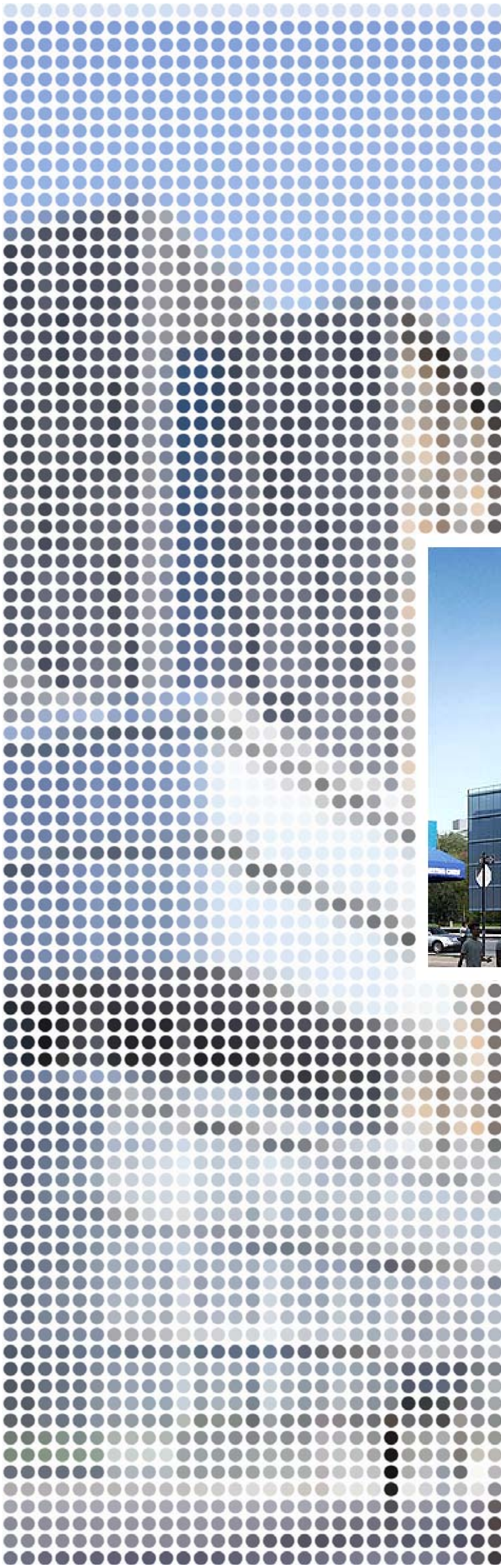

Gouverneur Healthcare Services

New York, NY

Final Report

Scott Rabold

Structural

Consultant: Dr. Ali Memari

4.7.2008

GOUVERNEUR HEALTHCARE SERVICES

NEW YORK, NY

structural SCOTT M. RABOLD

General Building Data

Name: Gouverneur Healthcare Services
Location: 227 Madison St. New York, NY 10002
Building Occupant: NYC HHC - Gouverneur
Function Type: Healthcare
Size: 75,000 sq. ft. addition / 275,000 sq. ft. renovation
Number of Stories: 5 and 13 stories

Primary Project Team

Owner: NYC Health and Hospitals Corporation
New York, NY
CM: Hunter-Roberts Construction Group
New York, NY
Architect: RMJM - Hillier
New York, NY
Structural Engineer Greenman-Pedersen
Scranton, PA
MEP Engineer: AKF Engineers
New York, NY

Architecture

13 story existing building, built approximately 35 years ago

Two components to the addition:
- -5 story ambulatory care facility
- -Expansion to longterm care facility

Facade of existing building is brick with punch widnows

Facade of new addition is glass curtainwall and Glass Fiber Reinforced Concrete Panels (GFRC)

Concrete roof on metal deck with insulation panels and rubberized-asphalt waterproofing membrane

Structural

Two-way concrete floor slab in existing building

Non-composite steel framing in new addition
- -Nearly all gravity members are castilated beams due to a small floor height

Moment-frame lateral system in 5 story ambulatory center

Braced- frame lateral system in 8 story addition (floors 6-13)

Mini-pile foundation w/ grade beams

MEP Systems

Variable Air and Constant Air AHU's w/ dedicated units for atrium and dialysis center

1250kW Emeregency Generator in penthouse

480 / 277V 3 Phase 4 Wire System

208 / 120V 3 Phase 4 Wire System

Indirect fluorescent lighting in office spaces

Direct lighting in emergency care facilities

http://www.engr.psu.edu/ae/thesis/portfolios/2009/smr331

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Executive Summary

The Gouverneur Healthcare Facility is a 75,000 sq. ft. addition to an existing, 13 story hospital. The addition is comprised of two portions; the lower five floors contain an ambulatory care center, while the upper floors are an expansion to the current long-term care residences. The existing hospital is a cast-in-place concrete structure with a tight, 11' floor-to-floor height. Due to the constraints imposed by this height, all gravity members of the addition's structure are cellular beams. Furthermore, moment frames comprise the majority of the lateral load resisting system. In order to save cost, and increase the ease of design and construction for all trades, a concrete structural system has been proposed to replace the existing steel frame design.

The proposed design utilizes a two-way flat plate floor system. The flat plate construction allows for unobstructed space between the ceiling and the slab above. This will allow significant freedom of design for all other systems, something that was lacking in the original design.

The slab is 12" thick and has a compressive strength, f'_c, of 6000psi. Typical bays for the lower portion are 22'x24' with 16" square columns. Columns supporting the upper portion of the building are 20" square. Deflections are typically limited to 0.80" (L/360) for immediate deflections, and 0.60" (L/480) for long term deflections in areas where large deflections would damage non-structural elements.

The column layout is shifted for the upper floors in order to control deflection and coordinate with the room layouts of the long-term care dormitories. At the 6th floor, a 60" transfer beam is designed to transmit the load of the shifted, upper column to the typical columns below. Otherwise the shifted column would extend through the center of the 4 story atrium in the lower portion of the building.

The lateral load resisting system is comprised of six shearwalls in total, three in each orthogonal direction, with an f'_c = 6000 psi. Shearwalls that extend from the foundation to the lower roof on the 6th floor are 16" thick to match the adjacent columns. Shearwalls that extend the full height of the building are 20" thick. Coupling beams that adjoin two piers of a shearwall are 36" deep and match the thickness of the shearwalls.

Wind and seismic loading were investigated in order to design the LFRS for strength and drift requirements per code, but also to limit deflections of the floors to half the distance between the existing and proposed structures. This design choice was made in order to conservatively limit the overall deflection of the structure to an "upper limit" that is an attempt to prevent damage to the structures during wind and, more critically, seismic events.

Although wind loading created the highest design forces in the shearwalls, the design of the LFRS was governed by seismic loading in order to meet the upper limit requirement for deflection. Therefore, the size and location of shearwall were designed to limit seismic deflections while also coordinating as best as possible with the room layouts of each floor.

Because the change in structural system created a potentially large impact on the room layout of the Gouverneur Healthcare Facility, an architectural investigation was conducted. Floors were redesigned to account for the added columns and the addition of shearwalls. Special care was taken to retain the same functional relationship between rooms. For example, spaces like the Sterile Prep Room and the Operation Procedure Rooms were kept adjacent. Furthermore, the long-term care residences for the upper floors were designed based off the layout of the rooms in the existing building.

In order to have a complete comparison of the viability of the proposed structure, a cost and schedule impact study was conducted. It was determined that the proposed structure saved $570,000 of the cost of the structure. However, when considering the total cost of the project, this savings is less than 1% saving. The proposed structure will also take longer to construct, with estimated construction time of 12 months, compared to 6.5 months for the existing addition. Because the Gouverneur Healthcare Facility is a hospital owned by the NYC HHC, immediate revenue generation is not an issue, and the longer estimated schedule time does not negatively affect the feasibility of the proposed design.

ACKNOWLEDGEMENTS

I'd like to thank the following people for all their help with thesis:

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Thank you to the entire AE Faculty, specifically all the structural professors whose classes prepared me for thesis:

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Dr. Geschwindner

Dr. Hanigan

Dr. Lepage

Thank you to the following engineers at Greenman-Pedersen Inc. for their help obtaining a thesis project and their willingness to answer questions:

Jeff Kelly

Marc Bowen

Thank you to my friends and family for their support over the last couple of years.

INTRODUCTION

The Gouverneur Health Services Modernization Project is an addition to an existing building and a renovation of the 35-year-old healthcare facility. The existing building is a 2-way floor system with square and rectangular columns. It is assumed that the lateral force resisting system consists of concrete moment frames. For the purpose of this thesis project, only the addition will be investigated in further detail. Furthermore, portions of the addition that wrap around the existing building and tie into the existing structure will be neglected for project. Consequently, it will be important to ensure that the addition is designed to ensure independent performance from the existing structure.

The addition that will be the main focus of this thesis project consists of two distinct portions. The first portion is the 5-story ambulatory care facility. This facility is approximately 115'x175' in plan, and sits on the western side of the site, connected to the existing building. The second portion is an expansion to the floor plan to the existing building in floors 6 through 13. It is roughly 50'x60' in plan, and extends upwards from the ambulatory center on the western side of the existing building. The portions may be referred to as lower addition and upper addition, or ambulatory addition and tower addition, respectively. See Figures below.

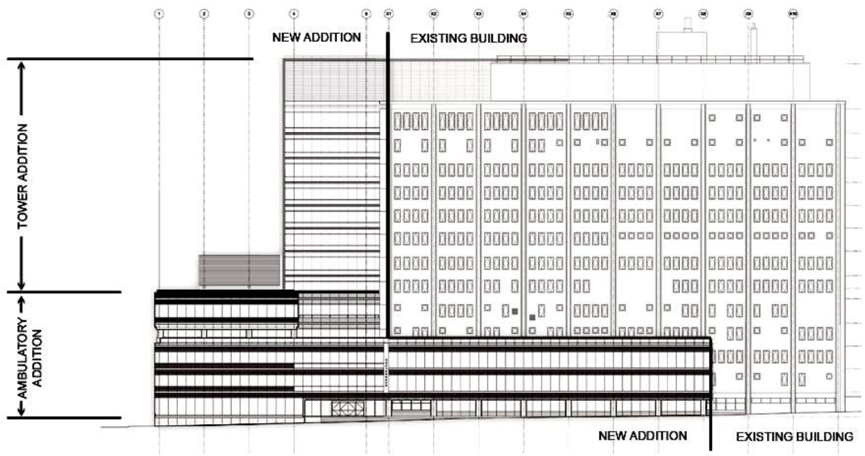


Fig 1. Gouverneur Layout Schematic

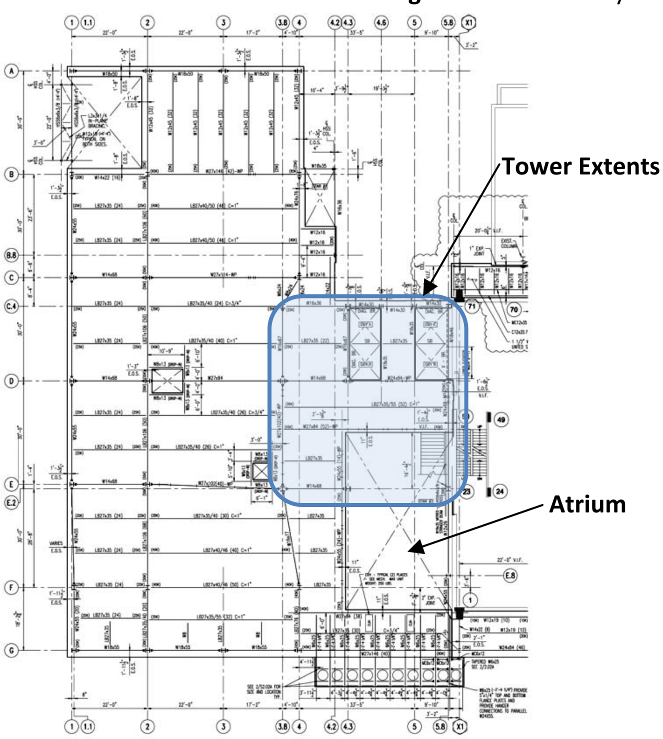


Fig 2. Typical Ambulatory Center Framing Plan

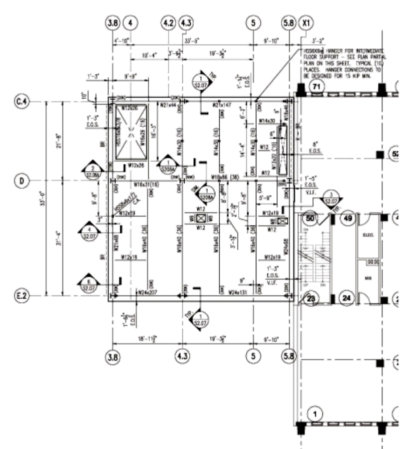


Fig 3. Typical Tower Addition Framing Plan

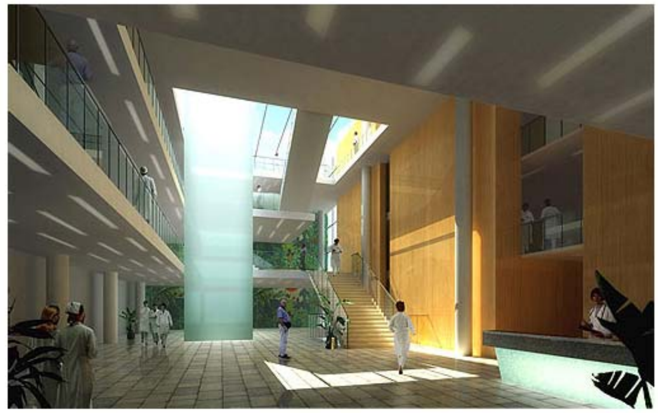


Fig 4. Exterior Rending

Fig 5. Atrium Rendering

EXISTING STRUCTURAL SYSTEM

Foundation

The Gouverneur Healthcare Facility bears on a pile foundation system, with 60-ton capacity, 12" drilled piles. Pile caps vary from 35" to 54" thick with the number of piles ranging from 2 to 16 piles per cap. The footprint for the cellar is smaller than the extents of the overall building so the depths of the pile caps vary. The depths of the caps are either 4'-6" below datum if the columns terminate in the cellar, or 16'-9" above datum if the columns terminate on the first floor.

The piles support grade beams that span between 15' and 40'. Their sizes range from 4'-0" to 8'-3" deep with reinforcing bars from #8 to #12 bars. A structural, two-way slab-on-grade spans between grade beams to make up the cellar floor.

Floor System

The floor system for Gouverneur Healthcare Services is a composite system that utilizes cellular beams for all gravity beams in the ambulatory addition. A 4 ¼" slab rests on a 2" LOK floor composite deck, and is tied to the beam with 5" long, ¾" diameter shear studs. Typical bays are 30'-0" by 44'-0" and almost all beams are nominally 27" deep to accommodate mechanical systems. The tower addition uses traditional W-shapes in a composite floor system. Beams are W16's in areas where clearance for mechanical equipment is not an issue, and W14's where clearance is an issue.

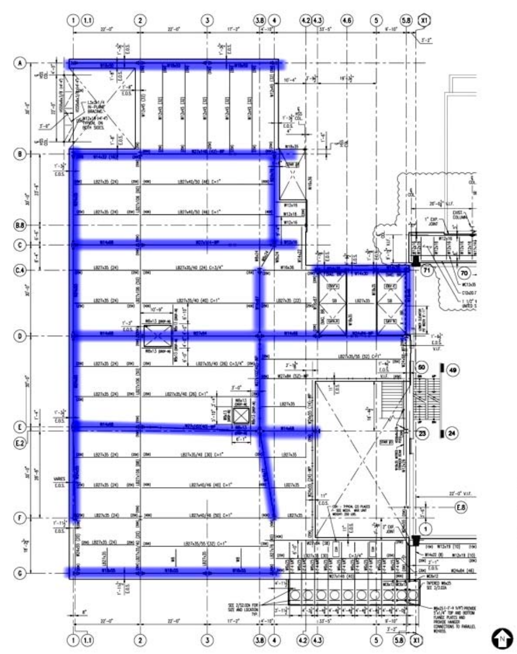


Fig 6. Typical Framing Plan Highlighting Moment Frames

Columns

Almost all columns in the Gouverneur Healthcare Services Building are W14 columns, regardless if it is a part of the lateral system or just a gravity column. Sizes range from W14x43 to W14x257, and are continuous from the foundation to the roof, with only column bearing on a transfer girder on the seventh floor. Columns are spliced on every other floor starting on the third floor. Base plates are typically 22" x 22" with bolts ranging in size from ¾" to 2".

Lateral System

Due to the vast use of glass curtain walls and irregular plan between floors, most of the lateral system in the Gouverneur Healthcare Services Building is moment resisting frames. For the interior moment frames, sizes are either W27's for long span beams or W14's for the shorter spans. Most beams in exterior moment frames are W18's and W24's. In the tower portion of the building, lateral loads are resisted by exterior moment frames in the

East-West direction, and braced frames in the North-South direction, both concentric and eccentric. Most braced frames are continuous from the roof to the column termination at the foundation. But at the interface of the upper addition and the lower addition, where one frame is discontinuous, loads transfer into columns in the floor below, and redistribute through the structure.

Wind loads transfer from curtain wall system to floor diaphragm. The floor diaphragm is rigid compared to structure so loads transfer to lateral frames based off of relative stiffness. Loads then transfer to foundations in the form of shear and axial load (tension and compression) in braced frames, and transfer to the foundation through shear, and axial loads in moment frames.

Man irregularities exist in the framing scheme for the lateral load resisting system. As can be seen in the figure below, both vertical and horizontal irregularities exist that may reduce the efficiency of the overall structural system.

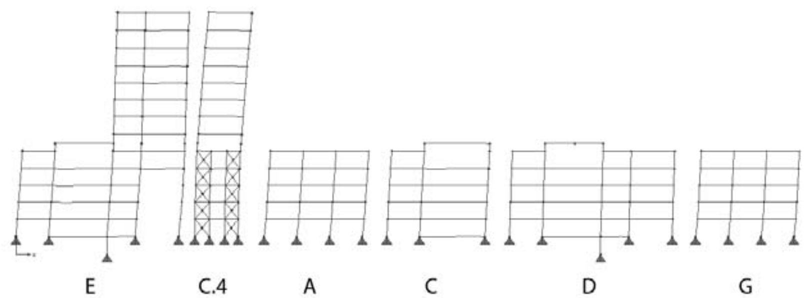


Fig 7. SAP Model – East-West Frames

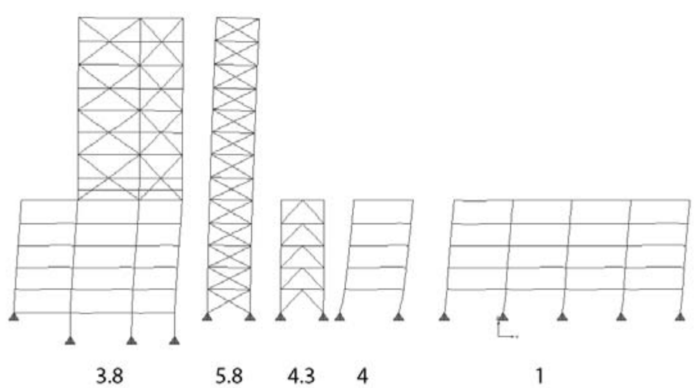


Fig 8. SAP Model – North-South Frames

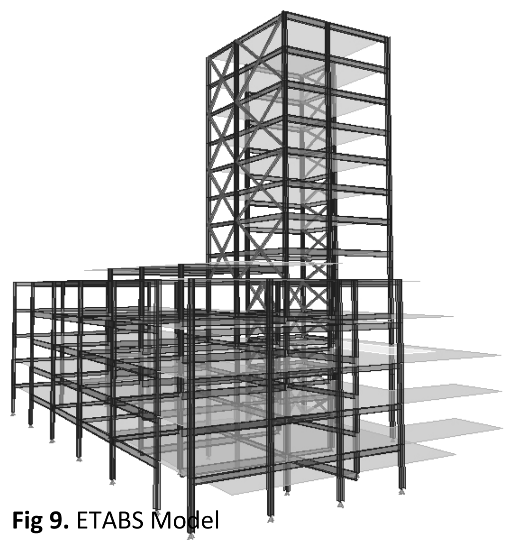


Fig 9. ETABS Model

ETABS Modeling

An ETABS model was also utilized to aid in the calculations of multiple portions of previous technical reports. This model was used to investigate the performance of the previous structure for deflection and drift criteria. It was determined that the building exhibited large torsional effects and certain lateral force resisting system was not as efficient as it could be. Frames were made irregular in order to interact agreeably with the architectural floorplan.

PROBLEM STATEMENT

Designers chose to utilize steel framing for the structure of the Gouverneur Healthcare Facility, with moment frames comprising the majority of the lateral system. Because the current design is an addition to an existing concrete building, many constraints were imposed on the design. A tight, 11ft floor-to-floor height restriction was required in order to match the existing building, constricting the usable space between the ceiling and structural system. In order to accommodate MEP systems, while remaining cost-effective, deep long-span cellular beams were employed for gravity members.

Moment frames used to resist lateral loads had bays that spanned these long distances, averaging 44ft. This long-span condition means that the moment frames are not as efficient as possible. The stiffness of the frames is reduced, resulting in heaver members in order to meet drift criteria. The added weight that is a result of long span lateral members may offset the cost benefits associated with utilizing this design. Specifically, long span members were utilized to save money on foundation costs, although the inefficient lateral system may negate these benefits. Furthermore, numerous web penetrations in the moment frames also cause more intensive fabrication.

The existence of the moment frames also restricts the design of other systems. To resist lateral loads, moment frame beams are designed to be 27" deep, nominally. Consequently, the beams take up the full ceiling cavity and mechanical systems cannot pass beyond this boundary. Effectively, this means that moment frames divide the plan into isolated zones, restricting the ability for MEP engineers to design flexibly.

Further complicating the structural design is the interaction between the tower addition, the ambulatory center, and the central atrium. The performance of the building is greatly impacted due to the placement of the tower directly above the atrium, and the dissimilar column grids between the tower and lower portion. These conditions created irregular braced-frames and moment-frames throughout the building, all leading to a significant amount of torsion under lateral loads, and a reduction in overall system efficiency.

PROPOSED SOLUTION

Depth Study – Structural System Redesign

In order to improve the coordination of systems, a two way flat-plate floor system is proposed for implementation in the Gouverneur Healthcare Facility. Replacing the current design with concrete construction will match the existing portion of the building and will allow adequate space for all systems within the floor height constraints. In order to ensure reasonable slab thickness and resistance of gravity loads, spans will be reduced from 44ft to 22ft. This will allow 12" slabs and create a typical 24ft x 22ft bay for the lower portion of the Gouverneur building. The structure will be designed according to the provisions specified in ACI 318-08 and methods learned in MAE courses will be used to develop a full three-dimensional E-TABS model will be used for lateral design.

Concrete shear walls are proposed to resist lateral loads. A more regular lateral design will be implemented in order to reduce the torsional effects present in the current design. The use of shearwalls will be more restrictive to the floorplan of the building than the moment frames currently employed; however, penetrations can be made at doorways and other openings. Despite this drawback, the inherent stiffness of shearwalls means fewer elements are needed to resist lateral loads, and will not negatively affect the function of the building if the structure and floorplan are designed in coordination.

Breadth Studies

Changing structural systems from steel to concrete may greatly impact the architectural design of the Gouverneur Healthcare Facility, making an architecture breadth a logical area of investigation. In order to accommodate the change in design of 44ft bays to 22ft bays, an extra row of columns will be needed. The use of shearwalls will also affect the architecture of the building as it will somewhat restrict the open plan that the original moment frame affords. The impact of these columns and walls on the floor layouts will be investigated, and rearranging of spaces will be performed as necessary, while still meeting IBC and ADA requirements. With the original design the floorplan dictated a need for structural irregularity. With the proposed design change it will be possible to simultaneously create a regular structural system and design the floorplan in coordination with one another.

It will also be necessary to design the architectural floorplan of the long-term care residence floors, because a final layout was not available at the time this thesis project commenced. The layout of these rooms will be based on the residences in the existing building.

Along with the architecture, the change in systems from steel to concrete will influence the construction process considerably. It will be necessary to investigate these impacts as a second breadth study to determine if the proposed changes to the structural system are feasible. Schedules and cost estimates will be evaluated for both the current design and the proposed design. Findings will be compared in order to understand the benefits and drawbacks of both systems.

STRUCTURAL SYSTEM REDESIGN

Design Goals –

In order to create a successful design, it is necessary to first outline certain design goals that must be met. The goals must be flexibile enough in order adjust to unexpected results, and prioritized to create a succesful project.

First and foremost, the structure must be designed to meet strength and servicability requirements. The building code and the specifications from ASCE7-05 and ACI318-08 will be used to meet the following requirements:

Meet Strength Requirements for all gravity and lateral members

Design floor slab to meet criteria for immediate and long term deflection

Design lateral system to meet drift criteria for wind and seismic loading and ensure deflection does not exceed the upper limit imposed by the expansion joint between buildings

In order for a concrete system to be cost effective, one of the most important design goals is to maintain regularity in structural design. This includes:

Flat plate construction

Column regularity

Shearwall regularity

Methodology –

In order to complete the structural design of the Gouverneur Healthcare Facility, multiple methods were employed, including hand calculations and computer programs. The utilization of each computer program is as follows:

ETABS – used to check deflections and obtain design forces for shearwalls.

RAM Concept – used to design slab reinforcing, check slab deflections, and obtain column takedowns.

PCA Column – used to design columns and check interaction diagram for shearwalls and design flexural reinforcing as necessary.

Excel – used to compile outup from ETABS, calculate load takedowns, design uniform reinforcing for shearwalls, design reinforcing for coupling beams, and compare deflections.

Note: Adequate information was unable to be obtained regarding the portion of the structure that wraps around the existing building on the southern side. Therefore, the scope of the structural design will neglect this space.

Materials –

Concrete	ASTM	Min Strength
Structural Concrete	-	6000 psi
Pile cap	-	4000 psi
Retaining walls	-	4000 psi
Interior Slabs	-	4000 psi
Reinforcing Steel	A615	60 ksi

Applicable Codes and Design Requirements –

Original Codes and References

The City of New York Building and Administrative Code
New York Electrical Code
All Applicable NFPA Codes
New York State Energy Code
AIA Guidelines for Design and Construction of Hospital and Health Care Facilities

Codes and Specifications used in Proposed Design

IBC 2006
ASCE7-05
ACI318-08

Deflection Criteria

Floor Deflection L/480 Total and L/360 Live (Table 9.5(b) ACI 318-08)

Lateral Deflection

Total Drift - 3" (at floors - due to 6" expansion joint between addition and existing building)

3 ½" (at roof – due to 7" expansion joint at roof)

Story Drift - H/500 for wind loading

$0.020h_{sx}$ for seismic loading (Table 12.21-1 ASCE7-05)

Design Load Combinations –

The load combinations considered in the design of the structure are taken from section 2.3.2 in ASCE7-05. The following loads are the basic combinations that are applicable to the design.

1. $1.4\,D$
2. $1.2\,D + 1.6\,L + 0.5\,L_r$
3. $1.2\,D + 1.6\,L_r + L$
4. $1.2D + 1.6W + L + 0.5\,L_r$
5. $1.2D + 1.0E + L$

Structural Design Layout - Overview

The initial step for the design of the structural system was to create a schematic column and shearwall layout. Initially, an extra row of columns was simply added to the existing design creating a typical 22'X30' bay. To meet the design goals previously outlined, multiple iterations of design changes were required. Eventually, a final bay size of **22'X24'** was determined to be most effective.

Columns in the portion of the building that support 6 floors are 16"x16" typical, and columns supporting the full 13 stories are 20"x20". All columns have an f'_c of 6000 psi. This is to match the required compressive strength needed by the floor slab to resist gravity loads, and the compressive strength needed by the shearwalls to maintain appropriate deflections.

The shearwalls are laid out in order to maximize regularity, to reduce torsional effects, to provide adequate stiffness for lateral loads, and to minimally impact the architectural floorplan. They are also designed to match the surrounding columns. For example, shearwalls B, G and 2 are 16" thick to match the columns they will be cast with. Shearwalls 4, 5.8, and D.8 are 20" thick to match the 20" columns in that portion of the building. The coupling beams that join the two piers of shearwall 2 and shearwall D.8 are 36" deep. They are 16" and 20" thick, respectively, to match the adjoining shearwalls.

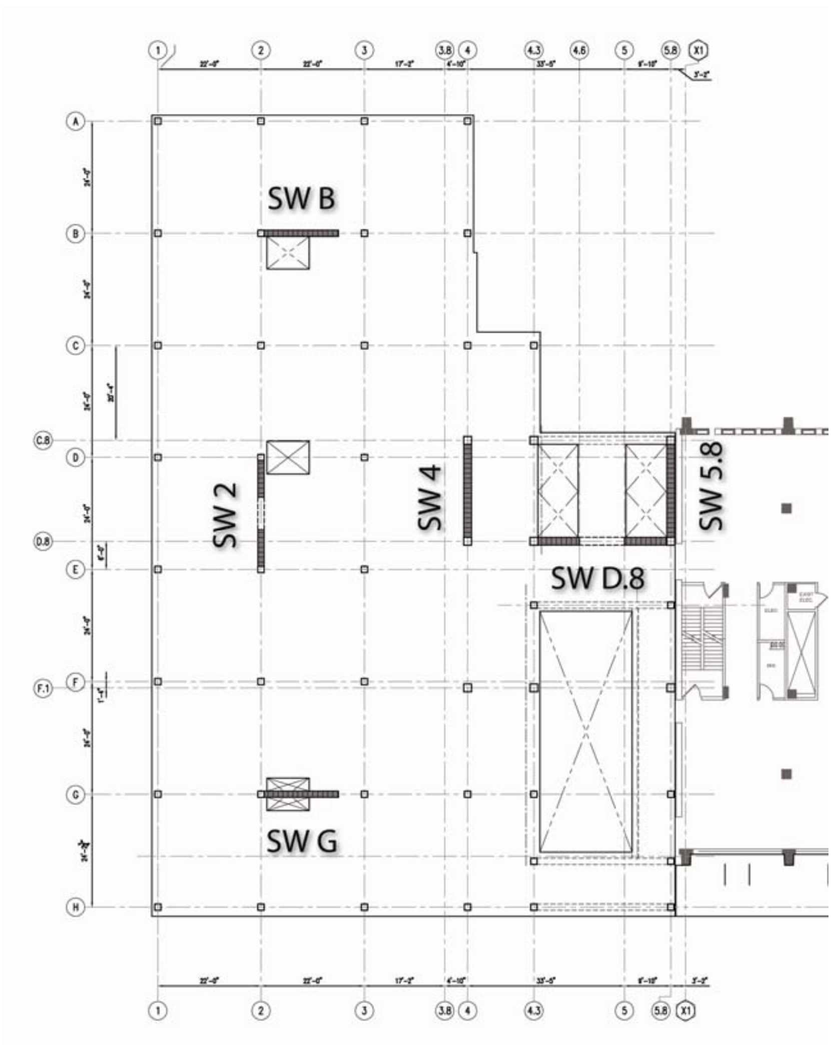


Fig 10. 4th Floor Structural Layout Plan

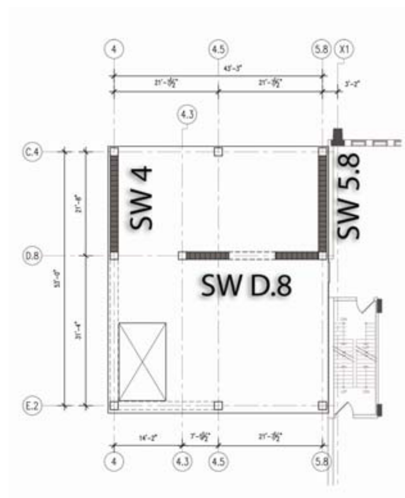


Fig 11. 9th Floor Structural Layout Plan

GRAVITY DESIGN

Gravity Loading –

Gravity loads were determined using the requirements from ASCE7-05 and superimposed dead loads provided by Greenman-Pedersen Inc. Due to the nature of the design process, and the unknown final floorplan layout during the structural design process, the majority of the gravity system was designed using the maximum live load that occurs on a given floor. For example, the 7th Floor consists entirely of long-term care residences and hallways. The residences could have been designed using 40psf live load as per ASCE7-05, however, the final layout of the rooms was unknown during the structural design process. Therefore, a blanket 80psf was used in the design of these floors. See Figure 12 for a summary of the loads used to design the gravity system.

Superimposed Dead Load (psf)	
Floor Load	
Ceiling	2
Floor Finish	2
Mech/Elect	10
Partitions	12
TOTAL	26
	(psf)
Penthouse Roof	
Deck/Insulation	8
Mechanical	10
Membrane	2
Fire Proofing	2
TOTAL	22
	(psf)

Live Load (psf)		
Live Load	As Designed	As per ASCE7
Dormatory Floors	80	40
Lobby	100	100
Lounge	100	100
Corridor 1st Floor	100	100
Corridor above 1st	80	80
Mechanical Rooms	150	-
Main Roof (Mech)	150	-

Wall assemblies	
1. Metal Panel	25
2. Glass Curtainwall	15
GFRC	40
	(psf)

Fig 12. Design Load Tables

Column Design –

Initially, load takedowns were performed using the preliminary column layout developed early in the design process. Takedowns were performed and members were sized using PCA Column. As the design progressed, the column layout was finalized in coordination with the final slab design. Takedowns were performed by hand and confirmed using RAM Concept output. See the figure at right for a sample of load takedowns performed by hand.

Typical columns supporting the six floors of the ambulatory care center are 16”x16” with an f'_c of 6000psi. Columns supporting the full 13 stories are 20”x20” with f'_c = 6000psi.

See Appendix D for column design summary.

Interior - Lower only		D2	Size	16
	# Floors	6	f'c	6
	Trib Area	660		
	Infl. Area	2640		
	DL	176		
	LL	80		
	LLr	150		
	Total DL	696960	**697.0**	
	Total LL	310200	310.2	
	LL Reduction	0.54		
	Reduced LL	168109	**168.1**	
	1.2D+1.6L	1.1E+06	**1105.3**	29.5
	1.4D	975744	**975.7**	
			(kips)	

Fig 13. Load Takedown – Column D/2 & D/3

The design of the columns was controlled largely by axial load, with the controlling load case being

$1.2\,D + 1.6\,L + 0.5\,L_r$

The unbalanced moment distributed to the columns, although small, were necessary in order to complete the design. These values were obtained from RAM Concept. See Appendix I for an example of the RAM output. The design forces were then input into PCA Column to design for the interaction between axial and moment.

Column Design - Typical Members - Interior Columns				
16x16	**D/2**		20x20	**D/4.3**
Pu=	1105		Pu=	1673
Mux=	30		Mux=	-57.7
Muy=	30		Muy=	28.9
Size	16x16		Size	20x20
Final Design				
Steel	(8) #14 Bars		Steel	(12) #14
Hoop	(4) @ 24	Hoop	(4) @	24

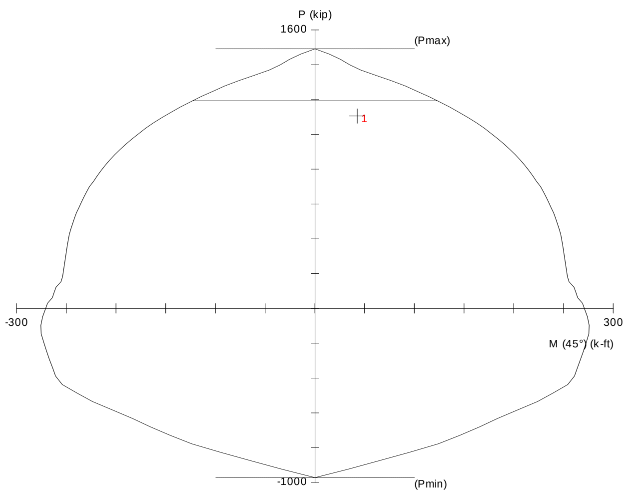


Fig 14. Interaction Diagram – Column D/2

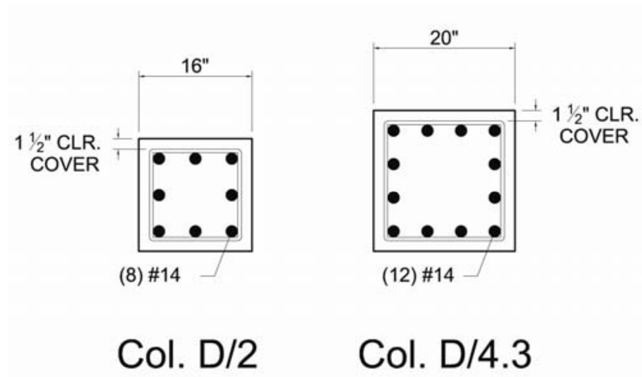


Fig 15. Design Loads and Detailing – Column D/2 & D/4.3

Column Design – Slender Columns

All columns were investigated for the need to consider slenderness as per ACI818-08 section 10.10. Only two columns, col. G/5.8 and col. F/5.8 were determined to have slenderness be a contributing factor to their design. These members span three stories, unbraced in the atrium space.

PCA Column was used to accurately determine the k value of each column by defining the connectivitey. Takedowns for axial loads and moment were determined using RAM Concept. Columns were initially investigated in order to maintain typical column dimensions.

After investigation, it was determined that column G/5.8, supporting 6 stories, was adequate for strength requirements using a 16"x16" column, with f'_c = 6000psi and (8) #14 bars for longitudonal reinforcing.

Column F/5.8, supporting the full 13 stories was inadequate and needed to be increased to 22"x22" with f'_c = 6000psi concrete and (12) #14 bars for reinforcing.


Fig 16. Atrium showing slender columns

Slenderness Considerations				Critical Members	
16x16		20x20		Must Consider	
k=	1	k=	1	Slenderness	
l=	396	l=	396		
r=	4.62	r=	5.77		
kl/r=	85.74	kl/r=	68.59		
				Typical Members	
16x16		20x20		Don't Consider	
k=	1	k=	1	Slenderness	
l=	132	l=	132		
r=	4.62	r=	5.77		
kl/r=	28.58	kl/r=	22.86		

Fig 17. Slenderness Calculation Table

Column Design - Slender Members					
16x16	**G/5.8**			20x20	**F/5.8**
P=	239.3			P=	1006.9
Mx=	7.1			Mx=	10.4
My=	67.7			My=	98.1
Load Takedown					
13-7	-			13-7	401.1
6	73.8			6	370
5	84.3			5	97.1
4	63.6			4	72.7
S.W.	17.6			S.W.	66
Final Design					
Size	**16x16**			Size	**22x22**
Steel	**(8) #14 Bars**			Steel	**(12) #14**
Hoop	(4) @	24	Hoop	(4) @	24

Fig 18. Slender Columns Design Summary

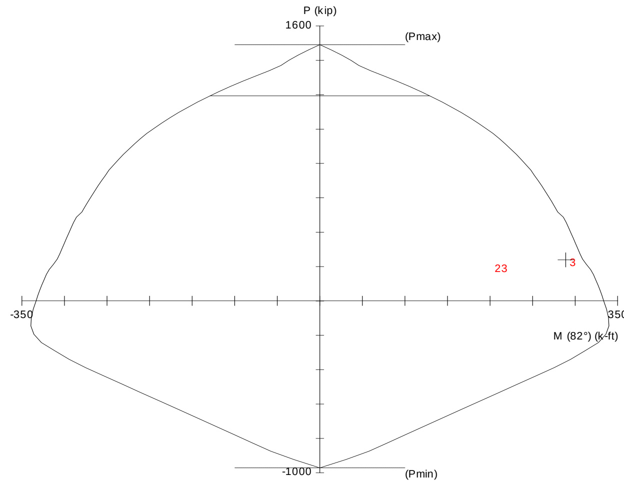


Fig 19. Column G/5.8 Interaction

Design of Two-Way Flat Plate Slab System –

Lower Portion Slab Design – Reinforcing

RAM Concept was used to design the reinforcing for the two-way flat plate slab system and check deflections. The preliminary column layout created a typical 22' x 30' bay in the lower portion of the Gouverneur Healthcare Facility. Initial slab thickness was determined to be 10" per minimum slab thickness specified in ACI318-08 Table 9.5(c).

However, running the model displayed a long-term deflection that exceeded the maximum allowable long-term deflection limit of 0.8". Further investigation revealed that many columns failed punching shear checks due to the relatively high Live Load for the specified slab thickness.

After an iterative process, the design was finalized at **22' x 24' bays and 12" slab thickness**. This design allowed for an adequate slab that did not require the use of edge beams or drop panels. Figures 16 and 17 show the bottom and top reinforcing for typical bays in the lower portion of the Gouverneur Facility, respectively.

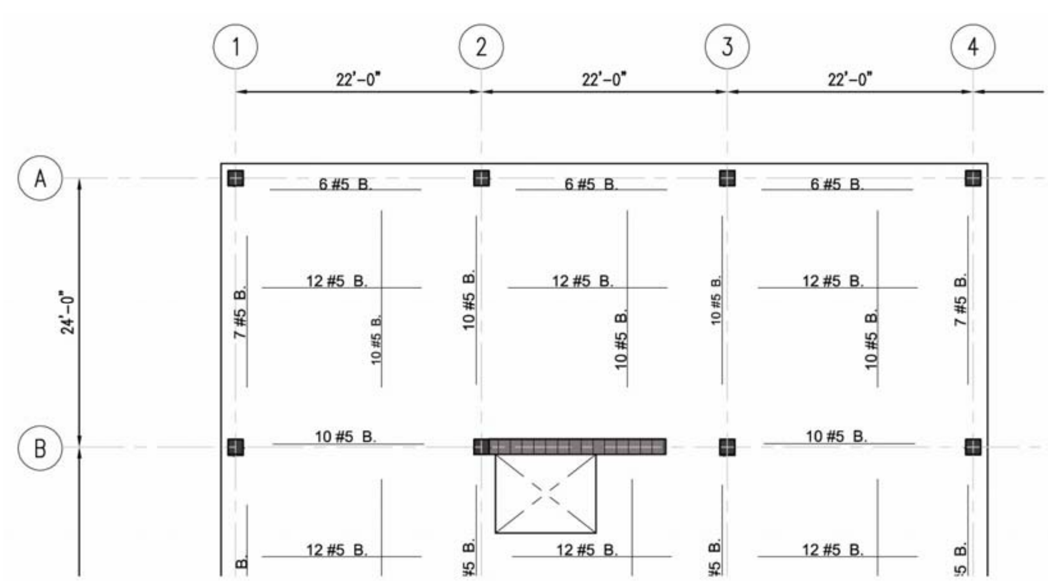


Fig 20. 3rd Floor – Bottom Reinforcing Partial Plan

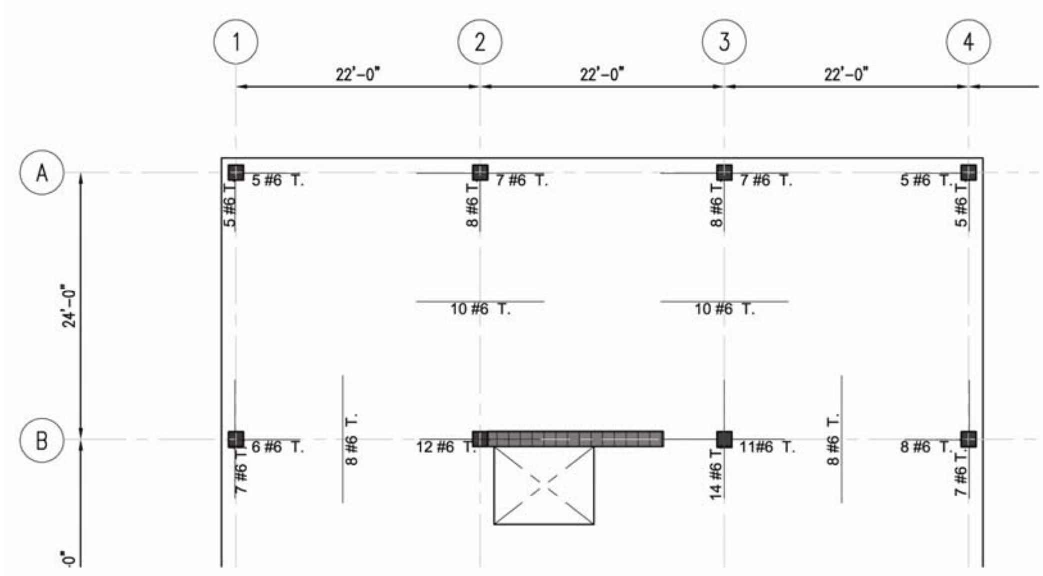


Fig 21. 3rd Floor – Top Reinforcing Partial Plan

Although most bay sizes for floors 2 through 6 are typical, each floor had elements unique to the level. This required that each floor be modeled explicitly. The fourth floor contains a catwalk that spans the long dimension of the atrium, the fifth floor is continuous over the top of the atrium and the slab cantilevers 7'-6" on the Southern side of the building. However, for typical bays, the reinforcing layout is largely the same, and is tabulated below. See Appendix G for full reinforcing plans.

Strip	**Span**	**Reinforcing Bars**
Exterior Column Strip	N-S	(7)#5 bottom and (7)#6 top bars
Interior Column Strip	N-S	(10)#5 bottom and (14)#6 top
Exterior Column Strip	E-W	(6)#5 bottom and (7)#6 top bars
Interior Column Strip	E-W	(12)#5 bottom and (10)#6 top
Middle Strip	N-S	(10)#5 bottom and (8)#6 top bars
Middle Strip	E-W	(12)#5 bottom and (10)#6 top bars

Lower Portion Slab Design – Deflection

As previously mentioned, deflection criteria was the governing factor for most of the design choices for the column layout and slab design. As per ACI318-08, immediate live load deflections are limited to L/360, which translates to 0.80" for slabs spanning 24' and 1.07" for 32' spans. Long-term plus live load deflections are limited to L/480 for most areas where partitions could be damaged by large deflections. The lower roof of the building has long-term deflection limits of L/240 because there are not any non-structural elements to be damaged by deflection.

The maximum long-term deflection for a 24' span is 0.5417" and occurs on the 4th Floor. The maximum long-term deflection for a 32' span is 0.81" and occurs on the Lower Roof (6th Floor – Roof), where the deflection limit is L/240. However, the max deflection for the 5th Floor is 0.7678" and is still within the allowable limit of 0.80".

24' Span

Max LL Deflection	
L/360	0.80
Max Long-Term + LL	
L/480	0.60
L/240	1.20

32' Span

Max LL Deflection	
L/360	1.07
Max Long-Term + LL	
L/480	0.80
L/240	1.60

Fig 22. Deflection Limits

The figures below depict a sample deflection map of the 3rd Floor, and the corresponding table for calculating immediate live load deflections and long-term deflections. A color gradient represents the magnitude of the deflection, with red being the maximum value which corresponds the Δd+l value given in the table. RAM Concept was utilized to obtain the Dead Load + Live Load deflections. A creep factor of 2.0 (ACI318-08 section 9.5.2.5) was used and a ratio of loads was used to calculate individual values.

See Appendix I for tabulation of deflection for all floors.

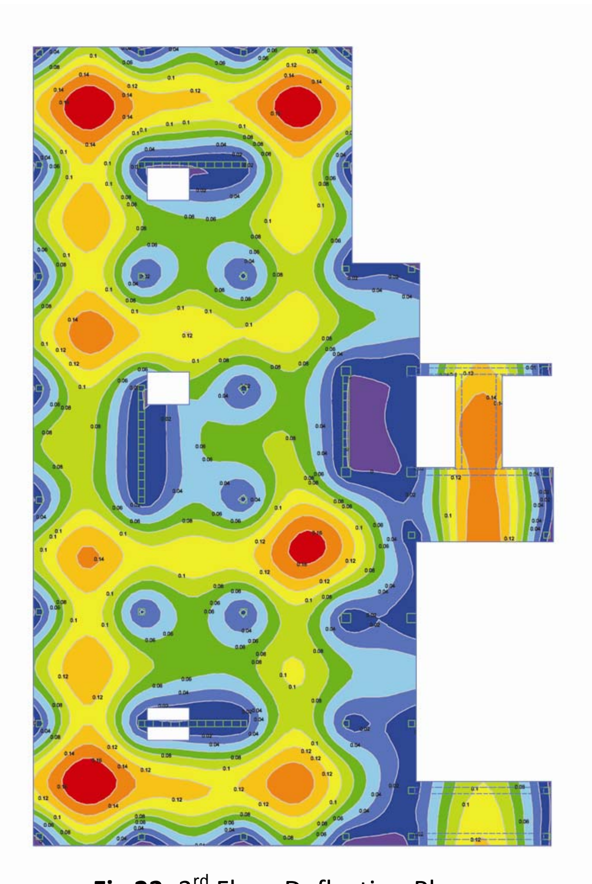


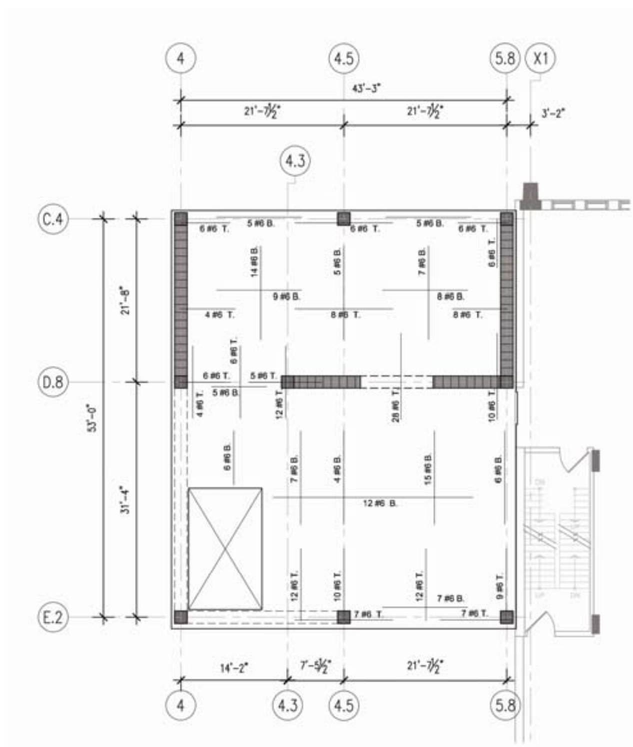
Fig 23. 3rd Floor Deflection Plan - D + L loading

Model	3rdflr	
slab	12	in
edge bm	no	
edge col	16	16
Δd+l	0.1775	
ΔLi	0.0555	ok
Δdt	0.2441	
Δ20%lt	0.0222	
Δ	0.4438	ok

Fig 24. 3rd Floor Deflection Table

Upper Portion Slab Design

The slab for the upper portion of the Gouverneur Healthcare Building (Floors 7-13) was designed using a 12” slab and #6 reinforcing bars top and bottom. In order to meet deflection criteria, the large span that occurs in lower floors, from column line 4.3 to column line 5.8, needed to be cut down. For architectural reasons, it was most logical to place a new column line halfway between the exterior columns of the upper floors. With this new layout, long-term deflections were kept to just within the L/480 limit of 0.7833”. See Figures on next page for a deflection plan and deflection calculation table.



See Appendix G for full reinforcing plan.

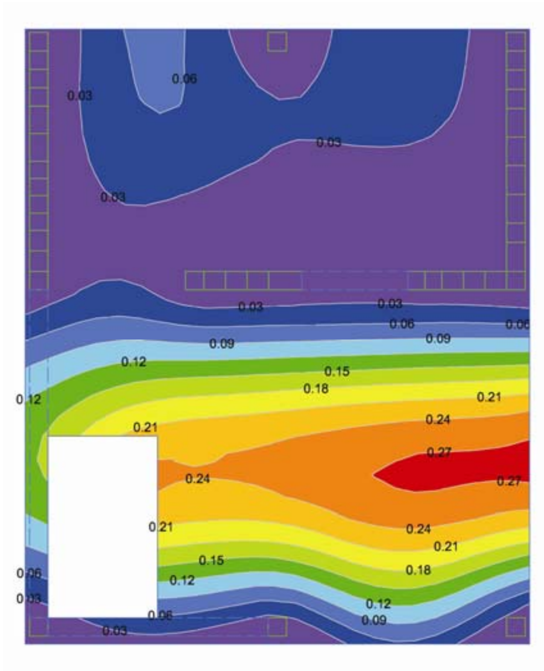


Fig 26. 7th-13th Floor Deflection Plan - D + L loading

Model	7thflr	
slab	12	in
edge bm	no	
edge col	20	20
Δd+l	0.2783	
ΔLi	0.0870	ok
Δdt	0.3827	
Δ20%lt	0.0348	
Δ	0.6958	ok

Fig 27. 7th -13th Floor Deflection Table

Transfer Beam Design –

The column shift of the upper floors conflicts with the architecture of the lower floors. Specifically, if the upper column (col F.1/4.5) were to be continuous to the foundation, it would drop into the middle of the atrium. In order to avoid this situation, a transfer beam was designed on the 6th floor. See Figure below for illustration of this condition.

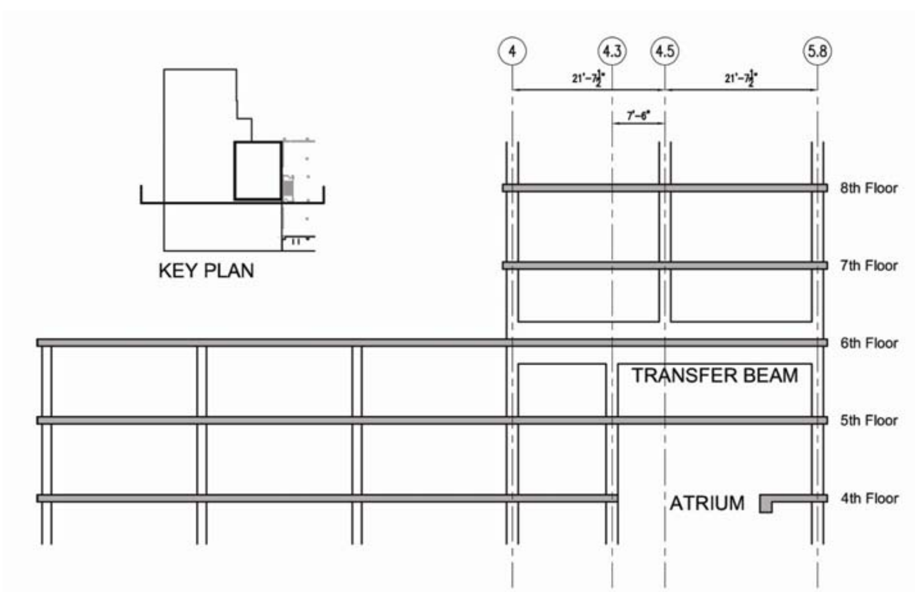
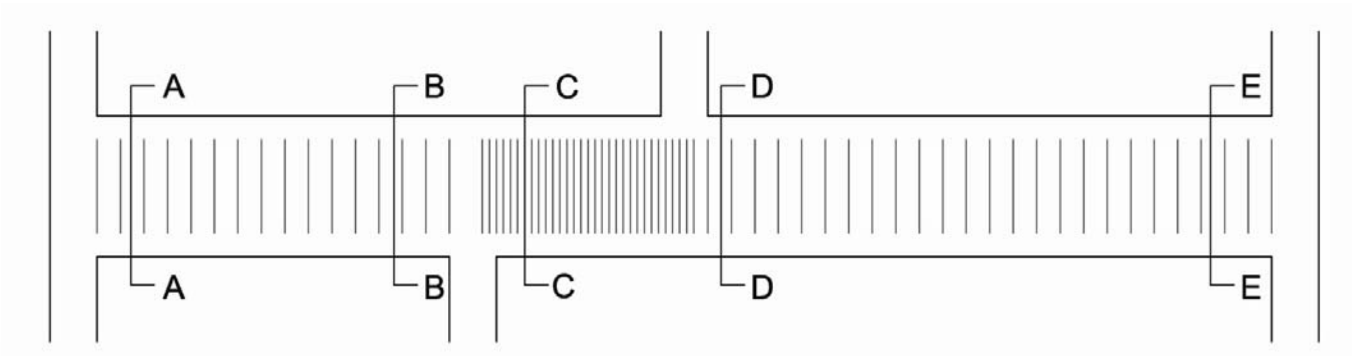
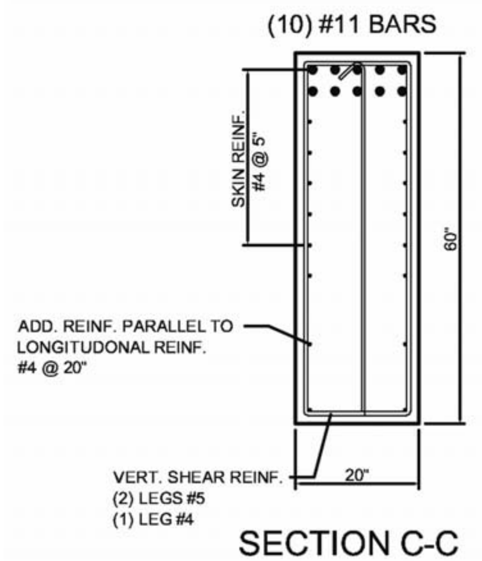


Fig 28. Building Section Showing Transfer Beam

Load takedowns revealed that the force on the beam is an 890k, factored reaction transmitted by the column from the floors above. The columns that the beam frames into are 20”x20”, therefore, to ease with construction, the width of the beam is designed to be 20”. The maximum moment is -3500 ft-k occurring at the nearest support; see Section C at figure below. The magnitude of the moment determined that the depth of the beam should be 60” in order to keep the amount of steel reinforcing at a reasonable amount.

In addition to typical flexural and shear design, additional deep beam considerations were taken into account. Because the beam depth exceeds 36”, skin reinforcing was provided as per ACI318-08 section 10.6.7. Section 10.7 in ACI318-08 also has provisions for deep beams that were considered in the design. See Appendix H for detailed design procedure.





Section	Long. Reinf.	Shear Reinf.	Skin Reinf.	Add. Horiz.
A-A	(5) #5 bot	(3) legs #4 @ 10"	#4 @ 5"	#4 @ 20"
B-B	(5) #11 top	(3) legs #4 @ 10"	#4 @ 5"	#4 @ 20"
C-C	(10) #11 top	(2) #5 & (1) legs #4 @ 4"	#4 @ 5"	#4 @ 20"
D-D	(12) #9 bot.	(3) legs #4 @ 10"	#4 @ 5"	#4 @ 20"
E-E	(8) #9 top.	(3) legs #4 @ 10"	#4 @ 5"	#4 @ 20"

Fig 29. Transfer Beam Design and Details

Normally, a deep transfer beam would be an issue in a two-flat plate system. However, because the beam occurs at the exterior wall of the upper floors, the top of the beam does not have to be level with the top of the slab. This allowed the full depth of the beam to extend 30” above the top of the slab, and 30” below the top of the slab. The top portion of the beam would be integrated into the exterior wall, and due to the slab thickness of 12”, the bottom of the beam extends only 18” from the bottom of the slab. Once it was decided to not count on the slab to contribute to the strength of the beam, a SAP model was created to obtain design output

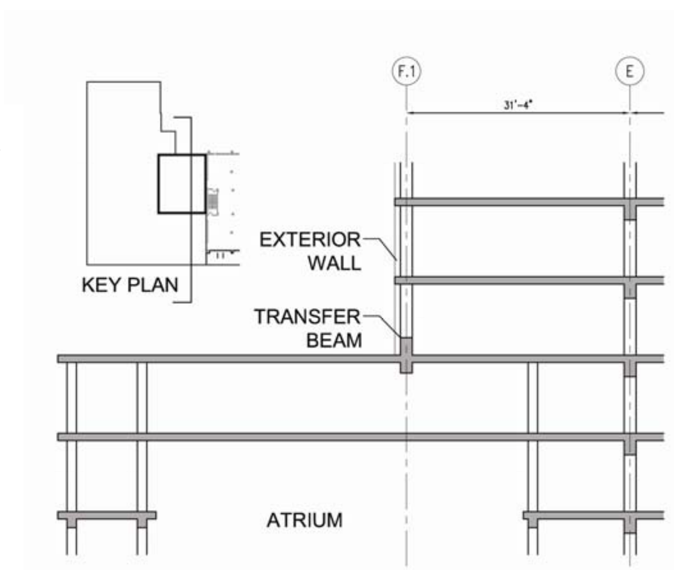


Fig 30. Section Showing Transfer Beam

Lateral System –

The lateral load resisting system of the Gouverneur Healthcare Facility is comprised of concrete shearwalls with f'_c=6000 psi. Shearwalls B, G, and 2 extend from the foundation to the 6th story roof and are 16" thick to match the size of the columns in that portion of the wall. Shearwalls D.8, 4 and 5.8 are 20" thick to match the columns that extend the full height of the building. See Figure.

The design of shearwalls was investigated for Seismic and Wind Loading. Equivalent Lateral Force Procedure provided in ASCE7-05 was used for seismic design and the Analytical Method in Chapter 6 from ASCE7-05 were used to investigate wind loading.

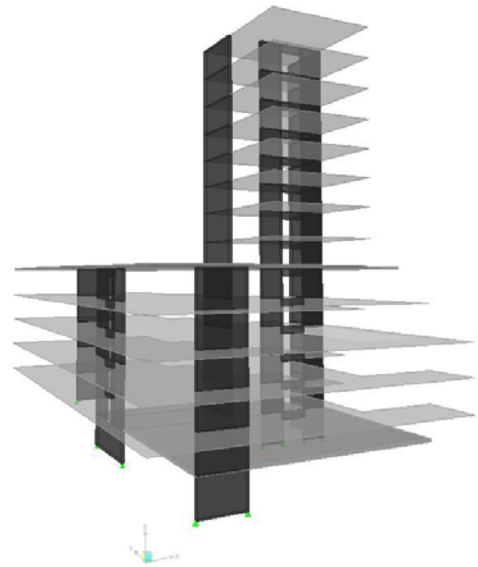


Fig 31. Etabs Model

ETABS Model – MAE Considerations –

A three-dimensional ETABS model was built using methods from MAE courses, most specifically AE 597A – Computer Modeling. For simplicity, only the diaphragms and shearwalls were modeled. Floors are assumed to be rigid diaphragms, except on the 1st and 6th floors where large shear reversals exist due to an abrupt change in the stiffness of a given floor. For these floors, the diaphragms were meshed and modeled as semi-rigid diaphragms. Note: basement walls account for the change in stiffness below the first floor but are omitted for clarity in the figure below. Shearwalls are modeled as membranes so they have no out-of-plane stiffness, and only resist in-plane forces. Walls are modeled using a 0.7 stiffness modifier, and coupling beams are modeled using a 0.35 stiffness modifier as per ACI318-08 section10.10.4.1.

Lateral load cases were calculated by hand and applied to the structure. Through analysis, it was determined that Seismic Loads controlled deflection criteria, and therefore controlled the size and layout of shearwalls. Wind Loads resulted in the highest shear forces in the walls but did not govern the design of the Lateral Force Resisting System due to the amplification of seismic deflections.

Seismic Loading –

As previously mentioned, seismic analysis was performed using the Equivalent Lateral Force Procedure specified in ASCE7-05

The seismic design criteria are as follows: (From ASCE7-05)

Occupancy Type:	IV
Importance Factor:	1.15
Seismic Design Category:	B
S_{DS}:	0.242
S_{D1}:	0.047
T_a:	0.874

Story	Floor Height	Floor Weight	$w_i h_i^k$	C_{vx}	Story Force	Story Shear
Main Roof	156.00	595.7	92922	0.096	29.046	29.046
13	140.3	571.2	80135	0.083	25.049	54.095
12	128.30	571.2	73292	0.076	22.910	77.005
11	116.3	571.2	66449	0.069	20.771	97.776
10	105.13	571.2	60052	0.062	18.771	116.548
9	93.9	571.2	53655	0.055	16.772	133.320
8	82.73	571.2	47258	0.049	14.772	148.092
7	70.8	571.2	40415	0.042	12.633	160.726
6	59.55	2478.0	147568	0.152	46.128	206.853
5	47.6	2489.8	118446	0.122	37.025	243.878
4	36.38	2486.9	90461	0.093	28.277	272.155
3	25.2	2486.9	62613	0.065	19.572	291.727
2	13.98	2489.8	34805	0.036	10.880	302.607
	(ft)	(kip)			302.61	
					Base Shear	

Fig 32. Seismic Loading

Seismic Deflections –

Deflection and force output from ETABS was compiled in Excel for the four individual seismic load cases.

EX (force in the X-dir)
EXMZ (force in the X-dir + Accidental Torsion)
EY (force in the Y-dir)
EYMZ (force in the Y-dir + Accidental Torsion)

ASCE7-05 requires amplification to the accidental torsion, defined by the equation:

$$\text{Amplification, } A= (d_{max}/(1.2*d_{avg}))^2 \quad \text{(Figure 12.8-1)}$$

Analysis from the ETABS model revealed A<1, therefore no amplification was necessary. Furthermore, the greatest deflection obtained from elastic analysis was from the EXMZ load case. Elastic analysis deflections are to be amplified to account for final deflections under seismic loads.

Final seismic deflection:

$$d_x=Cd\ d_{xe}/I *(T_a/T) \quad \text{(eq 12.8-15)}$$

The equation above was modified by a ratio of the approximate period used to calculate design forces and the period of the structure obtained by computer analysis. The modification is in response to ASCE7-05 Section 12.8.6.2, where it states that it is "permitted to determine the elastic drifts using seismic design forces based on the computed fundamental period of the structure not the upper limit (C_uT_a)…"

EXMZ Amplified Deflections					
	Displ X	Displ Y		Drift Ratio X	Drift Ratio Y
HI ROOF	3.2132	-1.0415		0.0019	0.0007
13	2.9393	-0.9365		0.0020	0.0007
12	2.6507	-0.8304		0.0021	0.0007
11	2.3508	-0.7236		0.0021	0.0007
10	2.0593	-0.6237		0.0022	0.0007
9	1.7601	-0.5250		0.0022	0.0007
8	1.4597	-0.4294		0.0021	0.0007
7	1.1470	-0.3331		0.0020	0.0006
LO ROOF	0.8821	-0.2513		0.0018	0.0005
5	0.6283	-0.1761		0.0015	0.0005
4	0.4212	-0.1175		0.0013	0.0004
3	0.2479	-0.0687		0.0010	0.0003
2	0.1164	-0.0302		0.0006	0.0002

Fig 33. Maximum Amplified Seismic Deflections

Values are well within the **allowable story drift ratio = 0.015 (**Table 12.21-1 ASCE7-05). However, the design of the shearwalls was controlled by the upper limit imposed by the joint between the existing structure and new structure.

Although the Equivalent Lateral Force Method is an appropriate way to design a structure, the structure of a building is not loaded statically as this method would imply. The response of a building to seismic loading is dynamic and depends on the natural frequency of a structure. Because accurate information is not known about the existing building, it was decided to conservatively limit the overall deflection of the proposed addition to half the distance of the expansion joint between the buildings. For floors, this means the building is allowed to deflect 3" due to a 6" expansion joint, and 3.5" at the roof due to a larger expansion joint at the roof.

The basis for this design choice comes from ASCE7-05. ASCE7-05 Section 12.12.3 states that portions of a structure that do not act integrally must be "separated structurally by a distance sufficient to avoid damaging contact under deflection (d_x)…"

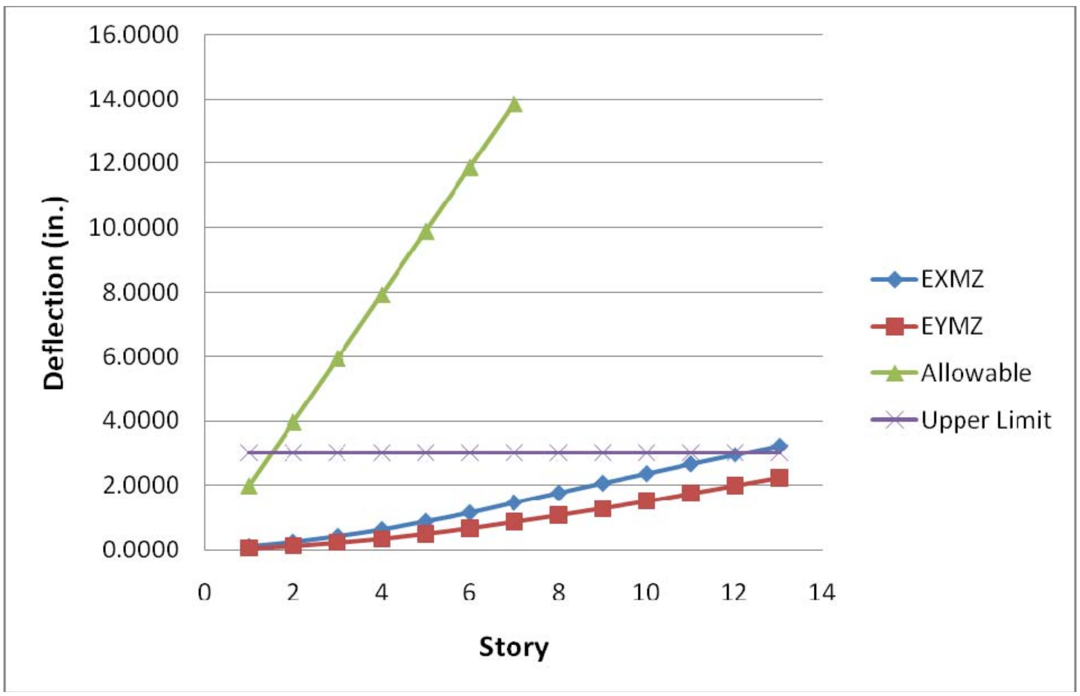


Fig 34. Deflections Compared to Allowable Limits

Wind Loads –

For the purpose of the proposed design, the Main Wind Force Resisting system is designed using the wind load provisions in Chapter 6 of ASCE7-05. Criteria used to calculate these loads are as follows:

Occupancy	IV
Basic Wind Speed - V	100mph
Importance Factor - I	1.15
Directionality - Kd	0.85
Internal Pressure Coeff. - GCpi	±0.18

The following figures are the wind load cases for the X and Y direction. They represent the two iterations of Case1 from ASCE7-05. The calculated results of the 12 iterations that are obtained from the four general cases can be found in Appendix B. Wind loads are split into two vertical "zones" that represent the impact of the smaller upper portion on the distribution of wind forces on the structure.

Wind X		Floor Elev. (ft)	Elev. above datum (ft)	Story Height (ft)	Story Force (kip)	Story Shear (kip)
Zone 2	main roof	154.00	171.01	11.98	23.6	23.6
	13	138.28	159.03	11.98	23.6	47.1
	12	126.30	147.05	11.98	23.4	70.6
	11	114.32	135.07	11.20	22.1	92.6
	10	103.13	123.88	11.20	21.2	113.8
	9	91.93	112.68	11.20	20.4	134.2
	8	80.73	101.48	11.98	20.8	155.0
	7	68.75	89.50	11.20	20.4	175.4
Zone 1	6	57.55	78.30	11.98	34.8	210.2
	5	45.57	66.32	11.20	47.6	257.8
	4	34.38	55.13	11.20	43.9	301.8
	3	23.18	43.93	11.20	41.0	342.7
	2	11.98	32.73	11.98	39.5	382.2
	Ground	0.00	20.75	0.00	0.0	382.2
		Datum	20.75		**382.2**	Total

Fig 35. Wind Loading X-dir (Case1a)

Wind Y		Floor Elev. (ft)	Elev. above datum (ft)	Story Height (ft)	Story Force (kip)	Story Shear (kip)
Zone 2	main roof	154.00	171.01	11.98	19.7	19.7
	13	138.28	159.03	11.98	19.7	39.4
	12	126.30	147.05	11.98	19.6	59.0
	11	114.32	135.07	11.20	18.5	77.5
	10	103.13	123.88	11.20	17.7	95.2
	9	91.93	112.68	11.20	17.1	112.3
	8	80.73	101.48	11.98	17.4	129.7
	7	68.75	89.50	11.20	17.0	146.7
Zone 1	6	57.55	78.30	11.98	34.2	180.9
	5	45.57	66.32	11.20	32.9	213.8
	4	34.38	55.13	11.20	30.4	244.2
	3	23.18	43.93	11.20	28.3	272.5
	2	11.98	32.73	11.98	27.3	299.8
	Ground	0.00	20.75	0.00	14.0	313.7
		Datum	20.75		**313.7**	Total

Fig 36. Wind Loading Y-dir (Case1b)

Wind Deflections –

The twelve wind cases were used to check deflections. Case 1 provided the highest values for X & Y deflections.

CASE1A					
Story	Diaphragm	Load	UX	UY	UZ
HI ROOF	D1	CASE1A	1.465	-0.385	0
13	D1	CASE1A	1.346	-0.346	0
12	D1	CASE1A	1.222	-0.307	0
11	D1	CASE1A	1.091	-0.268	0
10	D1	CASE1A	0.962	-0.232	0
9	D1	CASE1A	0.828	-0.196	0
8	D1	CASE1A	0.691	-0.161	0
7	D1	CASE1A	0.546	-0.126	0
LO ROOF	D2	CASE1A	0.425	-0.043	0
5	D1	CASE1A	0.313	-0.038	0
4	D1	CASE1A	0.213	-0.026	0
3	D1	CASE1A	0.126	-0.014	0
2	D1	CASE1A	0.054	-0.006	0
1	D1	CASE1A	0.000	0.000	0

Fig 37. Wind Deflection Case1a (X-dir)

CASE1B					
Story	Diaphragm	Load	UX	UY	UZ
HI ROOF	D1	CASE1B	-0.313	0.868	0
13	D1	CASE1B	-0.279	0.778	0
12	D1	CASE1B	-0.244	0.688	0
11	D1	CASE1B	-0.210	0.598	0
10	D1	CASE1B	-0.178	0.516	0
9	D1	CASE1B	-0.147	0.435	0
8	D1	CASE1B	-0.117	0.358	0
7	D1	CASE1B	-0.088	0.280	0
LO ROOF	D2	CASE1B	-0.067	0.189	0
5	D1	CASE1B	-0.049	0.137	0
4	D1	CASE1B	-0.032	0.092	0
3	D1	CASE1B	-0.018	0.052	0
2	D1	CASE1B	-0.008	0.022	0
1	D1	CASE1B	0.000	0.000	0

Fig 38. Wind Deflection Case1b (Y-dir)

This conclusion is expected since the structure does not have significant torsional problems. Case 1 controls wind deflection for similar reasons that the accidental torsion for seismic loading did not need to be amplified.

The values for deflection are well within the acceptable H/500 drift limitation. See Figure below for illustration. This is to be expected because seismic deflections controlled the design of the shearwalls.

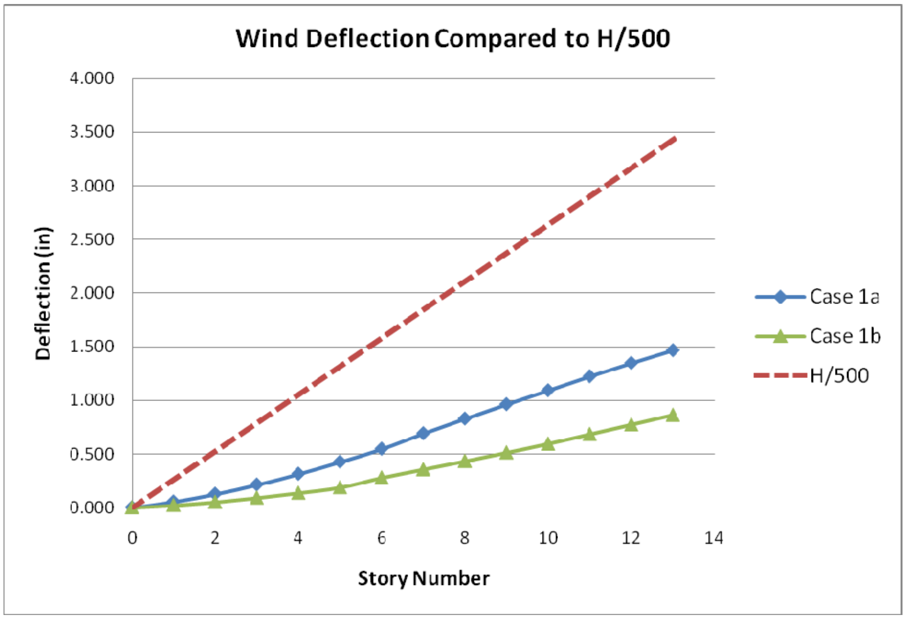


Fig 39. Wind Deflection Comparison

Shearwall Design and Detailing –

Factored load output from ETABS was compiled in Excel for all seismic and wind load cases. See figure below.

	Story	Pier	Load	Loc	P	V2	Vabs	M3	Mabs
Max V2	1	**SWG**	WIND1	Top	0	166.57	166.57	6510.079	6510.079
Max M3	1	**SWG**	WIND1	Bottom	0	166.57	166.57	8505.443	8505.443
							0		0
Min V2	3	**SWG**	WIND2	Top	0	-22.74	22.74	-358.1	358.0997
Min M3	2	**SWG**	WIND2	Bottom	0	-15.61	15.61	-799.73	799.7304

Fig 40. ETABS Output – Shearwall G

Wind1 represents a load factor of 1.6*W applied to wind case **Case1a** in the governing load combination for wind, and **Wind2** represents 1.6*W for **Case1b**. These combinations were the critical factored load for all shearwalls for shear (V2) and moment (M3). Despite the fact that wind loads resulted in the highest forces in the shearwalls, minimum reinforcing requirements controlled the design of all wall reinforcing.

This result was expected because the size of shearwalls was goverened by a need to limit seismic deflections to a limit far lower than is typically permitted. Consequently, shearwalls are oversized compared to a similar structure that would be "freestanding" as opposed to a structure that is adjacent to an existing building, as is the case with the Gouverneur Healthcare Facility.

Design Procedure –

Excel was utilized to design the uniform longitudonal and horizontal shear reinforcing for the shearwalls. ETABS output was automatically sorted to provide the maximum design forces for a given wall, and the corresponding load case, pier label, etc. (see figure on previous page). This output was then referenced in order to design the uniform longitudonal and horizontal shear reinforcing. This method provided a powerful means to design the walls. After any change in the design occurred, and the ETABS model was revised to reflect that change, the output from the model was put into Excel, and the reinforcing design would automatically update to provide the required spacing for the given reinforcing.

f'c	6000	**Max Permitted Shear**		
fy	60000	1472.35335	OKAY	
t	16			
Lw	198	Shear Strength		
Hw	132	Vc=	393	
d	158.4	0.5ϕVc	147	NEEDS SHEAR REINF

	Minimum Requirements		Req'd Reinforcing
	Horizontal	Vertical	Horizontal Initial
	ρt = 0.0025	ρl = 0.0025	Vs= -179.076
	S = 18	S = 18	Av/S= -0.019
(2) #4	Smin= 10.0	= 10.0	S= -21.229
(2) #5	S min= 15.5	= 15.5	S= -32.905

	Horizontal Final Design		Vertical Final Design	
(2) #4	S= 10	ρ= 0.0025	ρ= 0.0025	S= 10
(2) #5	S= 15.5	ρ= 0.0025	ρ= 0.0025	S= 15.5

Fig 41. Excel Design – Shearwall G

Base design output gives spacing at irregular intervals. Therefore, it is necessary to take the output and round down to the nearest typical spacing. For example, 16" shearwalls required #5's at 15.5" spacing. Therefore, the final design used the same bars at 12" O.C.

Minimum reinforcing requirements governed the shear strength requirements for all shearwalls. This result was expected due to the governing seismic deflection. The figure below shows the final design of all shearwalls in the lateral force resisting system.

Shearwall Schedule						
	Size		Reinforcement			
Wall	Thickness (in)	Length (in)	Long.	Space	Vert.	Spacing
SW2a	16	102	(2) #5	12"	(2) #5	12"
SW2b	16	102	(2) #5	12"	(2) #5	12"
SWB	16	198	(2) #5	12"	(2) #5	12"
SWG	16	198	(2) #5	12"	(2) #5	12"
SW4	20	260	(2) #5	12"	(2) #5	12"
SW5.8	20	260	(2) #5	12"	(2) #5	12"
SWD.8a	20	138	(2) #5	12"	(2) #5	12"
SWD.8b	20	138	(2) #5	12"	(2) #5	12"

Fig 42. Shearwall Schedule

Shearwalls also had to be investigated for the interaction of axial and momen. PCA Column was utilized to input the section properties, reinforcing and factored loads to investigate the design of the reinforcing.

Shearwall G was chosen to illustrate the interaction diagram because it was the shearwall nearest to capacity for the given loads.

For all shearwalls, the minimum reinforcing designed for shear considerations was adequate to resist flexural and axial interaction. Had the reinforcing been inadequate, additional flexural reinforcing would have been added to the ends of the shearwall.

See figure below for reinforcing details.

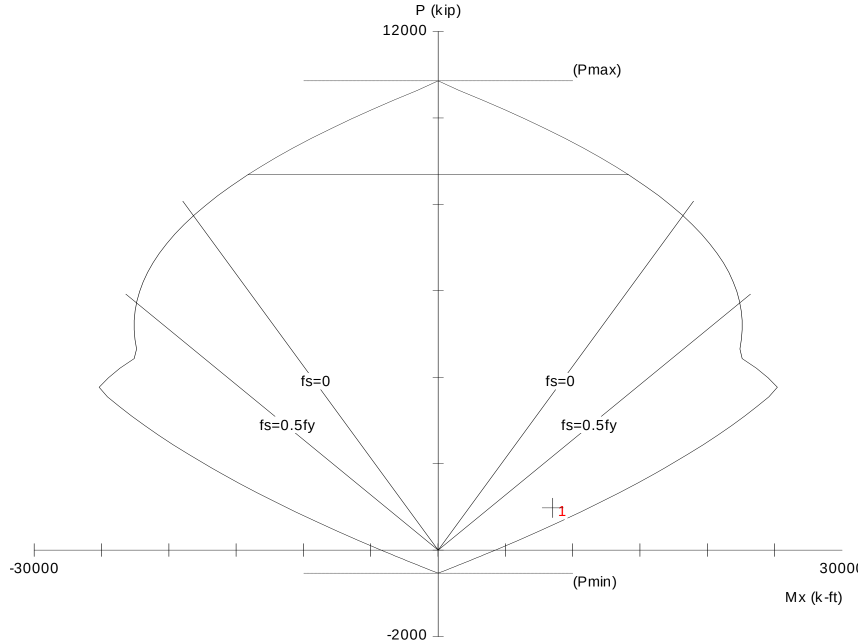


Fig 43. Shearwall G Interaction Diagram

Summary			
SWG		V2 max =	166.57
Lw =	198	M3 max =	8505.44
Tw =	16	Pu =	2187
use (2) #5 @ 12" O.C. each way			

Fig 44. Shearwall G Summary

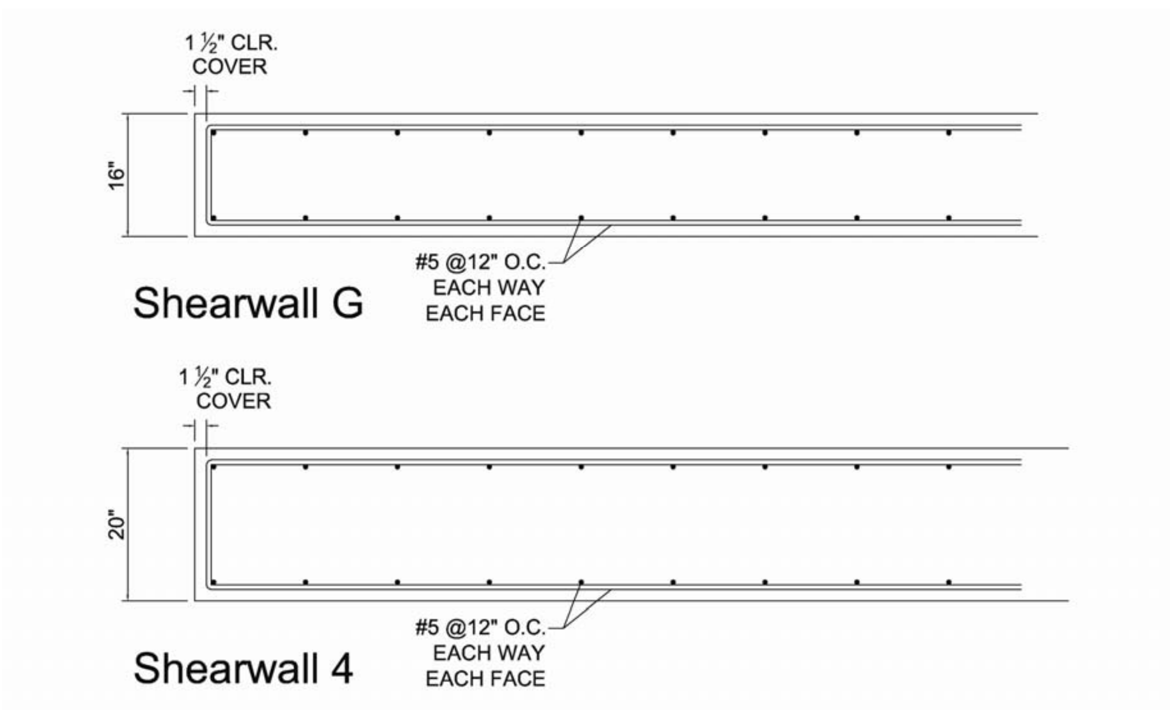


Fig 45. Shearwall Details

Coupling Beams –

Coupling beams connect the two piers of shearwall 2 and shearwall D.8. Beams are 36" deep in order to transfer forces between these two piers. Coupling beam CB1 spans between the two piers of shearwall D.8, and shearwall CB2 spans between the piers of shearwall 2. Due to the span to depth ratio of the coupling beams, it was determined that diagonal reinforcing was not requirement. Because the site of the Gouverneur Healthcare Facility is not in a high seismic zone, standard longitudonal reinforcing was designed as in a standard coupling beam. Steel is sized based on flexural strength and minimum requirements (ACI 318-08 equation 10-3). Longitudonal reinforcing is placed in the bottom and top of beams due to the possibility of load reversals.

ETABS output was compiled using Excel in the same manner as with the shearwalls in order to obtain maximum design forces.

CB8	Story	**Beam**	Load	Loc	P	V2	V3	T	M2	M3
Max M3	8	**CB1**	WIND1	0	0	80.96	0	0	0	421.8824
Min M3	8	**CB1**	WIND1	124.5	0	80.96	0	0	0	-418.056
Max V2	8	**CB1**	WIND1	0	0	80.96	0	0	0	421.8824
Min V2	11	**CB1**	WIND2	0	0	-3.29	0	0	0	-17.0833

Flexural Design							Shear Design				
eqt10-3	As,min=	2.56	ρ=	0.003873	ok			Vc=	102.2	S=	15
	not < 200bd/fy =	2.2	ρ=	0.003333	ok			0.5ϕVc	25.6	Av min =	0.29
	est. As~Mu/4.2d	3.04	ρ=	0.004612	ok			need shear reinf			0.25
							Vs req'd	-12.2912			
Use (4) #8 bars		a=	1.86		ε_s=	0.018		use min reinf			
As=	3.16	c=	2.48		ϕ=	0.9					
		ϕMn=	456.04	ok							

Fig 46. Coupling Beam CB1 Design

CB9	Story	**Beam**	Load	Loc	P	V2	V3	T	M2	M3
Max M3	5	**CB2**	WIND2	0	0	45	0	0	0	157.5092
Min M3	5	**CB2**	WIND2	84	0	45	0	0	0	-157.523
Max V2	5	**CB2**	WIND2	0	0	45	0	0	0	157.5092
Min V2	5	**CB2**	WIND1	0	0	-13.37	0	0	0	-46.661

Flexural Design							Shear Design				
	As,min=	2.04	ρ=	0.003098	ok			Vc=	81.8	S=	15
	not < 200bd/fy =	1.76	ρ=	0.002667	ok			0.5ϕVc	20.4	Av min =	0.23
	est. As~Mu/4.2d	1.14	ρ=	0.001722	ok			need shear reinf			0.2
							Vs req'd	-31.7974			
Use (5) #6 bars		a=	1.62		ε_s=	0.015		use min reinf			
As=	2.2	c=	2.16		ϕ=	0.9					
		ϕMn=	318.69	ok							

Fig 47. Coupling Beam CB2 Design

Summary CB1	36" x 20"
Long. Reinf	**Shear Reinf.**
Use (4) #8 bars	(2) legs #4 @ 15"
As= 3.16	

Summary CB2	36" x 16"
Long. Reinf	**Shear Reinf.**
Use (5) #6 bars	(2) legs #4 @ 15"
As= 2.2	

Fig 48. Couple Beam Design Summary

Parametric Study of Transmission of Lateral Forces –

For elements of the lateral system to resis forces, there needs to be a clear load path. In the Gouverneur Healthcare Facility, this means that lateral forces are distributed to the diaphragm and and subsequently distributed to the shearwalls. In order for forces to transfer from the diaphragm to the shearwalls, their needs to be an interface that is sufficient enough to transmit these forces. For most shearwalls, the wall is cast monolithically with the slab above, therefore satisfying this requirement. There exists a condition on the lower floors that that may conflict with this necessity. See figure below.

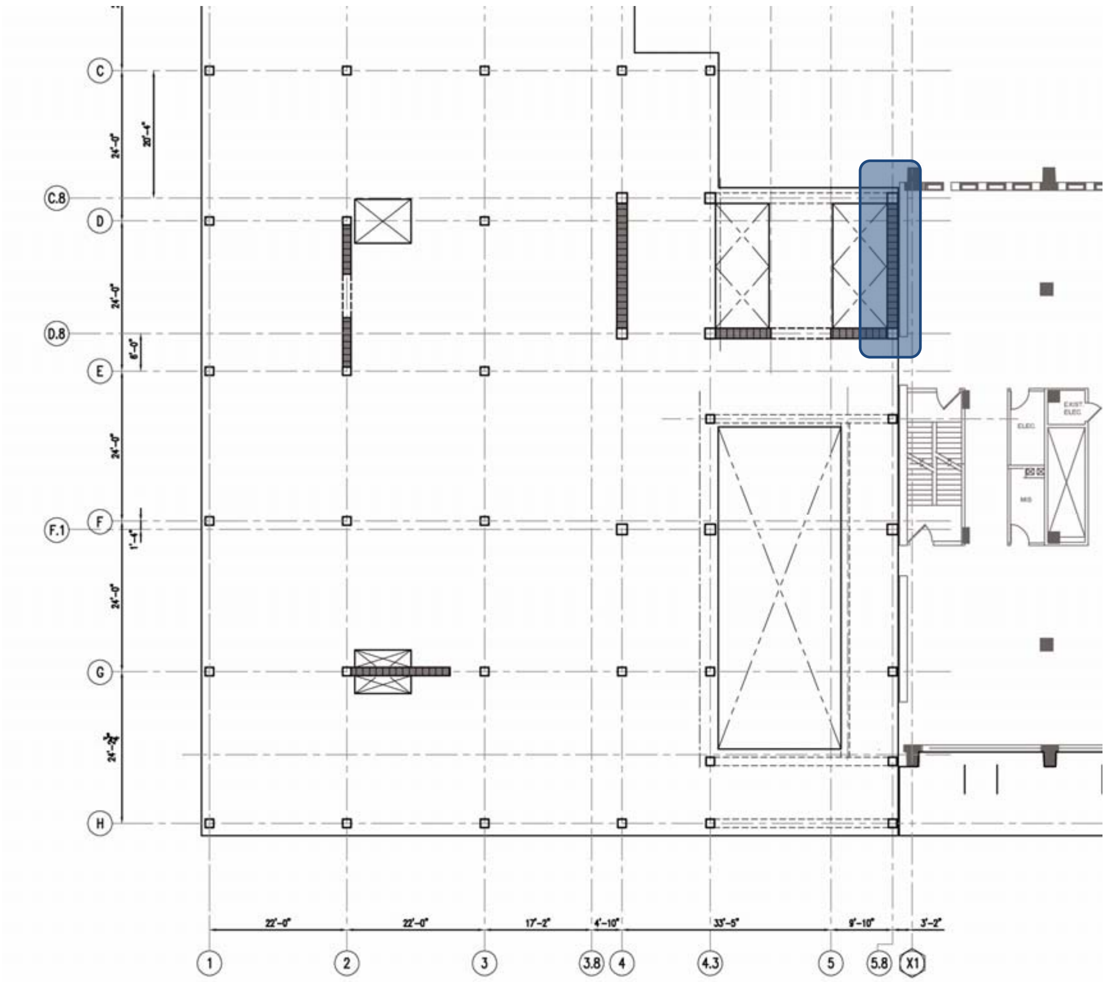


Fig 49. 4th Floor Plan Highlighting Shearwall 5.8

Due to the elevator shaft on the lower floors, shearwall 5.8 does not have a significant interface with the slab. In the ETABS model used to analyze lateral loads, shearwall 5.8 shares only a single node of connection with the floor diaphragm. However, this was enough connection to transmit forces between these elements. It would not necessarily be adequate in the actual behavior of the structure.

It may be possible to design a collector beam to transfer forces to the shearwall so it behaves in agreeance with the model. However, a decision was made to conduct a simple study to determine the behavior of the structure of shearwall 5.8 was isolated from the diaphragm in the lower floors of the building. For the study, Model1 is the original model where SW5.8 and the diaphragm are connected, and Model2 is the model where the two elements are isolated.

The figure at right provides evidence that Model 2 behaved as intended. Because the shear forces in the wall remain constant from the 6th Floor down, this means that the shearwall is independent from the diaphragm and additional lateral forces were not transferred.

From Model 2, the maximum shear and moment forces in each shear wall was compiled and compared to the output from Model 1. For most walls, the values obtained from Model 1 were larger than Model 2, although very close. In shearwall 4 the forces were higher in Model 2, and were then designed in accordence with these values.

WIND2	Model1 Vmax	Model 2 Vmax	Model1 Mmax	Model 2 Mmax
SW4	126	130	9592	9697
SW5.8	195	181	11196	11160
SW2A	69	67	1507	1482
SW2B	69	67	1507	1482

Fig 50. Maximum Design Force Comparison

SW5.8 Comparison Study		
Story	Model 1 V2 (kip)	Model 2 V2 (kip)
HI ROOF	15.89	15.89
13	31.78	31.78
12	47.58	47.58
11	62.5	62.5
10	76.77	76.78
9	90.56	90.56
8	104.58	104.58
7	118.28	118.28
LO ROOF	154.55	180.5
5	191.67	180.5
4	194.92	180.5
3	185.55	180.5
2	183.52	180.5
1	178.95	180.5

Fig 51. SW5.8 Comparison Study

The parametric study revealed that the original model is reasonably accurate for the modeling of the behavior of the structure. It was decided to use the original model for the majority of the analysis because force output was reasonably accurate, and deflection output may have been more accurate in Model 1.

When out-of-plane forces were applied to the structure of Model 2, SW 5.8 was put into axial compression due to its connection to SW D.8. When SW 5.8 was isolated from the diaphragm it behaved in an “unbraced” manner for six stories.

FOUNDATION DESIGN

A prelimenary study was conducted to investigate the impact of the new structural system on the foundations of the Gouverneur Healthcare Facility. The existing system utilized 12” diameter bored mini-piles with an compressive capacity of 100 tons.

The piles and pile caps for the proposed system was designed by comparison to the maximum load on existing piles that were obtained from the existing pile cap schedule. For example, the maximum load on Column D/2 is 1100 kips, therefore a five-pile, pile cap was used, whose maximum load was 1500 kips according the the existing schedule.

See figure on the following page for foundation plans.

An overturning analysis was also conducted to analyze the shearwall interaction with the foundation. The moment at the base of the shearwalls were conservatively resolved into a force couple by dividing by the length of the shearwall. The resulting tension was then substracted from the axial load on the pile cap to determine if there was any overturning. It was determined that none of the shearwalls created a net tension force that needed to be resisted by the pile foundation. See figure at right for sample overturning calculation table.

Overturning	(ft-k, k)	
Moment	2332.35	
Lw	138	
Force Couple	202.81	
Net Force	242.19	C

Fig 52. Overturning Calculation Table

See Appendix E for complete analysis.

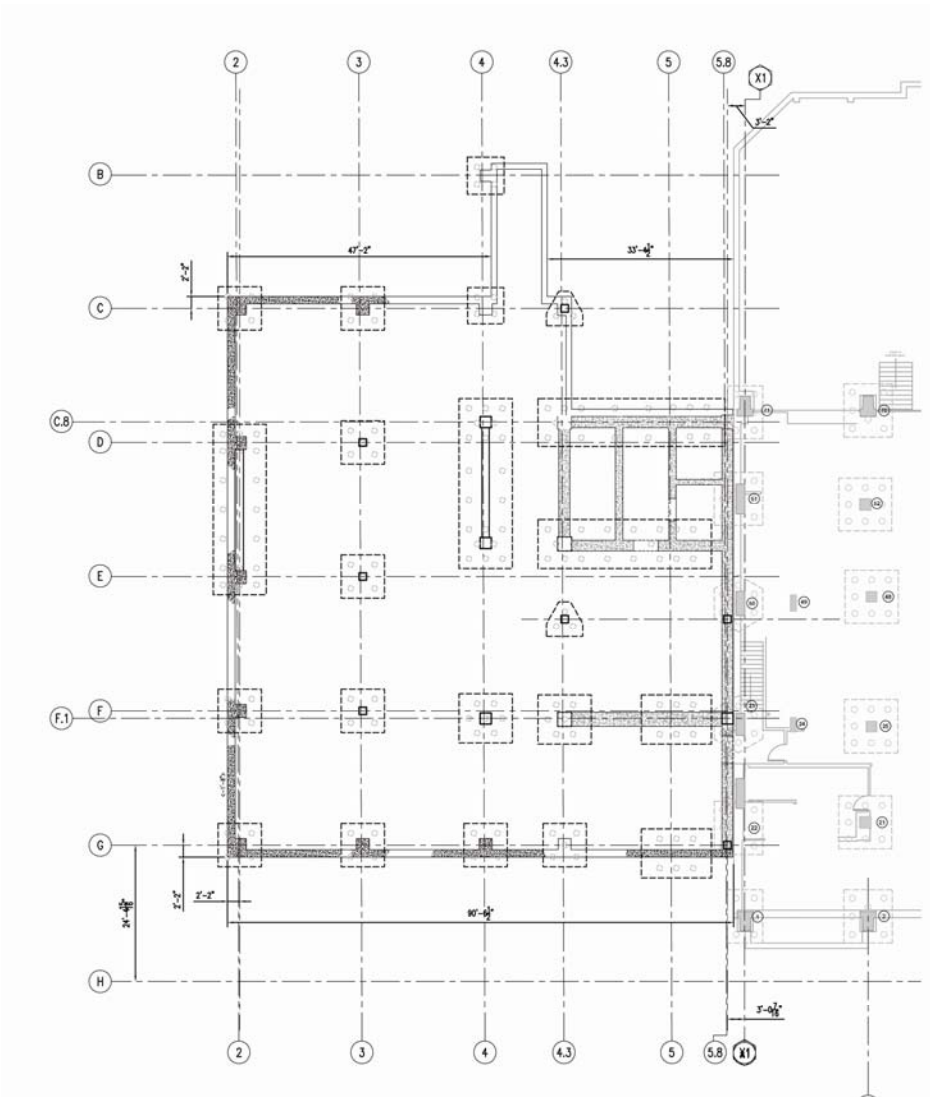


Fig 53. Foundation Plan

ARCHITECTURAL IMPACT

The structural system of the Gouverneur Healthcare Facility was redesigned from a streel framed structure to a concrete system. In order to compete this design change, an extra row of columns were added in both directions, changing typical bay sizes from 22’x30’ and 44’x30’ in the previous design to 22’x24’ typical in the proposed design. Along with this shift in bay size is the change of moment frames to shearwalls, which will arguably impact the architecture to a greater degree.

For these reasons, it was necessary to redesign the layout of the floorplans in order to comply with the new structural system. Because the Gouverneur Healthcare Facility functions as a hostpital, it was of utmost importance to maintain the same relationship between spaces in the redesign process. For example, the location of the Sterile Prep Room is important to the proper use of the Procedure Rooms where operations are performed. Spaces must also follow the same codes; hallways need to remain the proper width, accessible bathrooms need to maintain a 60” turnaround and dead-ends need to be avoided.

Floorplans for the existing layout of the tower addition were unavailable. The design of these spaces were based on the room layouts of the existing long-term dormitories.

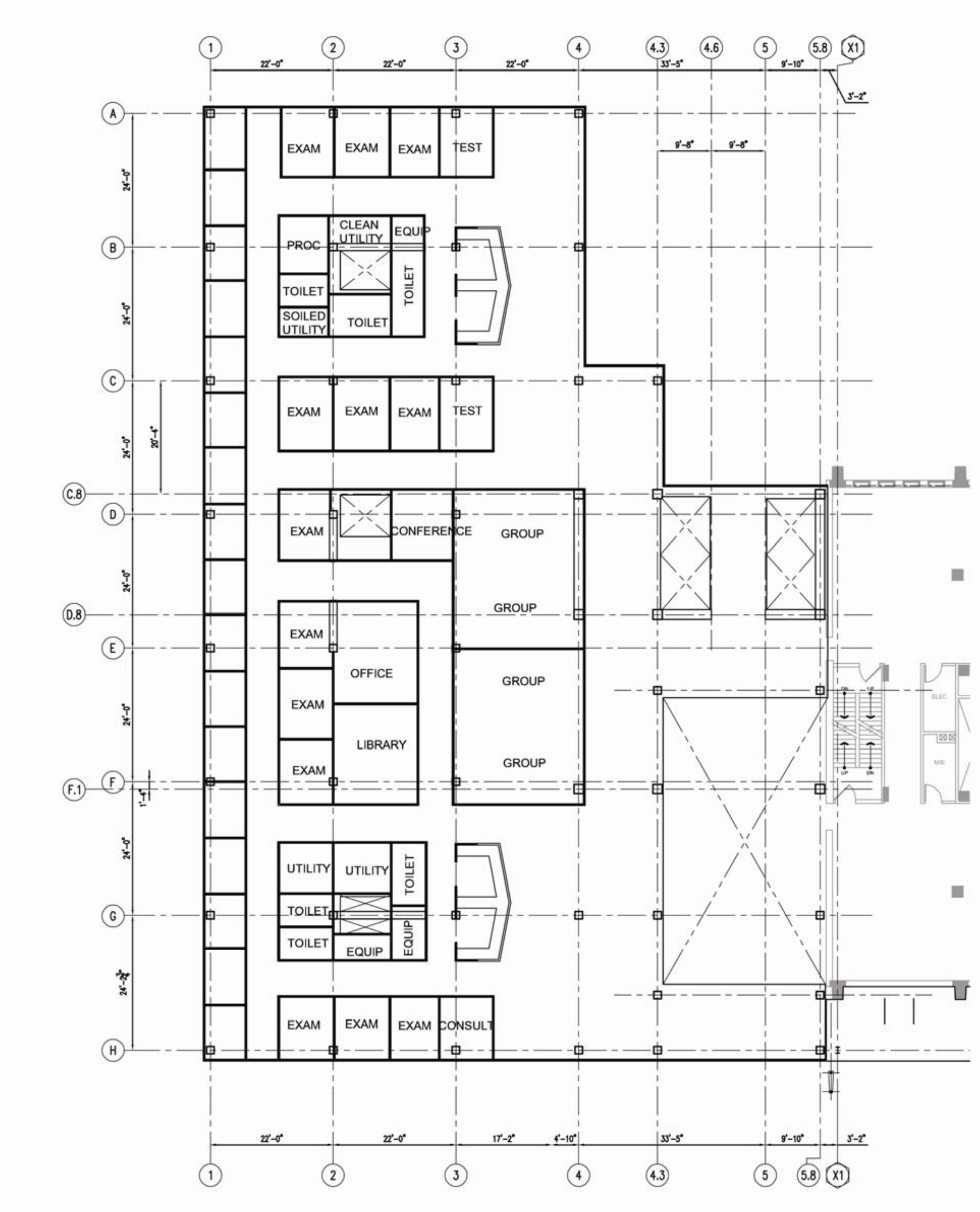


Fig 54. 2nd & 3rd Floor Room Layout Plan

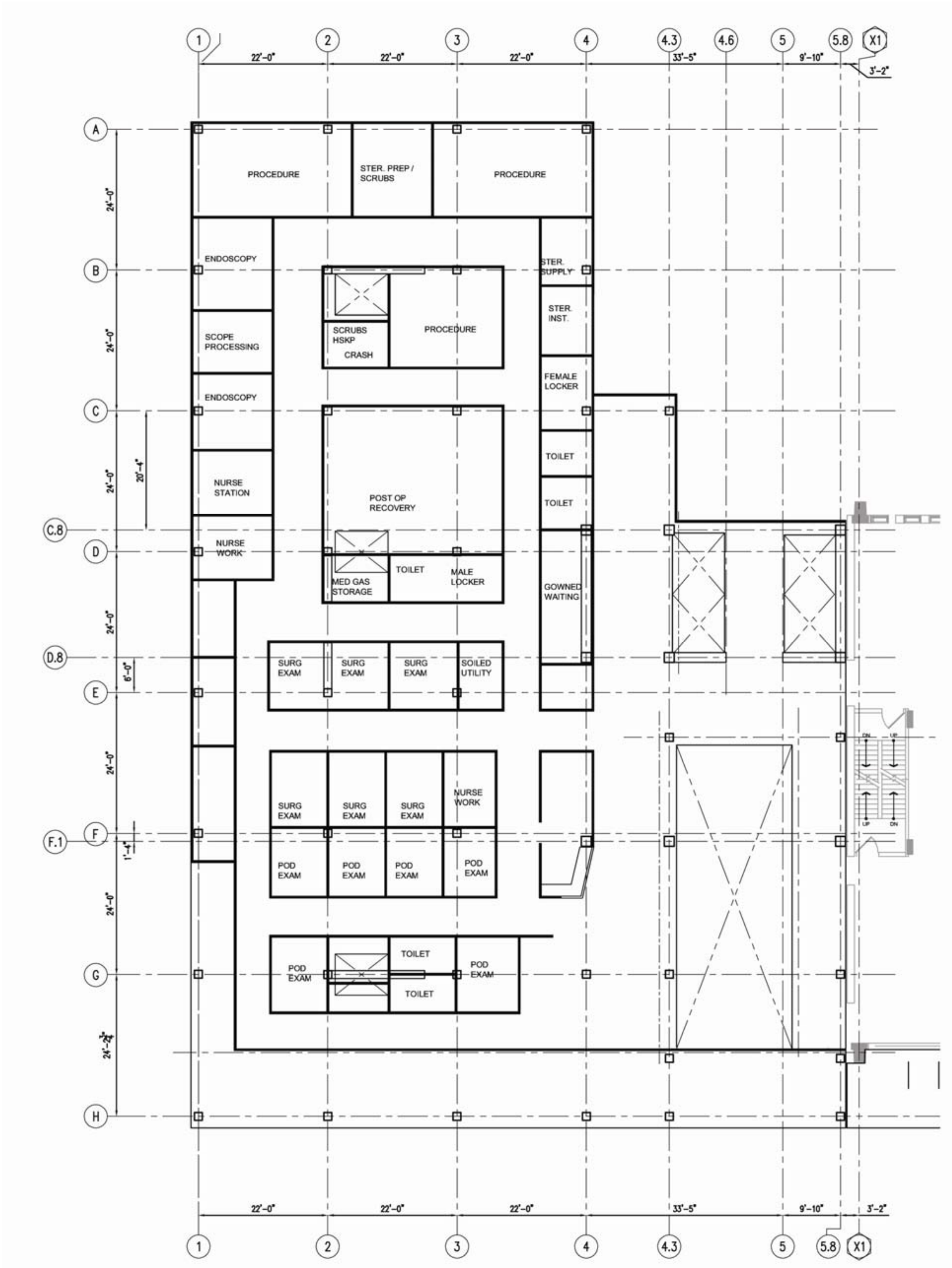


Fig 55. 4th Floor Room Layout Plan

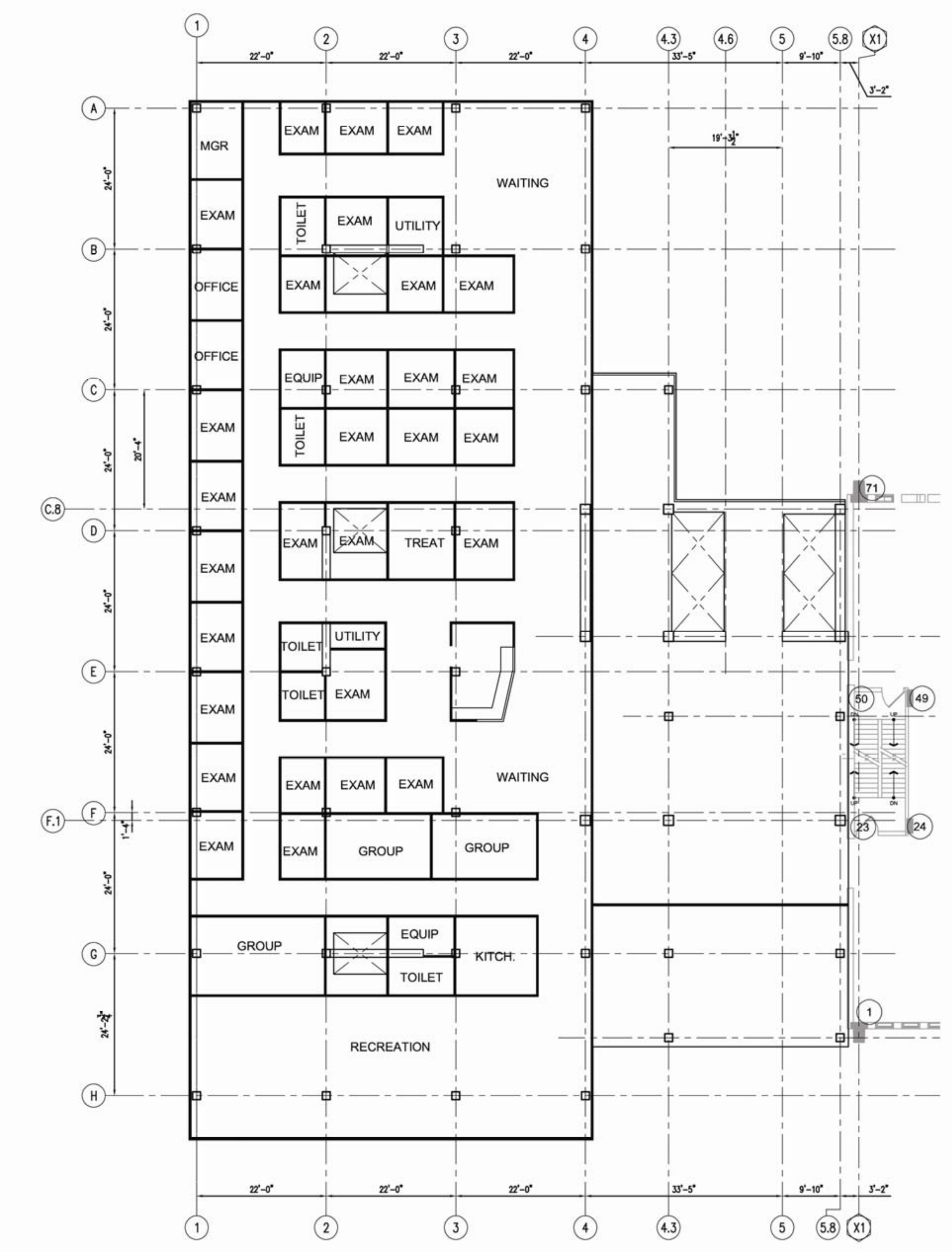


Fig 56. 5th Floor Room Layout Plan

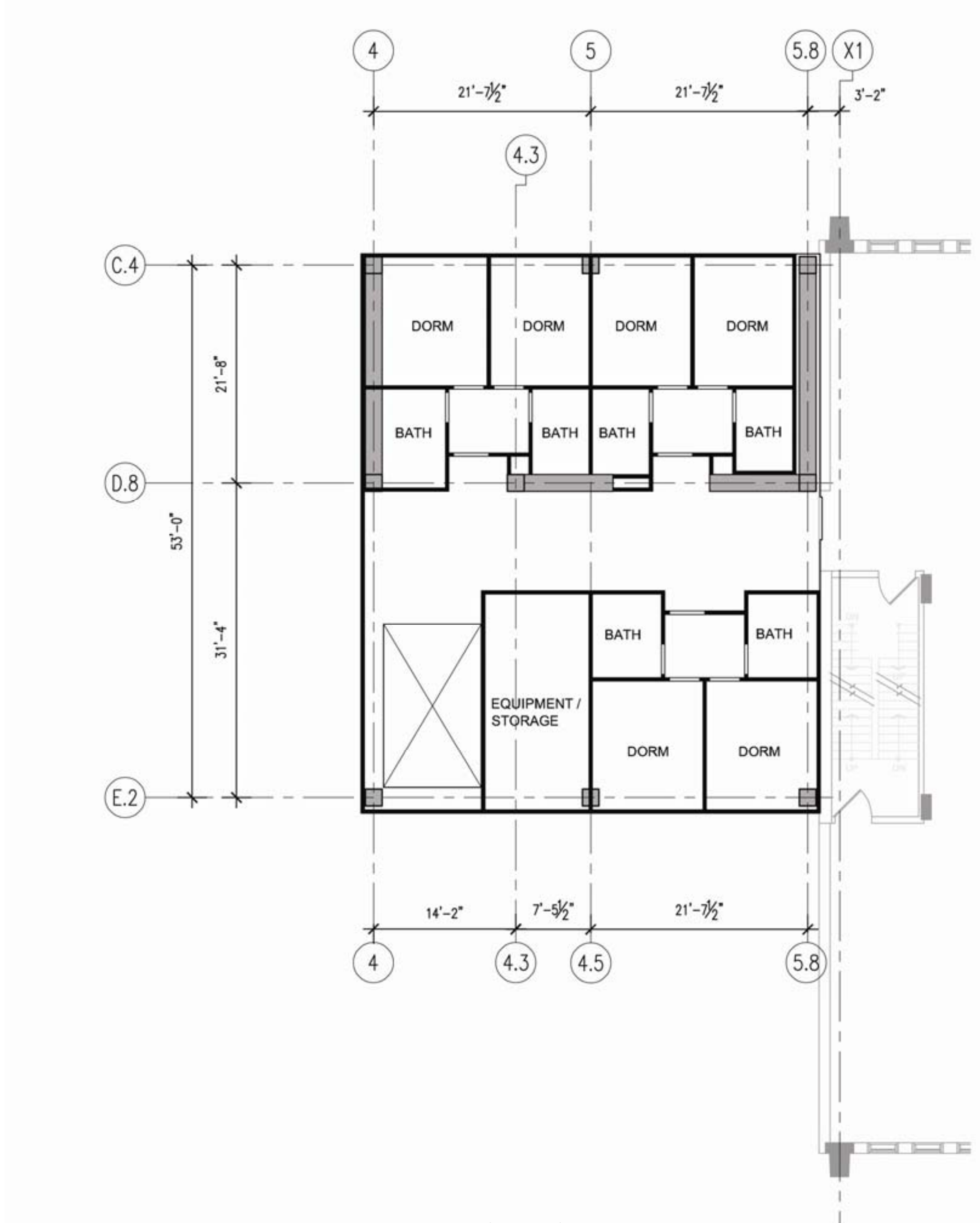


Fig 57. 7th – 13th Floor Room Layout

Construction Impact

An important measure of the success of a structural system is its impact on the cost of a project and the way it affects the construction schedule. In order to conduct an accurate comparison, cost analysis was developed for the portion of the foundation and super structure that changed as a result of the proposed redesign. Added cost due to the increased size of the pile foundation was obtained. Additionally, the cost of steel frame construction with slab on deck was compared to a two-way flat plate construction with shearwalls and square columns. An overhead and profit of 3% were used to calculate costs for the existing and proposed systems. To supplement the estimates, a schedule was developed for the existing and proposed design. All data concerning cost and duration information was obtained using 2008 RS Means Construction Cost Data.

For the existing structure, detailed takeoffs were done using output from RAM Structure and hand calculations. The categories used to estimate the cost of the existing structure is broken down into three main categories: Foundations, Structural Steel, Slab on Deck, and Equipment.
A detailed outline is as follows:

- **Foundation**
  - Piles
  - Formwork
  - Reinforcing
  - Concrete
- **Steel**
  - Structural Steel W-Shapes (including shear studs)
  - Structural Steel Cellular Members
- **Slab-on-Deck**
  - 3" Metal Deck
  - Welded Wire Fabric
  - Concrete
- **Equipment**
  - Pile Auger
  - Crane
  - Pump Truck

Gravity	Count	Weight (lb)	Weight (ton)
Total W Shape	551	388280	194.140
Studs	4777	47770	23.885
Total Cellular	258	345035	172.518
Studs	6781	67810	33.905
Column			
W-Shape	58	50856	25.428
HSS	26	18524	9.262
Frame			
Column	240	333282	166.641
Beams	209	429087	214.544
Brace	94	118163	59.082
Total Structural Steel		1599149.2	799.575
Total Cellular		379538.5	189.769

Fig 58. Existing System Takeoff Summary

For the proposed structure, detailed takeoffs were performed using output from RAM Concept and hand calculations. Categories for the estimate include Foundation, Columns, Walls, Slabs, and Equipment. A detailed breakdown is as follows:

- **Foundation**
  - Piles
  - Formwork
  - Reinforcing
  - Concrete
- **Columns, Walls, and Slabs**
  - Reinforcing
  - Formwork
  - Concrete
- **Equipment**
  - Pile Auger
  - Crane
  - Pump Truck

Slab		
Concrete	cuy	4116.88
Rebar	ton	151.22
Formwork	sf	108782.00
Column		
Concrete	cuy	307.73
Rear	ton	77.20
Formwork	sf	22616.00
Wall		
Concrete	cuy	815.63
Rebar	ton	29.44
Formwork	sf	30627.67
Overall		
Concrete	cuy	5240.24
Rebar	ton	257.86
Formwork	sf	162025.67

Fig 59. Proposed System Takeoff Summary

	Ext. Mat.	Ext. Labor	Ext. Equip.	**Ext. Total**	Ext. Mat.	Ext. Labor	Ext. Equip.	**Ext. Total O&P**	**Total**
STEEL	$ 2,499,334	$ 517,484	$ 153,911	$ 3,170,730	$ 2,748,275	$ 910,631	$ 169,192	$ 3,828,099	**$ 6,998,829**
SLAB ON DECK	$ 525,088	$ 151,849	$ 16,997	$ 693,933	$ 578,300	$ 254,510	$ 18,166	$ 850,975	**$ 1,544,909**
FOUNDATIO	$ 221,129	$ 103,059	$ 82,177	$ 406,364	$ 243,239	$ 161,164	$ 90,283	$ 494,687	**$ 901,050**
EQUIPMENT	$ -	$ 67,637	$ 340,282	$ 407,919	$ -	$ 102,395	$ 374,566	$ 476,960	**$ 884,879**
	$ 3,245,551	$ 840,028	$ 593,367	**$ 4,678,946**	$ 3,569,815	$ 1,428,699	$ 652,207	**$ 5,650,721**	
								Total	**$ 10,329,667**

Fig 60. Existing System Estimate Summary

	Ext. Mat.	Ext. Labor	Ext. Equip.	**Ext. Total**	Ext. Mat. O&P	Ext. Labor O&P	Ext. Equip. O&P	**Ext. Total O&P**	**Total**
CONCRETE	$ 807,762	$ -	$ -	$ 807,762	$ 892,491	$ -	$ -	$ 892,491	**$ 1,700,253**
FOUNDATIOI	$ 263,334	$ 146,147	$ 169,123	$ 578,604	$ 290,172	$ 226,057	$ 186,662	$ 702,891	**$ 1,281,495**
SLAB	$ 357,860	$ 907,391	$ 21,109	$ 1,286,361	$ 396,051	$ 1,423,441	$ 23,178	$ 1,842,670	**$ 3,129,031**
COLUMNS	$ 101,617	$ 317,591	$ 3,090	$ 422,297	$ 111,433	$ 500,739	$ 3,394	$ 615,565	**$ 1,037,863**
WALLS	$ 55,800	$ 317,502	$ 6,158	$ 379,460	$ 61,093	$ 494,496	$ 6,770	$ 562,359	**$ 941,820**
EQUIPMENT	$ -	$ 135,274	$ 633,027	$ 768,301	$ -	$ 204,789	$ 696,841	$ 901,630	**$ 1,669,930**
	$ 1,586,373	$ 1,823,905	$ 832,507	**$ 4,242,785**	$ 1,751,240	$ 2,849,522	$ 916,845	**$ 5,517,607**	
								Total	**$ 9,760,392**

Fig 61. Proposed System Estimate Summary

As can be seen in the figure above, the proposed cost of structure construction is less expensive than the existing. Although the cost savings represents a $570,000 or 5.5% decrease in structural cost, it does not represent a large decrease in overall project cost. The addition to the Gouverneur Healthcare Facility is only a small part in the modernization of the entire hospital. When including the construction of accompanying systems of the addition and the renovation of the existing hospital, the cost is estimated to top out at $160 million total project cost. The savings represents less than 1% savings. Therefore, in order to determine the success of the proposed redesign, other criteria need to be used.

Schedule Impact

Because of the proposed change from a steel system to a concrete system, it was anticipated that there would be a great impact to the construction schedule. In order to determine this impact, schedules were developed using Microsoft Project. Crew sizes and daily output from RS Means determined the durations of construction for a given portion of the system.

In the existing system, it was assumed that construction was sequenced to build each of the lower floors in two portions, while the upper floors were constructed in one sequence. This allowed a more efficient use of crew sizes while keeping construction durations to expected times.

In the proposed system, it was assumed that the slab would be constructed in three sequences, in order to maintain one day slab pouring durations for an individual portion. This allowed floors to be built as quickly as possible while keeping crew sizes at a minimum.

The duration of construction for the existing system was estimated to be 6 ½ months from the commencement of foundation drilling to the topping out of the structure. The duration for the proposed system was calculated to be 12 months and 1 week from foundation to topping out of structure. Although the timeframe of the proposed structure is significantly longer, it is not anticipated to be a great concern. The overall project, including renovation, is anticipated to last almost 4 years, and immediate revenue generation is not an issue as is the case in many other building projects.

See figures on the following page for sample sequencing for the 4th Floor.

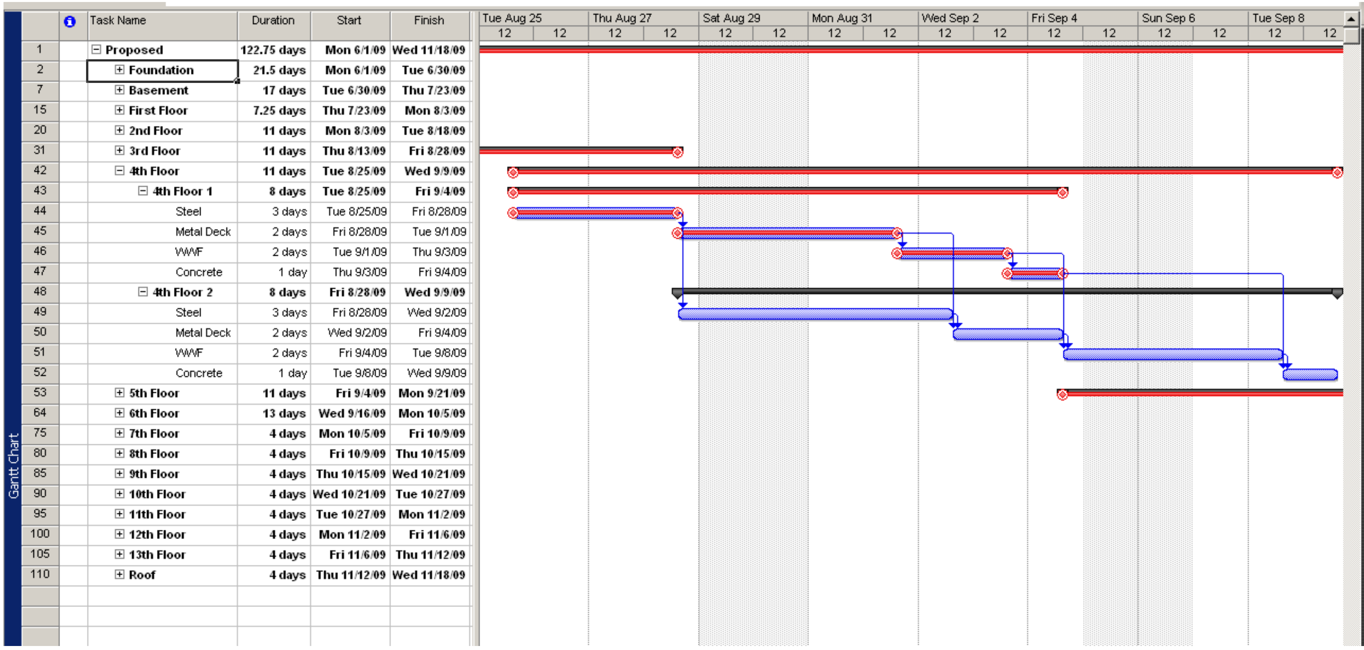


Fig 62. Existing System Construction Schedule

Each portion considers Steel, Metal Deck, WWF, and Concrete

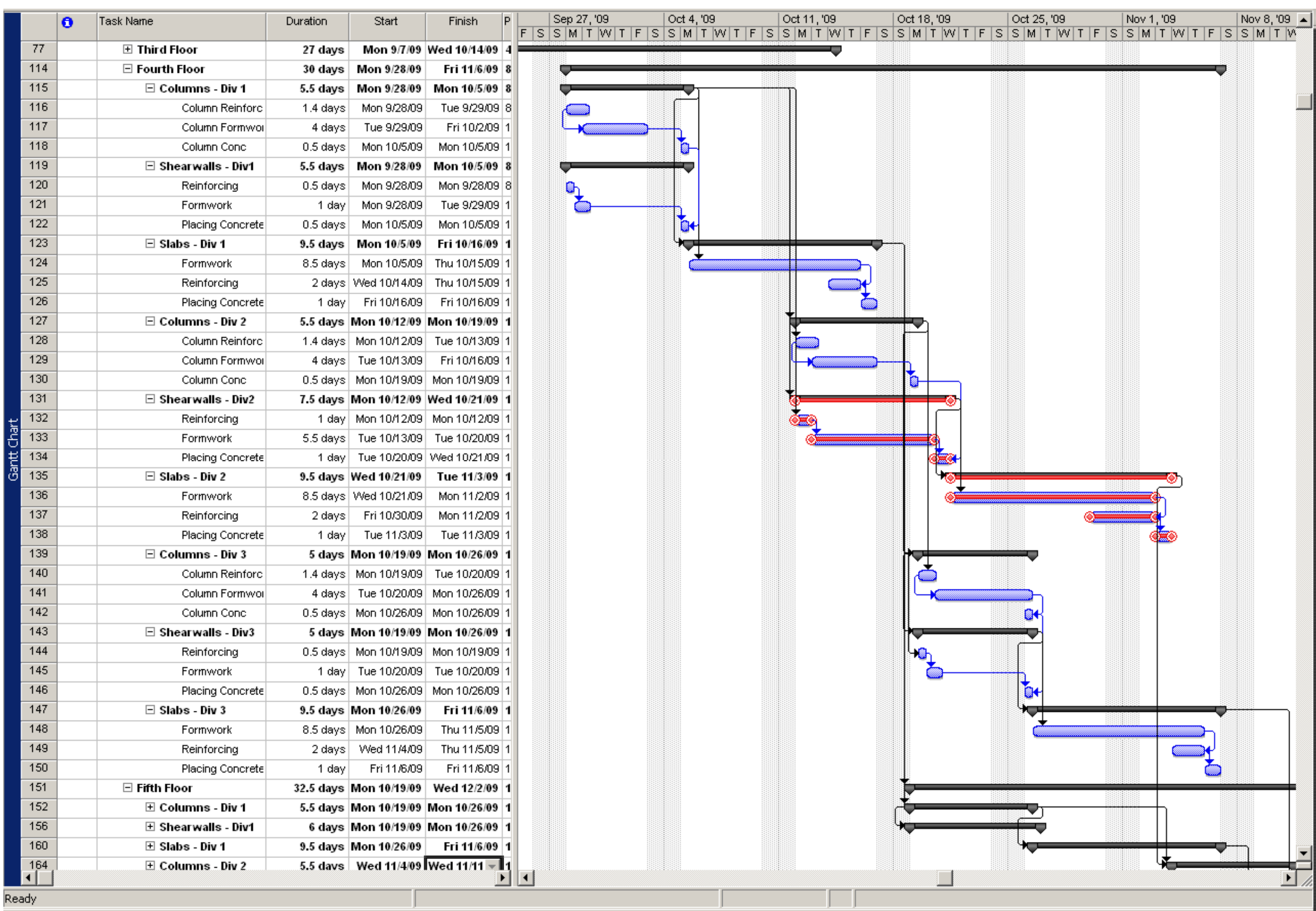


Fig 63. Proposed System Construction Schedule

Each portion considers Reinforcing, Formwork and Concrete for Columns, Shearwalls, and Slabs.

Conclusions and Recommendations

The proposed structural redesign is an attractive solution for many of the design problems necessitated by the project. Therefore, it is recommended that the proposed system be considered an appropriate alternative because the benefits include freedom of design for all systems, ease of construction and cost benefits.

The structure was designed as a 12” two-way flat plate floor system with 16” and 20” columns and shearwalls. Floor deflections are kept within the maximum limits of L/360 for immediate live load deflections and L/480 for long-term deflections. Lateral drifts are kept under the 3” upper limit imposed by the adjacent building.

A two-way flat plate floor system will create an unobstructed space between the drop ceiling the floor slab above with adequate room to run all mechanical systems. Because there are no moment frames dividing the floor into isolated zones, MEP engineers will be more able to design the systems more easily. They will be able to match the needs of the individual areas, not conform to the design of the structure. The scope of this thesis did not include an investigation of the impact on mechanical systems. However, mechanical systems are a large portion of both the construction and operating cost of a building; any opportunity to make the system more efficient is desirable.

Due to the regularity of the proposed system, the construction of the building will be made easier. Although concrete structures take longer to erect than steel, there are still benefits. The existing system has a significant number of moment connections and skewed beams, driving up labor and fabrications costs. The proposed system has a regular scheme with a fairly typical reinforcing layout. In the existing structure, laborers will also have to maneuver mechanical systems through the penetrations of the cellular beams. In the proposed system, there will be an unobstructed space below the slab to install MEP equipment more easily.

After conducting a cost comparison, it was determined that the proposed structure saved $570,000 of the cost of the structure. However, when considering the total cost of the project, this is a savings of less than 1%. A schedule comparison reveals an estimated construction time of 12 months, compared to 6.5 months for the existing addition. Because the Gouverneur Healthcare Facility is a hospital owned by the NYC HHC, immediate revenue generation is not an issue, and the longer estimated schedule time does not negatively affect the feasibility of the proposed design.

Overall, this thesis project reveals a need for coordination among engineers, designers and construction managers. It is assumed that design choices were originally made whose impact on other systems was not fully considered. The decision to use steel framing may have been made without considering the impact on MEP systems. During the schematic design process of the original addition, a row of columns was removed without considering the efficiency of moment frames, and column grids were skewed for architectural reasons without considering the layout of the framing. It was determined in this study, that if these systems are designed by a cohesive team, more coordinated solutions for the problems of a project can be achieved.

For example, architects may have originally wanted engineers to use moment frames to keep an open plan and to have unrestricted design freedom. Meanwhile, structural engineers needed to contain lateral deflections to a small amount due to the adjacent building, which is difficult due to the inherent flexibility of moment frames. This thesis report reveals that architects and engineers can manage their efforts to create a shearwall scheme that fit with the layout of the floorplan, while also providing adequate stiffness more efficiently.

APPENDIX A LIST OF FIGURES

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Wind E-W

		Height	Kz,Kh	q_z	q_h	$q_zG_fC_p$	$q_hG_fC_p$	$q_h(GC_{pi})$	Windward p_z	Leeward p_h	Total	Total	Overturning
		(ft)							(psf)	(psf)	(psf)	(kip)	(ft-k)
Zone 2		**154.00**	1.12	27.98	27.98	23.08	-14.43	5.04	28.12	-9.39	37.51	27.8	4091
		140.00	1.09	27.22	27.98	22.46	-14.43	5.04	27.50	-9.39	36.89	39.1	5083
		120.00	1.04	26.05	27.98	21.50	-14.43	5.04	26.53	-9.39	35.92	38.1	4189
		100.00	0.99	24.73	27.98	20.40	-14.43	5.04	25.44	-9.39	34.83	18.5	1754
		90.00	0.96	24.00	27.98	19.80	-14.43	5.04	24.83	-9.39	34.23	18.1	1542
		80.00	0.93	23.20	27.98	19.14	-14.43	5.04	24.18	-9.39	33.57	17.8	1334
		70.00	0.89	22.33	27.98	18.43	-14.43	5.04	23.46	-9.39	32.85	17.4	1132
		60.00	0.85	21.37	27.98	17.63	-14.43	5.04	22.67	-9.39	32.06	4.2	244
Zone 1	Lower Roof	***57.55***	0.84	21.12	21.12	15.29	-9.56	3.80	19.09	-5.76	24.85	31.7	1705
		50.00	0.81	20.29	21.12	14.69	-9.56	3.80	18.49	-5.76	24.25	41.0	1843
		40.00	0.76	19.03	21.12	13.78	-9.56	3.80	17.58	-5.76	23.34	39.4	1380
		30.00	0.70	17.53	21.12	12.70	-9.56	3.80	16.50	-5.76	22.25	18.8	517
		25.00	0.67	16.64	21.12	12.05	-9.56	3.80	15.85	-5.76	21.61	18.2	411
		20.00	0.62	15.61	21.12	11.31	-9.56	3.80	15.11	-5.76	20.86	17.6	308
		15.00	0.57	14.38	21.12	10.41	-9.56	3.80	14.22	-5.76	19.97	50.6	379
											Base Shea	**398.3**	**25912**
												(kip)	(ft-k)

Wind X

		Floor Elev.	lev. abov datum	Story Height	Story Force	Story Shear
		(ft)	(ft)	(ft)	(kip)	(kip)
Zone 2	main roof	154.00	171.01	11.98	23.6	23.6
	13	138.28	159.03	11.98	23.6	47.1
	12	126.30	147.05	11.98	23.4	70.6
	11	114.32	135.07	11.20	22.1	92.6
	10	103.13	123.88	11.20	21.2	113.8
	9	91.93	112.68	11.20	20.4	134.2
	8	80.73	101.48	11.98	20.8	155.0
	7	68.75	89.50	11.20	20.4	175.4
Zone 1	6	57.55	78.30	11.98	34.8	210.2
	5	45.57	66.32	11.20	47.6	257.8
	4	34.38	55.13	11.20	43.9	301.8
	3	23.18	43.93	11.20	41.0	342.7
	2	11.98	32.73	11.98	39.5	382.2
	Ground	0.00	20.75	0.00	0.0	382.2
		Datum	20.75		**382.2**	Total

Wind N-S

		Height	Kz,Kh	q_z	q_h	$q_zG_fC_p$	$q_hG_fC_p$	$q_h(GC_{pi})$	Windward p_z	Leeward p_h	Total	Total	Overturning
		(ft)							(psf)	(psf)	(psf)	(kip)	(ft-k)
Zone 2		**154.00**	1.12	27.98	27.98	23.41	-14.63	5.04	28.45	-9.60	38.05	23.3	3422
		140.00	1.09	27.22	27.98	22.78	-14.63	5.04	27.82	-9.60	37.42	32.7	4252
		120.00	1.04	26.05	27.98	21.80	-14.63	5.04	26.84	-9.60	36.44	31.9	3504
		100.00	0.99	24.73	27.98	20.70	-14.63	5.04	25.73	-9.60	35.33	15.4	1467
		90.00	0.96	24.00	27.98	20.08	-14.63	5.04	25.12	-9.60	34.71	15.2	1290
		80.00	0.93	23.20	27.98	19.42	-14.63	5.04	24.45	-9.60	34.05	14.9	1116
		70.00	0.89	22.33	27.98	18.69	-14.63	5.04	23.73	-9.60	33.32	14.6	947
		60.00	0.85	21.37	27.98	17.89	-14.63	5.04	22.92	-9.60	32.52	3.5	204
Zone 1	Lower Roof	***57.55***	0.84	21.12	21.12	15.88	-9.92	3.80	19.68	-6.12	25.80	21.9	1178
		50.00	0.81	20.29	21.12	15.25	-9.92	3.80	19.05	-6.12	25.18	28.3	1274
		40.00	0.76	19.03	21.12	14.31	-9.92	3.80	18.11	-6.12	24.24	27.2	954
		30.00	0.70	17.53	21.12	13.18	-9.92	3.80	16.98	-6.12	23.11	13.0	357
		25.00	0.67	16.64	21.12	12.51	-9.92	3.80	16.31	-6.12	22.44	12.6	284
		20.00	0.62	15.61	21.12	11.74	-9.92	3.80	15.54	-6.12	21.66	12.2	213
		15.00	0.57	14.38	21.12	10.81	-9.92	3.80	14.61	-6.12	20.74	35.0	262
											Base Shea	**301.6**	**20724**
												(kip)	(ft-k)

Wind Y

		Floor Elev.	lev. abov datum	Story Height	Story Force	Story Shear
		(ft)	(ft)	(ft)	(kip)	(kip)
Zone 2	main roof	154.00	171.01	11.98	19.7	19.7
	13	138.28	159.03	11.98	19.7	39.4
	12	126.30	147.05	11.98	19.6	59.0
	11	114.32	135.07	11.20	18.5	77.5
	10	103.13	123.88	11.20	17.7	95.2
	9	91.93	112.68	11.20	17.1	112.3
	8	80.73	101.48	11.98	17.4	129.7
	7	68.75	89.50	11.20	17.0	146.7
Zone 1	6	57.55	78.30	11.98	34.2	180.9
	5	45.57	66.32	11.20	32.9	213.8
	4	34.38	55.13	11.20	30.4	244.2
	3	23.18	43.93	11.20	28.3	272.5
	2	11.98	32.73	11.98	27.3	299.8
	Ground	0.00	20.75	0.00	14.0	313.7
		Datum	20.75		**313.7**	Total

Case 1.a			
Story	Fx	Fy	Mz
Main Roof	23.6		
13.00	23.6		
12.00	23.4		
11.00	22.1		
10.00	21.2		
9.00	20.4		
8.00	20.8		
7.00	20.4		
6.00	34.8		
5.00	47.6		
4.00	43.9		
3.00	41.0		
2.00	39.5		
Ground	-		

Case 1.b			
	Fx	Fy	Mz
Main Roof		19.7	
13.00		19.7	
12.00		19.6	
11.00		18.5	
10.00		17.7	
9.00		17.1	
8.00		17.4	
7.00		17.0	
6.00		34.2	
5.00		32.9	
4.00		30.4	
3.00		28.3	
2.00		27.3	
Ground		-	

Case 2.a			
	Fx	Fy	Mz
Main Roof	17.7		140.5009
13.00	17.7		140.5009
12.00	17.6		139.6602
11.00	16.6		131.6697
10.00	15.9		126.2933
9.00	15.3		121.5894
8.00	15.6		124.2695
7.00	15.3		121.4078
6.00	26.1		661.0274
5.00	35.7		904.8642
4.00	32.9		834.6685
3.00	30.7		778.7116
2.00	29.6		750.2388
Ground	-		-

Case2.b			
	Fx	Fy	Mz
Main Roof	17.7		-140.5
13.00	17.7		-140.5
12.00	17.6		-139.7
11.00	16.6		-131.7
10.00	15.9		-126.3
9.00	15.3		-121.6
8.00	15.6		-124.3
7.00	15.3		-121.4
6.00	26.1		-661.0
5.00	35.7		-283.9
4.00	32.9		-261.9
3.00	30.7		-244.4
2.00	29.6		-235.4
Ground	-		-

case2.c			
	Fx	Fy	Mz
Mail Roof		14.8	96.9
13.00		14.8	96.9
12.00		14.7	96.3
11.00		13.9	90.8
10.00		13.3	87.1
9.00		12.8	83.9
8.00		13.1	85.7
7.00		12.8	83.7
6.00		25.6	432.2
5.00		24.7	416.2
4.00		22.8	383.9
3.00		21.2	358.2
2.00		20.5	345.1
Ground		-	-

Case 2.d			
	Fx	Fy	Mz
Main Roof		14.8	-96.9
13.00		14.8	-96.9
12.00		14.7	-96.3
11.00		13.9	-90.8
10.00		13.3	-87.1
9.00		12.8	-83.9
8.00		13.1	-85.7
7.00		12.8	-83.7
6.00		25.6	-432.2
5.00		24.7	-416.2
4.00		22.8	-383.9
3.00		21.2	-358.2
2.00		20.5	-345.1
Ground		-	-

Case 3.a			
	Fx	Fy	Mz
Main Roof	17.7	14.8	
13.00	17.7	14.8	
12.00	17.6	14.7	
11.00	16.6	13.9	
10.00	15.9	13.3	
9.00	15.3	12.8	
8.00	15.6	13.1	
7.00	15.3	12.8	
6.00	26.1	25.6	
5.00	35.7	24.7	
4.00	32.9	22.8	
3.00	30.7	21.2	
2.00	29.6	20.5	
Ground	-	-	

Case 3.b			
	Fx	Fy	Mz
Main Roof	-17.7	-14.8	
13.00	-17.7	-14.8	
12.00	-17.6	-14.7	
11.00	-16.6	-13.9	
10.00	-15.9	-13.3	
9.00	-15.3	-12.8	
8.00	-15.6	-13.1	
7.00	-15.3	-12.8	
6.00	-26.1	-25.6	
5.00	-35.7	-24.7	
4.00	-32.9	-22.8	
3.00	-30.7	-21.2	
2.00	-29.6	-20.5	
Ground	-	-	

Case 4.a			
	Fx	Fy	Mz
Main Roof	13.3	11.1	178.2
13.00	13.3	11.1	178.2
12.00	13.2	11.0	177.2
11.00	12.4	10.4	167.0
10.00	11.9	10.0	160.2
9.00	11.5	9.6	154.2
8.00	11.7	9.8	157.6
7.00	11.5	9.6	154.0
6.00	19.6	19.2	820.7
5.00	26.8	18.5	991.7
4.00	24.7	17.1	914.8
3.00	23.1	15.9	853.4
2.00	22.2	15.4	822.2
Ground	-	-	-

Case 4.b			
	Fx	Fy	Mz
Main Roof	13.3	11.1	-178.2
13.00	13.3	11.1	-178.2
12.00	13.2	11.0	-177.2
11.00	12.4	10.4	-167.0
10.00	11.9	10.0	-160.2
9.00	11.5	9.6	-154.2
8.00	11.7	9.8	-157.6
7.00	11.5	9.6	-154.0
6.00	19.6	19.2	-820.7
5.00	26.8	18.5	-991.7
4.00	24.7	17.1	-914.8
3.00	23.1	15.9	-853.4
2.00	22.2	15.4	-822.2
Ground	-	-	-

Case 4.c			
	Fx	Fy	Mz
Main Roof	-13.3	-11.1	-178.1
13.00	-13.3	-11.1	-178.1
12.00	-13.2	-11.0	-177.0
11.00	-12.4	-10.4	-166.9
10.00	-11.9	-10.0	-160.1
9.00	-11.5	-9.6	-154.1
8.00	-11.7	-9.8	-157.5
7.00	-11.5	-9.6	-153.9
6.00	-19.6	-19.2	-819.9
5.00	-26.8	-18.5	-990.8
4.00	-24.7	-17.1	-914.0
3.00	-23.1	-15.9	-852.7
2.00	-22.2	-15.3	-821.5
Ground	-	-	-

Case 4.d			
	Fx	Fy	Mz
Main Roof	-13.3	-11.1	178.1
13.00	-13.3	-11.1	178.1
12.00	-13.2	-11.0	177.0
11.00	-12.4	-10.4	166.9
10.00	-11.9	-10.0	160.1
9.00	-11.5	-9.6	154.1
8.00	-11.7	-9.8	157.5
7.00	-11.5	-9.6	153.9
6.00	-19.6	-19.2	819.9
5.00	-26.8	-18.5	990.8
4.00	-24.7	-17.1	914.0
3.00	-23.1	-15.9	852.7
2.00	-22.2	-15.3	821.5
Ground	-	-	-

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Occupancy Type		IV
Occupancy Importance Factor		1.15
Site Class		B
Seismic Design Category		B
Height Above Grade [ft]	h_n	154.00
Short Period Spectral Response	S_S	0.363
Spectral Response at 1 Second	S_1	0.070
Maximum Short Period Spectral Reponse	S_{MS}	0.363
Maximum Spectral Reponse at 1 Second	S_{M1}	0.070
Design Short Period Spectral Response	S_{DS}	0.242
Design Spectral Response at 1 Second	S_{D1}	0.047
Period Parameter 1	C_t	0.02
Period Parameter 2	x	0.75
Response Modification Coefficient	R	4
Approximate Fundamental Period	T_a	0.874
Fundamental Period	T	
Long-Period Transition Period	T_L	6.000
Short-Period Transition Period	T_S	0.194
Seismic Response Coefficient	C_S	0.070
Maximum Required Cs Value	$C_{S.max}$	**0.015**
Max Cs per ASCE7-12.8.1.1	C_S	0.01
Effective Weight	**W**	19580
Base Shear	V	**302.61**
Overturning Moment	M	25247.1

Deflection Amplification Factor	C_d	4
System Overstrength Factor	Ω_0	2.5

Story	**Floor Height**	**Floor Weight**	$w_i h^k_i$	C_{vx}	**Story Force**	**Story Shear**
Main Roof	156.00	595.7	92922	0.096	29.046	29.046
13	140.3	571.2	80135	0.083	25.049	54.095
12	128.30	571.2	73292	0.076	22.910	77.005
11	116.3	571.2	66449	0.069	20.771	97.776
10	105.13	571.2	60052	0.062	18.771	116.548
9	93.9	571.2	53655	0.055	16.772	133.320
8	82.73	571.2	47258	0.049	14.772	148.092
7	70.8	571.2	40415	0.042	12.633	160.726
6	59.55	2478.0	147568	0.152	46.128	206.853
5	47.6	2489.8	118446	0.122	37.025	243.878
4	36.38	2486.9	90461	0.093	28.277	272.155
3	25.2	2486.9	62613	0.065	19.572	291.727
2	13.98	2489.8	34805	0.036	10.880	302.607
	(ft)	(kip)			302.61	
					Base Shear	

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Load Takedown			
Interior - Lower only	D2	Size	16
# Floors	6	f'c	6
Trib Area	660		
Infl. Area	2640		
DL	176		
LL	80		
LLr	150		
Total DL	696960	**697.0**	
Total LL	310200	310.2	
LL Reduction	0.54		
Reduced LL	168109	**168.1**	
1.2D+1.6L	1.1E+06	**1105.3**	29.5
1.4D	975744	**975.7**	
		(kips)	

Load Takedown			
Exterior - Lower only	1C	Size	16
# Floors	6	f'c	6
Trib Area	330		
Infl. Area	1320		
DL	176		
LL	80		
Total DL	348480	**348.5**	
Total LL	158400	158.4	
LL Reduction	0.66		
Reduced LL	104997	**105.0**	
1.2D+1.6L	586171.6	**586.2**	15.6
1.4D	487872	**487.9**	
		(kips)	

Load Takedown				
Interior - Lower Portion	4.3D		Size	20
# Floors	6		f'c	6
Trib Area	469			
Infl. Area	2550			
DL	176			
LL	80			
Total DL	495264	**495.3**		
Total LL	225120	225.1		
LL Reduction	0.55			
Reduced LL	123151	**123.2**		
1.2D+1.6L	791357.8	**791.4**		
1.4D	693369.6	**693.4**		
		(kips)		
			Total	**1673.3**
Interior - Upper Portion	4.3D			55.8
# Floors	7			
Trib Area	448			
Infl. Area	2550			
DL	176			
LL	80			
Total DL	551936	**551.9**		
Total LL	250880	250.9		
LL Reduction	0.55			
Reduced LL	137242	**137.2**		
1.2D+1.6L	881911.1	**881.9**		
1.4D	772710.4	**772.7**		
		(kips)		

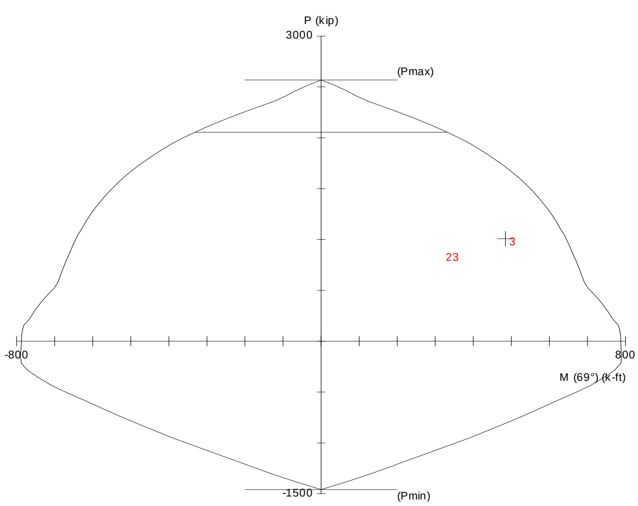
Slenderness Considerations				Critical Members	
16x16		20x20		Must Consider	
k=	1	k=	1	Slenderness	
l=	396	l=	396		
r=	4.62	r=	5.77		
kl/r=	85.74	kl/r=	68.59		
				Typical Members	
16x16		20x20		Don't Consider	
k=	1	k=	1	Slenderness	
l=	132	l=	132		
r=	4.62	r=	5.77		
kl/r=	28.58	kl/r=	22.86		

Column Design - Typical Members - Interior Columns				
16x16	**D/2**		20x20	**D/4.3**
Pu=	1105		Pu=	1673
Mux=	30		Mux=	-57.7
Muy=	30		Muy=	28.9
Size	16x16		Size	20x20
Final Design				
Steel	(8) #14 Bars		Steel	(12) #14
Hoop	(4) @ 24	Hoop	(4) @	24

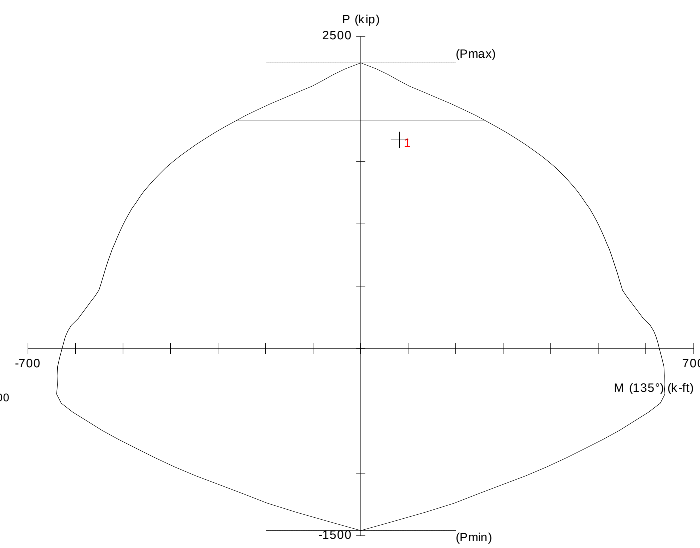
Column Design - Typical Members - Edge Columns				
16x16	**C1**		20x20	**D.8/4.3**
Pu=	628		Pu=	1060
Mux=	30		Mux=	-57.7
Muy=	60		Muy=	57.8
Size	16x16		Size	20x20
Final Design				
Steel	(4) #8 Bars		Steel	(8) #7
Hoop	(4) @ 16	Hoop	(4) @	14

Column Below Transfer		Column at Base		
20x20	**F.1/4.3**		20x20	**F.1/4.3**
Pu=	896		Pu=	1673
Mux=	56.4		Mux=	56.4
Muy=	155		Muy=	155
Size	20x20		Size	20x20
Final Design				
Steel	(8) #7 Bars		Steel	(12) #14
Hoop	(4) @ 14	Hoop	(4) @	24

Column Design - Slender Members				
16x16	**G/5.8**		20x20	**F/5.8**
P=	239.3		P=	1006.9
Mx=	7.1		Mx=	10.4
My=	67.7		My=	98.1
Load Takedown				
13-7	-		13-7	401.1
6	73.8		6	370
5	84.3		5	97.1
4	63.6		4	72.7
S.W.	17.6		S.W.	66
Final Design				
Size	**16x16**		Size	**22x22**
Steel	**(8) #14 Bars**		Steel	**(12) #14**
Hoop	(4) @ 24	Hoop	(4) @	24



Column F/5.8 Interaction Diagram



Column D/4.3 Interaction Diagram

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	Story	Pier	Load	Loc	P	V2	Vabs	M3	Mabs					
										f'c	6000	**Max Permitted Shear**		
Max V2	3	**SW4**	WIND2	Top	0	125.78	125.78	6388.25	6388.25	fy	60000	2416.74161	OKAY	
Max M3	1	**SW4**	WIND2	Bottom	0	44.94	44.94	9592.417	9592.417	t	20			
							0		0	Lw	260	Shear Strength		
Min V2	LO ROOF	**SW4**	EYMZ	Top	0	-70.3	70.3	5207.594	5207.594	Hw	132	Vc=	644	
Min M3	1	**SW4**	WIND1	Bottom	0	-40.28	40.28	-3672.13	3672.131	d	208	0.5ϕVc	242	NO REINF.

	Story	Pier	Load	Loc	P	V2	Vabs	M3	Mabs					
										f'c	6000	**Max Permitted Shear**		
Max V2	4	**SW5.8**	WIND2	Top	-408.16	194.92	194.92	5834.039	5834.039	fy	60000	2416.74161	OKAY	
Max M3	1	**SW5.8**	WIND2	Bottom	-705.02	178.95	178.95	11195.76	11195.76	t	20			
							0		0	Lw	260	Shear Strength		
Min V2	HI ROOF	**SW5.8**	EXMZ	Top	4.07	1.4	1.4	44.466	44.466	Hw	132	Vc=	644	
Min M3	1	**SW5.8**	WIND1	Bottom	-1012.87	49.24	49.24	-6438.56	6438.563	d	208	0.5ϕVc	242	NO REINF.

	Story	Pier	Load	Loc	P	V2	Vabs	M3	Mabs					
										f'c	6000	**Max Permitted Shear**		
Max V2	2	**SW2a**	WIND2	Top	164.46	69.23	69.23	677.7098	677.7098	fy	60000	758.485059	OKAY	
Max M3	2	**SW2a**	WIND2	Bottom	164.46	69.23	69.23	1507.06	1507.06	t	16			
							0		0	Lw	102	Shear Strength		
Min V2	2	**SW2a**	EX	Top	-44.33	-55.58	55.58	-34.7393	34.73925	Hw	132	Vc=	202	
Min M3	2	**SW2a**	EX	Bottom	-44.33	-55.58	55.58	-700.563	700.5626	d	81.6	0.5ϕVc	76	NO REINF.

	Story	Pier	Load	Loc	P	V2	Vabs	M3	Mabs					
										f'c	6000	**Max Permitted Shear**		
Max V2	2	**SW2b**	WIND2	Top	-164.46	69.23	69.23	677.7053	677.7053	fy	60000	758.485059	OKAY	
Max M3	2	**SW2b**	WIND2	Bottom	-164.46	69.23	69.23	1507.061	1507.061	t	16			
							0		0	Lw	102	Shear Strength		
Min V2	2	**SW2b**	EX	Top	44.33	-55.62	55.62	-34.3911	34.39108	Hw	132	Vc=	202	
Min M3	2	**SW2b**	EX	Bottom	44.33	-55.62	55.62	-700.703	700.7031	d	81.6	0.5ϕVc	76	NO REINF.

	Story	Pier	Load	Loc	P	V2	Vabs	M3	Mabs					
										f'c	4000	**Max Permitted Shear**		
Max V2	1	**SW2**	WIND2	Top	0	225.45	225.45	6056.567	6056.567	fy	60000	1590.75215	OKAY	
Max M3	1	**SW2**	WIND2	Bottom	0	225.45	225.45	8757.311	8757.311	t	16			
							0		0	Lw	262	Shear Strength		
Min V2	1	**SW2**	EX	Top	0	-121.49	121.49	-2221.29	2221.288	Hw	132	Vc=	424	
Min M3	1	**SW2**	EX	Bottom	0	-121.49	121.49	-3676.59	3676.593	d	209.6	0.5ϕVc	159	NEEDS SHEAR REINF

	Story	Pier	Load	Loc	P	V2	Vabs	M3	Mabs					
										f'c	6000	**Max Permitted Shear**		
Max V2	1	**SWG**	WIND1	Top	0	166.57	166.57	6510.079	6510.079	fy	60000	1472.35335	OKAY	
Max M3	1	**SWG**	WIND1	Bottom	0	166.57	166.57	8505.443	8505.443	t	16			
							0		0	Lw	198	Shear Strength		
Min V2	3	**SWG**	WIND2	Top	0	-22.74	22.74	-358.1	358.0997	Hw	132	Vc=	393	
Min M3	2	**SWG**	WIND2	Bottom	0	-15.61	15.61	-799.73	799.7304	d	158.4	0.5ϕVc	147	NEEDS SHEAR REINF

	Story	Pier	Load	Loc	P	V2	Vabs	M3	Mabs					
										f'c	6000	**Max Permitted Shear**		
Max V2	7	**SWD.8a**	WIND1	Top	531.66	135.38	135.38	-41.2133	41.21325	fy	60000	1282.73208	OKAY	
Max M3	1	**SWD.8a**	WIND1	Bottom	890.12	94.03	94.03	2822.811	2822.811	t	20			
							0		0	Lw	138	Shear Strength		
Min V2	1	**SWD.8a**	EXMZ	Top	411.9	-31.43	31.43	736.2432	736.2432	Hw	132	Vc=	342	
Min M3	11	**SWD.8a**	WIND1	Top	210.37	67.17	67.17	-747.401	747.4007	d	110.4	0.5ϕVc	128	NEEDS SHEAR REINF

	Story	Pier	Load	Loc	P	V2	Vabs	M3	Mabs					
										f'c	6000	**Max Permitted Shear**		
Max V2	7	**SWD.8b**	WIND1	Top	-188.79	149.13	149.13	-68.5164	68.51642	fy	60000	1282.73208	OKAY	
Max M3	1	**SWD.8b**	WIND1	Bottom	122.75	125.36	125.36	2332.352	2332.352	t	20			
							0		0	Lw	138	Shear Strength		
Min V2	1	**SWD.8b**	EXMZ	Top	28.52	-21.31	21.31	583.0954	583.0954	Hw	132	Vc=	342	
Min M3	11	**SWD.8b**	WIND1	Top	-206.51	83.11	83.11	-679.194	679.1937	d	110.4	0.5ϕVc	128	NEEDS SHEAR REINF

	Story	Pier	Load	Loc	P	V2	Vabs	M3	Mabs					
										f'c	6000	**Max Permitted Shear**		
Max V2	3	**SWB**	WIND1	Top	0	192.46	192.46	4779.201	4779.201	fy	60000	1472.35335	OKAY	
Max M3	1	**SWB**	WIND1	Bottom	0	4.28	4.28	7036.84	7036.84	t	16			
							0		0	Lw	198	Shear Strength		
Min V2	LO ROOF	**SWB**	WIND2	Top	0	-64.39	64.39	0	0	Hw	132	Vc=	393	
Min M3	1	**SWB**	WIND2	Bottom	0	-5.87	5.87	-1402.66	1402.661	d	158.4	0.5ϕVc	147	NEEDS SHEAR REINF

Minimum Requirements			Req'd Reinforcing
	Horizontal	Vertical	Horizontal Initial
	ρt = 0.0025	ρl = 0.0025	Vs= -483.208
	S = 18	S = 18	Av/S= -0.039
(2) #4	Smin= 8.0	= 8.0	S= -10.331
(2) #5	S min= 12.4	= 12.4	S= -16.013

	Horizontal Final Design		Vertical Final Design	
(2) #4	S= 8	ρ= 0.0025	ρ= 0.0025	S= 8
(2) #5	S= 12.4	ρ= 0.0025	ρ= 0.0025	S= 12.4

Summary	
SW4	V2 max = 125.78
Lw = 260	M3 max = 9592.42
Tw = 20	Pu = 2553.63
use (2) #5 @ 12" O.C. each way	

Overturning	(ft-k, k)	
Moment	9592.42	
Lw	260	
Force Couple	442.73	
Net Force	834.09	C

Minimum Requirements			Req'd Reinforcing
	Horizontal	Vertical	Horizontal Initial
	ρt = 0.0025	ρl = 0.0025	Vs= -394.567
	S = 18	S = 18	Av/S= -0.032
(2) #4	Smin= 8.0	= 8.0	S= -12.652
(2) #5	S min= 12.4	= 12.4	S= -19.610

	Horizontal Final Design		Vertical Final Design	
(2) #4	S= 8	ρ= 0.0025	ρ= 0.0025	S= 8
(2) #5	S= 12.4	ρ= 0.0025	ρ= 0.0025	S= 12.4

Summary	
SW5.8	V2 max = 194.92
Lw = 260	M3 max = 11195.76
Tw = 20	Pu = 2553.63
use (2) #5 @ 12" O.C. each way	

Overturning	(ft-k, k)	
Moment	11195.76	
Lw	260	
Force Couple	516.73	
Net Force	760.09	C

Minimum Requirements			Req'd Reinforcing
	Horizontal	Vertical	Horizontal Initial
	ρt = 0.0025	ρl = 0.0025	Vs= -113.506
	S = 18	S = 18	Av/S= -0.023
(2) #4	Smin= 10.0	= 10.0	S= -17.254
(2) #5	S min= 15.5	= 15.5	S= -26.743

	Horizontal Final Design		Vertical Final Design	
(2) #4	S= 10	ρ= 0.0025	ρ= 0.0025	S= 10
(2) #5	S= 15.5	ρ= 0.0025	ρ= 0.0025	S= 15.5

Summary	
SW2a	V2 max = 69.23
Lw = 102	M3 max = 1507.06
Tw = 16	Pu = 1089
use (2) #5 @ 12" O.C. each way	

Overturning	(ft-k, k)	
Moment	1507.06	
Lw	102	
Force Couple	177.30	
Net Force	367.20	C

Minimum Requirements			Req'd Reinforcing
	Horizontal	Vertical	Horizontal Initial
	ρt = 0.0025	ρl = 0.0025	Vs= -113.506
	S = 18	S = 18	Av/S= -0.023
(2) #4	Smin= 10.0	= 10.0	S= -17.254
(2) #5	S min= 15.5	= 15.5	S= -26.743

	Horizontal Final Design		Vertical Final Design	
(2) #4	S= 10	ρ= 0.0025	ρ= 0.0025	S= 10
(2) #5	S= 15.5	ρ= 0.0025	ρ= 0.0025	S= 15.5

Summary	
SW2b	V2 max = 69.23
Lw = 102	M3 max = 1507.06
Tw = 16	Pu = 1089
use (2) #5 @ 12" O.C. each way	

Overturning	(ft-k, k)	
Moment	1507.06	
Lw	102	
Force Couple	177.30	
Net Force	367.20	C

Minimum Requirements			Req'd Reinforcing
	Horizontal	Vertical	Horizontal Initial
	ρt = 0.0025	ρl = 0.0025	Vs= -135.162
	S = 18	S = 18	Av/S= -0.011
(2) #4	Smin= 10.0	= 10.0	S= -37.218
(2) #5	S min= 15.5	= 15.5	S= -57.687

	Horizontal Final Design		Vertical Final Design	
(2) #4	S= 10	ρ= 0.0025	ρ= 0.0025	S= 10
(2) #5	S= 15.5	ρ= 0.0025	ρ= 0.0025	S= 15.5

Summary	
SW2	V2 max = 225.45
Lw = 262	M3 max = 8757.31
Tw = 16	Pu = 2178
use (2) #5 @ 12" O.C. each way	

Overturning	(ft-k, k)	
Moment	8757.31	
Lw	262	
Force Couple	401.10	
Net Force	687.90	C

Minimum Requirements			Req'd Reinforcing
	Horizontal	Vertical	Horizontal Initial
	ρt = 0.0025	ρl = 0.0025	Vs= -179.076
	S = 18	S = 18	Av/S= -0.019
(2) #4	Smin= 10.0	= 10.0	S= -21.229
(2) #5	S min= 15.5	= 15.5	S= -32.905

	Horizontal Final Design		Vertical Final Design	
(2) #4	S= 10	ρ= 0.0025	ρ= 0.0025	S= 10
(2) #5	S= 15.5	ρ= 0.0025	ρ= 0.0025	S= 15.5

Summary	
SWG	V2 max = 166.57
Lw = 198	M3 max = 8505.44
Tw = 16	Pu = 2187
use (2) #5 @ 12" O.C. each way	

Overturning	(ft-k, k)	
Moment	8505.44	
Lw	198	
Force Couple	515.48	
Net Force	578.02	C

Minimum Requirements			Req'd Reinforcing
	Horizontal	Vertical	Horizontal Initial
	ρt = 0.0025	ρl = 0.0025	Vs= -168.498
	S = 18	S = 18	Av/S= -0.025
(2) #4	Smin= 8.0	= 8.0	S= -15.725
(2) #5	S min= 12.4	= 12.4	S= -24.373

	Horizontal Final Design		Vertical Final Design	
(2) #4	S= 8	ρ= 0.0025	ρ= 0.0025	S= 8
(2) #5	S= 12.4	ρ= 0.0025	ρ= 0.0025	S= 12.4

Summary	
SWD.8a	V2 max = 135.38
Lw = 138	M3 max = 2822.81
Tw = 20	Pu = 2522.8
use (2) #5 @ 12" O.C. each way	

Overturning	(ft-k, k)	
Moment	2822.81	
Lw	138	
Force Couple	245.46	
Net Force	1015.94	C

Minimum Requirements			Req'd Reinforcing
	Horizontal	Vertical	Horizontal Initial
	ρt = 0.0025	ρl = 0.0025	Vs= -150.870
	S = 18	S = 18	Av/S= -0.023
(2) #4	Smin= 8.0	= 8.0	S= -17.562
(2) #5	S min= 12.4	= 12.4	S= -27.221

	Horizontal Final Design		Vertical Final Design	
(2) #4	S= 8	ρ= 0.0025	ρ= 0.0025	S= 8
(2) #5	S= 12.4	ρ= 0.0025	ρ= 0.0025	S= 12.4

Summary	
SWD.8b	V2 max = 149.13
Lw = 138	M3 max = 2332.35
Tw = 20	Pu = 890
use (2) #5 @ 12" O.C. each way	

Overturning	(ft-k, k)	
Moment	2332.35	
Lw	138	
Force Couple	202.81	
Net Force	242.19	C

Minimum Requirements			Req'd Reinforcing
	Horizontal	Vertical	Horizontal Initial
	ρt = 0.0025	ρl = 0.0025	Vs= -145.884
	S = 18	S = 18	Av/S= -0.015
(2) #4	Smin= 10.0	= 10.0	S= -26.059
(2) #5	S min= 15.5	= 15.5	S= -40.392

	Horizontal Final Design		Vertical Final Design	
(2) #4	S= 10	ρ= 0.0025	ρ= 0.0025	S= 10
(2) #5	S= 15.5	ρ= 0.0025	ρ= 0.0025	S= 15.5

Summary	
SWB	V2 max = 192.46
Lw = 198	M3 max = 7036.84
Tw = 16	Pu = 2187
use (2) #5 @ 12" O.C. each way	

Overturning	(ft-k, k)	
Moment	7036.84	
Lw	198	
Force Couple	426.48	
Net Force	667.02	C

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CB1	Story	Beam	Load	Loc	P	V2	V3	T	M2	M3
Max M3	8	**CB1**	WIND1	0	0	80.96	0	0	0	421.8824
Min M3	8	**CB1**	WIND1	124.5	0	80.96	0	0	0	-418.056
Max V2	8	**CB1**	WIND1	0	0	80.96	0	0	0	421.8824
Min V2	11	**CB1**	WIND2	0	0	-3.29	0	0	0	-17.0833

f'c	6000	
fy	60000	
t	20	
ln	114.5	
h	36	
ln/h	3.18	may use diag
	1277	

Flexural Design							Shear Design					
eqt10-3	As,min=	2.56	ρ=	0.003873	ok			Vc=	102.2		S=	15
	not < 200bd/fy =	2.2	ρ=	0.003333	ok			0.5ϕVc	25.6		Av min =	0.29
	est. As~Mu/4.2d	3.04	ρ=	0.004612	ok				need shear reinf			0.25
								Vs req'd	-12.2912			
Use (4) #8 bars		a=	1.86			ε_s= 0.018			use min reinf			
As=	3.16	c=	2.48		ϕ=	0.9						
		ϕMn=	456.04	ok								

Summary CB1 36" x 20"	
Long. Reinf	**Shear Reinf.**
Use (4) #8 bars	(2) legs #4 @ 15"
As= 3.16	

CB2	Story	Beam	Load	Loc	P	V2	V3	T	M2	M3
Max M3	5	**CB2**	WIND2	0	0	45	0	0	0	157.5092
Min M3	5	**CB2**	WIND2	84	0	45	0	0	0	-157.523
Max V2	5	**CB2**	WIND2	0	0	45	0	0	0	157.5092
Min V2	5	**CB2**	WIND1	0	0	-13.37	0	0	0	-46.661

f'c	6000	
fy	60000	
t	16	
ln	84	
h	36	
ln/h	2.33	may use diag
	937	

Flexural Design							Shear Design					
	As,min=	2.04	ρ=	0.003098	ok			Vc=	81.8		S=	15
	not < 200bd/fy =	1.76	ρ=	0.002667	ok			0.5ϕVc	20.4		Av min =	0.23
	est. As~Mu/4.2d	1.14	ρ=	0.001722	ok				need shear reinf			0.2
								Vs req'd	-31.7974			
Use (5) #6 bars		a=	1.62			ε_s= 0.015			use min reinf			
As=	2.2	c=	2.16		ϕ=	0.9						
		ϕMn=	318.69	ok								

Summary CB2 36" x 16"	
Long. Reinf	**Shear Reinf.**
Use (5) #6 bars	(2) legs #4 @ 15"
As= 2.2	

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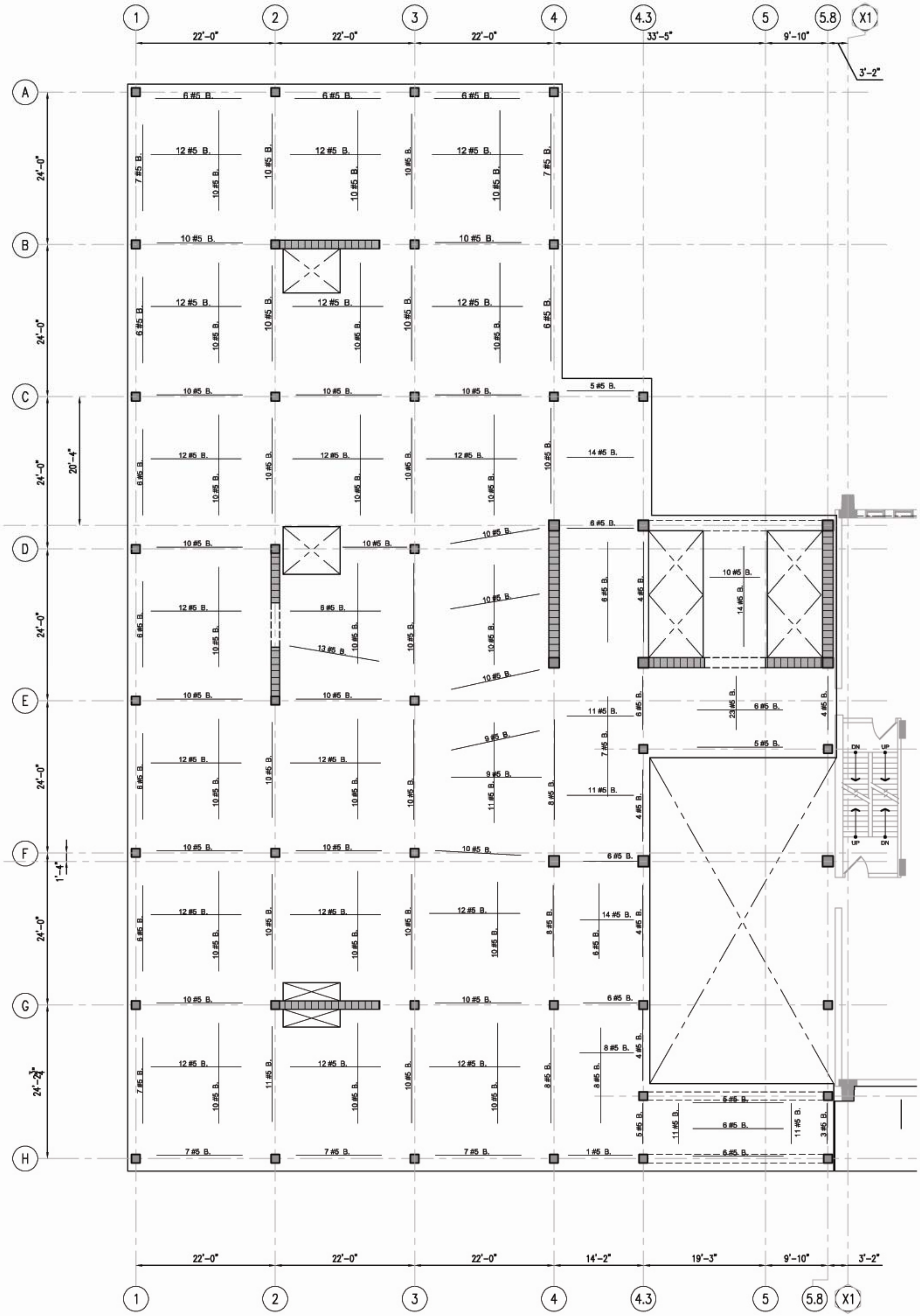
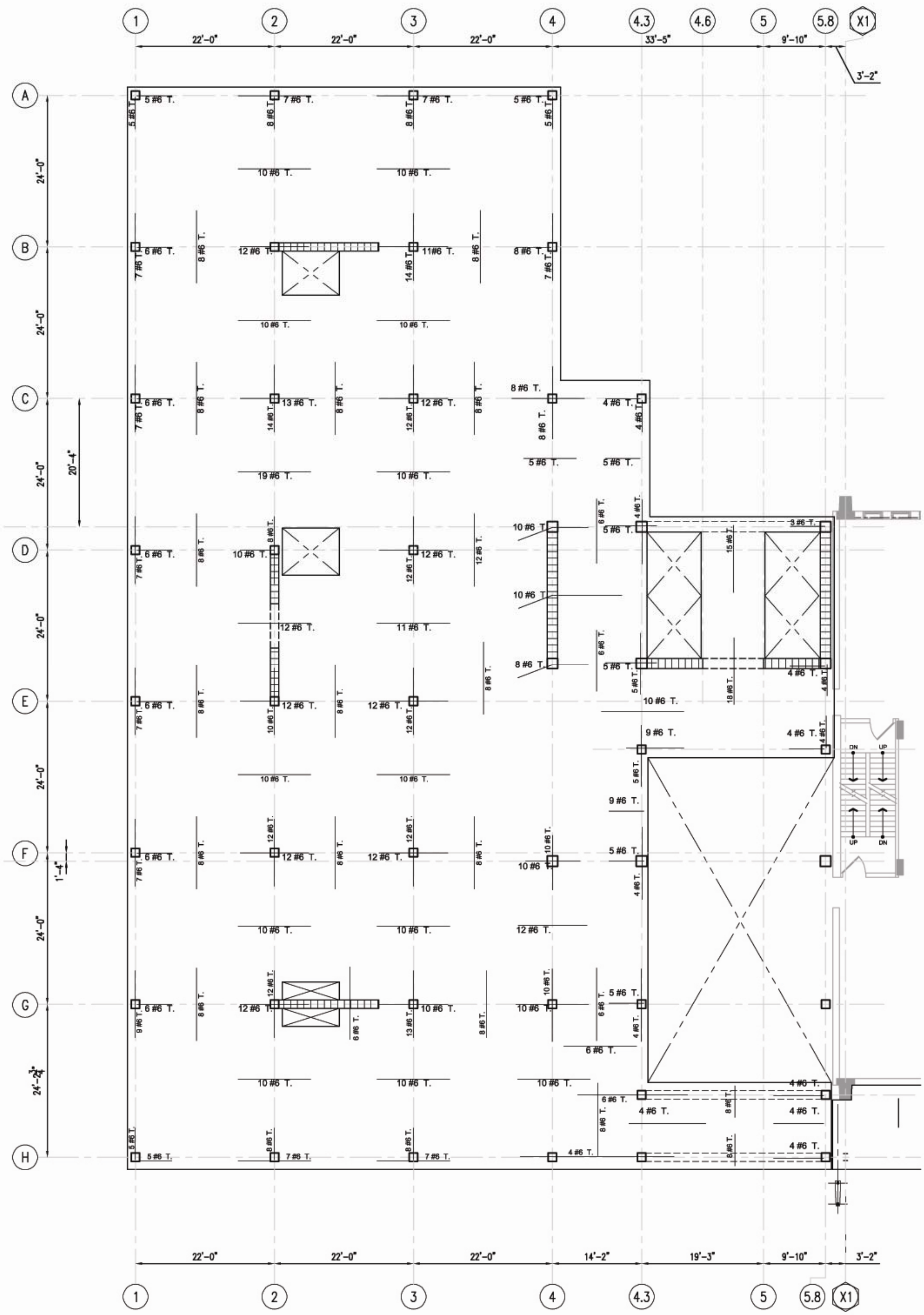
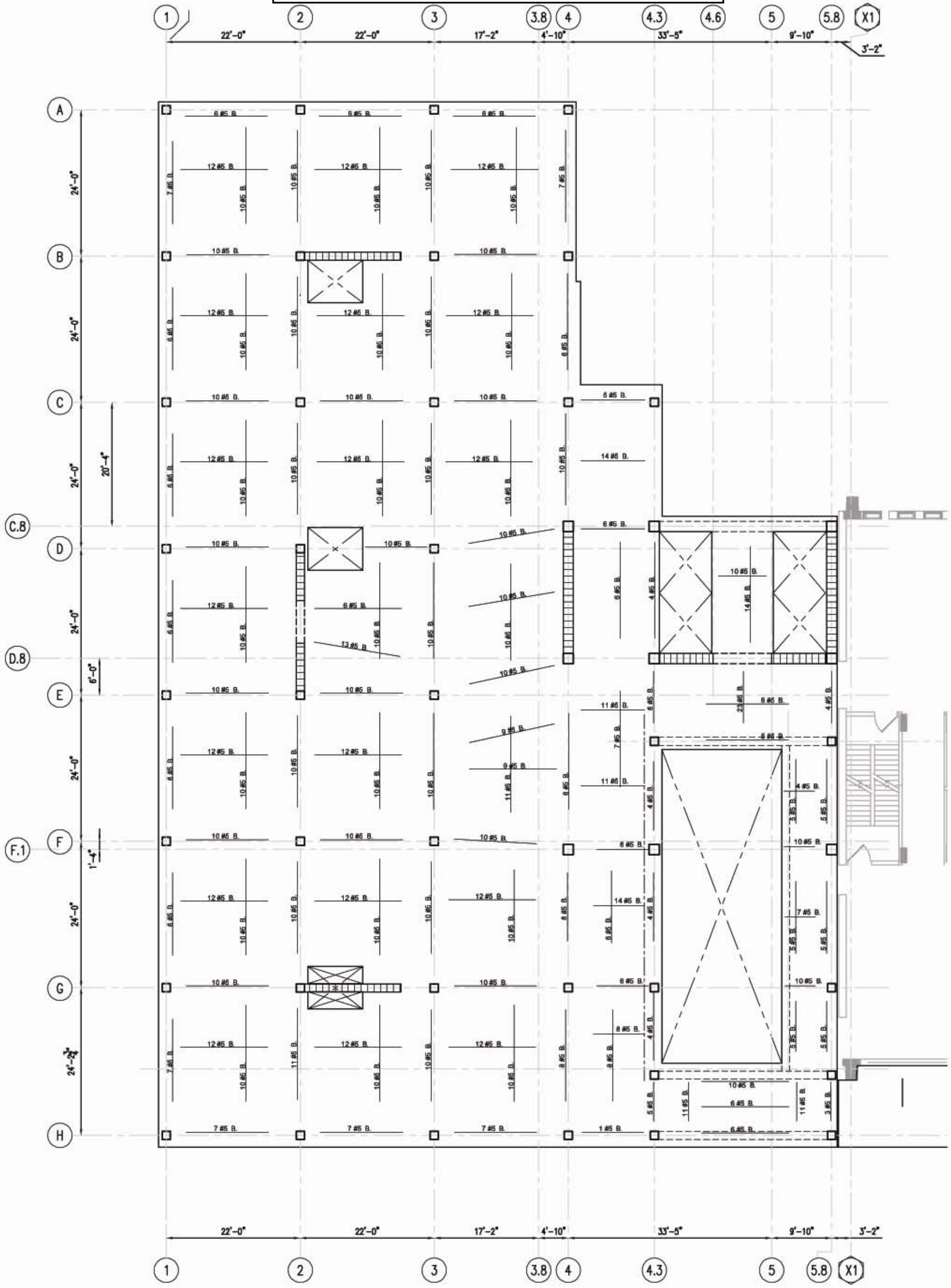
2nd & 3rd Floor Bottom Reinf. Plan



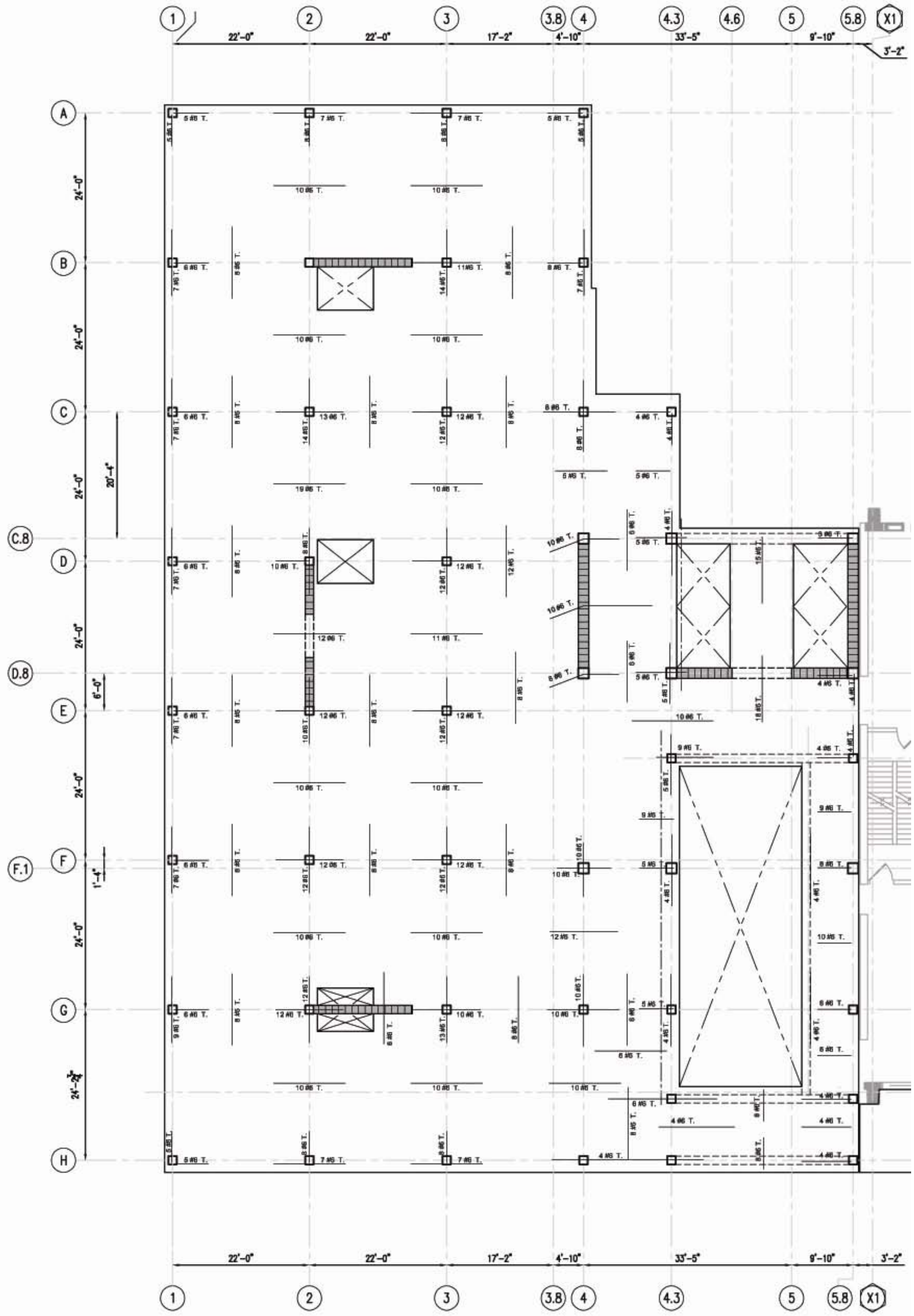
2nd & 3rd Floor Top Reinf. Plan



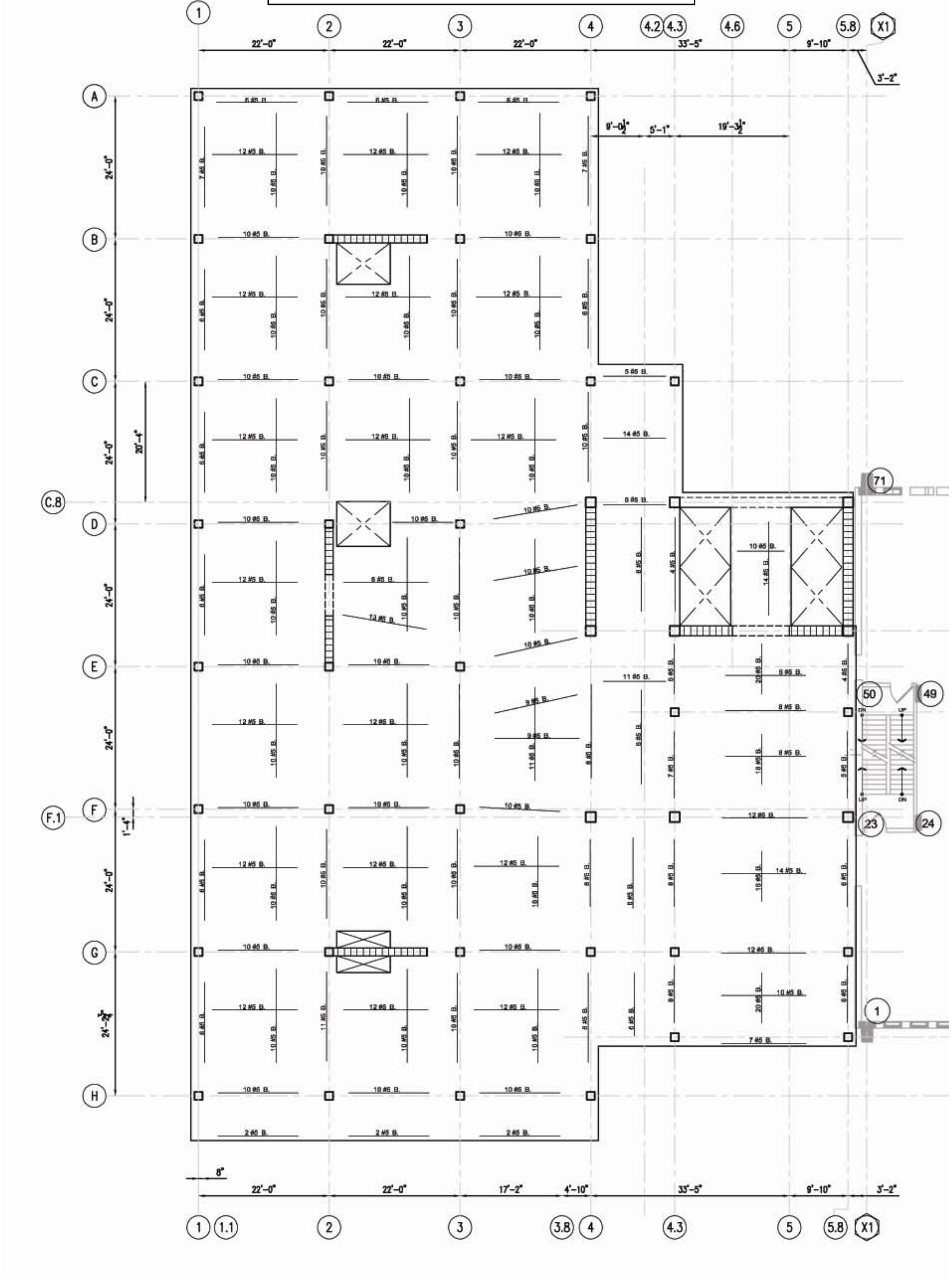
4th Floor Bottom Reinf. Plan



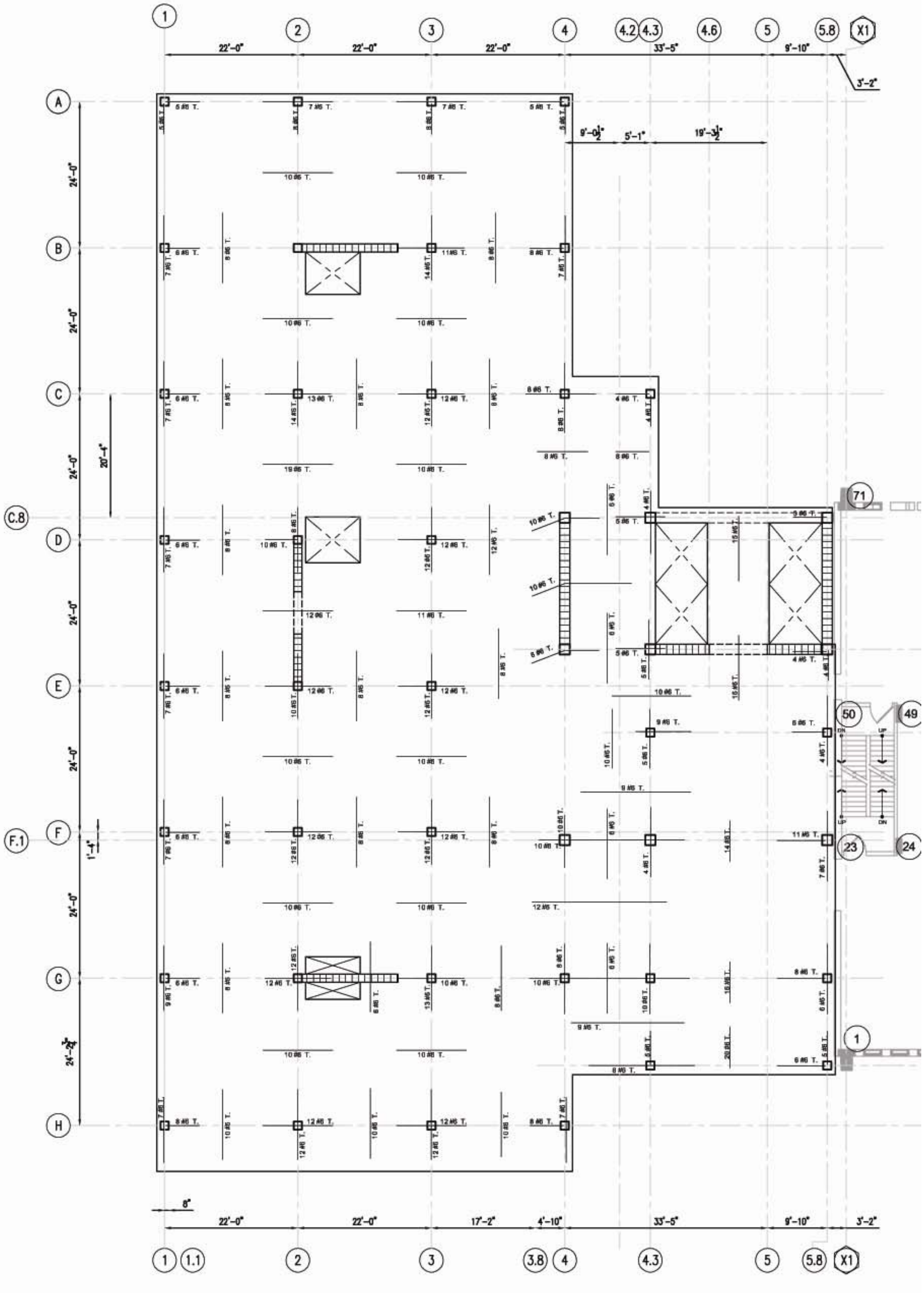
4th Floor Top Reinf. Plan



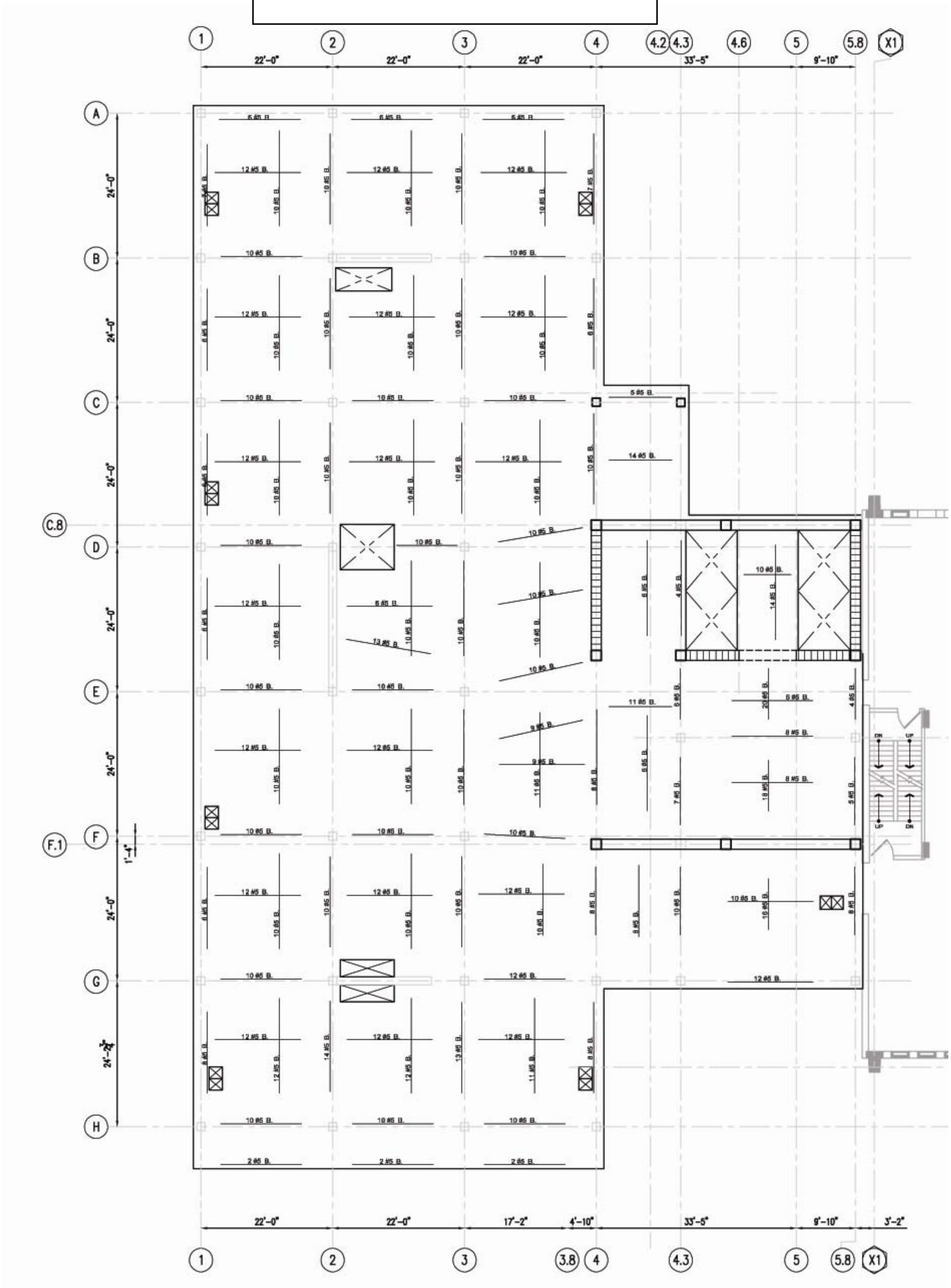
5th Floor Bottom Reinf. Plan



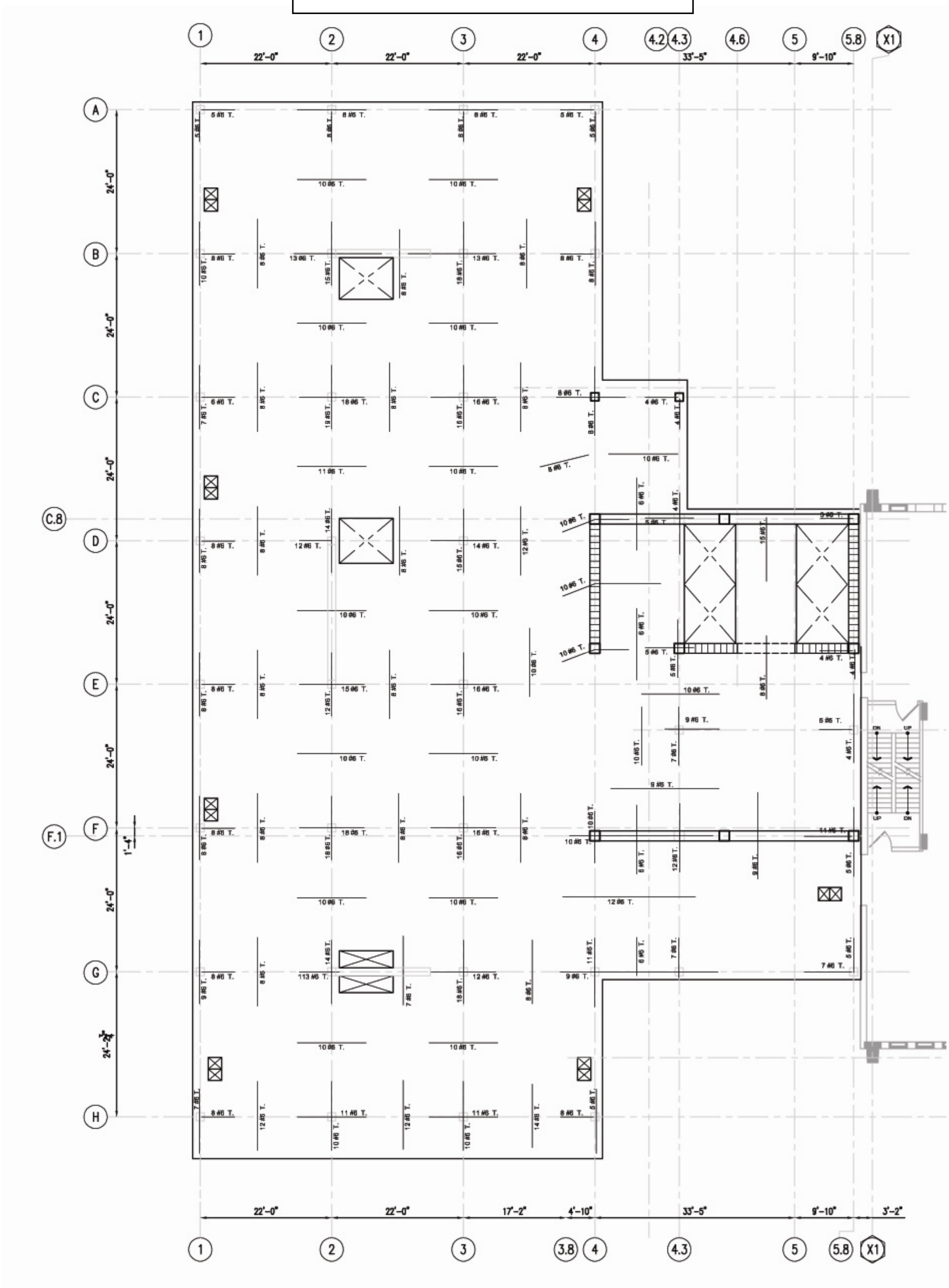
5th Floor Top Reinf. Plan

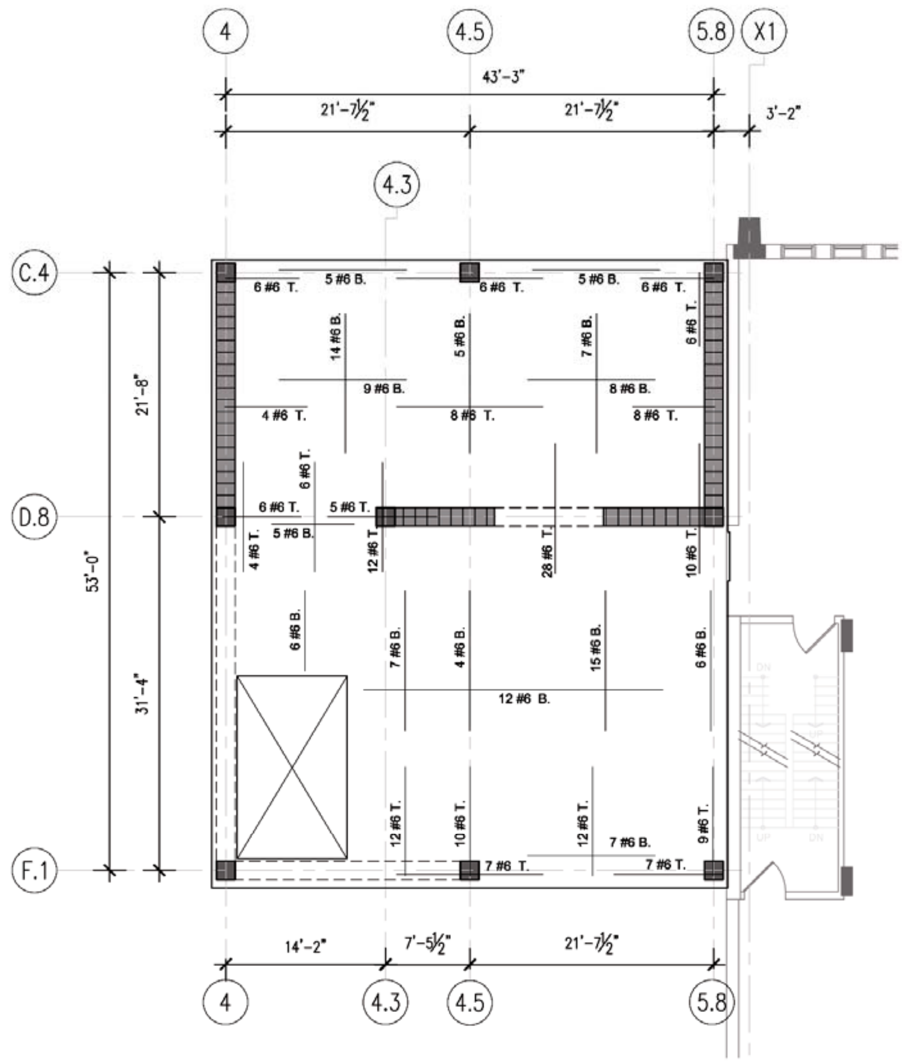


6th Floor Bottom Reinf. Plan



6th Floor Top Reinf. Plan





7th – 13th Floor Reinf. Plan

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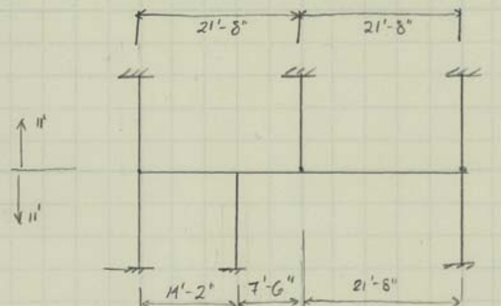
DESIGN TRANSFER BEAM

Assume: $f'_c = 6$ ksi
$t_{slab} = 12"$
col = 20 × 20

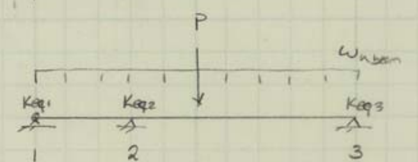


crit L.C. = 1.2D + 1.6L

th FLOOR

from RAM CONCEPT

P = 7 × 127 = 889 k



$w_u = 1.2(D) + 1.6(L) =$ psf

equivalent K_{col}

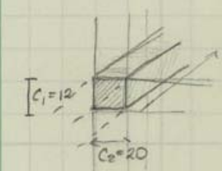
$$I_c = \frac{20(20)^3}{12} = 13300 \text{ in}^4 \qquad K_c = \frac{4E_c I_c}{L - 2t} = \frac{4E_c(13300)}{132 - 2(12)}$$

$$= 493\,E_c$$

slab contribution (torsion)

$$K_t = \Sigma \frac{9E_c C}{\ell_2(1 - c_2/\ell_2)} \qquad C = \Sigma\left(1 - 0.63\frac{x}{y}\right)\frac{x^3 y}{3}$$

$$= \left(1 - 0.63\frac{12}{20}\right)\left(\frac{12^3(20)}{3}\right) = 7165$$



$$K_t = 2\times\frac{9E_c(7165)}{188\left(1 - \frac{20}{188}\right)} = 768\,E_c$$

$$\frac{1}{K_{ec1}} = \frac{1}{K_{EC3}} = \frac{1}{2\times 493E_c} + \frac{1}{768E_c}$$

$$K_{EC1} = K_{EC3} = 431\,E_c$$

$$\frac{1}{K_{EC2}} = \frac{1}{493E_c} + \frac{1}{768E_c}$$

$$K_{EC2} = 300\,E_c$$

slab stiffness

- ignore beam contribution (~ same Relative stiffness)

$I_s = \frac{(188)(12)^3}{12} = 27072 \text{ in}^4$

$\ell_n = 170"$ for 1-2 & 2-1

$\ell_n = 350"$ for 2-3 & 3-2

$K_s = \frac{4E_c I_s}{\ell_n - \frac{c_1}{2}}$

$C_1 = 20"$

section 1-2 & 2-1

$K_s = \frac{4E_c(27072)}{170 - \frac{20}{2}} = 677 E_c$

section 2-3 & 3-2

$K_s = \frac{4E_c(27072)}{350 - 10} = 159 E_c$

Dist. Factor

$DF_{1-2} = \frac{K_s}{\Sigma K} = \frac{677}{677 + 431} = 0.61$

$DF_{2-1} = \frac{677}{677 + 300 + 159} = 0.60$

$DF_{2-3} = \frac{159}{677 + 300 + 159} = 0.14$

$DF_{3-2} = \frac{159}{159 + 431} = 0.27$

FEM

dist. load - conservatively assume CS % = 0.85
BM % = 0.85

LL reduction

$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL} A_T}}\right) = 80\left(0.25 + \frac{15}{\sqrt{1355}}\right) = 53 \text{ psf}$

$w_u = 1.2D + 1.6L = 1.2(157) + 1.6(53) = 273 \text{psf}$

$w_{u\,beam} = (0.85)(0.85)(273)(17.25') = \underline{3.4 \text{ K/ft}}$

$$FEM_{1-2} = \frac{w_{u\,beam}\,l^2}{12} = \frac{(3.4)(14.17)^2}{12} = 56.9 \text{ ft-k}$$

$$FEM_{2-1} = FEM_{1-2} = 56.9 \text{ ft-k}$$

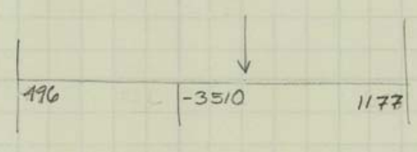
$$FEM_{2-3} = \frac{w_u l^2}{12} + \frac{Pab^2}{l^2} = \frac{(3.4)(29.17)^2}{12} + \frac{889(7.5)(21.67)^2}{(29.17)^2}$$

$$= 3920 \text{ ft-k}$$

$$FEM_{3-2} = \frac{w_u l^2}{12} + \frac{Pa^2 b}{l^2} = \frac{(3.4)(29.17)^2}{12} + \frac{889(7.5)^2(21.67)}{29.17^2}$$

$$= 1302 \text{ ft-k}$$

FROM Moment Dist.

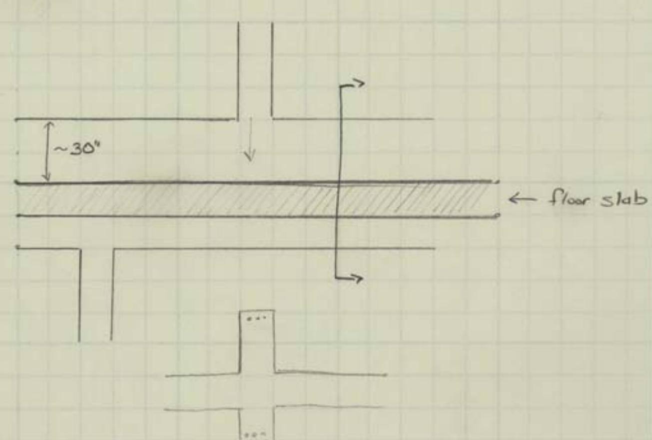


– need more slab contribution?

approximate depth & steel

$bd^2 = 18 M_u$ $\qquad A_s = M_u / 4.2d$

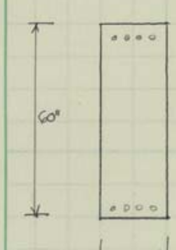
b	d	As
22	54"	15.5
36	42"	20.0
48	36"	23.3
→ 20	56	15 in^2

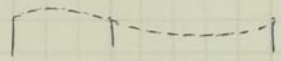


MOMENT DISTRIBUTION

1	2		3	
12	21	23	32	
0.61	0.6	0.14	0.27	Distrib. Factor
-56.90	56.90	-3920.00	1302.00	
-56.90			1302.00	
34.71			-351.54	
	17.35	-175.77		
	-4021.52			
	2412.91	563.01		
1206.45			281.51	
-735.94			-76.01	
	-367.97	-38.00		
	-405.97			
	243.58	56.84		
121.79			28.42	
-74.29			-7.67	
	-37.15	-3.84		
	-40.98			
	24.59	5.74		
495.8	2350.2	-3512.0	1176.7	Moment [ft.k]

Beam Design -





$f'_c = 6$ ksi

$f_y = 60$ ksi

$A_s \geq 15 in^2$ try (10) #11 bars

$A_s = 15.6 in^2$

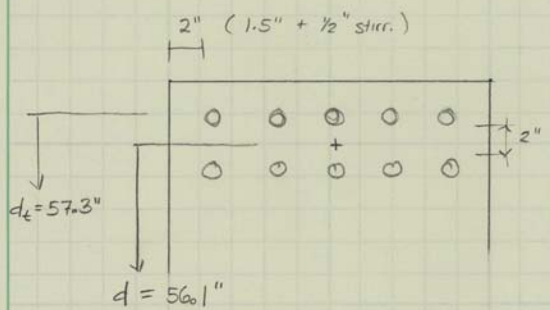
2 rows of 5 bars

max bars = 5 ∴ ok

min bars = 3

max spacing $= 15(\frac{4}{6}) - 2.5(2) = 5"$

Spacing $S_c = $ | 1" ; max $d_b = 1.41"$ ← controls ; 4/3 agg. = 1"



$S_c = [20 - (2)(2) - (5)(1.41)]/4 = 2.24"$ ∴ Ok

$A_{s\,min} = \frac{3\sqrt{f'_c}}{f_y} b_w d = 4.3 in^2$ ∴ ok

$= 200 b_w d / f_y = 3.74$ ∴ Ok

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{15.6(60\,000)}{0.85(6000)(20)} = 9.18"$$

$$c = a/\beta_1 = 9.18/0.75 = 12.24"$$

assume $\phi = 0.9$ $\phi M_n = \phi A_s f_y (d - \frac{a}{2}) = 0.9(15.6)(60)(56.1 - \frac{9.18}{2})$

$\boxed{= 3620 > 3510 \therefore ok}$

check $\varepsilon_s > 0.005$

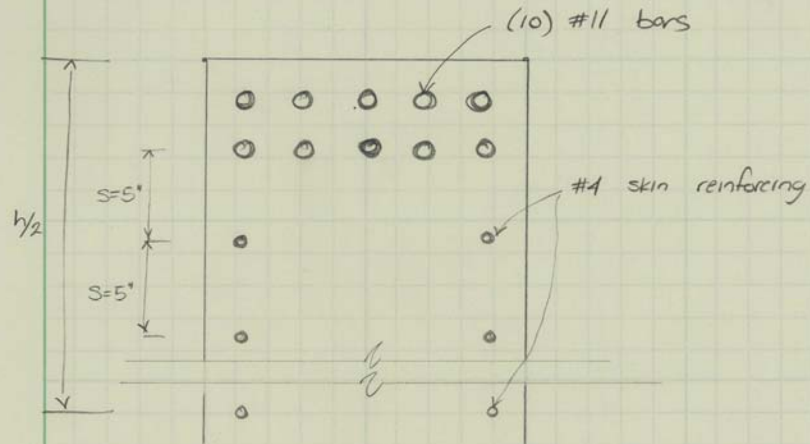
$$\varepsilon_s = \frac{\varepsilon_u}{c}(d_t - c) = \frac{0.003}{12.24}(57.3 - 12.24)$$

$$= 0.011 > 0.005 \therefore \phi = 0.9$$

Special requirements -

skin reinforcing - use # 4 bars -(same as shear)

$$S = 15\left(\frac{40000}{f_s}\right) - 2.5c_c = 15\left(\frac{4}{6}\right) - 2.5(2)$$
$$= 5''$$



Sections 11.7.1 A.3.3
12.10.6 11.7.4
11.7.5

most likely controls over skin reinforcing

From SAP OUTPUT $V_n = 818\,k < 10\sqrt{f'_c}\,b_w d = 10\sqrt{6000}\,(20)(56.1)/1000$
$= 870k$

$818 < 870$ ∴ ok

min shear reinf. perp. to tension reinf.

$d/5 = 11.2''$

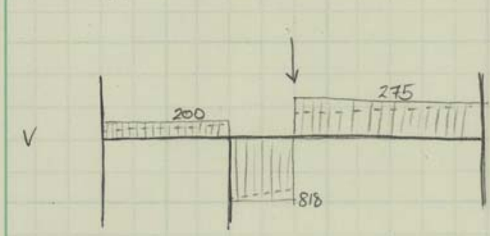
$A_v \geq 0.0025 b_w s$ $\quad s \leq 12$
$= 0.0025(20)(11) = 0.55$ $\quad$ $\boxed{\text{use (3) \#4 } A_v = 0.60}$

min shear reinf parallell to tension reinf.

$A_{vh} \geq 0.0015 b_w s_2$ $\quad S_2 \leq 11''$
$0.0015(20)(11) = 0.33$ $\quad$ $\boxed{\text{use (2)\#4 } A_v = 0.40}$

or A3.3 (use § 11.7.4 & 11.7.5, not A3.3)

Design additional shear reinf.



$V_{u1} = 200k$

$V_{u2} = 818k$

$V_{u3} = 275k$

$$V_c = 2\sqrt{f'_c}\, b_w d = 2\sqrt{6000}\,(20)(56.1)/1000$$
$$= 174\ k$$

1) $V_{s\,req'd} = V_u/\phi - V_c = \frac{200}{0.75} - 174 = \underline{93k}$

2) $= \frac{818}{0.75} - 174 = \underline{917k}$

3) $= \frac{275}{0.75} - 174 = \underline{193k}$

TRY (2) #4 $\quad V_s > 4\sqrt{f'_c}\, b_w d = 348\ k$

$\therefore$ 11.4.5.1 & 11.4.5.2 is halved

DNC (Deep Beam Controls)

$$A_{v\,min} = \max \begin{cases} 0.75\sqrt{f'_c}\, b_w s/f_y = 0.75\sqrt{6000}\,(20)(11)/60000 = 0.213'' \\ 50\, b_w s/f_y = 50\,(20)(11)/60000 = 0.183'' \end{cases}$$

$A_v = 3 \times 0.2 = 0.60\ in^2 > 0.213 \therefore$ OK ✓ (Deep Beam Controls) spacing for $V_{s1} = 93$

$$s = \frac{A_v f_y d}{V_s} = \frac{0.6(60)(56.1)}{93}$$

$= 21.7$ ∴ use 10" (3) legs #4

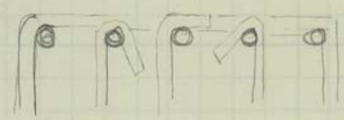
min = 11"

for $V_{s2} = 917$

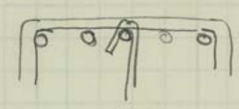
$s = 2.2"$ not feasible

try 5 legs? $s = \frac{1.0(60)(56.1)}{917} = 3.6"$

use 3"



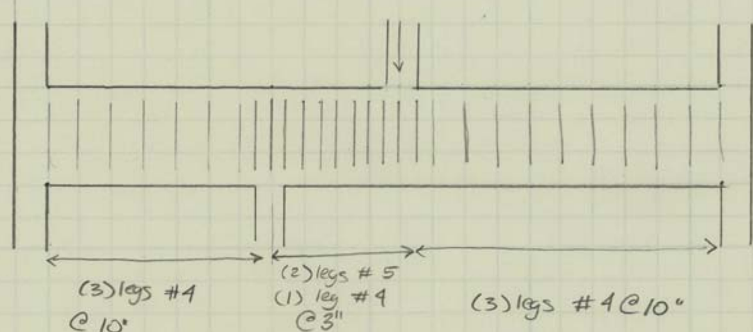
or (2) legs #5
(1) leg #4

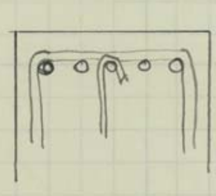


$s = 3"$

for $V_{s3} = 193$ K

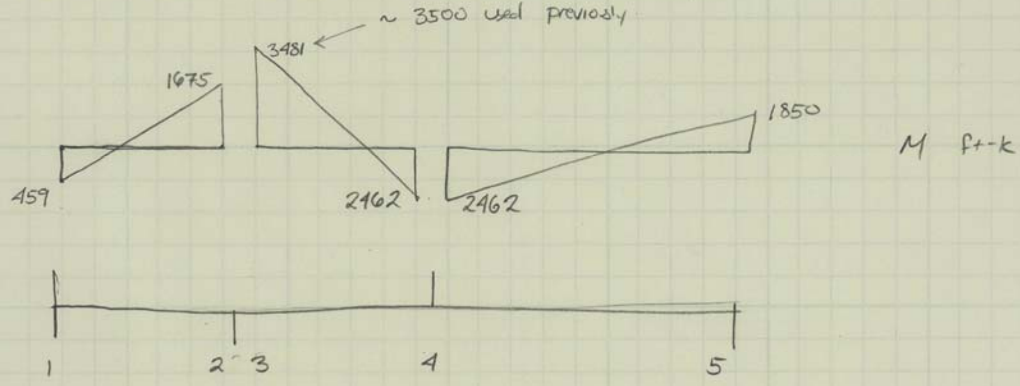
$s = \frac{0.6(60)(56.1)}{193} = 10.46$ ∴ use 10" (3) legs #4





other flexural Reinf.

(Moment Drawn on Tension side) (values from SAP output)



use $A_s = M_u / 4.2d$

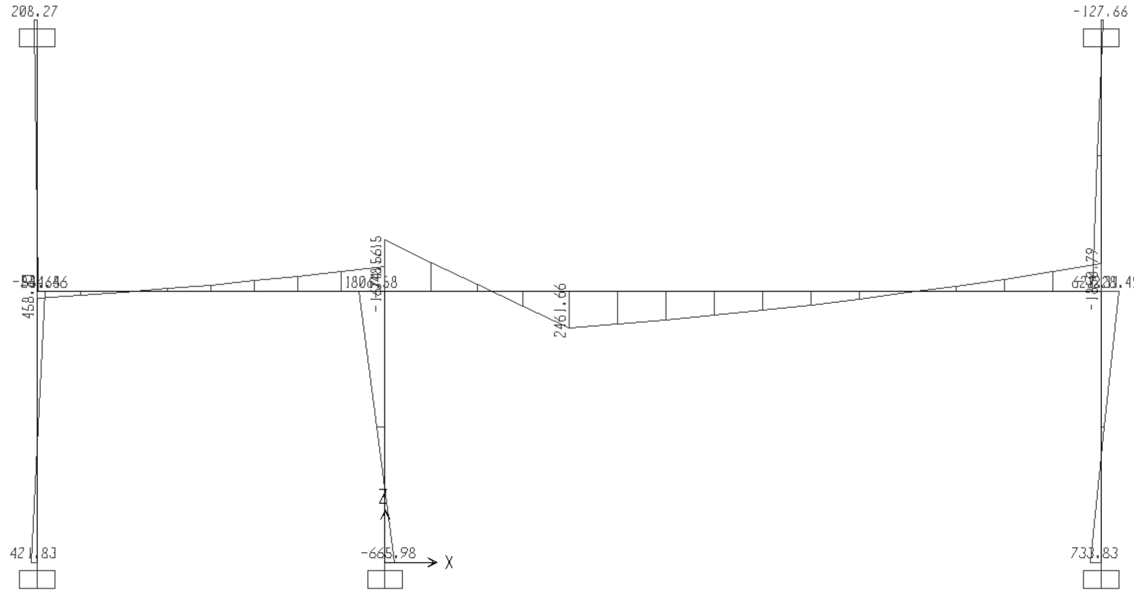
$A_{s_1} = 459 / 4.2(56.1) = 1.95\ in^2$ use (5) # 6 bars $A_s = 2.2''$
Bottom Barse

$A_{s_2} = 1675 / 4.2(56.1) = 7.10$ ∴ use (5) # 11 bars
$A_s = 7.7 in^2$
TOP BARS

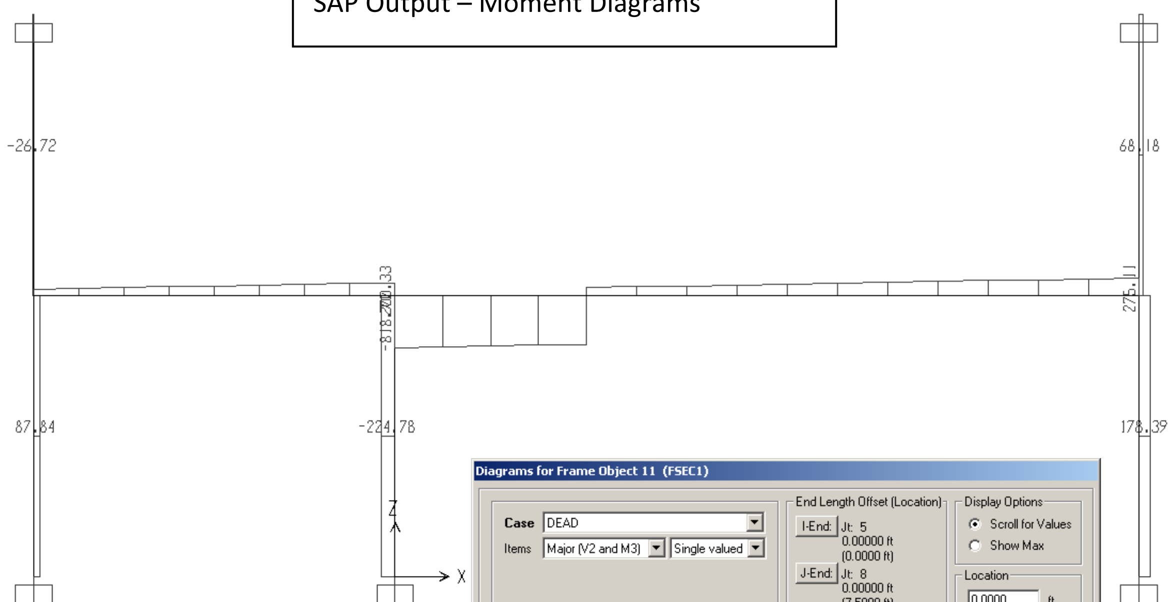
$A_{s_3} =$ — see previous use 2 rows (5) # 11 bars
$A_s = 15.4 in^2$
TOP BARS

$A_{s4} = 2462 / 4.2(56.1) = 10.5 in^2$ use 2 rows (6) # 9 bars
$A_s = 12.0 in^2$
Bottom Bars

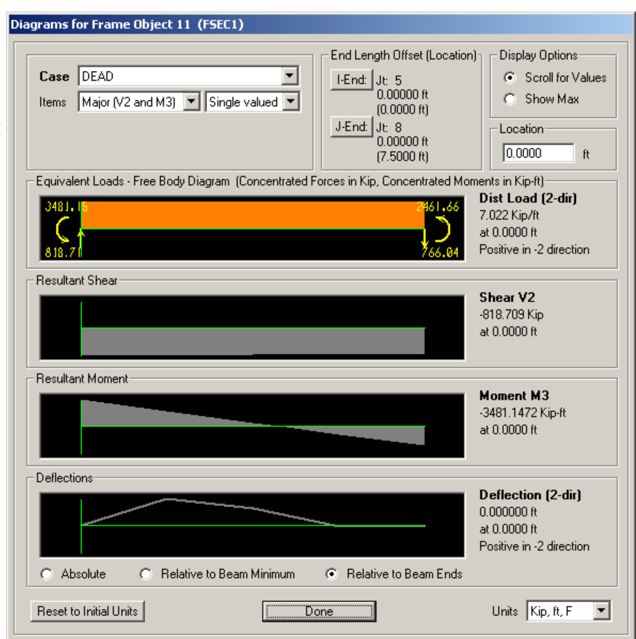
$A_{s5} = 1850 / 4.2(56.1) = 7.85 in^2$ use 2 rows (4) # 9 bars
$A_s = 8.0 in^2$
TOP Bars



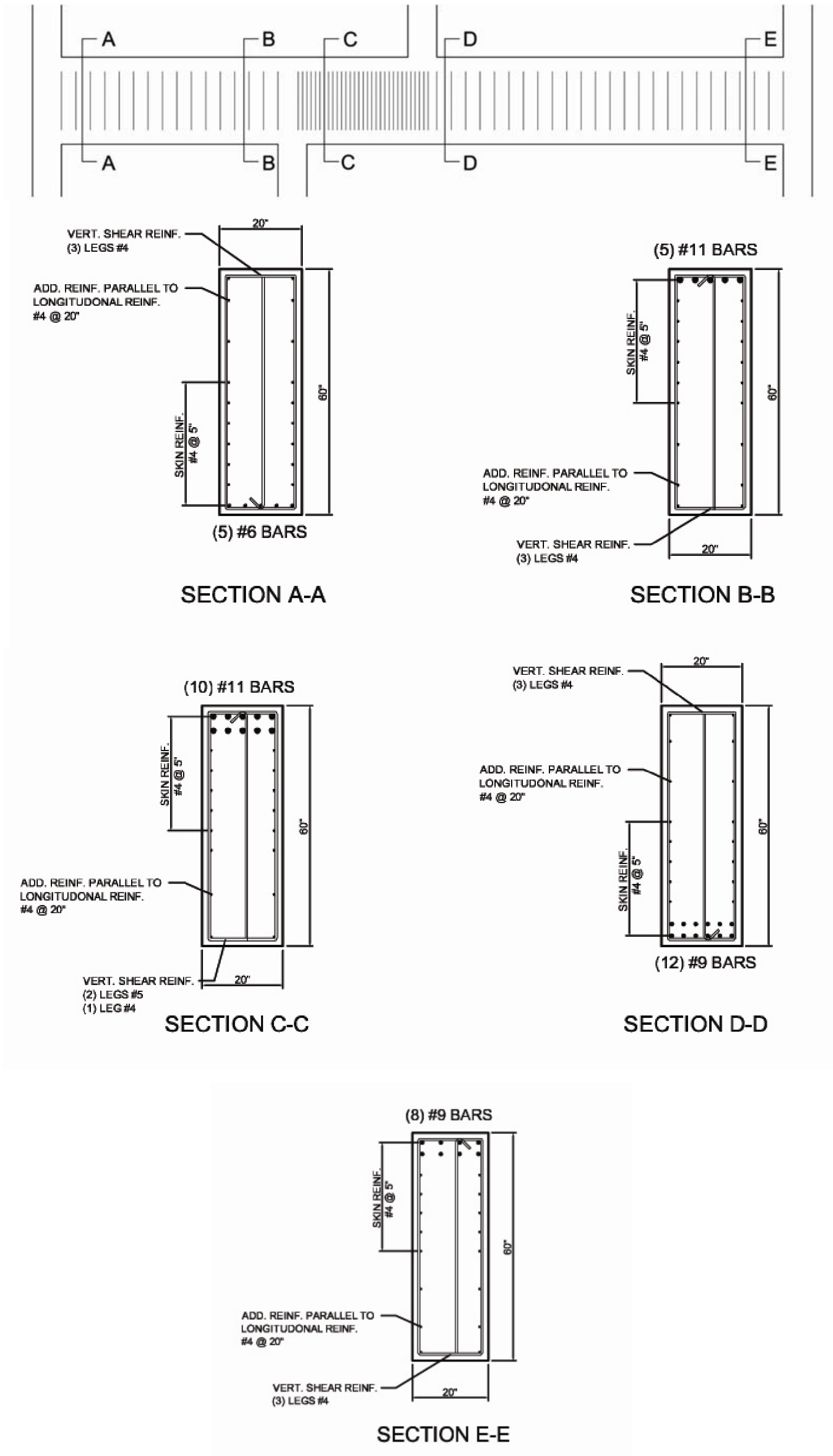
SAP Output – Moment Diagrams



SAP Output – Shear Diagrams



SAP Output Window


A
B
C
D
E
VERT. SHEAR REINF.
(3) LEGS #4
ADD. REINF. PARALLEL TO
LONGITUDONAL REINF.
#4 @ 20"
SKIN REINF.
#4 @ 5"
20"
60"
(5) #6 BARS
SECTION A-A
(5) #11 BARS
SECTION B-B
(10) #11 BARS
VERT. SHEAR REINF.
(2) LEGS #5
(1) LEG #4
SECTION C-C
(12) #9 BARS
SECTION D-D
(8) #9 BARS
SECTION E-E

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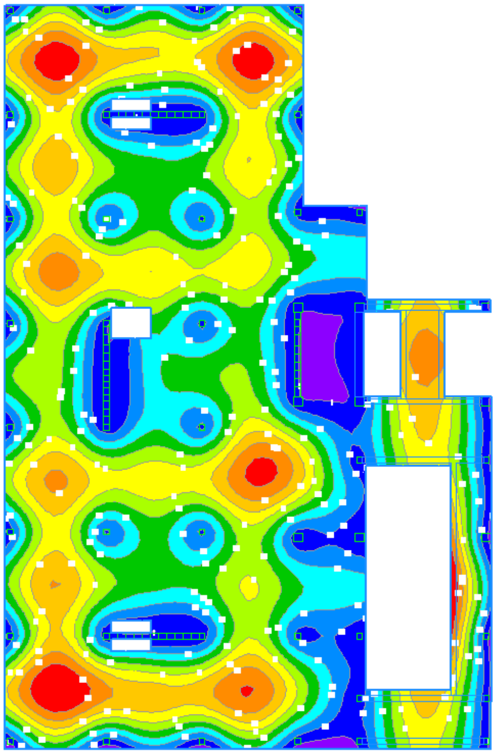
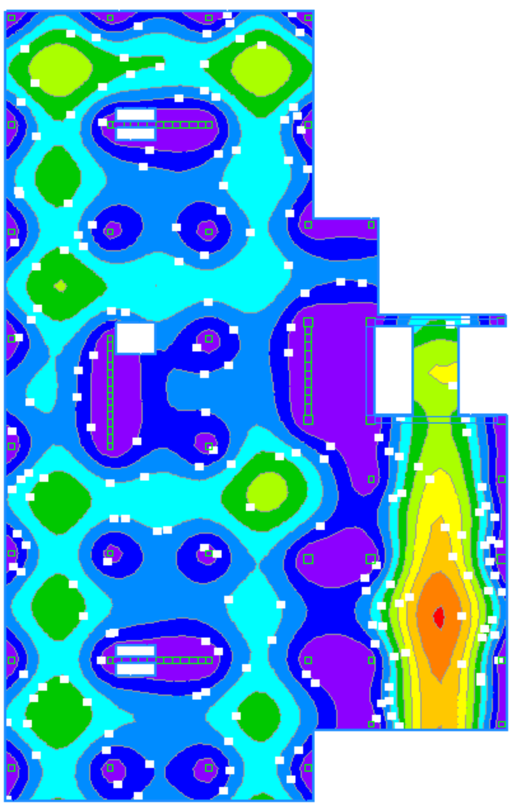
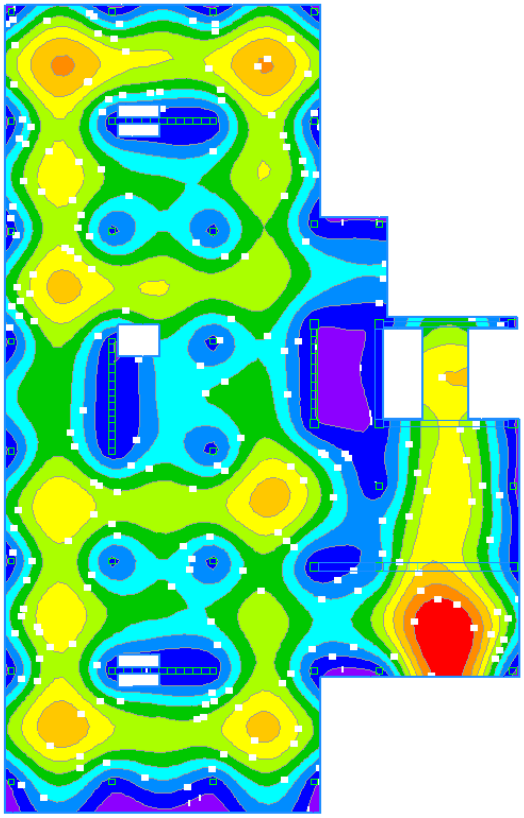


Model	4thflr	
slab	12	in
edge bm	no	
edge col	16	16
Δd+l	0.1926	
ΔLi	0.0602	ok
Δdt	0.2648	
Δ20%lt	0.0241	
Δ	0.4815	ok



Model	5thflr	
slab	12	in
edge bm	no	
edge col	20	20
Δd+l	0.2730	
ΔLi	0.0853	ok
Δdt	0.3754	
Δ20%lt	0.0341	
Δ	0.6825	ok



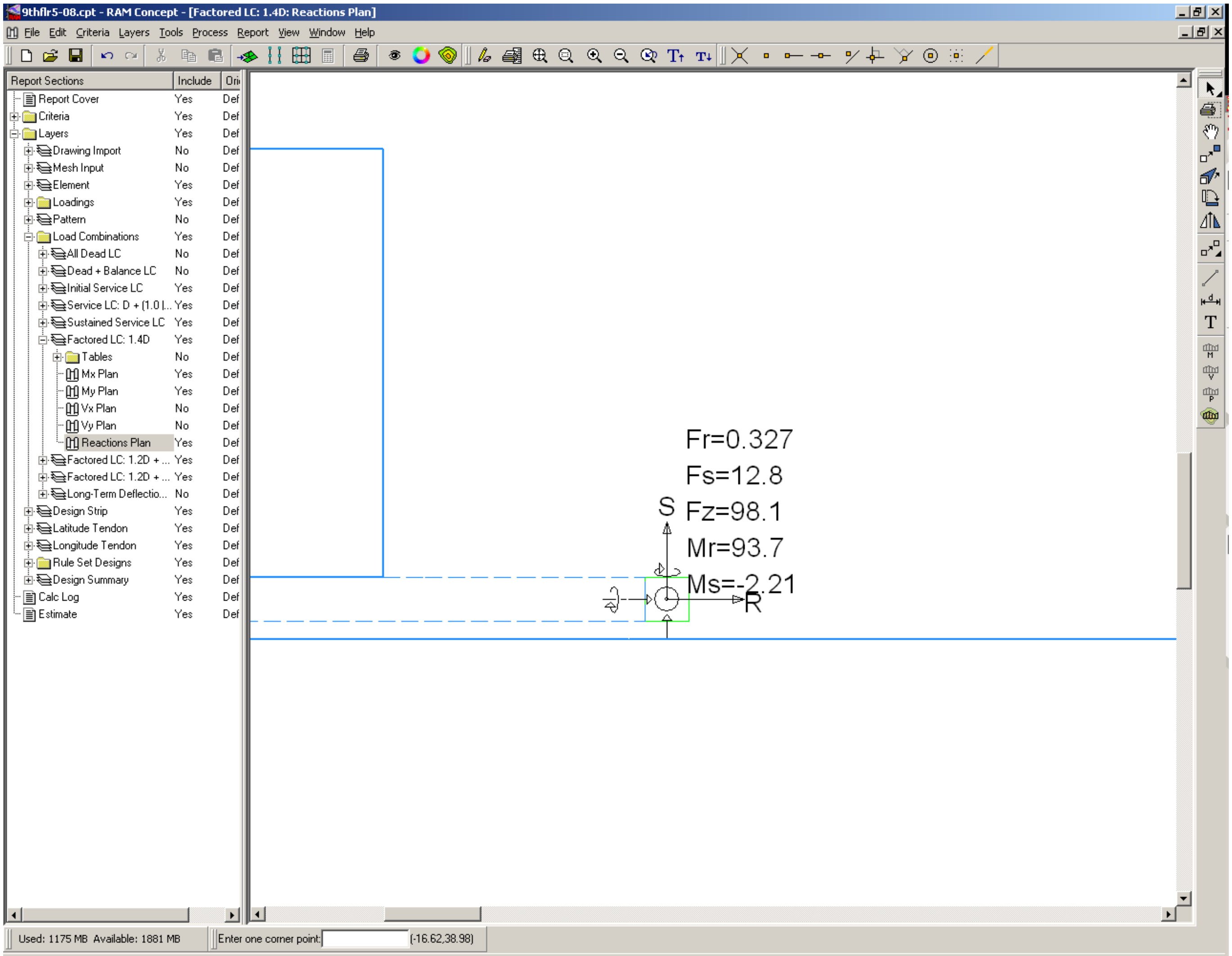
Model	6thflr	
slab	12	in
edge bm	no	
edge col	16	16
Δd+l	0.1900	
ΔLi	0.0594	ok
Δdt	0.2613	
Δ20%lt	0.0238	
Δ	0.4750	ok

Model	6thflr	
slab	12	in
edge bm	no	
edge col	16	16
Δd+l	0.2880	
ΔLi	0.0900	ok
Δdt	0.3960	
Δ20%lt	0.0360	
Δ	0.7200	ok

Alternate Design using 10" slab

Model	3rdflr	
slab	10	in
edge bm	no	
edge col	16	16
Δd+l	0.2547	
ΔLi	0.0796	ok
Δdt	0.3502	
Δ20%lt	0.0318	
Δ	0.6368	no good

Deflections are too large, 10" slab does not work. Use 12"



Sample RAM Concept “Reactions Plan” Output – Col F.1/4.5

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EX Elastic Analysis				
	Displ X	Displ Y	Drift Ratio X	Drift Ratio Y
HI ROOF	1.6257	-0.4980	0.0009	0.0003
13	1.4900	-0.4489	0.0010	0.0003
12	1.3486	-0.3993	0.0010	0.0003
11	1.2001	-0.3492	0.0011	0.0004
10	1.0550	-0.3021	0.0011	0.0003
9	0.9053	-0.2555	0.0011	0.0003
8	0.7543	-0.2101	0.0011	0.0003
7	0.5961	-0.1641	0.0010	0.0003
LO ROOF	0.4593	-0.1245	0.0009	0.0003
5	0.3279	-0.0878	0.0008	0.0002
4	0.2203	-0.0586	0.0007	0.0002
3	0.1297	-0.0342	0.0005	0.0001
2	0.0607	-0.0149	0.0003	0.0001

EXMZ Elastic Analysis				
	Displ X	Displ Y	Drift Ratio X	Drift Ratio Y
HI ROOF	1.6928	-0.5487	0.0010	0.0004
13	1.5485	-0.4934	0.0011	0.0004
12	1.3965	-0.4375	0.0011	0.0004
11	1.2385	-0.3812	0.0012	0.0004
10	1.0849	-0.3286	0.0012	0.0004
9	0.9273	-0.2766	0.0012	0.0004
8	0.7690	-0.2262	0.0012	0.0004
7	0.6043	-0.1755	0.0011	0.0003
LO ROOF	0.4647	-0.1324	0.0010	0.0003
5	0.3310	-0.0928	0.0008	0.0002
4	0.2219	-0.0619	0.0007	0.0002
3	0.1306	-0.0362	0.0005	0.0002
2	0.0613	-0.0159	0.0003	0.0001

EX Amplified Deflections				
	Displ X	Displ Y	Drift Ratio X	Drift Ratio Y
HI ROOF	3.0857	-0.9453	0.0018	0.0006
13	2.8283	-0.8522	0.0019	0.0007
12	2.5599	-0.7579	0.0020	0.0007
11	2.2779	-0.6628	0.0020	0.0007
10	2.0025	-0.5735	0.0021	0.0007
9	1.7185	-0.4850	0.0021	0.0006
8	1.4319	-0.3988	0.0021	0.0006
7	1.1315	-0.3114	0.0019	0.0006
LO ROOF	0.8718	-0.2364	0.0017	0.0005
5	0.6223	-0.1666	0.0015	0.0004
4	0.4181	-0.1112	0.0013	0.0003
3	0.2462	-0.0648	0.0010	0.0003
2	0.1153	-0.0282	0.0006	0.0002

EXMZ Amplified Deflections				
	Displ X	Displ Y	Drift Ratio X	Drift Ratio Y
HI ROOF	3.2132	-1.0415	0.0019	0.0007
13	2.9393	-0.9365	0.0020	0.0007
12	2.6507	-0.8304	0.0021	0.0007
11	2.3508	-0.7236	0.0021	0.0007
10	2.0593	-0.6237	0.0022	0.0007
9	1.7601	-0.5250	0.0022	0.0007
8	1.4597	-0.4294	0.0021	0.0007
7	1.1470	-0.3331	0.0020	0.0006
LO ROOF	0.8821	-0.2513	0.0018	0.0005
5	0.6283	-0.1761	0.0015	0.0005
4	0.4212	-0.1175	0.0013	0.0004
3	0.2479	-0.0687	0.0010	0.0003
2	0.1164	-0.0302	0.0006	0.0002

EY Elastic Analysis				
	Displ X	Displ Y	Drift Ratio X	Drift Ratio Y
HI ROOF	-0.3219	1.1233	0.0002	0.0008
13	-0.2908	1.0015	0.0002	0.0008
12	-0.2594	0.8804	0.0002	0.0008
11	-0.2276	0.7606	0.0002	0.0008
10	-0.1978	0.6508	0.0002	0.0008
9	-0.1682	0.5445	0.0002	0.0008
8	-0.1392	0.4432	0.0002	0.0007
7	-0.1096	0.3425	0.0002	0.0006
LO ROOF	-0.0860	0.2569	0.0002	0.0005
5	-0.0609	0.1800	0.0002	0.0005
4	-0.0403	0.1185	0.0001	0.0004
3	-0.0233	0.0667	0.0001	0.0003
2	-0.0104	0.0275	0.0001	0.0002

EYMZ Elastic Analysis				
	Displ X	Displ Y	Drift Ratio X	Drift Ratio Y
HI ROOF	-0.3474	1.1733	0.0003	0.0009
13	-0.3098	1.0448	0.0003	0.0009
12	-0.2721	0.9170	0.0003	0.0009
11	-0.2345	0.7906	0.0003	0.0009
10	-0.1996	0.6749	0.0003	0.0008
9	-0.1655	0.5630	0.0002	0.0008
8	-0.1328	0.4567	0.0002	0.0007
7	-0.1006	0.3513	0.0002	0.0007
LO ROOF	-0.0736	0.2627	0.0002	0.0006
5	-0.0518	0.1832	0.0001	0.0005
4	-0.0339	0.1204	0.0001	0.0004
3	-0.0189	0.0677	0.0001	0.0003
2	-0.0077	0.0278	0.0001	0.0002

EY Amplified Deflections				
	Displ X	Displ Y	Drift Ratio X	Drift Ratio Y
HI ROOF	-0.6110	2.1321	0.0004	0.0016
13	-0.5519	1.9010	0.0004	0.0016
12	-0.4923	1.6712	0.0004	0.0016
11	-0.4321	1.4437	0.0004	0.0016
10	-0.3755	1.2353	0.0004	0.0015
9	-0.3193	1.0335	0.0004	0.0014
8	-0.2642	0.8413	0.0004	0.0013
7	-0.2080	0.6501	0.0003	0.0012
LO ROOF	-0.1632	0.4877	0.0003	0.0010
5	-0.1155	0.3417	0.0003	0.0009
4	-0.0764	0.2249	0.0002	0.0007
3	-0.0442	0.1267	0.0002	0.0006
2	-0.0198	0.0522	0.0001	0.0004

EYMZ Amplified Deflections				
	Displ X	Displ Y	Drift Ratio X	Drift Ratio Y
HI ROOF	-0.6594	2.2270	0.0005	0.0017
13	-0.5881	1.9831	0.0005	0.0017
12	-0.5165	1.7406	0.0005	0.0017
11	-0.4450	1.5007	0.0005	0.0016
10	-0.3788	1.2811	0.0005	0.0016
9	-0.3141	1.0687	0.0005	0.0015
8	-0.2521	0.8669	0.0004	0.0014
7	-0.1909	0.6669	0.0004	0.0013
LO ROOF	-0.1397	0.4986	0.0003	0.0010
5	-0.0984	0.3477	0.0003	0.0009
4	-0.0643	0.2286	0.0002	0.0007
3	-0.0359	0.1285	0.0002	0.0006
2	-0.0147	0.0528	0.0001	0.0004