Franklin Square Hospital Center Patient Tower

Baltimore, MD

Final Report

Thomas Weaver

Structural Option

AE 497G/897G Senior Thesis

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Franklin Square Hospital Center

Patient Tower and Emergency Department Addition

9000 Franklin Square Drive, Baltimore MD

General Statistics:

Size: 356,000 SF
Number of Stories: 7
Function: Medical
Cost: $176 Million Project Cost
Construction Dates: November 07 - October 10
Delivery Method: Design—Bid—Build

Project Team:

Owner: Franklin Square Hospital Center
Project Manager: Lillibridge Healthcare Services
Construction Manager: Bovis Lend Lease
Architect: Wilmont/Sanz Inc.
Structural: Rathgeber/Gross Associates
Civil: Dewberry and Davis
MEP: Leach Wallace Associates

Architecture:

- Precast wall panels with exposed concrete, brick veneer, and stucco finish visually offset by exposed concrete bands between floors
- Sun shades extend out from the buildings face providing shelter for the large curtain wall sections
- Main entrance leads through a large three story atrium featuring the lobby
- Spaces Include:
    - 291 private inpatient rooms
    - Expanded emergency department
    - Dedicated pediatric emergency department and inpatient suite
    - Four new medical and surgical units
    - Expanded 50 bed critical care unit

Structure:

Framing System and Lateral System:
- Concrete columns, edge beams, and 10" slabs function as both the gravity system and moment frame lateral system.

Floor System:
- 10" concrete two-way slabs spanning a typical 30'x30' bay

Foundation:
- Drilled 4' concrete piers extending 42' below grade
- 24"x24" perimeter grade beams with 5" slab on grade ground floor

Roof System:
- 1.5" deep wide rib 20 gauge galvanized metal deck on cambered steel beams and steel columns

Future Expansion:
- Portion of ground floor that extends past the tower has oversized columns for future tower addition.
- Portion of roof system is strengthened for future heliport

MEP Systems:

Mechanical:
- Onsite central plant with two three-ton cooling towers, two chillers, and two fuel oil tank boilers
- Air handling units using variable percentage of outdoor air ranging from 10% to 50%
- Variable air volume terminal units with hot water reheat and variable volume return system

Electrical:
- 208Y/120V, 3 phase, 5 wire system
- Emergency 480Y/277V, 3 phase, 4 wire system using diesel engine generators

Lighting:
- Fluorescent lighting
- Ultrasonic and infrared occupancy sensors

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http://www.engr.psu.edu/ae/thesis/portfolios/2010/tww137

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Executive Summary

The goal of designing a more efficient lateral and gravity floor system for the Franklin Square Hospital Center Patient Tower was a difficult one considering the excellent job the design team did. When applying that goal to the Franklin Square Hospital Center as located in San Francisco California, the research and design necessary leads to unforeseen complexities. One such complexity involved the choice of lateral system to use while another was the choice of floor system to use and how to make everything work as a whole package.

The lateral system types explored were a specially reinforced concrete moment frame system and a specially reinforced concrete shear wall system. Both systems contained positives and negatives but the negatives of the moment frame design proved too great to overcome when research was completed. The moment frame system, while perfectly applicable to the original design in Baltimore, was far too large and imposing with column sizes of 34"x34" and beam sizes of 34"x36" when sized for San Francisco seismic forces. The optimal lateral force resisting system turned out to be a shear wall system with a centrally placed core and supplementary shear wall segments located in the northern and southern wings of the building. These walls at their thickest were 22" and at their thinnest 12". The footprint impact of the shear wall system on the interior spaces was far less than the massive moment frame impact. However, the interior architectural layout of the interior spaces needed rearrangement due to the transplant of the elevator core to the center of the building. The architectural changes resulted in space arrangements that are similar to the original plan design and still offer the necessary flexibility and accessibility required in a hospital.

The floor system change was far less complex than the lateral system change was. An 8" PT flat slab was implemented with 4'x4'x2" drop panels over columns. This much thinner floor system resulted in a building weight decrease of 5,800 kips or roughly 10%. In addition to the benefits of reduced building weight, the PT floor system also cost slightly less when just materials and labor are accounted for by close to $100 thousand. However, the length of construction is extended by four additional weeks with the post-tensioned system. With general conditions estimated around $40 thousand a week, the change in schedule costs an additional $160 thousand. Therefore the net increase in price of the post-tensioned floor system over the 10" regularly reinforced slab is $60 thousand or around 22 cents a square foot.

Given that the cost increase from the post-tensioned slab is less than the cost increase from a more substantial lateral system, the ideal structural system for the Franklin Square Hospital Center located in San Francisco, CA would be a concrete shear wall system with an 8" post-tensioned slab.

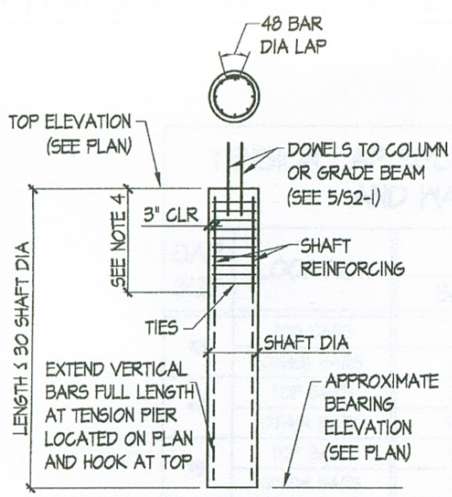
Introduction

The Franklin Square Hospital Center Patient Tower is a 7 story 356,000 square foot hospital addition that serves the existing Franklin Square Hospital campus and is a $175 million investment in the community. The new Patient Tower adds 291 private inpatient rooms with spacious modern designs for privacy, safety and convenience. Each private room offers a window for ample sunlight, a private bathroom for easier access and enhanced technology to support safe, quality care. An expanded emergency department provides greater privacy with walls, not curtains, separating patients. The expanded lab in the emergency department can also run more tests in less time. In addition there is easier access to CAT scan and diagnostic services. A dedicated pediatric emergency department offers child friendly intake and triage rooms with pediatric specialist doctors and nurses who understand the concerns of children that aren't feeling well and their parents. This pediatric emergency department is also connected to inpatient suites with larger rooms that better accommodate families spending the night. Four new medical and surgical units and an expanded 50 bed critical care unit round out the offerings of the new Franklin Square Hospital Center Patient Tower.

Existing Conditions—Structural Systems

Foundation System

The foundation system of the Franklin Square Hospital Patient Tower consists of drilled piers or caissons 4 feet in diameter and centered under columns or slightly offset under perimeter grade beams. The piers range in size from 1.5 feet in diameter to 5 feet in diameter. They are embedded a minimum of 20 feet into bedrock. The total typical depth of the piers is around 42 feet below grade pending geotechnical engineer inspection. See Figure 1, "Drilled Pier Reinforcing."



SHAFT DIAMETER	REINFORCEMENT	
	VERTICAL	TIES
1'-6"	6 #5	#3 @ 10"
2'-0"	6 #6	#3 @ 12"
2'-6"	6 #7	#3 @ 14"
3'-0"	7 #8	#3 @ 16"
3'-6"	7 #9	#4 @ 18"
4'-0"	7 #10	#4 @ 18"
4'-6"	9 #10	#4 @ 18"
5'-0"	9 #11	#4 @ 18"

NOTES:
1) SEE PLANS FOR DIAMETER OF SHAFT.
2) SHAFT AXIS SHALL NOT BE OUT OF PLUMB MORE THAN 3" IN 10'-0".
3) THE BOTTOM BEARING SHALL BE LEVEL AND ALL DISTURBED MATERIAL REMOVED.
4) EMBED ≥ 3x SHAFT DIAMETER (20'-0" MINIMUM).

Figure 1: Drilled Pier Reinforcing

The piers are required to be a normal weight concrete with a concrete compressive strength (f'_c) of 3000 psi. As previously mention, the piers directly support interior columns. See Figure 2, "Column Caisson Connection and Column Reinforcing."

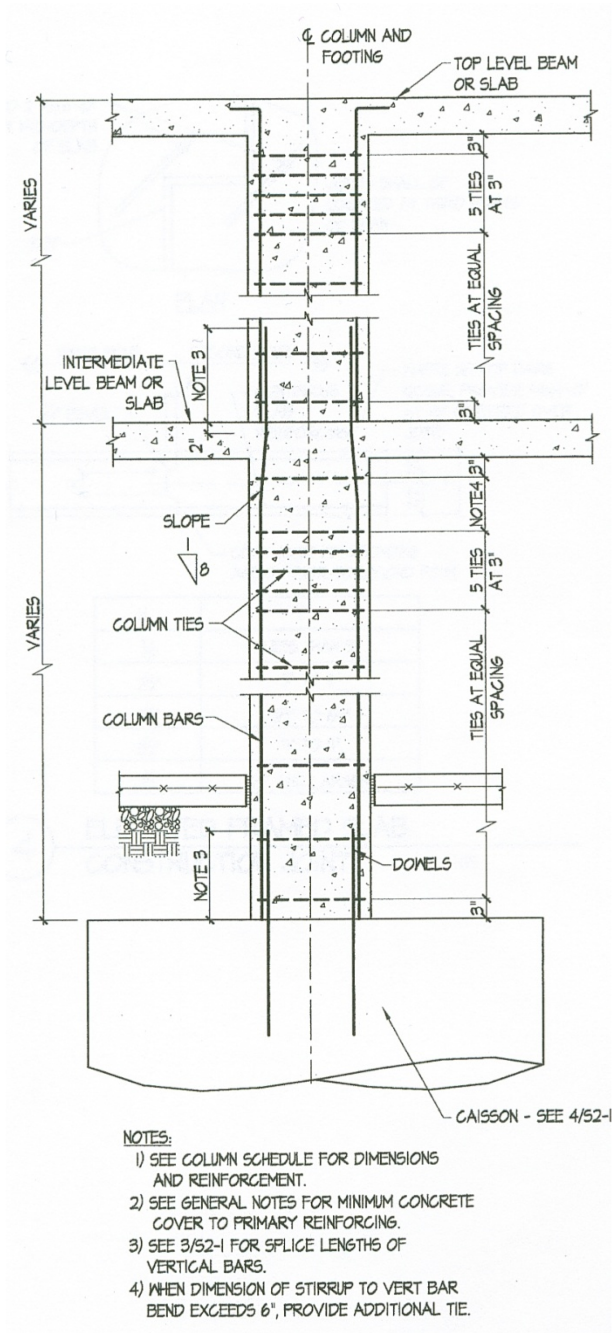


Figure 2: Typical Column Caisson Connection and Column Reinforcing

The piers also directly support perimeter grade beams. The typical grade beam is 24”x24” with some that are 36”x24”. See Figure 3, “Typical Grade Beam Caisson Connection.”

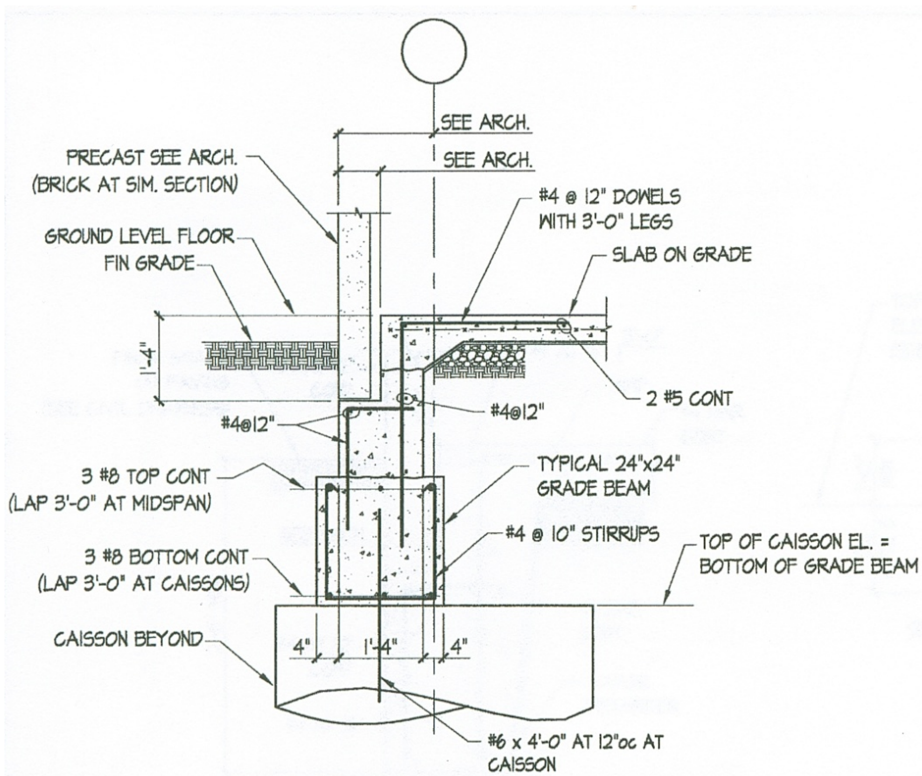


Figure 3: Typical Grade Beam Caisson Connection

While there are no sub grade levels in the structure, the west side of the ground floor can be considered below grade because the ground has been filled to provide on grade access to the first floor lobby. The existing hospital ground floor also resides on the level corresponding to the patient tower’s first floor. Lateral soil pressures from the foundation of the existing building are resisted by a 16” thick foundation wall in these areas. See Figure 4, “Typical Foundation Wall Section.”

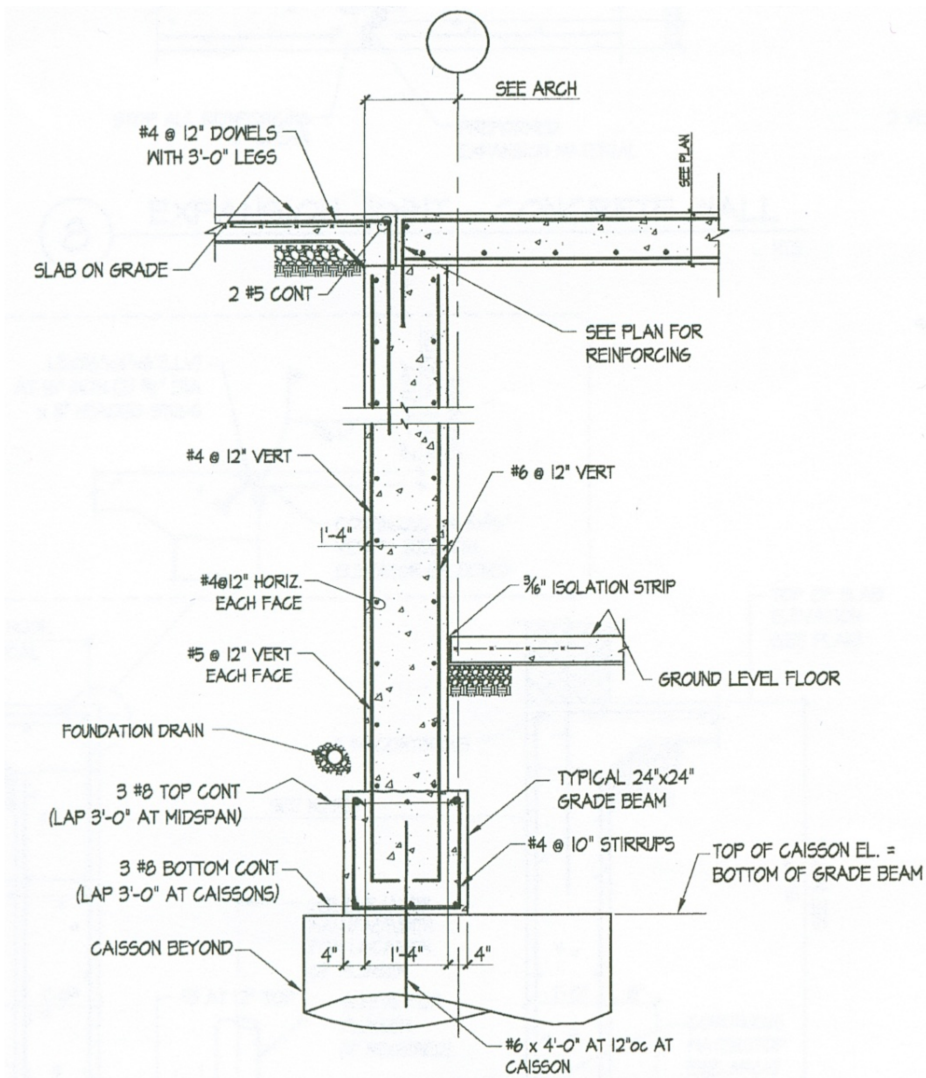


Figure 4: Typical Foundation Wall Section

The rest of the foundation consists of a 5 inch ground floor slab on grade of compressive strength equal to 3000 psi. The slab on grade is reinforced with 6x6-W2.9xW2.9 welded wire fabric over a 4 inch layer of clean, well-graded gravel or crushed stone.

Floor System

The buildings typical floor system is a 10" reinforced two way slab, or flat plate, spanning a typical 30'x30' bay. The reinforcing varies a great deal depending on location and span but for the most part there is a continuous bottom mat of #5 or #6 bars at 12" each way with continuous top reinforcing within the column strips with mostly #6 or #8 bars. See Appendix A for Floor Plans and Figure 5, "Slab Reinforcing Detail."

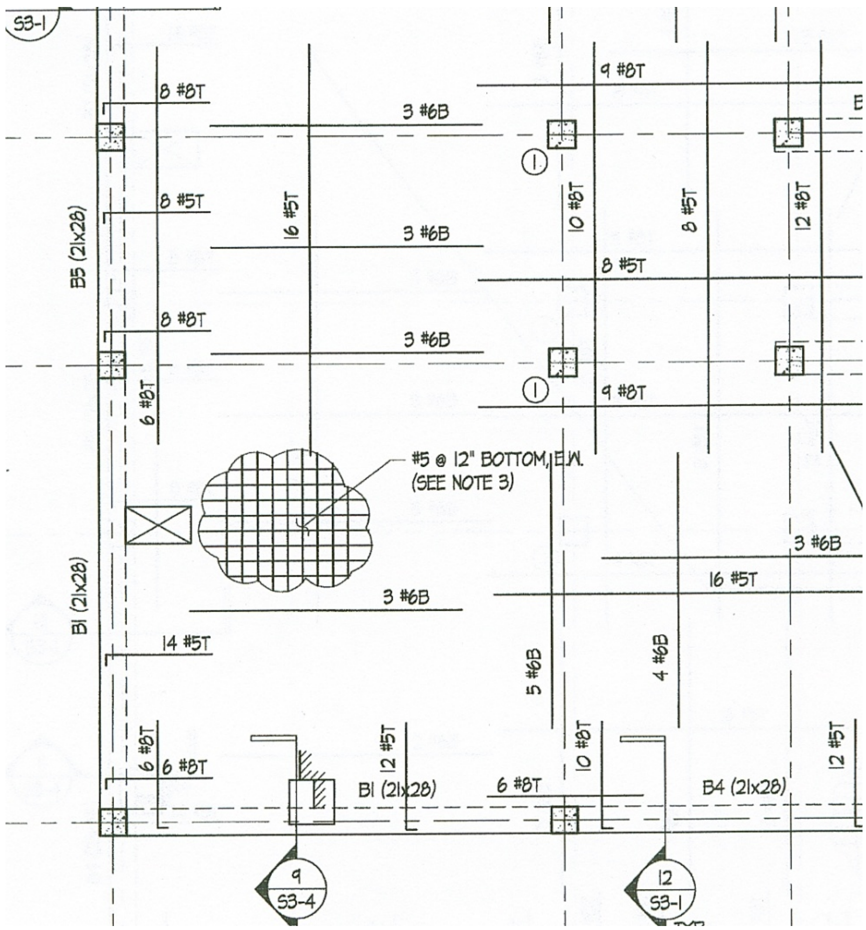


Figure 5: Slab Reinforcing Detail

The floor system also consists of edge beams that wrap the perimeter of the slab and surround openings such as stairs, elevators, and mechanical shafts. The typical edge beam is 21"x28" reinforced with #9 bars top and bottom. See Figure 6, "Portion of Concrete Beam Schedule."

CONCRETE BEAM SCHEDULE

MARK	SIZE: W (INCHES)	SIZE: D (INCHES)	REINFORCING: BOTTOM BARS	TOP BARS: LE	TOP BARS: FL	TOP BARS: RE	STIRRUPS: SIZE	STIRRUPS: TYPE	STIRRUPS: SPACING (INCHES)	STIRRUPS: END	REMARKS
B1	21	28	3#9	-	2#9	-	#4	S2	1@2, 12@12, R@18	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B2	12	28	3 #9	-	3#9	-	#4	S2	1@2, R@10	EE	
B3	10	28	3 #8	-	3#8	-	#4	S2	1@2, R@12	EE	
B4	26	20	3 #9	-	3#9	-	#4	S3	1@2, R@8 CANT. 1@2, R@8	EE	
B5	21	28	2#9	-	2#9	-	#4	S2	1@2, R@12	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B6	21	28	4#9	-	3#9	-	#4	S2	1@2, R@8	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B7	21	28	3#9	1#9	2#9	1#9	#4	S2	1@2, 18@8, R@12	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B8	21	28	3#9	-	2#9	3#9	#4	S2	1@2, 16@12, R@18	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B9	26	20	3#9	3#9	2#9	3#9	#4	S3	1@2, 20@8, R@18	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B10	22	20	4#9	5#10	2#10	5#10	#4	S3	1@2, 12@4, R@6	EE	
B11	26	20	3#9	3#9	2#9	3#9	#4	S3	1@2, 20@8, R@18	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B12	21	28	3#9	2#9	2#9	2#9	#4	S2	1@2, 14@12, R@18	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B13	26	20	5#9	5#9	-	7#10	#4	S3	1@2, 12@4, R@8	EE	
B14	20	20	3#9	6#9	-	6#9	#4	S3	1@2, R@6	EE	
B15	12	28	3#9	1#9	2#9	1#9	#4	S2	1@2, 6@8, R@12 CANT. 1@2, R@8	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B16	20	20	2#9	-	2#9	-	#4	S2	1@2, 6@8, R@12	EE	
B17	12	20	2#9	3#9	-	3#9	#4	S2	1@2, 16@6, R@12	EE	
B18	22	24	4#9	1#9	2#9	1#9	#4	S2	1@2, 15@10, R@18	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B19	22	24	4#9	-	2#9	-	#4	S2	1@2, 15@10, R@18	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B20	22	24	3#9	-	2#9	-	#4	S2	1@2, 5@10, R@18	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B21	21	28	3#9	1#9	2#9	1#9	#4	S2	1@2, 12@12, R@18	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B22	21	28	5#9	-	2#9	-	#4	S2	1@2, R@10	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B23	21	16	2#9	-	2#9	1#9	#4	S2	1@2, 16@6, R@12	EE	
B24	21	28	5#9	2#9	2#9	2#9	#4	S2	1@2, R@12	EE	
B25	30	28	3#9	4#9	4#9	-	#4	S3	1@2, 12@12, R@18 CANT. 1@2, R@12	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B26	21	28	5#9	2#9	2#9	-	#4	S2	1@2, 10@6, R@8	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B27	21	28	3#9	2#9	2#9	-	#4	S2	1@2, 10@6, R@12	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B28	21	28	2#9	-	2#9	2#9	#4	S2	1@2, R@8	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B29	21	28	5#9	1#9	2#9	1#9	#4	S2	1@2, 12@6, R@10	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B30	21	28	3#9	5#9	2#9	-	#4	S2	1@2, 16@4, R@12	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B31	21	28	3#9	-	2#9	5#9	#4	S2	1@2, 16@4, R@12	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B32	21	28	5#9	2#9	2#9	2#9	#4	S2	1@2, 10@6, R@12	EE	PROVIDE 2 #9 WEB BARS AT MID-DEPTH
B33	26	22	2#9	3#9	-	3#9	#3	S2	1@2, R@6	EE	

Figure 6: Portion of Concrete Beam Schedule

Columns

The columns are for the most part 21"x21" and 22"x22 with (8) #9 bars. Instead of changing column sizes as the building rises, the engineers specified different concrete compressive strengths for different levels and reduced the reinforcing to (8) #8's in spots. The ground to 3rd floor columns have a 28 day compressive strength of 7000 psi and the columns from the 3rd floor to the roof have a 28 day compressive strength of 5000 psi.

The portion of the tower that does not rise past the ground floor has oversized columns designed for future expansion. The Franklin Square Hospital Center Patient Tower was realized because the existing hospital had no capacity left for additional floors. Desperately needing space, the hospital commissioned the Patient Tower and supporting spaces. In the future when such a situation arises, the new Patient tower will be able to grow with the needs of the hospital. See Figure 2, "Typical Column Caisson Connection and Column Reinforcing" and see Figure 7, "Portion of Concrete Column Schedule."

LEVEL \ COLUMN		L-1 M3-1 P-1	K-2 L-2 K-12.4 L-12.4	J-7, J-8 K-7, K-8 L-7, L-8	M-3 N-3	M-6 M-7 M-8 M-9	M-4, M-5 M-10, M-11 N-4, N-5	N-12 P-6	N-6 N-7, N-8 N-9, N-10 N-11	P-3 P-4 P-5	M-12	J-9, L-6 K-9, L-9 H-6, J-6 K-6	F-4, F-5 F-6, F-10 F-11	G-4, G-5 G-6, G-10 G-11
PENTHOUSE ROOF	SIZE													
	VERTICAL BARS													
	TIES													
	REMARKS													
MAIN ROOF/ SEVENTH FLOOR	SIZE		30x12											
	VERTICAL BARS		6#8											
	TIES													
	REMARKS													
SIXTH FLOOR	SIZE		30x12									21x21	22x22	22x22
	VERTICAL BARS		6#8									8#9	8#9	8#9
	TIES													
	REMARKS													
FIFTH FLOOR	SIZE		30x12									21x21	22x22	22x22
	VERTICAL BARS		6#8									8#9	8#9	8#9
	TIES													
	REMARKS													
FOURTH FLOOR	SIZE		30x12	FUTURE EXPANSION	FUTURE EXPANSION	FUTURE EXPANSION	FUTURE EXPANSION	FUTURE EXPANSION	FUTURE EXPANSION	FUTURE EXPANSION	FUTURE EXPANSION	21x21	22x22	22x22
	VERTICAL BARS		6#8									8#9	8#9	8#9
	TIES													
	REMARKS													
THIRD FLOOR	SIZE		30x12									21x21	22x22	22x22
	VERTICAL BARS		6#8									8#9	8#9	8#9
	TIES													
	REMARKS													
SECOND FLOOR	SIZE		30x12									21x21	22x22	22x22
	VERTICAL BARS		6#10									8#9	8#9	8#9
	TIES													
	REMARKS													
FIRST FLOOR	SIZE		30x12									21x21	22x22	22x22
	VERTICAL BARS		6#10									8#9	8#9	8#9
	TIES													
	REMARKS													
GROUND FLOOR	SIZE	21x21	30x12	22x22	22x22	22x22	22x22	21x21	21x21	21x21	21x21	21x21	22x22	22x22
	VERTICAL BARS	12#10	6#10	8#10	8#10	8#9	8#10	8#11	8#11	8#11	8#10	8#9	8#9	8#9
	TIES	4#8		4#8	4#8	4#8	4#8	4#8	4#8	4#8	4#8			
	REMARKS													
DOWELS		12#7	6#7	8#8	8#8	8#8	8#8	8#8	8#8	8#8	8#8	8#7	8#7	8#7

Figure 7: Portion of Concrete Column Schedule

Roof System

The main roof system consists of cambered steel beams ranging from W12x14 to W21x73 and 1.5" deep, wide rib, 20 gauge galvanized metal deck with 3 ¼" lightweight concrete. Many of these beams are moment connected to the steel columns supporting them. A center portion of the roof contains a 10" reinforced concrete slab with concrete columns extending 2' above the surface for future placement of the helipad deck. See Appendix A for "Roof Framing Plan."

Wall System

The exterior façade is for the most part 7" precast concrete panels. Loads bearing connections occur at each level, with two per panel. The connections permit horizontal movement parallel to the panel except for a single non-load bearing connection which is fixed. Precast panel loads are supported only by the columns.

Lateral System

The Franklin Square Hospital Center Patient Tower utilizes the entire structure to resist lateral forces. Every column, slab and beam acts as an ordinary reinforced concrete moment frame resisting forces in both the North-South direction and the East-West direction. The large moments are carried down the building through the columns and directly into the drilled piers. The piers, with depths of 42 feet, are quite substantial and help greatly to give the building a rigid, fixed base.

In the case of wind, the force exerted on the precast panels is directly transferred to the columns and not the floor diaphragm. Once this occurs, the force is carried down the column and across the floor diaphragm to the remaining columns. The columns are expected to resist the lateral force through their moment capacity. The perimeter edge beams are stiffer than the diaphragm and function as more efficient moment frames. There are a total of 13 moment frames acting in each direction for a total of 26 moment frames in the structure. Some are very rigid and take much of the load while others are very flexible and do little in terms of lateral force resistance. The frames that reside on the perimeter of the building have beam elements consisting of substantial 21"x28" edge beams. These are the frames that take the majority of the lateral loads compared to the rest of the frames that have beam elements consisting of the slab cross-section. Figure 9, "Concrete Moment Frames Level 4" shows the typical floor and moment frame layout. The layout of the frames changes slightly on lower floors when the plan extents expand as shown in Figure 8, "Concrete Moment Frames Ground Level". The frame designations 1 through 12.4 and A through P are referred to heavily throughout this report and are visually identifiable on Figures 8 and 9 below.

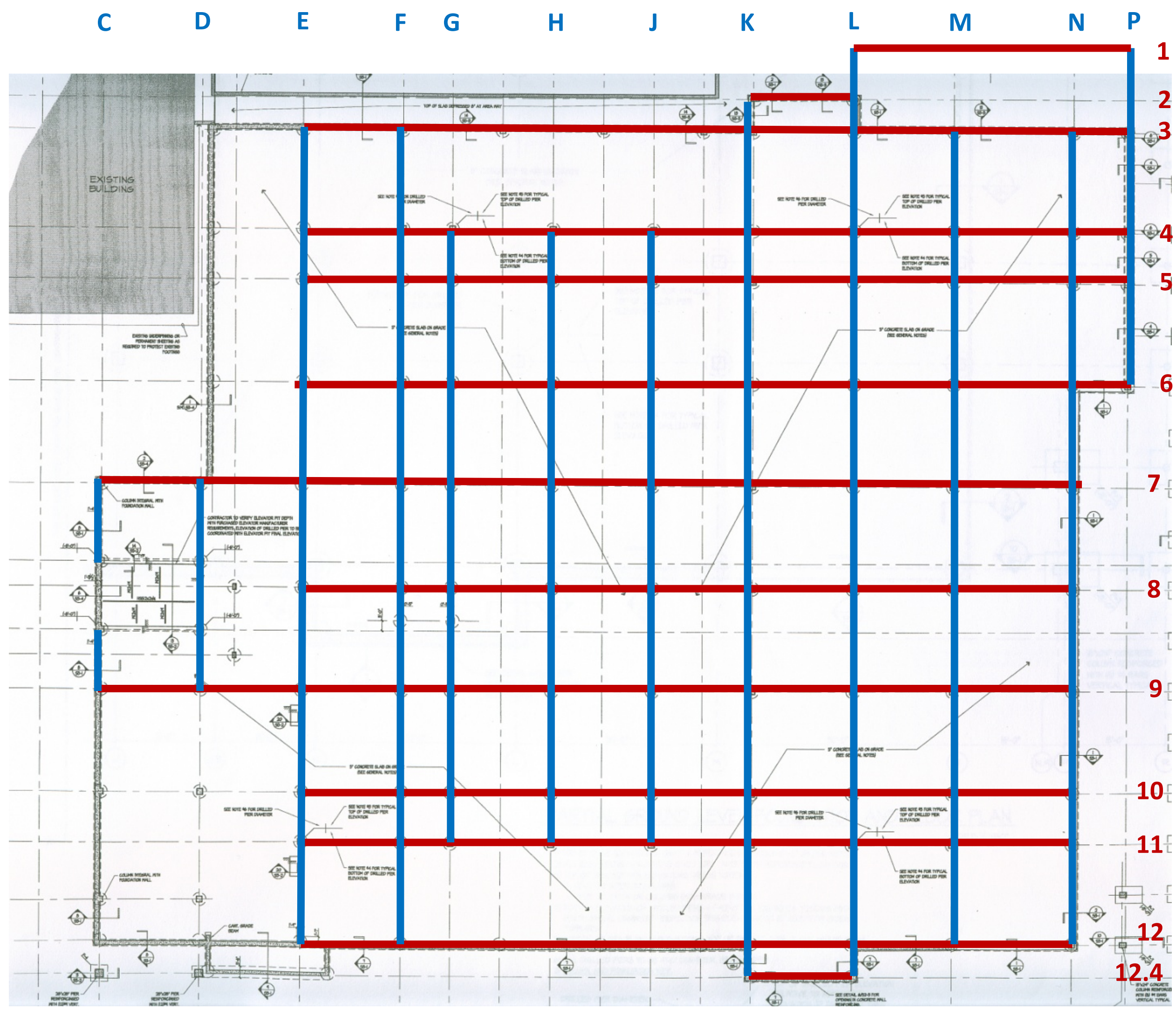


Figure 8: Concrete Moment Frames Ground Level

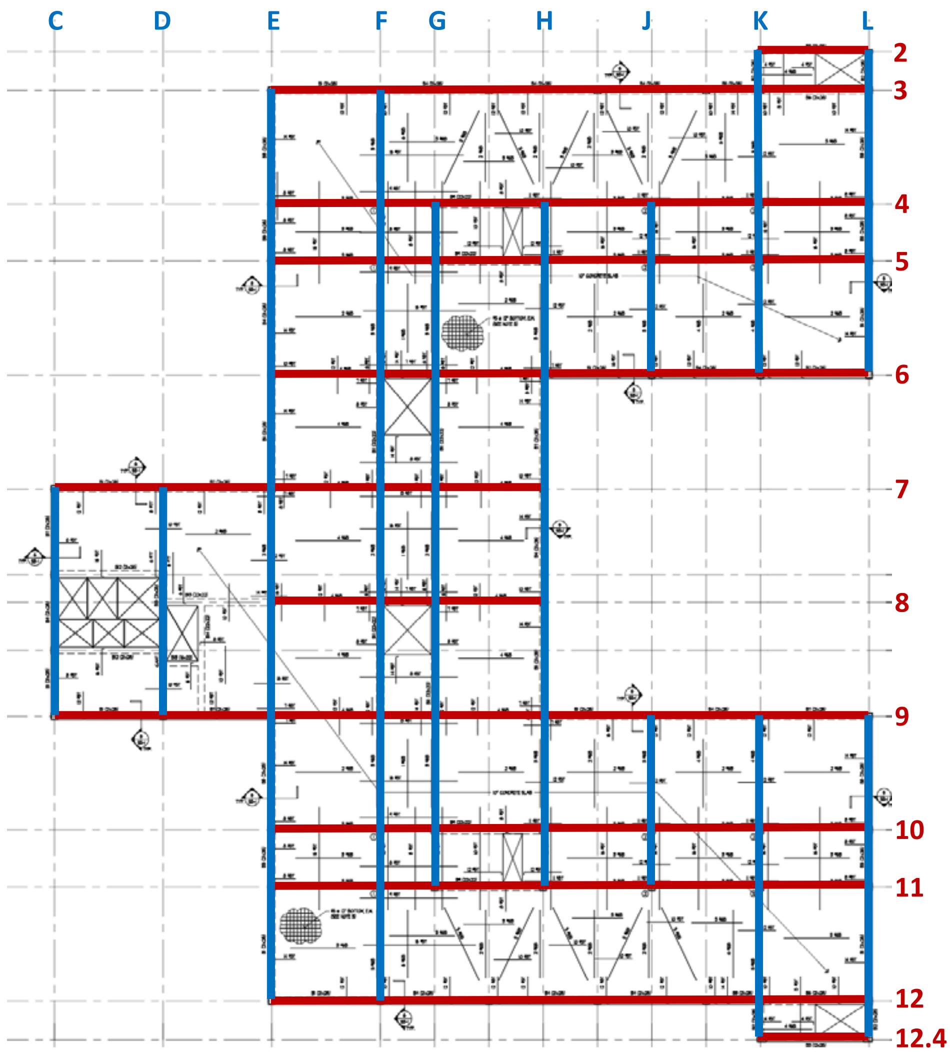


Figure 9: Concrete Moment Frames Level 4

Existing Conditions—Architecture

The facade of the Franklin Square Hospital Patient Tower features predominantly brick veneer precast concrete panels with aluminum curtain wall windows visually offset by exposed concrete bands between floor levels. Sun shades extend out from the buildings face to provide shelter for the larger curtain wall sections as seen in Figure 10, “Exterior Rendering (South Side Looking towards Main Entrance)”. The main entrance leads through a large three story atrium featuring the lobby. Figure 11, “Atrium (Looking towards Reception Desk)” and Figure 12, “Atrium (Looking Towards Main Entrance)” show the grandeur and openness of the lobby and show the engineering expertise used throughout the building.

Figure 10: Exterior Rendering (South Side Looking Towards Main Entrance)

Figure 11: Atrium (Looking towards Reception Desk)

Figure 12: Atrium (Looking towards Main Entrance)

Existing Conditions—Building Enclosure

The building is predominantly covered in a series of pre-cast wall panels with exterior surfaces ranging from exposed concrete to brick veneer to stucco. The precast panels are supported on 6" metal studs with a 3" cavity, 6" thermal insulation, vapor barrier, and 5/8" gypsum wall board. The remaining facade is a 2.5"x6" aluminum curtain wall system.
The roofing is of two main types. EP single-ply membrane roofing that includes a non-traffic-bearing sheet membrane system intended for weather exposure is used as the primary roofing on the patient tower and canopies. The second roofing type is a modified bitumen roofing system.

Existing Conditions—Sustainability Features

Aluminum sun shades extend from the building overhead many of the large aluminum curtain wall sections. The sixth floor of the south elevation incorporates a wall to wall aluminum curtain wall system. To help alleviate solar gain and glare in these spaces the roof is extended an additional five feet with an aluminum composite panel soffit acting like a large sun shade. The atrium also includes shrubbery, small trees, and a water wall making the space more enjoyable and promoting better indoor air quality as seen in Figure 11, “Atrium (Looking towards Reception Desk)” and Figure 12, “Atrium (Looking Towards Main Entrance)”.

Existing Conditions—Building Codes, Zoning and Design Standards

Building Codes

International Building Code 2006 with Baltimore County amendments, NFPA 10,1Life Safety Code 2006, International Fire Code 2006 with Baltimore County amendments, AIA Guidelines for Hospital and Health Care Facilities 2006, ADAAG Americans with Disabilities Act - Public Law 101-336, National Electrical Code 2005, International Mechanical Code 2006, National Standard Plumbing code 2003 with 2004 supplement, International Emergency Code 2006

Zoning

Baltimore Country Zoning Regulations. Franklin Square Hospital is located in zoning ordinance D.R. 5.5 of Baltimore County

General Codes and Standards

- “International Building Code 2006”, International Code Council with Baltimore County Amendments
- “Minimum Design Loads for Buildings and Other Structures, ASCE 7-05”, American Society of Civil Engineers

Concrete

- “Building Code Requirements for Reinforced Concrete, ACI 318”, American Concrete Institute
- “ACI Manual of Concrete Practice – Parts 1 through 5”
- “Manual of Standard Practice”, Concrete Reinforcing Steel Institute
- “PCI Design Handbook – Precast and Prestressed Concrete”, Prestressed Concrete Institute

Structural Steel

- “Manual of Steel Construction – Allowable Stress Design”, Ninth Edition
- “Manual of Steel construction – Load and resistance Factor Design”, Third Edition
- “Manual of Steel Construction, Volume II Connection”, ASD 9th Edition/LRFD 3rd Edition
- “Detailing for Steel construction”, American Institute of Steel Construction
- “Structural Welding Code ANSI/AWS D1.1, American Welding Society

Steel Deck

- “Design Manual Floor Decks and Roof Decks”, Steel Deck Institute

Existing Conditions—Material Specifications

Concrete

Application	f'c @ 28 days	Weight (PCF)
Slabs-On-Grade (Interior)	3000	145
Slabs-On-Grade (Exterior)	3500	145
Reinforced Slabs	5000	145
Reinforced Beams	5000	145
Fill on Metal Deck	4000	110
Columns (Ground to 3rd Floor)	7000	145
Columns (3rd Floor to Roof)	5000	145
Walls	4000	145
Grade Beams	3000	145
Footings	3000	145
Caissons	3000	145
Topping	3000	145

Structural Steel

Application	
Deformed Reinforcing Bars	ASTM A615, Grade 60
Rolled Shapes	ASTM A992, Grade 50
Channels, Angles and Plates	ASTM A36
Structural Pipe	ASTM A53, Grade B, F_y = 35 ksi
Round HSS Shapes	ASTM A500, Grade B, F_y = 42 ksi
Structural Tubing (Square and Rectangular HSS)	ASTM A500, Grade B, F_y = 46 ksi
High Strength Bolts	ASTM A325-N typical
Anchor Rods	ASTM F1554 Grade 36
Smooth & Threaded Rod	ASTM A36
Headed Shear Studs	ASTM A108
Welding Electrodes	AWS A5.1 OR A5.5, E70XX
Galvanized Metal Deck	ASTM A653
Painted Phosphated Metal Floor Deck	ASTM A611

Proposal

The current structural system consists a of two-way mildly reinforced concrete flat plate floor system and reinforced concrete moment frames. The proposed thesis focuses on lateral system design under stringent and harsh requirements by relocating the building to San Francisco, California which requires changes to both the gravity and lateral systems. A redesign of the entire floor system with a post-tensioned flat plate will be utilized to lower building self weight and a concrete moment frame lateral system will be analyzed in comparison with a concrete shear wall system in resisting lateral loads.

San Francisco, California was chosen as the new building site for its seismic history. For the purpose of this thesis, an intense lateral redesign was chosen which requires intense lateral loading. Located close to the San Andreas Fault and the Hayward Fault, San Francisco, Ca was a logical choice. The Hayward Fault is considered by some to be the most dangerous fault in America at this time with a 63% chance of a magnitude 6.7 or greater earthquake within the next 30 years. The past five large earthquakes of this fault have occurred on average about 140 years apart and the last occurred 142 years ago, October 21, 1868.

Additional topics covered by this proposal focus on other architectural engineering disciplines such as construction management and architecture. One of these studies will focus on a cost and scheduling comparison to determine adjustments to the construction schedule necessitated by the change from a mildly reinforced two way flat plate to a post-tensioned two way flat plate. The associated costs with a changed schedule will also be investigated. The second study involves an architectural redesign of support spaces, nurse’s stations and hallways to function around the addition of structural shear walls.

The MAE requirements for the project will be fulfilled through the construction and implementation of an improved and comprehensive ETABS building model. Methods taught in AE 597A: Computer Modeling including modified section properties, rigid end offsets, insertion points, panel zones and rigid and semi-rigid diaphragms will be included in the model. This model will be extremely useful for quickly and accurately comparing proposed lateral system designs and implementation.

Structural Investigation Studies—Lateral System Redesign

Introduction

The goal of this project was to investigate higher seismic loading than was required by the Franklin Square Hospital Center's location in Baltimore MD. To successfully accomplish this, the building was moved, to 845 Jackson Street, San Francisco, CA. This location was chosen for a number of reasons including the proximity to two of the more dangerous faults in the country at this time, the Hayward Fault and the San Andreas Fault and the challenging seismic requirements for buildings in this location. Figure 13, "Building Relocation site", shows the geographic location of the site and its proximity to the aforementioned faults.

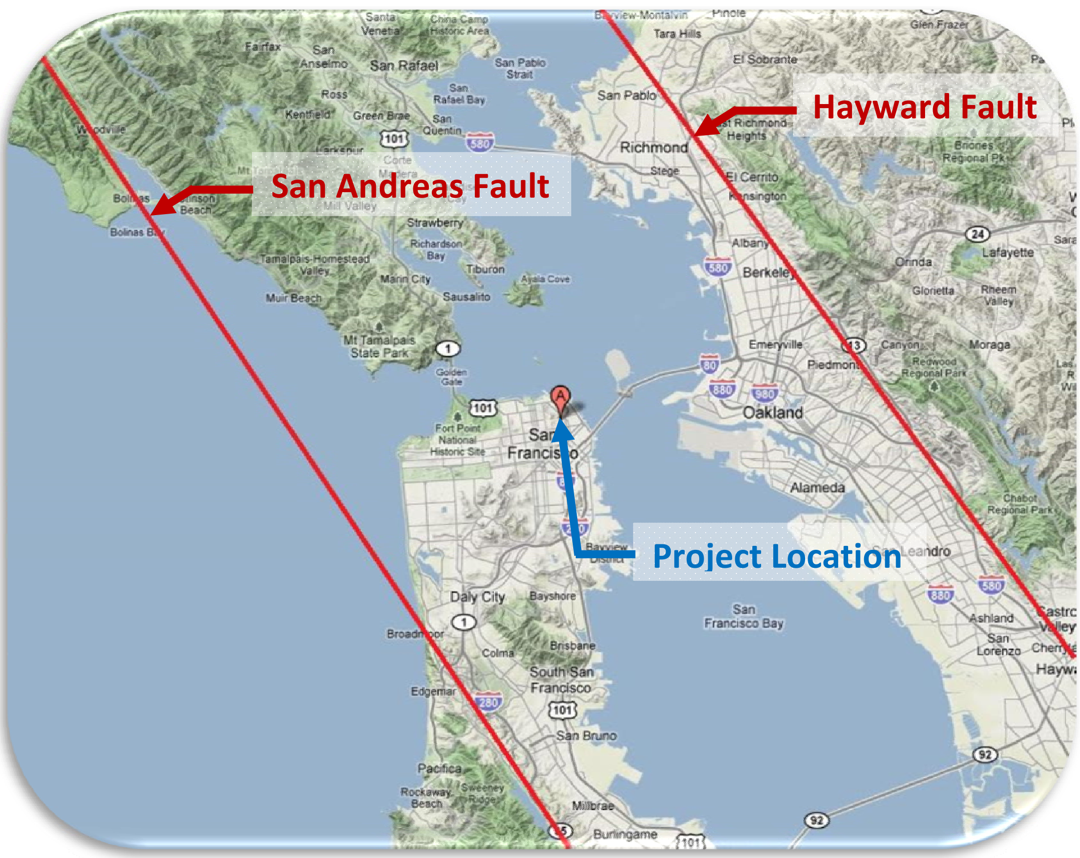


Figure 13: Proposed Building Relocation Site

For a valid study and comparison to be made, both a concrete moment frame system and a concrete shear wall system were analyzed and compared for feasibility and strength requirements. The two new designs included a change in the floor system to 8" PT slabs from the original 10" mildly reinforced slabs. This change in floor system is discussed later in this report in the Floor System Redesign Section.

ETABS 3D Building Model

It was determined early in the analysis of the Franklin Square Hospital Center Patient Tower that an advanced 3D model would be necessary to accurately determine member forces and stresses under lateral loading. The lateral system of the structure was modeled given the assumption below, and eight seismic load combinations were input in ETABS for detailed and accurate analysis. See Figure 14, Figure 15, and Figure 16 for the 3D rendering of the Original Ordinary Reinforced Concrete Moment Frame System for Baltimore MD, the Special Reinforced Concrete Moment Frame System for San Francisco CA, and the Special Reinforced Concrete Shear Wall System for San Francisco CA respectively.

ETABS Modeling Assumption:

- All Columns and Walls are Fixed at their bases
- Members not participating in Lateral Resistance were not modeled
- ACI 318-08 Modified Moments of Inertia for Columns, Beams, and Slabs
- All Column and Beam Connections were modeled with Rigid End Offsets equal to 1.0
- Beam Insertion Points were modeled correctly with Modified Stiffness from offsets
- Panel Zone's were Explicitly Modeled
- Rigid Diaphragms were created on Each Level
- Diaphragms were given Mass calculated from Story Weight

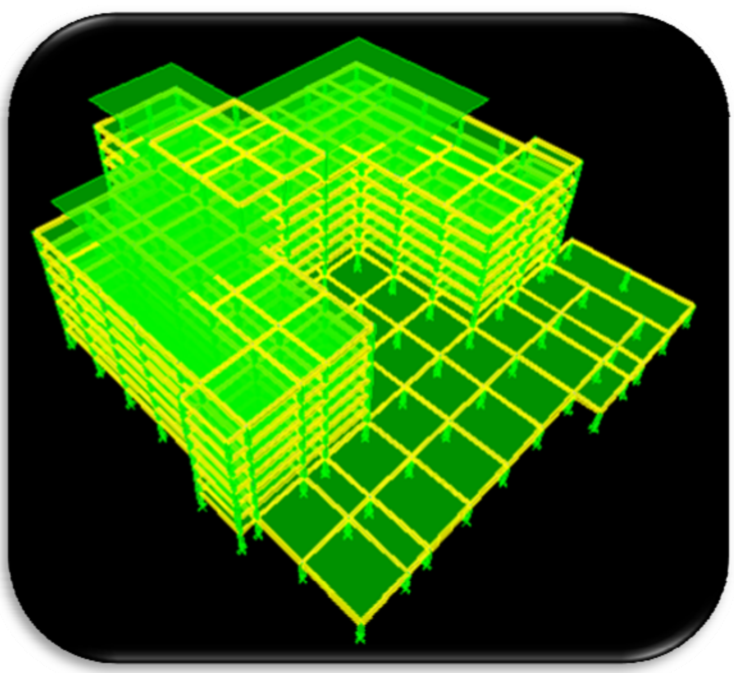


Figure 14: Original Concrete Moment Frame ETABS 3D Model

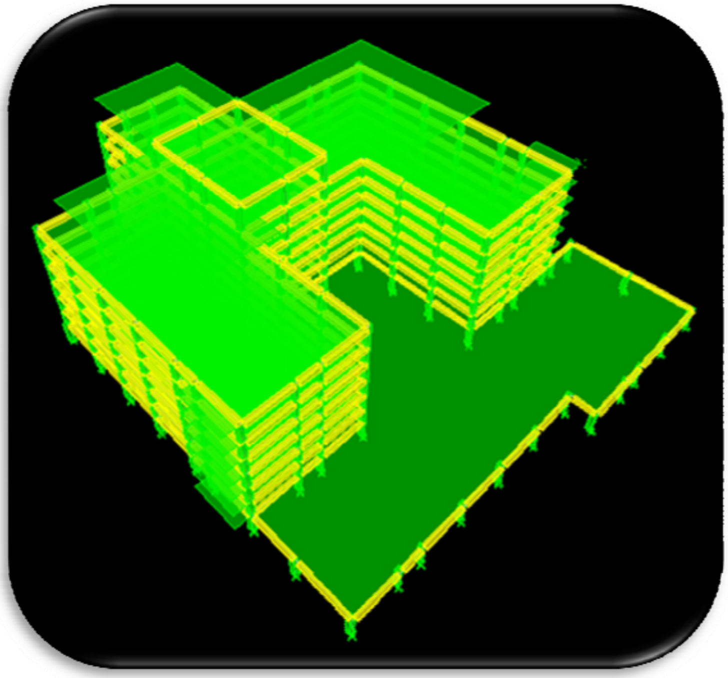


Figure 15: Modified Concrete Moment Frame ETABS 3D Model

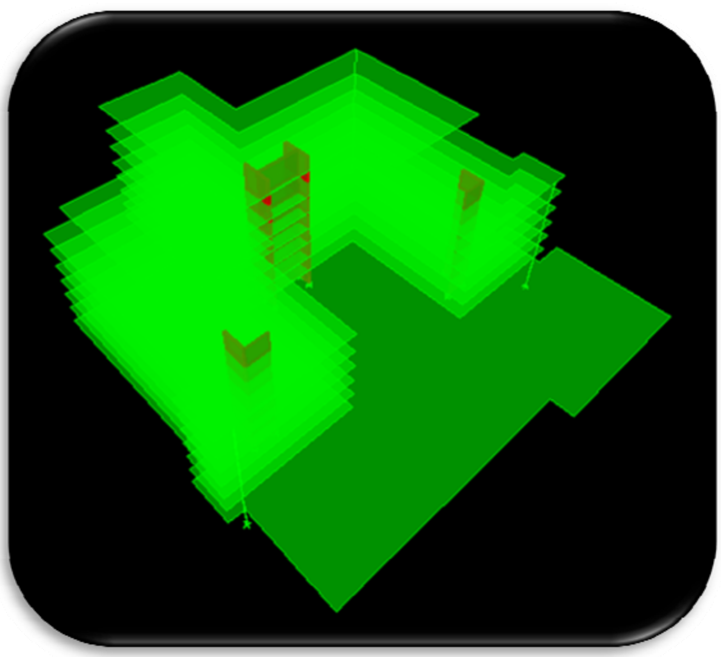


Figure 16: Concrete Shear Wall ETABS 3D Model

The design of the Special Reinforced Concrete Moment Frame System for San Francisco is noticeably different from the Ordinary Reinforced Concrete Moment Frame System for Baltimore. The differences stem from the change in floor system between the two. The original Baltimore Design uses 10" mildly reinforced slabs that act in flexure under lateral loading. With the change to an 8" post-tensioned slab, there was not enough reserve strength available in the slabs for them to aid in lateral force resistance therefore only perimeter beams and additional interior beams function as part of the lateral force resisting system.

With the entire lateral system modeled and the diaphragm masses assigned, a modal analysis was conducted resulting in the fundamental periods of the building for each design. In order to use the Equivalent Lateral Force Procedure, a number of requirements had to be met. These requirements included, but were not limited to, the absence of torsional and extreme torsional irregularities and the fundamental period of the structure to be under 3.5Ts or 1.446 seconds given the occupancy category of IV. These two requirements noted above proved to be the most difficult of all the requirements to meet and much time was spent reiterating the designs to meet the requirements.

For the Special Reinforced Concrete Moment Frame system, the column sizes needed to be at least 34"x34" while the beam sizes needed to be at least 34"x36" to meet the requirements. The columns on the lower four floors utilize 7000 psi concrete while the columns on the upper four floors utilize 5000 psi concrete. The moment frame beams utilize the same concrete strength as the post-tensioned slabs of 5000 psi. This design resulted in the 1st mode period of vibration of 1.4206 seconds which met the requirement of 1.446 seconds by the code for use of the Equivalent Later Force Procedure with an occupancy category of IV in SDC Category D. Additionally, the torsional amplification factors in all directions were 1.0 and no torsional irregularities were found.

The design of the Special Reinforced Concrete Shear Wall system proved to be more difficult in regards to meeting the requirement of torsional irregularities. It was decided early in the design process to move the elevator core of the building to coincide with the center of mass of the building to help reduce torsional issues. This helped greatly but the building was still quite flexible without supplemental walls in other locations. Two more shear wall groups were added on the 'wings' of the building to reduce torsional irregularities and stiffen the building. Through iteration, the design resulted in an H-shaped shear wall core with 22" walls, and two L-shaped walls placed in the 'wings' of the building. Both L-shaped shear walls are 22" thick in the East-West direction and only 12"thick in the North-South direction. Also, all walls are constructed of 7000 psi concrete. With this design, the 1st mode period of vibration met the requirement of 1.446 seconds with a period of 1.3101 seconds, the torsional amplification factors were 1.0 and no torsional irregularities were found in any direction.

Additionally, the center of masses and rigidities were calculated in ETABS and tabulated in Table 1, 2 and 3, "Centers of Mass and Rigidity" and displayed visually below for a typical level in Figure 17, 18, and 19, "Center of Mass and Rigidity Level 5"

Table 1: Original Center of Mass and Rigidity					(from c. of rigidity)	
Level	**X CM**	**Y CM**	**X CR**	**Y CR**	**ΔX (ft)**	**ΔY (ft)**
7	1527.0	1613.1	1564.9	1395.3	-3.2	18.2
6	1845.7	1543.9	1678.2	1549.2	14.0	-0.4
5	1845.7	1543.9	1643.1	1554.8	16.9	-0.9
4	1845.7	1543.9	1589.0	1561.5	21.4	-1.5
3	1845.7	1543.9	1497.9	1573.7	29.0	-2.5
2	1735.1	1556.7	1376.5	1600.1	29.9	-3.6
1	1735.1	1556.7	1285.0	1648.7	37.5	-7.7
Ground	2441.0	1633.5	1659.0	1625.9	65.2	0.6

Table 2: Moment Frame Center of Mass and Regidity					(from c. of rigidity)	
Level	**X CM**	**Y CM**	**X CR**	**Y CR**	**ΔX (ft)**	**ΔY (ft)**
7	1527.0	1613.1	1573.4	1406.9	-3.9	17.2
6	1845.7	1543.9	1715.9	1525.2	10.8	1.6
5	1845.7	1543.9	1735.0	1528.8	9.2	1.3
4	1845.7	1543.9	1753.1	1532.1	7.7	1.0
3	1845.7	1543.9	1778.3	1536.7	5.6	0.6
2	1845.7	1543.9	1819.9	1544.5	2.1	0.0
1	1845.7	1543.9	1902.7	1558.9	-4.8	-1.3
Ground	2441.0	1633.5	2053.3	1579.2	32.3	4.5

Table 3: Shear Wall Center of Mass and Regidity					(from c. of rigidity)	
Level	**X CM**	**Y CM**	**X CR**	**Y CR**	**ΔX (ft)**	**ΔY (ft)**
7	1527.0	1613.1	1821.3	1562.0	-24.5	4.3
6	1845.7	1543.9	1924.1	1562.0	-6.5	-1.5
5	1845.7	1543.9	1933.3	1562.0	-7.3	-1.5
4	1845.7	1543.9	1947.6	1562.0	-8.5	-1.5
3	1845.7	1543.9	1970.7	1562.0	-10.4	-1.5
2	1845.7	1543.9	2008.3	1562.0	-13.6	-1.5
1	1845.7	1543.9	2067.9	1562.0	-18.5	-1.5
Ground	2441.0	1633.5	2148.6	1562.0	24.4	6.0

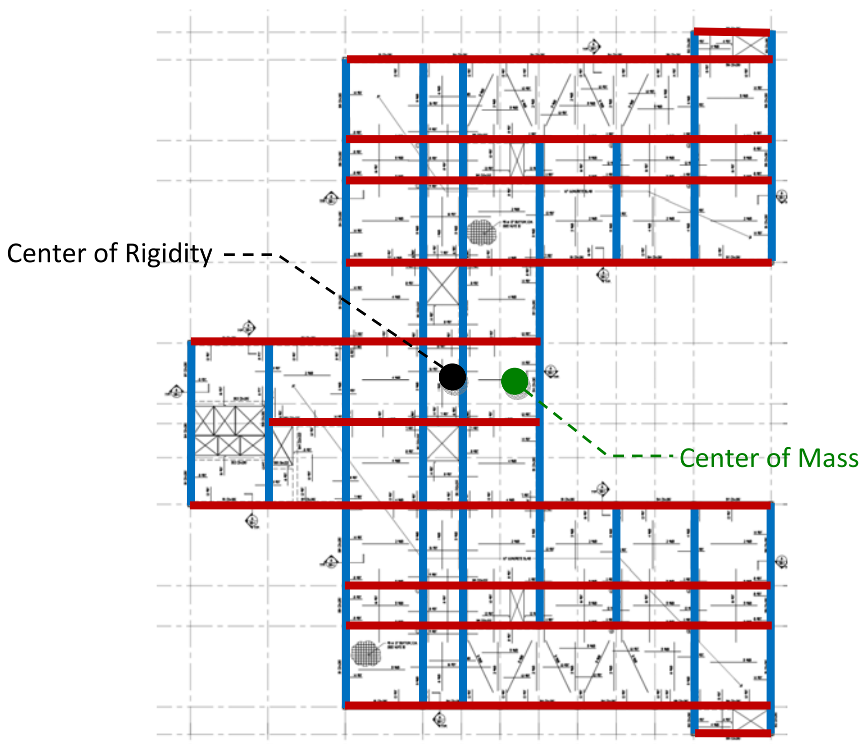


Figure 17: Original Concrete Moment Frame Design; Center of Mass and Rigidity Level 5

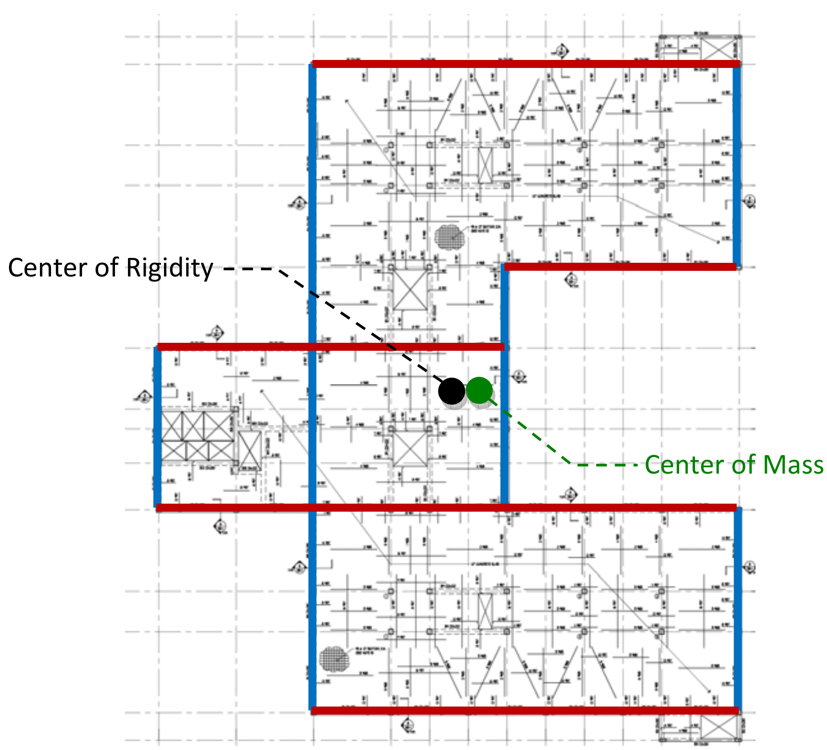


Figure 18: Modified Concrete Moment Frame Design; Center of Mass and Rigidity Level 5

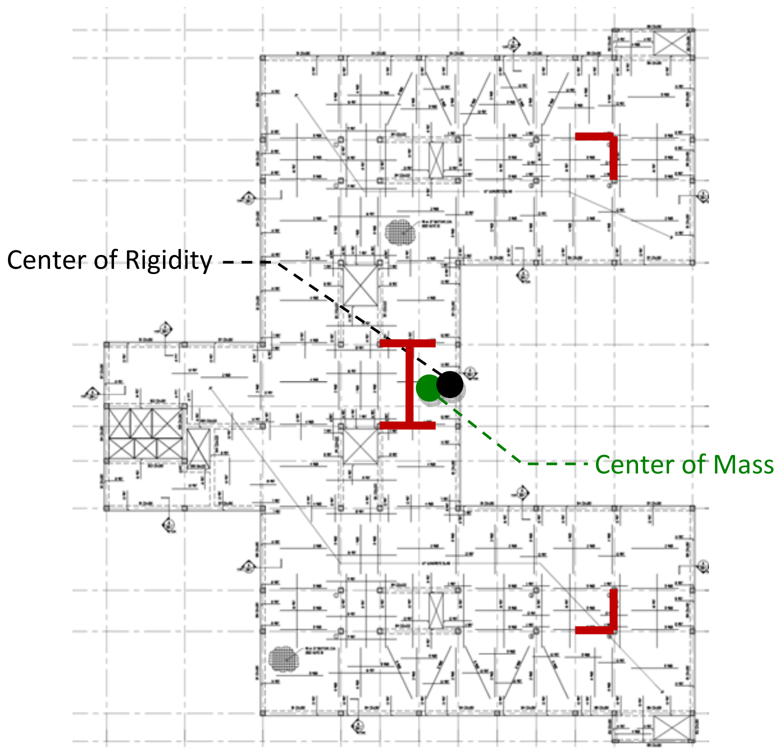


Figure 19: Concrete Shear Wall Desing; Center of Mass and Rigidity Level 5

Seismic Loads

Loads were determined based on Chapters 11 and 12 of ASCE 7-05 using the Equivalent Lateral Force Method along with class notes and example problems from an MAE course, AE597A Computer Modeling, taught by Dr. Andres Lapage. The spectral response coefficients were determined from the USGS Earthquake Hazard Program inputted with the exact latitude (37.7955˚N) and longitude (-122.4088˚S) of the site. Figure 20, "USGS Earthquake Hazard Program Screenshot" shows a screen shot of the program with the inputted data and the output of Site Class, Ss, and S1.

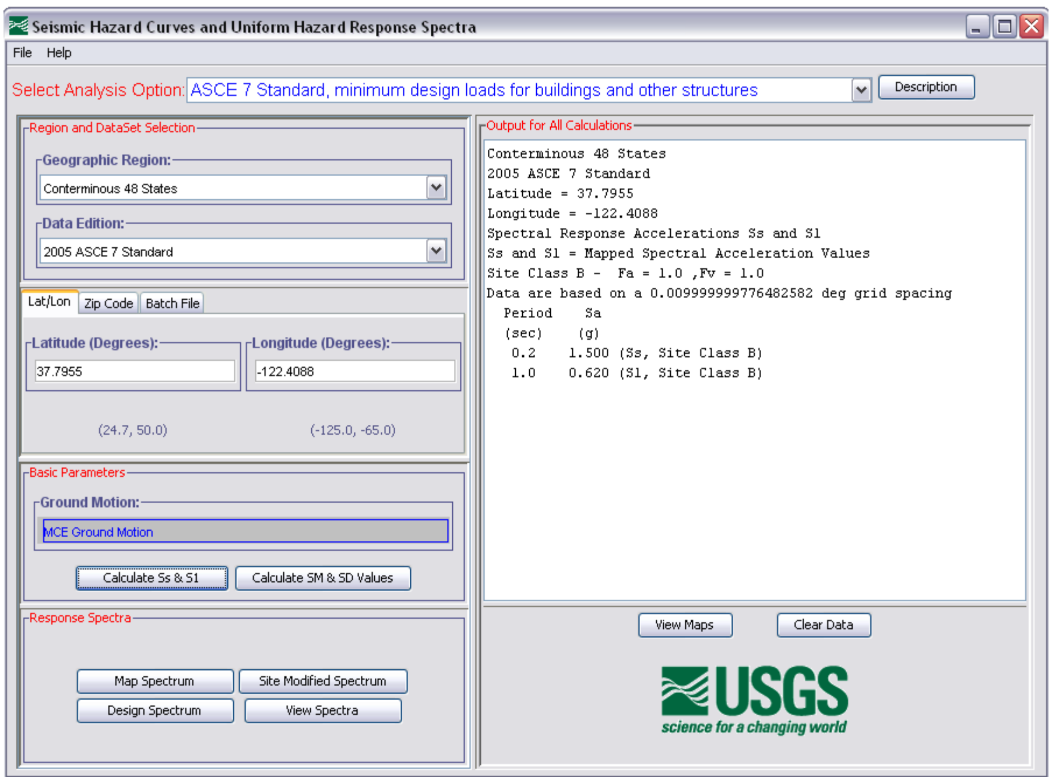


Figure 20: USGS Earthquake Hazard Program Screenshot

Table 4 details the basic seismic parameters of the original structure which is an Ordinary Reinforced Concrete Moment Frame system as constructed in Baltimore MD, Table 5 details the basic seismic parameters of a Special Reinforced Concrete Moment Frame system constructed in San Francisco CA, and Table 6 details the basic seismic parameters of a Special Reinforced Concrete Shear Wall system constructed in San Francisco CA.

Table 4: Original Seismic Parameters	
Spectral Response Coeff. S_s	0.176
Spectral Response Coeff. S_1	0.051
Soil Site Class	C
Seismic Design Category	A
Response Modification Factor	3
Importance Factor	1.5
Seismic Response Coeff. Cs	0.016
Total Building Weight	56,820 k
Design Base Shear	909 k

Table 5: Moment Frame Seismic Parameters	
Spectral Response Coeff. S_s	1.500
Spectral Response Coeff. S_1	0.620
Soil Site Class	B
Seismic Design Category	D
Response Modification Factor	8
Importance Factor	1.5
Seismic Response Coeff. Cs	0.0581
Total Building Weight	58,279 k
Design Base Shear	3386

Table 6: Shear Wall Seismic Parameters	
Spectral Response Coeff. S_s	1.500
Spectral Response Coeff. S_1	0.620
Soil Site Class	B
Seismic Design Category	D
Response Modification Factor	6
Importance Factor	1.5
Seismic Response Coeff. Cs	0.0775
Total Building Weight	51,209 k
Design Base Shear	3969

It should be noted that the total building weight for each system is fairly different. The original design for Baltimore consisted of comparatively small moment frame columns and beams and 10" mildly reinforced flat plate concrete slabs. As part two of the structural investigation study for this project, the building redesign for San Francisco also included the redesign of the floor system to an 8" post-tensioned slab. This thinner floor system decreased the total building weight for both lateral systems designed for San Francisco compared to a 10" mildly reinforced floor system. However, the total building weight for the Moment Frame system in San Francisco at 58,279 kips actually weighs more than the original design in Baltimore at 56,820 kips due to the need for much larger columns and beams in the moment frames. The weight gain from these larger elements actually surpasses the weight savings gained from the change in floor system. The lightest of all three systems is the Shear Wall System with an 8" Post-Tensioned Floor System at only 51,209 kips. The weight savings and design of the 8" Post-Tensioned Floor system was be discussed in detail later in this report.

As Detailed in the Seismic Parameter Tables, the Seismic Response Coefficient is interesting to compare between systems. The original structure as built in Baltimore, MD had a base shear of only 1.6% of the total building weight. The two systems designed for San Francisco CA had base shear values of 5.81% and 7.75% of the total building weight. While these numbers don't seem to be all that much larger in magnitude, the differences in the resulting loading on the structure and the strength requirements of the elements are huge. Tables 7, 8 and 9 detail the seismic load at each level and the overturning moment at the base for each system. Figures 21, 22, and 23 show the seismic load diagrams on the building's elevation. Seismic Hand Calculations are available for viewing in Appendix B.

Table 7: Original Seismic Load for Baltimore, MD

Level	Height, h_x (ft)	Story Weight, w_x (kips)	$w_x h_x^k$	$w_x h_x^k / \sum w_i h_i^k$	Lateral Force (kips)	Story Shear (kips)	Overturning Moment (ft-k)
Roof	105	2221	2280488	0.110	100	100	10516
Penthouse	87	6509	5051386	0.244	222	322	19300
Level 6	74	7094	4325728	0.209	190	512	14058
Level 5	62	7022	3289612	0.159	144	656	8957
Level 4	50	7233	2459303	0.119	108	764	5400
Level 3	38	7758	1752308	0.085	77	841	2924
Level 2	26	7592	974212	0.047	43	884	1112
Level 1	14	11123	567497	0.027	25	909	349
Ground	0	267	0	0	0	909	0
Total					**909 k**		**62,618 ftk**

Table 8: Moment Frame Seismic Loads for San Francisco, CA							
Level	**Height, h_x (ft)**	**Story Weight, w_x (kips)**	**$w_xh_x^k$**	**$w_xh_x^k$ / $\sum w_ih_i^k$**	**Lateral Force (kips)**	**Story Shear (kips)**	**Overturning Moment (ft-k)**
Roof	105	2218	1983404	0.105	357	357	37441
Penthouse	87	6520	4431074	0.235	797	1153	69306
Level 6	74	7579	4066837	0.216	731	1884	54104
Level 5	62	7451	3087600	0.164	555	2439	34416
Level 4	50	7662	2319210	0.123	417	2856	20847
Level 3	38	7812	1583857	0.084	285	3141	10820
Level 2	26	7558	880437	0.047	158	3299	4115
Level 1	14	10215	481875	0.026	87	3386	1213
Ground	0	1265	0	0	0	3386	0
Total					**3,386 k**		**232,262 ftk**

Table 9: Shear Wall Seismic Loads for San Francisco, CA							
Level	**Height, h_x (ft)**	**Story Weight, w_x (kips)**	**$w_xh_x^k$**	**$w_xh_x^k$ / $\sum w_ih_i^k$**	**Lateral Force (kips)**	**Story Shear (kips)**	**Overturning Moment (ft-k)**
Roof	105	2320	645012	0.109	432	432	45343
Penthouse	87	5963	1320559	0.223	884	1316	76919
Level 6	74	6477	1179388	0.199	790	2106	58431
Level 5	62	6353	933948	0.158	625	2731	38768
Level 4	50	6564	743986	0.126	498	3229	24905
Level 3	38	6714	546086	0.092	366	3595	13893
Level 2	26	6495	333844	0.056	224	3818	5811
Level 1	14	9250	224929	0.038	151	3969	2108
Ground	0	1072	0	0	0	3969	0
Total					**3,969 k**		**266,179 ftk**

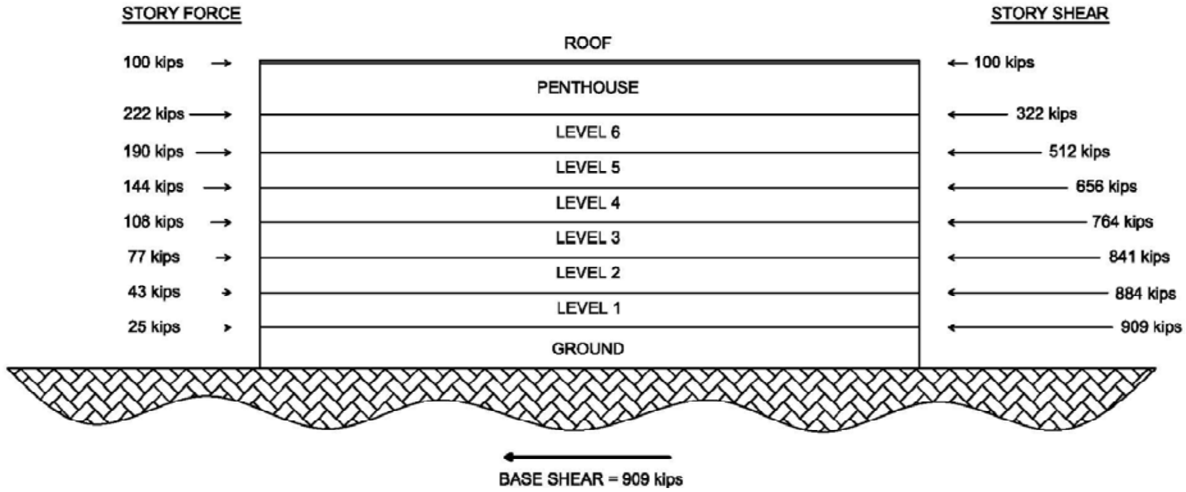


Figure 21: Seismic Load Diagram Original Moment Frame Design, Baltimore, MD

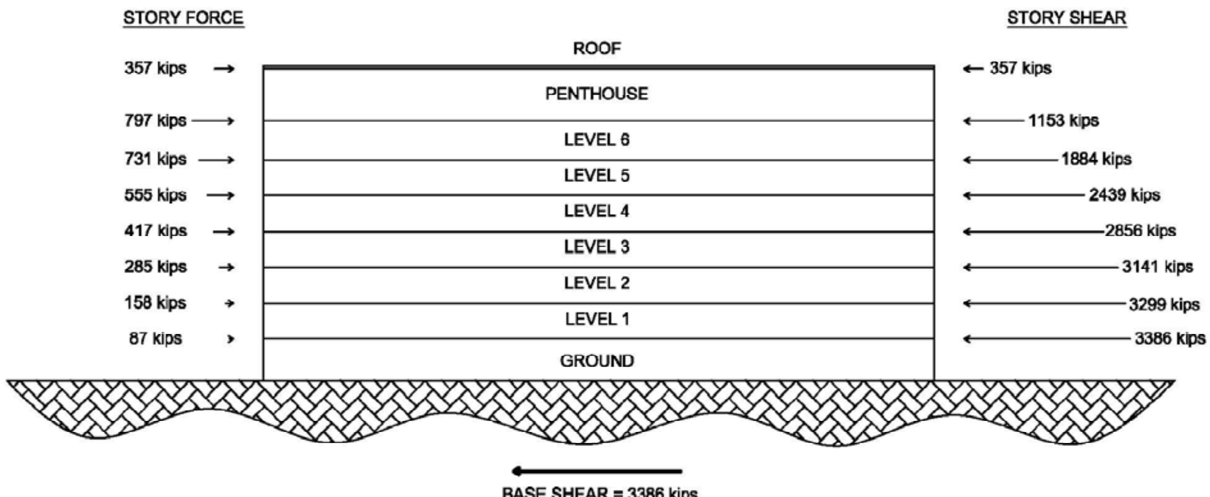


Figure 22: Seismic Load Diagram Moment Frame Design, San Francisco, CA

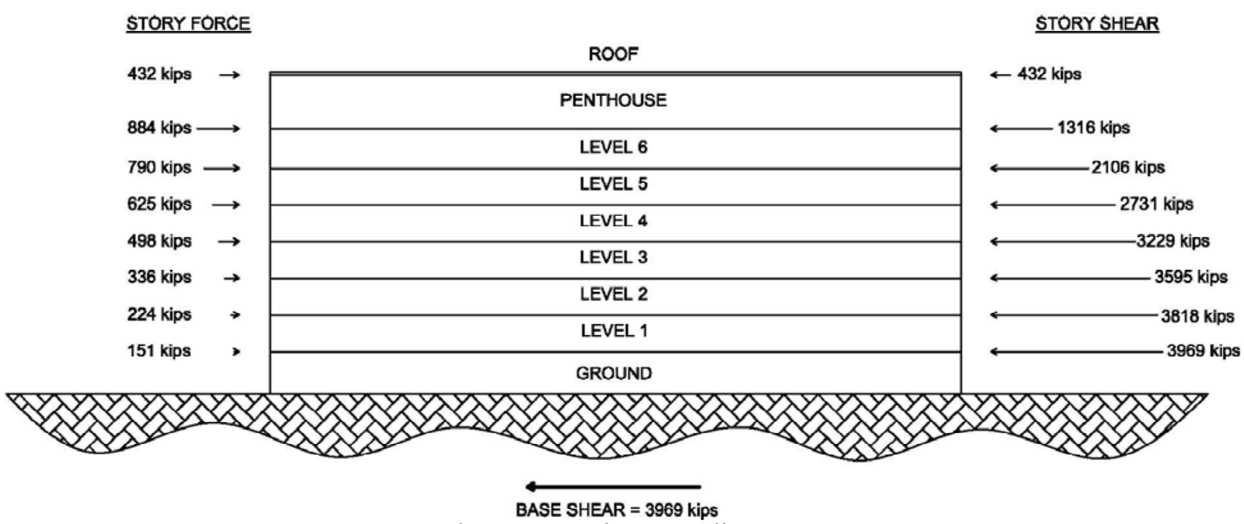


Figure 23: Seismic Load Diagram Shear Wall Design, San Francisco, CA

Seismic Load Cases and Combinations

For the most part, the seismic load cases are simple and straightforward. There are four main cases, one in each orthogonal direction plus two more with 100% in one direction and 30% in the other. In the case of seismic loading, the code mandates an applied accidental eccentricity of +/- 5% which essentially doubles the number of cases to eight. Tables 10 through 13 tabulate the applied diaphragm forces due to seismic response of the structure for the Special Reinforced Concrete Shear Wall Lateral System to show as an example. The load cases for both the original Ordinary Reinforced Concrete Moment Frame System and the Special Reinforced Concrete Moment Frame System are not show to save space and can be easily calculated from the Seismic Loading Tables and Graphs from the previous section.

Table 10: Shear Wall Seismic N-S (+/- 0.05)

Story	Fx	Fy
8	0	432
7	0	884
6	0	790
5	0	625
4	0	498
3	0	366
2	0	224
1	0	151

Table 11: Shear Wall Seismic E-W (+/- 0.05)

Story	Fx	Fy
8	432	0
7	884	0
6	790	0
5	625	0
4	498	0
3	366	0
2	224	0
1	151	0

Table 12: Sear Wall Seismic N-S + 0.3E-W (+/- 0.05)

Story	Fx	Fy
8	130	432
7	265	884
6	237	790
5	188	625
4	149	498
3	110	366
2	67	224
1	45	151

Table 13: Shear Wall Seismic E-W +0.3 E-W (+/- 0.05)

Story	Fx	Fy
8	432	130
7	884	265
6	790	237
5	625	188
4	498	149
3	366	110
2	224	67
1	151	45

The load combinations determined to apply to the structure came from ASCE 7-05. The combinations, listed below, were not all analyzed at this time but will need to be checked with further investigation of the structure.

ASCE 7-05 Load Combinations

1.2D + 1.6L + 0.5Lr
1.2D + 1.6Lr + (L or 0.8 W)
1.2D + 1.6W + L + 0.5Lr
1.2D + 1.0E + L + 0.2S
0.9 D + 1.6 W + 1.6H
0.9D + 1.0E + 1.6H

As lateral analysis is the main focus of this study, dead and live loading were not considered in the ETABS building model at this time but were calculated separately for those elements where it proved critical. The following load combinations were input into the ETABS building model for assessment.

1.0(SEISMIC 1)	*Seismic in the N-S direction (+0.05 Ecc) (Table 10 or Equivalent)*
1.0(SEISMIC 2)	*Seismic in the N-S direction (-0.05 Ecc) (Table 10 or Equivalent)*
1.0(SEISMIC 3)	*Seismic in the E-W direction (+0.05 Ecc) (Table 11 or Equivalent)*
1.0(SEISMIC 4)	*Seismic in the E-W direction (-0.05 Ecc) (Table 11 or Equivalent)*
1.0(SEISMIC 5)	*Seismic in the N-S direction + 0.3 E-W direction (+0.05 Ecc) (Table 12 or Equiv)*
1.0(SEISMIC 6)	*Seismic in the N-S direction + 0.3 E-W direction (-0.05 Ecc) (Table 12 or Equiv)*
1.0(SEISMIC 7)	*Seismic in the E-W direction + 0.3 N-S direction (+0.05 Ecc) (Table 13 or Equiv)*
1.0(SEISMIC 8)	*Seismic in the E-W direction + 0.3 N-S direction (-0.05 Ecc)(Table 13 or Equiv)*

After investigation, the lateral system of the structure is controlled by the load combinations for seismic, **1.2D + 1.0E + L + 0.2S** and **0.9D+1.0E+1.6H**. Wind forces were not evaluated for the design of the lateral systems in this report, as the original design in Baltimore MD was controlled by seismic loading over wind due to the large weight of the building. The seismic loads in San Francisco are much higher than those in Baltimore while the wind loads are less in San Francisco than in Baltimore. Therefore, is can be easily determined through inspection that wind loads are not going to control the design of the lateral system in San Francisco, CA.

Lateral System Strength Design

From the member force output in ETABS, the critical loading for each member was determined. Below, Table 14, "Special Reinforced Concrete Moment Frame Critical Design Forces" and Table 15, "Special Reinforced Concrete Shear Wall Critical Design Forces" display the critical design forces for each lateral system.

Table 14: Special Reinforced Concrete Moment Frame Critical Design Forces

Max Force By Section	Maximum Shear *(kips)*	Maximum Moment *(ft-kips)*
Columns	195	977
Beams	190	1460

Table 15: Special Reinforced Concrete Shear Wall Critical Design Forces

Max Shear By Section	Y-Direction Core *(kips)*	Y-Direction South *(kips)*	Y-Direction North *(kips)*	X-Direction Core South *(kips)*	X-Direction Core North *(kips)*	X-Direction South *(kips)*	X-Direction North *(kips)*
Level 4-7	2788	454	447	739	695	749	768
Level G-3	3717	536	522	1193	1191	1031	1009
Max Moment By Section	**Y-Direction Core *(ft-kips)***	**Y-Direction South *(ft-kips)***	**Y-Direction North *(ft-kips)***	**X-Direction Core South *(ft-kips)***	**X-Direction Core North *(ft-kips)***	**X-Direction South *(ft-kips)***	**X-Direction North *(ft-kips)***
Level 4-7	19168	1198	1194	22032	22084	12168	15424
Level G-3	56791	7002	7002	73859	74074	35711	35987

The design of the Special Reinforced Concrete Moment Frames was extremely straightforward and the reinforcing layout is easily constructible. The controlling load case for the columns was 1.2D + 1.0E + L. This resulted in a factored axial load of 830 kips and a factored moment of 977 ft-kips. Based on the required section size of 34"x34" from the Equivalent Later Force Procedure, the column only needed minimum longitudinal reinforcement for strength requirements. Twelve (12) #9 bars were implemented spaced equally around the perimeter of the column resulting in a longitudinal reinforcement ration 1.04% meeting the 1% minimum. Additionally, #5 spiral ties were implemented following code requirements. Figure 24, "Critical

Column Design Interaction Diagram" visually shows the imposed and allowed loads on the column while Figure 25, "Column Reinforcing" shows the reinforcing layout including vertical reinforcing and spiral confinement.

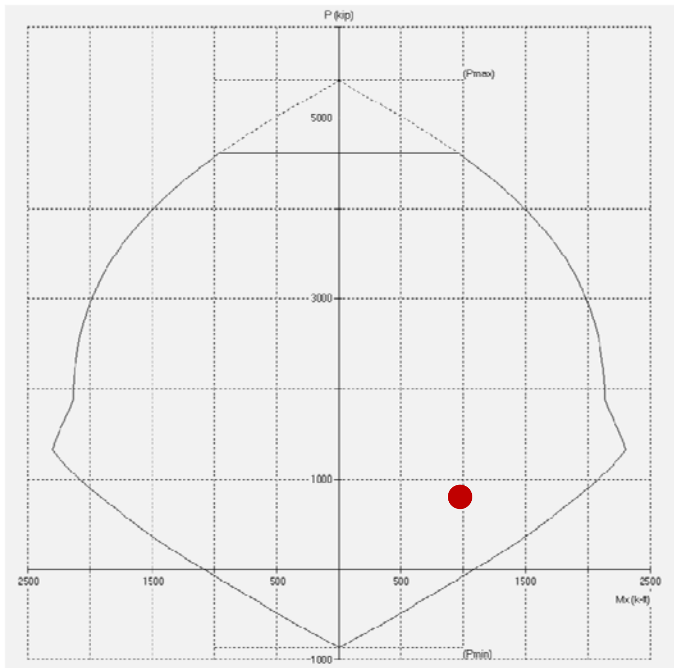


Figure 24: Critical Column Design Interaction Diagram

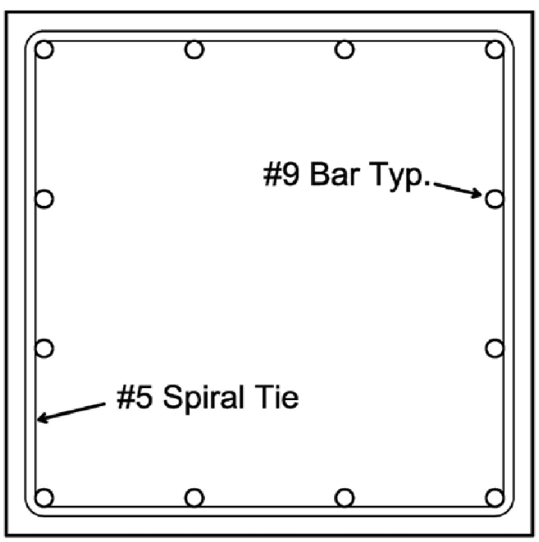


Figure 25: Column Reinforcing

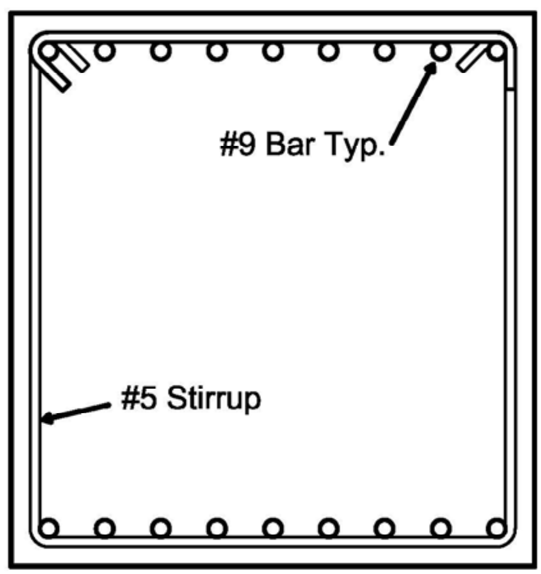


Figure 26: Beam Reinforcing

The design of the beams for the Reinforced Concrete Moment Frames was also quite easy and the reinforcing layout is easily constructible. The 34"x36" Beams required (9) #9 Bars top and bottom along with #5 stirrups at 12”. This reinforcing adequately resists the factored loads of 1460 ft-kips and 190 kips. For hand calculations of the Moment Frame Beams see Appendix B.

For the Shear Wall System, the sizing of the walls was first based on the requirements for use of the Equivalent Lateral Force Procedure. These wall sizes proved adequate to carry the factored design loads. Design of the reinforcing for strength was extremely straightforward and done by hand with the aid of a spreadsheet. Table 16, “Special Reinforced Shear Wall Provided Reinforcing” illustrates the provided horizontal, vertical, and flexure reinforcing along with the boundary element size provided as necessary.

Table 16: Special Reinforced Concrete Shear Wall Provided Reinfocing

Level 4 to Level 7	**N-S Direction Core**	**N-S Direction South**	**N-S Direction North**	**E-W Direction Core South**	**E-W Direction Core North**	**E-W Direction South**	**E-W Direction North**
Thickness of Wall (in)	22	12	12	22	22	22	22
Length of Wall (ft)	30	15	15	20	20	15	15
Horizontal Design	(2) #10 @ 14"	(2) #6 @ 18"	(2) #6 @ 18"	(2) #8 @ 18"	(2) #8 @ 18"	(2) #6 @ 12"	(2) #6 @ 12"
Vertical Design	(2) #8 @ 16"	(2) #4 @ 12"	(2) #4 @ 12"	(2) #6 @ 16"	(2) #6 @ 16"	(2) #6 @ 16"	(2) #6 @ 16"
Flexure Design (Each End)	(8) #11	(1) #11	(1) #11	(14) #11	(14) #11	(12) #11	(14) #11
Boundary Element Size	-	-	-	-	-	26X32	26X32
Level Ground to Level 3	**Y-Direction Core**	**Y-Direction South**	**Y-Direction North**	**X-Direction Core South**	**X-Direction Core North**	**X-Direction South**	**X-Direction North**
Thickness of Wall (in)	22	12	12	22	22	22	22
Length of Wall (ft)	30	15	15	20	20	15	15
Horizontal Design	(2) #11 @ 12"	(2) #6 @ 14"	(2) #6 @ 14"	(2) #8 @ 12"	(2) #8 @ 12"	(2) #8 @ 12"	(2) #8 @ 12"
Vertical Design	(2) #8 @ 12"	(2) #4 @ 12"	(2) #4 @ 12"	(2) #6 @ 16"	(2) #6 @ 16"	(2) #6 @ 16"	(2) #6 @ 16"
Flexure Design (Each End)	(24) #11	(6) #11	(6) #11	(52) #11	(52) #11	(34) #11	(34) #11
Boundary Element Size	-	-	-	36x60	36x60	30x44	30x44

The naming convention for the shear walls in Table 16 is as follows. N-S refers to the direction in which the shear wall is aligned. The walls with the term "Core" in them refer to the walls that make up the elevator core of the structure. The North and South term at the end of the name refers to which wing of the building the wall is located. For example, the N-S Direction Core shear wall is located in the core of the building and is aligned to the North-South axis; the E-W Direction South refers to the shear wall that is oriented in the East-West direction and resides in the Southern wing of the building. The provided reinforcing was designed to be easy to construct and followed a pattern. Easily distinguishable sizes are provided and spacing is kept relatively constant. This provided reinforcing easily fit in the walls and has no constructability issues. The shear wall that carries the most load, North-South Core Wall, uses (2) #11 at 12" for horizontal shear reinforcement and (2) # 10 at 14" on the upper floors. This same wall also uses (24) #11's at each end of the wall for flexure reinforcement on the lower floors while only needing (8) #11's on the upper floors.

Boundary elements were also required on some of the walls. On the lower floors, every wall aligned in the East-West direction needed boundary elements to deal with large flexural loads. These boundary elements were sized to limit the disruption to the architectural floor plan yet still provide adequate strength. The largest of these elements are 36"x60" and are located on the lower levels of the core shear walls in the East-West direction. Only two of the walls needed to have boundary elements continue to the upper floors. These walls required the elements due to their relatively short length of 15 feet. For the full design spreadsheets of the shear walls, see Appendix B.

Drift and Story Drift

When each lateral system was initially sized, maximum roof deflection was calculated for each design and iteration to roughly determine if code requirements were being met. When the initial designs met the requirements of maximum roof drift, the systems were designed for strength. After those procedures had been completed in full, a more detailed analysis of drift and story drift was undertaken.

Following sections 12.8.6 and 12.12 of ASCE 7-05, the elastic displacement of the structures was taken from ETABS for each load combination. These values were then amplified accordingly taking into account the deflection amplification factor and the importance of the structure. Additionally, the elastic deflections were modified according to 12.8.6.2 using the fundamental period of the structure in the corresponding direction without the upper limit of CuTa. Table 17, "Special Reinforced Concrete Moment Frame Story Drift vs. Allowable Story Drift" displays these calculated amplified drift values, story drift values, and allowable story drift values from the critical load combination. Table 18, "Special Reinforced Concrete Shear Wall Story Drift vs. Allowable Story Drift" displays these corresponding values for the shear wall system.

Table17: Special Reinforced Concrete Moment Frame Story Drift vs. Allowable Story Drift

1.0 E3 Load Case	Story Height (ft)	δ_{ex} (in)	$\delta_x = 8\delta_{ex}/1.5$ (in)	$\Delta_i = (\delta_{ex}-\delta_{ex-1})5/1.5$ (in)	$\Delta a = 0.010h_{sx}$ (in)
Level 8	18	2.275	12.136	1.670	2.16
Level 7	13	1.962	10.465	1.103	1.56
Level 6	12	1.755	9.362	1.296	1.44
Level 5	12	1.512	8.066	1.580	1.44
Level 4	12	1.216	6.487	1.787	1.44
Level 3	12	0.881	4.700	1.841	1.44
Level 2	12	0.536	2.859	1.717	1.44
Level 1	14	0.214	1.142	1.142	1.68

Table 18: Special Reinforced Concrete Shear Wall Story Drift vs. Allowable Story Drift					
1.0 E4 Load Case	Story Height (ft)	δ_{ex} (in)	$\delta_x = 5\delta_{ex}/1.5$ (in)	$\Delta_i = (\delta_{ex}-\delta_{ex-1})5/1.5$ (in)	$\Delta a = 0.010h_{sx}$ (in)
Level 8	18	2.415	8.049	1.847	2.16
Level 7	13	1.861	6.202	1.287	1.56
Level 6	12	1.475	4.915	1.154	1.44
Level 5	12	1.128	3.761	1.090	1.44
Level 4	12	0.802	2.672	0.978	1.44
Level 3	12	0.508	1.693	0.811	1.44
Level 2	12	0.265	0.882	0.581	1.44
Level 1	14	0.090	0.300	0.300	1.68

Both lateral systems had their controlling deflection values in the North-South direction. The moment frame system was controlled by seismic load in the North-South direction with positive 5% eccentricity while the Shear Wall System was controlled by seismic load in the North-South direction with a negative 5% eccentricity.

When the deflection calculations were made with the aid of spreadsheets, it was blatantly visible that the Moment Frame Design was not stiff enough, allowing too much deflection on floors 2 through 5. The design was already composed of 34”x34” columns and 34”x36” Beams and it was decided that members any larger would greatly impact the architecture of the building in a negative manner. Based on these points, refinement on the design of the Special Reinforced Concrete Moment Frame system was halted.

On the other hand, the Special Reinforced Concrete Shear Wall system performed very well with all story drift values well under the imposed code limit of 0.01hsx for a shear wall system with an occupancy category of IV.

Overturning and Impact on Foundations

A simplified overturning analysis was performed for both the moment frame system and the shear wall system. For the moment frame system, the overturning moment induced by seismic loading was 232,262 ft-kips while the building self weight was 58,279 kips. The load combination 0.9 D + 1.0 E was used as the controlling combination for this analysis. Given the average length of the moment frames in the short direction of the building at 165 ft, the resulting uplift force caused by the overturning moment was 1,408 kips. With a safety factor of 2, the building self weight was calculated as 26,226 kips. Based on these values, there is no worry of overturning in the moment frame lateral system.

The shear wall system was also analyzed for overturning. The overturning moment for this design was an un-factored 266,179 ft-kips while the un-factored building self weight was determined as 51,209 kips. Again, the load combination of 0.9 D + 1.0 E was used as the controlling combination for this analysis. In the North-south direction, the linear feet of shear was found to be critical at 60 feet resulting in an uplift force cause by the overturning moment of 4,436 kips. Again a safety factor of 2 was used giving the building self weight a value of 23,044 kips. Comparing these values, the uplift force of 4.4 thousand kips was clearly balanced out by the resisting downward load of the building at 23.0 thousand kips.

With the overturning analysis complete, the building was not at risk of overturning in any direction with either lateral system.

The original drilled pier foundation system was also investigated for validity in San Francisco. Much of the soils report for the new location was in reference to the mat slab of the building currently located on the site and it was determined that the cost and scheduling issues with a mat slab were undesirable for this project. The project location in San Francisco contained bedrock at an average depth of 70 feet. Compared to the depth of bedrock at the Baltimore, MD site of 50 feet, an increase in depth of 20 feet for the drilled piers was deemed very reasonable. Clearly the original 4 ft diameter pier size will need to be modified and designed for the new site and other seismic requirements, but at this time the applicability of a drilled pier foundation system seems very good.

Conclusions

After an exhausting analysis and numerous design iterations were performed, the validity and applicability of both the Special Reinforced Concrete Moment Frame System and the Special Reinforced Concrete Shear Wall System can be quantitatively compared.

The moment frame system utilizes very large 34"x34" concrete columns and 34"x36" beams required to meet building fundamental period requirements for use of the Equivalent Lateral Force Procedure. These sizes proved to be much more than needed regarding strength requirements as only minimum vertical reinforcing was needed in columns with (12) #9 bars and only (9) #9 bars were required in beams for flexure strength. Had a Modal Response Spectrum Analysis been performed, the requirement of the building period to fall below 3.5 Ts would have been lifted and more efficient member sizes could have been chosen. In the end, the moment frame lateral system proved to be undesirable as the story displacements were larger than those allowed by code even with the use of extremely large member sizes. These large member sizes also negatively impacted the concrete moment frame applicability in that a great deal of interior space was taken up by the large column sizes along with outward visibility from patient rooms being hampered by the very deep 36" moment frame beams.

However, the shear wall system performed above expectations easily meeting the required fundamental building period of 3.5Ts while utilizing 22" thick walls in most places. This value of 22" thick walls was deemed acceptable given the size of interior columns at 22"x22". The centrally placed elevator core allowed the majority of the shear walls in the structure to be located close to the center of mass therefore reducing torsional effects. The reinforcing required in these walls for strength requirements was extremely reasonable with the most horizontal reinforcement required in any of the walls at (2) #8 bars at 12". Additionally, the worst care flexural reinforcement required was (52) #11 bars located in a 36"x60" boundary element. Furthermore, all drift and story drift values were within code limit. Possible the greatest benefit of the shear wall system is its minimal consumed footprint when compared to the moment frame system. Much smaller 21"x21" and 22"x22" gravity-only-carrying columns were utilized throughout the structure which consume much less floor space than 34"x34" columns used in the moment frame system.

In the end, the Special Reinforced Concrete Shear Wall Lateral System clearly beats the Moment Frame System for structural applicability in the Franklin Square Hospital Center Patient Tower.

Structural Investigation Studies—Floor System Redesign

Introduction

In order to design a more efficient and less imposing lateral system, it was decided early in the proposal process that reducing the slab depth from the original 10" mildly reinforced flat plate to an 8" post-tensioned slab with drop panels would be a wise choice. The original buildings 10" slab consumed over 56% of the entire building weight! The opportunity to make a change to the floor system that could drastically benefit the lateral system while not making significant other changes to the structure, the post-tensioned slab really stood out over the other steel systems. From preliminary hand calculations, an 8" deep post-tensioned system would work given the larger spans of 30 feet while also being able to handle the odd span lengths when crossing the structure from 30 feet to 15 feet back to 30 feet just by changing the tendon drape. It was decided to run the uniform or distributed tendons in the North-South direction as this was the longer axis and the banded tendons would reside in the column strips in the East-West direction as this was the shorter of the principle axis. Adapt PT v8 was used for refinement and checking of preliminary hand calculations. Adapt PT v8 proved to work very well for designing strips of the slab in both directions with easy to follow input procedures and comprehensive output data. The biggest aid that Adapt PT v8 provided was in calculating additional mild reinforcing steel, quickly checking punching shear around columns, and calculating deflections. These two features proved invaluable during the many iterations performed to perfect the final design. Figure 27, "Adapt PT v8 Screenshot" visually displays the geometry of the structure including column sizes, slab thickness, drop panels, and the imposed loading.

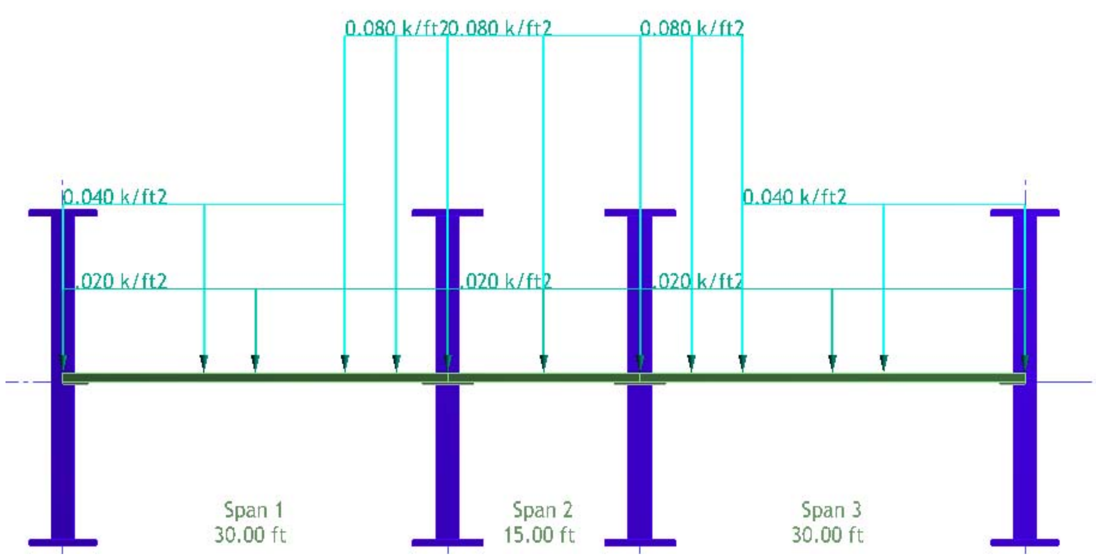


Figure 27: Adapt Pt v8 Screenshot

Post-Tensioned Layout for Moment Frame Configuration

The design of the post-tensioned slab for the moment frame lateral system was fairly uneventful except for one small issue discovered regarding the area of slab around the elevator core. ½" 7-wire un-grouted strands were used for the tendons as they are fairly common and standard. In the North-South direction, the design consists of these ½" tendons spaced at 12" stressed to 389.2 kips balancing anywhere from 60% of the dead load in the exterior bays to 96% in the interior spans. The induced compression in the slab was kept at a constant 270 psi in the direction of the uniform tendons to reduce longitudinal cracking between sections from pressure differences. The one area where the 8" post-tensioned slab could not be made to work at an 8 inch thickness was the wing of the building that houses the elevator core, the main staircase, and miscellaneous support spaces such as bathrooms and electrical closets. Due to the high loading and odd geometry, the thinnest post-tensioned slab that would meet stress limits was 11 inches deep. Obviously, this was unacceptable given the goal of reducing building weight, so the original 10" mildly reinforced slab was used in this area and a construction joint was employed between the two sections that would be filled solid once the post-tensioned slab had been fully stressed.

The banded tendons running in the East-West direction are of the same type and size as used in the uniform direction only these are pulled into the column strips to act as girders to the uniform tendons joist implementation. Tendons in these areas range anywhere from groups of 11 to groups of 33 stressed from 293 kips to 864 kips. The pre-compression in the slab was designed right up to the limit of 300 psi and the tendon profile was slightly modified so there would be no interferences with the uniform tendons. Balanced dead load in the end spans was 66% while 93% of the dead load was balanced in many of the inner spans.

In both directions, most all interior spans had maximum service load deflections under 0.25" while all exterior spans had maximum service load deflections under 0.4". Serviceability requirements never controlled over strength requirements at any time in the design of the post-tensioned slab.

During the design process, issues arose resulting from punching shear problems. In many places stud rails proved to not be enough so drop panels or shear caps were added to resolve these issues. The drop panels provided are 4'x4' and extend 2" below the bottom of the post-tensioned slab. Only a few select columns needed any additional shear reinforcement beyond the addition of the drop panels.

To view the tendon layout for the Moment Frame design, see Figure 28, "Typical Post-Tension Tendon Layout for Moment Frame System" on the next page. More information on the tendon profile for each span, see Appendix C.

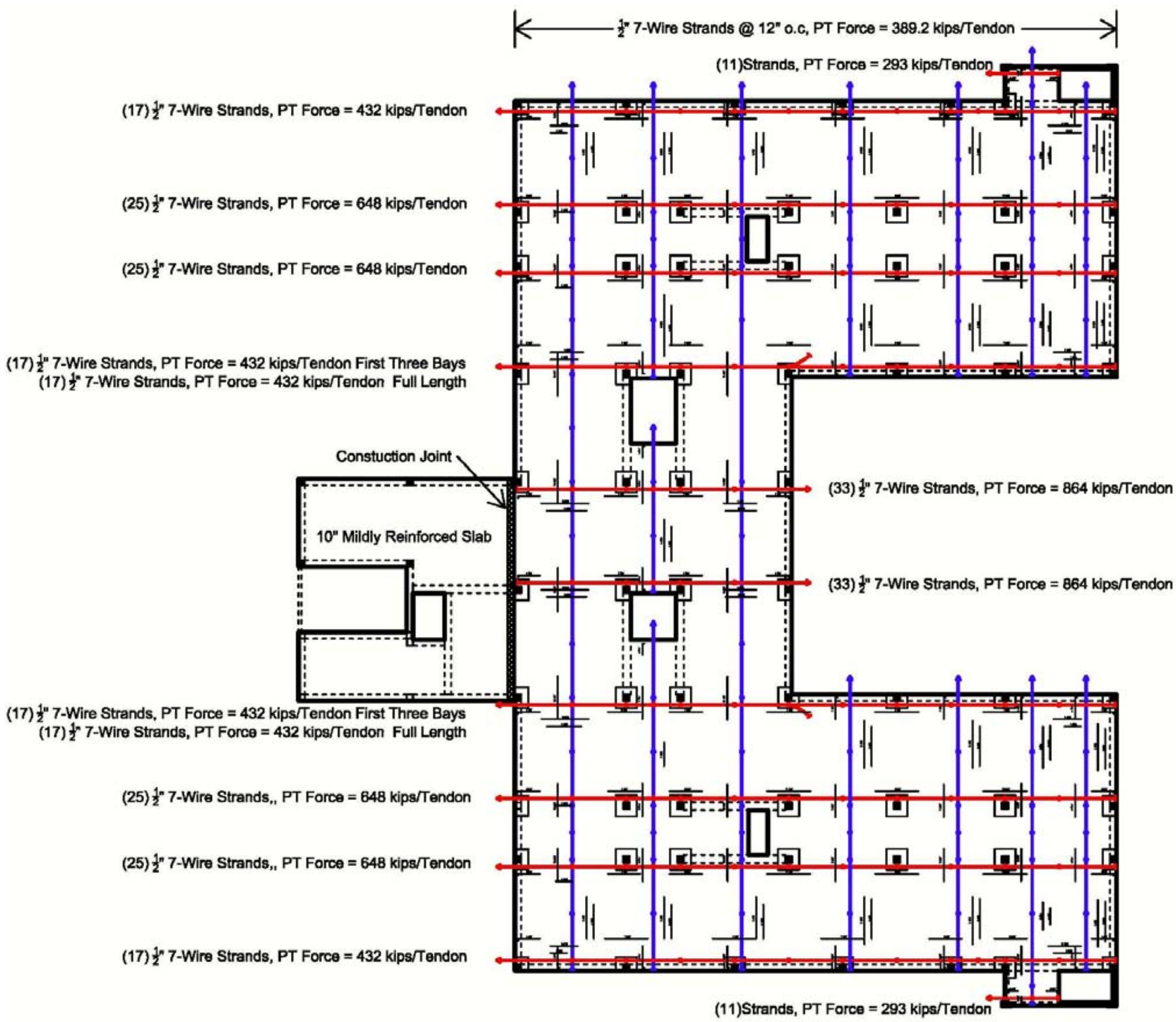


Figure 28: Typical Post-Tensioned Tendon Layout for Moment Frame System

Post-Tensioned Layout for Shear Wall Configuration

The design of the post-tensioned slab for the shear wall lateral system is almost identical to that of the Moment Frame System with a few small changes. The most notable is the ability for the entire level to be an 8" slab. With the elevator core moved, the odd geometric irregularities were removed from the west wing thus allowing an 8" post-tensioned slab to meet all stress requirements. The tendons around the elevator core and main stair tower also had to be slightly shifted and now these tendons follow a path that snakes around the core which is a common occurrence in post-tensioned slabs. All other properties of the slab design are similar between both lateral system designs. See Figure 29, "Typical Post-Tensioned Tendon Layout for Shear Wall System" to view how the design changed once shear walls were added to the plan.

Figure 28: Typical Post-Tensioned Tendon Layout for Shear Wall System

Conclusions

With the goal of substantially lowering building weight, the original 10" flat plate floor system of the Franklin Square Hospital Center San Francisco Version was redesigned to an 8" Post-Tensioned flat slab. This change resulted in a building weight loss of 5,800 kips or roughly a 10% decrease in total building weight. Original estimates had the weight loss around 15% of the total building weight but with the addition of drop panels, that estimate decreased to 10%.

Had this change in floor system not been made, the design of both lateral systems would have suffered. While strength design was not that controlling factor of either lateral design, the added mass of the building would have pushed both lateral systems over the limit in terms of fundamental building period allowed for use under the Equivalent Lateral Force Procedure with an occupancy category of IV resulting in a need for even larger concrete columns and beams for the moment frame system, and even thicker walls for the shear wall system.

Additionally a cost and schedule comparison was performed between the flat slab post-tensioned floor system and the original mildly reinforced flat plate. To see those results, view the Construction Cost and Scheduling Study of this report.

Architectural Floor Plan Study

Throughout the design of the shear wall lateral system, not only were structural necessities kept in mind, but architectural plan considerations were made. As discussed earlier, the elevator core was placed as close to the center of mass of the building as possible to reduce torsional effects. This drastically changed the architectural floor plans and space arrangements as the elevator core had originally resided in the west wing of the building. It was very difficult to keep the same logical space arrangements and flow that the original design had, but the new plan still contains many of the same design philosophies and ease of use expected in a hospital design.

Where the elevator core now resides had been occupied by patient rooms, a main corridor, and mechanical support spaces. The displaced patient rooms were transplanted to the area previously occupied by the public elevator lobby still within close proximity to the nurse's stations. A similarly large sized service elevator lobby was provided out of the way of traffic to accommodate stretchers and other large vehicles using the elevators. The main stair tower of the building was also moved with the elevator core for easier access and more logical vertical transportation routes. In addition, the mechanical closest and mechanical support spaces were kept within close proximity to the large mechanical openings in the slab near the center of the building for shorter pipe and wiring lengths.

In the original design, the public elevator lobby had a great deal of storefront glazing which overlooked the main entrance and lobby. In the new orientation, the public elevator lobby overlooks the lower first floor roof of the emergency department and out between the north and south wings of the building. In this space it is easy to grasp the sheer size of the Franklin Square Hospital Center and appreciate its great strength and robustness.

Figure 29, "Original Plan for Moment Frame Lateral System" shows the general layout of a typical level and details each space with its function. Figure 30, "Revised Plan for Shear Wall Lateral System" also shows a quick overview of the layout changes again with the functions of each space labeled. On the flowing pages, Figure 31, "Blow-Up of Central Section of Moment Frame Lateral System Plan Design" and Figure 32, "Blow-Up of Central Section of Shear Wall Lateral System Plan Design" show the plan changes in more detail of the central section of the building. Figure 33, "Blow-Up of West Wing of Moment Frame Lateral System Plan Design" and Figure 34, "Blow-Up of West Wing of Shear Wall Lateral System Plan Design" detail the space changes in the Western Section of the building.

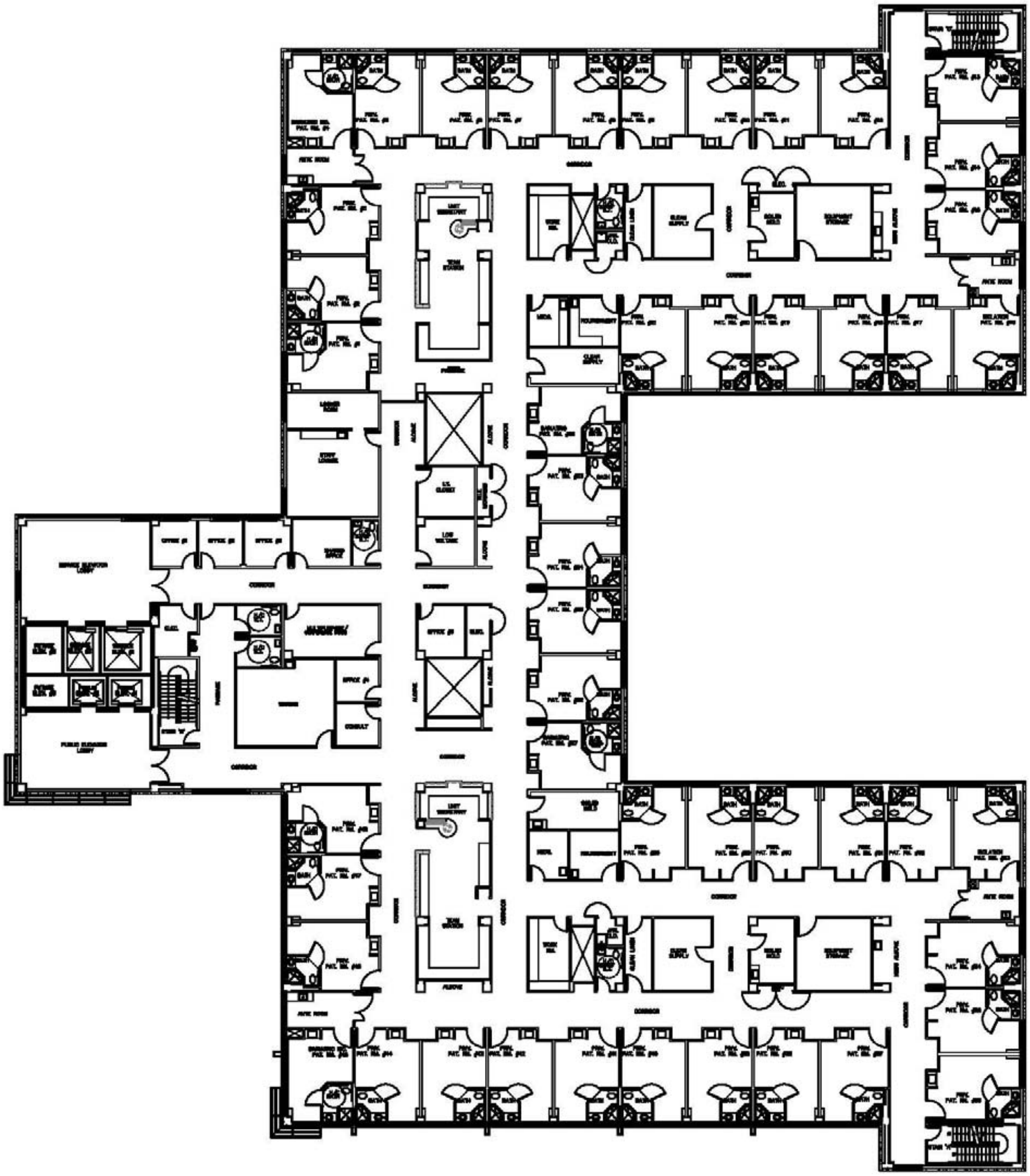


Figure 29: Original Plan for Moment Frame Lateral System

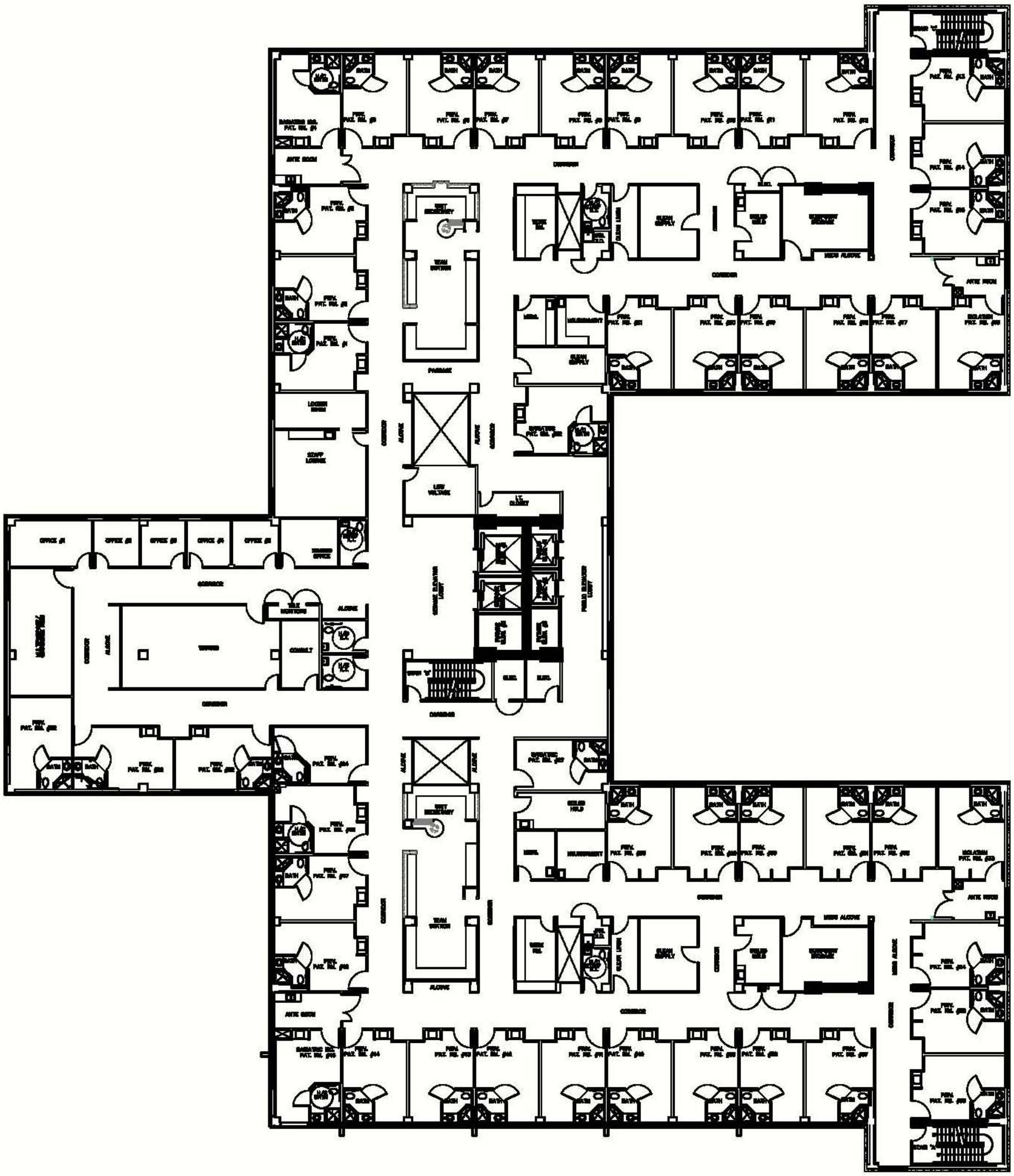


Figure 30: Revised Plan for Shear Wall Lateral System

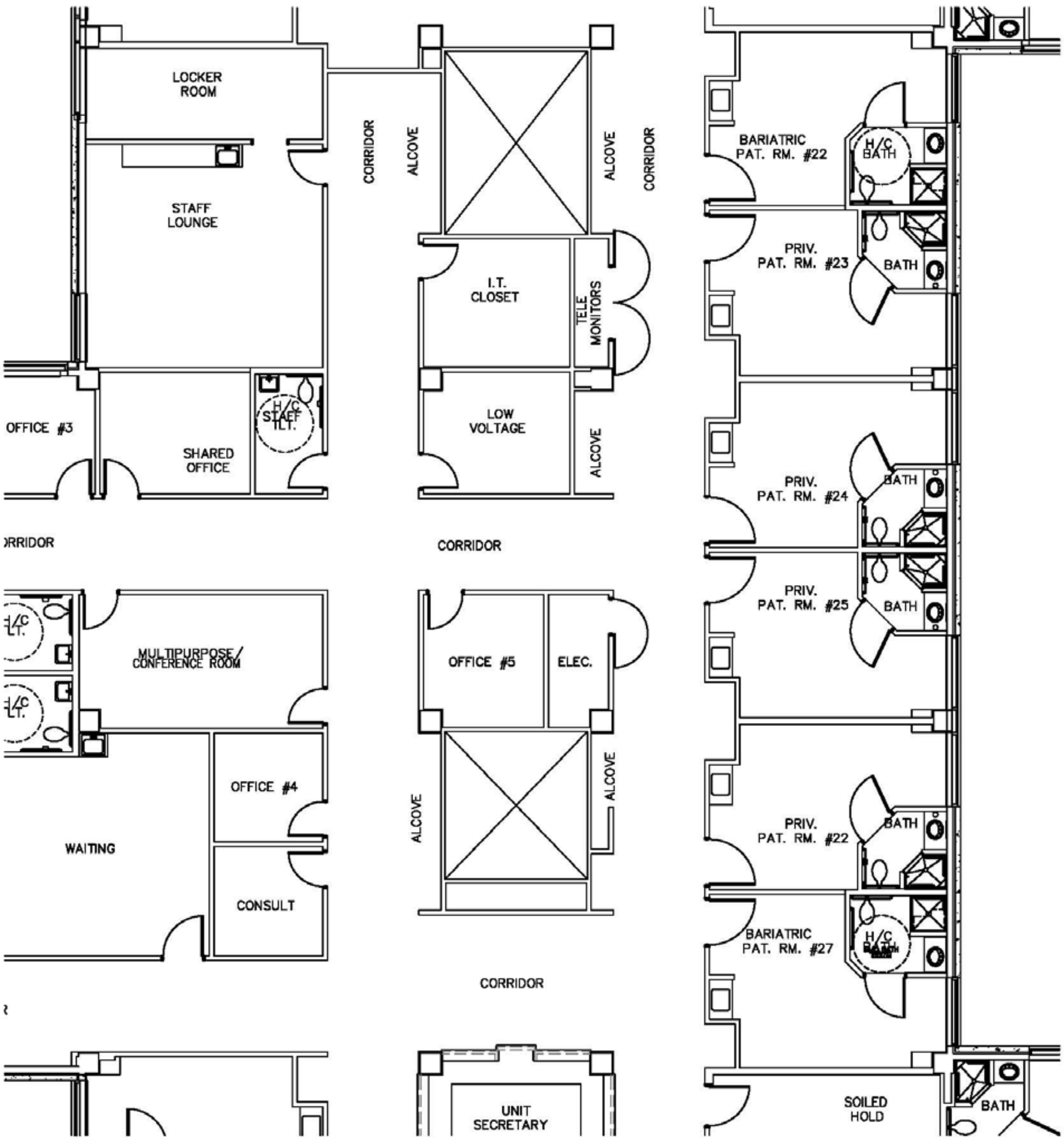


Figure 31: Blow-Up of Central Section of Moment Frame Lateral System Plan Design

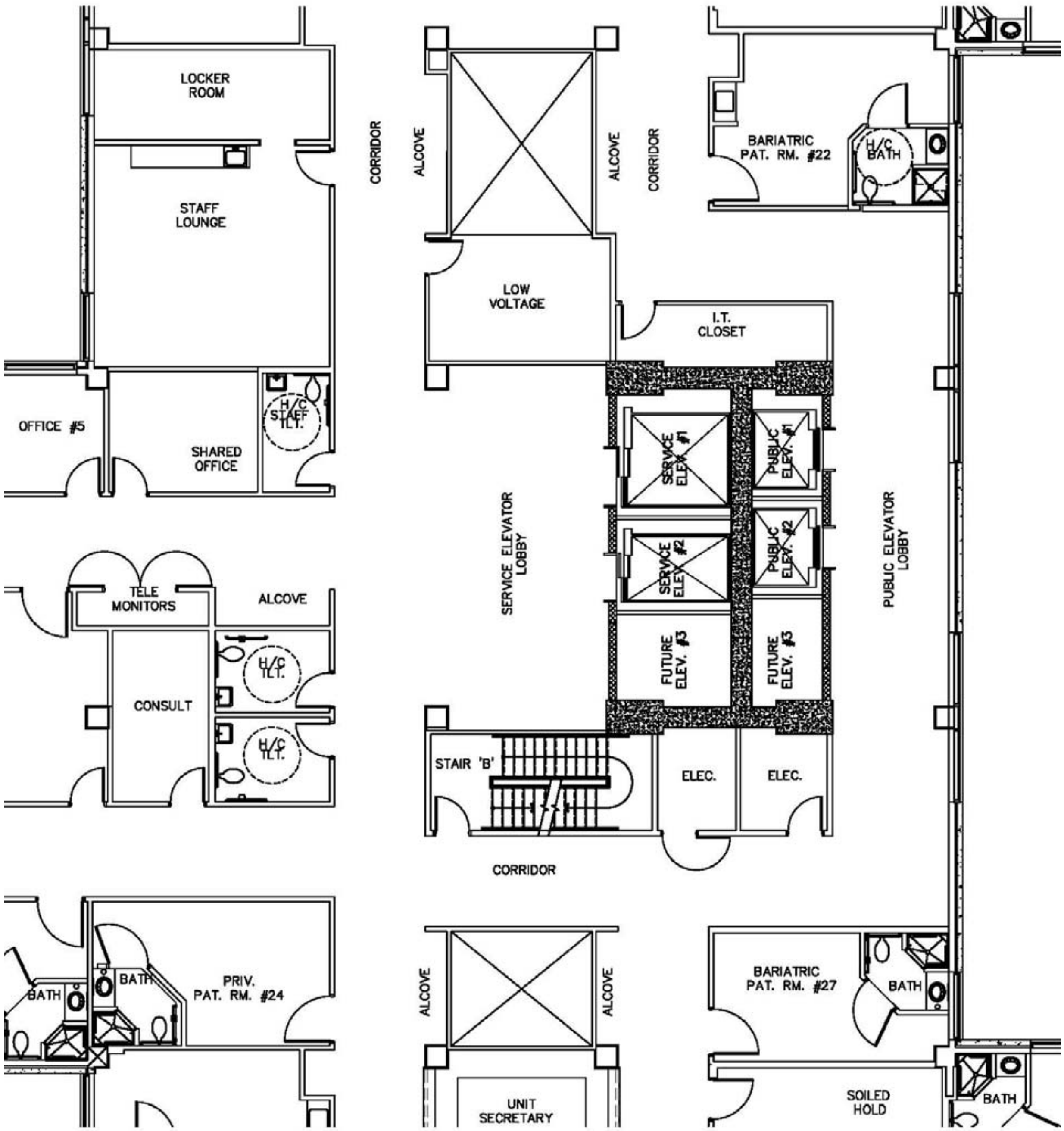


Figure 32: Blow-Up of Central Section of Shear Wall Lateral System Plan Design

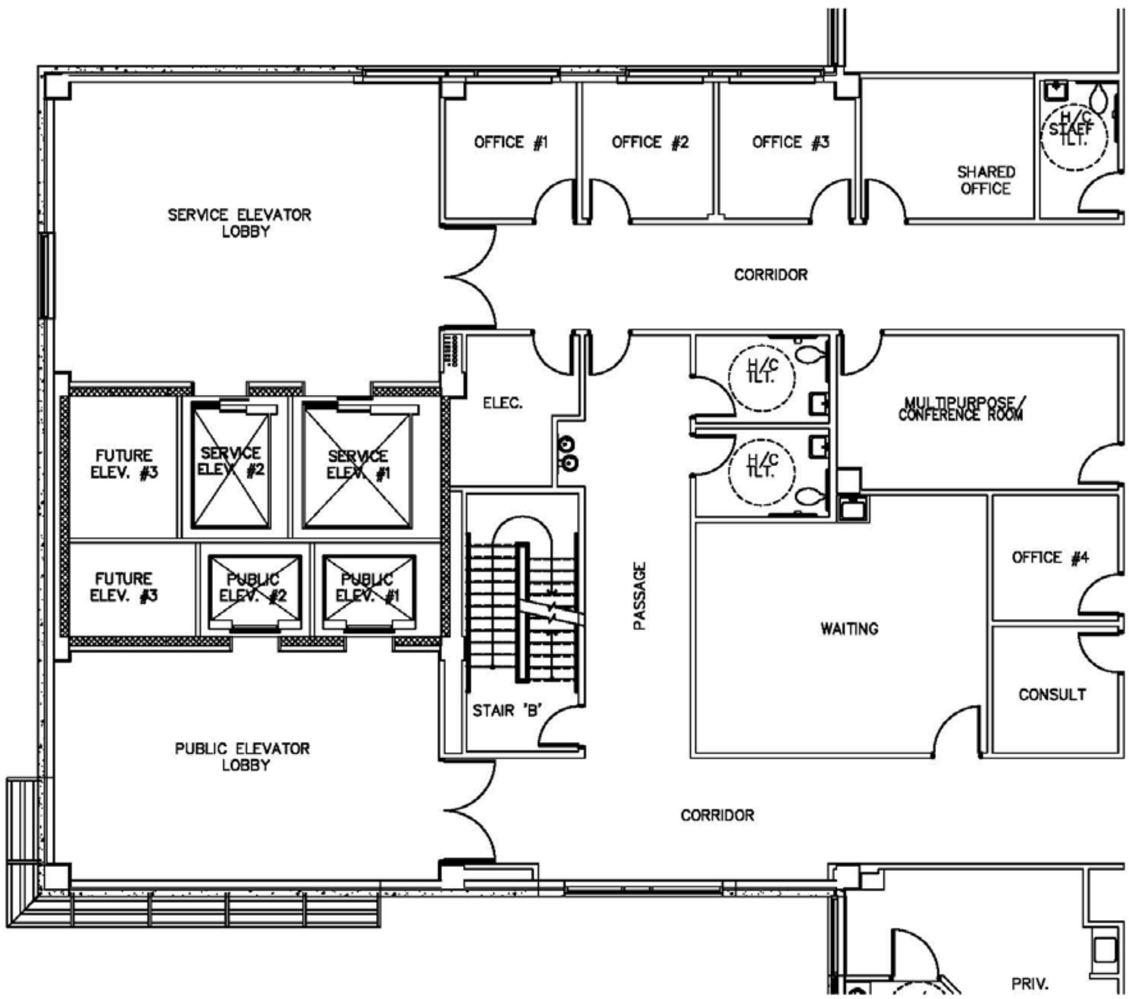


Figure 33: Blow-Up of Western Section of Moment Frame Lateral System Plan Design

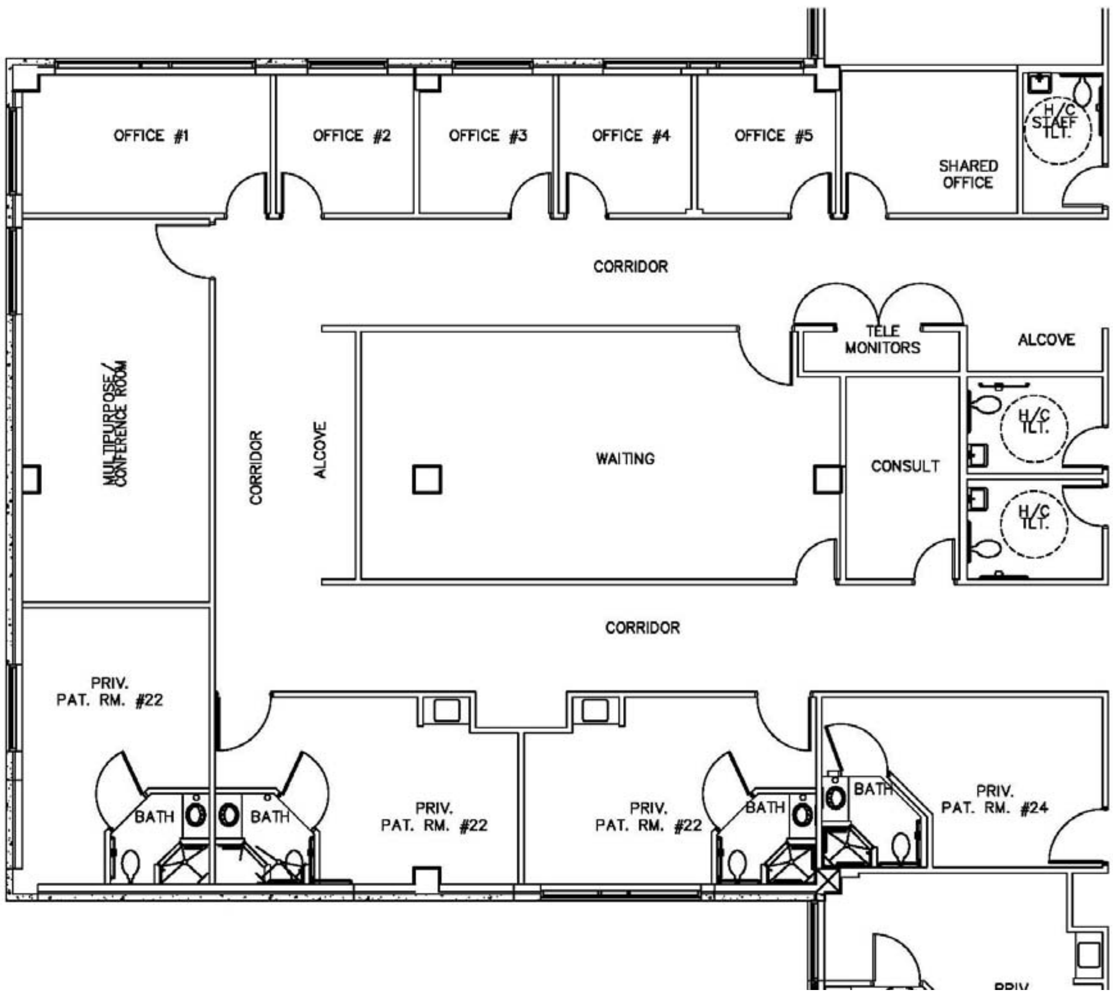


Figure 34: Blow-Up of Western Section of Shear Wall Lateral System Plan Design

Conclusions

The addition of shear walls to the layout of the Franklin Square Hospital Patient Tower was easily dealt with through logical and calculated changes to the interior spaces. The new elevator core, with its new central location, provides easy access to all levels and is easily found by occupants. When the Franklin Square Hospital Patient Tower expands in the future, the area between the north and south wings on the east side of the building will be in-filled and built up over the ground floor emergency department. With the elevator core now located in the center of the building, vertical transportation will be made much easier for all occupants verses the original design where an additional elevator core would be required or long walks would be required from one end of the building to the other to travel vertically.

The revised locations of the patient rooms provide identical function while still being in close proximity to nurses stations. Additionally, the numerous offices that were sprinkled throughout the plan in the original design are now logically all located adjacent to each other in the west wing with close proximity to the waiting room and consult spaces. With the more efficient use of space and slightly fewer cross-passages, many spaces increased in net square feet. For example, the waiting room on the typical floors has grown to match the size of the waiting room on the ground floor where there are less space constraints.

Overall, the revised architectural layout of the floor plans for accommodation of a shear wall lateral system is quite successful.

Construction Cost and Scheduling Study

To satisfy the second breadth topic, a cost and schedule comparison was constructed comparing the original 10” Flat Plate Floor System with the newly designed 8” Post-Tensioned Flat Slab Floor System. The items used in the cost analysis were composed only of those items which were of difference between the two systems and both systems were analyzed for the location of San Francisco, CA in the year of 2010. The cost items that varied between each system were elevated slab formwork, quantity of mild reinforcing steel, quantity of post-tensioned tendons, cubic yards of concrete, and cubic yards of concrete placing. These items included material and labor costs. The items not considered were curing costs and finishing costs as these were the same between both systems. Table 18, “Cost comparison of 10” Flat Plate vs. 8” P-T Flat Slab” itemizes each line item, the quantity, and the summarized total cost.

Table 18: Cost Comparison	10" Flat Plate		8" P-T Flat Slab	
Item	Quantity	Total Cost	Quantity	Total Cost
Elevated Slab Flat Plate Formwork	282,070 SF	$484,351.25	282,070 SF	$521,132.23
Mild Steel Reinforcing	295 Tons	$500,290.44	58 Tons	$97,955.74
Post-Tension Tendons	0 Lb	$0.00	324,268 Lb	$593,410.07
Concrete	9,221 CY	$1,301,671.65	7,222 CY	$1,019,457.52
Placing	9,221 CY	$206,187.15	7,222 CY	$161,483.92
Total		**$2,492,500.49**		**$2,393,439.48**

Interesting to note is the similarity in total cost between each system. Conventional wisdom seems to imply that the Post-Tensioned slab would cost considerable more but instead it costs $99,061 less than the 10” Flat Plate system. This price decrease makes sense when looking at the individual cost items. The formwork and total reinforcing for the post tensioned slab costs $227,856 more that the formwork and total reinforcing for the 10” flat plate. However, the total cost of concrete and concrete placing is $326,917 cheaper for the post-tensioned system. The cost of concrete and concrete placing is so very much less because the Post-Tensioned Floor System utilizes vastly less concrete by about 2,000 cubic yards than the conventional 10” slab.

For the schedule comparison, the daily output for each item and each crew was calculated and input into Microsoft Project for both systems. Figures 31 and 32 on the following pages detail the schedule timeline for each floor system. It was found that the Post-Tensioned System took a total of 22 more work days or close to 4 weeks to complete than the original 10” Flat Plate System.

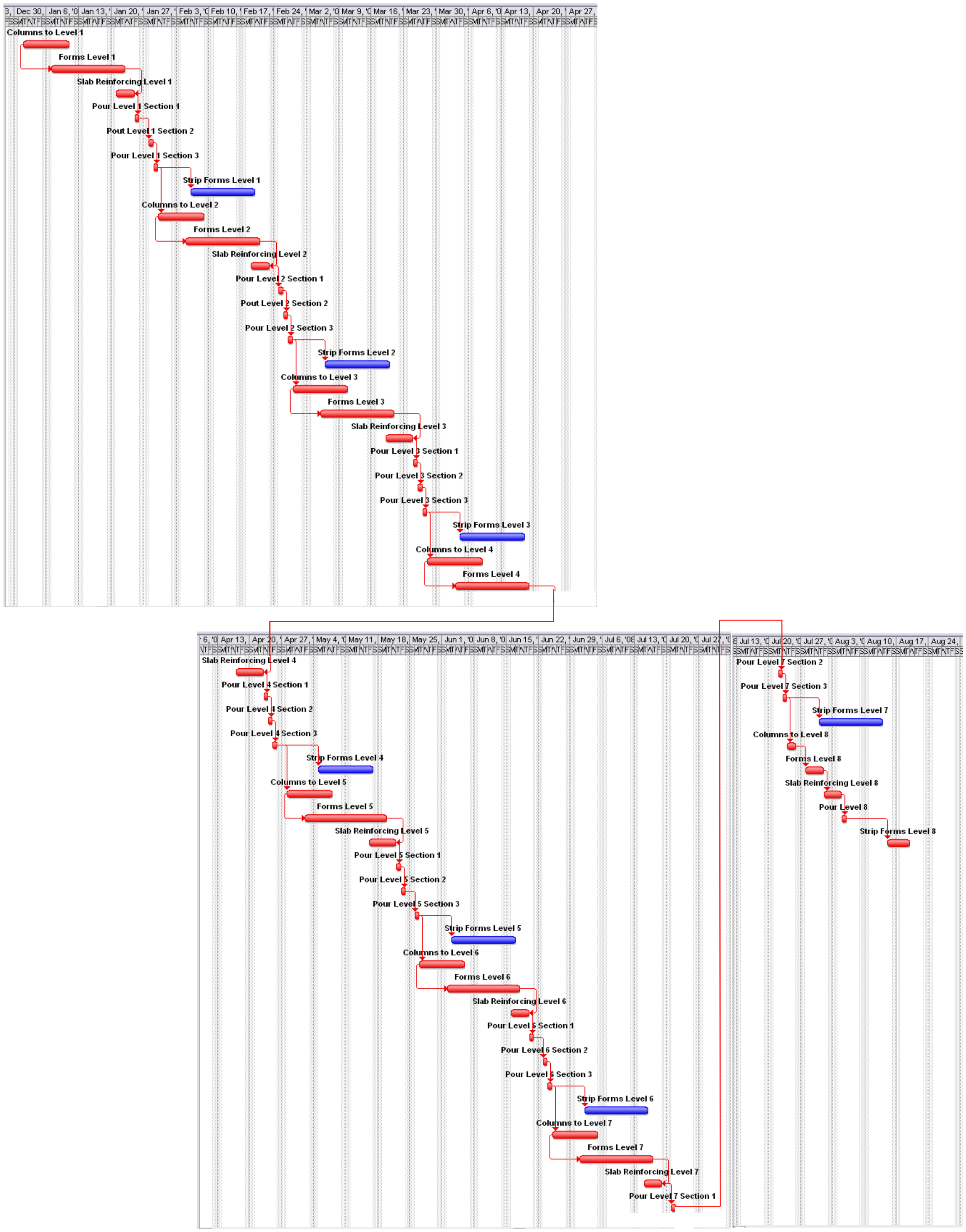


Figure 31: Ordinary Reinforced 10” Flat Plate Floor System Construction Schedule

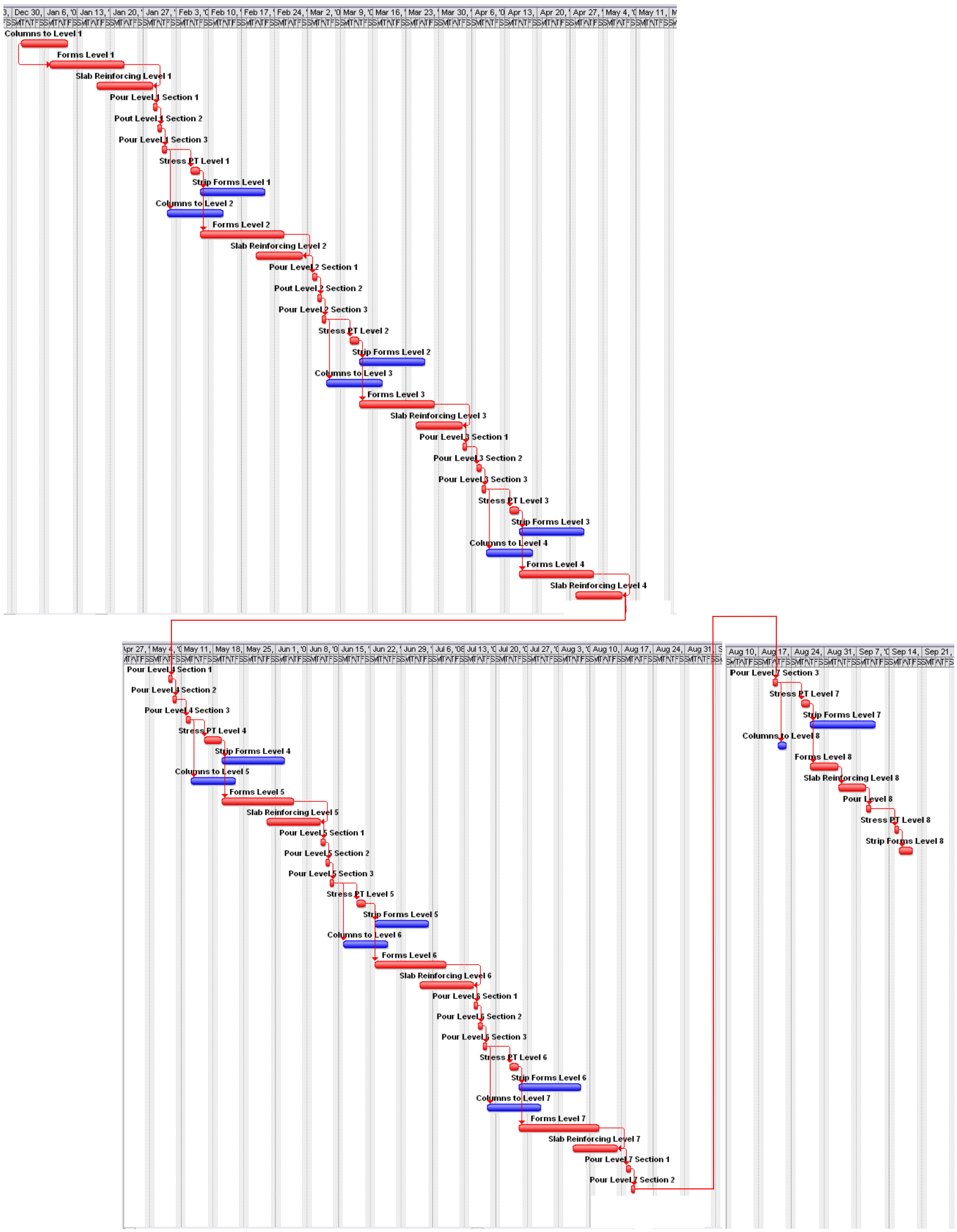


Figure 32: Post-Tensioned 8” Flat Slab Floor System Construction Schedule

The columns took on average 8 days per floor from start to finish and the forming for the level above was started once the columns below were half completed. The forming of the slab took 12 days per floor while slab reinforcing took 4 days for the 10" Flat Plate and 8 days for the 8" P-T Flat Slab. Each level was broken into 3 pours each containing on average 300 cubic yards and taking a day to complete each. The day after the slab was poured the columns to support the next floor began. All stripping activities were commenced after a 7 day break arrived and followed the pour sequence. Stripping took on average 10 days per floor.

With the 10" Flat Plate floor system, the critical path followed columns, slab forming, slab reinforcing, pouring of concrete and then forming of columns on the next floor. The stripping of the slab forms did not reside on the critical path in this system.

With the 8" PT Flat Slab floor system, the critical path followed columns, slab forming, slab reinforcing, pouring of concrete, a three day break before the PT tendons were stressed, and then the forming of the slab above. The items in this schedule not residing on the critical path were the column forming and stripping of the slab forms.

General Conditions for the project were estimated at $40,000 a week. With a 4 week longer construction schedule for the Post-Tensioned Floor system over the Original 10" Flat Plate, the added cost due to schedule changes was around $160,000.

When the cost reduction from the Post-Tensioned Floor System of $99,000 is added to the general conditions cost increase for a lengthened schedule, the Post-Tensioned System ends up costing roughly $61,000 more than the original 10" Flat Plate System. Converting this into cost per square foot, the Post-Tensioned Floor System costs roughly 22 cents more per square foot than the 10" Regularly Reinforced Flat Plate.

Conclusions

Given the benefits the much lighter P-T slab offers in terms of decreased building weight for more efficient lateral design, the increase in total cost of 22 cents per square foot is extremely minimal. Given this cost and scheduling study, there would be no reason to not choose the Post-Tensioned Flat Slab Floor System for use in the structure.

MAE Project Integration

The MAE requirements for the project were fulfilled through the knowledge taught in AE 597A: Computer Modeling and AE 538: Earthquake Resistant Design of Buildings.

An ETABS 3D Building Model was constructed according to the material taught by Dr. Andres Lepage in Computer Modeling which was extremely useful in properly modeling the lateral force resisting elements and modeling techniques used that result in accurate analysis output. This ETABS 3D building model was used to determine all member forces under seismic loading while also determining elastic story displacements. Accidental and inherent torsional effects were taken into account with this model which proved to be an exceptional time saver. Everything from modified section properties, rigid end offsets, insertion points, panel zone assignments, diaphragm mass assignments, and rigid and semi-rigid diaphragm assignments were used. This model proved invaluable for quickly and accurately comparing proposed lateral system designs and implementation.

In addition to the comprehensive ETABS 3D building model, the seismic provisions and design requirements taught in AE 538: Earthquake Resistant Design of Buildings by Dr. Ali M. Memari was used throughout the design of the lateral force resisting systems. Proper calculations of seismic loading as well as correct drift amplification and seismic detailing requirements were implemented through the material taught in this class.

Summary and Conclusions

The goal of this final thesis report was to investigate more severe seismic loading through moving the Franklin Square Hospital Center Patient Tower to San Francisco, CA. San Francisco was chosen as the new building site for its seismic history and close proximity to major fault systems. With the hopes of designing an efficient lateral system design, it was also determined that building weight would need to decrease. This was to be attempted through the change of the floor systems from a 10" regularly reinforced flat plate to an 8" post-tensioned flat slab system. The redesign of the lateral system would focus on two main system types—concrete moment frame and concrete shear wall systems. Additional topics that needed to be covered resulting from the structural changes included an architectural plan study focusing on the changes required from the addition of shear walls and a construction cost and schedule study to determine cost differences between the original 10" regularly reinforced flat plate floor system and the 8" post-tensioned flat slab. The MAE requirement for the project was to be fulfilled through the construction and implementation of an improved and comprehensive ETABS building model. Methods taught in AE 597A: Computer Modeling including modified section properties, rigid end offsets, insertion points, panel zones and rigid and semi-rigid diaphragms were to be included in the model.

After an exhausting analysis and numerous design iterations were performed, the validity and applicability of both the Special Reinforced Concrete Moment Frame System and the Special Reinforced Concrete Shear Wall System were determined. The moment frame system utilizes very large 34"x34" concrete columns with (12) #9 bars and 34"x36" beams with (9) #9 bars top and bottom. Because of the large member sizes required in the moment frame lateral system, it was determined that the Franklin Square Hospital Center Patient Tower would be best served by a lateral system that was not as imposing on the interior architectural spaces.

The shear wall lateral system, however, performed extremely well given the demanding seismic loads. Utilizing 22" thick walls in most places, the shear wall system composed of H-shaped shear walls, and additional smaller L-shaped walls performed surprisingly well meeting all strength, building period, and serviceability requirements. The centrally placed elevator core allowed the majority of the shear walls in the structure to be located close to the center of mass therefore reducing torsional effects. The reinforcing required in these walls for strength requirements was extremely reasonable with the most horizontal reinforcement required in any of the walls at (2) #8 bars at 12". Additionally, the worst care flexural reinforcement required was (52) #11 bars located in a 36"x60" boundary element. The greatest benefit of the shear wall system is its minimal consumed footprint when compared to the moment frame system. Much smaller 21"x21" and 22"x22" gravity-only-carrying columns were utilized throughout the structure which consume much less floor space than the 34”x34” columns used in the moment frame system.

With the goal of substantially lowering building weight, the original 10” flat plate floor system of the Franklin Square Hospital Center San Francisco Version was redesigned to an 8” Post-Tensioned flat slab. This change resulted in a building weight loss of 5,800 kips or roughly a 10% decrease in total building weight. Original estimates had the weight loss around 15% of the total building weight but with the addition of drop panels, that estimate decreased to 10%.

To deal with the architectural changes necessitated by the addition of shear walls, the architectural floor plans of the Franklin Square Hospital Patient Tower were modified with logical and calculated changes to the interior spaces. The new elevator core, with its new central location, provides easy access to all levels and is easily found by occupants. The revised locations of the patient rooms provide identical function while still being in close proximity to nurse’s stations. Overall space relationships were also largely undisturbed.

Given the benefits the much lighter P-T slab offers in terms of decreased building weight for more efficient lateral design, the increase in total cost of 22 cents per square foot is extremely minimal. While the cost of the post-tensioned system was less than the regularly reinforced slab, the increase in schedule length of 4 weeks increased general condition costs by $160 thousand, therefore making the post-tensioned system more expensive in the end. Given this cost and scheduling study, there would be no reason to not choose the Post-Tensioned Flat Slab Floor System for use in the structure.

In the end, the Special Reinforced Concrete Shear Wall Lateral System clearly beats the Moment Frame System for structural applicability in the Franklin Square Hospital Center Patient Tower. The differences in architectural plan configuration, while not having a monetary value, made little difference in the function needed for hospital tasks. Additionally, the total cost increase of 22 cents per square foot for the post-tensioned slab easily outweighs the added cost from a more intrusive and larger lateral system design.

Overall, if the Franklin Square Hospital Center Patient Tower were to be built in San Francisco, the ideal structure would contain a shear wall lateral system with a post-tensioned concrete floor system.

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Penn State Architectural Engineering Classmates

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References

The following software program were used to assist in the design and analysis calculations

- ETABS Nonlinear Version 9.5.0 – Extended 3D Analysis of Building System – Copyright © 1948-2008 Computer &Structures, Inc.
- ADAPT-PT Version 8.00 Plus – Copyright © ADAPT Corporation 1980-2008
- pcaColum v3.64 – Copyright ©1988-2005 Portland Cement Association
- AutoCAD 2010 – Copyright © 2009 autodesk, Inc.
- RSMeans CostWorks – Copyright © 2010 RSMeans

The following design codes were used to assist in the design and analysis calculations

- ASCE 7-50 Minimum Design Loads for Buildings and Other Structures" – American Society of Civil Engineers
- "ACI 318-08 Building Code Requirements for Structural Concrete" – American Concrete Institute
- "International Building Code 2006" – International Code Council

The following firm/companies provided design documents for the original building as designed and built in Baltimore, MD

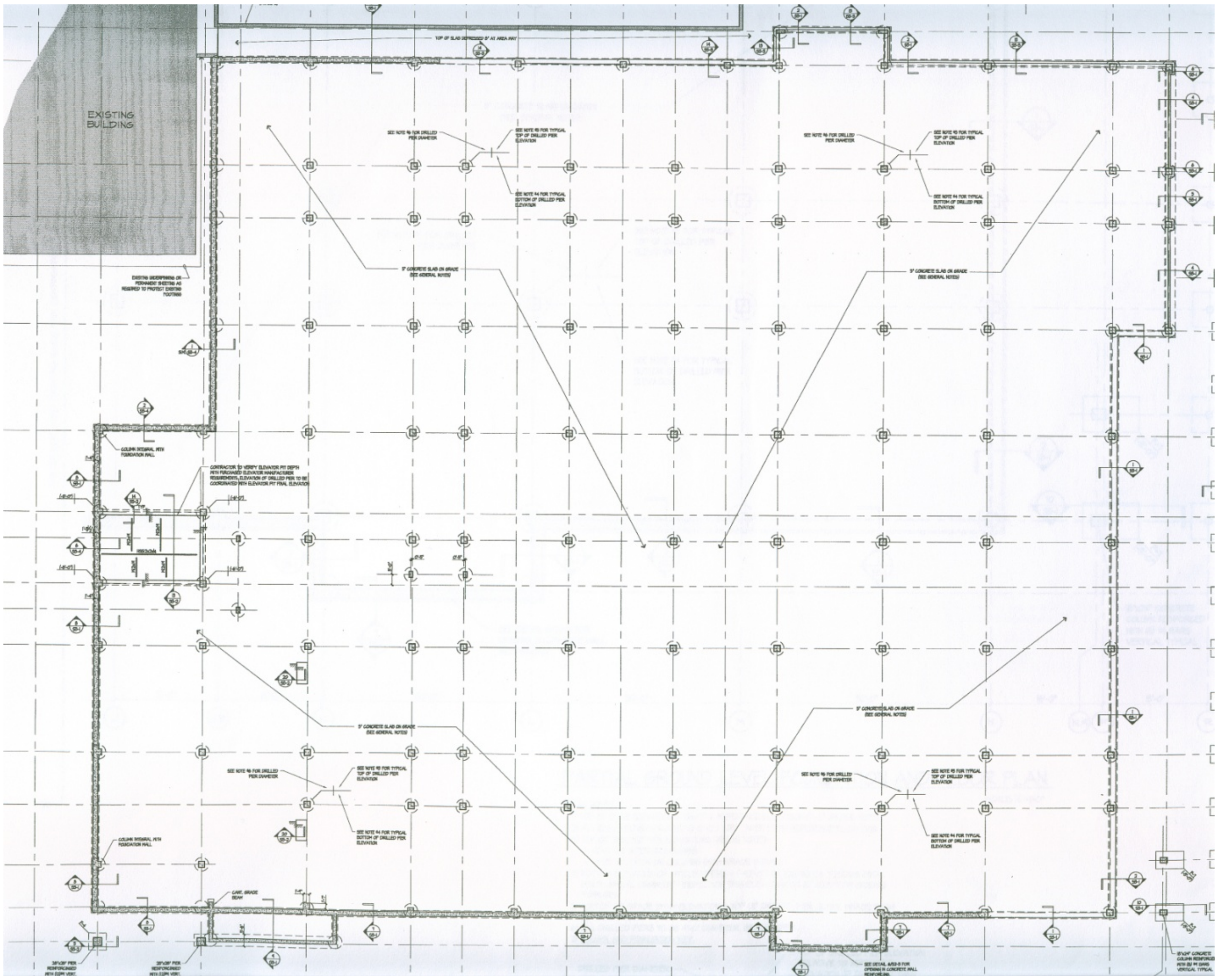
- Rathgeber/Gross Associates – Structural – Rockville MD
- Wilmont/Zanz Inc. – Architectural – Gaithersburg MD
- Bovis Lend Lease – Construction Management – Bethesda MD

The following works cited were used for concrete design and general seismic design research

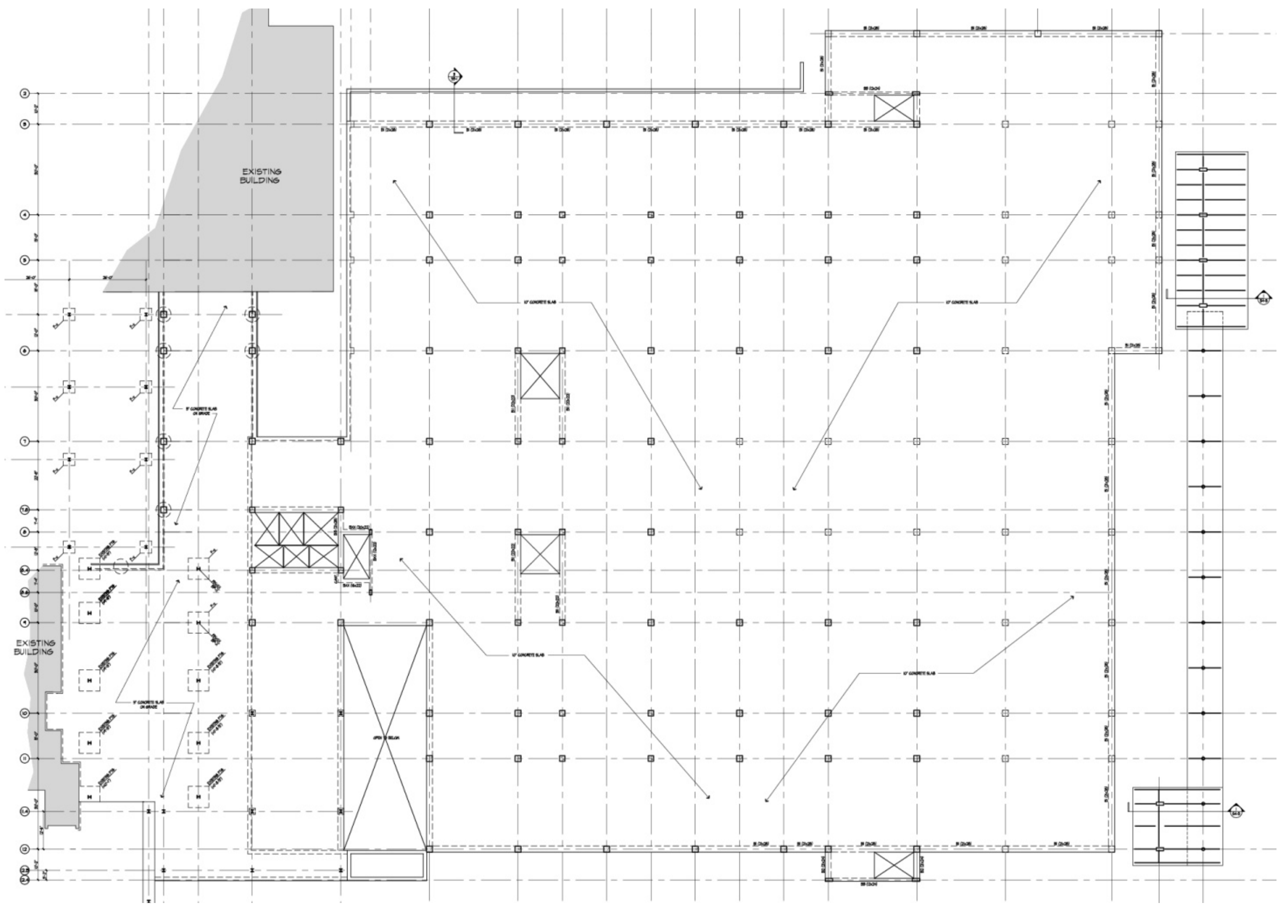
- Wight, James K., and James G. MacGregor. *Reinforced Concrete - Mechanics and Design*. 5th ed. Upper Saddle River, New Jersey: Pearson Education. Print.
- "Post-Tensioned Concrete Wall and Frames for Seismic Resistance – A Case Study of the David Brower Center." Web. 12 Jan 2010. <ftp://ftp2.bentley.com/dist/collateral/docs/ram_concept/cs_brower_seaoc_convention.pdf>

Appendix

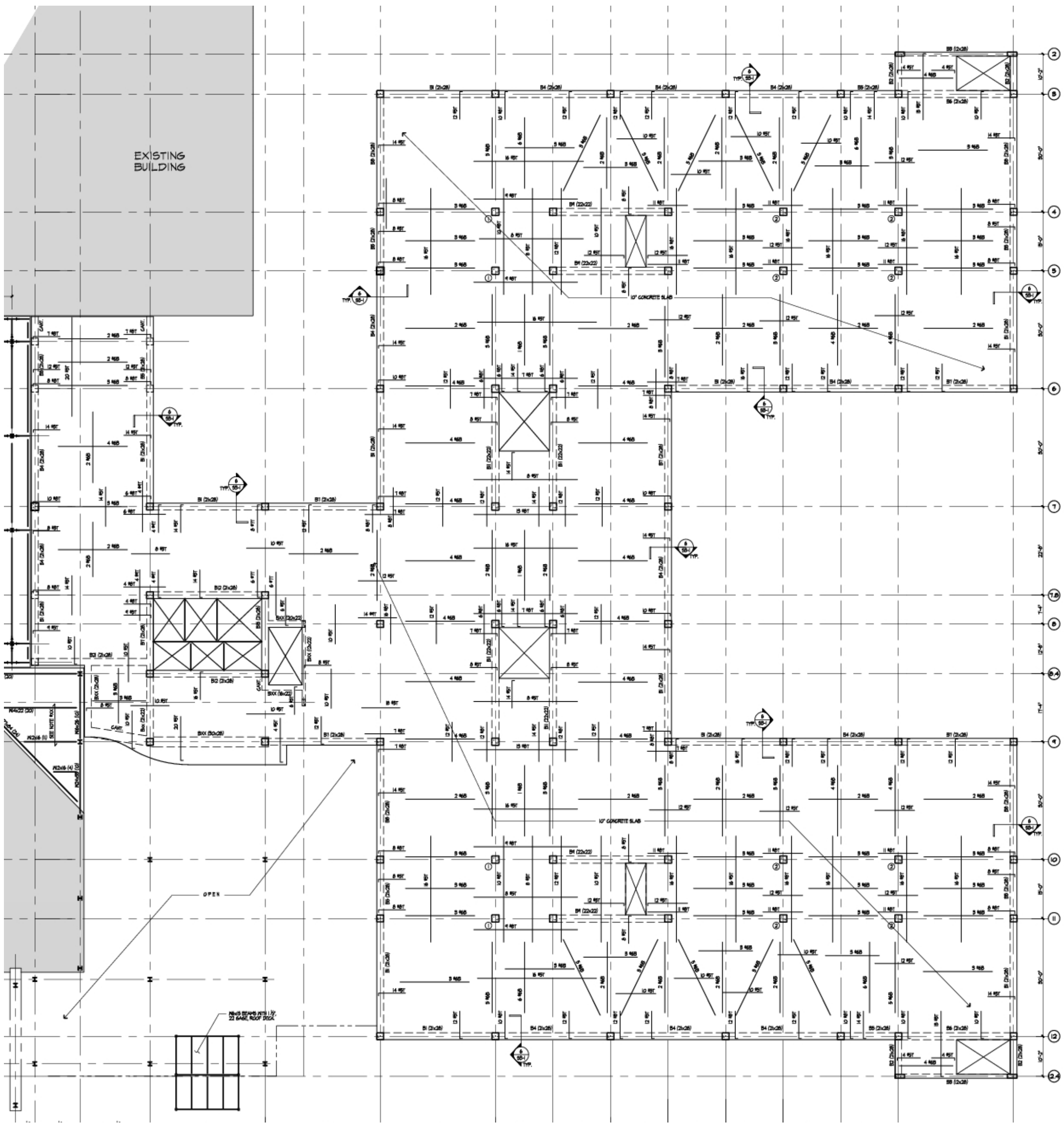
Appendix A: Typical Floor Plans



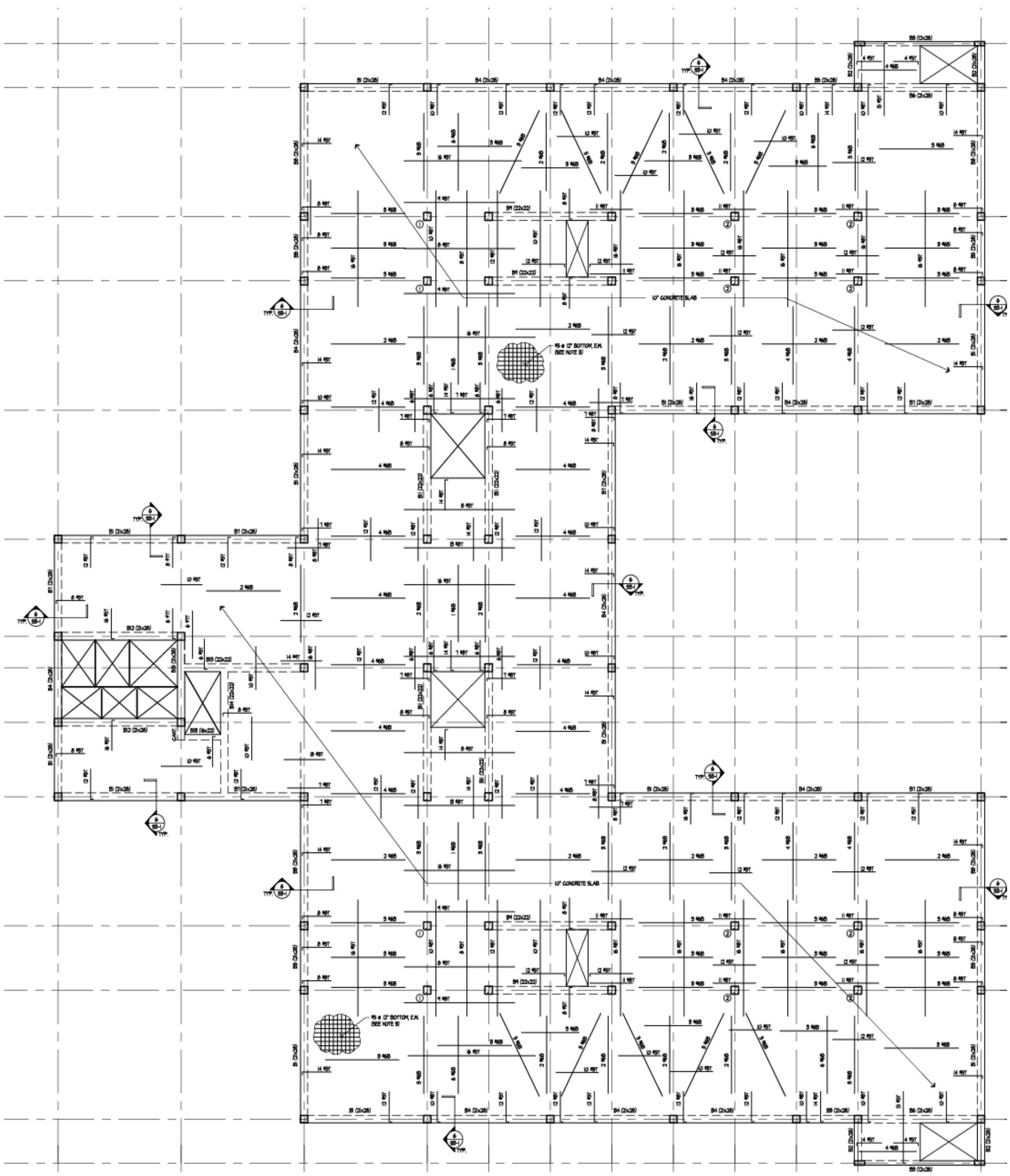
Ground Level



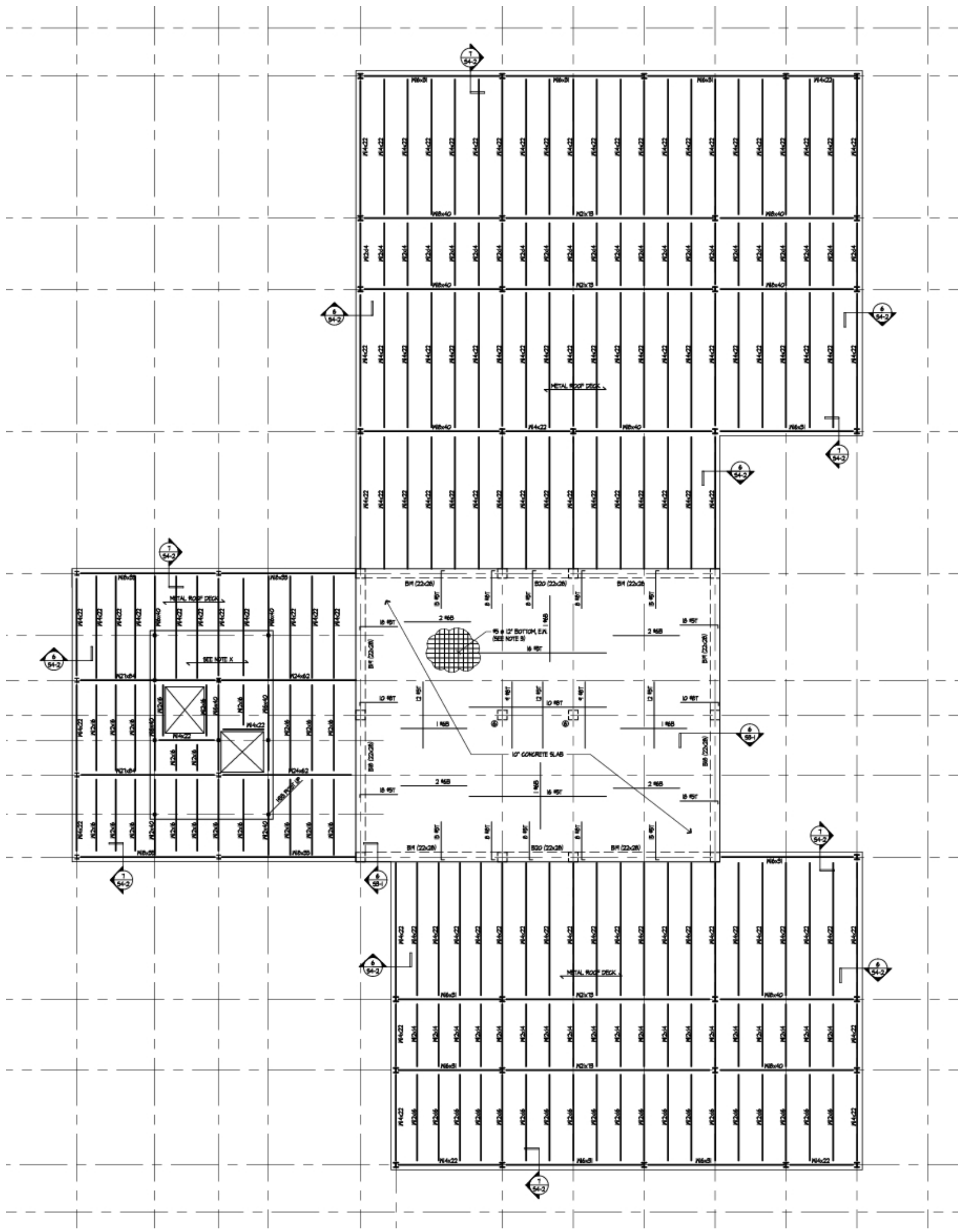
Level 1



Level 2 (Level 3 similar)



Level 4-7 (all similar)



Roof Level

Appendix B: Seismic Load and Strength Hand Calculations

SEISMIC LOAD CALCULATION - MOMENT FRAME SYSTEM PT SLAB

DETERMINE SEISMIC GROUND MOTION VALUES
FROM USGS EARTHQUAKE HAZARDS PROGRAM: LATITUDE: 37.7955
LONGITUDE: -122.4088

SITE CLASS B
$S_S = 1.500$ g
$S_1 = 0.620$ g

TABLE 11.4-1: $F_a = 1.0$
TABLE 11.4-2: $F_v = 1.0$

$S_{MS} = F_a S_S = 1.0(1.5) = 1.5$
$S_{M1} = F_v S_1 = 1.0(0.620) = 0.620$

$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(1.5) = 1.0$
$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(0.62) = 0.413$

DETERMINE SDC
OCCUPANCY CATEGORY IV
TABLE 11.6-1: SDC D
TABLE 11.6-2: SDC D $\rightarrow$ SDC D

PERMITTED ANALYTICAL PROCEDURES
$T_S = S_{D1}/S_{DS} = 0.413/1.0 = 0.413$ $\qquad T = 1.4206 < 3.5T_S = 1.446$ s

HORIZONTAL IRREGULARITIES TYPE 2 + 3 $\Rightarrow$ EQUIVALENT LATERAL FORCE PROCEDURE

EQUIVALENT LATERAL FORCE PROCEDURE
SPECIAL REINFORCED CONCRETE MOMENT FRAMES: $R = 8$, $\Omega = 3$, $C_d = 5.5$
IMPORTANCE FACTOR: $I = 1.5$

$T_a = C_t h_n^x$ $\quad C_t = 0.016$, $x = 0.9$, $h_n = 105$ ft $\quad T_a = (0.016)(105)^{0.9} = 1.0548$

$C_u T_a = 1.4(1.0548) = 1.4767$ $\quad T = 1.4206 < C_u T_a = 1.4767$ ∴ USE $T = 1.4206$

$T_L = 12$

$$C_s = \min \begin{cases} \dfrac{S_{DS}}{(R/I)} = \dfrac{1.0}{(8/1.5)} = 0.1875 \\[2ex] \dfrac{S_{D1}}{T(R/I)} = \dfrac{0.413}{1.4206(8/1.5)} = 0.05451 \\[2ex] \dfrac{S_{D1} T_L}{T^2(R/I)} = \dfrac{0.413(12)}{(1.4206)^2(8/1.5)} = 0.4605 \\[2ex] \dfrac{0.5 S_1}{(R/I)} = \dfrac{0.5(0.620)}{(8/1.5)} = \underline{0.0581} \leftarrow \text{GOVERNS} \end{cases}$$

$V_b = C_s W = 0.0581(58{,}279) = 3{,}386$ KIPS $\qquad A_x = 1.0$, $\rho = 1.0$

$k = 0.75 + 0.5T = 0.75 + 0.5(1.4206) = 1.4603$ $\qquad F_x = \dfrac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} V \rightarrow$ SEE SPREADSHEET

TORSIONAL AMPLIFICATION FACTOR

Y-DIRECTION LOADING

1.0E1: $\delta_{MAX} = 2.363968$

$$\delta_{AVG} = \frac{2.363968 + 2.025025}{2} = 2.1944965$$

$$A_x = \left[\frac{2.363968}{1.2(2.1944965)}\right]^2 = 0.805844 \rightarrow A_x = 1.0$$

1.0E2: $\delta_{MAX} = 2.170715$

$$\delta_{AVG} = \frac{2.0668 + 2.170715}{2} = 2.1187575$$

$$A_x = \left[\frac{2.170715}{1.2(2.1187575)}\right]^2 = 0.728921 \rightarrow A_x = 1.0$$

X-DIRECTION LOADING

1.0E3: $\delta_{MAX} = 2.214299$

$$\delta_{AVG} = \frac{2.214299 + 1.76619}{2} = 1.9902445$$

$$A_x = \left[\frac{2.214299}{1.2(1.9902445)}\right]^2 = 0.859602 \rightarrow A_x = 1.0$$

1.0E4: $\delta_{MAX} = 2.138661$

$$\delta_{AVG} = \frac{1.82368 + 2.138661}{2} = 1.9811705$$

$$A_x = \left[\frac{2.138661}{1.2(1.9811705)}\right]^2 = 0.809244 \rightarrow A_x = 1.0$$

CHECK TORSIONAL IRREGULARITIES

1.0E1: $\frac{\delta_{MAX}}{\delta_{AVG}} = 1.077226 < 1.2$ OK ✓

1.0E2: $\frac{\delta_{MAX}}{\delta_{AVG}} = 1.024523 < 1.2$ OK ✓

1.0E3: $\frac{\delta_{MAX}}{\delta_{AVG}} = 1.112576 < 1.2$ OK ✓

1.0E4: $\frac{\delta_{MAX}}{\delta_{AVG}} = 1.079494 < 1.2$ OK ✓

CHECK MAXIMUM ROOF DISPL

Y-DIRECTION LOADING: $\delta_y = \frac{C_d \delta_{ye}}{I} = \frac{5.5(2.363968)}{1.5} = 8.67'' < 10.44''$ OK ✓

X-DIRECTION LOADING: $\delta_x = \frac{5.5(2.214299)}{1.5} = 8.12'' < 10.44''$ OK ✓

SEISMIC LOAD CALCULATION – SHEAR WALL SYSTEM PT SLAB

DETERMINE SEISMIC GROUND MOTION VALUES

FROM USGS EARTHQUAKE HAZARDS PROGRAM: LATITUDE: 37.7955
LONGITUDE: −122.4058

SITE CLASS B

$S_S = 1.500g$

$S_1 = 0.620g$

TABLE 11.4-1: $F_a = 1.0$

TABLE 11.4-2: $F_v = 1.0$

$S_{DS} = \frac{2}{3} F_a S_S = \frac{2}{3}(1.0)(1.5) = 1.0$

$S_{D1} = \frac{2}{3} F_v S_1 = \frac{2}{3}(1.0)(0.62) = 0.413$

DETERMINE SDC

OCCUPANCY CATEGORY IV

TABLE 11.6-1: SDC D

TABLE 11.6-2: SDC D

$\Rightarrow$ SDC D

PERMITTED ANALYTICAL PROCEDURES

$T_S = S_{D1}/S_{DS} = 0.413/1.0 = 0.413$ $\quad T = 1.3101 < 3.5T_S = 1.446s$

HORIZONTAL IRREGULARITIES TYPE 2 + 3 $\Rightarrow$ EQUIVALENT LATERAL FORCE PROCEDURE

EQUIVALENT LATERAL FORCE PROCEDURE

SPECIAL REINFORCED CONCRETE SHEAR WALLS: $R = 6$, $\Omega = 2.5$, $C_d = 5$

IMPORTANCE FACTOR: $I = 1.5$

$T_a = C_t h_n^x$ $\quad C_t = 0.02$, $x = 0.75$, $h_n = 105$ ft $\quad T_a = (0.02)(105)^{0.75} = 0.656$

$C_u T_a = 1.4(0.656) = 0.9184$ $\quad T = 1.3101 > C_u T_a \therefore$ USE $\underline{T = 0.9184}$

$T_L = 12$ $\quad C_S = \text{MIN}\begin{cases} \dfrac{S_{DS}}{(R/I)} = \dfrac{1.0}{(6/1.5)} = 0.25 \\ \dfrac{S_{D1}}{T(R/I)} = \dfrac{0.413}{0.9184(6/1.5)} = 0.1124 \\ \dfrac{S_{D1}T_L}{T^2(R/I)} = \dfrac{0.413(12)}{(0.9184)^2(6/1.5)} = 1.469 \\ \dfrac{0.5S_1}{(R/I)} = \dfrac{0.5(0.620)}{(6/1.5)} = \underline{0.0775} \leftarrow \text{GOVERNS} \end{cases}$

$V_b = C_S W = 0.0775(51209) = 3969$ kips

$k = 0.75 + 0.5T = 0.75 + 0.5(0.9184) = 1.2092$ $\quad F_x = \dfrac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k} V \rightarrow$ SEE SPREADSHEET

$A_x = 1.0$, $\rho = 1.0$

TORSIONAL AMPLIFICATION FACTOR

Y-DIRECTION LOADING

1.0E1:

$$\delta_{MAX} = 0.723914$$

$$\delta_{AVG} = \frac{0.723914 + 0.670749}{2} = 0.6973305$$

$$A_x = \left[\frac{0.723914}{1.2(0.6973305)}\right]^2 = 0.748401 \rightarrow A_x = 1.0$$

1.0E2:

$$\delta_{MAX} = 0.819702$$

$$\delta_{AVG} = \frac{0.819702 + 0.574096}{2} = 0.696899$$

$$A_x = \left[\frac{0.819702}{1.2(0.696899)}\right]^2 = 0.960749 \rightarrow A_x = 1.0$$

X-DIRECTION LOADING

1.0E3:

$$\delta_{MAX} = 3.037324$$

$$\delta_{AVG} = \frac{3.037324 + 2.117864}{2} = 2.577594$$

$$A_x = \left[\frac{3.037324}{1.2(2.577594)}\right]^2 = 0.964252 \rightarrow A_x = 1.0$$

1.0E4:

$$\delta_{MAX} = 3.060055$$

$$\delta_{AVG} = \frac{2.132499 + 3.060055}{2} = 2.596277$$

$$A_x = \left[\frac{3.060055}{1.2(2.596277)}\right]^2 = 0.964704 \rightarrow A_x = 1.0$$

CHECK TORSIONAL IRREGULARITIES

1.0E1: $\frac{\delta_{MAX}}{\delta_{AVG}} = 1.038122 < 1.2$ OK ✓

1.0E2: $\frac{\delta_{MAX}}{\delta_{AVG}} = 1.176213 < 1.2$ OK ✓

1.0E3: $\frac{\delta_{MAX}}{\delta_{AVG}} = 1.178356 < 1.2$ OK ✓

1.0E4: $\frac{\delta_{MAX}}{\delta_{AVG}} = 1.178632 < 1.2$ OK ✓

CHECK MAXIMUM ROOF DISPL

Y-DIRECTION LOADING: $\delta_y = \frac{C_d \, \delta_{ye}}{I} = \frac{5(0.819702)}{1.5}\left(\frac{0.9184}{0.6816}\right) = 3.25'' < 10.44''$ OK ✓

$0.010 h_{sx}$

X-DIRECTION LOADING: $\delta_x = \frac{5(3.060055)}{1.5}\left(\frac{0.9184}{1.3101}\right) = 7.150'' < 10.44''$ OK ✓

SPECIAL REINFORCED CONCRETE MOMENT FRAME BEAM DESIGN

BEAM WIDTH: 34" BEAM HEIGHT: 36"

$f'_c = 5000$ PSI $f_y = 60{,}000$ PSI

DESIGN MOMENT: ± 1460 ft-k

ASSUME $d = 36'' - 1.5'' - \underbrace{0.625}_{\#5\ \text{STIRRUP}} - \underbrace{\frac{1.128''}{2}}_{\#9\ \text{BAR}} = 33.31''$

$$A_{s\,req'd} = \frac{(1460\ \text{ft-k})(12000)}{0.9(60{,}000)(33.31)} = 9.74\ in^2$$

TRY (9) #9 BARS $A_s = 10.15\ in^2$

$$a = \frac{10.15(60{,}000)}{0.85(5000)(34)} = 4.21'' \qquad c = \frac{a}{B_1} = 4.96''$$

$$\varepsilon_y = \frac{60}{29000} = 0.00207$$

$$\varepsilon_s = \left(\frac{34.936 - 4.96}{4.96}\right)0.003 = 0.0181 > 0.005 \therefore \phi = 0.9 \quad \underline{OK}\checkmark$$

$\boxed{\text{USE (9) \#9 BARS TOP AND BOTTOM FOR FLEXURE}}$ $\phi M_n = 1521'^k > M_u = 1460'^k$

DESIGN SHEAR: 190k

$$V_c = 2\sqrt{5000}\,(34)(33.31)/1000 = 160\ k$$

$$\phi V_n = 0.5\phi V_c = 0.5(0.75)(160) = 60\ K \qquad \text{NEED SHEAR REINF}$$

$$V_s = \frac{190}{0.75} - 160 = 93^k < 8\sqrt{f'_c}\,b_w d = 641\ k$$

$$4\sqrt{f'_c}\,b_w d = 320 \qquad V_s < 320 \therefore \quad S_{MAX} = \min\begin{cases} d/2 = 17.468'' \rightarrow \text{USE } 16'' \\ 24'' \end{cases}$$

$$A_{v\,min} = \text{MAX}\begin{cases} 0.75\sqrt{5000}\,(34)(16)/60{,}000 = 0.48 \\ 50(34)(16)/60{,}000 = 0.453 \end{cases}$$

USE #5 STIRRUPS @ 16" AS MINIMUM SHEAR REINFORCEMENT

$(A_v = 2 \times 0.31 = 0.62\ in^2 > 0.48\ in^2)$

$$S = (0.62)(60)(33.31)/93 = 13.32''$$

$$\phi V_n = \phi V_c + \phi V_s = 0.75\left(160 + \frac{0.62(60)(33.31)}{12}\right) = 197\ K$$

$\boxed{\text{USE \#5 STIRRUPS @ 12'' WHOLE LENGTH}}$ $\phi V_n = 197^k > V_u = 190^k$

OVERTURNING CALCULATIONS

MOMENT FRAME SYSTEM:

OVERTURNING MOMENT = 232,262 ftk
BUILDING WEIGHT = 58,279 k
MOMENT FRAME SPAN = 165 ft

LOAD COMBINATION = 0.9D + 1.0E

$P_u = 0.9(58,279) = 52,451\ k$

$M_u = 232,262\ ftk$

AVERAGE MOMENT FRAME = 165 ft IN SHORT DIRECTION

$P_u/2 = 26,226\ k$ (FACTOR OF SAFETY OF 2)

$M_u/165' = 1408\ k$

$P_u/2 > M_u/165'$ OK ✓

NO CONCERNS REGARDING UPLIFT

SHEAR WALL SYSTEM:

OVERTURNING MOMENT = 266,179 ftk
BUILDING WEIGHT = 51,209 k
LINEAR FEET OF SHEAR WALL IN SHORT DIR: 60 ft

LOAD COMBINATION = 0.9D + 1.0E

$P_u = 0.9(51,209) = 46,088\ k$

$M_u = 266,179\ ftk$

$P_u/2 = 23,044\ k$ (FACTOR OF SAFETY OF 2)

$M_u/60' = 4,436\ k$

$P_u/2 > M_u/60'$ OK ✓

Max Shear By Section	Y-Direction Core (kips)	Y-Direction South (kips)	Y-Direction North *(kips)*	X-Direction Core South *(kips)*	X-Direction Core North *(kips)*	X-Direction South *(kips)*	X-Direction North *(kips)*
Vu Level G-3	3717	536	522	1193	1191	1031	1009

Max Moment By Section	Y-Direction Core *(ft-kips)*	Y-Direction South (ft-kips)	Y-Direction North *(ft-kips)*	X-Direction Core South (ft-kips)	X-Direction Core North *(ft-kips)*	X-Direction South *(ft-kips)*	X-Direction North (ft-kips)
Mu Level G-3	56791	7002	7002	73859	74074	35711	35987

Shear Reinforcement							
Thickness of Wall (in)	22	12	12	22	22	22	22
Length of Wall (ft)	30	15	15	20	20	15	15
Height of Wall (ft)	50	50	50	50	50	50	50
f'c (psi)	7000	7000	7000	7000	7000	7000	7000
d (in)	288	144	144	192	192	144	144
ϕVn Max (kips)	3976	1084	1084	2651	2651	1988	1988
a	11.0	6.0	6.0	10.0	10.0	7.5	7.5
Vc (kips) (11.9.5)	1060	289	289	707	707	530	530
Vc (kips) (11.9.6)	1749	477	477	1166	1166	875	875
Vc (kips) (11.9.6)	71955	575	544	382	381	342	335
Vc (kips)	1060	289	289	382	381	342	335
ϕVc (kips)	795	217	217	287	286	257	252
0.5ϕVc (kips)	398	108	108	143	143	128	126
Vs Reqd (kips)	4161	498	479	1304	1302	1119	1094
Av/s (in2)	0.2408	0.0577	0.0554	0.1132	0.1130	0.1295	0.1266
Bar Size	(2) #11	(2) #6	(2) #6	(2) #8	(2) #8	(2) #8	(2) #8
s Max (Horiz) (in)	12.9560	15.2615	15.8866	13.9530	13.9826	12.2047	12.4831
s Provided (in)	12	14	14	12	12	12	12
ρt	0.0096	0.0052	0.0052	0.0060	0.0060	0.0060	0.0060
s Max Code (Horiz) (in)	18	18	18	18	18	18	18
Horizontal Design	**(2) #11 @ 12"**	**(2) #6 @ 14"**	**(2) #6 @ 14"**	**(2) #8 @ 12"**	**(2) #8 @ 12"**	**(2) #8 @ 12"**	**(2) #8 @ 12"**
ρl	0.0055	0.0025	0.0025	0.0025	0.0025	0.0025	0.0025
Bar Size (in2)	(2) #8	(2) #4	(2) #4	(2) #6	(2) #6	(2) #6	(2) #6
s Max (Vert) (in)	13.1363	13.3333	13.3333	16.0000	16.0000	16.0000	16.0000
s Max Code (Vert) (in)	18	18	18	18	18	18	18
s Provided (in)	12	12	12	16	16	16	16
Vertical Design	**(2) #8 @ 12"**	**(2) #4 @ 12"**	**(2) #4 @ 12"**	**(2) #6 @ 16"**	**(2) #6 @ 16"**	**(2) #6 @ 16"**	**(2) #6 @ 16"**

Flexure Reinforcement							
0.2f'c	1.4	1.4	1.4	1.4	1.4	1.4	1.4
fc	1.4	1.3	1.3	4.2	4.2	3.6	3.6
Boundary Element Needed	No	No	No	Yes	Yes	Yes	Yes
I Required (ft^4)	-	-	-	1831.82	1837.16	664.27	669.41
I Provided (ft^4)	-	-	-	1902.78	1902.78	678.09	678.09
Boundary Element Size	-	-	-	**36x60**	**36x60**	**30x44**	**30x44**
jd	259	130	130	173	173	130	130
As Reqd (1st Iteration)	48.69	12.01	12.01	94.98	95.26	61.23	61.71
a	22.32	10.09	10.09	43.54	43.66	28.07	28.28
jd	277	139	139	170	170	130	130
As Reqd (2nd Iteration)	45.59	11.20	11.20	96.42	96.73	61.06	61.58
Bar Size	(30) #11	(10) #11	(10) #11	(62) #11	(64) #11	(40) #11	(40) #11
As Provided (1st Iteration)	46.80	15.60	15.60	96.72	99.84	62.40	62.40
d (1st Iteration)	343.00	173.00	173.00	207.00	206.00	158.00	158.00
As Reqd (in2) (3rd Iteration)	36.79	8.99	8.99	79.29	79.91	50.23	50.62
Bar Size	(24) #11	(6) #11	(6) #11	(52) #11	(52) #11	(34) #11	(34) #11
As Provided (2nd Iteration)	40.56	9.36	9.36	90.48	93.60	56.16	59.28
d (2nd Iteration)	346.00	175.00	175.00	212.00	212.00	161.00	161.00
dt	357.00	177.00	177.00	237.00	237.00	177.00	177.00
a	18.59	7.87	7.87	41.47	42.90	25.74	27.17
c	21.87	9.25	9.25	48.79	50.47	30.28	31.97
εt	0.0460	0.0544	0.0544	0.0116	0.0111	0.0145	0.0136
Flexure Design (Each End)	**(24) #11**	**(6) #11**	**(6) #11**	**(52) #11**	**(52) #11**	**(34) #11**	**(34) #11**

Max Shear By Section	Y-Direction Core (kips)	Y-Direction South (kips)	Y-Direction North (kips)	X-Direction Core South (kips)	X-Direction Core North (kips)	X-Direction South (kips)	X-Direction North (kips)
Vu Level 4-7	2788	454	447	739	695	749	768

Max Moment By Section	Y-Direction Core (ft-kips)	Y-Direction South (ft-kips)	Y-Direction North (ft-kips)	X-Direction Core South (ft-kips)	X-Direction Core North (ft-kips)	X-Direction South (ft-kips)	X-Direction North (ft-kips)
Mu Level 4-7	19168	1198	1194	22032	22084	12168	15424

Thickness of Wall (in)	22	12	12	22	22	22	22
Length of Wall (ft)	30	15	15	20	20	15	15
Height of Wall (ft)	55	55	55	55	55	55	55
f'c (psi)	7000	7000	7000	7000	7000	7000	7000
d (in)	288	144	144	192	192	144	144
ϕVn Max (kips)	3976	1084	1084	2651	2651	1988	1988
a	11.0	6.0	6.0	10.0	10.0	7.5	7.5
Vc (kips) (11.9.5)	1060	289	289	707	707	530	530
Vc (kips) (11.9.6)	1749	477	477	1166	1166	875	875
Vc (kips) (11.9.6)	-	-	-	-	618	727	554
Vc (kips)	1060	289	289	707	618	530	530
ϕVc (kips)	795	217	217	530	463	398	398
0.5ϕVc (kips)	398	108	108	265	232	199	199
Vs Reqd (kips)	2922	389	380	455	463	601	626
Av/s (in2)	0.1691	0.0450	0.0440	0.0395	0.0402	0.0695	0.0725
Bar Size (in2)	(2) #10	(2) #6	(2) #6	(2) #8	(2) #8	(2) #6	(2) #6
s Max (Strength) (in)	15.0189	19.5623	20.0222	40.0306	39.2784	12.6535	12.1430
s (in)	14	18	18	18	18	12	12
ρt	0.0082	0.0041	0.0041	0.0040	0.0040	0.0060	0.0060
s Max (Code) (in)	18	18	18	18	18	18	18
Horizontal Design	**(2) #10 @ 14"**	**(2) #6 @ 18"**	**(2) #6 @ 18"**	**(2) #8 @ 18"**	**(2) #8 @ 18"**	**(2) #6 @ 12"**	**(2) #6 @ 12"**
ρl	0.0044	0.0025	0.0025	0.0025	0.0025	0.0025	0.0025
Bar Size (in2)	(2) #8	(2) #4	(2) #4	(2) #6	(2) #6	(2) #6	(2) #6
s Max (Vert) (in)	16.2647	13.3333	13.3333	16.0000	16.0000	16.0000	16.0000
s Max Code (Vert) (in)	18	18	18	18	18	18	18
s Provided (in)	16	12	12	16	16	16	16
Vertical Design	**(2) #8 @ 16"**	**(2) #4 @ 12"**	**(2) #4 @ 12"**	**(2) #6 @ 16"**	**(2) #6 @ 16"**	**(2) #6 @ 16"**	**(2) #6 @ 16"**
Flexure Reinforcement							
0.2f'c	1.40	1.40	1.40	1.40	1.40	1.40	1.40
fc	0.48	0.22	0.22	1.25	1.25	1.23	1.56
Boundary Element Needed	No	No	No	No	No	No	Yes
I Required	-	-	-	-	-	-	286.91
I Provided (ft^4)	-	-	-	-	-	-	584.28
Boundary Element Size	-	-	-	-	-	**26X32**	**26X32**
jd	259	130	130	173	173	130	130
As Reqd (1st Iteration)	16.43	2.05	2.05	28.33	28.40	20.86	26.45
a	7.53	1.73	1.72	12.99	13.02	9.56	12.12
jd	284	143	143	186	185	139	138
As Reqd (2nd Iteration)	14.99	1.86	1.85	26.39	26.46	19.42	24.85
Bar Size	(10) #11	(2) #11	(2) #11	(18) #11	(18) #11	(14) #11	(16) #11
As Provided (1st Iteration)	15.60	3.12	3.12	28.08	28.08	21.84	24.96
d (1st Iteration)	353.00	177.00	177.00	229.00	229.00	171.00	170.00
As Reqd (in2) (3rd Iteration)	12.07	1.50	1.50	21.38	21.43	15.81	20.16
Bar Size	(8) #11	(1) #11	(1) #11	(14) #11	(14) #11	(12) #11	(14) #11
As Provided (2nd Iteration)	12.48	1.56	1.56	21.84	21.84	18.72	21.84
d (2nd Iteration)	354.00	177.50	177.50	231.00	231.00	172.00	171.00
dt	357.00	177.00	177.00	237.00	237.00	177.00	177.00
a	5.72	1.31	1.31	10.01	10.01	8.58	10.01
c	6.73	1.54	1.54	11.78	11.78	10.09	11.78
εt	0.1561	0.3413	0.3413	0.0574	0.0574	0.0496	0.0421
Flexure Design (Each End)	**(8) #11**	**(1) #11**	**(1) #11**	**(14) #11**	**(14) #11**	**(12) #11**	**(14) #11**

Appendix C: Post-Tensioned Floor System Tendon Drape Spreadsheets

NORTH-SOUTH Uniform Tendons

Grid Line E-E.5	Span 1	Span 2	Span 3	Span 4	Span 5	Span 6	Span 7	Span 8	Span 9
Number of Strands	15	15	15	15	15	15	15	15	15
PT Force per Unit	25.9	25.9	25.9	25.9	25.9	25.9	25.9	25.9	25.9
PT Force	389.2	389.2	389.2	389.2	389.2	389.2	389.2	389.2	389.2
P/A	270	270	270	270	270	270	270	270	270
%DL Balanced	60	95	96	96	96	96	96	95	60
Control Point Left	4	7	7	7	7	7	7	7	7
Control Point Center	1.75	5.5	1	1	1	1	1	5.5	1.75
Control Point Right	7	7	7	7	7	7	7	7	4

Grid Line E.5-F & G-G.5	Span 1	Span 2	Span 3	Span 4	Span 5	Span 6	Span 7	Span 8	Span 9
Number of Strands	15	15	15	15	15	15	15	15	15
PT Force per Unit	25.9	25.9	25.9	25.9	25.9	25.9	25.9	25.9	25.9
PT Force	389.2	389.2	389.2	389.2	389.2	389.2	389.2	389.2	389.2
P/A	270	270	270	270	270	270	270	270	270
%DL Balanced	60	95	95	95	95	95	95	95	60
Control Point Left	4	7	7	7	7	7	7	7	7
Control Point Center	1.75	5.5	1	1	1	1	1	5.5	1.75
Control Point Right	7	7	7	7	7	7	7	7	4

Grid Line F-G P.1	Span 1	Span 2	Span 3	Span 4
Number of Strands	15	15	15	15
PT Force per Unit	25.9	25.9	25.9	25.9
PT Force	389.2	389.2	389.2	389.2
P/A	270	270	270	270
%DL Balanced	60	95	96	96
Control Point Left	4	7	7	7
Control Point Center	1.75	5.5	1	4
Control Point Right	7	7	7	4

Grid Line F-G P.2					**Span 5**	**Span 6**
Number of Strands					15	15
PT Force per Unit					25.9	25.9
PT Force					389.2	389.2
P/A					270	270
%DL Balanced					72	96
Control Point Left					4	7
Control Point Center					1	4
Control Point Right					7	4

Grid Line F-G P.3							**Span 7**	**Span 8**	**Span 9**
Number of Strands							15	15	15
PT Force per Unit							25.9	25.9	25.9
PT Force							389.2	389.2	389.2
P/A							270	270	270
%DL Balanced							72	95	60
Control Point Left							4	7	7
Control Point Center							1	5.5	1.75
Control Point Right							7	7	4

Grid Line G.5-H	**Span 1**	**Span 2**	**Span 3**	**Span 4**	**Span 5**	**Span 6**	**Span 7**	**Span 8**	**Span 9**
Number of Strands	15	15	15	15	15	15	15	15	15
PT Force per Unit	25.9	25.9	25.9	25.9	25.9	25.9	25.9	25.9	25.9
PT Force	389.2	389.2	389.2	389.2	389.2	389.2	389.2	389.2	389.2
P/A	270	270	270	270	270	270	270	270	270
%DL Balanced	60	95	96	96	96	96	96	95	60
Control Point Left	4	7	7	7	7	7	7	7	7
Control Point Center	1.75	5.5	1	1	1	1	1	5.5	1.75
Control Point Right	7	7	7	7	7	7	7	7	4

Grid Line H-K & K.5-L	**Span 1**	**Span 2**	**Span 3**
Number of Strands	30	30	30
PT Force per Unit	25.9	25.9	25.9
PT Force	778.4	778.4	778.4
P/A	270	270	270
%DL Balanced	60	95	60
Control Point Left	4	7	7
Control Point Center	1.75	5.5	1.75
Control Point Right	7	7	4

Grid Line K-K.5	Span 1	Span 2	Span 3	Span 4
Number of Strands	15	15	15	15
PT Force per Unit	25.9	25.9	25.9	25.9
PT Force	389.2	389.2	389.2	389.2
P/A	270	270	270	270
%DL Balanced	60	95	60	69
Control Point Left	4	7	7	4
Control Point Center	1.75	5.5	1.75	3.5
Control Point Right	7	7	4	4

EAST-WEST Banded Tendons

Grid Line 2 & 12.4	Span 1	Span 2	Span 3	Span 4	Span 5	Span 6	Span 7	Span 8
Number of Strands								11
PT Force per Unit								28.8
PT Force								293
P/A								300
%DL Balanced								89
Control Point Left								4
Control Point Center								2.75
Control Point Right								4

Grid Line 3 & 12	Span 1	Span 2	Span 3	Span 4	Span 5	Span 6	Span 7	Span 8
Number of Strands			17	17	17	17	17	17
PT Force per Unit			28.8	28.8	28.8	28.8	28.8	28.8
PT Force			432	432	432	432	432	432
P/A			300	300	300	300	300	300
%DL Balanced			67	93	93	93	89	54
Control Point Left			4	7	7	7	7	7
Control Point Center			1.75	1.75	1.75	1.75	5.75	1.75
Control Point Right			7	7	7	7	7	4

Grid Line 4, 5, 10, 11	Span 1	Span 2	Span 3	Span 4	Span 5	Span 6	Span 7	Span 8
Number of Strands			25	25	25	25	25	25
PT Force per Unit			28.8	28.8	28.8	28.8	28.8	28.8
PT Force			648	648	648	648	648	648
P/A			300	300	300	300	300	300
%DL Balanced			66	88	93	93	93	66
Control Point Left			4	7	7	7	7	7
Control Point Center			1.75	5.75	1.75	1.75	1.75	1.75
Control Point Right			7	7	7	7	7	4

Grid Line 6	Span 1	Span 2	Span 3	Span 4	Span 5	Span 6	Span 7	Span 8
Number of Strands			33	33	33	17	17	17
PT Force per Unit			28.8	28.8	28.8	28.8	28.8	28.8
PT Force			864(2*432)	864(2*432)	864(2*432)	432	432	432
P/A			300	300	300	300	300	300
%DL Balanced			66	88	93	93	93	66
Control Point Left			4	7	7	7	7	7
Control Point Center			1.75	5.75	1.75	1.75	1.75	1.75
Control Point Right			7	7	7	7	7	4

Grid Line 7 & 8	Span 1	Span 2	Span 3	Span 4	Span 5	Span 6	Span 7	Span 8
Number of Strands			33	33	33			
PT Force per Unit			28.8	28.8	28.8			
PT Force			864	864	864			
P/A			300	300	300			
%DL Balanced			66	88	66			
Control Point Left			4	7	7			
Control Point Center			1.75	5.75	1.75			
Control Point Right			7	7	4			

Grid Line 9	Span 1	Span 2	Span 3	Span 4	Span 5	Span 6	Span 7	Span 8
Number of Strands			33	33	33	17	17	17
PT Force per Unit			28.8	28.8	28.8	28.8	28.8	28.8
PT Force			864(2*432)	864(2*432)	864(2*432)	432	432	432
P/A			300	300	300	300	300	300
%DL Balanced			66	88	93	93	93	66
Control Point Left			4	7	7	7	7	7
Control Point Center			1.75	5.75	1.75	1.75	1.75	1.75
Control Point Right			7	7	7	7	7	4