CHRISTIANA HOSPITAL 2010 PROJECT

NEWARK, DE

Final Report Spring 2007

Joseph G. Sharkey
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.

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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.

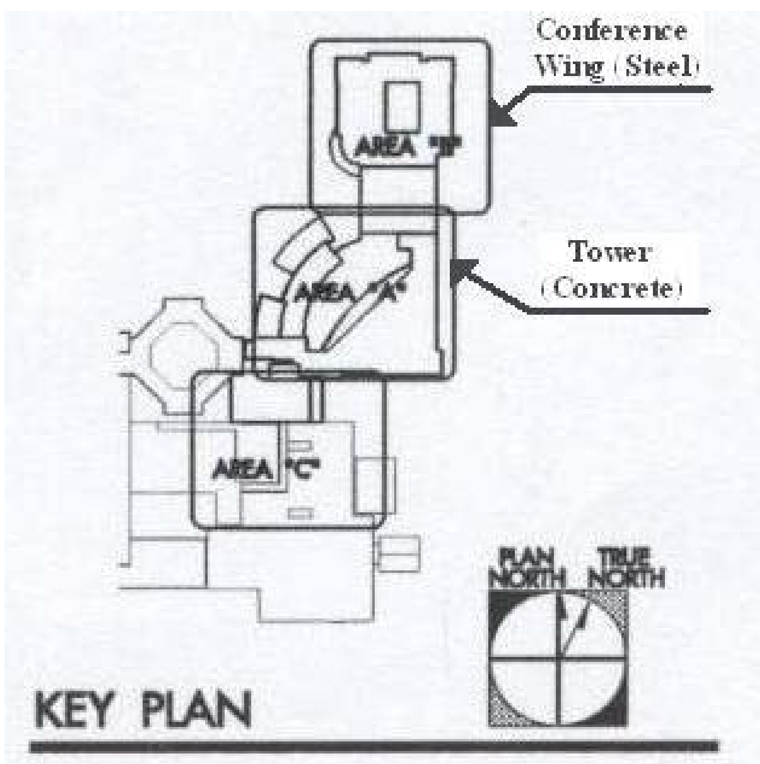
Secondly I will compare the existing structure to a structure using a two-way post-tensioned slab in the main tower and one-way post-tensioned beams and slab in the conference wing. Due to this change in the conference wing I will also make the necessary design changes to the rest of the wing which include 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:

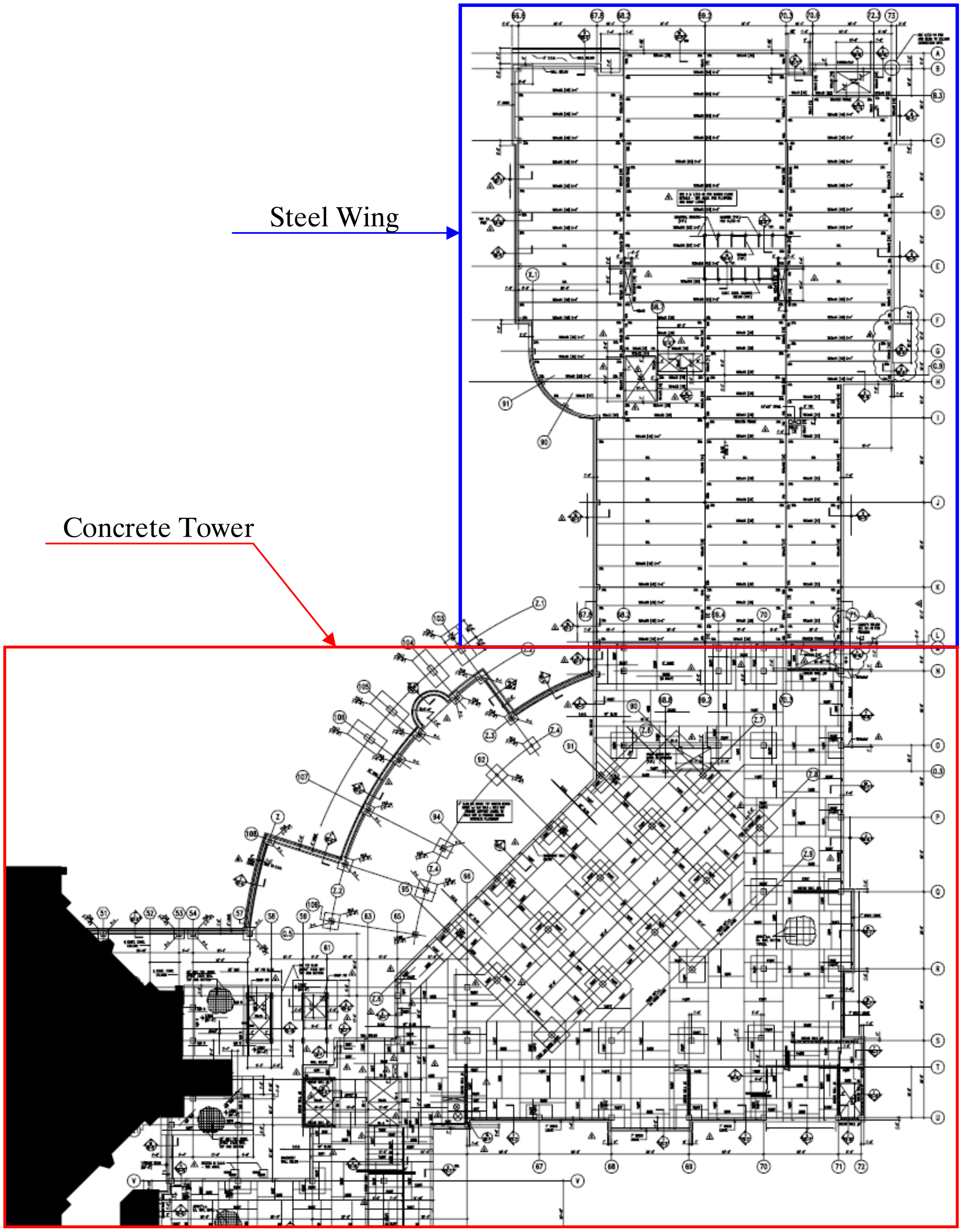
In the tower area a majority of the columns are 24"x24" reinforced concrete columns with only a few occurrences of 12"x24" columns. At the eighth floor nearly all the concrete columns stop and off of them W8 steel columns are posted. The 3 story conference wing is composed of W10 and W12 steel columns.

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.

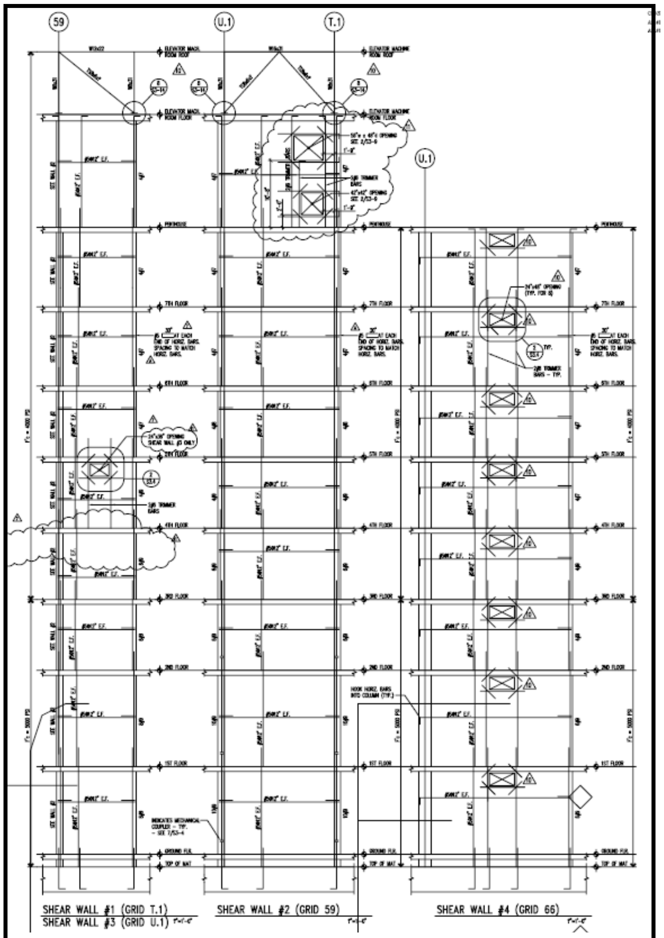


Figure 1 Shear Walls

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.

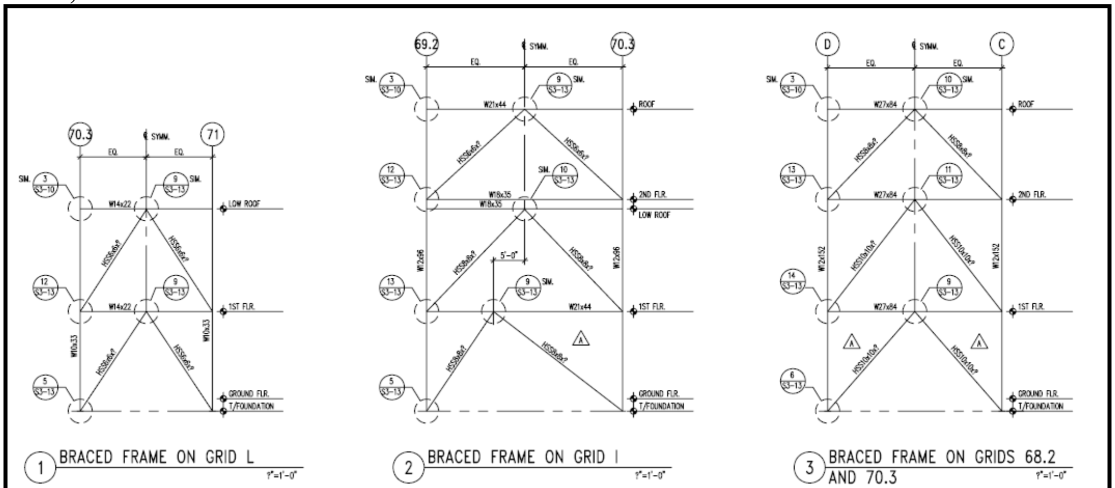


Figure 2 Concentrically Braced Frames

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.

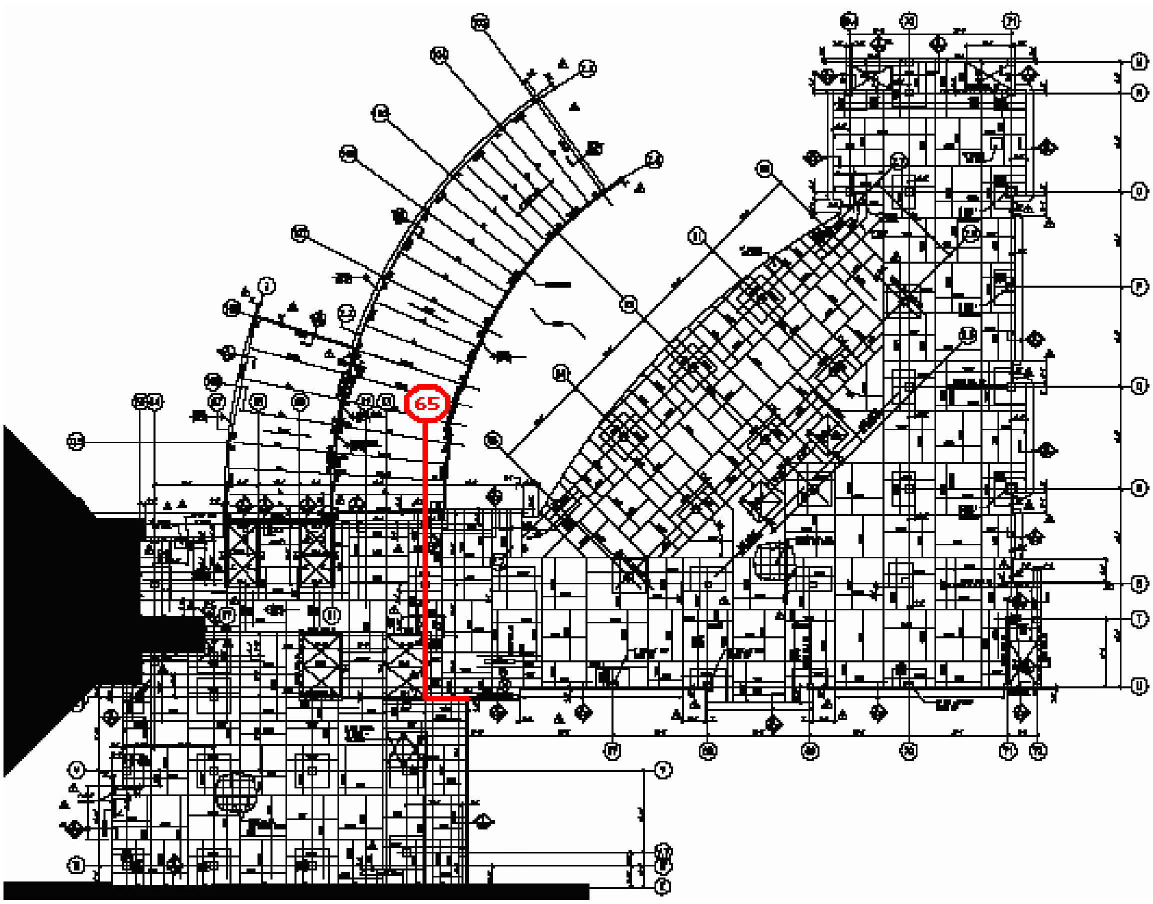


Figure 3 Expansion joint located on grid line 65

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 = Live Load
L_r = Roof Live Load
f_1 = 1.0 for live loads in excess of 100 psf and 0.5 for all other loads
S = Snow Load
W = Wind Load
f_2 = 0.2
E = Seismic or Earthquake Loading

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

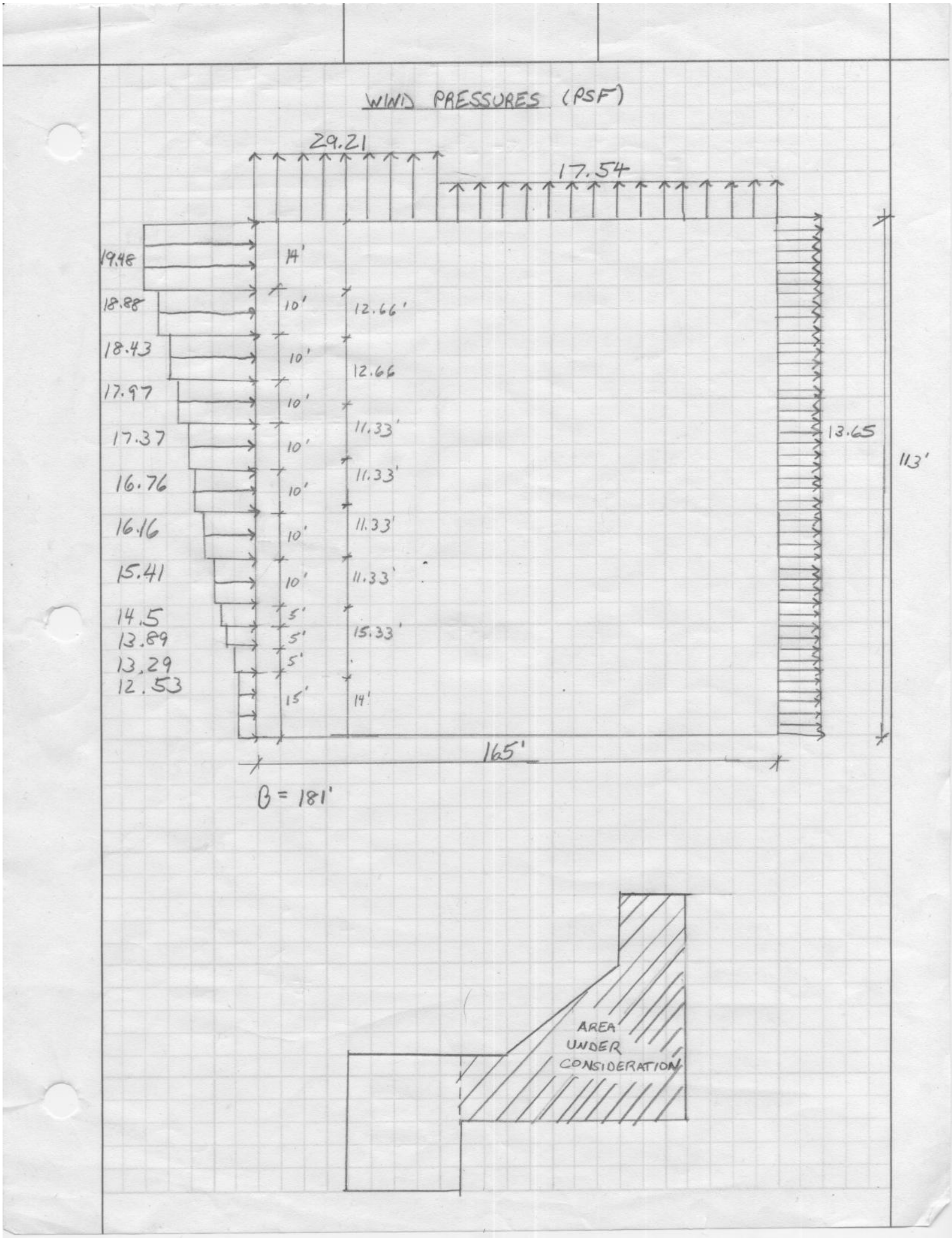
Snow Loading	
Item	**Value**
Ground Snow Load (P_g)	25 psf
Exposure Category	B
Roof Exposure	Partially Exposed
Exposure Factor (C_e)	1.0
Thermal Factor (C_t)	1.0
Occupancy Category	IV
Importance Factor (I_s)	1.2
Flat-Roof Snow Load $P_f = 0.7C_eC_tI_sP_g$	21 psf

Wind Loading

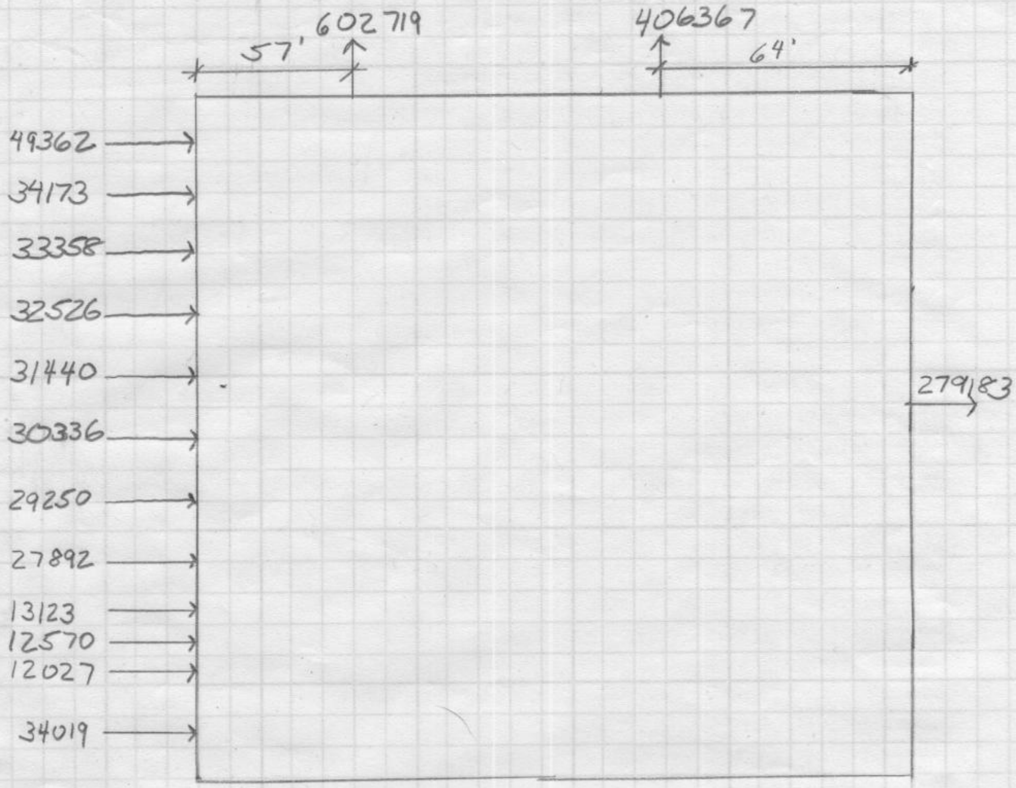
Assumptions: For the wind loading calculations, only one side of the building was calculated. The side chosen was the plan North face of the building. This was done because it is both the longest and tallest side of the building. By doing this the largest wind loads were found. For simplicity these loads 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

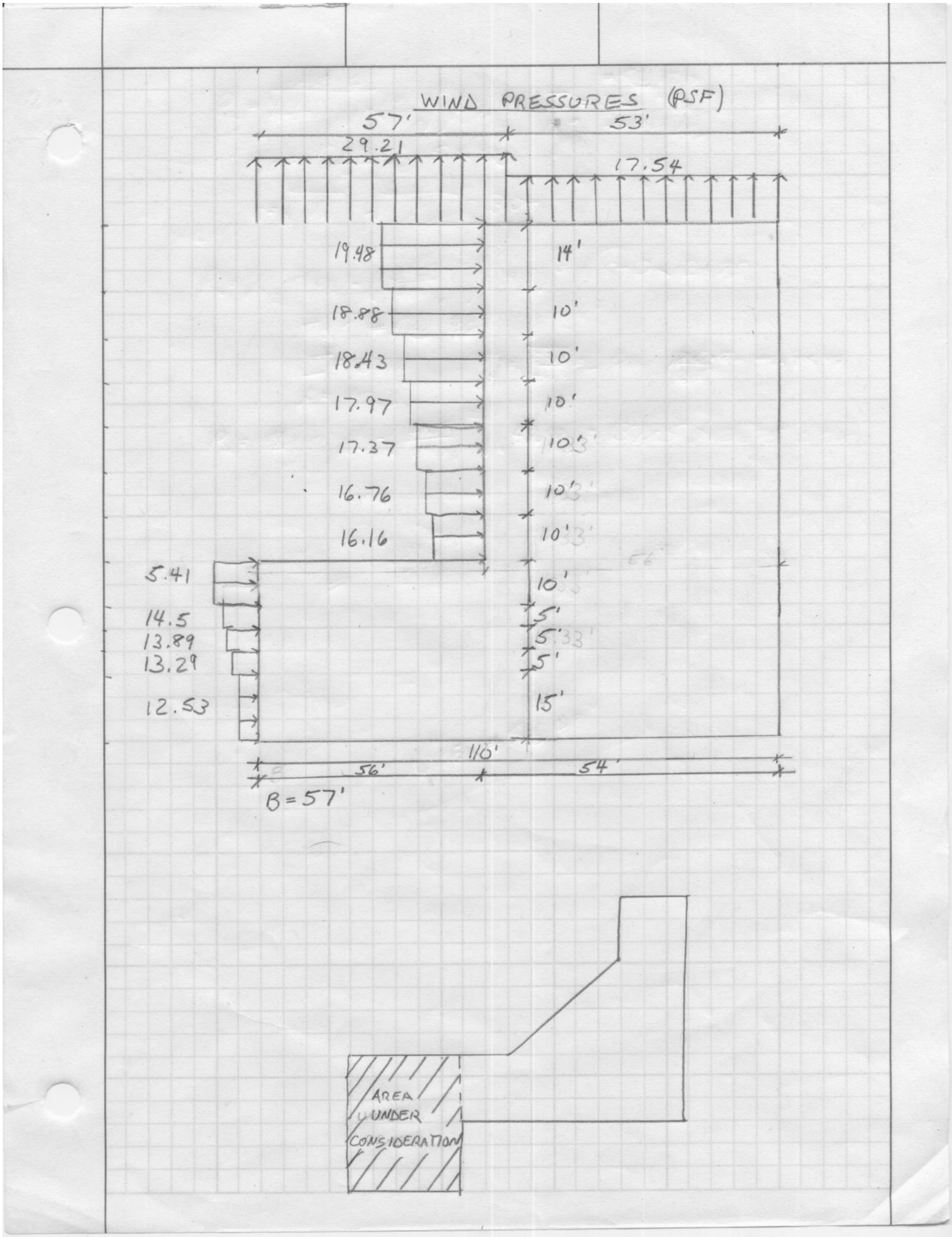

WIND PRESSURES (PSF)
29.21
17.54
19.48
18.88
18.43
17.97
17.37
16.76
16.16
15.41
14.5
13.89
13.29
12.53
14'
10'
5'
15'
12.66'
12.66
11.33'
15.33'
13.65
113'
165'
θ = 181'
AREA
UNDER
CONSIDERATION

WIND PRESSURES (lbs)

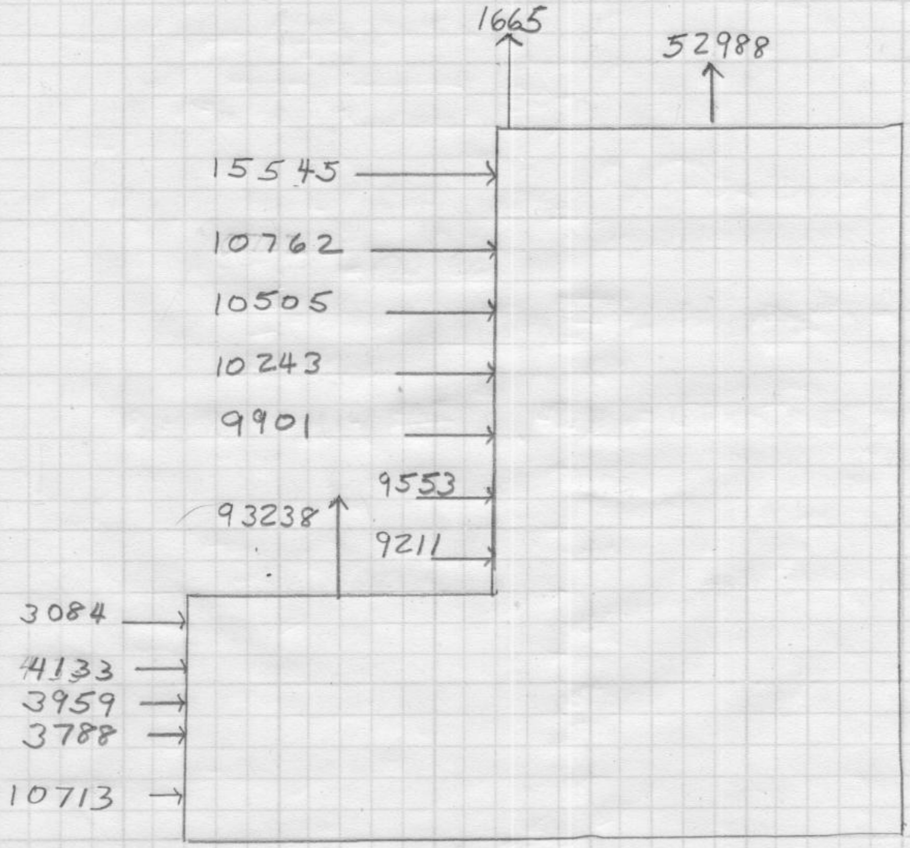


BASE SHEAR (V) = 619259 = 619.26 Kips

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
18.88
18.43
17.97
17.37
16.76
16.16
5.41
14.5
13.89
13.29
12.53
14'
10'
10'
10'
10'
10'
10'
10'
5'
5'
5'
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.

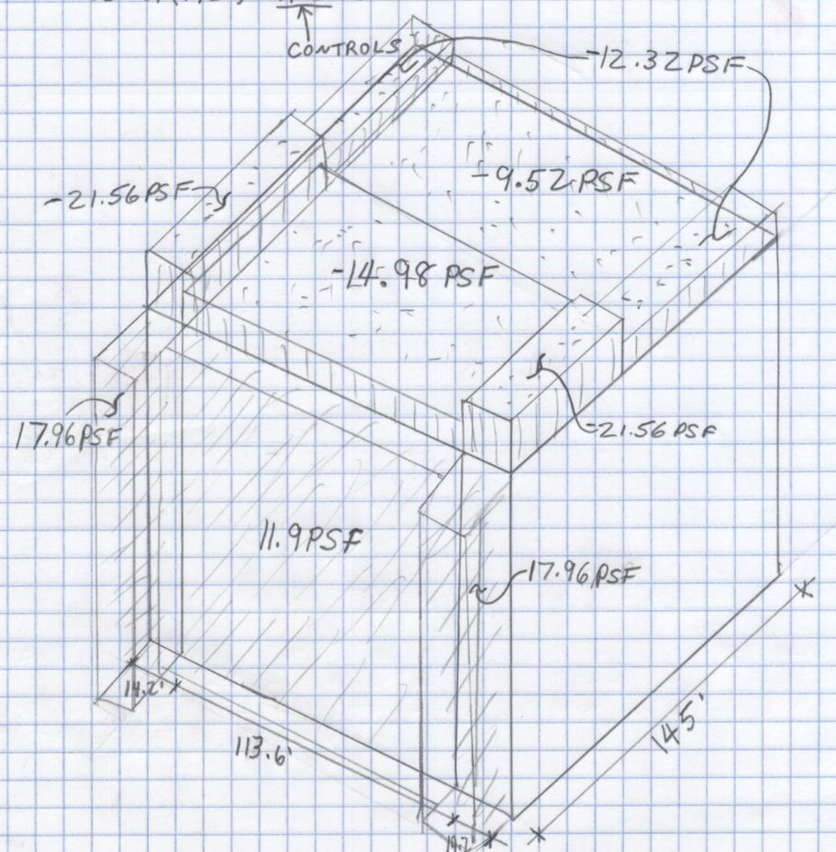
WIND PRESSURES (SIMPLIFIED METHOD)

IMPORTANCE FACTOR (I_w) = 1.15
IMPORTANCE CATEGORY = IV
EXPOSURE CATEGORY = B
HEIGHT + EXPOSURE ADJUSTMENT COEFFICIENT (λ) = 1.22
HEIGHT = 58.3' < 60'
V = 90 mph
ROOF ANGLE = 0-5°

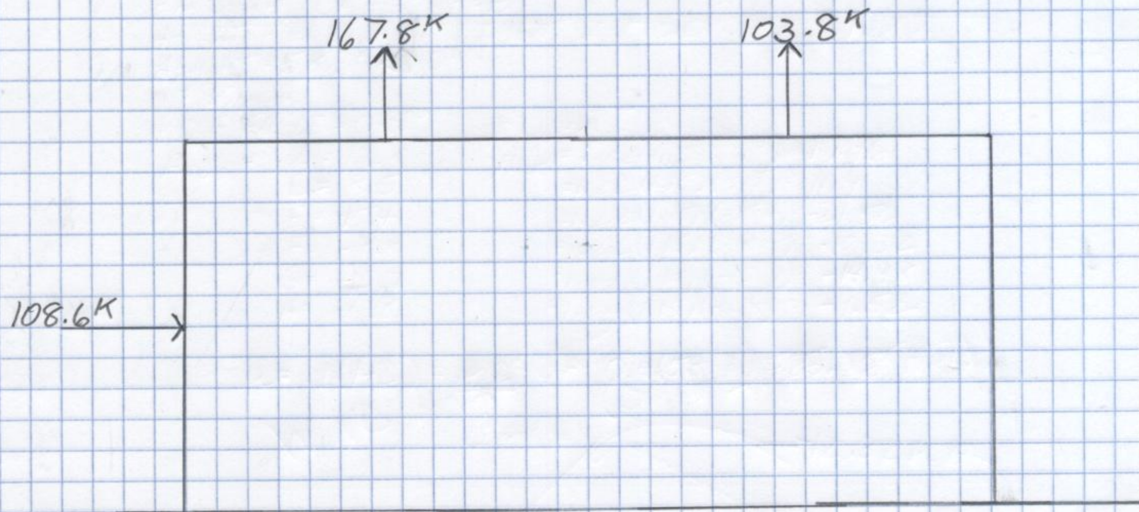
$P_{net} = \lambda I_w P_{net30}$

P_{s30} =		P_{net}
	A = 12.8	= 1.22(1.15)(12.8) = 17.96
	C = 8.5	= 1.22(1.15)(8.5) = 11.9
	E = -15.4	= 1.22(1.15)(-15.4) = -21.56
	F = -8.8	= 1.22(1.15)(-8.8) = -12.32
	G = -10.7	= 1.22(1.15)(-10.7) = -14.98
	H = -6.8	= 1.22(1.15)(-6.8) = -9.52

a = .1(142') = 14.2' or 0.4(58.3') = 23.32'



WIND PRESSURES (SIMPLIFIED METHOD) CONT.



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

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

$78.81 + 29.74 = 108.6^K$

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

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

$-123.4 - 44.4 = -167.8^K$

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

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

$-78.4 - 25.4 = -103.8^K$

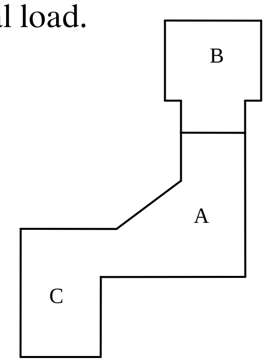
BASE SHEAR $(V) = 108.6^K$

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)

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

$C_d = 4.5$ $\quad T = 0.651$

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.

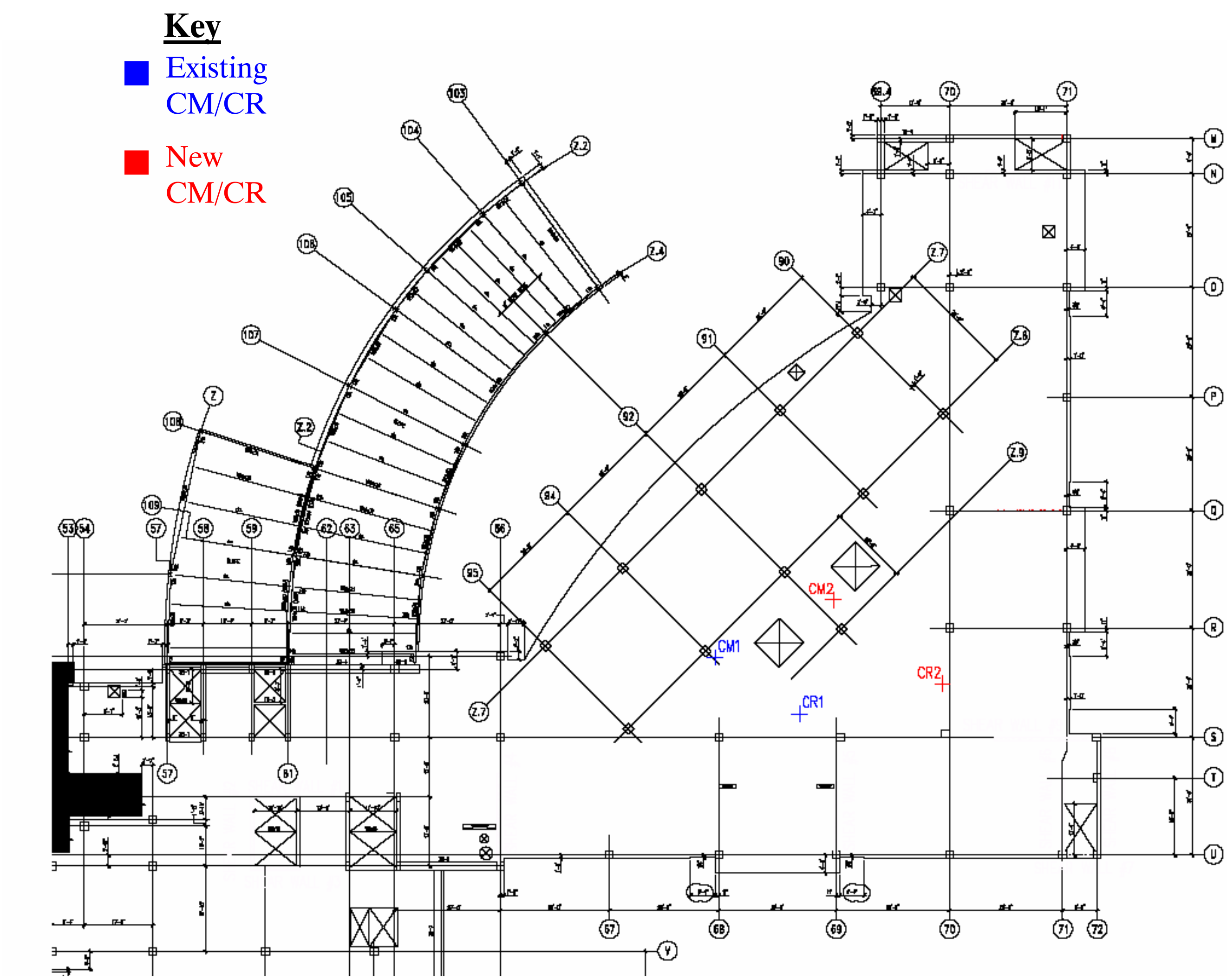


Figure 4 Locations of Center of Mass and Center of Rigidity

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.

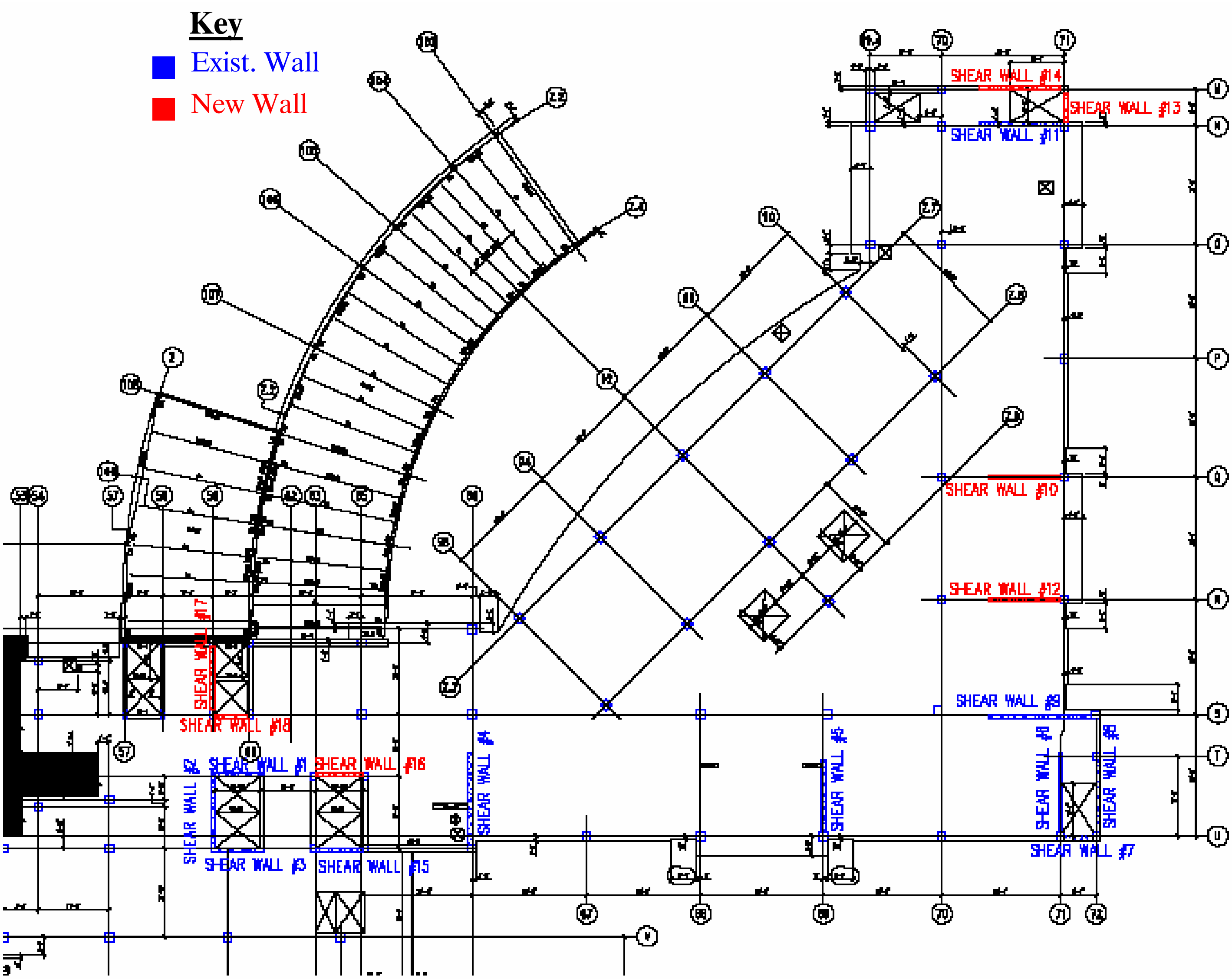


Figure 5 Main Tower Shear Wall Locations

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

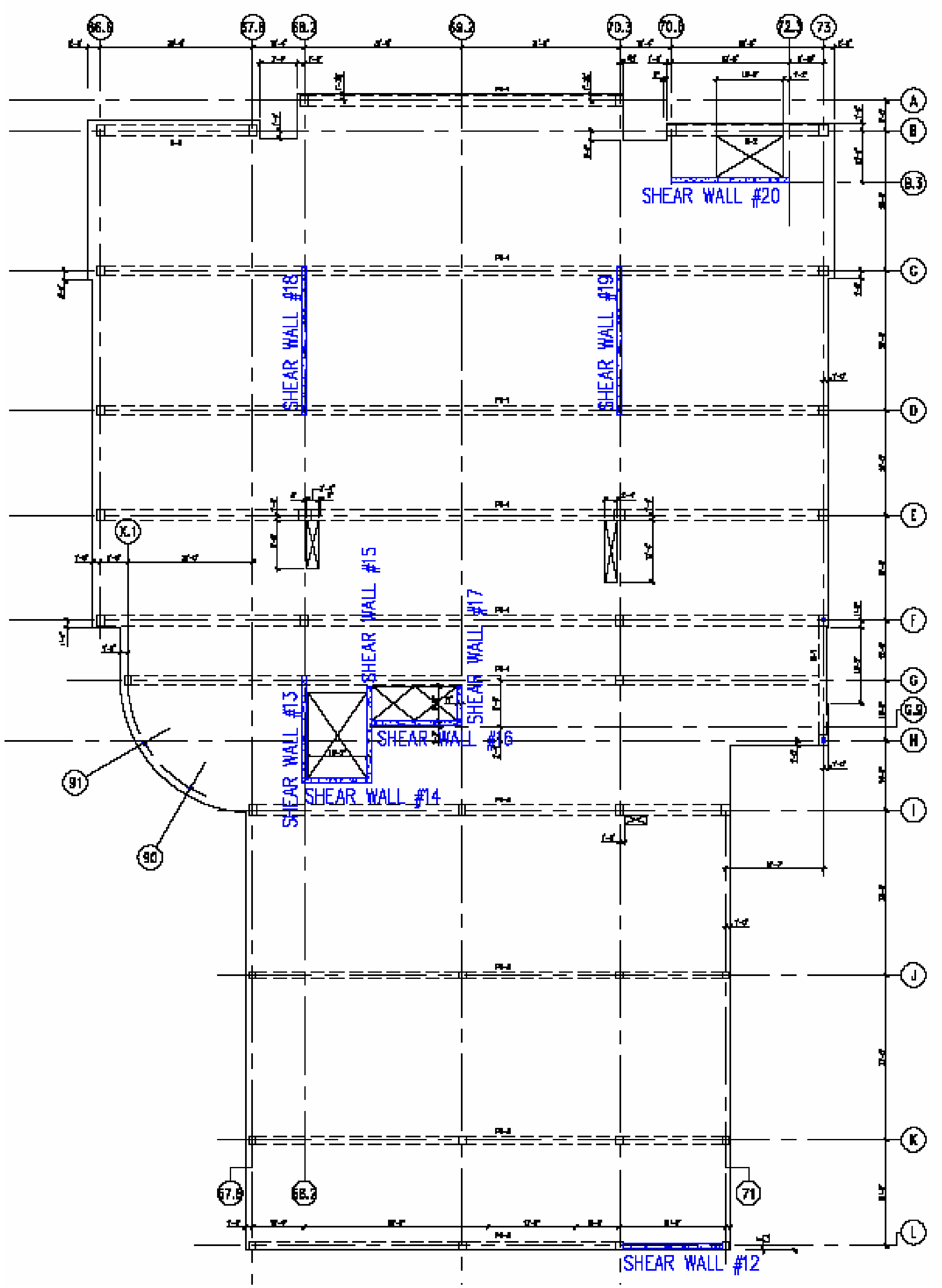


Figure 6 Conference Wing Shear Wall Locations

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) – $A_{ps} = 0.153 in^2$
- 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

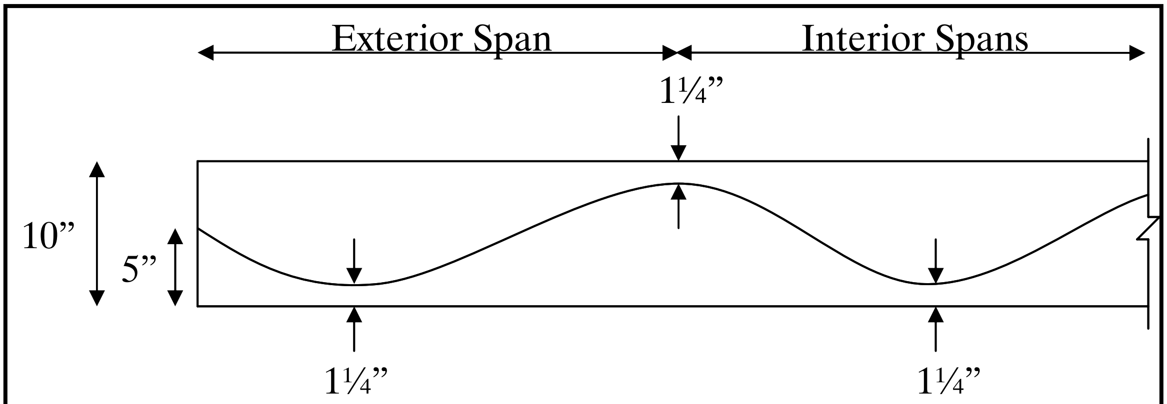


Figure 4 Slab Detail – Tendon Profile

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.

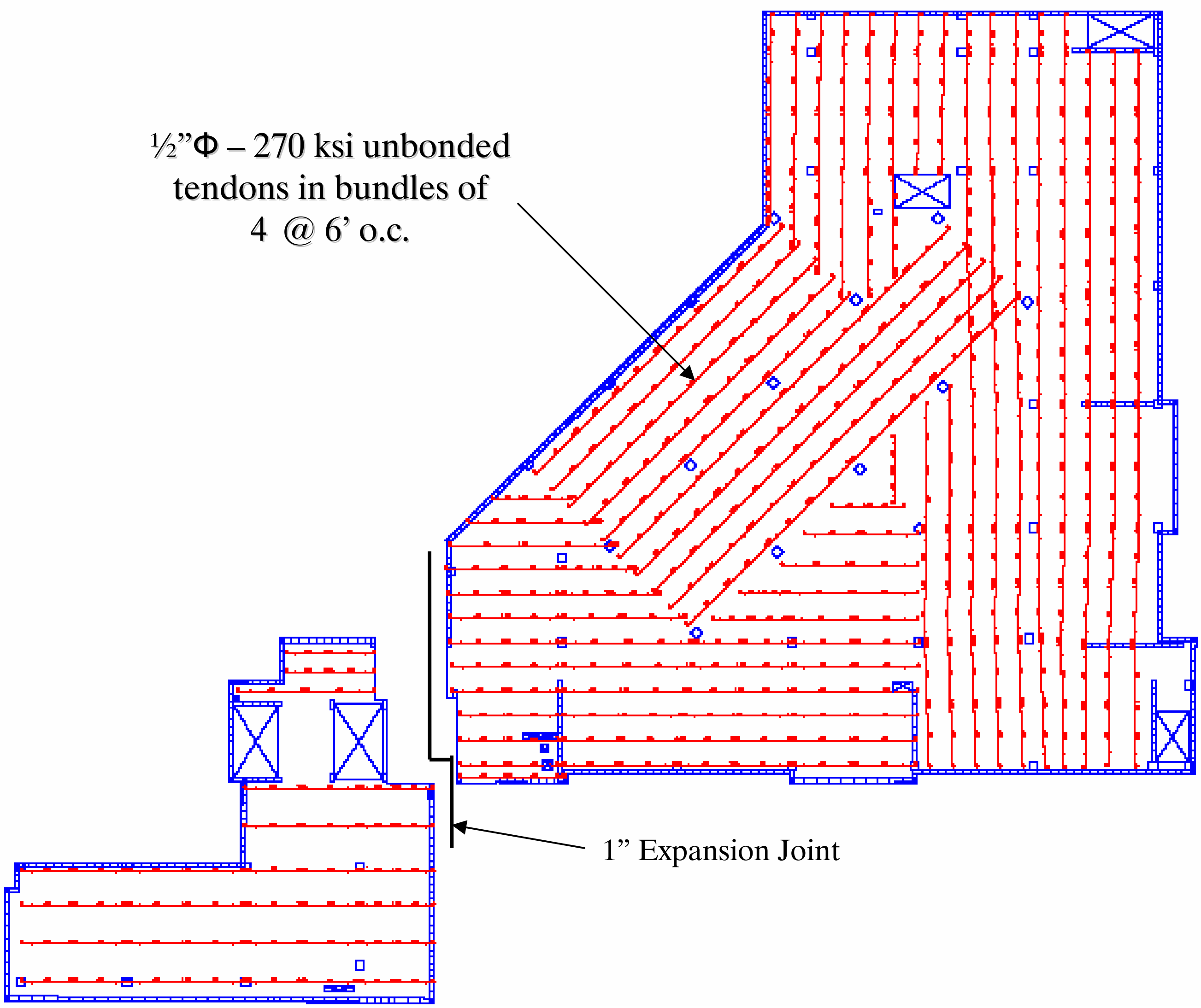


Figure 7 First Floor Uniformly Spaced Tendon Layout

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.

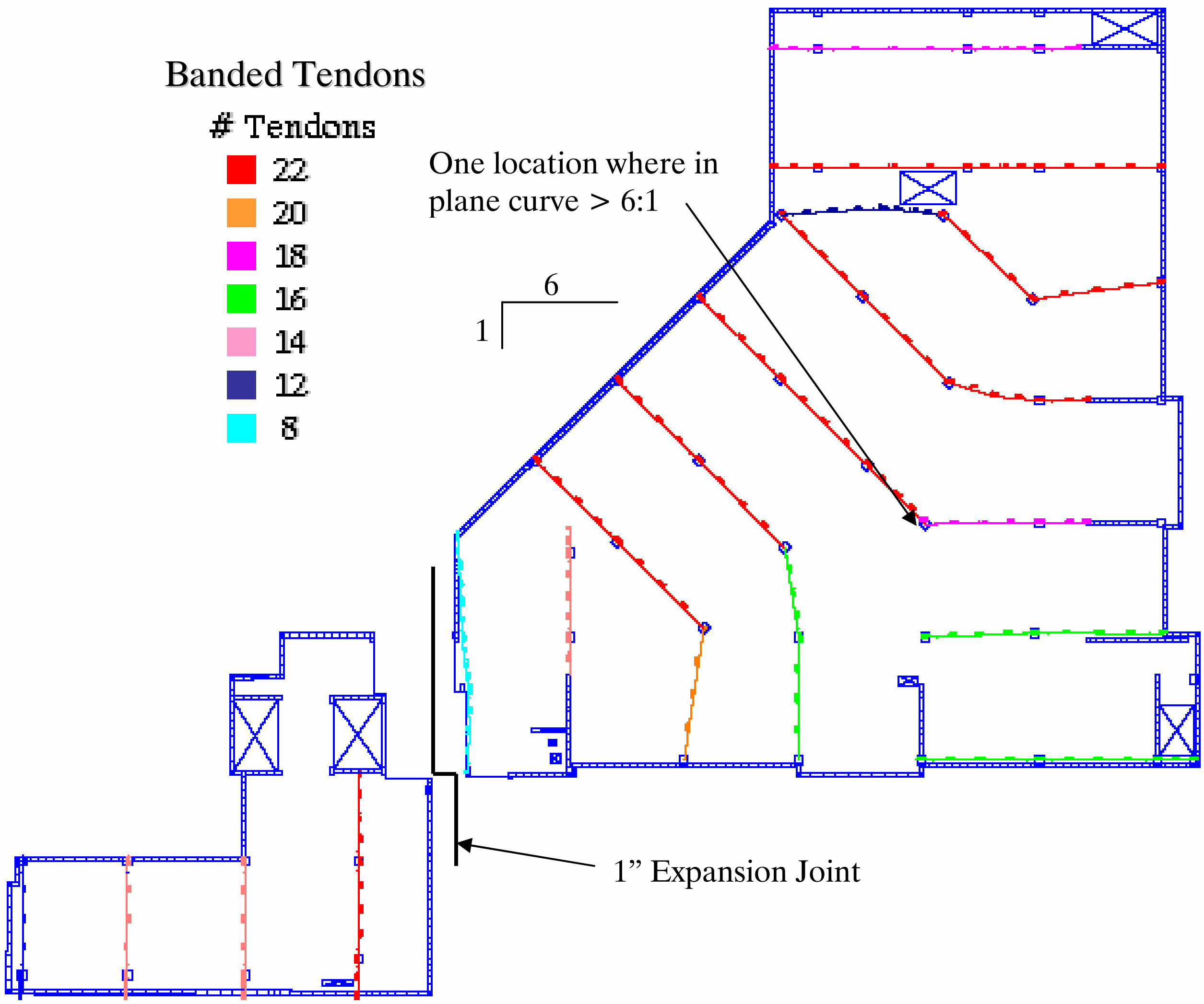


Figure 8 First Floor Banded Tendon Layout

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.

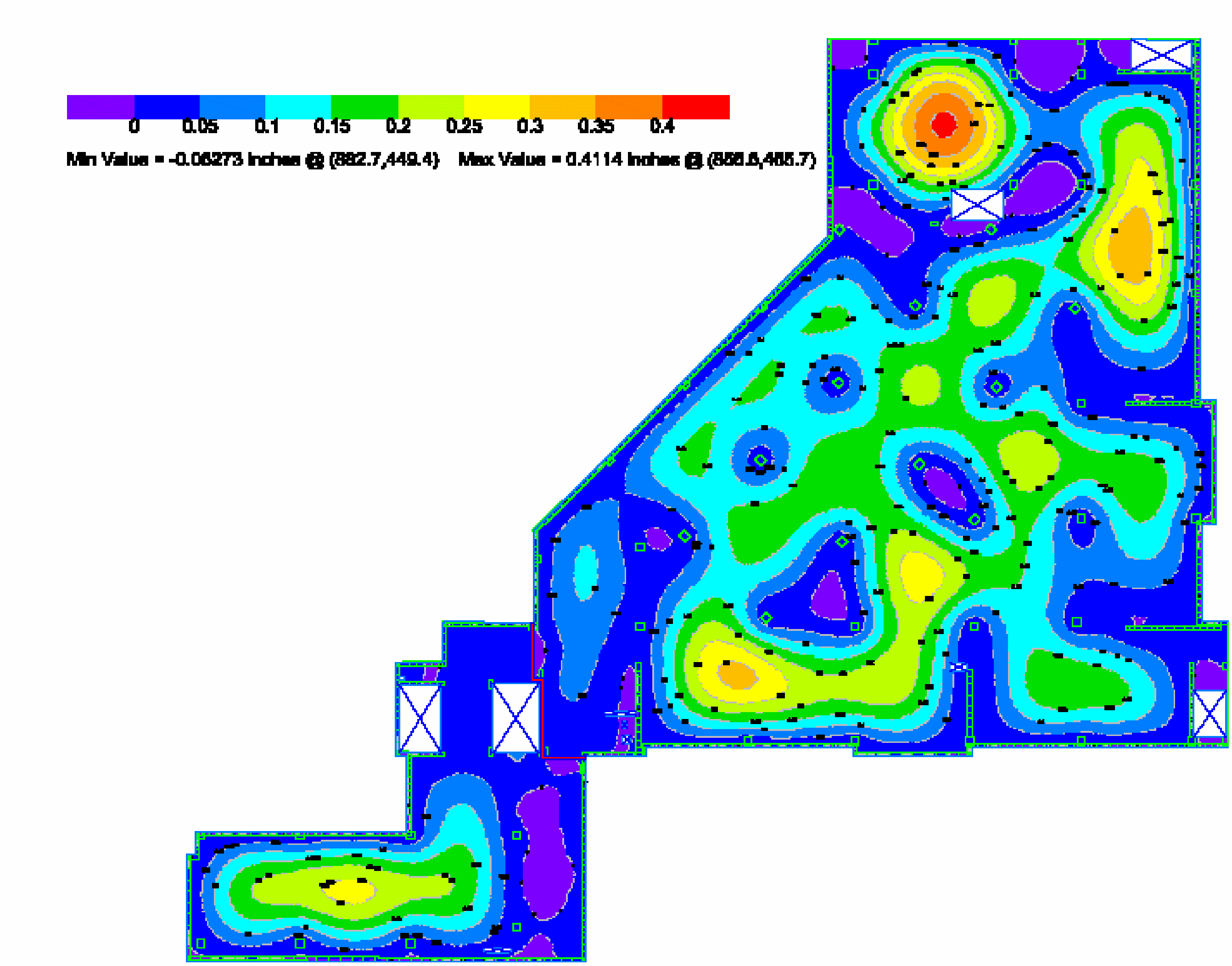


Figure 9 First Floor Sustained Service Load Deflection Plan

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.

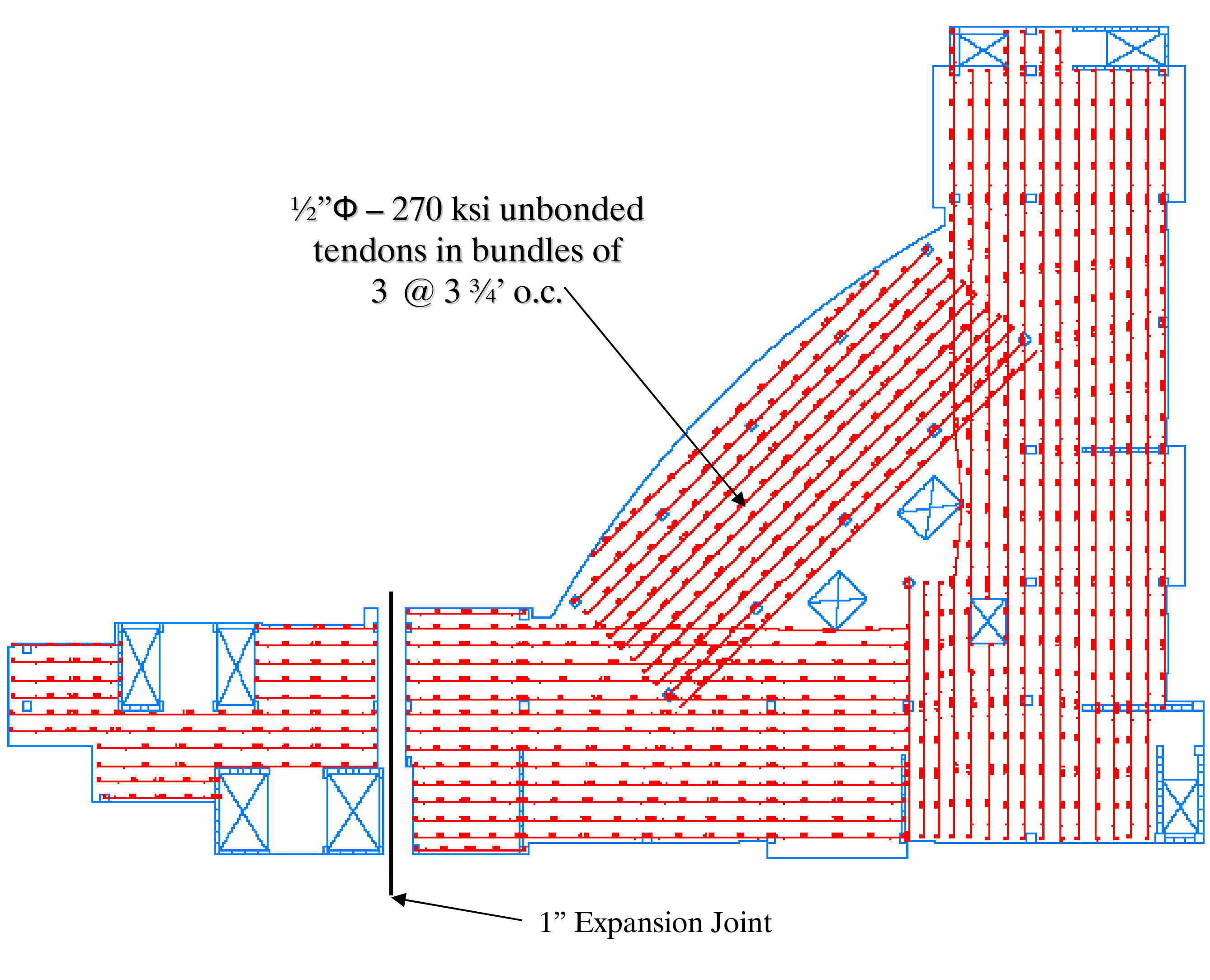


Figure 10 Typical Floor Uniformly Spaced Tendon Layout

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.

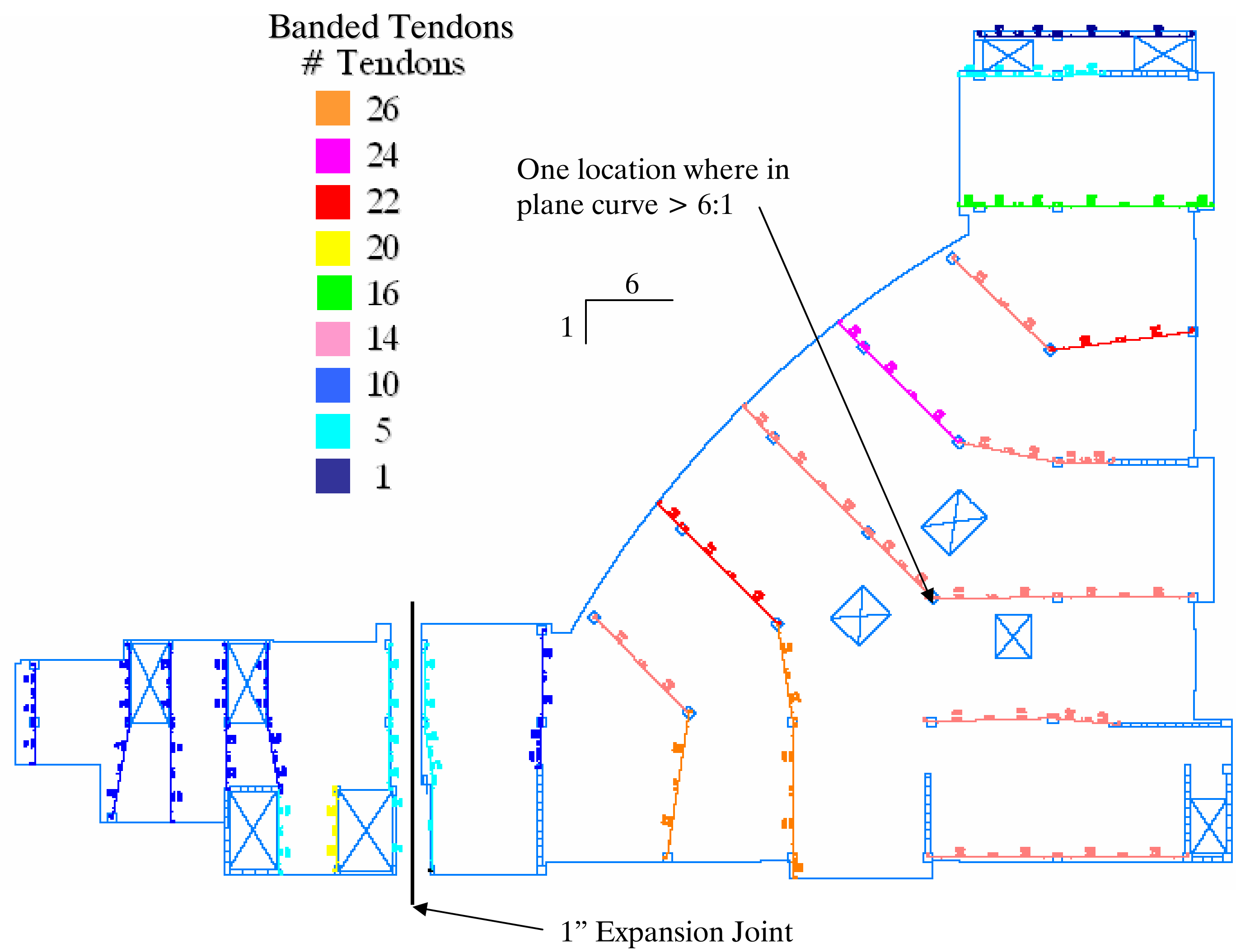


Figure 11 Typical Floor Banded Tendon Layout

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.

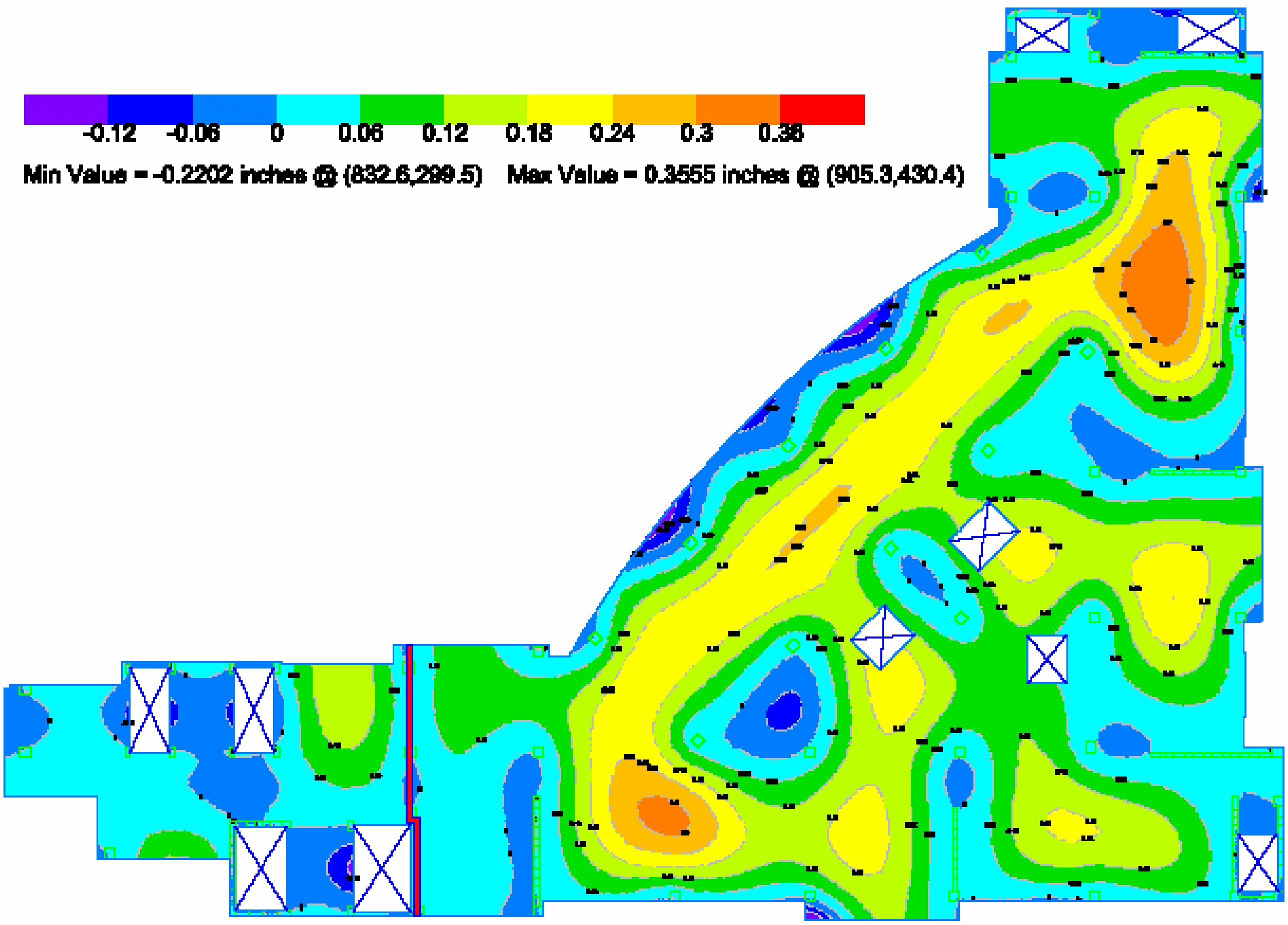


Figure 12 Typical Floor Sustained Service Load Deflection Plan

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	-	-	-	-	-

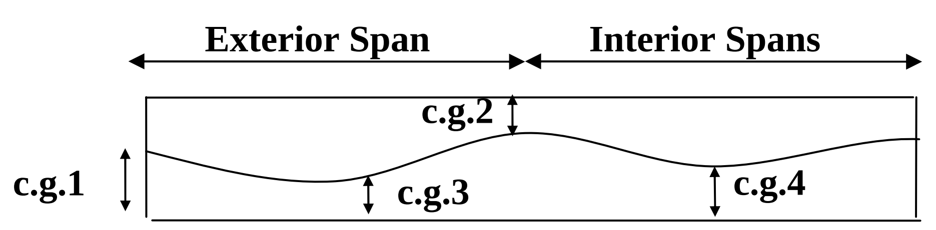


Figure 13 Slab Detail – Tendon Profile

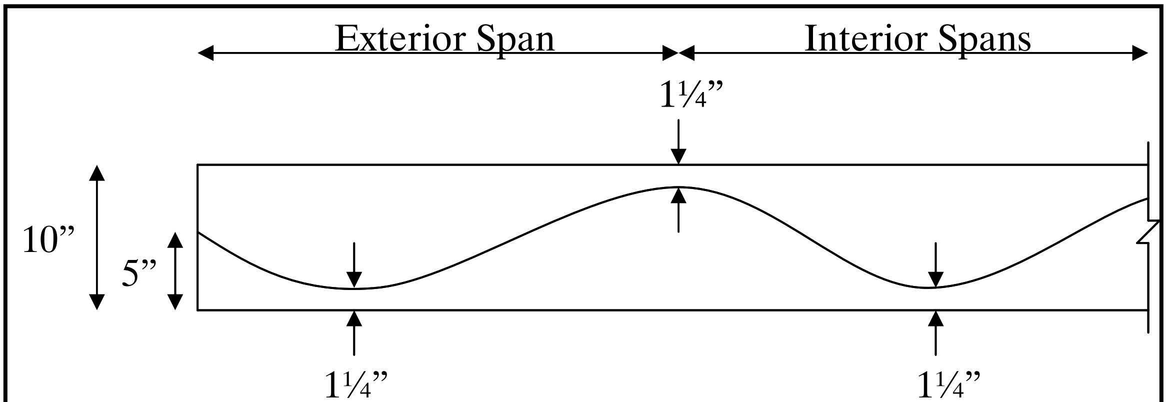


Figure 14 Slab Detail – Tendon Profile

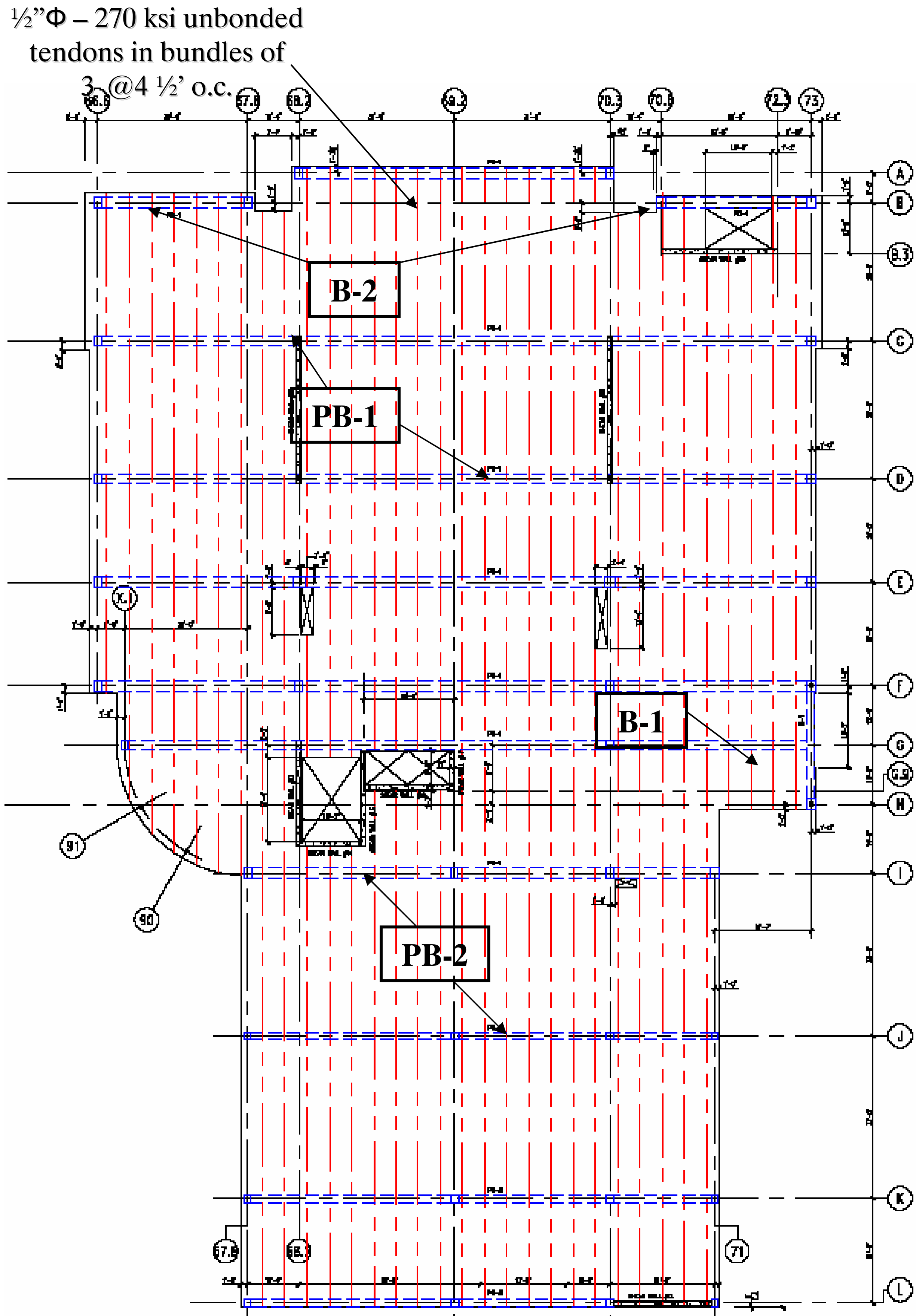


Figure 15 Conference Wing Uniformly Spaced Tendon and Beam Layout

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.

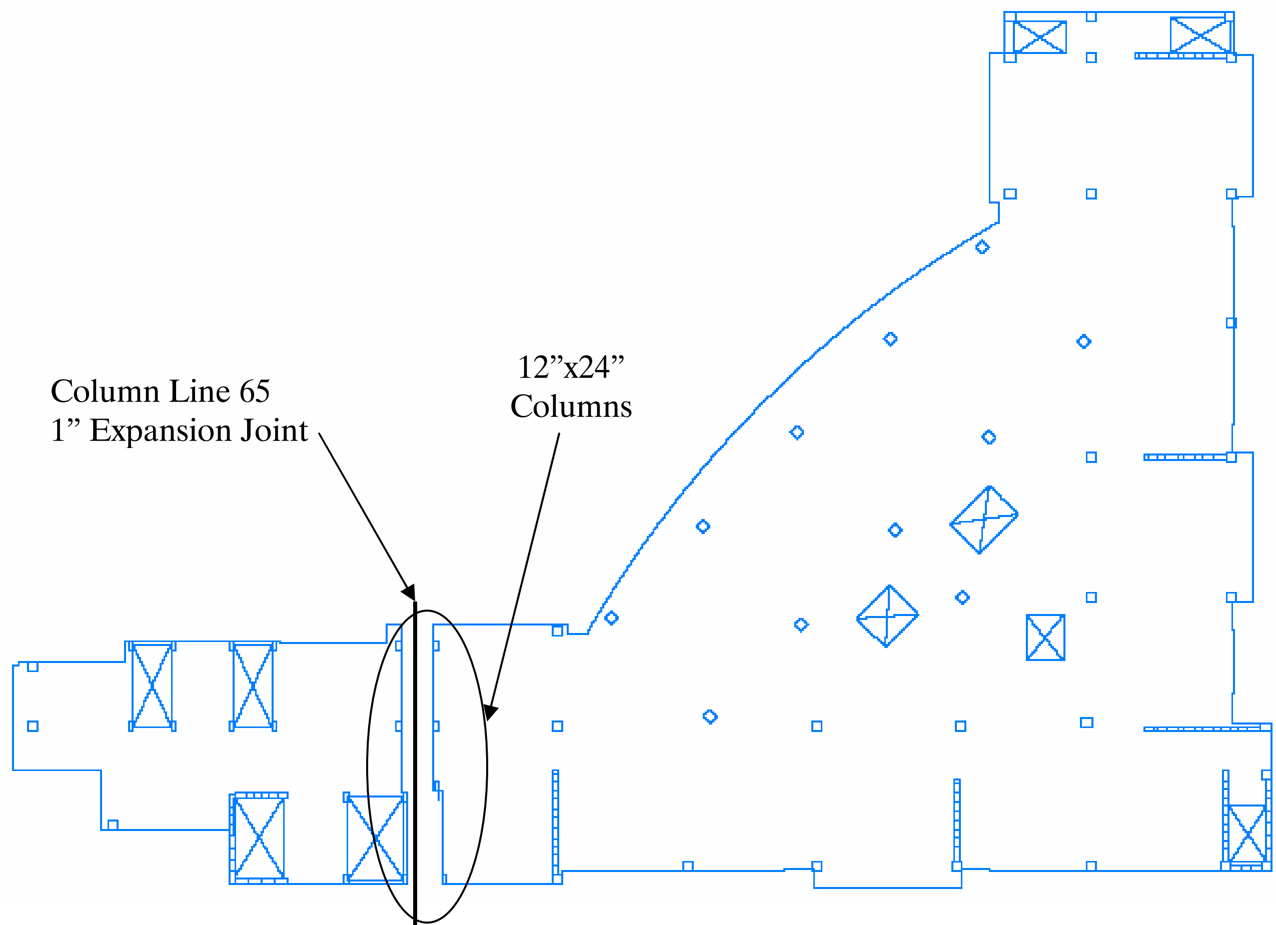


Figure 16 Main Tower Typical Floor Column Layout

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.

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**	

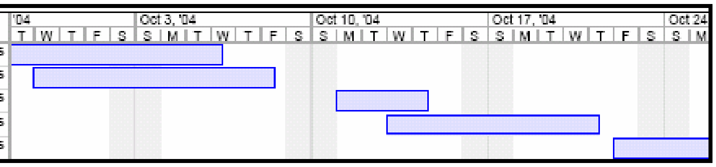
Total Structural System Cost With Added Shear Walls		
	My Design	**As Built**
Concrete		$9,320,230
Structural Steel/ Misc. Metals		$2,897,875
	$12,302,256	$12,218,105
Extra Cost of	**$84,151**	

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

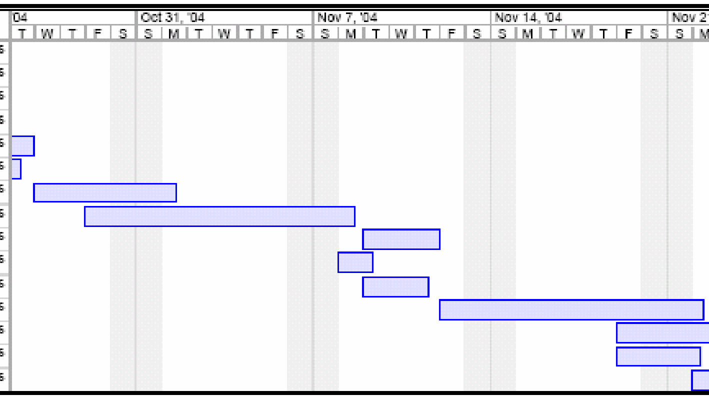
Main Tower Schedule (My Design)

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

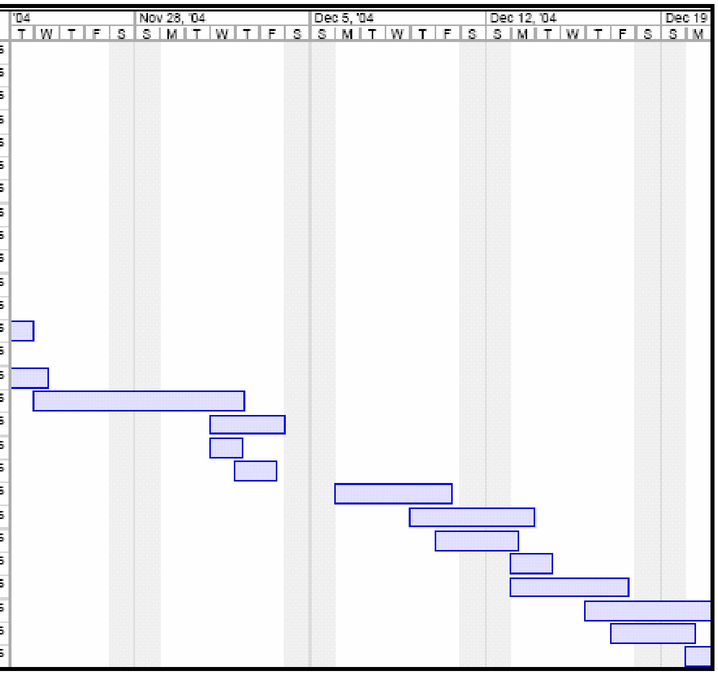
ID	Task Name	Duration
1	Form and Place Mat Foundations/S.O.G.	25.35 days
2	Form and Place Ground Floor Walls	7.61 days
3	Form and Place Ground Floor Columns	3.85 days
4	Form and Place 1st Floor Slab	6.27 days
5	Place and Stress Tendons	3 days



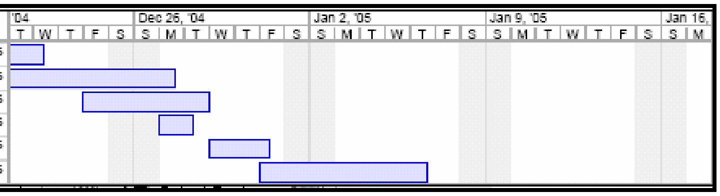
ID	Task Name	Duration
1	Form and Place Mat Foundations/S.O.G.	25.35 days
2	Form and Place Ground Floor Walls	7.61 days
3	Form and Place Ground Floor Columns	3.85 days
4	Form and Place 1st Floor Slab	6.27 days
5	Place and Stress Tendons	3 days
6	Form and Place 1st Floor Walls	1.39 days
7	Form and Place 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

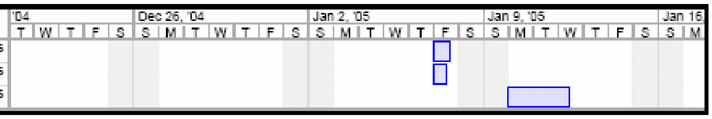


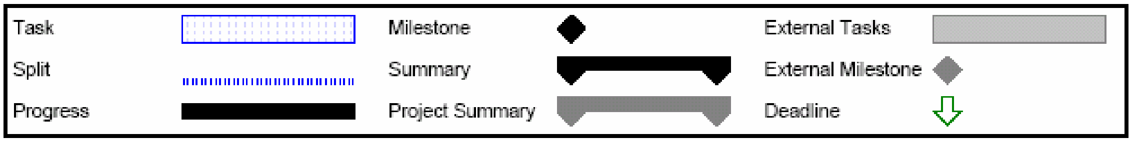
ID	Task Name	Duration
1	Form and Place Mat Foundations/S.O.G.	25.35 days
2	Form and Place Ground Floor Walls	7.61 days
3	Form and Place Ground Floor Columns	3.85 days
4	Form and Place 1st Floor Slab	6.27 days
5	Place and Stress Tendons	3 days
6	Form and Place 1st Floor Walls	1.39 days
7	Form and Place 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
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

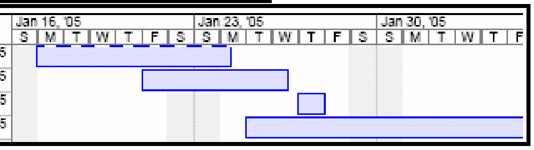




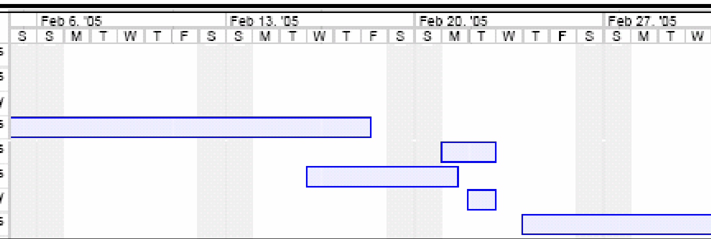


Conference Wing Schedule (My Design)

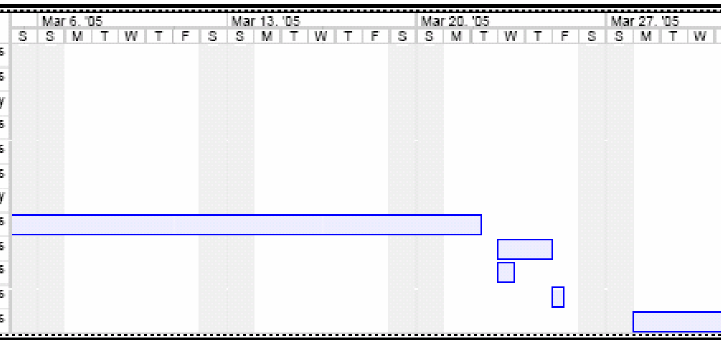
ID		Task Name	Duration	Start	Finish
1		Form and Place Spread/Wall Footings/S.O.G.	5.21 days	Mon 1/17/05	Mon 1/24/05
2		Form and Place Ground Floor Walls	3.65 days	Fri 1/21/05	Wed 1/26/05
3		Form and Place Ground Floor Columns	1 day	Thu 1/27/05	Thu 1/27/05
4		Form and Place 1st Floor Slab	18.2 days	Tue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

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