40 Gold Street Residential Building

New York, New York

THESIS FINAL REPORT

Jesse Cooper – Structural Option
Thesis Consultant: Dr. Boothby
Date of Submission: April 7th, 2010

40 GOLD STREET

NEW YORK, NEW YORK

JESSE T. COOPER STRUCTURAL

PROJECT INFORMATION

Location: 40 Gold Street NY, NY
Occupancy Type: Retail & Residential
Size: 8,000 SF Retail & 52,000 SF Residential
Stories Above Grade: 14 (1 Below Grade)
Construction Dates: June 2009 - January 2010
Project Cost: Figures Not Released
Project Delivery Method: Design-Bid-Build

PROJECT TEAM

Owner: Werber Management
Structural: Severud Associates
General Contractor: MJM Construction Services
MEP: TSF Engineering
Architect: Meltzer / Mandl Architects, P.C.

ARCHITECTURE

40 Gold street is an impressive architectural package that offers retail and residential space in lower Manhattan. The lowest two floors are primarily dedicated to retail space and serve as a podium on which a sleek 14 story residentail tower rests. The retail space is appropriately highlighted with the traditional floor-to-ceiling store-front windows and is complemented by a pre-fabricated assembly of dark stone cladding. The residential floors' facade has a distinctly different appearance from the first floor boasting a clean cut, organized pattern of two different shades of metal cladding on the East/West faces and a gold toned trespa panelling on the North/South faces. The trespa is a decorative exterior finish and creates an aesthetically pleasing juxtaposition with the metal cladding. The penthosue level is equipped with two recreation terraces, a trellis, and a recreation room enclosed by a windowall system.

MEP

3 300 MBG input Gas Fired Boilers.
5 Rooftop Toilet Exhaust fans at 900-1800 CFM each.
3 Rooftop Kitchen Exhast fans at 1800 CFM each.
2 TACO HWP Pumps at 85 GPM
2 RTAC units with 1995 CFM supply capacity
Packaged terminal gas Heating/Electric AC units.
2 AC-HU's on 1st floor and 1 AC-HU in Cellar

STRUCTURE

The structural system for 40 Gold Street is primarily of a composite steel frame with slab on metal deck to resist gravity loads, and a combination of of braced frames and moment frames function as the lateral resisting system. The foundation employs a system of thirteen 25' micro piles with 35 ton compression capacity and eighty eight 35' long micropiles with 75 ton compression capacity. At the cellar level, the floor is a 8" slab on grade with #5 bars @ 12" o.c. top and bottom running both directions. The s.o.g rests on 6" of crushed stone. At ground level where the retail and residential lobby spaces are located, the floor system is a 2"-18 gage metal deck with a one way 2-1/2" Light weight concrete topping (4,000 psi). This floor system is typical throughout the above residential levels. Overall, the framing plan is a uniform grid-like layout with few irregularities. The average bay size is approximately 15'8" x 14'. The beams and girders are all W shapes ranging from W10's to W14's. At the 2nd level, several beams project 2 feet outward and behave as cantilevers to support the 13 stories above. The vertical structural members are primarily W10's/W12's. HSS4x4x3/8 posts are used in various locations as well.

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Appendices: All Hand Calculations Can Be Found In The Appendices

References:

Figure / Description	Source
Cover Page / Abstract Rendering	Severud Associates and Meltzer/Mandl
F-18 / Waffle Slab Forms Image	http://www.designguide.com/staging/imagescustomers/10573/Image5/5.jpg
F-118 / Architectural Floor Plan	Severud Associates and Meltzer/Mandl
Existing Geotechnical Information	GZA GeoEnvironmental of New York - Patrick Mahon
Structural Concrete Design / Provisions / Equations	ACI 318 - 08 Handbook
URM and RM Masonry Wall Design/ Provisions / Equations	MSJC Handbook
F-143 / Story Drifts	ASCE7-05 Handbook
F-153 / Typical Elevated Slab Detail (Existing)	Severud Associates
F-169 / Existing Exterior Wall Detail	Severud Associates
F-172 / Site Plan	Severud Associates
Online Cost Analysis	RS Means Online Cost Works
Labor Output / Durations	2009 RS MEANS: Heavy Construction

Executive Summary:

Located in lower Manhattan New York City, 40 Gold Street is an impressive 13 story residential building. The 21' high first story is dedicated to all retail space, and serves as podium for the residential tower. The existing structural system is a steel frame system with slab on concrete metal decking. The scope of this thesis report encompasses a complete redesign of the structural system as a concrete structure with a flat plate waffle slab system. Also included is a construction management study and acoustic study.
The structural design depth was broken into three primary areas of design which included the slab design, the column design, and the shear wall lateral system design. Several types of slabs were designed due to changes in live loads and column layouts throughout the building. For example the 1st floor requires a 8" flat plate due to the 100 psf retail live load. For the most part the slab is designed as a waffle slab composed of a 3 ½ "slab and 4" wide x 8" deep (11.5" deep total) ribs spaced at 20" or 16". Top reinforcement is generally #5 @ 16" and the bottom reinforcement is either (1) #5 per rib, (2) #5 per rib, or (1) #6 per rib. The waffle slab self weight is approximately 46 psf (other slab types weigh more), which is a 35% increase from the existing 34 psf slab. . As shown on page 173, six different column sections are used in the final design (**A-F**). All columns are square tied columns ranging in size from 10x10 to 16x16. As mentioned in the column design section of the report, since 40 Gold Street is located in a non-seismic region, square tied columns were most appropriate. The primary disadvantage of the reinforced concrete columns is poor efficiency in terms of weight to strength ratio. The concrete columns weigh 340.2 kips ($\frac{87CY}{.037037} * 150pcf/1000$) whereas the columns in the original structure have a total weight of 260.7 kips. The strength behavior of the shear walls can be classified as tall, slender, flexural walls. The final lateral system consists of 7 shear walls, with 4 in the X direction and 3 in the Y direction. The shear walls are all 10" thick and range in length from 11'-0" to 19'-0".

The primary motive for electing to redesign the structure as a concrete structural system was to satisfy the client, who expressed his preference for a concrete structural system. Based on the research and data compiled in this report, it is evident that the concrete structural system offers a higher performing design solution for a New York City residential building. Obviously, other factors played a role in the selection of the structural system, since the existing system is a steel frame building with slab on metal decking. As will be explained, these factors included feasibility in terms of building weight and settlement potential, cost, construction duration, site congestion, and constructability.

In the end, designing a concrete structure with an acceptable weight (7,000 kips) was successfully achieved. With maintaining a low building weight being a primary goal, constructability of design was not dismissed but was not of high priority. Calculations and data suggest the concrete structural design is a viable option that yields a higher performing residential facility. However, the downside to the design is increased cost, increase construction duration, site congestion, and constructability issues. The cost analysis revealed the concrete structural system would cost $2,318,956, which is $579,921 more than the original structure ($1,739,035). As for the construction durations, the new concrete structural system requires (95) 8 hour work days, which is 2.5 times longer than the (37) 8 hour work days required for the existing structure. Advantages include improved acoustic performance, architecturally pleasing (leave exposed), improved floor-to-ceiling height, no additional fire proofing required, no framing interfering with MEP systems, and most importantly full client satisfaction (preferred concrete structure). In summary, the concrete structure is a feasible design solution offering several performance improvements; however, the increased construction cost and duration, and constructability problems all suggest the design is less reasonable than the existing structural design.

Acknowledgements:

Many individuals played a major role in providing advice, information, and time that helped with this thesis project. The efforts of the following individuals are greatly appreciated.

David Werber, President of Werber Management and Owner of 40 Gold Street
Larry Turk and David Stuart of Architectural Firm Meltzer / Mandl
Severud Associates Structural Engineers: Lev Tsukerman and Steve Reichwein
Professor Parfitt of Pennsylvania State University
Professor Thomas Boothby, Thesis Advisor and Professor of Pennslyvania State University
Professor Bob Holland of Pennsylvania State University

Thank You Very Much!

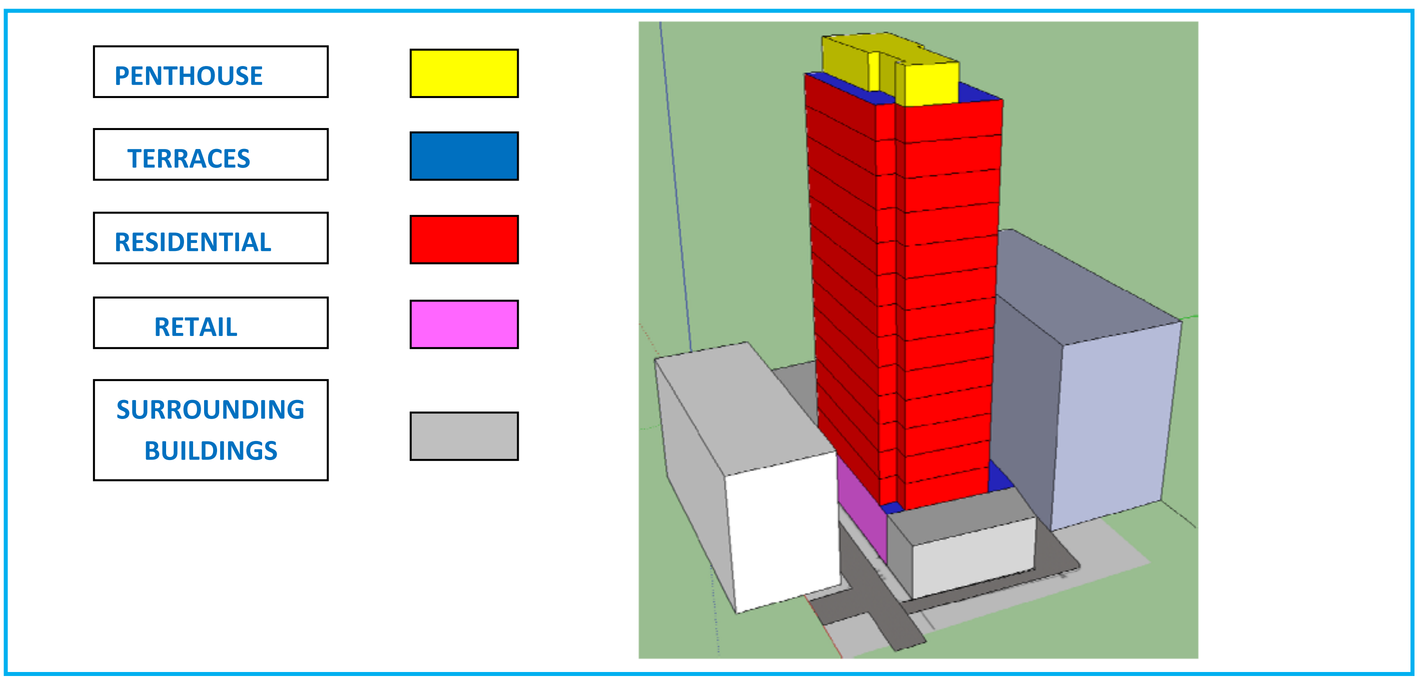
It has been an enlightening and invaluable experience. It was a pleasure working with everyone.

Introduction and Background Information:

40 Gold Street is an impressive architectural package that offers retail and residential space in lower Manhattan, which is one of the fastest growing residential sections of New York City. The construction of 40 Gold Street began in March 2009 and will conclude in January 2010. The building replaces an old two story brick building and is nestled tightly between two existing structures, a narrow alley (Eden's Alley), and Gold Street. The constricted area presented special restrictions and challenges that greatly affected the final design and construction process.

Standing 175' above grade, the 40 Gold Street Building is a 14 story structure comprised of 5,900 square feet of retail space and 62,000 Square feet of residential space. The lowest two floors are primarily dedicated to retail space and serve as a podium on which a sleek 14 story residential tower rests. The lowest floor, referred to as the cellar, is below grade and functions as extra retail space as well as space for mechanical and electrical equipment. Retail spaces are appropriately located at the ground level and are highlighted with traditional floor to ceiling storefront windows to attract customers from the nearby streets and sidewalks. The storefront glazing is complemented very nicely with a pre fabricated assembly of dark stone cladding and a large bronze plaque that boldly recognizes the building as 40 Gold Street. In addition to retail space, there is a residential lobby and mailroom.

The residential tower is comprised of 12 residential floors. Identical in layout, floors 2-9 are comprised of 2 studio apartments and 3 2 bedroom apartments that all encompass the vertical circulation node located at the core of the tower. Two elevators and a stairwell serve as the buildings vertical circulation. Floors 10-13 are identical as well, but have 4 2-bedroom apartments and no studio apartments. At the top of the building, a level referred to as the penthouse provides the building's residents with two spacious recreational terraces sheltered by a gold painted metal trellis, a large recreational room enclosed by a window wall system, a kitchenette, a laundry room, and bathrooms.



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Introduction Continued:

The trapezoidal shape of the building closely reflects the shape of the site, which is to be expected when working with such a constricted space. The interior spaces are laid out in a very rectangular manner, and the exterior shell is also very rectangular. The residential tower boasts a sleek modern appearance with metal exterior cladding and gold toned trespa paneling.

Building Envelope: Floors 2-14 are enclosed by a basic non-bearing exterior metal panel wall assembly. The general composition of the wall shown in figure F-169 is 2" metal cladding (exterior), air and moisture barrier, 5/8" exterior dens-glass sheathing, 6" metal studs, 6" batting insulation, and 5/8" gypsum board (interior).

The sub grade spaces, also referred to as the cellar, are enclosed by a cast-in-place concrete wall with Krystol waterproof admixture. Retail areas on the street level are enclosed by a large aluminum and glass storefront anchored to a basic CMU block wall assembly which consists of 2" stone panel (exterior), waterproofing membrane, 6" CMU block, 1" rigid insulation, 5/8" gypsum on 1 ½" furring channel (interior). The storefronts are also equipped with a roll-down gate for security purposes.

Sustainability Features: Although the overall design wasn't driven by sustainability, the 40 Gold Street building includes several green features throughout the design. The apartments are equipped with energy star appliances. In addition, the windows are assembled with low-emissive glass. The roofing materials are designed to prevent or minimize the heat island effect, and the building envelope is highly proficient for thermal and moisture protection. The exterior façade also has an 8" metal fin projecting out from above each of residential windows, which serves as a shading device.

Overall, the final design solution created by Architects Meltzer/Mandl and Structural Engineers Severud Associates makes the most of a small site, and is certainly playing a major role in the successful rebuilding of Lower Manhattan.

Thesis Proposal and Scope of Work:

Problem Statement

Project Goal: Redesign the structural system to optimize the buildings performance as a residential structure without significantly deviating from the final product envisioned by the owner.

40 Gold Street's existing structural system consists of steel framing and lightweight slab on composite metal decking. Located on a site with poor soil conditions, significant settlement potential exists. With two existing structures located within great proximity of the 40 Gold Street building, the ramifications of settlement would be profound. The structural design engineers, Severud, determined that a steel structural system was necessary to minimize total building weight and avoid disturbing the neighboring structures.

Although the existing steel structural system sufficiently meets the various requirements associated with a residential building, several aspects of the design require improvement in order to improve the building's performance. With the current steel frame structural system, a larger total floor construction depth results in an excessive floor-to-ceiling height. Also, the existing system requires additional fireproofing (spray on) and interferes with critical spaces often reserved for MEP equipment and raceways. Unlike the existing steel frame system, alternative concrete structural systems offer superior vibration and acoustic control. As part of this project, a solution to high ambient noise levels will be provided to ensure interior spaces are exposed to acceptable noise levels. Most importantly, Werber Management, the 40 Gold Street owners, expressed that, if practical, a concrete structural system was their first preference.

In summary, the challenge of this study requires designing and evaluating an alternative structural system that meets the clients request for a concrete system and appropriately addresses the potential settlement issues without sacrificing the building performance. In the end, this study should reveal if a practical and efficient concrete structural system can be designed for 40 Gold Street. Site congestion, construction cost, construction duration, and constructability will also be addressed to fully evaluate the practicality of the alternative structural design. Research and data results produced in this report will reveal if the proposed concrete structural system can yields enhanced performance and if it can be designed with a low enough total building weight to avoid both settlement issues and the need to redesign the foundation system.

Thesis Proposal and Scope of Work Continued:

Proposed Solution: To redesign the 40 Gold Street building using a two way flat plate (waffle slab) system.

This will also involve a redesign of the lateral system. After evaluating several concrete structural systems in Technical Report 2, the two-way flat plate system was identified as the best alternative structural system. With small average bay sizes of approximately 14'-0" x 16'-0" and the low residential live load (40 psf), preliminary design calculations revealed a two way flat plate floor depth of 8" could be achieved. This is a significant reduction from the existing 1'-7" floor depth. With the improved floor to ceiling height, owners and designers are presented with the option to include additional stories or higher ceilings without improving the total building height. The two-way flat plate system is also attractive because it requires simple construction, simple formwork, and widely available materials. Additional advantages include flexibility for partition location, superior vibration and acoustic control, and no additional fire resistance is required. Perhaps the most significant advantage is the ability to utilize the floor system as both an architectural and structural element. The concrete slab is commonly left exposed and serves as finished floors and ceilings in residential spaces.

In order to both verify and demonstrate that the two-way flat plate structural system improves the buildings overall performance in providing a good residential environment, a detailed continuation of the comparison study from Technical Report 2 will be conducted. Specifically, a detailed acoustic study will be performed to determine how to improve the acoustic performance of 40 Gold Street. Located in New York City, high ambient noise levels will require special exterior wall design considerations. In addition, a design solution will be devised to establish proper sound isolation between the penthouse and vertically adjacent residential spaces. With HVAC equipment, a laundry room, and a weight room in the penthouse, there are many sources of large magnitude noises and vibrations.

Based on the initial analysis conducted in Technical Report 2, a redesign of the structural system as a two-way flat plate system will result in a total building weight that greatly exceeds the current weight of 4,681,330 lbs. In fact, an 8" normal weight concrete slab weighs approximately 100 psf, which is nearly 3 times the weight of the current floor structure (34 psf). Therefore, an additional area of focus will involve maintaining an overall building weight that does not exceed the allowable loading of the existing micro pile foundation system. Light weight concrete waffle slab will therefore be the type of two way flat plate system used in the gravity system redesign.

Thesis Proposal and Scope of Work Continued:

After reviewing the geotechnical report and consulting ASCE7-05, the site class was determined as D. The current soil is composed primarily of medium dense silty sands and required a robust liquefaction analysis. Although it was ultimately determined that liquefaction is unlikely to occur, it is important to maintain a low building weight. It is important to note, the scope of this project does not include redesign of the foundation system, but simply determining if the total building weight can be limited to the soil bearing capacity and capacity of the micro pile foundation system.

By changing the gravity system to a concrete structural system, the lateral system will be affected. Redesign of the lateral system will be conducted. With a 14 story building and the two-way flat plate (waffle) system, the most logical option would be a series of shear walls at the core and perimeter. The core shear walls would be located around the elevator shaft and stair wells to avoid interfering with the residential spaces and hallways.

Although the concrete structural system may yield a better performing building, the cost, construction duration, and overall constructability may outweigh these performance benefits deeming the redesign unpractical. Therefore, a detailed cost and schedule analysis will be conducted to compare the material and labor costs associated with the construction of the structural system. In addition, with a constricted site with limited access, a brief investigation of site logistics and planning will play a monumental role in determining the feasibility and payoff of a concrete structural system.

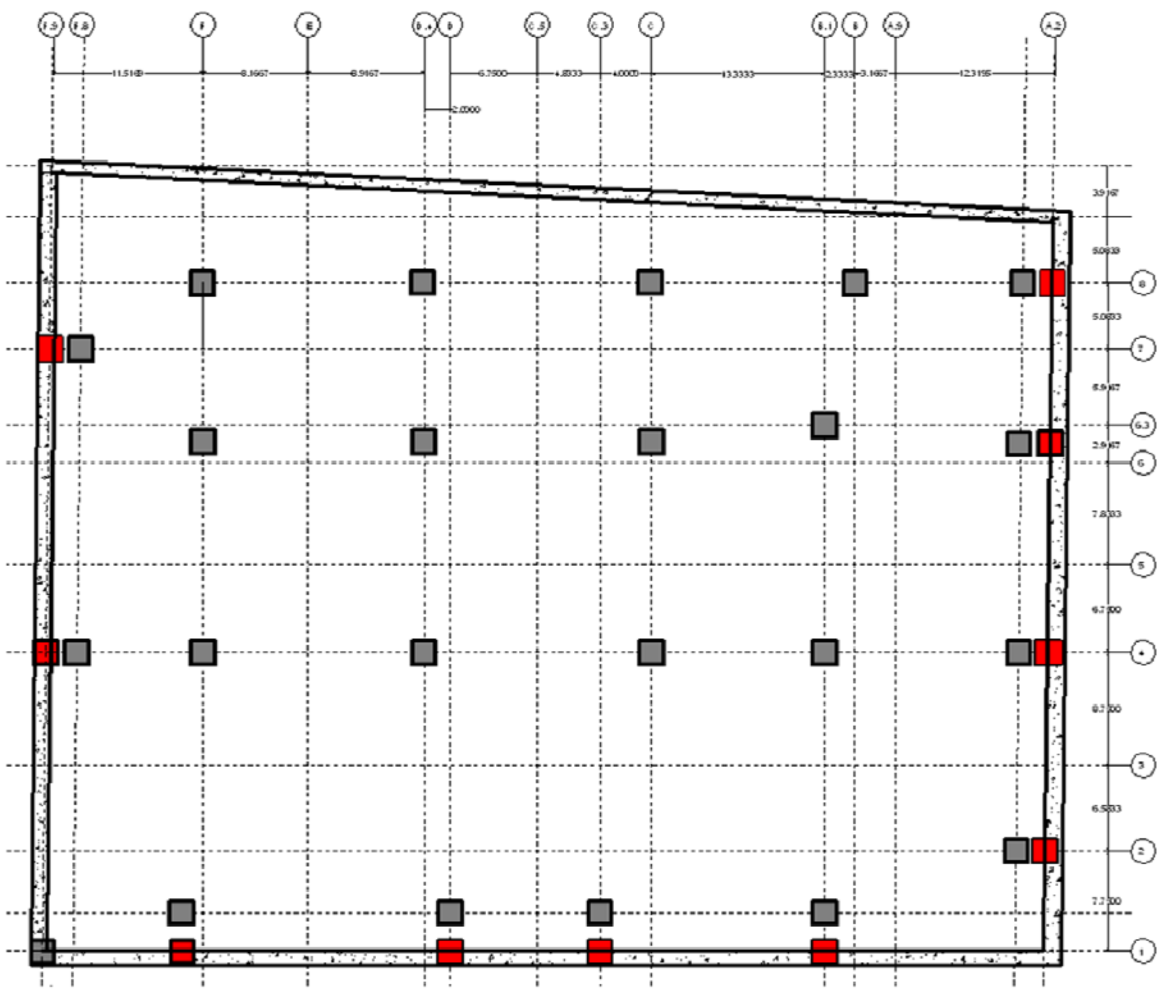
Proposed Column Layouts –

To begin, the general layout of the concrete columns was determined. The proposed concrete structure has different limitations compared to the existing steel frame system. Although the final column layouts, shown in figures F-2 through F-5, are similar to the existing column layouts, there are some significant modifications. As mentioned in the proposal, a primary design goal was to maintain as low a building weight as possible. As a result, a brief investigation of the "weight efficiency" of different column layouts was conducted. To be more specific, estimated weights were determined for a large bay column layout and for a small 15'x15' column layout. With the large bays, column sizes were approximated to require significantly large sections, but fewer columns were required. On the other hand, the smaller bay sizes required more columns; however, these columns required smaller sections. Upon completion of the investigation, it was concluded that the smaller bay size column layout would yield the least weight structure.

As one can see in the following column layout figures, a regular and uniform layout of columns was achieved. In some instances, the layout deviates from a rectangular grid layout due to architectural restraints. With doorways, hallways, windows, and essential residential living spaces scattered throughout the floor plans, the positioning of columns was difficult. In the end, all columns were located in non-critical locations where they did not interfere with circulation spaces or living areas. As a consequence of the change in apartment layouts at the upper floor levels, column 6-D.4 had to be discontinued at floor 10. As shown in figure F-5, this results in a large span of approximately 34'. As will be discussed later, this has significant ramifications on the slab design for floors 10 through 13, and generates large but manageable unbalanced moments in the surrounding columns.

To summarize the final column layouts, the bay sizes are approximately 15'x15', and there are four typical floor layouts. As shown in figure F-2, corbels are required at the perimeter of the first floor. Corbels were required to transfer loads to pile caps which are offset to the interior establishing greater distance from existing foundation systems.

First Floor Column Layout:



FIRST FLOOR COLUMN AND WALL LAYOUT (GROUND LEVEL) | RETAIL LEVEL: Live Load 100 PSF

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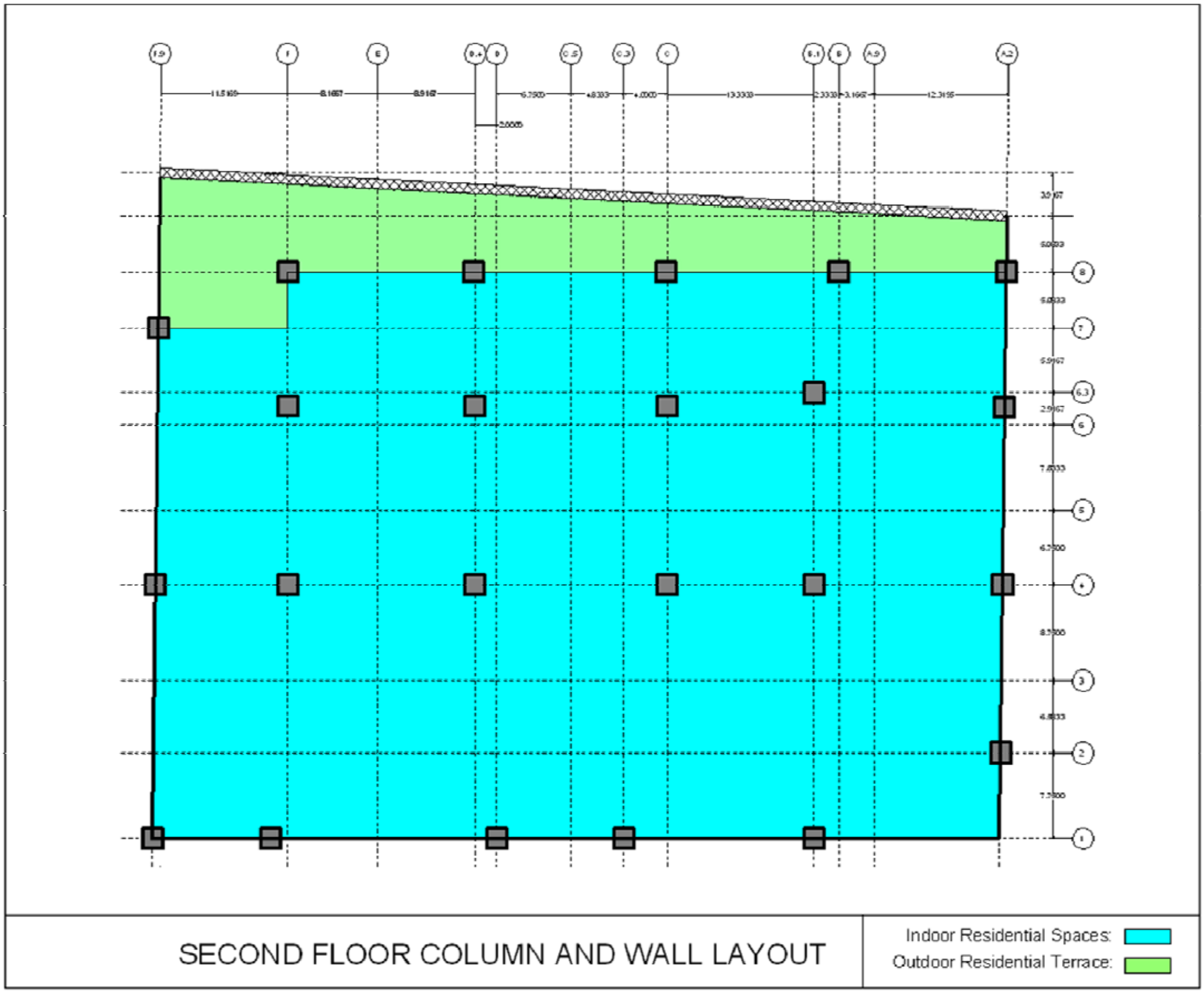
Note, the red columns represent the 2nd floor columns above. As one can see, they are offset from the first floor exterior columns. The foundation system was maintained at a distance away from the perimeter to avoid disrupting the existing foundations located at the site perimeter. As a result, corbels had to be designed to transfer the load between the 1st and 2nd floor columns.

Second Floor Column Layout:



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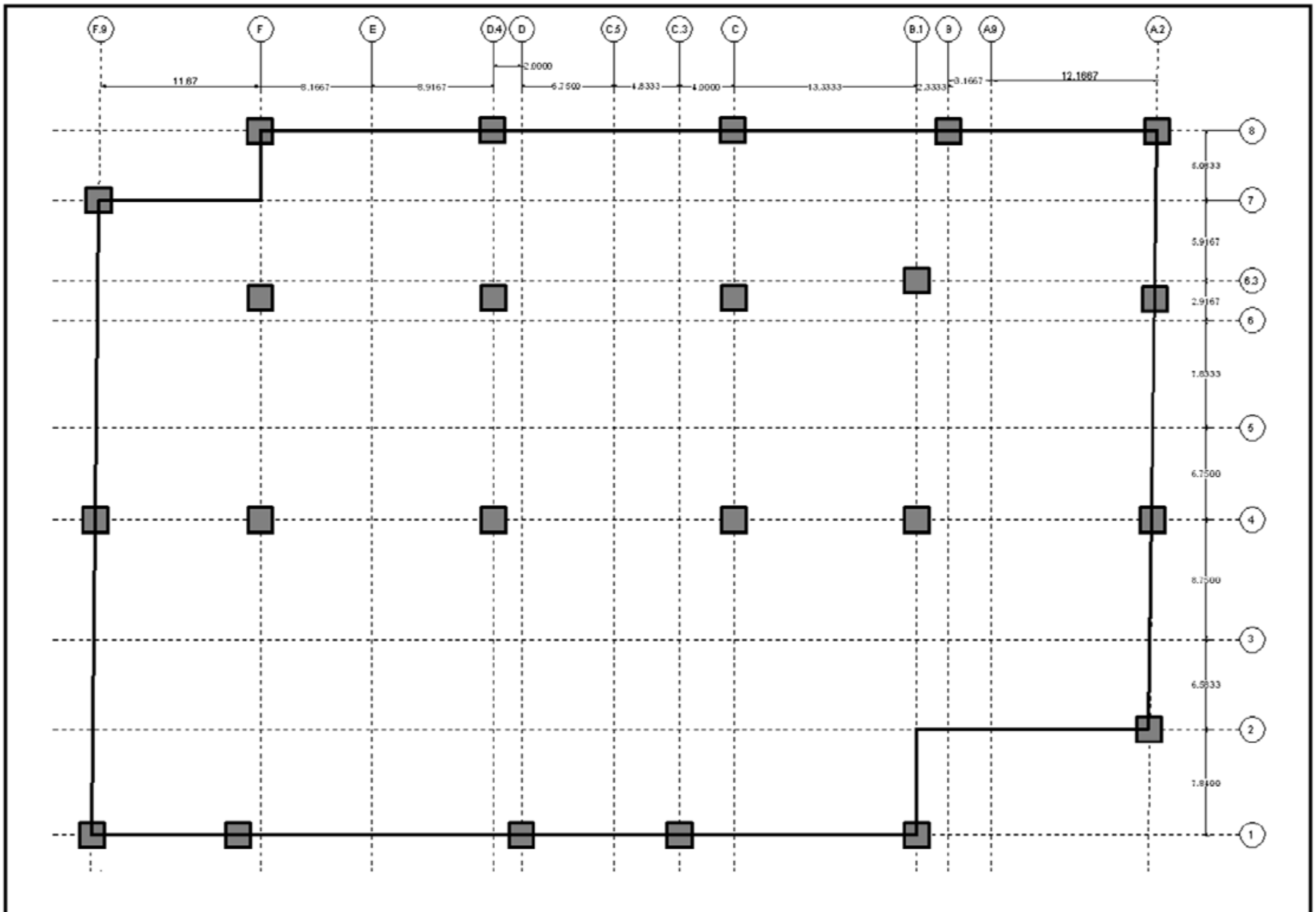
Column Layout – Floors 3 through 9:



PROPOSED COLUMN LAYOUT # 1	Jesse Cooper	
	FLOOR 3 through 9	

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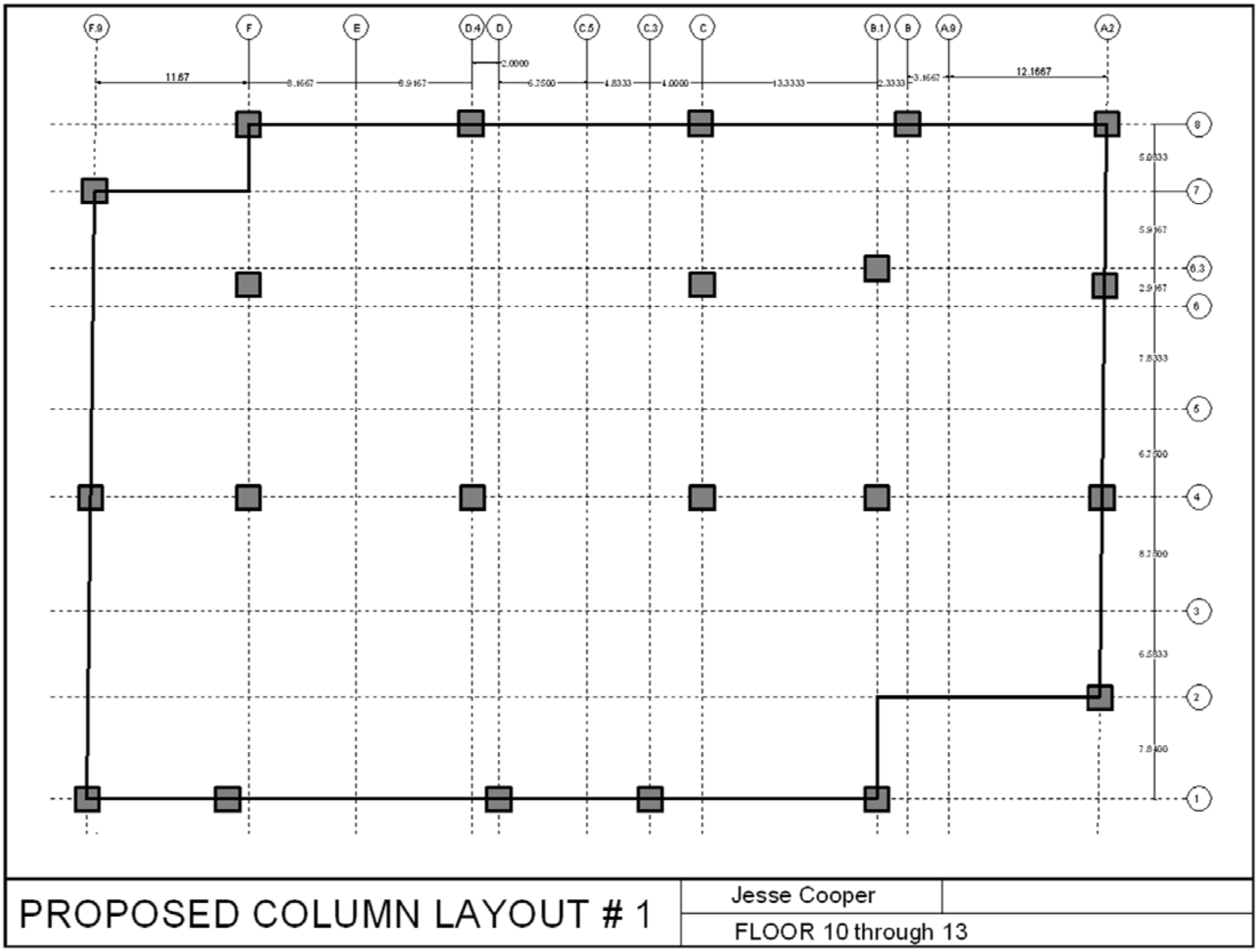
Column Layout – Floors 10 through 13:



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Slab Design Introduction:

Goal:

The primary goal of the slab design is to determine the least weight slab to support building loads. Two way waffle slab is the floor system of choice. However, from a constructability standpoint, employing too many variations of the waffle slab design should be avoided. In such cases, solid two way flat plates are an appropriate substitute.

Process:

Prior to computer aided design calculations, two sets of preliminary hand calculations, found in Appendix A, were completed to obtain a rough idea of the required slab dimensions and steel reinforcement. For the preliminary calculations, the Direct Design Method, outlined in Chapter 13.6 of ACI 318-08, was used to model two typical interior frames, one located at retail level and the other at a residential level. Although these preliminary calculations yielded two way flat plate designs, they still provided significant insight in determining a base waffle slab design.

With the aid of the preliminary calculations, the base waffle slab dimensions shown in figure F-7 were chosen. Several other factors influenced the final selection of this base waffle slab dimension. The 3 ½ "thick slab was chosen because it is the smallest slab thickness (for L.W.C.) that satisfies the required 2 hour fire rating. Also, in an attempt to fulfill a secondary design goal of improving floor to floor height, the rib depth was kept to 8" which amounts to a total slab thickness of 11 ½ " (4 ½" less than existing floor construction depth). A 4" rib width was chosen because it can provide the appropriate cover and spacing requirements for two #5 bars.

Once the base waffle slab design was chosen, each frame was carefully modeled and designed individually using spSLAB. Pages 19 – 61 show the modeled frames and the final design information. spSLAB operates by implementing the Equivalent Frame Method found in chapter 13.7 of ACI 318 -08. As expected several sections of the building either required a modified version of the base waffle slab design, or in some cases a solid two way flat plate was necessary.

Since the frames are designed individually as isolated frames, the true design challenge pertained to "piecing together" the frames based on their final designs. With non-uniform bay sizes and cantilevered slabs present intermittently throughout the structure, variation in slab depth and changes in the waffle slab rib spacing is prevalent. In addition, the spacing of ribs required exhaustive analysis to ensure a sufficient number of ribs were located within the column and middle strips of each frame (for placement of bottom reinforcement).

Deflection, shear, flexure, minimum cover, bar spacing, and maximum and minimum steel reinforcement requirements were all applied in the design of the slab systems. The material properties are displayed in figure F-6.

SPSLAB Input Information – Two Way Waffle Slab Design

Material Properties				
Concrete			Reinforcing Steel	
	Slabs / Beams	Columns		
Unit Density (lb/ft^3)	115	150	Yield Stress of Flexural Steel (ksi)	60
Compressive Strength (ksi)	5.95	5.95	Yield Stress of Stirrups (ksi)	60
Young's Modulus (ksi)	3139.2	4676.4	Young's Modulus (ksi)	29000
Rupture Modulus (ksi)	0.43389	0.57852		

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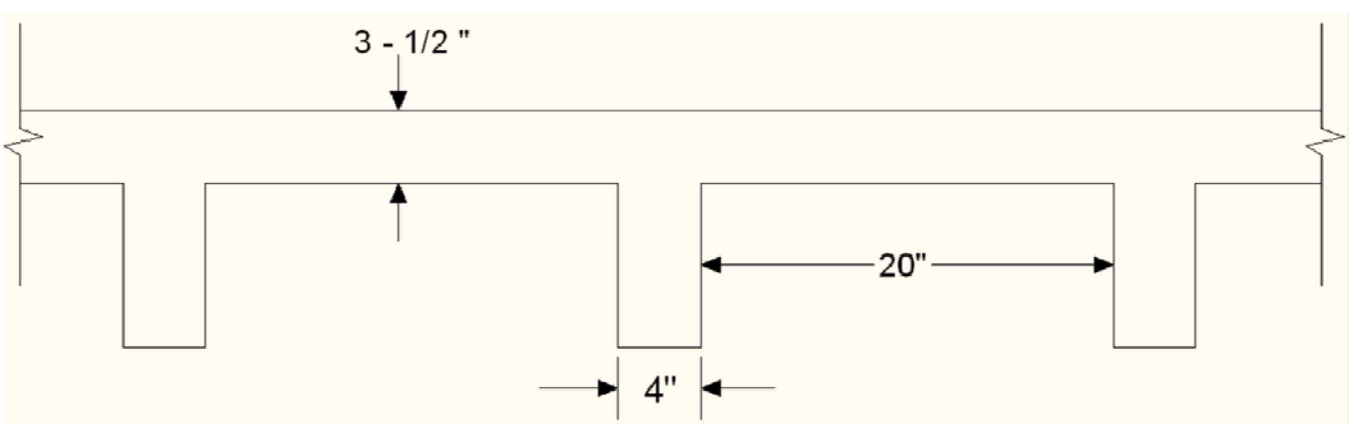
Base Waffle Slab Dimensions Used For Design:

Preliminary slab design calculations were completed using the base waffle slab dimensions shown in figure F-7 below, the 3.5" LWC slab/flange meets the two hour fire rating and was determined to be an appropriate dimension for the proposed column layout with an average bay size of 16.5' x 16.5'. Of course, larger bay sizes and/or irregular structural locations required adjustment to the base dimensions.



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PROFILE VIEW OF BASE WAFFLE SLAB DIMENSIONS

FLOORS 10 – 13: Slab Design Results

spSlab Design Results: Frame 8 (Floors 10 - 13)

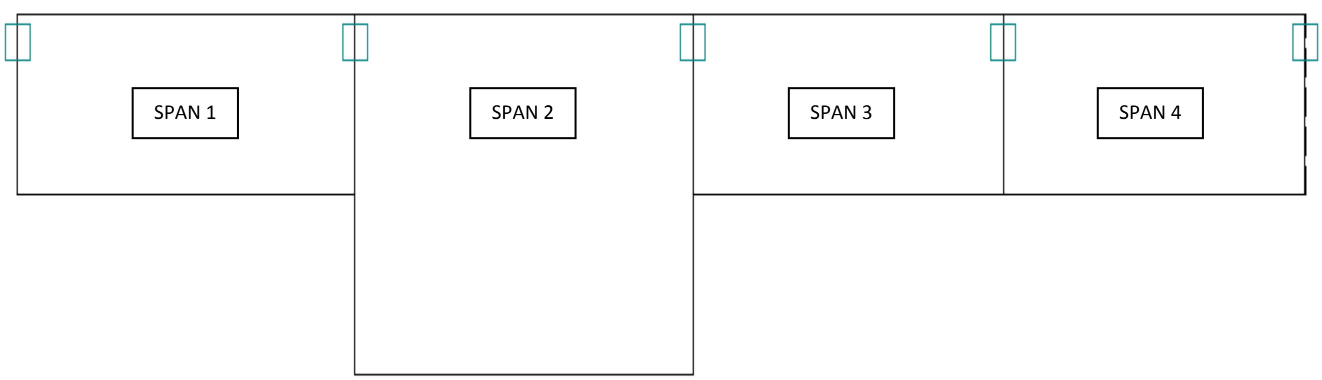
Top Reinforcement

Span	Strip	Left Side				Right Side				Continuous	
		Bars	Length	Bars	Length	Bars	Length	Bars	Length	Bars	Length
1	Column	2 - #5	5.82	1 - #5	3.78	2 - #5	6.86	1 - #5	3.78		
	Middle	2 - #5	4.09			7 - #5	6.86				
2	Column	2 - #5	5.82	1 - #5	3.78	2 - #5	5.82	1 - #5	3.78		
	Middle	7 - #5	4.86			7 - #5	4.11				
3	Column									3 - #5	15.6
	Middle	5 - #5	3.78							2 - #5	15.6
4	Column	2 - #5	5.23	1 - #5	3.42	2 - #5	5.23	1 - #5	3.42		
	Middle	2 - #5	4.74			2 - #5	3.69				

Bottom Reinforcement

Span	Strip	Long Bars			Waffle			Total Strip Width	Maximum Possible Number of Ribs Per Strip (20" clear spacing)		
		Bars	Start	Length	Ribs	Bars / Rib	As / Rib				
1	Column	2 - #5	0	17	1	2 - #5	0.62				
	Middle	2 - #5	0	17	2	1 - #5	0.31				
2	Column	4 - #5	0	17	2	2 - #5	0.62				
	Middle	4 - #5	0	17	4	1 - #5	0.31				
3	Column	2 - #5	0	15.6	1	2 - #5	0.62				
	Middle	2 - #5	0	15.6	2	1 - #5	0.31				
4	Column	2 - #5	0	15.2	1	2 - #5	0.62				
	Middle	2 - #5	0	15.2	2	1 - #5	0.31				

F -8



FLOORS 10 – 13: Slab Design Results (Continued)

spSlab Design Results: Frame D.4 Top (Floors 10 - 13)

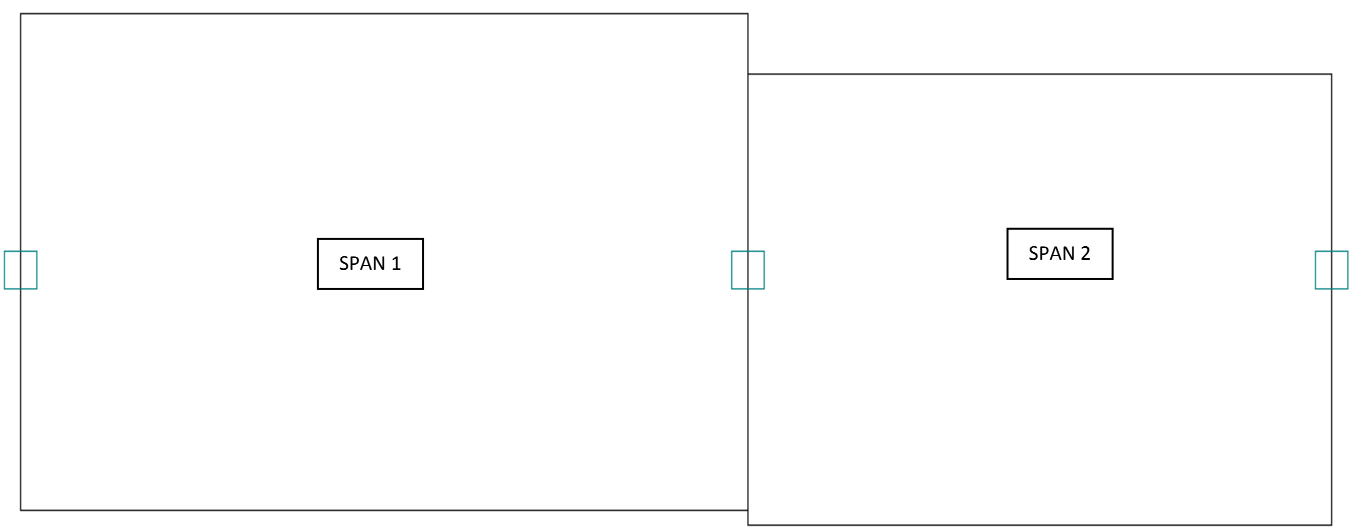
Top Reinforcement

Span	Strip	Left Side				Right Side				Continuous	
		Bars	Length	Bars	Length	Bars	Length	Bars	Length	Bars	Length
1	Column	5 - #5	9.45	2 - #5	5.98	10 - #5	9.45	9 - #5	5.98		
	Middle	7 - #5	6.51			7 - #5	8.18				
2	Column	10 - #5	9.2	9 - #5	4.88	5 - #5	7.64	1 - #5	4.88		
	Middle	7 - #5	9.2			6 - #5	5.3				

Bottom Reinforcement

Span	Strip	Long Bars			Short Bars		
		Bars	Start	Length	Bars	Start	Length
1	Column	11 - #5	0	28			
	Middle	5 - #5	0	28	2 - #5	4.2	19.6
2	Column	6 - #5	0	22.5			
	Middle	5 - #5	0	22.5	1 - #5	3.38	15.75

F -9



8” Flat Plate

FLOORS 10 – 13: Slab Design Results (Continued)

spSlab Design Results: Frame F.9 (Floors 10 - 13)

Top Reinforcement

Span	Strip	Left Side				Right Side			
		Bars	Length	Bars	Length	Bars	Length	Bars	Length
1	Column	2 - #5	7.9	1 - #5	5.04	2 - #5	7.9	2 - #5	5.04
	Middle	3 - #5	5.48			3 - #5	7.7		
2	Column	2 - #5	7.9	2 - #5	5.04	2 - #5	7.9	1 - #5	5.04
	Middle	3 - #5	7.7			3 - #5	5.48		

Bottom Reinforcement

Span	Strip	Long Bars			Waffle			Total Strip Width	Maximum Possible Number of Ribs Per Strip (20" clear spacing)	
		Bars	Start	Length	Ribs	Bars / Rib	As / Rib			
1	Column	2 - #5	0	23	2	1 - #5	0.31			
	Middle	2 - #5	0	23	1	2 - #5	0.62			
2	Column	2 - #5	0	23	2	1 - #5	0.31			
	Middle	2 - #5	0	23	1	2 - #5	0.62			

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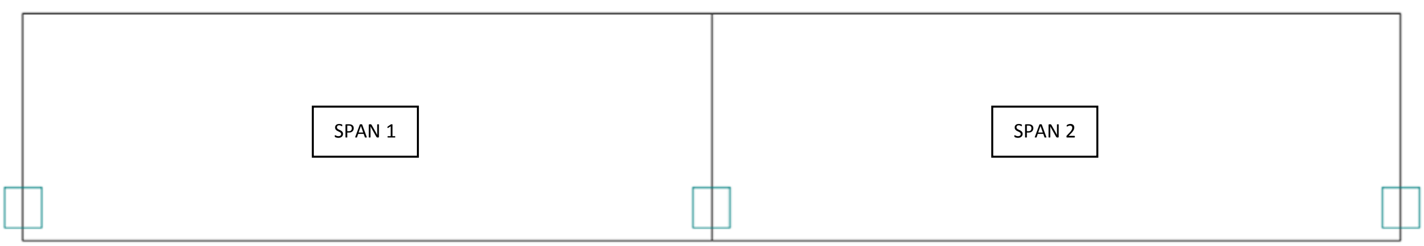
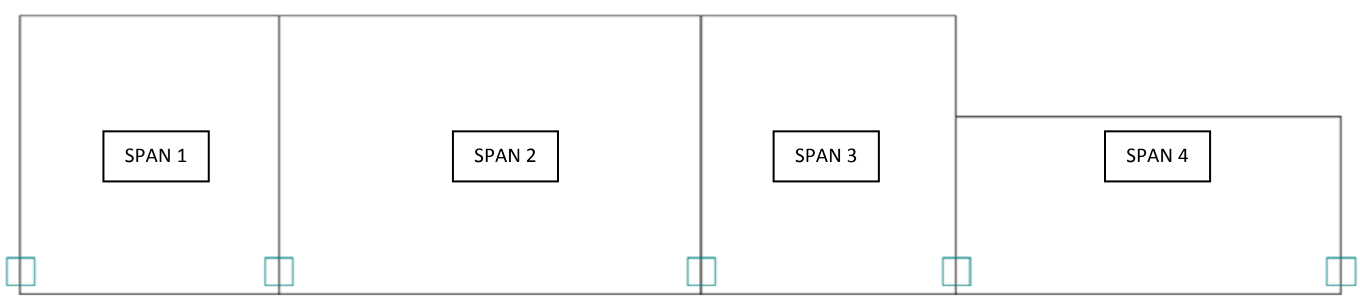


Note: This Frame is Typical for Floors 10 – 13, and 3 – 9

FLOORS 10 – 13: Slab Design Results (Continued)

spSlab Design Results: Frame 1 (Floors 10 - 13)											
Top Reinforcement											
Span	**Strip**	**Left Side**				**Right Side**				Continuous	
		Bars	**Length**	**Bars**	**Length**	**Bars**	**Length**	**Bars**	**Length**	**Bars**	**Length**
1	Column									3 - #5	11.67
	Middle									6 - #5	11.67
2	Column	2 - #5	6.48	1 - #5	4.18	2 - #5	6.48	1 - #5	4.18		
	Middle	6 - #5	5.19			6 - #5	4.69				
3	Column									3 - #5	11.5
	Middle									6 - #5	11.5
4	Column	2 - #5	5.93			2 - #5	5.93	2 - #5	3.84		
	Middle	6 - #5	5.09			3 - #5	4.16				
Bottom Reinforcement											
Span	**Strip**	**Long Bars**			**Waffle**			Total Strip Width	Maximum Possible Number of Ribs Per Strip (20" clear spacing)		
		Bars	**Start**	**Length**	**Ribs**	**Bars / Rib**	**As / Rib**				
1	Column	2 - #5	0	11.67	1	2 - #5	0.62				
	Middle	5 - #5	0	11.67	5	1 - #5	0.31				
2	Column	4 - #5	0	19	2	2 - #5	0.62				
	Middle	4 - #5	0	19	4	1 - #5	0.31				
3	Column	2 - #5	0	11.5	1	2 - #5	0.62				
	Middle	5 -- #5	0	11.5	5	1- #5	0.31				
4	Column	2 - #5	0	17.33	2	1 - #5	0.31				
	Middle	2 - #5	0	17.33	2	1 - #5	0.31				

F -11



Note: This Frame is Typical for
Floors 10 – 13, and 3 – 9

FLOORS 10 – 13: Slab Design Results (Continued)

spSlab Design Results: Frame A.2 (Floors 10 - 13)

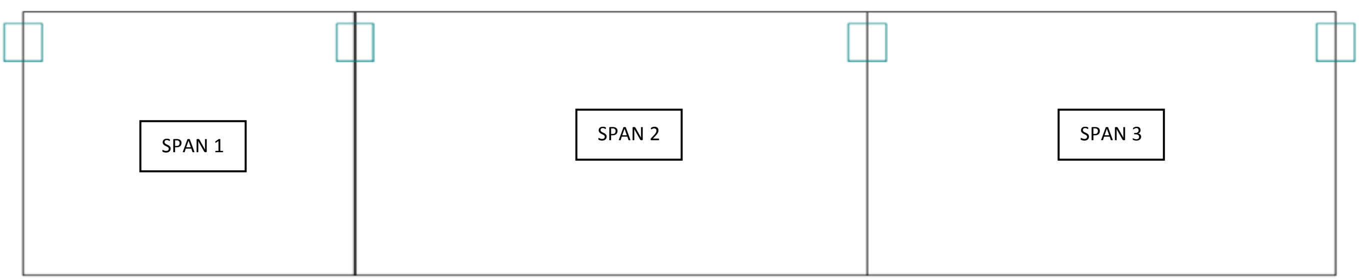
Top Reinforcement

Span	Strip	Left Side				Right Side				Continuous	
		Bars	Length	Bars	Length	Bars	Length	Bars	Length	Bars	Length
1	Column									3 - #5	11
	Middle									4 - #5	11
2	Column	2 - #5	5.82	1 - #5	3.78	3 - #5	5.82	1 - #5	3.78		
	Middle	4 - #5	4.61			3 - #5	5.61				
3	Column	3 - #5	5.33	1 - #5	3.48	3 - #5	5.33	1 - #5	3.48		
	Middle	3 - #5	5.27								

Bottom Reinforcement

Span	Strip	Long Bars			Waffle			Total Strip Width	Maximum Possible Number of Ribs Per Strip (20" clear spacing)	
		Bars	Start	Length	Ribs	Bars / Rib	As / Rib			
1	Column	2 - #5	0	11	1	2 - #5	0.62			
	Middle	3 - #5	0	11	3	1 - #5	0.31			
2	Column	4 - #5	0	17	2	2 - #5	0.62			
	Middle	2 - #5	0	17	2	1 - #5	0.31			
3	Column	4 - #5	0	15.5	2	2 - #5	0.62			
	Middle	2 - #5	0	15.5	2	1 - #5	0.31			

F -12



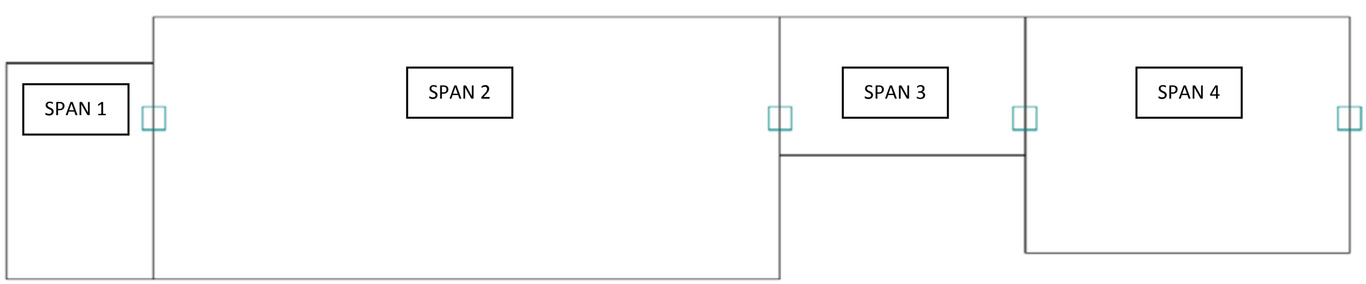
Note: This Frame is Typical for Floors 10 – 13, and 3 – 9

FLOORS 10 – 13: Slab Design Results (Continued)

spSlab Design Results: Frame 10 (Floors 10 - 13)

Top Reinforcement											
Span	Strip	Left Side				Right Side				Continuous	
		Bars	Length	Bars	Length	Bars	Length	Bars	Length	Bars	Length
1	Column					10 - #5	3.06	10 - #5	2.1	5 - #5	8
	Middle					2 - #5	2.25			5 - #5	8
2	Column	13 - #5	11.43	12 - #5	7.18	10 - #5	11.43	10 - #5	7.18		
	Middle	7 - #%	7.83			8 - #5	8.57				
3	Column	9 - #5	5.16	8 - #5	3.04					3 - #5	13.3
	Middle	5 - #5	3.28			4 - #5	3.28			3 - #5	13.3
4	Column	2 - #5	6.02	1 - #5	3.9	3 - #5	6.02	2 - #5	3.9		
	Middle	7 - #5	4.22			5 - #5	4.22				

Bottom Reinforcement										
Span	Strip	Long Bars			Waffle			Short Bars		
		Bars	Start	Length	Ribs	Bars / Rib	As / Rib	Bars	Start	Length
1	Column									
	Middle									
2	Column	11 - #5	0	34						
	Middle	4 - #5	0	34				4 - #5	0	28.9
3	Column	3 - #5	0	13.3						
	Middle	3 - #5	0	13.3						
4	Column	3 - #5	0	17.6	3	1 - #5	0.31			
	Middle	3 - #5	0	17.6	3	1 - #5	0.31			



F -13

Span 1 and 4 feature the basic waffle slab dimension shown on page 18, and spans 2 and 3 are designed as a 8” thick flat plate.

FLOORS 10 – 13: Slab Design Results (Continued)

spSlab Design Results: Frame C (Floors 10 - 13)

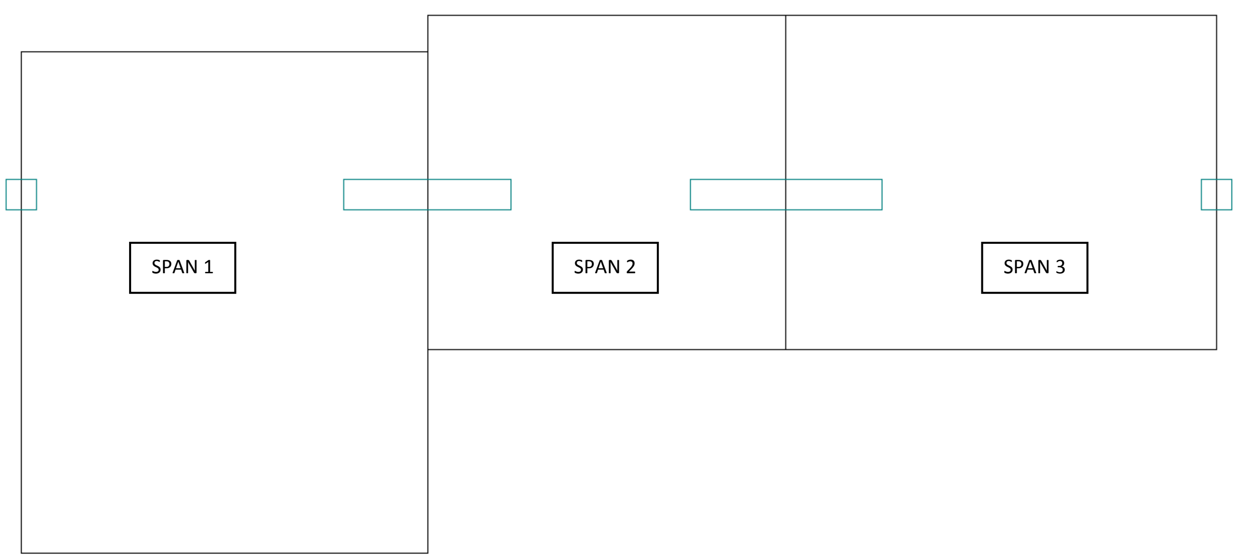
Top Reinforcement

Span	Strip	Left Side				Right Side				Continuous	
		Bars	Length	Bars	Length	Bars	Length	Bars	Length	Bars	Length
1	Column	4 - #5	4.87	1 - #5	3.2	4 - #5	7.75	1 - #5	6.08		
	Middle					10 - #5	6.33				
2	Column	4 - #5	5.98	1 - #5	5	4 - #5	6.48	1 - #5	5.5		
	Middle	10 - #5	5.31			5 - #5	5.65				
3	Column	4 - #5	8.41	1 - #5	6.68	4 - #5	5.04	1 - #5	3.3		
	Middle	5 - #5	6.94			5 - #5	3.57				

Bottom Reinforcement

Span	Strip	Long Bars			Waffle			Total Strip Width	Maximum Possible Number of Ribs Per Strip (20" clear spacing)	
		Bars	Start	Length	Ribs	Bars / Rib	As / Rib			
1	Column	6 - #5	0	17	3	2 - #5	0.62			
	Middle	7 - #5	0	17	7	1 - #5	0.31			
2	Column	6 - #5	0	15	3	2 - #5	0.62			
	Middle	4 - #5	0	15	4	1 - #5	0.31			
3	Column	6 - #5	0	18	3	2 - #5	0.62			
	Middle	4 - #5	0	18	4	1 - #5	0.31			

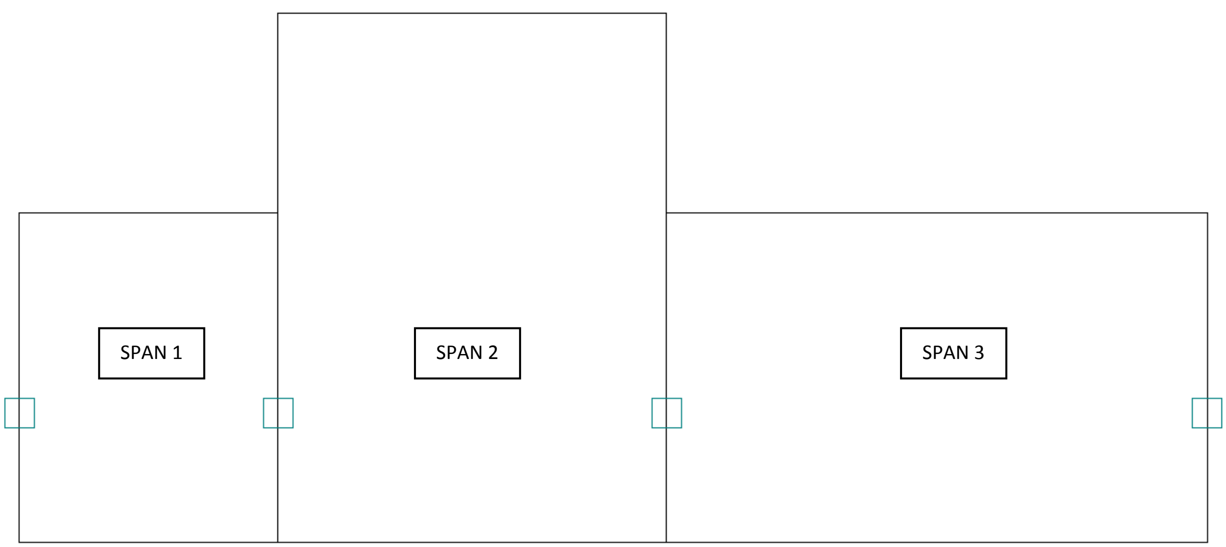
F -14



FLOORS 10 – 13: Slab Design Results (Continued)

spSlab Design Results: Frame F (Floors 10 - 13)											
Top Reinforcement											
Span	**Strip**	**Left Side**				**Right Side**				Continuous	
		Bars	**Length**	**Bars**	**Length**	**Bars**	**Length**	**Bars**	**Length**	**Bars**	**Length**
1	Column									4 - #5	11
	Middle					6 - #5	2.77			6 - #5	11
2	Column	3 - #5	5.66	1 - #5	3.68	4 - #5	6.58	3 - #5	3.68		
	Middle	12 - #5	3.98			11 - #5	6.58				
3	Column	4 - #5	7.8	3 - #5	4.98	4 - #5	7.8	1 - #5	4.98		
	Middle	11 - #5	7.23			5 - #5	5.41				
Bottom Reinforcement											
Span	**Strip**	**Long Bars**			**Waffle**			Total Strip Width	Maximum Possible Number of Ribs Per Strip (20" clear spacing)		
		Bars	**Start**	**Length**	**Ribs**	**Bars / Rib**	**As / Rib**				
1	Column	4 - #5	0	11	2	2 - #5	0.62				
	Middle	5 - #5	0	11	5	1 - #5	0.31				
2	Column	6 - #5	0	16.5	3	2 - #5	0.62				
	Middle	8 - #5	0	16.5	8	1 - #5	0.31				
3	Column	6 - #5	0	23	3	2 -#5	0.62				
	Middle	4 - #5	0	23	4	1 - #5	0.31				

F -15



FLOORS 10 – 13: Slab Design Results (Continued)

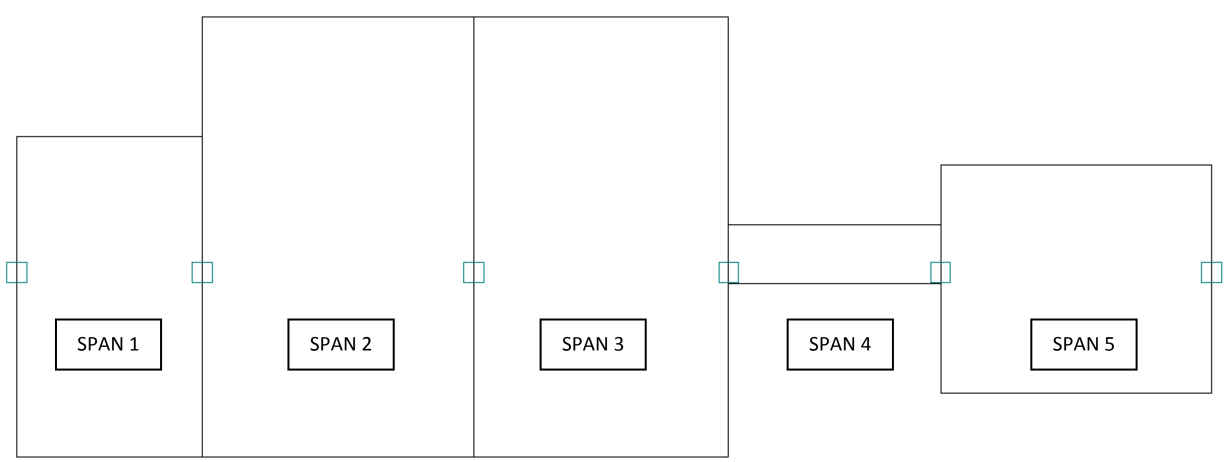
spSlab Design Results: Frame 4 (Floors 10 - 13)

Top Reinforcement

Span	Strip	Left Side				Right Side				Continuous	
		Bars	Length	Bars	Length	Bars	Length	Bars	Length	Bars	Length
1	Column									5 - #5	11.67
	Middle					5 - #5	2.92			10 - #5	11.67
2	Column	3 - #5	5.82	2 - #5	3.78	4 - #5	5.82	3 - #5	3.78		
	Middle	15 - #5	4.32			14 - #5	5.79				
3	Column	4 - #5	5.55	3 - #5	3.58	1 - #5	5.49	1 - #5	3.58		
	Middle	14-#5	5.55			18 - #5	3.87				
4	Column					1 - #5	4.6			2 - #5	13.3
	Middle	16-#5	3.28			7 - #5	3.28			2 - #5	13.3
5	Column	2-#5	5.82	1 - #5	3.78	4 - #5	5.82	1 - #5	3.78		
	Middle	9-#5	4.09			5 - #5	4.09				

Bottom Reinforcement

Span	Strip	Long Bars			Waffle			Total Strip Width	Maximum Possible Number of Ribs Per Strip (16" clear spacing)		
		Bars	Start	Length	Ribs	Bars / Rib	As / Rib				
1	Column	3 - #5	0	11.67	3	1 - #5	0.31				
	Middle	9 - #5	0	11.67	9	1 - #5	0.31				
2	Column	5 - #5	0	17	5	1 - #5	0.31				
	Middle	11 - #5	0	17	11	1 - #5	0.31				
3	Column	4 - #5	0	16	4	1 - #5	0.31				
	Middle	12 - #5	0	16	12	1 - #5	0.31				
4	Column	2 - #5	0	13.3	1	2 - #5	0.62				
	Middle	2 - #5	0	13.3	1	2 - #5	0.62				
5	Column	4 - #5	0	17	4	1 - #5	0.31				
	Middle	4 - #5	0	17	4	1 - #5	0.31				



Base Waffle Slab Design, but 16” Spaced Ribs Instead of 20”

F -16

Floor 10 – 13: Slab Design Summary

Overview: A typical floor plan was designed for residential floors 10 – 13. Due to architectural restraints, the column 6-D.4 found at lower levels, stops at floor 9 resulting in a large 34' span along frame 6 of floors 10 – 13 and wide strip widths along frame 4. As shown in figure F-20, the base waffle slab design (green), is maintained throughout the majority of the floor plan; however, the larger spans resulted in several frames requiring modified slab designs. Frame D.4, 6, and 4 required special design considerations.

Frame 6: Frame 6 at floors 10 – 13 has a large 32' span and a cantilever projecting out on the left exterior side. The basic waffle slab design used throughout the same floor cannot generate sufficient slab shear capacity or negative moment resistance. As a result, Span 1 and 4 feature the basic waffle slab dimension, and spans 2 and 3 are designed as an 8" thick flat plate.

Frame 4: The large transverse distance between adjacent frames results in wide column and middle strips at spans 2 and 3. The base waffle slab dimensions had to be modified in order to adjust for the increased flexural stresses. The above design is based on 16" clear spaced ribs as opposed to the typical 20" spacing.

Frame D.4: At floors 10 – 13, Frame D.4 consists of two larger spans that generate shear and flexural stresses that exceed the capacity of any practical design solution for the basic waffle slab used throughout the building. Instead of employing yet another waffle slab with modified dimensions, an 8" flat plate was designed, which is the preferred option from a constructability standpoint. As seen in figure F-18, coordination and construction of waffle slab formwork can be very tedious, and so maintaining uniform void dimensions is essential for simplifying the construction process. A third design option for frame D.4 was to simply remove the voids used in the base waffle slab design, which would result in a 11 ½ "thick solid slab. Although this simplified design solution would greatly minimize constructability and coordination issues, calculations (see figure F-17) revealed that such an overdesign would result in a significant amount of unnecessary dead weight. Please see figures F-21 through F-24 for section views of the final slab designs.

Floor 10 – 13: Slab Design Summary Continued:

WEIGHT COMPARISON STUDY – Slab Design Options For Frame D.4

Floor Design Investigation - Weight Comparison				
	8" Flat Plate w/ Transverse Ribs	vs.	Solid 11.5" Slab (no voids)	Weight Reduction
Weight / Floor	63,913 lbs		90,988 lbs	**27,075 lbs (27.1 kips)**
Total Weight / 4 Typical Floors	255,652 lbs		363,952 lbs	**108,300 lbs (108 kips)**

F -17

F -18

As shown on the final slab design figure on page 33, frame D.4 requires an 8” flat plate with transverse ribs running across it. Before selecting this as the final slab design, a comparison study of weight and constructability ramifications was conducted. From a constructability standpoint, simply employing an 11.5” flat plate would be an optimum, since it would only require omitting the use of the waffle void dome (shown in figure F-18). However, this design would also be a major overdesign from a structural standpoint. As is constantly mentioned, maintaining a low overall building weight is essential. As a result, the difference in weight between the 11.5” flat plate and the 8” flat plate with transverse ribs was calculated (Figure F-17)). Since this slab design is typical for frame D.4 on four floors, the difference in weight was significant. As shown, the 11.5” flat plate design would accumulate to nearly 108 additional kips. In conclusion, it was determined that reduction in weight outweighed the constructability issues.

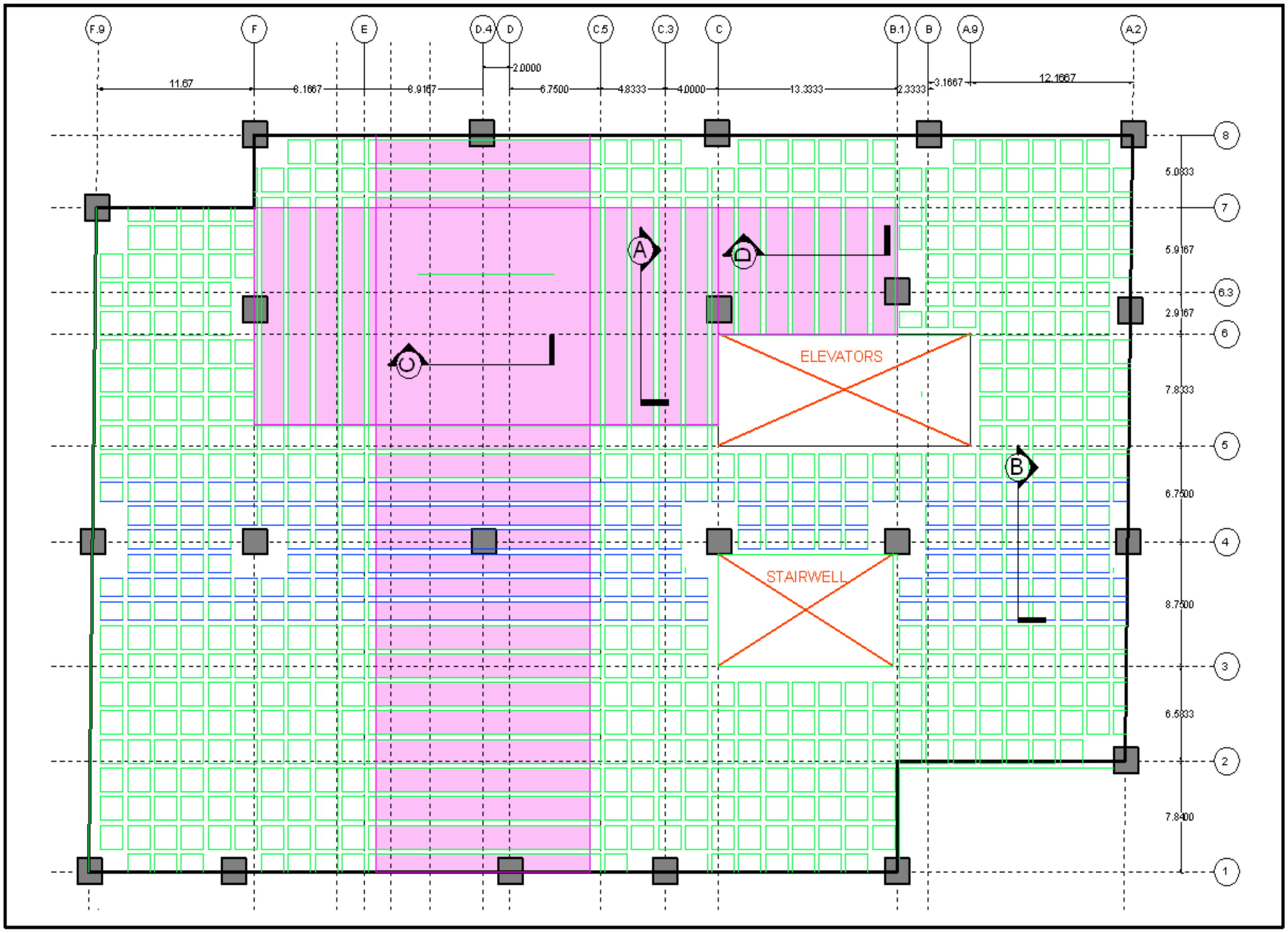
Floor 10 – 13: Slab Design Summary Continued:

The 8" flat plate design satisfies all flexural and shear stress requirements for frame D.4; however, the punching shear at column support 1 and 2 (see figure F-19) exceeds the allowable values of 173.6 kips. However, a thickened slab at the columns satisfied the punching shear issue.

Punching Shear Around Columns - Without Shear Reinforcment			
Support	Vu (kips)	ϕVc (kips)	Capacity Check
1	265.3	173.6	Exceeded
2	129.8	173.6	Exceeded
3	134.7	173.6	Okay

F -19

Plan View: Final Waffle Slab / Flat Plate Layout (Floors 10 – 13)



Base Waffle Slab: 3 ½” Slab, 4” x 8” ribs @ 20” clear space

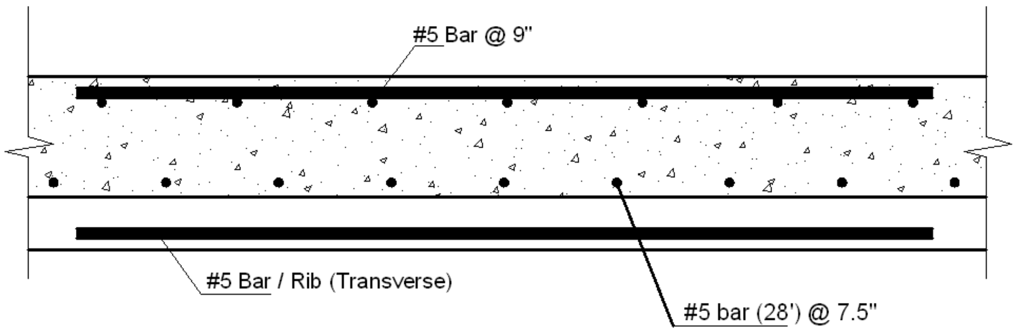
Modified Waffle Slab: 3 ½” Slab, 4” x 8” ribs @ 16” clear space

8” Flat Plate

F -20

Note: In some locations, ribs and an 8” flat plate are both present. See figures Figure F-24 (page 33) for clarification.

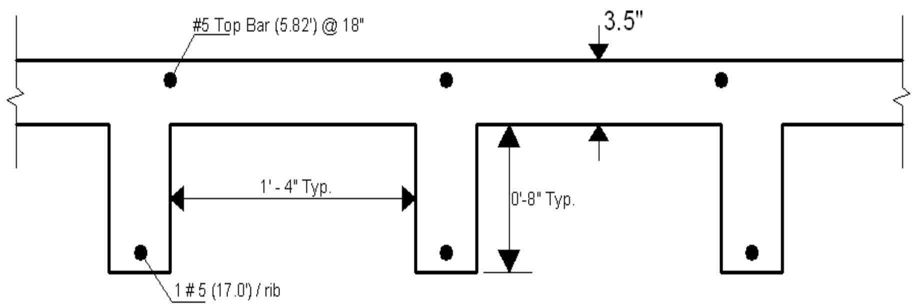
Slab Details (Correspond To Section Cuts On Figure F-20)



SECTION A - Modified Waffle Slab Detail

Note: Ribs Project 3.5" below bottom of 8" Slab.
Refer To Section D For A Side View Of This Slab Detail

F -21

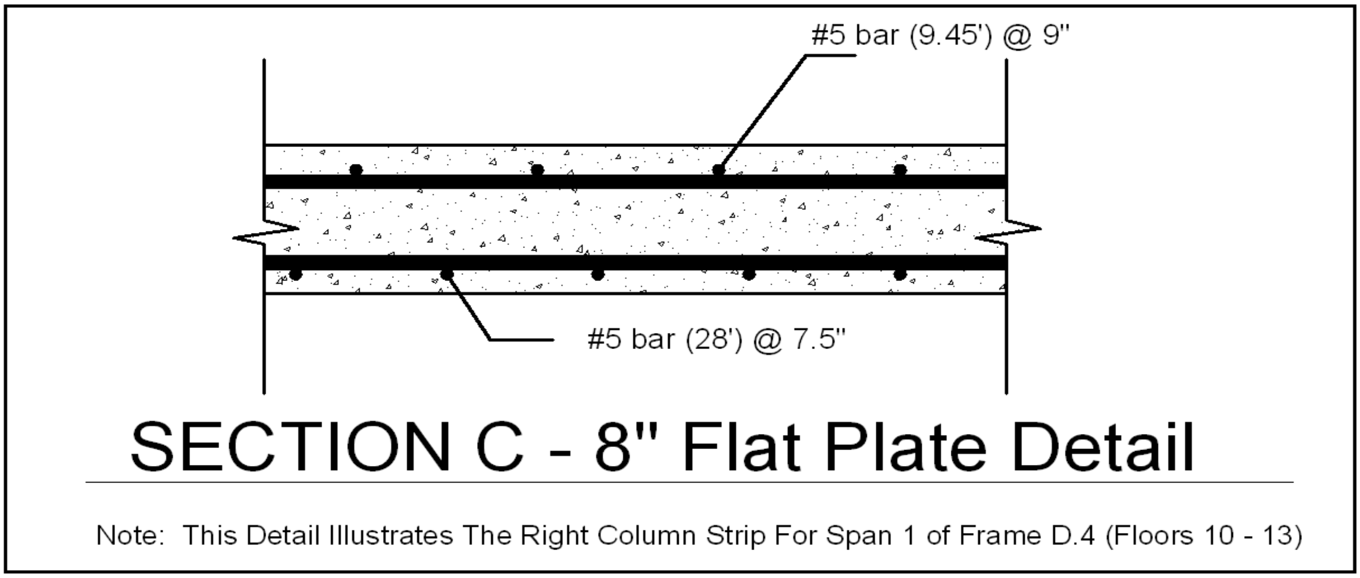


SECTION B - 16" Space Rib Waffle Slab Detail

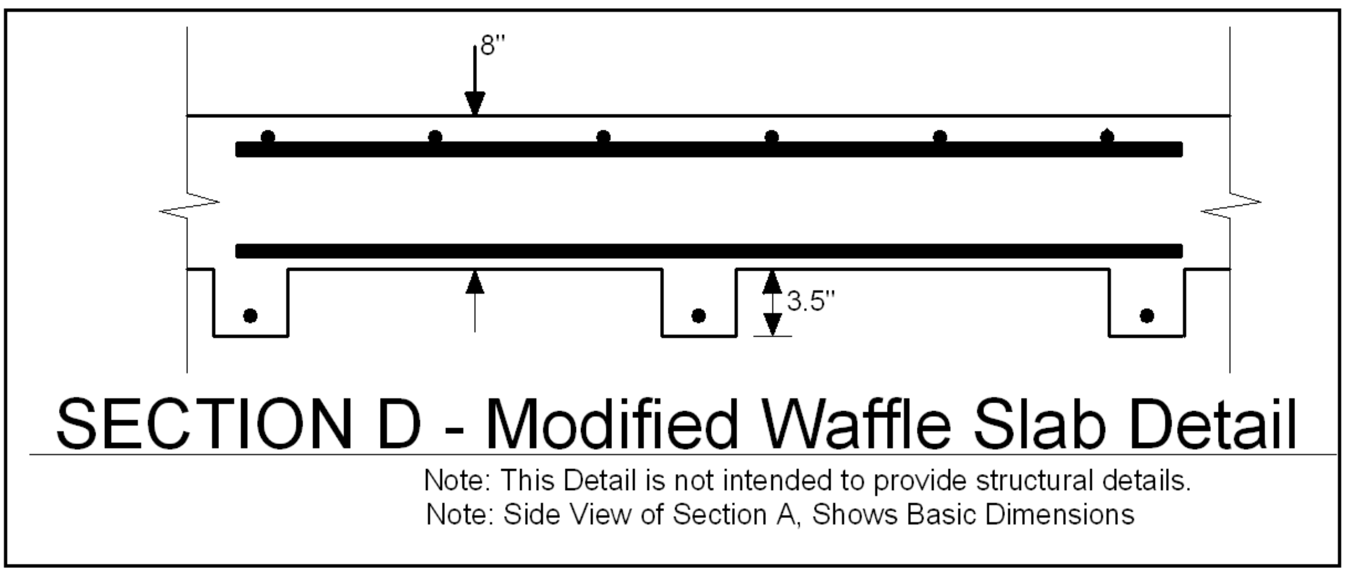
Note: This details the right column strip of Frame 4 - Span 5 (Floors 10 - 13)
Detail Only Shows Longitudinal Reinforcement

F -22

Slab Details (Correspond To Section Cuts On Figure ##)



F -23



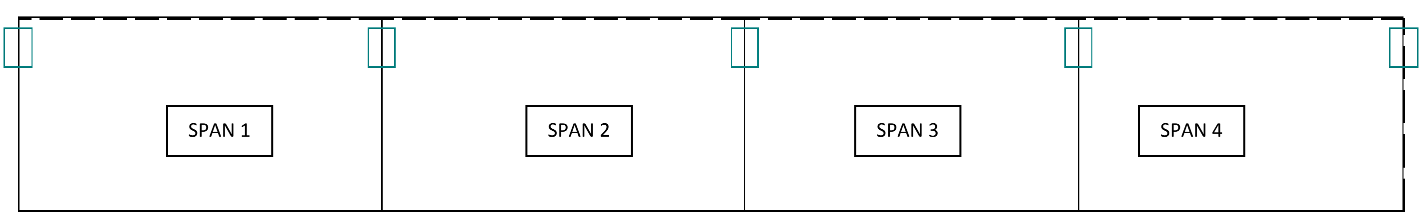
F -24

FLOORS 3 – 9: Slab Design Results

spSlab Design Results: Frame 8 (Floors 3 - 9)

Top Reinforcement									
Span	Strip	Left Side				Right Side			
		Bars	Length	Bars	Length	Bars	Length	Bars	Length
1	Column	2 - #5	5.82	1 - #5	3.78	2 - #5	5.82	1 - #5	3.78
	Middle	2 - #5	4.09			2 - #5	5.61		
2	Column	2 - #5	5.82	1 - #5	3.78	2 - #5	5.82	1 - #5	3.78
	Middle	2 - #5	5.61			2 - #5	4.86		
3	Column	2 - #5	5.36	1 - #5	3.5	2 - #5	5.36	1 - #5	3.5
	Middle	2 - #5	4.99			2 - #5	5.23		
4	Column	2 - #5	5.23	1 - #5	3.42	2 - #5	5.23	1 - #5	3.42
	Middle	2 - #5	4.98			2 - #5	3.69		
Bottom Reinforcement									
Span	Strip	Long Bars			Waffle				
		Bars	Start	Length	Ribs	Bars / Rib	As / Rib		
1	Column	2 - #5	0	17	1	2 - #5	0.62		
	Middle	2 - #5	0	17	2	1 - #5	0.31		
2	Column	2 - #5	0	17	1	2 - #5	0.62		
	Middle	2 - #5	0	17	2	1 - #5	0.31		
3	Column	2 - #5	0	15.6	1	2 - #5	0.62		
	Middle	2 - #5	0	15.6	2	1 - #5	0.31		
4	Column	2 - #5	0	15.2	1	2 - #5	0.62		
	Middle	2 - #5	0	15.2	2	1 - #5	0.31		

F -25



FLOORS 3 – 9: Slab Design Results (Continued)

spSlab Design Results: Frame F (Floors 3-9)

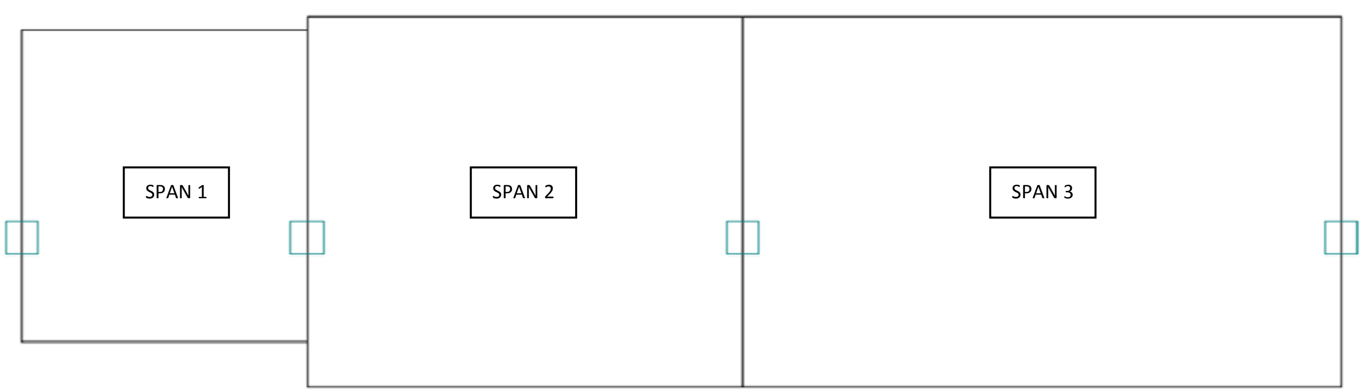
Top Reinforcement

Span	Strip	Left Side				Right Side				Continuous	
		Bars	Length	Bars	Length	Bars	Length	Bars	Length	Bars	Length
1	Column					1 - #5	3.84			4 - #5	11
	Middle					2 - #5	2.77			5 - #5	11
2	Column					2 - #5	5.74			5 - #5	16.75
	Middle	2 - #5	4.04							5 - #5	16.75
3	Column	4 - #5	7.8	3 - #5	4.98	4 - #5	7.8				
	Middle	5 - #5	6.73			5 - #5	5.41				

Bottom Reinforcement

Span	Strip	Long Bars			Waffle			Total Strip Width	Maximum Possible Number of Ribs Per Strip (20" clear spacing)	
		Bars	Start	Length	Ribs	Bars / Rib	As / Rib			
1	Column	4 - #5	0	11	2	2 - #5	0.62			
	Middle	4 -#5	0	11	4	1 - #5	0.31			
2	Column	6 - #5	0	16.75	3	2 - #5	0.62			
	Middle	4 - #5	0	16.75	4	1 - #5	0.31			
3	Column	6 - #5	0	23	3	2 - #5	0.62			
	Middle	4 - #5	0	23	4	1 - #5	0.31			

F -26



FLOORS 3 – 9: Slab Design Results Continued

spSlab Design Results: Frame D.4 (Floors 3-9)

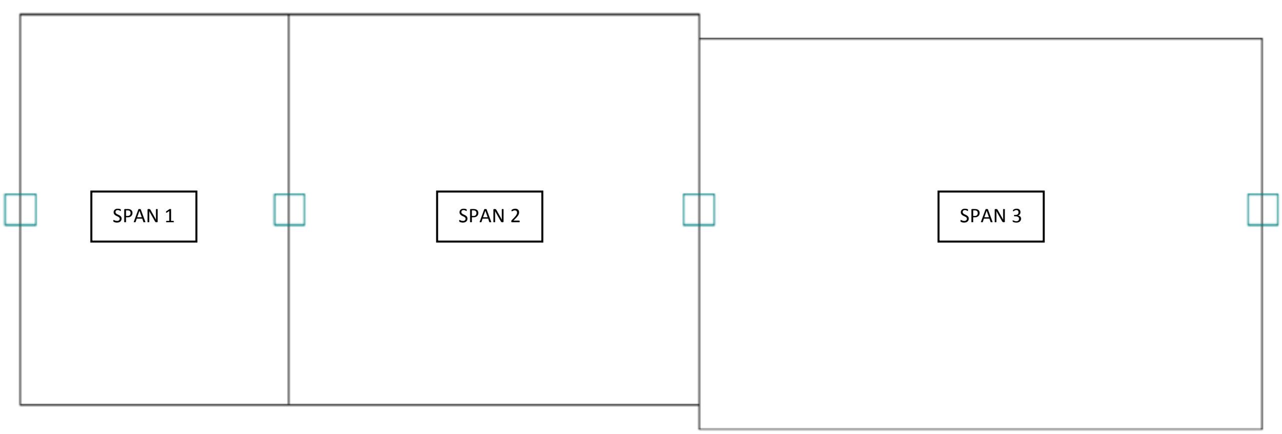
Top Reinforcement

Span	Strip	Left Side				Right Side				Continuous	
		Bars	Length	Bars	Length	Bars	Length	Bars	Length	Bars	Length
1	Column					2 - #5	3.84			4 - #5	11
	Middle									8 - #5	11
2	Column					1 - #5	5.74			6 - #5	16.75
	Middle	2 - #5	4.04							6 - #5	16.75
3	Column	4 - #5	7.8	3 - #5	4.98	4 - #5	7.8	2 - #5	4.98		
	Middle	6 - #5	6.48			6 - #5	5.41				

Bottom Reinforcement

Span	Strip	Long Bars			Waffle			Total Strip Width	Maximum Possible Number of Ribs Per Strip (20" clear spacing)	
		Bars	Start	Length	Ribs	Bars / Rib	As / Rib			
1	Column	4 - #5	0	11	2	2 - #5	0.62			
	Middle	6 - #5	0	11	6	1 - #5	0.31			
2	Column	4 - #5	0	16.75	4	1 - #5	0.31			
	Middle	4 - #5	0	16.75	4	1 - #5	0.31			
3	Column	4 - #5	0	23	4	1 - #5	0.31			
	Middle	4 - #5	0	23	4	1 - #5	0.31			

F -27



FLOORS 3 – 9: Slab Design Results Continued

spSlab Design Results: Frame 4 (Floors 3 - 9)

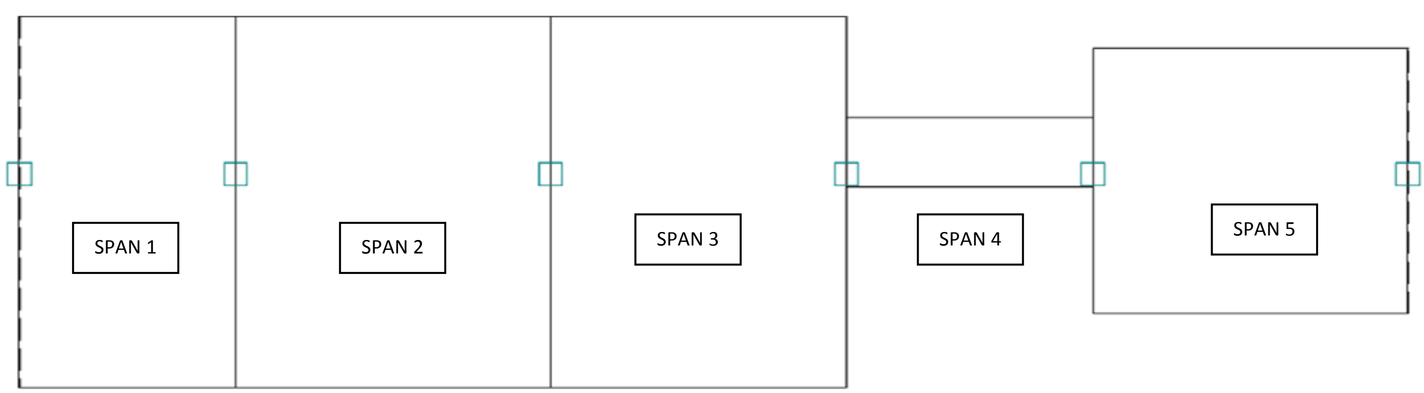
Top Reinforcement

Span	Strip	Left Side				Right Side				Continuous	
		Bars	Length	Bars	Length	Bars	Length	Bars	Length	Bars	Length
1	Column									5 - #5	11.67
	Middle									10 -#5	11.67
2	Column	3 - #5	5.82	2 - #5	3.78	4 - #5	5.82	2 - #5	3.78		
	Middle	10 - #5	4.81			9 - #5	5.55				
3	Column	4 - #5	5.55	2 - #5	3.58	1 - #5	5.49	1 - #5	3.58		
	Middle	9 - #5	5.55			13 - #5	3.87				
4	Column					1 - #5	4.6			2 - #5	13.3
	Middle	11 - #5	3.28			7 - #5	3.28			2 - #5	13.3
5	Column	2 - #5	5.82	1 - #5	3.78	4 - #5	5.82	1 - #5	3.78		
	Middle	9 - #5	4.09			5 - #5	4.09				

Bottom Reinforcement

Span	Strip	Long Bars			Waffle			Total Strip Width	Maximum Possible Number of Ribs Per Strip (16" clear spacing)		
		Bars	Start	Length	Ribs	Bars / Rib	As / Rib				
1	Column	3 - #5	0	11.67	3	1 - #5	0.31				
	Middle	9 - #5	0	11.67	9	1 - #5	0.31				
2	Column	5 - #5	0	17	5	1 - #5	0.31				
	Middle	7 - #5	0	17	7	1 - #5	0.31				
3	Column	4 - #5	0	16	4	1 - #5	0.31				
	Middle	8 - #5	0	16	8	1 - #5	0.31				
4	Column	2 - #5	0	13.3	1	2 - #5	0.62				
	Middle	2 - #5	0	13.3	1	2 - #5	0.62				
5	Column	4 - #5	0	17	4	1 - #5	0.31				
	Middle	4 - #5	0	17	4	1 - #5	0.31				

F -28



FLOORS 3 – 9: Slab Design Results Continued

spSlab Design Results: Frame C (Floors 3-9)

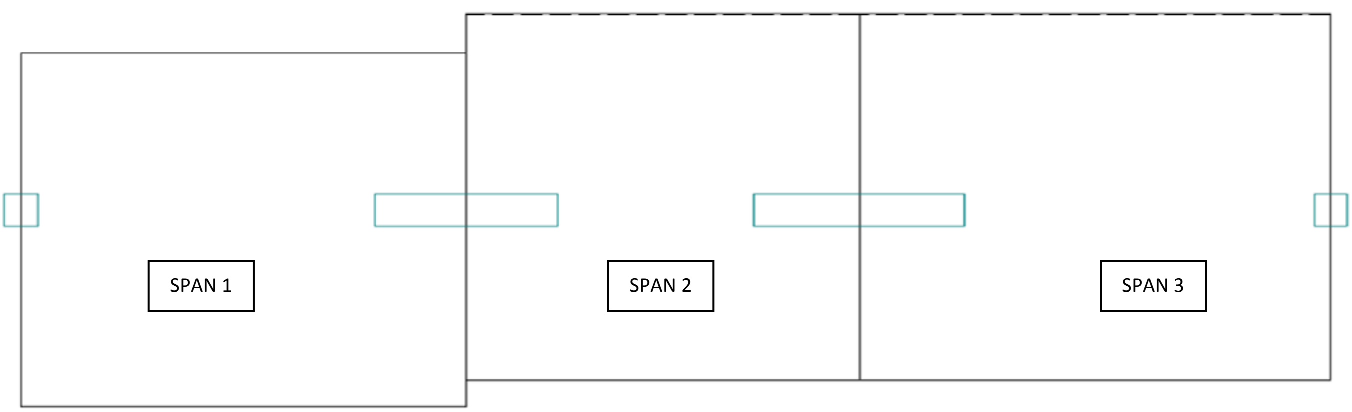
Top Reinforcement

Span	Strip	Left Side				Right Side				Continuous	
		Bars	Length	Bars	Length	Bars	Length	Bars	Length	Bars	Length
1	Column	3 - #5	4.87	2 - #5	3.2	3 - #5	7.75	2 - #5	6.08		
	Middle	5 - #5	3.46			5 - #5	6.33				
2	Column	3 - #5	5.98	2 - #5	5	4 - #5	6.48	1 - #5	5.5		
	Middle	5 - #5	5.15			5 - #5	5.65				
3	Column	4 - #5	8.41	1 - #5	3.2	4 - #5	5.04	1 - #5	3.3		
	Middle	5 - #5	6.94			5 - #5	3.57				

Bottom Reinforcement

Span	Strip	Long Bars			Waffle			Total Strip Width	Maximum Possible Number of Ribs Per Strip (20" clear spacing)	
		Bars	Start	Length	Ribs	Bars / Rib	As / Rib			
1	Column	3 - #5	0	17	3	1 - #5	0.31			
	Middle	3 -#5	0	17	3	1 - #5	0.31			
2	Column	6 - #5	0	15	3	2 -#5	0.62			
	Middle	4 - #5	0	15	4	1 - #5	0.31			
3	Column	6 - #5	0	18	3	2 - #5	0.62			
	Middle	4 -#5	0	18	4	1 - #5	0.31			

F -29



FLOORS 3 – 9: Slab Design Results Continued

spSlab Design Results: Frame 6 (Floors 3 - 9)

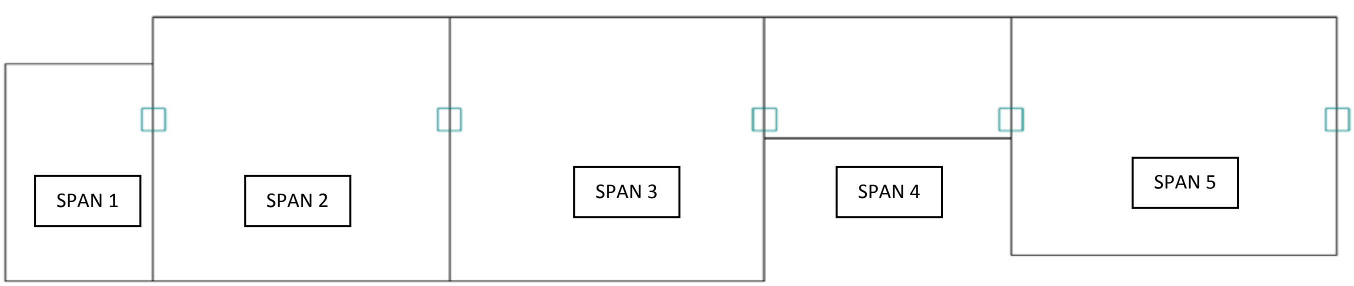
Top Reinforcement

Span	Strip	Left Side				Right Side				Continuous	
		Bars	Length	Bars	Length	Bars	Length	Bars	Length	Bars	Length
1	Column									5 - #5	8
	Middle					1 - #5	2.25			5 - #5	8
2	Column	3 - #5	5.8	2 - #5	3.58	3 - #5	5.55	2 - #5	3.58		
	Middle	6 - #5	3.87			6 - #5	5.55				
3	Column	3 - #5	5.82	2 - #5	3.78	2 - #5	5.82	1 - #5	3.78		
	Middle	6 - #5	5.11			8 - #5	4.09				
4	Column									3 - #5	13.33
	Middle	5 - #5	3.28			4 - #5	3.28			3 - #5	13.33
5	Column	2 - #5	6.02	1 - #5	3.9	3 - #5	6.02	2 - #5	3.9		
	Middle	7 - #5	4.62			5 - #5	4.22				

Bottom Reinforcement

Span	Strip	Long Bars			Waffle			Total Strip Width	Maximum Possible Number of Ribs Per Strip (20" clear spacing)		
		Bars	Start	Length	Ribs	Bars / Rib	As / Rib				
1	Column										
	Middle										
2	Column	3 - #5	0	16	3	1 - #5	0.31				
	Middle	4 - #5	0	16	4	1 - #5	0.31				
3	Column	6 - #5	0	17	3	2 - #5	0.62				
	Middle	4 - #5	0	17	4	1 - #5	0.31				
4	Column	2 - #5	0	13.33	1	2 - #5	0.62				
	Middle	2 - #5	0	13.33	2	1 - #5	0.31				
5	Column	3 - #5	0	17.6	3	1 - #5	0.31				
	Middle	3 - #5	0	17.6	3	1 - #5	0.31				

F -30

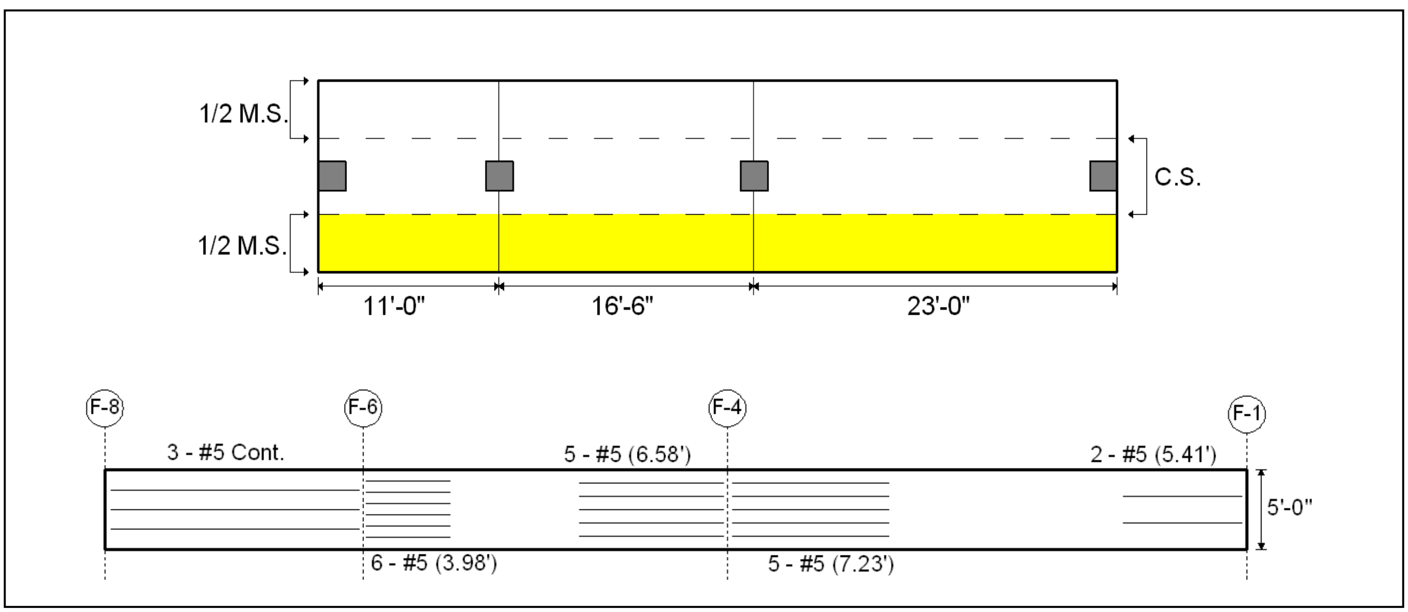


Floors 3 – 9: Slab Design Summary

Since floors 3 -9 account for over 50% of the entire structures floor system and are identical in loading and layout, both preliminary slab calculations and spSlab computer design methods started by focusing on these floors. The base waffle slab design depicted in figure F-7 on page 18, was specifically designed for this floor layout. With average bay sizes of approximately 15' x 15' and a live load of only 40 psf, the 3 ½" waffle slab with 4" x 8" ribs @ 20" clear spacing proves to be a very efficient and practical design solution. In fact, no modifications to the base waffle slab were required for this floor plan. In all frames, the required steel reinforcement was compatible with strip widths. At most, the 4" wide ribs require 2 #5 bars. As a result, procedures such as bundling and stacking of reinforcement were not necessary to satisfy minimum spacing and cover requirements. To view the final overall layout of the waffle slab at floors 3 – 9, please refer to figure F-35 on page 43.

In order to illustrate and help interpret the spSlab design results recorded in the above tables, please refer to the following figures F-31 through F-34. These figures are top bar and bottom bar reinforcement diagrams representing a typical interior frame (at floors 3 -9).

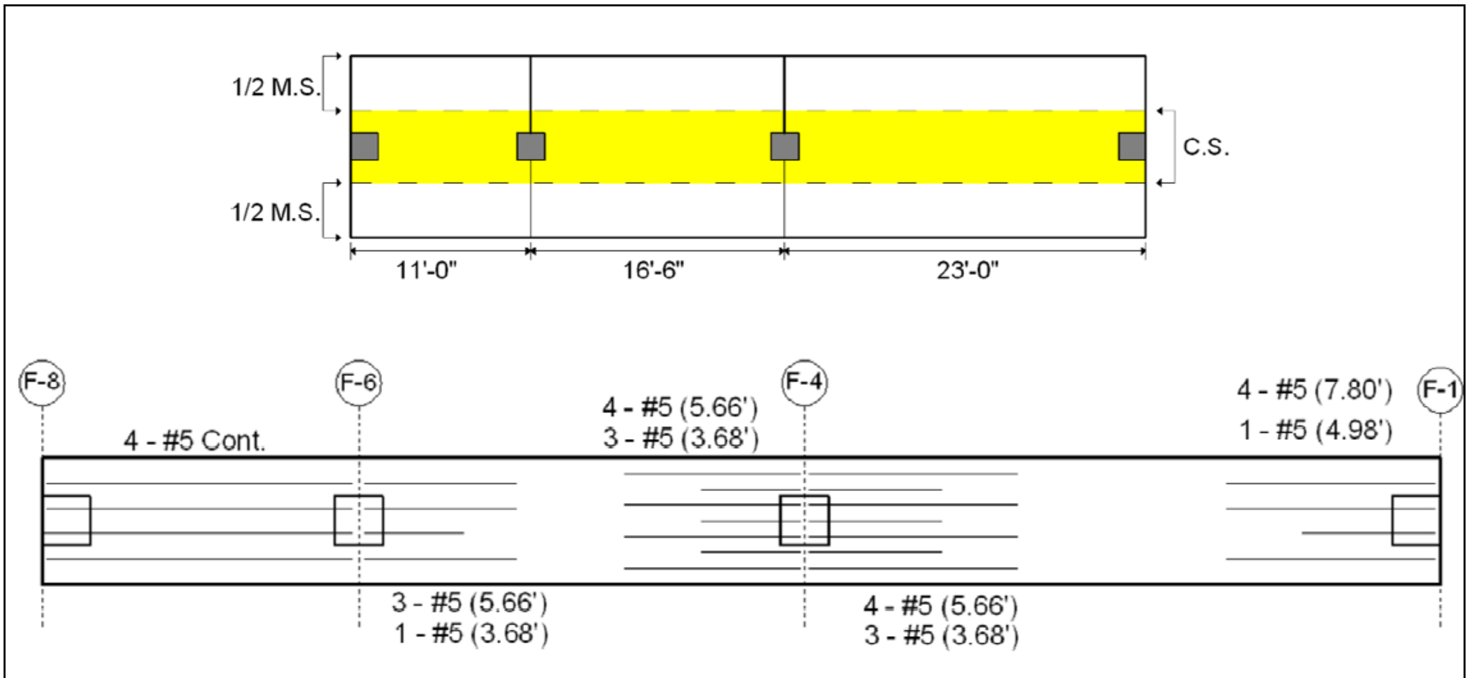
Top Bar Reinforcement –Plan View - ½ Middle Strip



F -31

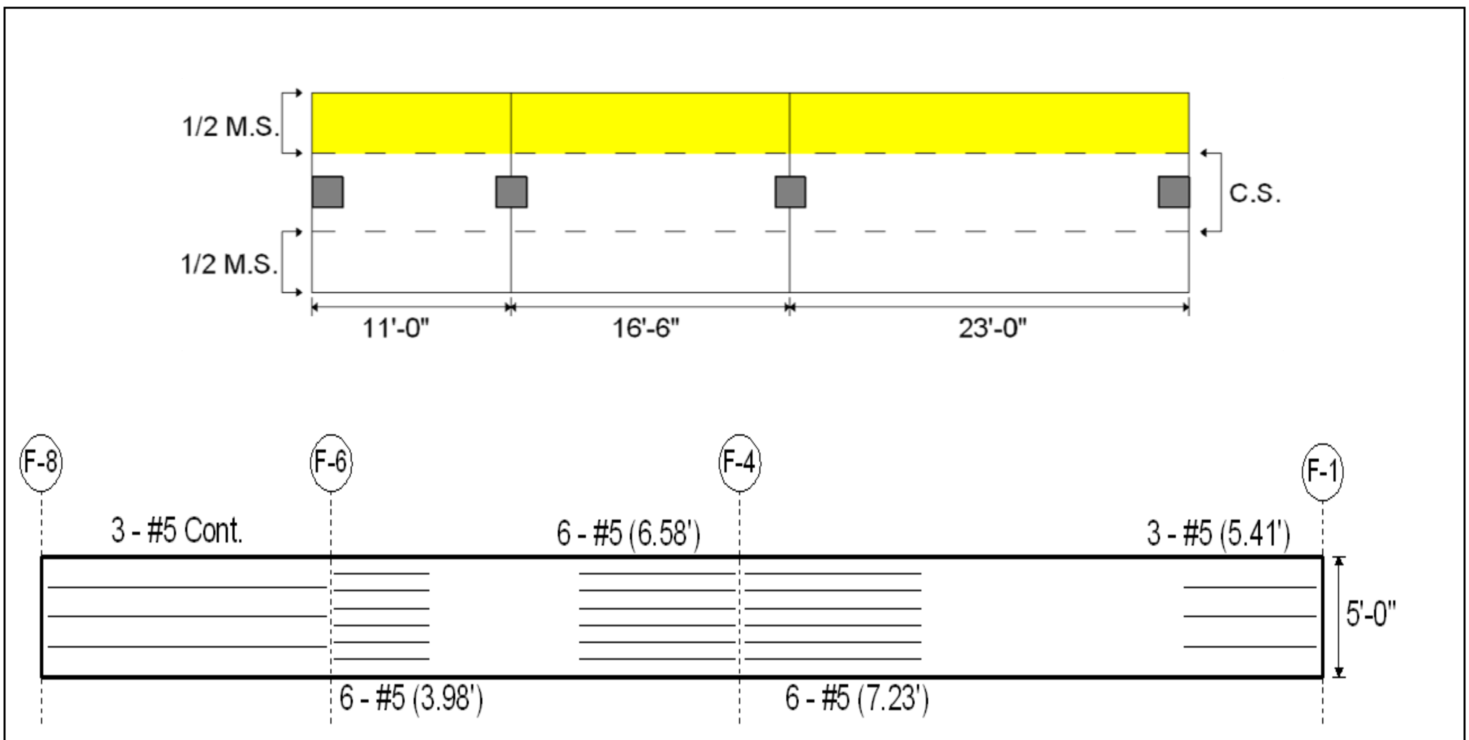
Floors 3 – 9: Slab Design Summary Continued

Top Bar Reinforcement – Plan View – Column Strip



F -32

Top Bar Reinforcement – Plan View – Other ½ Middle Strip

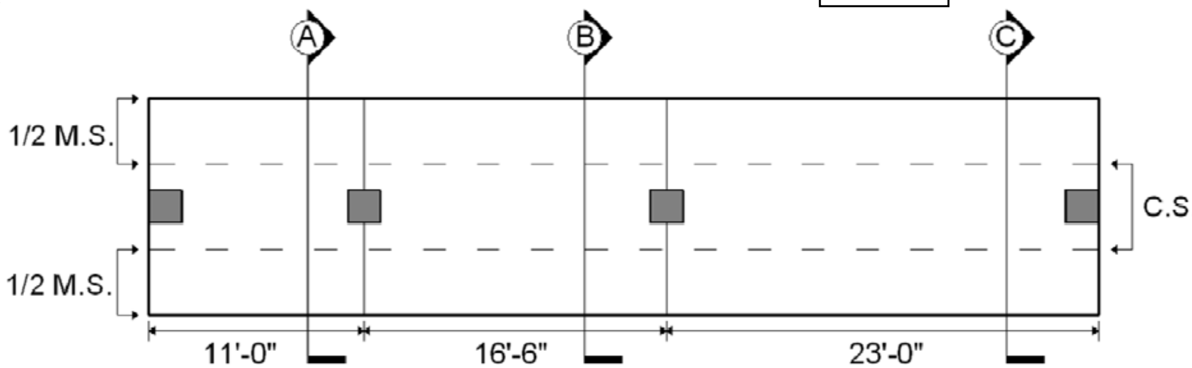


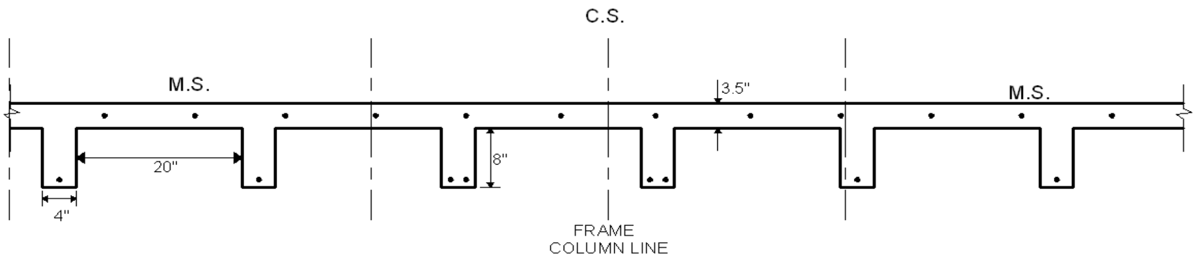
F -33

Floors 3 – 9: Slab Design Summary Continued

Bottom Reinforcement Details – Placement In Ribs – Typical Interior Frame

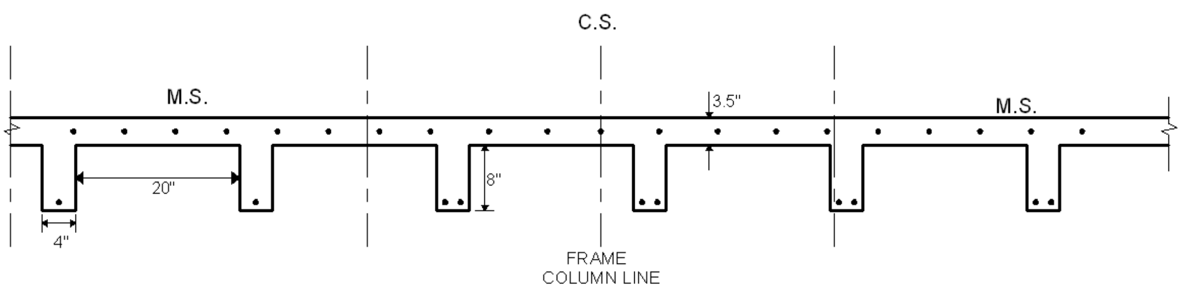
F -34





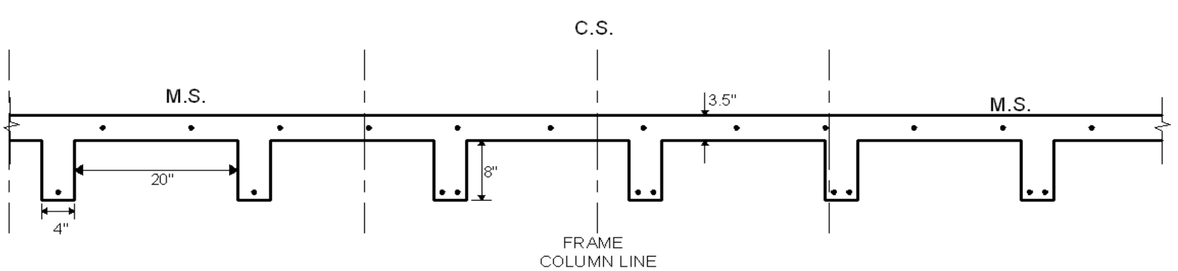
SECTION A - TYPICAL INTERIOR FRAME DETAIL

NOTE: ONLY SHOWS LONGITUDINAL REINFORCEMENT



SECTION B - TYPICAL INTERIOR FRAME DETAIL

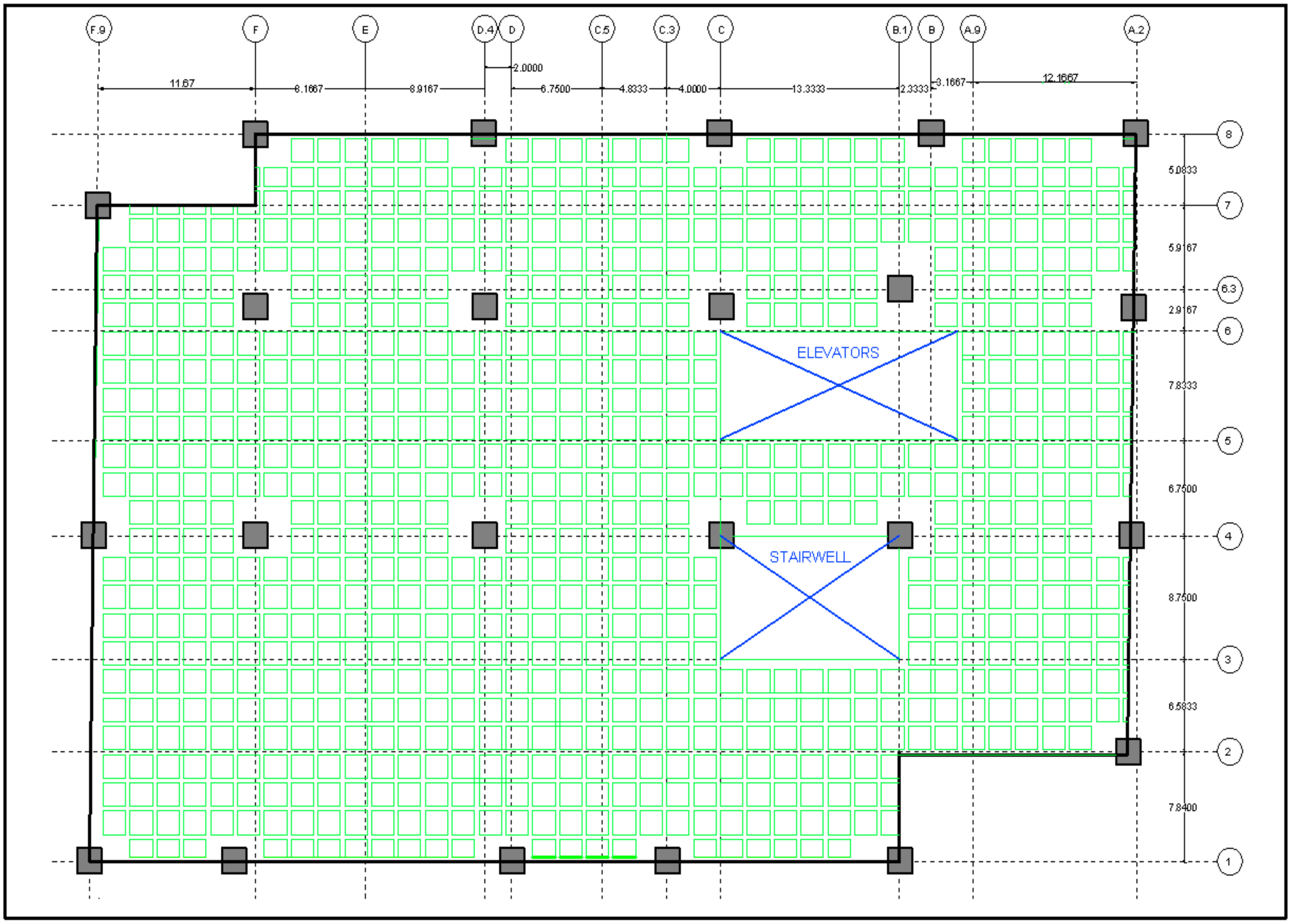
NOTE: ONLY SHOWS LONGITUDINAL REINFORCEMENT



SECTION C - TYPICAL INTERIOR FRAME DETAIL

NOTE: ONLY SHOWS LONGITUDINAL REINFORCEMENT

Plan View: Final Waffle Slab Layout (Floor 3 – 9)



F -35

Base Waffle Slab Design: 3 ½" Slab, 4" x 8" ribs @ 20" Clear Spacing

Note: Entire Floor Design Comprised of the base waffle slab design. As shown in above figure, to attain sufficient punching shear capacity the voids were left out around the columns. In other words, an 11 ½" thick slab resists punching shear at columns.

Floor 2: Slab Design Results

spSlab Design Results: Frame F.9 (Floor 2)

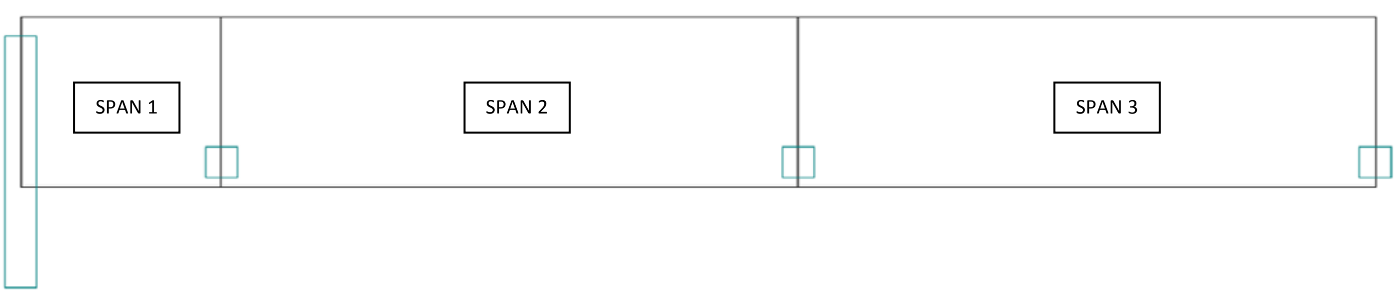
Top Reinforcement

Span	Strip	Left Side				Right Side				Continuous	
		Bars	Length	Bars	Length	Bars	Length	Bars	Length	Bars	Length
1	Column									2 - #5	8
	Middle									3 - #5	8
2	Column	2 - #5	7.8			3 - #5	7.8	1 - #5	4.98		
	Middle	3 - #5	5.98			2 - #5	7.48				
3	Column	3 - #5	7.8	1 - #5	4.98	3 - #5	7.8				
	Middle	2 - #5	7.23			2 - #5	5.41				

Bottom Reinforcement

Span	Strip	Long Bars			Waffle			Total Strip Width	Maximum Possible Number of Ribs Per Strip (20" clear spacing)	
		Bars	Start	Length	Ribs	Bars / Rib	As / Rib			
1	Column	2 - #5	0	8	1	2 - #5	0.62			
	Middle	4 - #5	0	8	2	2 - #5	0.62			
2	Column	2 - #6	0	23	1	2 - #6	0.88			
	Middle	2 - #5	0	23	2	1 - #5	0.31			
3	Column	2 - #6	0	23	1	2 - #6	0.88			
	Middle	2 - #5	0	23	2	1 - #5	0.31			

F -36



Floor 2: Slab Design Results Continued

spSlab Design Results: Frame F (Floors 2)

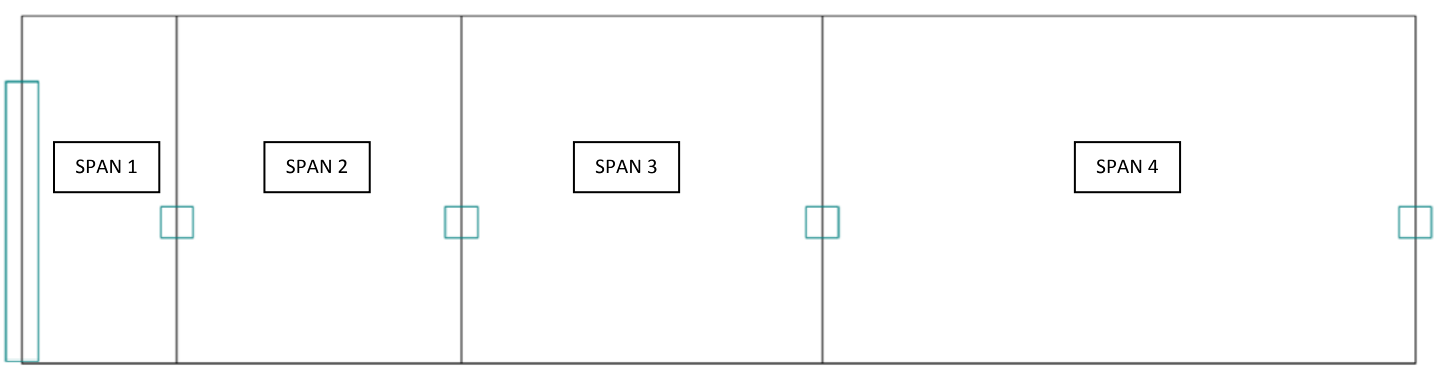
Top Reinforcement

Span	Strip	Left Side				Right Side				Continuous	
		Bars	Length	Bars	Length	Bars	Length	Bars	Length	Bars	Length
1	Column									3 - #5	6
	Middle									8 - #5	6
2	Column	2 - #5	3.84	1 - #5	2.58	3 - #5	3.93	2 - #5	2.58		
	Middle	8 - #5	3.18			6 - #5	3.93				
3	Column					1 - #5	4.83			5 - #5	14
	Middle	1 - #5	3.43							5 - #5	14
4	Column	3 - #5	7.8	3 - #5	4.98	3 - #5	7.8	2 - #5	4.98		
	Middle	5 - #5	6.48			5 - #5	5.41				

Bottom Reinforcement

Span	Strip	Long Bars			Waffle			Total Strip Width	Maximum Possible Number of Ribs Per Strip (20" clear spacing)		
		Bars	Start	Length	Ribs	Bars / Rib	As / Rib				
1	Column	2 - #5	0	6	1	2 - #5	0.62				
	Middle	5 - #5	0	6	5	1 - #5	0.31				
2	Column	4 - #5	0	11	2	2 - #5	0.62				
	Middle	4 - #5	0	11	4	1 - #5	0.31				
3	Column	3 - #5	0	14	3	1 - #5	0.31				
	Middle	6 - #5	0	14	3	2 - #5	0.62				
4	Column	6 - #5	0	23	3	2 - #5	0.62				
	Middle	3 - #5	0	23	3	1 - #5	0.31				

F -37



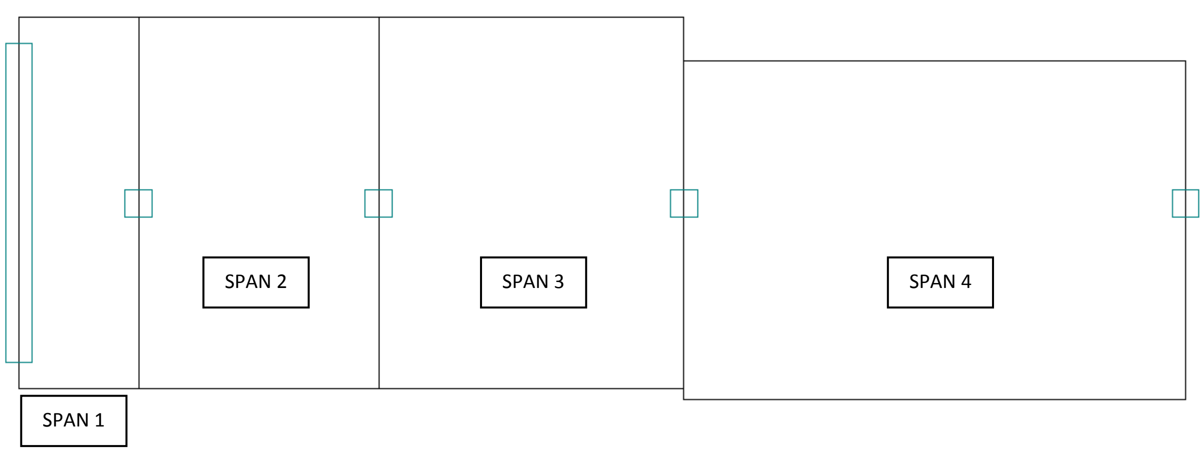
Floor 2: Slab Design Results Continued

spSlab Design Results: Frame D.4 (Floors 2)

Top Reinforcement											
Span	Strip	Left Side				Right Side				Continuous	
		Bars	Length	Bars	Length	Bars	Length	Bars	Length	Bars	Length
1	Column									2 - #5	5.5
	Middle									1- -#5	5.5
2	Column	2 - #5	3.84			3 - #5	3.93	2 - #5	2.58		
	Middle	10 - #5	3.18			8 - #5	3.93				
3	Column					2 - #5	4.83			5 0 #5	14
	Middle	1 - #5	3.43							7 - #5	14
4	Column	4 - #5	7.8	3 - #5	4.98	4 -#5	7.8	2 - #5	4.98		
	Middle	7 - #5	5.48			6 - #5	5.41				

Bottom Reinforcement										
Span	Strip	Long Bars			Waffle			Total Strip Width	Maximum Possible Number of Ribs Per Strip (20" clear spacing)		
		Bars	Start	Length	Ribs	Bars / Rib	As / Rib				
1	Column	2 - #5	0	5.5	1	2 - #5	0.62				
	Middle	7 - #5	0	5.5	7	1 - #5	0.31				
2	Column	4 - #5	0	11	2	2 - #5	0.62				
	Middle	6 - #5	0	11	6	1 - #5	0.31				
3	Column	6 - #5	0	14	3	2 - #5	0.62				
	Middle	5 - #5	0	14	5	1 - #5	0.31				
4	Column	6 - #5	0	23	3	2 - #5	0.62				
	Middle	4 - #5	0	23	4	1 - #5	0.31				

F -38



Floor 2: Slab Design Results Continued

spSlab Design Results: Frame C (Floors 2)

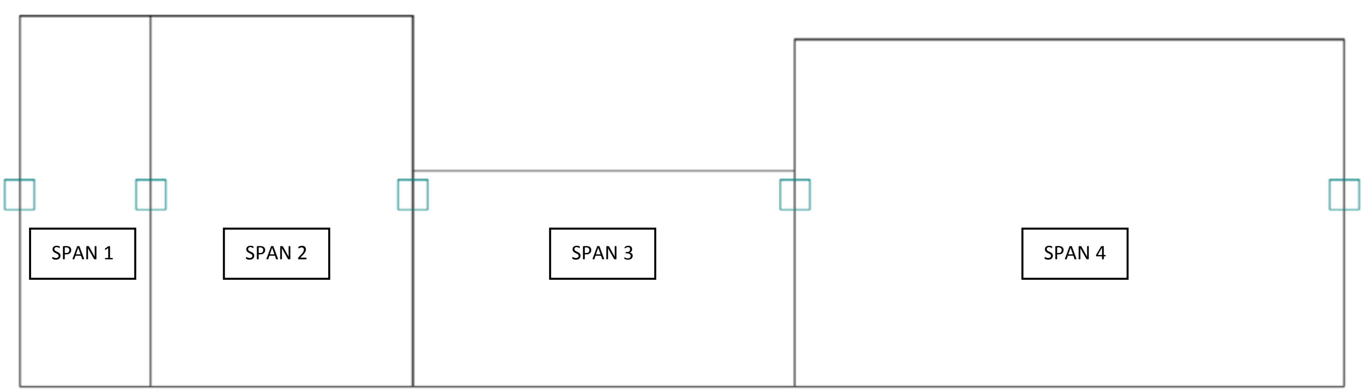
Top Reinforcement

Span	Strip	Left Side				Right Side				Continuous	
		Bars	Length	Bars	Length	Bars	Length	Bars	Length	Bars	Length
1	Column									2 - #5	5.5
	Middle									9 - #5	5.5
2	Column	2 - #5	3.84			3 - #5	3.84	1 - #5	2.58		
	Middle	9 - #5	3.18			8 - #5	3.43				
3	Column					2 - #5	5.49			4 - #5	16
	Middle	5 - #5	3.87			4 - #5	3.87			3 - #5	16
4	Column	3 - #5	7.8	3 - #5	4.98	5 - #5	7.8	2 - #5	4.98		
	Middle	7 - #5	5.98			4 - #5	5.41				

Bottom Reinforcement

Span	Strip	Long Bars			Waffle			Total Strip Width	Maximum Possible Number of Ribs Per Strip (20" clear spacing)		
		Bars	Start	Length	Ribs	Bars / Rib	As / Rib				
1	Column	2 - #5	0	5.5	1	2 - #5	0.62				
	Middle	6 - #5	0	5.5	6	1 - #5	0.31				
2	Column	4 - #5	0	11	2	2 - #5	0.62				
	Middle	5 - #5	0	11	5	1 - #5	0.31				
3	Column	4 - #5	0	16	2	2 - #5	0.62				
	Middle	2 - #5	0	16	2	1 - #5	0.31				
4	Column	8 - #5	0	23	4	2 - #5	0.62				
	Middle	3 - #5	0	23	3	1 - #5	0.31				

F -39



Floor 2: Slab Design Results Continued

spSlab Design Results: Frame A.2 (Floors 2)

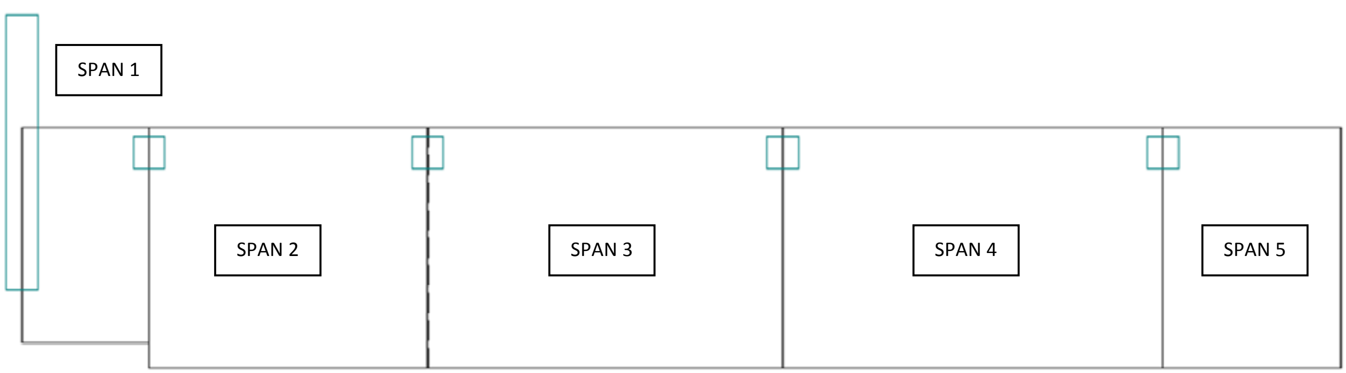
Top Reinforcement

Span	Strip	Left Side				Right Side				Continuous	
		Bars	Length	Bars	Length	Bars	Length	Bars	Length	Bars	Length
1	Column					1- #5	1.86			2 - #5	5
	Middle									5 - #5	5
2	Column									3 - #5	11
	Middle	1 - #5	2.77			2 - #5	4.83	2 - #5	3.18	4 - #5	11
3	Column	2 - #5	4.83	1 - #5	3.18	4 - #5	4.68				
	Middle	4 - #5	3.93								
4	Column									4 - #5	15
	Middle									4 - #5	15
5	Column									4 - #5	7
	Middle									4 - #5	7

Bottom Reinforcement

Span	Strip	Long Bars			Waffle			Total Strip Width	Maximum Possible Number of Ribs Per Strip (20" clear spacing)		
		Bars	Start	Length	Ribs	Bars / Rib	As / Rib				
1	Column	2 - #5	0	5	1	2 - #5	0.62				
	Middle	3 - #5	0	5	3	1 - #5	0.31				
2	Column	2 - #5	0	11	1	2 - #5	0.62				
	Middle	3 - #5	0	11	3	1 - #5	0.31				
3	Column	2 - #5	0	14	2	1 - #5	0.31				
	Middle	4 - #5	0	14	2	2 - #5	0.62				
4	Column	4 - #5	0	15	2	2 - #5	0.62				
	Middle	4 - #5	0	15	2	2 - #5	0.62				
5	Column										
	Middle										

F -40



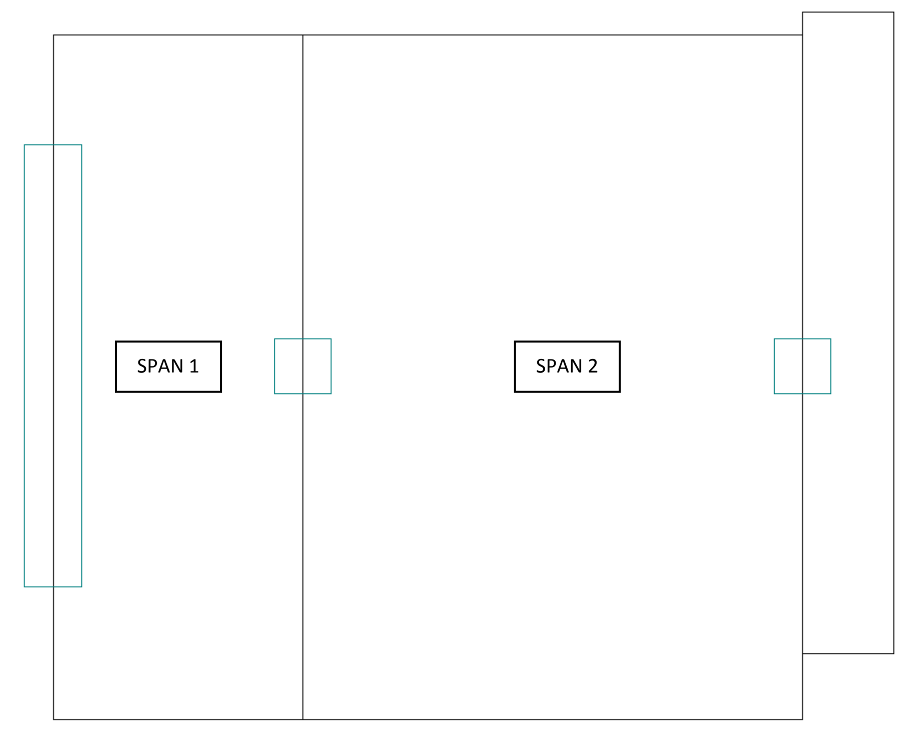
Due To Slight Cantilever, Span 5 is 11.5” Flat Plate (No voids)

Floor 2: Slab Design Results Continued

spSlab Design Results: Frame B.1 Top (Floor 2)											
Top Reinforcement											
Span	Strip	Left Side				Right Side				Continuous	
		Bars	Length	Bars	Length	Bars	Length	Bars	Length	Bars	Length
1	Column									2 - #5	5.5
	Middle									9 - #5	5.5
2	Column	2 - #5	3.84			5 - #5	3.84	5 - #5	2.58		
	Middle	9 - #5	3.18			16 - 35	2.77				
3	Column									10 - #5	2
	Middle									16 - #5	2

Bottom Reinforcement										
Span	Strip	Long Bars			Waffle			Total Strip Width	Maximum Possible Number of Ribs Per Strip (20" clear spacing)	
		Bars	Start	Length	Ribs	Bars / Rib	As / Rib			
1	Column	2 - #5	0	5.5	1	2 - #5	0.62			
	Middle	6 - #5	0	5.5	6	1 - #5	0.31			
2	Column	4 - #5	0	11	2	2 - #5	0.62			
	Middle	5 - #5	0	11	5	1 - #5	0.31			
3	Column									
	Middle									

F -41



Floor 2: Slab Design Results Continued

spSlab Design Results: Frame B.1 Bottom (Floor 2)

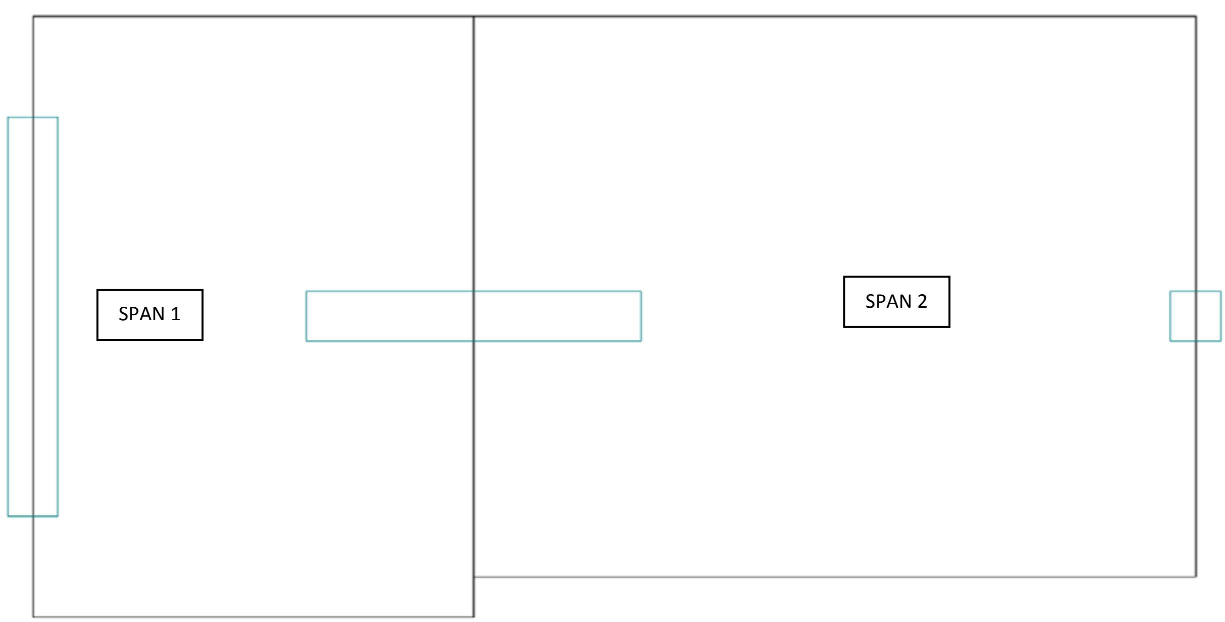
Top Reinforcement

Span	Strip	Left Side				Right Side				Continuous	
		Bars	Length	Bars	Length	Bars	Length	Bars	Length	Bars	Length
1	Column	3 - #5	2.92	1 - #5	1.87	3 - #5	6.22	1 - #5	5.41		
	Middle	7 - #5	1.99			7 - #5	5.53				
2	Column	3 - #5	8.53	1 - #5	6.81	4 - #5	4.98	1 - #5	3.27		
	Middle	7 - #5	7.07			5 - #5	3.53				

Bottom Reinforcement

Span	Strip	Long Bars			Waffle			Total Strip Width	Maximum Possible Number of Ribs Per Strip (20" clear spacing)	
		Bars	Start	Length	Ribs	Bars / Rib	As / Rib			
1	Column	4 - #5	0	11	2	2 - #5	0.62			
	Middle	5 - #5	0	11	5	1 - #5	0.31			
2	Column	6 - #5	0	18	3	2 - #5	0.62			
	Middle	4 - #5	0	18	4	1 - #5	0.31			

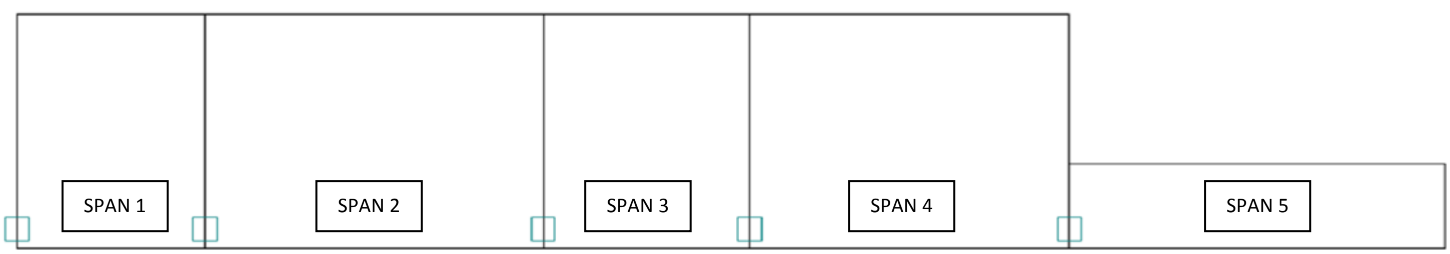
F -42



Floor 2: Slab Design Results Continued

spSlab Design Results: Frame 1 (Floor 2)											
Top Reinforcement											
Span	**Strip**	**Left Side**				**Right Side**				**Continuous**	
		Bars	**Length**	**Bars**	**Length**	**Bars**	**Length**	**Bars**	**Length**	**Bars**	**Length**
1	Column									3 - #5	10
	Middle									7 - #5	10
2	Column	2 - #5	6.15	1 - #5	3.98	2 - #5	6.15	1 - #5	3.98		
	Middle	7 - #5	4.67			6 - #5	4.67				
3	Column					2 - #5	3.84			3 - #5	11
	Middle					2 - #5	2.77			6 - #5	11
4	Column					5 - #5	5.82	4 - #5	3.78	5 - #5	17
	Middle	2 - #5	4.09			2 - #5	4.09			6 - #5	17
5	Column									14 - #5	20
	Middle	6 - #5	4.89							2 - #5	20
Bottom Reinforcement											
Span	**Strip**	**Long Bars**			**Waffle**			**Total Strip Width**	**Maximum Possible Number of Ribs Per Strip (20" clear spacing)**		
		Bars	**Start**	**Length**	**Ribs**	**Bars / Rib**	**As / Rib**				
1	Column	2 - #5	0	10	1	2 - #5	0.62				
	Middle	5 - #5	0	10	5	1 - #5	0.31				
2	Column	4 - #5	0	18	2	2 - #5	0.62				
	Middle	4 - #5	0	18	4	1 - #5	0.31				
3	Column	2 - #5	0	11	1	2 - #5	0.62				
	Middle	5 - #5	0	11	5	1 - #5	0.31				
4	Column	5 - #5	0	17							
	Middle	6 - #5	0	17							
5	Column										
	Middle										

F -43



For span 4 and 5, the base waffle slab design did not suffice. As a result, the voids were left out, and an 11.5” slab was used. This was required because the slab slight cantilevers generating greater flexural demand.

Floor 2: Slab Design Results Continued

spSlab Design Results: Frame 4 (Floor 2)

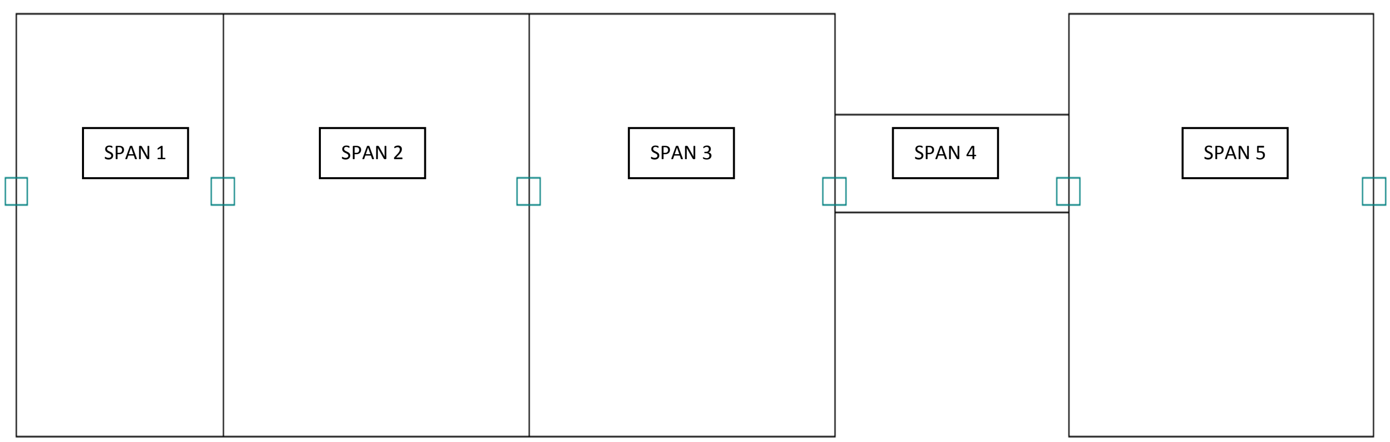
Top Reinforcement

Span	Strip	Left Side				Right Side				Continuous	
		Bars	Length	Bars	Length	Bars	Length	Bars	Length	Bars	Length
1	Column									4 - #5	11.5
	Middle									10 - #5	11.5
2	Column	3 - #5	5.82	1 - #5	3.78	4 - #5	5.86	2 - #5	3.78		
	Middle	10 - #5	4.61			8 - #5	5.86				
3	Column	4 - #5	5.82	2 - #5	3.78	1 - #5	5.82	1 - #5	3.78		
	Middle	8 - #5	5.61			12 - #5	4.09				
4	Column					1 - #5	4.5			2 -#5	13
	Middle	10 - #5	3.21			10 - #5	3.21			2 - #5	13
5	Column	2 - #5	5.82	1 - #5	3.78	4 - #5	5.82	2 - #5	3.78		
	Middle	12 - #5	4.09								

Bottom Reinforcement

Span	Strip	Long Bars			Waffle			Total Strip Width	Maximum Possible Number of Ribs Per Strip (20" clear spacing)		
		Bars	Start	Length	Ribs	Bars / Rib	As / Rib				
1	Column	4 - #5	0	11.5	2	2 - #5	0.62				
	Middle	7 - #5	0	11.5	7	1 - #5	0.31				
2	Column	4 - #5	0	17	4	1 - #5	0.31				
	Middle	5 - #5	0	17	5	1 - #5	0.31				
3	Column	4 - #5	0	17	4	1 - #5	0.31				
	Middle	5 - #5	0	17	5	1 - #5	0.31				
4	Column	2 - #5	0	13	1	2 - #5	0.62				
	Middle	2 - #5	0	13	1	2 - #5	0.62				
5	Column	4 - #5	0	17	4	1 - #5	0.31				
	Middle	5 - #5	0	17	5	1 - #5	0.31				

F -44



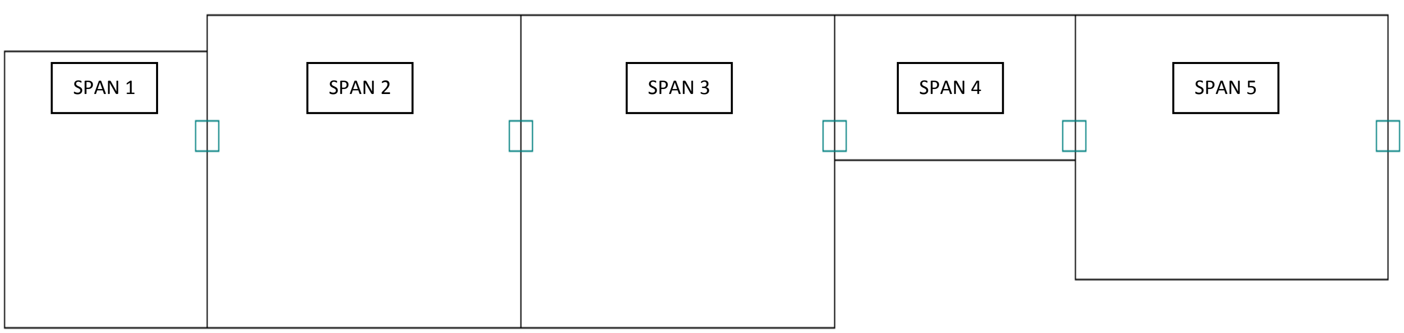
Floor 2: Slab Design Results Continued

spSlab Design Results: Frame 6 (Floor 2)

Top Reinforcement											
Span	Strip	Left Side				Right Side				Continuous	
		Bars	Length	Bars	Length	Bars	Length	Bars	Length	Bars	Length
1	Column									8 - #5	11
	Middle									5 - #5	11
2	Column	3 - #5	5.82							5 - #5	17
	Middle									5 - #5	17
3	Column	3 - #5	5.82	2 - #5	3.78	2 - #5	5.82	1 - #5	3.78		
	Middle	5 - #5	5.11			7 - #5	4.11				
4	Column									3 - #5	13
	Middle	4 - #5	3.21			3 - #5	3.21			3 - #5	13
5	Column	2 - #5	5.82	1 - #5	3.78	3 - #5	5.82	1 - #5	3.78		
	Middle	6 - #5	4.61			4 - #5	4.09				

Bottom Reinforcement											
Span	Strip	Long Bars			Waffle			Total Strip Width	Maximum Possible Number of Ribs Per Strip (20" clear spacing)		
		Bars	Start	Length	Ribs	Bars / Rib	As / Rib				
1	Column										
	Middle										
2	Column	3 - #5	0	17	3	1 - #5	0.31				
	Middle	3 - #5	0	17	3	1 - #5	0.31				
3	Column	3 - #5	0	17	3	1 - #5	0.31				
	Middle	3 -#5	0	17	3	1 - #5	0.31				
4	Column	2 - #5	0	13	1	2 - #5	0.62				
	Middle	2 -#5	0	13	2	1 - #5	0.31				
5	Column	4 - #5	0	17	2	2 - #5	0.62				
	Middle	3 - #5	0	17	3	1 - #5	0.31				

F -45



Due to column offsets, span 1 is considered a cantilever when modeling according to equivalent frame method. With larger moments, the base waffle slab was not used for span 1. The voids were left out.

Floor 2: Slab Design Results Continued

spSlab Design Results: Frame 8 (Floor 2)

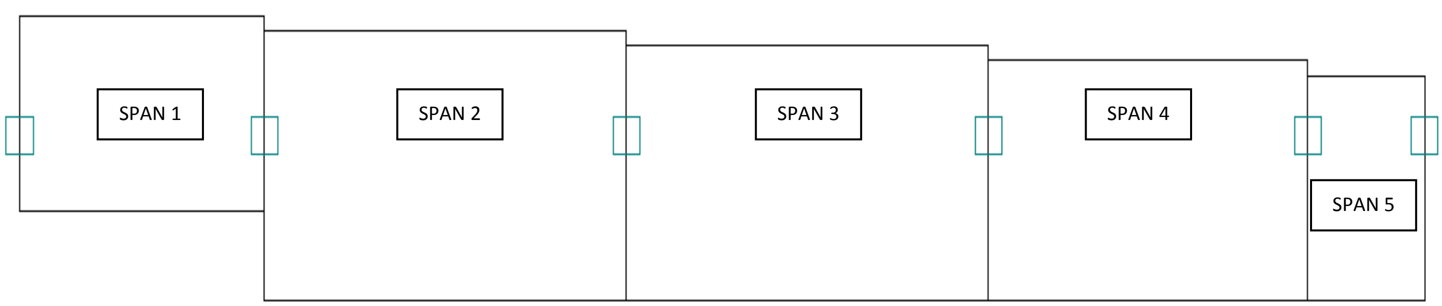
Top Reinforcement

Span	Strip	Left Side				Right Side				Continuous	
		Bars	Length	Bars	Length	Bars	Length	Bars	Length	Bars	Length
1	Column									3 - #5	11.5
	Middle					1 - #5	2.88			3 - #5	11.5
2	Column	2 - #5	5.82	1 - #5	3.78	2 - #5	5.82	1 - #5	3.78		
	Middle	4 - #5	4.61			4 - #5	5.36				
3	Column	2 - #5	5.82	1 - #5	3.78	2 - #5	5.82	1 - #5	3.78		
	Middle	4 - #5	5.36			4 - #5	5.11				
4	Column	2 - #5	5.23	1 - #5	3.78	2 - #5	5.16				
	Middle	4 - #5	5.23			4 - #5	3.98				
5	Column									2 - #5	5.5
	Middle									4 - #5	5.5

Bottom Reinforcement

Span	Strip	Long Bars			Waffle			Total Strip Width	Maximum Possible Number of Ribs Per Strip (20" clear spacing)		
		Bars	Start	Length	Ribs	Bars / Rib	As / Rib				
1	Column	2 - #5	0	11.5	1	2 - #5	0.62				
	Middle	2 - #5	0	11.5	2	1 - #5	0.31				
2	Column	2 - #5	0	17	2	1 - #5	0.31				
	Middle	2 - #5	0	17	2	1 - #5	0.31				
3	Column	2 - #5	0	17	2	1 - #5	0.31				
	Middle	2 - #5	0	17	2	1- #5	0.31				
4	Column	2 - #5	0	15	2	1 - #5	0.31				
	Middle	2 - #5	0	15	2	1 - #5	0.31				
5	Column	2 - #5	0	5.5	1	2 - #5	0.62				
	Middle	4 - #5	0	5.5	2	2 - #5	0.62				

F -46



Floor 2: Slab Design Results Continued

spSlab Design Results: Frame 9 (Floor 2)

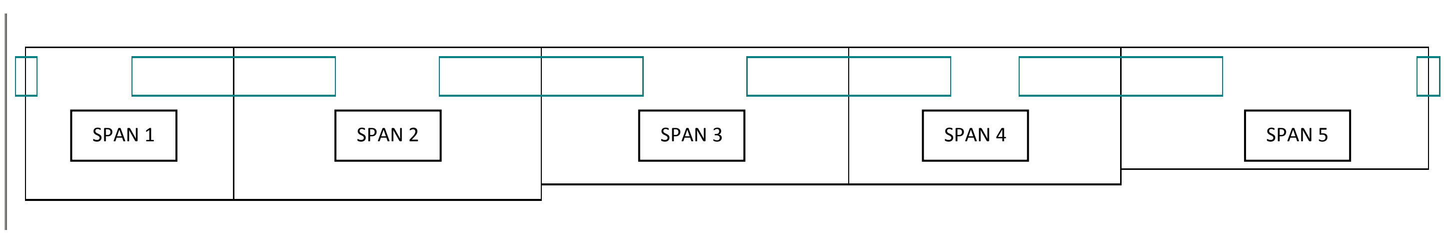
Top Reinforcement

Span	Strip	Left Side				Right Side				Continuous	
		Bars	Length	Bars	Length	Bars	Length	Bars	Length	Bars	Length
1	Column	2 - #5	2.36	1 - #5	1.68	2 - #5	7.36	1 - #5	6.68		
	Middle	2 - #5	1.78			2 - #5	6.78				
2	Column	2 - #5	7.52	1 - #5	6.78	2 - #5	7.52				
	Middle	2 - #5	6.89			2 - #5	6.89				
3	Column	2 - #5	7.52			2 - #5	7.52				
	Middle	2 - #5	6.89			2 - #5	6.89				
4	Column	2 - #5	6.86			2 - #5	6.86				
	Middle	2 - #5	6.63			2 - #5	6.63				
5	Column	2 - #5	9.17			2 - #5	4.17				
	Middle	2 - #5	7.99			2 - #5	2.99				

Bottom Reinforcement

Span	Strip	Long Bars			Waffle			Total Strip Width	Maximum Possible Number of Ribs Per Strip (20" clear spacing)		
		Bars	Start	Length	Ribs	Bars / Rib	As / Rib				
1	Column	2 - #5	0	11.5	1	2 - #5	0.62				
	Middle	2 - #5	0	11.5	1	2 - #5	0.62				
2	Column	2 - #5	0	17	1	2 - #5	0.62				
	Middle	2 - #5	0	17	1	2 - #5	0.62				
3	Column	2 - #5	0	17	1	2 - #5	0.62				
	Middle	2 - #5	0	17	1	2 - #5	0.62				
4	Column	2 - #5	0	15	1	2 - #5	0.62				
	Middle	2 - #5	0	15	1	2 - #5	0.62				
5	Column	2 - #5	0	17	1	2 - #5	0.62				
	Middle	2 - #5	0	17	1	2 - #5	0.62				

F -47

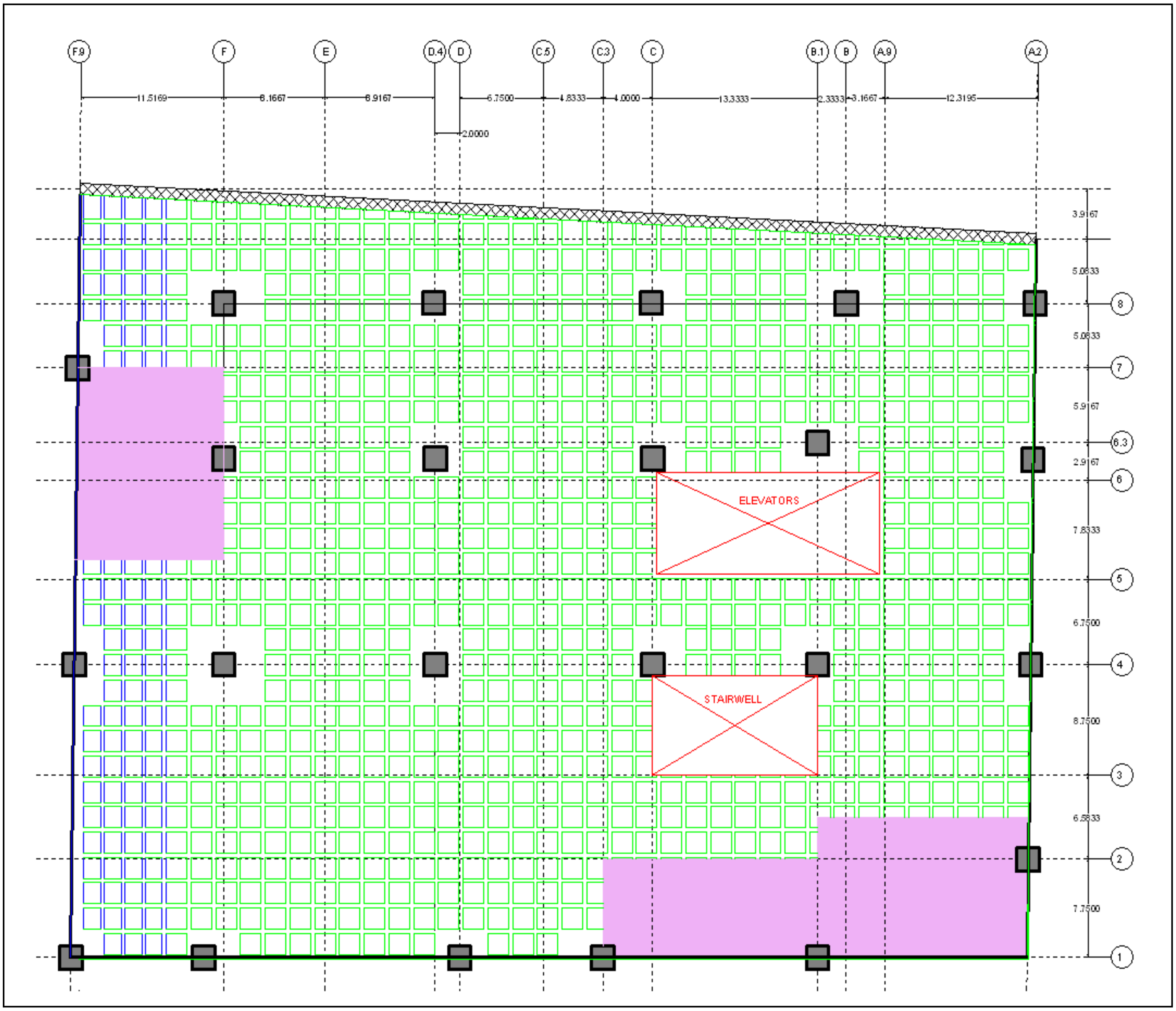


Floor 2: Slab Design Results Summary:

Floor 2 is very similar in the layout observed at floors 3 – 9. However, as shown in the final waffle slab layout on the following page, floor 2 extends beyond grid line 8. This section of floor 2 is an outdoor terrace that rests on an exterior reinforced CMU wall. The design live load for the terrace is 100 psf. In addition, the slab is slightly cantilevered out at the lower right corner of the plan. Please see figure F-48 on following page.

As is evident in figure F-48, these slight variations from the layout of floors 3 – 9 results in some significant slab design changes. In fact, a solid 11 ½" slab is employed in two separate locations, which are highlighted in pink in figure F-48. A smaller slab would have sufficed; however, since the sections requiring a solid slab are relatively small, simply leaving out the voids of the base waffle slab to create an 11 ½" thick slab was the best option. In addition, the slab along frame F.9 is designed as a waffle slab with 16" clear spaced ribs (shown in blue in figure F-48). Several options were considered when selecting the final slab design for these various locations. By changing the spacing of the ribs, greater flexural capacity was obtained without changing the overall depth of the floor. Varying the depth of the slab was avoided because it would have required special placement of continuous bottom bar reinforcement to accommodate for locations where there was a change in elevation of the bottom of slab.

Plan View – Final Slab Layout (Floor 2):



Base Waffle Slab: 3 ½" Slab, 4"x8" ribs @ 20" clear spacing:

Modified Waffle Slab: 16" rib clear spacing:

8" flat plate:

F -48

Note: Due to longer spans and/or "slightly" cantilevered slabs, the base waffle slab had to be substituted with a modified waffle slab design or in some cases an 11.5" (no voids) slab.

Floor 1: Slab Design Results

spSlab Design Results: Frame D.4 (Floor 1) - TYPICAL 1st FLOOR INTERIOR FRAME

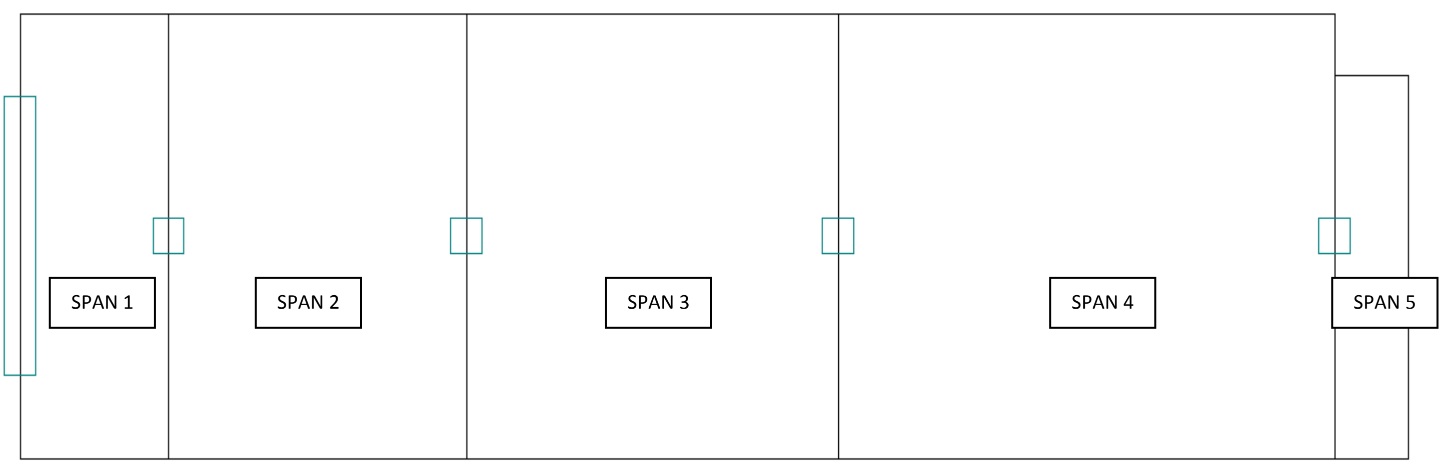
Top Reinforcement

Span	Strip	Left Side				Right Side				Continuous	
		Bars	Length	Bars	Length	Bars	Length	Bars	Length	Bars	Length
1	Column					2 - #5	2.19			3 - #5	6
	Middle									10 - #5	6
2	Column					1 - #5	4.17			5 - #5	12
	Middle	2 - #5	2.99							8 - #5	12
3	Column					3 - #5	5.16	3 - #5	3.38	6 - #5	15
	Middle	1 - #5	3.65							7 - #5	15
4	Column	6 - #5	6.81	6 - #5	4.38	4 - #5	6.81	4 - #5	4.38		
	Middle	7 - #5	6.05			7 - #5	4.75				
5	Column	2 - #5	1.63							6 - #5	3
	Middle	1 - #5	1.63							6 - #5	3

Bottom Reinforcement

Span	Strip	Long Bars			Short Bars		
		Bars	Start	Length	Bars	Start	Length
1	Column	3 - #5	0	6			
	Middle	8 - #5	0	6	2 - #5	0.90	4.2
2	Column	5 - #5	0	12			
	Middle	6 - #5	0	12	2 - #5	1.80	8.4
3	Column	6 - #5	0	15			
	Middle	5 - #5	0	15	2 - #5	2.25	10.5
4	Column	8 - #5	0	20			
	Middle	5 - #5	0	20	1 - #5	3.00	17
5	Column		0				
	Middle		0				

F -49



Floor 1: Slab Design Results Continued:

spSlab Design Results: Frame F.8 Top (Floor 1) - TYPICAL EXTERIOR FRAME

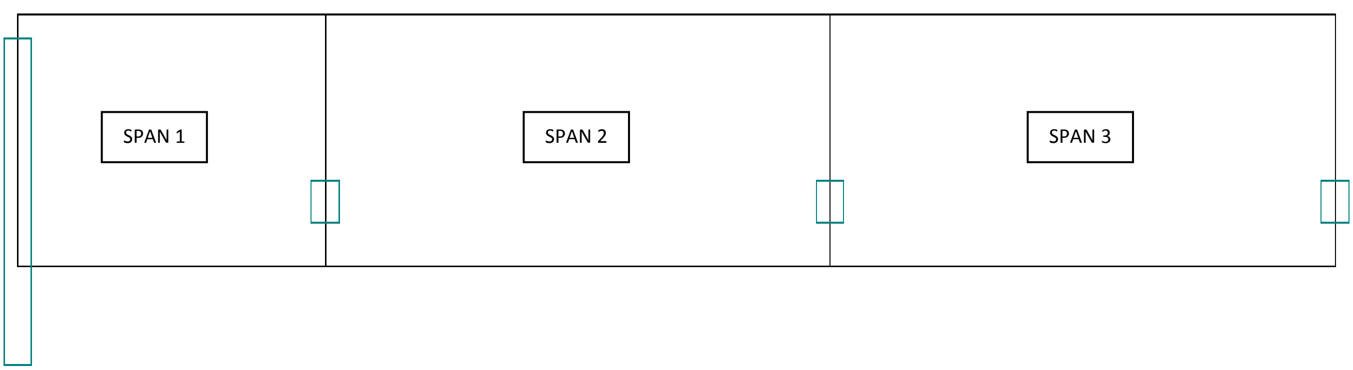
Top Reinforcement

Span	Strip	Left Side				Right Side				Continuous	
		Bars	Length	Bars	Length	Bars	Length	Bars	Length	Bars	Length
1	Column					2 - #5	4.83			4 - #5	14
	Middle	1 - #5	3.43							3 - #5	14
2	Column	3 - #5	7.8	3 - #5	4.98	5 - #5	8.23	5 - #5	4.98		
	Middle	3 - #5	6.23			4 - #5	8.23				
3	Column	5 - #5	7.98	5 - #5	4.98	3 - #5	7.8	1 - #5	4.98		
	Middle	4 - #5	7.98			3 - #5	5.41				

Bottom Reinforcement

Span	Strip	Long Bars			Short Bars		
		Bars	Start	Length	Bars	Start	Length
1	Column	4 - #5	0	14			
	Middle	3 - #5	0	14			
2	Column	4 - #5	0	23			
	Middle	3 - #5	0	23			
3	Column	May-35	0	23			
	Middle	2 - #5	0	23	2 - #5	3.45	16.1

F -50



Floor 1: Slab Design Results Summary

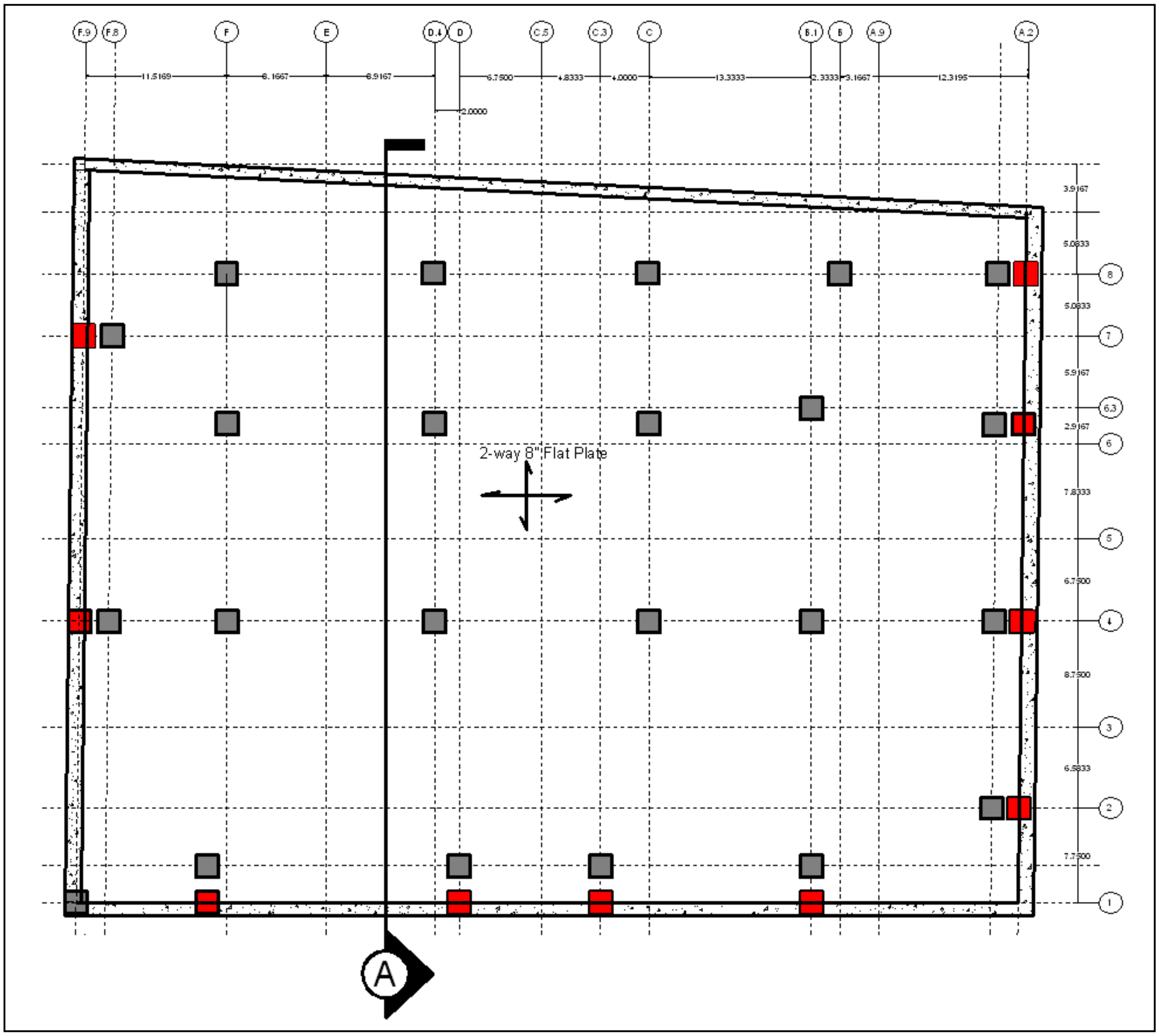
Floor 1 is entirely dedicated to retail services. As a result, the design live load is 100 PSF. With openings in the slab and a live load 2.5 times larger than that for the upper residential floors, the base waffle slab design was not a viable option. In order to avoid creating a construction management nightmare, no more modified waffle slab designs were employed either. Instead, hand calculations and spSlab software were used in combinations to determine the most efficient two way flat plate design.

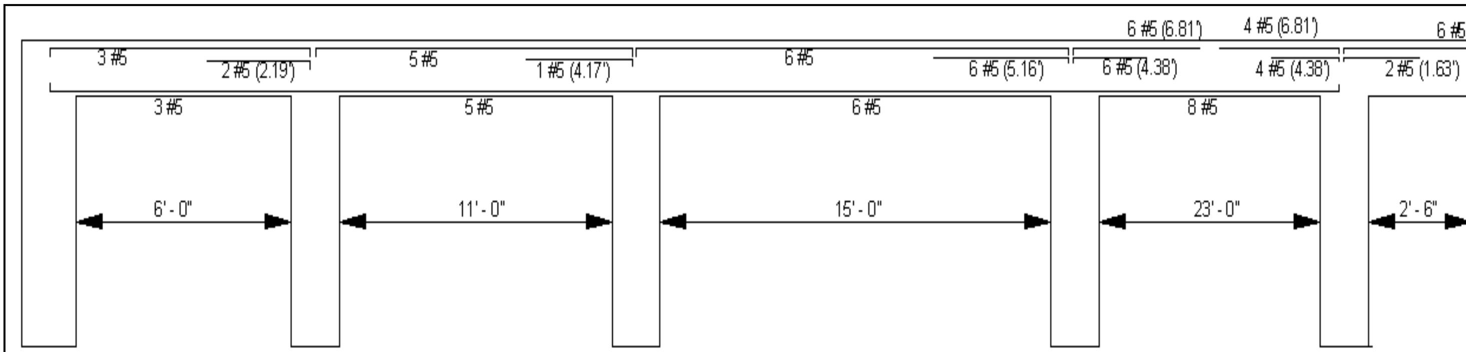
Based on calculation results, an 8" two way flat plate was determined as the most appropriate slab design. Figure F-23 on page 33, shows a general detail of an 8" flat plate. It is important to note, that the only load bearing exterior wall shown on figure F-51 is the top wall. This was designed as a masonry bearing wall.

Section A on the following page is not to scale; however, it does provide a good profile view of the 8" flat plate as well as important reinforcement extension values which apply to the majority of the first floor frames.

Plan View: Final Slab Layout (Floor 1):

F -51





SECTION A - REINFORCEMENT EXTENSIONS: TYPICAL FIRST FLOOR INTERIOR FRAME

Note: Drawing not to scale. Labeled dimensions are correct. Bottom reinforcement varies span to span. (2) # 5 Continuous bars are used as integrity steel to provide the slab some residual strength for punching shear failures. Only Longitudinal Reinforcement is Shown.

Reinforced Concrete Column Design:

Introduction: With the structural floor system designed, the vertical elements of the reinforced concrete structural system were designed next. As commonly done in practice, tied columns were selected for the design since the building is located in a nonseismic region. With a rather complicated slab design, it was important to avoid creating any more constructability issues. Using square tied columns with the same reinforcement on each side greatly reduces the potential for construction errors. Although all columns will be subject to combined axial load and bending, the tied columns are more than appropriate since they are not intended to participate in significant lateral load resistance. The lateral system consists of several reinforced concrete shear walls and no concrete moment frames. Based on these loading conditions, it was determined square tied columns would provide sufficient lateral restraint on the column core. With axial dominated load conditions, the primary design goal regarding the column ties is to properly space them to mitigate buckling potential of the vertical reinforcing bars. The bending moments in the columns are generated from the unbalanced moments transferred at the column-slab joints.

Process: The column design process operated under the assumption that "short column" behavior applied. As will be discussed with great detail in the following section, the process involved a series of steps that included exhaustive hand-calculations and computer-aided analysis. The basic steps of the design process are: Determine the unfactored live and dead axial loads for every column, calculate the bending moments for every column, determine the factored combined axial load and bending design conditions, design several columns via hand calculations, generate corresponding interaction diagrams via pcaColumn and hand calculations, and assign all columns final design section according to loading condition.

- Step One – Unfactored Dead Axial Loads (Page: 64 - 65): First, the column design loads had to be determined. Since the slab type, and thus slab dead load vary a lot throughout the building, both the calculation of loads and the design process were done primarily via hand calculations. Since columns resists loads within tributary areas on upper floors, load calculations began at the 13th floor. As shown in figure F-52 on page 64, the dead load (PSF) for each type of slab was calculated. Please see appendix B for related calculations. Next, for every column, the tributary areas were determined. In many cases however, the column tributary area had to be subdivided into different areas according to the slab type. For example, columns 6-F on floors 10-13 must resist a 78 SF section of Base Waffle Slab and a 130 SF section of 8"/Waffle Slab Overlay. Since a typical floor slab was designed for floors 10-13 as well as floors 3-9, the load calculation process was organized into just 4 tables, which also includes floors 1 and 2 (see figures F-53 through F-56). The green highlighted cells of table F-56 represent the unfactored dead axial load on 1st floor corbels. Although unnecessary for column design, the total dead load per floor was determined (yellow highlighted cells) for calculation of lateral story forces and the seismic base shear. As shown in figures F-53 through F-56, a miscellaneous superimposed dead load of 30 PSF was included in the design dead load. This value accounts for the unknown self weight of the columns, MEP loads, and partition loads.
- Step Two – Determine Unfactored Live Axial Loads (Page: 66 - 67): Next, the unfactored live axial loads were determined. In a similar manner to the dead load calculations, the tributary areas for each column were determined, and organized into four figures F-57 through F-60. The elevated floors, 2 through 13, function as residential spaces. The first floor functions as a retail space. As a result, the upper floors and 1st floor resist 40 PSF and 100 PSF live loads respectively.

Reinforced Concrete Column Design Continued:

- Step Three – Determine Bending Moments In Columns (Page: 68 - 72): With the axial loads calculated, the bending loads had to be determined. See page 68 and figures F-61 through F-68 for a detailed summary of the associated calculations and results.

- Step Four – Determine Factored Combined Axial and Bending Design Load Conditions: The governing load combination was 1.2D + 1.6L. Figures F-69 through F-81 display the design load conditions for every column present in the structure. With these combined loadings, the columns were designed.

- Step Five: Finally, the column design process began. For a detailed explanation of the calculations, assumptions, and theory involved in the design process, see pages 77 - 86. To quickly summarize the design process, it is important to mention the main steps of design. In step one, the entire field of loading conditions for each of the columns (figures F-69 through F-81) was analyzed. Next, from these loading conditions a set of 6 loading conditions were selected. The goal was to select a set of load conditions that most represented the various types of loadings present in the structure. The six set of loading conditions are displayed in table F-82 on page 77. The selected load conditions appropriately included nearly pure axial, nearly pure bending, as well as large and small magnitudes of loadings. To avoid exceeding the boundaries of practical design, a balance between constructability and structural efficiency had to be achieved. Therefore, just six column sections were designed instead of designing a specific column section for each and every loading condition. This strategy was appropriate since most of the columns shared similar loading conditions. The final column design sections can be found on page 78. Note, the column sections are labeled A – F. Once the six column sections were designed, the most efficient column section was assigned according to the loading conditions. See the last data column in figures F-69 through F-81 to view the design section chosen for each column (Red Letters: A – F).

Un-factored Dead Loads on Columns

APPROXIMATE SLAB WEIGHTS	
Waffle Slab	46
16" Waffle Slab	48.8
8" / Waffle Slab Overlay	82
8" Flat Plate	76
11.5" Slab (No Voids)	110
NOTE: L.W.C. = 115 PCF. See APPENDIX B For Determination of Slab Weights	

F-52

F-53

Floor 3- 9											
					TOTAL DEAD LOAD ON COLUMN INCLUDING ABOVE FLOORS						
Column	Slab Type	Tributary Area	Miscellaneous Dead Load	Total Dead Load Per Column (just from 9th Floor)	9th Floor Column	8th Floor Column	7th Floor Column	6th Floor Column	5th Floor Column	4th Floor Column	3rd Floor Column
1 - B.1	Waffle Slab	88.2	30	6703.2	33516	40219.2	46922.4	53625.6	60328.8	67032	73735.2
1 - C.3	Waffle Slab	93.6	30	7113.6	35568	42681.6	49795.2	56908.8	64022.4	71136	78249.6
1 - D.4	Waffle Slab	156	30	11856	81744	93600	105456	117312	129168	141024	152880
1 - F	Waffle Slab	156	30	11856	59280	71136	82992	94848	106704	118560	130416
1 - F.9	Waffle Slab	60	30	4560	22800	27360	31920	36480	41040	45600	50160
2 - A.2	Waffle Slab	96	30	7296	36480	43776	51072	58368	65664	72960	80256
4 - A.2	Waffle Slab	108.5	30	8246	42445.2	50691.2	58937.2	67183.2	75429.2	83675.2	91921.2
4 - B.1	Waffle Slab	125	30	9500	48900	58400	67900	77400	86900	96400	105900
4 - C	Waffle Slab	100	30	7600	39120	46720	54320	61920	69520	77120	84720
4 -D.4	Waffle Slab	250	30	19000	152280	171280	190280	209280	228280	247280	266280
4 -F	Waffle Slab	227.5	30	17290	88998	106288	123578	140868	158158	175448	192738
4 - F.9	Waffle Slab	92.5	30	7030	36186	43216	50246	57276	64306	71336	78366
6 - A.2	Waffle Slab	180	30	13680	68400	82080	95760	109440	123120	136800	150480
6 - B.1	Waffle Slab	51	30	3876	42228	46104	49980	53856	57732	61608	65484
6 - C	Waffle Slab	160	30	12160	140960	153120	165280	177440	189600	201760	213920
6 - D.4	Waffle Slab	238	30	18088	18088	36176	54264	72352	90440	108528	126616
6 - F	Waffle Slab	169	30	12844	94796	107640	120484	133328	146172	159016	171860
7 - F.9	Waffle Slab	50	30	3800	19000	22800	26600	30400	34200	38000	41800
8 - A.2	Waffle Slab	32.5	30	2470	12350	14820	17290	19760	22230	24700	27170
8 - B	Waffle Slab	87.5	30	6650	33250	39900	46550	53200	59850	66500	73150
8 - C	Waffle Slab	75	30	5700	28500	34200	39900	45600	51300	57000	62700
8 - D.4	Waffle Slab	70	30	5320	77260	82580	87900	93220	98540	103860	109180
8 - F	Waffle Slab	40	30	3040	15200	18240	21280	24320	27360	30400	33440
	Total Slab Dead Load (for each floor 3 - 9)			205678.8							

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Floor 13 Dead Load Calculations									
Column	Slab Type 1	Tributary Area 1 (SF)	Slab Type 2	Tributary Area 2 (SF)	Miscellaneous Dead Load (PSF)	Total Dead Load Per Column (lbs)	12th Floor Columns	11th Floor Columns	10th Floor Columns
1 - B.1	Waffle Slab	88.2	N/A	N/A	30	6703.2	13406.4	20109.6	26812.8
1 - C.3	Waffle Slab	93.6	N/A	N/A	30	7113.6	14227.2	21340.8	28454.4
1 - D.4	8"/Waffle Overlay	156	N/A	N/A	30	17472	34944	52416	69888
1 - F	Waffle Slab	156	N/A	N/A	30	11856	23712	35568	47424
1 - F.9	Waffle Slab	60	N/A	N/A	30	4560	9120	13680	18240
2 - A.2	Waffle Slab	96	N/A	N/A	30	7296	14592	21888	29184
4 - A.2	16" Waffle Slab	108.5	N/A	N/A	30	8549.8	17099.6	25649.4	34199.2
4 - B.1	16" Waffle Slab	125	N/A	N/A	30	9850	19700	29550	39400
4 - C	16" Waffle Slab	100	N/A	N/A	30	7880	15760	23640	31520
4 -D.4	8" / Waffle Overlay	297.5	N/A	N/A	30	33320	66640	99960	133280
4 -F	16" Waffle Slab	227.5	N/A	N/A	30	17927	35854	53781	71708
4 - F.9	16" Waffle Slab	92.5	N/A	N/A	30	7289	14578	21867	29156
6 - A.2	Waffle Slab	180	N/A	N/A	30	13680	27360	41040	54720
6 - B.1	Waffle Slab	51	8" / Waffle Overlay	51	30	9588	19176	28764	38352
6 - C	8" / Waffle Overlay	287.5	N/A	N/A	30	32200	64400	96600	128800
6 - F	Waffle Slab	78	8" / Waffle Overlay	130	30	20488	40976	61464	81952
7 - F.9	Waffle Slab	50	N/A	N/A	30	3800	7600	11400	15200
8 - A.2	Waffle Slab	32.5	N/A	N/A	30	2470	4940	7410	9880
8 - B	Waffle Slab	87.5	N/A	N/A	30	6650	13300	19950	26600
8 - C	Waffle Slab	75	N/A	N/A	30	5700	11400	17100	22800
8 - D.4	8" / Waffle Overlay	82.5	8" Slab	82.5	30	17985	35970	53955	71940
8 - F	Waffle Slab	40	N/A	N/A	30	3040	6080	9120	12160
				TOTAL FLOOR DEAD LOAD (for each floor 10 - 13)		255417.6			

Unfactored Dead Loads on Columns Continued

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Floor 2 Dead Load Calculations

Column	Slab Type 1	Tributary Area 1 (SF)	Slab Type 2	Tributary Area 2 (SF)	Miscellaneous Dead Load (PSF)	Total Dead Load Per Column (lbs)	Total Load Per Column (All Floors Above Included)
1 - B.1	11.5" Slab (no voids)	95	N/A	N/A	30	13300	87035.2
1 - C.3	11.5" Slab (no voids)	47	Waffle Slab	47	30	10152	88401.6
1 - D.4	Waffle Slab	156	N/A	N/A	30	11856	164736
1 - F	Waffle Slab	156	N/A	N/A	30	11856	142272
1 - F.9	16" Waffle Slab	60	N/A	N/A	30	4728	54888
2 - A.2	11.5" Slab (no voids)	48	Waffle Slab	48	30	10368	90624
4 - A.2	Waffle Slab	108.5	N/A	N/A	30	8246	100167.2
4 - B.1	Waffle Slab	125	N/A	N/A	30	9500	115400
4 - C	Waffle Slab	100	N/A	N/A	30	7600	92320
4 -D.4	Waffle Slab	297.5	N/A	N/A	30	22610	288890
4 -F	Waffle Slab	227.5	N/A	N/A	30	17290	210028
4 - F.9	16" Wafle Slab	92.5	N/A	N/A	30	7289	85655
6 - A.2	Waffle Slab	180	N/A	N/A	30	13680	164160
6 - B.1	Waffle Slab	102	N/A	N/A	30	7752	73236
6 - C	Waffle Slab	287.5	N/A	N/A	30	8625	222545
6 - D.4	Waffle Slab	238	N/A	N/A	30	18088	144704
6 - F	Waffle Slab	104	11.5" Slab (no voids)	104	30	22464	194324
7 - F.9	16" Waffle Slab	47	11.5" Slab (no voids)	100	30	17703.6	59503.6
8 - A.2	Waffle Slab	42	N/A	N/A	30	3192	30362
8 - B	Waffle Slab	84	N/A	N/A	30	6384	79534
8 - C	Waffle Slab	84	N/A	N/A	30	6384	69084
8 - D.4	Waffle Slab	95	N/A	N/A	30	7220	116400
8 - F	16" Waffle Slab	120	N/A	N/A	30	9456	42896
				TOTAL DEAD LOAD (for floor 2)		255743.6	

Floor 1 Dead Load Calculations

Column	Slab Type 1	Tributary Area 1 (SF)	Slab Type 2	Tributary Area 2 (SF)	Miscellaneous Dead Load (PSF)	Total Dead Load Per Column (lbs)	Total Load Per Column (All Floors Above Included)
1 - B.1	8" Flat Plate	95	N/A	N/A	30	10070	97105.2
1 - C.3	8" Flat Plate	94	N/A	N/A	30	9964	98365.6
1 - D.4	8" Flat Plate	156	N/A	N/A	30	16536	181272
1 - F	8" Flat Plate	156	N/A	N/A	30	16536	158808
1 - F.9	8" Flat Plate	60	N/A	N/A	30	6360	61248
2 - A.2	8" Flat Plate	96	N/A	N/A	30	10176	100800
4 - A.2	8" Flat Plate	108.5	N/A	N/A	30	11501	111668.2
4 - B.1	8" Flat Plate	125	N/A	N/A	30	13250	128650
4 - C	8" Flat Plate	100	N/A	N/A	30	10600	102920
4 -D.4	8" Flat Plate	297.5	N/A	N/A	30	31535	320425
4 -F	8" Flat Plate	227.5	N/A	N/A	30	24115	234143
4 - F.9	8" Flat Plate	92.5	N/A	N/A	30	9805	95460
6 - A.2	8" Flat Plate	180	N/A	N/A	30	19080	183240
6 - B.1	8" Flat Plate	102	N/A	N/A	30	10812	84048
6 - C	8" Flat Plate	287.5	N/A	N/A	30	30475	253020
6 - D.4	8" Flat Plate	238	N/A	N/A	30	25228	169932
6 - F	8" Flat Plate	208	N/A	N/A	30	22048	216372
7 - F.9	8" Flat Plate	147	N/A	N/A	30	15582	75085.6
8 - A.2	8" Flat Plate	42	N/A	N/A	30	4452	34814
8 - B	8" Flat Plate	84	N/A	N/A	30	8904	88438
8 - C	8" Flat Plate	84	N/A	N/A	30	8904	77988
8 - D.4	8" Flat Plate	95	N/A	N/A	30	10070	126470
8 - F	8" Flat Plate	120	N/A	N/A	30	12720	55616
				TOTAL DEAD LOAD (for floor 1)		338723	
				Note: Green Cells Represent Unfactored Dead Load On First Floor Corbels		TOTAL SLAB DEAD LOAD (ALL FLOORS)	3055888.6

Note: Cells Highlighted In Green Represent Unfactored Slab Dead Load On First Floor Corbels

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Unfactored Live Loads on Columns

13th Floor Live Load						
Column	Tributary Area (sf)	Live Load (psf)	Load On Column (lbs)	Live Load on 12th Floor Column	Live Load on 11th Floor Column	Live Load on 10th Floor Column
1 - B.1	88.2	40	3528	7056	10584	14112
1 - C.3	93.6	40	3744	7488	11232	14976
1 - D.4	156	40	6240	12480	18720	24960
1 - F	156	40	6240	12480	18720	24960
1 - F.9	60	40	2400	4800	7200	9600
2 - A.2	96	40	3840	7680	11520	15360
4 - A.2	108.5	40	4340	8680	13020	17360
4 - B.1	125	40	5000	10000	15000	20000
4 - C	100	40	4000	8000	12000	16000
4 -D.4	297.5	40	11900	23800	35700	47600
4 -F	227.5	40	9100	18200	27300	36400
4 - F.9	92.5	40	3700	7400	11100	14800
6 - A.2	180	40	7200	14400	21600	28800
6 - B.1	102	40	4080	8160	12240	16320
6 - C	287.5	40	11500	23000	34500	46000
6 - F	208	40	8320	16640	24960	33280
7 - F.9	50	40	2000	4000	6000	8000
8 - A.2	32.5	40	1300	2600	3900	5200
8 - B	87.5	40	3500	7000	10500	14000
8 - C	75	40	3000	6000	9000	12000
8 - D.4	82.5	40	3300	6600	9900	13200
8 - F	40	40	1600	3200	4800	6400

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9th Floor Live Load										
Column	Tributary Area (sf)	Live Load (psf)	Load on Column (lbs)	Total Load on Column (lbs)	Load on 8th Floor Column	Load on 7th Floor Column	Load on 6th Floor Column	Load on 5th Floor Column	Load on 4th Floor Column	Load on 3rd Floor Column
1 - B.1	88.2	40	3528	17640	21168	24696	28224	31752	35280	38808
1 - C.3	93.6	40	3744	18720	22464	26208	29952	33696	37440	41184
1 - D.4	156	40	6240	31200	37440	43680	49920	56160	62400	68640
1 - F	156	40	6240	31200	37440	43680	49920	56160	62400	68640
1 - F.9	60	40	2400	12000	14400	16800	19200	21600	24000	26400
2 - A.2	96	40	3840	19200	23040	26880	30720	34560	38400	42240
4 - A.2	108.5	40	4340	21700	26040	30380	34720	39060	43400	47740
4 - B.1	125	40	5000	25000	30000	35000	40000	45000	50000	55000
4 - C	100	40	4000	20000	24000	28000	32000	36000	40000	44000
4 -D.4	250	40	10000	57600	67600	77600	87600	97600	107600	117600
4 -F	227.5	40	9100	45500	54600	63700	72800	81900	91000	100100
4 - F.9	92.5	40	3700	18500	22200	25900	29600	33300	37000	40700
6 - A.2	180	40	7200	36000	43200	50400	57600	64800	72000	79200
6 - B.1	51	40	2040	18360	20400	22440	24480	26520	28560	30600
6 - C	160	40	6400	52400	58800	65200	71600	78000	84400	90800
6 - D.4	238	40	9520	9520	19040	28560	38080	47600	57120	66640
6 - F	169	40	6760	40040	46800	53560	60320	67080	73840	80600
7 - F.9	50	40	2000	10000	12000	14000	16000	18000	20000	22000
8 - A.2	32.5	40	1300	6500	7800	9100	10400	11700	13000	14300
8 - B	87.5	40	3500	17500	21000	24500	28000	31500	35000	38500
8 - C	75	40	3000	15000	18000	21000	24000	27000	30000	33000
8 - D.4	70	40	2800	16000	18800	21600	24400	27200	30000	32800
8 - F	40	40	1600	8000	9600	11200	12800	14400	16000	17600

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Unfactored Live Loads on Columns Continued:

2nd Floor Live Load					
Column	Tributary Area (sf)	Live Load (psf)	Tributary Area (sf)	Live Load (psf)	Total Load on Column (lbs)
1 - B.1	95	40	N/A	N/A	42608
1 - C.3	94	40	N/A	N/A	44944
1 - D.4	156	40	N/A	N/A	74880
1 - F	156	40	N/A	N/A	74880
1 - F.9	60	40	N/A	N/A	28800
2 - A.2	96	40	N/A	N/A	46080
4 - A.2	108.5	40	N/A	N/A	52080
4 - B.1	125	40	N/A	N/A	60000
4 - C	100	40	N/A	N/A	48000
4 -D.4	297.5	40	N/A	N/A	129500
4 -F	227.5	40	N/A	N/A	109200
4 - F.9	92.5	40	N/A	N/A	44400
6 - A.2	180	40	N/A	N/A	86400
6 - B.1	102	40	N/A	N/A	34680
6 - C	287.5	40	N/A	N/A	102300
6 - D.4	238	40	N/A	N/A	76160
6 - F	208	40	N/A	N/A	88920
7 - F.9	74	40	74	100	32360
8 - A.2	21	40	21	100	17240
8 - B	54	40	30	100	43660
8 - C	54	40	30	100	38160
8 - D.4	65	40	30	100	38400
8 - F	90	40	30	100	24200

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1st Floor Live Load			
Column	Tributary Area (sf)	Live Load (psf)	Total Load On Column (lbs)
1 - B.1	95	100	52108
1 - C.3	94	100	54344
1 - D.4	156	100	90480
1 - F	156	100	90480
1 - F.9	60	100	34800
2 - A.2	96	100	55680
4 - A.2	108.5	100	62930
4 - B.1	125	100	72500
4 - C	100	100	58000
4 -D.4	297.5	100	159250
4 -F	227.5	100	131950
4 - F.9	92.5	100	53650
6 - A.2	180	100	104400
6 - B.1	102	100	44880
6 - C	287.5	100	131050
6 - D.4	238	100	99960
6 - F	208	100	109720
7 - F.9	147	100	47060
8 - A.2	42	100	21440
8 - B	84	100	52060
8 - C	84	100	46560
8 - D.4	95	100	47900
8 - F	120	100	36200
Green Cells: Live Loads on First Floor Corbel Columns			

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Calculation of Design Moments In Columns:

After determining the axial loads on each column, the transfer of moments at the slab-column connections were analyzed to determine the design moments for each column. Calculation of moments was based upon Section 13.6.9 (Factored Moments in Columns and Walls) within the Direction Design Method provided in ACI 318-08.

For each interior column at every floor, the adjacent slab panels were analyzed and applicable dimensions were recorded for use in the following equations. The following two equations were used to determine design column moments. Equation 1 below yields the total unbalanced moment generated at the slab-column joint. Some of the moment is resisted by the slab; however, most of the moment, represented by equation 2, is transferred to the columns above and below the slab. Note, sample calculations are shown in Appendix C.

Governing Equations For Interior Columns:

Equation 1: $$M_{JOINT} = .65\left\{(q_{DU} + .5q_{LU})(L_2)({L_N}^2)\left(\frac{1}{8}\right) - (q'_{DU})(L'_2)(L'_N)^2\left(\frac{1}{8}\right)\right\}$$

Equation 2: $$M_{COLUMN} = .07\{(q_{DU} + .5q_{LU})(L_2)({L_N}^2) - (q'_{DU})(L'_2)(L'_N)^2\}$$

For each exterior column at every floor, the adjacent slab panels were analyzed and applicable dimensions were recorded for use in the following equation. Equation 3 below yields the approximate portion of negative exterior moment carried by the exterior column.

Equation 3: $$M_{COLUMN} = .30\,M_0 = .30\left\{(q_{DU} + q_{LU})(L_2)({L_N}^2)\left(\frac{1}{8}\right)\right\}$$

Next, the column moments were distributed to the above and below columns according to flexural stiffness. Column flexural stiffness were calculated as 4EI/L where$E = 57000\sqrt{E_c}$. Equation 4 and 5 show how the moments were divided between the columns.

Equation 4: $$M_{COLUMN\ ABOVE} = M_{COLUMN}\left\{\frac{K_{C\ ABOVE}}{K_{C\ ABOVE} + K_{C\ BELOW}}\right\}$$

Equation 5: $$M_{COLUMN\ BELOW} = M_{COLUMN}\left\{\frac{K_{C\ BELOW}}{K_{C\ ABOVE} + K_{C\ BELOW}}\right\}$$

Unbalanced Moment Transferred To Interior Columns At Each Slab:

This section contains a set of tables summarizing the calculation process and results pertaining to the interior column moments. The yellow highlighted cells represent the top and bottom design moments. Hand calculations were performed to both verify the process and demonstrate the procedure. Hand calculations, which can be found in Appendix C, correspond to the red outlined cells in the following tables.

FLOOR 10 - 13 - Interior Column Moments

Column	q_{DU} (ksf)	q_{LU} (ksf)	L_2 (ft)	L_n (ft)	q'_{DU} (KSF)	L'_2 (ft)	L'_N (ft)	M_{JOINT} (K-ft)	M_{COLUMN} (K-ft)	$K_{COLUMN\ ABOVE}$ (4EI/L)	$K_{COLUMN\ BELOW}$ (4EI/L)	$M_{C\ ABOVE}$ (K-ft)	$M_{C\ BELOW}$ (K-ft)
4 - B	0.0912	0.064	15	18.75	0.0912	15	13.75	31.773	27.374	615691839	615691839	13.687	13.687
4 - C	0.0912	0.064	16	20.75	0.0912	16	14.75	43.165	37.188	615691839	615691839	18.594	18.594
4 - D.4	0.0912	0.064	18	31.75	0.0912	18	20.75	124.204	107.007	615691839	615691839	53.503	53.503
4 - F	0.0912	0.064	18	21.75	0.0912	18	15.75	52.150	44.929	615691839	615691839	22.465	22.465
6 - B.1	0.0912	0.064	13	16.75	0.0912	13	8.75	29.134	25.100	615691839	615691839	12.550	12.550
6 - C	0.0912	0.064	12	31.75	0.0912	12	11.75	108.812	93.746	615691839	615691839	46.873	46.873
6 - F	0.0912	0.064	12	31.75	0.0912	12	8.75	114.281	98.457	615691839	615691839	49.229	49.229

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FLOOR 3 -9 - Interior Column Moments

Column	q_{DU} (ksf)	q_{LU} (ksf)	L_2 (ft)	L_n (ft)	q'_{DU} (KSF)	L'_2 (ft)	L'_N	M_{JOINT} (K-ft)	M_{COLUMN} (K-ft)	$K_{COLUMN\ ABOVE}$ (4EI/L)	$K_{COLUMN\ BELOW}$ (4EI/L)	$M_{C\ ABOVE}$ (K-ft)	$M_{C\ BELOW}$ (K-ft)
4 - B	0.0912	0.064	15	18.75	0.0912	15	13.75	31.773	27.374	615691839	615691839	13.687	13.687
4 - C	0.0912	0.064	16	20.75	0.0912	16	14.75	43.165	37.188	615691839	615691839	18.594	18.594
4 - D.4	0.0912	0.064	18	20.75	0.0912	18	13.75	52.362	45.112	615691839	615691839	22.556	22.556
4 - F	0.0912	0.064	18	21.75	0.0912	18	15.75	52.150	44.929	615691839	615691839	22.465	22.465
6 - B.1	0.0912	0.064	13	16.75	0.0912	13	8.75	29.134	25.100	615691839	615691839	12.550	12.550
6 - C	0.0912	0.064	12	15.75	0.0912	12	11.75	17.521	15.095	615691839	615691839	7.547	7.547
6 - D.4	0.0912	0.064	12	14.75	0.0912	12	8.75	19.326	16.650	615691839	615691839	8.325	8.325
6 - F	0.0912	0.064	14	15.75	0.0912	8	8.75	30.225	26.040	615691839	615691839	13.020	13.020

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FLOOR 2 - Interior Column Moments

Column	q_{DU} (ksf)	q_{LU} (ksf)	L_2 (ft)	L_n (ft)	q'_{DU} (KSF)	L'_2 (ft)	L'_N	M_{JOINT} (K-ft)	M_{COLUMN} (K-ft)	$K_{COLUMN\ ABOVE}$ (4EI/L)	$K_{COLUMN\ BELOW}$ (4EI/L)	$M_{C\ ABOVE}$ (K-ft)	$M_{C\ BELOW}$ (K-ft)
4 - B	0.1272	0.064	15	18.75	0.1272	15	13.75	38.903	33.516	615691839	293186590	22.704	10.812
4 - C	0.1272	0.064	16	20.75	0.1272	16	14.75	53.133	45.776	615691839	293186590	31.010	14.767
4 - D.4	0.1272	0.064	18	20.75	0.1272	18	13.75	65.077	56.066	615691839	293186590	37.980	18.086
4 - F	0.1272	0.064	18	21.75	0.1272	18	15.75	63.996	55.135	615691839	293186590	37.350	17.786
6 - B.1	0.1272	0.064	13	16.75	0.1272	13	8.75	36.891	31.783	615691839	293186590	21.531	10.253
6 - C	0.1272	0.064	12	15.75	0.1272	12	11.75	21.382	18.421	615691839	293186590	12.479	5.942
6 - D.4	0.1272	0.064	12	14.75	0.1272	12	8.75	24.275	20.914	615691839	293186590	14.167	6.746
6 - F	0.1272	0.064	14	15.75	0.1272	8	8.75	38.591	33.248	615691839	293186590	22.523	10.725
8 - B	0.1272	0.064	13	10.75	0.1272	13	3.75	17.543	15.114	615691839	293186590	10.239	4.875
8 - C	0.1272	0.064	8	16.75	0.1272	7	11.75	19.044	16.408	615691839	293186590	11.115	5.293
8 - D.4	0.1272	0.064	16	9.75	0.1272	16	3.75	17.349	14.947	615691839	293186590	10.125	4.821
8 - F	0.1272	0.064	10	15.75	0.1272	10	9.75	22.262	19.180	615691839	293186590	12.993	6.187

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Unbalanced Moment Transferred To Interior Columns At Each Slab Continued:

FLOOR 1 - Interior Column Moments													
Column	q_{DU} (ksf)	q_{LU} (ksf)	L_2 (ft)	L_n (ft)	q'_{DU} (KSF)	L'_2 (ft)	L'_N	M_{JOINT} (K-ft)	M_{COLUMN} (K-ft)	$K_{COLUMN\ ABOVE}$ (4EI/L)	$K_{COLUMN\ BELOW}$ (4EI/L)	$M_{C\ ABOVE}$ (K-ft)	$M_{C\ BELOW}$ (K-ft)
4 - B	0.1272	0.16	15	18.75	0.1272	15	13.75	59.469	51.235	293186590	615691839	16.527	34.707
4 - C	0.1272	0.16	16	20.75	0.1272	16	14.75	80.000	68.923	293186590	615691839	22.233	46.690
4 - D.4	0.1272	0.16	18	20.75	0.1272	18	13.75	95.302	82.106	293186590	615691839	26.486	55.620
4 - F	0.1272	0.16	18	21.75	0.1272	18	15.75	97.205	83.746	293186590	615691839	27.015	56.731
6 - B.1	0.1272	0.16	13	16.75	0.1272	13	8.75	51.116	44.038	293186590	615691839	14.206	29.832
6 - C	0.1272	0.16	12	15.75	0.1272	12	11.75	32.991	28.423	293186590	615691839	9.169	19.254
6 - D.4	0.1272	0.16	12	14.75	0.1272	12	8.75	34.457	29.686	293186590	615691839	9.576	20.110
6 - F	0.1272	0.16	14	15.75	0.1272	8	8.75	52.136	44.917	293186590	615691839	14.489	30.428
8 - B	0.1272	0.16	13	10.75	0.1272	13	3.75	23.402	20.162	293186590	615691839	6.504	13.658
8 - C	0.1272	0.16	8	16.75	0.1272	7	11.75	27.798	23.949	293186590	615691839	7.726	16.224
8 - D.4	0.1272	0.16	16	9.75	0.1272	16	3.75	23.281	20.057	293186590	615691839	6.470	13.587
8 - F	0.1272	0.16	10	15.75	0.1272	10	9.75	31.937	27.515	293186590	615691839	8.876	18.639

F-64

Unbalanced Moment Transferred To Exterior Columns At Each Slab:

This section contains a set of tables summarizing the calculation process and results pertaining to the exterior column moments. The yellow highlighted cells represent the top and bottom design moments. Hand calculations were performed to both verify the process and demonstrate the procedure. Hand calculations, which can be found in Appendix C, correspond to the **red** outlined cells in the following tables.

F-65

FLOOR 10 - 13 - Exterior Column Moments

Column	W_{DU} (ksf)	W_{LU} (ksf)	L_2 (ft)	L_N (ft)	M_O (k-ft)	$.3M_O$ (k-ft)	$K_{COLUMN\ ABOVE}$ (4EI/L)	$K_{COLUMN\ BELOW}$ (4EI/L)	$M_{C\ ABOVE}$ (K-ft)	$M_{C\ BELOW}$ (K-ft)
1 - B	0.0912	0.064	15	20.75	125.2937	37.58811	615691839	615691839	18.794053	18.79405313
1 - C.3	0.0912	0.064	15	20.75	125.2937	37.58811	615691839	615691839	18.794053	18.79405313
1 - D.4	0.0912	0.064	15	20.75	125.2937	37.58811	615691839	615691839	18.794053	18.79405313
1 - F	0.0912	0.064	14	20.75	116.9408	35.08223	615691839	615691839	17.541116	17.54111625
1 - F.9	0.0912	0.064	7	10.75	15.69339	4.708016	615691839	615691839	2.3540081	2.354008125
2 - A.2	0.0912	0.064	6	13.75	22.00688	6.602063	615691839	615691839	3.3010313	3.30103125
4 - A.2	0.0912	0.064	15	15.75	72.18619	21.65586	615691839	615691839	10.827928	10.82792813
4 - F.9	0.0912	0.064	17	10.25	34.64961	10.39488	615691839	615691839	5.1974419	5.197441875
6 - A.2	0.0912	0.064	12	15.75	57.74895	17.32469	615691839	615691839	8.6623425	8.6623425
7 - F.9	0.0912	0.064	5	20.75	41.76456	12.52937	615691839	615691839	6.2646844	6.264684375
8 - A.2	0.0912	0.064	6	13.75	22.00688	6.602063	615691839	615691839	3.3010313	3.30103125
8 - B	0.0912	0.064	13	9	20.4282	6.12846	615691839	615691839	3.06423	3.06423
8 - C	0.0912	0.064	13	9	20.4282	6.12846	615691839	615691839	3.06423	3.06423
8 - D.4	0.0912	0.064	16	23.75	175.085	52.5255	615691839	615691839	26.26275	26.26275
8 - F	0.0912	0.064	8	9	12.5712	3.77136	615691839	615691839	1.88568	1.88568

F-66

FLOOR 3 - 9 - Exterior Column Moments

Column	W_{DU} (ksf)	W_{LU} (ksf)	L_2 (ft)	L_N (ft)	M_O (k-ft)	$.3M_O$ (k-ft)	$K_{COLUMN\ ABOVE}$ (4EI/L)	$K_{COLUMN\ BELOW}$ (4EI/L)	$M_{C\ ABOVE}$ (K-ft)	$M_{C\ BELOW}$ (K-ft)
1 - B	0.0912	0.064	15	20.75	125.2937	37.58811	615691839	615691839	18.794053	18.79405313
1 - C.3	0.0912	0.064	15	20.75	125.2937	37.58811	615691839	615691839	18.794053	18.79405313
1 - D.4	0.0912	0.064	15	20.75	125.2937	37.58811	615691839	615691839	18.794053	18.79405313
1 - F	0.0912	0.064	14	20.75	116.9408	35.08223	615691839	615691839	17.541116	17.54111625
1 - F.9	0.0912	0.064	7	10.75	15.69339	4.708016	615691839	615691839	2.3540081	2.354008125
2 - A.2	0.0912	0.064	6	13.75	22.00688	6.602063	615691839	615691839	3.3010313	3.30103125
4 - A.2	0.0912	0.064	15	15.75	72.18619	21.65586	615691839	615691839	10.827928	10.82792813
4 - F.9	0.0912	0.064	17	10.25	34.64961	10.39488	615691839	615691839	5.1974419	5.197441875
6 - A.2	0.0912	0.064	12	15.75	57.74895	17.32469	615691839	615691839	8.6623425	8.6623425
7 - F.9	0.0912	0.064	5	20.75	41.76456	12.52937	615691839	615691839	6.2646844	6.264684375
8 - A.2	0.0912	0.064	6	13.75	22.00688	6.602063	615691839	615691839	3.3010313	3.30103125
8 - B	0.0912	0.064	13	9	20.4282	6.12846	615691839	615691839	3.06423	3.06423
8 - C	0.0912	0.064	13	9	20.4282	6.12846	615691839	615691839	3.06423	3.06423
8 - D.4	0.0912	0.064	16	9	25.1424	7.54272	615691839	615691839	3.77136	3.77136
8 - F	0.0912	0.064	8	9	12.5712	3.77136	615691839	615691839	1.88568	1.88568

Unbalanced Moment Transferred To Exterior Columns At Each Slab Continued:

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FLOOR 2 - Exterior Column Moments

Column	W_{DU} (ksf)	W_{LU} (ksf)	L_2 (ft)	L_N (ft)	M_O (k-ft)	$.3M_O$ (k-ft)	$K_{COLUMN\ ABOVE}$ (4EI/L)	$K_{COLUMN\ BELOW}$ (4EI/L)	$M_{C\ ABOVE}$ (K-ft)	$M_{C\ BELOW}$ (K-ft)
1 - B	0.1272	0.064	15	20.75	154.3567	46.307	615691839	293186590	31.369256	14.93774093
1 - C.3	0.1272	0.064	15	20.75	154.3567	46.307	615691839	293186590	31.369256	14.93774093
1 - D.4	0.1272	0.064	15	20.75	154.3567	46.307	615691839	293186590	31.369256	14.93774093
1 - F	0.1272	0.064	14	20.75	144.0662	43.21986	615691839	293186590	29.277972	13.94189154
1 - F.9	0.1272	0.064	7	10.75	19.33361	5.800082	615691839	293186590	3.9290877	1.870994154
2 - A.2	0.1272	0.064	6	13.75	27.11156	8.133469	615691839	293186590	5.5097692	2.623699597
4 - A.2	0.1272	0.064	15	15.75	88.93041	26.67912	615691839	293186590	18.072954	8.606168349
4 - F.9	0.1272	0.064	17	10.25	42.68689	12.80607	615691839	293186590	8.6750784	4.130989719
6 - A.2	0.1272	0.064	12	15.75	71.14433	21.3433	615691839	293186590	14.458363	6.884934679
7 - F.9	0.1272	0.112	20	9.75	56.84738	17.05421	615691839	293186590	11.552854	5.501358872
8 - A.2	0.1272	0.112	7	13.75	39.57078	11.87123	615691839	293186590	8.0418039	3.829430444

FLOOR 1 - Exterior Column Moments

Column	W_{DU} (ksf)	W_{LU} (ksf)	L_2 (ft)	L_N (ft)	M_O (k-ft)	$.3M_O$ (k-ft)	$K_{COLUMN\ ABOVE}$ (4EI/L)	$K_{COLUMN\ BELOW}$ (4EI/L)	$M_{C\ ABOVE}$ (K-ft)	$M_{C\ BELOW}$ (K-ft)
1 - B	0.1272	0.16	15	20.75	231.8579	69.55737	293186590	615691839	22.437862	47.11950998
1 - C.3	0.1272	0.16	15	20.75	231.8579	69.55737	293186590	615691839	22.437862	47.11950998
1 - D.4	0.1272	0.16	15	20.75	231.8579	69.55737	293186590	615691839	22.437862	47.11950998
1 - F	0.1272	0.16	14	20.75	216.4007	64.92021	293186590	615691839	20.942004	43.97820931
1 - F.9	0.1272	0.16	7	10.75	29.04086	8.712257	293186590	615691839	2.8104054	5.901851431
2 - A.2	0.1272	0.16	6	13.75	40.72406	12.21722	293186590	615691839	3.9410383	8.276180443
4 - A.2	0.1272	0.16	15	15.75	133.5817	40.0745	293186590	615691839	12.927257	27.14723982
4 - F.9	0.1272	0.16	17	10.25	64.11964	19.23589	293186590	615691839	6.2051268	13.03076631
6 - A.2	0.1272	0.16	12	15.75	106.8653	32.0596	293186590	615691839	10.341806	21.71779185
7 - F.9	0.1272	0.16	20	9.75	68.25488	20.47646	293186590	615691839	6.6053105	13.87115201
8 - A.2	0.1272	0.16	7	13.75	47.51141	14.25342	293186590	615691839	4.597878	9.65554385

F-68

Factored Design Axial and Flexural Column Loads (summary tables):

The following section of tables displays the factored axial and flexural loads on every column in the structure. The last column of each table contains a "column design specification letter" for each column. 6 Different reinforced concrete column sections (A – F) were designed. The corresponding column sections can be found on page 78.

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Floor 13 - Column Factored Design Loads

Column	Height	P_U = 1.2PD + 1.6 PL (kips)	$M_{U,TOP}$ (ft-k)	$M_{U,BOTTOM}$ (ft-k)	Column Design
1 - B.1	10' - 6"	13.69	18.794	18.794	A
1 - C.3	10' - 6"	14.53	18.794	18.794	A
1 - D.4	10' - 6"	30.95	18.794	18.794	B
1 - F	10' - 6"	24.21	17.54	17.54	B
1 - F.9	10' - 6"	9.31	2.35	2.35	B
2 - A.2	10' - 6"	14.90	3.3	3.3	B
4 - A.2	10' - 6"	17.20	10.828	10.828	B
4 - B.1	10' - 6"	19.82	13.687	13.687	B
4 - C	10' - 6"	15.86	18.594	18.594	A
4 -D.4	10' - 6"	59.02	53.503	53.503	C
4 -F	10' - 6"	36.07	22.465	22.465	A
4 - F.9	10' - 6"	14.67	5.197	5.197	B
6 - A.2	10' - 6"	27.94	8.66	8.66	B
6 - B.1	10' - 6"	18.03	12.55	12.55	B
6 - C	10' - 6"	57.04	46.873	46.873	C
6 - F	10' - 6"	37.90	49.229	49.229	F
7 - F.9	10' - 6"	7.76	6.264	6.264	B
8 - A.2	10' - 6"	5.04	3.3	3.3	B
8 - B	10' - 6"	13.58	3.064	3.064	B
8 - C	10' - 6"	11.64	3.064	3.064	B
8 - D.4	10' - 6"	26.86	26.26	26.26	A
8 - F	10' - 6"	6.21	26.26	26.26	C

F-70

Floor 12 - Column Factored Design Loads

Column	Height	P_U = 1.2PD + 1.6 PL (kips)	$M_{U,TOP}$ (ft-k)	$M_{U,BOTTOM}$ (ft-k)	Column Design
1 - B.1	10' - 6"	27.38	18.794	18.794	B
1 - C.3	10' - 6"	29.05	18.794	18.794	B
1 - D.4	10' - 6"	61.90	18.794	18.794	B
1 - F	10' - 6"	48.42	17.54	17.54	B
1 - F.9	10' - 6"	18.62	2.35	2.35	B
2 - A.2	10' - 6"	29.80	3.3	3.3	B
4 - A.2	10' - 6"	34.41	10.828	10.828	B
4 - B.1	10' - 6"	39.64	13.687	13.687	B
4 - C	10' - 6"	31.71	18.594	18.594	B
4 -D.4	10' - 6"	118.05	53.503	53.503	
4 -F	10' - 6"	72.14	22.465	22.465	B
4 - F.9	10' - 6"	29.33	5.197	5.197	B
6 - A.2	10' - 6"	55.87	8.66	8.66	B
6 - B.1	10' - 6"	36.07	12.55	12.55	B
6 - C	10' - 6"	114.08	46.873	46.873	C
6 - F	10' - 6"	75.80	49.229	49.229	C
7 - F.9	10' - 6"	15.52	6.264	6.264	B
8 - A.2	10' - 6"	10.09	3.3	3.3	B
8 - B	10' - 6"	27.16	3.064	3.064	B
8 - C	10' - 6"	23.28	3.064	3.064	B
8 - D.4	10' - 6"	53.72	26.26	26.26	B
8 - F	10' - 6"	12.42	26.26	26.26	C

Floor 11 - Column Factored Design Loads

Column	Height	P_U = 1.2PD + 1.6 PL (kips)	$M_{U,TOP}$ (ft-k)	$M_{U,BOTTOM}$ (ft-k)	Column Design
1 - B.1	10' - 6"	41.06592	18.794	18.794	B
1 - C.3	10' - 6"	43.58016	18.794	18.794	B
1 - D.4	10' - 6"	92.8512	18.794	18.794	B
1 - F	10' - 6"	72.6336	17.54	17.54	B
1 - F.9	10' - 6"	27.936	2.35	2.35	B
2 - A.2	10' - 6"	44.6976	3.3	3.3	B
4 - A.2	10' - 6"	51.61128	10.828	10.828	B
4 - B.1	10' - 6"	59.46	13.687	13.687	B
4 - C	10' - 6"	47.568	18.594	18.594	B
4 -D.4	10' - 6"	177.072	53.503	53.503	C
4 -F	10' - 6"	108.2172	22.465	22.465	B
4 - F.9	10' - 6"	44.0004	5.197	5.197	B
6 - A.2	10' - 6"	83.808	8.66	8.66	B
6 - B.1	10' - 6"	54.1008	12.55	12.55	B
6 - C	10' - 6"	171.12	46.873	46.873	C
6 - F	10' - 6"	113.6928	49.229	49.229	C
7 - F.9	10' - 6"	23.28	6.264	6.264	B
8 - A.2	10' - 6"	15.132	3.3	3.3	B
8 - B	10' - 6"	40.74	3.064	3.064	B
8 - C	10' - 6"	34.92	3.064	3.064	B
8 - D.4	10' - 6"	80.586	26.26	26.26	A
8 - F	10' - 6"	18.624	26.26	26.26	C

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Floor 10 - Column Factored Design Loads

Column	Height	P_U = 1.2PD + 1.6 PL (kips)	$M_{U,TOP}$ (ft-k)	$M_{U,BOTTOM}$ (ft-k)	Column Design
1 - B.1	10' - 6"	54.75456	18.794	18.794	B
1 - C.3	10' - 6"	58.10688	18.794	18.794	B
1 - D.4	10' - 6"	123.8016	18.794	18.794	B
1 - F	10' - 6"	96.8448	17.54	17.54	B
1 - F.9	10' - 6"	37.248	2.35	2.354	B
2 - A.2	10' - 6"	59.5968	3.3	3.301	B
4 - A.2	10' - 6"	68.81504	10.828	10.828	B
4 - B.1	10' - 6"	79.28	13.687	13.687	B
4 - C	10' - 6"	63.424	18.594	18.594	B
4 -D.4	10' - 6"	236.096	53.503	22.556	D
4 -F	10' - 6"	144.2896	22.465	22.465	B
4 - F.9	10' - 6"	58.6672	5.197	5.197	B
6 - A.2	10' - 6"	111.744	8.66	8.662	B
6 - B.1	10' - 6"	72.1344	12.55	12.55	B
6 - C	10' - 6"	228.16	46.873	7.547	C
6 - F	10' - 6"	151.5904	49.229	13.02	C
7 - F.9	10' - 6"	31.04	6.264	6.264	B
8 - A.2	10' - 6"	20.176	3.3	3.301	B
8 - B	10' - 6"	54.32	3.064	3.064	B
8 - C	10' - 6"	46.56	3.064	3.064	B
8 - D.4	10' - 6"	107.448	26.26	3.771	A
8 - F	10' - 6"	24.832	26.26	1.886	C

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Factored Design Axial and Flexural Column Loads (summary tables) Continued:

F-73

Floor 9 - Column Factored Design Loads					
Column	Height	P_U = 1.2PD + 1.6 PL (kips)	$M_{U,\,TOP}$ (ft-k)	$M_{U,\,BOTTOM}$ (ft-k)	Column Design
1 - B.1	10' - 6"	68.4432	18.794	18.794	B
1 - C.3	10' - 6"	72.6336	18.794	18.794	B
1 - D.4	10' - 6"	148.0128	18.794	18.794	B
1 - F	10' - 6"	121.056	17.54	17.54	B
1 - F.9	10' - 6"	46.56	2.354	2.354	B
2 - A.2	10' - 6"	74.496	3.301	3.301	B
4 - A.2	10' - 6"	85.65424	10.828	10.828	B
4 - B.1	10' - 6"	98.68	13.687	13.687	B
4 - C	10' - 6"	78.944	18.594	18.594	B
4 -D.4	10' - 6"	274.896	22.556	22.556	D
4 -F	10' - 6"	179.5976	22.465	22.465	B
4 - F.9	10' - 6"	73.0232	5.197	5.197	B
6 - A.2	10' - 6"	139.68	8.662	8.662	B
6 - B.1	10' - 6"	80.0496	12.55	12.55	B
6 - C	10' - 6"	252.992	7.547	7.547	C
6 - D.4	10' - 6"	36.9376	8.325	8.325	B
6 - F	10' - 6"	177.8192	13.02	13.02	B
7 - F.9	10' - 6"	38.8	6.264	6.264	B
8 - A.2	10' - 6"	25.22	3.301	3.301	B
8 - B	10' - 6"	67.9	3.064	3.064	B
8 - C	10' - 6"	58.2	3.064	3.064	B
8 - D.4	10' - 6"	118.312	3.77	3.77	B
8 - F	10' - 6"	31.04	1.886	1.886	C

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Floor 8 - Column Factored Design Loads					
Column	Height	P_U = 1.2PD + 1.6 PL (kips)	$M_{U,\,TOP}$ (ft-k)	$M_{U,\,BOTTOM}$ (ft-k)	Column Design
1 - B.1	10' - 6"	82.13184	18.794	18.794	B
1 - C.3	10' - 6"	87.16032	18.794	18.794	B
1 - D.4	10' - 6"	172.224	18.794	18.794	B
1 - F	10' - 6"	145.2672	17.54	17.54	B
1 - F.9	10' - 6"	55.872	2.354	2.354	B
2 - A.2	10' - 6"	89.3952	3.301	3.301	B
4 - A.2	10' - 6"	102.49344	10.828	10.828	B
4 - B.1	10' - 6"	118.08	13.687	13.687	B
4 - C	10' - 6"	94.464	18.594	18.594	B
4 -D.4	10' - 6"	313.696	22.556	22.556	F
4 -F	10' - 6"	214.9056	22.465	22.465	C
4 - F.9	10' - 6"	87.3792	5.197	5.197	B
6 - A.2	10' - 6"	167.616	8.662	8.662	B
6 - B.1	10' - 6"	87.9648	12.55	12.55	B
6 - C	10' - 6"	277.824	7.547	7.547	C
6 - D.4	10' - 6"	73.8752	8.325	8.325	B
6 - F	10' - 6"	204.048	13.02	13.02	C
7 - F.9	10' - 6"	46.56	6.264	6.264	B
8 - A.2	10' - 6"	30.264	3.301	3.301	B
8 - B	10' - 6"	81.48	3.064	3.064	B
8 - C	10' - 6"	69.84	3.064	3.064	B
8 - D.4	10' - 6"	129.176	3.77	3.77	B
8 - F	10' - 6"	37.248	1.886	1.886	C

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Floor 7 - Column Factored Design Loads					
Column	Height	P_U = 1.2PD + 1.6 PL (kips)	$M_{U,\,TOP}$ (ft-k)	$M_{U,\,BOTTOM}$ (ft-k)	Column Design
1 - B.1	10' - 6"	95.82048	18.794	18.794	B
1 - C.3	10' - 6"	101.68704	18.794	18.794	B
1 - D.4	10' - 6"	196.4352	18.794	18.794	B
1 - F	10' - 6"	169.4784	17.54	17.54	B
1 - F.9	10' - 6"	65.184	2.354	2.354	B
2 - A.2	10' - 6"	104.2944	3.301	3.301	B
4 - A.2	10' - 6"	119.33264	10.828	10.828	B
4 - B.1	10' - 6"	137.48	13.687	13.687	B
4 - C	10' - 6"	109.984	18.594	18.594	B
4 -D.4	10' - 6"	352.496	22.556	22.556	D
4 -F	10' - 6"	250.2136	22.465	22.465	C
4 - F.9	10' - 6"	101.7352	5.197	5.197	B
6 - A.2	10' - 6"	195.552	8.662	8.662	B
6 - B.1	10' - 6"	95.88	12.55	12.55	B
6 - C	10' - 6"	302.656	7.547	7.547	C
6 - D.4	10' - 6"	110.8128	8.325	8.325	B
6 - F	10' - 6"	230.2768	13.02	13.02	C
7 - F.9	10' - 6"	54.32	6.264	6.264	B
8 - A.2	10' - 6"	35.308	3.301	3.301	B
8 - B	10' - 6"	95.06	3.064	3.064	B
8 - C	10' - 6"	81.48	3.064	3.064	B
8 - D.4	10' - 6"	140.04	3.77	3.77	B
8 - F	10' - 6"	43.456	1.886	1.886	C

F-76

Floor 6 - Column Factored Design Loads					
Column	Height	P_U = 1.2PD + 1.6 PL (kips)	$M_{U,\,TOP}$ (ft-k)	$M_{U,\,BOTTOM}$ (ft-k)	Column Design
1 - B.1	10' - 6"	109.50912	18.794	18.794	B
1 - C.3	10' - 6"	116.21376	18.794	18.794	B
1 - D.4	10' - 6"	220.6464	18.794	18.794	C
1 - F	10' - 6"	193.6896	17.54	17.54	B
1 - F.9	10' - 6"	74.496	2.354	2.354	B
2 - A.2	10' - 6"	119.1936	3.301	3.301	B
4 - A.2	10' - 6"	136.17184	10.828	10.828	B
4 - B.1	10' - 6"	156.88	13.687	13.687	B
4 - C	10' - 6"	125.504	18.594	18.594	B
4 -D.4	10' - 6"	391.296	22.556	22.556	D
4 -F	10' - 6"	285.5216	22.465	22.465	C
4 - F.9	10' - 6"	116.0912	5.197	5.197	B
6 - A.2	10' - 6"	223.488	8.662	8.662	C
6 - B.1	10' - 6"	103.7952	12.55	12.55	B
6 - C	10' - 6"	327.488	7.547	7.547	F
6 - D.4	10' - 6"	147.7504	8.325	8.325	B
6 - F	10' - 6"	256.5056	13.02	13.02	C
7 - F.9	10' - 6"	62.08	6.264	6.264	B
8 - A.2	10' - 6"	40.352	3.301	3.301	B
8 - B	10' - 6"	108.64	3.064	3.064	B
8 - C	10' - 6"	93.12	3.064	3.064	B
8 - D.4	10' - 6"	150.904	3.77	3.77	B
8 - F	10' - 6"	49.664	1.886	1.886	C

Factored Design Axial and Flexural Column Loads (summary tables) Continued:

F-77

Floor 5 - Column Factored Design Loads

Column	Height	P_U = 1.2PD + 1.6 PL (kips)	$M_{U,TOP}$ (ft-k)	$M_{U,BOTTOM}$ (ft-k)	Column Design
1 - B.1	10' - 6"	123.19776	18.794	18.794	B
1 - C.3	10' - 6"	130.74048	18.794	18.794	B
1 - D.4	10' - 6"	244.8576	18.794	18.794	C
1 - F	10' - 6"	217.9008	17.54	17.54	C
1 - F.9	10' - 6"	83.808	2.354	2.354	B
2 - A.2	10' - 6"	134.0928	3.301	3.301	B
4 - A.2	10' - 6"	153.01104	10.828	10.828	B
4 - B.1	10' - 6"	176.28	13.687	13.687	B
4 - C	10' - 6"	141.024	18.594	18.594	B
4 -D.4	10' - 6"	430.096	22.556	22.556	D
4 -F	10' - 6"	320.8296	22.465	22.465	F
4 - F.9	10' - 6"	130.4472	5.197	5.197	B
6 - A.2	10' - 6"	251.424	8.662	8.662	C
6 - B.1	10' - 6"	111.7104	12.55	12.55	B
6 - C	10' - 6"	352.32	7.547	7.547	F
6 - D.4	10' - 6"	184.688	8.325	8.325	B
6 - F	10' - 6"	282.7344	13.02	13.02	C
7 - F.9	10' - 6"	69.84	6.264	6.264	B
8 - A.2	10' - 6"	45.396	3.301	3.301	B
8 - B	10' - 6"	122.22	3.064	3.064	B
8 - C	10' - 6"	104.76	3.064	3.064	B
8 - D.4	10' - 6"	161.768	3.77	3.77	B
8 - F	10' - 6"	55.872	1.886	1.886	C

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Floor 4 - Column Factored Design Loads

Column	Height	P_U = 1.2PD + 1.6 PL (kips)	$M_{U,TOP}$ (ft-k)	$M_{U,BOTTOM}$ (ft-k)	Column Design
1 - B.1	10' - 6"	136.8864	18.794	18.794	B
1 - C.3	10' - 6"	145.2672	18.794	18.794	B
1 - D.4	10' - 6"	269.0688	18.794	18.794	C
1 - F	10' - 6"	242.112	17.54	17.54	C
1 - F.9	10' - 6"	93.12	2.354	2.354	B
2 - A.2	10' - 6"	148.992	3.301	3.301	B
4 - A.2	10' - 6"	169.85024	10.828	10.828	B
4 - B.1	10' - 6"	195.68	13.687	13.687	B
4 - C	10' - 6"	156.544	18.594	18.594	B
4 -D.4	10' - 6"	468.896	22.556	22.556	D
4 -F	10' - 6"	356.1376	22.465	22.465	F
4 - F.9	10' - 6"	144.8032	5.197	5.197	B
6 - A.2	10' - 6"	279.36	8.662	8.662	C
6 - B.1	10' - 6"	119.6256	12.55	12.55	B
6 - C	10' - 6"	377.152	7.547	7.547	F
6 - D.4	10' - 6"	221.6256	8.325	8.325	C
6 - F	10' - 6"	308.9632	13.02	13.02	F
7 - F.9	10' - 6"	77.6	6.264	6.264	B
8 - A.2	10' - 6"	50.44	3.301	3.301	B
8 - B	10' - 6"	135.8	3.064	3.064	B
8 - C	10' - 6"	116.4	3.064	3.064	B
8 - D.4	10' - 6"	172.632	3.77	3.77	B
8 - F	10' - 6"	62.08	1.886	1.886	C

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Floor 3 - Column Factored Design Loads

Column	Height	P_U = 1.2PD + 1.6 PL (kips)	$M_{U,TOP}$ (ft-k)	$M_{U,BOTTOM}$ (ft-k)	Column Design
1 - B.1	10' - 6"	150.57504	18.794	31.369	B
1 - C.3	10' - 6"	159.79392	18.794	31.369	B
1 - D.4	10' - 6"	293.28	18.794	31.369	C
1 - F	10' - 6"	266.3232	17.54	29.278	C
1 - F.9	10' - 6"	102.432	2.354	3.929	B
2 - A.2	10' - 6"	163.8912	3.301	5.509	B
4 - A.2	10' - 6"	186.68944	10.828	18.07	B
4 - B.1	10' - 6"	215.08	13.687	22.704	C
4 - C	10' - 6"	172.064	18.594	31.01	B
4 -D.4	10' - 6"	507.696	22.556	37.98	E
4 -F	10' - 6"	391.4456	22.465	37.35	F
4 - F.9	10' - 6"	159.1592	5.197	8.675	B
6 - A.2	10' - 6"	307.296	8.662	14.458	F
6 - B.1	10' - 6"	127.5408	12.55	21.531	B
6 - C	10' - 6"	401.984	7.547	12.479	D
6 - D.4	10' - 6"	258.5632	8.325	14.167	C
6 - F	10' - 6"	335.192	13.02	22.523	F
7 - F.9	10' - 6"	85.36	6.264	11.55	B
8 - A.2	10' - 6"	55.484	3.301	8.042	B
8 - B	10' - 6"	149.38	3.064	10.239	B
8 - C	10' - 6"	128.04	3.064	11.115	B
8 - D.4	10' - 6"	183.496	3.77	10.125	B
8 - F	10' - 6"	68.288	1.886	12.993	C

Factored Design Axial and Flexural Column Loads (summary tables) Continued:

Floor 2 - Column Factored Design Loads					
Column	Height	P_U = 1.2PD + 1.6 PL (kips)	$M_{U,TOP}$ (ft-k)	$M_{U,BOTTOM}$ (ft-k)	Column Design
1 - B.1	21' - 0"	172.61504	14.937	22.437	B
1 - C.3	21' - 0"	177.99232	14.937	22.437	A
1 - D.4	21' - 0"	317.4912	14.937	22.437	F
1 - F	21' - 0"	290.5344	13.942	20.94	C
1 - F.9	21' - 0"	111.9456	1.871	2.81	B
2 - A.2	21' - 0"	182.4768	2.623	8.276	B
4 - A.2	21' - 0"	203.52864	8.606	27.147	A
4 - B.1	21' - 0"	234.48	10.812	16.527	C
4 - C	21' - 0"	187.584	14.767	22.233	C
4 -D.4	21' - 0"	553.868	18.086	26.486	E
4 -F	21' - 0"	426.7536	17.786	27.015	D
4 - F.9	21' - 0"	173.826	4.123	6.205	B
6 - A.2	21' - 0"	335.232	6.889	10.34	F
6 - B.1	21' - 0"	143.3712	10.253	14.206	B
6 - C	21' - 0"	430.734	5.942	9.169	D
6 - D.4	21' - 0"	295.5008	6.746	9.576	C
6 - F	21' - 0"	375.4608	10.725	14.489	F
7 - F.9	21' - 0"	123.18032	5.503	6.605	B
8 - A.2	21' - 0"	64.0184	3.823	4.597	B
8 - B	21' - 0"	165.2968	4.875	6.504	B
8 - C	21' - 0"	143.9568	5.293	7.726	B
8 - D.4	21' - 0"	201.12	4.821	6.47	A
8 - F	21' - 0"	90.1952	6.187	8.876	C

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Floor 1 - Column Factored Design Loads					
Column	Height	P_U = 1.2PD + 1.6 PL (kips)	$M_{U,TOP}$ (ft-k)	$M_{U,BOTTOM}$ (ft-k)	Column Design
1 - B.1	10' - 6"	199.89904	47.119	0	C
1 - C.3	10' - 6"	204.98912	47.119	0	C
1 - D.4	10' - 6"	362.2944	47.119	0	D
1 - F	10' - 6"	335.3376	43.978	0	F
1 - F.9	10' - 6"	129.1776	5.902	0	B
2 - A.2	10' - 6"	210.048	8.28	0	C
4 - A.2	10' - 6"	234.68984	27.15	0	C
4 - B.1	10' - 6"	270.38	34.707	0	C
4 - C	10' - 6"	216.304	46.69	0	C
4 -D.4	10' - 6"	639.31	55.62	0	E
4 -F	10' - 6"	492.0916	56.731	0	D
4 - F.9	10' - 6"	200.392	13.03	0	A
6 - A.2	10' - 6"	386.928	21.718	0	F
6 - B.1	10' - 6"	172.6656	29.832	0	C
6 - C	10' - 6"	513.304	19.254	0	E
6 - D.4	10' - 6"	363.8544	20.11	0	F
6 - F	10' - 6"	435.1984	30.428	0	D
7 - F.9	10' - 6"	165.39872	13.871	0	B
8 - A.2	10' - 6"	76.0808	9.656	0	B
8 - B	10' - 6"	189.4216	13.658	0	B
8 - C	10' - 6"	168.0816	16.224	0	B
8 - D.4	10' - 6"	228.404	13.587	0	C
8 - F	10' - 6"	124.6592	18.639	0	C

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Column Design Explanation – Hand Calculations (Appendix D)

Once all of the columns were assigned their design axial and flexural loads, the column design process began. First, every loading condition was examined to determine what the average (most common) and extreme loading conditions were. Next, to represent the entire collection of column loading conditions, 6 different loading conditions were chosen. This set of six loading conditions (Figure F-82)) ranged from small to large loads and from nearly pure axial loading to nearly pure bending loading. The six loadings were chosen to best represent the entire field of loading conditions.

Column Loading Conditions - Hand Calculations							
Condition	Axial (kips)	Flexure (ft-kips)	Design Section	Condition	Axial (kips)	Flexure (ft-kips)	Design Section
1	151.6	49.3	A	4	492.1	56.7	D
2	118.05	53.5	B	5	639	55.62	E
3	266.3	29.78	C	6	391.4	38	F

F-82

Six sets of hand calculations were performed, one for each of the six loading conditions shown above, to produce the required column sections. The final column designs are shown in the following section on page 78. The "short column" assumption was employed during for the hand calculations, thus slenderness effects were not accounted for in this process.

Appendix D contains all hand calculations pertaining to the "short column" section design for the above six loading conditions. All columns were designed as square tied columns. The first step in these hand calculations was to calculate eccentricity using the equation: $e = \frac{M_U}{Pu}$. Next, the column dimensions had to be determined. Since the columns were being designed as square columns, dimensions h and b were always equal simplifying the calculation process. The most economical range for ρ_g is from 1 to 2 percent. Therefore, an approximate target reinforcement of 1.5 percent was established. Considering the target reinforcement ratio, the known loading conditions, and e/h ratio values, design aids were analyzed to get an idea of the required column dimensions. In conjunction with this design aid analysis, the equation $A_g \geq \frac{P_U}{0.4*(f'_c + f_y\rho_g)}$ was used to determine column dimensions. Next, since the axial loads are the dominant loading condition, a pure compression chart: "simplified column design" figure was analyzed as well. This chart, found in Appendix D, and provides required reinforcement ratio for various square columns sizes based on the axial load P_u (kips).

After selecting the column dimensions, the corresponding γ value was calculated where: $\gamma = \frac{h-2d'}{h}$. Next, the values of $\emptyset \frac{P_N}{bh}\ ksi$ and $\emptyset \frac{M_N}{bh^2}$ ksi were determined. Using these two values in conjunction with column interaction diagram design aids, the required reinforcement ratio ρ_g was determined. However, since the design aids correspond to specific γ values of .6, .75, and .9, interpolation or extrapolation between charts was required to acquire the correct ρ_g value. Of course, required area of steel A_s was then determined via the equation: $A_s = \rho_g bh$.

Checks were done to ensure proper cover and reinforcement spacing could be provided without exceeding the b_{MIN} value. Also, the vertical spacing of the ties was determined as the minimum value of the following: 16*(Diameter$_{\text{LONGTIDUNIAL BAR}}$), 48 *(Diameter$_{\text{TIE}}$), and the least column dimension. Finally a check was done to ensure all longitudinal reinforcement was within 6" of a laterally restrained bar (otherwise need intermediate ties).

Column Sections Present In Final Gravity System Design:

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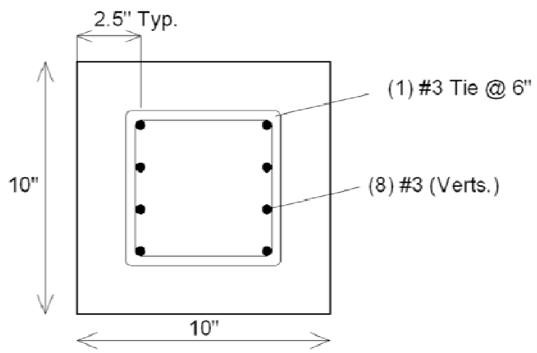
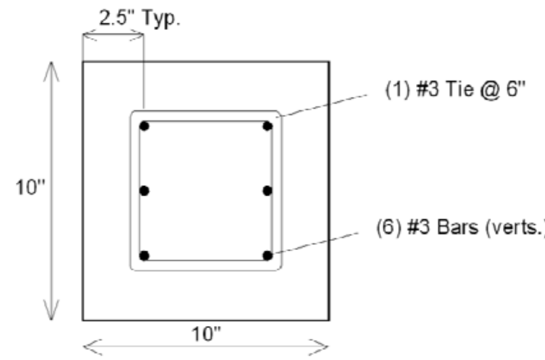


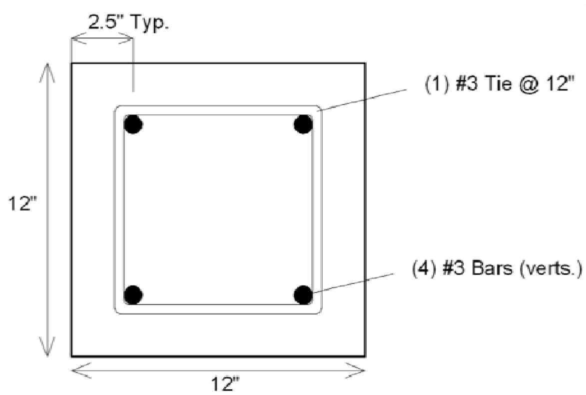
Column Section A

Note: No Intermediate Ties Required. All Longitudinal Reinforcement Is Within 6" Of A Laterally Restrained Bar.



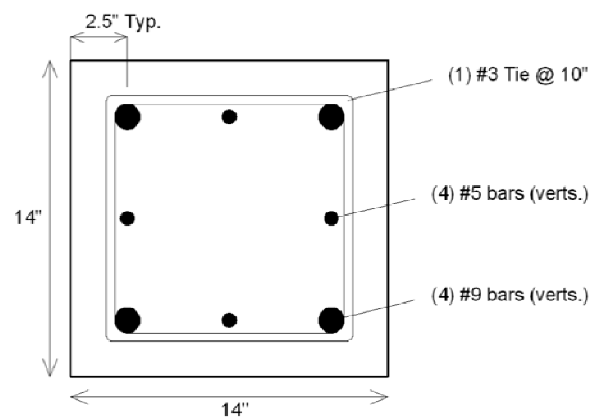
Column Section B

Note: No Intermediate Ties Required. All Longitudinal Reinforcement Is Within 6" Of A Laterally Restrained Bar.



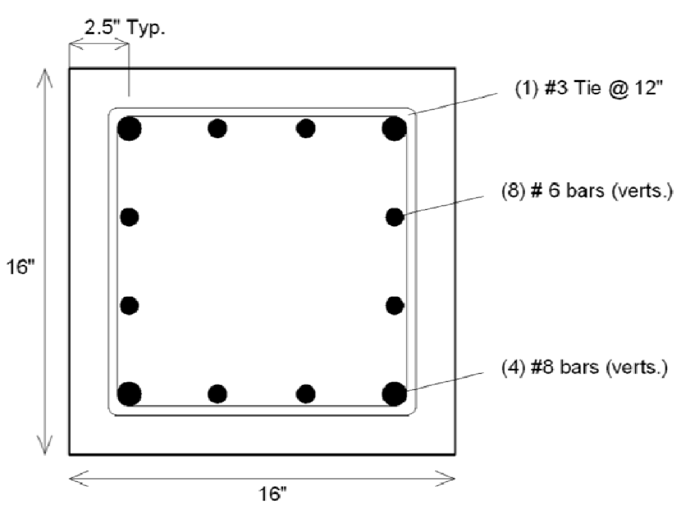
Column Section C

Note: No Intermediate Ties Required. All Longitudinal Reinforcement Is Within 6" Of A Laterally Restrained Bar.



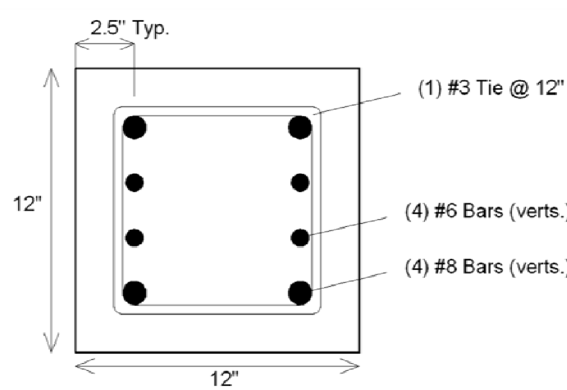
Column Section D

Note: No Intermediate Ties Required. All Longitudinal Reinforcement Is Within 6" Of A Laterally Restrained Bar.



Column Section E

Note: No Intermediate Ties Required. All Longitudinal Reinforcement Is Within 6" Of A Laterally Restrained Bar.



Column Section F

Note: No Intermediate Ties Required. All Longitudinal Reinforcement Is Within 6" Of A Laterally Restrained Bar.

Column Design – PCA and Interaction Diagrams:

The next step in the column design process involved modeling the above column sections using the computer program pcaCOLUMN. Each of the columns sections was modeled assuming f'_c = 4 ksi, f_y = 60 ksi, and E_c = 3605 ksi. Interaction diagrams were produced for each column section and can be found in the following section on pages 80 – 86.

As mentioned earlier, the design column load summary tables have a column containing "column design specification letters". Each column was assigned the most efficient column section by plotting the combined loading conditions (Mu,Pu) on the interaction diagrams found in the next section.

Column Design – Hand Calculation of Interaction Diagrams (Appendix E)

To avoid completely relying on the computer software to produce the interaction diagrams, hand calculations were performed to determine the interaction diagram for column section E. Refer to Appendix E for the full set of hand calculations. The hand calculated interaction diagram for column section E is on page 81.

In order to determine the interaction diagram by hand, the steel reinforcement has to be assigned area A*i* and depth d*i* values. The concept is to derive the interaction diagram by plotting several characteristic points. The first point to be determined pertained to pure axial strength in which C = ∞ and $\boldsymbol{\varepsilon_s = \varepsilon_c = 0.003}$. The pure compressive axial strength equation used in the hand calculations was:

$$\boxed{\boldsymbol{P_o = 0.85f'_c(bh - \sum A_{si}) + \sum A_{si}f_{si}}.}$$

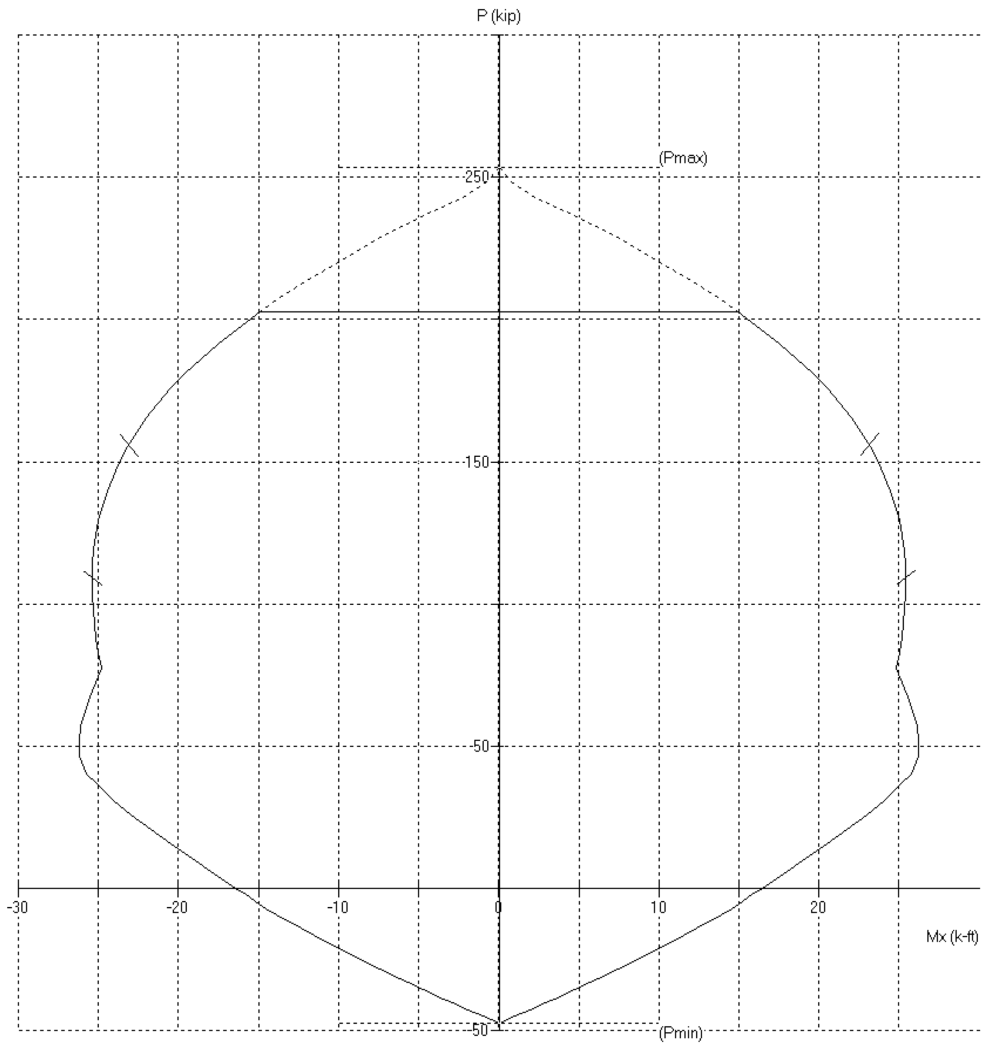
The balanced strain condition was the next significant point to be determined. Under this condition, both crushing of the concrete at the compression face and yielding of reinforcement at the tensile face occur simultaneously. Next, the points corresponding to pure bending, $\varepsilon_t = .005$, and pure tension were determined. Pure tension capacity was calculated via the equation:

$$\boxed{\boldsymbol{T_O = \sum A_{Si}f_{si}} \text{ where } \boldsymbol{\varepsilon_s = -\varepsilon_y}.}$$

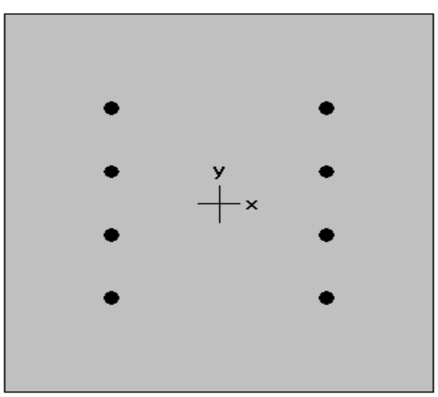
Note, the general case equation for flexural capacity was determined according to the following equation:

$$\boxed{\boldsymbol{M_n = 0.85f'_c b\beta_1 c\left(\frac{h}{2} - \frac{\beta_1 c}{2}\right) + \left[A_{si}f_{si}(\frac{h}{2} - d_i)\right]}}$$

Interaction Diagram (PCA COLUMN) – Column Section A

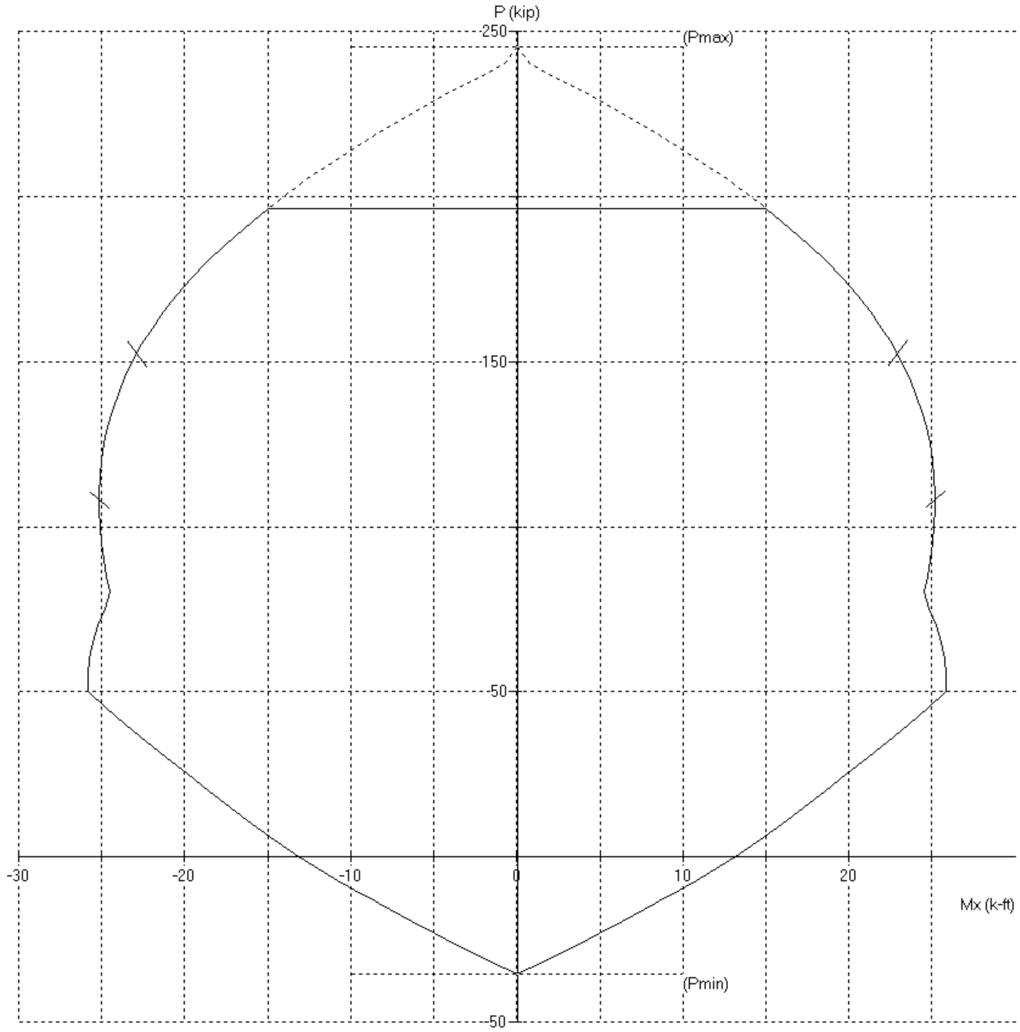


F-89

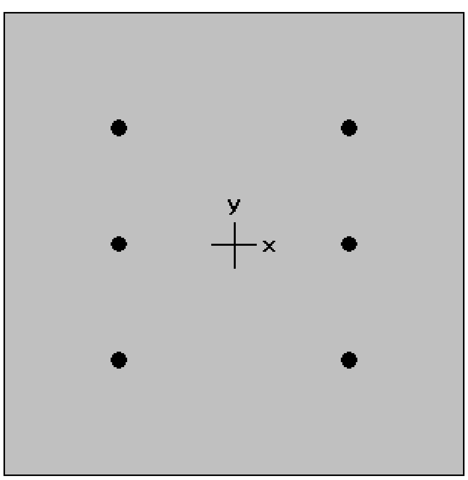


Column Material and Section Properties	
f'c	4 ksi
Ec	3605 ksi
fc	3.4 ksi
fy	60 ksi
Es	29000 ksi
Ag	100 in^2
Ix	833.33 in^4
Iy	833.33 in^4
Reinforcement	8 #3 Bars
Confinement	Tied
Clear Cover	2.31288 in

Interaction Diagram (PCA COLUMN) – Column Section B

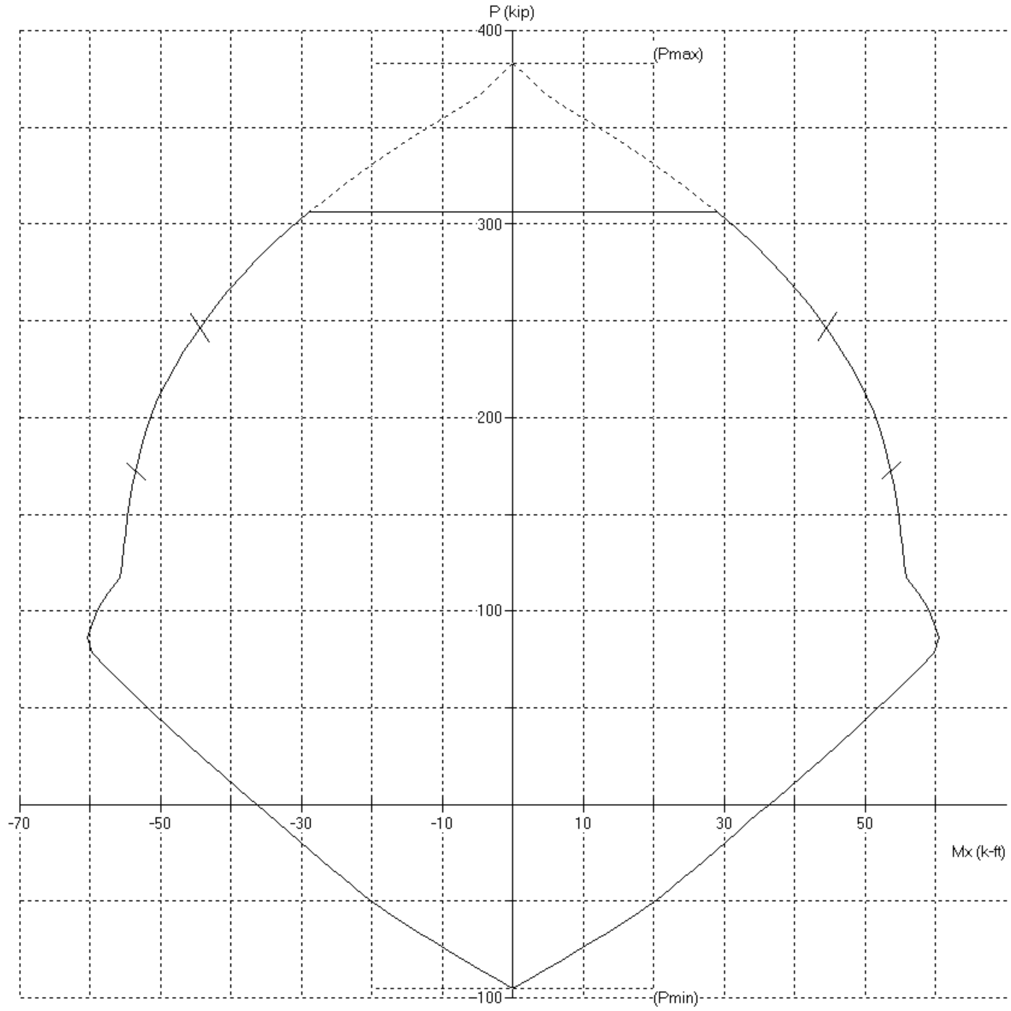


F-90

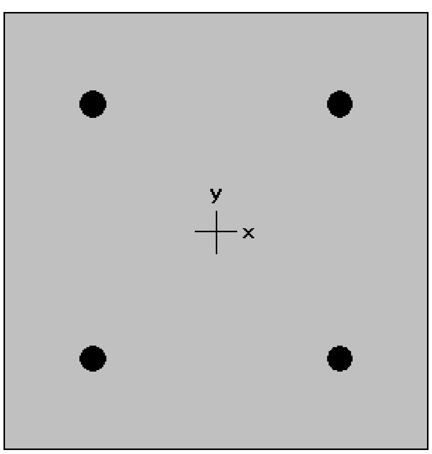


Column Material and Section Properties	
f'c	4 ksi
Ec	3605 ksi
fc	3.4 ksi
fy	60 ksi
Es	29000 ksi
Ag	100 in^2
Ix	833.33 in^4
Iy	833.33 in^4
Reinforcement	6 #3 Bars
Confinement	Tied
Clear Cover	2.31288 in

Interaction Diagram (PCA COLUMN) – Column Section C

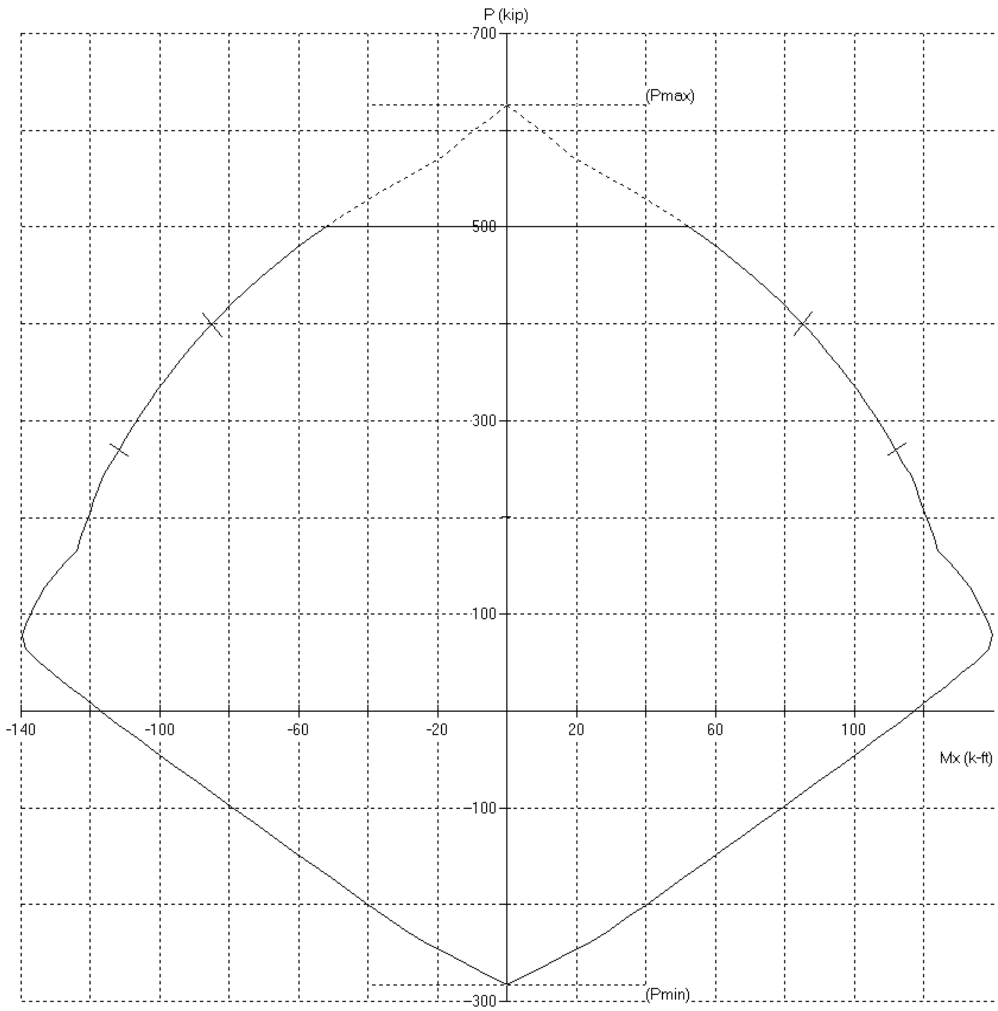


F-91

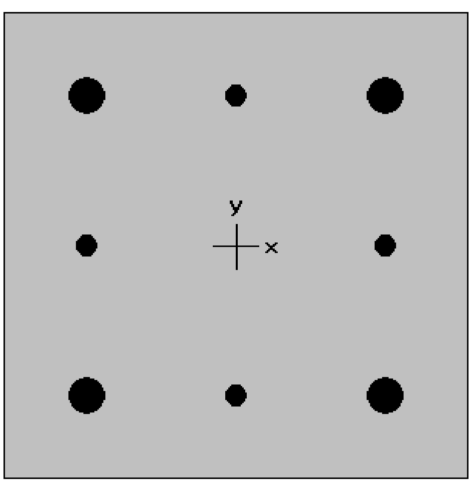


Column Material and Section Properties	
f'c	4 ksi
Ec	3605 ksi
fc	3.4 ksi
fy	60 ksi
Es	29000 ksi
Ag	144 in^2
Ix	1728 in^4
Iy	1728 in^4
Reinforcement	4 #6 Bars
Confinement	Tied
Clear Cover	2.12576 in

Interaction Diagram (PCA COLUMN) – Column Section D

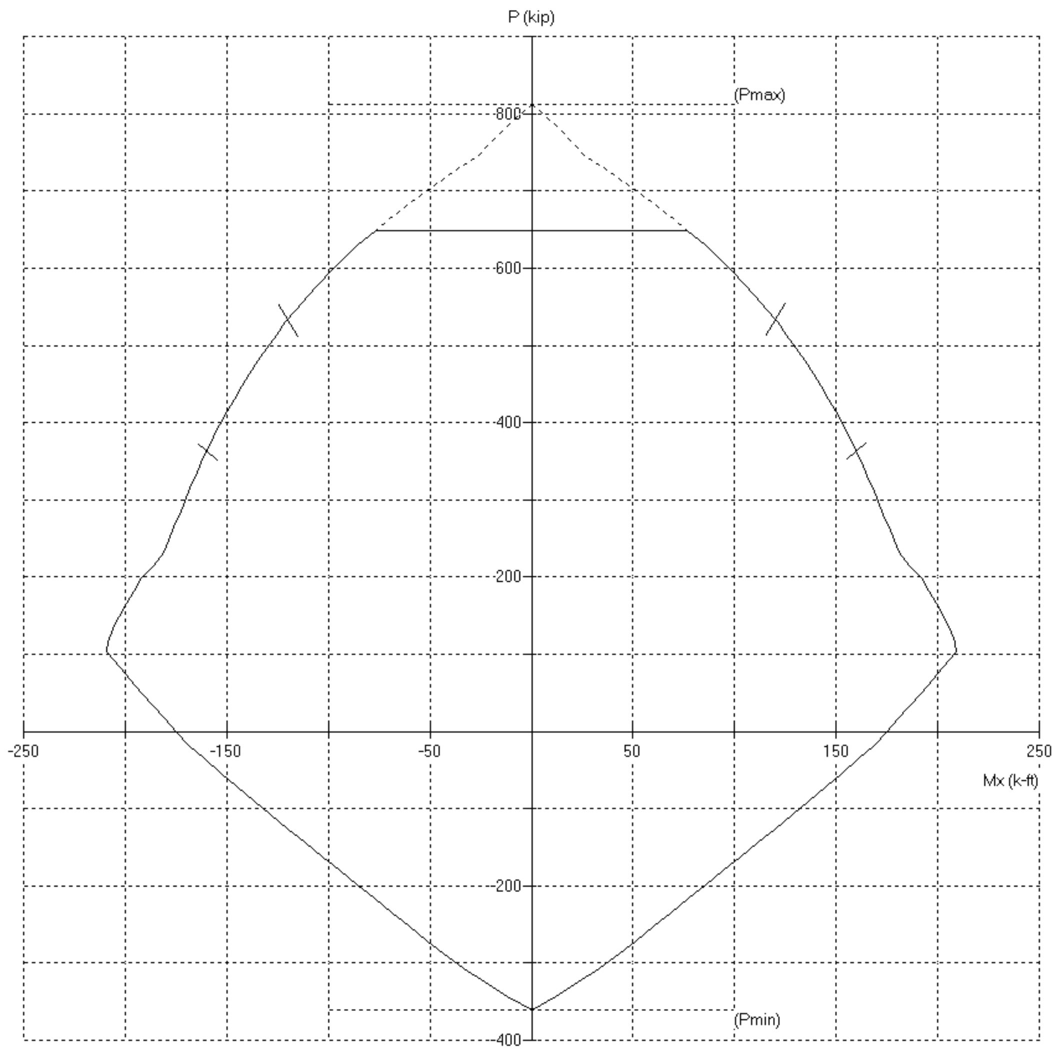


F-92

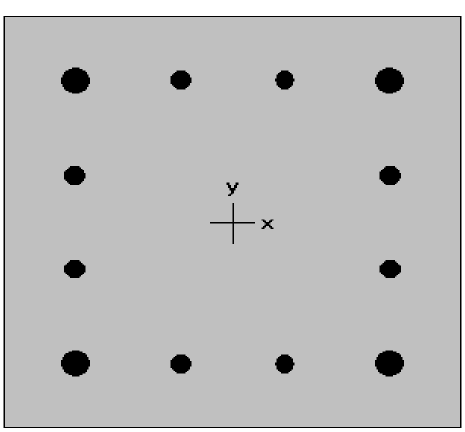


Column Material and Section Properties	
f'c	4 ksi
Ec	3605 ksi
fc	3.4 ksi
fy	60 ksi
Es	29000 ksi
Ag	196 in^2
Ix	3201.33 in^4
Iy	3201.33 in^4
Reinforcement	4 #5 Bars / 4 #9 Bars
Confinement	Tied
Clear Cover	1.93581 in

Interaction Diagram (PCA COLUMN) – Column Section E

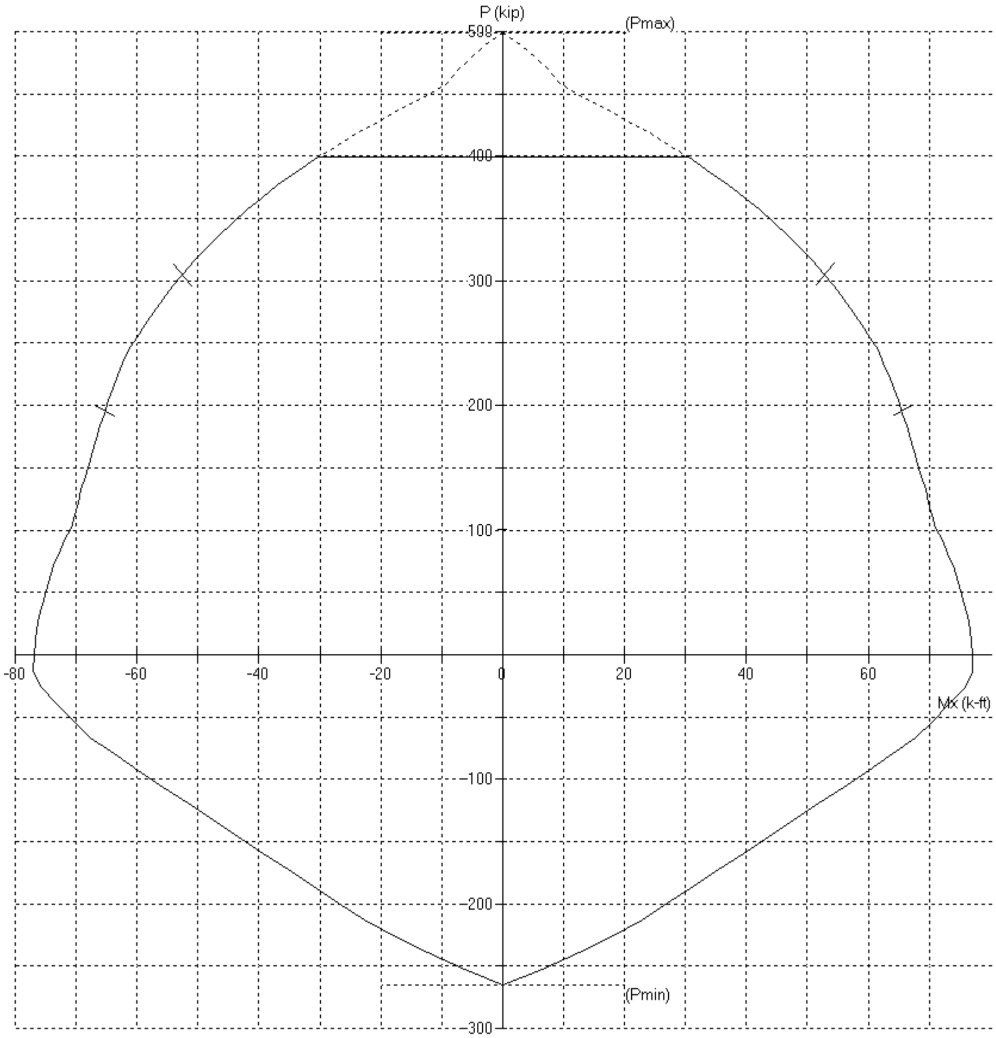


F-93

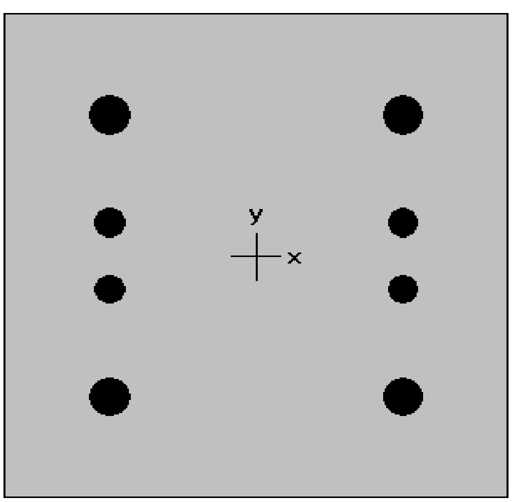


Column Material and Section Properties	
f'c	4 ksi
Ec	3605 ksi
fc	3.4 ksi
fy	60 ksi
Es	29000 ksi
Ag	256 in^2
Ix	5461.33 in^4
Iy	5461.33 in^4
Reinforcement	4 #8 Bars / 8 #6 Bars
Confinement	Tied
Clear Cover	1.99854 in

Interaction Diagram (PCA COLUMN) – Column Section F



F-94

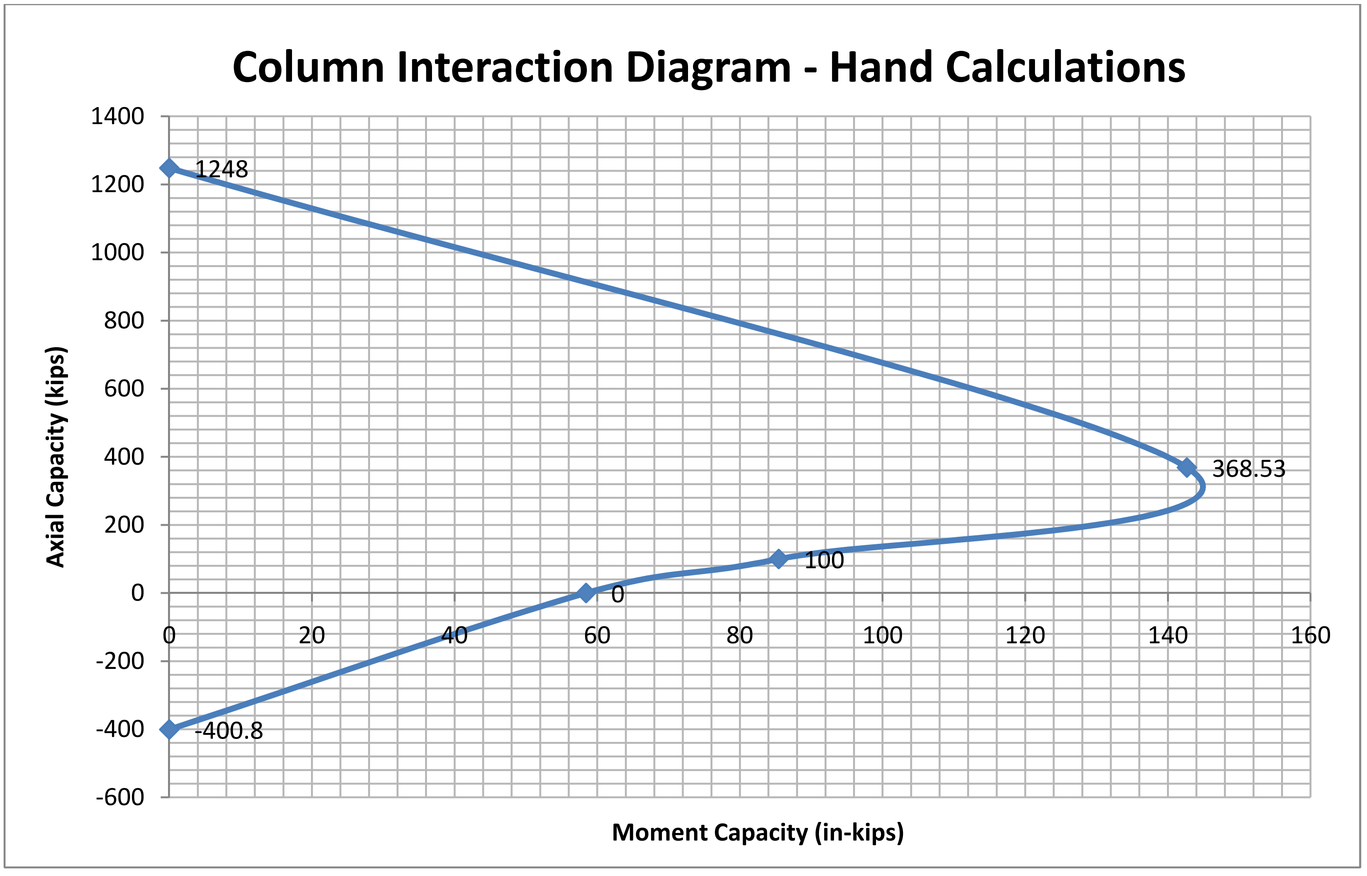


Column Material and Section Properties	
f'c	4 ksi
Ec	3605 ksi
fc	3.4 ksi
fy	60 ksi
Es	29000 ksi
Ag	144 in^2
Ix	1728 in^4
Iy	1728 in^4
Reinforcement	4 #8 Bars / 4 #6 Bars
Confinement	Tied
Clear Cover	1.99854 in

Column Design – Hand Calculations:

COLUMN INTERACTION DIAGRAM - HAND CALCULATIONS			
CONDITION	**Pn (Kips)**	**Mn (in-k)**	**Mn (ft-k)**
Pure Compression	1248	0	0
Pure Bending	368.53	142.6666667	1712
Balanced Strain	100	85.43833333	1025.26
ϵ_t = .005	0	58.43166667	701.18
Pure Tension	-400.8	0	0

F-95



Interaction Diagram. Hand calculated to supplement and verify accuracy of the computer aided design process. This interaction diagram is for Column Section E.

First Floor Concrete Corbel Design:

At the first floor, the structural design requires 10 reinforced concrete corbels. As shown in the final first floor layout on page 13, the 2nd floor columns (red) do not align with the first floor columns. In order to attain the desired residential square footage per floor, the above grade floor levels occupy the entire site. With poor soil conditions and adjacent existing structures located right at the perimeter of the site, a heavy concrete structure has great settlement potential. To avoid disturbing or damaging the nearby structures, the foundation had to be positioned a reasonable distance away from the perimeter of the site. As a result, the first floor columns had to be offset approximately 2' to the interior of the above second floor columns in order to align with the pile caps. In response to this misalignment of columns, corbels were designed to complete the vertical load path between floors one and two.

In the original structure, cantilever steel beams were used to transfer the loads at these locations. Similarly, the function of corbels relies upon the fundamental mechanics of cantilever design. The corbels designed for this structure are essentially a short reinforced concrete member that projects or cantilevers out from the first floor columns to support the second floor column loads. See figure F-81 to view the calculated design loads for each corbel.

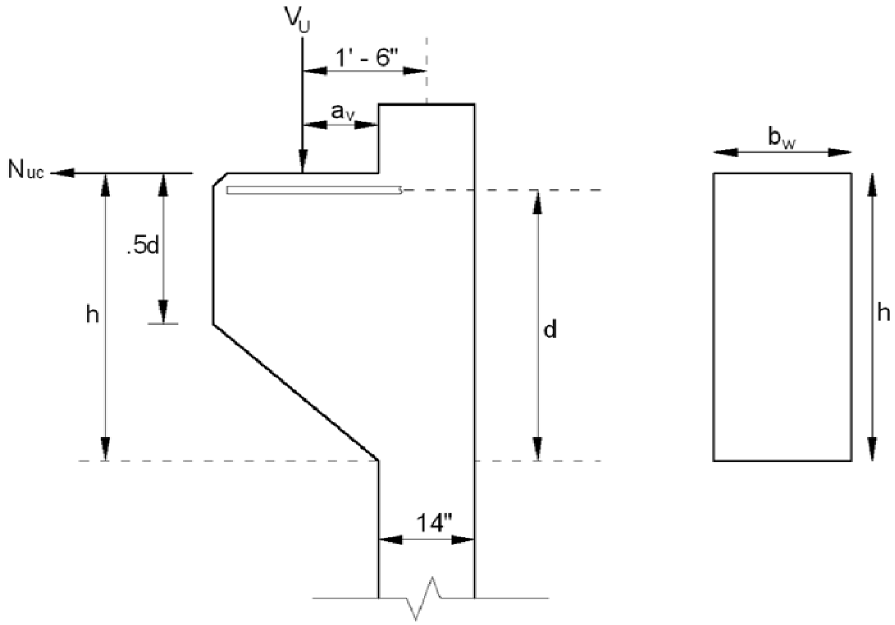
Design calculations were done according to the Provisions for Brackets and Corbels provided in section 11.8 of ACI 318-08 Code. On pages 87 - 90, a thorough and detailed explanation of corbel calculations can be found. When designing the corbels, several modes of failure had to be considered. As described in ACI section 11.8.1, corbels may fail by shearing along the vertical interface between the columns and corbel, by yielding of the tension tie, by the crushing of the compression strut, or by bearing/shearing failure confined to the area directly below the steel loading plate. Do to the multitude of potential failure modes, careful design and detailing of corbels is critical. The final design includes the specification of required primary tension reinforcement, required closed stirrups (or ties) parallel to the tension reinforcement, required shear-friction reinforcement, and required flexural reinforcement.

In addition, the basic dimensions of the corbel had to be carefully selected. First, the corbels had to project far enough out to provide a bearing surface for the second floor columns. Additionally, in order to apply the chapter 11.8 ACI provisions, verification that the shear-to-span depth ratios did not exceed unity was completed. Another dimensional limitation pertained to the depth of the outside corbel edge. The minimum depth of 0.5d for the outside edge bearing area was satisfied to eliminate the threat of premature failure due to major diagonal tension crack propagation.

As shown in the following page of calculations, the corbel behavior was controlled by shear. The final corbel detail design can be found on page 90. As required, the primary tension reinforcement is anchored by a structural weld to a transverse anchor bar. As shown on the following pages, the required reinforcement values are: 5.67 in^2 of shear-friction reinforcement, 1.9 in^2 of closed hoop reinforcement, 2.8 in^2 of flexural reinforcement, and 5.45 in of primary tension reinforcement

Corbel Design Calculations:

F-96



Corbel Design Information	
Dimensions	
a_v	11"
h	44"
d	40"
bw	20"
.5d	20"
Loads	
V_u	363 kips
N_{uc}	72.6 kips
Properties	
f'c	4 ksi
fy	60 ksi
Concrete	N.W.

Step One: Determine Basic Dimensions. Since the 2nd Floor Column bearing on corbel is 14" x 14" and column is offset 1'5" from the first story 14"x14" column, it was determined that a_v = 18" – (14"/2) = 11".

Next, depth of the outside edge at bearing area was determined. In order to avoid premature failure in which diagonal tension cracks propogate from the bearing area to the corbel's outside sloping face, the outside edge's minimum depth was calculated as:

Step Two: Several requirements must be satisfied in order to use the Corbel Design Provisions outlined in ACI Sections 11.8.3 and 11.8.4.

✓ (Satisfied)

✓ (satisfied, shear span-to-depth ratio is less than unity)

Step Three - Design of Shear Friction Reinforcement Avf: Based on ACI 11.8.3.2 (and 11.6), the capacity for shear friction reinforcement positioned orthogonally to the shear plane is expressed with the following equation:

As provided in ACI 11.6.4.3, the coefficient of friction (μ) for monolitichically placed concrete is 1.4λ. Also, for normal weight concrete, λ = 1.0. Now the required shear friction reinforcement A_{vf} to resist V_u can be calculated.

Corbel Design Calculations Continued:

Step Four - Calculation of Corbel Flexural Reinforcement: A_F was calculated to resist the factor moment generated simultaneously by the Shear Force V_u and Tensile Force N_{uc}. First, the Ultimate Factored Moment was determined using the following equation provided ACI Code Section 11.8.3.3, 10.2, and 10.3:

$$M_U = V_U a_V + N_{UC}(h-d) = (363*11)+(.2*363)(44-40") = 4283.4\ k-in$$

Next, the flexural reinforcement value A_F was calculated to resist the 4283.4 k-in moment.

Approximation: $A_F = \frac{M_U}{\emptyset f_y jd} = \frac{4283.4}{.75(60)(.85)(40)} = 2.8\ in^2$

Step Five – Calculation of Corbel Tension Reinforcement: First the design tensile force N_{UC} had to be established. For this design, no applied tensile force exists. However, to account for the tensile forces resulting from shrinkage, temperature fluctuation, and creep effects, a minimum tensile force (equation below) must be accounted for (ACI 11.8.3.4).

$$Minimum\ Tensile\ Force = 0.2V_U = 0.2*363 = 72.6\ kips$$

Reinforcement A_N was then determined:

$$\emptyset A_N f_y \geq N_{UC} \qquad A_N = \frac{72.6}{.75*60} = 1.613$$

Step Six - Determine Primary Tension Reinforcement: First, the total amount of reinforcement requried to cross the face of the support was determined by comparing two applicable cases (ACI 11.8.3.5):

Case 1: $A_{VF} + A_N = 5.762 + 1.613 = 7.375\ in^2$

Case 2: $1.5A_F + A_N = 1.5(2.8) + 1.613 = 5.813\ in^2$

Case 1 yielded the larger area, therfore case 1 controls. Since case 1 controls, the following two equations were applied in order to determine Primary Tension Reinforcement A_{SC} and the Required Closed Strirups A_h.

$$Primary\ Tension\ Reinforcement:\ A_{SC} = \frac{2A_{VF}}{3} + A_N = \frac{2(5.762)}{3} + 1.613 = 5.4543\ in^2$$

$$Close\ Hoop\ Reinforcement:\ A_h = \frac{A_{VF}}{3} = \frac{5.762}{3} = 1.921$$

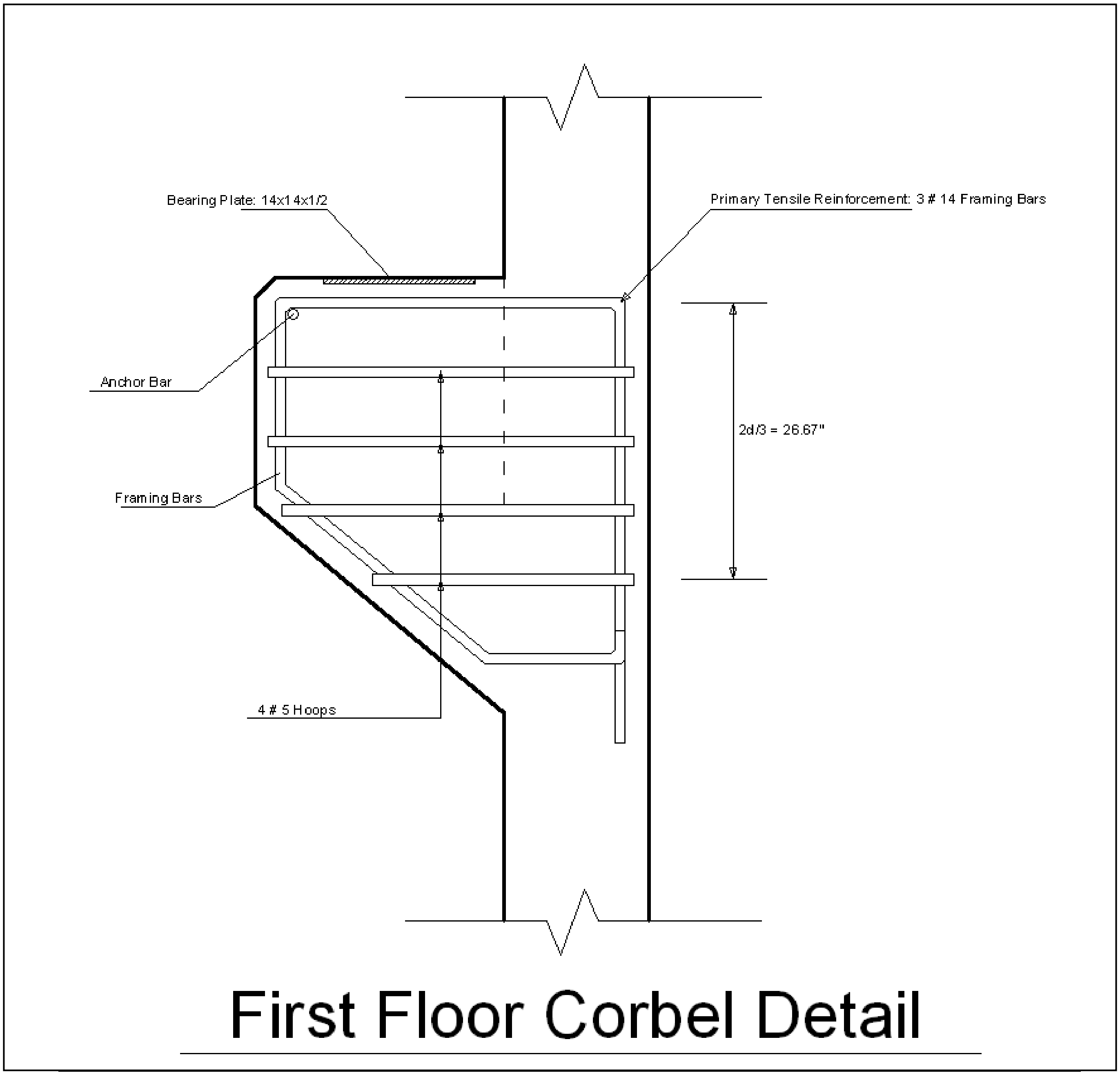
The Closed Hoop Reinforcement (A_N) is positioned parallel to the primary tension reinforcment. The closed hoops are evenly spaced througout the area within ***(2/3)d = 26.6667"*** of the primary tension reinforcment.

Step Seven – Reinforcement Limitations: Two limitations were checked and verified.

$\rho = \frac{A_{SC}}{b_w d} = \frac{5.4543}{20*40} = .006817 \geq \frac{0.04f'_c}{f_y} = .002667$ (Satisfied)

$A_{h=1.921} \geq .5(A_{sc} - A_N) = .5(5.4543 - 1.613) = 1.92$ (satisfied)

Final Corbel Design Detail:



F-97

Reinforced CMU Wall Design:

Introduction: The final elements of the structure's gravity system to be designed are first floor exterior masonry walls. Masonry walls were used in the original design as well. Concrete masonry is often left exposed serving both as a structural and architectural element. For 40 Gold Street, the masonry walls resist gravity loads and out of plane wind induced flexural loads. Also, since the masonry wall extends below grade down to the foundation, the masonry wall also behaves as a retaining wall by resisting surcharge loads and lateral earth pressures. Left exposed to the exterior, the masonry walls create an aesthetically pleasing store front façade. Complete design of the masonry wall involved three particular procedures. Lintel beams, un-reinforced masonry wall sections, and reinforced masonry wall sections had to be designed. The location of the masonry wall is shown in the first floor plan view (figure F-99).

Lintel Beam Design: As shown in the wall elevation in Appendix G, there is a 12' x 12' door opening. This doorway provides access to the retail space. It is the primary access point for delivery and loading/unloading services. Although this section provides a summary of the concrete masonry lintel design, the entire set of hand calculations is provided in appendix G. First, strength design method was selected as the design procedure. Factored loads were acquired via load combination 1.2D + 1.6L. To begin, important wall properties were identified. For example, it was assumed the wall is constructed with Type S PCL mortar and fully grouted, 8", hollow CMU. In addition, the lintel beam was assumed to extend 4" beyond the opening on each side. Therefore, the lintel beam length was identified as 12'-8". In addition, movement joints were assumed to be located at both ends of the beam. Next, the factored axial load was determined with the expression: $\boldsymbol{W_U = 1.2P_D + 1.6P_L + 1.2(S.W.\,of\ wall\ above)}$. Next, the max design moment and shear were determined via the following equations: $\boldsymbol{M_U = \frac{W_U l^2}{8}}$ and $\boldsymbol{V_U = \frac{W_U l}{2}}$. Before design of reinforcement, a shear capacity check had to be completed: $\boldsymbol{V_U \leq \emptyset V_N}$. The governing equation to calculate the masonry shear capacity is: $\boldsymbol{V_{NM} = \left[4 - 1.75\left(\frac{M_U}{V_U d_V}\right)\right] A_N \sqrt{f'_m} + .25P_U}$. To be conservative, the axial load was not considered. As shown in appendix G, no shear reinforcement was required since the capacity exceeded the design shear value. Before proceeding to the design of flexural reinforcement, the validity of the expression $\boldsymbol{V_N \leq 4A_N\sqrt{f'_m}}$ was verified, as required. To begin the flexural design process, "jd" was first approximated as .9d. Next, the required moment capacity was determined via the equation: $\boldsymbol{M_N = \frac{M_U}{\emptyset}}$. Next, the moment capacity equation was rearranged to determine the required area of steel reinforcement: $\boldsymbol{A_{S,required} = \frac{M_N}{f_Y * 0.9 * d}}$. Before specifying the provided steel reinforcement, the cracked moment capacity had to be considered by investigating the validity of the expression: $\boldsymbol{M_N \geq 1.3M_{CR}}$. Since the expression held true, the flexural reinforcement was specified according to the un-cracked moment capacity. The final design requires (1) #4. Note, for all calculated values, assumptions, and results, please see appendix G.

Reinforced CMU Wall Design (continued):

1 Story Un-reinforced Concrete Masonry Wall: A portion of the exterior concrete masonry wall does not extend below grade. This portion of the wall did not require steel reinforcement. Please see appendix F for related figures and the entire set of hand calculations. Subject to out of plane flexure loads (wind) and axial loads, several design criteria were considered. Load combinations .6D + W and L+ D were considered, and the ASD method provided by MSJC Code was followed.

To begin, the unity equation was considered: $\frac{f_a}{F_A} + \frac{f_b}{F_B} \leq 1.0$. First loads were factored according to the load combination .6D + W. As shown in appendix #, the axial load at mid-height was calculated as $P_{MidHeight} = .6(P_D + s.w.)$ Note, the self weight for normal weight fully grouted CMU was calculated using 10PSF/inch of wall thickness. With the mid height axial load known, the axial stress was obtained via: $f_a = \frac{P}{A_n}$. Next, the slenderness ratio h/r had to be calculated to determine the allowable compressive load F_A. The radius of gyration "r" was calculated as $\sqrt{I/A} = \sqrt{\frac{bt^3}{12*A}}$. Since the slenderness ratio exceeds 99, the allowable compressive stress value was determined according to: $F_A = 0.25f'_m\left(\frac{70}{h/r}\right)^2$ where **f'_m= 1500 psi**. Calculation of the moment at mid-height of wall followed next. The expression applied was: $M_{MidHeight} = 0.6M_{eccentric} + 1.0M_{Wind}$. With the moment calculated, the neutral axis depth C and the net section moment of inertia I were determined. Finally, the bending stress was calculated according to the equation $f_b = \frac{MC}{I}$, and the allowable bending stress was determined as $\frac{f'_m}{3} = 500\ psi$. With the four terms of the unity equation known, it was determined that the unity requirement was satisfied. The same process was completed for the load combination L + D, and once again the unity equation requirement was satisfied.

In the next phase of the design process, tensile stresses were checked for both load combinations. The tensile stress of 56.84 psi exceeded the 40 psi allowable capacity for Type S mortar, PCL solid units. However, the wall was the redesigned as fully grouted hollow units. As a result, the tensile stress capacity was increased to 65 psi, which exceeds the actual tensile stresses.

Finally, a stability check was performed to ensure the buckling load limit is not exceeded. The governing equation was: $P \leq \frac{1}{4}P_e = \frac{1}{4} * \left(\frac{\pi^2 E_m I}{h^2}\right)\left(1 - \frac{0.577e}{r}\right)^3$. Considering both load combinations, the axial load P = 1,505 lb/ft which did not exceed the 23.2 kip/ft buckling load limit. As shown in appendix F, the single story portion of the masonry wall can be safely and efficiently designed as a 8" unreinforced CMU wall. The specifics of the wall include Type S PCL mortar, and fully grouted hollow units.

2 Story Reinforced Concrete Masonry Wall: (See F-98 on page 95). The design of this wall masonry wall system involved extensive calculations and analysis. See appendix H to view the calculations, charts, and results. Not only is the wall subject to axial loads and out of plane flexure loads from wind, but the wall must also resist equivalent lateral earth pressures. Additionally, construction equipment located on grade will induce surcharge loads on the sub grade portion of the wall. The calculation and analysis of the wall system was broken down into three phases which include determination of design loads, attempt to design as URM wall, design as reinforced masonry wall.

To begin, the geotechnical report was reviewed to determine the soil properties. According to the report, the soil layer of interest is composed of loose cohesion less soil. With a angle of friction **$\emptyset = 32.5^{\circ}$**. The dry unit weight was calculated to be 110 PCF via the equation: $\frac{W_S}{V}$. With the unit weight known, the equivalent lateral earth pressures were determined. First the term K_O was calculated as **$1 - \sin\phi = 1 - \sin(32.5) = .462$**. As shown in Appendix H, a free body diagram was created to establish the force distribution characteristic of surcharge and soil pressures. The surcharge pressure imposes a triangular load distribution on the wall system, whereas the soil pressure is distributed uniformly over the height of the wall. The resultant soil and surcharge forces were calculated with the respective equations: $\boldsymbol{K_O Q H}$ and $\frac{1}{2}\boldsymbol{K_O \gamma H^2}$.

Next, considering the wind and previously calculated equivalent lateral pressures, the critical flexural load was determined. The critical section is located at the base of the wall. After establishing both the max axial and max flexure loads, an attempt at URM wall design was completed according to the ASD method provided in MSJC Code. Of course, due to the large flexural loads, no URM wall designs were sufficient. For example, the calculations shown in appendix H show that the combined axial and flexural unity requirement was unable to be satisfied.

To begin the reinforced masonry wall design, axial/flexure interaction diagrams were examined. With the design loads known, a specification of steel reinforcement was selected. 1 # 8 bars @ 16" were chosen. Exhaustive hand calculations were carried out to generate the corresponding interaction diagram, which is shown on page 96. To determine the approximate interaction diagram, 5 major points were determined which include: pure flexure, pure compression, balance point, a point above the balance point, and a point below the balance point. The Strength Design procedure was followed. First, the pure flexural point was determined with the equation: $\boldsymbol{M_N = A_S f_y \left(d - \frac{a}{2}\right)}$. Next, the pure compression value was determined according to the expression: $\boldsymbol{P_O = 0.8(0.8f'_m)(A_{Conc.} - A_{Steel}) + A_S f_Y}$. Note, the first .8 term was included to account for the minimum design eccentricity. Next, the balance point was determined by analyzing the stress distribution in which $\boldsymbol{C = d\left(\frac{\varepsilon_{MIN}}{\varepsilon_{MIN} + \varepsilon_Y}\right)}$. The axial and flexural capacities at the balance condition were determined via the respective expressions: $\boldsymbol{P_O = C - T = .8f'_m ab - A_S f_Y}$ and $\boldsymbol{M_n = T\left(d - \frac{h}{2}\right) + C\left(\frac{h}{2} - \frac{a}{2}\right)}$. Note, the bars were assumed to be placed in the middle of the cell, and so **d = h/2**. Next, the intermediate points were determined by selecting neutral axis depth values above and below C_B.

Reinforced CMU Wall Design (continued):

According to MSJC 3.3.4.1.1, slenderness effects also had to be considered in order to adjust the interaction diagram. First the slenderness ratio h/r was calculated and compared to 99 to determine which slenderness modifier applied. Since h/r > 99, the applicable slenderness modifier was $\left(\frac{70r}{h}\right)^2$. Each of the axial capacities in the interaction diagram was then reduced by the slenderness factor (calculated to be (0.431). The interaction diagram adjusted for slenderness effects can be found on page 96. Finally, an iterative set of hand calculations was completed to include secondary moment effects in the design. Before proceeding, applicability of MSJC equations was determined by checking $\frac{P_U}{A_g} < 0.2f'_m$, which held true. Next, the cracked moment of inertia I_{CR} was calculated as **40%(I_{GROSS}).** The equation $M_{U2} = M_{U1} + P_U\delta_{U1}$ was applied for several iterations until a negligible increase in moment was observed. Also note, the deflection due to factored loads was determined with the equations provided by MSJC 3.3.5.4:

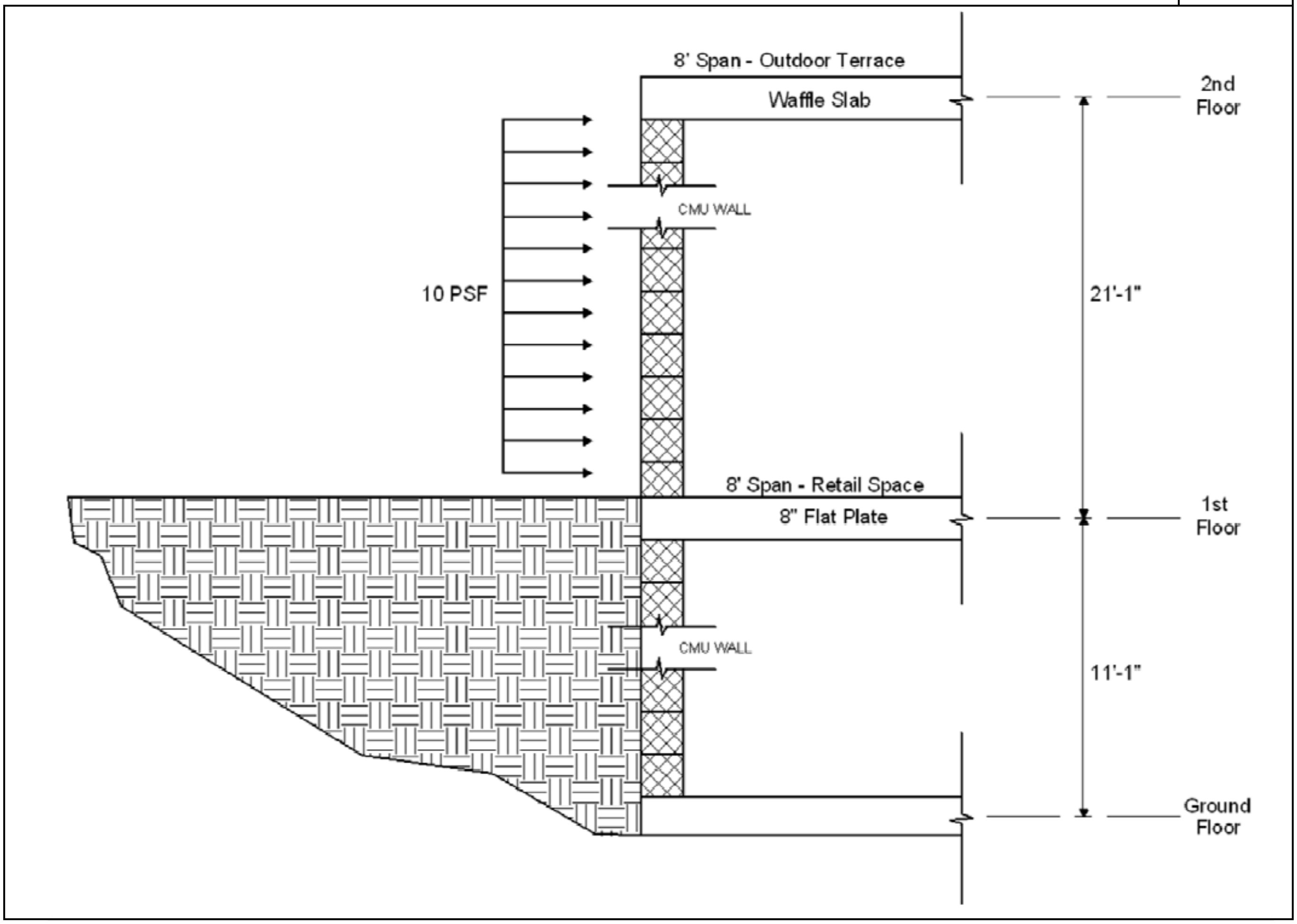
For $M_{Service} < M_{CR}$, $\delta_S = \frac{5M_{Cr}h^2}{48E_mI_Y}$

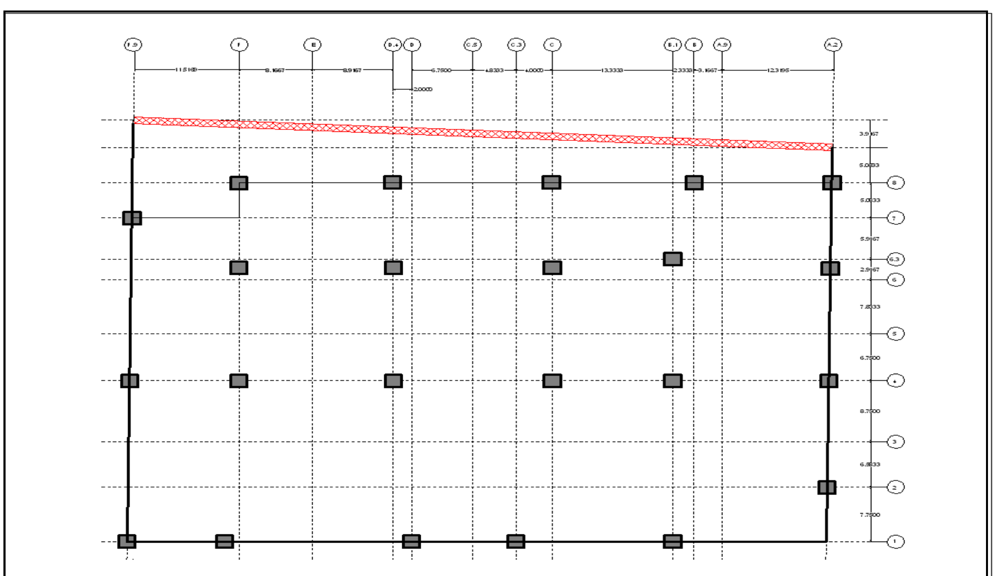
For $M_{Cr} < M_{Service} < M_N$, $\delta = \frac{5M_{Cr}h^2}{48E_mI_Y} + \frac{5(M_{SERVICE}-M_{CR})}{48E_mI_{Cr}}$

The adjusted design loads were plotted on the modified interaction diagram, and is within the capacity boundary. Please see page 95, 96 and Appendix H for all related information and diagrams.

Reinforced CMU Wall Design:

F-98





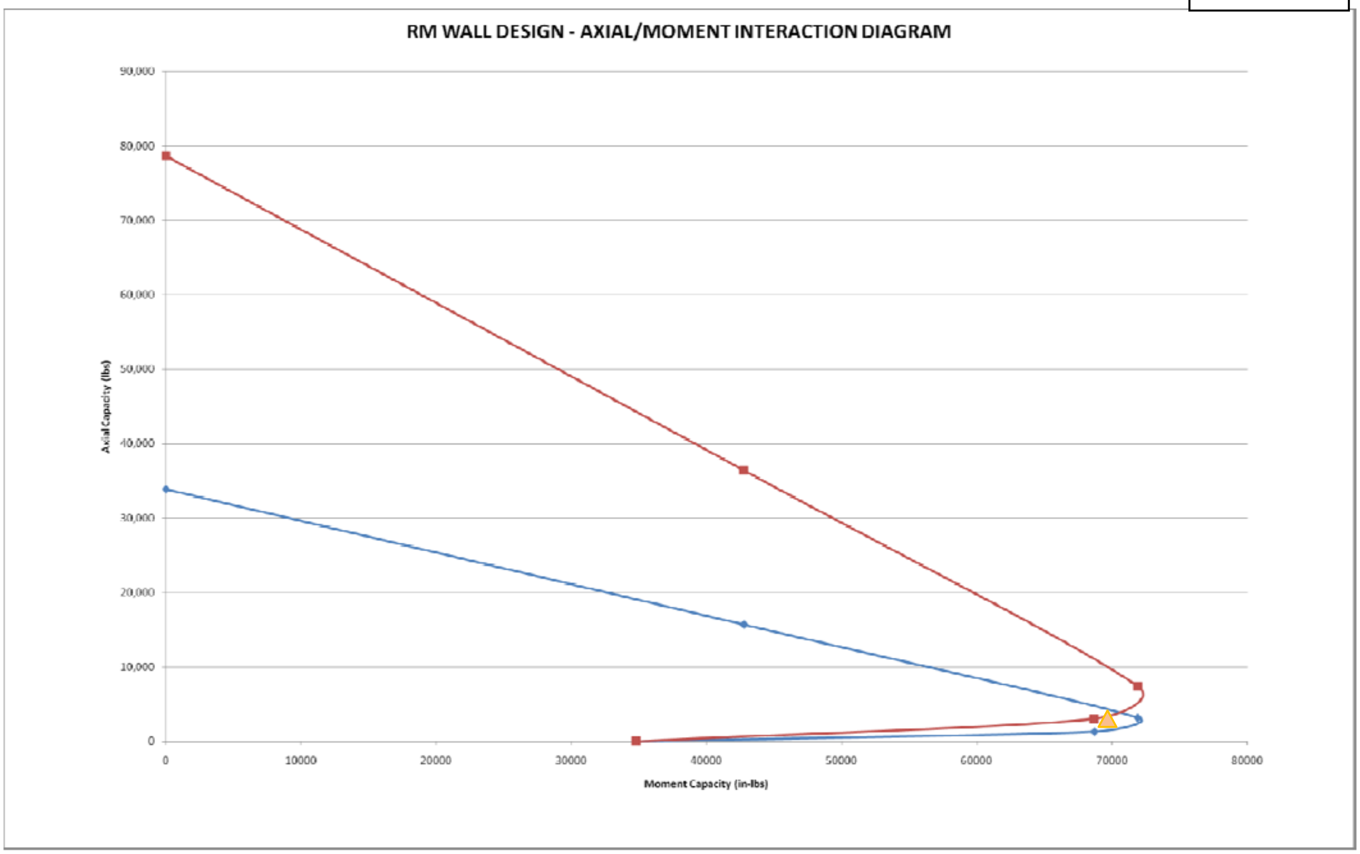
SECOND FLOOR COLUMN AND WALL LAYOUT

F-99

Reinforced Masonry Wall Design – Final Design and Interaction Diagram:

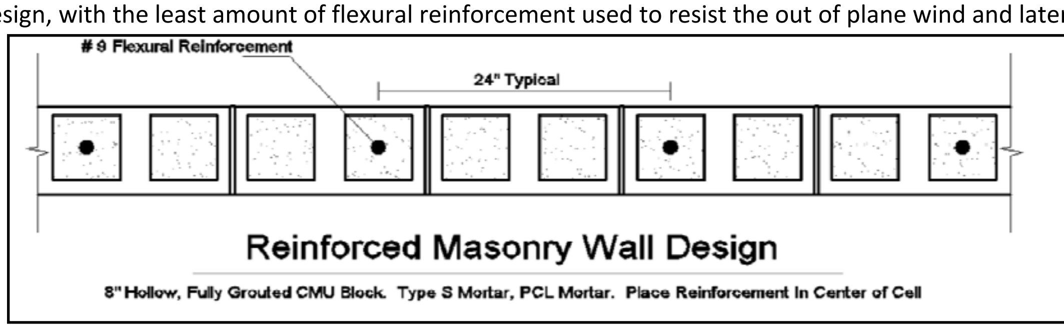
8" Full Grouted CMU Blocks. Type S PCL Mortar. Hollow Units. # 9 @ 24" Steel Reinforcement (Placed in Center of Cells)			
Condition	Axial Capacity (lbs)	Axial Capacity Adjusted For Slenderness (lbs)	Moment Capacity (in-lbs)
Pure Axial Compression	78,675	33,909	0
Point Above Balance Condition	36,420	15,697	42,765
Balance Condition	7402	3190	71,887
Point Below Balance Condition	3040	1310	68,683
Pure Flexure	0	0	34,800

F-100



Basic Interaction Diagram: Adjusted For Slenderness Effects:

Adjusted For 2nd Order Effects, the Combined Axial and Bending Design Loads were plotted on the diagram (△) As shown, the loading condition (M,P) = (70,927 in-lbs ,4,353 lbs) is barely within the adjust interaction diagram (blue). As one can see, this is a very efficient design, with the least amount of flexural reinforcement used to resist the out of plane wind and lateral earth forces.



F-101

Lateral System – Calculation of Building Weight

With the Gravity System completely designed, the total building weight and weight per floor were calculated to determine the seismic forces. Figures F-53 through F-56 display the total weight due to slab and superimposed dead loads. The following section of tables summarizes both the calculation process and results regarding the column dead weight. In order to determine the effective column weight at each floor, only ½ of the column above and ½ of the column below each floor were included. For each column, a 150 PCF self weight was used for normal weight concrete. With the height and gross section area of each column known, the weight of the columns was determined. As shown in the tables, the weight per floor due to columns gradually decreases from the 1st floor level (90,606 lbs) to the top floor level (12,993.75 lbs) of structure. The total weight due to column self weight is **410,943.78 lbs**.

Building Weight Due to Columns - Broken Down Per Floor For Seismic Calculations

Floor Level	Column Below - Specification Letter	Column Below - Gross Section Area (in^2)	# Identical Columns	Column Height (ft)	Total Weight Per Column (lbs)	Column Above - Specification Letter	Column Above - Gross Section Area (in^2)	# Identical Columns	Column Height (ft)	Total Weight Per Column (lbs)	Total Weight Per Floor (lbs)
1st	A	100	1	21	2187.5	A	100	3	10.5	1640.625	1st Floor Columns = 90,606.25
	B	100	5	21	10937.5	B	100	9	10.5	4921.875	
	C	144	9	21	28350	C	144	5	10.5	3937.5	
	D	196	3	21	12862.5	D	196	2	10.5	2143.75	
	E	256	2	21	11200	E	256	1	10.5	1400	
	F	144	3	21	9450	F	144	2	10.5	1575	
2nd	A	100	3	10.5	1640.625	A	100	0	10.5	0	2nd Floor Columns = 31,500
	B	100	9	10.5	4921.875	B	100	13	10.5	7109.375	
	C	144	5	10.5	3937.5	C	144	5	10.5	3937.5	
	D	196	2	10.5	2143.75	D	196	1	10.5	1071.875	
	E	256	1	10.5	1400	E	256	1	10.5	1400	
	F	144	2	10.5	1575	F	144	3	10.5	2362.5	
3rd	A	100	0	10.5	0	A	100	0	10.5	0	3rd Floor Columns = 29,334.38
	B	100	13	10.5	7109.375	B	100	14	10.5	7656.25	
	C	144	5	10.5	3937.5	C	144	5	10.5	3937.5	
	D	196	1	10.5	1071.875	D	196	1	10.5	1071.875	
	E	256	1	10.5	1400	E	256	0	10.5	0	
	F	144	3	10.5	2362.5	F	144	1	10.5	787.5	
4th	A	100	0	10.5	0	A	100	0	10.5	0	4th Floor Columns = 28,240.63
	B	100	14	10.5	7656.25	B	100	15	10.5	8203.125	
	C	144	5	10.5	3937.5	C	144	5	10.5	3937.5	
	D	196	1	10.5	1071.875	D	196	1	10.5	1071.875	
	E	256	0	10.5	0	E	256	0	10.5	0	
	F	144	1	10.5	787.5	F	144	2	10.5	1575	
5th	A	100	0	10.5	0	A	100	0	10.5	0	5th Floor Columns = 29,334.38
	B	100	15	10.5	8203.125	B	100	16	10.5	8750	
	C	144	5	10.5	3937.5	C	144	5	10.5	3937.5	
	D	196	1	10.5	1071.875	D	196	1	10.5	1071.875	
	E	256	0	10.5	0	E	256	0	10.5	0	
	F	144	2	10.5	1575	F	144	1	10.5	787.5	
6th	A	100	0	10.5	0	A	100	0	10.5	0	6th Floor Columns = 28,612.5
	B	100	16	10.5	8750	B	100	18	10.5	9843.75	
	C	144	5	10.5	3937.5	C	144	4	10.5	3150	
	D	196	1	10.5	1071.875	D	196	1	10.5	1071.875	
	E	256	0	10.5	0	E	256	0	10.5	0	
	F	144	1	10.5	787.5	F	144	0	10.5	0	
7th	A	100	0	10.5	0	A	100	0	10.5	0	7th Floor Columns = 27,846.88
	B	100	18	10.5	9843.75	B	100	18	10.5	9843.75	
	C	144	4	10.5	3150	C	144	4	10.5	3150	
	D	196	1	10.5	1071.875	D	196	0	10.5	0	
	E	256	0	10.5	0	E	256	0	10.5	0	
	F	144	0	10.5	0	F	144	1	10.5	787.5	

Note: Column Weights for Floors 8 - 13 are displayed on next Table. Columns were designed using Normal Weight Concrete. Therefore, a weight of 150 PCF was used in the self weight calculations.

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Lateral System – Calculation of Building Weight Continued:

Building Weight Due to Columns - Broken Down Per Floor For Seismic Calculations												
Floor Level	Column Below - Specification Letter	Column Below - Gross Section Area (in^2)	# Identical Columns	Column Height (ft)	Total Weight Per Column (lbs)	Column Above - Specification Letter	Column Above - Gross Section Area (in^2)	# Identical Columns	Column Height (ft)	Total Weight Per Column (lbs)	Total Weight Per Floor (lbs)
8th	A	100	0	10.5	0	A	100	0	10.5	0	9th Floor Columns = 27,365.63
	B	100	18	10.5	9843.75	B	100	20	10.5	10937.5	
	C	144	4	10.5	3150	C	144	2	10.5	1575	
	D	196	0	10.5	0	D	196	1	10.5	1071.875	
	E	256	0	10.5	0	E	256	0	10.5	0	
	F	144	1	10.5	787.5	F	144	0	10.5	0	
9th	A	100	0	10.5	0	A	100	1	10.5	546.875	10th Floor Columns = 26,862.5
	B	100	20	10.5	10937.5	B	100	17	10.5	9296.875	
	C	144	2	10.5	1575	C	144	3	10.5	2362.5	
	D	196	1	10.5	1071.875	D	196	1	10.5	1071.875	
	E	256	0	10.5	0	E	256	0	10.5	0	
	F	144	0	10.5	0	F	144	0	10.5	0	
10th	A	100	1	10.5	546.875	A	100	1	10.5	546.875	11th Floor Colulmns = 26,271.88
	B	100	17	10.5	9296.875	B	100	17	10.5	9296.875	
	C	144	3	10.5	2362.5	C	144	4	10.5	3150	
	D	196	1	10.5	1071.875	D	196	0	10.5	0	
	E	256	0	10.5	0	E	256	0	10.5	0	
	F	144	0	10.5	0	F	144	0	10.5	0	
11th	A	100	1	10.5	546.875	A	100	0	10.5	0	11th Floor Columns = 25,987.5
	B	100	17	10.5	9296.875	B	100	18	10.5	9843.75	
	C	144	4	10.5	3150	C	144	4	10.5	3150	
	D	196	0	10.5	0	D	196	0	10.5	0	
	E	256	0	10.5	0	E	256	0	10.5	0	
	F	144	0	10.5	0	F	144	0	10.5	0	
12th	A	100	0	10.5	0	A	100	5	10.5	2734.375	12th Floor Columns = 25,987.5
	B	100	18	10.5	9843.75	B	100	13	10.5	7109.375	
	C	144	4	10.5	3150	C	144	3	10.5	2362.5	
	D	196	0	10.5	0	D	196	0	10.5	0	
	E	256	0	10.5	0	E	256	0	10.5	0	
	F	144	0	10.5	0	F	144	1	10.5	787.5	
13th	A	100	5	10.5	2734.375	A	100	0	10.5	0	13th Floor Columns = 12993.75
	B	100	13	10.5	7109.375	B	100	0	10.5	0	
	C	144	3	10.5	2362.5	C	144	0	10.5	0	
	D	196	0	10.5	0	D	196	0	10.5	0	
	E	256	0	10.5	0	E	256	0	10.5	0	
	F	144	1	10.5	787.5	F	144	0	10.5	0	

F-102 Continued

In addition to the column weight, the building envelope dead load had to be considered. Once again, ½ the story above and ½ the story below contributed to each story level. The perimeter of each story level and a 20 PSF dead load weight were used to calculate the dead load. The following table summarizes these calculations and results.

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Building Envelope Dead Load Broken Down By Level (lbs)					
Floor	h/2 below (ft)	h/2 above (ft)	Dead Load (PSF)	Perimeter of Building at Corresponding Story Elevation	Weight = 20*Perimeter*(h/2 below + h/w above) (lbs)
Ground	0	10.5	20	274	57540
1	10.5	5.25	20	274	86310
2	5.25	5.25	20	270	56700
3	5.25	5.25	20	270	56700
4	5.25	5.25	20	270	56700
5	5.25	5.25	20	270	56700
6	5.25	5.25	20	270	56700
7	5.25	5.25	20	270	56700
8	5.25	5.25	20	270	56700
9	5.25	5.25	20	270	56700
10	5.25	5.25	20	260	54600
11	5.25	5.25	20	260	54600
12	5.25	5.25	20	260	54600
13	5.25	5.25	20	260	54600
Penthouse	5.25	5.25	20	252	52920
Bulkhead Roof	5.25	0	20	156	16380
				Total Building Envelope (lbs)	885,150

Lateral System – Calculation Of Building Weight Continued:

Once the self weight contributions from all the structure's components were considered, the values were combined together. The following table shows the total dead load weight per floor as well as the total overall building weight. As shown in the table, the total building weight for the new concrete building design is **7,758 kips**. This is approximately 1.66 times heavier than the original 4,681 kips steel structure.

Final Building Weight (Shear Walls Not Included)		
Floor	**Weight (lbs)**	**Weight (Kips)**
Ground	100,000	100
1	515,639	515.639
2	343,944	343.944
3	291,713	291.713
4	290,619	290.619
5	291,713	291.713
6	290,992	290.992
7	290,244	290.244
8	289,744.63	289.74463
9	289,242	289.242
10	336,289	336.289
11	336,006	336.006
12	336006	336.006
13	325018	325.018
Roof	100,000	100
Total Building Weight:	**4427169.63**	**4427.16963**

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Final Building Weight (Shear Walls Included)		
Floor	**Weight (lbs)**	**Weight (Kips)**
Ground	305,000	305
1	720,639	720.639
2	548,944	548.944
3	496,713	496.713
4	495,619	495.619
5	752,963	752.963
6	495,992	495.992
7	495,244	495.244
8	494,744.63	494.74463
9	494,242	494.242
10	541,289	541.289
11	541,006	541.006
12	541006	541.006
13	530018	530.018
Roof	305,000	305
Total Building Weight:	**7758419.63**	**7758.41963**

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Seismic Force Calculations:

The primary goal of these calculations is to determine the seismic base shear V. V is a function of the design response accelerations S_{DS} and S_{D1}, the importance factor I, modification coefficient R, the fundamental period of the structure T, and the effective seismic weight W.

Flow Chart 6.1 – consideration of seismic design requirements.

11.1.2 Every structure, and portion therefore, including nonstructural components, shall be designed and constructed to resist the effects of earthquake motions

Is the structure a detached 1 or 2 story family dwelling? NO
Is the structure an agricultural storage structure? NO
Does the structure require special considerations? NO
Conclusion: Seismic requirements of ASCE/SEI 7-05 must be considered.

Flow Chart 6.2 – Seismic ground motion values (11.4)

Determine the parameters S_s and S_1 from the 0.2 and 1.0 sec spectral response accelerations on figure 22-1 through 22-14 (11.4.1):

New York City: S_s = 0.35 S_1 = .062

Is $S_s \leq 0.15$ and $S_1 \leq .04$? No

Is the structure seismically isolated or does it have damping systems on site with $S_1 \geq 0.6$? NO

Determine Site Class of Soil In Accordance with 11.4.2 and Chapter 20.

Table 20.3-1 based on upper 100 ft of site profile: Therefore Review Geotechnical Report. According to geotechnical report, the fill stratum consists of heterogeneous mixtures of fine to coarse sand, gravel, silt, brick, and concrete fragments. Immediately below the fill, there are silty sands of medium density, which extends beyond the deepest boring penetration. A robust liquefaction analysis was done due to poor/loose site conditions. Ultimately it was concluded that liquefaction is unlikely to occur under seismic ground shaking.

To be conservative, the site soil class is categorized as: Soil Site Class D

Determine S_{MS} and S_{M1}:

$$S_{MS} = F_a S_s = 1.52 * .35 = .53$$

$$S_{M1} = F_V S_1 = 2.4 * .062 = .1488$$

Table 11.4-1: Using Linear Interpolation Between Values: F_a = 1.52
Table 11.4-2: F_v = 2.4

Linear Interpolation Calculation: $F_a = 1.6 - \frac{(0.35-0.25)}{(0.50-0.25)}(1.6-1.4) = 1.52$

Seismic Force Calculations Continued:

Determine S_{DS} and S_{D1}:

$$S_{DS} = \frac{2S_{MS}}{3} = \frac{2(.532)}{3} = .3547$$

$$S_{D1} = \frac{2S_{M1}}{3} = \frac{2(.1488)}{3} = .0992$$

Flowchart 6.4 – Seismic Design Category (SDC):

Occupancy = II

$S_1 \geq 0.75$? NO

Table 11.6-1: SDC based on short period response acceleration parameter and occupancy category: SCD = C

Flow Chart 6.8 – Equivalent Lateral Force Procedure

Determine The Response Modification Factor R

Table 12.2-1: Detailed Plain Concrete Shear Walls: R = 2
*System Overstrength Factor: $\Omega_o^g = 2.5$
*No specified Building Height Limitation

Considering structural properties and deformational characteristics, calculate Fundamental Period of Building:

$$T_a = C_t h_n^x = .02 * 186^{.75} = 1.01$$

According to 12.8-7, structural system is under "all other structural systems". Therefore: $C_t = .02$ and $X = .75$

Determine T_s: $T_S = \frac{S_{D1}}{S_{DS}} = .\frac{0992}{.3547} = .2797$

Is $T \leq 3.5T_S$? NO

Determine the Long Period Transition Period: Figure 22-15, New York City: $T_L = 6$

Is $T \geq T_L$? NO

Determine Seismic Response Coefficient (C_S):

For $T \leq T_L$, $C_S = \frac{S_{D1}}{T(\frac{R}{I})} = \frac{0.0992}{1.001(\frac{2}{1})} = .04955$

Seismic Force Calculations Continued:

CHECK: $C_S \geq 1.0$, OKAY

CHECK: $C_S \leq \frac{S_{DS}}{\left(\frac{R}{I}\right)} = \frac{0.3547}{\left(\frac{2}{1}\right)} = .17735$ OKAY

Calculate Base Shear V: $V = C_S * W = .049 * (7502\ kips) = 367.6\ kips$

Determine lateral seismic story forces F_X at each level: (Exponent Related to Structure K = 2)

$$\boxed{F_X = \frac{W_X h_X^k}{\sum W_i h_i^k} * V}$$

Note: Seismic story forces are organized in the following Table:

SEISMIC STORY FORCES – CALCULATIONS AND RESULTS

Floor Level	Height (feet)	Total Weight (kips)	Exponent Related To Structure	Weight*Height^k	Wx*hxk / (∑Wx*hxk)	Base Shear (kips)	Lateral Seismic Force	Story Shear
	hx	Wx	K	Wx*hx^k		V	Fx (kips)	Vx (kips)
Ground / 1st	0	305	2	0	0	367.60	0	367.6
2nd Floor	21.667	720.6	2	338292.0754	0.005688961	367.60	2.091262126	367.6
3rd Floor	32.4167	548.9	2	576807.4147	0.009700005	367.60	3.565722014	365.5087379
4th Floor	43.1667	496.7	2	925532.8933	0.015564422	367.60	5.721481603	361.9430159
5th Floor	53.9167	495.6	2	1440714.423	0.024228083	367.60	8.906243232	356.2215343
6th Floor	64.667	496.7	2	2077110.436	0.034930173	367.60	12.84033148	347.315291
7th Floor	75.4167	495.2	2	2816538.462	0.047364923	367.60	17.41134552	334.4749595
8th Floor	86.167	494.7	2	3673024.759	0.061768208	367.60	22.70599321	317.063614
9th Floor	96.9167	494.2	2	4641944.858	0.07806226	367.60	28.69568689	294.3576208
10th Floor	107.667	541.2	2	6273689.38	0.105502842	367.60	38.7828446	265.6619339
11th Floor	118.4167	541	2	7586180.528	0.12757463	367.60	46.89643409	226.8790893
12th Floor	129.167	541.6	2	9036116.082	0.151957782	367.60	55.85968074	179.9826552
13th Floor	139.9167	530	2	10375641.96	0.174484206	367.60	64.14039416	124.1229745
Penthouse	150.667	305	2	6923666.191	0.116433316	367.60	42.80088696	59.98258035
Roof	162.667	50	2	1323027.644	0.022248978	367.60	8.178724261	17.18169339
Bulkhead Roof	170.667	50	2	1456361.244	0.024491211	367.60	9.002969131	9.002969131
				∑Wx*hx^k= 59,464,648				

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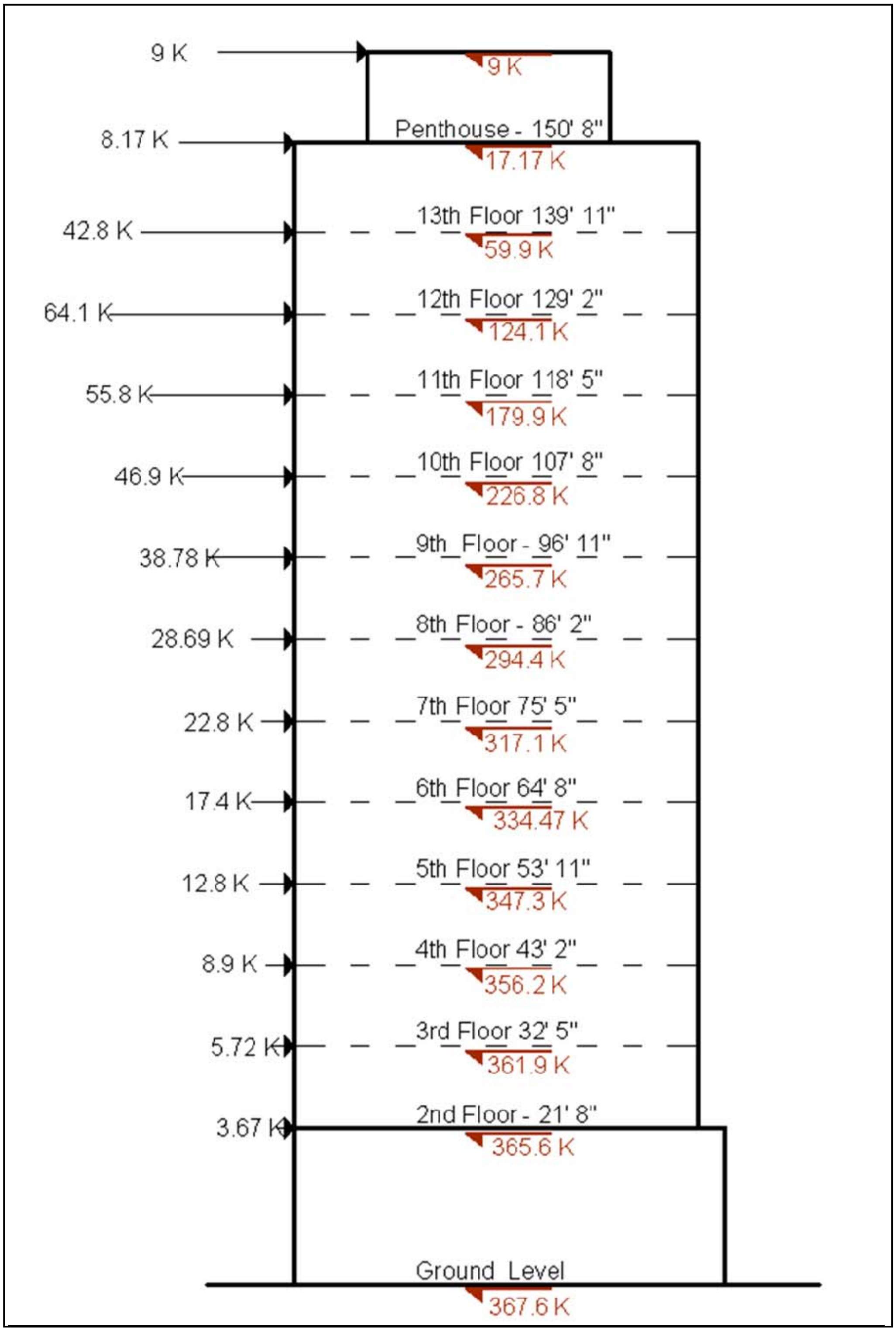
Seismic Force Diagram:

This diagram shows the calculated seismic design forces. Note the base shear, shown on the diagram, is 367 kips. For comparison purposes, a diagram of the original seismic story forces and story shears are shown on the next page.



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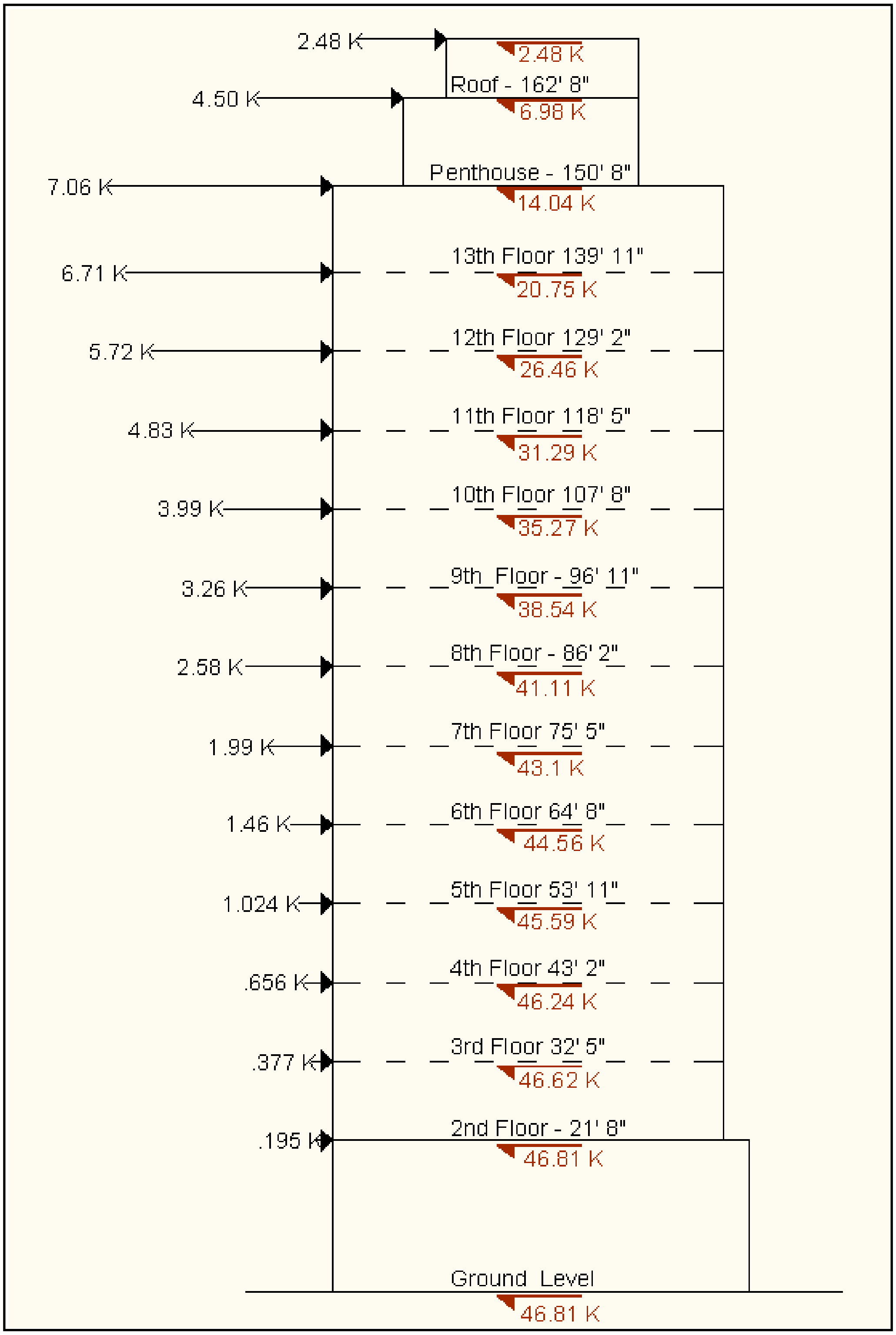
Seismic Design Force Diagram For Original Steel Structure Design:



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Wind Load Calculations:

The actual wind loads calculated for the 40 Gold Street Design were done according to ASCE7-02. For the Thesis calculations, wind load pressures were obtained by following Method 2 for the main wind-force resisting system for enclosed buildings and referencing the IBC 2006 1609.1.1 and Chapter 6 of ASCE/SEI 7-05 (ASCE7). These wind calculations are summarized in the following tables and diagrams.

Summary of Wind Calculations: Variables and Classifications (EAST/WEST DIRECTION)					
Basic Wind Speed (V)	110 mph	damping ratio (b)	1.5	Qp	29.47
Wind Directionality Factor (Kd)	0.85	natural frequency (n)	0.363	GCpn (windward)	1.5
Importance Factor (I)	1	Z	102.4	GCpn(Leeward)	-1
Exposure Category	B	Iz	0.4968	Pp(windward)	44.207
Topographic Factor (Kzt)	1	Lz	466.74	Pp(leeward)	-29.47
Alpha	7	Q	0.838	Cp (windward)	0.8
Zg	1200	Vz	96.357	Cp(leeward)	-0.5
a	0.50	N1	1.7583	Cp(side walls)	-0.7
b	0.84	Rn	0.0962	Gcpi	0.18 / -0.18
c	0.3	Rb	0.4837	Mean Roof Height	170' 8"
l	320	Rl	0.2575	Enclosure Type	Fully Enclosed
e	0.3333	R	0.09226	Rigidity	Flexible
Zmin	30	Gr	3.788	Parapet	4' high parapet
alpha	0.25	Gf	0.8845	Topography	No Hill / No Escarpment

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Summary of Wind Calculations: Variables and Classifications (NORTH/SOUTH DIRECTION)					
Basic Wind Speed (V)	110 mph	damping ratio (b)	1.5	Qp	0
Wind Directionality Factor (Kd)	0.85	natural frequency (n)	0.363	GCpn (windward)	1.5
Importance Factor (I)	1	Z	102.4	GCpn(Leeward)	-1
Exposure Category	B	Iz	0.4968	Pp(windward)	0
Topographic Factor (Kzt)	1	Lz	466.74	Pp(leeward)	0
Alpha	7	Q	0.8449	Cp (windward)	0.8
Zg	1200	Vz	96.357	Cp(leeward)	-0.42
a	0.50	N1	1.7583	Cp(side walls)	-0.7
b	0.84	Rn	0.0962	Gcpi	0.18 / -0.18
c	0.3	Rb	0.2605	Mean Roof Height	170' 8"
l	320	Rl	0.1961	Enclosure Type	Fully Enclosed
e	0.3333	R	0.07179	Rigidity	Flexible
Zmin	30	Gr	3.788	Parapet	No Parapet
alpha	0.25	Gf	0.8877	Topography	No Hill / No Escarpment

***Note:** The Orange highlighted cells represent values in the North/South Wind Pressure Calculations that differ from the East/West Wind Pressure Calculations. These differences were due to the changes in building dimensions.

Wind Design Load Tables:

(WIND PRESSURES)

Calculated Wind Pressures for the EAST / WEST Direction													
			B = 78' 2-1/2"				L = 56' 9-1/2"						
Story	**Story Height**	**Height**	**k_z**	**k_{zt}**	**k_d**	**V**	**I**	**$q_{z\,(psf)}$**	**G_f**	**C_{pw}**	**C_{pL}**	**$p_{z\,(windward)}$**	**$p_{z\,(Leeward)}$**
2	21' 8"	21'8"	0.7	1.00	0.85	110	1.00	18.43	0.8845	0.8	-0.5	**7.587**	**-7.9476**
3	10' 9"	32'5"	0.7145	1.00	0.85	110	1.00	18.81	0.8845	0.8	-0.5	**7.857**	**-7.9476**
4	10' 9"	43'2"	0.7158	1.00	0.85	110	1.00	18.85	0.8845	0.8	-0.5	**7.881**	**-7.9476**
5	10' 9"	53'11"	0.8256	1.00	0.85	110	1.00	21.74	0.8845	0.8	-0.5	**9.927**	**-7.9476**
6	10' 9"	64'8"	0.8687	1.00	0.85	110	1.00	22.87	0.8845	0.8	-0.5	**10.730**	**-7.9476**
7	10' 9"	75'5"	0.9117	1.00	0.85	110	1.00	24.00	0.8845	0.8	-0.5	**11.531**	**-7.9476**
8	10' 9"	86'2"	0.9485	1.00	0.85	110	1.00	24.97	0.8845	0.8	-0.5	**12.216**	**-7.9476**
9	10' 9"	96'11"	0.98075	1.00	0.85	110	1.00	25.82	0.8845	0.8	-0.5	**12.817**	**-7.9476**
10	10' 9"	107'8"	1.009	1.00	0.85	110	1.00	26.57	0.8845	0.8	-0.5	**13.344**	**-7.9476**
11	10' 9"	118'5"	1.036	1.00	0.85	110	1.00	27.28	0.8845	0.8	-0.5	**13.847**	**-7.9476**
12	10' 9"	129'2"	1.063	1.00	0.85	110	1.00	27.99	0.8845	0.8	-0.5	**14.350**	**-7.9476**
13	10' 9"	139'11"	1.089	1.00	0.85	110	1.00	28.67	0.8845	0.8	-0.5	**14.834**	**-7.9476**
Penthouse	10' 9"	150'8"	1.111	1.00	0.85	110	1.00	29.25	0.8845	0.8	-0.5	**15.244**	**-7.9476**
Roof	12' 0"	162'8"	1.135	1.00	0.85	110	1.00	29.88	0.8845	0.8	-0.5	**15.691**	**-7.9476**
Bulkhead Roof	8' 0"	170'8"	1.151	1.00	0.85	110	1.00	30.31	0.8845	0.8	-0.5	**15.989**	**-7.9476**

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Calculated Wind Pressures for the North / South Direction													
			B = 56' 9-1/2"				L = 78' 9-1/2"						
Story	**Story Height**	**Height**	**k_z**	**k_{zt}**	**k_d**	**V**	**I**	**$q_{z\,(psf)}$**	**G_f**	**C_{pw}**	**C_{pL}**	**$p_{z\,(windward)}$**	**$p_{z\,(Leeward)}$**
2	21' 8"	21'8"	0.7	1.00	0.85	110	1.00	18.43	0.8877	0.8	-0.42	**7.634**	**-5.84391**
3	10' 9"	32'5"	0.7145	1.00	0.85	110	1.00	18.81	0.8877	0.8	-0.42	**7.905**	**-5.84391**
4	10' 9"	43'2"	0.7158	1.00	0.85	110	1.00	18.85	0.8877	0.8	-0.42	**7.929**	**-5.84391**
5	10' 9"	53'11"	0.8256	1.00	0.85	110	1.00	21.74	0.8877	0.8	-0.42	**9.982**	**-5.84391**
6	10' 9"	64'8"	0.8687	1.00	0.85	110	1.00	22.87	0.8877	0.8	-0.42	**10.788**	**-5.84391**
7	10' 9"	75'5"	0.9117	1.00	0.85	110	1.00	24.00	0.8877	0.8	-0.42	**11.592**	**-5.84391**
8	10' 9"	86'2"	0.9485	1.00	0.85	110	1.00	24.97	0.8877	0.8	-0.42	**12.280**	**-5.84391**
9	10' 9"	96'11"	0.98075	1.00	0.85	110	1.00	25.82	0.8877	0.8	-0.42	**12.883**	**-5.84391**
10	10' 9"	107'8"	1.009	1.00	0.85	110	1.00	26.57	0.8877	0.8	-0.42	**13.412**	**-5.84391**
11	10' 9"	118'5"	1.036	1.00	0.85	110	1.00	27.28	0.8877	0.8	-0.42	**13.916**	**-5.84391**
12	10' 9"	129'2"	1.063	1.00	0.85	110	1.00	27.99	0.8877	0.8	-0.42	**14.421**	**-5.84391**
13	10' 9"	139'11"	1.089	1.00	0.85	110	1.00	28.67	0.8877	0.8	-0.42	**14.907**	**-5.84391**
Penthouse	10' 9"	150'8"	1.111	1.00	0.85	110	1.00	29.25	0.8877	0.8	-0.42	**15.319**	**-5.84391**
Roof	12' 0"	162'8"	1.135	1.00	0.85	110	1.00	29.88	0.8877	0.8	-0.42	**15.768**	**-5.84391**
Bulkhead Roof	8' 0"	170'8"	1.151	1.00	0.85	110	1.00	30.31	0.8877	0.8	-0.42	**16.067**	**-5.84391**

F-112

Table F-111 Is Illustrated in Diagram F-115 and F-116

Table F-112 Is Illustrated in Diagram F-114 and F-117

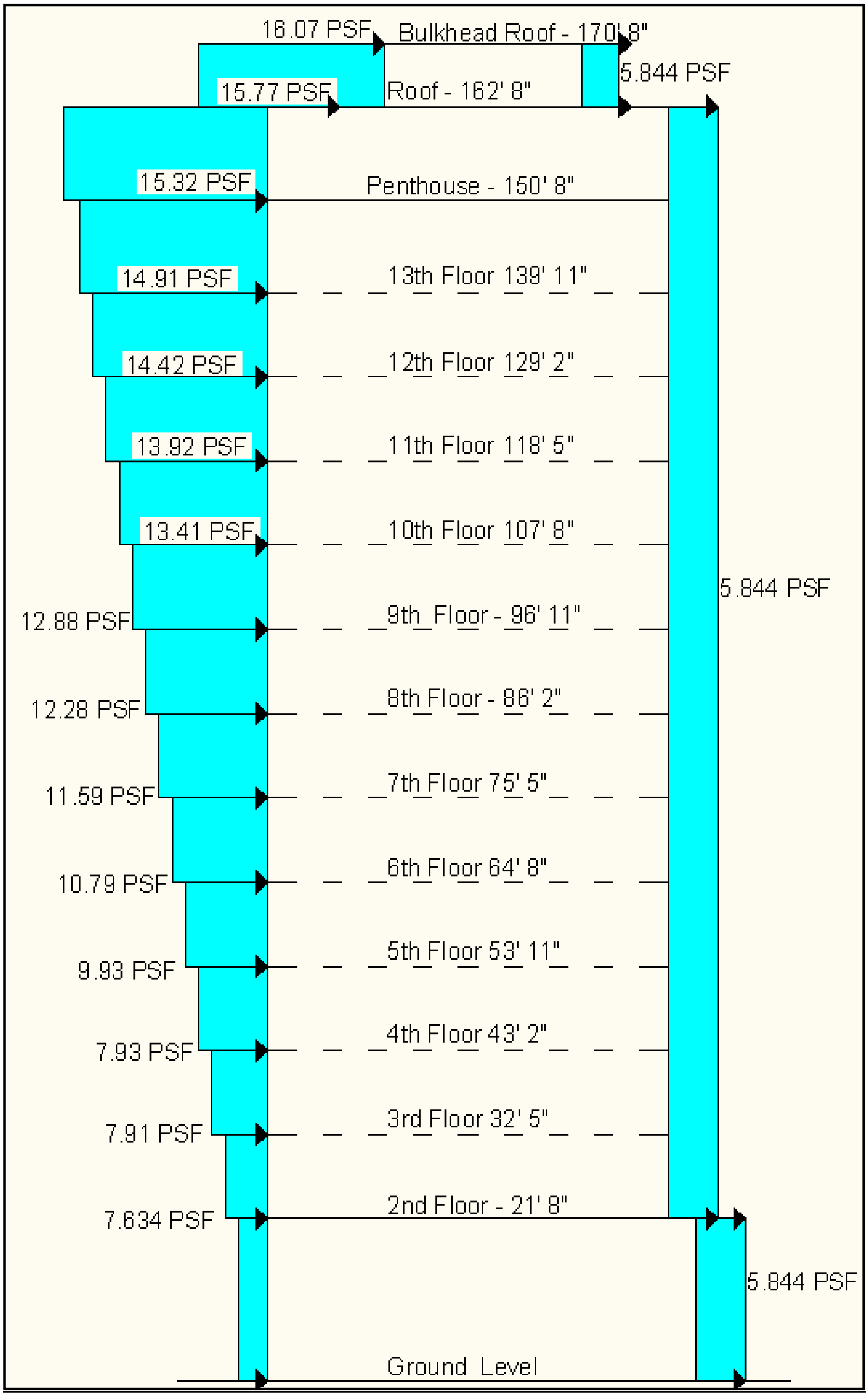
WIND CALCULATIONS: STORY SHEARS AND OVERTURNING MOMENT

WIND FORCES, STORY SHEARS, OVERTURNING MOMENT										
EAST/WEST DIRECTION										
story	story height	height from ground	Tributary Area Below	Tributary Area Above	Pz below	Pz Above	Fx	Vx	Moment Contribution Fx*(height)	Overturing Moment (Kip*ft)
	ft	ft	SF	SF	PSF	PSF	kips	kips		
2	21.667	21.667	845	419	7.587	7.857	9.703	139.328	210.2370244	
3	10.75	32.417	419	419	7.857	7.881	6.594	129.625	213.7648946	
4	10.75	43.167	419	419	7.881	9.927	7.462	123.031	322.0928152	
5	10.75	53.917	419	419	9.927	10.73	8.655	123.031	466.6668935	
6	10.75	64.667	419	419	10.73	11.531	9.327	106.914	603.1723245	
7	10.75	75.417	419	419	11.531	12.216	9.950	97.587	750.3986221	
8	10.75	86.167	419	419	12.216	12.817	10.489	87.637	903.7907561	
9	10.75	96.917	419	419	12.817	13.344	10.961	77.148	1062.351722	
10	10.75	107.667	419	419	13.344	13.847	11.393	66.186	1226.653253	
11	10.75	118.417	419	419	13.847	14.35	11.815	54.793	1399.042738	
12	10.75	129.167	419	419	14.35	14.834	12.228	42.979	1579.466476	
13	10.75	139.917	419	419	14.834	15.244	12.603	30.751	1763.329457	
Penthouse	10.75	150.667	419	468	15.244	15.691	13.731	18.148	2068.751926	
Roof	12	162.667	200	40	15.691	15.989	3.778	4.417	614.5168859	
Bulkhead	8	170.667	40	0	15.989	0	0.640	0.640	109.1517865	13293.38758
NORTH/SOUTH DIRECTION										
2	21.667	21.667	614	305	7.634	7.905	7.098	98.568	153.7988878	
3	10.75	32.417	305	305	7.905	7.929	4.829	91.470	156.5536873	
4	10.75	43.167	305	305	7.929	9.982	5.463	86.641	235.8150618	
5	10.75	53.917	305	305	9.982	10.788	6.335	86.641	341.5561075	
6	10.75	64.667	305	305	10.788	11.592	6.826	74.843	441.4104753	
7	10.75	75.417	305	305	11.592	12.28	7.281	68.017	549.1081603	
8	10.75	86.167	305	305	12.28	12.883	7.675	60.736	661.3071674	
9	10.75	96.917	305	305	12.883	13.412	8.020	53.061	777.2719171	
10	10.75	107.667	305	305	13.412	13.916	8.335	45.041	897.4087517	
11	10.75	118.417	305	305	13.916	14.421	8.643	36.706	1023.452671	
12	10.75	129.167	305	305	14.421	14.907	8.945	28.064	1155.403982	
13	10.75	139.917	305	305	14.907	15.319	9.219	19.119	1289.885029	
Penthouse	10.75	150.667	305	125	15.319	15.768	6.643	9.900	1000.925328	
Roof	12	162.667	125	40	15.768	16.067	2.614	3.256	425.1594846	
Bulkhead	8	170.667	40	0	16.067	0	0.643	0.643	109.6842676	9218.740978

F-113

Wind Load Diagram N/S Direction

WINDWARD

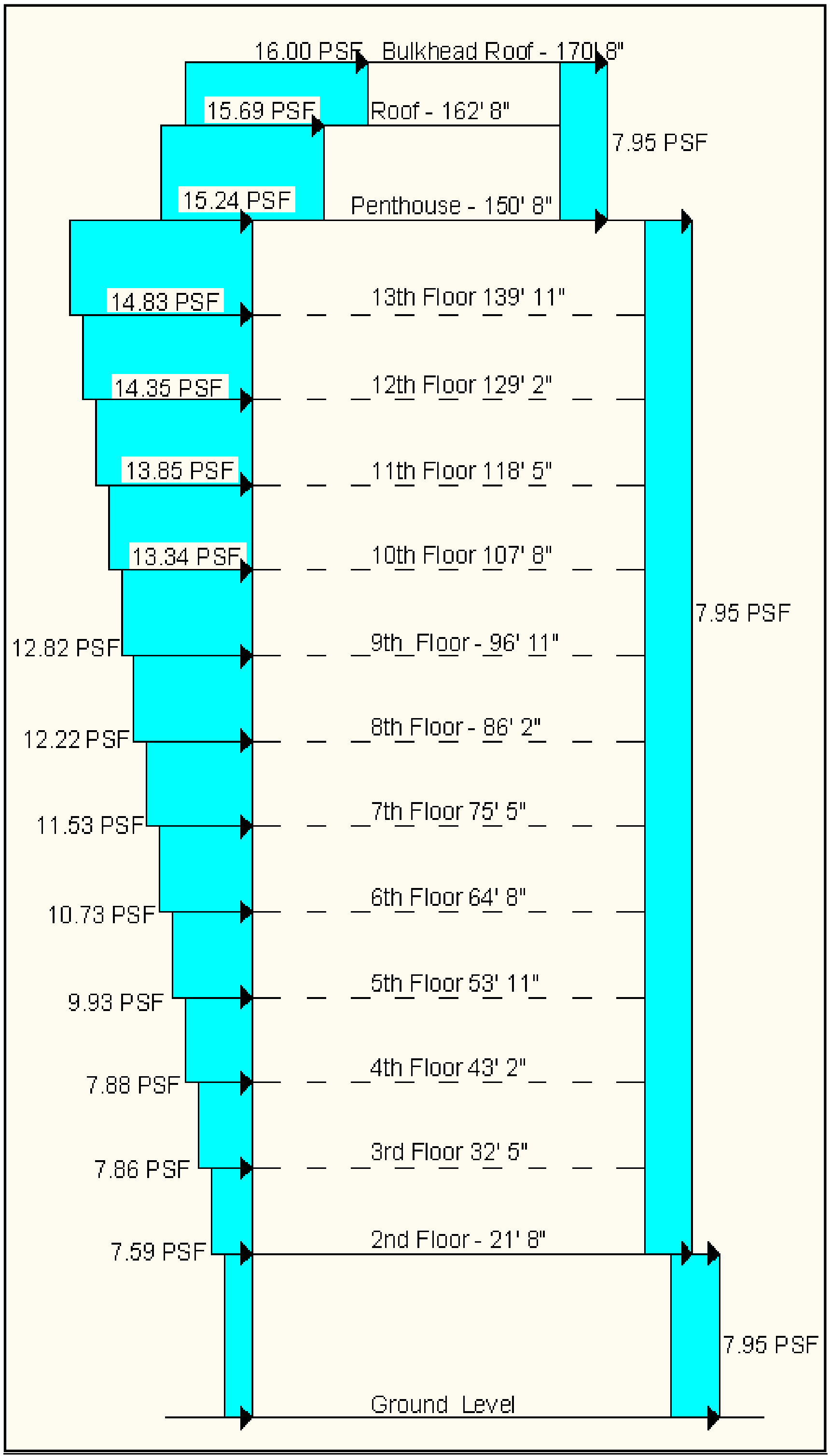


LEEWARD

F-114

Wind Load Diagram E/W Direction:

WINDWARD

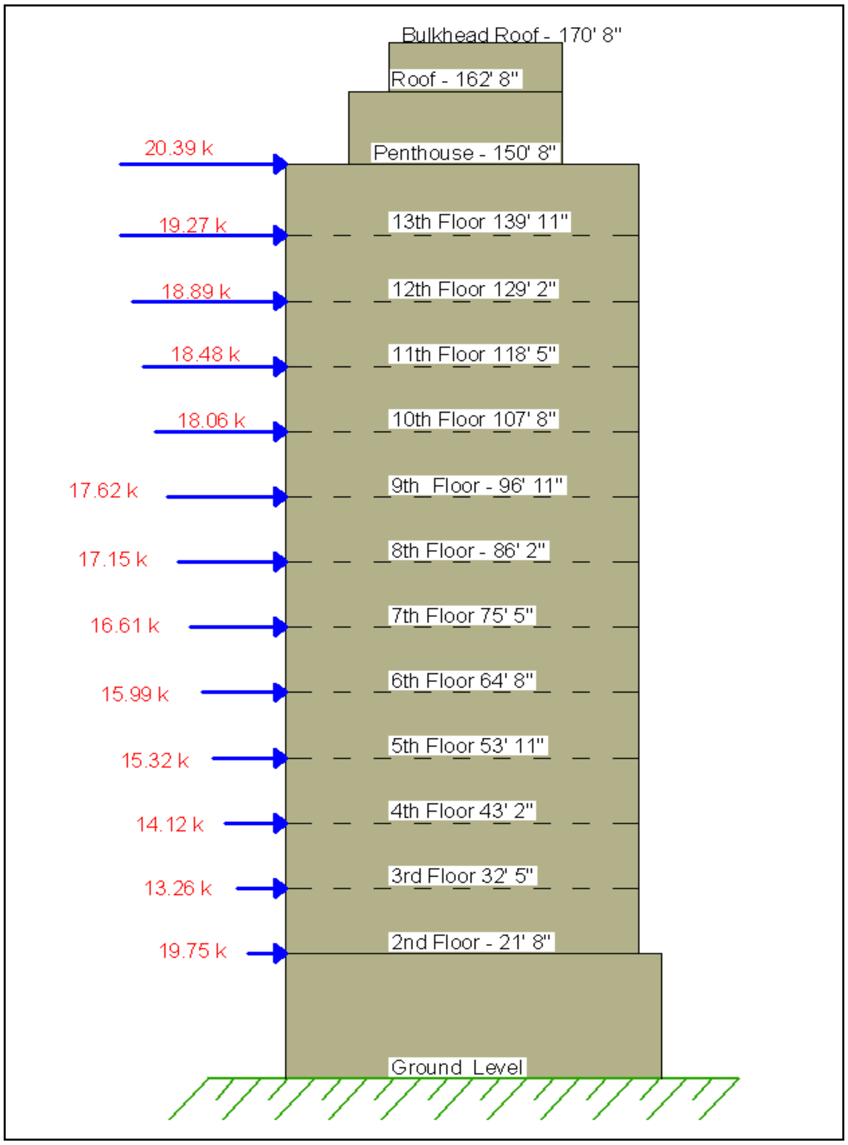


LEEWARD

F-115

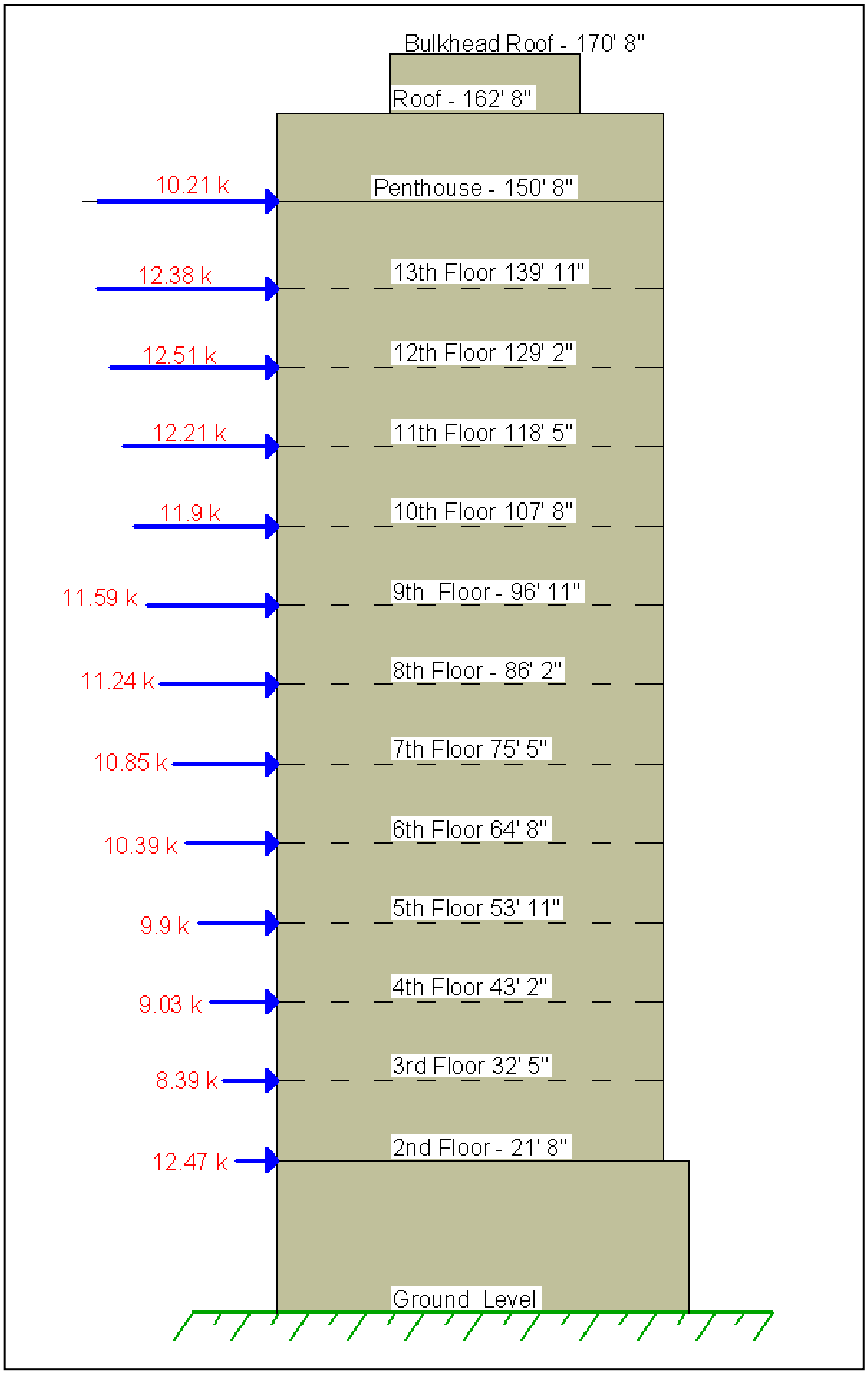
<u>East / West Wind Diagram (X Direction)</u>:

The next two diagrams represent the calculated wind story forces. Note, these forces were determined using ½ the tributary area above and below each story level.



F-116

<u>North/South Wind Diagram (Y Direction)</u>:



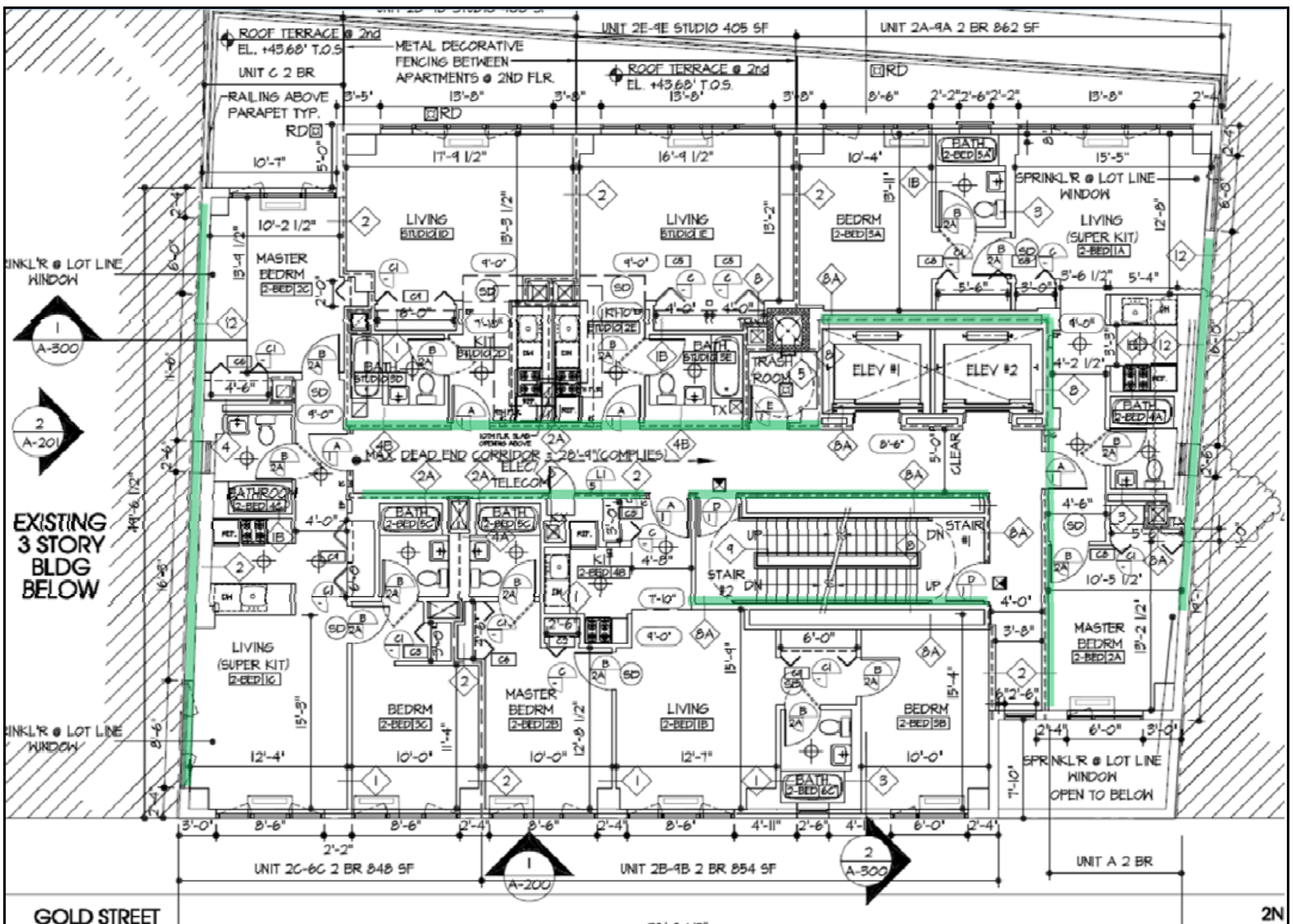
F-117

<u>Preliminary Lateral System Design</u>

Prior to modeling and designing the lateral system in ETABs for the above lateral loads, an in-depth, multi-faceted analysis was performed to determine the most efficient shear wall layout. First, a thorough examination of the architectural drawings was conducted to determine what locations were most feasible for placement of reinforced concrete shear walls. The figure F-118 below shows the preliminary shear wall layout overlaying a typical architectural plan. As one can see, the optimum location for shear walls was determined to be at the exterior, along the perimeter of the main residential corridor, and around the central stairwell and elevator shaft. As a result of architectural restraints, the shear wall layout lacks geometric symmetry. Therefore, to minimize torsion effects, distribution of lateral stiffness throughout the structure is integral to counteracting the lateral system's geometric irregularities.

<u>PRELIMINARY SHEAR WALL LAYOUT – BASED ON ARCHITECTURAL RESTRAINTS</u>

Note: This figure shows shear walls (green) overlaying a typical Architectural Plan.



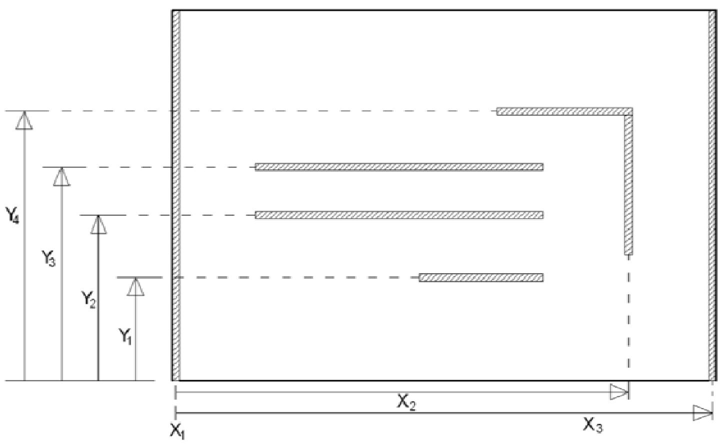
F-118

Preliminary Lateral System Design Calculations:

Next, preliminary hand calculations were performed to determine the center of rigidity, center of mass, and eccentricity associated with the preliminary shear wall layout. These calculations are summarized on the following pages. The purpose of these hand calculations was to determine the direct shear forces and torsion effects present in the shear wall system under lateral loading. Based upon the results, conclusions were drawn as how to distribute lateral stiffness throughout the building to efficiently minimize torsion effects. By properly distributing the lateral stiffness, the eccentricity of the Center of rigidity with respect to the center of mass can be reduced.

Preliminary Hand Calculations - Lateral Load Distribution and Design: These hand calculations are a brief investigation of the direct shear forces, the torsion effects, and the eccentricity of center of rigidity for the proposed shear wall layout illustrated in figure F-118.

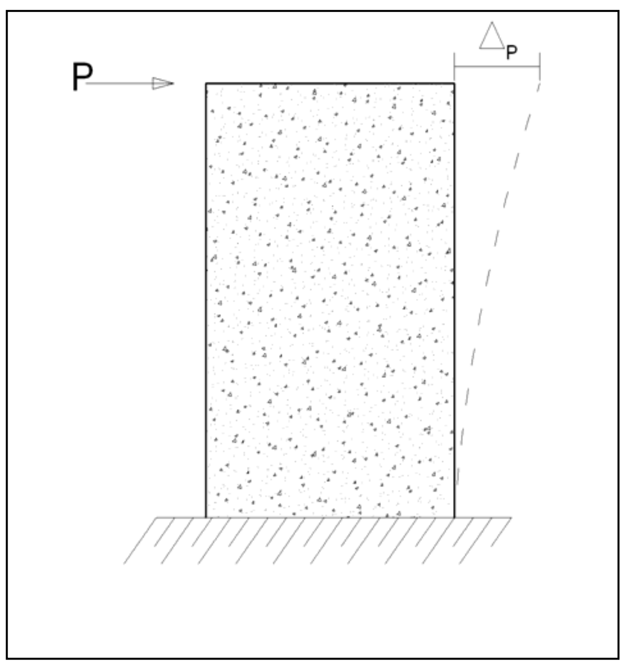
Step One -Determine The Shear Wall Lengths and Locations (refer to diagram below):



DIMENSIONS OF SHEAR WALL LAYOUT		
Shear Wall	Location	Length
W_1	$Y_1 =$ 14.5'	17.3'
W_2	$Y_2 =$ 23.25'	48'
W_3	$Y_3 =$ 30.0'	48'
W_4	$Y_4 =$ 37.8'	22'
W_5	$X_5 =$ 0'	52'
W_6	$X_6 =$ 64.0'	23'
W_7	$X_7 =$ 76.0'	52'

Step Two - Determine Stiffness Values:

The concept behind the calculation of stiffness values K relates to the equation: . Beside shear wall W_5 and W_7, the shear walls deflections are all controlled by flexure since their dimensions satisfy the equation: categorizing them as "tall walls". The governing equation used in the hand calculations of the shear wall stiffness values was: . It was first assumed all walls have the same thickness, height, modulus of elasticity. Therefore, the stiffness values K are proportional to for flexure and for shear.



Preliminary Hand Calculations - Lateral Load Distribution and Design Continued:

Comparing the wind and seismic story forces, it appears from initial observations that the seismic forces will control the design of the shear walls since they are so much larger. As shown on the seismic story force diagram, the story shear for floor 8 is 294.4 kips. For preliminary calculations, floor 8 was arbitrarily chosen for analysis of lateral force distribution.

Step Three - Determine Center of Rigidity Location (X, Y):

$$\boxed{X = \frac{\sum K_{iy}X_i}{\sum K_{iy}}} = \frac{\left(\frac{Et}{384} * 0\right) + \left(\frac{Et}{1469} * 64\right) + \left(\frac{Et}{384} * 76\right)}{\left(\frac{2Et}{384}\right) + \left(\frac{Et}{1469}\right)} = \frac{0.2414Et}{0.005889Et} = \mathbf{40.99'}$$

$$\boxed{Y = \frac{\sum K_{ix}Y_i}{\sum K_{ix}}} = \frac{\left(\frac{Et}{3434} * 14.5\right) + \left(\frac{Et}{169} * 23.25\right) + \left(\frac{Et}{169} * 30\right) + \left(\frac{Et}{1678 * 37.8}\right)}{\left(\frac{2Et}{169}\right) + \left(\frac{Et}{1678}\right) + \left(\frac{Et}{3434}\right)} = \frac{03418Et}{0.01272Et} = \mathbf{26.867'}$$

C.O.R. = (40.99’,26.87’)

C.O.M. = (38’,26’)

Eccentricity x = 2.99’

Eccentricity y = .867’

Step Four – Determine Direct Shear Forces:

Governing equation: $F_{iy} = \frac{K_{iy}}{\sum I_{iy}} P_y$

$$F_{5Direct} = \frac{\left(\frac{Et}{384}\right)}{\frac{2Et}{384} + \frac{Et}{1469}} * 294.4 = \frac{0.0026041667}{.005889} * (294) = \mathbf{130\ kips}$$

$$F_{6Direct} = \frac{\left(\frac{Et}{1469}\right)}{\frac{2Et}{384} + Et/1469} * 294.4 = \frac{0.0026041667}{.005889} * (294) = \mathbf{34\ kips}$$

$$F_{7Direct} = \frac{\left(\frac{Et}{384}\right)}{\frac{2Et}{384} + \frac{Et}{1469}} * 294.4 = \frac{0.0026041667}{.005889} * (294) = \mathbf{130\ kips}$$

Preliminary Hand Calculations - Lateral Load Distribution and Design Continued:

Step Five – Determine Torsion Forces and Total Forces

Governing equations: $F_{it} = \frac{K_i d_i P_y e_x}{\sum K_i d_i^2}$ $\quad F_{itotal} = F_{i,Direct} \pm F_{i,Torsion}$

$F_{1t} = \frac{\left(\frac{Et}{3434}\right)(11.5)(294)(2.99)}{8.234Et} = \mathbf{.3575\ kips}$ $\quad F_{1,Total} = 0 + .3575 = \mathbf{.3575\ kips}$

$F_{2t} = \frac{\left(\frac{Et}{169}\right)(2.5)(294)(2.99)}{8.234Et} = \mathbf{1.579\ kips}$ $\quad F_{2,Total} = 0 + 1.579 = \mathbf{1.579\ kips}$

$F_{3t} = \frac{\left(\frac{Et}{169}\right)(4)(294)(2.99)}{8.234Et} = \mathbf{2.526\ kips}$ $\quad F_{3,Total} = 0 + 2.526 = \mathbf{2.526\ kips}$

$F_{4t} = \frac{\left(\frac{Et}{1678}\right)(11.8)(294)(2.99)}{8.234Et} = \mathbf{.75075\ kips}$ $\quad F_{4,Total} = 0 + .75075 = \mathbf{.75075\ kips}$

$F_{5t} = \frac{\left(\frac{Et}{384}\right)(38)(294)(2.99)}{8.234Et} = \mathbf{10.56\ kips}$ $\quad F_{5,Total} = 130 + 10.56 = \mathbf{140.56\ kips}$

$F_{6t} = \frac{\left(\frac{Et}{1460}\right)(26)(294)(2.99)}{8.234Et} = \mathbf{1.889\ kips}$ $\quad F_{1,Total} = 34 - 1.889 = \mathbf{32.11\ kips}$

$F_{7t} = \frac{\left(\frac{Et}{384}\right)(38)(294)(2.99)}{8.234Et} = \mathbf{10.56\ kips}$ $\quad F_{1,Total} = 130 - 10.56 = \mathbf{119.47\ kips}$

At lower levels where story shears are even higher, torsion effects become increasingly more significant. Therefore, a critical design goal is to reduce or even eliminate torsions lateral force induced torsion effects.

Based on these lateral force distribution calculations, enough eccentricity exists to generate significant torsion effects.

Solution: Currently the walls are all assumed to have equivalent thicknesses. To reduce eccentricity, calculations were performed to determine how to manipulate shear wall thicknesses to make the center of rigidity coincide with the center of mass location. First, in each direction a single wall with the fewest limitations (architectural, special, structural) on its size, thickness, and length was selected.

X Direction: W_5 Y Direction: W_1

Wall W_5 and W_1 were chosen because they are not in critical locations where changes in wall thickness would impose problems on the surrounding areas. The goal of the calculations is to determine how much these walls should be thickened to minimize eccentricity.

Preliminary Hand Calculations - Lateral Load Distribution and Design Continued:

Mathematically, this required setting the X and Y center of rigidity equations equal to the known center of mass coordinates. For the X direction, stiffness $K_5 = \frac{(Ft)E}{384}$ and for the Y direction, stiffness $K_1 = \frac{(Ft)E}{3434}$. Note the term "F" in the equation represents the factor of wall thickness t (t = wall thickness of other shear walls). By solving for F, as is shown in the following calculations, it is now known by what factor the thickness of Wall W_5 and Wall W_1 should be magnified by in order to reduce or even eliminate eccentricity.

$$X = 38 = \frac{\left[(K_1 * 0) + \left(\frac{Et}{1469} * 64\right) + \left(\frac{Et}{384} * 76\right)\right]}{K_1 + \frac{Et}{1469} + \frac{Et}{384}} = \frac{\left[\left(\frac{FEt}{384} * 0\right) + \left(\frac{Et}{1469} * 64\right) + \left(\frac{Et}{384} * 76\right)\right]}{\frac{FEt}{384} + \frac{Et}{1469} + \frac{Et}{384}} = \frac{(0 + .2414837191)}{\frac{F}{384} + .0032849019}$$

$\boldsymbol{F = 1.2}$

Conclusion: Wall W_5 should be 1.2 times thicker than the other two walls to reduce or eliminate eccentricity.

$$Y = 26 = \frac{\left[\left(\frac{FEt}{3434} * 14.5\right) + \left(\frac{Et}{169} * 23.25\right) + \left(\frac{Et}{169} * 30\right) + \left(\frac{Et}{1678} 37.8\right)\right]}{\frac{FEt}{3434} + \frac{2Et}{169} + \frac{Et}{1678}} = \frac{(.00422F + .1376 + .1775 + .0225268)}{.0002912F + (.005917 * 2) + .0005959}$$

F_1 = 4.315

Conclusion: As one can see, the factor F for wall W_1 is 4.315. This is not a practical solution. Designing a shear wall at 4 times the thickness of the other shear walls will yield an overdesigned shear wall. Since wall W_1 is so short in length, it has negligible influence on the location of the center of rigidity, which is why the thickness had to be altered to such a significant degree to reduce eccentricity. As a result, the same calculation process was performed for wall W_2.

$$Y = 26 = \frac{\left[\left(\frac{Et}{3434} * 14.5\right) + \left(\frac{FEt}{169} * 23.25\right) + \left(\frac{Et}{169} * 30\right) + \left(\frac{Et}{1678} 37.8\right)\right]}{\frac{Et}{3434} + \frac{FEt}{169} + \frac{Et}{1678} + \frac{Et}{169}} = \frac{(.13757F + .204264)}{.0068043129 + .005917F}$$

F_2 = 1.78

Conclusion: Wall W_2 must be 1.79*t thick to reduce or eliminate eccentricity.

Lateral System Design Explanation:

With the seismic and wind story forces calculated, the next phase of the lateral system design involved creating a 3-D model of the proposed lateral system in ETABS. Figures F-120 through F-122 provide views of the initial ETABS model. As shown in figure F-121, four shear walls run in the X direction, and three shear walls are oriented in the Y direction. Restricted by the architectural design, very few options were available for the positioning of the shear walls. All interior shear walls are aligned along corridors or around vertical circulation nodes such as the elevator shaft and stairwell. Because the two facades normal to the y direction have setbacks, multi-story shear walls could only be located along the two facades normal to the x direction.

To model the shear walls, a 12" membrane thickness was used. Also, for increased accuracy, the wall meshing was assigned as "subdivide objects with maximum size of 24". The material properties and modifiers were properly defined. The shear walls were modeled with f'c = and N.W.C. To ensure distribution of lateral forces was governed by relative stiffness as opposed to tributary areas, rigid diaphragm modeling was employed. Mass and Uniform loads were assigned to each diaphragm as well. For more detailed information regarding the diaphragm inputs, please see figure F- 123. It was also verified that the base joints were restrained in all 6 degrees of freedom.

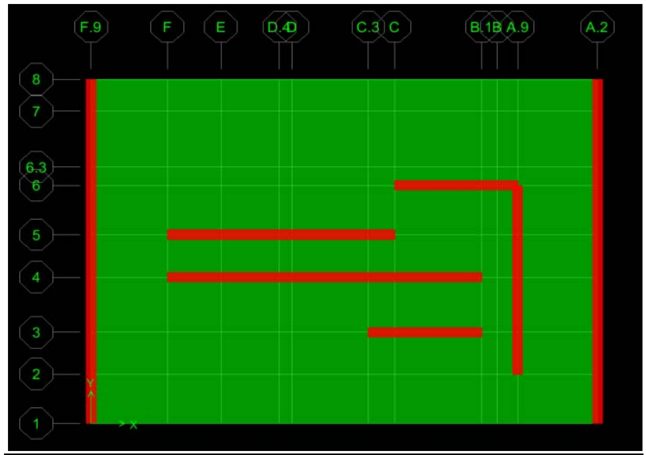
Next, 38 load combinations comprised of the 4 wind cases and the 7 ASCE7-05 load combinations were established. Using the wind and seismic story forces calculated on the previous pages, the 38 load combinations were input into the model. 38 load combinations are required to account for the various combinations of eccentricity and torsion scenarios.

Once the loads were assigned to the model, the analysis was run. As is common in most design procedures, the lateral system design process was iterative. In fact, upon analysis of the 3D output data, it became apparent that the preliminary shear wall lengths were huge over designs. As a result, the shear walls were remodeled, with the final shear wall layout arranged as shown in figure F-124 on page 114.

After remodeling the shear walls and running the model analysis again, each shear wall was examined under all 38 load combinations to determine the controlling loading condition. Figures F-128 through F-134 display the controlling loading case for each of the seven shear walls. As one can see, the design story shears range from 72.14 kips to 172 kips. Since the shear walls are 13 stories tall, the story forces have large moment arms generating design moments as large as 17,277 ft-kips. Seismic Load combinations in a single direction controlled for all seven shear walls.

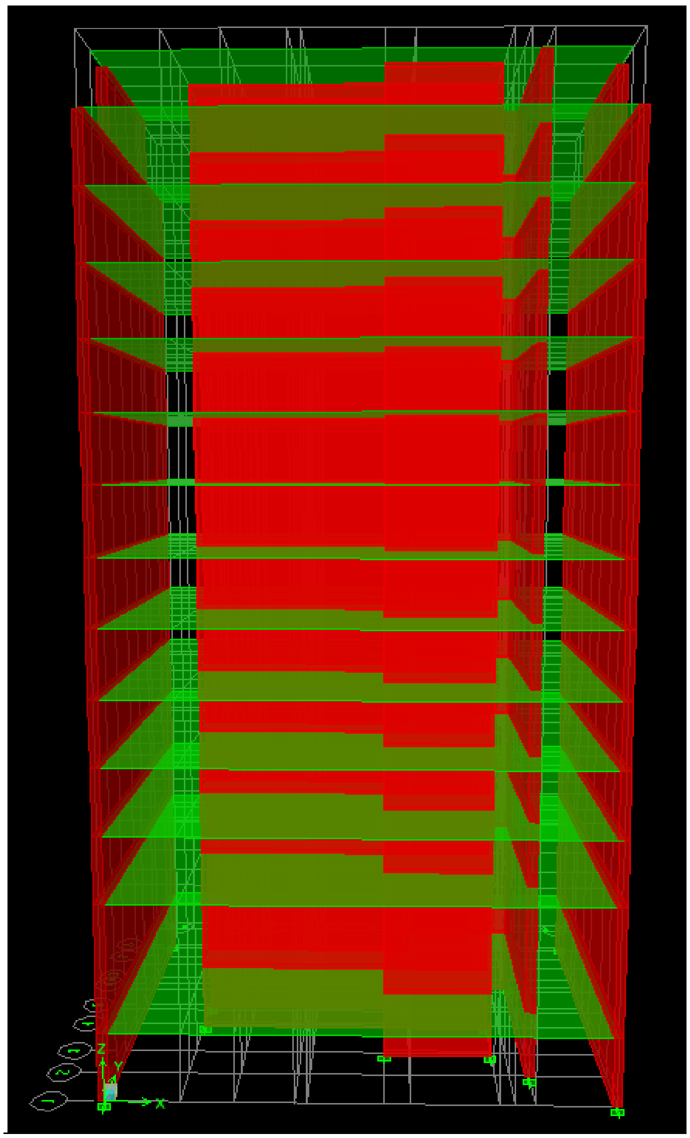
ETABS – 3D LATERAL SYSTEM MODEL

A 3D ETABS model of the shear wall lateral system was constructed. To help explain how the lateral system was modeled, a plan view, isometric view, and elevation view are provided. (Note this is a preliminary Layout)



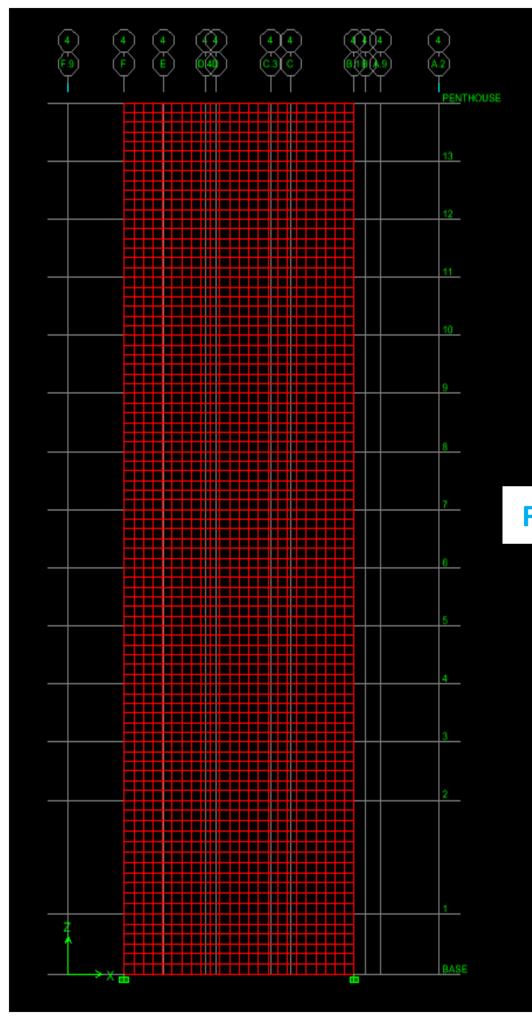
Plan View of Initial Shear Wall Layout

F-121



3D View Of ETABS Model: Diaphragms Are Green, Shear Walls Are Red.

F-120



F-122

Elevation of A Shear Wall – Modeled As 10" Membrane. Meshed as subdivided objects with maximum Size of 24"

3D ETABS MODEL – INPUT DATA

During the modeling process, careful attention was given to the diaphragm modeling. Uniform dead loads due to the weight of the slab, building envelope, and superimposed loads were applied to each diaphragm. Uniform Roof Live and Live Loads were also applied to diaphragms where necessary. In doing so, overturning moment and column strength checks could be easily completed with the aid of the 3D model.

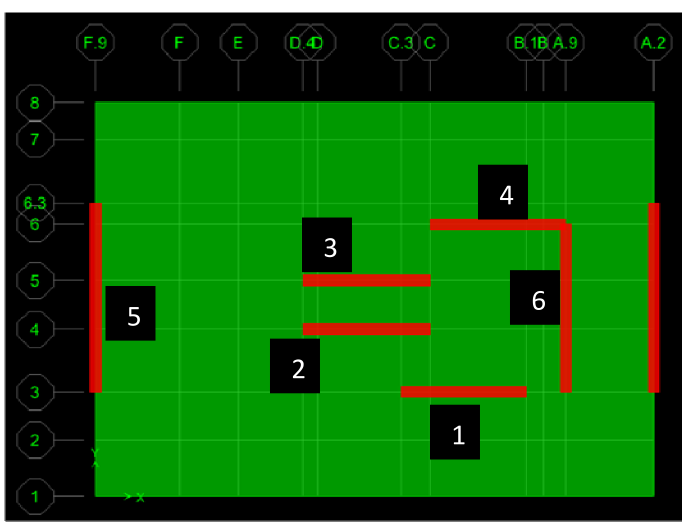
A mass was also assigned to each diaphragm. The unit conversion required mass = PSF / (12^3 * 32.2* 1000).

Calculation of Mass Definitions - Diaphgram Assignments in ETABS F-123

Floor Level	Slab / Superimposed Load (kips)	Envelope Dead Load (kips)	Floor Area (SF)	KSF	Unit Conversion: KSF/32.2/12^3
1	338.7	86.31	3952	0.107543016	1.93278E-06
2	255.74	70	3952	0.082424089	1.48134E-06
3	205.67	56.7	3952	0.06638917	1.19316E-06
4	205.67	56.7	3952	0.06638917	1.19316E-06
5	205.67	56.7	3952	0.06638917	1.19316E-06
6	205.67	56.7	3952	0.06638917	1.19316E-06
7	205.67	56.7	3952	0.06638917	1.19316E-06
8	205.67	56.7	3952	0.06638917	1.19316E-06
9	205.67	56.7	3952	0.06638917	1.19316E-06
10	255.42	54.6	3952	0.078446356	1.40985E-06
11	255.42	54.6	3952	0.078446356	1.40985E-06
12	255.42	54.6	3952	0.078446356	1.40985E-06
13	255.42	54.6	3952	0.078446356	1.40985E-06
Penthouse	100	52.9	3952	0.038689271	6.9533E-07
Roof	50	16.4	2100	0.031619048	5.68263E-07

Note, the shear wall layout shown in figure F-121 was a preliminary design. The final shear wall layout modeled in ETABS is shown here. The original shear wall layout shown on the previous page was a huge overdesign.

Note, shear walls are referred to as Walls 1 through 7. This Figure shows the shear wall labels, which correspond to the final details 1 through 7 on pages 128 and 129.



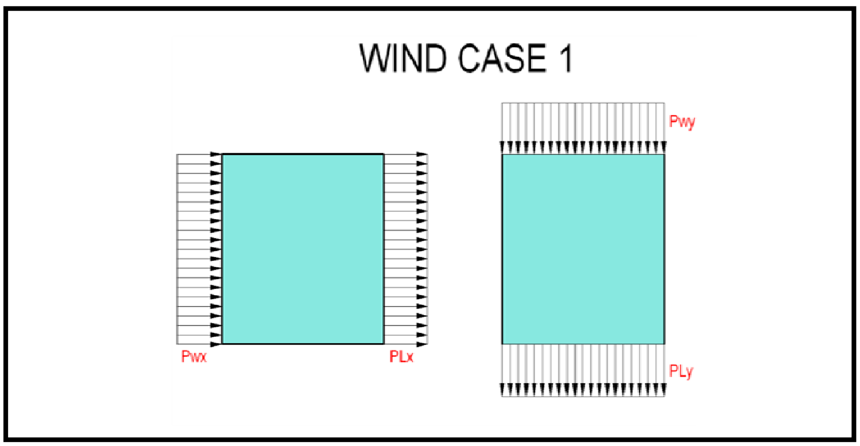
F-124

LOAD CASES AND COMBINATIONS

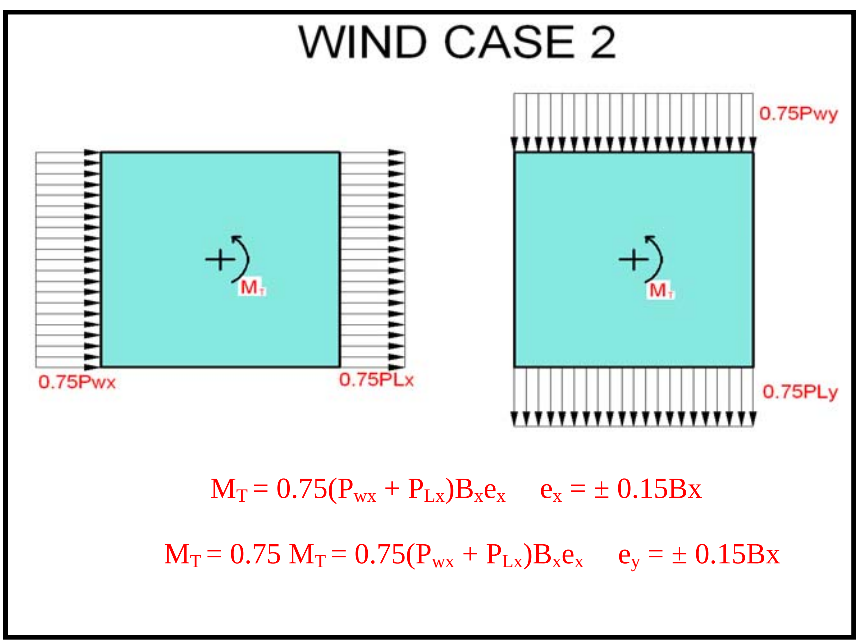
In this report, strength, drift, story drift, and overturning moment checks were performed by analyzing the structure under the following ASCE7-05 (section 2.3.2) load combinations:

1. 1.4 (D + F)
2. 1.2 (D + F + T) + 1.6(L+H) + 0.5(L_r or S or R)
3. 1.2D + 1.6(L_r or S or R) + (L or .8W)
4. 1.2D + 1.6W + L + 0.5(L_r or S or R)
5. 1.2D 1.0E + L + 0.2S
6. .9D + 1.6W + 1.6H
7. .9D + 1.0E + 1.6H

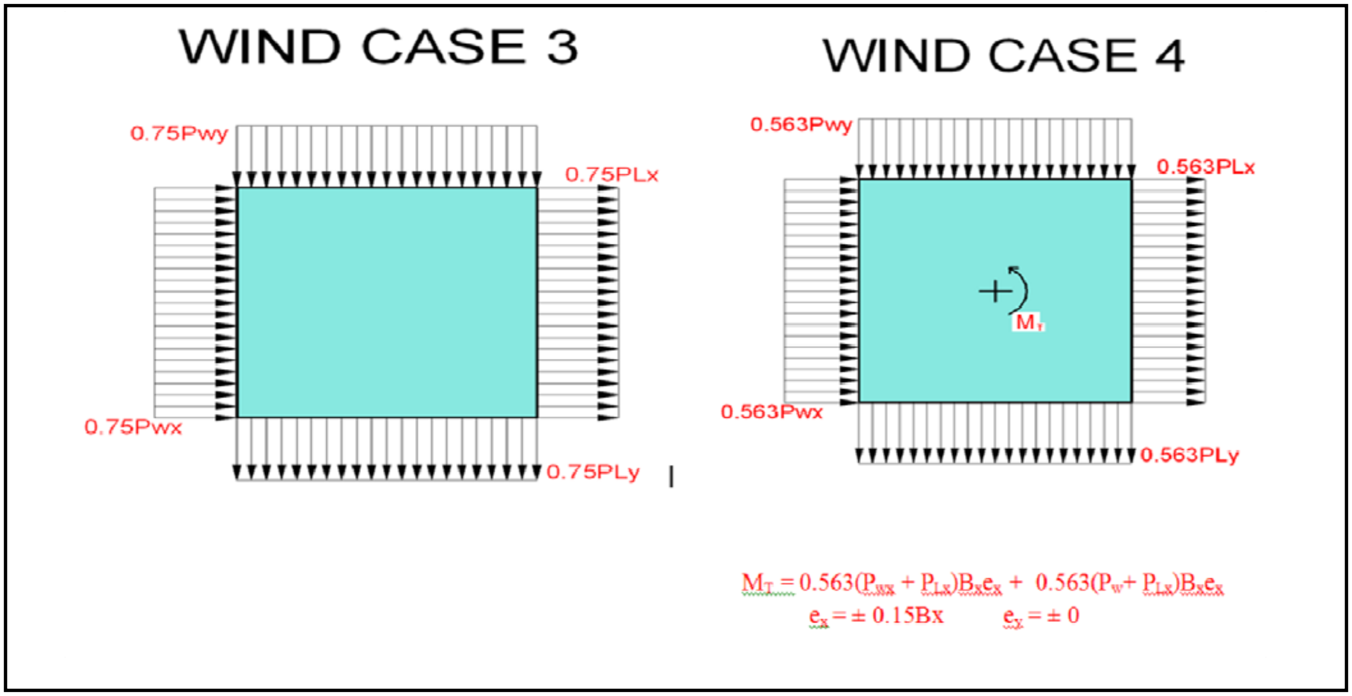
Design wind pressures were applied according to the following 4 ASCE7-05 (figure 6.9) wind loading cases:



F-125



F-126



F-127

Considering the 7 ASCE7-05 load combinations and 4 wind cases, the following 38 loading conditions were considered in the lateral design:

1-3:	Load Combination 3,4,6: Wind Case 1, X Direction
4-6:	Load Combination 3,4,6: Wind Case 1, Y Direction
7-9:	Load Combination 3,4,6: Wind Case 2, X Direction, $+e_y$
10-12:	Load Combination 3,4,6: Wind Case 2, X Direction, $-e_y$
13-15:	Load Combination 3,4,6: Wind Case 2, Y Direction $+ e_y$
16-18:	Load Combination 3,4,6: Wind Case 2, Y Direction $– e_y$
19-21:	Load combination 3,4,6: Wind Case 3
22-24:	Load Combination 3,4,6: Wind Case 4, $+e_x$ and $+e_y$
25-27:	Load Combination 3,4,6: Wind Case 4, $+e_x$ and $–e_y$
29-31:	Load Combination 3,4,6: Wind Case 4, $-e_x$ and $–e_y$
32-34:	Load Combination 3,4,6: Wind Case 4, $-e_x$ and $+ e_y$
35:	Load Combination 5
36:	Load Combination 7

3D ETAB MODEL - OUTPUT

These tables display the controlling load combination for each shear wall. Each table shows the max story shear and max bending moment for the corresponding shear wall. To determine these controlling load cases, exhaustive analysis of 3D ETABS output was done. Every shear and flexure value for all seven shear walls was noted to determine max values for each of the load combinations.

F-128

Wall 1				
Story	Story Shear (kips)	Story Force (Kips)	Story Height From Base (ft)	Moment (ft-kips)
BASE	**72.14**			
2	65.43	6.71	21	140.91
3	62.8	2.63	31	81.53
4	61.2	1.6	41	65.6
5	58.4	2.8	51	142.8
6	56.19	2.21	61	134.81
7	54.12	2.07	71	146.97
8	49.89	4.23	81	342.63
9	45	4.89	91	444.99
10	37.4	7.6	101	767.6
11	28.8	8.6	111	954.6
12	17.9	10.9	121	1318.9
13	6.64	11.26	131	1475.06
PENTHOUSE		6.64	141	936.24
			TOTAL MOMENT	**6952.64**

F-129

Wall 2				
Story	Story Shear (kips)	Story Force (Kips)	Story Height From Base (ft)	Moment (ft-kips)
BASE	**73.69**			
2	67.08	6.61	21	138.81
3	64.2	2.88	31	89.28
4	62.4	1.8	41	73.8
5	60.6	1.8	51	91.8
6	58.3	2.3	61	140.3
7	55.1	3.2	71	227.2
8	51	4.1	81	332.1
9	45.8	5.2	91	473.2
10	38.4	7.4	101	747.4
11	29.6	8.8	111	976.8
12	18.09	11.51	121	1392.71
13	6.32	11.77	131	1541.87
PENTHOUSE		6.32	141	891.12
			TOTAL MOMENT	**7116.39**

Shear Wall Design Loads

F-130

Wall 3				
Story	Story Shear (kips)	Story Force (Kips)	Story Height From Base (ft)	Moment (ft-kips)
BASE	73.187			
2	66.1	7.087	21	148.827
3	63.06	3.04	31	94.24
4	61.25	1.81	41	74.21
5	59.44	1.81	51	92.31
6	57.11	2.33	61	142.13
7	54.04	3.07	71	217.97
8	50.03	4.01	81	324.81
9	44.94	5.09	91	463.19
10	37.68	7.26	101	733.26
11	28.64	9.04	111	1003.44
12	17.55	11.09	121	1341.89
13	5.81	11.74	131	1537.94
PENTHOUSE		5.81	141	819.21
			TOTAL MOMENT	6993.427

F-131

Wall 4				
Story	Story Shear (kips)	Story Force (Kips)	Story Height From Base (ft)	Moment (ft-kips)
BASE	172			
2	154.6	17.4	21	365.4
3	150.5	4.1	31	127.1
4	149.6	0.9	41	36.9
5	144.67	4.93	51	251.43
6	142.87	1.8	61	109.8
7	136.54	6.33	71	449.43
8	125.95	10.59	81	857.79
9	112.67	13.28	91	1208.48
10	95.74	16.93	101	1709.93
11	76.05	19.69	111	2185.59
12	53.34	22.71	121	2747.91
13	24.03	29.31	131	3839.61
PENTHOUSE		24.03	141	3388.23
			TOTAL MOMENT =	17277.6

Shear Wall Design Loads

Wall 5				
Story	Story Shear (kips)	Story Force (Kips)	Story Height From Base (ft)	Moment (ft-kips)
BASE	**153.8**			
2	142.6	11.2	21	235.2
3	141.65	0.95	31	29.45
4	139.2	2.45	41	100.45
5	135.4	3.8	51	193.8
6	129.8	5.6	61	341.6
7	122.9	6.9	71	489.9
8	113.2	9.7	81	785.7
9	101.2	12	91	1092
10	85.9	15.3	101	1545.3
11	66.7	19.2	111	2131.2
12	43.2	23.5	121	2843.5
13	17.7	25.5	131	3340.5
PENTHOUSE		17.7	141	2495.7
			TOTAL MOMENT	**15624.3**

F-132

Wall 6				
Story	Story Shear (kips)	Story Force (Kips)	Story Height From Base (ft)	Moment (ft-kips)
BASE	**116**			
2	113.07	2.93	21	61.53
3	110.4	2.67	31	82.77
4	104.56	5.84	41	239.44
5	103.57	0.99	51	50.49
6	102.8	0.77	61	46.97
7	100.88	1.92	71	136.32
8	95.6	5.28	81	427.68
9	85.77	9.83	91	894.53
10	72.83	12.94	101	1306.94
11	57.51	15.32	111	1700.52
12	39.19	18.32	121	2216.72
13	14.9	24.29	131	3181.99
PENTHOUSE		14.9	141	2100.9
			TOTAL MOMENT	**12446.8**

F-133

Wall 7				
Story	Story Shear (kips)	Story Force (Kips)	Story Height From Base (ft)	Moment (ft-kips)
BASE	**101.3**			
2	94.09	7.21	21	151.41
3	90.14	3.95	31	122.45
4	83.74	6.4	41	262.4
5	80.88	2.86	51	145.86
6	77.5	3.38	61	206.18
7	73.25	4.25	71	301.75
8	66.7	6.55	81	530.55
9	59.8	6.9	91	627.9
10	50.7	9.1	101	919.1
11	32.1	18.6	111	2064.6
12	22.4	9.7	121	1173.7
13	8.14	14.26	131	1868.06
PENTHOUSE		8.14	141	1147.74
			TOTAL MOMENT	**9521.7**

F-134

Shear Wall Design Explanation:

Using the max story forces and shears retrieved from the 3D ETABS model output, hand calculations were performed to design each shear wall. The entire set of calculations can be found in Appendix I. However the following pages provide a concise summary of the shear wall design calculations. With the length of the shear walls primarily controlled by architectural constraints, there was more "design flexibility" available for the wall thickness dimension. As a result, the modeled shear wall length was maintained, however trial and error was used to determine the most efficient wall thickness. As shown on Figure F-122, all the shear walls were designed with a 10" thickness. As was evident by the max shear and flexure values in figures F-28 through F-34, flexure controlled the shear wall design. The strength behavior of the shear walls was also predicted using the basic ratio $\frac{h_w}{l_w}$. In all cases, the shear walls were flexure controlled and categorized as slender elements where the ratio $\frac{h_w}{l_w} < 3$.

Next, each shear wall was modeled including the flexure, shear, and axial loads. In phase one of the calculations, the flexural reinforcement was designed for each shear wall. In step one, jd is approximated as $jd = 0.9(d) = 0.9 * (0.8l_w)$. Next, the equation $M_U = \emptyset M_N = \emptyset A_S f_Y jd$ is used to determine an approximate A_s value. Next, the corresponding "a" value was determined via the equation: $0.85f'_c ab = A_S f_Y$. Next, "jd" was recalculated according to the expression: $jd = d - \frac{a}{2}$. With the new "jd" value, the required A_s value was recalculated using the equation: $M_U = \emptyset M_N = \emptyset A_S f_Y jd$. As shown in appendix I, flexural reinforcement was then selected to satisfy the required area of steel. The flexural reinforcement is arranged in two curtains on both ends of the shear wall. The minimum spacing and cover requirements were satisfied as well. Finally, the d_t value was determined and tension control checks were performed to ensure $\varepsilon_t = \varepsilon_{CU}\left(\frac{d_t - C}{C}\right) > .005$.

Next, the shear walls were designed for the shear forces. To begin, the shear dimensions had to be checked to ensure the applied load $V_U \leq \emptyset V_{N,max} = \emptyset 10\sqrt{f'_c}hd$. In all cases, the shear wall dimensions were sufficient and did not require modification. Next, the critical shear locations were determined according to the expression:

$a \leq minimum\left\{\frac{l_W}{2} \text{ and } \frac{h_W}{2}\right.$.

Using the conservative and simplified equation $V_c = 2\sqrt{f'_c}hd$, the shear strength contribution of the concrete was determined. The concrete capacity was then compared to V_u to determine shear reinforcement requirements. In all cases, $\emptyset V_C \geq V_U$. This was expected, since flexure controls in slender walls. As a result, no calculation of V_s is necessary to determine shear reinforcement. Since the equation $V_U > \frac{1}{2}\emptyset V_C$ was satisfied in all cases, chapter 11 ACI code provisions are applicable for determination of shear reinforcement.

First, the horizontal (transverse) shear reinforcement was determined according to the minimum transverse shear reinforcement ratio: $\rho_t = \frac{A_V}{SH} > .0025$. In all cases, (2) # 4 bars spaced at 16" satisfied the reinforcement ratio.

Shear Wall Design Explanation:

However, the spacing limitations had to be checked as well. For all seven shear walls, the spacing was specified as 16" which is less than the maximum allowable spacing: $\leq \left\{ \frac{l_W}{5} \; or \; 3h \; or \; 18" \right.$. Finally, the vertical shear reinforcement was determined for each shear wall. The governing equation for vertical shear reinforcement:

$\rho_l = \frac{A_v}{SH} \geq .0025 + \; +.5\left(2.5 - \frac{h_W}{l_W}\right)(\rho_t - .0025)$. However, since $h_w/l_w \geq 2.5$, the ratio $\rho_l = \frac{A_V}{SH} > .0025$. As it turns out, all seven shear walls require (2) # 4 bars (verts.) spaced at 16".

See Figures F-135 through F-140 for the final shear wall design details.

M/AE Application: In order to properly model, design, and analyze the shear wall lateral system, a significant portion of the process required application of knowledge and techniques acquired from MAE coursework. When modeling the lateral system, rigid diaphragm modeling was applied. As instructed in AE 597 A, basic concepts such as shear deformation and property modifiers were used. To adjust the shear walls for cracked moment of inertia, modifiers were used in ETABS to reduce the gross section moment of inertia to 75%. Additionally, shell elements and meshing was integrated into the model. Specifically, 10" membrane thickness and 10" bending thicknesses were used. As warned in AE 597 A, fine meshing requires significantly more time to run analysis; however, the fine meshing produces much more accurate drift results. This was especially important to integrate into the model since the shear walls are 141' tall. Without properly meshing the shear walls (subdivided with 24" maximum), they would have behaved in an unrealistic and more rigid manner. By meshing the shear walls, each subdivision can translate relative to the adjacent subdivision. Additionally, all base nodes were restrained in all 6 degrees of freedom. Several diaphragm techniques taught in AE 597 A were also used to properly model the diaphragms. Some of these techniques include assignment of mass definitions (required proper unit conversions) and uniform loads.

In addition to AE 597 A coursework, several concepts pertaining to AE 592: Building Enclosures was integrated in to the thesis report. Specifically, knowledge regarding window details and building facades was applied to the acoustic study. In order to properly design the exterior wall detail in figure F-171, knowledge regarding placement of wall components from AE 597 A was applied. Although the focus of the study was sound isolation, other concepts had to be considered to create a properly functioning wall system. For instance, placement of air and moisture barrier and its effects on the dew point had to be accounted for in the study. Understanding of window details was vital as well. A large section of AE 597 is dedicated just to glazing systems. In AE 597, it was emphasized that laminated glass units perform best for sound isolation. The mechanics behind laminated glass performance was applied as well. For instance, as taught in class, using two different thicknesses of glass expands the range of frequencies it can resist and eliminates the chance of resonance in the entire glazing unit since the thicknesses differ. Finally, the building enclosure class stresses the importance of being able to identify the various components of a window detail. Often times, failure of building facades has to do with failed seals or thermal bridges in the window system. These similar concepts can be applied to the acoustic study. For example, it was suggested that soft neoprene gaskets should be used as well as sound-absorbing material lining in the head, sill, and window jambs. Also, for sound isolation, sealed windows were recommended. Knowing the advantages and disadvantages of sealed windows versus operable windows enabled production of a well thought out design solution. For example, a major consequence of sealed or fixed windows is the need for alternate forms of ventilation including PTAC units or central air conditioning. As one can see, a multitude of concepts and skills pertaining to MAE coursework was required to properly complete the thesis project.

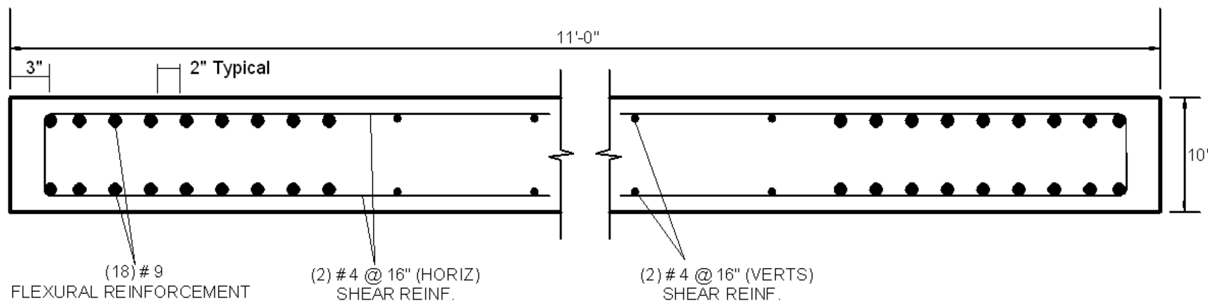
Final Shear Wall Design Details

Using the max story forces and shears retrieved from the 3D ETABS model output, hand calculations were performed to design each shear wall according to ACI chapter 11 Provisions. The entire set of calculations can be found in Appendix I. The previous pages provide a concise summary of the shear wall design calculations.

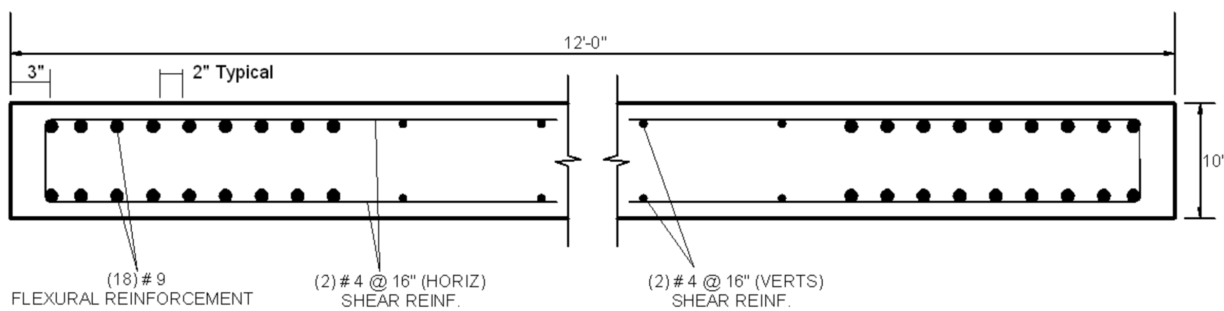
Final Shear Wall Designs:

F-135 : F-137

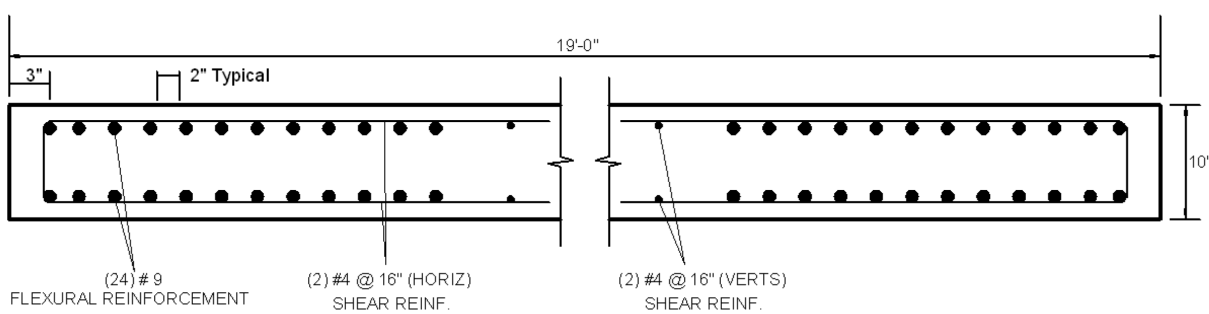
SHEAR WALL 1 DETAIL



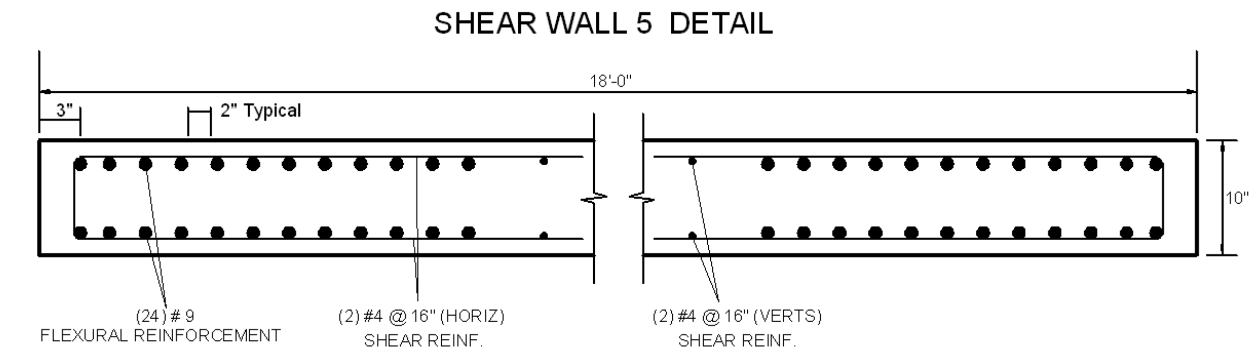
SHEAR WALL 2 AND 3 DETAIL

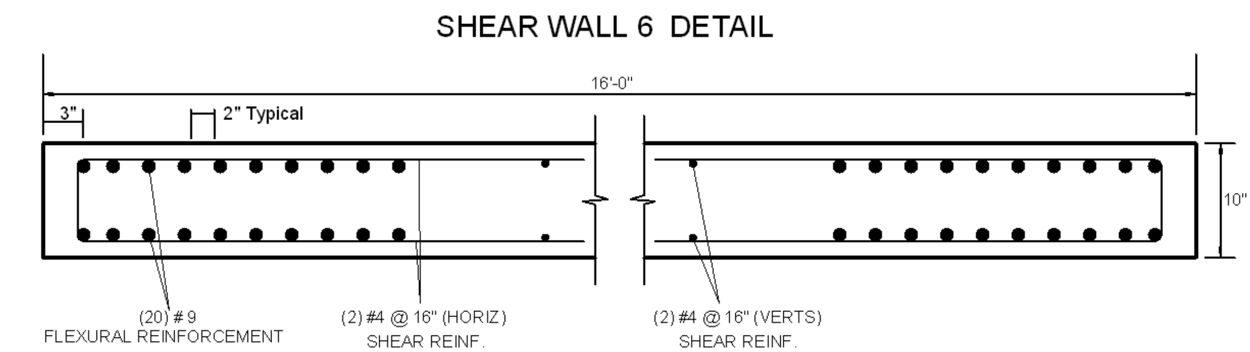


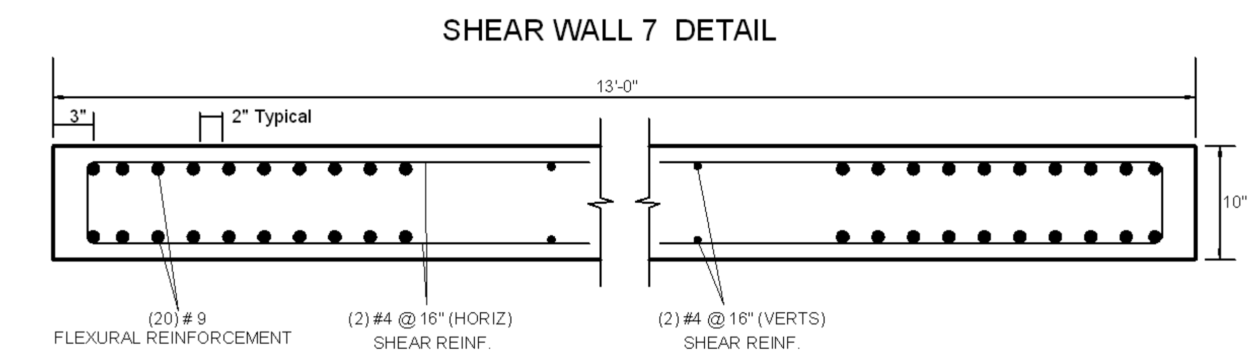
SHEAR WALL 4 DETAIL



Final Shear Wall Designs Details







F-138 : F-140

Wind Drifts - Serviceability Check:

Confirmation of the lateral system requires serviceability checks. Using unfactored wind loads,, story drifts and total building drifts were recorded and examined for each of the four ASCE7-05 wind cases. Wind case 1 controlled in both the X and Y directions. Tables F-141 and F-142, show controlling story drift and total drift values which were compared to the allowable drift: $\Delta_{WIND} = H / 400$.

Actual Wind Drift (Unfactored Loads): N-S Direction							
CONTROLLING WIND CASE: ASCE7-05 Wind Case 1 - X Direction							
Story	Story Height (in.)	Actual Story Drift (in.)	Allowable Story Drift Δwind = H / 400 (in.)	Serviceability Check Actual < Allowable	Actual Total Drift (in.)	Allowable Total Drift Δwind = H/400 (in.)	Serviceability Check Actual < Allowable
Penthouse	1692	0.00598	0.3	OK	0.047116	4.23	OK
13	1572	0.00591	0.3	OK	0.047116	3.93	OK
12	1452	0.0058	0.3	OK	0.041136	3.63	OK
11	1332	0.005659	0.3	OK	0.029426	3.33	OK
10	1212	0.004923	0.3	OK	0.029426	3.03	OK
9	1092	0.00478	0.3	OK	0.023767	2.73	OK
8	972	0.003745	0.3	OK	0.018844	2.43	OK
7	852	0.00314	0.3	OK	0.014064	2.13	OK
6	732	0.002489	0.3	OK	0.010319	1.83	OK
5	612	0.001949	0.3	OK	0.007179	1.53	OK
4	492	0.001541	0.3	OK	0.00469	1.23	OK
3	372	0.0012	0.3	OK	0.002741	0.93	OK
2	252	0.0008	0.63	OK	0.002	0.63	OK

F-141

Actual Wind Drift (Unfactored Loads): E-W Direction							
CONROLLING WIND CASE: ASCE7-05 Wind Case 4, and eccentricity combination: + ex and - ey							
Story	Story Height (in.)	Actual Story Drift (in.)	Allowable Story Drift Δwind = H / 400 (in.)	Serviceability Check Actual < Allowable	Actual Total Drift (in.)	Allowable Total Drift Δwind = H/400 (in.)	Serviceability Check Actual < Allowable
Penthouse	1692	0.1109	0.3	OK	0.1723359	4.23	OK
13	1572	0.0098819	0.3	OK	0.1723359	3.93	OK
12	1452	0.00967	0.3	OK	0.0614359	3.63	OK
11	1332	0.008553	0.3	OK	0.051554	3.33	OK
10	1212	0.007585	0.3	OK	0.041884	3.03	OK
9	1092	0.006489	0.3	OK	0.033331	2.73	OK
8	972	0.005198	0.3	OK	0.025746	2.43	OK
7	852	0.00422	0.3	OK	0.019257	2.13	OK
6	732	0.00318	0.3	OK	0.014059	1.83	OK
5	612	0.00276	0.3	OK	0.009839	1.53	OK
4	492	0.002019	0.3	OK	0.006659	1.23	OK
3	372	0.00188	0.3	OK	0.93	0.93	OK
2	252	0.0016	0.63	OK	0.63	0.63	OK

F-142

Seismic Story Drift- Stability Check:

3D output data for ASCE 7-05 Load Combinations 6 (1.2D + 1.0E + 1.0L) and 7 (.9D + 1.0E) were examined to verify that seismic induced story drifts do not exceed the allowable .02hsx. Table F-144, displays the actual and allowable drift values proving the lateral system is properly designed. Therefore, it is fair to assume the 40 Gold Street structure will not sustain any permanent damage due to small or moderate seismic activity. More importantly, in the event of severe seismic activity, structural failure will be avoided; however, the seismic induced stresses will exceed the yield strength of various structural members resulting in inelastic deformation (permanent damage). Since the building has a low overall building weight and is located in New York City, an area of little seismic activity, seismic story drifts did not come close to exceeding the allowable drift.

Governing Equation: $\Delta_{SEISMIC} = .020h_{SX}$

F-143

TABLE 12.12-1 ALLOWABLE STORY DRIFT, Δ_a[a,b]

Structure	Occupancy Category		
	I or II	III	IV
Structures, other than masonry shear wall structures, 4 stories or less with interior walls, partitions, ceilings and exterior wall systems that have been designed to accommodate the story drifts.	$0.025h_{sx}$[c]	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures [d]	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
All other structures	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$

Actual and Allowable Seismic Story Drift:

Seismic Story Drift							
Story	Story Height (in)	X Direction			Y Direction		
		Actual Drift	Allowable = .01Hsx	Check Actual < Allowable	Actual Drift	Allowable = .01Hsx	Check Actual < Allowable
Penthouse	1692	0.00921	1.2	OK	0.183	1.2	OK
13	1572	0.00912	1.2	OK	0.1827	1.2	OK
12	1452	0.008876	1.2	OK	0.1639	1.2	OK
11	1332	0.00834	1.2	OK	0.147	1.2	OK
10	1212	0.007189	1.2	OK	0.13889	1.2	OK
9	1092	0.0068	1.2	OK	0.1278	1.2	OK
8	972	0.0063	1.2	OK	0.122	1.2	OK
7	852	0.006289	1.2	OK	0.00888	1.2	OK
6	732	0.00587	1.2	OK	0.007701	1.2	OK
5	612	0.005489	1.2	OK	0.00703	1.2	OK
4	492	0.005289	1.2	OK	0.005498	1.2	OK
3	372	0.004689	1.2	OK	0.00523	1.2	OK
2	252	0.0012	2.52	OK	0.00287	2.52	OK

F-144

Construction Management Breadth:

Introduction: With the redesign of the steel frame structure being a concrete structural system, the two construction processes for each of the designs differ significantly. Everything from equipment, labor, material, construction duration, and cost differ. The goal of the construction management breadth is to employ a detailed cost and schedule analysis to determine which structural design includes the largest duration and cost. The scope of the cost and schedule analysis is limited only to the primary structural components, but is still very effective in providing important cost and duration values for comparison purposes. The construction management breadth was subdivided into 4 sections: material takeoffs, cost analysis, durations, and construction schedules. The following pages summarize the process and explain any related assumptions or calculations required to accomplish the construction management breadth.

The Process:

Takeoffs: First, drawings and details were closely examined to determine the quantity of each major structural material that is incorporated into the building. To begin, the **new concrete structural design** was considered. Takeoff values were determined for concrete, formwork, and rebar. In order to apply takeoff values to a cost and schedule analysis, the units must be compatible with RS Means. As a result, quantities of concrete were recorded in cubic yards (CY), formwork is in square footage of contact area (SFCA), and steel reinforcement in (Tons). As displayed in figure F-145, quantities of concrete were subdivided according to type of application (column, slab, wall) and according to the compressive strengths. Later in the analysis, when determining duration and output values, knowing the type and application of concrete is essential for producing accurate results. As expected, the elevated slabs account for nearly 64% of the structures concrete. It is important to note, the micro pile foundation system was not included in the analysis since it was not redesigned. As displayed in figure F-145, the elevated slabs require 938 CY of L.W.C, the columns require 85 CY of N.W.C., and the walls require 439 CY of N.W.C.

Next, as displayed in table F-146 and F-147, the total amount of formwork in square footage of contact area was determined for elevated slabs, columns, and shear walls. This required exhaustive analysis of drawings and details. Since the elevated slabs vary from floor to floor, it was important to determine form work takeoffs per floor. In a high rise structure such as 40 Gold Street, the duration of the concrete construction is greatly influenced by the inability to overlap the construction of floors. Even with shoring, the curing process still imposes a limitation on the overlapping of concrete floor construction. Although premade domes are used to form the voids in waffle slabs, SFCA of form work was instead calculated for each void. This resulted in large SFCA values and introduces possible error to the construction analysis; however, it enabled compatibility between takeoff values and RS Means. As one can see, the elevated slabs, columns, and shear walls required 82,461 SFCA, 11,881 SFCA, and 25,920 SFCA of job built plywood respectively.

Next, as shown in tables F-148 and F-150, the total tonnage of steel reinforcement was determined according to bar size, application, and floor level. With reinforcement present in columns, slabs, and walls, and sizes ranging between #3 to #9,

Construction Management Breadth Continued:

the corresponding duration and cost values encompassed a large range of values. In order to determine the tonnage of steel reinforcement, SP-Slab results tables in the slab design section of the report were analyzed to determine takeoffs values in linear feet. For instance, floor ten columns require 1155 linear feet of # 3 vertical rebar and 1125 feet of #3 hoop ties (see figure F- 148). Next, the lb/ft property for each bar type was used to first convert to lbs, and then ultimately tons. The final quantities of steel reinforcement for the columns, slabs, and walls were 9.68, 57.9, and 78.55 tons respectively. Finally, total square footage of floor area per level was determined. This was required in order to determine cost and duration of concrete floor finishing.

After acquiring all necessary takeoff values for the new concrete design, the **original structure** was then considered. For the original steel frame structure, material takeoff quantities were determined for structural steel beams, structural steel columns, welded wire fabric, steel reinforcement, structural concrete, and metal floor decking. First, as built drawings provided by Severud Associates were analyzed to determine the total linear feet (by floor) for each structural steel column size present in the structure. Figures F-151 and F-152 display total overall linear feet values for each column size. For example, some of the most common column sizes in the structure are W10x33, W10x39, and W12x120 whose corresponding takeoffs quantities are 627, 652, and 371 linear feet respectively.

Another major structural component is the 2" 18 gauge composite metal floor decking. For each floor, the square footage of decking was determined and recorded in table F-153. As shown, 54,375 SF of metal decking is required for the 13 elevated slabs. In addition to metal decking, the welded wire fabric takeoffs had to be considered. The welded wire fabric is 6x6 W2.9xW2.9. The table F-154 displays the total amount of welded wire fabric per floor, with a total quantity of 543.75 CSF (100's SF).

Although the existing structure contains much less steel reinforcement than the new structural design, the reinforcement was still considered. The slab is reinforced with #4 bars @ 12" spacing. In addition, (2) # 6 bars are present on either side of partitions that bear on the slab at mid-span. As shown in table ##, the total amount of reinforcement required is just 20.6 tons, which is only 14 % of the 146 tons required for the new structural design.

Page 138 shows the typical floor assembly in the existing building. As one can see, the design includes a 2 ½" concrete topping. Overall, the existing structure requires 402 CY of light weight concrete (f'_c = 4000 psi).

Finally, takeoff quantities were determined for the structural steel beams. Similar to the columns, the total linear feet of each beam size was determined at each floor level. For the most part, the beams are W10's or W12's, with the largest beam size being a W24x306. Large beams are present in the structure, but are limited in number. Some of the more common beam sizes are W8x10, W 12x22, and W12x30, which have takeoff values of 1222, 2422, and 1704 linear feet respectively.

Cost Analysis: RS Means Online Cost Works was utilized to determine the corresponding construction costs for each of the aforementioned takeoff values. On page 149 and 150, tables F-159 and F-160 provide a summary of the cost calculations and results for the new and original structural design.

Construction Management Breadth Continued:

First, preliminary input information specific to the project had to be provided to facilitate proper unit cost estimates. The estimate type was categorized as unit cost, the building was listed as commercial new structure, a standard union was assumed, and localization was assigned to New York, New York. Finally, general contractor's markup on subs, the general conditions, and general contractor's overhead and profit were all assumed to be 10%.

A cost estimate was first completed for the new structural design. As shown in table F-159, cost values were determined for both structural concrete and placing concrete. The structural concrete costs simply represent the purchasing of the concrete. The placing of concrete includes both the cost of labor and equipment required to pump the concrete in place. Using the cubic yard quantities and unit cost values, the total cost for the elevated slab L.W.C , the N.W.C for the columns, and the N.W.C., for the walls were calculated as $176,550, $10,728, and $55,532 respectively. The concrete was assumed to be ready mix concrete. The price for placing concrete varied depending on application. Assuming the concrete placement method is pumping, the columns cost $9,316, the elevated slabs cost $43,089, and the walls cost unknown. Next, the cost of formwork and the related labor costs were determined. Assuming 4 use job built plywood, the total cost associated with formwork amounts to approximately$1,497,186. With several variations of waffle slab dominating the structural design, the structural cast in place concrete forming is labor intensive. Although these values appear somewhat too large, it was expected that the form work would account for a large portion of the total cost. Next, steel reinforcement in place costs were determined. In total, the steel reinforcement cost nearly $500,000. Finally, the concrete floor finishing was included in the unit cost estimate. It was assumed that the floor area per level was large enough to require the power screed, bull float; machine float & trowel (ride on). The final "Extended Total Overhead and Profit" amounts to $2,328,956.

A cost estimate was then completed for the original structural design. First the steel decking and welded wire fabric were considered. The 54,375 square feet of decking costs $288,731. Next, the 554 CSF of 6x6 W2.9xW29 welded wire fabric adds an additional $62,391 to the project cost. Cost of purchasing and placing the concrete amounts to $88,673 which is just 5 % of the total cost. Once again, the concrete was assumed to be ready mix and placed using a concrete pump as opposed to a crane and bucket. Accounting for the majority of the project cost, the structural steel columns and beams are projected to cost $530,811 and $665,956 respectively. The "Extended Total Overhead and Profit" cost for the original structure is approximately $1,739,035. As one can see, when comparing just the structural components of the two structures, the original structure amounts to just 74.99% of the new design.

Construction Management Breadth Continued:

Durations: In order to generate construction schedules, the duration of each labor task had to be determined. Using 2009 RS Means Heavy Construction, typical crews and daily output values were determined. A summary of this information can be found on pages 151 through 157. With the quantity (takeoff value) and the daily output known, the duration of each construction task was determined. Pages 151 through 157 summarize the calculations and results pertaining to the various construction task durations. For the most part, the typical crew obtained from RS Means was maintained; however, in some instances the crews had to be increased to achieve more practical duration times.

Generating Schedules: By analyzing figures F-163 and F-164, and considering other factors such as overlap potential and site congestion issues, construction schedules were generated for the two structural designs. Microsoft project was used to generate the schedules, which can be found in figures F-165 through F-168. Please see pages 158 – 164 for explanations regarding the scheduling strategy as well as a construction schedule and cost comparison.

Material Takeoffs – For New Structural Design

Total Cubic Yards of Concrete - Takeoff Values			
Slabs: f'c = 5.95 Ksi L.W.C.			
Floor	Cubic Feet / Floor	Cubic Yards / Floor	Total Cubic Yards
10 thru 13	2102	77.85	311.41
3 thru 9	1782	66	462
2	1579	58.48	58.48
1	2859	105.89	105.89
			937.78
Columns: f'c = 4 ksi N.W.C.			
13	173.25	6.416666667	6.416666667
12	173.25	6.416666667	6.416666667
11	173.25	6.416666667	6.416666667
10	177.04	6.557037037	6.557037037
9	181.12	6.708148148	6.708148148
8	183.75	6.805555556	6.805555556
7	187.54	6.945925926	6.945925926
6	193.9567	7.183581481	7.183581481
5	197.19	7.303333333	7.303333333
4	198.33	7.345555556	7.345555556
3	211.75	7.842592593	7.842592593
2	218.753	8.101962963	8.101962963
1	500	18.51851852	18.51851852
			84.04369259
Shear Walls: f'c = 4 ksi N.W.C.			
Wall 1	1292.5	3.682336179	47.87037032
Wall 2	1410	4.017094013	52.22222217
Wall 3	1410	4.017094013	52.22222217
Wall 4	2233	6.361823355	82.70370362
Wall 5	2115	6.02564102	78.33333326
Wall 6	1880	5.356125351	69.62962956
Wall 7	1527.5	4.351851848	56.57407402
			439.5555551

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Form Work For Elevated Slabs Total Square Footage of Contact Area					
Floors 10 - 13					
Type of Void	# of Voids	SFCA / Void	Total SFCA For Voids	Total SFCA For Floor Area	Overall SFCA / Floor
20" X 20"	390	4.44	1731.6	3446	5838
16" X 20"	165	4	660		
				Cummulative SFCA For Floors 10 -13: 23350	
Floors 3 - 9					
20" X 20"	748	4.44	3321.12	3446	6767
				Cummulative SFCA For Floors 3-9: 47369	
Floor 2					
20" X 20"	763	4.44	3387.72	4015	7726.72
16" X 20"	81	4	324		
				SFCA For Floor 2: 7727	
Floor 1					
N/A	N/A	N/A	N/A	4015	4015
				SFCA For Floors 1: 4015	
				Total SFCA For All Elevated Slabs: 82,461	

Material Takeoffs – For New Structural Design (Continued)

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Form Work For Concrete Columns Total Square Footage of Contact Area				
	Column	# Columns	SFCA / Column	TOTAL SFCA
Floor 13	A	5	35	175
	B	13	35	455
	C	3	42	126
	F	1	42	42
			Cumulative SFCA For Columns: 798	
Floor 12	B	18	35	630
	C	4	42	168
			Cumulative SFCA For Columns: 798	
Floor 11	A	1	35	35
	B	17	35	595
	C	4	42	168
			Cumulative SFCA For Columns: 798	
Floor 10	A	1	35	35
	B	17	35	595
	C	3	42	126
	D	1	49	49
			Cumulative SFCA For Columns: 805	
Floor 9	B	20	35	700
	C	2	42	84
	D	1	49	49
			Cumulative SFCA For Columns: 833	
Floor 8	B	18	35	630
	C	4	42	168
	F	1	42	42
			Cumulative SFCA For Columns: 840	
Floor 7	B	18	35	630
	C	4	42	168
	D	1	49	49
			Cumulative SFCA For Columns: 847	
Floor 6	B	16	35	560
	C	5	42	210
	D	1	49	49
	F	1	42	42
			Cumulative SFCA For Columns: 861	
Floor 5	B	15	35	525
	C	5	42	210
	D	1	49	49
	F	2	42	84
			Cumulative SFCA For Columns: 868	
Floor 4	B	14	35	490
	C	5	42	210
	D	1	49	49
	F	3	42	126
			Cumulative SFCA For Columns: 875	
Floor 3	B	13	35	455
	C	5	42	210
	D	1	49	49
	E	1	56	56
	F	3	42	126
			Cumulative SFCA For Columns: 896	
Floor 2	A	3	35	105
	B	9	35	315
	C	5	42	210
	D	2	49	98
	E	1	56	56
	F	3	42	126
			Cumulative SFCA For Columns: 910	
Floor 1	A	1	66.67	66.67
	B	5	66.67	333.35
	C	9	80	720
	D	3	93.3	279.9
	E	2	56	112
	F	3	80	240
			Cumulative SFCA For Columns: 1752	
			Total Overall SFCA For All Columns: 11,881	

This table displays the total number of each reinforced concrete column size per floor. Refer to page 73 for the corresponding column dimensions. The then show the calculated amount of SFCA needed for columns at each floor.

Note: The total amount of formwork required for shear walls is 20,000 square feet of contact area.

Material Takeoffs – For New Structural Design (Continued)

Column Steel Reinforcement Takeoffs
Total Tonage of Steel Reinforcement

	Bar Size (Type)	Lineal Feet of Bar	lb/ft of bar	lbs	Tonage
Floor 13	# 3 (verts.)	1239	0.376	465.864	0.232932
	# 6 (verts.)	168	1.502	252.336	0.126168
	# 8 (verts.)	42	2.76	115.92	0.05796
	# 3 (hoop ties)	1370	0.376	515.12	0.25756
			Cumulative Tonage of Steel: .67462		
Floor 12	# 3 (verts.)	1134	0.376	426.384	0.213192
	# 6 (verts.)	168	1.502	252.336	0.126168
	# 3 (hoop ties)	1176	0.376	442.176	0.221088
			Cumulative Tonage of Steel: .5604		
Floor 11	# 3 (verts.)	1155	0.376	434.28	0.21714
	# 6 (verts.)	168	1.502	252.336	0.126168
	# 3 (hoop ties)	1120	0.376	421.12	0.21056
			Cumulative Tonage of Steel: .5539		
Floor 10	# 3 (verts.)	1155	0.376	434.28	0.21714
	# 5 (verts.)	42	1.043	43.806	0.021903
	# 6 (verts.)	126	1.502	189.252	0.094626
	# 9 (verts.)	42	3.4	142.8	0.0714
	# 3 (hoop ties)	1125	0.376	423	0.2115
			Cumulative Tonage of Steel: .6166		
Floor 9	# 3 (verts.)	1260	0.376	473.76	0.23688
	# 5 (verts.)	42	1.043	43.806	0.021903
	#6 (verts.)	84	1.502	126.168	0.063084
	# 9 (verts.)	42	3.4	142.8	0.0714
	# 3 (hoop ties)	760	0.376	285.76	0.14288
			Cumulative Tonage of Steel: .5536		
Floor 8	# 3 (verts.)	1134	0.376	426.384	0.213192
	# 6 (verts.)	210	1.502	315.42	0.15771
	# 8 (verts.)	42	2.76	115.92	0.05796
	# 3 (hoop ties)	1146	0.376	430.896	0.215448
			Cumulative Tonage of Steel: .64431		
Floor 7	# 3 (verts.)	1134	0.376	426.384	0.213192
	# 5 (verts.)	42	1.043	43.806	0.021903
	# 6 (verts.)	168	1.502	252.336	0.126168
	# 9 (verts.)	42	3.4	142.8	0.0714
	# 3 (hoop ties)	1152	0.376	433.152	0.216576
			Cumulative Tonage of Steel: .6492		

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Material Takeoffs – For New Structural Design (Continued)

Floor 6	# 3 (verts.)	1008	0.376	379.008	0.189504
	# 5 (verts.)	42	1.043	43.806	0.021903
	# 6 (verts.)	252	1.502	378.504	0.189252
	# 8 (verts.)	42	2.76	115.92	0.05796
	# 9 (verts.)	42	3.4	142.8	0.0714
	# 3 (hoop ties)	1093	0.376	410.968	0.205484
			Cumulative Tonage of Steel: .7355		
Floor 5	# 3 (verts.)	945	0.376	355.32	0.17766
	# 5 (verts.)	42	1.043	43.806	0.021903
	# 6 (verts.)	294	1.502	441.588	0.220794
	# 8 (verts.)	84	2.76	231.84	0.11592
	# 9 (verts.)	42	3.4	142.8	0.0714
	# 3 (hoop ties)	1067	0.376	401.192	0.200596
			Cumulative Tonage of Steel: .8083		
Floor 4	# 3 (verts.)	882	0.376	331.632	0.165816
	# 5 (verts.)	42	1.043	43.806	0.021903
	# 6 (verts.)	336	1.502	504.672	0.252336
	# 8 (verts.)	126	2.76	347.76	0.17388
	# 9 (verts.)	42	3.4	142.8	0.0714
	# 3 (hoop ties)	1037	0.376	389.912	0.194956
			Cumulative Tonage of Steel: .8803		
Floor 3	# 3 (verts.)	819	0.376	307.944	0.153972
	# 5 (verts.)	42	1.043	43.806	0.021903
	# 6 (verts.)	336	1.502	504.672	0.252336
	# 8 (verts.)	168	2.76	463.68	0.23184
	# 9 (verts.)	42	3.4	142.8	0.0714
	# 3 (hoop ties)	1008	0.376	379.008	0.189504
			Cumulative Tonage of Steel: .9210		
Floor 2	# 3 (verts.)	819	0.376	307.944	0.153972
	# 5 (verts.)	84	1.043	87.612	0.043806
	# 6 (verts.)	420	1.502	630.84	0.31542
	# 8 (verts.)	126	2.76	347.76	0.17388
	# 9 (verts.)	84	3.4	285.6	0.1428
	# 3 (hoop ties)	976	0.376	366.976	0.183488
			Cumulative Tonage of Steel: 1.013		
Floor 1	# 3 (verts.)	399	0.376	150.024	0.075012
	# 5 (verts.)	126	1.043	131.418	0.065709
	# 6 (verts.)	672	1.502	1009.344	0.504672
	# 8 (verts.)	210	2.76	579.6	0.2898
	# 9 (verts.)	126	3.4	428.4	0.2142
	# 3 (hoop ties)	827	0.376	310.952	0.155476
			Cumulative Tonage of Steel: 1.305		
		TOTAL TONAGE OF STEEL FOR COLUMNS = 9.682			

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Material Takeoffs – For New Structural Design (Continued)

Elevated Slab Steel Reinforcement Takeoffs Total Tonage of Steel Reinforcement					
Floor	**Top Reinforcement (Bar size / Lineal Feet)**	**Bottom Reinforcement (Bar Size / Lineal Feet)**	**Total Reinforcement (Bar Size / Lineal Feet)**	**Lb / ft**	**Tonage of Steel Reinforcement**
13	# 5 / 4142	# 5 / 4982	# 5 / 9124	1.043	4.758
12	# 5 / 4142	# 5 / 4982	# 5 / 9124	1.043	4.758
11	# 5 / 4142	# 5 / 4982	# 5 / 9124	1.043	4.758
10	# 5 / 4142	# 5 / 4982	# 5 / 9124	1.043	4.758
9	# 5 / 3667	# 5 / 3750	# 5 / 7417	1.043	3.868
8	# 5 / 3667	# 5 / 3750	# 5 / 7417	1.043	3.868
7	# 5 / 3667	# 5 / 3750	# 5 / 7417	1.043	3.868
6	# 5 / 3667	# 5 / 3750	# 5 / 7417	1.043	3.868
5	# 5 / 3667	# 5 / 3750	# 5 / 7417	1.043	3.868
4	# 5 / 3667	# 5 / 3750	# 5 / 7417	1.043	3.868
3	# 5 / 3667	# 5 / 3750	# 5 / 7417	1.043	3.868
2	# 5 / 5065	# 5 / 4038	# 5 / 9103	1.043	4.747
	N/A	# 6 / 92	# 6 / 92	1.502	0.0691
1	# 5 / 7255.87	# 5 / 6299.4	# 5 / 13555	1.043	7.069
		Total Tonage Of Elevated Slab Steel Reinforcement = 57.9931			

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Shear Wall Steel Reinforcement Takeoffs Total Tonage of Steel Reinforcement				
Shear Wall	Vertical Shear Reinforcement (Bar size / Lineal Feet)	Horizontal Shear Reinforcement (Bar Size / Lineal Feet)	Flexural Reinforcement (Bar Size / Lineal Feet)	Total Weight of Reinforcement (tonage)
1	# 4 / 1410	# 4 / 2068	# 9 / 5076	9.79
2	# 4 / 1551	# 4 / 2256	# 9 / 5076	9.9
3	# 4 / 1551	# 4 / 2256	# 9 / 5076	9.9
4	# 4 / 1974	# 4 / 3572	# 9 / 6768	13.36
5	# 4 / 1974	# 4 / 3384	# 9 / 6768	13.23
6	# 4 / 1692	# 4 / 3008	# 9 / 5640	22315.6
7	# 4 / 1833	# 4 / 2444	# 9 / 5640	22033
		Total Tonage of Shear Wall Reinforcement = 44,400		

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Material Takeoffs – Components Common To Both New and Original Design (Continued):

Several components of the structural system were not redesigned. These components include the perimeter basement level concrete walls and the slab on grade. Below are the calculations and results of the material takeoff values for the slab on grade and the concrete basement walls.

Slab on Grade: 8" L.W.C Slab reinforced with # 5 bars @ 12" spacing running both directions.

Concrete: $(slab\ depth)(slab\ width)(slab\ length)(CY\ conversion\ factor)$

$$\left(\frac{8}{12}\right) * (60') * (80') * (.037037037) = \boxed{\mathbf{118.5\ Cubic\ Yards}}$$

Steel Reinforcement: $\text{Tons} = \left(\frac{slab\ length}{bar\ spacing}\right) * (Length\ of\ bar) * (lb/ft) * \left(\frac{ton}{2000\ lbs}\right)$

5 bar: 1.043 lb /ft

$$\left(\frac{80}{1}\right) * (60') * (1.403) * \left(\frac{1\ ton}{2000\ lbs}\right) = 3.367\ Tons$$

$$\left(\frac{60}{1}\right) * (80') * (1.403) * \left(\frac{1\ ton}{2000\ lbs}\right) = 3.367\ Tons$$

Total Tons of Steel Reinforcement in Slab on Grade: $\boxed{6.74 \text{ Tons}}$

4 Concrete Perimeter Basement Walls: 12" Thick x 12' high Reinforced N.W.C. Walls. Reinforced With (2) # 4 @ 12" (horizontals) and (2) # 5 @ 12" (verticals).

Formwork: $Total\ Wall\ Perimeter * 2 * Wall\ Height$

$$[2(60') + 2(80')] * 2 * 12' = \boxed{6720 \text{ SFCA}}$$

Concrete: $(Total\ Wall\ Perimeter) * (Wall\ Thickness) * (Wall\ Height) * (CY\ Conv.factor)$

$$[2(60) + 2(80)] * (1') * (12') * (.037037037) = \boxed{\mathbf{124.44\ Cubic\ Yards}}$$

Steel Reinforcement: $2 * \left(\frac{Total\ Wall\ Length}{Bar\ Spacing}\right) * (Wall\ Height) * \left(\frac{lb}{ft}\right) * \left(\frac{Ton}{2000\ lbs}\right)$

$$2 * \left(\frac{280}{1}\right) * (12') * (.668) * \left(\frac{ton}{2000}\right) = \boxed{\mathbf{2.244\ Tons}}$$

$$2 * \left(\frac{280}{1}\right) * (12') * (1.043) * \left(\frac{ton}{2000}\right) = \boxed{\mathbf{3.504\ Tons}}$$

Material Takeoffs – For Original Design:

Linear Feet Takeoffs – Structural Steel Columns

Floor	Column Shape	# Identical Columns	Column Height	Total Linear Feet
1st				
	W10x33	1	21.1	21.1
	W12x96	1	23.2	23.2
	W12x120	16	23.2	371.2
	W12x106	1	23.2	23.2
	W10x77	1	23.2	23.2
	W10x54	1	23.2	23.2
	W14x132	3	31.67	95.01
	W10x60	1	23.2	23.2
	W14x132	1	23.2	23.2
	W14x109	1	31.67	31.67
	W10x88	1	31.67	31.67
2nd - 3rd				
	W10x88	10	21.5	215
	W10x77	10	21.5	215
	W10x59	1	21.5	21.5
	W10x49	2	21.5	43
	W10x54	1	21.5	21.5
	W10x68	3	21.5	64.5
4th-5th				
	W10x68	10	21.5	215
	W10x49	3	21.5	64.5
	W10x39	1	21.5	21.5
	W10x54	5	21.5	107.5
	W10x45	3	21.5	64.5
	W10x60	2	21.5	43
	W10x77	2	21.5	43
	W10x35	1	21.5	21.5
6th-7th				
	W10x68	7	21.5	150.5
	W10x60	3	21.5	64.5
	W10x54	2	21.5	43
	W10x39	6	21.5	129
	W10x33	2	21.5	43
	W10x49	5	21.5	107.5
	W10x33	1	29.75	29.75

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Material Takeoffs – For Original Design (Continued):

Linear Feet Takeoffs – Structural Steel Columns Continued

8th-9th				
	W10x54	3	21.5	64.5
	W10x45	2	21.5	43
	W10x49	6	21.5	129
	W10x33	4	21.5	86
	W10x60	3	21.5	64.5
	W10x39	8	21.5	172
10th-11th				
	W10x49	7	21.5	150.5
	W10x39	6	21.5	129
	W10x45	3	21.5	64.5
	W10x33	9	21.5	193.5
	W10x54	1	21.5	21.5
12th-13th				
	W10x39	3	19	57
	W10x33	6	19	114
	W10x45	4	19	76
	W10x39	2	21.5	43
	W10x33	5	21.5	107.5
	W10x45	4	21.5	86
	W10x54	1	21.5	21.5
	W10x49	1	21.5	21.5
Penthouse/Roof				
	W10x39	3	19.1	57.3
	W10x39	3	9.4	28.2
	W10x39	1	14.5	14.5
	W10x33	1	11.7	11.7
	W10x33	2	9.4	18.8
	W10x33	1	19.1	19.1
	W10x33	3	24.5	73.5

F-151 Continued

Structure Steel Column Takeoffs Summary of Column Type and Lineal Feet	
Column Type	Lineal Feet
W10 x 33	626.95
W10 x 35	21.5
W10 x 39	651.5
W10 x 45	364
W10 x 49	516
W10 x 54	302.7
W10 x 59	21.5
W10 x 60	172
W10 x 68	430
W10 x 77	281.2
W10 x 88	246.67
W12 x 96	23.92
W12 x 106	23.2
W12 x 120	371.2
W14 x 109	31.67
W14 x 132	23.2

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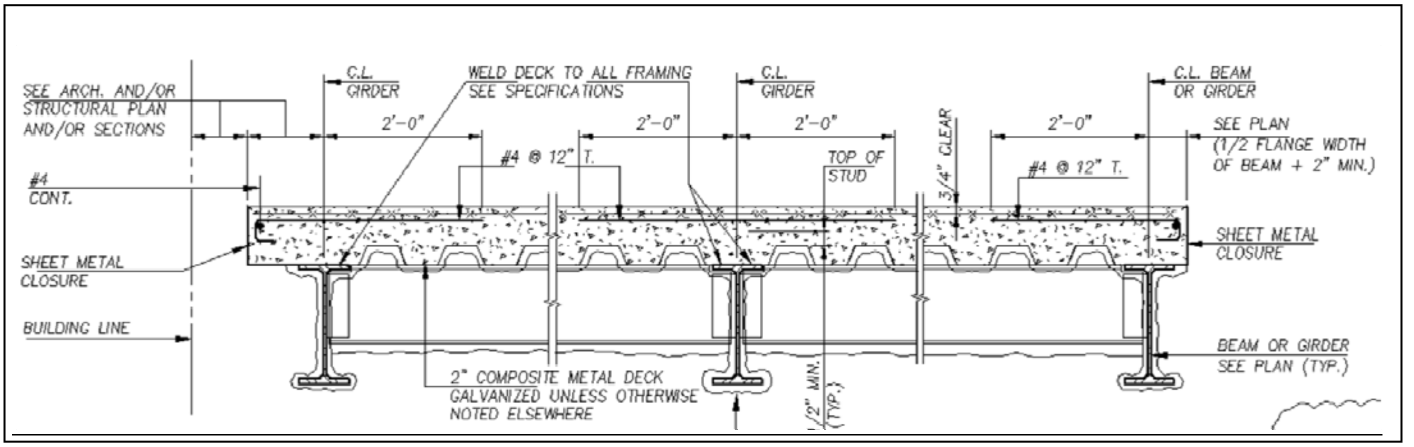
Material Takeoffs – For Original Design (Continued):



Typical Elevated Slab – 2” Composite Metal Deck (18 Guage). 2-1/2” L.W.C. Topping.

Metal Decking:

Elevated Slab Takeoffs - Steel Decking 2" Composite Metal Decking - 18 Gauge	
Floor	Square Feet of Metal Decking
13	4125
12	4125
11	4125
10	4125
9	4125
8	4125
7	4125
6	4125
5	4125
4	4125
3	4125
2	4500
1	4500
Total	54375

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Welded Wire Fabric:

Elevated Slab Takeoffs - Weld Wire Fabric 6 x 6 / W3 x W3 W.W.F.		
Floor	SF	CSF
13	4125	41.25
12	4125	41.25
11	4125	41.25
10	4125	41.25
9	4125	41.25
8	4125	41.25
7	4125	41.25
6	4125	41.25
5	4125	41.25
4	4125	41.25
3	4125	41.25
2	4500	45
1	4500	45
Total	54375	543.75

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Material Takeoffs – For Original Design (Continued):

Elevated Slab Takeoffs - Steel Reinforcement Total Tonage						
Floor	Lineal Feet (# 6 Bars)	Lineal Feet (# 4 Bars)	lbs (# 6 Bars)	lbs (# 4 bars)	Total lbs	Tons
13	1300	1800	1952.6	1202.4	3155	1.5775
12	1300	1800	1952.6	1202.4	3155	1.5775
11	1300	1800	1952.6	1202.4	3155	1.5775
10	1300	1800	1952.6	1202.4	3155	1.5775
9	1300	1800	1952.6	1202.4	3155	1.5775
8	1300	1800	1952.6	1202.4	3155	1.5775
7	1300	1800	1952.6	1202.4	3155	1.5775
6	1300	1800	1952.6	1202.4	3155	1.5775
5	1300	1800	1952.6	1202.4	3155	1.5775
4	1300	1800	1952.6	1202.4	3155	1.5775
3	1300	1800	1952.6	1202.4	3155	1.5775
2	1300	2000	1952.6	1336	3288.6	1.6443
1	1300	2000	1952.6	1336	3288.6	1.6443
Total	1300	2000	1952.6	1336	3288.6	20.6411

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Material Takeoffs – For Original Design (Continued):

Lineal Feet Takeoffs - Structural Steel Beams FIRST FLOOR			
Beam	# of Identical Beams	Beam Length	Total Lineal Feet Per Floor
W12x22	4	14	56
W12x22	1	17.5	17.5
W12x22	5	8.75	43.75
W12x22	1	6.6	6.6
W12x22	1	12	12
W12x26	4	17.5	70
W12x26	2	12	24
W10x15	6	6.75	40.5
W10x15	1	8.75	8.75
W10x15	2	7.8	15.6
W10x15	6	5.1	30.6
W10x15	2	12	24
W14x22	1	12.6	12.6
W14x22	3	13.9	41.7
W14x22	1	14.3	14.3
W14x22	1	21	21
W14x22	1	15.6	15.6
W12x19	2	13.9	27.8
W12x19	1	14.5	14.5
W12x19	1	10	10
W12x19	1	17.4	17.4
W10x12	5	5.1	25.5
W16x26	2	23.1	46.2
W21x182	1	23.1	23.1
W21x182	2	14.4	28.8
W24x176	1	12.5	12.5
W12x16	1	5	5
W12x16	1	8.5	8.5
W24x279	2	13	26
W18x35	1	11.6	11.6
W18x35	1	18.5	18.5
W10x30	2	6.6	13.2
W16x45	1	17.1	17.1
W10x17	2	8.9	17.8
W24x306	1	15.1	15.1
W16x36	1	17.6	17.6
W16x31	1	14.1	14.1
W24x131	1	14.1	14.1
W24x250	1	15.4	15.4
W8x24	1	8.75	8.75
W12x35	1	17.6	17.6
W14x30	1	14.6	14.6
W18x119	1	23.1	23.1
W21x201	1	23.1	23.1
W8x15	2	7.75	15.5
W12x35	1	17.6	17.6
W24x279	1	17.5	17.5
W8x13	1	10.1	10.1
W8x13	2	2.9	5.8

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Material Takeoffs – For Original Design (Continued):

Lineal Feet Takeoffs - Structural Steel Beams Floor 2 - 13				
Beam Size	# of Identical Beams	Length of Beam	Total Lineal Feet Per Floor	Cumulative Over All Typical Floors
W12x22	3	15.75	47.25	567
W12x22	1	14	14	168
W12x22	1	11	11	132
W12x22	3	17	51	612
W12x22	3	14.7	44.1	529.2
W12x22	2	11.6	23.2	278.4
W8x10	4	7	28	336
W8x10	9	2.3	20.7	248.4
W8x10	2	8.1	16.2	194.4
W8x10	3	4.3	12.9	154.8
W8x10	2	12	24	288
W10x12	4	6.75	27	324
W10x12	2	7.8	15.6	187.2
W10x15	1	13.11	13.11	157.32
W10x15	3	11	33	396
W10x15	1	14.4	14.4	172.8
W12x30	2	14.4	28.8	345.6
W12x30	2	13.11	26.22	314.64
W12x30	1	15	15	180
W12x30	1	17.5	17.5	210
W12x30	1	28	28	336
W12x30	1	8.75	8.75	105
W12x30	1	7.9	7.9	94.8
W12x16	3	15.4	46.2	554.4
W12x16	1	22.6	22.6	271.2
W12x16	1	17.5	17.5	210
W12x16	1	23.1	23.1	277.2
W12x16	2	10.4	20.8	249.6
W12x19	5	23.1	115.5	1386
W12x19	2	15.4	30.8	369.6
W12x19	1	10.9	10.9	130.8
W8x13	3	8.9	26.7	320.4
W12x35	1	25	25	300
W12x35	1	17	17	204
W12x26	2	15.67	31.34	376.08
W12x26	3	17.6	52.8	633.6
W12x26	1	11.8	11.8	141.6
W12x26	1	19	19	228
W12x30	1	14	14	168
W12x30	1	13	13	156
W10x17	1	10	10	120

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Material Takeoffs – For Original Design (Continued):

Structural Steel Beam Takeoffs Summary Beam Type and Lineal Feet	
Beam Size	Lineal Feet
W8 x 10	1221.6
W8 x 13	336.3
W8 x 15	15.5
W8 x 24	8.75
W10 x 12	511.2
W10 x15	845.45
W10 x 17	137.8
W10 x 30	13.2
W12 x 16	1755.9
W12 x 19	1886.4
W12 x 22	2422.45
W12 x 26	1473
W12 x30	1704
W12 x35	521.6
W14 x 22	105.2
W14 x 30	14.6
W16 x 26	46.2
W16 x 31	14.1
W16 x 36	17.6
W16 x45	17.1
W18 x 15	30.1
W18 x 119	23.1
W21 x 182	51.9
W21 x 201	23.1
W24 x 131	14.1
W24 x 176	12.5
W24 x 250	15.4
W24 x 279	43.5
W24 x 306	15.1

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RS MEANS Unit Cost Estimates:

RS MEANS Online Cost Works Analysis - Unit Cost Construction Estimate NEW STRUCTURAL DESIGN					
Description	**Quantity**	**Units**	**Crew**	**Extended Total**	**Extended Total O&P**
Structural Concrete					
L.W. , 5000 psi, Elevated Slabs	938	CY		$161,242	$176,550
N.W. , 4000 psi, Ready Mix, Columns	85	CY		$9,803	$10,728
N.W. , 4000 psi, Ready Mix, Walls	440	CY		$50,745	$55,532
Placing Concrete (Labor,Equipment Included)					
12" Square Columns, Pumped	85	CY	C-20	$6,397	$9,316
Elevated Slabs, less than 6", Pumped	833	CY	C-20	$26,981	$38,735
Elevated Slabs, 6" - 10"	105	CY	C-20	$2,973	$4,354
Walls				$0	$0
Structural Cast In Place Concrete Forming					
Elevated Slab - flat plate, job built plywood, 4 use	82461	SFCA	C-2	$636,598	927,686
Columns - job built plywood, 12"x12" columns, 4 use	11881	SFCA	C-2	$127,958	193,660
Shear Walls - 8' - 16' High, Job Built Plywood, 4 use	25920	SFCA	C-2	$248,054	375,840
Steel Reinforcement In Place					
Elevated Slab Steel Reinforcement (#4 - #7)	58	Ton	4 Rodm	$156,309	$200,433
Shear Walls, Steel Reinforcement (#4 - #7)	79	Ton	4 Rodm	$195,893	$253,733
Columns, Steel Reinforcement (#3 - #7)	6	Ton	4 Rodm	$20,793	$28,619
Columns, Steel Reinforcement (#8 - #18)	3.68	Ton	4 Rodm	$10,437	$13,868
Concrete Floor Finishing					
Power Screed, Bull Float, Machine Float & Trowel (Ride On)	48230	SF	C-10E	$21,221	$29,902
			Total	**$1,675,404**	**$2,318,956**

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RS MEANS Unit Cost Estimates:

RS MEANS ONLINE COST WORKS - UNIT COST CONSTRUCTION ESTIMATE ORIGINAL STRUCTURE (Steel Frame - Slab On Metal Decking)					
Item	**Quantity**	**Unit**	**Crew**	**Extended Total**	**Extended Total O&P**
Steel Decking - Floor Decking					
Non Cellular Composite Deck, Galvanized, 2" deep, 18 guage	54375	SF	E-4	$243,056	$288,731
Welded Wire Fabric					
6x6 W2.9xW2.9 (6x6) Sheets, 42 lb per CSF	544	CSF	4 Rodm	$44,281	$62,391
Structural Concrete					
L.W.C 4000 psi, ready mix	402	CY	N/A	$63,420	69,980
Placing Concrete					
Elevated Slab < 6" Thick, pumped	402	CY	C-20	13,021	18,693
Steel Reinforcement - In Place					
Elevated Slab: #4 - #7 , uncoated	21	Ton	4 Rodm	$56,594	$72,570
Concrete Floor Finishing					
Power Screed, Bull Float, Machine Float & Trowel (Ride On)	48230	SF	E-10	$21,221	$29,903
Structural Steel Framing - Columns					
W10 x 45	1664	LF	E-2	$136,963	$153,288
W10 x 68	1970	LF	E-2	$239,118	$266,856
W12 x 120	418	LF	E-2	$87,997	$97,799
W14 x 120	55	LF	E-2	$11,579	$12,868
Structural Steel Members - Beams					
W8 x 10	1582	LF	E-2	$41,179	$50,861
W10 x 15	1507	LF	E-2	$52,488	$62,254
W12 x 22	9763	LF	E-2	$428,400	$493,129
W14 x 30	119	LF	E-2	$6,807	$7,773
W16 x 40	95	LF	E-2	$7,128	$8,078
W18 x 106	208	LF	E-2	$39,305	$43,861
			Total:	**$1,492,557**	**$1,739,035**

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Construction Durations, Crews, and Outputs:

In order to properly compare the new and original structural designs, non-structural matters had to be considered including construction labor and material cost estimates as well as analysis of crew requirements and approximation of daily output and construction duration.

In order to generate a construction schedule, RS Means Heavy Construction Cost Data Handbook (23rd Edition) was referenced to determine the duration, labor requirements, and overlap potential of various construction tasks. First, structural material takeoffs were completed for both the new and original design. For each of these takeoff values, typical crew requirements and daily outputs were obtained from RS Means 2009. The following set of data summarizes the breakdown of daily output according to material type and labor task.

RS MEANS 03 11: Concrete Forming

Forms In Place, Columns

*12"x12" Columns, 4 Use, Job Built Plywood
Crew: C-1
Daily Output: 225 SFCA

*16"x16" Columns, 4 Use, Job Built Plywood
Crew: C-1
Daily Output: 235 SFCA

Forms In Place, Elevated Slabs

*Flat Plate, 15', 4 Use, 15' Max Columns, Job Built Plywood
Crew: C -2
Daily Output: 560 SFCA

Forms In Place, Walls

*8' – 16' High, 4 use, Job Built Plywood
Crew: C-2
Daily Output: 395 SFCA

RS MEANS 03 21: Reinforcing Steel, In Place

Columns

*#3 - #7 Reinforcing Steel
Crew: 4 Rodm
Daily Output: 1.5 Tons

*#8 - #18 Reinforcing Steel
Crew: 4 Rodm
Daily Output: 2.3 Tons

Elevated Slabs

*#4 - #7 Reinforcing Steel
Crew: 4 Rodm
Daily Output: 2.9 Tons

Construction Durations, Crews, and Outputs (continued):

Slab On Grade

*#3 - #7 Reinforcing Steel

Crew: 4 Rodm

Daily Output: 2.3 Tons

Walls

*#3 - #7 Reinforcing Steel

Crew: 4 Rodm

Daily Output: 3 Tons

RS Means: Construction Duration Requirements Continued:

RS MEANS 03 31: Structural Concrete – Placing Concrete

Columns

*12" Thick, Square Column

Crew: C-20

Daily Output: 60 CY

Walls

*8" Thick, Pumped

Crew: C-20

Daily Output: 100 CY

Elevated Slabs

*Less Than 6" Thick, Pumped

Crew: C-20

Daily Output: 140 CY

*6" – 10" Thick, Pumped

Crew: C-20

Daily Output: 160 Cy

RS MEANS 03 35: Concrete Finishing of Floors

Elevated Slabs

*Power Screed, Bull Float, Machine Float & Trowel (Ride on)

Crew: C-10E

Daily Output: 4000 SF

RS MEANS 03 22: Welded Wire Fabric

*6x6 W2.9xW2.9 (6x6): 42 lbs per CSF

Crew: 2 Rodm

Daily Output: 29 CSF

Construction Durations, Crews, and Outputs (continued):

RS MEANS 05 12: Strutural Steel Framing

Structure Steel Column RS MEANS Typical Crew and Daily Output		
Column Type	Typical Crew	Daily Output (LF / Day)
W10 x 33	E 10	600
W10 x 35	E 10	600
W10 x 39	E 10	600
W10 x 45	E 10	600
W10 x 49	E 10	600
W10 x 54	E 10	600
W10 x 59	E 10	600
W10 x 60	E 10	600
W10 x 68	E 10	600
W10 x 77	E 10	600
W10 x 88	E 10	600
W12 x 96	E 10	880
W12 x 106	E 10	880
W12 x 120	E 10	880
W14 x 109	E 10	880
W14 x 132	E 10	880

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RS Means: Construction Duration Requirements Continued:

Structural Steel Beams RS MEANS Daily Output and Typical Crew		
Beam Size	Typical Crew	Daily Output (LF / DAY)
W8 x 10	E - 2	600
W8 x 13	E - 2	600
W8 x 15	E - 2	600
W8 x 24	E - 2	550
W10 x 12	E - 2	600
W10 x15	E - 2	600
W10 x 17	E - 2	600
W10 x 30	E - 2	600
W12 x 16	E - 2	880
W12 x 19	E - 2	880
W12 x 22	E - 2	880
W12 x 26	E - 2	880
W12 x30	E - 2	880
W12 x35	E - 2	810
W14 x 22	E - 2	990
W14 x 30	E - 2	900
W16 x 26	E - 2	900
W16 x 31	E - 2	900
W16 x 36	E - 2	900
W16 x45	E - 2	800
W18 x 15	E - 2	1000
W18 x 119	E - 5	900
W21 x 182	E - 5	1036
W21 x 201	E - 5	1036
W24 x 131	E - 5	1036
W24 x 176	E - 5	1036
W24 x 250	E - 5	1036
W24 x 279	E - 5	1036
W24 x 306	E - 5	1036

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Duration of Construction Process – New Structural Design:

CONSTRUCTION SCHEDULE - DURATION OF EVENTS				
ITEM	QUANTITY	CREW	DAILY OUTPUT	DURATION (DAYS)
S.O.G. Steel Reinforcement				
S.O.G. Placing Concrete				
Basement Walls - Form Work				
Basement Columns - Form Work				
Basement Walls - Steel Reinforcement				
Basement Columns - Steel Reinforcement				
Basement Walls - Placing Concrete				
Basement Columns - Place Concrete				
1st Floor Elevated Slab - Form Work (SFCA)	4015	(4)C-2	2240	1.792410714
1st Floor Columns - Form Work (SFCA)	1752	(8)C-1	940	1.863829787
1st Floor Shear Walls - Formwork (SFCA)	1600	(4)C-2	1580	1.012658228
1st Floor Elevated Slab - Steel Reinforcement (Tons)	7.069	4 Rodm	2.9	2.437586207
1st Floor Columns - Steel Reinforcement (Tons)	1.305	4 Rodm	1.5	0.87
1st Floor Shear Walls - Steel Reinforcement (Tons)	6	4 rodm	3	2
1st Floor Elevated Slab - Placing Concrete (CY)	105.89	C-20	140	0.756357143
1st Floor Columns - Placing Concrete (CY)	18.51	C-20	60	0.1851
1st Floor Shear Walls - Placing Concrete (CY)	62	C-20	100	0.62
1st Floor Elevated Slab - Concrete Finishing (SF)	3500	C-10E	4000	0.875
2nd Floor Elevated Slab - Form Work (SFCA)	7727	(6)C-2	3360	2.299702381
2nd Floor Columns - Form Work (SFCA)	910	(4)C-1	900	1.011111111
2nd Floor Shear Walls - Formwork (SFCA)	1851	(4)C-2	1580	1.171518987
2nd Floor Elevated Slab - Steel Reinforcement (Tons)	4.81	4 Rodm	2.9	1.65862069
2nd Floor Columns - Steel Reinforcement (Tons)	1.013	4 Rodm	1.5	0.675333333
2nd Floor Shear Walls - Steel Reinforcement (Tons)	6	4 rodm	3	2
2nd Floor Elevated Slab - Placing Concrete (CY)	58.46	C-20	140	0.417571429
2nd Floor Columns - Placing Concrete (CY)	8.1	C-20	60	0.135
2nd Floor Shear Walls - Placing Concrete (CY)	62	C-20	100	0.62
2nd Floor Elevated Slab - Concrete Finishing (SF)	3500	C-10E	4000	0.875
3rd Floor Elevated Slab - Form Work (SFCA)	6767	(6)C-2	3360	2.013988095
3rd Floor Columns - Form Work (SFCA)	896	(3)C-1	705	1.270921986
3rd Floor Shear Walls - Formwork (SFCA)	1851	(4)C-2	1580	1.171518987
3rd Floor Elevated Slab - Steel Reinforcement (Tons)	3.868	4 Rodm	2.9	1.333793103
3rd Floor Columns - Steel Reinforcement (Tons)	0.921	4 Rodm	1.5	0.614
3rd Floor Shear Walls - Steel Reinforcement (Tons)		4 Rodm	3	0
3rd Floor Elevated Slab - Placing Concrete (CY)	462	(3)C-20	420	1.1
3rd Floor Columns - Placing Concrete (CY)	7.84	C-20	60	0.130666667
3rd Floor Shear Walls - Placing Concrete (CY)	62	C-20	100	0.62
3rd Floor Elevated Slab - Concrete Finishing (SF)	3500	C-10E	4000	0.875
4th Floor Elevated Slab - Form Work (SFCA)	6767	(6)C-2	3360	2.013988095
4th Floor Columns - Form Work (SFCA)	875	(3)C-1	705	1.241134752
4th Floor Shear Walls - Formwork (SFCA)	1851	(4)C-2	1580	1.171518987
4th Floor Elevated Slab - Steel Reinforcement (Tons)	3.868	4 Rodm	2.9	1.333793103
4th Floor Columns - Steel Reinforcement (Tons)	0.8803	4 Rodm	1.5	0.586866667
4th Floor Shear Walls - Steel Reinforcement (Tons)	6	4 rodm	3	2
4th Floor Elevated Slab - Placing Concrete (CY)	462	(3)C-20	420	1.1
4th Floor Columns - Placing Concrete (CY)	7.35	C-20	60	0.1225
4th Floor Shear Walls - Placing Concrete (CY)	62	C-20	100	0.62
4th Floor Elevated Slab - Concrete Finishing (SF)	3500	C-10E	4000	0.875

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Duration of Construction Process – New Structural Design (Continued):

5th Floor Elevated Slab - Form Work (SFCA)	6767	(6)C-2	3360	2.013988095
5th Floor Columns - Form Work (SFCA)	868	(3)C-1	705	1.231205674
5th Floor Shear Walls - Formwork (SFCA)	1851	(4)C-2	1580	1.171518987
5th Floor Elevated Slab - Steel Reinforcement (Tons)	3.868	4 Rodm	2.9	1.333793103
5th Floor Columns - Steel Reinforcement (Tons)	0.8083	4 Rodm	1.5	0.538866667
5th Floor Shear Walls - Steel Reinforcement (Tons)	6	4 Rodm	3	2
5th Floor Elevated Slab - Placing Concrete (CY)	462	(3)C-20	420	1.1
5th Floor Columns - Placing Concrete (CY)	7.3	C-20	60	
5th Floor Shear Walls - Placing Concrete (CY)	62	C-20	100	1.033333333
5th Floor Elevated Slab - Concrete Finishing (SF)	3500	C-10E	4000	35
6th Floor Elevated Slab - Form Work (SFCA)	6767	(6)C-2	3360	1.69175
6th Floor Columns - Form Work (SFCA)	861	(3)C-1	705	1.221276596
6th Floor Shear Walls - Formwork (SFCA)	1851	(4)C-2	1580	1.171518987
6th Floor Elevated Slab - Steel Reinforcement (Tons)	3.868	4 Rodm	2.9	1.333793103
6th Floor Columns - Steel Reinforcement (Tons)	0.7355	4 Rodm	1.5	0.490333333
6th Floor Shear Walls - Steel Reinforcement (Tons)	6	4 rodm	3	2
6th Floor Elevated Slab - Placing Concrete (CY)	462	(3)C-20	420	1.1
6th Floor Columns - Placing Concrete (CY)	7.18	C-20	60	0.119666667
6th Floor Shear Walls - Placing Concrete (CY)	62	C-20	100	0.62
6th Floor Elevated Slab - Concrete Finishing (SF)	3500	C-10E	4000	0.875
7th Floor Elevated Slab - Form Work (SFCA)	6767	(6)C-2	3360	2.013988095
7th Floor Columns - Form Work (SFCA)	847	(3)C-1	705	1.20141844
7th Floor Shear Walls - Formwork (SFCA)	1851	(4)C-2	1580	1.171518987
7th Floor Elevated Slab - Steel Reinforcement (Tons)	3.868	4 Rodm	2.9	1.333793103
7th Floor Columns - Steel Reinforcement (Tons)	0.6492	4 Rodm	1.5	0.4328
7th Floor Shear Walls - Steel Reinforcement (Tons)		4 Rodm	3	0
7th Floor Elevated Slab - Placing Concrete (CY)	462	(3)C-20	420	1.1
7th Floor Columns - Placing Concrete (CY)	6.95	C-20	60	0.115833333
7th Floor Shear Walls - Placing Concrete (CY)	62	C-20	100	0.62
7th Floor Elevated Slab - Concrete Finishing (SF)	3500	C-10E	4000	0.875
8th Floor Elevated Slab - Form Work (SFCA)	6767	(6)C-2	3360	2.013988095
8th Floor Columns - Form Work (SFCA)	840	(3)C-1	705	1.191489362
8th Floor Shear Walls - Formwork (SFCA)	1851	(4)C-2	1580	1.171518987
8th Floor Elevated Slab - Steel Reinforcement (Tons)	3.868	4 Rodm	2.9	1.333793103
8th Floor Columns - Steel Reinforcement (Tons)	0.64331	4 Rodm	1.5	0.428873333
8th Floor Shear Walls - Steel Reinforcement (Tons)	6	4 rodm	3	2
8th Floor Elevated Slab - Placing Concrete (CY)	462	(3)C-20	420	1.1
8th Floor Columns - Placing Concrete (CY)	6.8	C-20	60	0.113333333
8th Floor Shear Walls - Placing Concrete (CY)	62	C-20	100	0.62
8th Floor Elevated Slab - Concrete Finishing (SF)	3500	C-10E	4000	0.875

F-163 Continued

Duration of Construction Process – New Structural Design:

9th Floor Elevated Slab - Form Work (SFCA)	6767	(6)C-2	3360	2.013988095
9th Floor Columns - Form Work (SFCA)	833	(3)C-1	705	1.181560284
9th Floor Shear Walls - Formwork (SFCA)	1851	(4)C-2	1580	1.171518987
9th Floor Elevated Slab - Steel Reinforcement (Tons)	3.868	4 Rodm	2.9	1.333793103
9th Floor Columns - Steel Reinforcement (Tons)	0.5536	4 Rodm	1.5	0.369066667
9th Floor Shear Walls - Steel Reinforcement (Tons)	6	4 Rodm	3	2
9th Floor Elevated Slab - Placing Concrete (CY)	462	(3)C-20	420	1.1
9th Floor Columns - Placing Concrete (CY)	6.7	C-20	60	0.111666667
9th Floor Shear Walls - Placing Concrete (CY)	62	C-20	100	0.62
9th Floor Elevated Slab - Concrete Finishing (SF)	3500	C-10E	4000	0.875
10th Floor Elevated Slab - Form Work (SFCA)	5838	(6)C-2	3360	1.7375
10th Floor Columns - Form Work (SFCA)	805	(3)C-1	705	1.141843972
10th Floor Shear Walls - Formwork (SFCA)	1851	(4)C-2	1580	1.171518987
10th Floor Elevated Slab - Steel Reinforcement (Tons)	4.758	4 Rodm	2.9	1.640689655
10th Floor Columns - Steel Reinforcement (Tons)	0.6166	4 Rodm	1.5	0.411066667
10th Floor Shear Walls - Steel Reinforcement (Tons)	6	4 rodm	3	2
10th Floor Elevated Slab - Placing Concrete (CY)	311	(3)C-20	420	0.74047619
10th Floor Columns - Placing Concrete (CY)	6.56	C-20	60	0.109333333
10th Floor Shear Walls - Placing Concrete (CY)	62	C-20	100	0.62
10th Floor Elevated Slab - Concrete Finishing (SF)	3500	C-10E	4000	0.875
11th Floor Elevated Slab - Form Work (SFCA)	5838	(6)C-2	3360	1.7375
11th Floor Columns - Form Work (SFCA)	798	(3)C-1	705	1.131914894
11th Floor Shear Walls - Formwork (SFCA)	1851	(4)C-2	1580	1.171518987
11th Floor Elevated Slab - Steel Reinforcement (Tons)	4.758	4 Rodm	2.9	1.640689655
11th Floor Columns - Steel Reinforcement (Tons)	0.5539	4 Rodm	1.5	0.369266667
11th Floor Shear Walls - Steel Reinforcement (Tons)	6	4 Rodm	3	2
11th Floor Elevated Slab - Placing Concrete (CY)	311	(3)C-20	420	0.74047619
11th Floor Columns - Placing Concrete (CY)	6.42	C-20	60	0.107
11th Floor Shear Walls - Placing Concrete (CY)	62	C-20	100	0.62
11th Floor Elevated Slab - Concrete Finishing (SF)	3500	C-10E	4000	0.875
12th Floor Elevated Slab - Form Work (SFCA)	5838	(6)C-2	3360	1.7375
12th Floor Columns - Form Work (SFCA)	798	(3)C-1	705	1.131914894
12th Floor Shear Walls - Formwork (SFCA)	1851	(4)C-2	1580	1.171518987
12th Floor Elevated Slab - Steel Reinforcement (Tons)	4.758	4 Rodm	2.9	1.640689655
12th Floor Columns - Steel Reinforcement (Tons)	0.5604	4 Rodm	1.5	0.3736
12th Floor Shear Walls - Steel Reinforcement (Tons)	6	4 rodm	3	2
12th Floor Elevated Slab - Placing Concrete (CY)	311	(3)C-20	420	0.74047619
12th Floor Columns - Placing Concrete (CY)	6.41	C-20	60	0.106833333
12th Floor Shear Walls - Placing Concrete (CY)	62	C-20	100	0.62
12th Floor Elevated Slab - Concrete Finishing (SF)	3500	C-10E	4000	0.875
13th Floor Elevated Slab - Form Work (SFCA)	5838	(6)C-2	3360	1.7375
13th Floor Columns - Form Work (SFCA)	798	(3)C-1	705	1.131914894
13th Floor Shear Walls - Formwork (SFCA)	1851	(4)C-2	1580	1.171518987
13th Floor Elevated Slab - Steel Reinforcement (Tons)	4.758	4 Rodm	2.9	1.640689655
13th Floor Columns - Steel Reinforcement (Tons)	0.675	4 Rodm	1.5	0.45
13th Floor Shear Walls - Steel Reinforcement (Tons)	6	4 Rodm	3	2
13th Floor Elevated Slab - Placing Concrete (CY)	311	(3)C-20	420	0.74047619
13th Floor Columns - Placing Concrete (CY)	6.41	C-20	60	0.106833333
13th Floor Shear Walls - Placing Concrete (CY)	62	C-20	100	0.62
13th Floor Elevated Slab - Concrete Finishing (SF)	3500	C-10E	4000	0.875

Durations Used to Determine Construction Schedule Of New Concrete Structure Shown in Figures F-165 and F-166

F-163 Continued

Duration of Construction Process – Original Structural Design:

Used to generate the construction schedule shown in figure F-167 and F-168.

CONSTRUCTION SCHEDULE - DURATION OF EVENTS				
ITEM	QUANTITY	CREW	DAILY OUTPUT	DURATION (DAYS)
1st Floor Columns (LF)	689	E-10	880	0.782954545
1st Floor Beams (LF)	1008	E-5	900	1.12
1st Floor - Decking (SF)	4500	E - 4	3,380	1.331360947
1st Floor - Steel Reinforcement (Tons)	1.6433	4 Rodm	2.9	0.566655172
1st Floor - WWF (CSF)	45	2 Rodm	29	1.551724138
2nd/3rd Floor Columns (LF)	581	E-10	880	0.660227273
2nd/3rd Floor Beams (LF)	956	E-5	880	1.086363636
4th/5th Floor Columns (LF)	581	E-10	600	0.968333333
4th/5th Floor Beams (LF)	956	E-2	880	1.086363636
6th/7th Floor Columns (LF)	567	E-10	600	0.945
6th/7th Floor Beams (LF)	956	E-2	880	1.086363636
8th/9th Floor Columns (LF)	559	E-10	600	0.931666667
8th/9th Floor Beams (LF)	956	E-2	880	1.086363636
9th/10th Floor Columns (LF)	559	E-10	600	0.931666667
9th/10th Floor Beams (LF)	956	E-2	880	1.086363636
11th/12th Floor Columns (LF)	526	E-10	600	0.876666667
11th/12th Floor Beams (LF)	956	E-2	880	1.086363636
13th Floor Beams (LF)	956	E-2	880	1.086363636
2nd Floor Decking (SF)	4500	E - 4	3,380	1.331360947
3rd Floor Decking (SF)	4125	E - 4	3,380	1.220414201
4th Floor Decking (SF)	4125	E - 4	3,380	1.220414201
5th Floor Decking (SF)	4125	E - 4	3,380	1.220414201
6th Floor Decking (SF)	4125	E - 4	3,380	1.220414201
7th Floor Decking (SF)	4125	E - 4	3,380	1.220414201
8th Floor Decking (SF)	4125	E - 4	3,380	1.220414201
9th Floor Decking (SF)	4125	E - 4	3,380	1.220414201
10th Floor Decking (SF)	4125	E - 4	3,380	1.220414201
11th Floor Decking (SF)	4125	E - 4	3,380	1.220414201
12th Floor Decking (SF)	4125	E - 4	3,380	1.220414201
13th Floor Decking (SF)	4125	E - 4	3,380	1.220414201
2nd Floor Slab Steel Reinforcement (Tons)	1.6433	4 Rodm	2.9	0.566655172
2nd Floor WWF (CSF)	45	2 Rodm	29	1.551724138
3rd Floor Slab Steel Reinforcement (Tons)	1.5755	4 Rodm	2.9	0.543275862
3rd Floor WWF (CSF)	41.25	2 Rodm	29	1.422413793
4th Floor Slab Steel Reinforcement (Tons)	1.5755	4 Rodm	2.9	0.543275862
4th Floor WWF (CSF)	41.25	2 Rodm	29	1.422413793
5th Floor Slab Steel Reinforcement (Tons)	1.5755	4 Rodm	2.9	0.543275862
5th Floor WWF (CSF)	41.25	2 Rodm	29	1.422413793
6th Floor Slab Steel Reinforcement (Tons)	1.5755	4 Rodm	2.9	0.543275862
6th Floor WWF (CSF)	41.25	2 Rodm	29	1.422413793
7th Floor Slab Steel Reinforcement (Tons)	1.5755	4 Rodm	2.9	0.543275862
7th Floor WWF (CSF)	41.25	2 Rodm	29	1.422413793
8th Floor Slab Steel Reinforcement (Tons)	1.5755	4 Rodm	2.9	0.543275862
8th Floor WWF (CSF)	41.25	2 Rodm	29	1.422413793
9th Floor Steel Reinforcement (Tons)	1.5755	4 Rodm	2.9	0.543275862
9th Floor WWF (CSF)	41.25	2 Rodm	29	1.422413793
10th Floor Slab Steel Reinforcement (Tons)	1.5755	4 Rodm	2.9	0.543275862
10th Floor WWF (CSF)	41.25	2 Rodm	29	1.422413793
11th Floor Slab Steel Reinforcement (Tons)	1.5755	4 Rodm	2.9	0.543275862
11th Floor WWF (CSF)	41.25	2 Rodm	29	1.422413793
12th Floor Slab Steel Reinforcment (Tons)	1.5755	4 Rodm	2.9	0.543275862
12th Floor WWF (CSF)	41.25	2 Rodm	29	1.422413793
13th Floor Slab Steel Reinforcement (Tons)	1.5755	4 Rodm	2.9	0.543275862
13th Floor WWF (CSF)	41.25	2 Rodm	29	1.422413793
1st Floor Place Concrete (CY)	33.33	C-20	140	0.238071429
1st Floor Concrete Floor Finishing (SF)	3700	C-10E	4000	0.925
2nd Flooor Place Concrete (CY)	30.44	C-20	140	0.217428571
2nd Floor Concrete Floor Finishing (SF)	3500	C-10E	4000	0.875
3rd Floor Place Concrete (CY)	30.44	C-20	140	0.217428571
3rd Floor Concrete Floor Finishing (SF)	3500	C-10E	4000	0.875
4th Floor Place Concrete (CY)	30.44	C-20	140	0.217428571
4th Floor Concrete Floor Finishing (SF)	3500	C-10E	4000	0.875
5th Floor Place Concrete (CY)	30.44	C-20	140	0.217428571
5th Floor Concrete Floor Finishing (SF)	3500	C-10E	4000	0.875
6th Floor Place Concrete (CY)	30.44	C-20	140	0.217428571
6th Floor Concrete Floor Finishing (SF_	3500	C-10E	4000	0.875
7th Floor Place Concrete (CY)	30.44	C-20	140	0.217428571
7th Floor Concrete Floor Finishing (SF)	3500	C-10E	4000	0.875
8th Floor Place Concrete (CY)	30.44	C-20	140	0.217428571
8th Floor Concrete Floor Finishing (SF)	3500	C-10E	4000	0.875
9th Floor Place Concrete (CY)	30.44	C-20	140	0.217428571
9th Floor Concrete Floor Finishing (SF)	3500	C-10E	4000	0.875
10th Floor Place Concrete (CY)	30.44	C-20	140	0.217428571
10th Floor Concrete Floor Finishing (SF)	3500	C-10E	4000	0.875
11th Floor Place Concrete (CY)	30.44	C-20	140	0.217428571
11th Floor Concrete Floor Finishing (SF)	3500	C-10E	4000	0.875
12th Floor Place Concrete (CY)	30.44	C-20	140	0.217428571
12th Floor Concrete Floor Finishing (SF)	3500	C-10E	4000	0.875
13th Floor place Concrete (CY)	30.44	C-20	140	0.217428571
13th Floor Concrete Floor Finishing (SF)	3500	C-10E	4000	0.875

F-164

Construction Schedule Explanation – New Structural Design:

As is immediately evident upon observing the construction schedule on the following page, the construction process has a long duration with limited overlap potential. This is characteristic of a typical high rise concrete structure. For the most part, each successive floor is schedule in a finish to start manner. Even with shoring, the need to cure and finish concrete, as well as the need to strip and construct formwork restricts builders from overlapping construction of successive floors.

A significant amount of overlap exists between the form work, reinforcement, and placement of concrete. Since the schedule is organized into a simplified cycle of F/R/P for elevated slabs, then columns, then walls, this overlap is not clearly visible but does exist and helps to reduce durations. Note, the small blue bar on the schedules below represents a concrete floor finishing which has a finish to finish relationship to the F/R/P of elevated slabs. When observing table F-163, it's evident the duration times would not be practical without overlapping the formwork, reinforcement, and placement of concrete. Assuming the site is large enough to accommodate multiple trades simultaneously, the start of the reinforcement can slightly lag behind form work to expedite the process.

As illustrated in the following construction schedule, a typical construction cycle for an entire floor level consists first of the F/R/P for elevated slabs (blue). No events overlap with the slab F/R/P. The column and wall F/R/Ps are assumed to be in a Finish to Start Relationship with the slab construction.

As shown by the schedule, the Walls (Green) overlap with the Columns (Red) since neither of them are dependent upon the other's progress completion. Since the site is small, a limit on the number of crew members on site has to be carefully monitored. In response to this restriction, the start of shear wall construction (for each floor) lags a ½ day behind the commencement of column construction to reduce crew numbers on site.

As shown by the close up view of the construction schedule in figure F-166, a typical level takes approximately 7 (8) hour days. In total, construction of the new structural design takes (95) 8 hour work days. In New York City, where construction occurs at a very high rate, this would not suffice, especially in such a competitive market. Therefore, an increase in crews can be expected, as well as increase in hours per day. However, the goal of this analysis was not to fit the schedule within a specific time frame. Instead, the goal was to compare duration and cost of two different structures. As a result, to avoid introducing another variable, it was critical that crews not be changed significantly from the RS Means typical crew listing. Only in some cases, were duration times so outstanding that a large boost in crew numbers was required. For example, table F-163 show various tasks in which the typical crew is multiplied by a factor as large as 8.

CONSTRUCTION SCHEDULES – New Structural Design

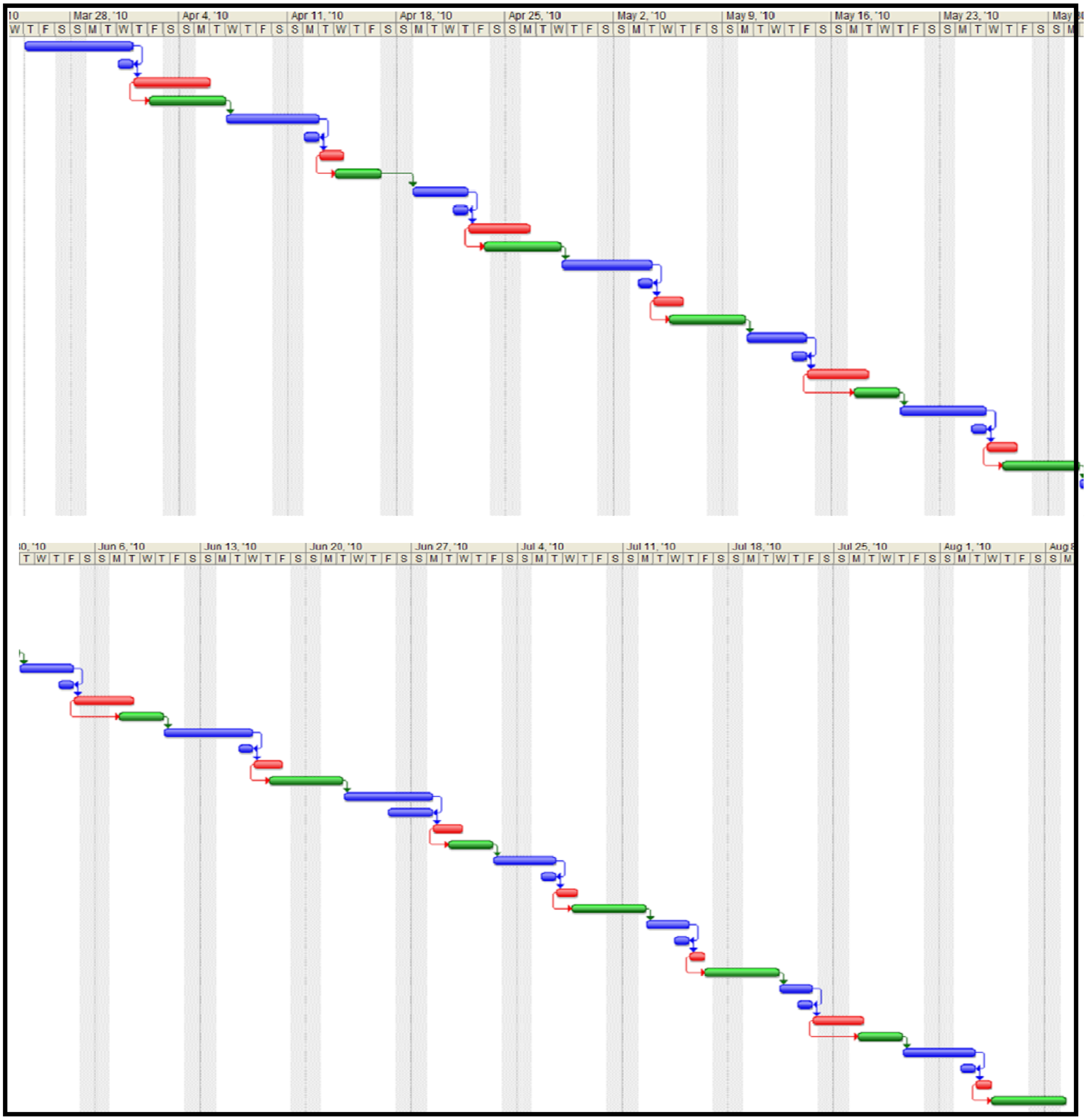
NEW STRUCTURAL DESIGN: CONSTRUCTION SCHEDULE

Total Duration: (95) 8 Hour Work Days

F/R/P Slabs:

F/R/P Columns:

F/R/P Shear Walls:

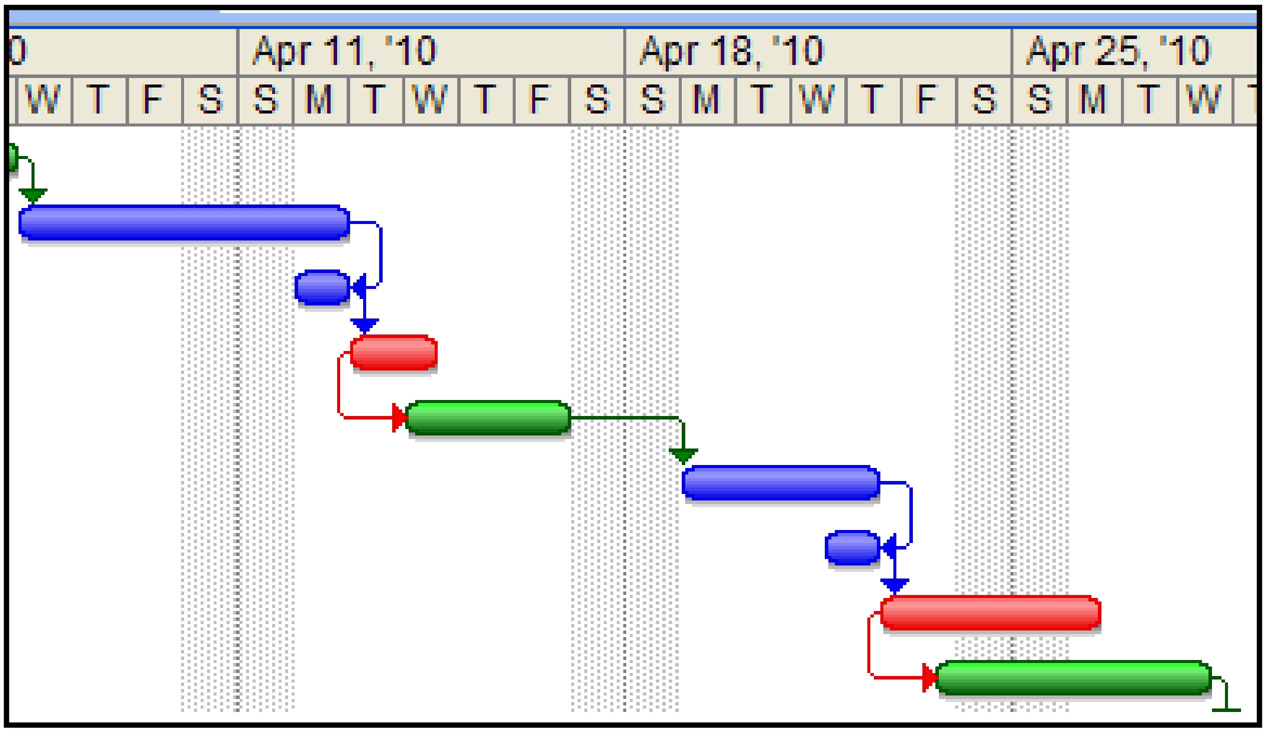


F-165

Construction Schedule – New Structural Design Continued:

Construction Schedule – New Design

Close Up View of Construction Schedule showing 2 Cycles of The Construction process for a Typical Floor.



F-166

Construction Schedule Explanation – Original Structural Design:

Unlike the concrete structural design, the existing 13 floor steel frame building can be constructed via a more compact process. Since the actual construction schedule and cost information was not available, a schedule and estimate were determined for the original structure as well. Once the steel frame is erected, a great deal of overlap between construction tasks exists. As shown in the following schedule diagram, the construction process is extremely delinearized. The schedule is organized into five specific tasks: Steel frame erection (blue), steel decking (red), steel reinforcement (black), welded wire fabric (green), and placement of concrete (gray).

In the close up view of the schedule (figure F-168), the second floor metal decking (first red bar) starts as soon as the second floor steel framing is in place. Each successive level of decking is done on immediately on a finish-to-start basis. As shown on the schedule diagrams, the 2nd floor steel reinforcement and welded wire fabric are done upon completion of second floor decking. Much like the steel decking, the successive floors of steel reinforcement and welded wire fabric are completed in a finish-to-start fashion. Since the decking is below the reinforcement, and the reinforcement is positioned below the welded wire fabric, this sequence of tasks must be maintained. Next, the placement of concrete at the second floor begins upon completion of the second floor welded wire fabric and steel reinforcement. At this point, the rest of the slabs are poured one after another.

As one can see, for several days during construction, decking, steel reinforcement, welded wire fabric, and concrete pouring are all occurring simultaneously (or at least overlap). Note, half of the steel frame is erected right away, and the top half doesn't begin until later. The strategy behind this decision was governed by site congestion potential. By looking at the close up view, one will notice that on April 11, the concrete pouring begins as the first half of the steel frame erection ceases. In other words, dividing up the steel frame erection guarantees that at no single time will concrete pouring and steel frame erection occur. This strategy enables maximization of overlap potential without creating site congestion issues.

Overall, the construction process for the steel frame building only requires 37 (8) hour work days. This is less than ½ the time to construct the concrete structure. Reduced construction duration not only limits general conditions costs but improves reputation in the construction market and enables earlier occupation by future tenants.

Construction Schedule – Original Structural Design:

Original Structural Design: Construction Schedule

Total Duration: (37) 8 Hour Work Days

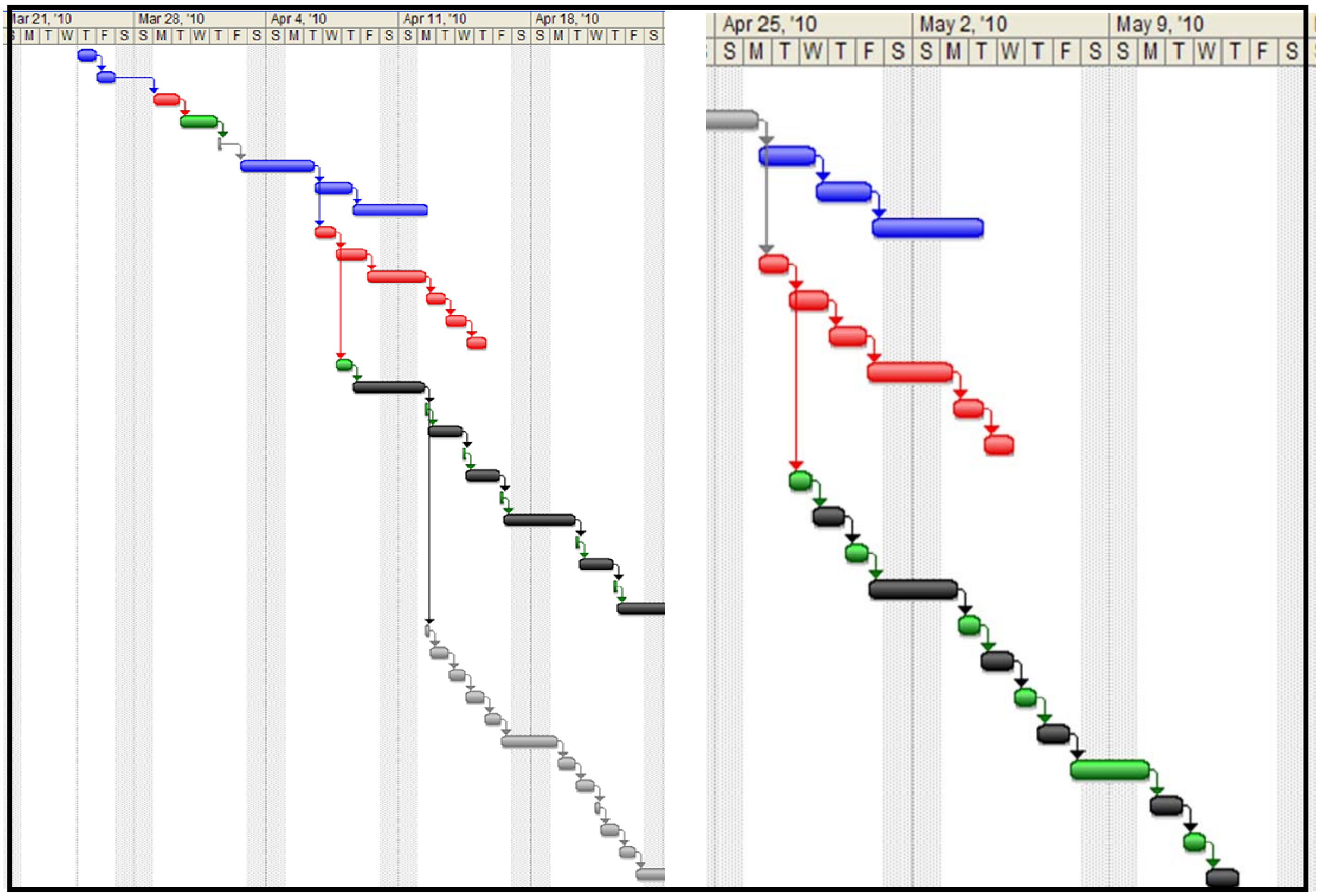
Steel Framing:

Steel Reinforcement:

Decking:

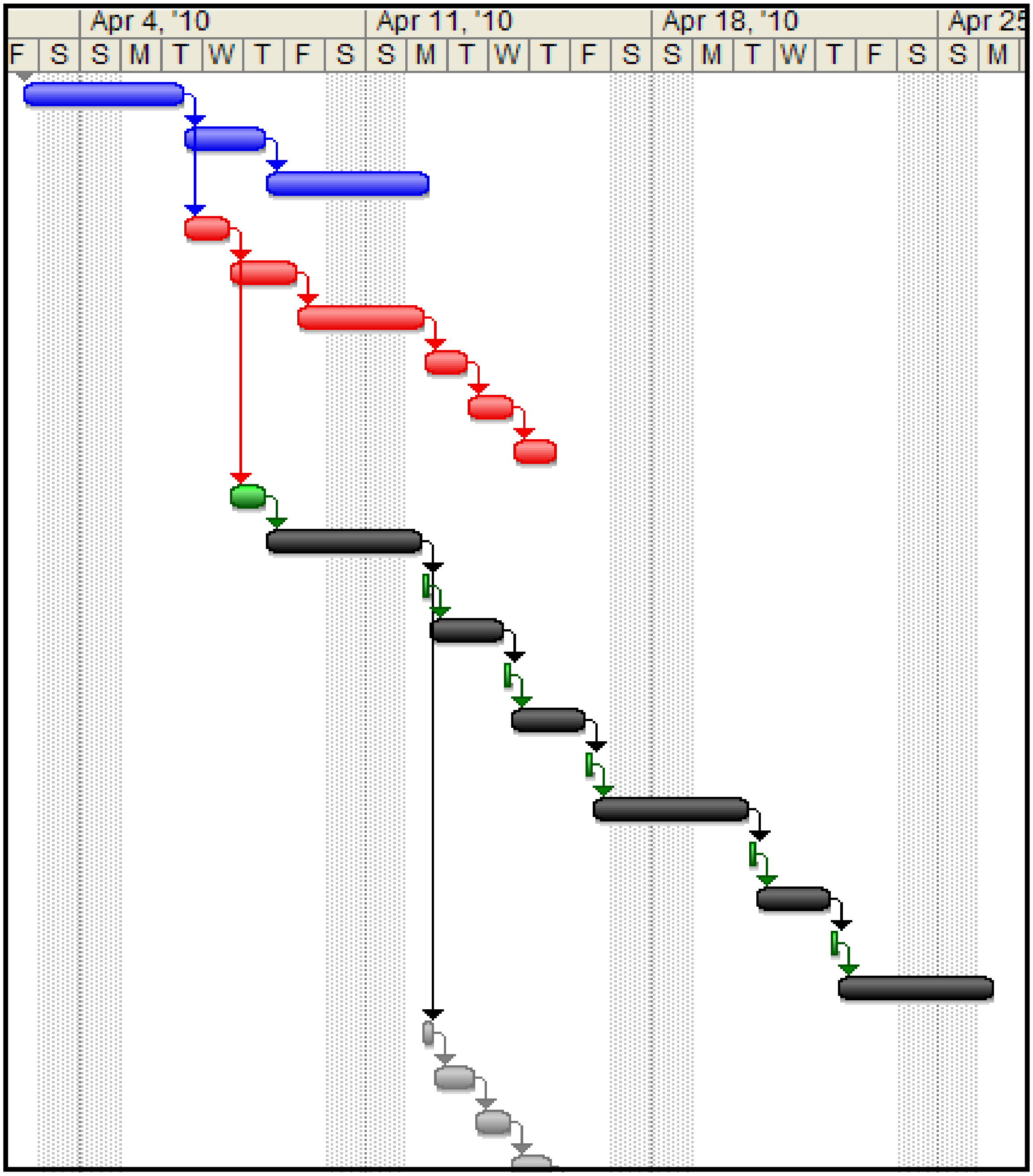
Placing Concrete:

WWF:



F-167

Construction Schedule – Original Structural Design Continued:



This Section of the Construction schedule shows how overlapping of framing, decking, steel reinforcement, and concrete placement results in a compact schedule. This allows for fast tracking the project meaning short labor times and reduced general costs.

F-168

Acoustic Breadth:

Aside from the first floor retail spaces, 40 Gold Street is comprised of 13 residential floors. Overall, the building includes 40 two bedroom apartments and 18 studio apartments, which amounts to approximately 62,000 square feet of residential space. As the owner emphasized during the design phase of the existing structure, there is a fine line between failing and thriving of any New York City residential endeavor. Although this applies to various sectors of the residential market, it is specifically relevant to the direct relation between customer satisfaction and the quality of a building's design. In a competitive area such as New York City, residential spaces have to be designed and constructed without overlooking any details. Since residential buildings are noise-sensitive, the acoustic performance of 40 Gold Street is of upmost importance. Located in a densely populated and highly active urban environment, ambient noise levels for 40 Gold Street are very high. After visiting the site and surrounding areas in the summer of 2009, several specific sources of noise were identified. With two alley ways running immediately up against the construction site perimeter, several private businesses are located within close proximity of the site. As a result, delivery trucks and waste management vehicles will regularly generate noise near the site. In addition, two highly travelled streets are located close to the building. Also, the site is located near a major subway station. Although a few blocks away, the World Trade Center ground zero site generates considerable noise as it is very active with construction workers, tourists, and spectators. In addition to high ambient noise levels, another source of sound exists as result of the building function. Located on top of the 13th floor residential spaces, a penthouse encloses a recreational room with exercise equipment, a laundry room, and a mechanical room. The goal of this acoustic study is to determine what steps should be taken to ensure interior spaces are exposed to an acceptable level of noise. Typically, for private residential occupancy, an acceptable noise level is 30-35 dB, which is nearly equivalent to a Noise Rating (NR) of 30.

The calculation of sound transmission loss and performance is very complex. The frequency of noises varies, and the sound transmission coefficients of wall components within a composite wall assembly are not additive. As a result, extensive research was done pertaining to field and laboratory performance tests of various materials and sound isolation techniques. By applying the knowledge obtained from the research and past architectural engineering coursework, several design solutions were produced in response to the acoustical requirements of 40 Gold Street. The two specific areas of focus are sound isolation of interior spaces from high ambient noises and sound isolation of residential spaces from the aforementioned penthouse noises.

With an ambient noise level at the upwards of 80 dB (approximated), the acoustic design and performance of the exterior façade is crucial in order to guarantee that the building's interior environment satisfies acoustic design criterion. One way to measure sound isolation performance of a construction assembly is Transmission Loss (TL), which is expressed: $\boldsymbol{TL = 10\log\left(\frac{1}{\tau}\right)}$, where τ = the sound transmission coefficient. To improve the TL of the existing exterior wall assembly shown in figure F-169, specific materials were selected according to their sound transmission

Acoustic Breadth Continued:

coefficients. In general, massive, airtight, and reduced stiffness (limp) are characteristics of an excellent sound absorbing material. As one can see in figure F-169, the existing wall assembly consists of 2" metal trespa panels (exterior), air and moisture barrier, 5/8" exterior dens-glass sheathing, 6" metal studs, 6" batt insulation in stud cavity, and 5/8" Gypsum board (interior).

The first step to improving the transmission loss performance of the wall assembly involves increasing the mass. As shown in figure F-171, a double wall can be used to make the wall more massive and airtight. Although the wall will occupy more space, the use of a double wall is extremely effective in sound attenuation. In fact, a double in weight typically increases transmission losses by 5 dB. Also, since ambient sound levels significantly decrease with elevation, the double wall assembly should only be required for the lower half of the structure. Next, by using a double layer of gypsum wall board on the interior, the STC rating can be improved by approximately 3 dB. It is important to note, that gypsum wall board provides mass and reduced stiffness. Even if a wall assembly has a large mass, significant transmission loss cannot be established without obstructing sound transmission paths. To address this problem, research suggests that staggering the studs and using resilient channels to support the metal studs is essential. By staggering the studs as shown in figure F-171 and by using resilient channel attachments shown in figure F-171, an improvement of 6 – 10 dB is often observed. Since several different types of materials are used in the wall design, a larger array of frequencies can be absorbed by the wall assembly. Although the wall weight, cost, and size increase significantly, the long term benefits of customer satisfaction will surely pay off. The final suggested wall detail is shown in figure F-171.

Nearly 20 % of the exterior wall system is glazing. Compared to typical exterior wall construction, the windows perform poorly in sound attenuation. As a result, this acoustic study also involves suggested improvements and redesign of the window systems. Before recommending a window detail for sound isolation, the severity by which glazing reduces a wall systems acoustic performance must be discussed. For composite wall systems, sound transmission lost is expressed via the equation: $TL_{Composite} = 10log\left(\frac{\sum Surface\ Area}{\sum Surface\ Area * \tau}\right)$. As an approximation, assuming typical values of $\tau_{Glazing}$ = 10^{-2} and τ_{Wall} = 10^{-5}, a wall with 20% glazing has a TL:

$$\boxed{TL_{Composite} = 10log\left[\frac{100}{(80 * 10^{-5} + 20 * 10^{-2})}\right] = 10log(498) = 26.97\ dB}$$

As one can see, if glazing design does not address sound isolation issues, the glazing greatly reduces the sound transmission across the wall. First, a single lite of glass does not suffice. Insulating glass units perform well because the air space and different thicknesses of glass enable decent sound attenuation. However, laminated glass units are superior for sound attenuation. A laminated glass unit is two lites of glass glued together with a clear plastic interlayer.

Acoustic Breadth Continued:

It is important to use lites of different thicknesses to widen the range of frequencies the glazing unit can absorb.

Laminated glass is limp and can improve STC ratings by 3 dB. Although some windows should remain operable, as preferred by most residences, the majority of windows should be designed as sealed units. Sealed windows can provide an additional 5 dB of sound transmission loss. As a consequence of using sealed windows, an alternate method of ventilation such as PTAC units must be used. Next, soft neoprene gaskets should be used as well as sound-absorbing material lining in the head, sill, and window jambs.

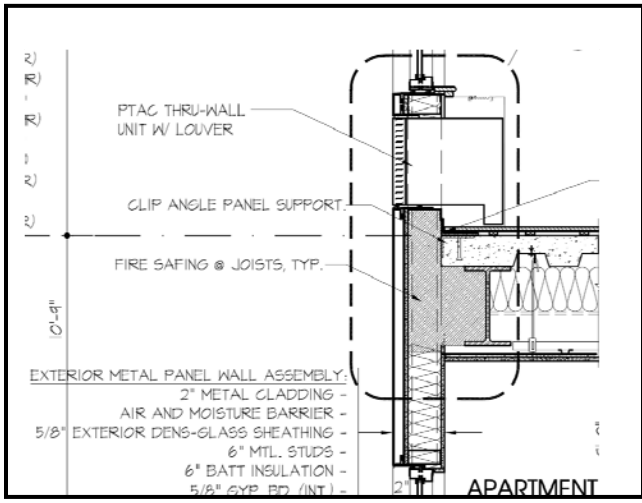
Finally, the sound isolation between the penthouse and 13^{th} floor residential spaces was considered. As mentioned before, there is exercise equipment, a laundry room, and HVAC equipment. As one can see, there are several sources of sound and vibration. The vibrations are constant and large in magnitude. In order to prevent vibration transmission to the below residential spaces, a conventional floor and ceiling construction is inadequate. Although labor intensive and expensive, a floated concrete slab with suspended ceiling should be used. This greatly increases the floor construction depth which requires increasing the 13^{th} floor story height. Figure F-170 shows the suggested floor and ceiling assembly. Once again, this is only required for a single location, which is between the penthouse and the 13^{th} floor residential space. The primary components include a 3" floated concrete slab (top),isolation pads with fiberboard in between, concrete structural slab 4", isolation hanger, and resiliently suspended gypsum board with fibrous insulation.

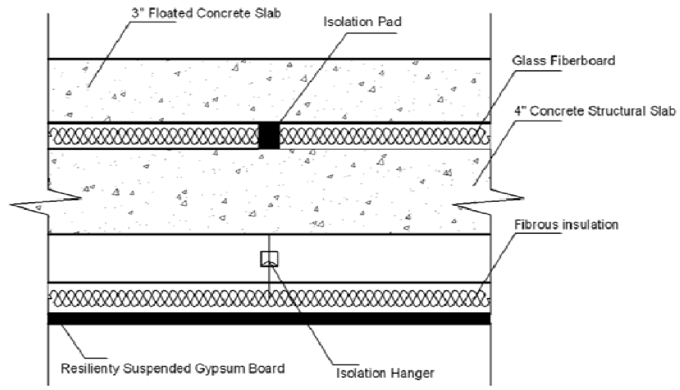
It is important not to simply rely on the floor construction to mitigate vibration and sound transmission. Therefore, the HVAC equipment (boilers and compressors etc.) and laundry room machines should be positioned on 4" concrete housekeeping pedestals. In addition, standing steel springs should be used as the first line of defense in dissipating the energy and vibrations. Occasionally, large amplitude vibrations originating from aging or malfunctioning mechanical equipment can short circuit the springs. As a result, ribbed neoprene pads are also suggested to reinforce performance in sound and vibration transmission loss.

Finally, the existing and new structural floor systems were compared to determine which performs better in sound attenuation between vertically adjacent rooms. The original floor system was 2" 18 gauge composite metal decking with 2" light weight concrete topping. Based on calculations, the original structure is lighter and less effective in vibration control. The structural steel frame is not considered as conducive to sound isolation as solid concrete. In fact, the existing structure requires suspended ceilings and additional material for sound isolation. However, the additional material is also required for fire rating and because the structural components are not architecturally pleasing when left exposed. Based on research, the new concrete floor system performs much better in sound isolation.

Acoustic Breadth Continued – Design Details and Figures:

F-170



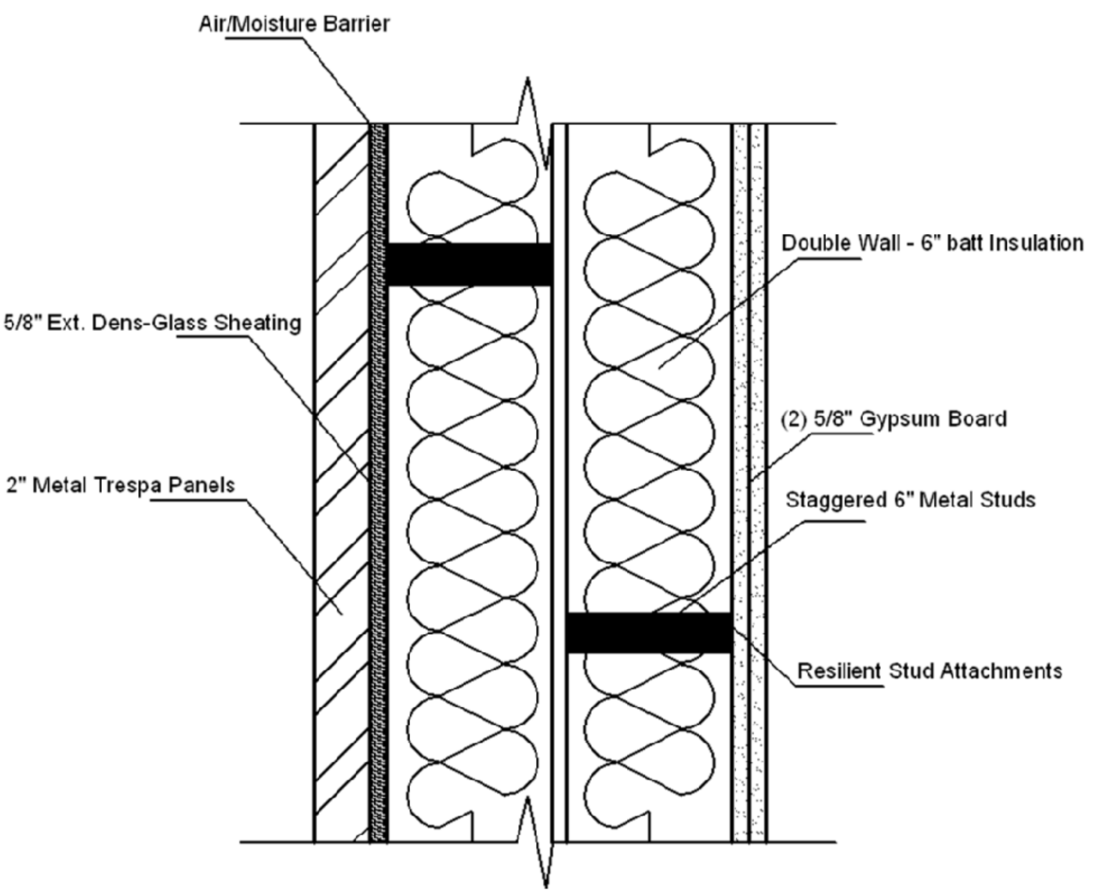


Floated Concrete Slab With Suspended Ceiling

Design For Sound Isolation Between Penthouse and Residential Spaces

F-169

F-171



Suggested Exterior Wall Design

Horizontal Section: Final Design For Sound Isolation of Interior Spaces From High Ambient Noise Levels

Results and Conclusion:

After completing the breadth studies and redesign of the existing structural system (steel framing and slab on metal decking) as a concrete structural system with a flat plate waffle slab, results were carefully examined. As mentioned in the proposal, the goal of the report was to draw conclusions regarding the feasibility and practicality of a concrete structural redesign. This endeavor originated when informed that the owner of 40 Gold Street expressed that if possible, he preferred a concrete structural system. Based on the research and data results produced in this report, the existing and new structural designs were compared in order to draw conclusions regarding the benefits and disadvantages of each system.

The first phase of the report is the structural depth, which is comprised of three specific areas of structural design: slab design, column and corbel design, and the shear wall lateral system design. Several strategies were used to produce an efficient and lightweight slab design, which included small 15'x15 bay sizes, waffle slab design, and the use of high strength lightweight concrete. Several types of slabs were designed due to changes in live loads and column layouts throughout the building. For example the 1st floor requires a 8" flat plate due to the 100 psf retail live load. For the most part the slab is designed as a waffle slab composed of a 3 ½ "slab and 4" wide x 8" deep (11.5" deep total) ribs spaced at 20" or 16". Top reinforcement is generally #5 @ 16" and the bottom reinforcement is either (1) #5 per rib, (2) #5 per rib, or (1) #6 per rib. The waffle slab self weight is approximately 46 psf (other slab types weigh more), a 35% increase from the existing 34 psf slab. Using hand calculations and SpSlab, deflection, flexure, punching shear, two way shear, cover, and various steel reinforcement requirements were checked. Occasionally punching shear around columns and large moments due to cantilevers required special attention. Additional reinforcement and/or thickening of slab resolved all these issues. Upon completion of the slab design, the original and new floor systems were compared. Unlike the existing structure, there is no framing below the slab. The new concrete structural design has a maximum floor depth of 11.5" (8" in some places), whereas the existing floor structure has a floor depth of 14.5" to 16.5". As a result, the new structural design has an improved floor-to-ceiling height. In fact, this provides the designer the option to either maintain the existing building height and increase ceiling heights in the apartments, or lower the building height and maintain the existing ceiling height. For this redesign, the latter option was selected. The building height was reduced by nearly 5 ½', which ultimately results in reduced building weight and lower lateral system design loads. Based on research the underside of the waffle slab system is often left exposed serving as an aesthetic ceiling surface. As for the original structure, leaving the steel decking and framing exposed is not an option, and so a drop down ceiling is used. Additionally, the original structure requires additional fireproofing, whereas the new concrete structure does not. Finally, the waffle slab design does not interfere with critical MEP spaces like the steel framing does in the original structure. As one can see, the redesigned floor system offers a multitude of advantages over the existing structure without significantly increasing the buildings weight.

Results and Conclusions Continued:

Next, the aid of PCAcolumn and excel spreadsheets, exhaustive hand calculations were completed to design every single column in the structure. As shown on page 173, six different column sections are used in the final design (**A-F**). All columns are square tied columns ranging in size from 10x10 to 16x16. As mentioned in the column design section of the report, since 40 Gold Street is located in a non-seismic region, square tied columns were most appropriate. Tables F-69 through F-81 (pages 73:76) display the selected column section for every single column. In total, there are 11 A columns, 192 B columns, 40 C columns, 6 D Columns, 1 E column, 8 F columns, and 10 corbels. Designed for the modes of failure outlined in section 11.8 of ACI 318-08, a single design for all 10 corbels was determined. The final corbel design shown on page 85, proved to be an efficient design solution for transferring the load between the unaligned 1st and 2nd floor columns (see figure F-2). In fact, the existing design requires large section cantilever beams as large as W24x306. Also, large moment connections were used which is expensive and labor intensive. Concrete column sizes are comparable to the existing W-shape columns which are primarily W10X33, W10X45 and W12x96 (extend two stories). Perhaps the most significant advantage of the concrete columns over the existing steel columns is no need for connection design and labor. Also, no additional fire proofing is required for the concrete columns. The primary disadvantage is poor efficiency in terms of weight to strength of the columns. The concrete columns weigh 340.2 kips ($\frac{87CY}{.037037} * 150pcf/1000$) whereas the columns in the original structure have a total weight of 260.7 kips (calculated in technical report 1). Overall, reinforced concrete columns and corbels offer several advantages over the existing vertical structural elements; however, additional but acceptable amount of weight is added to the building.

Preliminary hand calculations were used to minimize torsion and establish the most ideal distribution of lateral stiffness throughout the building. Without question, the shear wall system created significant issues that are nonexistent in the design of steel moment and brace frames. With setbacks, changing floor plans, and architectural restraints, the placement of shear walls was extremely challenging. According to Severud Associates, the structural engineers of the existing structure, occasionally a brace frame interfered with a corridor or door way; however, the angle of the cross braces were simply modified (For example, braces no longer a perfect X shape).

By applying 38 load combinations consisting of the 7 ASCE load combinations and 4 Wind cases, using preliminary calculations, employing 3D ETABS analysis, and working through exhaustive hand calculations, an accurate and efficient design was produced. As is common in practice, the lateral system was iterative requiring relocation and resizing of the shear walls.

It is important to note, the gravity system was designed as though the shear walls did not resist gravity loads. In doing so, redundancy was introduced into the structure. To be more specific, if the lateral system were to fail under lateral loading; catastrophic failure (collapse) would likely be avoided because the columns would still have the capacity

Results and Conclusions Continued:

to resist the gravity loads. Since the existing structure includes many structural components responsible for both resistances of gravity and lateral loads, there is likely less redundancy.

As shown in Appendix I and tables F-128 through F-134, the shear wall design was controlled by flexure. The strength behavior of the shear walls can be classified as tall slender, flexural walls. The final lateral system consists of 7 shear walls, with 4 in the X direction and 3 in the Y direction. The shear walls are all 10" thick and range in length from 11'-0" to 19'-0". The existing lateral system is comprised of 6 braced frames and 7 moment frames. After comparing the two lateral systems, it was concluded that the shear wall system is less efficient and introduces significantly more weight to the building. Even with relatively small story shears, large amounts of flexural reinforcement are required because the shear walls are so tall and slender that flexural loads are as high as 17,277 ft-kips. For a building of just 4000 sf per floor, the shear wall design appears excessive but is in fact necessary.

With the structural system completely redesigned, the two structures were carefully examined and compared. As mentioned, drawing conclusions regarding the feasibility of the proposed concrete structure was a primary objective in this report. This feasibility study specifically pertains to the buildings weight. With poor soil conditions and existing buildings located at the site's perimeter, it was important to maintain a building weight that would not cause settlement or require redesign of the foundation system. Certainly, soil remediation and underpinning can provide a solution to the issue; however, this requires money and involvement of additional personnel. Therefore, the structural system was greatly governed by the importance of maintaining a low building weight. According to calculations, the new concrete structural system weighs approximately 7,000 kips (once length of overdesigned shear walls reduced). With an existing building of weight of 4,681 kips, the new concrete structural design is 1.5 times heavier. According to the geotechnical report, structural loads of 75 tons can be supported by a nominal 9.625" diameter micro pile. Considering both the micro pile capacity and soil bearing capacity, research and calculations suggests the building weight will not exceed theses capacities. As a result, the new structural design passed the feasibility test, and settlement issues can be dismissed.

In addition to the structural depth, several important conclusions were drawn regarding the construction management study. First of all, the existing steel structure involves shorter construction costs and durations. Only considering the structural system and the components that were redesigned, a detailed cost and schedule analysis were performed to determine which structure was more practical from a construction management point of view. Since the new design is a high rise concrete structure, minimal overlap potential exists between constructions of successive floors. This observation is apparent in figures F-165 – F-168. The cost analysis revealed the concrete structural system would cost $2,318,956, which is $579,921 more than the original structure ($1,739035). Note, only major structural components were considered. Therefore, the total costs are somewhat insignificant, but the cost difference is

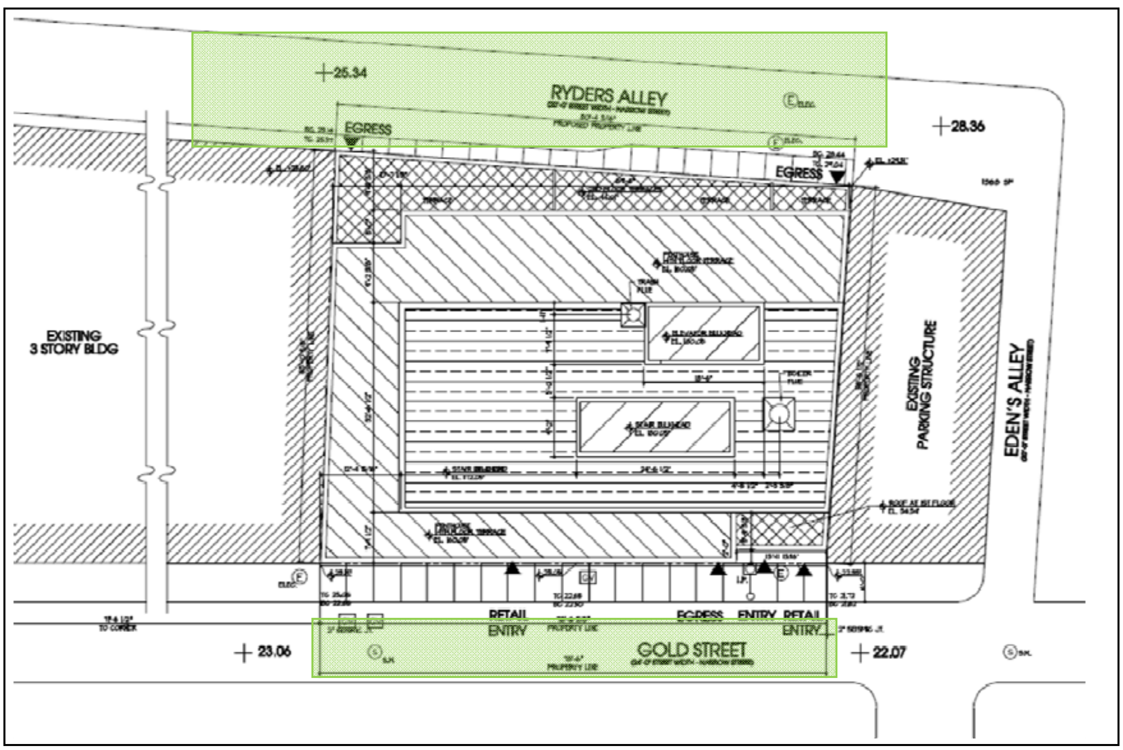
Results and Conclusions Continued:

extremely relevant. As for the construction durations, the new concrete structural system requires (95) 8 hour work days, which is 2.5 times longer than the (37) 8 hour work days required for the existing structure. As a consequence, general conditions costs are significantly larger due to the increase in construction duration. Another disadvantage of the new concrete structural design regards constructability. With several types of slabs including a waffle slab, coordination and skill of laborers is important. As discussed in the report, the site is extremely small as shown in figure F-172. In fact, the structure is designed to occupy the entire site. With formwork, storage bins, steel reinforcement, concrete trucks, concrete pumps, concrete finishing equipment, and many types of trades required for construction, congestion is a major issue. In fact, the only available space is on the two adjacent alleys shown in figure F-172. As a consequence, the grade level slab has to be placed and cured right away. The first floor slab will serve as critical storage space, and as a result requires design for supporting the construction equipment. Based upon this construction management study, the new concrete structural system is much less practical than the existing design. Constructability, duration, cost, and congestion are all major disadvantages that should not be overlooked.

SITE PLAN: Limited Space imposes major congestion issues

F- 172

The site plan is extremely congested. There is limited space for preassembly activities, storage and parking. The only available space is along Gold Street and Ryder Alley. As a result, the grade level slab must serve as space for construction equipment and activities.



Results and Conclusions Continued:

Finally, conclusions were drawn regarding the acoustic breadth study results. Based on a comparative study of the two structural systems, the concrete structural system provides better sound transmission loss performance. However, in this report special design details were produced to ensure interior spaces are exposed to acceptable noise levels. With high ambient noise levels, it was concluded that the exterior walls should be redesigned. Although not required for the higher level floors, the exterior walls should be designed as double walls with staggered metal studs, resilient stud attachments, and double layer of gypsum board on the interior side (see figure F-171). For sound isolation of residential spaces from the penthouse exercise room and mechanical equipment, a floated concrete slab with resilient suspended ceiling should be used as shown in figure F-170.

Final Remarks: As mentioned the main reason for electing to redesign the structure as a concrete structural system was to satisfy the client, who preferred a concrete structural system. Based on the research and data compiled in this report, it is evident that the concrete structural system offers a higher performing design solution for a New York City residential building. Obviously, other factors played a role in the selection of the structural system, since the existing system in a steel frame building with slab on metal decking. As explained, these factors included feasibility in terms of building weight and settlement potential, cost, construction duration, site congestion, and constructability.

In the end, designing a concrete structure with an acceptable weight (7,000 kips) was successfully achieved. With maintain a low weight being a primary goal, constructability of design was not dismissed but was not of high priority. Calculations and data suggest the concrete structural design is a viable option that yields a higher performing residential facility. However, the downside to the design is increased cost, increase construction duration, site congestion, and constructability issues. Advantages include improved acoustic performance, architecturally pleasing (leave exposed), improved floor-to-ceiling height, no additional fire proofing required, no framing interfering with MEP systems, and most importantly full client satisfaction (preferred concrete structure). In conclusion, the concrete structure is structurally feasible and offers several performance enhancements; however, the increased cost, increased construction duration, and constructability issues all suggest the design lacks practicality. The philosophy of design can be summarized as this: Design is only restricted by the necessity to produce a safe design, for there is no single solution to any design problem. In fact, an infinite number of solutions exist, all of which have disadvantages and advantages. With this said, what is really important is proper collaboration between all individuals involved in order to yield a final product that satisfies the client, and most importantly, functions safely.

Appendix A – Preliminary Slab Design Calculations

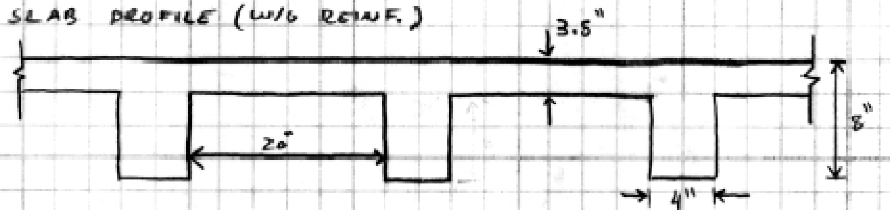
Column Layout #1	Hand Calculations	Jesse T. Cooper

DESIGN OF 5-SPAN FRAME ALONG COLUMN LINE C4

IMPORTANT INFORMATION:

L.W.C FOR SLAB (115 PSF)
ASSUME COLUMNS ARE 15" x 15"
WAFFLE SLAB DIMENSIONS: 3.5" THICK SLAB, 8" Deep Ribs, 20" Clear space between ribs, and 4" THICK RIBS @ BOTTOM

WAFFLE SLAB PROFILE (W/O REINF.)



Concrete compressive strength: $f'_c = 5.95$ ksi

$f_y = 60{,}000$

PROCESS: WILL USE DIRECT DESIGN METHOD AND/OR EFM TO DETERMINE SLAB THICKNESS AND STEEL REINFORCEMENT. EXCEL SPREADSHEETS WILL BE USED TO EXPEDITE THE DESIGN PROCESS OF OTHER FRAMES. SPSLAB WILL BE THE MAIN WAY OF DESIGN, BUT THESE CALCULATIONS PROVIDE VERIFICATION OF COMPUTER OUTPUT.

STEP ONE: SINCE IT IS A WAFFLE SLAB, THE S.W. CALCULATION MUST ACCOUNT FOR THE VOID AREAS.

BASED ON ABOVE PROFILE: AVERAGE DEPTH = $3.5''(20/24) + 8''(4/24)$

Avg. depth = 4.25"

$\therefore$ S.W. of slab = $115 \text{ PCF}(4.25/12)$

S.W. of slab = 40 PSF

TABLE 9.5(c): NO DROP PANELS, NO EDGE BEAMS, $f_y = 60{,}000$

l_n varies across frame, average $l_n \approx 15'$ CONSERVATIVE = 16

$$l_n/33 = \frac{(16 \times 12 - 15)}{33} = 5.36$$

FOR CALCULATIONS – will use $t = 5.36'' \rightarrow t = 6''$

COLUMN LAYOUT #1 | HAND CALCULATIONS | JESSE T. COOPER

STEP TWO: DETERMINE TOTAL STATIC MOMENT

DEAD LOAD: PARTITION LOAD + MEP + S.W SLAB
12 + 3 + 40 = 55 PSF

LIVE LOAD: RESIDENTIAL: 40 PSF

$W_u = 1.6W_L + 1.2W_D = 1.6(40) + 1.2(55) = 130$ PSF

$M_o = \frac{1}{8}(W_u)(l_2)(l_n^2) = \frac{1}{8}(130)(19.6)(16 - \frac{15}{12})^2(\frac{1}{1000})$

$M_o = 69.30$ l-k

STEP THREE: DETERMINE POSITIVE + NEGATIVE MOMENTS
"LONGITUDINAL DISTRIBUTION" OF TOTAL STATIC MOMENT.

ACI 13.6.3.2 : INTERIOR SPAN

$M_u^- = .65 M_o = .65(69.3) = -45.05$ l-k

$M_u^+ = .35 M_o = .35(69.3) = 24.3$ l-k

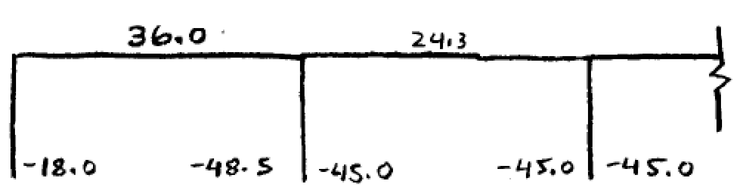
13.6.3.3: END SPAN

INT. $M_u^- = .70 M_o = .70(69.3) = -48.5$ l-k

$M_u^+ = .52 M_o = .52(69.3) = 36.04$ l-k

EXT. $M_u^- = .26 M_o = .26(69.3) = 18.02$ l-k

Summary of Longitudinal Moments - DIAGRAM



STEP FOUR: TRANSVERSE MOMENT DISTRIBUTION

ITEM / DESCRIPTION	FRAME – 2^{ND} FLOOR, ℄ 4
1. TOTAL TRANSVERSE WIDTH	19.6' x 12 = 235"
2. Column Strip width	117"
3. Middle Strip width	117"
4. TORSIONAL CONSTANT C $C = \Sigma \left(1 - .63\frac{x}{y}\right)\frac{x^3y}{3}$	NO BEAMS: C = 0
5. $I_s = \ell t^3/12$	$(235)\,6^3/12 =$ 4,230
6. $B_t = C/2I_s$	0
7. $\alpha_1 = I_b/I_s$	0
8. ASPECT RATIO ℓ_2/ℓ_1	19.6/15.2 = 1.28
9. $\alpha_1\,\ell_2/\ell_1$	0
10. M^+ % TO C.S.	TABLE 13.6.4.4: 60 %
11. M^- % TO C.S. INTERIOR SPAN	TABLE 13.6.4.1: 75 %
12. M^- % TO C.S. END SPAN	TABLE 13.6.4.2: 100 %

EXTERIOR M^- (END SPAN)

$-18 \rightarrow$ 100 % TO C.S. = −18 1-k
$-18 \rightarrow$ 0 % TO M.S.

END SPAN M^- INT

$-48.5 \rightarrow$ 75 % TO C.S. = −36.37
$-48.5 \rightarrow$ 25 % TO M.S. = −12.13 1-k

END SPAN M^+

$36 \rightarrow$ 60 % TO C.S. = 21.6 1-k
$36 \rightarrow$ 40 % TO M.S. = 14.4 1-k

INTERIOR M^+

$24.3 \rightarrow$ 60 % TO C.S. = +14.58
$24.3 \rightarrow$ 40 % TO M.S. = +9.72

INTERIOR M^-

$-45 \rightarrow$ 75 % TO C.S. = −33.75 1-k
$-45 \rightarrow$ 25 % TO M.S. = −11.25 1-k

STEP FIVE: REINFORCEMENT DESIGN OF C.S. SLAB

ITEM / DESCRIPTION	EXTERIOR SPAN			INT. SPAN	
	M^-_{EXT}	M^+	M^-_{INT}	M^-	M^+
1. M_u (ft-k)	−18	21.6	−36.375	−33.75	14.58
2. CS width, b	117"	117"	117"	117"	117"
3. EFFECTIVE DEPTH * CLEAR COVER: .75 * $\phi_{\#5}$ = .625 d_{avg} = 6 − .75 − 1.5(.625) d = 4.3125"	4.3125"	4.3125"	4.3125"	4.3125"	4.3125"
4. $M_u \cdot 12 / b$	1.85	2.22	3.73	3.46	1.49
5. $M_n = M_u / \phi$	−20	24	−40.42	−37.5	16.2
6. $R = \frac{M_n \times 12 \times 1000}{b \cdot d^2}$	110	132	222	207	89.3
7. $\rho_{REQUIRED}$ (SEE NEXT PAGE FOR DETAILED CALCULATIONS OF $\rho_{REQUIRED}$)	.00135	.00229	.003785	.00352	.002763
8. $A_{s,required} = \rho b d$	.933	1.155	1.9	1.77	1.39
9. $A_{s,min} = .002bt$ .002(117)(6") = 1.404	1.404	1.404	1.404	1.4041	1.404
10. $N = \frac{\text{larger of 8 and 9}}{\#5 \text{ bar area} = .31}$	4.5	4.5	6.1	5.7	4.5
11. $N_{min} = \frac{\text{STRIP WIDTH}}{2t}$	9.75	9.75	9.75	9.75	9.75
12. LARGER OF 10 and 11 FOR REINF. REQUIREMENTS	10 #5 TOP BARS	10 #5 BOT BARS	10 #5 TOP BARS	10 #5 TOP BARS	10 #5 BOT BARS

ρ REQUIRED CALCULATIONS

$$R = \rho f_y \left(1 - .59\rho \frac{f_y}{f'_c}\right) = M_u / \phi b d^2$$

M^- EXT

$$110 = \rho 60{,}000 \left(1 - .59\rho\left(\frac{60}{5.95}\right)\right)$$

$$110 = 60{,}000\rho - 356{,}974.79\rho^2$$

$$0 = -356{,}975\rho^2 + 60{,}000\rho - 110$$

$$\frac{-60{,}000 \pm \sqrt{(60{,}000)^2 - (4 \cdot -356{,}975 \cdot -110)}}{2(-356{,}975)}$$

$$\boxed{\rho = .00185}$$

END SPAN M^+

$$\frac{-60{,}000 \pm \sqrt{(60{,}000)^2 - (4 \cdot -356{,}975 \cdot -132)}}{2(-356{,}975)}$$

$$\boxed{\rho = .002229}$$

END SPAN (M^- INT)

$$\frac{-60{,}000 \pm \sqrt{(60{,}000)^2 - (4 \cdot -356{,}975 \cdot -222)}}{2(-356{,}975)}$$

$$\boxed{\rho = .003785}$$

INTERIOR SPAN : M^-

$$\frac{-60{,}000 \pm \sqrt{(60{,}000)^2 - (4 \cdot -356{,}975 \cdot -207)}}{2(-356{,}975)}$$

$$\boxed{\rho = .00352}$$

INTERIOR SPAN : M^+

$$\frac{-60{,}000 \pm \sqrt{(60{,}000)^2 - (4 \cdot -356{,}975 \cdot -89)}}{2(-356{,}975)}$$

$$\boxed{\rho = .002763}$$

Column Layout #1 | Hand Calculations | Jesse Cooper

STEP SIX: DESIGN OF REINF. - MIDDLE STRIP -

	End Span			Interior Span	
	M^-_{EXT}	M^+	M^-_{INT}	M^-	M^+
1.) M_u (ft-k)	0	14.4	-12.13	-11.25	9.72
2.) M.S. width = b	117"	117"	117"	117"	117"
3.) Effective depth $6 - .75 - 1.5(.625) = 4.3125$	4.3125"	4.3125"	4.3125"	4.3125"	4.3125"
4.) $M_u \cdot 12/b$	0	1.47	1.24	1.15	.997
5.) $M_n = M_u/\phi{=}.9$	0	16	13.5	12.5	10.8
6.) $R = M_n/bd^2$ X12 X1000	0	88.25	74.5	68.9	59.56
7.) ρ required (see next page for calcs)	0	.00148	.00125	.001156	.001
8.) $A_{s,REQD} = \rho bd = \rho(117)(4.3125)$	0	.7467	.631	.58327	.504
9.) $A_{s\,min} = .002bt$ $.002(117)(6)$	1.404	1.404	1.404	1.404	1.404
10.) Larger of 8 or 9 #5 bar area	4.53	4.53	4.53	4.53	4.53
11.) $N_{min} = \frac{\text{width of strip}}{2(t)}$ $\frac{117}{2(6)} = 9.75$	9.75 10 #5 top bars	9.75 10 #5 bottom bars	9.75 10 #5 top bars	9.75 10 #5 top bars	9.75 10 #5 bottom bars

PREQUIRED CALCULATIONS – MIDDLE STRIP

$f'_c = 5.95$

$f_y = 60{,}000$

M^-_{EXT}:

$$\frac{-60{,}000 \pm \sqrt{60{,}000^2 - (4 \cdot 356{,}975 \cdot 0)}}{2(-356{,}975)} \qquad \boxed{\rho_{REQ} = 0}$$

M^+ EXT

$$\frac{-60{,}000 \pm \sqrt{60{,}000^2 - (4 \cdot -356{,}975 \cdot -88.25)}}{(2 \cdot -356{,}975)} \qquad \boxed{\rho_{REQ} = .00148}$$

ENDSPAN/INT. M^-

$$\frac{-60{,}000 \pm \sqrt{60{,}000^2 - (4 \cdot -356{,}975 \cdot -74.5)}}{(2 \cdot -356{,}975)} \qquad \boxed{\rho_{REQ} = .00125}$$

INTERIOR SPAN M^-

$$\frac{-60000 \pm \sqrt{60{,}000^2 - (4 \cdot -356{,}975 \cdot -68.4)}}{(2 \cdot -356{,}975)}$$

$$\boxed{\rho_{REQ} = .001156}$$

INTERIOR SPAN M^+

$$\frac{-60{,}000 \pm \sqrt{60{,}000^2 - (4 \cdot -356{,}975 \cdot -59.56)}}{(2 \cdot -356{,}975)}$$

$$\boxed{\rho_{REQ} = .001}$$

COLUMN LAYOUT #1 | HAND CALCULATIONS | JESSE COOPER

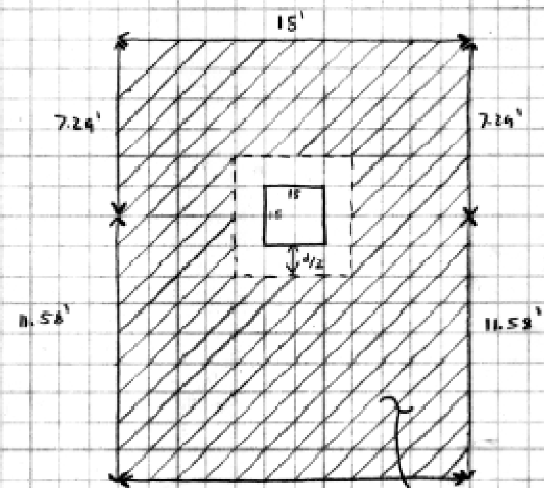
PUNCHING SHEAR CHECK

$d/2 = 4.3125/2 = 2.156$

CRITICAL SECTION, b_o = critical section perimeter

$b_o = (15 + 2.156) \times 4$

$b_o = 68.625''$



$V_c = 4\sqrt{f'_c}\, b_o d$ where $\lambda = 1$

$V_c = 4\sqrt{5{,}950}\,(68.625)(4.3125)(1/1000) = \boxed{91.3^k = V_c}$

$V_c = \left(2 + \frac{4}{\beta}\right)\lambda\sqrt{f'_c}\, b_o d \qquad \beta = 15/15 = 1$

$< 6\sqrt{f'_c}\, b_o d$ WON'T GOVERN

$V_c = \left(\frac{\alpha_s d}{b_o} + 2\right)\sqrt{f'_c}\, b_o d \qquad \alpha_s$ = INTERIOR column = 40

$\left(\frac{40(4.3125)}{68.6} + 2\right) \therefore 4.51\sqrt{f'_c}\, b_o d$ WON'T GOVERN

$V_u = \left[(11.58 + 7.29)(15) - (17.156/12)^2\right](w_u)$

$w_u = .130$ ksf

$V_u = (281\ ft^2)(.130\ ksf) = \boxed{37\ \text{KIPS} = V_u}$

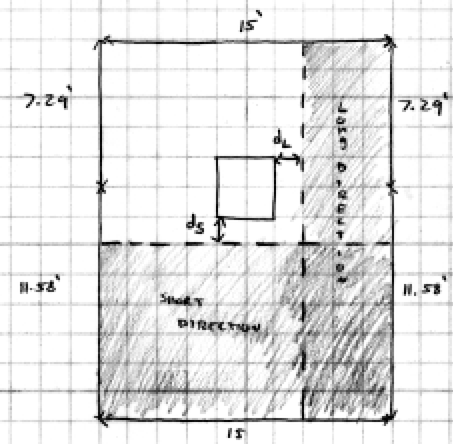
A 6" SLAB W/ NO WAFFLE SLAB VOIDS WILL WORK FOR PUNCH SHEAR

$\phi V_c = .9(91.3) = 82.17^k > 37^k$ OKAY

WIDE BEAM ACTION

$d_L = 4.9375$
$d_S = 4.3125$
$t = 6''$

$$V_u \le \phi V_n = \phi V_c = \phi 2\sqrt{f'_c}\, b_w \cdot d$$



LONG DIRECTION

$$\phi V_c = (.75)(2)\sqrt{5950}\,(11.58 + 7.29)(12)(4.9375)(1/1000)$$

$$\phi V_c = 129.4^k$$

$$V_u = \left[15/2 - \left(\tfrac{15}{12}/2\right) - \left(4.9375/12\right)\right](7.29 + 11.58)(.130)$$

$$V_u = (121.96\ SF)(.130\ KSF) = 15.9\ K$$

$$\boxed{\phi V_c = 129.4 > V_u = 16^k \quad \text{OKAY} \checkmark}$$

SHORT DIRECTION

$$V_u = 15 \times \left(11.58 - (15/12/2) - 4.3125/12\right)(.130\ KSF)$$

$$V_u = 159\ SF\ (.130\ KSF) = 21\ K$$

$$\boxed{\phi V_c = 129^k > V_u = 21^k \quad \text{OKAY} \checkmark}$$

Appendix B – Calculation of Slab Dead Loads

Base Waffle Slab:

Average Depth $= \frac{((2)*(11.5)+(2)*(11.5)+(20)*(3.5))}{24} = \boxed{4.8333333 \text{ inches}}$

Weight (PSF) $= (115 \text{ PCF})*(4.833333/12) = \boxed{46 \text{ PSF}}$

Modified Waffle Slab:

Average Depth $= \frac{((2)*(11.5)+(2)*(11.5)+(16)*(3.5))}{20} = \boxed{5.1 \text{ inches}}$

Weight (PSF) $= (115 \text{ PCF}) * (5.1/12) = \boxed{48.875 \text{ PSF}}$

8” / Waffle Slab Overlay:

Average Depth $= \frac{((2)*(11.5)+(2)*(11.5)+(20)*(8))}{24} = \boxed{8.583333 \text{ inches}}$

Weight (PSF) $= (115) * (8.583333 / 12) = \boxed{82.25 \text{ PSF}}$

Appendix C: Hand Calculations – Moment Transfer at Column Slab Connections

TRANSFER OF MOMENTS TO COLUMNS

FLOOR 2 – EXTERIOR COLUMN 1-D.4

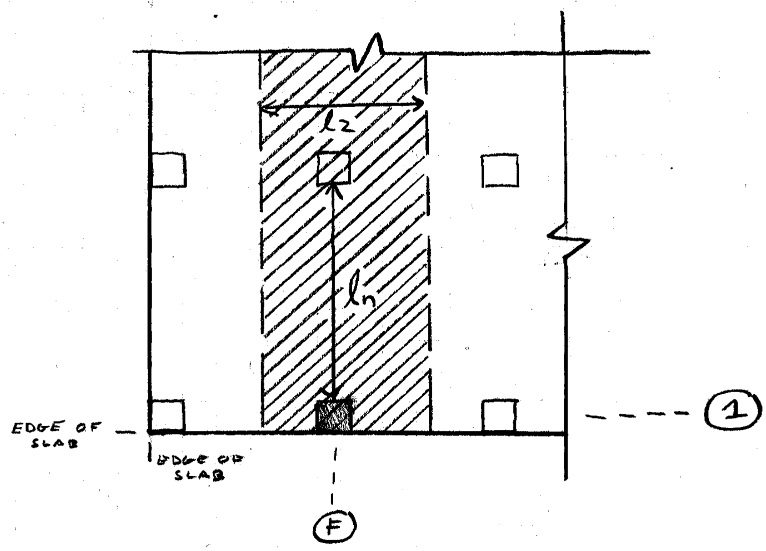
$W_{DU} = 1.2$ (SLAB + MISCELLANEOUS)
$= 1.2(76 + 30) = 127.2$ PSF
$= .1272$ KSF

$W_{LU} = 1.6$ (RESIDENTIAL) $= 64$ PSF
$= .064$ KSF

$L_2 = 14'-0''$

$L_n = 22' - 15''/12 = 20.75'$

DIAGRAM OF ADJACENT SLAB PANELS:



STATIC MOMENT: $M_o = \dfrac{w_u l_2 l_n^2}{8}$

$$M_o = \frac{(.1272 + .064)(14')(20.75')^2}{8}$$

$$\boxed{M_o = 144.0662 \text{ l-k}}$$

MOMENT TRANSFERRED TO COLUMNS: $.3M_o$

$.3M_o = .3(144.0662) = \boxed{43.219 \text{ l-k}}$

2.

DISTRIBUTE M_{column} BASED ON COLUMN FLEXURAL STIFFNESSES

$$K_{column} = \frac{4EI}{L}$$

$$K_{column\ above} = \frac{4 \cdot 57000\sqrt{5900} \cdot 15'' \times 15''^3 (1/12)}{10' \times 12''}$$

$$= 615691839$$

$$K_{column\ below} = \frac{4 \cdot 57000\sqrt{5900} \cdot 15'' \times 15''^3 (1/12)}{21' \times 12''}$$

$$= 293186590$$

$$M_{c\ above} = M_{column}\left(\frac{K_{c\ above}}{K_{c\ above} + K_{c\ below}}\right)$$

$$= 43.219\ \text{1-k}\left(\frac{615691839}{615691839 + 293186590}\right)$$

$$= .6774\,(43.219\ \text{1-k}) = \boxed{29.27\ \text{1-k}}$$

$$M_{c\ below} = M_{column}\,(1 - .6774)$$

$$= 43.219\,(.3326)$$

$$= \boxed{13.94\ \text{1-k}}$$

1.

TRANSFER OF MOMENTS TO COLUMNS:

FLOOR 12 - EXTERIOR COLUMN 1-C.3

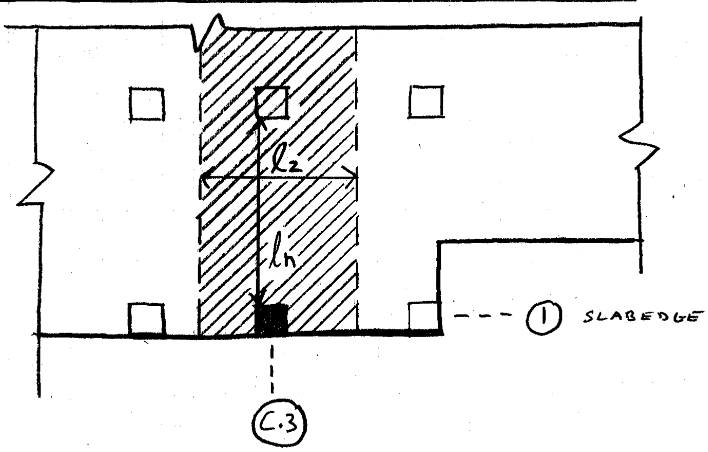
$W_{DU} = 1.2\ (\text{SLAB} + \text{MISCELLANEOUS})$
$= 1.2\ (46 + 30)$
$= .0912\ \text{KSF}$

$W_{LU} = 1.6\ (\text{RESIDENTIAL}) = 1.6\ (40\ \text{PSF})$
$= .064\ \text{KSF}$

$L_2 = 15'$

$L_n = 20.75'$

DIAGRAM OF ADJACENT SLAB PANELS:



STATIC MOMENT: $M_o = \dfrac{W_u\, l_2\, l_n^2}{8}$

$$M_o = \frac{(.0912 + .064)(15)(20.75)^2}{8}$$

$$\boxed{M_o = 125.29\ \text{l-k}}$$

MOMENT TRANSFERS TO COLUMNS: $.3M_o$

$$.3M_o = .3\,(125.79\ \text{l-k}) = \boxed{37.58\ \text{l-k}}$$

2

DISTRIBUTE M_{COLUMN} BASED ON COLUMN FLEXURAL STIFFNESS

$$K_{column} = \frac{4EI}{L}$$

$$K_{column\ above} = \frac{4 \cdot 57000\sqrt{5900}\ 15''(15''^3)(1/12)}{10 \cdot 12}$$

$$K_{C\ ABOVE} = K_{C\ BELOW} = 615691839$$

$$M_{C\ ABOVE} = M_{column}\left(\frac{K_{ABOVE}}{K_{ABOVE} + K_{BELOW}}\right)$$

$$= .3M_o\left(\frac{615691839}{2 \times 615691839}\right)$$

$$= 37.58\ \text{l-k}\ (1/2)$$

$$= \boxed{18.79\ \text{l-k}}$$

$$M_{C\ BELOW} = 37.58\ \text{l-k}\ (1/2)$$

$$= \boxed{18.79\ \text{l-k}}$$

(1.)

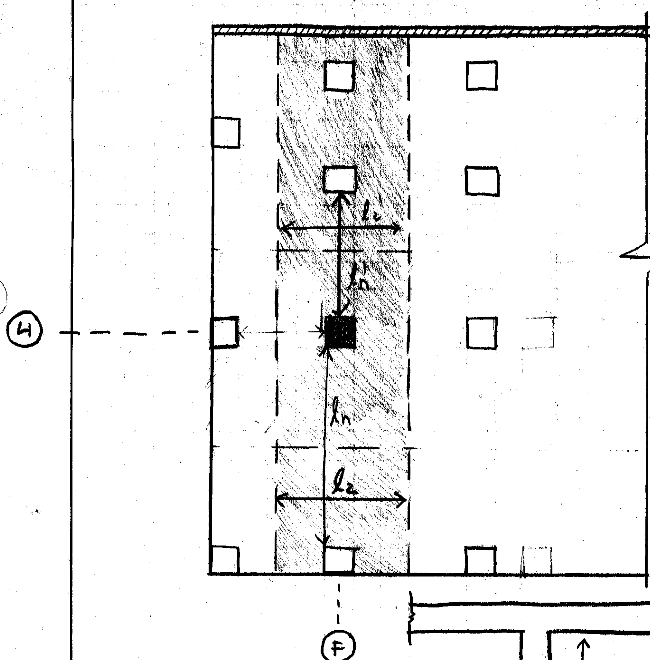
TRANSFER OF MOMENT TO COLUMNS

1^{ST} FLOOR – INTERIOR COLUMN 4 – F

FACTORED DEAD LOAD: q_{DU} = 1.2 (SLAB + MISCELLANEOUS)
= 1.2 (76 + 30) = 127.2 PSF
= .1272 KSF

FACTORED LIVE LOAD: q_{LU} = 1.6 (RETAIL)
= 1.6 (100 PSF) = 160 PSF
= .160 KSF

DIAGRAM OF ADJACENT SLAB PANELS

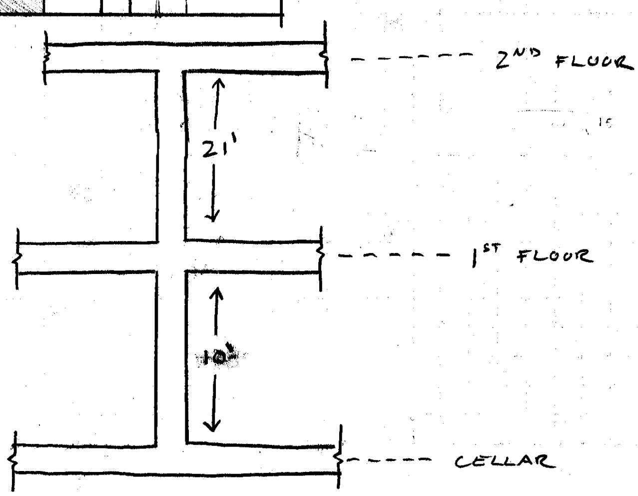


IMPORTANT DIMENSIONS

LONG SPAN
ℓ_2 = 18'
ℓ_n = 21.75'

SHORT SPAN
ℓ_2' = 18'
ℓ_n' = 15.75'

ELEVATION VIEW



2.

MOMENT AT JOINT:

$$M_{JOINT} = 0.65\left[\frac{(q_{DU} + 0.5 q_{LU}) l_2 l_n^2}{8} - \frac{q_{DU}' l_2' (l_n')^2}{8}\right]$$

$$= 0.65\left[\frac{(.1272 + 0.5(.160))(18)(21.75)^2}{8} - \frac{(.1272)(18)(15.75)^2}{8}\right]$$

$$M_{JOINT} = 0.65\,(149.54) = \boxed{97.205^{\,1\text{-}K}}$$

SOME MOMENT GOES TO COLUMN

MOST OF MOMENT GOES TO COLUMNS:

$$M_{COLUMN} = .07\left[(q_{DU} + .5 q_{LU})\, l_2 l_n^2 - q_{DU}' l_2' l_n'^2\right]$$

$$= .07\left[(.1272 + .5(.16))(18)(21.75)^2 - (.1272)(18)(15.75)^2\right]$$

$$= .07\,(1196.37) = \boxed{83.746^{\,1\text{-}K}}$$

DISTRIBUTE TO ABOVE / BELOW COLUMNS ACCORDING TO FLEXURAL STIFFNESS

$$K_{column} = \frac{4EI}{L}$$

ABOVE COLUMN: $K = \dfrac{4 \cdot 57000\sqrt{5{,}900} \cdot 15''(15'')^3/12}{21 \times 12}$

$K = 293186589$

BELOW COLUMN: $K = \dfrac{4 \cdot 57000\sqrt{5{,}900} \cdot 15''\,(15'')^3\,(1/12)}{10 \times 12}$

$K = 615691839$

1.

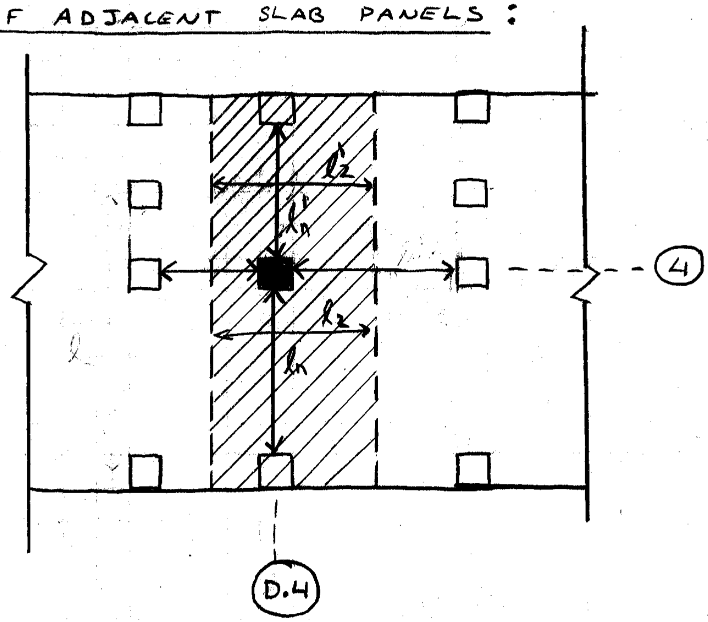
TRANSFER OF MOMENTS TO COLUMNS

FLOOR 11 - INTERIOR COLUMN 4 - D.4

FACTORED DEAD LOAD: $q_{DU} = 1.2\,(\text{SLAB} + \text{MISCELLANEOUS})$
$= 1.2\,(46 + 30) = 91.2 \text{ PSF} = .0912 \text{ KSF} = q'_{DU}$

FACTORED LIVE LOAD: $q_{LU} = 1.6\,(\text{RESIDENTIAL})$
$= 1.6\,(40) = 64 \text{ PSF} = .064 \text{ KSF}$

DIAGRAM OF ADJACENT SLAB PANELS:



LONG SPAN: l_2 = WIDTH OF SLAB PANEL = 18'
l_n = CLEAR SPAN OF SLAB PANEL = 33'-0" − 15"/12
= 31.75'

SHORT SPAN: $l'_2 = 18'$
$l'_n = 20.75'$

MOMENT TRANSFERRED TO JOINT:

$$M_{JOINT} = .65\left[\frac{(q_{DU} + 0.5q_{LU})\,l_2\,l_n^2}{8} - \frac{q'_{DU}\,l'_2\,(l'_n)^2}{8}\right]$$

2.

$$M_{JOINT} = .65\left[\frac{(.0912 + (0.5 \times .064))(18)(31.75)^2}{8} - \frac{(.0912)(18)(20.75)^2}{8}\right]$$

$$M_{JOINT} = .65(191.087) = \boxed{124.204 \text{ ft-k}}$$

SOME MOMENT GOES TO SLAB:

MOST OF MOMENT GOES TO COLUMNS:

$$M_{COLUMN} = .07\left[(q_{DU} + 0.5 q_{LU})\, l_2 l_n^2 - q'_{DU}\, l_2' (l_n')^2\right]$$

$$M_{COLUMN} = .07\left[(.0912 + 0.5(.064))(18)(31.75)^2 - (.0912)(18)(20.75)^2\right]$$

$$M_{COLUMN} = .07(1528.67) = \boxed{107.007 \text{ ft-k}}$$

M_{COLUMN} IS DISTRIBUTED ABOVE AND BELOW BASED ON FLEXURAL STIFFNESS OF COLUMNS:

$$K_{column\ Above} = \frac{4EI}{L} = \frac{4 \times 57{,}000\sqrt{f'_c} \times (15'' \cdot 15''^3/12'')}{(10' \times 12'')}$$

$f'_c = 5{,}900$

$K_{column\ Above} = 615691839$ "

$K_{column\ Below} = 615691839$ $''/in$

$$M_{column\ Above} = M_{column} \times \left(\frac{K_{c\ Above}}{K_{c\ Above} + K_{c\ Below}}\right)$$

$$= 107.007\left(\frac{1}{2}\right)$$

$$\boxed{\begin{aligned} M_{COL.\ ABOVE} &= 53.503 \text{ ft-k} \\ M_{COL.\ BELOW} &= 53.503 \text{ ft-k} \end{aligned}}$$

Appendix D: Hand Calculations – Tied Square Column Design

Columns 1.

PRELIMINARY COLUMN DESIGN

ARCHITECTURAL CONSTRAINTS – COLUMNS ARE ALL LOCATED IN AREAS WHERE THEY WILL NOT INTERFERE WITH THE RESIDENTIAL SPACES/WALKWAYS. HOWEVER MANY COLUMNS ARE LOCATED IN PLACES IN WHICH THE COLUMN DIMENSIONS NEED TO BE MINIMIZED. SQUARE TIED COLUMNS OF Approximately 15" X15" ARE PREFERRED. However columns shouldn't exceed 22" X22" TO Avoid interfering with critical residential spaces / walkways! (To avoid blocking windows / doorways etc.).
In summary, 22 x 22 square tied column or smaller are most compatible with the existing architectural layout!

MINIMIZING WEIGHT OF BUILDING – WHEN CHOOSING COLUMN SECTIONS DURING DESIGN, MINIMIZING WEIGHT OF Building is a key design goal.

PRELIMINARY CALCULATIONS – 1ST CHOOSE A combined axial and flexural loading that is most representative of a typical column loading! Example, choose a loading observed in many columns (NOT AN EXTREME OR UNIQUE COLUMN LOADING).

FLOOR 10 - COLUMN 6-F

$P_u = 151.6^k \quad M_u = 49.3^k$

$e = M_u / P_u = 49.3 \times 12 / 151.6 = 3.902$

Assume $d' = 2.5''$

STEP ONE: DETERMINE COLUMN DIMENSIONS

h	γ	e/h
10	.5	.3902
12	.583	.3252
14	.643	.2787

$$\gamma = \frac{h - 2d'}{h}$$

ANALYZE PURE COMPRESSION CHART, "SIMPLIFIED COLUMN DESIGN" FIGURE TO GET AN IDEA OF COLUMN DIMENSIONS AND REINF. RATIO.

NOT CONSIDERING MOMENT - 10 X 10 or 12 X 12 WILL SUFFICE

TRY A 10 X 10, NOW CHECK FOR AXIAL + MOMENT

STEP TWO: DESIGN AIDS, ASSUME 2 FACES

$$\frac{\phi M_n}{bh^2} = \frac{49.3}{10^3} = .05 \qquad \frac{\phi P_n}{bh} = \frac{151.6}{10^2} = 1.516$$

CHART: $\rho_g = .75\%$ (E-4-60-.6)

DESIGN AID: E-4-60-.75

$$\frac{\phi M_n}{bh^2} = .05 \qquad \frac{\phi P_n}{bh} = 1.516$$

CHART: $\rho_g = .6\%$

EXTRAPOLATE FROM CHARTS

γ	ρ_g
.5	?
.6	.75 %
.75	.6 %

$$\frac{.75 - .6}{.6 - .75} = \frac{\rho_g - .75}{.5 - .6}$$

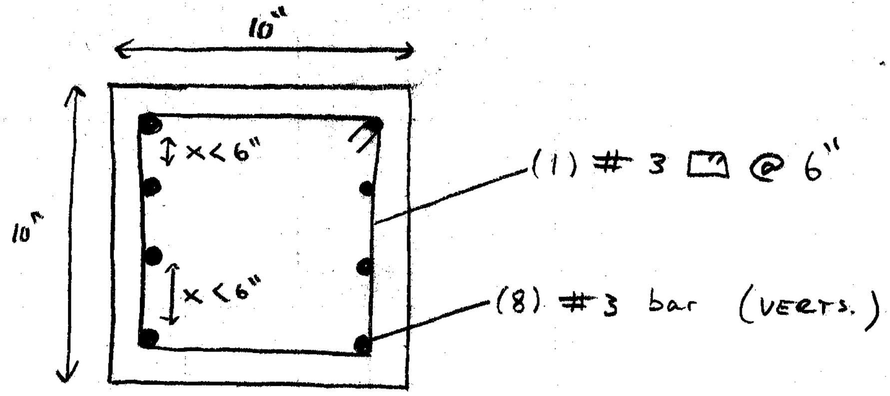
$\rho_g = .85\%$

$A_{s\ \text{REQUIRED}} = \rho_g bh = .0085(10 \times 10) = .85\ in^2$

TRY (8) #3 :

$A_{s\ \text{PROVIDED}} = 8 \times .11 = .88\ in^2 > A_{s\ \text{REQUIRED}} = .85\ in^2$

OKAY ✓



CHECK bmin :

$b_{min} = 2(\text{cover}) + 2(\phi\ \text{TIE}) + 4(\#3\ \phi) + 3(1.5)(\#3\ \phi)$

$b_{min} = 2(1.5") + 2(3/8) + 4(3/8) + 3(1.5)(3/8)$

$b_{min} = 6.9375" < b_{actual} = 10"$ OKAY ✓

DETERMINE SPACING OF TIES

$$S \le \min \begin{cases} 16 \times \phi \text{ of Longitudinal bar} = 16(3/8) = 6" \\ 48 \times \text{TIE } \phi = 48(3/8) = 18" \\ \text{LEAST COLUMN DIMENSION} = 10" \end{cases}$$

THEREFORE, SPACE TIES AT 6" VERTICAL SPACING

FLOOR 12 – COLUMN 4 - D.4

$P_u = 118.05^k \quad M_u = 53.5^{k-1}$

$e = M_u / P_u = \dfrac{53.5 \times 12}{118.05} = 5.41$

Assume $d' = 2.5"$

STEP ONE: DETERMINE COLUMN DIMENSIONS

h	γ	e/h
10	.5	.541
12	.583	.451
14	.643	.386

$\gamma = \dfrac{h - 2d'}{h}$

ANALYZE PURE COMPRESSION CHART "SIMPLIFIED COLUMN DESIGN" FIGURE TO GET AN IDEA OF COLUMN DIMENSIONS b, h and REINFORCEMENT RATIO.

W/O MOMENT, 10 x 10 column requires VERY LITTLE REINFORCEMENT.

TRY A 10 x 10, SHOULD WORK WITH MOMENT AS WELL!

STEP TWO: DESIGN AIDS, ASSUME 2 FACES

DESIGN AID: E-4-60-.6

$\dfrac{\phi M_n}{bh^2} = \dfrac{53.5^{k\text{-}k}}{10^3} = .0535 \qquad \dfrac{\phi P_n}{bh} = \dfrac{118.05^k}{10^2} = 1.181$

CHART: ρ_g required $= .5\% = .005$

DESIGN AID: E-4-60-.75

$\dfrac{\phi M_n}{bh^2} = .0535 \qquad \dfrac{\phi P_n}{bh} = 1.181$

CHART: ρ required $= .28\% = .0028$

EXTRAPOLATE FROM CHARTS

γ	ρ_g
.5	$\boxed{?}$
.6	.005
.75	.0028

$\dfrac{.75 - .6}{} = \dfrac{.6 - .5}{.005 - \rho_g}$

$\rho_g = .00647$

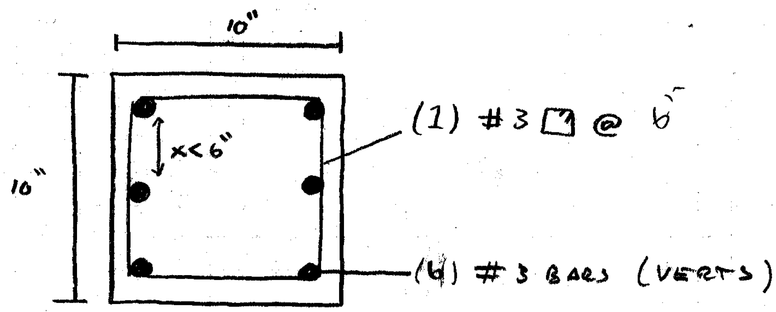
FLOOR 12
4-D.4

(2.)

$A_s = \rho_g \times bh = .00647 \times 100 = .647 \text{ in}^2$

TRY (6) #3 = $6 \times .11 \text{ in}^2$ $A_s = .66 \text{ in}^2$

A_s provided = .66 > A_s REQUIRED = .647 in^2



CHECK bmin :

bmin = 2(COVER) + 2(TIE Ø) + 2(#3 Ø) + 1.5(#3 Ø)

bmin = 2(1.5") + 2(3/8") + 2(3/8") + 1.5(3/8")

bmin = 3" + 6/8" + 6/8" + .5625"

bmin = 5.0625" < b = 10" OKAY ✓

DETERMINE REQUIRED VERTICAL SPACING OF TIES

$$S \le \min \begin{cases} 16 \times \text{Longitudinal bar } \varnothing = 16(3/8) = 6" \\ 48 \times \text{TIE } \varnothing = 48(3/8) = 18" \\ \text{LEAST COLUMN DIMENSION} = 10" \end{cases}$$

SPACE TIES AT 6" VERTICALLY

NO INTERMEDIATE TIES REQUIRED, ALL LONGITUDINAL BARS ARE AT CORNERS. OR w/in 6".

COLUMN DESIGN
1 - F

1

FLOOR 3 - COLUMN - 1-F

$P_u = 266.3^k$ $M_u = 29.78^{1-k}$

$e = M_u/P_u = \dfrac{29.78 \times 12}{266.3} = 1.34119''$

$d' = 2.5''$

STEP ONE DETERMINE WHAT column dimensions to use

h	γ	e/h
12	.583	.1118
14	.643	.0958
16	.688	.08387

$\gamma = \dfrac{h - 2d'}{h}$

ANALYZE PURE COMPRESSION CHART "SIMPLIFIED COLUMN DESIGN" FIGURE TO GET AN IDEA OF COLUMN DIMENSIONS b AND h AS WELL AN APPROXIMATE TARGET REINF. RATIO ρ_g.

W/O MOMENT, USE 10 x 10 w/ 2.6 % reinforcement

THEREFORE, TRY 12 x 12 COLUMN

STEP TWO: USE DESIGN AIDS TO DETERMINE b, h, and ρ
ASSUME REINF. ON 2 FACES

DESIGN AID: E-4-60-0.6

$\dfrac{\phi M_n}{bh^2} = \dfrac{29.78}{12^3} = .0172$ $\dfrac{\phi P_n}{bh} = \dfrac{266.3}{12 \times 12} = 1.85$

$\rho_g = .98\%$

DESIGN AID: E-4-60-.75

$\dfrac{\phi M_n}{bh^2} = .0172$ $\dfrac{\phi P_n}{bh} = 1.85$ $\rho_g = .96\%$

EXTRAPOLATE

γ	ρ_g
.583	$\boxed{?}$
.6	.98
.75	.96

$\dfrac{.98 - .96}{.6 - .75} = \dfrac{\rho_g - .98}{.583 - .6} =$

$\rho_g = .982\%$

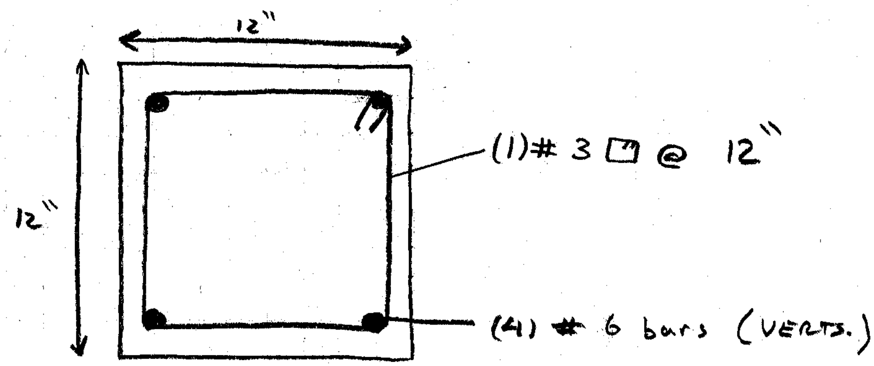
FL3
1-F.

2

$A_s = \rho_g \cdot bh = .982\% (12 \times 12) = 1.42 \text{ in}^2$

TRY (4) # 6 bars = 1.76 in^2

$1.76 \text{ in}^2 > 1.42 \text{ in}^2$ OKAY ✓



CHECK bmin:

$b_{min} = 2(\text{cover}) + 2(\text{TIE } \varnothing) + 2(\#6\ \varnothing) + 1.5(\#6\ \varnothing)$

$b_{min} = 2(1.5) + 2(3/8) + 2(6/8) + 1.5(6/8)$

$b_{min} = 3 + .75 + 1.5 + 1.125$

$b_{min} = 6.375 < b = 12"$ OKAY ✓

DETERMINE VERTICAL SPACING OF TIES

$$S \leq \min \begin{cases} 16 \text{ longitudinal } d_b = 16(6/8) = 12" \\ 48 \times \text{TIE } \varnothing = 48(3/8) = 18" \\ \text{LEAST COLUMN DIMENSION} = 12" \end{cases}$$

USE TIES @ 12" VERTICAL SPACING ✓

FLOOR 1 – COLUMN 4-F

$P_u = 492.1^K \qquad M_u = 56.7^{1-K}$

$e = M_u/P_u = \dfrac{56.7 \times 12}{492.1} = 1.38''$

Assume $d' = 2.5''$

STEP ONE: ANALYZE PURE COMPRESSION CHART "SIMPLIFIED COLUMN DESIGN" FIGURE TO GET AN IDEA OF COLUMN DIMENSIONS.

NOT CONSIDERING MOMENT, $14'' \times 14''$ COLUMN W/ $\rho_g = 2.5\%$ IS RECOMMENDED.

TRY 14×14, SINCE MOMENT IS PRETTY LOW!

STEP TWO: USE DESIGN AIDS TO DETERMINE b, h, AND ρ. ASSUME REINFORCEMENT ON 4 FACES

$\gamma = \dfrac{h - 2d'}{h} = \dfrac{14 - 2(2.5)}{14} = .6428$

DESIGN AID: R-4-60-0.6

$\dfrac{\phi M_n}{bh^2} = \dfrac{56.7}{14(14^2)} = .021 \qquad \dfrac{\phi P_n}{bh} = \dfrac{492.1}{14 \times 14} = 2.5107$

$\rho_g = 2.7\%$

DESIGN AID: R-4-60-.75

$\dfrac{\phi M_n}{bh^2} = .021 \qquad \dfrac{\phi P_n}{bh} = 2.5107$

ρ_g required $= 2.6\%$

INTERPOLATE BETWEEN CHARTS

γ	ρ_g
.60	2.7%
.6428	$\boxed{?}$
.75	2.6%

$2.7 - \left(\dfrac{.6428 - .60}{.75 - .6}\right)\left(2.7 - 2.6\right)$

$2.671 = \rho_g$

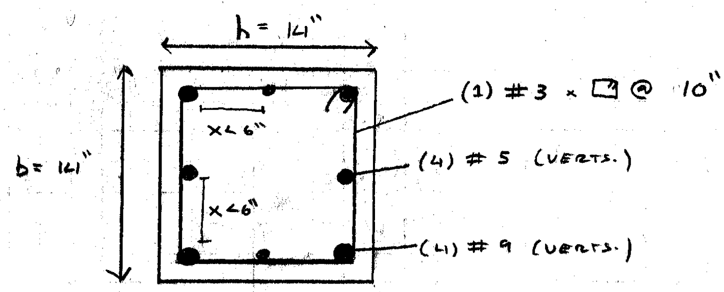
Floor 1 Column 4-F

(2)

$A_s = \rho_g \cdot bh = (.02671)(14 \times 14) = 5.236 \text{ in}^2$

TRY (4) #9 : $A_s = 4 \times 1.00 \text{ in}^2 = 4 \text{ in}^2$
AND
(4) #5 : $A_s = 4 \times .31 \text{ in}^2 = 1.24 \text{ in}^2$

$5.24 \text{ in}^2 > A_{s\,required} = 5.236 \text{ in}^2$ OKAY ✓



CHECK bmin:

$b_{min} = 2(\text{cover}) + 2(\text{tie } \phi) + 1(\#5\ \phi) + 2(\#9\ \phi) + 3(1.5)(\#9\ \phi)$

$b_{min} = 2(1.5") + 2(.375) + 1(5/8) + 2(9/8) + 2(1.5)(9/8)$

$b_{min} = 3 + .75 + .625 + 2.25 + 3.375$

$b_{min} = 10" < 14"$ OKAY ✓

NO INTERMEDIATE TIES REQUIRED - ALL VERTICAL REINFORCEMENT IS W/IN 6" OF LATERALLY RESTRAINED Longitudinal bars.

VERTICAL SPACING OF TIES

$$s \le \min \begin{cases} 16 d_b = 16(5/8) = 10" \\ 48 d_{tie} = 48(3/8) = 18" \\ \text{LEAST COLUMN DIMENSION} = 14" \end{cases}$$

SPACE TIES VERTICALLY @ 10" ✓

FLOOR 1: Column 4-D.4   Assume "short column"

$P_u = 639^k$ $M_u = 55.62^{1\text{-}k}$

$e = M_u/P_u = \frac{55.62 \times 12}{639} = 1.04''$

Assume $d' = 2.5''$

STEP ONE: ANALYZE PURE COMPRESSION "SIMPLIFIED COLUMN DESIGN" FIGURE. TO GET AN IDEA OF COLUMN DIMENSIONS b and h AS well as AN APPROXIMATE TARGET REINF. RATIO ρ_g.

FOR 639^k LOAD, Assume $f'_c = 4000$ $f_y = 60{,}000$

$16'' \times 16''$ column: $\rho_g = 2.4\%$

STEP TWO: USE DESIGN AIDS TO DETERMINE b, h and ρ

Assume REINF. ON 4 FACES

$\gamma = \frac{h - 2d'}{h} = \frac{16 - 2(2.5)}{16} = .6875$

DESIGN AID: R-4-60-0.6

$\frac{\phi P_n}{bh} = \frac{639}{16 \times 16} = 2.496$

$\frac{\phi M_n}{bh} = \frac{55.62}{16 \times 16^2} = .013$

REQUIRED $\rho_g = 2.7\%$

DESIGN AID: R-4-60-0.75

$\frac{\phi P_n}{bh} = 2.496$

$\frac{\phi M_n}{bh^2} = .013$

REQUIRED $\rho_g = 2.5\%$

INTERPOLATION:

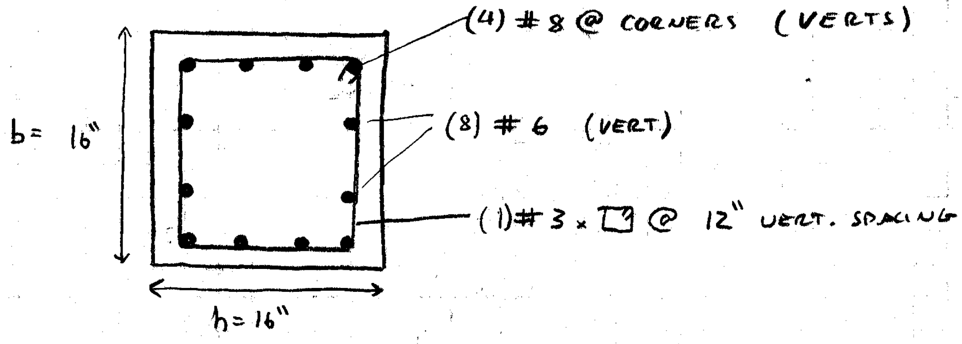
γ	ρ_g
.6	2.7
.6875	$\boxed{?}$
.75	2.5

$2.7 - \left(\frac{.6875 - .6}{.75 - .6}\right)(.2) = 2.7 - .116667$

$\boxed{\rho_g = 2.583\%}$

$A_s = \rho_g \times bh$

$A_s = .02583 \times (16 \times 16) = 6.613 \text{ in}^2$



USE (4) #8 @ CORNERS : $4(.79) = 3.16$
USE (8) #6 : $8(.44) = 3.52$
$\Big\}$ 6.68

$A_{s\,provided} = 6.68 \text{ in}^2 > A_{s\,required} = 6.613 \text{ in}^2$ OKAY ✓

CHECK b_{min}:

$b_{min} = 2\,\text{cover} + 2(\text{TIE}\,\varnothing) + 2(\#8\,\varnothing) + 2(\#6\,\varnothing) + 3(1.5)(\#8\,\varnothing)$

$b_{min} = 2(1.5) + 2(3/8) + 2(1) + 2(6/8) + 3(1.5)(1)$

$b_{min} = 3 + .75 + 2 + 1.5 + 4.5$

$b_{min} = 11.75 < b = 16"$ OKAY

NO INTERMEDIATE TIES REQUIRED! ALL VERT. BARS LESS THAN 6" FROM A LATERALLY SUPPORTED LONG. BAR.

#3 ⊏ @ 12" VERTICAL SPACING

$$\text{minimum} \le \begin{cases} 16 \times \text{longitudinal } d_b = 16(6/8) = 12" \\ 48 \times \text{tie } \varnothing = 48(3/8) = 18" \\ \text{LEAST COLUMN DIMENSION} = 16" \end{cases}$$

FLOOR 3 Column 4-F – DESIGN TIED SQUARE COLUMN

$P_u = 391.4^k$ $M_u = 38$ l-k

$e = M_u/P_u = 38 \times 12 / 391 = 1.17''$

Assume $d' = 2.5''$

STEP ONE: ANALYZE PURE COMPRESSION "SIMPLIFIED COLUMN DESIGN" FIGURE TO GET AN IDEA OF column dimensions b and h as well as an approximate target reinforcement ratio ρ_g.

FOR 391.4^k LOAD, 12" × 12" COLUMN REQUIRES
$\rho_g = 2.5\% = .0025$.

STEP TWO: USE DESIGN AIDS TO DETERMINE b, h and ρ

ASSUME REINFORCEMENT BARS ON 2 FACES

$$\gamma = \frac{h - 2d'}{h} = \frac{12 - 2(2.5)}{12} = .58333$$

DESIGN AID: E-4-60.6 ($\gamma = .60$)

$$\frac{\phi P_n}{bh} = \frac{391.4}{144} = 2.736 \qquad \frac{\phi M_n}{bh^2} = \frac{38}{12(12^2)} = .022$$

REQUIRES: $\rho_g = 3.25\%$

DESIGN AID: E-4-60-.75 ($\gamma = .75$)

$$\frac{\phi P_n}{bh} = 2.736 \qquad \frac{\phi M_n}{bh^2} = .022$$

$\rho_g = 3.1\%$ (LESS STEEL, BECAUSE LARGER LEVER ARM)

SINCE ACTUAL $\gamma = .58333$, MUST EXTRAPOLATE DESIGN AID VALUES FOR ρ_g.

γ	ρ
.75	3.1 %
.60	3.25 %
.5833	$\boxed{?}$

$$\frac{.75 - .60}{3.1 - 3.25} = \frac{.60 - .5833}{3.25 - \rho_g}$$

$$-1 = \frac{.016667}{3.25 - \rho_g}$$

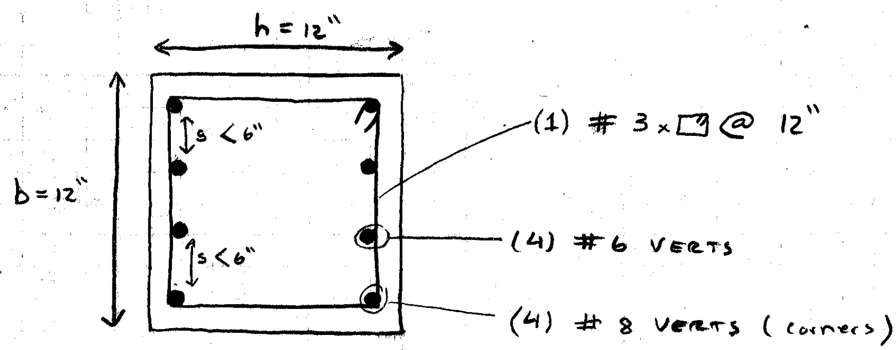
$$\rho_g - 3.25 = .016667$$

$$\boxed{\rho_g = 3.27\%}$$

$A_s = .0327(12 \times 12) = 4.7088 \text{ in}^2$

$4 \# 6 = 4 \times (.44) = 1.76 \text{ in}^2$
and
$4 \# 8 = 4 \times (.79) = 3.16 \text{ in}^2$

$\left.\right\}\ A_s = 4.92 > 4.7$ OKAY



$b_{min} = 2(\text{cover}) + 2(\text{TIE } \phi) + 2(\#8\ \phi) + 2(\#6\ \phi) + 3(1.5)(\#8\ \phi)$

$b_{min} = 2(1.5) + 2(.375) + 2(1) + 2(.75) + 3(1.5)(1)$

$b_{min} = 11.75" < 12"$ OKAY ✓

#3 TIES USED TO RESTRAIN VERTICAL BARS FROM BUCKLING OUT THROUGH COLUMN SURFACE

ACI CODE SECTION: 7.10.5

min. size = #3

TO ENSURE TIES CAN DEVELOP ADEQUATE FORCES TO RESTRAIN BUCKLING OF LONG. BARS!

VERTICAL SPACING OF TIES $\leq$ 16 longitudinal d_b

$\leq$ 48 tie diameters

$\leq$ least dimension of column

NOT SEISMIC REGION, CLOSER SPACING NOT NECESSARY!

NO INTERMEDIATE TIES REQUIRED, ALL Longitudinal BARS ARE w/in LESS THAN 6" FROM A LATERALLY SUPPORTED Longitudinal bar

$16(6/8) = 12"$

$48(3/8) = 18"$

Column dim = 12"

Pure Compressive Chart: This design aid was only used for preliminary hand calculation and estimation of column dimensions.

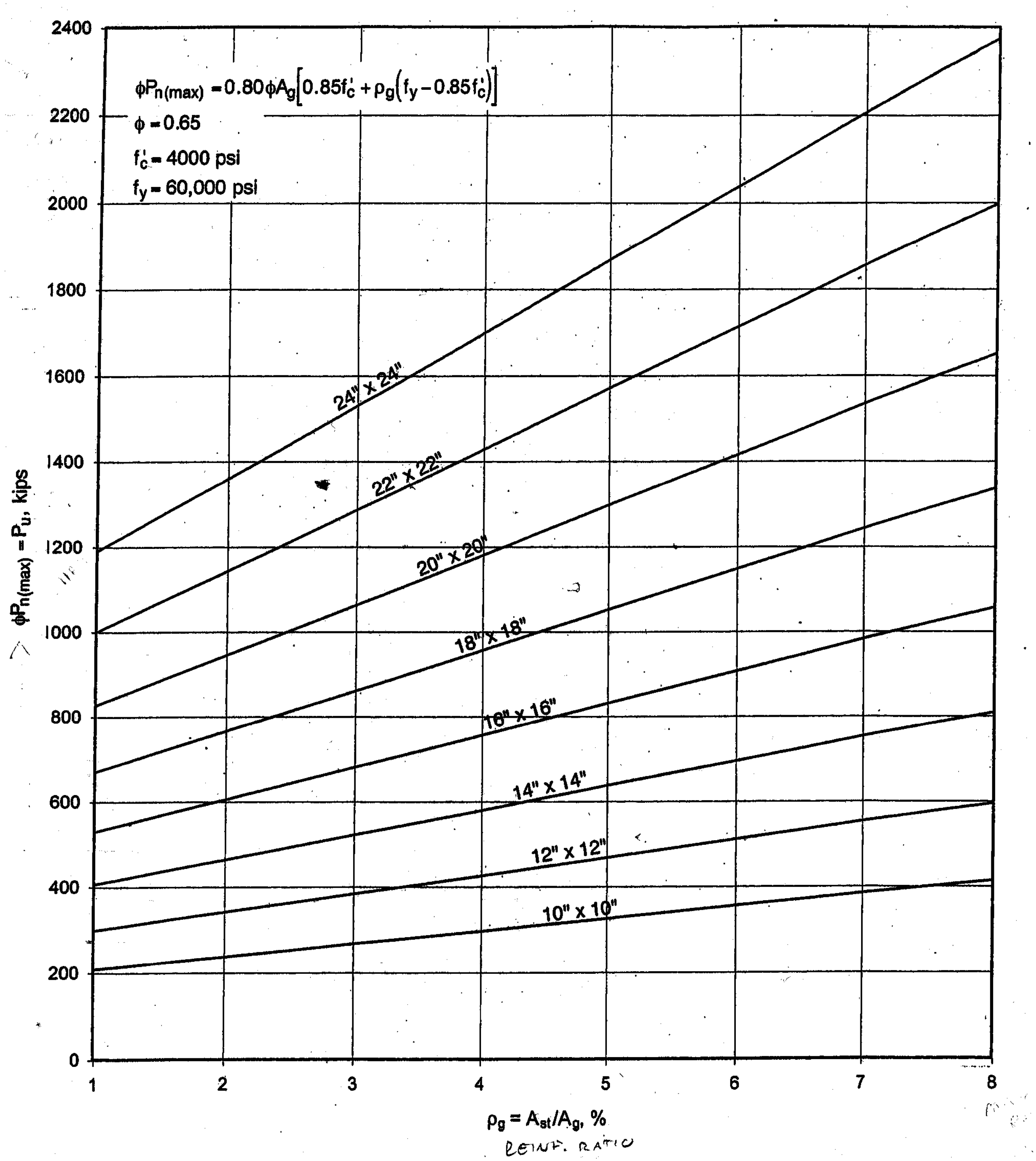


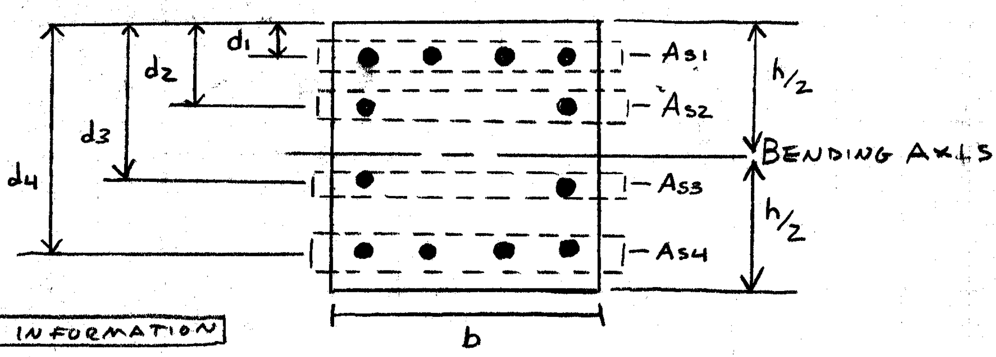
Figure 5-1 Design Chart for Nonslender, Square Tied Columns

Appendix E: Hand Calculations – Column Interaction Diagrams

INT. DIAGRAM

ONE

DETERMINE INTERACTION DIAGRAM FOR COLUMN SECTION



IMPORTANT INFORMATION

$A_{s1} = 2(\#8) + 2(\#6) = 2(.79) + 2(.44) = 2.46 \text{ in}^2$

$A_{s2} = 2(\#6) = 2(.44) = .88 \text{ in}^2$

$A_{s3} = 2(\#6) = 2(.44) = .88 \text{ in}^2$

$A_{s4} = 2(\#8) + 2(\#6) = 2(.79) + 2(.44) = 2.46 \text{ in}^2$

$b = 16''$ $d_1 = 2.5''$ $f'_c = 4$ ksi $\sum A_{si} = 6.68 \text{ in}^2$

$h = 16''$ & $d_2 = 6.16667''$ $f_y = 60$ ksi

$d_3 = 9.8333''$

$d_4 = 13.5''$

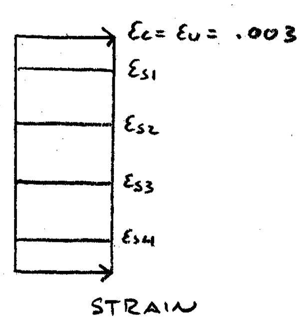
STEP ONE: PURE AXIAL STRENGTH

$c = \infty \therefore \varepsilon_s = \varepsilon_c = 0.003$

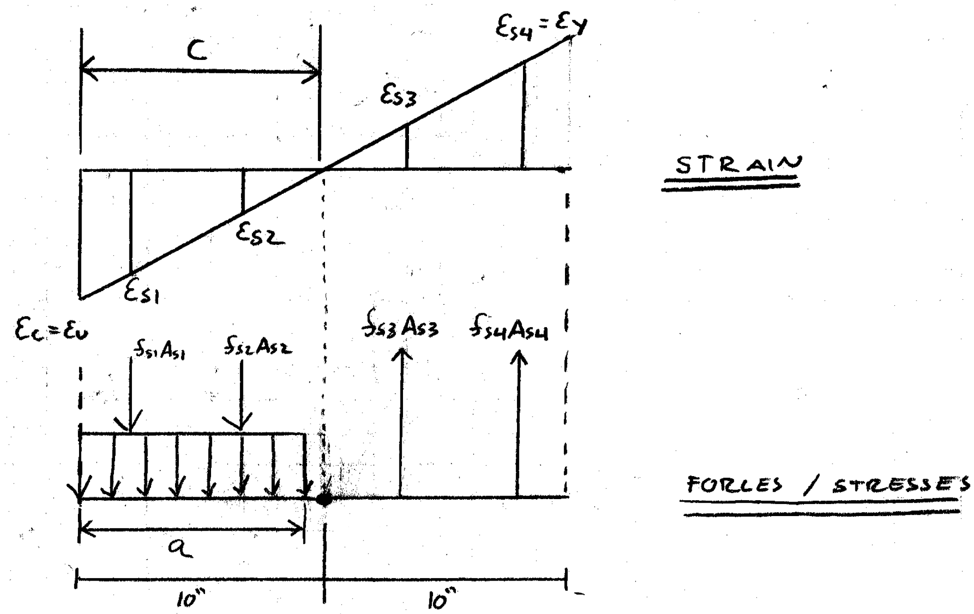
$P_o = 0.85 f'_c (bh - \sum A_{si}) + \sum A_{si} f_{si}$

$P_o = 0.85(4)(16 \cdot 16 - 6.68) + (6.68 \times 60)$

$\boxed{P_o = 1248^k}$



STEP TWO: BALANCED STRAIN CONDITION



$\varepsilon_y = f_y/E = 60\text{ ksi}/29000\text{ ksi} = .002068$

$$c = \frac{.003}{.003 + .002068}\, d_{imax} = \frac{.003}{.005068}(13.5") = 7.989"$$

$$\varepsilon_{s1} = \frac{.003}{c}(c - d_1) = \frac{.003}{7.989}(7.989 - 2.5) = .002061 < \varepsilon_y = .002068$$

$\therefore\ f_{s1} = .002061(29000) = \underline{59.7\text{ ksi}}$ COMPRESSION

$$\varepsilon_{s2} = \frac{.003}{c}(c - d_2) = \frac{.003}{7.989}(7.989 - 6.16) = .000686 < \varepsilon_y = .002068$$

$\therefore\ f_{s2} = .000686(29000) = \underline{19.92\text{ ksi}}$ COMPRESSION

$$\varepsilon_{s3} = \frac{.003}{c}(c - d_3) = \frac{.003}{7.989}(7.989 - 9.8333) = |-.000693| < \varepsilon_y$$

$\therefore\ f_{s3} = -.000693(29000) = -\underline{20.08\text{ ksi}}$ TENSION

$$\varepsilon_{s4} = \frac{.003}{c}(c - d_4) = -.002069 < \varepsilon_y = .002068$$

$\therefore\ f_{s4} = -\underline{60\text{ ksi}}$ TENSION

$$P_b = 0.85 \cdot f'_c \cdot b \cdot B_1 \cdot c + \sum A_{si} f_i$$

$$P_b = 0.85(4)(16)(.85)(7.989) + (2.46)(59.7) + .88(19.92)$$
$$- .88(20.08) - 2.46(60)$$

$$P_b = \underline{\underline{368.53}}^{k}$$

$$M_b = 0.85 f'_c \cdot b \cdot B_1 \cdot c \cdot \left(\frac{h}{2} - \frac{B_1 \cdot c}{2}\right) + \sum\left[A_{si} f_{si}\left(\frac{h}{2} - d_i\right)\right]$$

$$0.85(4)(16)(.85)(7.989)\left(8 - \frac{.85(7.989)}{2}\right)$$

$$+ (2.46)(59.7)\left(\frac{16}{2} - 2.5\right) + .88(19.92)\left(\frac{16}{2} - 6.1667\right)$$

$$+ .88(-20.08)\left(\frac{16}{2} - 9.83\right) + 2.46(-60)\left(\frac{16}{2} - 13.5\right)$$

$$M_b = 1716.5^{1-k} + 807.741 - 811.8 + 32.137 - 82.34$$

$$M_b = \underline{\underline{1712}}^{1-k}$$

STEP THREE: PURE BENDING CONDITION $(M_o, 0)$

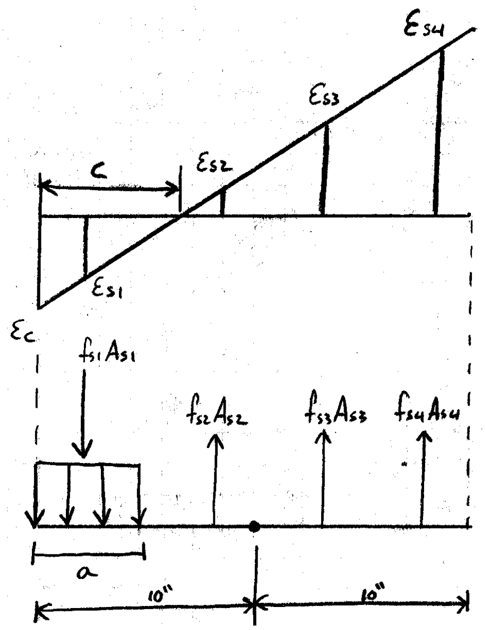
ASSUMPTIONS: $\varepsilon_{s1} < \varepsilon_y \therefore \varepsilon_{s1} = \frac{.003}{c}(c - 2.5)$ NO YIELD

$\varepsilon_{s2} < \varepsilon_y \therefore \varepsilon_{s2} = \frac{.003}{c}(c - 6.167)$ NO YIELD

$\varepsilon_{s3} > \varepsilon_y \therefore \varepsilon_{s3} = \varepsilon_y$ YIELDS

$\varepsilon_{s4} > \varepsilon_y \therefore \varepsilon_{s4} = \varepsilon_y$ YIELDS

MUST CHECK THESE ASSUMPTIONS LATER, FOR VALIDITY!



C IS MUCH MORE SHALLOW HERE

STRAIN

$f_{s1} = \frac{.003}{c}(c-2.5)(29000)$

$f_{s2} = \frac{.003}{c}(c-6.17)(29000)$

$f_{s3} = -60$

$f_{s4} = -60$

$\sum F = 0 = .85(4)(16)(.85)(c) + 12.46 f_{s1} + .88 f_{s2} + .88 f_{s3} + 2.46 f_{s4}$

$\sum F = 0 = 46.24c + 214.02 - \frac{535.05}{c} + 76.6 - \frac{472.4}{c} - 200.4$

$0 = 46.24c^2 + 90.2c - 1007.45$

$$\frac{-90.2 \pm \sqrt{(90.2)^2 - (4 \cdot 46.24 \cdot -1007.45)}}{2(46.24)}$$

$c = 3.793"$

VERIFY ASSUMPTIONS

$f_{s1} = \frac{.003}{3.793}(3.793-2.5)(29000) = 29.66 \text{ ksi} < 60 \text{ ksi}$ OKAY ✓
COMPRESSION

$f_{s2} = \frac{.003}{3.793}(3.793-6.16)(29000) = -54.29 \text{ ksi} < -60 \text{ ksi}$ OKAY ✓
TENSION

$f_{s3} = \frac{.003}{3.793}(3.793-9.83)(29000) = -138 > -60 \text{ ksi}$ OKAY ✓

$f_{s4} = \frac{.003}{3.793}(3.793-13.5)(29000) = -222 > -60 \text{ ksi}$ OKAY ✓

ASSUMPTIONS WERE CORRECT!

INT. DIAGRAM

FIVE

$$M_o = .85(f'_c)\, b\, B_1 c\left(\frac{h}{2} - \frac{B_1 c}{2}\right) + \sum\left[A_{si} f_{si}\left(\frac{h}{2} - d_i\right)\right]$$

$$M_o = .85(4)(16)(.85)(3.793)\left(8 - \frac{.85(3.793)}{2}\right) + 2.46(29.66)(8 - 2.5)$$
$$+ .88(54.29)(8 - 6.16)$$
$$- .88(60)(8 - 9.83)$$
$$- 2.46(60)(8 - 13.5)$$

$$M_o = 1120.37 + 401.3 + 87.91 - 96.6 - 811.8$$

$$M_o = \underline{\underline{701.18}}^{\text{FK}}$$

STEP FOUR: PURE TENSION $(0, T_o)$

$$T_o = \sum A_{si} f_{si}$$

$C = -\infty$, THEREFORE $\varepsilon_s = -\varepsilon_y$

FOR EQUATION ABOVE, $f_{si} = -f_y$

$$T_o = (2.46\text{ in}^2)(-60\text{ ksi}) + (.88\text{ in}^2)(-60\text{ ksi}) + (.88\text{ in}^2)(-60\text{ ksi}) + (2.46\text{ in}^2)(-60\text{ ksi})$$

$$T_o = \underline{\underline{-400.8}}^{K}$$

STEP FIVE: DETERMINE POINT ON INTERACTION DIAGRAM WHERE $\varepsilon_t = .005$

$$\varepsilon_t = \varepsilon_{s4} = .005$$

$$C = \frac{.003}{.003 + .005}(13.5) = 5.0625''$$

	INT. DIAGRAM		SIX

$$\varepsilon_{s1} = \frac{.003}{5.063}(5.063 - 2.5) = .001518 < \varepsilon_y \qquad f_{s1} = 44.04 \text{ ksi (C)}$$

$$\varepsilon_{s2} = \frac{.003}{5.063}(5.063 - 6.17) = -.000656 < \varepsilon_y \qquad f_{s2} = -19.02 \text{ ksi (T)}$$

$$\varepsilon_{s3} = \frac{.003}{5.063}(5.063 - 9.83) = -.00282 > -\varepsilon_y \qquad f_{s3} = -60 \text{ ksi (T)}$$

$$\varepsilon_{s4} = -60 \text{ ksi (T)}$$

$$M_n = 0.85(4)(16)(.85)(5.0625)\left(8 - \frac{.85(5.0625)}{2}\right) + 2.46(44)(8-2.5)$$
$$+ .88(-19.02)(8-6.17)$$
$$+ .88(-60)(8-9.83)$$
$$+ 2.46(-60)(8-13.5)$$

$$M_n = 1369 + 595.32 - 30.63 - 96.63 - 811.8$$

$$M_n = \underline{\underline{1025.26^{\text{k-ft}}}}$$

$$P_n = .85(4)(16)(.85)(5.0625) + 2.46(44) + .88(-19.02) + .88(-60) + 2.46(-60)$$

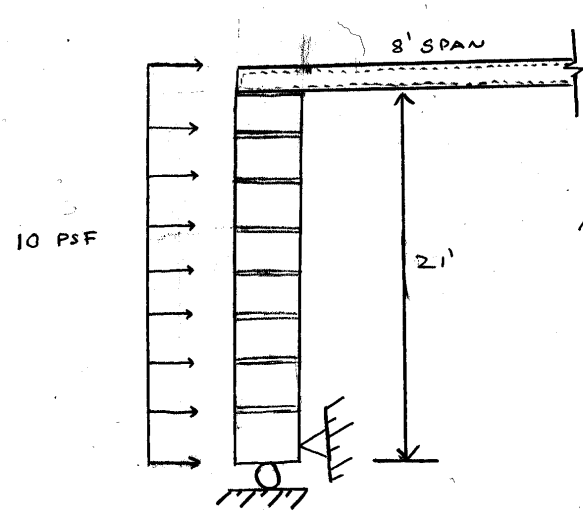
$$P_n = 234 + 108.24 - 16.74 - 52.8 - 147.6$$

$$P_n = \underline{\underline{100^{\text{k}}}}$$

Appendix F: Hand Calculations – 1 Story Unreinforced Concrete Masonry Wall Design

(1.)

DESIGN – 1^{ST} STORY EXTERIOR URM WALL



D = 76 PSF
L = 100 PSF (TERRACE)

WAFFLESLAB = 46 PSF
MISCELLANEOUS DL = 30 PSF

ASD: DUE TO LARGE WALL HEIGHT WILL CHECK LOAD COMBO $\boxed{.6D + W}$ and $\boxed{L + D}$

STEP ONE: SPECIFY MATERIAL AND WALL PROPERTIES

SOLID MASONRY UNIT – CONCRETE MASONRY
- → NORMAL WEIGHT: 10 PSF / INCH
- → MORTAR TYPE S: FULL BED MORTAR
- → FULLY GROUTED
- → PORTLAND CEMENT (PCL)

STEP TWO: CHECK UNITY EQUATION: $\frac{f_a}{F_A} + \frac{f_b}{F_B} \leq 1.0$ $\boxed{.6D + W}$

$P_{@\,MIDHEIGHT} = P_D + S.W. = (76\text{ PSF})(8'/2)(.6) + .6\,S.W.$

$S.W. = 10\text{ lb/ft}^2/\text{INCH} = 10 \times (8'' - 3/8'') = 76.3\text{ PSF}$

$P_{@\,MIDHEIGHT} = 182.4 + (21'/2 \times 76.3)(.6) = 663\text{ lb/FT}$

$f_a = P/A_n = \frac{663\text{ lb/FT}}{91.5} = 7.25$

$h = 21' \times 12'' = 252''$

$r = \sqrt{I/A} = \sqrt{\frac{bt^3/12}{91.5}} = \sqrt{\frac{12(7.63)^3/12}{91.5}} = 2.203\text{ in/ft}$

SLENDERNESS RATIO $= h/r = 252/2.203 = 114 > 99$

$\therefore\ F_A = 0.25 f'_m \left(\frac{70}{h/r}\right)^2$

$f'_m = 1500\text{ psi}$

2.

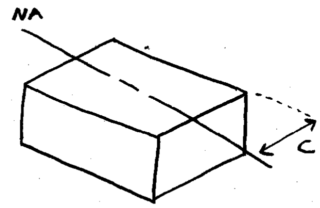
$$F_A = 0.25\,(1500)\left(\frac{70}{114}\right)^2 = 141 \text{ psi}$$

$$M_{MID\ HEIGHT} = .6\,\cancel{M_{ECC}}^{\,0} + M_{WIND}$$

$$= \frac{10 \cdot 21^2}{8} \times 12 = 551.25 \times 12 = 6615 \text{ lb-in}$$

$$c = 7.63''/2 = 3.815''$$

$$I = bt^3/12 = 12\,(7.63)^3/12 = 444 \text{ in}^4/\text{ft}$$



$$f_b = \frac{Mc}{I} = \frac{6615 \cdot 3.815''}{444} = 56.84 \text{ psi}$$

$$F_B = \tfrac{1}{3} f'_m = \tfrac{1}{3}(1500) = 500 \text{ psi}$$

$$\frac{f_a}{F_A} + \frac{f_b}{F_B} = \frac{7.25}{141} + \frac{56.84}{500} = .167 < 1.0 \text{ OKAY}$$

CHECK $\boxed{L + D}$

$$P_{@MIDHEIGHT} = 176 \text{ psf}\,(8/2) + (21/2 \times 76.3 \text{ psf})$$

$$= 1505.15 \text{ lb/ft}$$

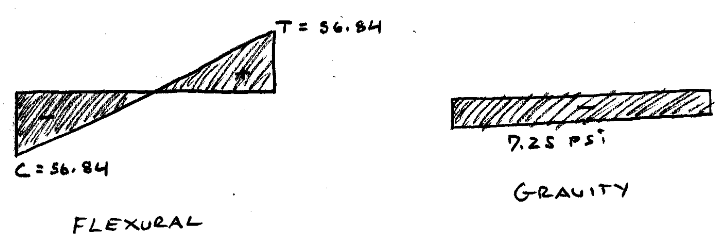
$$f_a = P/A_n = \frac{1505.15}{91.5} = 16.45 \text{ psi}$$

$$M = 0 \qquad f_b = 0$$

$$\frac{f_a}{F_A} + \frac{f_b}{F_B} = \frac{16.45}{141} + 0 = .11667 \le 1.0 \text{ OKAY}$$

STEP THREE CHECK TENSILE STRESS $\boxed{.6D + W}$

$$\text{TENSION} = 56.84 - 7.25 = 49.59 \text{ psi}$$

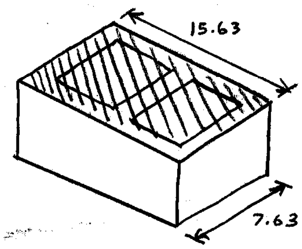


Allowable F_T: TYPE S MORTAR, PCL $F_T = 40$ psi (SOLID UNIT)

$f_t = 49.59$ psi $>$ 40 psi ✗ NOT GOOD, FAILS IN TENSION

NOW TRY A HOLLOW UNIT – FULLY GROUTED

$F_T = 65$ psi



STEP ONE: CHECK UNITY EQUATION $\boxed{.6D + W}$

$P_{@\,MID\,HEIGHT} = .6D + .6\,S.W.$

$= .6(76\ \text{psf} \times 4) + .6(21/2 \cdot 91) = 755.7$

$f_a = P/A = 755.7/91.5 = 8.25$ psi

$h = 21' \times 12'' = 252''$

$r = \sqrt{I/A} = 2.203$

$h/r = 114 > 99 \qquad F_A = .25 f_m' \left(\frac{70}{h/r}\right)^2 = 141$ psi

$f_m' = 1500$ psi

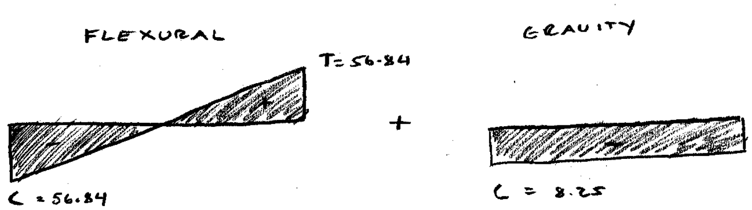
$M_{@\,MID\,HEIGHT} = \cancel{M_{ECC}}^{\,0}(.6) + M_{WIND} = 6615$ lb-in

$f_b = \frac{Mc}{I} = \frac{6615 \times 3.815}{444} = 56.84$ psi

$F_B = 1/3\ f_m' = 500$ psi

$\frac{f_a}{F_A} + \frac{f_b}{F_B} = \frac{8.25}{141} + \frac{56.84}{500} = .172 \le 1.0$ OKAY ✓

STEP TWO: CHECK TENSILE STRENGTH



MAX TENSION $= 56.84 - 8.25 = 48.59$ psi $< F_T = 65$ psi OKAY ✓

4.

STEP THREE: CHECK STABILITY: BUCKLING LOAD LIMIT

$$P \leq \tfrac{1}{4} P_e$$

$$P_e = \left(\frac{\pi^2 E_m I}{h^2}\right)\left[1 - 0.577\frac{e}{r}\right]^3$$

e = effective eccentricity = 0

$$P_e = \left(\frac{\pi^2 (900)(1500)(444)}{(12 \times 21^2)}\right)\left(\tfrac{1}{1000}\right) = 93.2\ ^{k}/_{ft}$$

$$\frac{P_e}{4} = 23.2\ ^{k}/_{FT}$$

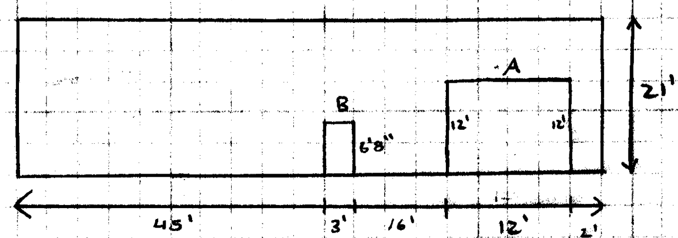
CHECK: D + L

P = 1505.15 lb/ft = $\boxed{1.5\ ^{k}/_{FT} < 23.2\ ^{k}/_{FT} \quad \text{OKAY} \checkmark}$

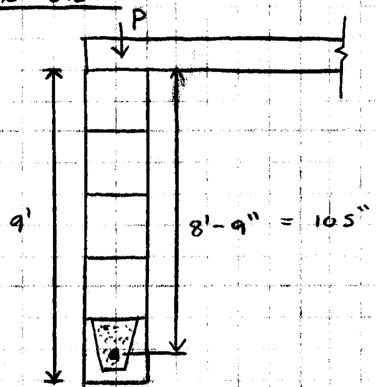
Appendix G: Hand Calculations – Reinforced Concrete Masonry Bond Beam Design

DESIGN OF REINFORCED MASONRY BOND BEAM

ELEVATION VIEW:



LINTEL A: SECTION VIEW



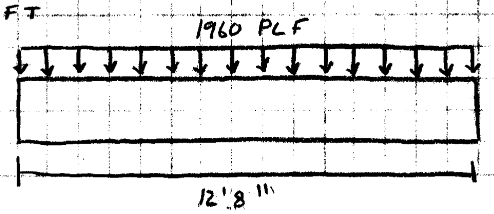
KNOWN INFORMATION: P_D = 76 PSF
P_L = 100 PSF
FULLY GROUTED, Hollow CMU, 8" BLOCKS!
f_m' = 1500 psi
TYPE S (PCL MORTAR)
LINTEL EXTENDS 4" PAST OPENING (BOTH SIDES)
Assume movement joints AT ENDS OF BEAM

STEP ONE: STRENGTH DESIGN: L.C. = 1.2D + 1.6L

$W_U = 1.2P_D + 1.6P_L + 1.2$ (ABOVE WALL WEIGHT)

$= 1.2\,(8'\text{span}/2)(76\text{ PSF}) + 1.6\,(8/2 \cdot 100\text{ PSF}) + 1.2\,(8.75')(91\text{ PSF})$

W_u = 1960 lbs / FT



STEP TWO: $M_u = \frac{W_u \ell^2}{8}$

$\ell = 12' + 4" + 4" = 12.67'$

$M_u = \frac{(1960)(12.67)^2}{8} \times 12 = 471{,}954$ in-lbs

STEP THREE: $V_u = \frac{W_u \ell}{2} = \frac{1960 \cdot 12.67}{2} = 12{,}416$ lbs

STEP FOUR: SHEAR CAPACITY CHECK $V_u \leq \phi V_n$

$V_{nm} = \left[4 - 1.75\left(\frac{M_u}{V_u d_v}\right)\right] A_n \sqrt{f_m'} + .25 P_u$

P_u: $0 \rightarrow$ NO AXIAL LOAD

$V_{nm} = \left[4 - 1.75(1)\right] A_n \sqrt{f_m'}$

$V_{nm} = 2.25(7.63 * 105)\sqrt{1500} = 69{,}813$ lbs

$\phi V_{nm} = .8(69{,}813) = 55{,}851 > V_u = 12{,}416$ lbs

NO SHEAR REINFORCEMENT NEEDED

STEP FIVE: $V_m = V_n$ ~~WITH~~ $\frac{M_u}{V_u d_v} \geq 1.0$

$V_n \leq 4 A_n \sqrt{f_m'} = 124{,}113$

$V_n = 69{,}813 \leq 124{,}113$ OKAY ✓

STEP SIX:

$M_n = \rho b d^2 f_y \left(1 - \frac{.625 \rho f_y}{f_m'}\right)$ where $\rho = \frac{A_s}{bd}$

$\phi M_n = .8\rho(7.63)(105^2)(65{,}000)\left(1 - \frac{.625\rho(65{,}000)}{1500}\right) \geq 471{,}954$

$-27.08\rho^2 + \rho - 1.0789 \times 10^{-4} \geq 0$

$\rho = 1.08 \times 10^{-4} = \frac{A_s}{bd}$

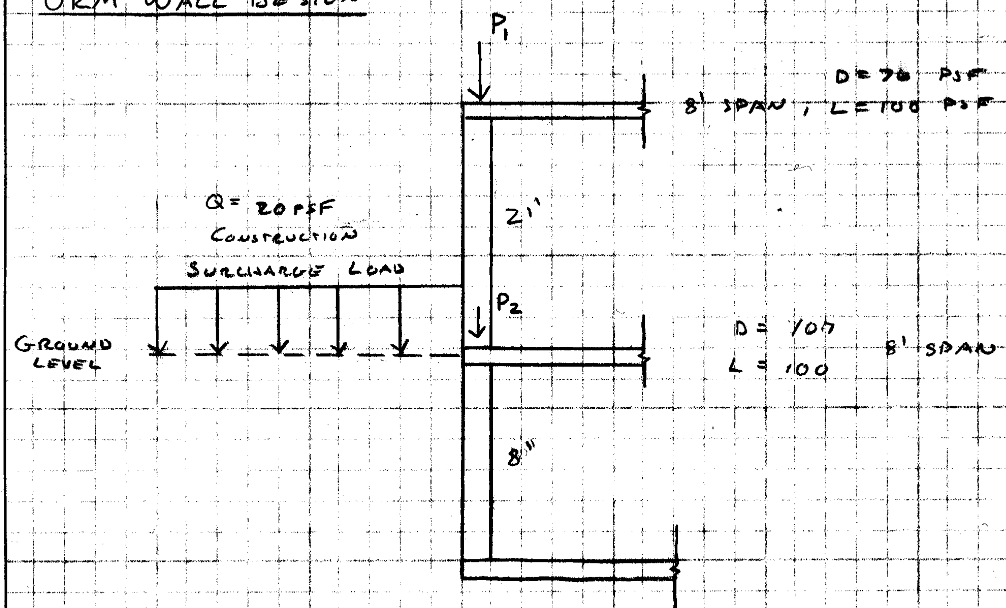
$A_s = 1.08 \times 10^{-4}(7.63)(105) = .0865$ in^2

USE A #3 bar = $A_s = .11$ in^2

$\phi = 3/8"$

Appendix H: Hand Calculations – 2 Story Reinforced Concrete Masonry Wall Design

URM WALL DESIGN



GEOTECH REPORT

Loose soil, cohesionless, $\phi = 32.5°$

DRY UNIT WEIGHT $= \gamma_d = \frac{W_s}{V}$ $\qquad G_s = 2.67$

$\gamma_D = 110$ PCF

DEPTH OF WATER TABLE > H of wall

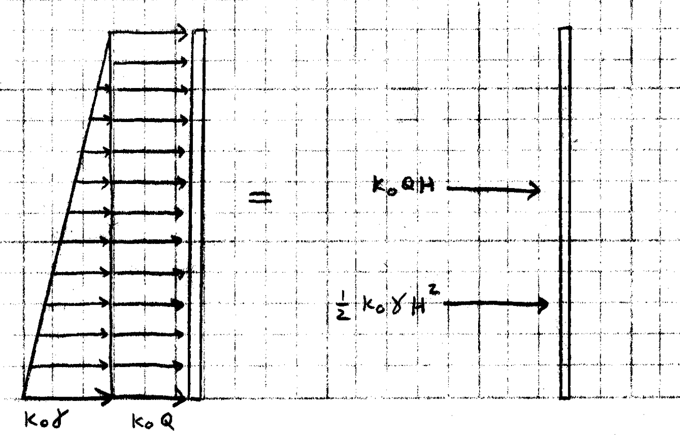
STEP ONE: DETERMINE LATERAL EARTH PRESSURE

FBD $\qquad$ RESULTANTS

$k_o = 1 - \sin \phi$

$k_o = 1 - \sin 32.5°$

$k_o = .462$



2

Surcharge Force = $K_o Q H = (.462)(20)(8) = 73.9$ lb/ft

Soil pressure = $\frac{1}{2} K_o \gamma H^2 = (\frac{1}{2})(105)(8^2)(.462) = 1552$ lb/ft

STEP TWO: FIND CRITICAL SECTION – FLEXURAL (M_{max})

$\sum M_{Bot} = 0 = K_o Q H (5') + (\frac{1}{2} K_o \gamma H^2)(10/3)$

$= 369.5 + 5174 = 5543.5$ lb-ft.

STEP THREE: SPECIFY WALL PROPERTIES

Hollow 8" CMU, FULLY GROUTED
TYPE S MORTAR PCL

STEP FOUR: CHECK UNITY EQUATION $\frac{f_a}{F_A} + \frac{f_b}{F_B} \leq 1.0$

CHECK L+R ASD

$P_{@\,bottom} = (176\ psf)(8\frac{1}{2}) + (207\ psf)(8\frac{1}{2}) + (91\ psf \times 31')$

$P_{@\,bottom} = 4,353$ lb/ft of wall

$P/A = 4,353/91.5 = 47.6$ psi/ft of wall $= f_a$

$h/r = \frac{10 \times 12}{\sqrt{I/A}} = \frac{120}{\sqrt{444/91.5}} = 54.4 < 99$

$\therefore F_A = 0.25 f_m' \left(1 - \left(\frac{h/r}{140}\right)^2\right) = 0.25(1500)\left(1 - \left(\frac{54.4}{140}\right)^2\right)$

$F_A = 318.4$ psi

$f_b = \frac{Mc}{I} = \frac{66,522 \cdot (7.63/2)}{444} = 571$ psi

$F_B = \frac{1}{3} f_m' = 500$ psi

$\frac{f_a}{F_A} + \frac{f_b}{F_B} = \frac{47.6}{318.4} + \frac{571}{500} = 1.29 > 1$ NOT GOOD ×

NEED TO DESIGN A REINFORCED MASONRY WALL

RM Calculations Are Not Shown. However The final Interaction Diagram Adjusted For Slenderness Effects Can be found on page ##. The final design required #9 bars @ 24" placed in center of cells. Type S PCL Mortar. Fully Grouted 8" Hollow CMU

Appendix I: Design Of All 7 Multi-Story Reinforced Concrete Shear Walls

WALL 1

HEIGHT OF WALL = $h_w = 141' = 1692''$

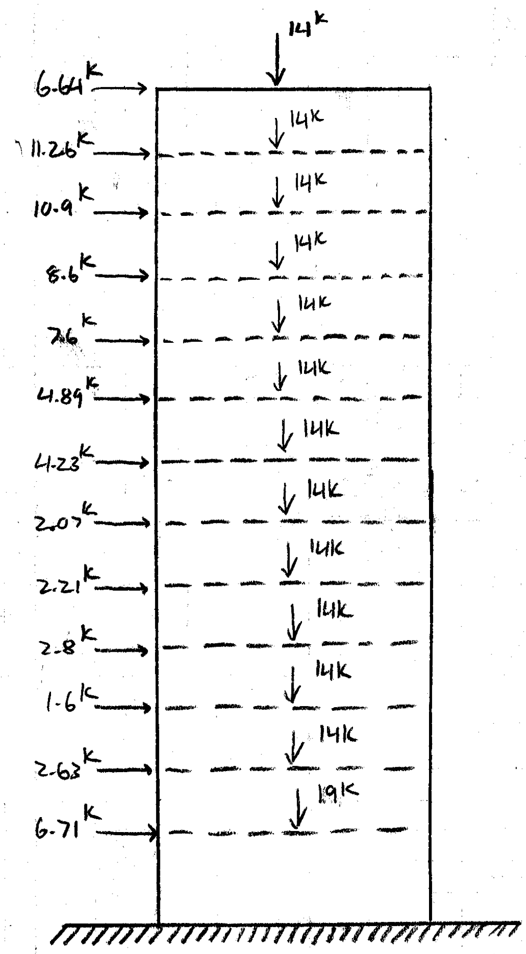
LENGTH OF WALL = $l_w = 11'$

THICKNESS $h = 10''$

$\boxed{\text{STRENGTH BEHAVIOR}}$: $h_w/l_w = 141/11 = 12.8 > 3$

∴ SHEAR wall → "SLENDER FLEXURAL WALL"

STEP ONE: MODEL WALL W/ LOADS



$M_u = 6{,}952.64^{\,l\text{-}k}$

$V_u = 72.14^k$

$N_u = .9d = .9(14 \times 12 + 19) = 168^k$

STEP TWO: "SLENDER WALL" DESIGN FOR FLEXURE 1ST

$M_n = A_s f_y (d - a/2) = A_s f_y jd$

$C = T: \quad 0.85 f'_c ab = A_s f_y$

$jd = .9(.8 l_w) = .9(.8)(11 \times 12) = .9(105.6) = 95.04''$

$M_u = \phi M_n = \phi A_s f_y jd$

$(6952)(12000) = .9(A_s)(60000)(95.04)$

$A_s = 16.26 \text{ in}^2$

$.85 f'_c ab = A_s f_y$

$$a = \frac{(16.26)(60)}{.85(4)(10)} = 28.69''$$

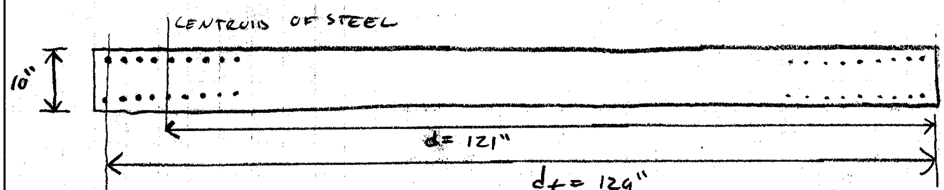
RECALCULATE jd: $jd = d - a/2 = 105.6 - \frac{28.69}{2} = 91.26''$

RECALCULATE As:

$(6952)(12000) = .9 A_s (60{,}000)(91.26'')$

$A_s = 16.929 \text{ in}^2$ TRY (18) #9

A_s PROVIDED $= 18 \text{ in}^2 > 16.929 \text{ in}^2$ OKAY



$C = T: \quad 0.85 f'_c ab = A_s f_y$

$$a = \frac{(18)(60)}{.85(4)(10)} = 31.76''$$

NEUTRAL AXIS DEPTH C:

$$C = \frac{a}{\beta} = \frac{31.76}{.85} = 37.37''$$

CHECK TENSION CONTROL

$$\varepsilon_t = .003\left(\frac{d_t - c}{c}\right) > .005$$

$$\varepsilon_t = .003\left(\frac{129 - 37.37}{37.37}\right)$$

$\varepsilon_t = .00735 > .005$ OK ✓

$\boxed{\text{USE (18) \#9 bars AS FLEXURE REINFORCEMENT}}$

STEP THREE – DESIGN WALL FOR SHEAR $V_u = 72.14^k$

• CHECK MAX PERMITTED SHEAR STRENGTH:

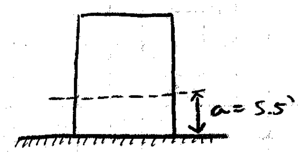
$$V_u \le \phi V_{n\,MAX} = \phi 10 \sqrt{f'_c}\, hd = \frac{.75(10)\sqrt{4000}(10)(.8 \times 11 \times 12)}{1000}$$

$V_u = 72.14^k \le 501^k$

OKAY, WALL DIMENSIONS WILL WORK!

• DETERMINE LOCATION OF CRITICAL SHEAR SECTION

$$a \le \text{minimum} \begin{cases} l_w/2 = 11'/2 = 5.5' \ \ast \text{ CONTROLS} \\ h_w/2 = 141'/2 = 70.5' \end{cases}$$



• DETERMINE SHEAR STRENGTH CONTRIBUTION OF CONCRETE:
~ CONSERVATIVE / SIMPLIFIED METHOD

$$V_c = 2\sqrt{f'_c}\, hd = 2\sqrt{4000}(10)(.8 \times 11 \times 12) = 133.6^k$$

$$\phi V_c = .75(133.6^k) = 100^k > 72.14^k = V_u$$

∴ DONT NEED TO CALCULATE V_s TO DETERMINE SHEAR REINF.

• CHECK $V_u > \frac{\phi V_c}{2} = \frac{.75(133.6)}{2} = 50^k$

$V_u = 72.14^k > 50^k$

∴ USE CHAPTER 11 PROVISIONS ACI TO DESIGN REINFORCEMENT ✓

• CHECK MINIMUM HORIZONTAL (TRANSVERSE) SHEAR REINF. RATIO ρ_t:

$$\rho_t = \frac{A_v}{sh} > .0025$$

TRY (2) #4

$$\rho_t = \frac{2(.2)}{s(10)} > .0025$$

$s = 16" < 18"$ OKAY ✓

CHECK MINIMUM SPACING

$$s = 16" \le \min \begin{cases} l_w/5 = 26" \\ 3h = 30" \\ 18" \ \ast \text{ CONTROLS} \end{cases}$$

$$\boxed{\text{USE (2) \#4 @ 16" FOR HORIZ. REINFORCEMENT}}$$

• DESIGN VERTICAL SHEAR REINFORCEMENT

$$\rho_\ell = \frac{A_v}{s h} \geq .0025 + 0.5\left(2.5 - \frac{h_w}{\ell_w}\right)\left(\rho_t - .0025\right)$$

$h_w/\ell_w = \frac{141}{11} = 12.8 > 2.5 \quad \therefore$ EQUATION WON'T GOVERN ✓

$\rho_\ell \geq .0025 \quad \therefore$ USE (2) #4 @ 16"

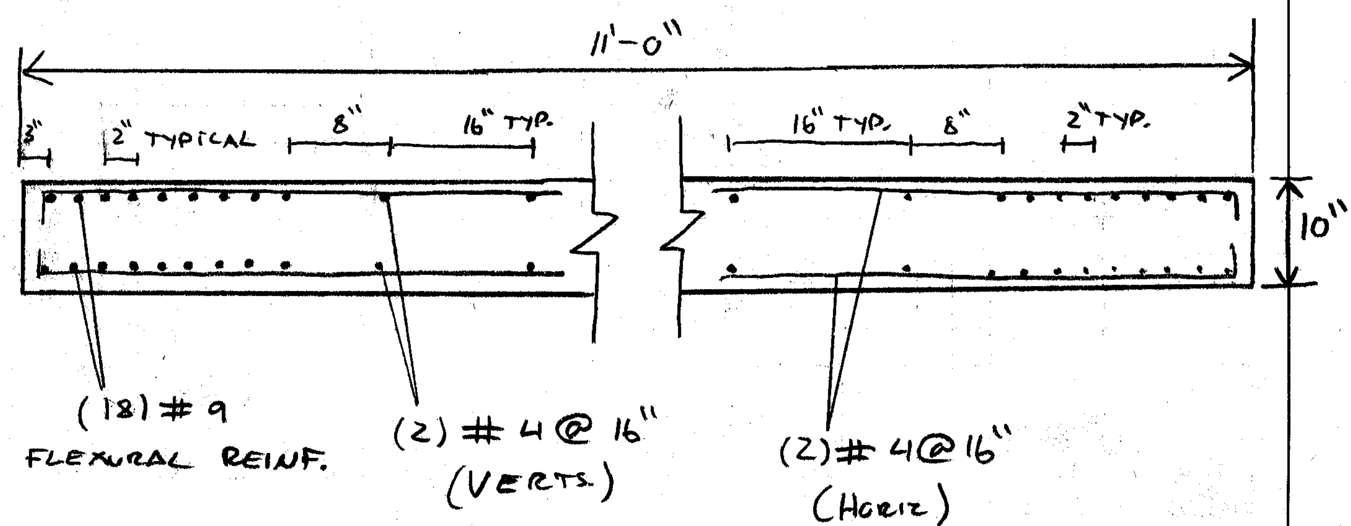
CHECK MINIMUM SPACING...

$$s = 16" \leq \begin{cases} \ell_w/3 = 44" \\ 3h = 30" \\ 18" = 18" \text{ governs *} \end{cases}$$

16" < 18" OKAY ✓

$\boxed{\text{USE (2) \#4 @ 16" FOR VERTICAL REINFORCEMENT}}$

FINAL SHEAR WALL DESIGN:



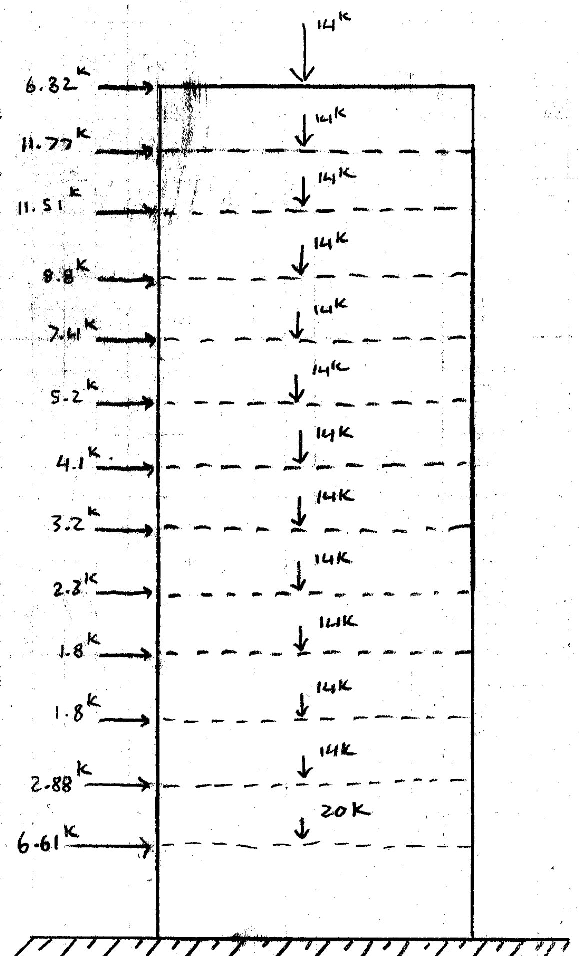
WALL 2

HEIGHT OF WALL : $h_w = 141'$
LENGTH OF WALL : $l_w = 12'$
THICKNESS $h = 10"$

STRENGTH BEHAVIOUR : $h_w / l_w = 141/12 = 11.75 > 3$

∴ SHEAR WALL → "SLENDER WALL" → FLEXURE CONTROLS

STEP ONE : MODEL WALL WITH LOADS



$M_u = 7116.39$ '-k

$V_u = 73.69^k$

$N_u = .9d = .9(14 \times 12 + 19) = 168.3^k$

STEP TWO: "SLENDER WALL" DESIGN FOR FLEXURE 1st

$M_n = A_s f_y (d - a/2) = A_s f_y \, jd$

C = T: $0.85 f'_c a b = A_s f_y$

$jd = .9(.8 l_w) = .9(.8)(12 \times 12) = .9(115.2) = 103.68$

$M_u = \phi M_n = \phi A_s f_y \, jd$

$(7116)(12000) = .9(A_s)(60000)(103.68)$

$A_s = 15.252$

$.85 f'_c a b = A_s f_y$

$a = \dfrac{(15.252)(60)}{(.85)(4)(10)} = 26.92''$

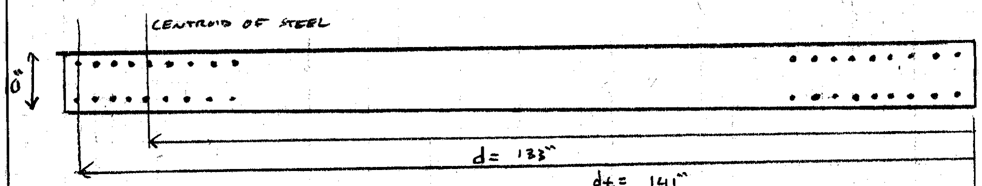
RECALCULATE jd: $jd = d - a/2 = 103.68 - (26.92/2) = 90.22''$

RECALCULATE As:

$(7116)(12000) = .9 A_s (60{,}000)(90.22)$

$A_s = 17.53 \text{ in}^2$ TRY (18) #9

$A_{s\,PROVIDED} = 18 \text{ in}^2 > 17.53 \text{ in}^2$ OKAY ✓



C = T: $0.85 f'_c a b = A_s f_y$

$a = \dfrac{(18)(60)}{.85(4)(10)} = 31.7647''$

NEUTRAL AXIS DEPTH, c:

$c = \dfrac{a}{\beta} = \dfrac{31.76}{.85} = 37.37''$

CHECK TENSION CONTROL

$\varepsilon_t = .003 \left(\dfrac{d_t - c}{c} \right) > .005$

$\varepsilon_t = .003 \left(\dfrac{141 - 37}{37} \right)$

$\varepsilon_t = .008319 > .005$ OKAY ✓

$\boxed{\text{USE (18) \#9 bars – FLEXURE REINFORCEMENT}}$

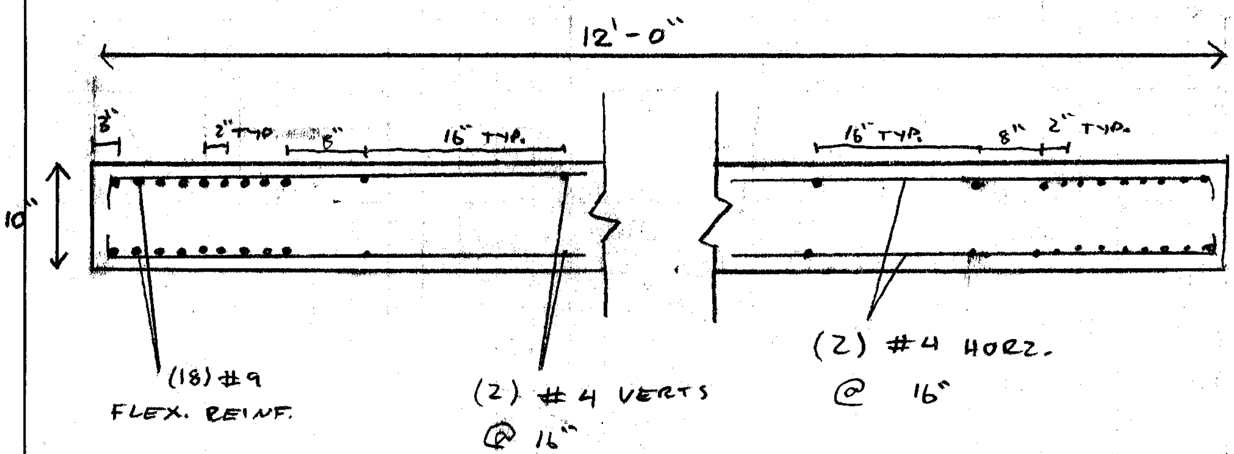
STEP THREE: DESIGN WALL FOR SHEAR

USE MINIMUM HORIZ. + VERTICAL SHEAR REINFORCEMENT

(2) #4 @ 16" → VERT

(2) #4 @ 16" → HORIZONTAL

FINAL SHEAR WALL DESIGN – WALL 2 AND 3 IDENTICAL



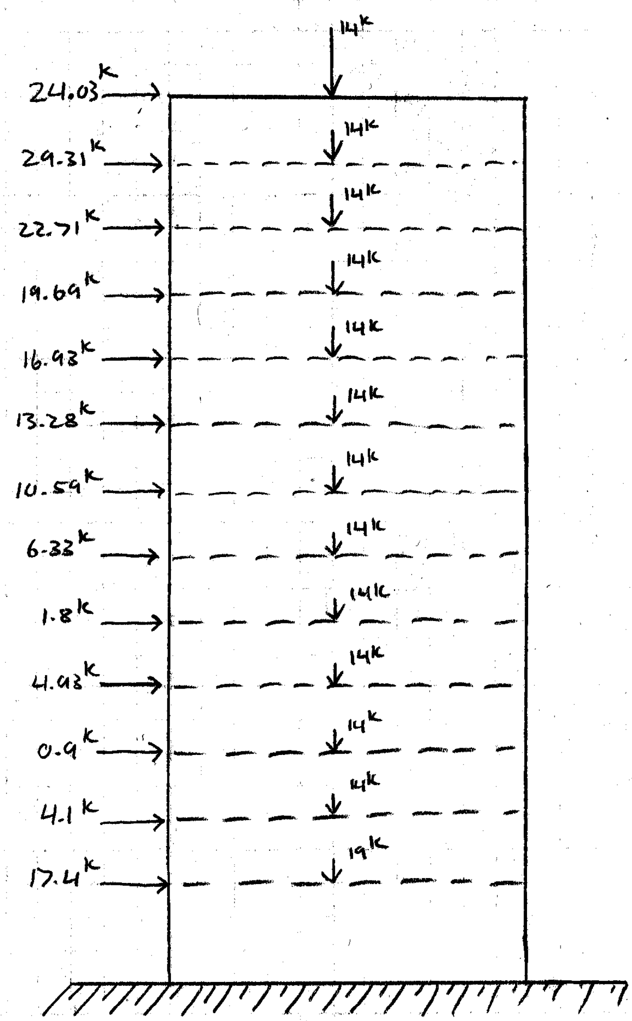
WALL 4

HEIGHT OF WALL $h_w = 141' = 1692''$
LENGTH OF WALL $\ell_w = 19' = 228''$
THICKNESS $h = 10''$

$\boxed{\text{STRENGTH BEHAVIOR}}$: $h_w/\ell_w = 141/19 = 7.42 > 3$

∴ SHEAR WALL → "SLENDER FLEXURAL WALL"

STEP ONE MODEL WALLS W/ LOADS :



$M_u = 17{,}277^{k\text{-}'}$

$V_u = 172^k$

$N_u = .9(12 \times 14 + 19) = 168.3^k$

STEP TWO: DESIGN FOR FLEXURE 1st

$M_n = A_s f_y (d - a/2) = A_s f_y \cdot jd$

$C = T: \quad 0.85 f'_c a b = A_s f_y$

$jd = .9d = .9(.8 l_w) = .9(.8)(19 \times 12) = 164.16''$

$M_u = \phi M_n = \phi A_s f_y jd$

$(17,277)(12,000) = .9 A_s (60,000)(164.16)$

$A_s = 23.38 \text{ in}^2$

$.85 f'_c a b = A_s f_y$

$$a = \frac{(23.28)(60,000)}{.85(4000)(10)} = 41.27''$$

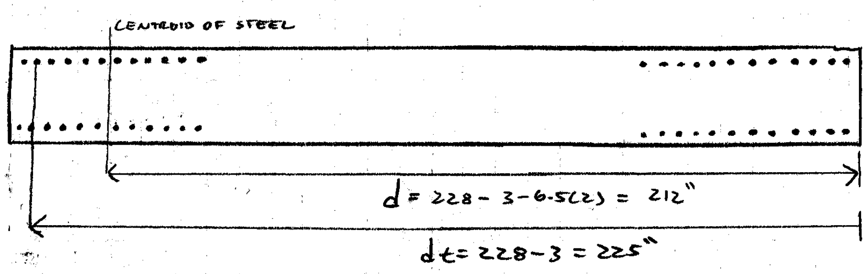
RECALCULATE jd: $jd = d - \frac{a}{2} = 182.4'' - \frac{41.27''}{2} = 161.77''$

RECALCULATE A_s:

$(17,277)(12,000) = .9 A_s (60,000)(161.77)$

$A_{s\ REQUIRED} = 23.73 \text{ in}^2$

TRY (24) #9 $A_{s\ PROVIDED} = 24 \text{ in}^2 > 23.73 \text{ in}^2$



$C = T$:

$0.85 f'_c a b = A_s f_y$

$$a = \frac{(24)(60)}{.85(4)(10)} = 42.35''$$

NEUTRAL AXIS: c

$$c = \frac{42.35}{.85} = 49.8''$$

CHECK TENSION CONTROL CRITERIA:

$$\varepsilon_t = .003\left(\frac{d_t - c}{c}\right) > .005$$

$$= .003\left(\frac{225 - 49.8}{49.8}\right) = .01 > .005 \text{ OKAY}$$

$$\boxed{\text{USE (24) \#9 FOR FLEXURAL REINFORCEMENT}}$$

STEP THREE: DESIGN HORIZONTAL SHEAR REINFORCEMENT

• CHECK MAXIMUM PERMITTED SHEAR STRENGTH

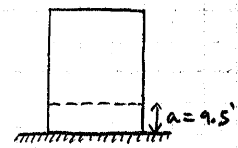
$$V_u \leq \phi V_{n\,max} = \phi\, 10\sqrt{f'_c}\; hd = \frac{.75\,(10)\sqrt{4000}\,(10)(.8 \times 19 \times 12)}{1000}$$

$$V_u = 172^k \leq 865.19^k$$

SHEAR WALL DIMENSIONS ARE OKAY ✓

• DETERMINE LOCATION OF CRITICAL SHEAR SECTION

$$a \leq \text{minimum} \begin{cases} l_w/2 = 19/2 = 9.5' & \text{* CONTROLS} \\ h_w/2 = 141/2 = 70.5' \end{cases}$$



• DETERMINE SHEAR STRENGTH CONTRIBUTION OF CONCRETE:

V_c IS PERMITTED TO BE LESSER OF EQUATIONS 11-27 AND 11-28

"INCLINED CRACKING STRENGTH"

$$\frac{M_u}{V_u} - \frac{l_w}{2} = \frac{17{,}277}{172} - \frac{19}{2} = 90.9 > 0$$

$$\left(\frac{M_u}{V_u} - \frac{l_w}{2}\right) \rightarrow \text{THIS TERM IS POSITIVE} \;\therefore\; \text{EQUATION 11-27 GOVERNS}$$

$$V_c = 3.3\sqrt{f'_c}\; hd + \frac{N_u d}{4\, l_w}$$

$$V_c = \frac{3.3\sqrt{4000}\,(10)(.8)(19 \times 12)}{1000} + \frac{(168.3^k)(.8)(19 \times 12)}{4\,(19 \times 12)}$$

$$V_c = 380.69^k + 33.66^k$$

$V_c = 414.35^k$ CAPACITY

$$\phi V_c = .75(414.35^k) = 310.76^k > V_u = 172^k$$

$\therefore$ NO NEED TO CALCULATE V_s TO DETERMINE THE SHEAR REINFORCEMENT

- CHECK $V_u > \frac{1}{2}\phi V_c$

$$V_u = 172^k > \frac{1}{2}(.75)(414.35^k) = 155.38^k \checkmark$$

∴ DESIGN SHEAR REINFORCEMENT ACCORDING TO ACI CHAPTER 11 PROVISIONS.

- CHECK MINIMUM HORIZONTAL REINFORCEMENT RATIO ρ_t

$$\rho_t = \frac{A_v}{sh} \geq .0025$$

TRY (2)# 4 $\quad \frac{(2)(0.2)}{(.0025)(10)} \geq s = 16''$

(2) # 4 @ 16" (ONE ON EACH FACE) SATISFIES MINIMUM RATIO OF HORIZONTAL SHEAR REINFORCEMENT TO GROSS CONCRETE AREA OF VERTICAL SECTION.

- CHECK MINIMUM SPACING

$$s \leq \text{Minimum}\begin{cases} l_w/5 = 19\times12/5 = 45.6'' \\ 3h = 3(10) = 30'' \\ 18'' \ast \text{ CONTROLS} \end{cases} \quad s = 16'' < 18'' \checkmark \text{ OKAY}$$

$$\boxed{\begin{array}{l}\text{USE (2) \# 4 @ 16''} \\ \text{FOR HORIZONTAL REINFORCEMENT}\end{array}}$$

- DESIGN VERTICAL SHEAR REINFORCEMENT

$$\rho_l = \frac{A_v}{sh} \geq .0025 + 0.5\left(2.5 + h_w/l_w\right)\left(\rho_t - .0025\right)$$

$h_w/l_w > 2.5$ ∴ EQUATION ACI 11-30 WON'T GOVERN

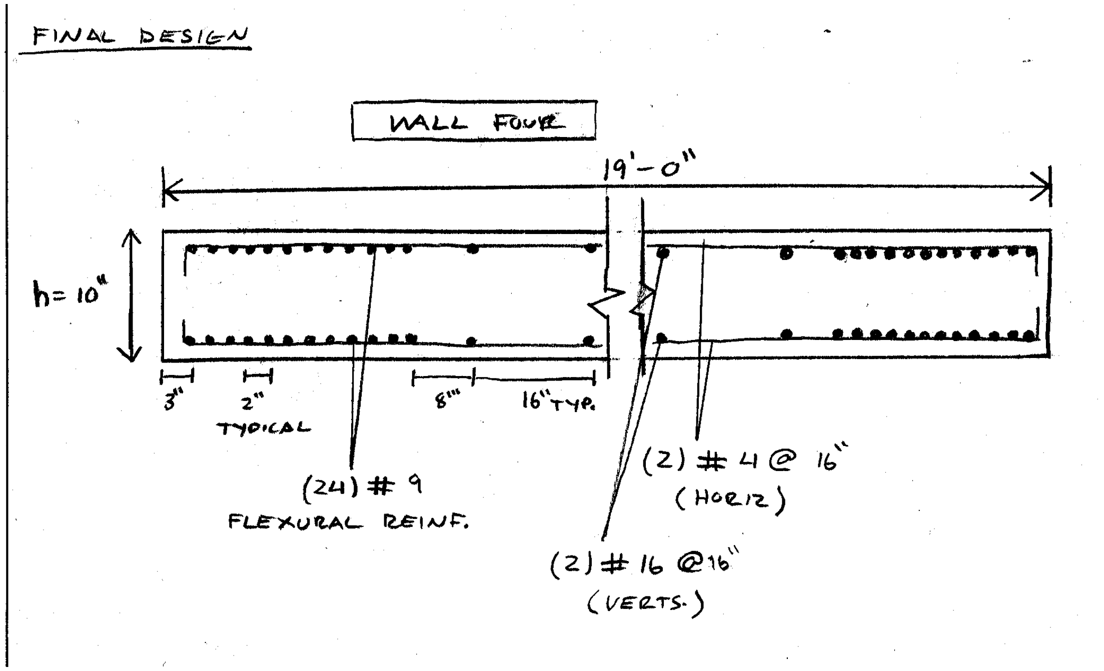
$\rho_l \geq .0025 \quad$ (2) # 4 @ 16" WORKS ✓

CHECK SPACING LIMITATIONS:

$$s \leq \begin{cases} l_w/3 = 76'' \\ 3h = 30'' \\ 18'' \ast \text{ CONTROLS} \end{cases} \quad 16'' < 18'' \checkmark \text{ OKAY}$$

$$\boxed{\begin{array}{l}\text{VERTICAL REINFORCEMENT:} \\ \text{(2) \# 4 @ 16''}\end{array}}$$

↳ NEEDED TO RESIST COUPLE FORCES, CONTROL TEMP./SHRINKAGE, AND CRACK CONTROL


FINAL DESIGN
WALL FOUR
19'-0"
h= 10"
3"
2"
TYPICAL
8"
16" TYP.
(24) # 9
FLEXURAL REINF.
(2) # 4 @ 16"
(HORIZ)
(2) # 16 @ 16"
(VERTS.)

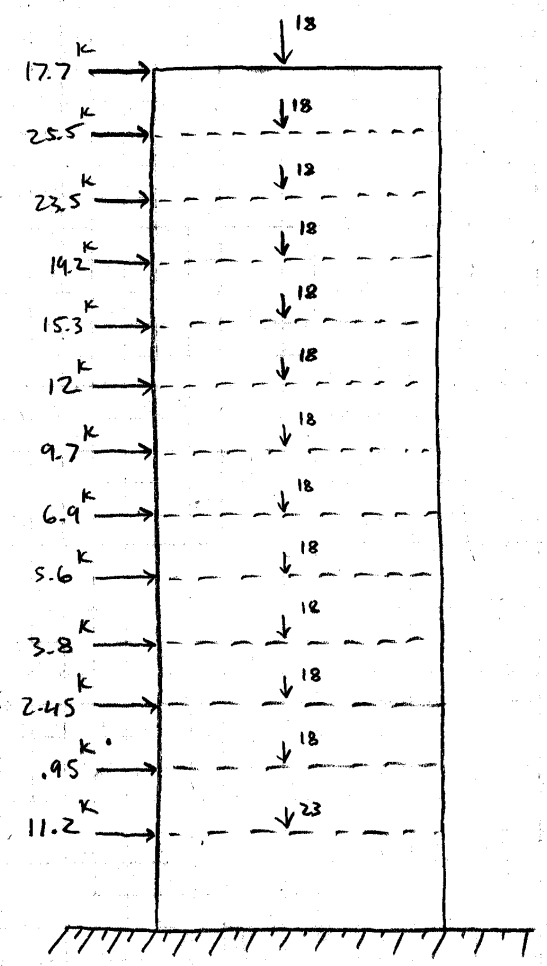
WALLS

HEIGHT OF WALL $h_w = 141' = 1692''$
LENGTH OF WALL $L_w = 18' = 216''$
THICKNESS = 10"

$\boxed{\text{STRENGTH BEHAVIOR}}$: $h_w / \ell_w = 7.83 > 3$

∴ SHEAR WALL → "SLENDER FLEXURAL WALL"

STEP ONE: MODEL WALL W/ LOADS



$M_U = 15{,}624^k$

$V_U = 153.8^k$

$N_U = .9D + .9(18 \times 12) + .9(23) = 215.1^k$

STEP TWO: FLEXURE DOMINATED, THEREFORE DESIGN FOR FLEXURE 1ST

$M_n = A_s f_y (d - a/2)$

$C = T: \quad .85 f'_c ab = A_s f_y$

LET $jd = .9d = .9(.8 l_w) = .9(.8 \times 18 \times 12) = .9(172.8) = 155.52''$

$M_u = \phi M_n = \phi A_s f_y jd$

$(15{,}624)(12{,}000) = .9 A_s (60{,}000)(155.28)$

$A_s = 22.36 \text{ in}^2$

$C = T: \quad 0.85 f'_c ab = A_s f_y$

$$a = \frac{(22.36)(60)}{.85(3)(10)} = 52.5''$$

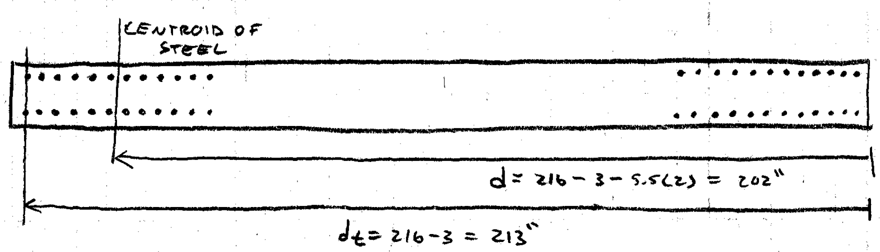
RECALCULATE jd: $d - a/2 = 172.8 - \frac{52.5}{2} = 146.55''$

RECALCULATE A_s:

$(15{,}624)(12{,}000) = .9 A_s (60{,}000)(146.55)$

$23.69 \text{ in}^2 = A_s$

$\boxed{\text{TRY } (24)\ \#9}$ $\quad A_s$ provided $= 24 \text{ in}^2 > 23.69 \text{ in}^2$ OKAY ✓



$C = T:$

$0.85 f'_c ab = A_s f_y$

$$a = \frac{(24)(60)}{.85(4)(10)} = 42.35''$$

$$b = \frac{42.35}{.85} = 49.8''$$

CHECK TENSION CONTROL

$$\varepsilon_t = \varepsilon_{cu}\left(\frac{d_t - c}{c}\right) > .005$$

$$\varepsilon_t = .003\left(\frac{213 - 49.8}{49.8}\right) = .009 > .005$$

OKAY ✓

STEP THREE: DESIGN WALL FOR SHEAR: $V_u = 153.8^k$

• CHECK MAXIMUM PERMITTED SHEAR STRENGTH

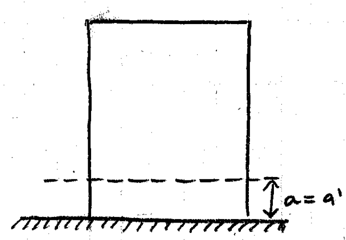
$$V_u \le \phi V_{n\,max} = \phi 10\sqrt{f'_c}\,hd = \frac{.75(10)\sqrt{f'_c}\,(10)(.8 \times 18 \times 12)}{1000}$$

$$V_u = 153.8^k \le 819^k$$

OKAY wall DIMENSIONS WILL WORK!

• DETERMINE LOCATION OF CRITICAL SHEAR SECTION

$$a \le \text{minimum} \begin{cases} l_w/2 = 9' & \text{* CONTROLS} \\ h_w/2 = 70.5' \end{cases}$$



• DETERMINE SHEAR STRENGTH CONTRIBUTION OF CONCRETE:

$$\frac{M_u}{V_u} - \frac{l_w}{2} = \frac{15624}{158} - \frac{18}{2} = 89.88 > 0$$

EQUATION ACI 11-27 will govern over 11-28

$$V_c \le 3.3\sqrt{f'_c}\,hd + \frac{N_u d}{4 l_w}$$

$$\le \frac{3.3\sqrt{4000}\,(10)(18 \times 12 \times .8)}{1000} + \frac{(215)(.8)(18 \times 12)}{4(18 \times 12)}$$

$$V_c \le 360.65^k + 43.02^k$$

$$V_c \le 403.67^k \qquad \phi V_c = 302.75^k < V_u = 158.3^k$$

NO NEED TO CALCULATE V_s TO DETERMINE SHEAR REINFORCEMENT.

• CHECK $V_u = 158.3^k > \frac{1}{2}\phi V_c = \frac{1}{2}(.75)(403.67) = 151.38^k$

USE CHAPTER 11 (ACI) DESIGN PROVISIONS

• CHECK MINIMUM HORIZONTAL (TRANSVERSE) SHEAR REINF. RATIO

$$\rho_t = \frac{A_v}{s h} \geq .0025$$

TRY (2) # 4

$$\frac{2(.2)}{(.0025)(10)} = s = 16''$$

• CHECK SPACING LIMITATIONS:

$$s \leq \text{minimum} \begin{cases} l_w/5 = 18 \times 12/5 = 43'' \\ 3h = 3(10) = 30'' \\ 18'' = * \text{ CONTROLS} \end{cases}$$

$s = 16'' < 18''$ OKAY ✓

$$\boxed{\text{USE (2) \# 4 @ 16'' (ONE ON EACH FACE) FOR HORIZONTAL REINFORCEMENT}}$$

• DETERMINE VERTICAL SHEAR REINFORCEMENT:

$$\rho_l = \frac{A_v}{s h} \geq .0025 + 0.5\left(2.5 - \frac{h_w}{l_w}\right)(\rho_t - .0025)$$

SINCE $h_w/l_w = 141/18 = 7.8 > 2.5 \rightarrow$ EQUATION WON'T GOVERN

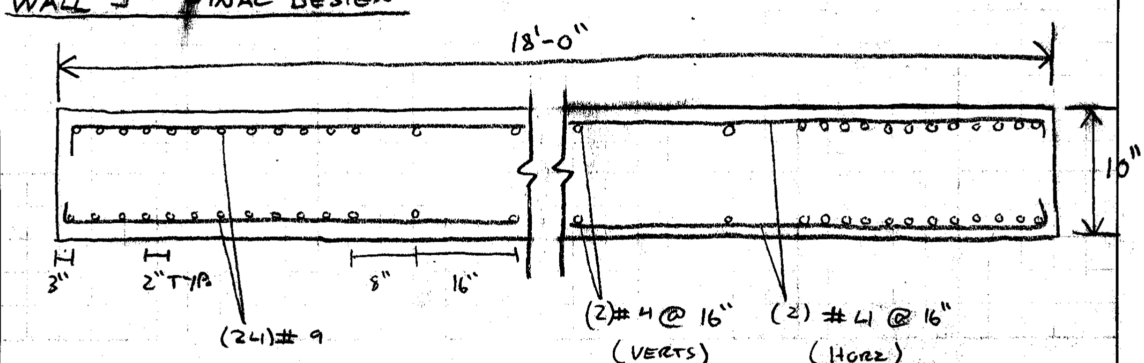
$$\rho_l \geq .0025$$

TRY (2) # 4 $\quad \frac{2(.2)}{.0025(10)} = s = 16''$

TRY (2) # 4 @ 16''

CHECK SPACING: $\quad s \leq \min \begin{cases} l_w/3 = 72'' \\ 3h = 30'' \\ 18'' = * \text{ CONTROLS} \end{cases}$ $\quad 16'' < 18''$ OKAY ✓

WALL 5 – FINAL DESIGN



WALL 6.

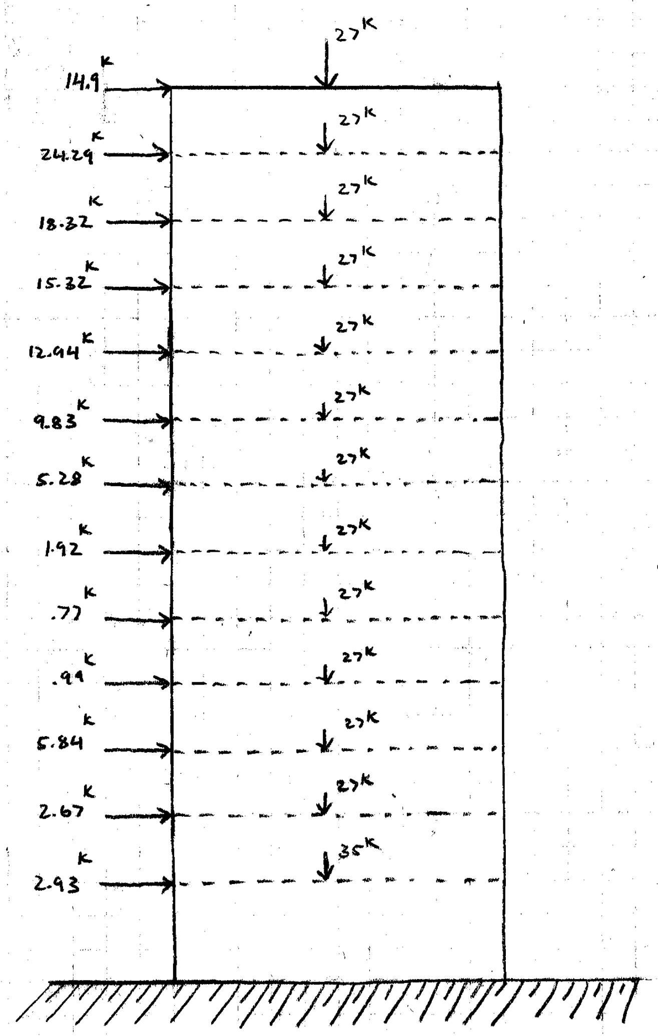
HEIGHT OF WALL $h_w = 141' = 1692''$
LENGTH OF WALL $L_w = 18' = 216''$
THICKNESS $h = 10''$

$\boxed{\text{STRENGTH BEHAVIOR}}$ – $h_w/l_w = 141/18 = 7.83 > 3$

∴ SHEAR WALL = "SLENDER OR FLEXURAL WALL"

STEP ONE: MODEL WALL W/ LOADS

AXIAL + FLEXURAL + SHEAR LOADS



STEP TWO: FLEXURAL DOMINATED SHEAR WALL, SO BEGIN BY DESIGNING FOR FLEXURE.

$$M_n = A_s f_y (d - a/2)$$

$C = T$: $.85 f'_c a b = A_s f_y$

LET $jd = 0.9d \approx 0.9(0.8 \ell_w) = 0.9(0.8 \cdot 16)$

$jd = 11.52' = 138.24"$

$M_u = \phi M_n = \phi A_s f_y jd$

$(12{,}446)(12{,}000) = .9(A_s)(60{,}000)(138.24)$

$A_s = 20 \text{ in}^2$

$.85 f'_c a b = A_s f_y$

$.85(4000)(a)(10) = (20)(60{,}000)$ $\quad a = 35.3"$

RECALCULATE jd:

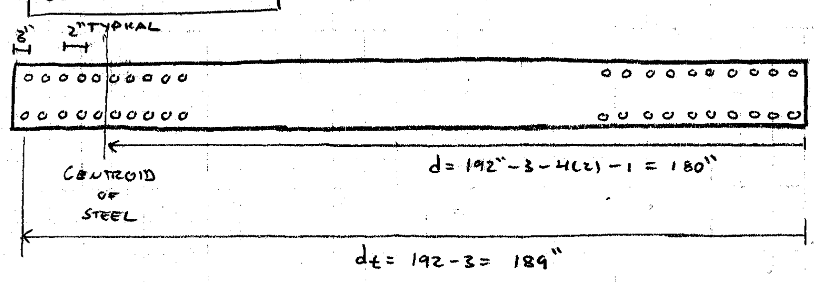
$$jd = d - a/2 = 153.6" - \frac{35.3}{2} = 135.95"$$

RECALCULATE REQUIRED A_s:

$(12{,}446)(12000) = 0.9(A_s)(60{,}000)(135.95)$

$A_{s\,REQ'D} = 20.34 \text{ in}^2$

TRY $\boxed{20 \ \#9 \text{ BARS}} \approx 20 \text{ in}^2$



$C = T$: $0.85 f'_c a b = A_s f_y$

$$a = \frac{(20)(60)}{.85(4)(10)} = 35.29"$$

$$c = \frac{a}{\beta} = \frac{35.29}{.85} = 41.5"$$

CHECK TENSION CONTROL

$$\varepsilon_t = \varepsilon_{cu}\left(\frac{d_t - c}{c}\right) > .005$$

$$\varepsilon_t = .003\left(\frac{189 - 41.5}{41.5}\right) = \boxed{.0106 > .005 \text{ OKAY}}$$

STEP THREE: DESIGN WALL FOR SHEAR $V_u = 116^{kips}$

• CHECK MAX PERMITTED SHEAR STRENGTH:

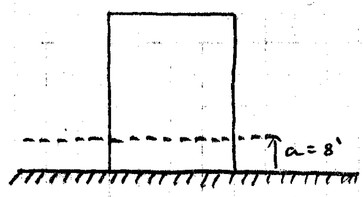
$V_u \le \phi V_{n max} = \phi 10 \sqrt{f'_c}\, hd$

$V_u = 116 \le .75 (10) \sqrt{4000}\, (10)\, (.8 \times 16 \times 12)\, (1/1000)$

$V_u = 116^k \le 728.6^k$ OKAY WALL DIMENSIONS ARE OKAY.

• DETERMINE LOCATION OF CRITICAL SHEAR SECTION:

$a \le \text{minimum} \begin{cases} l_w/2 = 16'/2 = 8' & * \text{ controls} \\ h_w/2 \quad 141'/2 = 70.5' \end{cases}$



• DETERMINE SHEAR STRENGTH CONTRIBUTION OF CONCRETE:

~ CONSERVATIVE EQUATION: $V_c = 2\sqrt{f'_c}\, hd = \dfrac{2\sqrt{4000}(10)(.8)(16\times12)}{1000}$

$V_c = 194^{kips}$

~ ALTERNATE EQUATIONS: (11-27)

$V_c \le 3.3\sqrt{f'_c}\, hd + \dfrac{N_u d}{4 l_w}$ $\qquad N_u = .9(27^k(12) + 35^k) = 323^k$

$V_c \le \dfrac{3.3\sqrt{4000}(10)(.8)(16\times12)}{1000} + \dfrac{323^k(.8)(16\times12)}{4(16\times12)}$

$V_c \le 385^{kips}$

• CHECK: $\dfrac{M_u}{V_u} - \dfrac{l_w}{2} < 0$

$\dfrac{12{,}446}{116} - \dfrac{141}{2} = 36.79 \not< 0$ ∴ EQ. 11-27 SHOULD CONTROL OVER EQ. 11-28 (BUT WILL CHECK)

$V_c = \left[.6\sqrt{4000} + \dfrac{192\left(1.25\sqrt{4000} + .2\dfrac{(323)}{192(10)}\right)}{36.79}\right] 10(.8)(192)\,(1/1000)$

$V_c = 692^{kips}$

STEP THREE: DESIGN WALL FOR SHEAR $V_u = 116^{kips}$

- CHECK MAX PERMITTED SHEAR STRENGTH:

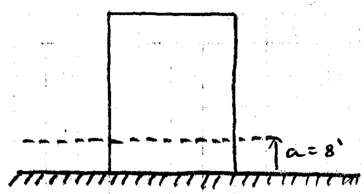
$$V_u \leq \phi V_{n max} = \phi 10 \sqrt{f'_c}\, h d$$

$$V_u = 116 \leq .75 (10) \sqrt{4000}\, (10) (.8 \times 16 \times 12) (1/1000)$$

$V_u = 116^k \leq 728.6^k$ OKAY WALL DIMENSIONS ARE OKAY.

- DETERMINE LOCATION OF CRITICAL SHEAR SECTION:

$$a \leq \text{minimum} \begin{cases} l_w/2 = 16'/2 = 8' & \text{* CONTROLS} \\ h_w/2 \quad 141'/2 = 70.5' \end{cases}$$



- DETERMINE SHEAR STRENGTH CONTRIBUTION OF CONCRETE:

~ CONSERVATIVE EQUATION: $V_c = 2\sqrt{f'_c}\, h d = \dfrac{2\sqrt{4000}\,(10)(.8)(16 \times 12)}{1000}$

$V_c = 194^{kips}$

~ ALTERNATE EQUATIONS: (11-27)

$$V_c \leq 3.3\sqrt{f'_c}\, h d + \frac{N_u d}{4 l_w} \qquad N_u = .9 (27^k (12) + 35^k) = 323^k$$

$$V_c \leq \frac{3.3\sqrt{4000}\,(10)(.8)(16 \times 12)}{1000} + \frac{323^k (.8)(16 \times 12)}{4(16 \times 12)}$$

$V_c \leq 385^{kips}$

- CHECK: $\dfrac{M_u}{V_u} - \dfrac{l_w}{2} < 0$

$\dfrac{12,446}{116} - \dfrac{141}{2} = 36.79 < 0$ ∴ EQ. 11-27 SHOULD CONTROL OVER EQ. 11-28 (BUT WILL CHECK)

$$V_c = \left[.6\sqrt{4000} + \frac{192\left(1.25\sqrt{4000} + .2 \dfrac{(323)}{192\,(10)}\right)}{36.79} \right] 10(.8)(192)\,(1/1000)$$

$V_c = 692^{kips}$

$V_c = 194^{kips}$

$\phi V_c = .75(194) = 145.5^{kips} > V_u = 116^{kips}$

NO CALCULATION OF V_s FOR SHEAR REINFORCEMENT REQUIRED.

• CHECK $V_u > \frac{1}{2}\phi V_c$

$V_u = 116 > \frac{1}{2}(.75)(194) = 72.75^k$ YES ✓

USE CHAPTER 11 PROVISIONS OF ACI CODE

• CHECK MINIMUM HORIZONTAL (TRANSVERSE) SHEAR REINF. RATIO

$$\rho_t = \frac{A_v}{Sh} > .0025$$

TRY (2) # 4 $\qquad \rho_t = \frac{2(.2)}{S(10)} > .0025$

$$\frac{2(.2)}{.0025(10)} = S = 16''$$

USE (2) #4 @ 16"

• CHECK SPACING LIMITATIONS:

$$S \le \text{minimum} \begin{cases} l_w/5 = 16\times12/5 = 38'' \\ 3h = 3(10) = 30'' \\ 18'' \ \text{* CONTROLS} \end{cases} \qquad 16'' < 18'' \ \text{OKAY ✓}$$

$\boxed{\text{USE (2) \# 4 (ONE ON EACH FACE) @ 16" SPACING FOR HORIZONTAL SHEAR REINFORCEMENT}}$

• DETERMINE VERTICAL SHEAR REINF.

$$\rho_l = \frac{A_v}{Sh} \ge .0025 + .5\left(2.5 - \frac{h_w}{l_w}\right)(\rho_t - .0025)$$

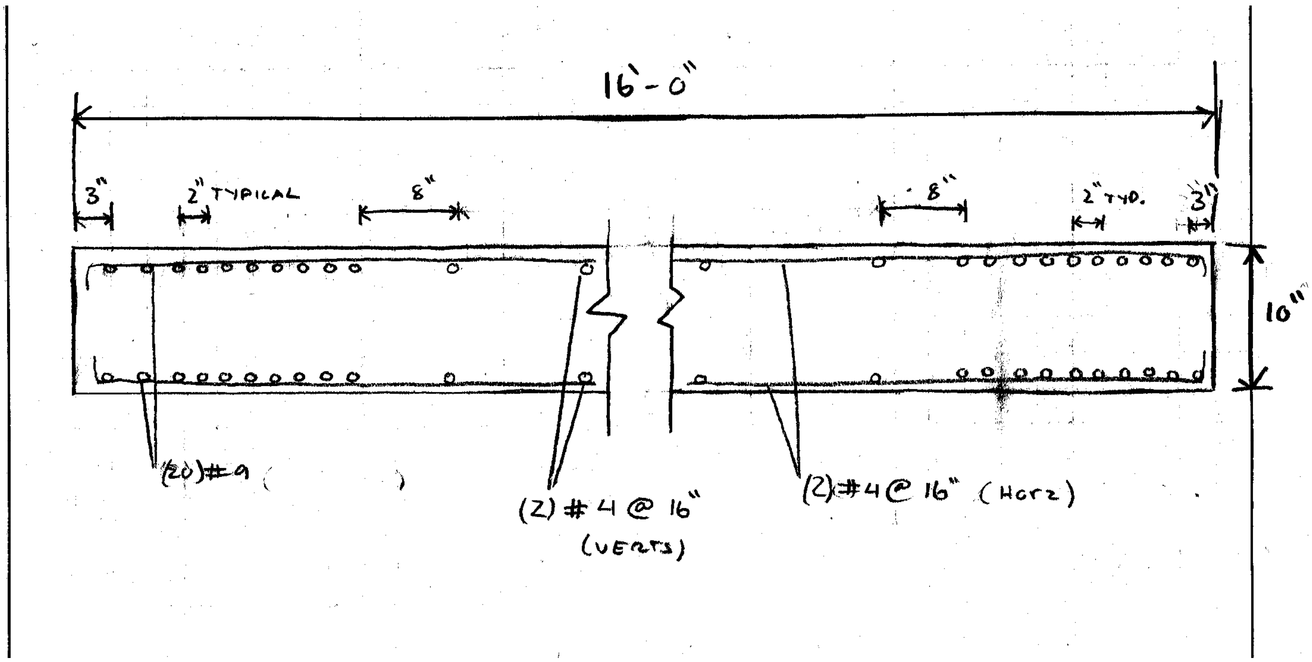
W/OUT CONTROL: $h_w/l_w > 2.5$

$$\therefore \rho_l = \frac{A_v}{Sh} \ge .0025 \qquad \text{minimum } S = \begin{cases} l_w/3 = 64'' \\ 3h = 30'' \\ 18'' \ \text{* CONTROLS} \end{cases} \ \text{OKAY ✓}$$

TRY (2) #4

$$\frac{2(.2)}{10(.0025)} = S = 16''$$

$\boxed{\text{USE (2) \# 4 @ 16" (one on each face) FOR VERTICAL SHEAR REINF.}}$


16'-0"
3"
2" TYPICAL
8"
8"
2" TYP.
3"
10"
(20)#9
(2) #4 @ 16"
(VERTS)
(2)#4@ 16" (HORZ)

WALL 7

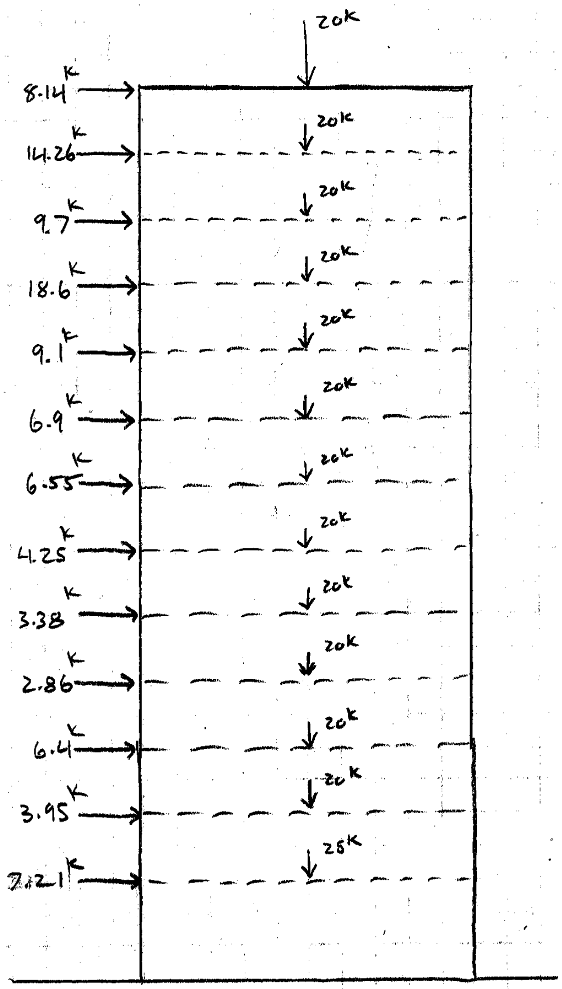
HEIGHT OF WALL $h_w = 141' = 1692''$
LENGTH OF WALL $l_w = 13' = 156''$
THICKNESS OF WALL $h = 10''$

$\boxed{\text{STRENGTH BEHAVIOR}}$ $h_w / l_w = 1692/156 = 10.8 > 3$

$\therefore$ SHEAR WALL = "SLENDER OR FLEXURAL WALL"

STEP ONE: MODEL WALL W/ LOADS

AXIAL + FLEXURAL + SHEAR LOADS



$N_u = [12(20) + 25]\,0.9 = 238.5^k$

$M_u = 9521.7^{k\text{-}'}$ $V_u = 101.3^k$

STEP TWO: DESIGN FOR FLEXURE 1ST, SINCE TALL SLENDER WALL

$M_n = A_s f_y (d - a/2) = A_s f_y \cdot jd$

$C = T$: $.85 f'_c ab = A_s f_y$

LET $jd = .9d = .9(.8 \ell_w) = .9(124.8) = 112.32''$

$M_u = \phi M_n = \phi A_s f_y jd$

$(9{,}521)(12{,}000) = .9(A_s)(60{,}000)(124.8)$

$A_s = 16.95 \text{ in}^2$

$.85 f'_c ab = A_s f_y$

$$a = \frac{(16.95)(60{,}000)}{.85(4000)(10)} = 29.9''$$

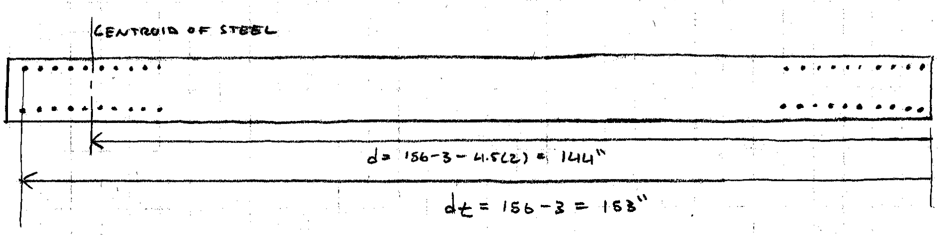
RECALCULATE jd: $jd = d - a/2 = 124.8 - \frac{29.9}{2} = 109.85''$

RECALCULATE A_s:

$(9{,}521)(12{,}000) = .9(A_s)(60{,}000)(109.85)$

$A_s = 19.2 \text{ in}^2$

TRY 20 #9 A_s PROVIDED $= 20 \text{ in}^2 > 19.2 \text{ in}^2$ OKAY ✓



$C = T$: $0.85 f'_c ab = A_s f_y$

$$a = \frac{(20)(60)}{.85(4)(10)} = 35''$$

$$c = \frac{a}{\beta} = \frac{35}{.85} = 41''$$

CHECK TENSION CONTROL

$$\varepsilon_t = \varepsilon_{cu}\left(\frac{d_t - c}{c}\right) > .005$$

$$.003\left(\frac{153 - 41}{41}\right) = .008 > .005$$

WALL 7

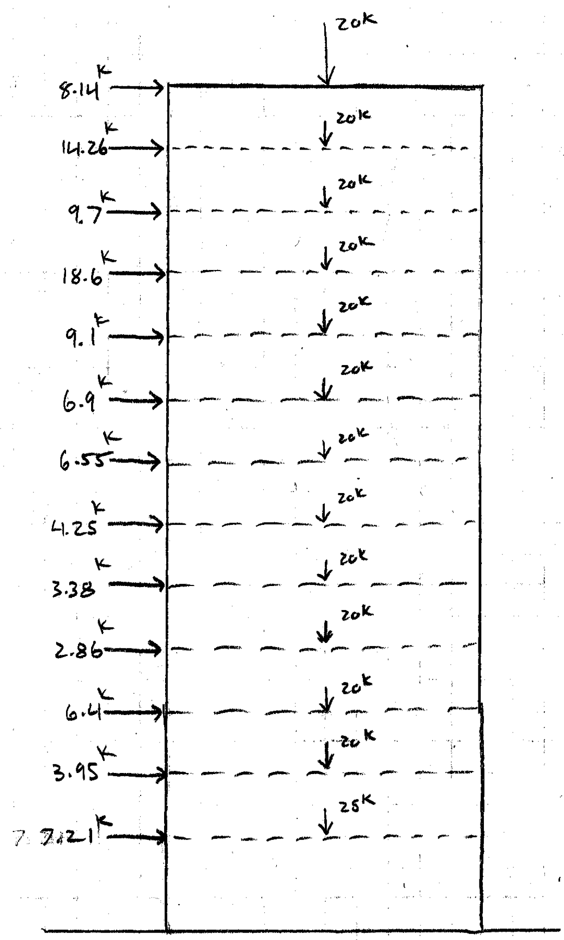
HEIGHT OF WALL $h_w = 141' = 1692''$
LENGTH OF WALL $\ell_w = 13' = 156''$
THICKNESS OF WALL $h = 10''$

$\boxed{\text{STRENGTH BEHAVIOR}}$ $h_w/\ell_w = 1692/156 = 10.8 > 3$

∴ SHEAR WALL = "SLENDER OR FLEXURAL WALL"

STEP ONE: MODEL WALL W/ LOADS

AXIAL + FLEXURAL + SHEAR LOADS



$N_u = [12(20) + 25]\,0.9 = 238.5^k$

$M_u = 9521.7^{k\text{-}1}$ $V_u = 101.3^k$

STEP THREE: DESIGN WALL FOR SHEAR: $V_u = 101.3$ KIPS

- CHECK MAX PERMITTED SHEAR STRENGTH

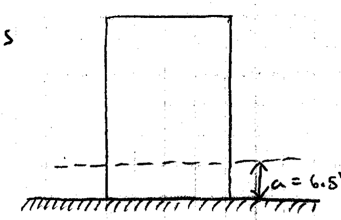
$$V_u \leq \phi V_{n\,max} = \phi 10\sqrt{f'_c}\, hd = \frac{.75(10)\sqrt{4000}(10)(.8)(13\times12)}{1000}$$

$$V_u = 101.3^k \leq 591.97^k$$

OKAY, wall dimensions are okay ✓

- DETERMINE LOCATION OF CRITICAL SHEAR SECTION

$$a \leq \text{minimum}\begin{cases} l_w/2 = 6.5' & * \text{ CONTROLS} \\ h_w/2 = 70.5' \end{cases}$$



- DETERMINE SHEAR STRENGTH CONTRIBUTION OF CONCRETE:

~ CONSERVATIVE EQUATION: $V_c = 2\sqrt{f'_c}\, hd$

$$2\sqrt{4000}\,(10)(.8\cdot 13\times12)\,(1/1000)$$

$$V_c = 158^k$$

$$\phi V_c = .75(158) = 118.5^k > V_u = 101.3^k$$

NO NEED TO CALCULATE V_s FOR SHEAR REINF. REQUIRED.

- CHECK $V_u > \frac{1}{2}\phi V_c$

$$101^k > \tfrac{1}{2}(.75)(158) = 59.25^k$$

∴ DESIGN ACCORDING TO ACI CHAPTER 11 PROVISIONS

- CHECK MINIMUM HORIZONTAL (TRANSVERSE) SHEAR REINF. RATIO

$$\rho_t = \frac{A_v}{sh} > .0025$$

TRY (2) #4

$$\frac{2(.2)}{s(10)} = .0025$$

$$s = 16$$

CHECK MINIMUM SPACING

$$s = 16 \leq \begin{cases} l_w/5 = 31.2'' \\ 3h = 30'' \\ 18'' = 18'' & * \text{ CONTROLS} \end{cases}$$

$s = 16 \leq 18$ OKAY ✓

$$\boxed{\text{USE (2) \#4 @ 16'' SPACING FOR HORIZONTAL REINFORCEMENT}}$$

- DETERMINE VERTICAL REINFORCEMENT

$$\rho_\ell = \frac{A_v}{s h} \geq .0025 + .5\left(2.5 - \frac{h_w}{\ell_w}\right)\left(\rho_t - .0025\right)$$

SINCE $h_w/\ell_w > 2.5$, EQUATION DOES NOT GOVERN

$$\therefore \quad \rho_\ell = \frac{A_v}{s h} \geq .0025$$

TRY (2) # 4 $\rightarrow$ $\dfrac{2(.2)}{(.0025)(10)} = s = 16"$

CHECK MINIMUM SPACING:

$$s \leq \text{minimum} \begin{cases} \ell_w/3 = 13 \times 12/3 = 52" \\ 3h = 30" \\ 18" \ \ast \text{ CONTROLS} \end{cases}$$

$16 \leq 18$ OKAY

$$\boxed{\text{USE (2) \# 4 (ONE ON EACH FACE) AT 16" SPACING FOR VERTICAL REINFORCEMENT}}$$

WALL 7 - FINAL DESIGN