems. For the main tower the designs of the columns required less reinforcing because of the lighter post-tensioned design. In the conference wing the original steel columns all required to be redesigned as concrete columns. These concrete columns were all significantly larger than the original steel columns because of the size of the members framing into them and the increased weight of the structure. The new sizes of the concrete columns, their reinforcing, loading, and the interaction diagrams used for design can be viewed in Appendix C.

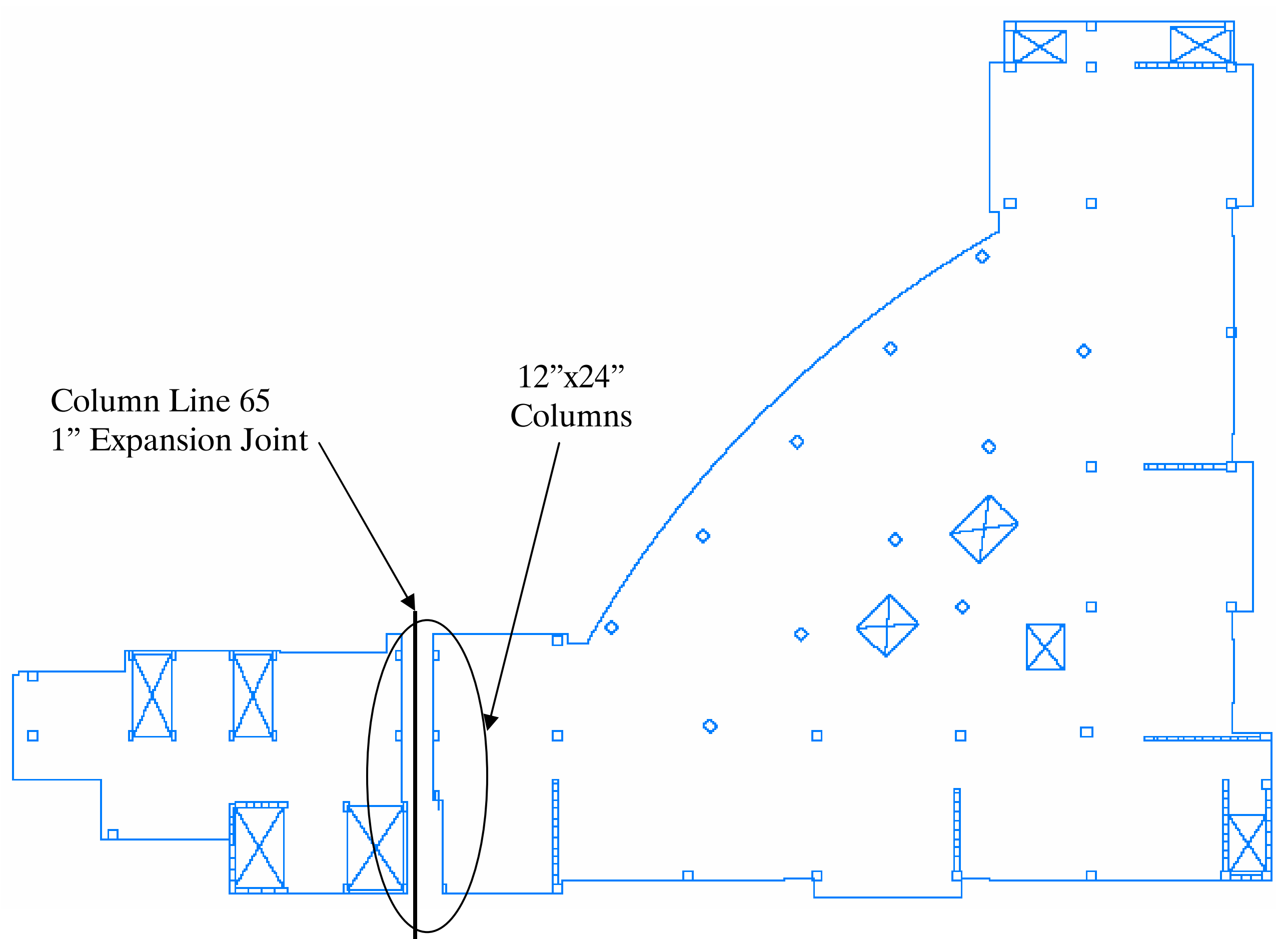


Figure 16 Main Tower Typical Floor Column Layout

Impact on Foundations

The foundations of the Christiana Hospital as mentioned earlier are currently a mat foundation under the main tower and spread footings under the conference wing. With the new post-tensioned design the building weight was reduced which in turn imposed a lighter load on the foundations. After reanalyzing the foundations not many large changes can be made because the soils low bearing pressure (4000psf).

The reason there is a mat foundation is because the spread footings required to support the main tower would be so large they would have to overlap. Due to this a mat foundation was chosen. Even though the building is now lighter, the loads on each column have not been reduced enough to allow spread footings to be used and therefore a mat foundation must also be used under the main tower in my design.

In the case of the conference wing there is some change in footing sizes. For my concrete design all the footings were required to be sized larger while some were forced to be made into combined footings. All of these changes have been taken into account in my schedule and cost estimate.

Construction Management Breadth

The final comparison made between my design and the original design of the Christiana Hospital Project was a cost and schedule comparison of the structural frames. Cost estimates were done using some data from Suncoast Post-Tension Corp. in Woodbridge, VA, and the computer program ICE. For scheduling the project RS means was used to find how many hours it would take typical crews to complete each task and later put into schedule format. In my schedule ranges from 1 to 3 crews were used. The cost and scheduling information for the actual structure is factual data from the records of the construction manager on the project.

Total Structural System Cost Without Added Shear Walls		
	My Design	**As Built**
Concrete		$9,320,230
Structural Steel/ Misc. Metals		$2,897,875
Total	$12,086,085	$12,218,105
Savings of	**$132,020**	

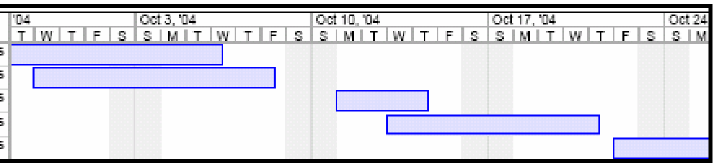
Total Structural System Cost With Added Shear Walls		
	My Design	**As Built**
Concrete		$9,320,230
Structural Steel/ Misc. Metals		$2,897,875
	$12,302,256	$12,218,105
Extra Cost of	**$84,151**	

Project Schedule				
	Main Tower		**Conference Wing**	
	Start Date	Finish Date	Start Date	Finish Date
As Built	9/1/2004	3/1/2005	1/17/2005	3/11/2005
My Design	9/1/2004	1/12/2005	1/17/2005	3/31/2005
Time Savings		49 Days		
Time Lost				20 Days

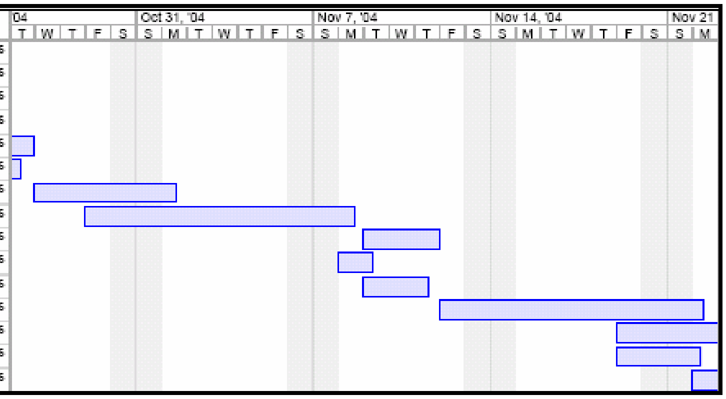
Main Tower Schedule (My Design)

ID	Task Name	Duration
1	Form and Place Mat Foundations/S.O.G.	25.35 days

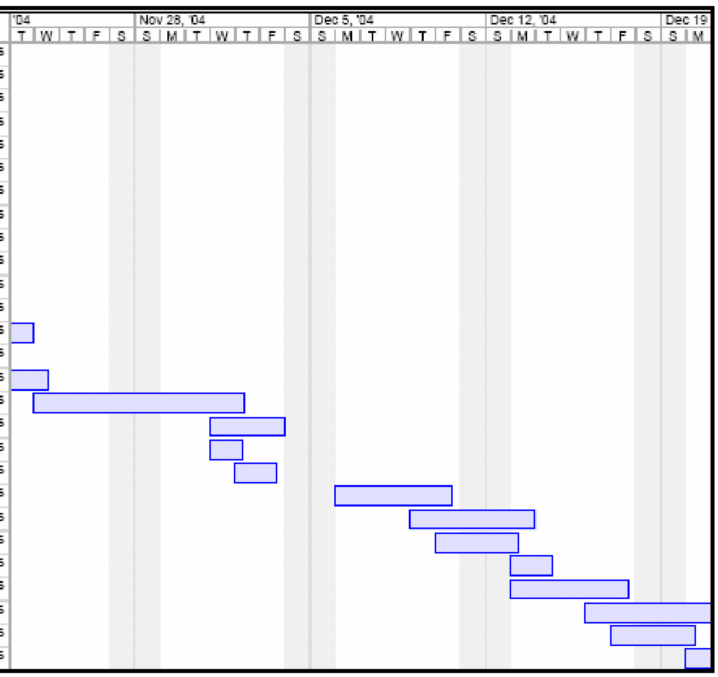
ID	Task Name	Duration
1	Form and Place Mat Foundations/S.O.G.	25.35 days
2	Form and Place Ground Floor Walls	7.61 days
3	Form and Place Ground Floor Columns	3.85 days
4	Form and Place 1st Floor Slab	6.27 days
5	Place and Stress Tendons	3 days



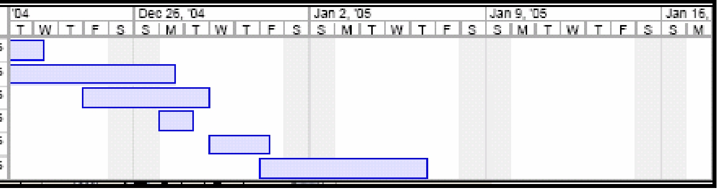
ID	Task Name	Duration
1	Form and Place Mat Foundations/S.O.G.	25.35 days
2	Form and Place Ground Floor Walls	7.61 days
3	Form and Place Ground Floor Columns	3.85 days
4	Form and Place 1st Floor Slab	6.27 days
5	Place and Stress Tendons	3 days
6	Form and Place 1st Floor Walls	1.39 days
7	Form and Place 1st Floor Columns	3.69 days
8	Form and Place 2nd Floor Slab	6.78 days
9	Place and Stress Tendons	3 days
10	Form and Place 2nd Floor Walls	1.03 days
11	Form and Place 2nd Floor Columns	2.62 days
12	Form and Place 3rd Floor Slab	6.24 days
13	Place and Stress Tendons	3 days
14	Form and Place 3rd Floor Walls	1.03 days
15	Form and Place 3rd Floor Columns	2.57 days



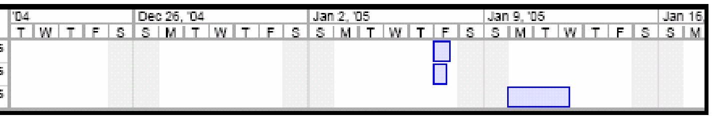
ID	Task Name	Duration
1	Form and Place Mat Foundations/S.O.G.	25.35 days
2	Form and Place Ground Floor Walls	7.61 days
3	Form and Place Ground Floor Columns	3.85 days
4	Form and Place 1st Floor Slab	6.27 days
5	Place and Stress Tendons	3 days
6	Form and Place 1st Floor Walls	1.39 days
7	Form and Place 1st Floor Columns	3.69 days
8	Form and Place 2nd Floor Slab	6.78 days
9	Place and Stress Tendons	3 days
10	Form and Place 2nd Floor Walls	1.03 days
11	Form and Place 2nd Floor Columns	2.62 days
12	Form and Place 3rd Floor Slab	6.24 days
13	Place and Stress Tendons	3 days
14	Form and Place 3rd Floor Walls	1.03 days
15	Form and Place 3rd Floor Columns	2.57 days
16	Form and Place 4th Floor Slab	6.19 days
17	Place and Stress Tendons	3 days
18	Form and Place 4th Floor Walls	1.03 days
19	Form and Place 4th Floor Columns	1.98 days
20	Form and Place 5th Floor Slab	4.95 days
21	Place and Stress Tendons	3 days
22	Form and Place 5th Floor Walls	1.03 days
23	Form and Place 5th Floor Columns	1.96 days
24	Form and Place 6th Floor Slab	4.95 days
25	Place and Stress Tendons	3 days
26	Form and Place 6th Floor Walls	1.15 days
27	Form and Place 6th Floor Columns	2.19 days

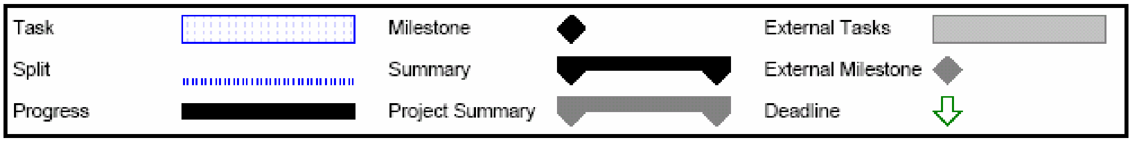


ID	Task Name	Duration
27	Form and Place 6th Floor Columns	2.19 days
28	Form and Place 7th Floor Slab	4.95 days
29	Place and Stress Tendons	3 days
30	Form and Place 7th Floor Walls	1.15 days
31	Form and Place 7th Floor Columns	2.19 days
32	Form and Place 8th Floor Slab	4.91 days



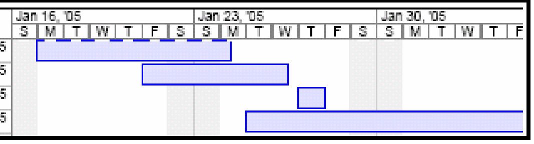
ID	Task Name	Duration
33	Form and Place 8th Floor Walls	0.92 days
34	8th Floor Steel Columns	0.55 days
35	Roof Steel Beams/Girders	2.28 days



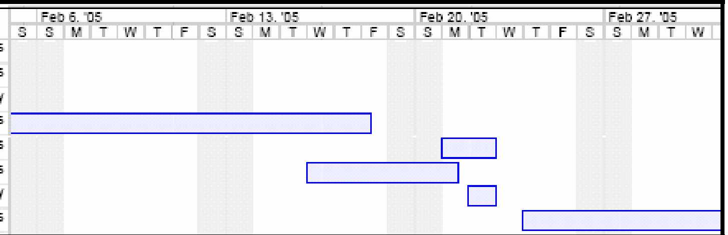


Conference Wing Schedule (My Design)

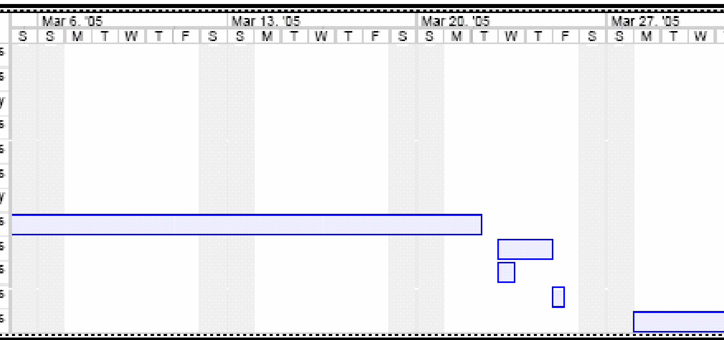
ID		Task Name	Duration	Start	Finish
1		Form and Place Spread/Wall Footings/S.O.G.	5.21 days	Mon 1/17/05	Mon 1/24/05
2		Form and Place Ground Floor Walls	3.65 days	Fri 1/21/05	Wed 1/26/05
3		Form and Place Ground Floor Columns	1 day	Thu 1/27/05	Thu 1/27/05
4		Form and Place 1st Floor Slab	18.2 days	Tue 1/25/05	Fri 2/18/05



ID		Task Name	Duration
1		Form and Place Spread/Wall Footings/S.O.G.	5.21 days
2		Form and Place Ground Floor Walls	3.65 days
3		Form and Place Ground Floor Columns	1 day
4		Form and Place 1st Floor Slab	18.2 days
5		Place and Stress Tendons	2 days
6		Form and Place 1st Floor Walls	3.65 days
7		Form and Place 1st Floor Columns	1 day
8		Form and Place 2nd Floor Slab	18.2 days



ID		Task Name	Duration
1		Form and Place Spread/Wall Footings/S.O.G.	5.21 days
2		Form and Place Ground Floor Walls	3.65 days
3		Form and Place Ground Floor Columns	1 day
4		Form and Place 1st Floor Slab	18.2 days
5		Place and Stress Tendons	2 days
6		Form and Place 1st Floor Walls	3.65 days
7		Form and Place 1st Floor Columns	1 day
8		Form and Place 2nd Floor Slab	18.2 days
9		Place and Stress Tendons	2 days
10		Form and Place 2nd Floor Walls	0.72 days
11		2nd Floor Steel Columns	0.33 days
12		Roof Steel Beams/Girders	3.27 days



Acoustics Breadth

The main attraction to the conference wing in this project is a large conference room on the first floor. Being that this type of room will be mainly used for lectures, conferences, etc. it is essential for the room to be correctly designed acoustically so that information transmitted by way of sound can reach the listener most effectively.

Currently the room has been designed using ½" thick acousticotton panels, wood panels, and 5/8" gypsum along the walls, high traffic carpet and heavily upholstered seats on the floor, and 4'x4' Armstrong Optima acoustical ceiling tiles on the ceiling. Upon initial inspection this amount of sound absorptive materials seemed to be too high which in turn would deliver a much shorter than desirable reverberation time (the time it takes in seconds for average sound in a room to decrease by 60 decibels).

In this type of space the optimum reverberation time is between 0.7 and 1.1 seconds. As predicted earlier the amount of absorptive material in this space is too high giving reverberation times as short as 0.31 seconds at 4000 Hz and only as long as 0.53 seconds at 500 Hz. With this low of a reverberation time sound dies too quickly making it difficult to understand speech.

With further investigation I found that a much more desirable reverberation time could be achieved by using much less absorptive materials which also would greatly reduce the cost of the room. By removing 90% of the acousticotton paneling and all of the Armstrong ceiling tiles and replacing them with 5/8" gypsum the reverberation time was increased to 0.66 seconds at 4000Hz and 1.14 seconds at 500 Hz. With cost information found from local distributors the price of this room alone was reduced by $12,591. The only downfall to this design is that by removing all the ceiling tiles and replacing them with gypsum the room's versatility is taken away. Being a conference room, new wiring will most likely need to be run with changes in technology and removable ceiling tiles lend themselves to this need much better than gypsum.

The second item I looked at was transmission loss. Because this room is located next to a corridor it requires a Sound Transmission Coefficient (STC) of 40. The walls in the current design of the building call for a 3½" sound attenuation blanket which gives an STC of 49 bringing the wall up and over an STC of 42 that allows the wall to be considered quiet for this spatial relationship.

Calculations, material properties, and cost comparisons can be viewed in Appendix D.

Conclusions

Sectioning Structure with Expansion Joint:

The attempt made to reduce the loads in the shear walls by means of dividing the main tower into two separate structures showed to be a very uneconomical design. By separating the structure the eccentricity between the center of mass and the center of rigidity actual increased thus increasing the magnitude of load on each shear wall. The portion of the load on each wall caused by this torsional effect was so high that extra shear walls were required to be put in place adding extra time to the schedule and cost to the project making the as built design the best method of design in this area.

Post Tension Design vs. Reinforced Concrete

Main Tower:

By designing the main tower's floor systems as post-tensioned instead of a reinforced concrete slab with drop panels two things were capable of being achieved. First, the project schedule was capable of being decreased by 49 days and the cost was decreased by $132,020 or 1%. These benefits were mainly from the fact that the floor system was capable of being designed without drop panels which saves on labor costs, formwork, and schedule. While both of these outcomes are beneficial I feel they are not large enough of changes to make a post-tensioned design more practical. The reason for my conclusion is that in hospitals, penetrations in slabs are very common and post-tensioned slabs do not lend themselves well to this. Slab penetrations which are preplanned are not as problematic but those which require any sort of drilling after the slab has been placed can pose problems. These problems arise when tendons are hit and broken by drilling equipment which then requires a very pricey fix.

Post Tension Design vs. Steel Design

Conference Wing:

The design of the conference wing as a post-tensioned slab and beam system with concrete columns and shear walls also showed to be not as practical as the original steel design. Due to the added dead load of the structure both columns and floor thicknesses needed to be increased. Along with the added mass of the structure it also added an extra 20 days to the projects schedule which is a 37% increase to the steel design schedule.

Acoustic Design:

In my acoustical analysis of the major conference room in the conference wing of the Christiana Hospital Project it was found that the amount of sound absorptive materials used to line both the walls and ceiling was too high and lead to the room having a much shorter reverberation time than the desired range of 0.7-1.1 seconds. My design, which decreased the amount of acousticotton used and completely deleted the use of acoustical ceiling tiles, allowed the room to have a longer reverberation time which fell within the desired range of 0.7-1.1 seconds. Along with achieving the desired reverberation time it also allowed the room to be designed for a much lower price.

Acknowledgements

- Cagley & Associates
    - Frank Malits
    - Joe Ajello
    - James Lakey
- Wilmot Sanz
    - Sheila Williams
- Suncoast Post-Tension
- Acoustical Panel Resources
- Armstrong
- Marjam Supply
- AE Faculty
    - Dr. Ali Memari
    - Dr. Andres Lepage
    - Professor Kevin Parfitt

Appendix A
Shear Wall Design

Shear Wall Forces (With Expantion Joint After Adding Required Walls)							
Wall #	Story	V (k)	M (ft-k)	Wall #	Story	V (k)	M (ft-k)
1	ROOF	4.89	88.042	6	ROOF	13.48	1.1233333
	EIGHTH STORY	35.96	543.566		EIGHTH FLOOR	61.65	5.1375
	SEVENTH STORY	61.63	1324.201		SEVENTH FLOOR	154.16	12.846667
	SIXTH STORY	83.58	2271.43		SIXTH FLOOR	225.76	18.813333
	FIFTH STORY	100.69	3412.537		FIFTH FLOOR	282.5	23.541667
	FOURTH STORY	114	4704.484		FOURTH FLOOR	326.05	27.170833
	THIRD STORY	121.38	6080.153		THIRD FLOOR	372.68	31.056667
	SECOND STORY	136.72	8176.503		SECOND FLOOR	410.05	34.170833
	FIRST FLOOR	126.15	9942.546		FIRST FLOOR	443.25	36.9375
2	ROOF	9.16	164.894	7	ROOF	3.08	0.2566667
	EIGHTH STORY	59.32	709.516		EIGHTH FLOOR	7.7	0.6416667
	SEVENTH STORY	107.9	2076.223		SEVENTH FLOOR	13.88	1.1566667
	SIXTH STORY	157.71	3863.623		SIXTH FLOOR	16.64	1.3866667
	FIFTH STORY	178.1	5882.127		FIFTH FLOOR	20.9	1.7416667
	FOURTH STORY	241.12	8614.811		FOURTH FLOOR	17.62	1.4683333
	THIRD STORY	291.28	11915.989		THIRD FLOOR	42.8	3.5666667
	SECOND STORY	339.24	17117.619		SECOND FLOOR	-25.56	-2.13
	FIRST FLOOR	387.54	22543.137		FIRST FLOOR	129.4	10.783333
3	ROOF	11.52	207.421	8	ROOF	16.02	1.335
	EIGHTH STORY	15.48	403.533		EIGHTH FLOOR	60.98	5.0816667
	SEVENTH STORY	19.58	651.525		SEVENTH FLOOR	153.17	12.764167
	SIXTH STORY	20.91	888.562		SIXTH FLOOR	224.55	18.7125
	FIFTH STORY	22.86	1147.663		FIFTH FLOOR	281.15	23.429167
	FOURTH STORY	57.96	1804.491		FOURTH FLOOR	324.51	27.0425
	THIRD STORY	87.79	2799.427		THIRD FLOOR	371.04	30.92
	SECOND STORY	110.61	4495.478		SECOND FLOOR	408.85	34.070833
	FIRST FLOOR	113.74	6087.877		FIRST FLOOR	439.8	36.65
4	ROOF	12.05	1.0041667	9	ROOF	227.76	18.98
	EIGHTH FLOOR	133.3	11.108333		EIGHTH FLOOR	384.96	32.08
	SEVENTH FLOOR	290.38	24.198333		SEVENTH FLOOR	581.3	48.441667
	SIXTH FLOOR	417.85	34.820833		SIXTH FLOOR	745.39	62.115833
	FIFTH FLOOR	519.07	43.255833		FIFTH FLOOR	880.76	73.396667
	FOURTH FLOOR	598.77	49.8975		FOURTH FLOOR	1005.96	83.83
	THIRD FLOOR	676.56	56.38		THIRD FLOOR	1149.6	95.8
	SECOND FLOOR	717.94	59.828333		SECOND FLOOR	1353.12	112.76
	FIRST FLOOR	676.39	56.365833		FIRST FLOOR	823.48	68.623333
5	ROOF	-6.34	-0.528333	10	ROOF	49.57	4.1308333
	EIGHTH FLOOR	54.81	4.5675		EIGHTH FLOOR	97.5	8.125
	SEVENTH FLOOR	136.25	11.354167		SEVENTH FLOOR	160.24	13.353333
	SIXTH FLOOR	197.95	16.495833		SIXTH FLOOR	160.24	13.353333
	FIFTH FLOOR	246.57	20.5475		FIFTH FLOOR	160.24	13.353333
	FOURTH FLOOR	284.17	23.680833		FOURTH FLOOR	255.84	21.32
	THIRD FLOOR	324.57	27.0475		THIRD FLOOR	245.59	20.465833
	SECOND FLOOR	353.15	29.429167		SECOND FLOOR	435.8	36.316667
	FIRST FLOOR	424.75	35.395833		FIRST FLOOR	435.8	36.316667

Shear Wall Forces (With Expantion Joint After Adding Required Walls)							
Wall #	Story	V (k)	M (ft-k)	Wall #	Story	V (k)	M (ft-k)
11	ROOF	119.39	9.949167	15	ROOF	8.5	153.026
	EIGHTH FLOOR	181.5	15.125		EIGHTH STORY	10.87	290.756
	SEVENTH FLOOR	271.17	22.5975		SEVENTH STORY	13.53	462.088
	SIXTH FLOOR	344.35	28.69583		SIXTH STORY	14.03	621.061
	FIFTH FLOOR	399.47	33.28917		FIFTH STORY	13.8	777.498
	FOURTH FLOOR	430.97	35.91417		FOURTH STORY	41.45	1247.234
	THIRD FLOOR	439.81	36.65083		THIRD STORY	64.24	1975.336
	SECOND FLOOR	429.53	35.79417		SECOND STORY	79.16	3189.1
	FIRST FLOOR	407.37	33.9475		FIRST FLOOR	92.19	4479.733
12	ROOF	57.71	4.809167	16	ROOF	3.12	56.158
	EIGHTH FLOOR	113.94	9.495		EIGHTH STORY	26.05	56.158
	SEVENTH FLOOR	184.94	15.41167		SEVENTH STORY	44.22	386.133
	SIXTH FLOOR	237.68	19.80667		SIXTH STORY	59.9	946.307
	FIFTH FLOOR	276.52	23.04333		FIFTH STORY	72.07	1625.211
	FOURTH FLOOR	306.17	25.51417		FOURTH STORY	82.2	2442.044
	THIRD FLOOR	290.58	24.215		THIRD STORY	90.94	3373.646
	SECOND FLOOR	276.93	23.0775		SECOND STORY	93.09	4404.259
	FIRST FLOOR	497.58	41.465		FIRST FLOOR	109.65	5831.622
13	ROOF	58.26	4.855	17	ROOF	-31.25	-472.522
	EIGHTH FLOOR	109.09	9.090833		EIGHTH STORY	-54.63	-863.06
	SEVENTH FLOOR	178.14	14.845		SEVENTH STORY	-74.06	-1256.55
	SIXTH FLOOR	238.08	19.84		SIXTH STORY	-82.04	-1491.19
	FIFTH FLOOR	285.55	23.79583		FIFTH STORY	-107.3	-1859.8
	FOURTH FLOOR	318.2	26.51667		FOURTH STORY	-132.9	-2296.84
	THIRD FLOOR	339.2	28.26667		THIRD STORY	-153.8	-2800.75
	SECOND FLOOR	309.16	25.76333		SECOND STORY	-161.1	-3555.13
	FIRST FLOOR	268.36	22.36333		FIRST FLOOR	-160.9	-3677.9
14	ROOF	-90.81	-7.5675	18	ROOF	-1.11	39.706
	EIGHTH FLOOR	77.04	6.42		EIGHTH STORY	25.76	255.609
	SEVENTH FLOOR	143.39	11.94917		SEVENTH STORY	44.1	677.067
	SIXTH FLOOR	196.43	16.36917		SIXTH STORY	61.4	1187.429
	FIFTH FLOOR	239.02	19.91833		FIFTH STORY	77.8	1835.307
	FOURTH FLOOR	267.47	22.28917		FOURTH STORY	80.38	2451.4
	THIRD FLOOR	287.89	23.99083		THIRD STORY	80.72	3015.326
	SECOND FLOOR	302.01	25.1675		SECOND STORY	89.53	3907.391
	FIRST FLOOR	342.37	28.53083		FIRST FLOOR	93.02	4605.752

Shear Wall Forces (Post-Tensioned Conference Wing)			
Wall #	Story	V (k)	M (ft-k)
19	SECOND STORY	30.7	552.57
	FIRST FLOOR	105.9	2035.153
20	THIRD STORY	36.34	401.063
	SECOND STORY	81.17	1383.472
	FIRST FLOOR	133.52	2457.171
21	THIRD STORY	86.49	673.003
	SECOND STORY	134.03	1485.795
	FIRST FLOOR	150.91	1780.027
22	THIRD STORY	-64.42	-522.838
	SECOND STORY	-76.77	-1354.448
	FIRST FLOOR	-132.26	-2492.499
23	THIRD STORY	51.46	536.867
	SECOND STORY	127.4	2046.526
	FIRST FLOOR	187.63	3618.693
24	THIRD STORY	15.01	113.387
	SECOND STORY	32.8	306.455
	FIRST FLOOR	53.76	673.614
25	THIRD STORY	91.48	1311.237
	SECOND STORY	170.72	4384.221
	FIRST FLOOR	229.98	7603.877
26	THIRD STORY	105.41	1510.87
	SECOND STORY	177.75	4710.306
	FIRST FLOOR	241.47	8090.857
27	THIRD STORY	138.88	1990.617
	SECOND STORY	227.89	6092.708
	FIRST FLOOR	348.38	10969.995

12" Concrete Shear Wall Schedule								
Floor	1	2	3	4	5	6	7	8
Length	11.7	18.5	11.7	23.5	18.58	19.7	8.75	19.7
Boundary Element	T.1-59, T.1-61	T.1-59, U.1-59	U.1-59,U.1-61	U.66	U-69	U-71	U-71, U-72	T-72
8	#5@18"	#5@18"	#5@18"		#5@18"	#5@18"	#5@18"	#5@18"
7				#5@18"				
6								
5								
4								
3								
2				#5@16"				
1								
G								

12" Concrete Shear Wall Schedule								
Floor	9	10	11	12	13	14	15	16
Length	26.2	18.5	20.67	18.5	9	20.67	11.4	11.4
Boundary Element	S-72	Q-71	N-71	R-71	N-71, M-71	M-71	U.1-63, U.1-65	T.1-63, T.1-65
8			#5@18"	#5@18"	#5@18"	#5@18"	#5@18"	#5@18"
7	#5@14"	#5@18"						
6								
5								
4	#5@10"							
3								
2	#5@16"							
1								
G								

12" Concrete Shear Wall Schedule						
Wall #	17	18	19	20	21	22
Length	17.5	9.25	20.67	19.75	12.2	19.75
Boundary Element	R.2-59, S-59	S-59, S-61	-	-	-	-
8	#5@18"	#5@18"				
7						
6						
5						
4						
3						
2				#5@18"	#5@18"	#5@18"
1			#5@18"			
G						

12" Concrete Shear Wall Schedule					
Wall #	23	24	25	26	27
Length	18.25	9.67	28	28	28
Boundary Element	-	-	-	-	-
8					
7					
6					
5					
4					
3					
2	#5@18"	#5@18"	#5@18"	#5@18"	#5@18"
1					
G					

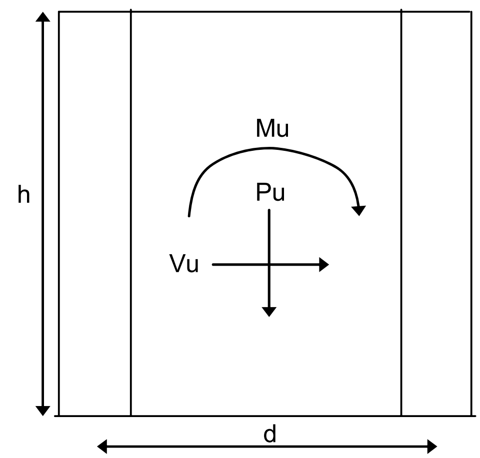
The above schedules give the length, reinforcement, and boundary element locations for all shear walls. To view the reinforcement designed for the shear wall boundary elements see the column schedules in Appendix C.

Shear Wall Design

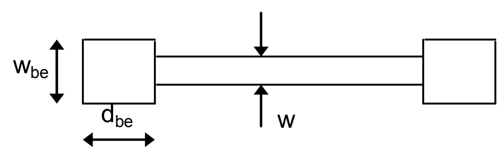
Engineer:	Joe Sharkey
Date:	3/19/2007
Job:	Christiana Hospital Project
Shear Wall #	1 - Ground Floor through 2nd

Material Properties	
Concrete Strength - f'c (psi) =	5000
Reinforcement Strength - fy (psi) =	60000

Wall Dimensions	
Length - d (ft) =	11.7
Width - w (in) =	16
Height - h (ft) =	118



Boundary Element Dimensions	
Length - d_{be} (in) =	18
Width - w_{be} (in) =	18



Wall Loads	
Pu (kip) =	813
Mu (ft-kip) =	9943
Vu (kip) =	126

Boundary Element	
Axial Force - Pu_{be} (kip) =	1256.329

ACI 21.7.6.3

Boundary Element Check	
Ag (ft^2) =	17.6
Ig (in^4) =	255.552
Extreme Fiber Comp. - Fc (ksi) =	2.104066

Boundary Elemement Needed - fc>0.2 f'c

ACI 21.7.2.2

Longitudinal & Transverse Reinforcement

One Curtain of Reinf. Req.		
Acv (in^2/ft) =	192	
Longitudinal - ρ_l, Transverse - ρt >= 0.0025		
$As_{lreq'd}$ (in^2/ft) =	0.48	
$As_{supplied}$ (in^2) =	0.62	#5 Bars
Bar Diameter (in) =	0.625	
Required Spacing - $S_{req'd}$ (in) =	15.5	**OK**
Spacing Supplied - $S_{supplied}$ (in) =	15	

Shear Capacity Check

$\alpha_c = h_w/l_w$	2	**hw/lw>2 therefore use 2**
Acv_{total} (in^2) =	2534.4	
Transverse - ρt =	0.002583	
Nominal Shear Capacity - Vn (kip) =	751.2503	
Shear Capacity - ΦVn (kip) =	450.7502	**OK**

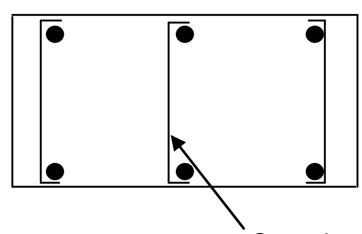
Boundary Element Capacity Check

Ast (in^2) =	18.72	12-#11
ρ_{st} =	0.057778	**OK**
Pn(max) (kip) =	1936.512	
Axial Load Capacity - ΦPn (kip) =	1355.558	**OK**

Check With Interaction Diagram

Determine Confinement Reinforcement for Boundary Elements

Max. Allowable Vert. Spacing - Smax (in) =	4	
Vert. Spacing Supplied - $S_{supplied}$ (in) =	4	
Short Direction (in) =	18	
Long Direction (in) =	18	
Bar Diameter (in) =	0.625	#5 Bar
Cover from center of Vert. Reinf. To Col. Face (in) =	3	
As of one Bar (in^2) =	0.31	
Area Bounded by out-to-out of hoops - Ach (in^2) =	192.5156	
Short Direction		
Number of Crossties In Short Derection =	4	
hc (in) =	13.25	
Req'd Reinf. In Short Direction - Ash (in^2) =	0.904949	
As provided (in^2) =	1.24	**OK**
Long Direction		
Number of Crossties In Short Derection =	4	
hc (in) =	13.25	
Req'd Reinf. In Short Direction - Ash (in^2) =	0.904949	
As provided (in^2) =	1.24	**OK**



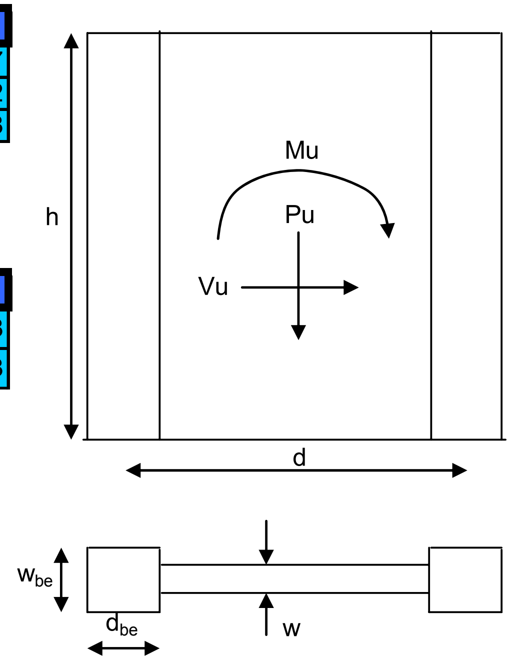
Crosstie

Shear Wall Design

Engineer:	Joe Sharkey
Date:	3/19/2007
Job:	Christiana Hospital Project
Shear Wall #	1 - 3rd through 8th

Material Properties	
Concrete Strength - f'c (psi) =	4000
Reinforcement Strength - fy (psi) =	60000

Wall Dimensions	
Length - d (ft) =	11.7
Width - w (in) =	12
Height - h (ft) =	118



Boundary Element Dimensions	
Length - d_{be} (in) =	18
Width - w_{be} (in) =	18

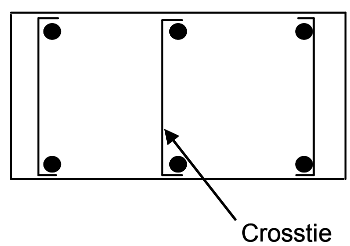
Wall Loads	
Pu (kip) =	586
Mu (ft-kip) =	4704
Vu (kip) =	114

Boundary Element	
Axial Force - Pu_{be} (kip) =	695.0513

ACI 21.7.6.3

Boundary Element Check		
Ag (ft^2) =	13.2	
Ig (in^4) =	191.664	
Extreme Fiber Comp. - Fc (ksi) =	1.433176	**Boundary Elemement Needed - fc>0.2 f'c**

	Longitudinal & Transverse Reinforcement		
ACI 21.7.2.2	**One Curtain of Reinf. Req.**		
	Acv (in^2/ft) =	144	
	Longitudinal - ρ_l, Transverse - ρt >= 0.0025		
	$As_{lreq'd}$ (in^2/ft) =	0.36	
	$As_{supplied}$ (in^2) =	0.62	#5 Bars
	Bar Diameter (in) =	0.625	
	Required Spacing - $S_{req'd}$ (in) =	20.66667	**NOT OK Spacing Must Be Less Than 18in**
	Spacing Supplied - $S_{supplied}$ (in) =	18	
	Shear Capacity Check		
	$\alpha_c = h_w/l_w$	2	**hw/lw>2 therefore use 2**
	Acv_{total} (in^2) =	1900.8	
	Transverse - ρt =	0.00287	
	Nominal Shear Capacity - Vn (kip) =	567.7943	
	Shear Capacity - ΦVn (kip) =	340.6766	**OK**
	Boundary Element Capacity Check		
	Ast (in^2) =	6.24	4-#11
	ρ_{st} =	0.019259	**OK**
	Pn(max) (kip) =	1163.827	
	Axial Load Capacity - ΦPn (kip) =	814.679	**OK**
	Check With Interaction Diagram		
	Determine Confinement Reinforcement for Boundary Elements		
	Max. Allowable Vert. Spacing - Smax (in) =	4	
	Vert. Spacing Supplied - $S_{supplied}$ (in) =	4	
	Short Direction (in) =	18	
	Long Direction (in) =	18	
	Bar Diameter (in) =	0.625	#5 Bar
	Cover from center of Vert. Reinf. To Col. Face (in) =	3	
	As of one Bar (in^2) =	0.31	
	Area Bounded by out-to-out of hoops - Ach (in^2) =	192.5156	
	Short Direction		
	Number of Crossties In Short Derection =	3	
	hc (in) =	13.25	
	Req'd Reinf. In Short Direction - Ash (in^2) =	0.723959	
	As provided (in^2) =	0.93	**OK**
	Long Direction		
	Number of Crossties In Short Derection =	3	
	hc (in) =	13.25	
	Req'd Reinf. In Short Direction - Ash (in^2) =	0.723959	
	As provided (in^2) =	0.93	**OK**

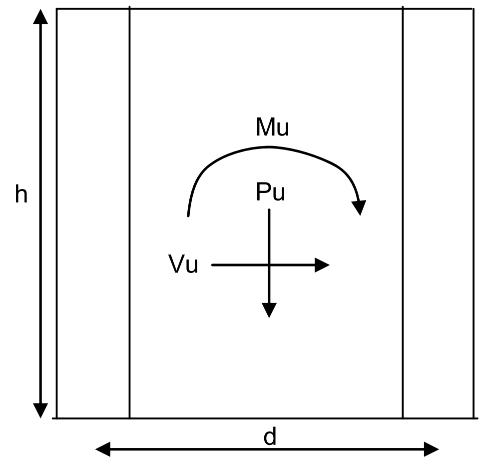


Shear Wall Design

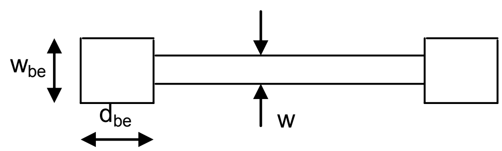
Engineer:	Joe Sharkey
Date:	3/19/2007
Job:	Christiana Hospital Project
Shear Wall #	5 - Ground Floor through 2nd

Material Properties	
Concrete Strength - f'c (psi) =	5000
Reinforcement Strength - fy (psi) =	60000

Wall Dimensions	
Length - d (ft) =	18.58
Width - w (in) =	12
Height - h (ft) =	118



Boundary Element Dimensions	
Length - d_{be} (in) =	30
Width - w_{be} (in) =	30



Wall Loads	
Pu (kip) =	2253
Mu (ft-kip) =	25605
Vu (kip) =	425

Boundary Element

Axial Force - Pu_{be} (kip) = 2504.595

ACI 21.7.6.3

Boundary Element Check

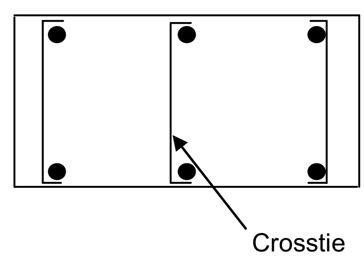
Ag (ft^2) = 21.08

Ig (in^4) = 780.6036

Extreme Fiber Comp. - Fc (ksi) = 3.143103 **Boundary Elemement Needed - fc>0.2 f'c**

ACI 21.7.2.2

Longitudinal & Transverse Reinforcement		
One Curtain of Reinf. Req.		
Acv (in^2/ft) =	144	
Longitudinal - ρ_l, Transverse - ρt >= 0.0025		
$As_{lreq'd}$ (in^2/ft) =	0.36	
$As_{supplied}$ (in^2) =	0.62	#5 Bars
Bar Diameter (in) =	0.625	
Required Spacing - $S_{req'd}$ (in) =	20.66667	**NOT OK Spacing Must Be Less Than 18in**
Spacing Supplied - $S_{supplied}$ (in) =	18	
Shear Capacity Check		
$\alpha_c = h_w/l_w$	2	**hw/lw>2 therefore use 2**
Acv_{total} (in^2) =	3035.52	
Transverse - ρt =	0.00287	
Nominal Shear Capacity - Vn (kip) =	952.0714	
Shear Capacity - ΦVn (kip) =	571.2428	**OK**
Boundary Element Capacity Check		
Ast (in^2) =	12.48	8-#11
ρ_{st} =	0.013867	**OK**
Pn(max) (kip) =	3616.608	
Axial Load Capacity - ΦPn (kip) =	2531.626	**OK**
Check With Interaction Diagram		
Determine Confinement Reinforcement for Boundary Elements		
Max. Allowable Vert. Spacing - Smax (in) =	4	
Vert. Spacing Supplied - $S_{supplied}$ (in) =	4	
Short Direction (in) =	30	
Long Direction (in) =	30	
Bar Diameter (in) =	0.625	#5 Bar
Cover from center of Vert. Reinf. To Col. Face (in) =	3	
As of one Bar (in^2) =	0.31	
Area Bounded by out-to-out of hoops - Ach (in^2) =	669.5156	
Short Direction		
Number of Crossties In Short Direction =	3	
hc (in) =	25.25	
Req'd Reinf. In Short Direction - Ash (in^2) =	0.869245	
As provided (in^2) =	0.93	**OK**
Long Direction		
Number of Crossties In Short Direction =	3	
hc (in) =	25.25	
Req'd Reinf. In Short Direction - Ash (in^2) =	0.869245	
As provided (in^2) =	0.93	**OK**



Shear Wall Design

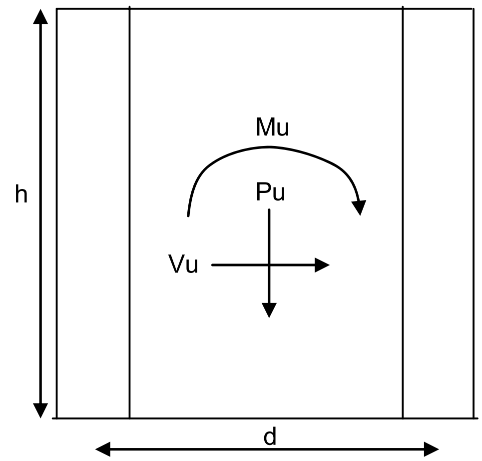
Engineer:	Joe Sharkey
Date:	3/19/2007
Job:	Christiana Hospital Project
Shear Wall #	5 - 3rd and 4th Floors

Material Properties

Concrete Strength - f'c (psi) =	4000
Reinforcement Strength - fy (psi) =	60000

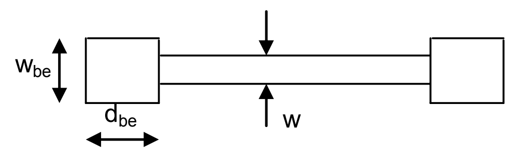
Wall Dimensions

Length - d (ft) =	18.58
Width - w (in) =	12
Height - h (ft) =	77.33



Boundary Element Dimensions

Length - d_{be} (in) =	24
Width - w_{be} (in) =	24



Wall Loads

Pu (kip) =	1522
Mu (ft-kip) =	10564
Vu (kip) =	284

Boundary Element

Axial Force - Pu_{be} (kip) = 1329.568

ACI 21.7.6.3

Boundary Element Check

Ag (ft^2) = 20.58

Ig (in^4) = 726.3649

Extreme Fiber Comp. - Fc (ksi) = 1.552844 **Boundary Elemement Needed - fc>0.2 f'c**

ACI 21.7.2.2

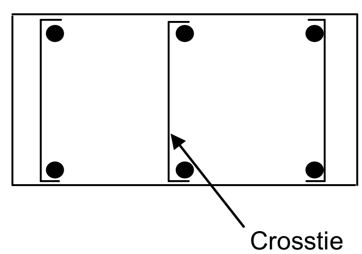
Longitudinal & Transverse Reinforcement		
One Curtain of Reinf. Req.		
Acv (in^2/ft) =	144	
Longitudinal - ρ_l, Transverse - ρt >= 0.0025		
$As_{lreq'd}$ (in^2/ft) =	0.36	
$As_{supplied}$ (in^2) =	0.62	#5 Bars
Bar Diameter (in) =	0.625	
Required Spacing - $S_{req'd}$ (in) =	20.66667	**NOT OK Spacing Must Be Less Than 18in**
Spacing Supplied - $S_{supplied}$ (in) =	18	

Shear Capacity Check		
$\alpha_c = h_w/l_w$	2	**hw/lw>2 therefore use 2**
Acv_{total} (in^2) =	2963.52	
Transverse - ρt =	0.00287	
Nominal Shear Capacity - Vn (kip) =	885.2429	
Shear Capacity - ΦVn (kip) =	531.1458	**OK**

Boundary Element Capacity Check		
Ast (in^2) =	12.48	8-#11
ρ_{st} =	0.021667	**OK**
Pn(max) (kip) =	2131.814	
Axial Load Capacity - ΦPn (kip) =	1492.27	**OK**

Check With Interaction Diagram

Determine Confinement Reinforcement for Boundary Elements		
Max. Allowable Vert. Spacing - Smax (in) =	4	
Vert. Spacing Supplied - $S_{supplied}$ (in) =	4	
Short Direction (in) =	24	
Long Direction (in) =	24	
Bar Diameter (in) =	0.625	#5 Bar
Cover from center of Vert. Reinf. To Col. Face (in) =	3	
As of one Bar (in^2) =	0.31	
Area Bounded by out-to-out of hoops - Ach (in^2) =	395.0156	
Short Direction		
Number of Crossties In Short Direction =	3	
hc (in) =	19.25	
Req'd Reinf. In Short Direction - Ash (in^2) =	0.705582	
As provided (in^2) =	0.93	**OK**
Long Direction		
Number of Crossties In Short Direction =	3	
hc (in) =	19.25	
Req'd Reinf. In Short Direction - Ash (in^2) =	0.705582	
As provided (in^2) =	0.93	**OK**

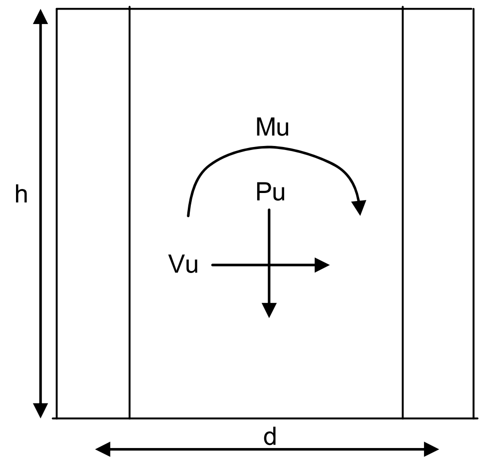


Shear Wall Design

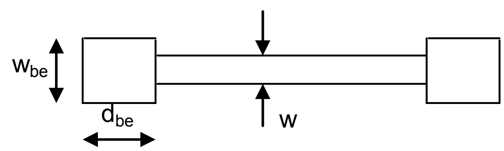
Engineer:	Joe Sharkey
Date:	3/19/2007
Job:	Christiana Hospital Project
Shear Wall #	5 - 5th through 8th

Material Properties	
Concrete Strength - f'c (psi) =	4000
Reinforcement Strength - fy (psi) =	60000

Wall Dimensions	
Length - d (ft) =	18.58
Width - w (in) =	12
Height - h (ft) =	77.33



Boundary Element Dimensions	
Length - d_{be} (in) =	24
Width - w_{be} (in) =	24



Wall Loads	
Pu (kip) =	1080
Mu (ft-kip) =	4549
Vu (kip) =	198

Boundary Element	
Axial Force - Pu_{be} (kip) =	784.8332

ACI 21.7.6.3

Boundary Element Check	
Ag (ft^2) =	20.58
Ig (in^4) =	726.3649
Extreme Fiber Comp. - Fc (ksi) =	0.811953

Boundary Elemement Needed - fc>0.2 f'c

ACI 21.7.2.2

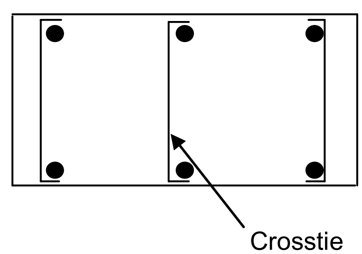
Longitudinal & Transverse Reinforcement		
One Curtain of Reinf. Req.		
Acv (in^2/ft) =	144	
Longitudinal - ρ_l, Transverse - ρt >= 0.0025		
$As_{lreq'd}$ (in^2/ft) =	0.36	
$As_{supplied}$ (in^2) =	0.62	#5 Bars
Bar Diameter (in) =	0.625	
Required Spacing - $S_{req'd}$ (in) =	20.66667	NOT OK Spacing Must Be Less Than 18in
Spacing Supplied - $S_{supplied}$ (in) =	18	

Shear Capacity Check		
$\alpha_c = h_w/l_w$	2	hw/lw>2 therefore use 2
Acv_{total} (in^2) =	2963.52	
Transverse - ρt =	0.00287	
Nominal Shear Capacity - Vn (kip) =	885.2429	
Shear Capacity - ΦVn (kip) =	531.1458	OK

Boundary Element Capacity Check		
Ast (in^2) =	6.24	4-#11
ρ_{st} =	0.010833	OK
Pn(max) (kip) =	1849.267	
Axial Load Capacity - ΦPn (kip) =	1294.487	OK

Check With Interaction Diagram

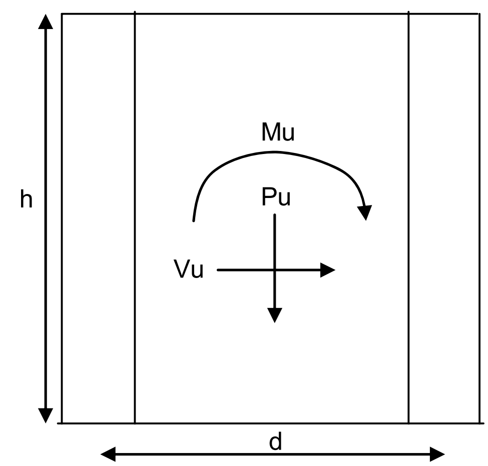
Determine Confinement Reinforcement for Boundary Elements		
Max. Allowable Vert. Spacing - Smax (in) =	4	
Vert. Spacing Supplied - $S_{supplied}$ (in) =	4	
Short Direction (in) =	24	
Long Direction (in) =	24	
Bar Diameter (in) =	0.625	#5 Bar
Cover from center of Vert. Reinf. To Col. Face (in) =	3	
As of one Bar (in^2) =	0.31	
Area Bounded by out-to-out of hoops - Ach (in^2) =	395.0156	
Short Direction		
Number of Crossties In Short Direction =	3	
hc (in) =	19.25	
Req'd Reinf. In Short Direction - Ash (in^2) =	0.705582	
As provided (in^2) =	0.93	OK
Long Direction		
Number of Crossties In Short Direction =	3	
hc (in) =	19.25	
Req'd Reinf. In Short Direction - Ash (in^2) =	0.705582	
As provided (in^2) =	0.93	OK



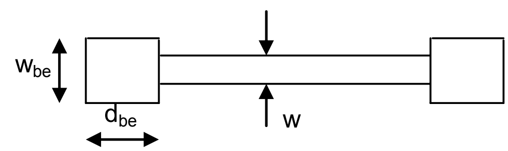
Shear Wall Design	
Engineer:	Joe Sharkey
Date:	3/19/2007
Job:	Christiana Hospital Project
Shear Wall #	11 - Ground Floor through 2nd

Material Properties	
Concrete Strength - f'c (psi) =	5000
Reinforcement Strength - fy (psi) =	60000

Wall Dimensions	
Length - d (ft) =	20.67
Width - w (in) =	12
Height - h (ft) =	118



Boundary Element Dimensions	
Length - d_{be} (in) =	24
Width - w_{be} (in) =	24



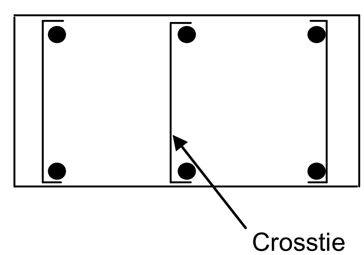
Wall Loads	
Pu (kip) =	1745
Mu (ft-kip) =	20917
Vu (kip) =	407

Boundary Element	
Axial Force - Pu_{be} (kip) =	1884.45

ACI 21.7.6.3

Boundary Element Check		
Ag (ft^2) =	22.67	
Ig (in^4) =	970.8973	
Extreme Fiber Comp. - Fc (ksi) =	2.230382	**Boundary Elemement Needed - fc>0.2 f'c**

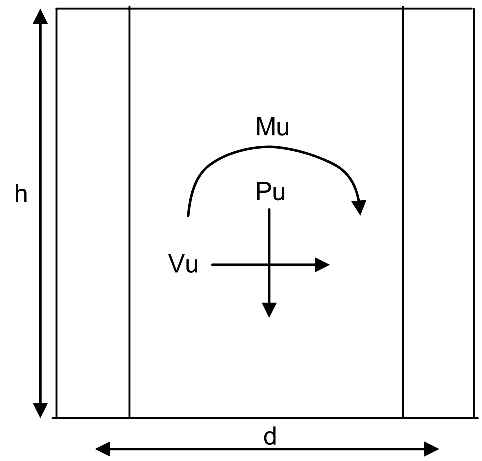
	Longitudinal & Transverse Reinforcement		
ACI 21.7.2.2	**One Curtain of Reinf. Req.**		
	Acv (in^2/ft) =	144	
	Longitudinal - ρ_l, Transverse - ρt >= 0.0025		
	$As_{lreq'd}$ (in^2/ft) =	0.36	
	$As_{supplied}$ (in^2) =	0.62	#5 Bars
	Bar Diameter (in) =	0.625	
	Required Spacing - $S_{req'd}$ (in) =	20.66667	**NOT OK Spacing Must Be Less Than 18in**
	Spacing Supplied - $S_{supplied}$ (in) =	16	
	Shear Capacity Check		
	$\alpha_c = h_w/l_w$	2	**hw/lw>2 therefore use 2**
	Acv_{total} (in^2) =	3264.48	
	Transverse - ρt =	0.003229	
	Nominal Shear Capacity - Vn (kip) =	1094.16	
	Shear Capacity - ΦVn (kip) =	656.4961	**OK**
	Boundary Element Capacity Check		
	Ast (in^2) =	18.72	12-#11
	ρ_{st} =	0.0325	**OK**
	Pn(max) (kip) =	2793.312	
	Axial Load Capacity - ΦPn (kip) =	1955.318	**OK**
	Check With Interaction Diagram		
	Determine Confinement Reinforcement for Boundary Elements		
	Max. Allowable Vert. Spacing - Smax (in) =	4	
	Vert. Spacing Supplied - $S_{supplied}$ (in) =	4	
	Short Direction (in) =	24	
	Long Direction (in) =	24	
	Bar Diameter (in) =	0.625	#5 Bar
	Cover from center of Vert. Reinf. To Col. Face (in) =	3	
	As of one Bar (in^2) =	0.31	
	Area Bounded by out-to-out of hoops - Ach (in^2) =	395.0156	
	Short Direction		
	Number of Crossties In Short Direction =	4	
	hc (in) =	19.25	
	Req'd Reinf. In Short Direction - Ash (in^2) =	0.881978	
	As provided (in^2) =	1.24	**OK**
	Long Direction		
	Number of Crossties In Short Direction =	4	
	hc (in) =	19.25	
	Req'd Reinf. In Short Direction - Ash (in^2) =	0.881978	
	As provided (in^2) =	1.24	**OK**



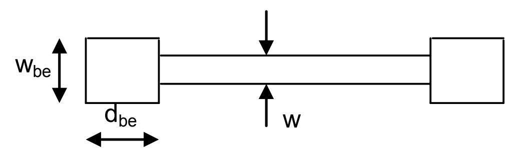
Shear Wall Design	
Engineer:	Joe Sharkey
Date:	3/19/2007
Job:	Christiana Hospital Project
Shear Wall #	11 - 3rd through 8th

Material Properties	
Concrete Strength - f'c (psi) =	4000
Reinforcement Strength - fy (psi) =	60000

Wall Dimensions	
Length - d (ft) =	20.67
Width - w (in) =	12
Height - h (ft) =	77.33



Boundary Element Dimensions	
Length - d_{be} (in) =	24
Width - w_{be} (in) =	24



Wall Loads	
Pu (kip) =	1192
Mu (ft-kip) =	11670
Vu (kip) =	431

Boundary Element

Axial Force - Pu_{be} (kip) = 1160.586

ACI 21.7.6.3

Boundary Element Check

Ag (ft^2) =	22.67
Ig (in^4) =	970.8973
Extreme Fiber Comp. - Fc (ksi) =	1.311285

Boundary Elemement Needed - fc>0.2 f'c

Longitudinal & Transverse Reinforcement

ACI 21.7.2.2 **Two Curtains of Reinf. Req.**

Acv (in^2/ft) =	144	
Longitudinal - ρ_l, Transverse - ρt >= 0.0025		
$As_{lreq'd}$ (in^2/ft) =	0.36	
$As_{supplied}$ (in^2) =	0.62	#5 Bars
Bar Diameter (in) =	0.625	
Required Spacing - $S_{req'd}$ (in) =	20.66667	**NOT OK Spacing Must Be Less Than 18in**
Spacing Supplied - $S_{supplied}$ (in) =	18	

Shear Capacity Check

$\alpha_c = h_w/l_w$	2	**hw/lw>2 therefore use 2**
Acv_{total} (in^2) =	3264.48	
Transverse - ρt =	0.00287	
Nominal Shear Capacity - Vn (kip) =	975.1437	
Shear Capacity - ΦVn (kip) =	585.0862	**OK**

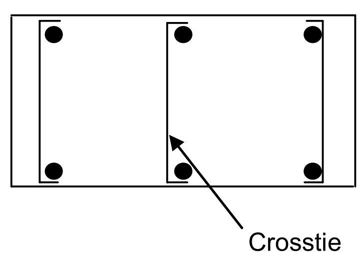
Boundary Element Capacity Check

Ast (in^2) =	6.24	4-#11
ρ_{st} =	0.010833	**OK**
Pn(max) (kip) =	1849.267	
Axial Load Capacity - ΦPn (kip) =	1294.487	**OK**

Check With Interaction Diagram

Determine Confinement Reinforcement for Boundary Elements

Max. Allowable Vert. Spacing - Smax (in) =	4	
Vert. Spacing Supplied - $S_{supplied}$ (in) =	4	
Short Direction (in) =	24	
Long Direction (in) =	24	
Bar Diameter (in) =	0.625	#5 Bar
Cover from center of Vert. Reinf. To Col. Face (in) =	3	
As of one Bar (in^2) =	0.31	
Area Bounded by out-to-out of hoops - Ach (in^2) =	395.0156	
Short Direction		
Number of Crossties In Short Direction =	3	
hc (in) =	19.25	
Req'd Reinf. In Short Direction - Ash (in^2) =	0.705582	
As provided (in^2) =	0.93	**OK**
Long Direction		
Number of Crossties In Short Direction =	3	
hc (in) =	19.25	
Req'd Reinf. In Short Direction - Ash (in^2) =	0.705582	
As provided (in^2) =	0.93	**OK**



Shear Wall Design

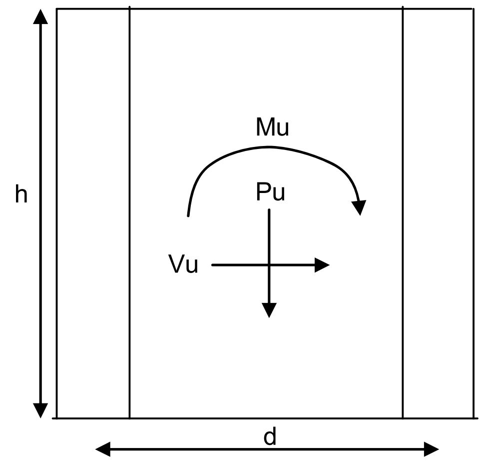
Engineer:	Joe Sharkey
Date:	3/19/2007
Job:	Christiana Hospital Project
Shear Wall #	12 - Ground Floor through 2nd

Material Properties

Concrete Strength - f'c (psi) =	5000
Reinforcement Strength - fy (psi) =	60000

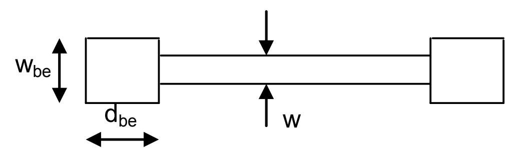
Wall Dimensions

Length - d (ft) =	18.5
Width - w (in) =	12
Height - h (ft) =	118



Boundary Element Dimensions

Length - d_{be} (in) =	26
Width - w_{be} (in) =	26



Wall Loads

Pu (kip) =	2148
Mu (ft-kip) =	28628
Vu (kip) =	498

Boundary Element

Axial Force - Pu_{be} (kip) = 2621.459

ACI 21.7.6.3

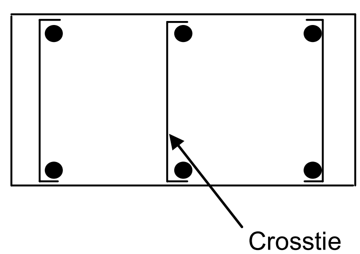
Boundary Element Check

Ag (ft^2) = 20.66667

Ig (in^4) = 735.5802

Extreme Fiber Comp. - Fc (ksi) = 3.514568 **Boundary Elemement Needed - fc>0.2 f'c**

Longitudinal & Transverse Reinforcement		
ACI 21.7.2.2 — **Two Curtains of Reinf. Req.**		
Acv (in^2/ft) =	144	
Longitudinal - ρ_l, Transverse - ρt >= 0.0025		
$As_{lreq'd}$ (in^2/ft) =	0.36	
$As_{supplied}$ (in^2) =	0.62	#5 Bars
Bar Diameter (in) =	0.625	
Required Spacing - $S_{req'd}$ (in) =	20.66667	**NOT OK Spacing Must Be Less Than 18in**
Spacing Supplied - $S_{supplied}$ (in) =	18	
Shear Capacity Check		
$\alpha_c = h_w/l_w$	2	**hw/lw>2 therefore use 2**
Acv_{total} (in^2) =	2976	
Transverse - ρt =	0.00287	
Nominal Shear Capacity - Vn (kip) =	933.4033	
Shear Capacity - ΦVn (kip) =	560.042	**OK**
Boundary Element Capacity Check		
Ast (in^2) =	37.44	24-#11
ρ_{st} =	0.055385	**OK**
Pn(max) (kip) =	3968.224	
Axial Load Capacity - ΦPn (kip) =	2777.757	**OK**
Check With Interaction Diagram		
Determine Confinement Reinforcement for Boundary Elements		
Max. Allowable Vert. Spacing - Smax (in) =	4	
Vert. Spacing Supplied - $S_{supplied}$ (in) =	4	
Short Direction (in) =	26	
Long Direction (in) =	26	
Bar Diameter (in) =	0.625	#5 Bar
Cover from center of Vert. Reinf. To Col. Face (in) =	3	
As of one Bar (in^2) =	0.31	
Area Bounded by out-to-out of hoops - Ach (in^2) =	478.5156	
Short Direction		
Number of Crossties In Short Direction =	4	
hc (in) =	21.25	
Req'd Reinf. In Short Direction - Ash (in^2) =	0.876992	
As provided (in^2) =	1.24	**OK**
Long Direction		
Number of Crossties In Short Direction =	4	
hc (in) =	21.25	
Req'd Reinf. In Short Direction - Ash (in^2) =	0.876992	
As provided (in^2) =	1.24	**OK**

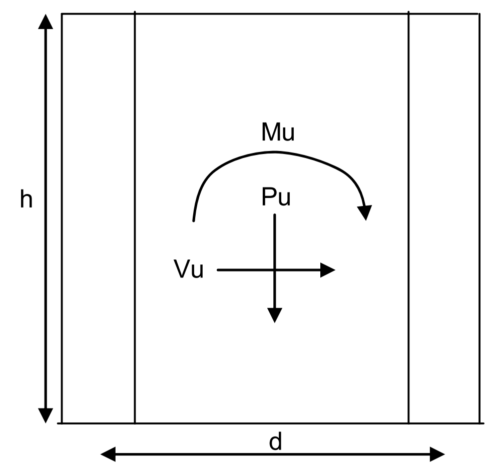


Shear Wall Design

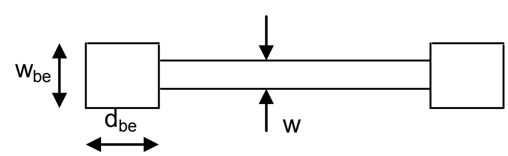
Engineer:	Joe Sharkey
Date:	3/19/2007
Job:	Christiana Hospital Project
Shear Wall #	12 - 3rd through 8th

Material Properties	
Concrete Strength - f'c (psi) =	4000
Reinforcement Strength - fy (psi) =	60000

Wall Dimensions	
Length - d (ft) =	18.5
Width - w (in) =	12
Height - h (ft) =	77.33



Boundary Element Dimensions	
Length - d_{be} (in) =	24
Width - w_{be} (in) =	24



Wall Loads	
Pu (kip) =	1492
Mu (ft-kip) =	14122
Vu (kip) =	306

Boundary Element	
Axial Force - Pu_{be} (kip) =	1509.351

ACI 21.7.6.3

Boundary Element Check	
Ag (ft^2) =	20.5
Ig (in^4) =	717.9271
Extreme Fiber Comp. - Fc (ksi) =	1.905579

Boundary Elemement Needed - fc>0.2 f'c

Longitudinal & Transverse Reinforcement

ACI 21.7.2.2

One Curtain of Reinf. Req.		
Acv (in^2/ft) =	144	
Longitudinal - ρ_l, Transverse - ρt >= 0.0025		
$As_{lreq'd}$ (in^2/ft) =	0.36	
$As_{supplied}$ (in^2) =	0.62	#5 Bars
Bar Diameter (in) =	0.625	
Required Spacing - $S_{req'd}$ (in) =	20.66667	**NOT OK Spacing Must Be Less Than 18in**
Spacing Supplied - $S_{supplied}$ (in) =	18	

Shear Capacity Check

$\alpha_c = h_w/l_w$	2	**hw/lw>2 therefore use 2**
Acv_{total} (in^2) =	2952	
Transverse - ρt =	0.00287	
Nominal Shear Capacity - Vn (kip) =	881.8017	
Shear Capacity - ΦVn (kip) =	529.081	**OK**

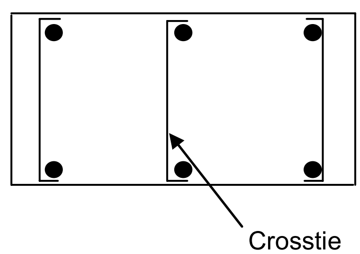
Boundary Element Capacity Check

Ast (in^2) =	18.72	12-#11
ρ_{st} =	0.0325	**OK**
Pn(max) (kip) =	2414.362	
Axial Load Capacity - ΦPn (kip) =	1690.053	**OK**

Check With Interaction Diagram

Determine Confinement Reinforcement for Boundary Elements

Max. Allowable Vert. Spacing - Smax (in) =	4	
Vert. Spacing Supplied - $S_{supplied}$ (in) =	4	
Short Direction (in) =	24	
Long Direction (in) =	24	
Bar Diameter (in) =	0.625	#5 Bar
Cover from center of Vert. Reinf. To Col. Face (in) =	3	
As of one Bar (in^2) =	0.31	
Area Bounded by out-to-out of hoops - Ach (in^2) =	395.0156	



Short Direction

Number of Crossties In Short Direction =	3	
hc (in) =	19.25	
Req'd Reinf. In Short Direction - Ash (in^2) =	0.705582	
As provided (in^2) =	0.93	**OK**

Long Direction

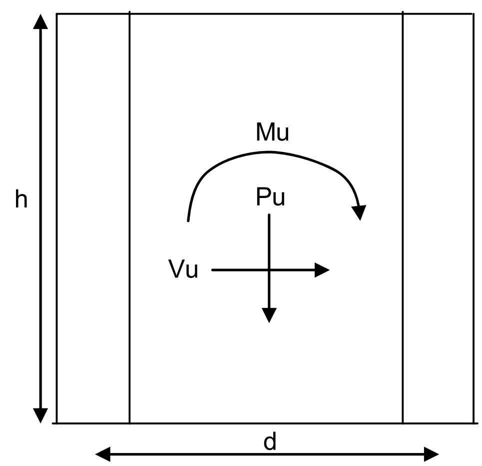
Number of Crossties In Short Direction =	3	
hc (in) =	19.25	
Req'd Reinf. In Short Direction - Ash (in^2) =	0.705582	
As provided (in^2) =	0.93	**OK**

Shear Wall Design

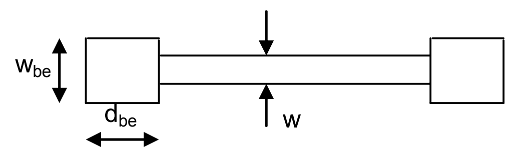
Engineer:	Joe Sharkey
Date:	3/19/2007
Job:	Christiana Hospital Project
Shear Wall #	25,26 - Ground through 2nd Floor

Material Properties	
Concrete Strength - f'c (psi) =	5000
Reinforcement Strength - fy (psi) =	60000

Wall Dimensions	
Length - d (ft) =	28
Width - w (in) =	12
Height - h (ft) =	46.3



Boundary Element Dimensions	
Length - d_{be} (in) =	24
Width - w_{be} (in) =	12



Wall Loads	
Pu (kip) =	2625
Mu (ft-kip) =	8091
Vu (kip) =	241

Boundary Element	
Axial Force - Pu_{be} (kip) =	1601.464

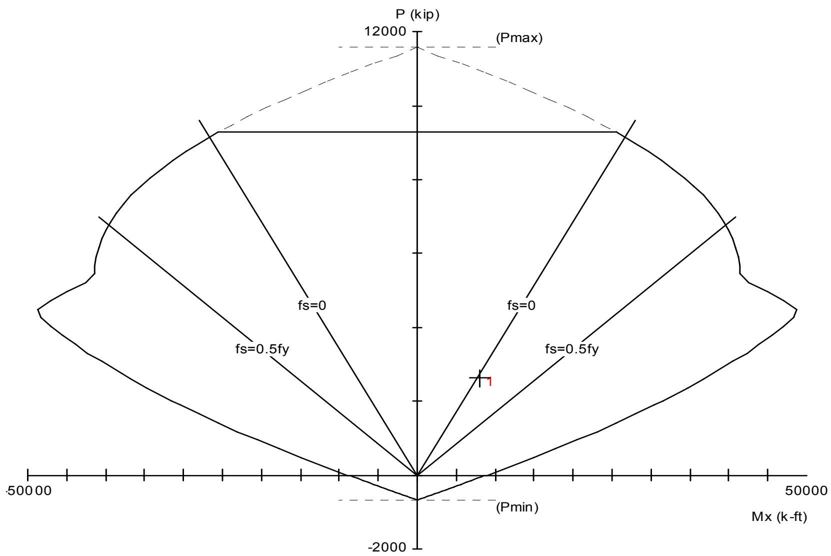
ACI 21.7.6.3

Boundary Element Check	
Ag (ft^2) =	30
Ig (in^4) =	2250
Extreme Fiber Comp. - Fc (ksi) =	0.982222

OK Without Boundary Element

ACI 21.7.2.2	**Longitudinal & Transverse Reinforcement**		
	One Curtain of Reinf. Req.		
	Acv (in^2/ft) =	144	
	Longitudinal - ρ_l, Transverse - ρt >= 0.0025		
	$As_{lreq'd}$ (in^2/ft) =	0.36	
	$As_{supplied}$ (in^2) =	0.62	#5 Bars
	Bar Diameter (in) =	0.625	
	Required Spacing - $S_{req'd}$ (in) =	20.66667	**NOT OK Spacing Must Be Less Than 18in**
	Spacing Supplied - $S_{supplied}$ (in) =	18	
	Shear Capacity Check		
	$\alpha_c = h_w/l_w$	1.543333	
	Acv_{total} (in^2) =	4320	
	Transverse - ρt =	0.00287	
	Nominal Shear Capacity - Vn (kip) =	1215.442	
	Shear Capacity - ΦVn (kip) =	729.2653	**OK**

Check With Interaction Diagram



Appendix B
Post-Tensioning Design

1	First Floor	Two-Way P-T Design	Main Tower

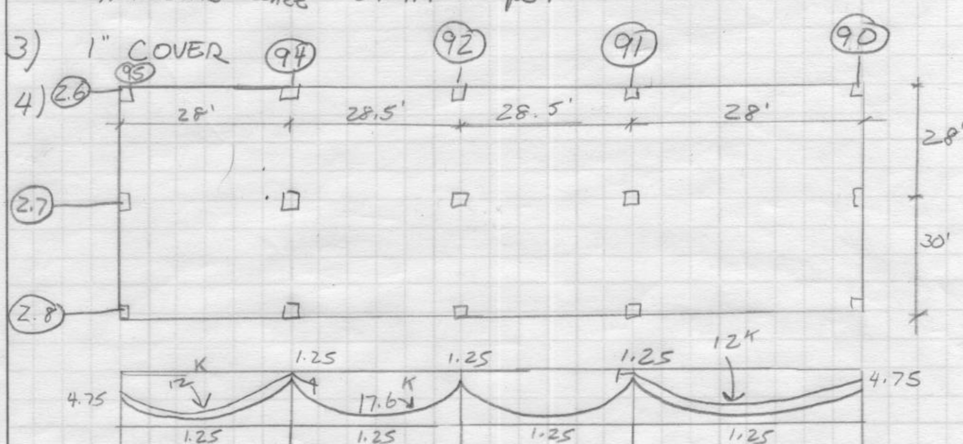
1) SLAB THICKNESS: 9.5"
LIVE LOAD = 100 psf   DL = 15 psf
COLUMN SIZE = 24" SQUARE
$f'_c = 5000$ psi
1/2" ϕ - 270 Ksi STRAND (ASTM A416) $A_{ps} = 0.153\ in^2$

2) LOAD TO BE BALANCED
9 1/2" SLAB = 119 psf
DL = 15 psf
LL = 100 psf
234 psf ↓

$W_{pre} = 1.0 \times 119 = 119$ psf ↑
NET LOAD = $W_{net} = 234 - 119 = 115$ psf

3) 1" COVER

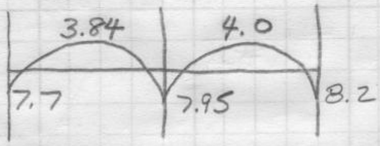
4)



Wpre	119	119	119	119
Mpre	11.7	12.1	12.1	11.7
a (in)	4.75	8.25	8.25	4.75
F (K)	29.6	17.6	17.6	29.6
F/A (psi)	260	154	154	260

FLEXURAL ANALYSIS

$w_{pre} = 119$			
	$\frac{1}{28}$	$\frac{1}{28.5}$	
0	0.5	0.5	0
-7.8	7.8	-8.1	8.1
+.1 ←	+.15	+.15 →	+.1
-7.7	7.95	7.95	8.2



AVG. STRESS $A = 12 \times 9.5 = 114 \text{ in}^2$ $S = \frac{bh^2}{6} = \frac{12(9.5)^2}{6} = 180.5 \text{ in}^3$

- NEG. MOMENT

INT SPAN

$$f = -154 \pm \frac{12 \times 8.2 \times 1000}{180.5} = -154 \pm 545 = 391.2 < 6\sqrt{f'c} = 424$$
$$= -699 < .6(5000) = 3000$$

EXT. SPAN

$$f = -260 \pm \frac{12 \times 7.95 \times 1000}{180.5} = -260 \pm 528.5 = 268.5 < 424$$
$$= -788.5 < 3000$$

- POS. MOMENT

INT SPAN

$$f = -154 \pm \frac{12 \times 3.84 \times 1000}{180.5} = -154 \pm 255.3 = 101.3 < 3\sqrt{f'c} = 212$$
$$= -409.3 < 3000$$

EXT SPAN

$$f = -260 \pm \frac{12 \times 4 \times 1000}{180.5} = -260 \pm 266 = 6 < 212$$
$$= -526 < 3000$$

3

5) ULTIMATE STRENGTH CHECK

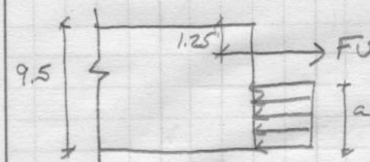
$w_u = 7.2(134) = 160.8$
$1.6(100) = 160$
320.8

$d = 9.5 - 1.25 = 8.25$

$17.6^k \times \frac{1 in^2}{175k} \times \frac{1}{.153} = 0.66 \frac{STRANDS}{ft} \times \frac{28+30}{2} = 19.14 \approx 20$

(ACI) $\rho_p = \frac{A_{ps}}{bd} = \frac{0.66(.153)}{12(8.25)} = 0.00102$

(ACI 18-5) $f_{su} = 175 + \frac{1.0(5)}{100(.00102)} + 10 = 224\ ksi$



$F_{ULT} = \frac{224}{175} \times 20 = 25.6$

$f_{cULT} = .85(5000) = 4250 psi = 4.25 ksi$

$a = \frac{25.6}{12 \times 4.25} = 0.5$

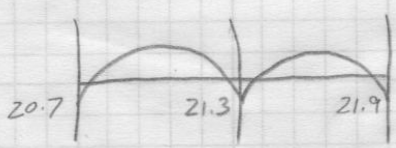
$jd = 9.5 - \frac{.5}{2} - 1.25 = 8''$

$M_u = .9 \times 25.6 \times \frac{8}{12} = 15.36^{1k}$

6) REQUIRED MOMENT

320.8

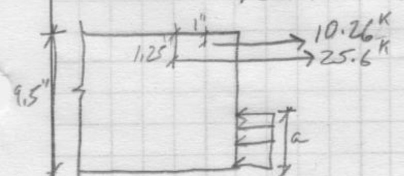
	1/28	1/28.5	
0	0.5	0.5	0
+20.9	20.9	-21.7	21.7
.2 ←	+.4	+.4 →	.2
20.7	21.3	-21.3	21.9



$> 15.36^{1k}$ REQUIRES REINF.

$A_{smin} = .0015 \times 9.5 \times 12 \times 29 = 5\ in^2$

$\frac{5}{29} = 0.172\ in^2/ft$ $F_u = 0.172 \times 60 = 10.26^k$



$a = \frac{35.86}{12 \times 4.25} = 0.7$

$jd_p = 9.5 - 1.25 - \frac{.7}{2} = 7.9$
$jd_R = 9.5 - 1 - \frac{.7}{2} = 8.15$

$M_u = \frac{.9}{12}\left[25.6(7.9) + 10.26(8.15)\right] = 21.4^{1k} \approx 21.9 \therefore OK$

7) SHEAR DESIGN @ COLUMNS

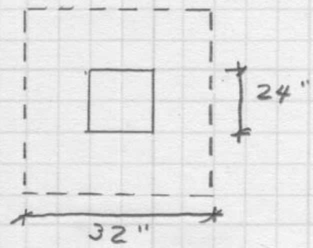
$t/2 = \frac{9.5}{2} = 4.5''$

$b' = 4.5 \times 32 = 144''$

$d = 9.5 - 1.25 = 8.25''$

$f_{pc} = \frac{260+154+154+154}{4}$

$= 180.5\ psi$



$V_{cw} = b'd(3.5\sqrt{f'_c} + 0.3 f_{pc})$

$= \frac{144(8.25)}{1000}(3.5\sqrt{5000} + 0.3(180.5)) = 358.3^{K}$

REQUIRED CAPACITY

$1.2D + 1.6L$

$1.2(115_{psf}) + 1.6(100_{psf}) = 298_{psf}$

$\left(\frac{28}{2} + \frac{30}{2}\right) \times \left(\frac{28.5 \times 2}{2}\right) - \frac{32}{12} \times \frac{32}{12} = 570.5^{\phi}$

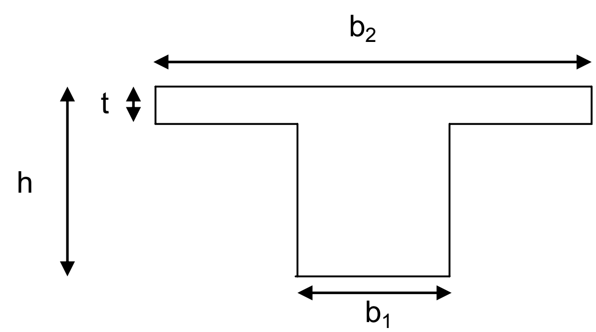
$\frac{298_{psf} \times 570.5 sf}{1000} = 170^{K} < 358.3^{K} \therefore OK$

PT Beam-and-Slab Design (Conference Wing)	
Engineer:	Joe Sharkey
Date:	3/14/2007
Job:	Christiana Hospital Project
Beam #:	PB-1

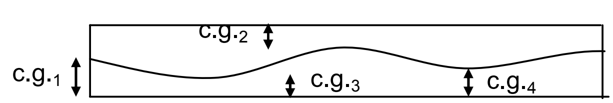
Load			
Live Load (psf) =	100		
Superimposed Dead Load (psf) =	15		
Slab Weight (psf) =	187.5		
Prestressing - w_{pslab} (psf) =	-187.5		
Net Load - w_{nslab} (psf) =	100		
Slab Weight (plf) =	5250	x 1.2	6300
Beam Weight (plf) =	675	x 1.2	810
Live Load (plf) =	2800	x 1.6	4480
Prestressing - w_{pbeam} (plf) =	-4740		
Net Load - w_{nbeam} (plf) =	3985		

Concrete Properties	
Concrete Weight (pcf) =	150
Concrete Strength - f'c (psi) =	5000

Beam/Slab Dimensions	
Slab Thickness - t (in) =	15
Beam Height - h (in) =	42
Beam Width - b_1 (in) =	24
Span (ft) =	62
Beam Spacing (ft) =	28
Effective Flange Width - b_2 (in) =	264
Total Beam Area (in^2) =	4608
Y_{top} (in) =	10.453125
Y_{bottom} (in) =	31.546875
I (in^4) =	359197.875
S_{top} (in^3) =	34362.72646
S_{bottom} (in^3) =	11386.16345
S_{slab} (in^3) =	450



Prestressing	
Slab % Prestress =	100
Beam % Prestress =	80
Unbonded Strand Type =	1/2" Φ - 270ksi (ASTM A461)
Prestressing - w_{pslab} (psf) =	-187.5
Prestressing - w_{pbeam} (plf) =	-4740
$c.g._{slab\ 1}$ (in) =	7.5
$c.g._{slab\ 2}$ (in) =	1.25
$c.g._{slab\ 3}$ (in) =	1.25



c.g.$_{beam\ 1}$ (in) =	10.453125
c.g.$_{beam\ 2}$ (in) =	4
c.g.$_{beam\ 3}$ (in) =	4
c.g.$_{beam\ 4}$ (in) =	7.2265625

Design Stresses

Slab

Interior Spans

M_p (ft-kip) =	18.375	
a (in) =	12.5	
F (k/ft) =	17.64	
# of Strands/ft =	0.658823529	
F/A (psi) =	98	
CL M_n (ft-kip) =	7.127272727	
Va/3 (ft-kip) =	0.466666667	
M (ft-kip) =	6.660606061	
S (in^3) =	450	
f (psi) =	79.61616162	< 6√f'c therefore OK
	-275.6161616	< .6f'c therefore OK

Exterior Spans

M_p (ft-kip) =	18.375	
a (in) =	16.875	
F (k/ft) =	13.06666667	
# of Strands/ft =	0.488017429	
F/A (psi) =	72.59259259	
CL M_n (ft-kip) =	7.84	
Va/3 (ft-kip) =	0.466666667	
M (ft-kip) =	7.373333333	
S (in^3) =	450	
f (psi) =	124.0296296	< 6√f'c therefore OK
	-269.2148148	< .6f'c therefore OK

Beam

All Spans

M_p (ft-kip) =	2277.57	
a (in) =	30.7734375	
F (k/ft) =	888.1308759	
F/A (psi) =	192.7367352	
# of Strands =	29	
$F_{e\ supplied}$ (kip) =	892.4296875	
S_{top} (in^3) =	34362.72646	
S_{bottom} (in^3) =	11386.16345	

Positive Moment

M^+ (ft-kip) =	233	
f (psi) =	52.82446834	< 6√f'c therefore OK
	-274.1039691	< .6f'c therefore OK

Negative Moment	
M^- (ft-kip) =	268
f (psi) =	-99.14695548
	-475.1848148

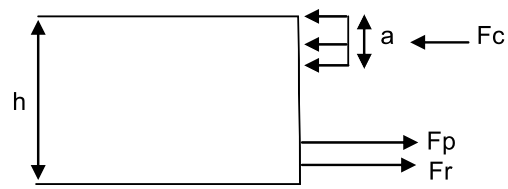
< 6√f'c therefore OK

< .6f'c therefore OK

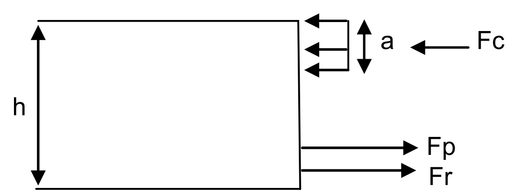
Ultimate Strength Design

Slab

Ultimate Strength @ Interior Span			
M^+ max (ft-kip/ft) =	18		
M^- max (ft-kip/ft) =	25		
A_{smin} (in^2/ft) =	0.27		
$A_{ssupplied}$ (in^2/ft) =	0.31	1-#5	**Min Req Steel Met**
Rebar Cover (in) =	1		
Fp (kips) =	17.64		
Fr (kips) =	18.6		
a (in) =	0.710588235	**Compression within slab therefore OK**	
Mu (ft-kips/ft) =	36.75556059	**OK**	

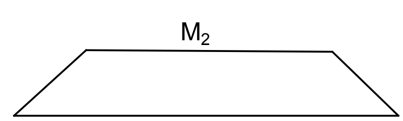


Ultimate Strength @ Exterior Span			
M^+ max (ft-kip/ft) =	18		
M^- max (ft-kip/ft) =	36		
A_{smin} (in^2/ft) =	0.27		
$A_{ssupplied}$ (in^2/ft) =	0.6	1-#7	**Min Req Steel Met**
Rebar Cover (in) =	1		
Fp (kips) =	13.06666667		
Fr (kips) =	36		
a (in) =	0.962091503	**Compression within slab therefore OK**	
Mu (ft-kips/ft) =	49.50475163	**OK**	



Beam

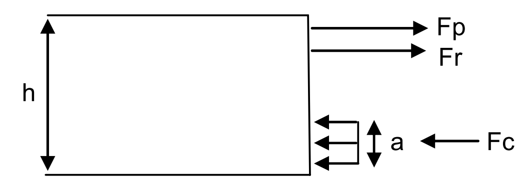
M^+ max (ft-kip) =	2560
M^- max (ft-kip) =	3400
Secondary Moments	
Wp (klf) =	4.762942978
Mp = $M_1 + M_2$ (ft-kip) =	1830.875281
M_1 (ft-kip) =	479.9133606
M_2 (ft-kip) =	1350.96192



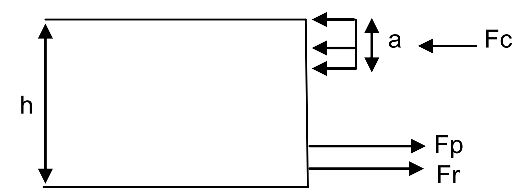
M^+ total (ft-kip) =	3235.48096		
M^- total (ft-kip) =	2049.03808		

f_{su} not to exceed f_{sy} =			235
ρ @ midspan exterior =	0.000442285	f_{su} (ksi) =	235
ρ @ support =	0.004865132	f_{su} (ksi) =	195.2772

ltimate Strength @ Interior Support			
A_{smin} (in^2) =	3.0285		
$A_{ssupplied}$ (in^2) =	3.16	4-#8	Min Req Steel Met
Rebar Cover (in) =	2		
Fp (kips) =	1198.40558		
Fr (kips) =	189.6		
a (in) =	1.237081622	Compression within slab therefore OK	
Mu (ft-kips) =	3919.865547	OK	



Ultimate Strength @ Midspan			
A_{smin} (in^2) =	1.0035		
$A_{ssupplied}$ (in^2) =	6	6-#9	Min Req Steel Met
Rebar Cover (in) =	2		
Fp (kips) =	995.8353335		
Fr (kips) =	360		
a (in) =	13.29250327	Compression within beam therefore OK	
Mu (ft-kips) =	3242.28899	OK	

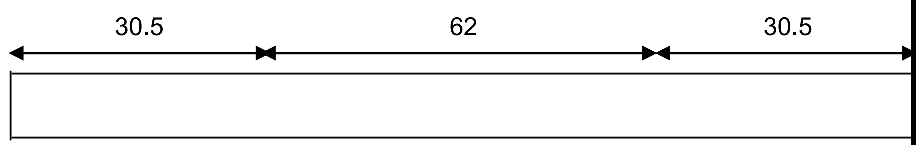


Shear			
Line Load on Beam (klf) =	12.544		
Vu (kips) =	443.7027097		
Vu (psi) =	453.6837522		
Vc (psi) =	321.0965718	NOT OK Need Shear Reinf	
Stirrup Spacing - s (in) =	5		
fy (psi) =	60000		
Cross-Sectional Area of Steel -Av (in^2) =	0.4	2-#4	
Max s (in) =	18.85618083		
d (in) =	38		
Vc from stirrups (psi) =	135.7142857		
Vc total with stirrups =	456.8108575	OK	

PB-1 Moment Distribution

		(plf)		(plf)
Beam Weight =	150	1050	x 1.2 =	1260
Slab Weight =	150	5250	x 1.2 =	6300
Dead Load =	15	420	x 1.2 =	504
Live Load =	100	2800	x 1.6 =	4480
Total =	415			12544

Beam Dim.	b (in)	h (in)		
	24	42	Slab Thickness (in) =	15
			Tributary Width (ft) =	28



DF	0	0.67027	0.32973	0.32972973	0.67027	0
FEM	-972.4213	972.4213	-4018.261	4018.261333	-972.4213	972.4213
		2041.536	1004.304			
	1020.768			502.152		
				-1169.878443	-2378.114	
			-584.9392			-1189.057
		392.0674	192.8719			
	196.0337			96.43592573		
				-31.79779173	-64.63813	
			-15.8989			-32.31907
		10.65656	5.242339			
	5.328279			2.621169318		
				-0.864277451	-1.756892	
						-0.878446
Total (ft-kips)	249.7086	3417.352	-3416.352	3417.259646	-3416.26	-249.833
Positive Moment	-125.1895		2610.586		-124.5813	

PT Beam-and-Slab Design (Conference Wing)

Engineer:	Joe Sharkey
Date:	3/14/2007
Job:	Christiana Hospital Project
Beam #:	PB-2

Load

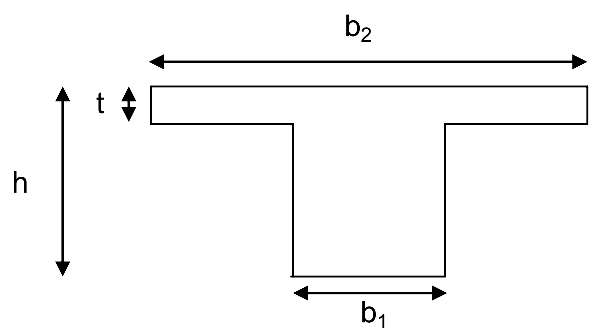
Live Load (psf) =	100	
Superimposed Dead Load (psf) =	15	
Slab Weight (psf) =	187.5	
Prestressing - w_{pslab} (psf) =	-187.5	
Net Load - w_{nslab} (psf) =	100	
Slab Weight (plf) =	6187.5 x 1.2	7425
Beam Weight (plf) =	506.25 x 1.2	607.5
Live Load (plf) =	3300 x 1.6	5280
Prestressing - w_{pbeam} (plf) =	-6024.375	
Net Load - w_{nbeam} (plf) =	3969.375	

Concrete Properties

Concrete Weight (pcf) =	150
Concrete Strength - f'c (psi) =	5000

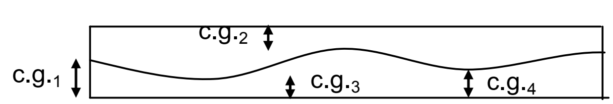
Beam/Slab Dimensions

Slab Thickness - t (in) =	15
Beam Height - h (in) =	42
Beam Width - b_1 (in) =	18
Span (ft) =	41.8
Beam Spacing (ft) =	33
Effective Flange Width - b_2 (in) =	258
Total Beam Area (in^2) =	4356
Y_{top} (in) =	9.842975207
Y_{bottom} (in) =	32.15702479
I (in^4) =	292500.595
S_{top} (in^3) =	29716.68514
S_{bottom} (in^3) =	9096.009252
S_{slab} (in^3) =	450



Prestressing

Slab % Prestress =	100
Beam % Prestress =	90
Unbonded Strand Type =	1/2" Φ - 270ksi (ASTM A461)
Prestressing - w_{pslab} (psf) =	-187.5
Prestressing - w_{pbeam} (plf) =	-6024.375
$c.g._{slab\ 1}$ (in) =	7.5
$c.g._{slab\ 2}$ (in) =	1.25
$c.g._{slab\ 3}$ (in) =	1.25



c.g.$_{beam\ 1}$ (in) =	9.842975207
c.g.$_{beam\ 2}$ (in) =	2.5
c.g.$_{beam\ 3}$ (in) =	2.5
c.g.$_{beam\ 4}$ (in) =	6.171487603

Design Stresses

Slab

Interior Spans

M_p (ft-kip) =	25.5234375	
a (in) =	12.5	
F (k/ft) =	24.5025	
# of Strands/ft =	0.91512605	
F/A (psi) =	136.125	
CL M_n (ft-kip) =	9.9	
Va/3 (ft-kip) =	0.55	
M (ft-kip) =	9.35	
S (in^3) =	450	
f (psi) =	113.2083333	< 6√f'c therefore OK
	-385.4583333	< .6f'c therefore OK

Exterior Spans

M_p (ft-kip) =	25.5234375	
a (in) =	16.875	
F (k/ft) =	18.15	
# of Strands/ft =	0.677871148	
F/A (psi) =	100.8333333	
CL M_n (ft-kip) =	10.89	
Va/3 (ft-kip) =	0.55	
M (ft-kip) =	10.34	
S (in^3) =	450	
f (psi) =	174.9	< 6√f'c therefore OK
	-376.5666667	< .6f'c therefore OK

Beam

All Spans

M_p (ft-kip) =	1315.753622	
a (in) =	33.3285124	
F (k/ft) =	473.73982	
F/A (psi) =	108.7556979	
# of Strands =	15	
$F_{e\ supplied}$ (kip) =	499.927686	
S_{top} (in^3) =	29716.68514	
S_{bottom} (in^3) =	9096.009252	

Positive Moment

M^+ (ft-kip) =	233	
f (psi) =	198.6318523	< 6√f'c therefore OK
	-202.8442541	< .6f'c therefore OK

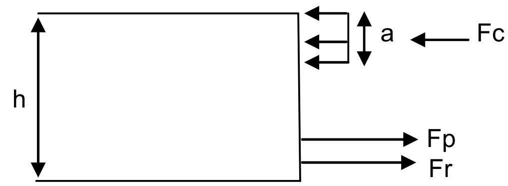
Negative Moment	
M^- (ft-kip) =	268
f (psi) =	-0.533667562
	-462.3173435

< 6√f'c therefore OK
< .6f'c therefore OK

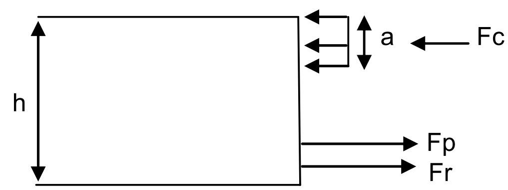
Ultimate Strength Design

Slab

Ultimate Strength @ Interior Span			
M^+ max (ft-kip/ft) =	18		
M^- max (ft-kip/ft) =	25		
A_{smin} (in^2/ft) =	0.27		
$A_{ssupplied}$ (in^2/ft) =	0.31	1-#5	**Min Req Steel Met**
Rebar Cover (in) =	1		
Fp (kips) =	24.5025		
Fr (kips) =	18.6		
a (in) =	0.845147059	**Compression within slab therefore OK**	
Mu (ft-kips/ft) =	43.43215496	**OK**	

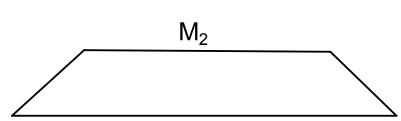


Ultimate Strength @ Exterior Span			
M^+ max (ft-kip/ft) =	18		
M^- max (ft-kip/ft) =	36		
A_{smin} (in^2/ft) =	0.27		
$A_{ssupplied}$ (in^2/ft) =	0.6	1-#7	**Min Req Steel Met**
Rebar Cover (in) =	1		
Fp (kips) =	18.15		
Fr (kips) =	36		
a (in) =	1.061764706	**Compression within slab therefore OK**	
Mu (ft-kips/ft) =	54.36114154	**OK**	



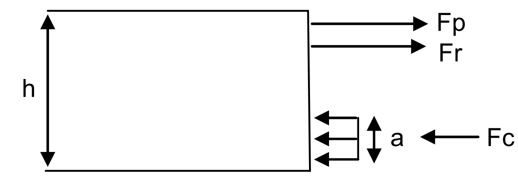
Beam

M^+ max (ft-kip) =	1957
M^- max (ft-kip) =	2243
Secondary Moments	
Wp (klf) =	6.357396457
Mp = M_1 + M_2 (ft-kip) =	1110.789739
M_1 (ft-kip) =	305.9130503
M_2 (ft-kip) =	804.8766883

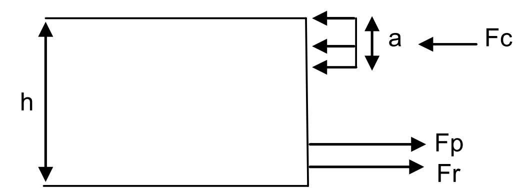


M^{+} total (ft-kip) =	2359.438344		
M^{-} total (ft-kip) =	1438.123312		
f_{su} not to exceed f_{sy} =			235
ρ @ midspan exterior =	0.000225199	f_{su} (ksi) =	235
ρ @ support =	0.003227848	f_{su} (ksi) =	200.4902

timate Strength @ Exterior Support			
A_{smin} (in^2) =	2.315305785		
$A_{ssupplied}$ (in^2) =	3	3-#9	**Min Req Steel Met**
Rebar Cover (in) =	2		
Fp (kips) =	671.331464		
Fr (kips) =	180		
a (in) =	0.776408084	**Compression within slab therefore OK**	
Mu (ft-kips) =	2504.032688	**OK**	



Ultimate Strength @ Midspan			
A_{smin} (in^2) =	0.708694215		
$A_{ssupplied}$ (in^2) =	6.08	6-#9	**Min Req Steel Met**
Rebar Cover (in) =	2		
Fp (kips) =	572.7462845		
Fr (kips) =	364.8		
a (in) =	12.25550699	**Compression within beam therefore OK**	
Mu (ft-kips) =	2360.281929	**OK**	

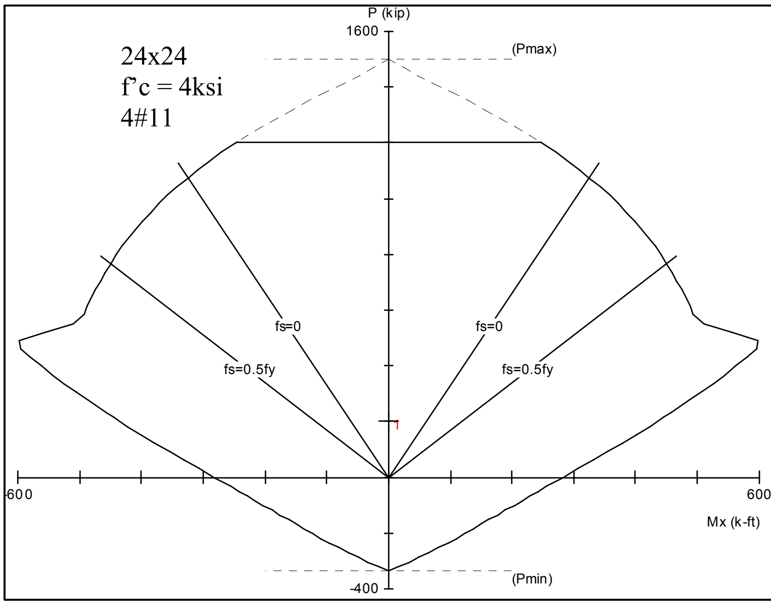


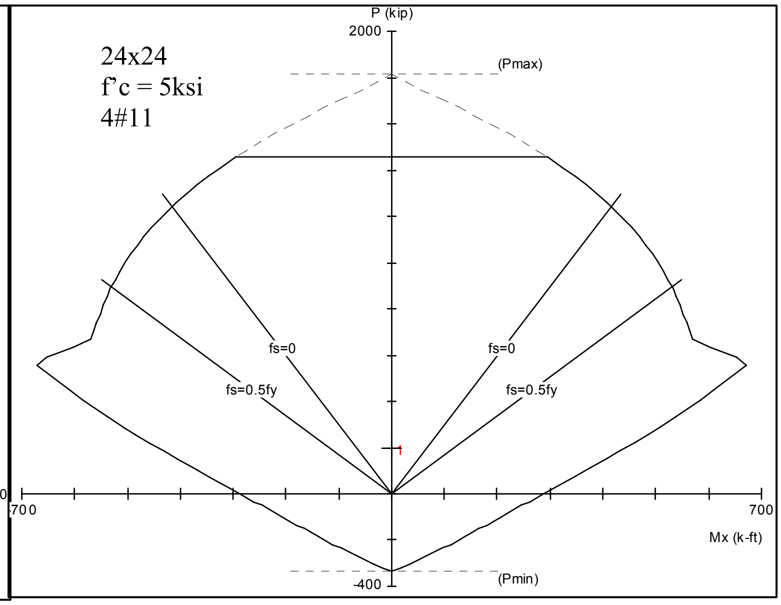
Shear			
Line Load on Beam (klf) =	14.244		
Vu (kips) =	351.3598871		
Vu (psi) =	479.018251		
Vc (psi) =	392.7612852	**NOT OK Need Shear Reinf**	
Stirrup Spacing - s (in) =	10		
fy (psi) =	60000		
Cross-Sectional Area of Steel -Av (in^2) =	0.4	2-#4	
Max s (in) =	25.14157444		
d (in) =	39.5		
Vc from stirrups (psi) =	94.04761905		
Vc total with stirrups =	486.8089043	**OK**	

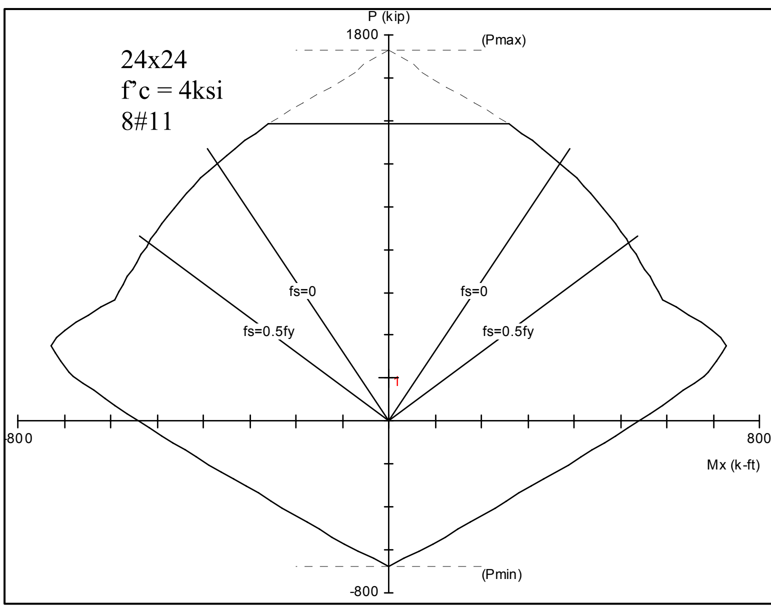
PB-2 Moment Distribution

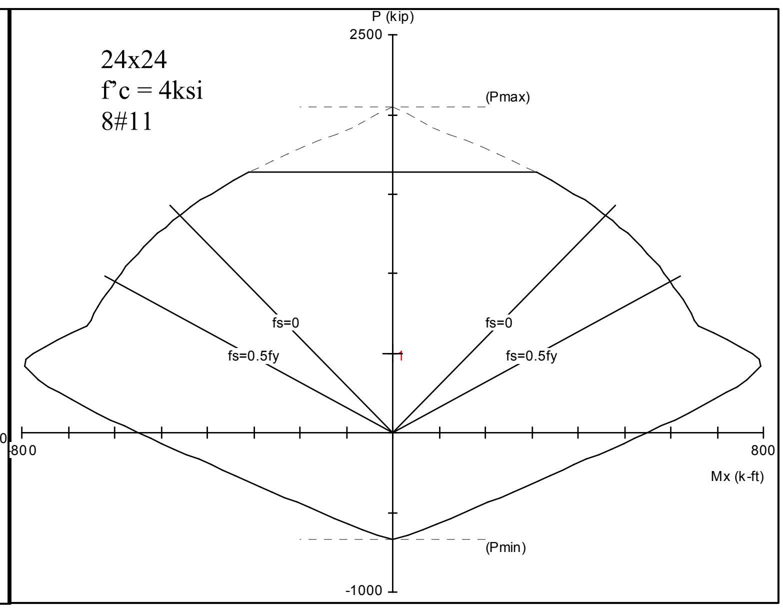
				(plf)		(plf)
		Beam Weight =	150	666.6666667	x 1.2 =	800
		Slab Weight =	150	6187.5	x 1.2 =	7425
		Dead Load =	15	495	x 1.2 =	594
		Live Load =	100	3300	x 1.6 =	5280
		Total =	415			14099
Beam Dim.	b (in)	h (in)				
	16	40		Slab Thickness (in) =	15	
				Tributary Width (ft) =	33	
	41.8			31.5	21.25	
DF	1	0.429741	0.570259	0.402843602	0.597156	1
FEM	-2052.861	2052.861	-1165.811	1165.811063	-530.5483	530.5483
	2052.861					
		1026.431				
		-822.3009	-1091.18			
	-411.1504			-545.5900898		
				-36.12405954	-53.54861	
			-18.06203			-26.7743
		7.761991	10.30004			-503.774
	3.880995			5.150019403	-251.887	
				99.39641486	147.3406	
			49.69821			73.67028
		-21.35735	-28.34086			
	-10.67867			-14.17043022		
	417.9481			5.708467151	8.461963	
						4.230982
						-77.90127
Total (ft-kips)	0	2243.396	-2243.396	680.1813844	-680.1814	0
Positive Moment	1957.594		286.928		455.7318	

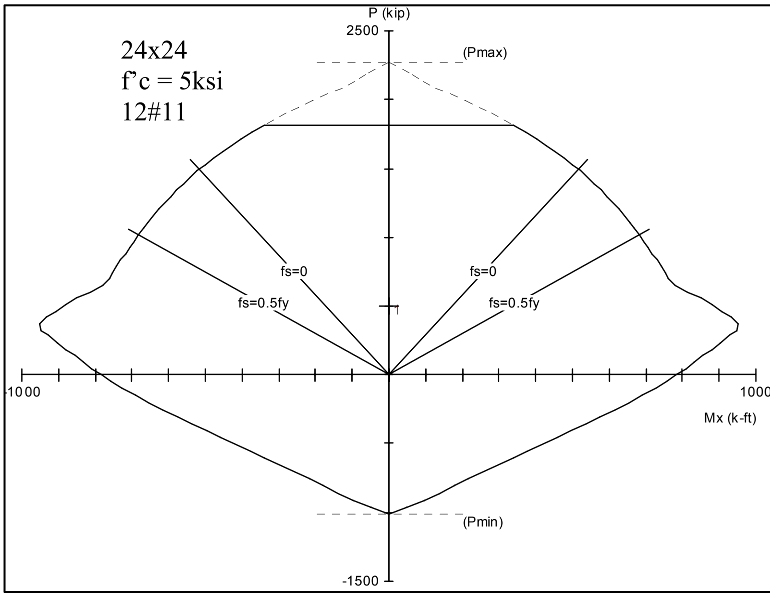
Appendix C
Column Design

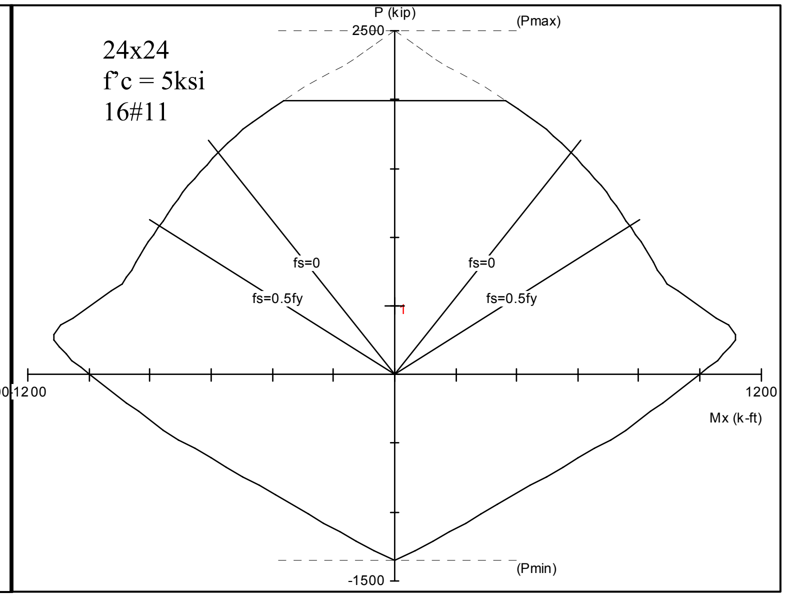


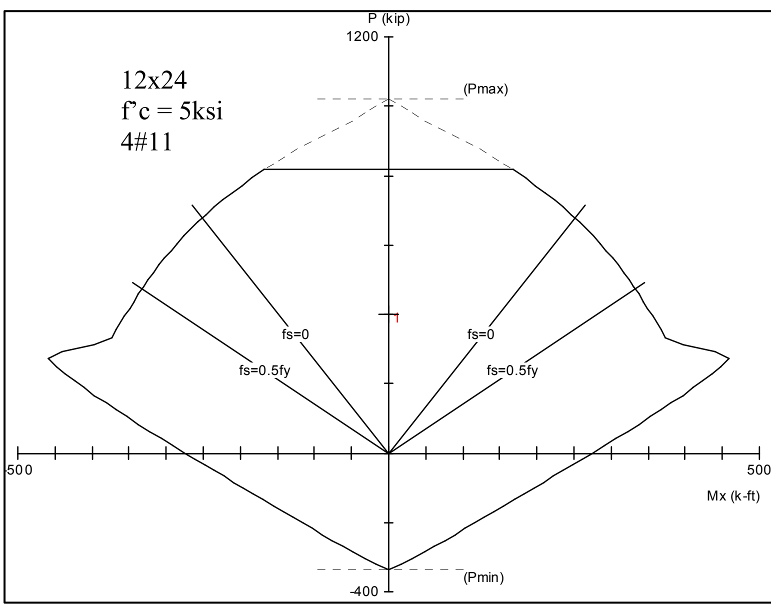


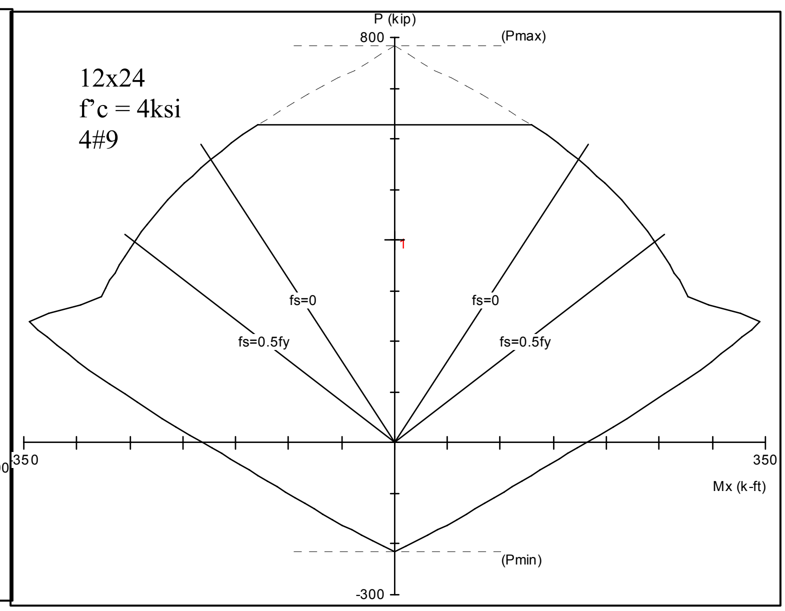


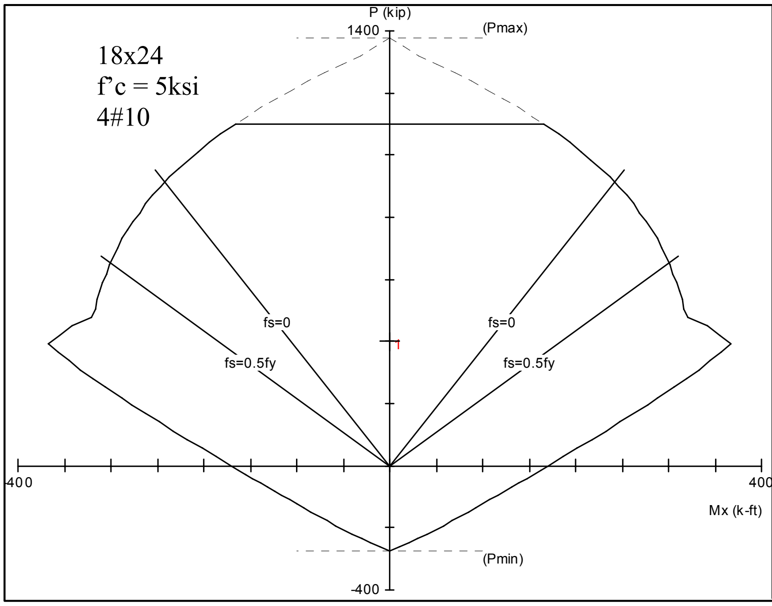


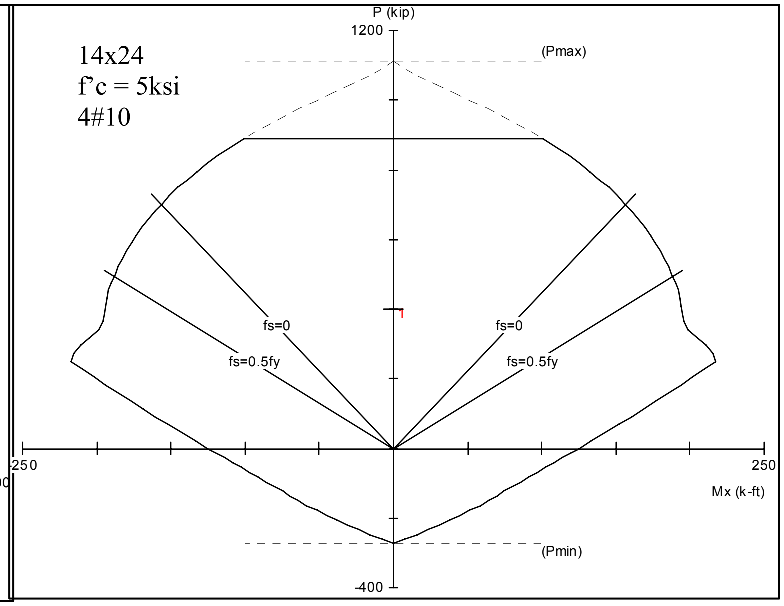

24x24
f'c = 5ksi
12#11
P (kip)
2500
-1500
(Pmax)
(Pmin)
fs=0
fs=0.5fy
-1000
1000
Mx (k-ft)

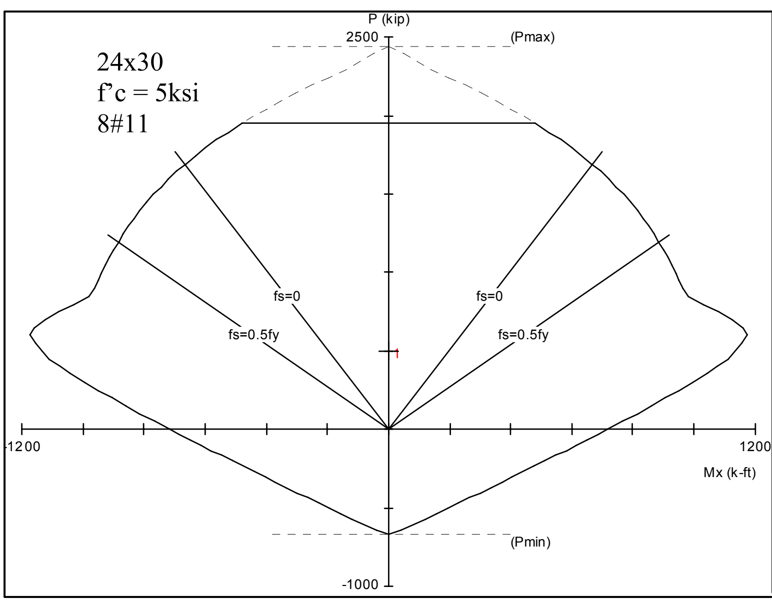

24x24
f'c = 5ksi
16#11
P (kip)
2500
-1500
(Pmax)
(Pmin)
fs=0
fs=0.5fy
-1200
1200
Mx (k-ft)

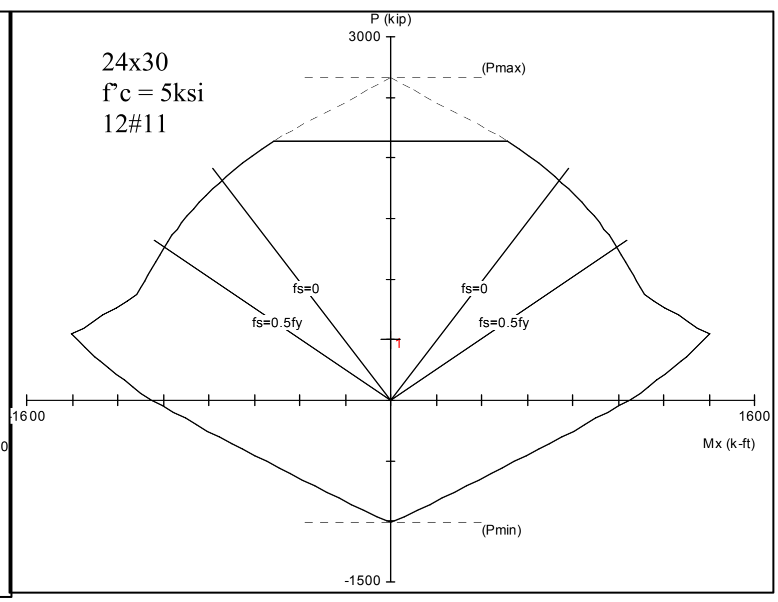

12x24
f'c = 5ksi
4#11
P (kip)
1200
-400
(Pmax)
(Pmin)
fs=0
fs=0.5fy
-500
500
Mx (k-ft)

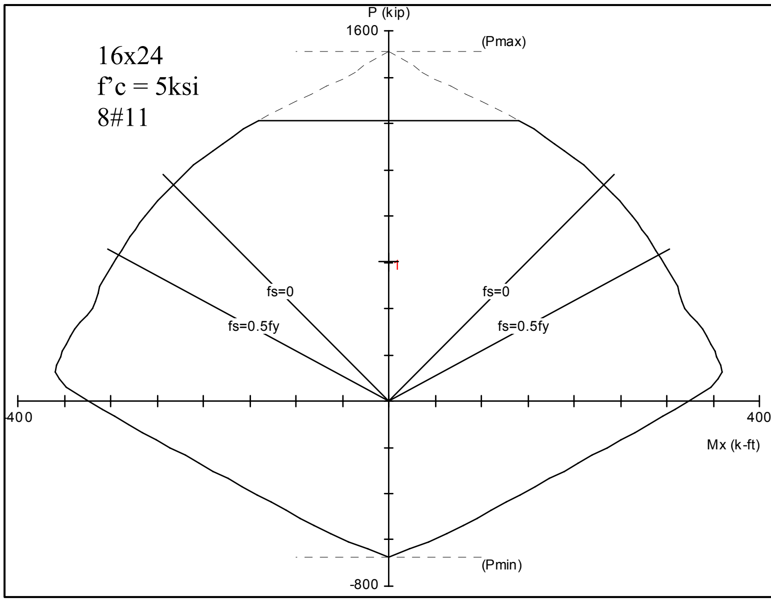

12x24
f'c = 4ksi
4#9
P (kip)
800
-300
(Pmax)
(Pmin)
fs=0
fs=0.5fy
-350
350
Mx (k-ft)

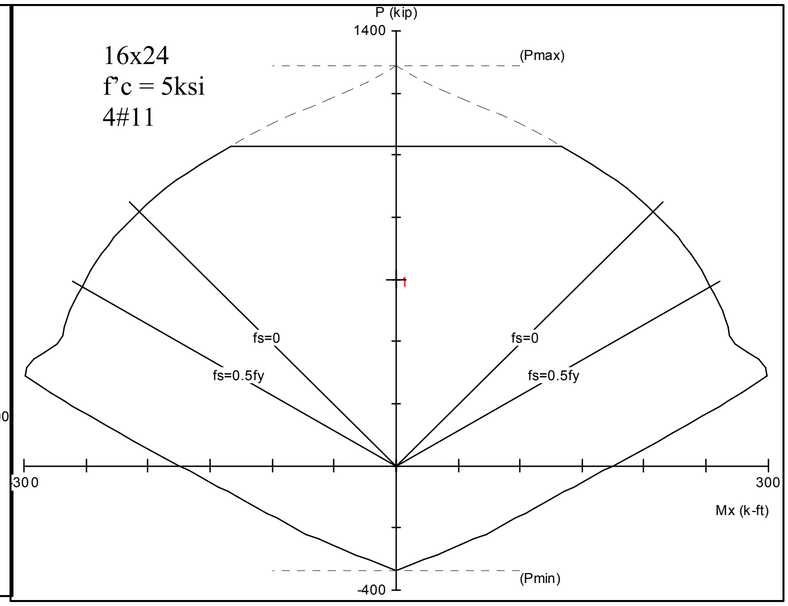

18x24
f'c = 5ksi
4#10
P (kip)
1400
(Pmax)
fs=0
fs=0
fs=0.5fy
fs=0.5fy
-400
400
Mx (k-ft)
(Pmin)
-400

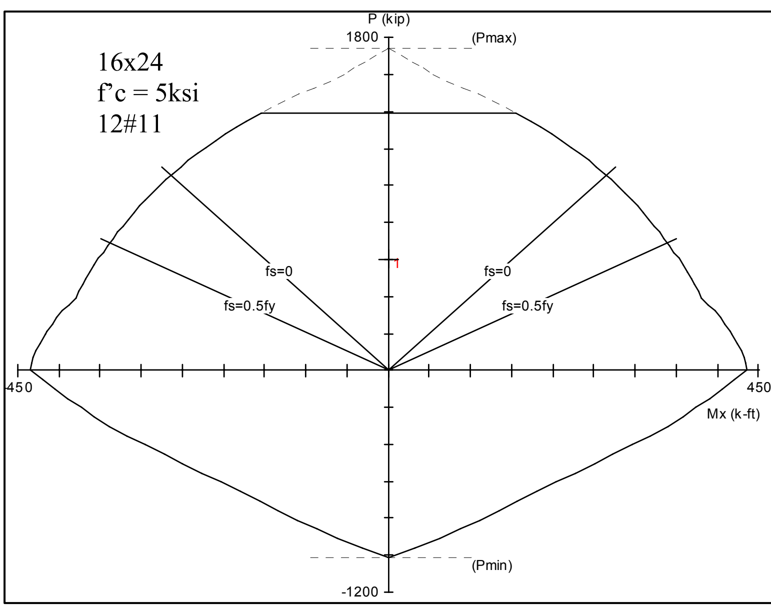

14x24
f'c = 5ksi
4#10
P (kip)
1200
(Pmax)
fs=0
fs=0
fs=0.5fy
fs=0.5fy
-250
250
Mx (k-ft)
(Pmin)
-400

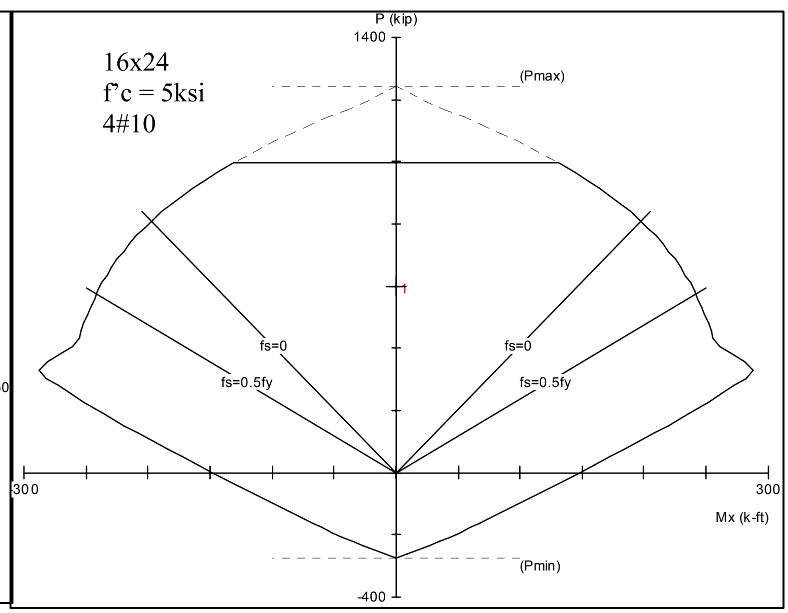

24x30
f'c = 5ksi
8#11
P (kip)
2500
(Pmax)
fs=0
fs=0
fs=0.5fy
fs=0.5fy
-1200
1200
Mx (k-ft)
(Pmin)
-1000

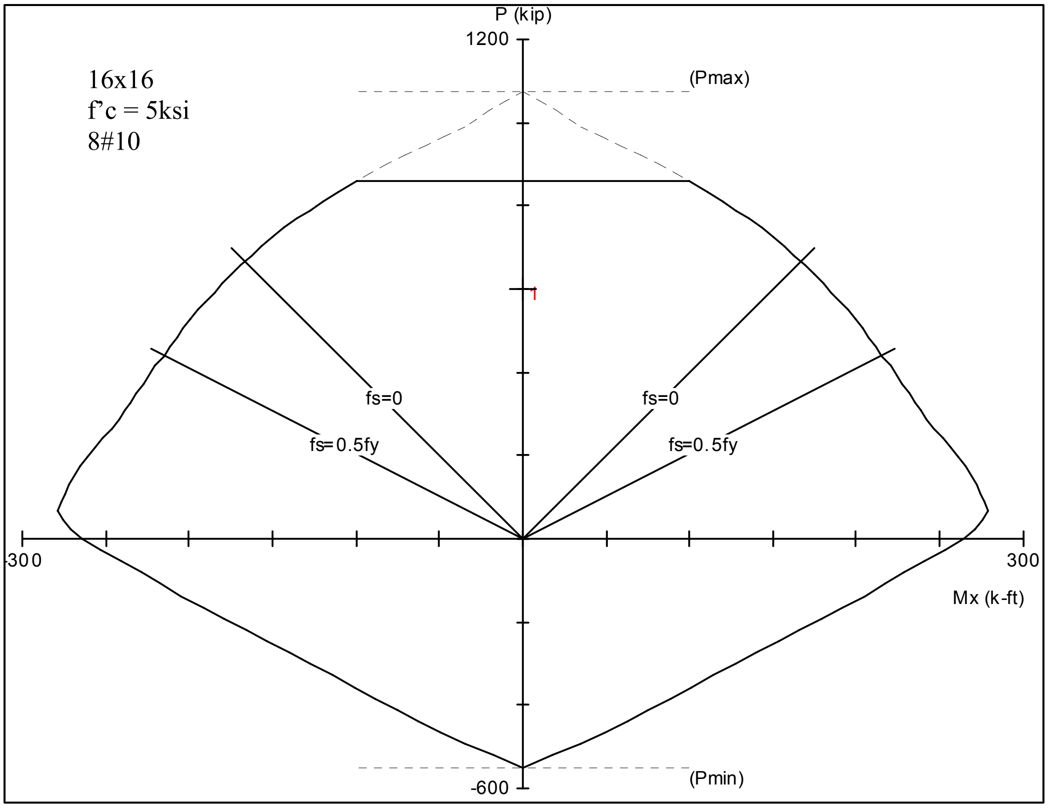

24x30
f'c = 5ksi
12#11
P (kip)
3000
(Pmax)
fs=0
fs=0
fs=0.5fy
fs=0.5fy
-1600
1600
Mx (k-ft)
(Pmin)
-1500

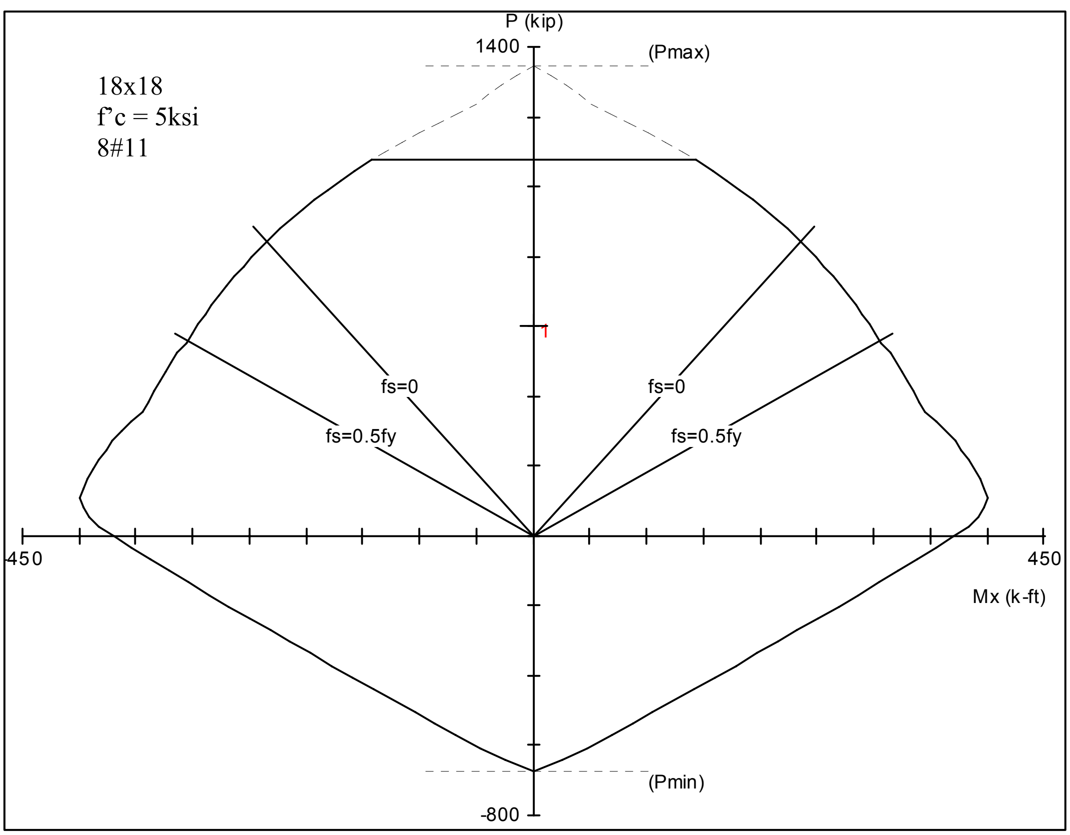

16x24
f'c = 5ksi
8#11
P (kip)
1600
(Pmax)
fs=0
fs=0
fs=0.5fy
fs=0.5fy
400
400
Mx (k-ft)
(Pmin)
-800


16x24
f'c = 5ksi
4#11
P (kip)
1400
(Pmax)
fs=0
fs=0
fs=0.5fy
fs=0.5fy
300
300
Mx (k-ft)
(Pmin)
-400


16x24
f'c = 5ksi
12#11
P (kip)
1800
(Pmax)
fs=0
fs=0
fs=0.5fy
fs=0.5fy
450
450
Mx (k-ft)
(Pmin)
-1200


16x24
f'c = 5ksi
4#10
P (kip)
1400
(Pmax)
fs=0
fs=0
fs=0.5fy
fs=0.5fy
300
300
Mx (k-ft)
(Pmin)
-400


P (kip)
1200
16x16
f’c = 5ksi
8#10
(Pmax)
fs=0
fs=0
fs=0.5fy
fs=0.5fy
-300
300
Mx (k-ft)
(Pmin)
-600


P (kip)
1400
(Pmax)
18x18
f’c = 5ksi
8#11
fs=0
fs=0
fs=0.5fy
fs=0.5fy
-450
450
Mx (k-ft)
(Pmin)
-800

COLUMN SCHEDULE – E-TOWER (AREAS 'A' & 'C')

LEVEL \ COLUMN	M-67.8	M-68.2	M-69.4	M-70	M-71	N-67.8	N-68.2	N-69.4	N-70	N-71	O-68.2
ELEV. MACHINE ROOM ROOF											
ROOF											
ELEV. MACH. ROOM			W8x31	W8x31	W8x31			W8x31	W8x31	W8x31	
EIGHTH FLOOR											
SEVENTH FLOOR			24"x24" 4#11 40k	24"x24" 4#11 90k	24"x24" 4#11 #5@4" M.C.			24"x24" 4#11 160k	24"x24" 4#11 160k	24"x24" 4#11 #5@4" M.C.	
SIXTH FLOOR			24"x24" 4#11 60k	24"x24" 4#11 145k	24"x24" 4#11 #5@4" M.C.			24"x24" 4#11 225k	24"x24" 4#11 250k	24"x24" 4#11 #5@4" M.C.	
FIFTH FLOOR			24"x24" 4#11 80k	24"x24" 4#11 200k	24"x24" 4#11 #5@4" M.C.			24"x24" 4#11 295k	24"x24" 4#11 335k	24"x24" 4#11 #5@4" M.C.	
FOURTH FLOOR			24"x24" 4#11 105k	24"x24" 4#11 255k	24"x24" 4#11 #5@4" M.C.			24"x24" 4#11 360k	24"x24" 4#11 420k	24"x24" 4#11 #5@4" M.C.	
THIRD FLOOR			24"x24" 4#11 125k	24"x24" 4#11 310k	24"x24" 4#11 #5@4" M.C.			24"x24" 4#11 420k	24"x24" 4#11 505k	24"x24" 4#11 #5@4" M.C.	
SECOND FLOOR			24"x24" 4#11 145k	24"x24" 4#11 365k	24"x24" 12#11 #5@4" M.C.			24"x24" 4#11 445k	24"x24" 4#11 590k	24"x24" 12#11 #5@4" M.C.	
FIRST FLOOR	W10x33	W10x33	24"x24" 4#11 185k	24"x24" 4#11 420k	24"x24" 12#11 #5@4" M.C.	W10x33	W10x33	24"x24" 4#11 485k	24"x24" 4#11 675k	24"x24" 12#11 #5@4" M.C.	W10x33
GROUND FLOOR / TOP OF FOUNDATION	24"x24" 4#11	24"x24" 4#11	24"x24" 4#11 240k	24"x24" 4#11 475k	24"x24" 12#11 #5@4" M.C.	24"x24" 4#11	24"x24" 4#11	24"x24" 4#11 700k	24"x24" 4#11 810k	24"x24" 12#11 #5@4" M.C.	24"x24" 4#11 390k
WORKING LOAD			190k	365k	1920k			535k	630k	3140k	295k

LEVEL \ COLUMN	O-69.4	O-70	O-71	O.3-68.6	P-71	Q-70	Q-71	Q.5-51	Q.5-53	Q.5-54	Q.5-57
ELEV. MACHINE ROOM ROOF											
ROOF											
ELEV. MACH. ROOM	W8x31	W8x31	W8x31		W8x31	W8x31	W8x31				
EIGHTH FLOOR											
SEVENTH FLOOR	24"x24" 4#11 185k	24"x24" 4#11 415k	24"x24" 4#11 360k		24"x24" 4#11 360k	24"x24" 4#11 310k	24"x24" 4#11 #5@4" M.C.				
SIXTH FLOOR	24"x24" 4#11 265k	24"x24" 4#11 580k	24"x24" 4#11 510k		24"x24" 4#11 490k	24"x24" 4#11 420k	24"x24" 4#11 #5@4" M.C.				
FIFTH FLOOR	24"x24" 4#11 340k	24"x24" 4#11 745k	24"x24" 4#11 655k		24"x24" 4#11 615k	24"x24" 4#11 530k	24"x24" 4#11 #5@4" M.C.				
FOURTH FLOOR	24"x24" 4#11 415k	24"x24" 4#11 915k	24"x24" 4#11 800k		24"x24" 4#11 740k	24"x24" 4#11 640k	24"x24" 8#11 #5@4" M.C.				
THIRD FLOOR	24"x24" 4#11 490k	24"x24" 8#11 1080k	24"x24" 8#11 940k		24"x24" 4#11 860k	24"x24" 4#11 750k	24"x24" 8#11 #5@4" M.C.				
SECOND FLOOR	24"x24" 4#11 570k	24"x24" 8#11 1245k	24"x24" 8#11 1085k		24"x24" 4#11 985k	24"x24" 4#11 860k	24"x24" 16#11 #5@4" M.C.				
FIRST FLOOR	24"x24" 4#11 625k	24"x24" 8#11 1415k	24"x24" 8#11 1210k		24"x24" 4#11 1110k	24"x24" 4#11 970k	24"x24" 16#11 #5@4" M.C.	W10x33	W10x33	W10x33	W10x49
GROUND FLOOR / TOP OF FOUNDATION	24"x24" 4#11 835k	24"x24" 8#11 1585k	24"x24" 8#11 1385k	12"x24" 4#9	24"x24" 4#11 1280k	24"x24" 4#11 1060k	24"x24" 16#11 #5@4" M.C.				
WORKING LOAD	690k	1220k	1075k	200K	995k	815k	3060k				

LEVEL \ COLUMN	R-70	R-71	R.1-66	R.2-57	R.2-58	R.2-59	R.2-61	R.2-65	R.3-54	S-54	S-57
ELEV. MACHINE ROOM ROOF											
ROOF											
ELEV. MACH. ROOM	W8x31	W8x31		W8x31	W8x31	W8x31	W8x31			W8x31	W8x31
EIGHTH FLOOR				12"x24" 4#9	12"x24" 4#9	12"x24" 4#9	12"x24" 4#9				12"x24" 4#9
SEVENTH FLOOR	24"x24" 4#11 500k	24"x24" 8#11 #5@4" M.C.	24"x24" 4#11 165k	12"x24" 4#9 60k	12"x24" 4#9 60k	12"x24" 4#9 60k	12"x24" 4#9 65k	24"x24" 4#11 85k	24"x24" 4#11 60k	24"x24" 4#11 125k	12"x24" 4#9 110k
SIXTH FLOOR	24"x24" 4#11 670k	24"x24" 8#11 #5@4" M.C.	24"x24" 4#11 270k	12"x24" 4#9 95k	12"x24" 4#9 90k	12"x24" 4#9 90k	12"x24" 4#9 115k	24"x24" 4#11 190k	24"x24" 4#11 105k	24"x24" 4#11 185k	12"x24" 4#9 165k
FIFTH FLOOR	24"x24" 4#11 840k	24"x24" 8#11 #5@4" M.C.	24"x24" 4#11 375k	12"x24" 4#9 130k	12"x24" 4#9 115k	12"x24" 4#9 115k	12"x24" 4#9 160k	24"x24" 4#11 290k	24"x24" 4#11 145k	24"x24" 4#11 240k	12"x24" 4#9 215k
FOURTH FLOOR	24"x24" 4#11 1010k	24"x24" 12#11 #5@4" M.C.	24"x24" 4#11 475k	12"x24" 4#9 165k	12"x24" 4#9 145k	12"x24" 4#9 145k	12"x24" 4#9 210k	24"x24" 4#11 390k	24"x24" 4#11 185k	24"x24" 4#11 295k	12"x24" 4#9 270k
THIRD FLOOR	24"x24" 8#11 1175k	24"x24" 12#11 #5@4" M.C.	24"x24" 4#11 575k	12"x24" 4#9 200k	12"x24" 4#9 170k	12"x24" 4#9 170k	12"x24" 4#9 255k	24"x24" 4#11 485k	24"x24" 4#11 225k	24"x24" 4#11 350k	12"x24" 4#9 320k
SECOND FLOOR	24"x24" 8#11 1365k	26"x26" 24#11 #5@4" M.C.	24"x24" 4#11 670k	12"x24" 4#11 275k	12"x24" 4#11 195k	12"x24" 4#11 195k	12"x24" 4#11 335k	24"x24" 4#11 620k	24"x24" 4#11 265k	24"x24" 4#11 405k	12"x24" 4#11 375k
FIRST FLOOR	24"x24" 16#11 1550k	26"x26" 24#11 #5@4" M.C.	24"x24" 4#11 780k	12"x24" 4#11 350k	12"x24" 4#11 225k	12"x24" 4#11 225k	12"x24" 4#11 380k	24"x24" 4#11 730k	24"x24" 4#11 340k	24"x24" 4#11 465k	12"x24" 4#11 425k
GROUND FLOOR / TOP OF FOUNDATION	24"x24" 16#11 1820k	26"x26" 24#11 #5@4" M.C.	24"x24" 4#11 980k	12"x24" 4#11 350k	12"x24" 4#11 225k	12"x24" 4#11 225k	12"x24" 4#11 380k	24"x24" 4#11 790k	24"x24" 4#11 340k	24"x24" 4#11 465k	12"x24" 4#11 425k
WORKING LOAD	1395k	2620k	573k	275k	180k	180k	300k	620k	620k	265k	330k

F'c = 4000 PSI

F'c = 5000 PSI

CONCRETE COLUMN SCHEDULE KEY

COL. DIM.	24"x24"	
	8#11	NO. & SIZE OF VERT. REINF
TIES	#5@4"	
VERT. LOAD	500k	
	M.C.	M.C. INDICATES FULL MECHANICAL COUPLERS

CONCRETE COLUMN SCHEDULE – E-TOWER (AREAS 'A' & 'C')

LEVEL \ COLUMN	S-58	S-59	S-61	S-65	S-66	S-68	S-69	S-70	S-72	T-72	T.1-59	T.1-61
ELEV. MACHINE ROOM ROOF												
ROOF		W8x35	W8x35									
ELEV. MACH. ROOM	W8x31	14"x24" 4#11 #5@4" M.C.	14"x24" 4#11 #5@4" M.C.	W8x31	W8x31	W8x31	W8x31	W8x31	W8x31	24"x24" 4#11 #5@4" M.C.		
EIGHTH FLOOR	12"x24" 4#9											
SEVENTH FLOOR	12"x24" 4#9 80k	14"x24" 4#11 #5@4" M.C.	14"x24" 4#11 #5@4" M.C.	24"x24" 4#11 155k	24"x24" 4#11 235k	24"x24" 4#11 455k	24"x24" 4#11 380k	24"x24" 4#11 380k	30"x30" 8#11 #5@4" M.C.	24"x24" 4#11 #5@4" M.C.	20"x20" 4#11 #5@4" M.C.	18"x18" 4#11 #5@4" M.C.
SIXTH FLOOR	12"x24" 4#9 115k	14"x24" 4#11 #5@4" M.C.	14"x24" 4#11 #5@4" M.C.	24"x24" 4#11 260k	24"x24" 4#11 345k	24"x24" 4#11 620k	24"x24" 4#11 515k	24"x24" 4#11 515k	30"x30" 8#11 #5@4" M.C.	24"x24" 4#11 #5@4" M.C.	20"x20" 4#11 #5@4" M.C.	18"x18" 4#11 #5@4" M.C.
F'c = 4000 PSI FIFTH FLOOR	12"x24" 4#9 155k	14"x24" 8#11 #5@4" M.C.	14"x24" 8#11 #5@4" M.C.	24"x24" 4#11 370k	24"x24" 4#11 455k	24"x24" 4#11 780k	24"x24" 4#11 655k	24"x24" 4#11 655k	30"x30" 8#11 #5@4" M.C.	24"x24" 4#11 #5@4" M.C.	20"x20" 4#11 #5@4" M.C.	18"x18" 4#11 #5@4" M.C.
FOURTH FLOOR	12"x24" 4#9 190k	14"x24" 8#11 #5@4" M.C.	14"x24" 8#11 #5@4" M.C.	24"x24" 4#11 475k	24"x24" 4#11 560k	24"x24" 4#11 945k	24"x24" 4#11 790k	24"x24" 4#11 790k	30"x30" 16#11 #5@4" M.C.	24"x24" 8#11 #5@4" M.C.	20"x20" 4#11 #5@4" M.C.	18"x18" 4#11 #5@4" M.C.
THIRD FLOOR	12"x24" 4#9 225k	14"x24" 12#11 #5@4" M.C.	14"x24" 12#11 #5@4" M.C.	24"x24" 4#11 580k	24"x24" 4#11 665k	24"x24" 8#11 1105k	24"x24" 8#11 925k	24"x24" 8#11 925k	30"x30" 16#11 #5@4" M.C.	24"x24" 8#11 #5@4" M.C.	20"x20" 4#11 #5@4" M.C.	18"x18" 4#11 #5@4" M.C.
SECOND FLOOR	12"x24" 4#11 255k	14"x24" 12#11 #5@4" M.C.	14"x24" 12#11 #5@4" M.C.	24"x24" 4#11 680k	24"x24" 4#11 775k	24"x24" 8#11 1270k	24"x24" 8#11 1060k	24"x24" 8#11 1060k	36"x32" 40#11 #5@4" M.C.	24"x24" 24#11 #5@4" M.C.	20"x22" 12#11 #5@4" M.C.	18"x18" 12#11 #5@4" M.C.
F'c = 5000 PSI FIRST FLOOR	12"x24" 4#11 295k	14"x24" 12#11 #5@4" M.C.	14"x24" 12#11 #5@4" M.C.	24"x24" 4#11 785k	24"x24" 4#11 890k	24"x24" 12#11 1435k	24"x24" 8#11 1195k	24"x24" 8#11 1195k	36"x32" 40#11 #5@4" M.C.	24"x24" 24#11 #5@4" M.C.	20"x22" 12#11 #5@4" M.C.	18"x18" 12#11 #5@4" M.C.
GROUND FLOOR / TOP OF FOUNDATION	12"x24" 4#11 295k	14"x24" 12#11 #5@4" M.C.	14"x24" 12#11 #5@4" M.C.	24"x24" 4#11 895k	24"x24" 4#11 1045k	24"x24" 12#11 1670k	24"x24" 8#11 1395k	24"x24" 8#11 1395k	36"x32" 40#11 #5@4" M.C.	24"x24" 24#11 #5@4" M.C.	20"x22" 12#11 #5@4" M.C.	18"x18" 12#11 #5@4" M.C.
WORKING LOAD	225k	1310k	1310k	700k	810k	1280k	1070k	1070k	4350k	2250k	1600k	1600k

LEVEL \ COLUMN	T.1-63	T.1-65	T.2-56	T.3-54	U-67	U-68	U-69	U-70	U-71	U-72	U.1-52.8	U.1-56
ELEV. MACHINE ROOM ROOF												
ROOF	W8x31	W8x31										
ELEV. MACH. ROOM					W8x31	W8x31	W8x31	W8x31	24"x24" 4#11 #5@4" M.C.	24"x24" 4#11 #5@4" M.C.		
EIGHTH FLOOR	12"x24" 4#9	12"x24" 4#9										
SEVENTH FLOOR	12"x24" 4#9 95k	12"x24" 4#9 150k	24"x24" 4#11 75k		24"x24" 4#11 350k	24"x24" 4#11 370k	24"x24" 4#11 #5@4" M.C.	24"x24" 4#11 30k	24"x24" 4#11 #5@4" M.C.	24"x24" 4#11 #5@4" M.C.		
SIXTH FLOOR	12"x24" 4#9 130k	12"x24" 4#9 220k	24"x24" 4#11 125k		24"x24" 4#11 470k	24"x24" 4#11 515k	24"x24" 4#11 #5@4" M.C.	24"x24" 4#11 165k	24"x24" 4#11 #5@4" M.C.	24"x24" 4#11 #5@4" M.C.		
F'c = 4000 PSI FIFTH FLOOR	12"x24" 4#9 170k	12"x24" 4#9 285k	24"x24" 4#11 175k		24"x24" 4#11 590k	24"x24" 4#11 655k	24"x24" 4#11 #5@4" M.C.	24"x24" 4#11 595k	24"x24" 4#11 #5@4" M.C.	24"x24" 4#11 #5@4" M.C.		
FOURTH FLOOR	12"x24" 4#9 210k	12"x24" 4#9 355k	24"x24" 4#11 220k		24"x24" 4#11 705k	24"x24" 4#11 795k	24"x24" 8#11 #5@4" M.C.	24"x24" 4#11 850k	24"x24" 8#11 #5@4" M.C.	24"x24" 8#11 #5@4" M.C.		
THIRD FLOOR	12"x24" 4#9 245k	12"x24" 4#9 420k	24"x24" 4#11 300k	24"x24" 4#11 35k	24"x24" 4#11 825k	24"x24" 4#11 940k	24"x24" 8#11 #5@4" M.C.	24"x24" 4#11 975k	24"x24" 8#11 #5@4" M.C.	24"x24" 8#11 #5@4" M.C.	24"x24" 4#11 60k	24"x24" 4#11 110k
SECOND FLOOR	12"x24" 4#11 285k	12"x24" 4#11 485k	24"x24" 4#11 370k	24"x24" 4#11 80k	24"x24" 8#11 940k	24"x24" 8#11 1080k	30"x30" 8#11 #5@4" M.C.	24"x24" 8#11 1110k	24"x24" 24#11 #5@4" M.C.	24"x24" 24#11 #5@4" M.C.	24"x24" 4#11 130k	24"x24" 4#11 250k
F'c = 5000 PSI FIRST FLOOR	12"x24" 4#11 325k	12"x24" 4#11 550k	24"x24" 8#11 445k	24"x24" 8#11 120k	24"x24" 16#11 1060k	24"x24" 16#11 1225k	30"x30" 8#11 #5@4" M.C.	24"x24" 16#11 1200k	24"x24" 24#11 #5@4" M.C.	24"x24" 24#11 #5@4" M.C.	24"x24" 4#11 195k	24"x24" 4#11 360k
GROUND FLOOR / TOP OF FOUNDATION	12"x24" 4#11 375k	12"x24" 4#11 645k	24"x24" 8#11 445k	24"x24" 8#11 120k	24"x24" 16#11 1225k	24"x24" 16#11 1425k	30"x30" 8#11 #5@4" M.C.	24"x24" 16#11 1300k	24"x24" 24#11 #5@4" M.C.	24"x24" 24#11 #5@4" M.C.	24"x24" 4#11 195k	24"x24" 4#11 360k
WORKING LOAD	290k	500k	345k	95k	950k	1105k	2505k	1005k	2250k	2250k	150k	275k

LEVEL \ COLUMN	U.1-59	U.1-61	U.1-63	U.1-65	U.1-66	V-52.8	V-56	V-60	V-64	V.7-64	V.7-66	W-52.8
ELEV. MACHINE ROOM ROOF												
ROOF			W8x31	W8x31								
ELEV. MACH. ROOM					W8x31							
EIGHTH FLOOR			12"x24" 4#9	12"x24" 4#9								
SEVENTH FLOOR	20"x20" 4#11 #5@4" M.C.	18"x18" 4#11 #5@4" M.C.	12"x24" 4#11 #5@4" M.C.	12"x24" 4#11 #5@4" M.C.	24"x24" 4#11 #5@4" M.C.							
SIXTH FLOOR	20"x20" 4#11 #5@4" M.C.	18"x18" 4#11 #5@4" M.C.	12"x24" 4#11 #5@4" M.C.	12"x24" 4#11 #5@4" M.C.	24"x24" 4#11 #5@4" M.C.							
F'c = 4000 PSI FIFTH FLOOR	20"x20" 4#11 #5@4" M.C.	18"x18" 4#11 #5@4" M.C.	12"x24" 4#11 #5@4" M.C.	12"x24" 4#11 #5@4" M.C.	24"x24" 4#11 #5@4" M.C.							
FOURTH FLOOR	20"x20" 4#11 #5@4" M.C.	18"x18" 4#11 #5@4" M.C.	14"x24" 8#11 #5@4" M.C.	14"x24" 8#11 #5@4" M.C.	24"x24" 12#11 #5@4" M.C.							
THIRD FLOOR	20"x20" 4#11 #5@4" M.C.	18"x18" 4#11 #5@4" M.C.	14"x24" 8#11 #5@4" M.C.	14"x24" 8#11 #5@4" M.C.	24"x24" 12#11 #5@4" M.C.	24"x24" 4#11 85k	24"x24" 4#11 160k	24"x24" 4#11 170k	24"x24" 4#11 125k	24"x24" 4#11 115k	24"x24" 4#11 130k	24"x24" 4#11 95k
SECOND FLOOR	20"x22" 12#11 #5@4" M.C.	20"x20" 4#11 #5@4" M.C.	14"x24" 12#11 #5@4" M.C.	14"x24" 12#11 #5@4" M.C.	30"x30" 24#11 #5@4" M.C.	24"x24" 4#11 210k	24"x24" 4#11 415k	24"x24" 4#11 430k	24"x24" 4#11 315k	24"x24" 4#11 290k	24"x24" 4#11 255k	24"x24" 4#11 235k
F'c = 5000 PSI FIRST FLOOR	20"x22" 12#11 #5@4" M.C.	20"x20" 4#11 #5@4" M.C.	14"x24" 12#11 #5@4" M.C.	14"x24" 12#11 #5@4" M.C.	30"x30" 24#11 #5@4" M.C.	24"x24" 4#11 295k	24"x24" 4#11 575k	24"x24" 4#11 595k	24"x24" 4#11 470k	24"x24" 4#11 430k	24"x24" 4#11 345	24"x24" 4#11 340k
GROUND FLOOR / TOP OF FOUNDATION	20"x22" 12#11 #5@4" M.C.	20"x20" 4#11 #5@4" M.C.	14"x24" 12#11 #5@4" M.C.	14"x24" 12#11 #5@4" M.C.	30"x30" 24#11 #5@4" M.C.	24"x24" 4#11 365k	24"x24" 4#11 710k	24"x24" 4#11 780k	24"x24" 4#11 635k	24"x24" 4#11 610k	24"x24" 4#11 445k	24"x24" 4#11 460k
WORKING LOAD	1600k	880k	1310k	1310k	3145k	275k	535k	585k	480k	460k	75k	350k

CONCRETE COLUMN SCHEDULE KEY

COL. DIM. → 24"x24"
8#11 ← NO. & SIZE OF VERT. REINF
TIES → #5@4"
VERT. LOAD → 500k
M.C. ← M.C. INDICATES FULL MECHANICAL COUPLERS

COLUMN SCHEDULE – E-TOWER (AREAS 'A' & 'C')

LEVEL \ COLUMN	W-56	W-60	Z-108	Z.2-103	Z.2-104	Z.2-105	Z.2-106	Z.2-107	Z.2-108	Z.2-109	Z.3-103	Z.4-92	Z.4-103	Z.4-107	Z.4-108	Z.4-109	Z.6-91	Z.6-92	Z.6-94	Z.6-95
ROOF																				
EIGHTH FLOOR																				
SEVENTH FLOOR																				
SIXTH FLOOR																				
FIFTH FLOOR																				
FOURTH FLOOR																				
THIRD FLOOR	24"x24" 4#11	24"x24" 4#11																		
SECOND FLOOR	24"x24" 4#11	24"x24" 4#11																		
FIRST FLOOR	24"x24" 4#11	24"x24" 4#11	W10x39	W10x49	W10x49	W10x49	W10x33	W10x33	W12x96	W12x96	W10x33	W14x99	W14x99	W14x99	W14x99	W14x99	W10x33	W10x33	W10x33	W10x33
GROUND FLOOR / TOP OF FOUNDATION	24"x24" 4#11	24"x24" 4#11													24"x24" 4#11	24"x24" 4#11	24"x24" 4#11	24"x24" 4#11	24"x24" 4#11	24"x24" 4#11
WORKING LOAD	450k	460k															205k	235k	235k	160k

COLUMN / LEVEL	Z.7-9.0	Z.7-91	Z.7-92	Z.7-94	Z.7-95	Z.8-90	Z.8-91	Z.8-92	Z.8-94	Z.8-95	Z.9-92	LEVEL
ROOF												ROOF
EIGHTH FLOOR	W8x31	W8x31	W8x31	W8x31	W8x31	W8x31	W8x35	W8x35	W8x35	W8x31	W8x31	EIGHTH FLOOR
SEVENTH FLOOR	24"x24" 4#11	24"x24" 4#11	24"x24" 4#11	24"x24" 4#11	24"x24" 4#11	24"x24" 4#11	24"x24" 4#11	24"x24" 4#11	24"x24" 4#11	24"x24" 4#11	24"x24" 4#11	SEVENTH FLOOR
SIXTH FLOOR	24"x24" 4#11	24"x24" 4#11	24"x24" 4#11	24"x24" 4#11	24"x24" 4#11	24"x24" 4#11	24"x24" 4#11	24"x24" 4#11	24"x24" 4#11	24"x24" 4#11	24"x24" 4#11	SIXTH FLOOR
FIFTH FLOOR	24"x24" 4#11	24"x24" 4#11	24"x24" 4#11	24"x24" 4#11	24"x24" 4#11	24"x24" 4#11	24"x24" 4#11	24"x24" 4#11	24"x24" 4#11	24"x24" 4#11	24"x24" 4#11	FIFTH FLOOR
FOURTH FLOOR	24"x24" 4#11	24"x24" 4#11	24"x24" 4#11	24"x24" 4#11	24"x24" 4#11	24"x24" 4#11	24"x24" 4#11	24"x24" 4#11	24"x24" 4#11	24"x24" 4#11	24"x24" 4#11	FOURTH FLOOR
THIRD FLOOR	24"x24" 4#11	24"x24" 8#11	24"x24" 8#11	24"x24" 8#11	24"x24" 4#11	24"x24" 8#11	24"x24" 4#11	24"x24" 4#11	24"x24" 4#11	24"x24" 8#11	24"x24" 4#11	THIRD FLOOR
SECOND FLOOR	24"x24" 4#11	24"x24" 8#11	24"x24" 8#11	24"x24" 8#11	24"x24" 4#11	24"x24" 8#11	24"x24" 8#11	24"x24" 8#11	24"x24" 8#11	24"x24" 8#11	24"x24" 4#11	SECOND FLOOR
FIRST FLOOR	24"x24" 4#11	24"x24" 12#11	24"x24" 12#11	24"x24" 12#11	24"x24" 4#11	24"x24" 8#11	24"x24" 12#11	24"x24" 12#11	24"x24" 12#11	24"x24" 12#11	24"x24" 4#11	FIRST FLOOR
GROUND FLOOR / TOP OF FOUNDATION	24"x24" 4#11	24"x24" 12#11	24"x24" 12#11	24"x24" 12#11	24"x24" 4#11	24"x24" 8#11	24"x24" 12#11	24"x24" 12#11	24"x24" 12#11	24"x24" 12#11	24"x24" 4#11	GROUND FLOOR / TOP OF FOUNDATION
WORKING LOAD	710k	1310k	1185k	1285k	760k	1140k	1230k	1175k	1195k	1260k	830k	WORKING LOAD

F'c = 4000 PSI (third floor to roof)

F'c = 5000 PSI (ground floor to third floor)

CONCRETE COLUMN SCHEDULE KEY

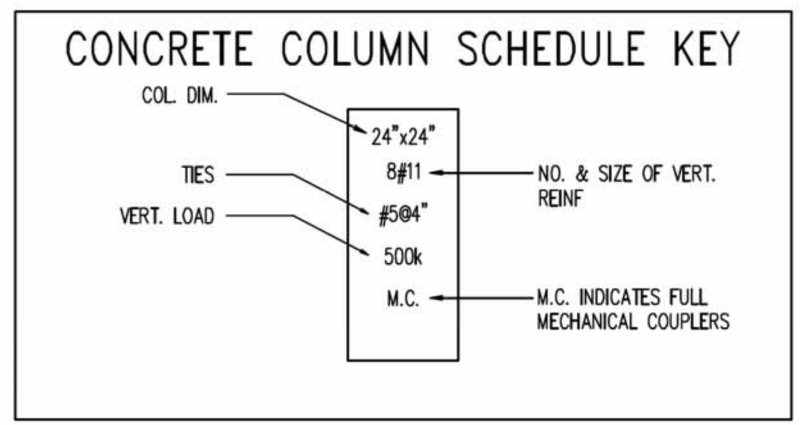
COL. DIM.	24"x24"	
	8#11	NO. & SIZE OF VERT. REINF
TIES	#5@4"	
VERT. LOAD	500k	
	M.C.	M.C. INDICATES FULL MECHANICAL COUPLERS

STEEL COLUMN SCHEDULE – CONFERENCE WING (AREA 'B')

COLUMN / LEVEL	A-68.2	A-70.3	B-66.6	B-67.8	B-70.6	B-73	C-66.6	C-73	D-66.6	D-73	E-66.6	E-68.2
ROOF												
SECOND FLOOR	24"x14" 4#10	24"x14" 4#10	24"x14" 4#10	24"x14" 4#10	24"x14" 4#10	24"x14" 4#10	24"x14" 4#10	24"x14" 4#10	24"x14" 4#10	24"x14" 4#10	24"x14" 4#10	24"x30" 8#11
FIRST FLOOR / TOP OF FOUNDATION	24"x14" 4#10	24"x14" 4#10	24"x14" 4#10	24"x14" 4#10	24"x14" 4#10	24"x14" 4#10	24"x14" 4#10	24"x14" 4#10	24"x14" 4#10	24"x14" 4#10	24"x14" 4#10	24"x14" 8#11
GROUND FLOOR / TOP OF FOUNDATION	24"x14" 4#10	24"x14" 4#10	24"x14" 4#10	24"x14" 4#10	24"x14" 4#10	24"x14" 4#10	24"x14" 4#10	24"x14" 4#10	24"x14" 4#10	24"x14" 4#10	24"x14" 4#10	30"x130 12#11
WORKING LOAD	540k	540k	540k	540k	125k	125k	540k	540k	540k	540k	540k	1425k

COLUMN / LEVEL	E-70.3	E-73	F-66.6	F-68.2	F-70.3	F-73	G-70.3	H-73	I-67.8	I-69.2	I-70.3	I-71
ROOF												
SECOND FLOOR	24"x30" 8#11	24"x14" 4#10	24"x14" 4#10	24"x16" 4#11	24"x16" 4#11	24"x14" 4#10	24"x16" 8#11	24"x14" 4#10	24"x16" 4#10	24"x16" 4#10	24"x16" 4#10	24"x16" 4#10
FIRST FLOOR / TOP OF FOUNDATION	24"x14" 8#11	24"x14" 4#10	24"x14" 4#10	24"x16" 4#11	24"x16" 4#11	24"x14" 4#10	24"x16" 8#11	24"x14" 4#10	24"x16" 4#10	24"x16" 4#10	24"x16" 4#10	24"x16" 4#10
GROUND FLOOR / TOP OF FOUNDATION	30"x130 12#11	24"x14" 4#10	24"x14" 4#10	24"x16" 8#11	24"x16" 8#11	24"x14" 4#10	24"x16" 12#11	24"x14" 4#10	24"x16" 4#10	24"x16" 4#10	24"x16" 4#10	24"x16" 4#10
WORKING LOAD	1425k	540k	540k	860k	860k	540k	920k	540k	220k	475k	650k	650k

COLUMN / LEVEL	J-67.8	J-69.2	J-70.3	J-71	K-67.8	K-69.2	K-70.3	K-71	L-67.8	L-69.2	L-70.3	L-71	COLUMN / LEVEL
													STAIR ROOF
ROOF													ROOF
SECOND FLOOR													SECOND FLOOR
FIRST FLOOR / TOP OF FOUNDATION	16"x16" 8#10	18"x18" 8#11	18"x18" 8#11	16"x16" 8#10	16"x16" 8#10	18"x18" 8#11	18"x18" 8#11	16"x16" 8#10	16"x16" 8#10	16"x16" 8#10	16"x16" 8#10	16"x16" 8#10	FIRST FLOOR / TOP OF FOUNDATION
GROUND FLOOR / TOP OF FOUNDATION	16"x16" 8#10	18"x18" 8#11	18"x18" 8#11	16"x16" 8#10	16"x16" 8#10	18"x18" 8#11	18"x18" 8#11	16"x16" 8#10	16"x16" 8#10	16"x16" 8#10	16"x16" 8#10	16"x16" 8#10	GROUND FLOOR / TOP OF FOUNDATION
WORKING LOAD	460k	655k	545k	255k	390k	560k	450k	220k	190k	460k	380k	110k	WORKING LOAD



Appendix D
Acoustics Design

As Built Room

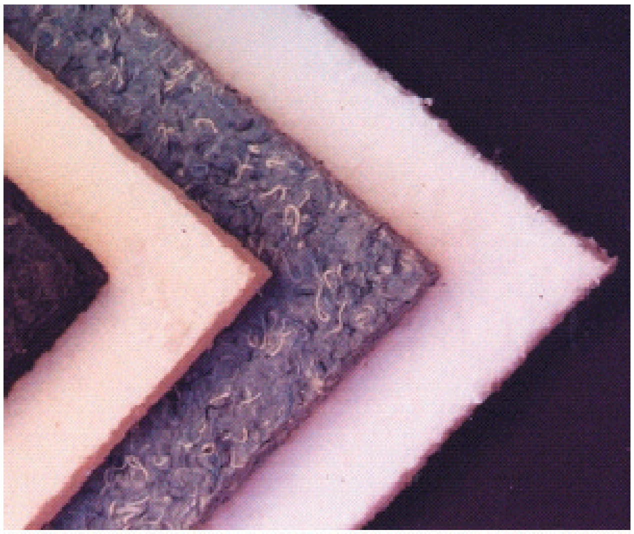
Surface	Material	Area (ft^2)	Absorption Coefficient (Hz) 500	1000	2000	4000	Sα 500	1000	2000	4000	Price/ft^2	Cost
Floor	Carpet on Concret	3304	0.14	0.37	0.6	0.65	462.56	1222.48	1982.4	2147.6		
Floor	Audience in Upolstered Seats	1220.6	0.88	0.96	0.93	0.85	1074.128	1171.776	1135.158	1037.51		
Wall	5/8" Gypsum on Metal Studs	724.45	0.05	0.03	0.03	0.03	36.2225	21.7335	21.7335	21.7335	$0.32	$231.82
Wall	Wood Paneling and Doors	619.75	0.17	0.09	0.1	0.11	105.3575	55.7775	61.975	68.1725		
Wall	Softwall - 1/2" Acousticotton	1755.14	0.22	0.54	0.81	1	386.1308	947.7756	1421.6634	1755.14	$1.50	$2,632.71
Wall	Glass	35.33	0.18	0.12	0.07	0.04	6.3594	4.2396	2.4731	1.4132		
Wall	Curtin	98	0.4	0.4	0.5	0.5	39.2	39.2	49	49		
Ceiling	Armstrong Optima 3255 - 4'x4' Tile	2843.19	0.84	1.01	1.02	0.97	2388.2796	2871.6219	2900.0538	2757.8943	$4.10	$11,657.08
Ceiling	5/8" Gypsum	4169.75	0.05	0.03	0.03	0.03	208.4875	125.0925	125.0925	125.0925	$0.32	$1,334.32
												$15,855.93
						a=?Sα	4706.7253	6459.6966	7699.5493	7963.556		
Target Reverb. Time: 0.7-1.1 sec.					Volume (ft^3): 49939.59	$T_{60\ (sec.)}$	0.53	0.39	0.32	0.31		

My Design

Surface	Material	Area (ft^2)	Absorption Coefficient (Hz)					Sα				Price/ft^2	Cost
			500	1000	2000	4000		500	1000	2000	4000		
Floor	Carpet on Concret	3304	0.14	0.37	0.6	0.65		462.56	1222.48	1982.4	2147.6		
Floor	Audience in Upolstered Seats	1220.6	0.88	0.96	0.93	0.85		1074.128	1171.776	1135.158	1037.51		
Wall	5/8" Gypsum on Metal Studs	2289.57	0.05	0.03	0.03	0.03		114.4785	68.6871	68.6871	68.6871	$0.32	$732.66
Wall	Wood Paneling and Doors	619.75	0.17	0.09	0.1	0.11		105.3575	55.7775	61.975	68.1725		
Wall	Softwall - 1/2" Acousticotton	192	0.22	0.54	0.81	1		42.24	103.68	155.52	192	$1.50	$288.00
Wall	Glass	35.33	0.18	0.12	0.07	0.04		6.3594	4.2396	2.4731	1.4132		
Wall	Curtin	98	0.4	0.4	0.5	0.5		39.2	39.2	49	49		
Ceiling	Armstrong Optima 3255 - 4'x4' Tile	0	0.84	1.01	1.02	0.97		0	0	0	0	$4.10	$0.00
Ceiling	5/8" Gypsum	7012.94	0.05	0.03	0.03	0.03		350.647	210.3882	210.3882	210.3882	$0.32	$2,244.14
													$3,264.80
							a=∑Sα	2194.9704	2876.228	3665.601	3774.771		
Target Reverb. Time: 0.7-1.1 sec.					Volume (ft^3):								
					49939.59		$T_{60 \text{ (sec.)}}$	1.14	0.87	0.68	0.66		

AcoustiCotton®

Semi Rigid Acoustical Board



SoftWalls™ INC.
Acoustical & Tackable Stretch Fabric Systems

7620-D Rickenbacker Drive, Gaithersburg, MD 20879
301-212-9880 • 301-384-9629 fax
info@softwalls.com

Performance Characteristics:

Fire Hazard Classification:

Unfaced (ASTM-E84) Flame Spread Class A or 1,
Smoke Developed Class A or 1.

Acoustical Performance:

		Coefficients and Frequencies						
Density	Thickness	125	250	500	1000	2000	4000	NRC
3#	½"	.01	.06	.22	.54	.81	1.00	0.40
4.5#	⅞"	.04	.33	.86	1.01	1.04	1.02	0.80

*Noise Reduction Coefficient as per ASTM C423, 1999 Standard.
"A" Type Mounting.

PRODUCT/DESIGN*	PAGES	UNIT SIZE TESTED	SOUND ABSORPTION COEFFICIENTS[C] – E-400 MOUNTING 125Hz	250Hz	500Hz	1000Hz	2000Hz	4000Hz	SOUND ABSORPTION[F] NRC[E]	AC[H]	SOUND TRANSMISSION[G] CAC MINIMUM[E]
GENERAL APPLICATION CEILINGS (CONTINUED)											
SHASTA (Nonperforated)	134-135	24" x 48" x 5/8"	0.62	0.36	0.29	0.76	0.66	0.77	0.50	–	–
SHASTA (Perforated)	134-135	24" x 48" x 5/8"	0.72	0.65	0.66	0.73	0.73	0.68	0.70	–	–
STRATUS	124-125	24" x 24" x 3/4"	0.46	0.53	0.57	0.61	0.93	0.99	0.70	–	25
TUNDRA	128-129	24" x 24" x 5/8"	0.27	0.31	0.56	0.65	0.50	0.37	0.50	–	33[B]
ULTIMA	130-131	24" x 48" x 3/4"	0.32	0.34	0.76	0.87	0.86	0.84	0.70	–	35
ULTIMA VECTOR	132-133	24" x 48" x 3/4"	0.40	0.33	0.72	0.92	0.87	0.82	0.70	–	33
SPECIAL PERFORMANCE CEILINGS											
ARMATUFF	38-39	24" x 24" x 5/8"	0.31	0.26	0.39	0.61	0.75	0.81	0.50	–	33[B]
CIRRUS Open Plan	50-51	24" x 24" x 7/8"	0.33	0.36	0.75	0.93	0.96	0.94	0.75	170	35
Clean Room MYLAR (Field Units)	64-65	24" x 48" x 3/4"	0.27	0.29	0.52	0.78	0.70	0.56	0.55	–	35[A]
FINE FISSURED Ceramaguard (Perforated)	84-85	24" x 48" x 5/8"	0.30	0.27	0.44	0.66	0.85	0.82	0.55	–	40
FINE FISSURED Open Plan	88-89	24" x 48" x 3/4"	0.30	0.33	0.66	0.94	0.90	0.87	0.70	170	35[A]
OPTIMA Open Plan	104-107	24" x 24" x 1"	0.65	0.91	0.84	1.01	1.02	0.97	0.95	190	–
OPTIMA Open Plan CAC Backing	104-107	24" x 48" x 1"	0.30	0.59	0.92	1.04	1.03	0.94	0.90	200	27