The Harry and Jeanette Weinberg Center

Mercy Hospital Medical Office Building
Baltimore, MD

Structural System Analysis and
Post-Tensioned Concrete Redesign

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Architectural Engineering – Building
Structures Option

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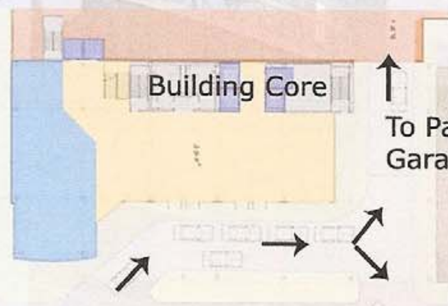
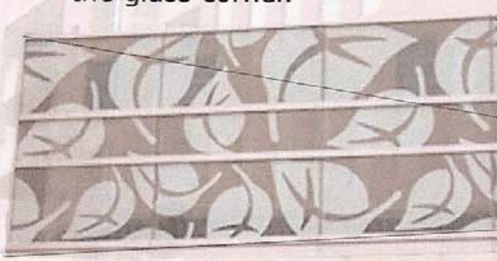
The Harry and Jeanette Weinberg Center at Mercy Hospital

Project Information
Location - Baltimore, MD
Owner - Mercy Medical Center
Architect - RTKL
Engineer
Structural - RTKL
MEP - RMF Engineering Inc.
General Contractor - Harkins Builders



Architecture
Use: Medical Office Building providing Outpatient Services
Stories: Six above grade, 1 below
Facade: Brick and glass curtain wall
Roof: Insulated metal deck with tar and gravel surface
Features: -Drive through patient drop off with access to parking garage
-Elevated walkway allows access to Mercy Medical Center across E. Saratoga Street
-An RTKL designed leaf motif on the glass corner.



Structural System
Type: Structural steel frame with slab-on-deck flooring utilizing composite beam action
Foundation: -Caissons that bear on bedrock
-A few spread footings
-Retaining wall system
Lateral Force System: 3 braced frames that enclose the building core
Features: -Lower level framing carries some lateral earth pressure
-Drive through is supported by steel beams

Mechanical System
Fire Suppression: Wet pipe sprinklers
HVAC System: -Steam is purchaced from Trigen, Chilled Water is purchaced from ComfortLink
-Steam/Chilled Water is distributed to air handling units located on each floor in the Building Core
-Variable Air Volume boxed distribute hot/cold air as needed

Electrical/Lighting Systems
Main Power: 13,000 Volt Dry Transformer provides The Weinberg Center with power
Lighting is run on 277 Volt grid
Motors are run on a 480 Volt grid
Emergency Generators provide power to building/elevators in case of a blackout

Kevin Clouser - Structural Option
www.arche.psu.edu/thesis/eportfolio/2007/portfolios/KDC153

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Executive Summary

The following report describes the investigation and redesign of the Harry and Jeanette Weinberg Center, Mercy Hospital's Medical Office Building, located in Baltimore, MD. The current structural system is made up of a steel frame composite action slab on deck and braced frames. Previous technical assignments and investigations showed that this steel structure works well for the current building conditions. For this project a zoning regulation change was imposed to limit the height of the building. This new condition would then require that a structural system be redesigned so that the building will meet the new height limitation that is 10 feet less than what is currently in place.

A post-tensioned one-way concrete slab and beam system was proposed for the redesigned structure. Through various consultations this system was singled out because of its abilities to have shallow floor depths and allow concrete to span relatively large distances. This would allow the current column grid layout to remain, insuring that architecturally the floor plan remains as is.

To accomplish this redesign Risa3D was used along with ACI 318, ASCE 7-02 and IBC 2003 code provisions. A slab depth of 8 inches was used and post tensioned to 24 kips/ft with ½" diameter tendons 1'-6" on center. A frame analysis was then performed to find worse case load conditions outlined in ASCE and IBC. The beams have (6) ½" diameter tendons providing a total of 173.4 kips of post-tensioning force. ACI code provisions for minimum required bonded reinforcement controlled the design of many of the beam sections; however, additional reinforcement is provided to enable the beams to work with the columns to resist lateral loads in a concrete sway frame. Columns were then designed using Risa3D which uses the PCA load contour method to determine worse case loadings and design reinforcement as required. After the ultimate strength design was completed, all structural members were then analyzed for serviceability requirements. Limitations were placed at 0.02Hsx for seismic story sway per ASCE 7-02 code requirements and an industry standard of H/400 for service wind loading. Gravity deflections of slabs and beams were compared to the industry standard of L/360.

Acoustical and lighting systems were then designed for a similar conference room to investigate for changes. It was determined that lowering the ceiling heights did indeed impact each of these systems. Reverberation times dropped below required limits and forced tile floors and wooden seats to be installed to bring the times up to an acceptable value of 0.96 seconds. Lighting spacing criteria was changed enough that additional lights would need to be added to provide adequate lighting for the room.

The current steel structure is estimated as costing $1.84 million and requiring 11960 labor hours while the redesigned structure is estimated at $1.9 million and requiring 26000 labor hours. These were both determined to be reasonable given that the percent of the structural system cost of the whole project went from 9.2% to 9.5% showing that the two systems are economically competitive. Also, given that the overall building height was reduced by more than 11 feet this report concludes that under the building height limitations a post-tensioned concrete system is a preferred alternative to steel construction.

Acknowledgements

Acknowledgements to the following people and organizations for their contributions and support in helping me to develop and complete this report.

Mercy Medical Center, Inc. specifically Ms. Judy Weiland

For allowing me to use The Harry and Jeanette Weinberg Center, Mercy Hospital Medical Office Building in my studies.

RTKL Associates, Inc. and Mr. Bob Knight

For their help in both answering questions and sponsoring me with building drawings that allowed me to complete my study.

Building Description

Building Description

The Harry and Jeanette Weinberg Center at Mercy Hospital is located at 227 St. Paul Place, Baltimore, MD 21202 (a few blocks from the Inner Harbor and National Aquarium). An elevated walkway connects Floor 2 of the Weinberg Center to Mercy Hospital across Saratoga Street. The Harry and Jeanette Weinberg Center at Mercy Hospital is designed to show Mercy as one of the nation's top healthcare facilities. Focusing on outpatient facilities, the Weinberg Center offers leadership in healthcare facilities and services by the nation's top physicians. Combining these in a building constructed with quality material and special attention to design allows for a relaxing environment well suited for healing. This puts The Weinberg Center at the forefront of outpatient services.

Of particular interest in the building's design are several key features. Mercy's nationally recognized women's health care center is located on the top floor and features an elegant entrance complete with skylights. The Weinberg Center was designed to coordinate with the central business district, where it is located. At the same time it stands apart with its glass facade corner that directs people to its entrance. Located off the main street is a drive-through entrance where patients exit vehicles and the driver can then park their vehicle in the parking garage located behind the Weinberg Center.

The Weinberg Center includes several different types of exterior wall systems. A brick facade with strip windows covers most of the building's exterior. A glass facade is used at the main entrance and is adorned with an RTKL designed leaf motif that is associated with Mercy Medical Center. Glass is also used along both sides adjacent to the entrance, which includes the drive-through. An architectural metal panel system is used on the top floor.

The roofing system is comprised of two main roof assemblies. A metal slab with 4-3/4" insulation, fireproofing, water protection, and an aggregate surface is used on most of the roof. In a few areas concrete slab replaces the metal decking and fire-proofing. In both cases the roof has a fire rating of one hour.

Primary Project Team

Owner:	Mercy Medical Center
	Website: www.mdmercy.com
Architect:	RTKL Associates, Inc.
	www.rtkl.com
Structural Engineer:	RTKL Associates, Inc.
Mechanical Electrical and Plumbing Engineer:	RMF Engineering, Inc.
	www.rmf.com
General Contractor:	Harkins Builders
	www.harkinsbuilders.com

Dates of Construction:	October 2002 through June 2004
Project Delivery Method:	Design-Bid-Build
Estimated Cost:	$20 Million

Codes, Regulations, and Standards

- 1997 Baltimore City Building Code
- Maryland Building Performance Standards as amended effective 7 April 1997
- 1996 BOCA
- 1996 National Electrical Code
- 1996 International Mechanical Code
- 1996 National Standard Plumbing Code
- CABO Model Energy Code MEC 95 1301.1
- 1996 National Fuel Gas Code
- 1997 Baltimore City Fire Prevention Code
- Fire Suppression NFPA 13 – 1994 Edition and Fire Detection Alarm NFPA 72 – 1993 Edition
- Elevators: ASME A17.1, 1996 Edition with Latest Amendments
- Accessibility: CABO/ANSI 117.1 1992 edition, more stringent of requirements as referenced from BOCA Chapter 11, and ADA.

Building Systems

Lighting/Electrical System

The electricity of The Weinberg Center is provided by one 13,000 volt dry transformer. The lighting system is run from a 277 volt grid while all motors are supplied from a 480 volt system. Emergency generators are present to supply backup power.

Mechanical System

The mechanical system for The Weinberg Center is made up of air handling units located on each floor. These units are fed steam or chilled water in order to heat or cool the building. The steam and chilled water are piped from production plants that supply downtown Baltimore. Steam is purchased from Trigen, soon to be Johnson Controls. Chilled water is produced at ComfortLink by adding large quantities of ice to very large vats of water. The air handlers on each floor use the piped-in water to heat or cool air simultaneously and then distribute the conditioned air to variable air volume (VAV) boxes which mix the hot and cool air to desired levels before releasing the air into the building interior.

Fire Protection

The Weinberg Center follows 1994 NFPA 13 fire code. Columns have a two hour fire rating, beams and slabs have a 1-1/2 hour fire rating and roof construction has a one hour fire rating. The fire suppression system provided is a wet-pipe system that allows for conversion to a pre-action valve controlled zoned system

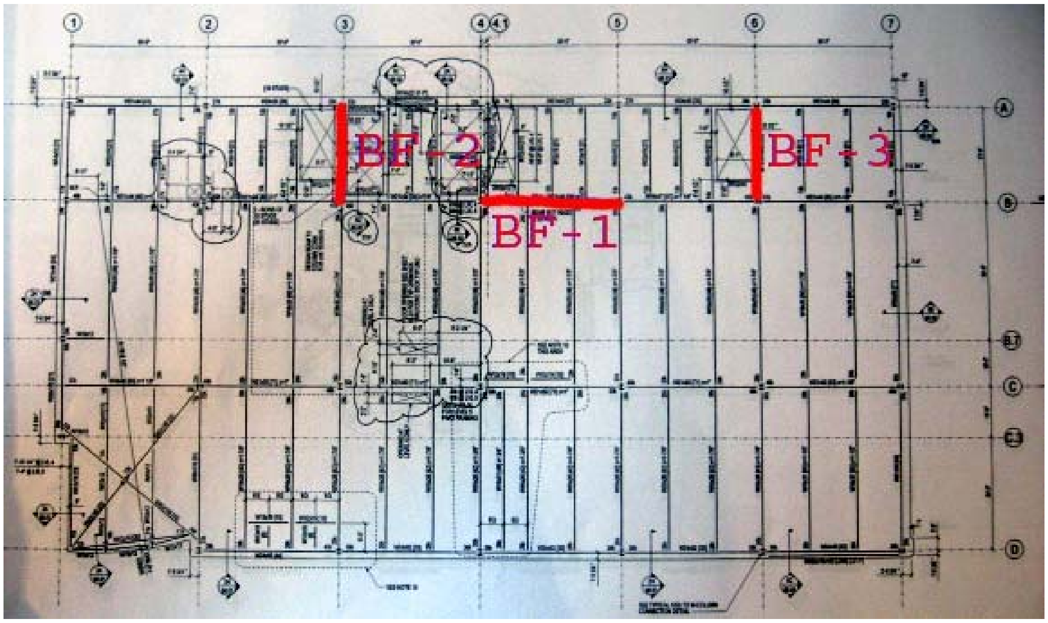
Transportation

Vertical transportation is achieved through four elevators located at the building core. There are two stairwells that run the full height of the building located at either end of the building core. A grand staircase runs from Floor 1 to Floor 2 through the atrium. Access across the elevated walkway is also located on Floor 2. Floor 1 provides access to the street level, where cars can drive through the patient drop-off and into the parking garage.

Existing Structural System

Existing Structural System

The structural system of the Weinberg Center spans six floors above grade and one below grade. Floor-to-floor heights are 20'-0" from the basement to lLvel 1, 14'-0" for Level 1 through Level 5 and 15'-0" for Level 6. The building is constructed using steel framing with composite action slab on deck. The columns are set on a maximum bay size of 30'-0" North-South (N-S) by 40'-0" East-West (E-W). Lateral forces are resisted by braced frames located at the building's core. All structural steel is A572 Grade 50. The existing floor plan is shown in the figure below. A better floor plan can be found in the structural system analysis and redesign, Figure 1.1.



Typical Existing Structural Floor Plan

Foundation

The foundation is composed of straight shaft drilled caissons, spread footing, slab-on-grade, and a concrete retaining wall along the west elevation. Caissons bear on rock and on average are 54'-0" deep. A bearing capacity for the caissons is set at 90 kips per square foot (ksf). Spread footings are all 12" thick. Assumed bearing pressures for spread footings and slab-on-grade is 2.0ksf. Slab-on-grade is divided into quadrants between column areas and is typically 6" thick with a maximum thickness of 10" in the North-West corner. The concrete retaining wall is 15" thick, 22'-0" high and carries minimal loads from the floor above.

Summary of foundation materials:
Caisson diameters: 3'-6" 4'-0" 4'-6" 7'-0"
Spread footings dimensions: 4'-0"x4'-0" 4'-6"x4'-6" 5'-6"x5'-6"
Deformed Bar Reinforcing Strength: fy=60ksi

Concrete Strengths	f 'c
Drilled Caissons:	3500psi
Spread Footings:	3000psi
Walls & Piers:	4000psi
Slab-on-grade:	3000psi

Columns

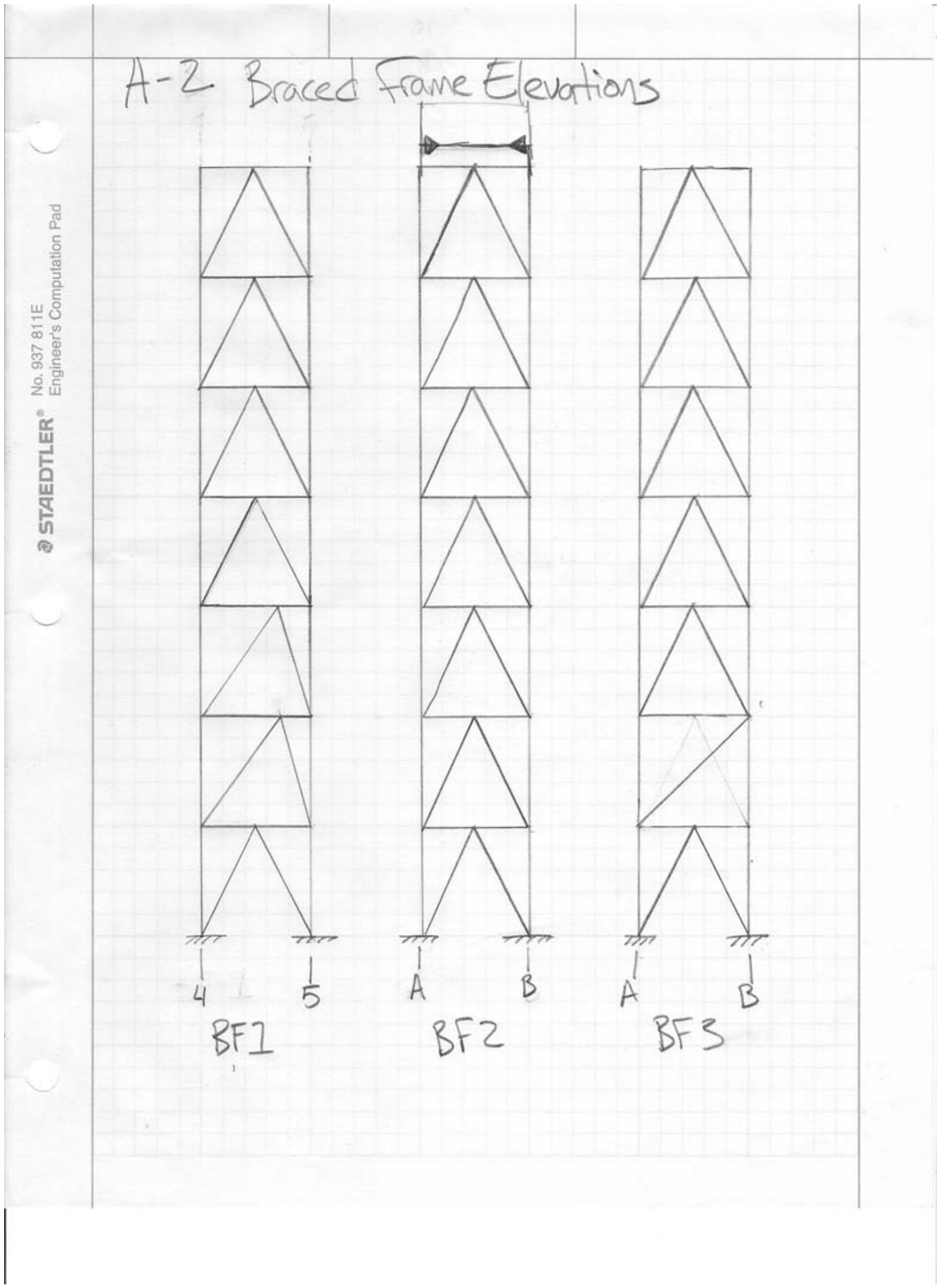
The columns of the Weinberg Center are all W14 shapes. They range in size from a W14x24 at the penthouse level to W14x283 in the basement. Columns are typically spliced at Floor 1, Floor 3 and Floor 5. The longest columns are 29'-1" tall and are located on the top floors. All columns are ASTM A572 GR50.

Floor System

The floor system is constructed of simply supported girders (typical sizes are W21x50 or W21x44) that span 30'-0" column to column in the N-S direction and simply supported infill beams (typical sizes are W16x26 and W18x35) span 35'-0", 40'-0" and 21'-0" at 10'-0" on center in the E-W direction. Infill beams that span more that 30'-0" are cambered upward in the middle by 1-7/8". Girders that span 30'-0" are cambered up in the middle by 1" to 1-1/8". A one-way slab-on-deck utilizing composite action is used to carry floor loads to the beams. The slab is 3.25" lightweight concrete (strength f'c=3000 psi) on a 2"-20 gage deck with 6x6-W1.4xW1.4 welded wire fabric. The maximum span for the slab on deck is 10'-0", the typical beam spacing. The main lobby on Floor 1 is two stories high so Floor 2 runs only around the North, West and South walls. The glass/aluminum corner is framed out by running a diagonal beam to truncate the corner, and cantilevering beams off the diagonal to the facade. The cantilevered beams are moment-connected into the diagonal girder; opposite the cantilevered beams is another moment-connected beam tying into the structural system to balance any torsion effects (See previous figure for typical framing plan). All structural steel is fy=50ksi while all plates and angles are fy=36ksi steel. The roof is framed out in a similar manner as the floors except that none of the roof beams are cambered and the majority of them are not composite action. The roof girders range from a W21x44 to a W24x62 while the beams range from W16X26 to W18x40. The high roof framing for the glass/aluminum corner is more simplified than the floor framing and composes of W14 and smaller shapes.

Lateral Force Resisting System

The lateral force resisting system is composed of three braced frames that run the entire height of the building around the building core. Four smaller braced frames are located at the top of the glass/aluminum corner, and a few moment frames are located at the penthouse level. The three main frames are chevron-braced with the exception of one diagonal brace. Two of the braced frames carry lateral load in the E-W direction while the remaining braced frame carries the load in the N-S direction. The load is distributed to the braced frames through the framing on each floor. Elevations of the three braced frames are shown in the following image.



Problem Statement and Solution Overview

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Problem

Local zoning regulations have been changed to limit the overall building height of the Harry and Jeanette Weinberg Center. Given these regulations, a new structural system must be designed that will reduce the buildings height by ten feet, limiting it to 75'-0" above the street level or less. The current height is 84'-4" and the existing structural steel building has been deemed too tall to work within the zoning regulation.

Solution Overview

The solution to this problem will be solved by investigating whether a concrete structure would have been better for the Weinberg Center project with limitations placed on the overall building height. The structural system for the Weinberg Center will be redesigned with a height limitation set to 12'-0" floor-to-floor height. This height limitation will only go into effect after Floor 2 since existing elevations must be maintained on Floors 1 and 2. These elevations must be kept to allow for the continued access to the street, parking garage and the elevated walkway that make the Weinberg Center unique and functional.

A post-tensioned concrete slab and beam system will be designed. Beams will have a depth of approximately 24". Post-tensioning will be added to the slab and beams in order to minimize thicknesses and allow the installation of Mechanical, Electrical and Plumbing (MEP) equipment in the cavity above the suspended ceiling. This system will allow roughly 2'-0"+/- for MEP equipment. This depth has been determined through consultations with peers to be sufficient to allow the installation of MEP equipment. Approximately two feet is not ideal for the installation for MEP equipment, however, the circumstances of the redesigned structure are less than ideal. Height limitations often sacrifice ideal construction for what must be done to ensure an end product that meets the owner's needs.

The proposed alternate system will have a height restriction of 12'-0" floor-to-floor for Floors 2 through 6. The intent of this design is two-fold. First, it is intended to allow an investigation of how height restrictions impact the design of a building. Second, it is of interest to determine if a concrete system would have been an economically competitive structure if local zoning regulations had limited the height of the Weinberg Center. Height restrictions in zoning regulations can adversely affect the design and economics of building projects.

Post-Tensioned Concrete Structural System Redesign

Structural System Analysis and Redesign

Materials used in the analysis and redesign are outlined in Table A. The selection of a majority of the materials used is based on industry standards and is commonly used in similar construction projects. A higher strength concrete was chosen for use throughout the entire building. Reasons for this decision are discussed in more depth later but it is sufficient for now to say that increasing the strength of concrete to 6000psi for the slab and beams was done to help control shear and flexural strength design.

Table A. Structural Component Materials

Structural Component	Material
Slab	6000psi Light Weight Concrete (LWC)
Beam	6000psi LWC
Columns	6000psi LWC
Reinforcing Steel	ASTM Grade 60
Post-Tensioning Tendons	Unbonded Seven-Wire Strand, f_{pu} = 270ksi

Gravity System

The existing structural steel framing system was replaced with a cast-in-place post-tensioned concrete system. A post-tensioned system was chosen in order to limit the floor-to-floor heights to meet the imposed zoning regulations. The redesigned systems column grid is shown in Figure 1.1 while an overall building elevation shown in Figure 1.2. The existing column grid was altered slightly, but essentially kept as is so that the existing architectural layouts could remain to provide the building owner and tenants with an open workspace for the buildings interior.

There are seven beam frames that support the 6-30 foot spans of slab. Each frame is seven stories high, including the roof, and three bays wide. The beam spans are not equal, but instead run continuously across 35 foot, 40 foot and 21 foot spans. Post-tensioning was used to minimize the depth of beams and the slab and to control deflection across the long spans. A span/depth ratio of twenty was used to obtain initial depths for each beam. A span/depth ratio of 45 was used as a starting point for the slab design.

Loads used in the design of the gravity system are described in Table B. Beams along the exterior face of the building carry the additional 180plf line load from the building's facade system. A complete list of load cases solved for using Risa3D is provided in Appendix E.

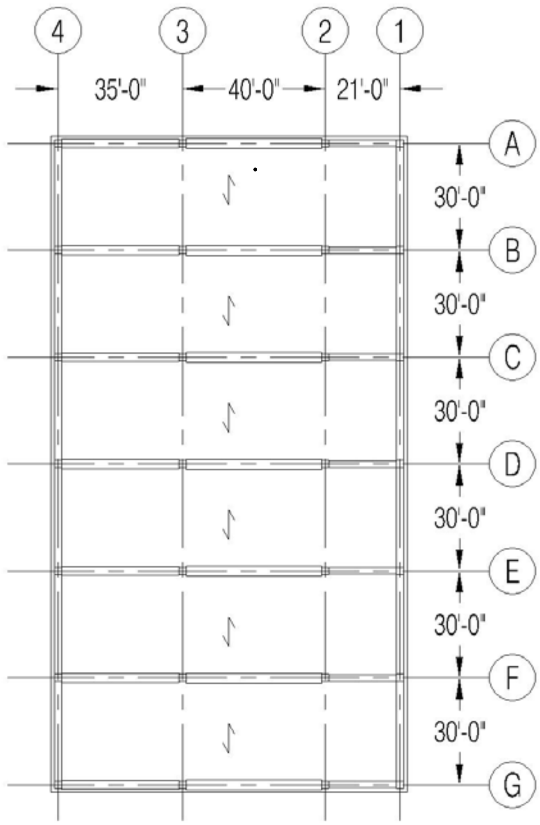


Figure 1.1

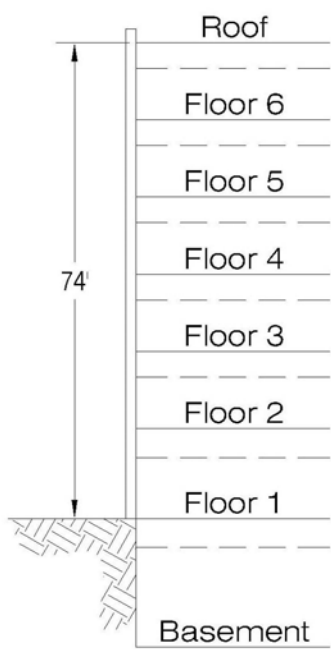


Figure 1.2

Table B: Gravity Loads		
	Superimposed Dead Load	Live Load
Floor 1-6	15 psf	100 psf
Roof	8 psf	30 psf
Wall Dead Load	15psf = 180plf line load	

Slab

The slab was designed and used in a frame analysis utilizing Risa3D to complete the design of the rest of the building's structure and check loadings on the slab. Using notes and design guides for post-tensioned slabs the following slab design was determined to be sufficient to carry gravity loads. In analyzing the slab and by discussing the results with faculty and peers it was determined that using a high strength concrete would allow a higher safety margin for shear and flexural strengths. Concrete with a 28 day compressive strength of 6000 pounds-per-square-inch (psi) was selected. Higher strength concretes were considered, but when cost data for the different strengths was compared it was determined that 6000psi concrete does not cost much more than 3000psi or 4000psi and can still provide the extra strength needed to design a small floor depth gravity system. Concrete over 6000psi compressive strength becomes progressively more expensive and uneconomic to gain additional strength.

A section of the redesigned floor slab is shown in Figure 2. A total slab thickness of eight inches is used. This thickness allows the rated fire protection of 1-1/2 hours to be maintained with the proper clear cover to bonded reinforcing and post-tensioning tendons. Bonded reinforcing has a required ¾" clear cover while the post-tensioning tendons have a required 1" clear cover. Post-Tensioning cables ½" in diameter are spaced at 1'-3" on center for exterior slab spans and 1'-8" on center for interior slab spans. These cables would then be tensioned to a required force of 33 kips (.8fpuAps). Additional bonded reinforcement of #4 bars at 12" on center is provided, per ACI code requirements, at the top face of the slab over supports and along the bottom face of the slab clear spans.

The prestressing forces in the concrete slab, after tensioning has been applied, comply with Class U flexural members. The service load prestressing stresses are 374psi tensile at the bottom of the slab and 920psi compressive at the top. This ensures that the full section uncracked properties can be used in design and deflection calculations. Detailed calculations can be found in Appendix A.

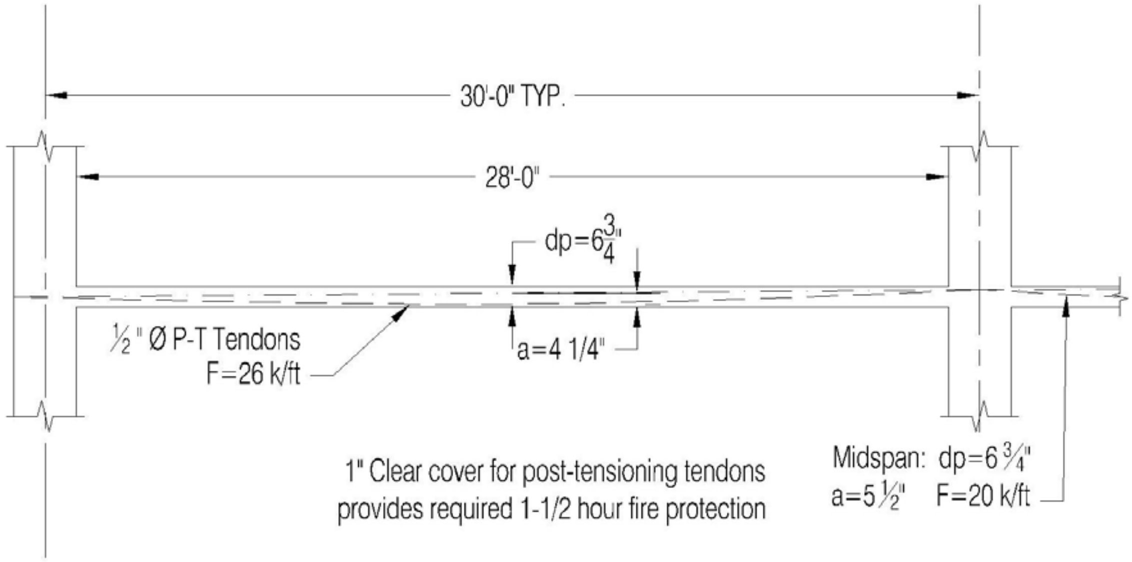


Figure 2

Beams

Beams were designed by modeling frames in Risa3D and finding worse case member loads. Each frame was modeled separately and denoted using the column grid lines described in Figure 1. Elevations of the frames used in the gravity system are given in Figures 5.1 through 5.7. Frames 1 through 4 will be discussed along with the lateral system design since they were controlled by lateral force analysis. In general, beams of equivalent length were designed to be similar. Thirty-five foot beams are 24" deep by 30" wide, 40 foot beams are 26" deep by 34" wide and 21 foot beams are 18" deep by 22" wide. Widths do not include the 8" slab that allows the beams to be designed as T-beam sections.

A total of six unbonded post-tensioning tendons are used in the design of each beam. Post-tensioning tendons are ½" diameter and tensioned to a required force of 28.9 kips per tendon. Beams have been designed as Class U flexural members with service load prestressing stresses summarized in Table C. Prestressing design calculations can be found in Appendix A. A profile of each beam's post-tensioning tendons is shown in Figures 3.1 through 3.3. Sections showing bonded reinforcing layouts of typical beams are provided in Figures 4.1 through 4.6. Bonded Reinforcing Schedules are provided in Tables D.1 through D.7. Bonded reinforcement was mostly controlled by ACI's minimum required bonded reinforcement since the unbonded post-tensioning tendons provide significant load carrying capacity to resist ultimate load design. Several of the beams have large enough factored moments to need the addition of more than the minimum bonded requirement and have been designed to handle the higher loads. Appendix B contains Tables for each beams factored loads and a design calculation example for flexural reinforcement. Number 4 reinforcing bars were used in the design of shear for the beams. Shear reinforcing schedules are given in Tables D.8 through

D.14. Values described in the shear reinforcing schedules are given as number of bars at a particular spacing in inches.

Table C: Summary of Service Load Stresses				
	Extreme Fiber Stresses (psi)			
Beam	Positive Moment		Negative Moment	
	Compression	Tension	Compression	Tension
21 foot	-331	365	-1376	414
35 foot	-344	459	-1160	364
40 foot	-335	441	-1023	336
Allowable	-2700	465	-2700	465

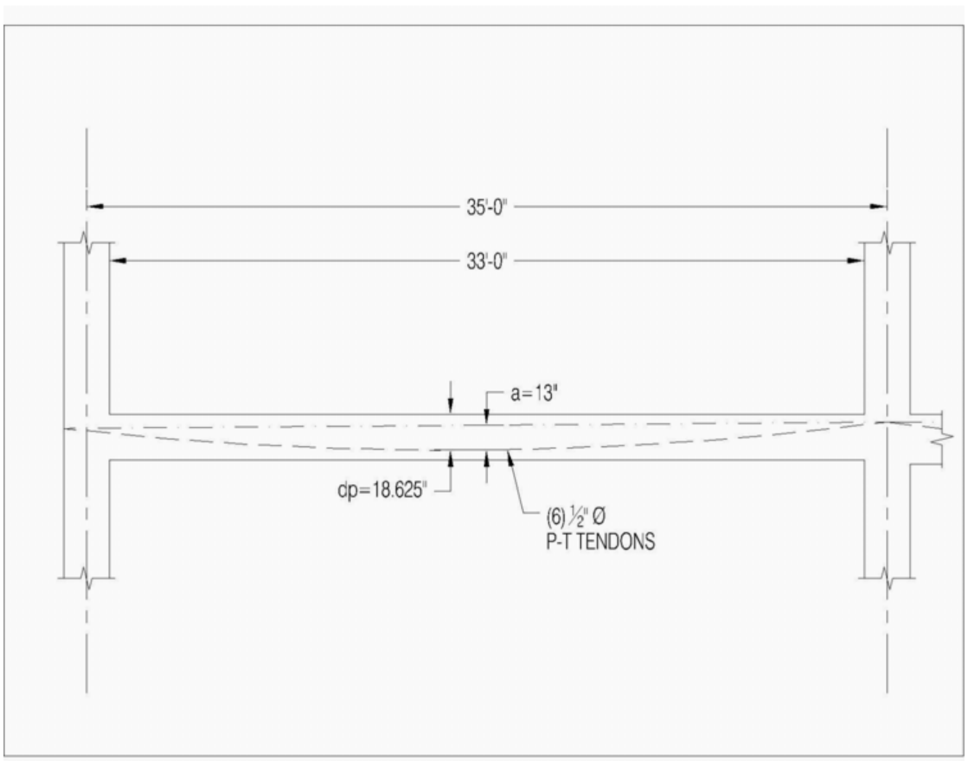


Figure 3.1 35' Beam Post-Tensioning Tendon Profile

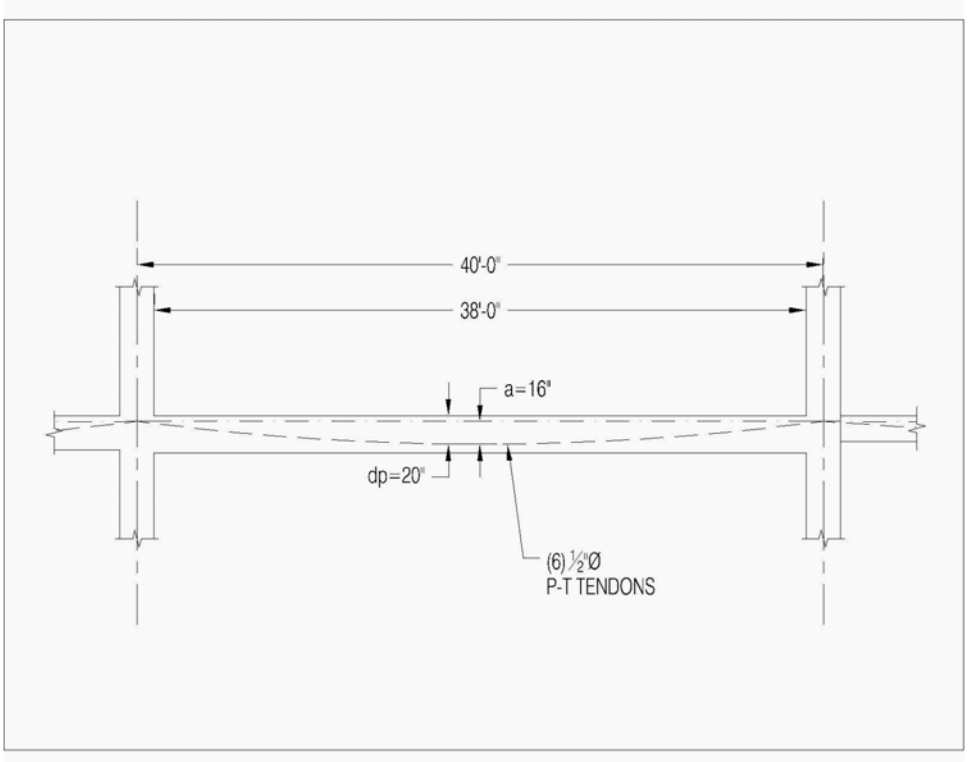


Figure 3.2 40' Beam Post-Tensioning Tendon Profile

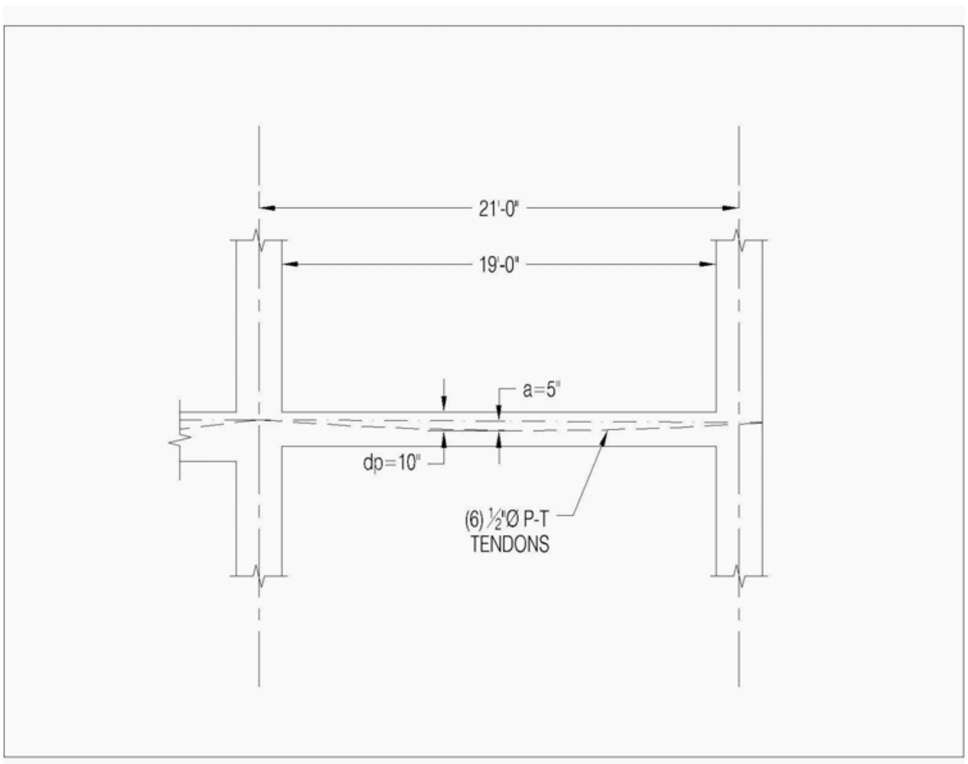


Figure 3.3 21' Beam Post-Tensioning Tendon Profile

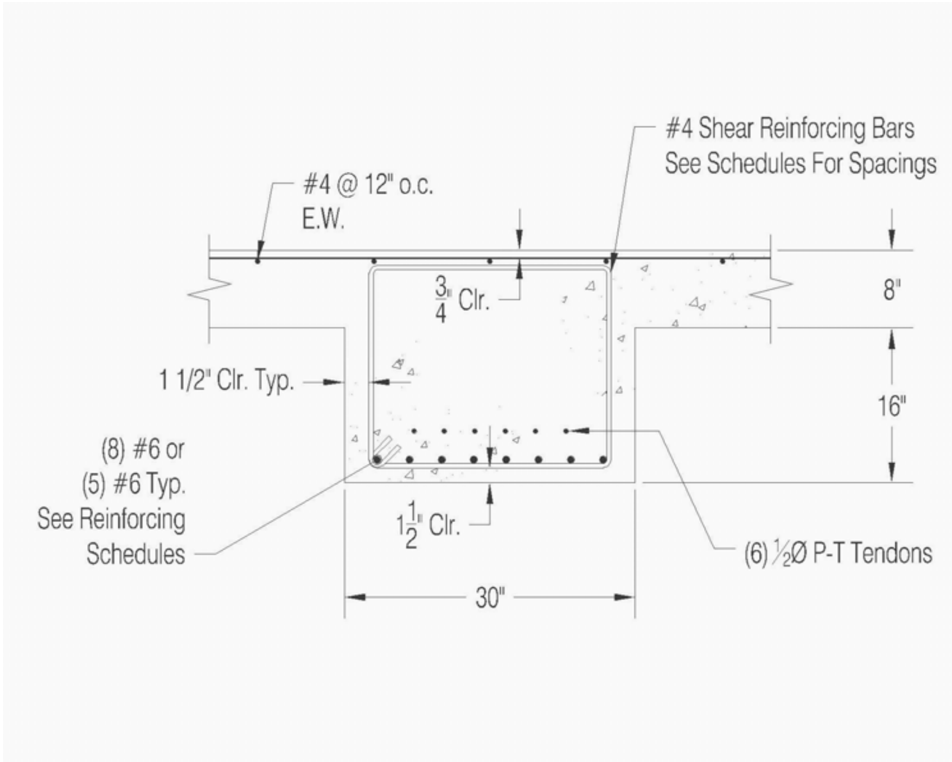


Figure 4.1 35’ Beam Midspan Reinforcement

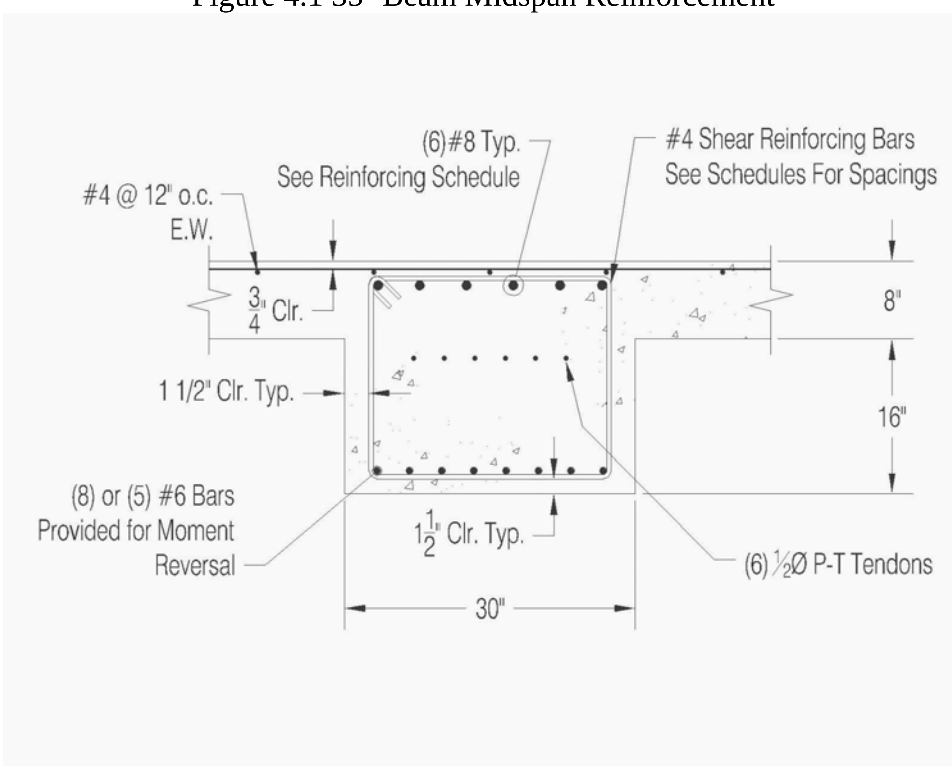


Figure 4.2 35’ Beam Support Reinforcement

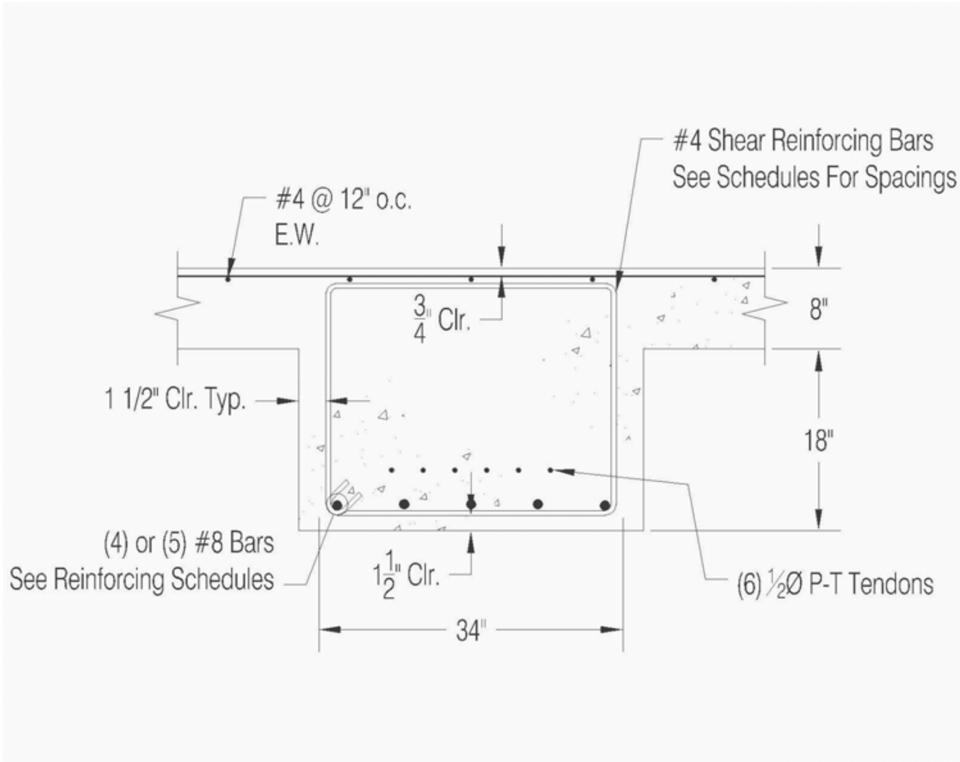


Figure 4.3 40’ Beam Midspan Reinforcement

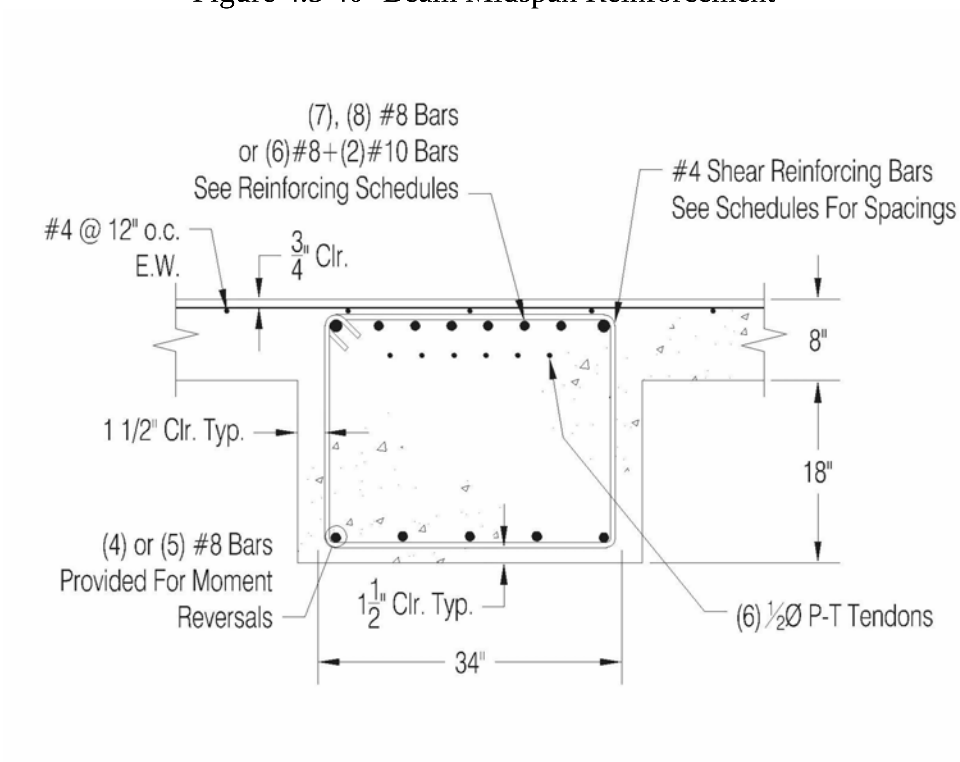


Figure 4.4 40’ Beam Support Reinforcement

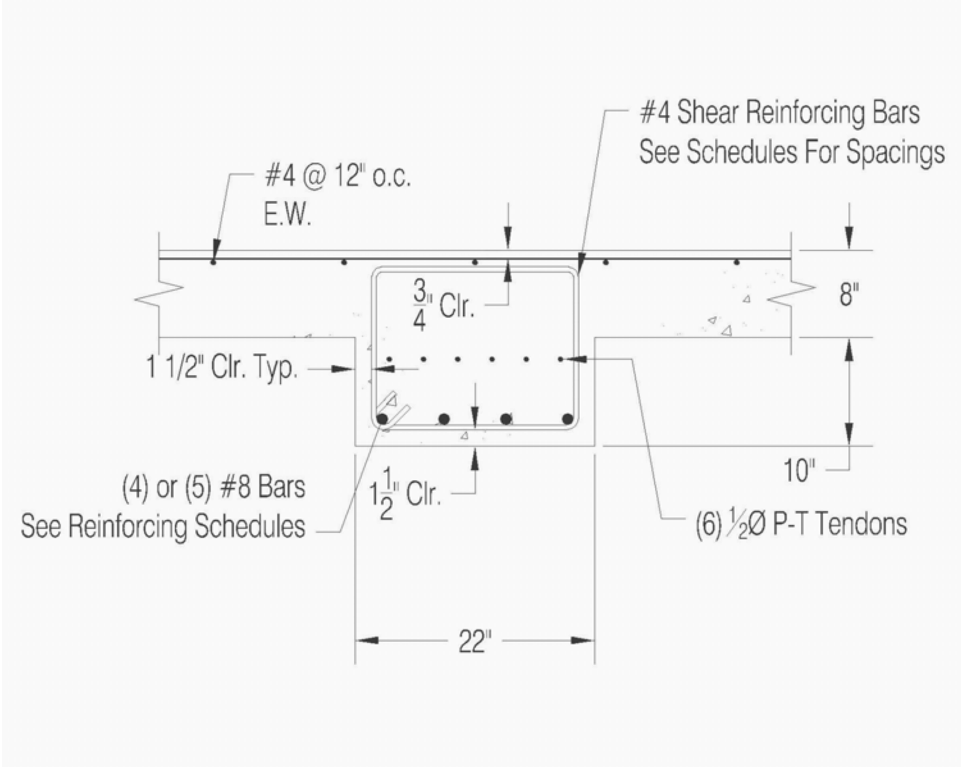


Figure 4.5 21’ Beam Midspan Reinforcement

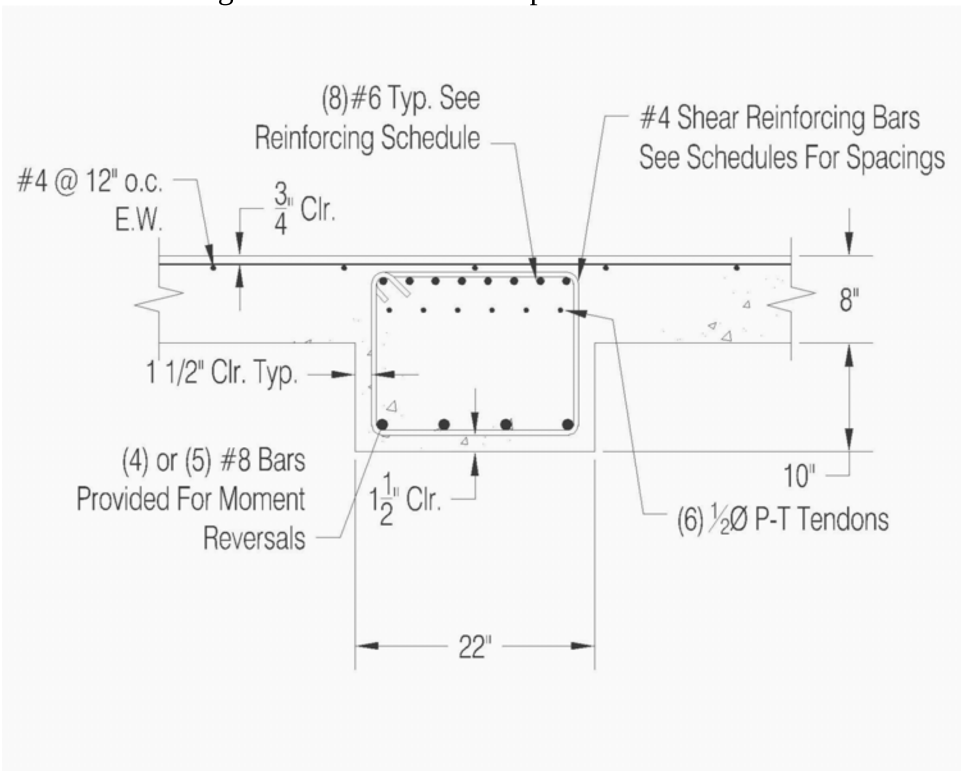


Figure 4.6 21’ Beam Support Reinforcement

Figure 5.1 Frame A

Figure 5.2 Frame B

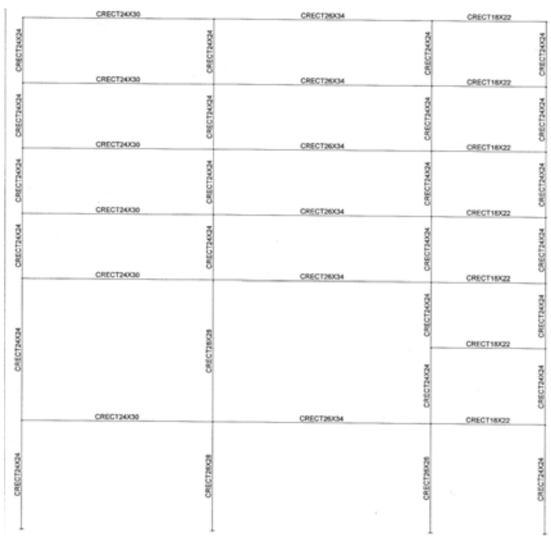


Figure 5.3 Frame C

Figure 5.4 Frame D

Figure 5.5 Frame E

Figure 5.6 Frame F

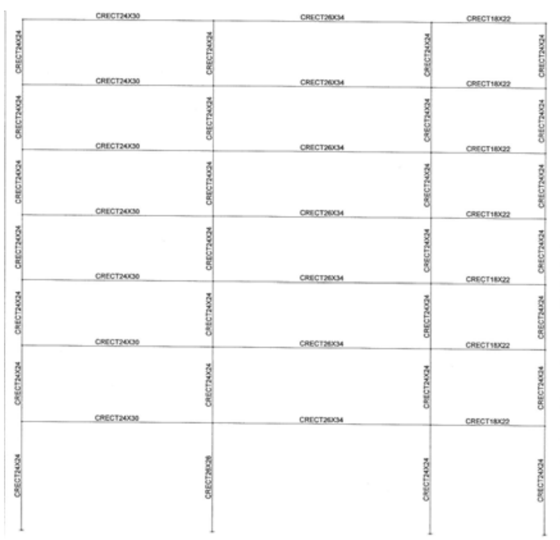


Figure 5.7 Frame G

Table D.1: Frame A Bonded Reinforcing Bars									
	35' Beam			40' Beam			21' Beam		
	Left	Mid	Right	Left	Mid	Right	Left	Mid	Right
Roof	(6) #8	(5) #6	(6) #8	(7) #8	(4) #8	(7) #8	(8) #6	(4) #8	(8) #6
6	(6) #8	(5) #6	(6) #8	(7) #8	(4) #8	(7) #8	(8) #6	(4) #8	(8) #6
5	(6) #8	(5) #6	(6) #8	(7) #8	(4) #8	(7) #8	(8) #6	(4) #8	(8) #6
4	(6) #8	(5) #6	(6) #8	(7) #8	(4) #8	(7) #8	(8) #6	(4) #8	(8) #6
3	(6) #8	(5) #6	(6) #8	(7) #8	(4) #8	(7) #8	(8) #6	(4) #8	(8) #6
2	(6) #8	(5) #6	(6) #8	(7) #8	(4) #8	(7) #8	(8) #6	(4) #8	(8) #6
1	(6) #8	(5) #6	(6) #8	(7) #8	(4) #8	(7) #8	(8) #6	(4) #8	(8) #6

Table D.2: Frame B Bonded Reinforcing Bars									
	35' Beam			40' Beam			21' Beam		
	Left	Mid	Right	Left	Mid	Right	Left	Mid	Right
Roof	(6) #8	(5) #6	(6) #8	(7) #8	(4) #8	(7) #8	(8) #6	(4) #8	(8) #6
6	(6) #8	(8) #6	(6) #8	(7) #8	(5) #8	(7) #8	(8) #6	(4) #8	(8) #6
5	(6) #8	(8) #6	(6) #8	(7) #8	(5) #8	(7) #8	(8) #6	(4) #8	(8) #6
4	(6) #8	(8) #6	(6) #8	(8) #8	(5) #8	(7) #8	(8) #6	(4) #8	(8) #6
3	(6) #8	(8) #6	(6) #8	6#8+2#10	(5) #8	(7) #8	(8) #6	(4) #8	(8) #6
2				6#8+2#10	(5) #8	(7) #8	(8) #6	(4) #8	(8) #6
1	(6) #8	(8) #6	(6) #8	6#8+2#10	(5) #8	(7) #8	(8) #6	(4) #8	(8) #6

Table D.3: Frame C Bonded Reinforcing Bars									
	35' Beam			40' Beam			21' Beam		
	Left	Mid	Right	Left	Mid	Right	Left	Mid	Right
Roof	(6) #8	(5) #6	(6) #8	(7) #8	(4) #8	(7) #8	(8) #6	(4) #8	(8) #6
6	(6) #8	(8) #6	(6) #8	(7) #8	(5) #8	(7) #8	(8) #6	(4) #8	(8) #6
5	(6) #8	(8) #6	(6) #8	(7) #8	(5) #8	(7) #8	(8) #6	(4) #8	(8) #6
4	(6) #8	(8) #6	(6) #8	(8) #8	(5) #8	(7) #8	(8) #6	(4) #8	(8) #6
3	(6) #8	(8) #6	(6) #8	6#8+2#10	(5) #8	(7) #8	(8) #6	(4) #8	(8) #6
2							(8) #6	(4) #8	(8) #6
1	(5) #8	(8) #6	(6) #8	6#8+2#10	(5) #8	(7) #8	(8) #6	(4) #8	(8) #6

Table D.4: Frame D Bonded Reinforcing Bars									
	35' Beam			40' Beam			21' Beam		
	Left	Mid	Right	Left	Mid	Right	Left	Mid	Right
Roof	(6) #8	(5) #6	(6) #8	(7) #8	(4) #8	(7) #8	(8) #6	(4) #8	(8) #6
6	(6) #8	(8) #6	(6) #8	(7) #8	(5) #8	(7) #8	(8) #6	(4) #8	(8) #6
5	(6) #8	(8) #6	(6) #8	(7) #8	(5) #8	(7) #8	(8) #6	(4) #8	(8) #6
4	(6) #8	(8) #6	(6) #8	(8) #8	(5) #8	(7) #8	(8) #6	(4) #8	(8) #6
3	(6) #8	(8) #6	(6) #8	6#8+2#10	(5) #8	(7) #8	(8) #6	(4) #8	(8) #6
2							(8) #6	(4) #8	(8) #6
1	(6) #8	(8) #6	(6) #8	6#8+2#10	(5) #8	(7) #8	(8) #6	(4) #8	(8) #6

Table D.5: Frame E Bonded Reinforcing Bars									
	35' Beam			40' Beam			21' Beam		
	Left	Mid	Right	Left	Mid	Right	Left	Mid	Right
Roof	(6) #8	(5) #6	(6) #8	(7) #8	(4) #8	(7) #8	(8) #6	(4) #8	(8) #6
6	(6) #8	(8) #6	(6) #8	(7) #8	(5) #8	(7) #8	(8) #6	(4) #8	(8) #6
5	(6) #8	(8) #6	(6) #8	(7) #8	(5) #8	(7) #8	(8) #6	(4) #8	(8) #6
4	(6) #8	(8) #6	(6) #8	(7) #8	(5) #8	(7) #8	(8) #6	(4) #8	(8) #6
3	(6) #8	(8) #6	(6) #8	(8) #8	(5) #8	(7) #8	(8) #6	(4) #8	(8) #6
2							(8) #6	(4) #8	(8) #6
1	(6) #8	(8) #6	(6) #8	6#8+2#10	(5) #8	(7) #8	(8) #6	(4) #8	(8) #6

Table D.6: Frame F Bonded Reinforcing Bars									
	35' Beam			40' Beam			21' Beam		
	Left	Mid	Right	Left	Mid	Right	Left	Mid	Right
Roof	(6) #8	(5) #6	(6) #8	(7) #8	(4) #8	(7) #8	(8) #6	(4) #8	(8) #6
6	(6) #8	(8) #6	(6) #8	(7) #8	(5) #8	(7) #8	(8) #6	(4) #8	(8) #6
5	(6) #8	(8) #6	(6) #8	(7) #8	(5) #8	(7) #8	(8) #6	(4) #8	(8) #6
4	(6) #8	(8) #6	(6) #8	(8) #8	(5) #8	(7) #8	(8) #6	(4) #8	(8) #6
3	(6) #8	(8) #6	(6) #8	(8) #8	(5) #8	(7) #8	(8) #6	(4) #8	(8) #6
2	(6) #8	(8) #6	(6) #8	(8) #8	(5) #8	(7) #8	(8) #6	(4) #8	(8) #6
1	(6) #8	(8) #6	(6) #8	6#8+2#10	(5) #8	(7) #8	(8) #6	(4) #8	(8) #6

Table D.7: Frame G Bonded Reinforcing Bars									
	35' Beam			40' Beam			21' Beam		
	Left	Mid	Right	Left	Mid	Right	Left	Mid	Right
Roof	(6) #8	(5) #6	(6) #8	(7) #8	(4) #8	(7) #8	(8) #6	(4) #8	(8) #6
6	(6) #8	(5) #6	(6) #8	(7) #8	(4) #8	(7) #8	(8) #6	(4) #8	(8) #6
5	(6) #8	(5) #6	(6) #8	(7) #8	(4) #8	(7) #8	(8) #6	(4) #8	(8) #6
4	(6) #8	(5) #6	(6) #8	(7) #8	(4) #8	(7) #8	(8) #6	(4) #8	(8) #6
3	(6) #8	(5) #6	(6) #8	(7) #8	(4) #8	(7) #8	(8) #6	(4) #8	(8) #6
2	(6) #8	(5) #6	(6) #8	(7) #8	(4) #8	(7) #8	(8) #6	(4) #8	(8) #6
1	(6) #8	(5) #6	(6) #8	(7) #8	(4) #8	(7) #8	(8) #6	(4) #8	(8) #6

Table D.8: Frame A Shear Reinforcing Schedule												
	35' Beam				40' Beam				21' Beam			
	Region 1	Region 2	Region 3	Region 4	Region 1	Region 2	Region 3	Region 4	Region 1	Region 2	Region 3	Region 4
Roof	7 @10			9 @10	8 @11			7 @11	8 @7			5 @7
6	13@10			13@10	13@11			13@11	11 @7			11 @7
5	14@10			13@10	14@11			13@11	12 @7			12 @7
4	14@10			14@10	15@11			13@11	13 @7			13 @7
3	15@10			14@10	15@11			13@11	14 @7			14 @7
2	15@10			15@10	16@11			15@11	15 @7			15 @7
1	15@10			16@10	16@11			15@11	16 @7			15 @7

Table D.9: Frame B Shear Reinforcing Schedule												
	35' Beam				40' Beam				21' Beam			
	Region 1	Region 2	Region 3	Region 4	Region 1	Region 2	Region 3	Region 4	Region 1	Region 2	Region 3	Region 4
Roof	12@10			15@10	14@11			13@11	12 @7			10 @7
6	9 @6	12 @9	12 @9	12 @5	14 @5	13 @9	13 @9	13 @5	14 @7			13 @7
5	9 @6	13 @9	12 @9	12 @5	14 @5	14 @9	13 @9	13 @5	14 @7			14 @7
4	9 @6	13 @9	13 @9	12 @5	14 @5	14 @9	14 @9	13 @5	15 @7			15 @7
3	9 @6	13 @9	13 @9	12 @5	14 @5	14 @9	14 @9	13 @5	15 @7			15 @7
2					13 @5	15 @9	15 @9	14 @5	19 @6			15 @7
1	8 @6	14 @9	13 @9	12 @5	13 @5	15 @9	14 @9	13 @5	19 @6			15 @7

Table D.10: Frame C Shear Reinforcing Schedule

	35' Beam				40' Beam				21' Beam			
	Region 1	Region 2	Region 3	Region 4	Region 1	Region 2	Region 3	Region 4	Region 1	Region 2	Region 3	Region 4
Roof	12@10"			15@10"	14@11			13@11	12 @7			10 @7
6	9 @6	12 @9	12 @9	12 @5	14 @5	13 @9	13 @9	13 @5	14 @7			13 @7
5	9 @6	13 @9	12 @9	12 @5	14 @5	14 @9	13 @9	13 @5	14 @7			14 @7
4	(@6	13 @9	13 @9	12 @5	14 @5	14 @9	14 @9	13 @5	15 @7			15 @7
3	9 @6	14 @9	13 @9	13 @5	14 @5	15 @9	14 @9	13 @5	19 @6			15 @7
2									38 @6			
1	9 @6	13 @9	13 @9	13 @5	14 @5	15 @9	14 @9	13 @5	20 @6			15 @7

Table D.11: Frame D Shear Reinforcing Schedule

	35' Beam				40' Beam				21' Beam			
	Region 1	Region 2	Region 3	Region 4	Region 1	Region 2	Region 3	Region 4	Region 1	Region 2	Region 3	Region 4
Roof	12@10			15@10	14@11			13@11	12 @7			10 @7
6	9 @6	12 @9	12 @9	12 @5	14 @5	13 @9	13 @9	13 @5	13 @7			13 @7
5	9 @6	12 @9	12 @9	12 @5	14 @5	13 @9	13 @9	13 @5	14 @7			14 @7
4	9 @6	13 @9	12 @9	12 @5	14 @5	14 @9	12 @9	13 @5	15 @7			14 @7
3	9 @6	13 @9	13 @9	13 @5	14 @5	14 @9	14 @9	13 @5	15 @7			15 @7
2									15 @7			20 @6
1	8 @6	13 @9	13 @9	13 @5	13 @5	15 @9	14 @9	13 @5	19 @6			15 @7

Table D.12: Frame E Shear Reinforcing Schedule

	35' Beam				40' Beam				21' Beam			
	Region 1	Region 2	Region 3	Region 4	Region 1	Region 2	Region 3	Region 4	Region 1	Region 2	Region 3	Region 4
Roof	12@10			15@10	14@11			13@11	12 @7			10 @7
6	9 @6	12 @9	12 @9	12 @5	14 @5	13 @9	13 @9	13 @5	13 @7			13 @7
5	9 @6	12 @9	12 @9	12 @5	14 @5	13 @9	13 @9	13 @5	14 @7			13 @7
4	9 @6	13 @9	12 @9	12 @5	14 @5	14 @9	13 @9	13 @5	15 @7			14 @7
3	9 @6	13 @9	12 @9	13 @5	14 @5	14 @9	14 @9	13 @5	15 @7			15 @7
2									15 @7			19 @6
1	8 @6	13 @9	13 @9	13 @5	13 @5	15 @9	14 @9	13 @5	19 @6			14 @7

Table D.13: Frame F Shear Reinforcing Schedule

	35' Beam				40' Beam				21' Beam			
	Region 1	Region 2	Region 3	Region 4	Region 1	Region 2	Region 3	Region 4	Region 1	Region 2	Region 3	Region 4
Roof	12@10			14@10	14@11			13@11	12 @7			10 @7
6	9 @6	12 @9	12 @9	12 @5	14 @5	13 @9	13 @9	13 @5	13 @7			13 @7
5	9 @6	13 @9	12 @9	12 @5	14 @5	14 @9	13 @9	13 @5	14 @7			14 @7
4	9 @6	13 @9	12 @9	12 @5	14 @5	14 @9	13 @9	13 @5	15 @7			15 @7
3	9 @6	13 @9	13 @9	12 @5	14 @5	14 @9	14 @9	13 @5	15 @7			15 @7
2	9 @6	13 @9	13 @9	12 @5	14 @5	14 @9	14 @9	13 @5	15 @7			15 @7
1	9 @6	13 @9	13 @9	13 @5	14 @5	15 @9	14 @9	13 @5	19 @6			15 @7

Table D.14: Frame G Shear Reinforcing Schedule												
	35' Beam				40' Beam				21' Beam			
	Region 1	Region 2	Region 3	Region 4	Region 1	Region 2	Region 3	Region 4	Region 1	Region 2	Region 3	Region 4
Roof	7 @10			9 @10	8 @11			7 @11	7 @7			5 @7
6	13@10			13@10	13@11			13@11	11 @7			10 @7
5	14@10			13@10	14@11			13@11	12 @7			11 @7
4	14@10			13@10	14@11			13@11	13 @7			12 @7
3	14@10			14@10	15@11			13@11	13 @7			13 @7
2	15@10			14@10	15@11			14@11	14 @7			13 @7
1	14@10			15@10	15@11			14@11	15 @7			14 @7

Lateral System

Seismic and wind loadings used for the design of the sway frames are given in Table E. A complete list of load cases solved utilizing Risa3D is provided in Appendix E. These loads were divided and applied to each story of each frame by using relative rigidities of the frames and by taking into account torsion caused by both the eccentricity of the center of rigidity to center of mass as well as incidental torsion requirement of 5% of the buildings width. Equivalent lateral force analysis was used in the design of the seismic loads on building frames. A fundamental period of 0.95 seconds was used in the determination of seismic loads on the structure. Other values used in the determination of the seismic loads include an Ss=0.22g, S1=0.07g, redundancy factor (R) of 3 and a drift amplification factor (C_d) of 2.5.

The design of concrete sway frames required the additional design of the beam-column interfaces to address the extra load that would be transferred from the columns into the beams. To accomplish this, doubly reinforced beam sections have been designed for beam-column interfaces to handle moment reversals from seismic loads. In many of the beams, additional reinforcing was required above the minimum required bonded reinforcing because of the significantly large loads these beams would experience. Reinforcing schedules for Frames A through G are given in previous Tables D. Columns were designed using Risa3D and are discussed later.

Table E: Lateral Loads			
Wind Loads on Structure		Seismic Loads on Structure	
Basic Wind Speed = 90mph		Ss=0.22	S1=0.07
Exposure B		Site Class D	R=3 Cd=2.5
Height (ft)	Load (psf)	Floor	Load (k)
0-15	9.81	R	113.6
15-20	10.41	6	112
20-30	10.89	5	98.4
30-40	11.37	4	78.1
40-50	12.09	3	59.3
50-60	12.69	2	30.9
60-70	13.17	1	14.7
70-80	13.65		
80-90	14.13		
90-94	14.77		

Frames 1 through 4 were experiencing large story drifts from seismic forces. To counter these effects beams were added to the exterior Frames 1 and 4. These beams are not post-tensioned since suitable depths were found that would not interfere with minimizing floor depths. Elevations of Frames 1 through 4 are shown in Figures 5.8 through 5.11 while Figure 6.1 and 6.2 show typical beam/column interface sections of Frames 1 and 4. These sections show both bottom and top rebar to provide adequate reinforcing for moment reversals from seismic load effects, similar to the other seven frame designs. Only the bottom reinforcement would be needed at the midspan of a beam.

Columns were designed using Risa3D which has an integrated columns design feature that utilizes the PCA Load Contour method for design. A summary of column sizes and reinforcing schedule is provided in Table F.1 through F.7. Dimensions are given as inches by inches. Column sizes are either 24, 26 or 28 inches square. Columns that had slenderness and/or increased lateral load effects had to be increased in size from what was originally planned.

Figure 5.8 Frame 1 Elevation

Figure 5.9 Frame 2 Elevation

Figure 5.10 Frame 3 Elevation

Figure 5.11 Frame 4 Elevation

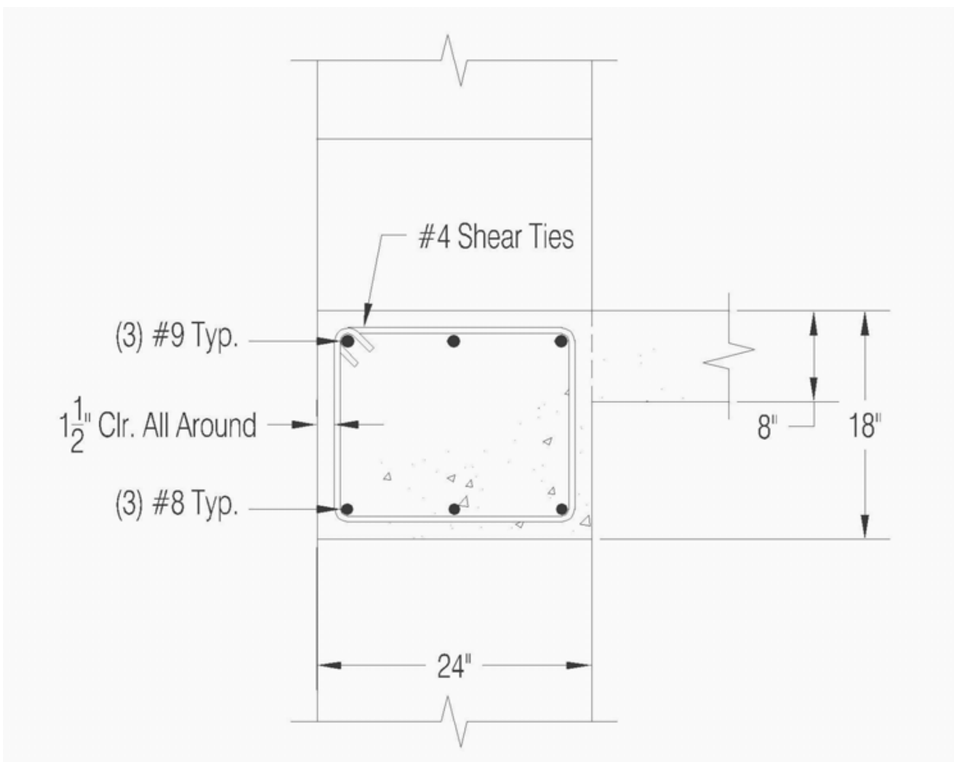


Figure 6.1 Frame 1 Beam Section

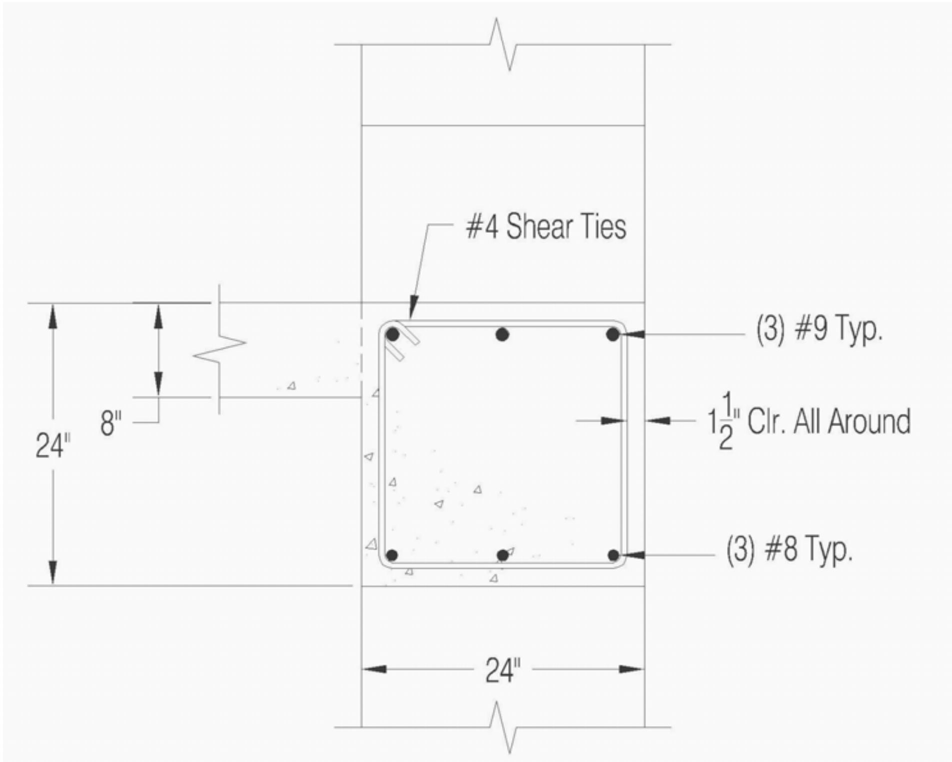


Figure 6.2 Frame 4 Beam Section

Table F.1: Frame A Column Schedule

Floor	Grid Line 4				Grid Line 3				Grid Line 2				Grid Line 1			
	Pu(k)	Mu('k)	Dim	Steel	Pu(k)	Mu('k)	Dim	Steel	Pu(k)	Mu('k)	Dim	Steel	Pu(k)	Mu('k)	Dim	Steel
6	42	203	24x24	8#8	102	230	24x24	8#8	75	228	24x24	8#8	32	63	24x24	8#8
5	103	286	24x24	8#8	216	300	24x24	8#8	169	117	24x24	8#8	60	35	24x24	8#8
4	148	315	24x24	8#8	324	345	24x24	8#8	225	121	24x24	8#8	91	33	24x24	8#8
3	208	330	24x24	8#8	432	381	24x24	8#8	338	118	24x24	8#8	122	33	24x24	8#8
2	260	348	24x24	8#8	539	393	24x24	8#8	414	116	24x24	8#8	153	36	24x24	8#8
1	314	372	24x24	8#8	649	470	24x24	8#8	509	138	24x24	8#8	185	38	24x24	8#8
B	358	460	24x24	8#8	1167	482	24x24	8#9	585	156	24x24	8#8	206	20	24x24	8#9

Table F.2: Frame B Column Schedule

Floor	Grid Line 4				Grid Line 3				Grid Line 2				Grid Line 1			
	Pu(k)	Mu('k)	Dim	Steel	Pu(k)	Mu('k)	Dim	Steel	Pu(k)	Mu('k)	Dim	Steel	Pu(k)	Mu('k)	Dim	Steel
6	69	360	24x24	8#9	167	116	24x24	8#8	117	409	24x24	8#9	37	116	24x24	8#8
5	160	171	24x24	8#8	345	62	24x24	8#8	271	182	24x24	8#8	95	63	24x24	8#8
4	233	171	24x24	8#8	517	67	24x24	8#8	397	189	24x24	8#8	143	61	24x24	8#8
3	323	188	24x24	8#8	688	31	24x24	8#8	533	179	24x24	8#8	192	57	24x24	8#8
2	396	173	24x24	8#8	860	228	24x24	8#8	669	187	24x24	8#8	240	60	24x24	8#8
1					949	231	26x26	8#9	816	194	24x24	8#8	289	69	24x24	8#8
B	488	161	24x24	8#8	1145	356	26x26	8#9	962	208	26x26	8#9	328	38	24x24	8#8

Table F.3: Frame C Column Schedule

Floor	Grid Line 4				Grid Line 3				Grid Line 2				Grid Line 1			
	Pu(k)	Mu('k)	Dim	Steel	Pu(k)	Mu('k)	Dim	Steel	Pu(k)	Mu('k)	Dim	Steel	Pu(k)	Mu('k)	Dim	Steel
6	69	351	24x24	8#9	168	115	24x24	8#8	125	407	24x24	8#9	37	115	24x24	8#8
5	159	168	24x24	8#8	347	63	24x24	8#8	346	62	24x24	8#8	94	61	24x24	8#8
4	232	169	24x24	8#8	518	61	24x24	8#8	397	184	24x24	8#8	143	64	24x24	8#8
3	322	184	24x24	8#8	691	62	24x24	8#8	542	216	24x24	8#8	183	55	24x24	8#8
2	395	169	24x24	8#8	870	106	28x28	8#9	677	77	24x24	8#8	239	82	24x24	8#8
1									725	155	24x24	8#8	291	64	24x24	8#8
B	485	158	24x24	8#8	1071	30.4	28x28	8#9	871	87	26x26	8#9	329	39	24x24	8#8

Table F.4: Frame D Column Schedule

Floor	Grid Line 4				Grid Line 3				Grid Line 2				Grid Line 1			
	Pu(k)	Mu('k)	Dim	Steel	Pu(k)	Mu('k)	Dim	Steel	Pu(k)	Mu('k)	Dim	Steel	Pu(k)	Mu('k)	Dim	Steel
6	68	352	24x24	8#9	167	115	24x24	8#8	125	407	24x24	8#9	37	115	24x24	8#8
5	159	169	24x24	8#8	347	63	24x24	8#8	271	181	24x24	8#8	94	61	24x24	8#8
4	233	169	24x24	8#8	519	61	24x24	8#8	398	184	24x24	8#8	143	64	24x24	8#8
3	321	184	24x24	8#8	691	62	24x24	8#8	542	216	24x24	8#8	183	55	24x24	8#8
2	394	169	24x24	8#8	870	106	28x28	8#9	677	77	24x24	8#8	239	82	24x24	8#8
1									725	155	24x24	8#8	291	64	24x24	8#8
B	485	158	24x24	8#8	1071	30	28x28	8#9	872	87	26x26	8#9	329	40	24x24	8#8

Table F.5: Frame E Column Schedule

Floor	Grid Line 4				Grid Line 3				Grid Line 2				Grid Line 1			
	Pu(k)	Mu('k)	Dim	Steel	Pu(k)	Mu('k)	Dim	Steel	Pu(k)	Mu('k)	Dim	Steel	Pu(k)	Mu('k)	Dim	Steel
6	68	352	24x24	8#9	168	115	24x24	8#8	125	407	24x24	8#9	37	115	24x24	8#8
5	159	169	24x24	8#8	347	63	24x24	8#8	271	181	24x24	8#8	94	63	24x24	8#8
4	233	169	24x24	8#8	519	61	24x24	8#8	398	184	24x24	8#8	143	64	24x24	8#8
3	322	184	24x24	8#8	691	61	24x24	8#8	542	216	24x24	8#8	183	55	24x24	8#8
2	395	169	24x24	8#8	870	106	28x28	8#9	677	77	24x24	8#8	239	82	24x24	8#8
1									725	155	24x24	8#8	291	64	24x24	8#8
B	485	158	24x24	8#8	1071	30	28x28	8#9	871	87	24x24	8#9	329	39	24x24	8#8

Table F.6: Frame F Column Schedule

Floor	Grid Line 4				Grid Line 3				Grid Line 2				Grid Line 1			
	Pu(k)	Mu('k)	Dim	Steel	Pu(k)	Mu('k)	Dim	Steel	Pu(k)	Mu('k)	Dim	Steel	Pu(k)	Mu('k)	Dim	Steel
6	68	363	24x24	8#9	167	117	24x24	8#8	117	412	24x24	8#9	37	117	24x24	8#8
5	160	172	24x24	8#8	346	164	24x24	8#8	271	183	24x24	8#8	95	64	24x24	8#8
4	234	173	24x24	8#8	517	62	24x24	8#8	398	189	24x24	8#8	144	61	24x24	8#8
3	324	170	24x24	8#8	688	62	24x24	8#8	542	183	24x24	8#8	192	61	24x24	8#8
2	397	166	24x24	8#8	860	61	24x24	8#8	669	181	24x24	8#8	241	58	24x24	8#8
1	488	204	24x24	8#8	1810	120	24x24	8#10	815	213	24x24	8#8	290	69	24x24	8#8
B	560	109	24x24	8#8	2152	150	26x26	12#10	1698	264	24x24	12#10	329	39	24x24	8#8

Table F.7: Frame G Column Schedule																
Floor	Grid Line 4				Grid Line 3				Grid Line 2				Grid Line 1			
	Pu(k)	Mu('k)	Dim	Steel	Pu(k)	Mu('k)	Dim	Steel	Pu(k)	Mu('k)	Dim	Steel	Pu(k)	Mu('k)	Dim	Steel
6	42	203	24x24	8#8	102	79	24x24	8#8	76	228	24x24	8#8	33	63	24x24	8#8
5	103	103	24x24	8#8	217	43	24x24	8#8	169	118	24x24	8#8	60	35	24x24	8#8
4	148	103	24x24	8#8	324	42	24x24	8#8	245	122	24x24	8#8	91	33	24x24	8#8
3	208	102	24x24	8#8	432	42	24x24	8#8	338	118	24x24	8#8	122	33	24x24	8#8
2	261	98	24x24	8#8	540	40	24x24	8#8	414	116	24x24	8#8	153	32	24x24	8#8
1	315	121	24x24	8#8	650	49	24x24	8#8	509	138	24x24	8#8	185	38	24x24	8#8
B	350	66	24x24	8#8	766	84	24x24	8#8	586	71	24x24	8#8	207	25	24x24	8#8

Serviceability

Serviceability requirements of the gravity system were set at an industry standard of L/360. Deflections were determined from hand calculation of the post-tensioned beams and slab. These members were designed as Class U flexural members and have been analyzed as such using an analysis method provided in “Design of Concrete Structures: Thirteenth Edition” by Nilson, Darwin, and Dolan. Deflections are summarized in Table G. Appendix C contains a sample deflection check of the 40’-0” beam; other deflections were completed in the same way. Maximum deflection at the center of the slab and beam spans should be considered together; in this way total upward deflection at center span of the slab would be reduced from the effect of the beams deflecting down.

Table G: Deflection Summary				
Building Element	Deflections			Deflection Ratio
	Short Term	Long Term	With Live Load	L/
35' Beams	-0.034	-0.304	-0.753	557
40' Beams	-0.122	-0.471	-1.04	461
21' Beams	-0.053	-0.169	-0.338	746
Slab	0.139"	1.0"	.999"	360

Maximum observed story drift is summarized in Figure 7.1 and 7.2 for seismic and wind loadings. Detailed tables of maximum observed story drift can be found in Appendix D. Allowable story drift for seismic loads was determined using ASCE 7-02 and a deflection amplification factor (Cd) of 2.5. The allowable seismic story drift form ASCE was determined to be 0.02Hsx. Wind drift limitations were compared to the industry standard of H/400. When seismic drift limitations for Frames A through G were checked they were determined to be acceptable. Frames 1 through 4 experienced large story drifts and a larger beam section had to be designed to counter these effects. Final designed story drifts were determined to be acceptable by industry standards and ASCE code requirements.

Figure 7.1 Maximum Observed Seismic Drift

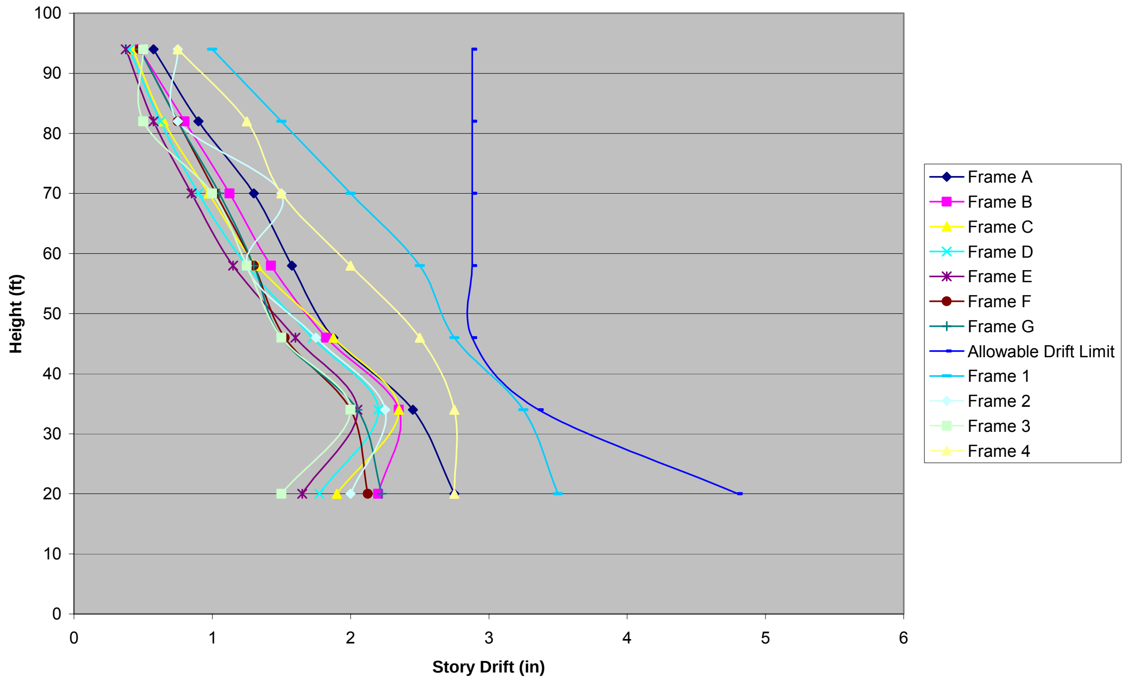
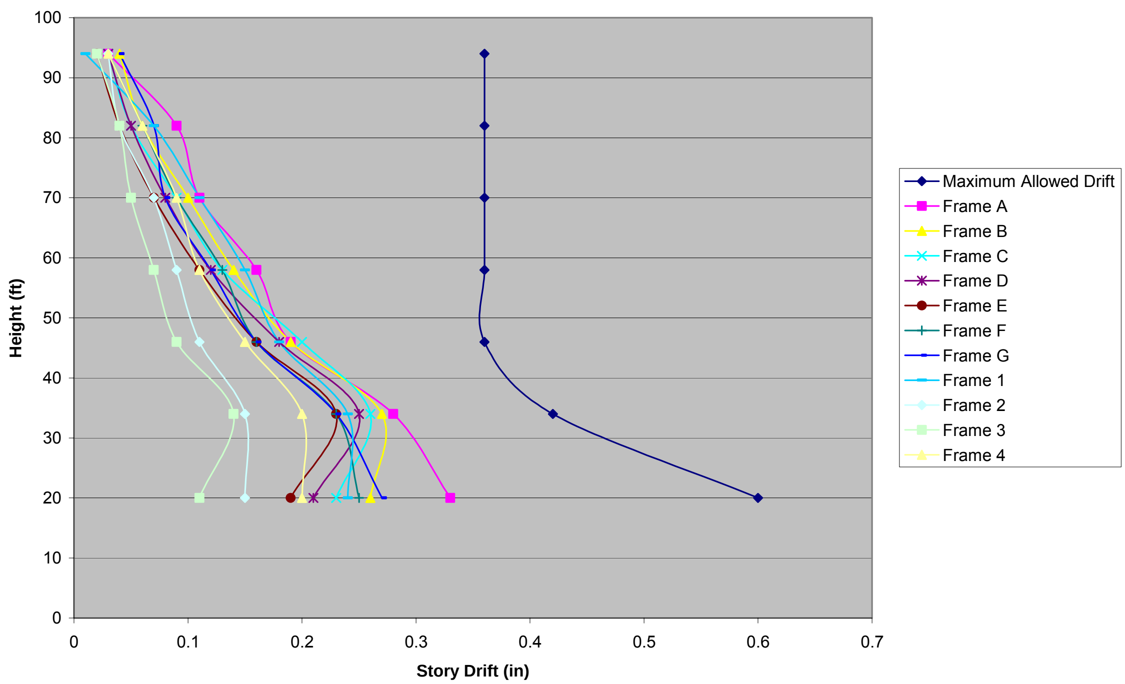


Figure 7.2 Maximum Observed Wind Drift



Foundations

Soil conditions of the site were determined previously to be less than ideal. While no actual geotechnical reports could be obtained for the site, general design assumptions were determined from the building documents of the soil conditions. An allowable soil bearing capacity of 2000psi is listed under the structural general notes. Existing caissons were designed using an allowable bearing capacity of 90ksf. Caissons have been drilled to an average depth of 54'-0". The caissons were redesigned based on the given criteria. Risa3D determined maximum and minimum foundation loads from joint reactions. A summary of these loads can be found in Table I below. It should be noted that no uplift will occur as a result of wind or seismic forces. This is common when using concrete construction since the concrete is heavy and will naturally counter uplift effects.

Table I: Summary of Foundation Loads

Values in Kips	Column Line 4		Column Line 3		Column Line 2		Column Line 1	
	Max	Min	Max	Min	Max	Min	Max	Min
Frame A	619.7	99.0	1210.3	325.2	962.0	248.4	344.7	40.0
Frame B	856.7	172.9	2004.9	473.6	1718.7	402.8	601.7	97.2
Frame C	853.2	175.3	1850.7	456.9	1542.3	367.3	602.5	99.9
Frame D	853.2	177.9	1850.7	457.0	1542.3	367.7	602.5	103.0
Frame E	853.2	180.5	1850.7	457.2	1542.3	368.1	602.5	106.2
Frame F	1002.7	196.7	2152.4	519.1	1713.7	402.8	603.5	103.0
Frame G	608.6	110.1	1210.3	325.8	962.0	250.1	344.7	50.2

For the redesign, additional caisson sizes were used to save material and construction time. Due to the gravity loads being spread out over the building's foundation system, some caissons were increased in size while others were reduced. A comparison of caisson sizes is shown below in Table J.1 and J.2. Caisson sizes are given as CXY where X and Y measure feet and inches of diameter respectively. Since the lateral system is now spread over the entire building footprint there is no need for any of the 7'-0" caissons that are used in the original system. This saves time and money since 7'-0" diameter caissons are expensive and take longer to drill and pour than most others. However, the redesign calls for many of the caissons to be increased in capacity and thus in size. The number of caissons that need increased in size offsets the use of smaller sizes to the point where using a concrete system would increase the cost of the foundations.

Table J.1 Existing Caisson Schedule

	Column Line 4	Column Line 3	Column Line 2	Column Line 1
Frame A	C36	C36	C40	C36
Frame B	C36	C36	C36	C36
Frame C	C36	C36	C46	C46
Frame D	C36	C36	C36	C70
Frame E	C36	C36	C36	C36
Frame F	C36	C40	C40	C36
Frame G	C36	C36	C36	C70

Table J.2 Redesigned Caisson Schedule				
	Column Line 1	Column Line 2	Column Line 3	Column Line 4
Frame A	C36	C46	C40	C36
Frame B	C36	C56	C50	C36
Frame C	C36	C56	C50	C36
Frame D	C36	C56	C50	C36
Frame E	C36	C56	C50	C36
Frame F	C40	C60	C50	C36
Frame G	C36	C46	C40	C36

Economics

The purpose of the redesign of the Harry and Jeanette Weinberg Center was to develop a new structural system that could be built under zoning restrictions that limit the overall building height. Doing this allows for exploration and design of a post-tensioned concrete system; however, this redesign would have been pointless if the redesigned concrete system ended up being vastly more expensive than what has been built. Thus the criterion of designing an economically competitive concrete system was stipulated to insure that a redesign was not impractical.

RS Means 2003 (The same year that the Weinberg Center was completed) was used to estimate the cost and labor hours of the structural system as-built and as redesigned. Changing the structural systems would create the major cost differences between the two buildings. The cost and labor differences of then two structures were used for comparison of the two systems. A summary of each systems takeoff per floor is given in Table K.

Table K: Current and Redesigned Takeoffs				
	Current System Takeoff		Redesigned System Takeoff	
	Labor Hours	Cost	Labor Hours	Cost
Caissons	2302	$201,069	2811	$254,502
Basement	394	$27,100	983	$60,817
Floor 1	1407	$247,607	3517	$248,348
Floor 2	1020	$173,527	2208	$154,618
Floor 3	1326	$232,735	3424	$243,066
Floor 4	1337	$234,631	3424	$243,066
Floor 5	1339	$234,877	3424	$243,066
Floor 6	1377	$242,282	3424	$242,411
Roof	1458	$246,731	2811	$206,153
Totals	11960	$1,840,559	26026	$1,896,047

Given that the redesigned system has a height limitation and as such members had to be designed that aren't as effective as what they could be. The redesigned system would cost $55,500 more than the existing steel building. However, compared to the total project cost of $20 million, a structural system cost of $1.9 million is reasonable. The structure as-built is estimated at $1.84 million which is 9.2% of the overall project cost. The redesigned structure cost would be 9.5% of the overall system cost. A typical structural system would cost less than 10% of the total project cost. With this in mind either system is equally viable to work under their representative situations.

A drawback to using a concrete system is the amount of labor involved in construction. The difference in the total labor hours between the two projects during construction is more than two-fold. The as-built system required an estimated 12,000 labor hours while the redesigned would require an estimated 26,000. These figures, while vastly different, are not surprising. Steel systems only require a few people to hoist and connect beams in place, steel deck is placed and shear studs welded in place, then concrete poured to create a floor. A concrete system requires workers to build formwork, rebar and post-tensioning tendons must be place in the correct locations in the formwork, and finally concrete is poured to create the floor. There is much more hands-on-work for a concrete system to be constructed. While this increases the construction time it does not double it. It just means that more people would be required to be working on site to construct the concrete system than with the steel system.

Summary

The program Risa3D was used in the design and analysis of a post-tensioned concrete system for the Weinberg Center. The column grid was kept as is so that no architectural considerations of the building would be altered. The slab was designed and used to develop a preliminary design for beams. Computer models of each building frame were created and analyzed using Risa3D. From Risa3D worse case load conditions were determined for beams and columns. Risa3D was used to design the columns of the sway frames while the beams were analyzed by hand to fully take advantage of the post-tensioning tendons that were present in the beams from the gravity system design.

A slab depth of eight inches is used. Post-tensioning tendons ½” diameter are spaced at 1’-6” on center to provide 24k/ft of slab width. Beams vary in depth and width, 24”x30”, 26”x34”, and 18”x22”. Six post-tensioned tendons were used in the beams to provide a total of 173.4 kips of post-tensioning force. Beams were designed to act as part of a concrete sway frame and have been designed as doubly reinforced over column supports. Columns are typically 24”x24” except where loads or slenderness effects controlled the column design. Adequate cover is provided so that the required fireproofing rating is maintained for all floor systems and columns.

After gravity and lateral forces were analyzed and members designed as appropriate, a check of the redesigned building’s serviceability requirements was completed. Slab and beams were compared to the industry standard of Length (L)/360 for an allowable deflection ratio. Seismic drift was limited by ASCE 7-02 code requirements and a drift magnification factor of 2.5. Using these code requirements, beams for Frames 1 and 4 had to be increased in size and reinforced as appropriate to handle increased seismic loads. This was done to keep seismic drift under acceptable limitations set forth by ASCE.

Using the redesigned concrete building, new foundation caissons were designed to handle the change in loadings. Joint reactions were obtained from Risa3D output and used to design the new caissons. The existing condition of poor quality soil was assumed so the redesigned caissons, like the ones used for the steel building, were drilled to a depth where they would rest on bedrock. By doing this 90 kips per square foot bearing capacity was obtained. Additional caisson sizes were added to what was used for the steel structure. This was done to save material and the labor costs of installing larger caissons.

A post-tensioned concrete structure is a good choice for the structural system of the Weinberg Center. Since concrete was used higher seismic loads will be observed. The increased weight of the structure increased many of the caisson sizes that are used in the foundation of the structure. Given these impacts on the building’s structure, a smaller floor depth is easily obtained with post-tensioned concrete. The increased loads can be designed around to allow the building to keep the same column grid and floor areas.

The post-tensioned concrete system is economically competitive with the steel structure that was built. The percent of the cost of the structure to the total project cost would increase from 9.2% to 9.5%. This increase shows that project costs would not increase to such a degree that using a concrete structure would not be viable. The only drawback to using a concrete structure would be the increase in the amount of work that would need to be completed by the construction workers. The amount of labor to construct the post-tensioned concrete structure more than doubles from what would be used for the steel structure, but would not double the construction timeline for the structural system. A concrete system would take longer to construct yet it would cost relatively the same as the steel.

Acoustical Design of a Conference Room

Acoustical Design of a Conference Room

To understand how a change in height will impact the buildings performance and other systems the acoustical design of a conference room will be analyzed for the height as-built and the height that it would be if the redesigned structure had been used. The current steel system has a floor-to-floor height of 14'-0" for a typical story, of which 9'-8" is the clear height of the ceiling. The redesigned system with 12'-0" floor-to-floor only allows an 8'-0" ceiling height. This would obviously have repercussions on many of the building's other systems. For instance, the reduction of volume in a conference room would lower the reverberation time of speech. It is of interest to determine if this reduction would be enough to elicit any change in a conference room design. To determine this two conference rooms were designed so that direct comparisons could be drawn from the two different designs. In each case the target reverberation time is between 0.7 seconds and 1.1 seconds. However, attention will be focused on trying to keep this reverberation time on the low end since a small room performs better with less reverberation.

The architectural blueprints that were produced for the Weinberg Center leave each floor blank so that tenants can determine where they want to place walls and such. To work around this I chose a conference room design that is very similar to one located at the company I have interned with for two summers. This conference room is approximately 40'-0" x 16'-0", and like the Weinberg Center has a large ribbon window that takes up most of the exterior wall of the building.

The first design is the steel buildings conference room with floor to ceiling height of 9'-8". This room would have a total room volume of 6188.8 ft^3. The following spreadsheet outlines calculations for this room's reverberation time. The steel buildings reverberation times have been calculated using finishing materials that closely approximate what is used in the chosen conference room. A final reverberation time of 0.74 seconds was achieved using surface finishes as follows:

Ceiling	Suspended gypsum non-acoustical tiles
Wall	Painted gypsum wallboard
Doors	Wood
Window	¼" Thk. glass pane
Floor	Medium weight carpet

Seating	People seated in upholstered chairsSteel Building Conference Room Design					
			Absorption Coefficients		Sα	
Surface	Material	Area	500Hz	1000Hz	500Hz	1000Hz
Ceiling	Gyp Susp	640.00	0.05	0.04	32.00	25.60
Wall	GWB	824.00	0.07	0.05	57.68	41.20
Door	Wood	84.00	0.09	0.06	7.56	5.04
Window	Glass	175.00	0.04	0.03	7.00	5.25
Floor	Carpet	320.00	0.06	0.15	19.20	48.00
	Seated People	320.00	0.88	0.96	281.60	307.20
				a=	405.04	432.29
				T=	0.76	0.72
				Tavg=	0.74	ok

The second design of this conference room is for the redesigned concrete system using a ceiling height of 8'-0". The redesigned system's room has a total volume of 5120 ft^3. The following spreadsheets outline calculations for this room. The first spreadsheet shows the reverberation time calculations by not changing any of the room's finishing materials. This was done to determine if indeed a redesign would be needed to achieve acceptable performance of the room. The previous room's surfaces will provide a reverberation time of 0.61 seconds, which is too low. New materials were chosen that increase the reverberation time to an acceptable level. The final redesigned room's reverberation time is calculated at 0.97 seconds. While this is longer than what was calculated for the first room, it is still in acceptable values range. The materials used in the redesigned room are as follows:

Ceiling	Suspended gypsum non-acoustical tiles
Wall	Painted gypsum wallboard
Doors	Wood
Window	¼" Thk. glass pane
Floor	Terrazzo or Tiles floors
Seating	People seated in wooden chairs

Concrete Building Using Same Finishing Materials						
			Absorption Coefficients		Sα	
Surface	Material	Area	500Hz	1000Hz	500Hz	1000Hz
Ceiling	Gyp Susp	640.00	0.05	0.04	32.00	25.60
Wall	GWB	637.00	0.07	0.05	44.59	31.85
Door	Wood	84.00	0.09	0.06	7.56	5.04
Window	Glass	175.00	0.04	0.03	7.00	5.25
Floor	Carpet	320.00	0.06	0.15	19.20	48.00
	Seated People	320.00	0.88	0.96	281.60	307.20
				a=	391.95	422.94
				T=	0.65	0.61
				Tavg=	0.63	Too Low

Concrete Building Redesigned Conference Room						
			Absorption Coefficients		Sα	
Surface	Material	Area	500Hz	1000Hz	500Hz	1000Hz
Ceiling	Gyp Susp	640.00	0.05	0.04	32.00	25.60
Wall	GWB	637.00	0.07	0.05	44.59	31.85
Door	Wood	84.00	0.09	0.06	7.56	5.04
Window	Glass	175.00	0.04	0.03	7.00	5.25
Floor	Tile Floor	320.00	0.02	0.02	6.40	6.40
	Seated People	320.00	0.40	0.76	128.00	243.20
				a=	225.55	317.34
				T=	1.14	0.81
				Tavg=	0.97	ok

Summary

This study was done to determine if a conference room would need to be redesigned for acoustical considerations. It may also be determined from the finding of this study that other rooms would need similar attention to design. It should not be assumed that existing materials would still work under the new conditions. It is the findings of this study that a change in the story heights of the Weinberg Center would indeed elicit a change in the acoustics of the conference room studied. If the materials are kept the same, then lower-than-acceptable reverberation times would be present. Reverberation times that are too low can adversely affect the quality of speech perceived by listeners. Sine the amount of change between the two rooms in minimal few changes would be needed. A change in the flooring material and seating types would be sufficient to keep reverberation times in the room at acceptable levels.

Lighting Design of a Conference Room

Lighting Design of a Conference Room

To further understand how a change in height will impact the building's performance and other systems the lighting design of a conference room was analyzed for the height as-built and the height of the conference room in the redesigned structure. The current steel system has a floor-to-floor height of 14'-0" for a typical story, of which 9'-8" is the clear height to the ceiling. The redesigned system with 12'-0" floor-to-floor only allows an 8'-0" ceiling height. As shown in the study of the conference room's acoustics, this change in height has effects on many of the building's other systems.

In this case, the reduction in height of the room would adversely affect the lighting design of the conference room. This would cause the room to need a different lighting layout from what is currently provided. Since no lighting schematics could be obtained for the Weinberg Center, a lighting system was designed for the steel structure with a ceiling height of 9'-8". This will then be compared to a design of the lighting system for the redesigned concrete system with a ceiling height of 8'-0".

In each case the required number of luminaires would be twelve. This amount of luminaires provides the required 30 footcandles of illuminance at 36" above the floor, the height of a conference room table. Assumptions used in the design include a 12-month cleaning interval in a clean environment. A Phillips compact florescent lamp is used in the design of each conference room. Total lumen output by this luminaire is 3600 lumens. A light loss factor of 0.74 was calculated in each case. This would not necessarily be typical, but the room cavity ratios are close enough to not drastically affect light loss throughout the room. A Coefficient of Utilization (CU) of 0.60 was calculated for the steel structure lighting, while a CU value of 0.64 was calculated for the redesigned concrete structure. This is to be expected because a lower value would be expected for a taller room. More lights would be needed to illuminate a surface that is farther away. This is seen in the exact number of luminaires needed for each system, 11.94 for the original system and 11.17 for the redesigned. However, both these numbers are rounded up so that twelve are needed for each system.

The spacing criterion for each building would be different. The compact florescent downlight has a s/mh value of 1.5. This spacing criterion ensures that a uniform lighting distribution is provided on the surface that needs to be illuminated. This means that for the original system the lamps could be spaced at a maximum of ten feet to achieve a uniform lighting layout. For the redesigned system the spacing changes to 7.5 feet. This is enough of a difference to change the lighting layout of each system. The original system could have a layout very similar to what is shown below in Figure 8.1. The redesigned structure would have a layout similar to what is shown in Figure 8.2. The original system could have a spacing of ten feet and use the required twelve luminaires. The redesigned system would need an additional 4 luminaires to meet the required spacing criteria.

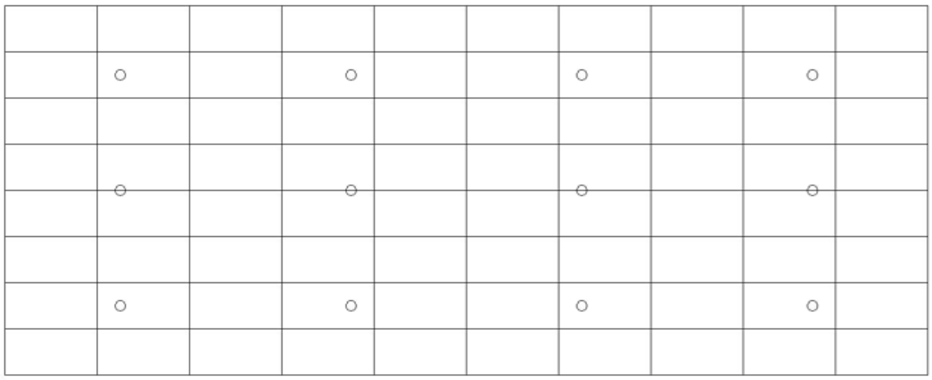


Figure 8.1 shows the original steel system lighting layout. A total of 12 luminaires are provided at a maximum spacing of 10 feet.

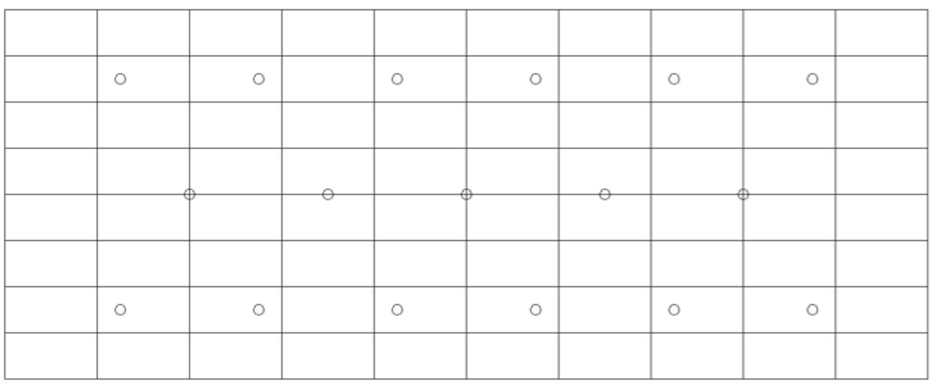


Figure 8.2 shows the redesigned systems lighting layout. A total of 17 luminaires are provided at a maximum spacing of 6 feet.

Assumptions made about each system are outlined in the following spreadsheet as well as the basic calculation outline of the two systems. In general each system's walls, ceiling and floors were kept similar so that a change in height would be the only contributing factor to the systems differences.

Lighting Design of Original System					
L=	40.00	P ceiling=	0.80		
W=	16.00	P walls=	0.60	P window=	0.06
H=	9.67	Pavg walls=	0.51		
H work plane=	3.00	P floor=	0.25		
CCR=	0.00				
RCR=	2.92				
FCR=	1.31				
LLF's					
	Ballast Factor	1.00			
	Lamp Lumen Depreciation	0.86			
	Lumen Dirt Depreciation	IV-Downlight w/ open bottom			
		Clean w/ 12 mo cleaning interval for luminaires			
		0.89			
	RSDD	12 mo cleaning interval			
		0.97			
Total LLF	0.74				
P floor cavity	0.225				
Factor for Pfc=.225		1.02			
	0.59				
CU=	0.60				
	2 lamps per luminaire	3600.00	total lumen output		
	Required Illuminance is 30fc - Cat. D conference Room				
	# luminaires required				
	11.92	Need 12 Lights			
	Spacing Criteria	1.5 s/mh			
		10.01	feet		

Lighting Design of Redesigned System					
L=	40.00	P ceiling=	0.80		
w=	16.00	P walls=	0.60	P window=	0.06
h=	8.00	Pavg walls=	0.49		
hwp=	3.00	P floor=	0.25		
CCR=	0.00				
RCR=	2.19				

FCR=	1.31			
	Ballast Factor	1.00		
	Lamp Lumen Depreciation	0.86		
	Lumen Dirt Depreciation	IV-Downlight w/ open bottom		
		Clean, 12 mo cleaning interval for luminaires		
		0.89		
	RSDD	12 mo cleaning interval		
		0.97		
Total LLFs	0.74			
	P floor cavity	0.225		
	Factor for Pfc=.225			1.02
			0.63	
	CU=		0.64	
	2 lamps per luminaire		3600.00	
	Required Illuminance is 30fc - Cat. D conference Room			
	# luminaires required			
	11.17		Still Need 12 Lights	
	Spacing Criteria		1.5 s/mh	
			7.50	feet
				but spacing requirement changes

Summary

This study was done to determine if the lighting system layout of the conference room would need to be redesigned. The concrete structure with its lower ceiling height would necessitate a different spacing of the luminaries. It can be assumed from the findings that similar changes would need to be made for other rooms. The redesigned system would require lamps placed at smaller intervals which would in turn raise the number of total lamps needed to illuminate each room as well as this conference room.

Conclusion and Recommendation

Conclusion

The steel system that is currently in place was found, in previous technical assignments, to be a very good system that balanced both economics and practicality of building for the Weinberg Center. The one drawback to this system is that deeper floor depths are required to contain the slab, beams, girders and MEP equipment. This results in a tall building height requirement for the building. The redesigned concrete post-tensioned system would minimize the floor depths needed to contain the slab, beams, and MEP equipment. The following Figure 9 shows a direct comparison between the two systems. The redesigned concrete system requires only 12 foot floor-to-floor heights while the existing steel needs 14 feet. This results in the redesigned concrete system being slightly more than 11 feet shorter than the steel building. Eleven feet may not seem significant, after all what is 74 feet when compared to 85 feet? However, certain municipalities may limit the height of buildings. As most building owners and realtors look to maximize square footage, reducing floor-to-floor heights from 14'-0" to 12'-0" could mean the addition of another story of usable office space.

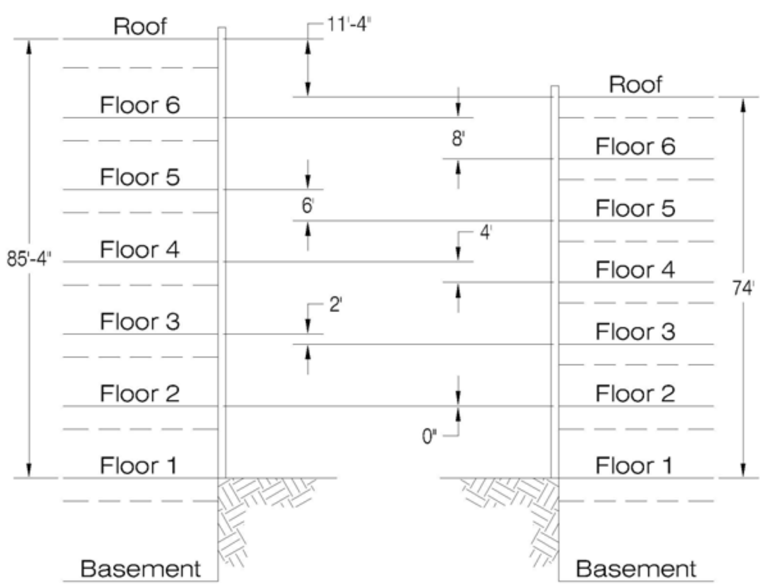


Figure 9 Steel vs. Concrete System Height Comparison

Equally important are the project's economics. Building in concrete is a more time consuming process that requires more labor than an equivalent steel building. However, labor costs associated with concrete are less than those associated with steel. This offsets the added labor required and keeps the construction cost down. In this way concrete can become an economically competitive structural system. The redesigned concrete system would cost more to build, but would still keep the structural system cost below 10% of the total project cost. The concrete system would take longer to construct. If this project was on a short time-table to be constructed this would be a major drawback. However, since this particular project was a design-bid-build delivery method that spanned two years of design and construction, a concrete system could work for this project. Even though the time it would take to complete this building is longer, it would cost little more that the steel system. If indeed this building had to be built under strict zoning regulations that limited the height or if the building owner wanted to add an additional story to the building without changing its height, a concrete post-tensioned system would be a viable and economic solution.

Studies were done to see how designing the building to work under the imposed zoning regulations would impact several of the building's other systems. The first that was studied was the acoustical design of a conference room. It was determined that a decrease in the ceiling height would have enough of an impact on the reverberation time to necessitate change to a tile or stone floor and wooden chairs. The lighting system was also studied to see what sort of impact the shorter ceiling height would have on lighting quality of the same conference room. It was determined that the spacing criteria decreases with the decreasing ceiling height. This would require lights to be placed at closer intervals and thus increase the number of lights it takes to provide a quality lighting system. No unusual changes were found in the design of this conference room. Changes that would have to be made for the acoustic properties of the rooms would be comparable to each other and would not drastically affect the interior finishing costs. The lighting systems, however, would increase in cost from the steel structure to the concrete. This is to be expected and would need to be completed in order for the building to be built under the imposed zoning height limitation.

The redesigned concrete post-tensioned system was built to satisfy imposed zoning restrictions on the overall building height. These restrictions would make a comparable steel building more expensive because of the weight of beams that would be required to minimize the floor depths. Concrete can be designed as a post-tensioned structure which would significantly reduce the depth of the floor systems and allow the building height to be reduced. Through the use of post-tensioning a redesigned concrete structure can be built at a comparable price to that of a steel structure.

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Appendix A: Post-Tensioning Design Calculation and Secondary Moment Effects

Post Tens. Floor Design

Assume Light Weight Conc
$f'_c = 4000\text{psi}$

1) Det. Slab Thickness

span/Depth ratio ≈ 45

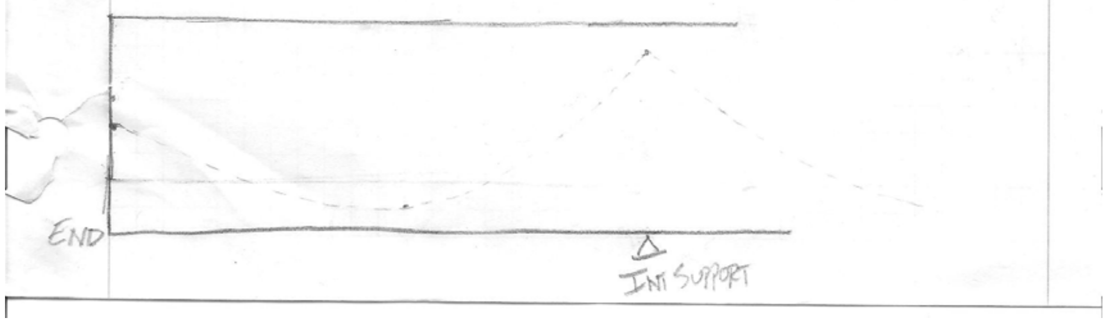
$$\frac{30'(12)}{45} = 8''$$

2) Det. load to be ballanced by prestressing

80% of DL.

8" Lightweight Slab:	120pcf(8/12) =	80psf
Partition Load		20psf
MEP		15psf
	TDL =	115psf
Live Load	LL =	100psf	TL = 215psf
Prestressing .7(115)	w_p =	-80psf
	w_n =	135psf

3) Det. Req'd Cover to Post-Tensioned Steel



STAEDTLER® No. 937 811E Engineer's Computation Pad

Min Slab Reinf: .004A

$A = \frac{1}{2}(12)(8) = 48\,in^2$

$.004A = .004(48) = .192\,in^2/ft\text{-width}$

use #4 @12" $A_s = 0.20\,in^2/ft$

Max. Spacing:

$s \le \frac{540}{f_s} - 2.5C_c$ & $s \le 12\left(\frac{36}{f_s}\right)$

if $f_s = .6f_y$ & $C_c = .75"$

$s \le 13.125$ & $s < 12"$

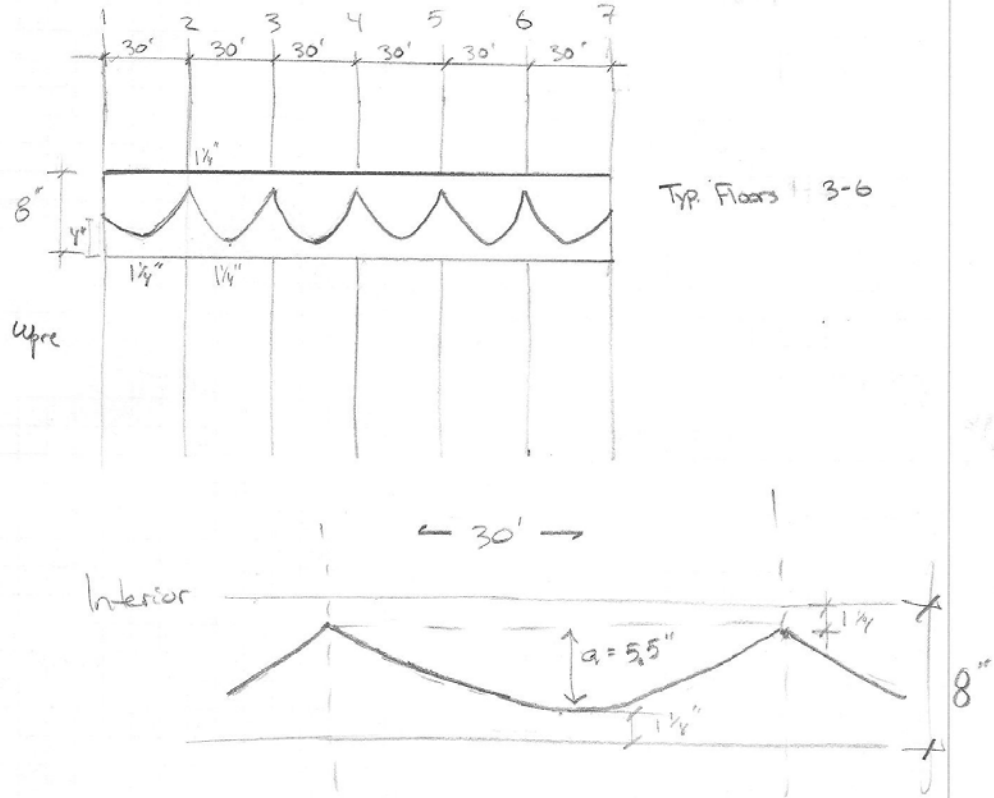
#4 @ 12" is okay

Temp & Shrinkage:

$\rho_{min} = 0.0018 \Rightarrow 0.0018 = \frac{A_s}{(12)(7.25 - \frac{4/8}{2})} = .1512\,\frac{in^2}{ft} \Rightarrow$ use $.192\,in^2/ft$

$\boxed{\text{#4@12" is okay}}$

2-hr fire-rating = 1" clr T&B



Interior Spans: $M_{pre} = \frac{.080^{k/ft}(30)^2}{8} = 9'^{k}$

$a = 8 - 2.5 = 5.5''$

$F = \frac{w_{pre} L^2}{8a} = \frac{M_{pre}}{a} = \frac{9^k}{\frac{5.5''}{12}} = 19.6^k/ft$

$F/A = \frac{19.6^k}{8 \times 12} = 204 \text{ psi}$

$M_n = \frac{w_u l_n^2}{16} = \frac{.135(28)^2}{16} = 6.6'^k$

$S = \frac{12 \times 8^2}{6} = 128 in^3$

$f = \frac{F}{A} \pm \frac{M}{S} = -204 psi \pm \frac{6600'lb' \times 12}{128 in^3} =$

$= -204 + 619 = 415$ ← can get away with $f'_c = 6000$ psi conc → allow = 465 psi

$-204 - 619 = -823$

max Tens: $6\sqrt{f'_c} = 465$ psi

$M_n = \frac{w_u l_n^2}{16} = \frac{.123(28)^2}{16} = 6.0\,'k$

$S = 128\,in^3$

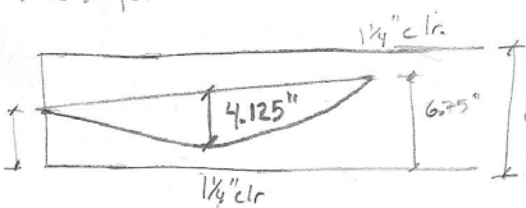
$f = -204 \pm \frac{6000 \times 12}{128} = (-204 \pm 562.5)\,psi$

$= 358.5\,psi < 465\,psi$ -ok

$= -766.5\,psi$

Exterior Spans.

$M = \frac{w_u l_n^2}{14}$



$M_p = 9\,'k$

$a = 4.125''$

$F = \frac{12 \times 9}{4.125} = 26.18$

$F/A = \frac{26.18}{8 \cdot 12} = 273\,psi$

$M_{\mathcal{E}} = \frac{.123 \times 28^2}{14} = 6.9\,'k$

$S = \frac{12 \cdot 8^2}{6} = 128\,in^3$

$F = -273\,psi \pm \frac{6900 \times 12}{128} = -273 \pm 646.9$

$= +373.9\,psi < 379\,psi$ -ok

$= -919.9\,psi$

Exterior Spans: ½"φ @ 1.3' to 26 k/ft

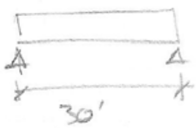
Interior Spans: ½"φ @ 1.7' to 19.6 k/ft

$6\sqrt{f'_c} = 379\,psi$ -close

use $f'_c = 6000\,psi$ - not much more expensive

$\phi V_c = \phi 2\sqrt{f'_c}\, b\, d_p$

$= 0.75(2)(\sqrt{4000})(12)(8-1.25) = 7684^{\#}/\text{ft-width}$



$w_u = 1.2(115) + 1.6(100) = 298\text{psf}$

$V_u = \dfrac{wl^2/8}{l} + \dfrac{wl}{2} = \dfrac{5}{8}wl$

$V_u = \dfrac{5}{8}w_u l = \dfrac{5}{8}(298)(30') = 5587.5^{\#}/\text{ft-width}$

$\phi V_c = 7684 > V_u = 5587.5$ – ok

Shear good!! :)

Tendon Spacings:

Need 20k/ft in Interior spans

26k/ft in Exterior spans.

Seven Wire Strands:

$.8 f_{pu} A_{ps}'$

$\frac{1}{2}$" diameter regular Strand:

$\dfrac{33}{20^k/\text{ft}} = 1'\text{-}8''$ spacing

$\dfrac{33}{26} = 1'\text{-}3''$ spacing

21'-0" Beam Post-Tensioning Deisgn

Spacing=	30.00	
Span,L(ft)=	21.00	
L/20=	12.60	

Live Load=	100.00	psf
f'c=	6000.00	psi

t=	8.00
b=	22.00
h=	18.00
bs=	150.00

Section Properties							
	Area		ybar		d	Ad^2	bh^3/12
16t*t	1024.00	x	4.00	4096.00	1.39	1990.92	5461.33
b*h	396.00	x	9.00	3564.00	3.61	5148.24	10692.00
	1420.00			7660.00		7139.15	16153.33
Yt=	5.39				St=	4317.93	
Yb=	12.61		I=	23292.49	Sb=	1847.78	

Loads						
Slab	80.00	x	30.00	=	2400.00	plf
Beam	1.53	x	120.00	=	183.33	plf
			density of concrete			

Dead Load	wd=	2583.33	x1.2	3100.00
Live Load	wl=	3000.00	x1.6	4800.00
	TL=	5583.33	wUL=	7900.00
Prestress	wpre=	2583.33		
Net Load	wn=	3000.00		

From Prestressing Layout:		
a=	5.00	in

From Analysis		
Mpre=	71.00	ft-k
M+=	75.00	ft-k
M-=	-193.00	ft-k

F=	170.40	k			
	Use	6.00	1/2" dia.	@	0.70	fpuAps
	Fact=	173.40	k		

Positive Moment Check								
		-	208.43	=	-330.55	<	-2700.00	ok
P/A=	-122.11							
		+	487.07	=	364.96	<	464.76	ok

Negative Moment Check								
		+	536.37	=	414.26	<	464.76	ok
	-122.11							
		-	1253.39	=	-1375.51	<	-2700.00	ok

35'-0" Beam Post-Tensioning Design

Spacing=	30.00		Live Load=	100.00	psf
Span,L(ft)=	35.00				
L/20=	21.00		f'c=	6000.00	psi

t=	8.00
b=	30.00
h=	24.00
bs=	158.00

Section Properties								
	Area		ybar			d	Ad^2	bh^3/12
16t*t	1024.00	x	4.00	4096.00		3.30	11169.97	5461.33
b*h	720.00	x	12.00	8640.00		4.70	15886.18	34560.00
	1744.00			12736.00			27056.15	40021.33
Yt=	7.30					St=	9185.23	
Yb=	16.70		I=	67077.48		Sb=	4017.28	

Loads						
Slab	80.00	x	30.00	=	2400.00	plf
Beam	3.33	x	120.00	=	400.00	plf
			density of concrete			
Dead Load	wd=	2800.00	x1.2	3360.00		
Live Load	wl=	3000.00	x1.6	4800.00		
	TL=	5800.00	wUL=	8160.00		
Prestress	wpre=	2800.00				
Net Load	wn=	3000.00				

From Prestressing Layout:		
a=	12.00	in

From Analysis		
Mpre=	176.00	ft-k
M+=	187.00	ft-k
M-=	-355.00	ft-k

F=	176.00	k				
	Use	6.00	1/2" dia.	@	0.70	fpuAps
	Fact=	173.40	k			

Positive Moment Check								
		-	244.31	=	-343.73	<	-2700.00	ok
P/A=	-99.43							
		+	558.59	=	459.16	<	464.76	ok

Negative Moment Check								
		+	463.79	=	364.36	<	464.76	ok
	-99.43							
		-	1060.42	=	-1159.85	<	-2700.00	ok

40'-0" Beam Post-Tensioning Design

Spacing=	30.00	Live Load=	100.00	psf
Span,L(ft)=	40.00			
L/20=	24.00	f'c=	6000.00	psi

t=	8.00
b=	34.00
h=	26.00
bs=	162.00

Section Properties								
	Area		ybar			d	Ad^2	bh^3/12
16t*t	#####	x	4.00	4096.00		4.17	17804.62	5461.33
b*h	884.00	x	13.00	11492.00		4.83	20624.36	49798.67
	#####			15588.00			38428.98	55260.00
Yt=	8.17					St=	11467.70	
Yb=	17.83		I=	93688.98		Sb=	5254.51	

Loads							
	Slab	80.00	x	30.00	=	2400.00	plf
	Beam	4.25	x	120.00	=	510.00	plf
			density of concrete				
	Dead Load	wd=	2910.00	x1.2	3492.00		
	Live Load	wl=	3000.00	x1.6	4800.00		
		TL=	5910.00	wUL=	8292.00		
	Prestress	wpre=	2910.00				
	Net Load	wn=	3000.00				

From Prestressing Layout:		
a=	16.00	in

From Analysis		
Mpre=	226.00	ft-k
M+=	233.00	ft-k
M-=	-408.00	ft-k

F=	169.50	k			
	Use	6.00	1/2" dia.	@	0.70	fpuAps
	Fact=	173.40	k		

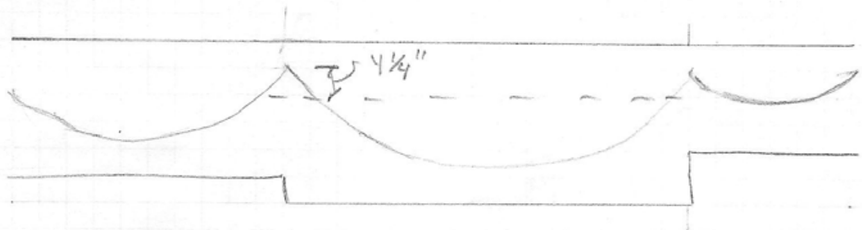
Positive Moment Check							
		-	243.82	=	-334.70	<	-2700.00	ok
P/A=	-90.88							
		+	532.11	=	441.23	<	464.76	ok

Negative Moment Check								
		+	426.94	=	336.06	<	464.76	ok
	-90.88							
		-	931.77	=	-1022.65	<	-2700.00	ok

Secondary Moment Calculations

Calculation of Secondary Moments

Typical Prestressing Layout



40'-Beam: $W_p = \frac{8 \times 165^k \times 16''}{12 \times 40^2} = 1.1\ ^k/_F$

$M_p = \frac{1.1 \times 40^2}{10} = 176'k = M_1 + M_2$

$M_1 = 165 \times \frac{4.25}{12} = 58.4'k$

$M_2 = 176'k - 58.4'k = \underline{117.6'k}$

35' Beam: $W_p = \frac{8 \times 165^k \times 13''}{12 \times 35^2} = 1.2\ ^k/_{ft}$

$M_p = \frac{1.2 \times 35^2}{10} = 147'k = M_1 + M_2$

$M_1 = 147'k \times \frac{3.25}{12} = 39.8'k$

$M_2 = 147 - 39.8 = \underline{107.2'k}$

21' Beam: $W_p = \frac{8 \times 165^k \times 5.25''}{12 \times 21^2} = 1.3\ ^k/_{ft}$

$M_p = \frac{1.3 \times 21^2}{10} = 57.3'k = M_1 + M_2$

$M_1 = \frac{165 \times 1.5}{12} = 20.6'k$

$M_2 = 57.3 - 20.6 = \underline{36.7'k}$

Appendix B: Ultimate Strength Beam Design Samples

Ultimate Design Moments from RISA Output (Envelope Solution)									
Frame A									
	35' Beam			40' Beam			21' Beam		
	Left	Mid	Right	Left	Mid	Right	Left	Mid	Right
Roof	-236	157.9	-280	-376	230.7	-275	-101	40.1	-69
6	-417	243.3	-379	-568	337.4	-496	-131	77.1	-128
5	-453	241.3	-419	-615	336.1	-537	-154	75.6	-147
4	-487	241.7	-456	-659	336.3	-582	-174	77.3	-165
3	-516	241.7	-488	-697	336.2	-618	-192	82.2	-180
2	-543	241.2	-518	-731	335.7	-653	-208	92.1	-194
1	-536	247.8	-540	-740	348	-647	-227	105.8	-198
Frame B									
	35' Beam			40' Beam			21' Beam		
	Left	Mid	Right	Left	Mid	Right	Left	Mid	Right
Roof	-357	278.6	-490	-585	392.4	-475	-160	77.5	-210
6	-666	451.5	-710	-933	614	-885	-209	149.5	-212
5	-695	447.6	-712	-960	609.8	-883	-235	146.9	-208
4	-728	447.1	-718	-999	611	-860	-255	147.4	-231
3	-757	454.6	-711	-1060	607.9	-883	-277	147.6	-250
2				-1054	627.7	-992	-299	146.6	-264
1	-718	474.9	-789	-1114	616.4	-922	-311	149.3	-252
Frame C									
	35' Beam			40' Beam			21' Beam		
	Left	Mid	Right	Left	Mid	Right	Left	Mid	Right
Roof	-343	278.5	-494	-585	392.1	-470	-161	77.7	-140
6	-656	451.5	-720	-940	613.7	-878	-213	149.5	-207
5	-671	447.3	-723	-941	609.8	-875	-230	146.9	-206
4	-705	447.9	-726	-989	609.4	-878	-248	147.8	-222
3	-743	450.6	-737	-1065	616	-876	-282	146	-244
2							-261	159.7	-296
1	-698	470.2	-776	-1091	632.6	-874	-319	147.6	-240
Frame D									
	35' Beam			40' Beam			21' Beam		
	Left	Mid	Right	Left	Mid	Right	Left	Mid	Right
Roof	-341	278.5	-494	-578	392.1	-471	-161	77.7	-125
6	-660	451.6	-720	-940	613.7	-878	-213	149.5	-220
5	-666	447.3	-723	-941	609.8	-878	-228	146.9	-220
4	-695	447.9	-726	-979	609.4	-832	-245	147.8	-220
3	-727	450.6	-737	-1051	616	-862	-276	146	-238
2							-268	159.7	-288
1	-686	470.2	-776	-1077	632.6	-859	-313	147.6	-234
Frame E									
	35' Beam			40' Beam			21' Beam		
	Left	Mid	Right	Left	Mid	Right	Left	Mid	Right

Roof	-338	278.4	-493	-578	392.1	-470	-161	77.8	-96
6	-656	451.5	-720	-940	613.7	-878	-213	149.5	-208
5	-670	447.3	-723	-941	609.8	-876	-225	146.9	-200
4	-687	447.9	-726	-970	609.4	-822	-242	147.8	-214
3	-720	450.6	-737	-1037	616	-849	-271	146	-233
2							-260	159.7	-281
1	-674	470.2	-776	-1062	632.6	-845	-307	147.6	-227
Frame F									
	35' Beam			40' Beam			21' Beam		
	Left	Mid	Right	Left	Mid	Right	Left	Mid	Right
Roof	-489	278.5	-357	-583	392.5	-475	-159	77.4	-104
6	-668	451.6	-707	-932	614	-886	-207	149.4	-214
5	-692	447.4	-710	-952	610	-884	-231	146.9	-214
4	-719	448.3	-714	-988	610.6	-883	-248	147.4	-229
3	-740	449.3	-710	-1020	610.5	-881	-264	147.4	-239
2	-759	447.5	-736	-1047	609.4	-906	-280	147.3	-249
1	-735	459.4	-757	-1064	626.9	-888	-303	148.5	-245
Frame G									
	35' Beam			40' Beam			21' Beam		
	Left	Mid	Right	Left	Mid	Right	Left	Mid	Right
Roof	-227	157.8	-279	-368	230.3	-266	-97	40.2	-65
6	-403	243.3	-364	-551	337.4	-478	-124	77.1	-83
5	-431	241.3	-398	-590	336.1	-512	-144	75.6	-137
4	-459	241.7	-429	-626	336.3	-548	-160	75.9	-151
3	-482	241.7	-456	-657	336.2	-578	-175	77.3	-163
2	-385	241.2	-481	-685	335.4	-606	-189	80.9	-174
1	-494	247.8	-501	-693	346.4	-599	-206	86.3	-176

Sample Calculation of 40'-0" Beam Midspan Flexure Reinforcement

Ultimate Design of 40' Beam.

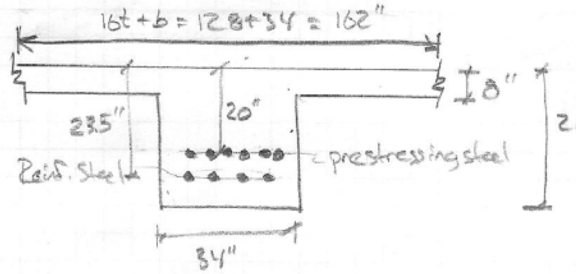
Frame C Bottom 40' span: $M^+ = 613.2'k$ $f_{pu} = 270$ ksi

$M^- = 1179'k$ $f'_c = 6000$ psi

$A_p = 6 \times .153 = .918 \text{ in}^2$

$f_y = 60{,}000$

$f_{py} = 243{,}000$



Assume #8 Bars, 1.5" cover, #4 Ties: $d = 26 - 1.5 - \frac{1}{2} - \frac{1.0}{2} = 23.5''$

Stress from effective prestress is $.7 f_{pu} = 189{,}000$ psi

$f_{ps} = f_{pe} + 10000 + \frac{f'_c}{100 \rho_p}$ $f_{pe} = 28.9$ $\rho_p = \frac{A_{ps}}{b d_p}$

$f_{ps} = 189{,}000 + 10{,}000 + \frac{6000}{100(.00028)}$ $f_{pe} = P_e / A_{ps} = 189 > .5(270) = 135$ - ok

$f_{ps} = 413{,}286$ $\rho_p = \frac{.418}{(128 \cdot 8) + (34 \cdot 20)} = .00028$

not greater than $f_{py} = .9 f_{pu} = 243{,}000$ psi

$189{,}000 + 60{,}000 = 249{,}000$

USE $f_{ps} = 243{,}000$ psi

$\beta_1 = .75$ for $f'_c = 6000$ psi

$\frac{f_{py}}{f_{pu}} = \frac{243}{270} = .9 > .85 \therefore \gamma_p = 0.4$

$M_n = A_p f_{ps}\left(d_p - \frac{a}{2}\right) + A_s f_y\left(d - \frac{a}{2}\right)$

Min Bonded Reinf.

$.004A =$

$.004(17.75 \times 34) = 2.41$

$4\#8 \quad A_s = 3.95 \, in^2$

$a = \dfrac{(.918)(243000) + (3.95)(60{,}000)}{.85(6000)(162)}$

$a = .557'' < 8''$ slab depth ← Assumption ok

$c = \frac{.557}{.75} = .74'' < .375 d_t = 7.5'' - ok \quad \phi = 0.9$

20" - prestress, conservative

$M_n = A_p f_{ps}\left(d_p - \frac{a}{2}\right) + A_s f_y\left(d - \frac{a}{2}\right)$

$M_n = (.918)(243)\left(20 - \frac{.557}{2}\right) + (3.95)(60)\left(23.5 - \frac{.557}{2}\right)$

$M_n = 9902.8''k = 825'k$

$\phi M_n = .9(825) = 742.5'k$

$M_u^+ = 632.6'k + \frac{117.6}{2} = 691.4'k$

$M_u = 691'k < \phi M_n = 742.5'k$ → ok

$\boxed{\text{Use Min Bonded Steel } (5)\#8}$

Use in rest.

$(4)\#8 \quad A_{s\,min} = 3.16 \, in^2$

$\phi M_n - \frac{M_2}{2} = 573'k$ → good for some

Min. Bonded Steel Governs for 40' Beam

Positive Moment, Highest moment is

$M_u^+ = 691.4 < \phi M_n = 742.5$

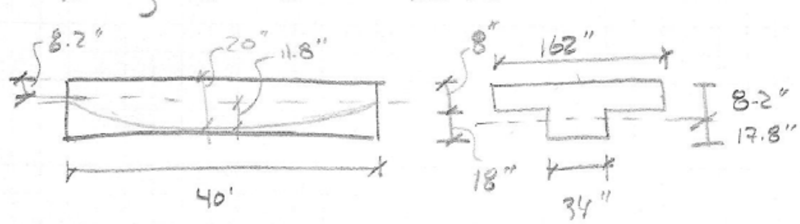
using min bonded reinf

Need (5) #8 for all 40' beams midspan.

Appendix C: Deflection Calculation Example

Deflection Check

Assuming Class U ∴ uncracked member

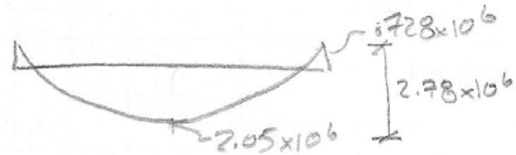


prestress force = 173.4k → After losses : 147.4k

ecc at supports = 4.2" Live Load = 3000 lb/ft

ecc at midspan = 11.8" Dead Load = 1460 lb/ft

$C_c = 2.0$ $E_c = 3360E6$ $I_c = 93689$



$$\Delta_{pi} = \frac{(-2.78E6)(240)(2/3)(240)(5/8) + (.728E6)(240)(120)}{(3.36E6)(93689)}$$

$\Delta_{pi} = -0.145''$

$$\Delta_o = \frac{5wl^4}{384EI} = \frac{5(1460)(40)(12)}{384(3.36E6)(93689\times12)} = 0.267''$$

$\Delta_{pi} + \Delta_o = -0.145 + 0.267 = .122''$ – down initial

$$\Delta_{pe} = -0.145\left(\frac{0.145+0.123}{2}\right)\times 2 + 0.267(1+2.0)$$

$$\Delta_{pe} = -0.123 - 0.207 + 0.801 = 0.471''$$

$$\Delta = 0.471'' + 0.267\left(\frac{3000}{1460}\right) = 1.04'' \text{ - down long term.}$$

$$\frac{40\times 12}{1.04} = 461 > 360$$

$$\frac{L}{461} < \frac{L}{360} \text{ - ok}$$

- Actual member is fixed-fixed ∴ will reduce deflection from pin-pin assumption.

Appendix D: Story Drift Tables

Frame A								
	Seismic					Wind		
	Total Drift	Story Drift	Cd/I	Amplified Drift	Allow. Drift	T Drift	Story Drift	Allow. Drift
Roof	4.57	0.23	2.5	0.575	2.88	1.19	0.03	0.36
Floor 6	4.34	0.36	2.5	0.9	2.88	1.16	0.09	0.36
Floor 5	3.98	0.52	2.5	1.3	2.88	1.07	0.11	0.36
Floor 4	3.46	0.63	2.5	1.575	2.88	0.96	0.16	0.36
Floor 3	2.83	0.75	2.5	1.875	2.88	0.8	0.19	0.36
Floor 2	2.08	0.98	2.5	2.45	3.36	0.61	0.28	0.42
Floor 1	1.1	1.1	2.5	2.75	4.8	0.33	0.33	0.6

Frame B								
	Seismic					Wind		
	Total Drift	Story Drift	Dc/I	Amplified Drift	Allow. Drift	T Drift	Story Drift	Allow. Drift
Roof	4.08	0.19	2.5	0.475	2.88	1.06	0.04	0.36
Floor 6	3.89	0.32	2.5	0.8	2.88	1.02	0.06	0.36
Floor 5	3.57	0.45	2.5	1.125	2.88	0.96	0.1	0.36
Floor 4	3.12	0.57	2.5	1.425	2.88	0.86	0.14	0.36
Floor 3	2.55	0.73	2.5	1.825	2.88	0.72	0.19	0.36
Floor 2	1.82	0.94	2.5	2.35	3.36	0.53	0.27	0.42
Floor 1	0.88	0.88	2.5	2.2	4.8	0.26	0.26	0.6

Frame C								
	Seismic					Wind		
	Total Drift	Story Drift	Cd/I	Amplified Drift	Allow. Drift	T Drift	Story Drift	Allow. Drift
Roof	3.8	0.17	2.5	0.425	2.88	0.99	0.03	0.36
Floor 6	3.63	0.26	2.5	0.65	2.88	0.96	0.05	0.36
Floor 5	3.37	0.39	2.5	0.975	2.88	0.91	0.09	0.36
Floor 4	2.98	0.53	2.5	1.325	2.88	0.82	0.13	0.36
Floor 3	2.45	0.75	2.5	1.875	2.88	0.69	0.2	0.36
Floor 2	1.7	0.94	2.5	2.35	3.36	0.49	0.26	0.42
Floor 1	0.76	0.76	2.5	1.9	4.8	0.23	0.23	0.6

Frame D								
	Seismic					Wind		
	Total Drift	Story Drift	Cd/I	Amplified Drift	Allow. Drift	T Drift	Story Drift	Allow. Drift
Roof	3.55	0.16	2.5	0.4	2.88	0.92	0.03	0.36
Floor 6	3.39	0.25	2.5	0.625	2.88	0.89	0.05	0.36
Floor 5	3.14	0.36	2.5	0.9	2.88	0.84	0.08	0.36
Floor 4	2.78	0.5	2.5	1.25	2.88	0.76	0.12	0.36
Floor 3	2.28	0.69	2.5	1.725	2.88	0.64	0.18	0.36
Floor 2	1.59	0.88	2.5	2.2	3.36	0.46	0.25	0.42
Floor 1	0.71	0.71	2.5	1.775	4.8	0.21	0.21	0.6

Frame E								
	Seismic					Wind		
	Total Drift	Story Drift	Cd/I	Amplified Drift	Allow. Drift	T Drift	Story Drift	Allow. Drift
Roof	3.3	0.15	2.5	0.375	2.88	0.82	0.02	0.36
Floor 6	3.15	0.23	2.5	0.575	2.88	0.8	0.04	0.36
Floor 5	2.92	0.34	2.5	0.85	2.88	0.76	0.07	0.36
Floor 4	2.58	0.46	2.5	1.15	2.88	0.69	0.11	0.36
Floor 3	2.12	0.64	2.5	1.6	2.88	0.58	0.16	0.36
Floor 2	1.48	0.82	2.5	2.05	3.36	0.42	0.23	0.42
Floor 1	0.66	0.66	2.5	1.65	4.8	0.19	0.19	0.6

Frame F								
	Seismic					Wind		
	Total Drift	Story Drift	Cd/I	Amplified Drift	Allow. Drift	T Drift	Story Drift	Allow. Drift
Roof	3.68	0.19	2.5	0.475	2.88	0.95	0.03	0.36
Floor 6	3.49	0.3	2.5	0.75	2.88	0.92	0.06	0.36
Floor 5	3.19	0.41	2.5	1.025	2.88	0.86	0.09	0.36
Floor 4	2.78	0.52	2.5	1.3	2.88	0.77	0.13	0.36
Floor 3	2.26	0.61	2.5	1.525	2.88	0.64	0.16	0.36
Floor 2	1.65	0.8	2.5	2	3.36	0.48	0.23	0.42
Floor 1	0.85	0.85	2.5	2.125	4.8	0.25	0.25	0.6

Frame G								
	Seismic					Wind		
	Total Drift	Story Drift	Cd/I	Amplified Drift	Allow. Drift	T Drift	Story Drift	Allow. Drift
Roof	3.74	0.19	2.5	0.475	2.88	0.97	0.04	0.36
Floor 6	3.55	0.3	2.5	0.75	2.88	0.93	0.07	0.36
Floor 5	3.25	0.42	2.5	1.05	2.88	0.86	0.08	0.36
Floor 4	2.83	0.52	2.5	1.3	2.88	0.78	0.12	0.36
Floor 3	2.31	0.6	2.5	1.5	2.88	0.66	0.16	0.36
Floor 2	1.71	0.82	2.5	2.05	3.36	0.5	0.23	0.42
Floor 1	0.89	0.89	2.5	2.225	4.8	0.27	0.27	0.6

Frame 1								
	Seismic					Wind		
	Total Drift	Story Drift	Cd/I	Amplified Drift	Allow. Drift	T Drift	Story Drift	Allow. Drift
Roof	6.6	0.4	2.5	1	2.88	1	0.01	0.36
Floor 6	6.2	0.6	2.5	1.5	2.88	0.99	0.07	0.36
Floor 5	5.6	0.8	2.5	2	2.88	0.92	0.11	0.36
Floor 4	4.8	1	2.5	2.5	2.88	0.81	0.15	0.36
Floor 3	3.8	1.1	2.5	2.75	2.88	0.66	0.18	0.36
Floor 2	2.7	1.3	2.5	3.25	3.36	0.48	0.24	0.42
Floor 1	1.4	1.4	2.5	3.5	4.8	0.24	0.24	0.6

Frame 2								
	Seismic					Wind		
	Total Drift	Story Drift	Cd/I	Amplified Drift	Allow. Drift	T Drift	Story Drift	Allow. Drift
Roof	4.1	0.3	2.5	0.75	2.88	0.64	0.03	0.36
Floor 6	3.8	0.3	2.5	0.75	2.88	0.61	0.04	0.36
Floor 5	3.5	0.6	2.5	1.5	2.88	0.57	0.07	0.36
Floor 4	2.9	0.5	2.5	1.25	2.88	0.5	0.09	0.36
Floor 3	2.4	0.7	2.5	1.75	2.88	0.41	0.11	0.36
Floor 2	1.7	0.9	2.5	2.25	3.36	0.3	0.15	0.42
Floor 1	0.8	0.8	2.5	2	4.8	0.15	0.15	0.6

Frame 3								
	Seismic					Wind		
	Total Drift	Story Drift	Cd/I	Amplified Drift	Allow. Drift	T Drift	Story Drift	Allow. Drift
Roof	3.3	0.2	2.5	0.5	2.88	0.52	0.02	0.36
Floor 6	3.1	0.2	2.5	0.5	2.88	0.5	0.04	0.36
Floor 5	2.9	0.4	2.5	1	2.88	0.46	0.05	0.36
Floor 4	2.5	0.5	2.5	1.25	2.88	0.41	0.07	0.36
Floor 3	2	0.6	2.5	1.5	2.88	0.34	0.09	0.36
Floor 2	1.4	0.8	2.5	2	3.36	0.25	0.14	0.42
Floor 1	0.6	0.6	2.5	1.5	4.8	0.11	0.11	0.6

Frame 4								
	Seismic					Wind		
	Total Drift	Story Drift	Cd/I	Amplified Drift	Allow. Drift	T Drift	Story Drift	Allow. Drift
Roof	5.4	0.3	2.5	0.75	2.88	0.84	0.03	0.36
Floor 6	5.1	0.5	2.5	1.25	2.88	0.81	0.06	0.36
Floor 5	4.6	0.6	2.5	1.5	2.88	0.75	0.09	0.36
Floor 4	4	0.8	2.5	2	2.88	0.66	0.11	0.36
Floor 3	3.2	1	2.5	2.5	2.88	0.55	0.15	0.36
Floor 2	2.2	1.1	2.5	2.75	3.36	0.4	0.2	0.42
Floor 1	1.1	1.1	2.5	2.75	4.8	0.2	0.2	0.6

Appendix E: Load Cases

1.4D
1.2D+1.6L+.5Lr
1.2D+1.6L+.5Lr-P1
1.2D+1.6L+.5Lr-P2
1.2D+1.6L+.5Lr-P3
1.2D+1.6L+.5Lr-P4
1.2D+1.6Lr+.8L
1.2D+1.6S+.8L
1.2D+1.6Lr+.8W
1.2D+1.6Lr+.8W (R)
1.2D+1.6S+.8W
1.2D+1.6S+.8W (R)
1.2D+1.6W+L+.5Lr
1.2D+1.6W+L+.5Lr-P1
1.2D+1.6W+L+.5Lr-P2
1.2D+1.6W+L+.5Lr-P3
1.2D+1.6W+L+.5Lr-P4
1.2D+1.6W (R)+L+.5Lr
1.2D+1.6W (R)+L+.5Lr-P1
1.2D+1.6W (R)+L+.5Lr-P2
1.2D+1.6W (R)+L+.5Lr-P3
1.2D+1.6W (R)+L+.5Lr-P4
1.2D+1.6W+L+.5S
1.2D+1.6W+L+.5S-P1

1.2D+1.6W+L+.5S-P2
1.2D+1.6W+L+.5S-P3
1.2D+1.6W+L+.5S-P4
1.2D+1.6W (R)+L+.5S
1.2D+1.6W (R)+L+.5S-P1
1.2D+1.6W (R)+L+.5S-P2
1.2D+1.6W (R)+L+.5S-P3
1.2D+1.6W (R)+L+.5S-P4
1.2D+1.0E+L+.2S
1.2D+1.0E+L+.2S-P1
1.2D+1.0E+L+.2S-P2
1.2D+1.0E+L+.2S-P3
1.2D+1.0E+L+.2S-P4
1.2D+1.0E (R)+L+.2S
1.2D+1.0E (R)+L+.2S-P1
1.2D+1.0E (R)+L+.2S-P2
1.2D+1.0E (R)+L+.2S-P3
1.2D+1.0E (R)+L+.2S-P4
.9D+1.6W
.9D+1.6W (R)
.9D+1E
.9D+1E (R)
Wind Drift
Seismic Drift

Key:

D – Dead Load
L – Live Load
Lr – Roof Live Load
S – Snow Load
W – Wind Load
E – Earthquake Loads
R – Reverse
P1 – Live Load Pattern 1
P2 – Live Load Pattern 2
P3 – Live Load Pattern 3
P4 – Live Load Pattern 4