The Regent

950 N. Glebe Road
Arlington, VA

Architect: Cooper Carry Architects

Senior Thesis Final Report
Spring 2006

Prepared By:	**Kristin Ruth**
Option:	**Structural**
Date:	**April 3, 2006**
Consultant:	**Dr. Memari**

The Regent
950 N. Glebe Road, Arlington, VA

Kristin Ruth
Structural Option

General Building Information
- Size: 265,243 SF (Tower)
  158,889 SF (Garage)
- Height: 176.32 FT
- Building Code: 2000 ICC International Building Code
- Zoning: C-O-2.5
- Number of Stories:
  Above Grade – 12
  Below Grade – 3
- Dates of Construction:
  Notice to Proceed – 1 – 5 - 05
  Substantial Completion – 7 – 5 - 06
  Final Completion – 9 – 5 - 06
- Cost: Approximately $32,000,000

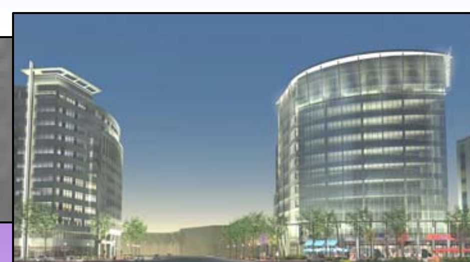
Primary Project Team
- Owner: JBG/950 N. Glebe, Ltd. Partnership
- Architect: Cooper Carry Architects
- Contractor: Glen Construction Company
- Structural Engineer: Structural Design Group, Ltd.
- MEP Engineer: Tolk, Inc.
- Civil Engineer: VIKA, Inc.
- Landscape Architect: Parker Rodriquez
- Traffic Consultant: Wells and Associates, LLC

Structural
- Parking Garage: Concrete columns, girders, beams, and slab
- Superstructure: Steel framing
- Tower Floors: Concrete slab on metal deck
- Envelope: Glass curtain wall and precast panels
- Lateral Force Resisting System: Five central braced frames

Architecture
- 3 level concrete Parking Garage below grade
- 1st level Retail space
- 11 stories of Office space on levels 2 - 12
- Roof terrace access from the 2nd level
- Office levels are open floor plans with a typical central core
- Elevators: 6 tower elevators, 2 parking garage elevators
- Fire Protection: Building is fully sprinklered

Lighting
- Exterior Lighting: Uplights accenting the top of the building
- Interior Lighting: Wall washers and sconces, TIR LED lighting, uplights, cove lighting, recessed lighting, linear strip lighting, and spotlights
- Ground Lighting: Floodlights, bollards, and 12' pole grade fixtures along the sidewalk
- Garage Lighting: Fluorescent strip fixtures wall and ceiling mounted

Electrical
- Power enters two main switchboards each connecting to their respective distribution centers and busways that feed the upper floor panels
- Power distribution: 480/277V and 208/120V
- Emergency power: 400KW (500KVA) standby generator 3 phase, 4 wire, 277/480V, 0.8PF connected to four automatic transfer switches



Construction
- Type 1A Construction
- Delivery Method: Design – Bid – Build
- Steel piles and wood lagging used during excavation
- Cranes used on site for concrete, steel, and precast erection

Mechanical
- VAV System
- Sunken Mechanical Roof Penthouse houses two cooling towers, outdoor air handling unit - OAHU-1, air handling unit - AHU-PH-1, a condenser water filtration system, two compression tanks, two hot water pumps, two hot water boilers, electric unit heaters, and an exhaust fan
- Central Plant houses two water chilling units, a plate-type heat exchanger, a chilled water pump, two condenser water pumps, two condenser water tenant pumps, air handling units - AHU-1-1 and AHU-1-2, and a condenser water treatment system

CPEP: http://www.arche.psu.edu/thesis/eportfolio/current/portfolios/kdr136/

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Executive Summary

Executive Summary

The Regent is a 12-story office building located at 950 North Glebe Road in Arlington, VA. There is retail space on the first floor and a 3-level concrete parking garage below grade.

This report provides an overview of and introduction to The Regent as well as a detailed description of the existing steel system design. Alternative floor system designs from Technical Report 2 are reviewed and a summary of the proposal introduces the structural depth and breadth topics.

The structural depth study included a design of The Regent using a cast-in-place concrete system with wide module joists. The scope of the design includes the CIP joists, CIP girders, CIP columns, CIP shearwall, representative spread footings, and the roof design.

There were two purposes for completing this structural depth study. The first purpose was to gain a better understanding of CIP structural system design through the study of design processes, design codes, structural analysis methods, and becoming more familiar with the use of structural analysis and concrete design software. The second purpose was to compare the CIP concrete system design with the existing structural steel system design in order to determine which system more effectively meets the project design team's goals which include minimal material, labor, and equipment costs, a quick erection schedule, and preservation of the architectural design intensions. It was predicted in the proposal that that steel system would better accommodate the design goals, and the system comparison results confirmed this prediction.

The construction management breadth study included a cost and schedule analysis for a typical floor and representative spread footings for both the steel and concrete systems. The costs for the concrete system were significantly higher than the costs for the steel system for both the typical floor costs and the spread footing costs. The concrete system takes approximately twice as long to erect as the steel system.

The mechanical breadth study included an analysis of the impact of the CIP concrete floor system depth on the existing mechanical layout for a typical floor. It was determined that the concrete system exceeded the allowable floor system depth by 4". It was concluded that if the CIP concrete system were to be used, there were three options; the mechanical ductwork would have to be reduced from a 12" depth to an 8" depth, the floor to floor height would be reduced from 9' to 8'-8", or the number of floors would have to be reduced in order to meet the 9' floor to ceiling height requirement and the overall building height limitations.

Overall, it was concluded that the steel system is a more efficient structural design for The Regent in terms of cost, schedule, and preservation of the architectural design.

Building Statistics and Overview of The Regent

General Building Statistics

Building Name: The Regent

Location and Site: 950 North Glebe Road, Arlington, VA 22203 (1.79 acre site)

Occupancy or Function Types

Use Type	Occupancy Type	Construction Type	Levels
Principal:			
Business (Highrise)	Group B	1A	2-12
Other:			
Retail	Group M	1A	1
Parking Garage	Group S2	1A	G3-G1

Size

Parking – Levels G3-G1: 158,889 SF

Garage:

Standard Parking Spaces: 369
Compact Parking Spaces: 50
Handicap Parking Spaces: 9
Onsite Parking: 18
Total 446 parking spaces

Level 1: 26,259 SF

Retail (South): 7,927 SF
Retail (North): 7,363 SF
Office/Retail: 485 SF
Loading Dock: 1,988 SF
Other: 8,496 SF

Office – Levels 2-12: 238,984 SF

Total Square Footage
Tower = 265,243 SF
Garage = 158,889 SF

Number of Stories

Above Grade: 11 stories of Office
1 story of Retail
Below Grade: 3 stories of Parking
Building Height (Roof to Average Grade, not including Penthouse): 176.32'

Primary Project Team

Owner/Developer:	JBG/950 North Glebe, Ltd. Partnership
Architect:	Cooper Carry Architects http://www.coopercarry.com/index.aspx
Contractor:	Glen Construction Company http://www.glencon.com/
Structural Engineer:	Structural Design Group, Ltd. http://www.sdg-ltd.com/
MEP Engineer:	Tolk, Inc. http://www.tolk.net/
Civil Engineer:	VIKA Incorporated
Landscape Architect:	Parker Rodriguez http://www.parkerrodriguez.com/aboutus.html
Traffic Consultant:	Wells and Associates, LLC http://www.mjwells.com/
Attorney:	Walsh, Colucci, Stackhouse, Emrich, and Lubeley,Inc.

Dates of Construction:

Notice to Proceed	January 5, 2005
Substantial Completion	July 5, 2006
Final Completion	September 5, 2006

Actual Cost: ≈ $32,000,000

Subtotal of the divisions and labor (no general conditions) = $31,739,500

Project Delivery Method: Design-Bid-Build

Architecture

The Regent is a state-of-the-art, 12-story office/retail building currently under construction at 950 North Glebe Road in Arlington, VA. Below the 12-story steel structure, there is a three-level concrete parking garage below grade. The main lobby, loading dock, central plant, and retail space are located on the 1^{st} floor.

Architect: Cooper Carry Architects

Glebe Road is a prime location for The Regent's office and retail space. It is located just across the street from the Ballston metrorail station at the Arlington Gateway, local to Interstate 66, and not far across the Potomac River from Washington D.C..

The Regent is a steel structure above grade and it boasts its North-facing, curved glass curtain wall façade on the southwest quadrant of the intersection of North Glebe Road and North Fairfax Drive. The South, East and West façades of the building are clad in glass and precast concrete panels. The building height varies on its South side and changes height at the 6^{th} and 10^{th} levels.

The core of the building includes an elevator lobby, five passenger elevators and one service elevator that run from the 1^{st} to the 12^{th} floors, two passenger elevators that run from the lowest parking level, G3, to the 1^{st} floor, a mechanical room, electrical room, telephone room, service vestibule, restrooms, and two stairwells. This central core is typical on levels 2-12. The office spaces on the 2^{nd} through 12^{th} floors are open floor plans with no interior structural partitions. There are roof terraces on top of the 1^{st}, 5^{th}, and 9^{th} floors. Other architectural features include the non-structural, exterior steel roof brow that spans the 11^{th} and 12^{th} floors and a non-structural steel canopy on the 1^{st} level around the retail spaces.

Since The Regent is built to its maximum height allowance, its penthouse is sunken into the 12^{th} story and as a result the 12^{th} story has both single story and two story spaces. The typical floor to floor height for levels 2-11 is 13' with a 9' floor to ceiling height. The floor to floor height of the 1^{st} level is 18' and the floor to floor height in parking garage is 10'.

Major National Model Codes

Zoning	2001 Arlington County Zoning Ordinance
Building	2000 ICC International Building Code
Life Safety	NFPA 101 Life Safety Code 1985 Plus NFPA Volumes 1-16
Mechanical	2000 ICC International Mechanical Code
Plumbing	2000 ICC International Plumbing Code
Electrical	1999 NEC National Electric Code
Energy	2000 ICC International Energy Conservation Code

Zoning

2001 Arlington County Zoning Ordinance
Existing Conditions Zoning – C-O-2.5
New Zoning – C-O-2.5

Historical Requirements

The site previously housed a 4-story glass and marble building surrounded by onsite parking. This building had no historical value and no preservation was required. It was demolished and the whole site was stripped in order to build The Regent. The zoning did not change and remains to be C-O-2.5. In conclusion, there was no historical building or zoning requirements.

Building Envelope

The building envelope consists of a curved glass curtain wall tied to steel columns on the Northern side of the building which faces the corner of North Glebe Road and North Fairfax Drive. The South, East and West exterior walls are predominantly clad in precast concrete panels and glass windows. The precast concrete panels are connected to the steel columns.

The roof is relatively flat with slopes ranging from 2% to 4.6%. The roof construction is 3" x 22 gage, deep rib, type N painted roof deck. The most common steel roofing members are W16 x 26's and W14 x 22's. The roofing system is a TPO roofing system. The TPO membrane is on 5/8" perlite board on top of R-17 rigid insulation.

The penthouse is at the top of the building and is sunken down one story into the two-story-high twelfth floor. This design maximized the amount of rentable office space while not exceeding maximum height restriction.

The steel member roof overhang on the Northern side of the building is a self-supporting, cantilevered roof brow. Specific pieces of steel in the brow are designed to support a window washing system. The bracket members along the top of the structure, on the Northern façade at the 11th and 12th floors, are non-structural, architectural members and they do not support the roof overhang.

Construction

The Regent is a design-bid-build project and is currently under construction. The notice to proceed was given on January 5, 2005. The substantial completion is scheduled for July 5, 2006 and final completion is scheduled for September 5, 2006.

The cost of this project, including the subtotal of the divisions (no general conditions) and labor is $31,739,500.

The 4-story building that existed on the site needed to be demolished and the site cleared in order to begin The Regent's construction. Demolition and construction are both in the General Contractors scope of work. A sheeting and shoring system, which included the installation of steel piles and wood lagging, was used during excavation.

The concrete contractor is using a typical tower crane within the building's central core. The installation of the concrete requires a formwork and shoring system.

The precast and steel subcontractors are going to use a "track" boom crane to erect the steel and precast panels.

The construction type for all use types in The Regent is 1A.

Electrical

Power for The Regent enters two main switchboards, MS1 and MS2, via two sets of (8) – 4" conduits, each with (4) #750 MCM and (1) #400 MCM ground from the utility transformer vault located on garage level G2. The Switchboard Room is located on level G1. MS1 and MS2 are 3 phase, 4 wire, 277/480V, 3000A bus with a fault current rating of 100,000 A. The retail spaces are fed separately from a utility transformer via (3) 4" conduits each with (4) #600 MCM wires. MS1 and MS1 are each connected to 3P, 3000AF/3000AT breakers with ground fault protection.

MS1 is connected to a distribution center which handles loads from HG1-A,B, HG1-C,D, WCU-2, ATS #3 (elevators), a 37.3KVA bus, future receptacles, and future lighting. MS1 also feeds a 2500A busway, which feeds the panels in the electrical closets on floors 3-12 and transformers convert the voltage from 277/480V to 120/208V. The 2500A busway is connected to a 3P, 2500AF/2500AT breaker with ground fault protection.

MS2 is connected to a distribution center which handles loads from WCU-2, MCCCP (mechanical panel), ATS #1 (FP-1), ATS #2, ATS #4, and a 1500 KVA bus. MS2 also feeds a 2500A busway which feeds panels in electrical closets on floors 3-12. The

2500A busway is connected to a 3P, 2500AF/2500AT breaker with ground fault protection.

Emergency power is provided by a 400KW (500KVA) standby generator, 3 phase, 4 wire, 277/440V, 0.8PF which is housed in a weatherproof acoustical enclosure on level G1. The generator is connected to four automatic transfer switches; ATS #1, ATS #2, ATS #3, and ATS #4. ATS #1 is a 3P-600A, 480V automatic transfer switch that feeds the fire pump. ATS #2 is a 3P-400A, 480V automatic transfer switch that feeds all of the life safety panels. ATS #3 is a 3P-400A, 480V automatic transfer switch that feeds the elevators that run from the lobby to the twelfth floors. Finally, ATS #4 is a 3P-200A, 480V automatic transfer switch that feeds the two garage elevators.

Lighting

The top of the structure is lit with uplights surface mounted to the trusses, two fixtures per truss. The fixtures use a 35 watt PAR 20 lamp and remote ballast.

Architect: Cooper Carry Architects

The main lobby lighting is a combination of recessed fixed downlights, linear strip lights, wall washers, light spotlights, wall/slot cove lights, TIR LED lighting, and architectural uplights.

Typical floor lobby lighting includes wall sconces, downlights, and fluorescent lighting in the coves.

The lighting above the retail store front consists of 8” long surface mounted fixtures that use 2 – T5HO 3000K, 54 W lamps.

The ground lighting includes above grade floodlights on the Northern end of the building, bollards around the traffic circle on the West side of the building, and single head - 12’ pole grade fixtures along the sidewalk.

The stairwells are lit with 4” x 5” x 48” wall mounted fixtures with 2 – F32T8 lamps with electronic ballasts. The fixtures in the garage portion of the stairwell need to be damp listed.

The lighting in the parking garage consists of fluorescent strip fixtures; wall and ceiling mounted, with 2 – T8 lamps.

The restrooms have recessed spotlights and walls sconces. The tenant corridors have recessed wall washers.

Other lighting fixtures used inside and outside the building include, 2' x 4' recessed fixtures with 3 – T8's, recessed ceiling downlights, recessed fountain lighting, and recessed spotlights.

Mechanical

The Regent features a state-of-the-art VAV system.

The Regent has a mechanical Penthouse that is sunken into the double height twelfth floor space, a Central Plant on the first floor, and mechanical rooms on floors 2-12. The Penthouse features two cooling towers, outdoor air handling unit - OAHU-1, air handling unit - AHU-PH-1, a condenser water filtration system, two compression tanks, two hot water pumps, two hot water boilers, electric unit heaters, and an exhaust fan. The Central Plant houses two water chilling units, a plate-type heat exchanger, a chilled water pump, two condenser water pumps, two condenser water tenant pumps, air handling units - AHU-1-1 and AHU-1-2, and a condenser water treatment system. There are also mechanical rooms in the central core of each floor which house each of the air handling units for floors 2-12. Electric unit heaters can be found throughout the building and parking garage.

In the parking garage on each level, there are three garage supply fans on the East side of the building and three garage exhaust fans on the West side of the building. There are exhaust fans in the Central Plant, Penthouse, Fire Pump Room, Switchboard Room, Transformer Vault, Water Pump Room, Telephone Room, and Level G3 Storage. A ventilation fan is also provided in the Central Plant.

There are 17 air handling units throughout The Regent. Their designation, total air volume capacities, locations in the building, areas they service, and types are summarized in the following table:

AHU	Total Air Volume – Max CFM	Location	Areas of Service	Type
AHU-1-1	3,300	Central Plant - Level 1	Central Plant	Chilled Water AHU
AHU-1-2	6,500	Central Plant - Level 1	Main Lobby	Chilled Water AHU
AHU-1-3	500	Fire Command Room - Level 1	Fire Command Room	Chilled Water AHU
AHU-(2-5)-1	19,000	Mechanical Rooms - Levels 2-5	Floors 2-5	Chilled Water AHU
AHU-(6-9)-1	16,500	Mechanical Rooms - Levels 6-9	Floors 6-9	Chilled Water AHU
AHU-(10,11)-1	14,000	Mechanical Rooms - Levels 10-11	Floors10-11	Chilled Water AHU
AHU-12-1	19,000	Mechanical Room - Level 12	Floor 12	Chilled Water AHU
OAHU-1	40,000	Penthouse	Outside Air System	Chilled Water AHU
AHU-G3-1	1,400	Level G3	Level G3 - Machine Room	Packaged Air Cooled AHU
AHU-PH-1	5,200	Penthouse	Penthouse, Elevator, Machine Room	Split Type DX AHU

The design conditions for The Regent are listed in the table below.

	Dry Bulb (ºF)	Wet Bulb (ºF)
Outside Design Conditions		
Summer Outside Air Temperature	95	78
Winter Outside Air Temperature	10	-----
Coincident Summer Outside Air Condition For Conditioning Outside Air	93	75
Cooling Inside Design Conditions		
Inside Temperature (Offices and Lobbies)	73	57.5
Elevator Machine Rooms and Equipment Rooms	80	68
Heating Inside Design Conditions		
Inside Temperature (Office and Lobbies)	75	55
Penthouse and Equipment Rooms	65	58

Structural

The Regent's structure consists of three levels of concrete parking below grade and twelve levels of steel framing above grade.

Below Grade

There is a 3-level concrete parking garage below grade. The typical bay size for the three levels of below grade parking is 30' x 30'. The most common column sizes are 16" x 24"and 28" x 36" and the most common beam sizes are 12" x 24", 12" x 18", 8" x 18", and 18" x 30". All of the columns are of design strength f'c = 5000 psi although a few are f'c = 7000 psi. The slab on grade for level G3 is a 4" slab. The other parking slabs are 8" with a strength of f'c = 4000psi. The plaza slab is 12" thick because it is designed to handle emergency vehicles which require design loads of 350 psf. The 1st floor building slab is 9".

Above Grade

There are two typical bay sizes for the steel superstructure above grade; 30' x 30' and approximately 40' x 30'. The most common column size is W14 x 145, 99, and 176. The most common beam sizes are W18 x 50, W18 x 46, and W16 x 26 with cambers ranging from ¾" to 2" which are designed to 75% dead load. The most common girder sizes are W18 x 65, W24 x 55, W24 x 62, and W24 x 55.

The typical floor slab is 3 ¼" light weight concrete with an f'c = 3000 psi on top of a 3" – 20 gage composite steel deck for a total slab thickness of 6 ¼".

The lateral force resisting system for The Regent is a combination of five braced frames with Frame #4 and Frame #5 running North and South and Frames #1, #2, and #3 running East and West. The five braced frames are in the central core of the building and run from the 1st to the 12th floors.

Fire Protection

The Regent is a fully sprinklered building. The hourly fire rating for different areas throughout the building are listed in the table below.

Area	Hour Rating
Structural	
Floors	2
Beams	2
Columns at Perimeter	3
Columns at Interior	3
Roof Construction	1.5
Exterior Non-bearing Walls	Non-combustible
Interior Non-bearing Walls	Non-combustible
Exit Stair Enclosures	2
Horizontal Exit Corridors	2
Elevator Hoistways	2
Elevator Machine Rooms	2
Mechanical Shafts	2
Mechanical Rooms	0
Electrical Rooms	1
Core Walls and Corridor Adjacent to Tenant Space	0
Transformer Vault (Walls, Floors, Ceiling)	2
Switchgear Room	2
Pump Room	2
Emergency Generator Room	2

The Life Safety Code used is NFPA 101 Life Safety Code, 1985 plus NFPA Volumes 1-16. Also, all applicable requirements from the Arlington County Fire Prevention Division for a highrise must be provided.

There is a fire command room on the first floor and it houses a fire alarm communicator panel, terminal cabinet, control panel, fire system annunciator, and a fire alarm transponder or transmitter. There are also fire alarm closets located in the core of the building on levels 2-12. There is a fire pump room on parking garage level G1 which houses a fire pump, jockey pump, and their respective controllers. There is a dry pump room on level G1 and dry pump valve cabinets on levels G1-G3.

Other fire detection and prevention devices used throughout the building include ionization smoke detectors, photoelectric smoke detectors, duct smoke detectors, fixed-temperature heat detectors, rate-of-rise heat detectors, sprinkler water flow detectors, sprinkler valve tamper switches, fire alarm manual pull stations, fire alarm gongs and bells, fire alarm audible devices, fire alarm strobe (ADA), fire service telephone handsets, and fire service telephone jacks.

Transportation

The vertical transportation for The Regent is broken down into three categories of elevators. The first set of elevators, cabs #1 and #2, are passenger parking shuttle elevators that are located in the core of the building and run from the lowest parking level G3 to the first floor. Another set of elevators, cabs #3, #4, #6, #7, and #8, are office tower passenger elevators that are also located in the core of the building and service levels 1 through 12. Cab #5 is an office tower swing/service elevator which is located with cabs #3, #4, #6, #7 and #8 in the core of the building and also runs from levels 1 through 12. Cab #5 has two doors; one that opens to the elevator lobbies and one that opens to the service vestibules located in the core of each floor.

Telecommunications

The telephone service comes into the building through the main telephone room which is located in parking garage level G3. In addition there are telephone rooms located in the core of each level 2-12. There are telephone outlets, data outlets, and a combination of data and telephone outlets throughout the building. There is also fire service telephone handsets located in all of the stairwells. Flame retardant, ¾” thick 4’ x 8’ plywood telephone boards are provided in the main telephone room and remote telephone closets.

Existing Steel System Design

Existing Steel Framing Design

The Regent is located at 950 North Glebe Road in Arlington, Virginia. The building is a 12-story spec office building with retail space on the first level. There is also a 3-story parking garage below grade. The building is designed to a maximum allowable height of 176 feet.

Gravity Framing System Description

Foundations

The foundations for The Regent consist of square footings ranging in size from 4' x 4' to 9' x 9' with depths ranging from 24" to 50" respectively. They are located on a 30' x 30' square grid. The two allowable bearing pressures for the square footings are 25 ksf and 40 ksf. The southwest quarter of the building has allowable bearing pressures of 25 ksf while the other three quarters of the building have a 40 ksf allowable bearing pressure. The larger square footings are located in the central core of the building below the elevator shafts. There are also continuous 24" wide, 12" deep concrete footings under the 12" thick continuous walls. The slab on grade is 4" thick reinforced with 6 x 6, 10/10 WWF. The concrete strength for all foundations, walls, and slabs on grade is a minimum of 3000 psi.

Concrete Parking Garage Below Grade

There is a 3-level concrete parking garage below grade. The typical bay size for the three levels of below grade parking is 30' x 30'. The most common column sizes are 16" x 24"and 28" x 36" and the most common beam sizes are 12" x 24", 12" x 18", 8" x 18", and 18" x 30". All of the columns are of design strength f'c = 5000 psi, although a few are f'c = 7000 psi and the 28-day design strength of the beams is f'c = 4000 psi. The parking garage slabs are 8" thick with a typical drop panel size of 10' x 10' x 5 ½" and a 28-day strength of 4000 psi.

Plaza and 1st Floor Slabs

The Plaza level slab is 12" thick with 10' x 10' x 12" drop panels. The design loads for the Plaza level include a 350 PSF live load which accounts for the weight of a fire truck loading. The first floor slab is 9" thick with 10' x 10'x 5 ½" drop panels. The Plaza and 1st floor slabs are both of strength f'c = 4000 psi.

Steel Framing Above Grade

There are two typical bay sizes for the steel superstructure above grade; 30' x 30' and approximately 43' - 46' x 30'. From North to South the columns are at a 30' spacing. From East to West the columns are spaced at 46', 30' and 43', respectively. The most common column sizes are W14 x 145, W14 x 99, and W14 x 176.

The most common beam sizes are W18 x 50, W18 x 46, and W16 x 26 with cambers ranging from ¾” to 2” which are designed to 75% dead load. The most common girder sizes are W18 x 65, W24 x 55, W24 x 62, and W24 x 55.

The typical floor slab is 3 ¼” light weight concrete with an f’c = 3000 psi and is reinforced with 6 x 6 10/10 WWF on top of a 3” – 20 gage composite steel deck for a total slab thickness of 6 ¼”. Headed shear studs, ¾” in diameter and 5” in length, allow for composite action between the slab on deck and the supporting beams.

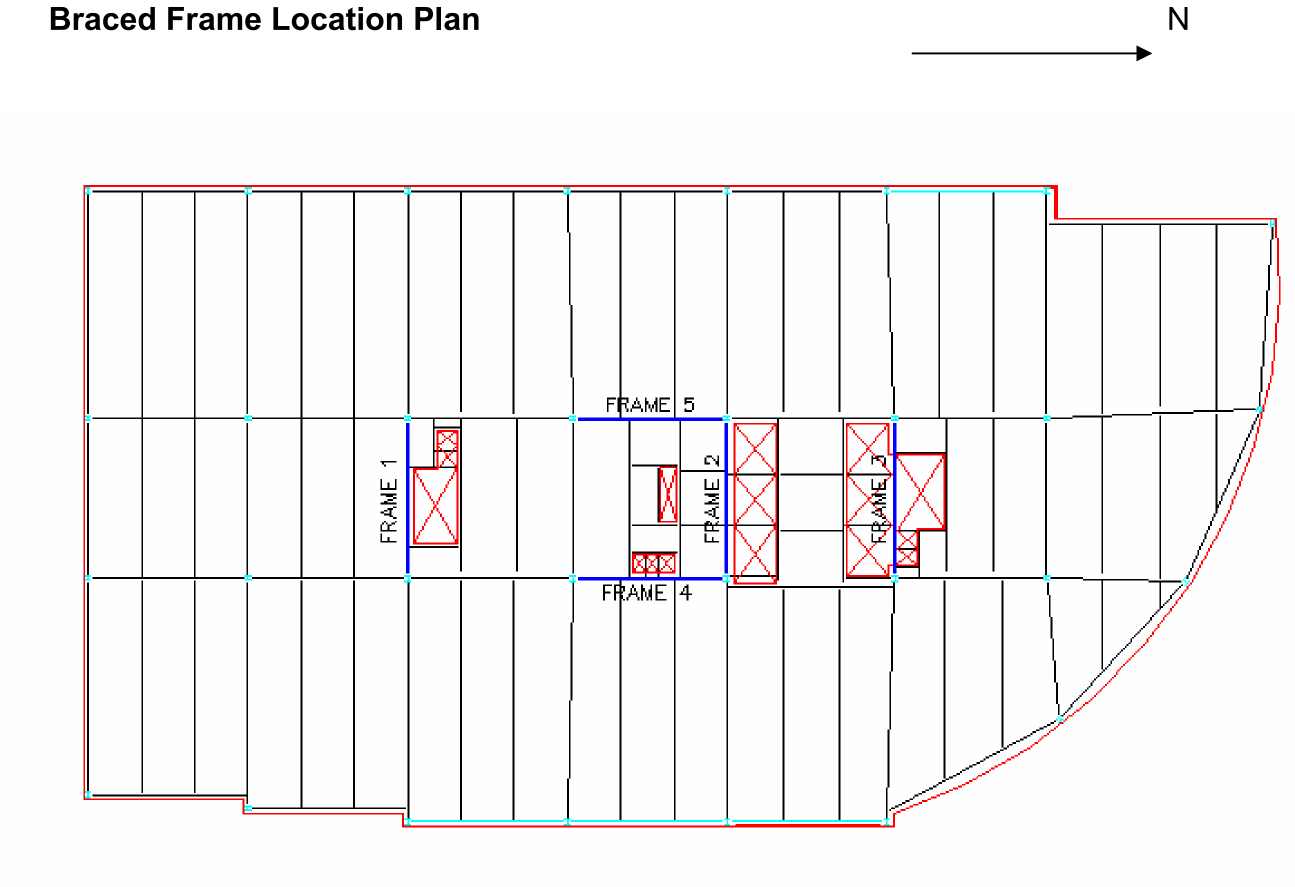
There is an elevator core running up the center of the building and through the center of each floor. The roof deck construction is 3” x 22 gage, deep rib, type N, painted roof deck.

Lateral System Description

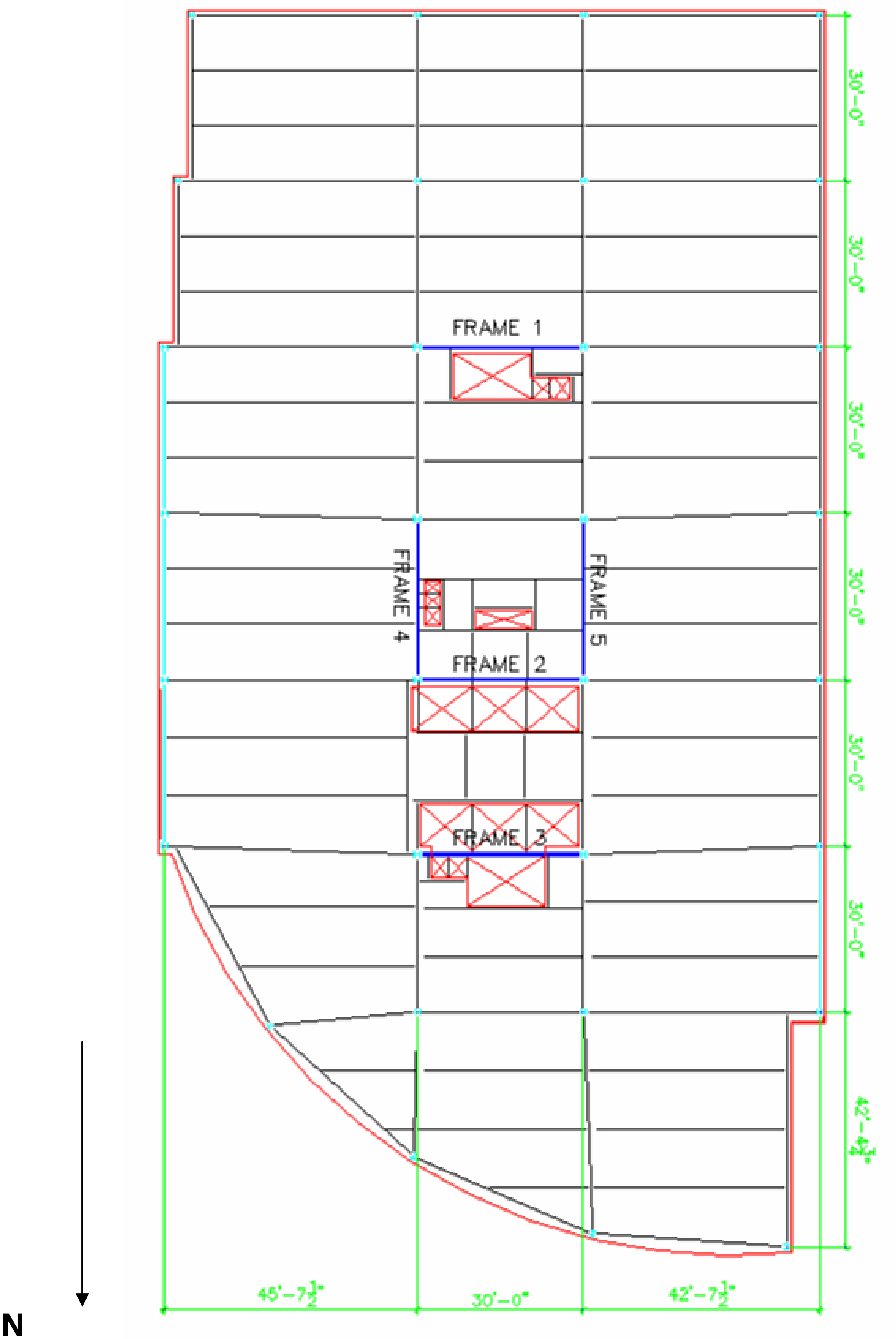
The lateral load resisting system for The Regent consists of five braced frames at the core of the building. There are two braced frames, Frame #4 and Frame #5, that span along the building’s north / south axis, and three braced frames, Frame #1, Frame #2, and Frame #3, that span along the building’s east / west axis. Frame #1, Frame #3, and Frame #5 have chevron style bracing and Frame #2 and Frame #4 have single diagonal bracing. The braced frames are approximately 30’ in width and run the full height of the building from the first floor to the penthouse roof.

The typical diagonal steel members used in the braced frames are HSS 8” x 8”’s, 10” x 10”’s, and 12” x 12”’s with thicknesses ranging from 3/8” to 5/8”. The columns in the braced frames are all 14” wide flange members ranging in size from W14 x 233’s and W14 x 257’s near the base to W14 x 53’s to W14 x 72’s at the top.

Braced Frame Location Plan

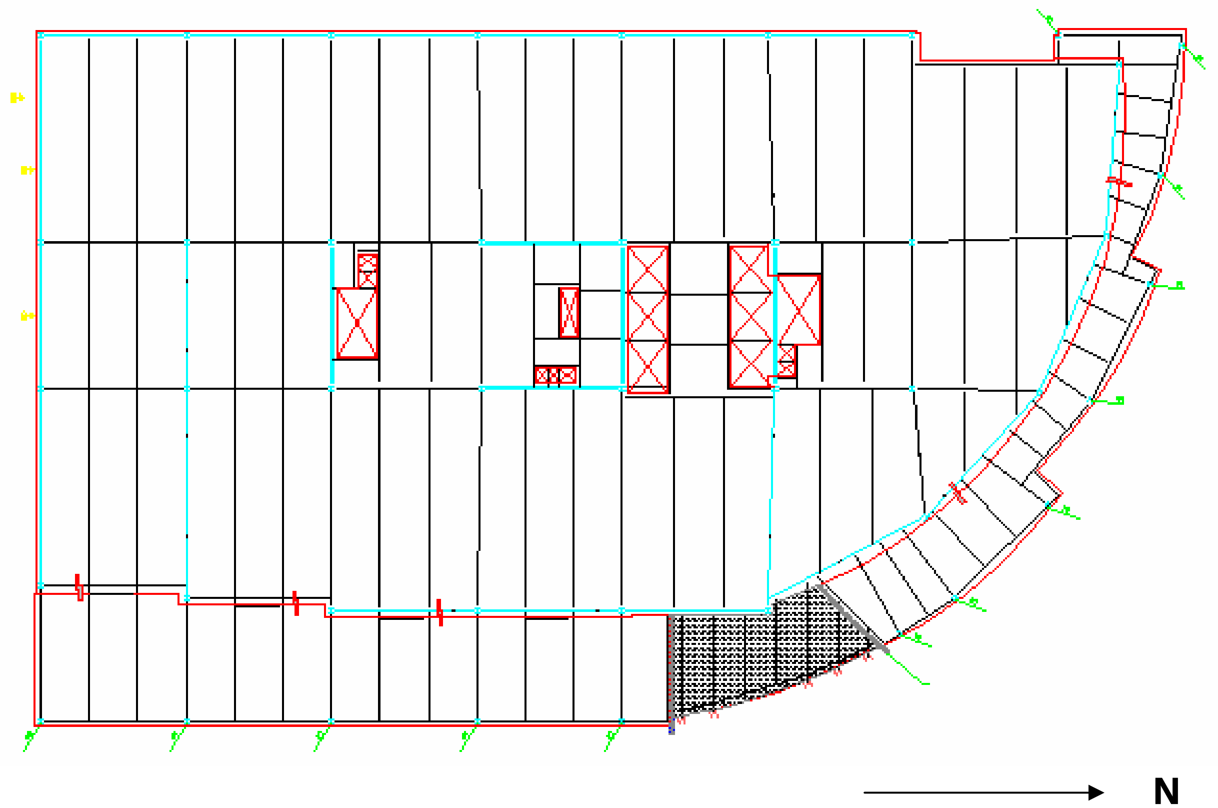


Enlarged Typical Framing Plan with Dimensions

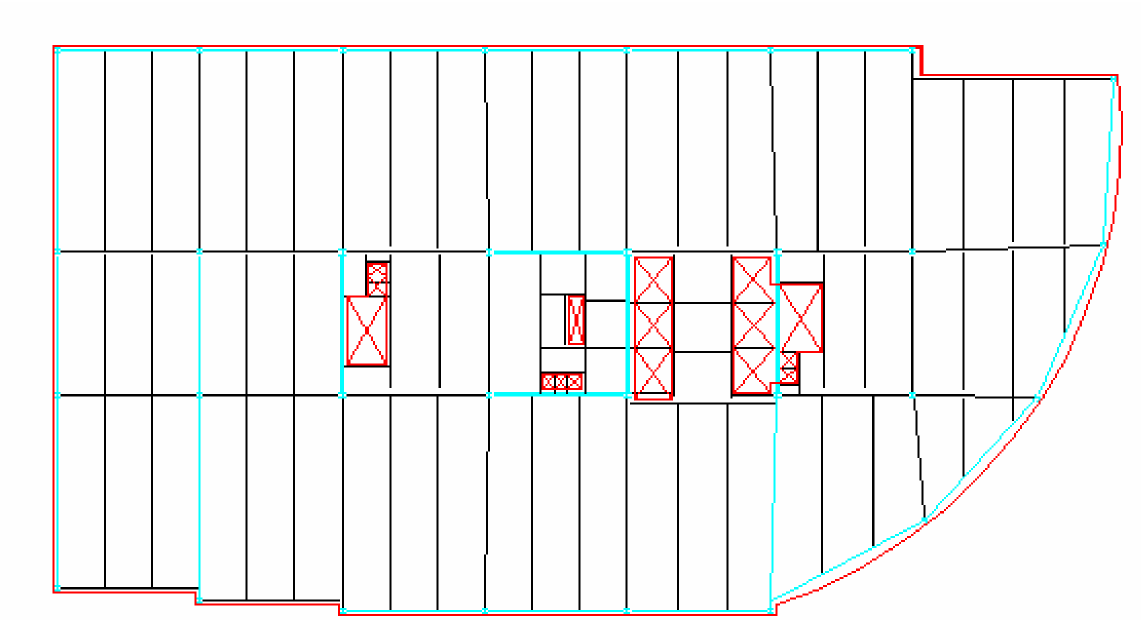


Typical Framing Plans and Elevations

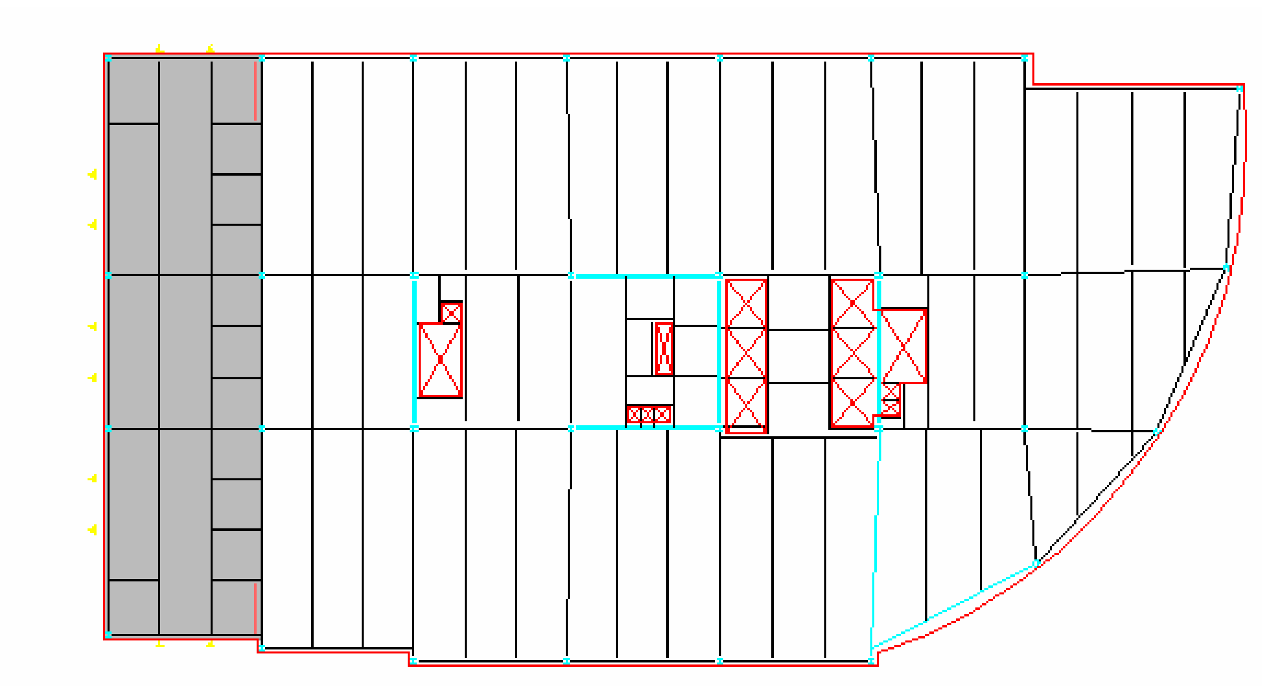
2nd Floor Faming Plan



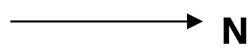
3rd – 5th Floor Framing Plan



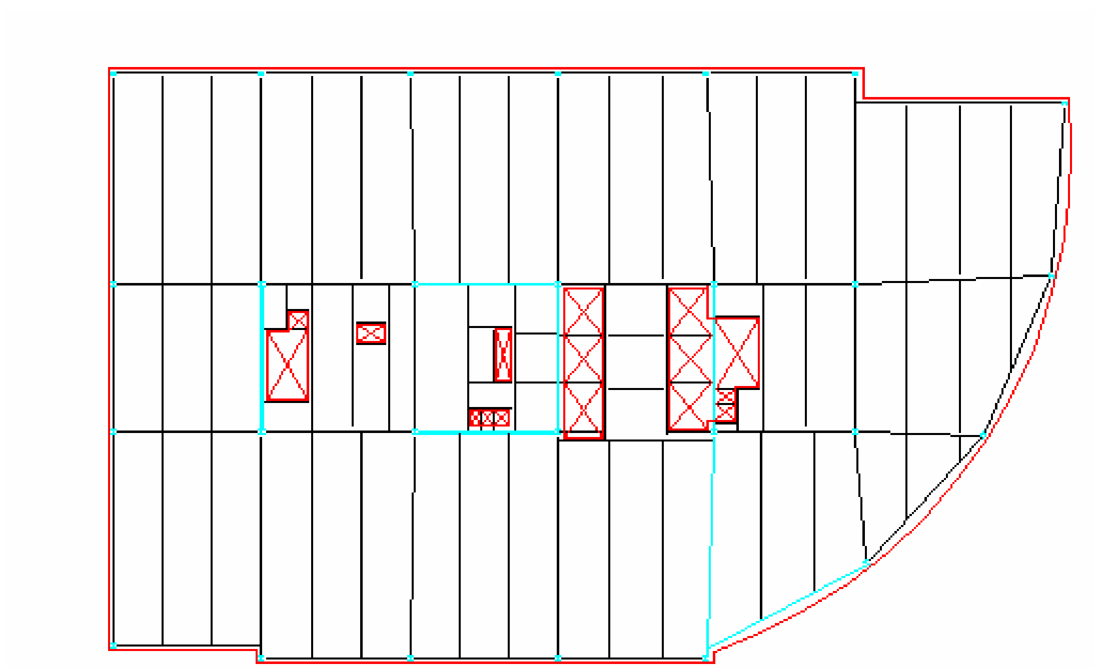
6th Floor Framing Plan



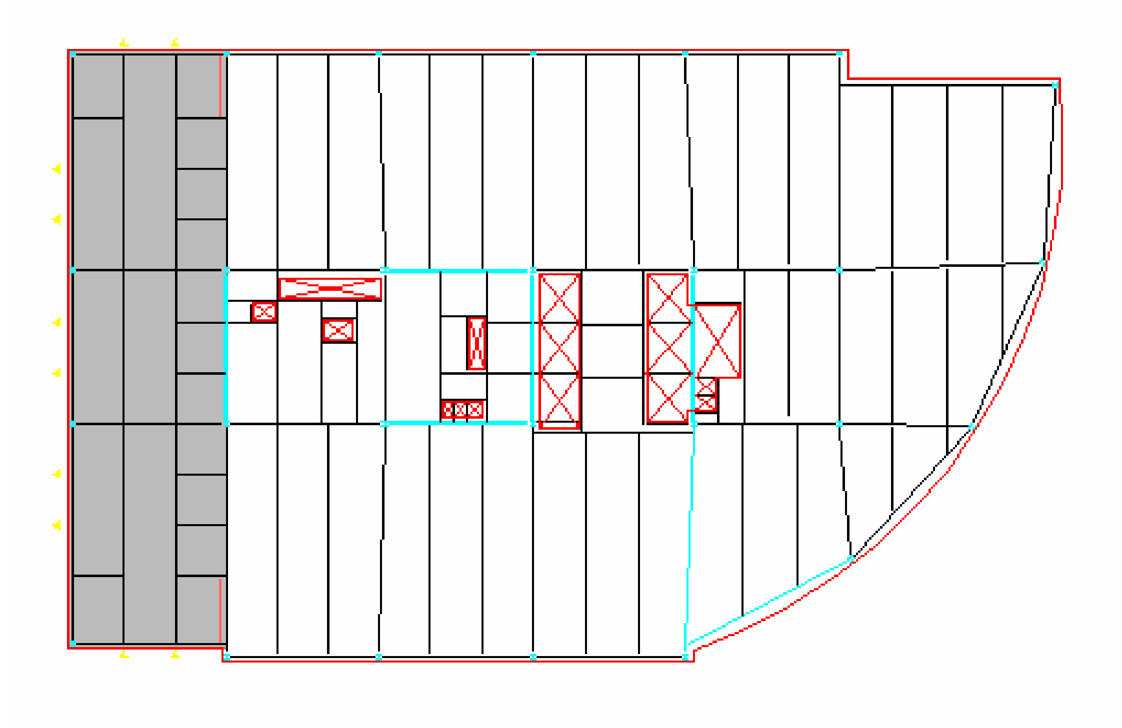
Note: Shaded area is roof construction



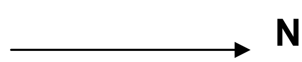
7-9th Floor Framing Plan



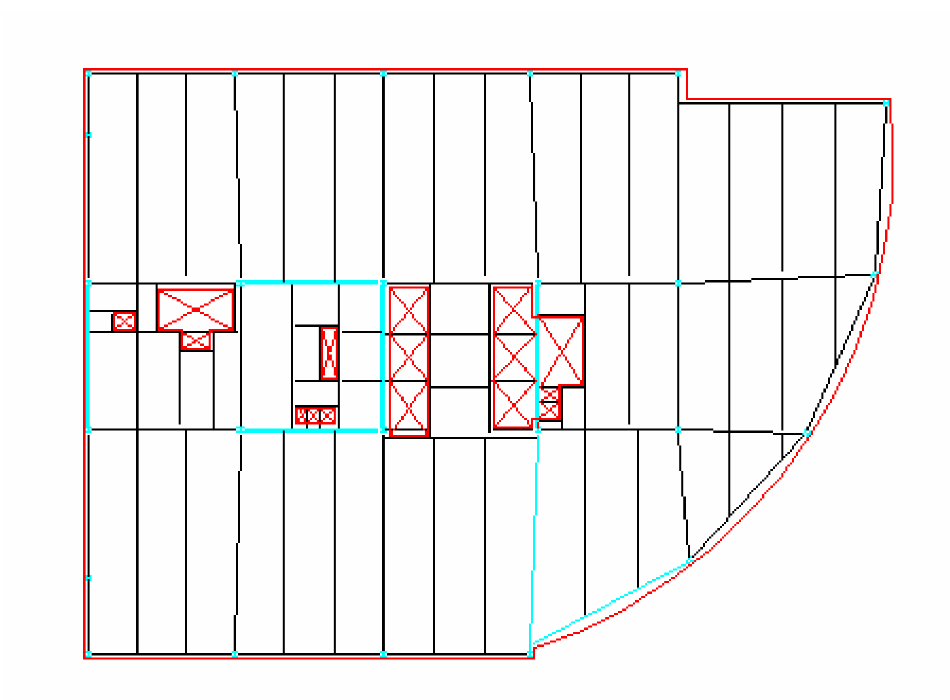
10^{th} Floor Framing Plan



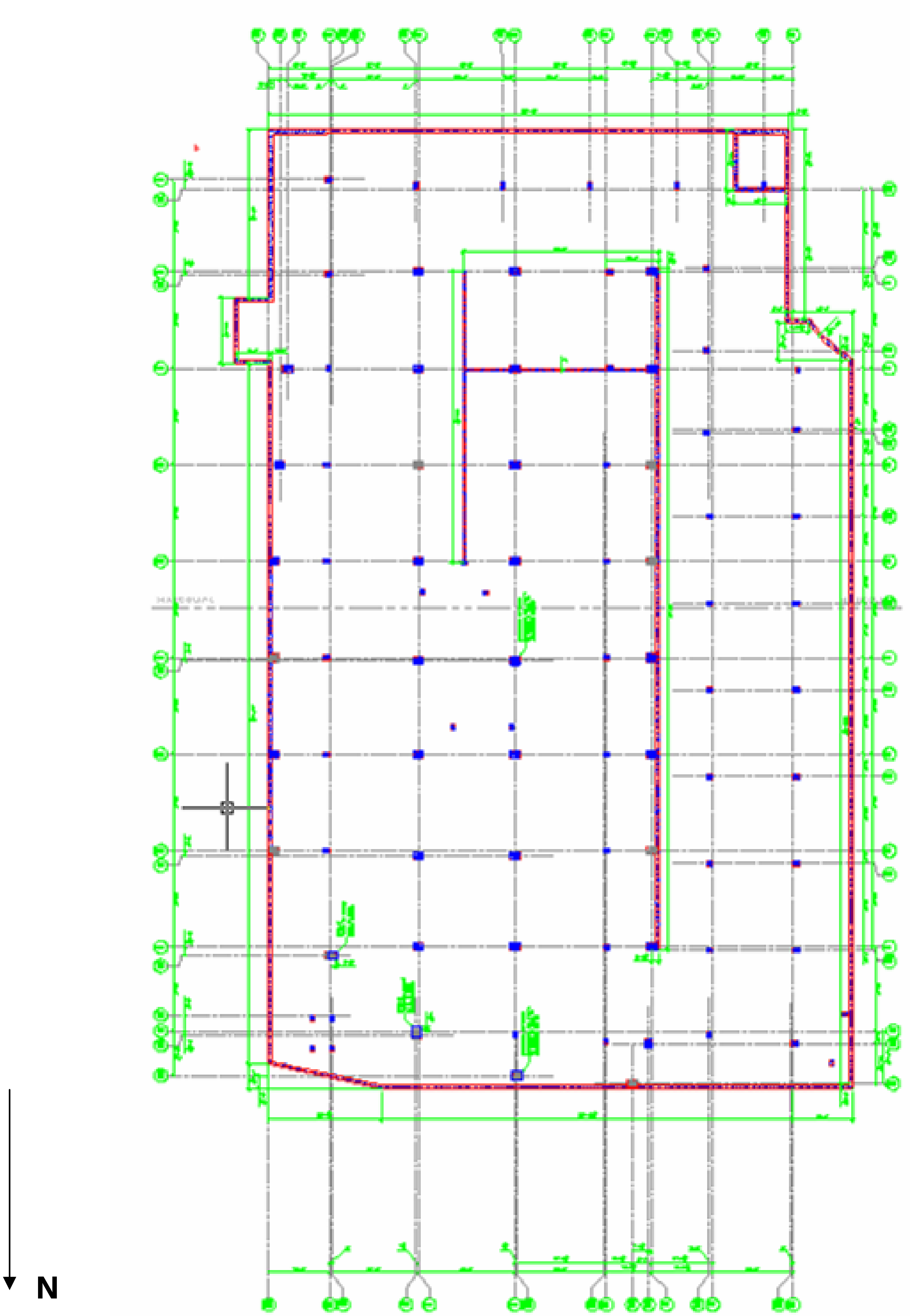
Note: Shaded area is roof construction



11^{th} and 12^{th} Floor Framing Plan



Concrete Column and Wall Layout for the Parking Levels Below Grade



Elevations

Architect: Cooper Carry Architects

The Regent's Southeastern corner and East Elevation looking across Glebe Road

Architect: Cooper Carry Architects

The Regent's Northern Elevation as seen from Glebe Road across North Fairfax Drive

Codes and Code Load Requirements

The 2000 ICC International Building Code (IBC 2000) was used for the structural design of The Regent. IBC 2000 incorporates many of the design load procedures of ASCE 7. ASCE 7-02 was used for calculating the snow loads and roof live loads. The live loads were taken from Table 1607.1 of IBC 2000. The equations, tables, and procedures used to calculate the design loads listed in this report were taken from ASCE 7-02. LRFD was used for the existing structural design.

Steel System Loads

Gravity Loads

Dead Loads

- Roof
  - 3" - 22 Gage Metal Deck — 5 PSF
  - Insulation — 3 PSF
  - Misc. DL — 10 PSF
  - Roofing — 20 PSF

- Typical Floor
  - 3 ¼" lt. wt. slab on 3" - 20 gage metal deck (United Steel Deck design manual p. 40) — 46 PSF*
  - Concrete Ponding — 10 PSF*
    *included because of the long steel spans and cambers
  - Misc. DL — 15 PSF
    (mechanical ducts, sprinklers, ceiling, plumbing, etc.)

- Construction Loads
  - 3 ¼" lt. wt. slab on 3" -20 gage metal deck — 46 PSF*
  - Concrete Ponding — 10 PSF*

*NOTE: The slab on metal deck will be unshored during construction.

Live Loads (IBC 2000, Table 1607.1)

- **Corridors** — **100 PSF**
- **Stairs** — **100 PSF**
- **Mechanical Spaces** — **150 PSF**
- **Offices** — **100 PSF***
  ***Includes 20 PSF Partition Load**
  - Lobbies and 1st Floor Corridors — 100 PSF *Critical Case
  - Offices — 50 PSF
  - Corridors above 1st Floor — 80 PSF
- **Retail – 1st Level** — **100 PSF**
- **Terrace Above 1st Floor Retail** — **100 PSF**
  - Deck (Roof/Patio) – same as occupancy served (Office) — 100 PSF
  - Balcony – exterior — 100 PSF
- **Loading Dock** — **350 PSF**
  - *Designed for Arlington Fire Dept. Tower 75-1987 (total weight = 66,320#) — 350 PSF *Critical Case
- **Parking Garage (Garages having trucks and busses)** — **50 PSF**
  - IBC 2000 1607.6
  - Truck and bus access provided to loading dock on 1st level
- **Plaza Deck (Fire Truck Loading)** — **350 PSF**
  - Vehicular Driveways — 250 PSF
  - *Designed for Arlington Fire Dept. Tower 75-1987 (total weight = 66,320#) — 350 PSF *Critical Case

- **Snow Load** — 30 PSF
- **Construction Live Load** (unreducible) — 20 PSF
- **Roof Live Load** (as calculated per ASCE 7-02) — ~~12 PSF~~
  - Snow Load controls — 30 PSF
  - Mechanical — 150 PSF

Snow Load and Roof Live load calculations can be found in Appendix I.

Lateral Loads

The wind and seismic loads calculations are included in Appendix I.

Load Cases and Controlling Lateral Forces

Load Combinations Involving Wind Loads (W) and Seismic Loads (E)

ASCE 7-02 (Sec. 2.3.2)

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

Check 1.6W vs. 1.0E

Red = Controlling E-W Lateral Force, Blue = Controlling N-S Lateral Force

	1.6W (N-S)	1.6W (E-W)	1.0E (N-S/E-W)
Roof	60.16	**93.72**	**60.96**
12	82.32	**128.64**	**84.58**
11	45.55	**74.59**	**70.55**
10	44.91	**83.57**	**73.27**
9	43.95	**82.05**	**63.70**
8	42.77	**80.14**	**54.40**
7	41.42	**77.98**	**45.40**
6	40.19	**87.89**	**42.28**
5	**38.78**	**107.92**	32.75
4	**37.07**	**82.13**	23.74
3	**35.06**	**78.43**	15.36
2	**37.64**	**85.79**	7.94

After reviewing all of the load combinations for ASCE 7-02, it was determined that wind will control the lateral design in the east / west direction and seismic will control the north / south direction from the roof down to the 6th floor at which point wind will control. Only the load combinations involving wind and seismic were considered to calculate the worst case lateral loading since they are the only two loads considered in a lateral direction.

Alternative Floor System Design Considerations

Four alternative floor system designs were analyzed and designed in Technical Report 2: Pro-Con Structural Study of Alternate Floor Systems. These four alternative floor systems include:

- Hollow-Core Planks with Steel Framing System
- One-way Wide Module Joists, Multiple Spans, with Cast-In-Place Framing System
- Precast Double Tees with Precast Framing System
- Two-way Flat Slab with Drop Panels with Cast-In-Place Framing System

Each alternative floor system design was discussed and their advantages and disadvantages were compared amongst each other and to the existing floor framing system.

A system comparison chart was compiled for and is reproduced from Technical Report 2: Pro-Con Structural Study of Alternate Floor Systems below.

Existing and Alternative Floor System Comparison Chart

System	Pros	Cons	Considerations
Existing Composite Slab on Metal Deck with Composite Steel Beams and Steel Framing	• Lighter structure • Quick construction • Smaller foundations • Relatively small depths • Smaller columns sizes • Can efficiently accommodate longer spans	• Concrete ponding over the long spans • Lots of beams	• None at this point
Precast Hollow-Core Planks / Steel Framing	• Quick construction • Relatively smaller foundations • Lighter structure • Smaller column sizes • Quality control • Relatively small depths • Less steel beams needed per bay • Good fire rating • Good acoustical value	• Lots of deliveries to a downtown site • Angle detailing to support the planks • Deeper, heavier steel members	• Composite action between the steel beams and the hollow-core planks • Prefabrication of angles to the webs • Adding infill beams to get smaller beam and plank sizes
Precast Double Tees / Precast Framing	• Quick construction • Quality control • Good fire resistance • Can accommodate longer spans • Less labor intensive • Less labor costs • Good acoustical value • Double tee self weight comparable to slab on deck weight	• Larger foundations • Deep flooring system • Heavy beams and columns • Lots of deliveries to a downtown site	• Smaller bay sizes • Shallower supporting members (not flush)
CIP One-way Wide Module Joists / CIP Framing	• Uniform depth • Rigid floor system • Slab and supporting beam depths are less than existing depths • Can accommodate longer spans • Good fire rating	• Larger foundations • Heavy structure • Labor intensive • Longer construction time • More field labor intensive • Larger column sizes • Forming and shoring system required	• Smaller bay sizes, more columns
CIP Two-way Flat Slab with Drop Panels / CIP Framing	• Good fire resistance	• Not practical from a constructability, cost, labor, standpoint for the existing bay sizes • Very heavy structure • Larger foundations • Larger column sizes • Extensive forming and shoring systems required	• Two-way post-tensioning • Smaller bay sizes, more columns

Based off of the initial study, all of the alternative floor systems were selected to be studied further except the Two-way Flat Slab with Drop Panels with Cast-In-Place Framing System for the following reasons:

- Not practical from a constructability, cost, and labor standpoint for the existing bay sizes (minimum slab depth = 16.5", 21" at the drop panels)
- Very heavy structure, significantly heavier than the existing design (≈210PSF vs 56 PSF)
- Would require significantly larger foundations
- Larger column sizes required
- Extensive forming and shoring systems required

The initial design team goals and the original design were then taken into consideration. They are listed below:

- Cost
- Quick construction
- Typical floor to floor height 13' (existing system)
- Typical floor to ceiling height = 9' (existing system)
- Keep existing column layout to keep open floor layout for tenant flexibility
- Lighter structure = lighter foundations = less cost (existing system)
- Maximum height restrictions ≈ 181' (existing system)

System	Reasons for Elimination
Precast Double Tees with Precast Framing System	• The depth of this system was exactly 4' which is significantly deeper than the existing system, which has a maximum depth of 30.25". This means that the floor to ceiling height would be reduced. **(DEPTH)**
Precast Hollow-Core Planks / Steel Framing	• In order to minimize the depth of the floor system, the planks would require angles connected to the web of the steel beams. Fabrication and detailing of the angles would be very expensive. Also, the size of the beams increased significantly over the existing system due to the loss of composite action between the concrete on deck and the beams. **(COST, DEPTH)**
One-way Wide Module Joists / CIP Framing	• The weight of this system is significantly greater than the existing system. Also, since everything in this system is cast-in-place, this system would take long to erect. However, the depth of this system is comparable to the existing system. **(TIME, WEIGHT)**

Based off of the previously mentioned initial design team goals and alternative floor system research and analysis, it is determined that the existing structural system is the most efficient design to meet the needs of the building, the project team, the schedule, and the site.

Proposal

Proposal Problem Statement

Based off of the study, research, analysis, and designs of the existing system and the four alternative systems, it was determined that the existing steel system is the most efficient design to meet the needs of the building, the project team, the schedule, the budget, and the site. Ideas for a redesign of the existing structure to make it a more efficient structure are difficult to find, if they even exist.

Having studied the existing steel structure last semester, I wanted to challenge myself this semester by proposing to do a design of this building using a concrete system. Although my initial conclusions are that the existing steel design is the most appropriate for this building, I wanted to do a structural design of The Regent using a concrete system in order to make comparisons between the two systems.

The criteria for the existing design were discussed in the previous section. A concrete system design shall be selected that meets as many of the criteria as possible in order to make a fair comparison between the concrete system and the existing steel system.

Comparisons between the two systems will be based on the following:

- Cost
- Schedule
- Labor
- Floor to floor height
- Floor to ceiling height
- Weight
- Impact on the foundations

Proposed Solution to the Problem

Floor System

In reviewing the results of the alternative floor systems involving concrete design in Technical Report 2, it has been decided to explore the following concrete system in the redesign of The Regent.

- One-way Joists, Wide Module, with all Cast-In-Place Framing

In comparison to the other concrete systems considered, this concrete system is expected to be the lightest in weight and the shallowest in depth and also able to accommodate longer spans.

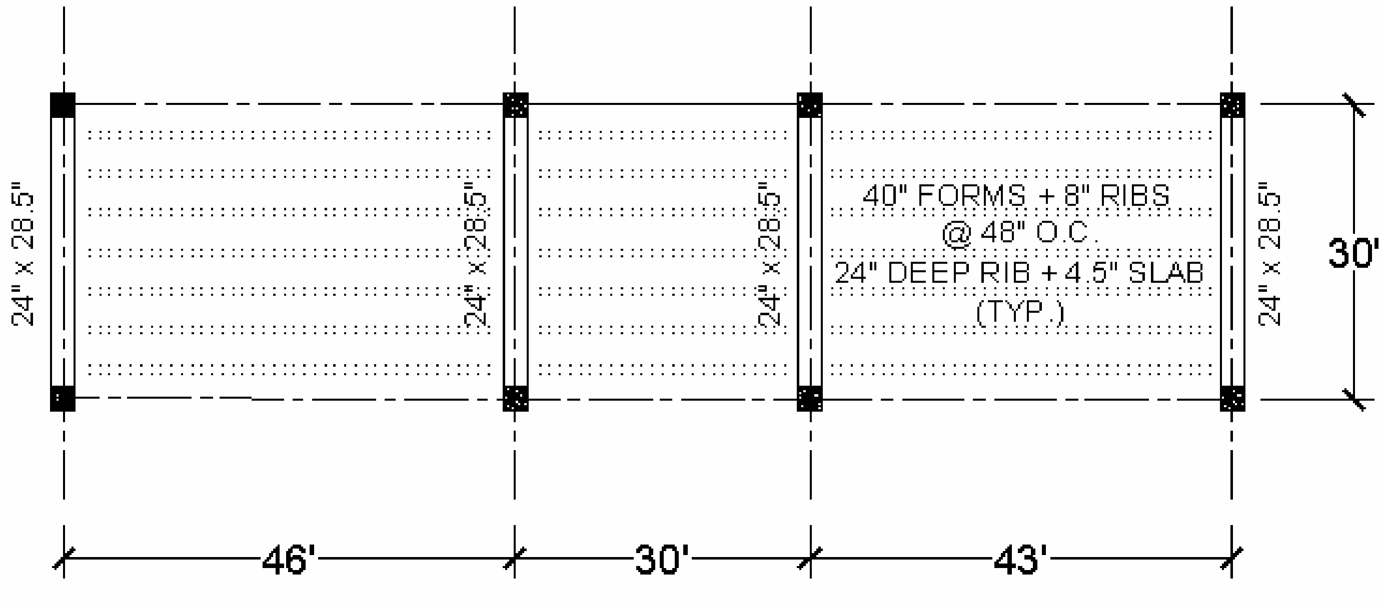
The goal is to keep the same column layout as the existing steel system in order to keep the original architectural design intention of an open floor plan.

The starting point for the design of the wide module joists was the results of the preliminary design from Technical Report 2.

The one-way joists with CIP framing system was preliminarily designed in Technical Report 2 using the CRSI Design Handbook. The preliminary design for the joists and the girders is sketched below.

One-way Joists, Wide Module, with Cast-In-Place Framing

Typical Floor Framing Plan for One-way Wide Module Joists with Cast-In-Place Framing System Design



Joist Selection: 40" Forms + 8" Ribs @ 48" o.c.
24" Deep Rib + 4.5 "Top Slab = 28.5" Total Depth
f'c = 4,000 psi
fy = 60,000 psi

End Span: 764 PLF < 873 PLF ∴ OK

Top Bars: #7 @ 9"
Bottom Bars: 1 - #10 and 1-#10
Stirrups: #3 @ 13" for 204"

Interior Span: 764 PLF < 926 PLF ∴ OK

Top Bars: #6 @ 7"
Bottom Bars: 1 - #8 and 1-#9
Stirrups: #3 @ 13" for 167"

Interior Beam Selection:
24” x 28.5”
Top: (5) #14
Bottom: (2) #14
Stirrups (Closed): (16) #5, 1@2”, 25@7”
12.5 PLF > 10.83 PLF ∴ **OK**

Exterior Beam Selection:
24” x 28.5”
Top: (4) #14
Bottom: (2) #14
Stirrups (Closed): (23) #5, 1@2”, 22@8”
10.1 PLF > 6.9 PSF ∴ **OK**

Lateral Force Resisting System

The existing structure utilizes a series of 5 braced frames; 2 spanning in the north / south direction and 2 spanning in the east / west direction. Since the redesign will be an all concrete system, a series of concrete shearwalls will be designed as the lateral force resisting system. These shearwalls will ideally be placed around the elevator core, concrete stairwells, and/or in the existing braced frame locations.

Proposal Solution Method

The design of the concrete structure will be based off of ACI 318-02: Building Code Requirements for Structural Concrete. Analysis for gravity loads will be completed by hand calculations and/or through the use of structural analysis and design software such as PCACOL. Analysis of lateral loads will be completed using ETABS. Live load loading patterns will be considered and used to properly design the concrete gravity system.

Scope of Structure to be Designed (Above Grade Superstructure Only)

- Floor System - One-way Joists, Wide Module
- Cast-In-Place Beams
- Cast-In-Place Columns
- Lateral Load Resisting Shearwalls
- Representative Foundations
- Roof

Breadth Analyses

Construction Management

Since two of the key factors in selecting the existing structural system were cost and speed of erection, a construction management breath analysis will be conducted to estimate the cost and scheduling differences between the existing steel system and the concrete system. Since it already has been initially predicted that the existing system is the most cost effective and the quickest to erect, the cost and schedule comparison will be used to determine approximately how much time and money was saved by going with the steel system, if the initial assumption was correct.

Mechanical

Since the concrete system design will most likely have a new depth and framing layout, the mechanical system sizes and layout may not be compatible with the new spatial requirements and layout of the new concrete system. The impact of the concrete system design on the mechanical system layout will be analyzed, and if there are conflicts with space and layout between the new concrete structure and the existing mechanical system, solutions to the conflicts will be proposed.

Structural Depth Study:

Cast-In-Place Concrete Design of The Regent

Structural Depth Study Overview

This structural breadth study is a structural design of The Regent using a cast-in-place concrete system. The scope of this depth study includes the design of the CIP wide module joists, girders, columns, shearwalls, roof system, and two representative spread footing designs. In most cases, similar members were designed together according to the worst case loading. One of the main purposes of this depth study is to get experience designing a concrete system and to become more familiar with design processes, codes, and the computer design and analysis programs associated with designing a concrete system.

This depth study is broken down into the following six sections each with a corresponding Appendix which contain the necessary calculations and spreadsheets used for design.

1. CIP Joist Designs	Appendix A
2. CIP Girder Designs	Appendix B
3. CIP Column Designs	Appendix C
4. CIP Shearwall Designs	Appendix D
5. Representative Spread Footing Designs	Appendix E
6. Roof Design	Appendix F

Each section describes the design procedures, references, and computer programs used for the concrete system design. Also, each section summarizes the loads and final designs for each piece of the structure. More detailed design calculations for each section can be found in their corresponding Appendices. Most of the calculations and spreadsheets were included in this report and/or the Appendices. If further calculations and/or computer output are necessary in order to understand or clarify the design processes used, they are available upon request.

Codes and Code Load Requirements

The 2000 ICC International Building Code (IBC 2000) was used for the steel structural design of The Regent and was also used for the concrete design of The Regent. IBC 2000 incorporates many of the design load procedures of ASCE 7. ASCE 7-02 was used for calculating the design wind loads, seismic loads, snow loads and roof live loads for the cast-in-place concrete system. The live loads were taken from Table 1607.1 of IBC 2000 and are the same as for the steel system. The equations, tables, and procedures used to calculate the design loads listed in this section were taken from ASCE 7-02. ACI 318-02 was used for the design of the cast-in-place concrete system. LRFD was used.

Loads

Dead Loads

- Roof 38 PSF
  - Metal Roof Deck
  - Steel Joists
  - Insulation
  - Built-up Roof (5-ply felt and gravel)
  - SDL

- Typical Floor
  - 24” Joists w/ 4.5” Slab 119 PSF
  - 16” Joists w/ 4.5” Slab 95 PSF
  - SDL 15 PSF

- Construction Loads
  - 24” Joists w/ 4.5” Slab 119 PSF
  - 16” Joists w/ 4.5” Slab 95 PSF

Live Loads (IBC 2000, Table 1607.1)

- **Corridors** **100 PSF**
- **Stairs** **100 PSF**
- **Mechanical Spaces** **150 PSF**
- **Offices** **100 PSF***
  ***Includes 20 PSF Partition Load**
  - Lobbies and 1st Floor Corridors 100 PSF *Critical Case
  - Offices 50 PSF
  - Corridors above 1st Floor 80 PSF
- **Retail – 1st Level** **100 PSF**
- **Terrace Above 1st Floor Retail** **100 PSF**
  - Deck (Roof/Patio) – same as occupancy served (Office) 100 PSF
  - Balcony – exterior 100 PSF
- **Loading Dock** **350 PSF**
  - *Designed for Arlington Fire Dept. Tower 75-1987 (total weight = 66,320#) 350 PSF *Critical Case
- **Parking Garage (Garages having trucks and busses)** **50 PSF**
  - IBC 2000 1607.6
  - Truck and bus access provided to loading dock on 1st level
- **Plaza Deck (Fire Truck Loading)** **350 PSF**
  - Vehicular Driveways 250 PSF
  - *Designed for Arlington Fire Dept. Tower 75-1987 (total weight = 66,320#) 350 PSF *Critical Case

- **Snow Load** 30 PSF

- **Construction Live Load** (unreducible) 20 PSF

- **Roof Live Load** (as calculated per ASCE 7-02) ~~12 PSF~~
  - Snow Load controls 30 PSF
  - Mechanical 150 PSF

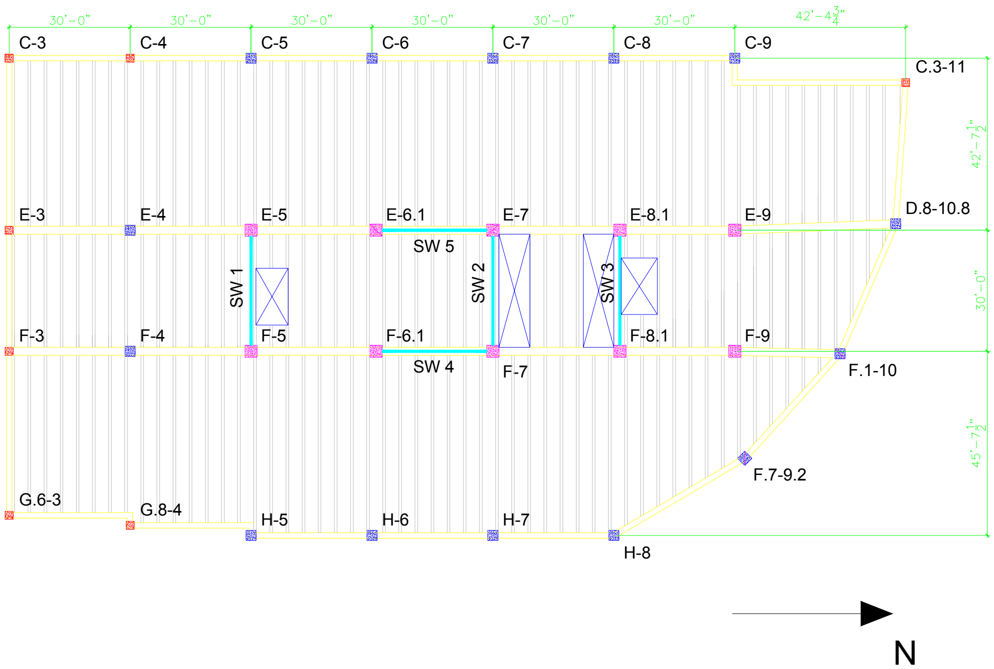
Snow Load and Roof Live load calculations can be found in Appendix I.

Lateral Loads

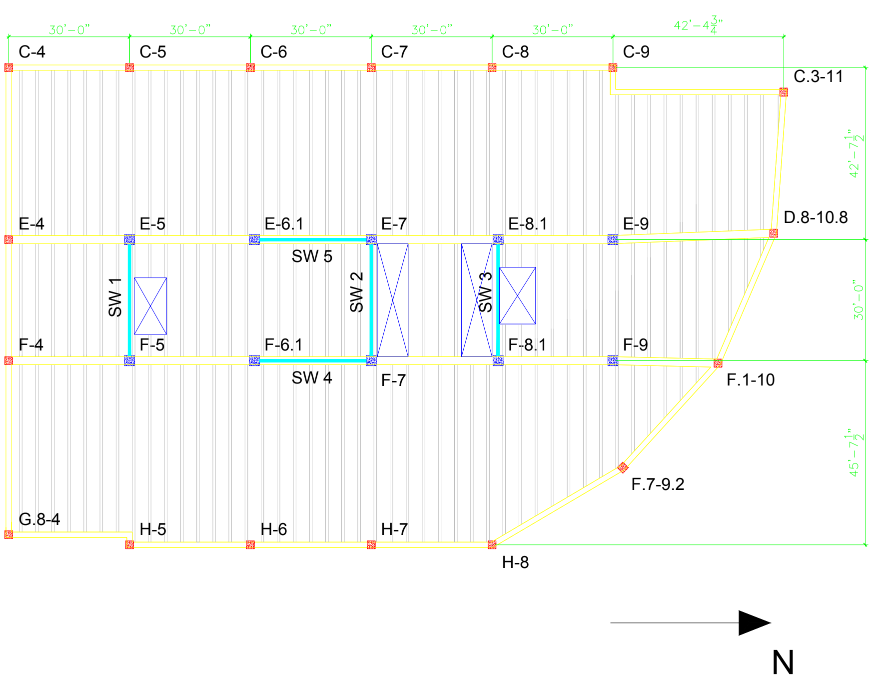
The wind and seismic load calculations for the concrete system can be found in Appendix I.

Cast-In-Place Concrete Design Plans

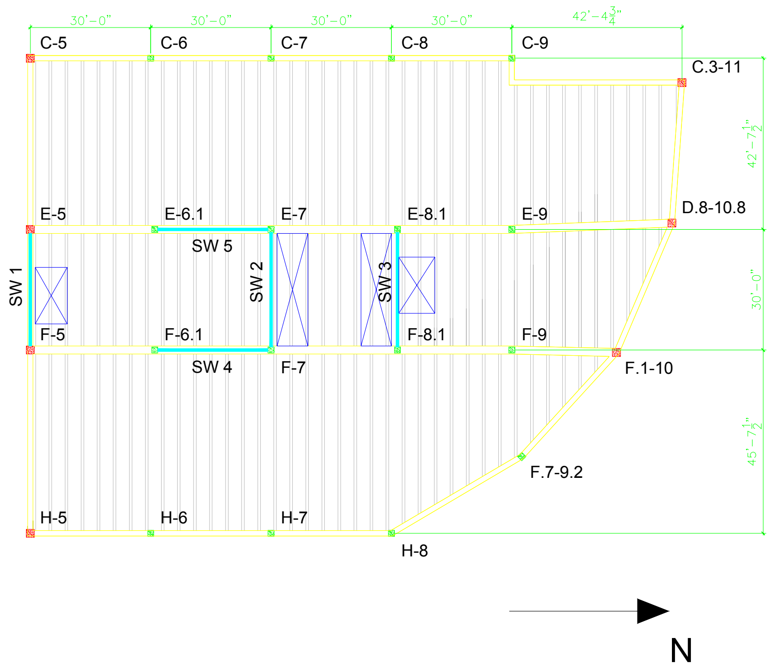
2nd – 5th Floor Plan



6th – 9th Floor Plan



10th – 12th Floor Plan



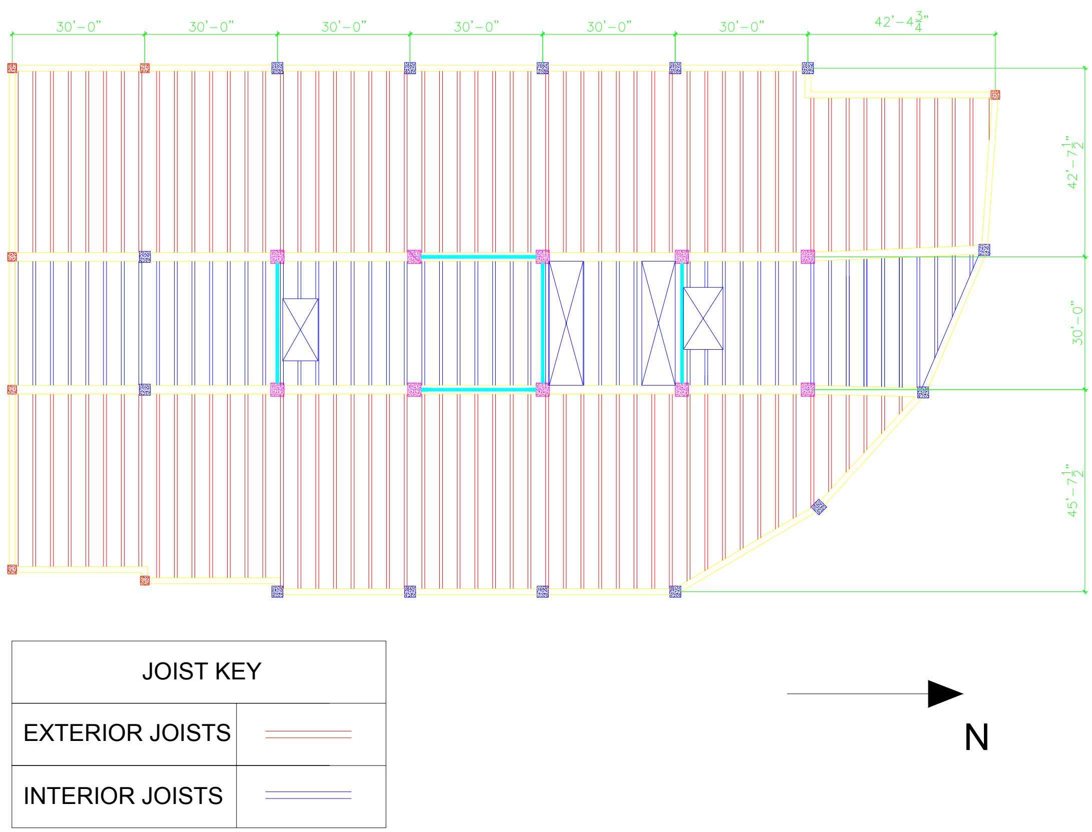
CIP Floor Joist Designs

The cast-in-place floor system consists of one-way, wide module joists that span in the East/West direction across the 46', 30', and 43' bays, respectively. The joists were chosen to span in this direction as a result of a preliminary design of the floor system which yielded a more efficient design if the joists spanned in the East/West direction and the girders spanned in the North/South direction.

For this floor system design, the 43' span was assumed to be equivalent to the 46' span. Therefore, there were two different spans to design for; a 46' span and a 30' span. The 46' span joists span the two exterior bays and will be referred to as the exterior joists. The 30' span joists span the 30' interior bay and will be referred to as the interior joists. The joists will be cast monolithically with girders which run in the North/South direction. The joists were designed as continuous across the three bays.

The following plan shows which joists are considered exterior joists and which joists are considered interior joists.

Joist Designation Plan



A wide module joist system with a 4.5” slab was selected in order to better accommodate the longer spans and also to meet the minimum slab requirements for a fire resistance rating. The CRSI Design Handbook was used to find initial trial joist sizes that were able to span 46’ and 30’ while being able to carry the gravity design loads.

Initial Joist Sizes

Span	Form Size	Rib Size	Total Width	Rib Depth	Total Depth
46’	40”	8”	48”	24”	28.5”
30’	40”	8”	48”	16”	20.5”

The design gravity loads for both interior (30’ span) and exterior (46’ span) joists are listed below. The live loads could not be reduced because the tributary area for each joist was less than 400 SF, which is the minimum tributary area to be able to consider live load reduction according to ASCE 7-02, Section 4.8.1.

Gravity Loads

Span	Self Weight (including slab) (PSF)	SDL (PSF)	Live Load (PSF)		Tributary Width
46’	119	15	100	Office	4’
30’	95	15	150	Mechanical	4’

Since the interior joists span across the center bay of the building which includes the mechanical space for each floor, a different joist size was selected in order to try to minimize the floor depth above those mechanical spaces. Also, the 30’ span can use a smaller joist size (16”) than the 46’ span (24”), which would save material, time, and labor over using the 24” joists across the entire floor.

The design moments and shears found in ACI 8.3.3 for a one-way slab system could not be used because the larger of two adjacent spans (46’) was greater than the shorter span (30’) by more than 20%. Moment distribution, with live load pattern loading, was used to calculate the design moments and shears for the joists. The design moments and shears are summarized below and the moment distribution calculations are included in Appendix A.

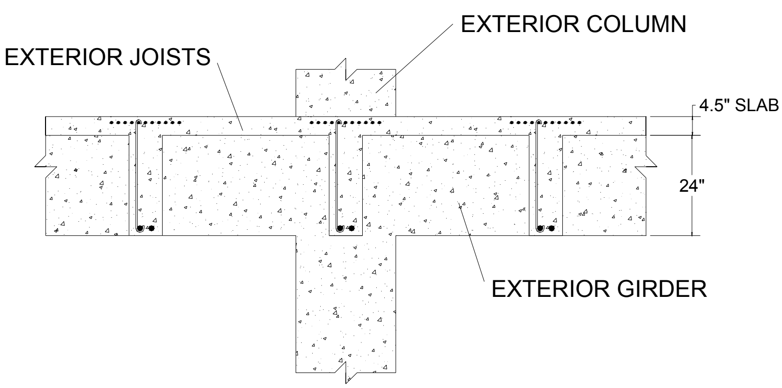
46’Span (24”) Joists

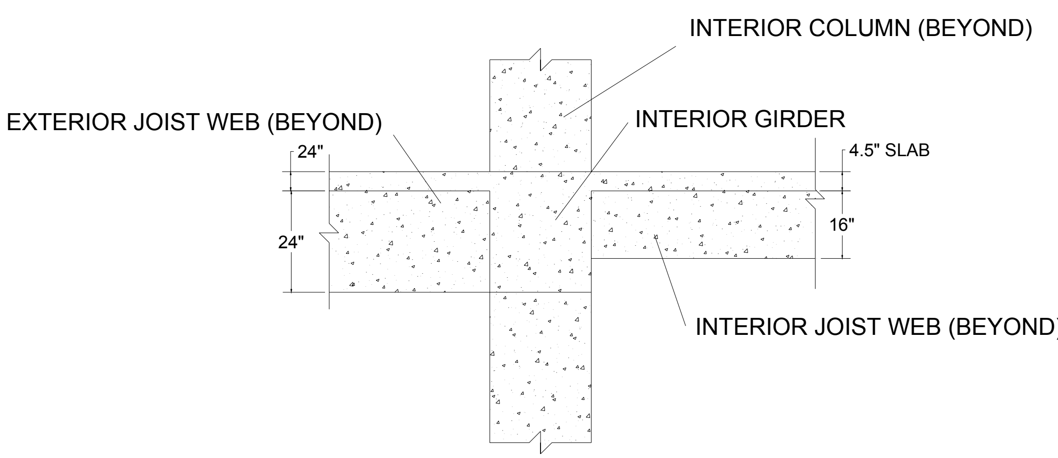
$\mathbf{M_u^+}$	279 ft-k
$\mathbf{M_u^-}$	199 ft-k
$\mathbf{V_u}$	33.9 k

30' Span (16") Joists

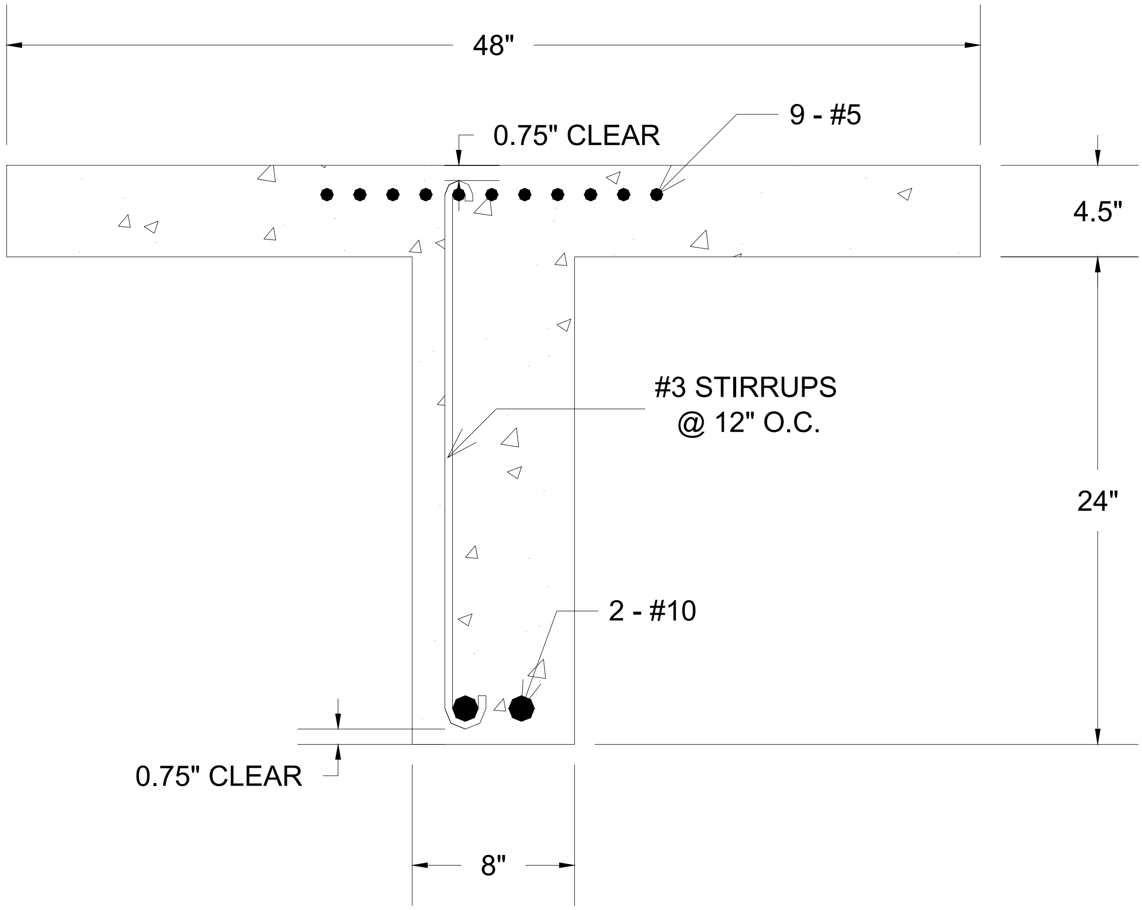
M_u^+	23.65 ft-k
M_u^-	199 ft-k
V_u	33.9 k

The joists were designed as tee beams with a flange thickness of 4.5" (slab thickness) for flexure and shear using ACI 318-02, Chapters 10 and 11, respectively. The concrete strength selected for design is f'c = 4,000 psi, which is a common concrete strength for office buildings and the reinforcement is 60 ksi steel. The joists will be cast monolithically with the girders and the columns. The calculations for the design of the joists for flexure and shear are included in Appendix A. Punching shear was not a concern since the joists frame into girders and columns as shown in the two details below.

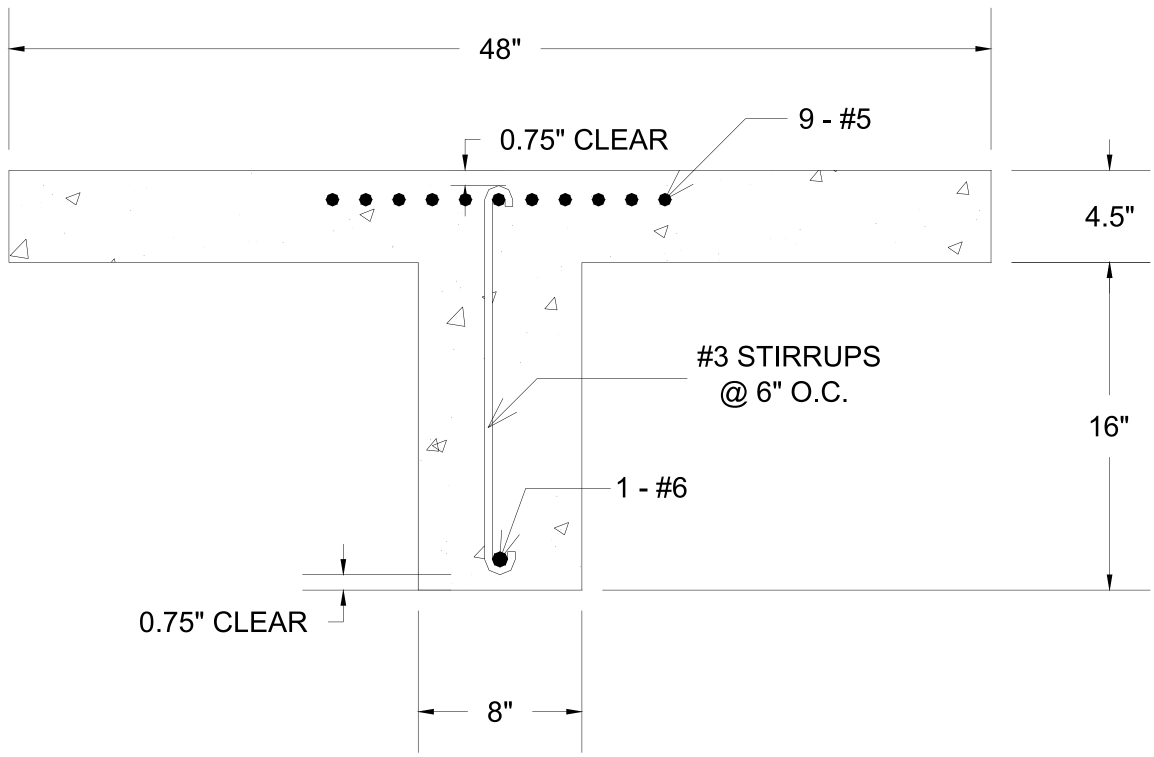




Exterior Joist Design



Interior Joist Design



After the joists were designed, their anticipated deflections were compared to an allowable deflection for the total load of l/360 and an allowable deflection for live load of l/480. The both joist designs met these design criteria.

The following tables summarize and compare the actual and allowable loads and deflections for both joist designs.

Summary of Actual and Allowable Loads and Deflections

46'/43' Span (24") Joists   24 + 8 + 40 Joists

$\mathbf{M_u^+}$	279 ft-k	ϕM_n^+	292 ft-k	OK
$\mathbf{M_u^-}$	199 ft-k	ϕM_n^-	199 ft-k	OK
$\mathbf{V_u}$	33.9 k	ϕV_n	40.6 k	OK
$\mathbf{\Delta_{TL}}$	0.75"	$\mathbf{\Delta_{TL,allow}}$ **(l/360)**	1.5"	OK
$\mathbf{\Delta_{LL}}$	0.325"	$\mathbf{\Delta_{TL,allow}}$ **(l/480)**	1.15"	OK

30' Span (16") Joists 16 + 8 + 40 Joists

M_u^+	23.65 ft-k	ϕM_n^+	36 ft-k	OK
M_u^-	199 ft-k	ϕM_n^-	199 ft-k	OK
V_u	33.9 k	ϕV_n	38.6	OK
Δ_{TL}	0.41"	**$\Delta_{TL,allow}$ (l/360)**	1"	OK
Δ_{LL}	0.24"	**$\Delta_{TL,allow}$ (l/480)**	0.75"	OK

In conclusion, the all of the design moments, shears, and deflections are less than the allowable, therefore both joist designs are okay.

The following is a final schedule of the cast-in-place concrete wide module joists.

CIP One-Way, Wide Module Pan Joist Schedule

	Size							
Span	Forms	Ribs	Rib Depth	Slab Depth	Total Depth	I_g	A	Self Weight
24 + 8 + 40	40"	8"	24"	4.5"	28.5"	32,297 in^4	456 in^2	119 PSF
16 + 8 + 40	40"	8"	16"	4.5"	20.5"	12,128 in^4	381 in^2	95 PSF

	Reinforcement		Stirrups		
Span	Bottom Bars	Top Bars	Size	Type	Spacing
24 + 8 + 40	(2) #10	(9) #5	#3	Single Leg	12"
16 + 8 + 40	(1) #6	(9) #5	#3	Single Leg	6"

f'c =	4,000	psi
fy =	60,000	psi

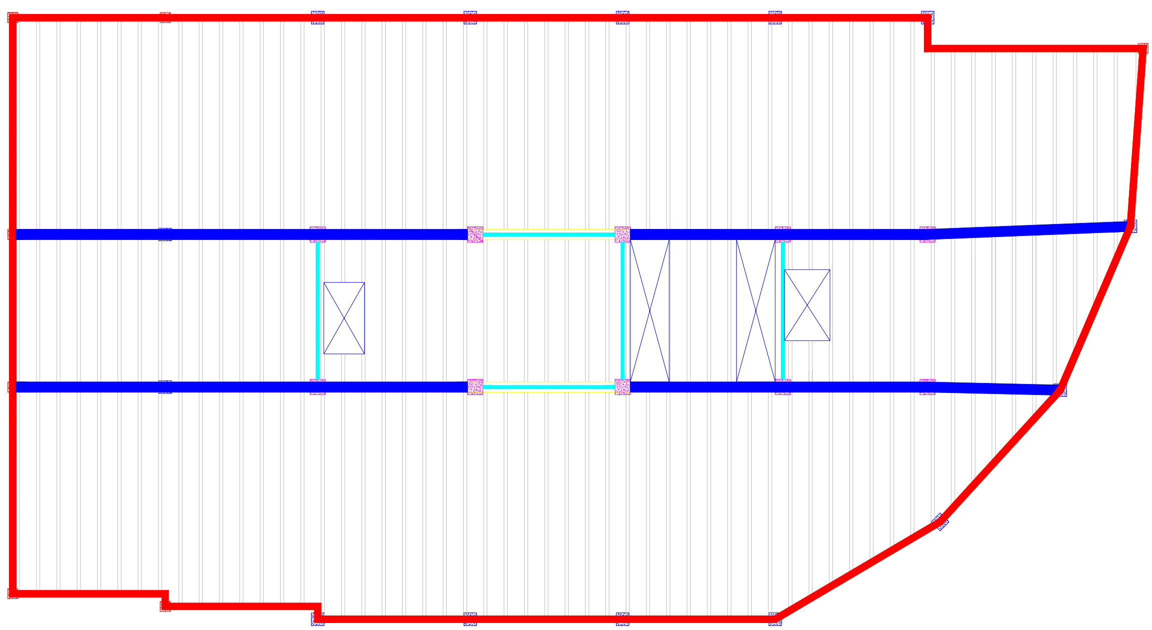
CIP Girder Designs

The building was assumed to be approximately symmetric about its North/South axis for simplicity of design. As a result, in the North/South direction, two different girder sizes needed to be designed in order to carry the loads of the CIP wide module joists which span in the East/West direction; an exterior girder size and an interior girder size. Both the interior and exterior girders spanning in the North/South direction all have a span of 30’, since the columns are spaced at 30’ o.c. in the North/South direction.

The exterior girders that span in the East/West direction are not carrying much load from the joists because the joists span parallel or almost parallel to these girders. However, these exterior girders are necessary for carrying the façade loads. The loads of the exterior girders spanning in the East/West direction are significantly less than the exterior girders spanning in the North/South direction. For simplicity of design, the girders spanning in the East/West direction will be assumed to have same design as the exterior girders spanning in the North/South direction, even though in reality they could be designed for the lighter loads they are actually carrying.

The following plan shows which girders are considered interior girders and which girder are considered exterior girders.

Girder Designation Plan



GIRDER KEY	
EXTERIOR GIRDER	(red line)
INTERIOR GIRDER	(blue line)

A design requirement for both the interior and exterior girders was to have a girder depth of 24" plus the 4.5" slab depth for a total girder depth of 28.5". This depth requirement was necessary in order for the girders to same depth as the 24" joists which will be cast monolithically with both the exterior and interior girders. The 28.5" girder depth can then accommodate the bottom joist reinforcement that will either continue through the girder or hook into the girder.

The initial trial size for the both the interior and exterior girders as determined from the CRSI Design Handbook was 24" x 28.5" including the 4.5" top slab. This initial trial size was based off of a load combination of 1.4D + 1.7L which exceeds the current load combination of 1.2D + 1.6L. The results of hand calculations concluded that a width of

16” would work for the exterior girder and a width of 24” would work for the interior girder.

The design gravity loads for the interior and exterior girder are summarized below. The office live load of 100 PSF was reduced based off of the tributary area for each girder; however, the mechanical live load of 150 PSF could not be reduced because it exceeded 100 PSF.

Gravity Loads

Girder	Self Weight (PLF)	SDL (PSF)	Joists and Slab (PSF)	Façade (PLF)	LL (PSF)	Reduced LL (PSF)	Tributary Width (FT)	Space
Exterior	400	15	119	310	100	65	23	Office
Interior	600	15	119 95	N/A	100 150	54 N/A	38	Office Mechanical

Since the girders met the requirements of ACI 318-02, Section 8.3.3, these moment and shear equations were used to find the design moments and shears for both the interior and exterior girders. The girders also have a design moment due to the 25% seismic load that was applied to the girder and column moment frame system as a requirement of ASCE 7-02, Chapter 9, Section 9.5.2.2.1. The frames were designed to take 25% of the seismic load in the event that the shearwalls would fail. Since the girders have moments from live, dead, and seismic loads, three different load combinations were calculated in order find the worst case moments on the girders.

Load Combinations

1. 1.2D + 1.6L
2. 1.23D + L + E
3. 0.93D + E

$$E = \rho Q_E + 0.2 S_{DS} D$$
$$E = (1)Q_E + 0.2(0.153)D$$
$$E = Q_E + 0.03D$$

The controlling load combination was 1.23D + E + L.

The torsional loads for the exterior girder were taken as the fixed end moments from the exterior joists and the torsional loads for the interior girder were taken as the difference in fixed end moments of the 16” and 24” joists it supports on either side.

Detailed calculations for the design moments and shear are included in Appendix B. A summary of the design moments, shear, and torsion are listed below.

Design Loads for the Girders

Interior Girders

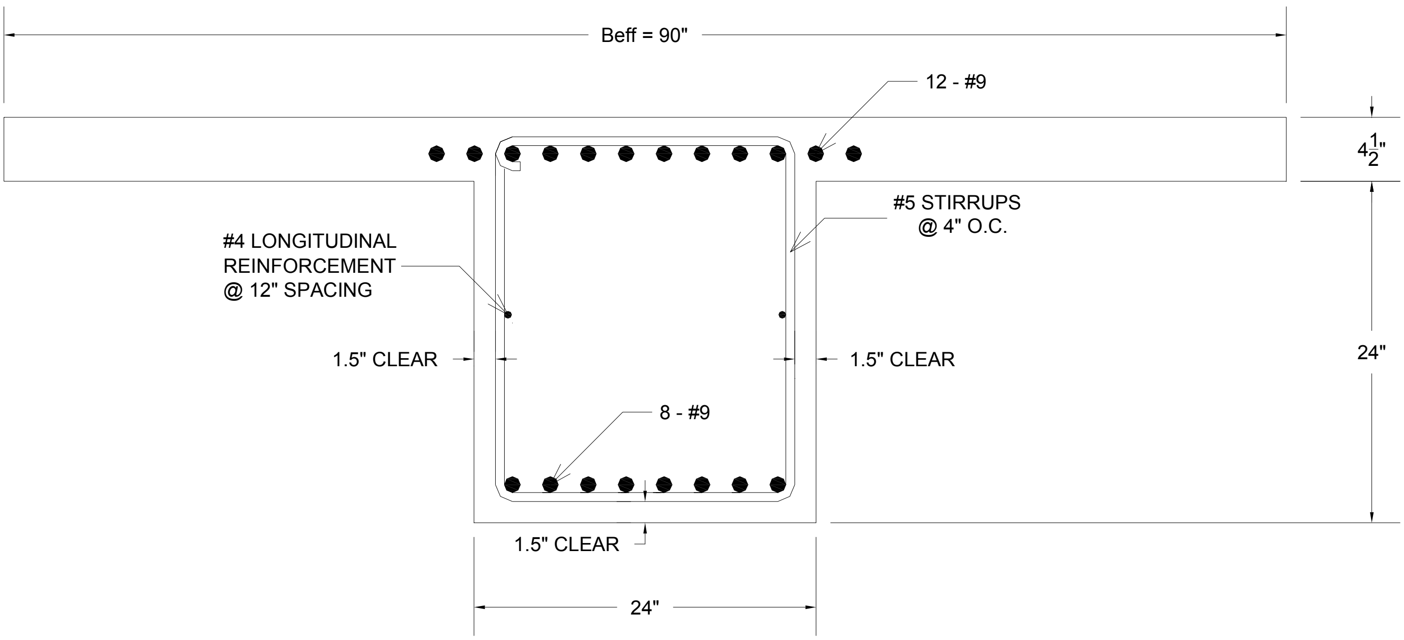
M_u^+	782 ft-k
M_u^-	1094 ft-k
V_u	200 k
T_u	69.7 ft-k

Exterior Girders

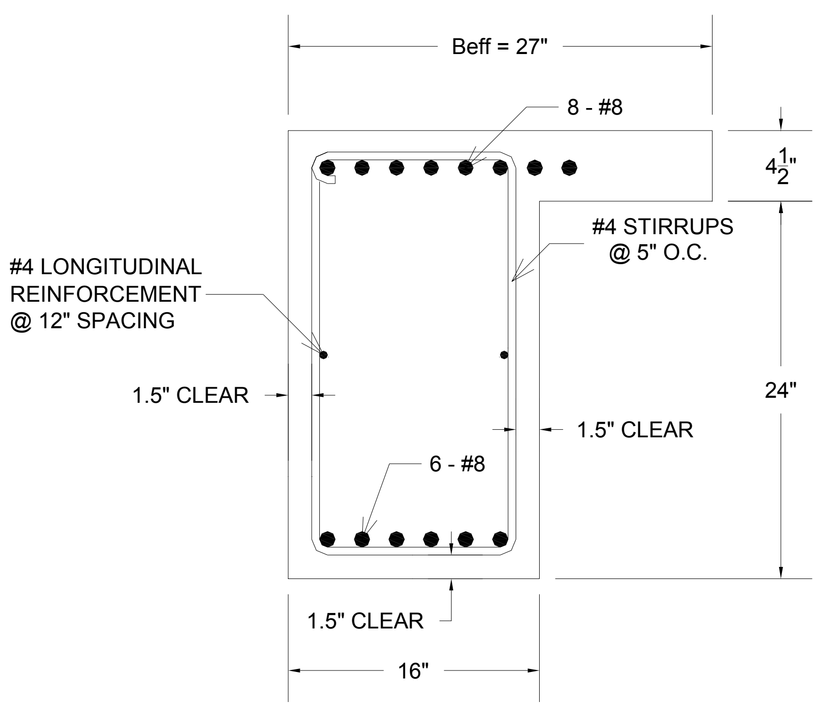
M_u^+	448 ft-k
M_u^-	627 ft-k
V_u	115 k
T_u	114 ft-k

The girders were designed as tee beams with a flange thickness of 4.5” (slab thickness) for flexure, shear, and torsion using ACI 318-02, Chapters 10 and 11, respectively. The concrete strength selected for design is f’c = 4,000 psi, which is a common concrete strength for office buildings and is the same as the joists. The flexural and shear reinforcement is 60 ksi steel. The development lengths for the flexural reinforcement and hooks shall be based off of the provisions of ACI 318-02, Chapter 12. The girders will be cast monolithically with the joists and the columns. The calculations for the design of the girders for flexure, shear, and torsion are included in Appendix B.

Interior Girder Design



Exterior Girder Design



After the girders were designed, their anticipated deflections were compared to an allowable deflection for the total load of l/360 and an allowable deflection for live load of l/480. The both girder designs met these design criteria. The girder deflection calculations are included in Appendix B.

The following tables summarize and compare the actual and allowable loads and deflections for both girder designs.

Summary of Actual and Allowable Loads and Deflections

Interior Girders

$\mathbf{M_u^+}$	782 ft-k	ϕM_n^+	900 ft-k	OK
$\mathbf{M_u^-}$	1094 ft-k	ϕM_n^-	1111 ft-k	OK
$\mathbf{V_u}$	200 k	ϕV_n	318 k	OK
$\mathbf{T_u}$	69.7 ft-k	ϕT_n	92.8 ft-k	OK
$\mathbf{\Delta_{TL}}$	0.91”	$\mathbf{\Delta_{TL,allow}}$ **(l/360)**	1”	OK
$\mathbf{\Delta_{LL}}$	0.36”	$\mathbf{\Delta_{TL,allow}}$ **(l/480)**	0.75”	OK

Exterior Girders

$\mathbf{M_u^+}$	448 ft-k	ϕM_n^+	522 ft-k	OK
$\mathbf{M_u^-}$	627 ft-k	ϕM_n^-	644 ft-k	OK
$\mathbf{V_u}$	115 k	ϕV_n	177 k	OK
$\mathbf{T_u}$	114 ft-k	ϕT_n	150 ft-k	OK
$\mathbf{\Delta_{TL}}$	0.81”	$\mathbf{\Delta_{TL,allow}}$ **(l/360)**	1”	OK
$\mathbf{\Delta_{LL}}$	0.23”	$\mathbf{\Delta_{TL,allow}}$ **(l/480)**	0.75”	OK

In conclusion, the all of the design moments, shears, and deflections are less than the allowable, therefore both girder designs are okay.

The following is a final schedule of the cast-in-place concrete wide module joists.

CIP Girder Schedule

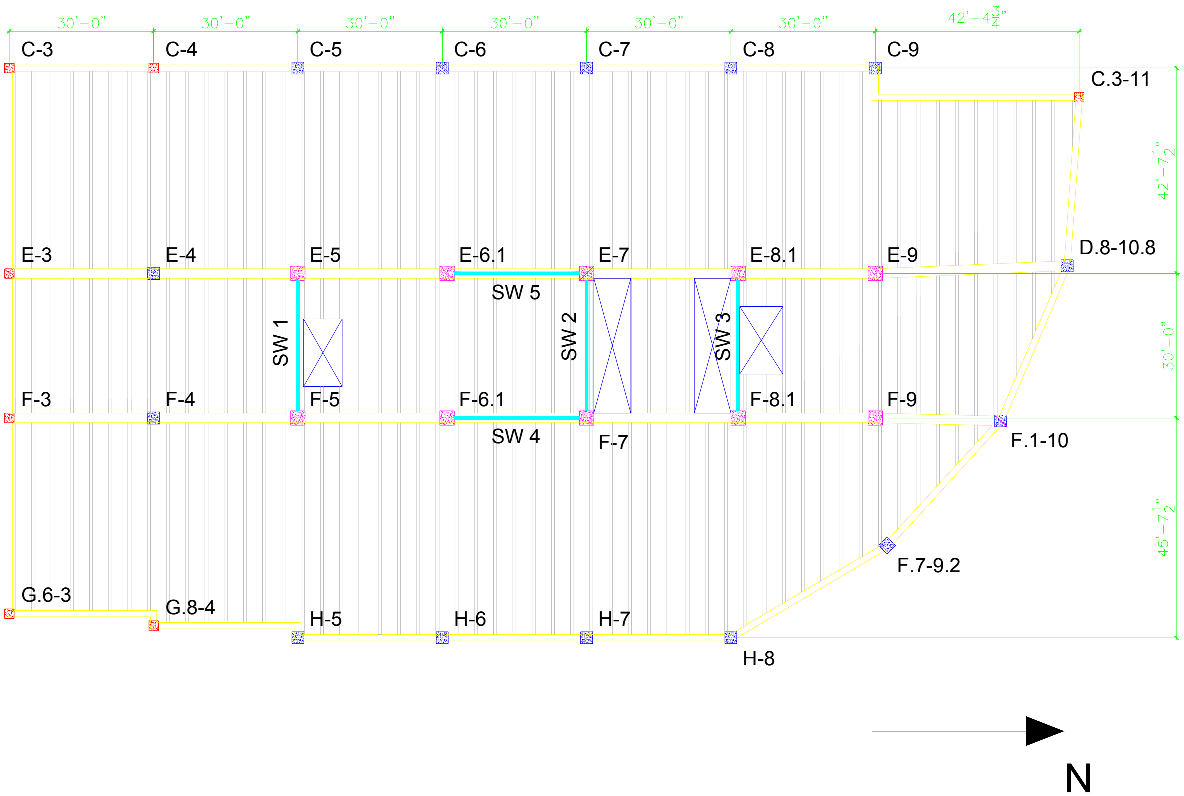
Girder	Size		Reinforcement		Stirrups			Longitudinal Reinforcement
	B	H	Bottom Bars	Top Bars	Size	Type	Spacing	
Interior	24"	28.5”	(8) #9	(12) #9	#5	Closed w/ 2 legs	4"	#4 @ 12"
Exterior	16"	28.5”	(6) #8	(8) #8	#4	Closed w/ 2 legs	5"	#4 @12"

f'c =	4,000	psi
fy =	60,000	psi

CIP Column Designs

Since The Regent is a spec office building, an open floor plan with minimal column interruption is desirable. The original long span steel system design with composite beams can easily, and relatively efficiently, accommodate the 46' span between columns in the East/West direction in order to keep an open floor plan between the perimeter of the building and the core of the building. In order to keep the original design intensions of an open floor plan, the original column locations from the steel system were also used for the cast-in-place concrete design, even though a smaller spacing between columns in the East/West direction could possibly result in a more efficient concrete design.

Column Location Plan



The column gravity loads considered for design are summarized below.

Dead Loads	
Façade	
Glass Curtain Wall	15 PSF
Precast Panels	20 PSF
Roof (Steel Joists and Metal Deck)	38 PSF
Typical Floor	
24” Joists and 4.5” Slab	119 PSF
16” Joists and 4.5” Slab	95 PSF
SDL	15 PSF
Girders	600 PLF
Live Loads	
Roof	
Mechanical	150 PSF
Snow	30 PSF
Typical Floor	
Mechanical	150 PSF
Office	100 PSF

The tributary area for each column was calculated and the column axial live and dead loads for each level were calculated and compiled in spreadsheets. The column live loads were reduced according to ASCE 7-02, Section 4.8.1 where applicable. An example of an individual column loading spreadsheet can be found in Appendix C.

Although the shearwalls were designed to take 100% of the lateral load, the columns were designed to take 25% of the seismic load which is a requirement of ASCE 7-02, Chapter 9, Section 9.5.2.2.1. The columns were designed to take 25% of the seismic load in the event that the shearwalls would fail. The axial loads due to the lateral seismic loading were found using the portal method for the 2nd Floor, which yielded the most conservative axial force. The axial loads induced into the column due to the 25% seismic loads were very small compared to the axial loads due to the dead and live gravity loads. The controlling load combination for axial loading was 1.2D + 1.6L.

Columns F-8.1, F-7, F-6.1, E-6.1, E-7, E-8.1, F-5, and E-5 were considered as the boundary elements for the shearwalls. These eight columns have an additional axial load due to the resisting force couple necessary to resist the moment caused by the lateral loads applied to the shearwalls. The calculations for the additional axial loads applied to the boundary element columns can be found in Appendix D.

The exterior column moments in the East/West direction, also referred to as the x-direction, are a result of the fixed end moments of the 24” exterior joists. The interior column moments in the East/West direction are a result of the difference in the fixed end moments of the 16” and 24” joists. A design moment (ft-k/ft) was calculated and multiplied 3’ which is the largest width of the columns.

The column moments for all of the columns in the North/South direction, also referred to as the y-direction, are a result of the difference in the girder moments at the column locations.

The load combinations considered for finding M_{ux} and M_{uy} for each column are listed below.

Load Combinations:

1. 1.2D + 1.6L	$E = \rho Q_E + 0.2 S_{DS} D$
2. 1.23D + L + E	$E = (1)Q_E + 0.2(0.153)D$
3. 0.93D + E	$E = Q_E + 0.03D$

Columns listed together had similar or exact loadings and were designed as similar columns.

The following tables summarize the column design moments in each direction.

Column Moments in the N-S Direction

				M_u		
Column(s)	M_D	M_L	M_E	Load Case 1	Load Case 2	Load Case 3
F-4, E-4	300	143	39	**589**	552	319
F-5, E-5	300	143	39	**589**	552	319
F-9, E-9	44	21	39	87	**115**	80
F-8.1, F-7, F-6.1, E-6.1, E-7, E-8.1	44	21	39	87	**115**	80
G.6-3, C-3	227	86	12	**410**	378	224
G.8-4, C-4	227	86	24	**410**	390	236
F-3, E-3	300	143	20	**589**	532	299
H-5, C-5	227	86	24	**410**	390	236
H-6, H-7, H-8, F.7-9.2, C-9, C-8, C-7, C-6	33	13	24	61	**78**	55
F.1-10, D.8-10.8	300	143	20	**589**	532	299
C.3-11	227	86	12	**410**	378	224

Column Moments in the E-W Direction

				M_u		
Column(s)	M_D	M_L	M_E	Load Case 1	Load Case 2	Load Case 3
F-4, E-4	24	23	39	66	**92**	63
F-5, E-5	24	23	39	66	**92**	63
F-9, E-9	24	23	39	66	**92**	63
F-8.1, F-7, F-6.1, E-6.1, E-7, E-8.1	24	23	39	66	**92**	63
G.6-3, C-3	72	54	12	**173**	155	79
G.8-4, C-4	72	54	24	**173**	167	91
F-3, E-3	24	23	20	66	**73**	43
H-5, C-5	72	54	24	**173**	167	91
H-6, H-7, H-8, F.7-9.2, C-9, C-8, C-7, C-6	72	54	24	**173**	167	91
F.1-10, D.8-10.8	24	23	20	66	**73**	43
C.3-11	72	54	12	**173**	155	79

Since the building height changes at the 6th and 10th levels, the column were designed in three different sections, in order to have more efficient column designs.

Section	Floors Supported
Top	10-12
Middle	6-12
Bottom	1-12

The following table summarizes all of the column design loads.

Column Loading Summary

	P_u (k) (excluding self weight)			**M_u (ft-k)**	
	10-12	**6-12**	**1-12**	**Mx**	**My**
F-4	**0**	**538**	**1901**	92	589
E-4	0	538	1901		
F-5	**425**	**1537**	**3044**	92	589
E-5	425	1537	3044		
F-9	486	1263	2234	92	115
E-9	**750**	**1979**	**3516**		
F-8.1	709	1840	3255	92	115
F-7	**740**	**1930**	**3418**		
F-6.1	740	1930	3418		
E-6.1	740	1930	3418		
E-7	740	1930	3418		
E-8.1	682	1791	3177		
G.6-3	**0**	**0**	**508**	173	410
C-3	0	0	508		
G.8-4	**0**	**396**	**1403**	173	410
C-4	0	396	1403		
F-3	**0**	**0**	**700**	73	589
E-3	0	0	700		
H-5	**281**	**1071**	**2142**	173	410
C-5	281	1071	2142		
H-6	**479**	**1278**	**2281**	173	78
H-7	479	1278	2281		
H-8	480	1273	2269		
F.7-9.2	333	872	1548		
C-9	495	1308	2328		
C-8	479	1278	2281		
C-7	479	1278	2281		
C-6	479	1278	2281		
F.1-10	328	845	1493	73	589
D.8-10.8	**529**	**1382**	**2453**		
C.3-11	**314**	**815**	**1445**	173	410

The columns were designed using PCACOL for each of the three sections for each column. Column self weight was included in the design even though it was not included in the previous table of design loads.

A concrete strength of f'c = 4000 psi was initially intended and used for the column design, however, the column sizes were very large. In order to help minimize the column sizes, a concrete strength of **f'c = 5000 psi** was used. The difference in material cost for 4000 psi concrete versus 5000 psi concrete is $91 versus $96, respectively.

The size and vertical spacing of the ties was based off of ACI 318-02, Sections 7.10.5.1 and 7.10.5.2, respectively.

The cover requirement for each column is 1.5" and the ratio of steel is between 1% and 6%.

The reinforcement is to be placed at equal spacings.

The following table summarizes the column designs.

Column Design Summary

NOTE: f'c = 5000 psi

Column(s)	Level 10-12	Level 6-9	Level 1-5
F-4, E-4	________	24" x 24" (8) - #10 #3 ties @ 18" o.c.	30" x 30" (8) - #10 #3 ties 18" o.c.
F-5, E-5	24" x 24" (12) - #9 #3 ties @18" o.c.	30" x 30" (12) - #8 #3 ties @16" o.c.	36" x 36" (24) - #10 #3 ties @18" o.c.
F-9, E-9	18" x 18" (4) - #9 #3 ties @18" o.c.	30" x 30" (8) - #10 #3 ties @18" o.c.	36" x 36" (16) - #11 #4 ties @22" o.c.
G.6-3, C-3	________	________	24" x 24" (8) - #8 #3 ties @16" o.c.
G.8-4, C-4	________	24" x 24" (8) - #8 #3 ties @16" o.c.	24" x 24" (12) - #9 #3 ties @18" o.c.
F-3, E-3	________	________	24" x 24" (12) - #8 #3 ties @16" o.c.
H-5, C-5	24" x 24" (8) - #9 #3 ties @18" o.c.	24" x 24" (8) - #9 #3 ties @18" o.c.	30" x 30" (8) - #10 #3 ties @18" o.c.
H-6, H-7, H-8, F.7-9.2, C-9, C-8, C-7, C-6	18" x 18" (4) - #9 #3 ties @18" o.c.	24" x 24" (8) - #9 #3 ties @18" o.c.	30" x 30" (12) - #10 #3 ties @18" o.c.
F.1-10, D.8-10.8	24" x 24" (8) - #10 #3 ties @18" o.c.	24" x 24" (16) - #10 #3 ties @18" o.c.	30" x 30" (12) - #10 #3 ties @18" o.c.
C.3-11	24" x 24" (8) - #9 #3 ties @18" o.c.	24" x 24" (8) - #9 #3 ties @18" o.c.	24" x 24" (12) - #10 #3 ties @18" o.c.
F-8.1, F-7, F-6.1, E-6.1, E-7, E-8.1	18" x 18" (4) - #9 #3 ties @18" o.c.	30" x 30" (8) - #10 #3 ties @18" o.c.	36" x 36" (28) - #11 #4 ties @22" o.c.

The following table summarizes the column loadings and capacities for each of the three sections; Levels 1-12, 6-12, 10-12.

Column Loadings and Capacities (Level 1-12)

Column	$P_u(k)$*	$\phi P_n(k)$	$M_{ux}(ft-k)$	$\phi M_x(ft-k)$	$M_{uy}(ft-k)$	$\phi M_y(ft-k)$
F-4, E-4	2,018	2,338	92	108	589	693
F-5, E-5	3,983	4,036	92	92	589	587
F-9, E-9	3,713	3,864	92	95	115	118
G.6-3, C-3	546	620	173	197	410	467
G.8-4, C-4	1,492	1,510	173	177	410	420
F-3, E-3	751	760	73	76	589	604
H-5, C-5	2,301	2,459	173	184	410	439
H-6, H-7, H-8, F.7-9.2, C-9, C-8, C-7, C-6	2,469	2,618	173	178	78	81
F.1-10, D.8-10.8	2,613	2,618	73	75	589	611
C.3-11	1,577	1,629	173	180	410	429
F-8.1, F-7, F-6.1, E-6.1, E-7, E-8.1	4,332	4,448	92	93	115	116

***NOTE:** P_u values include column self weight and shearwall boundary element loads (where applicable).

Column Loadings and Capacities (Level 6-12)

Column	$P_u(k)$*	$\phi P_n(k)$	$M_{ux}(ft-k)$	$\phi M_x(ft-k)$	$M_{uy}(ft-k)$	$\phi M_y(ft-k)$
F-4, E-4	538	578	92	96	589	608
F-5, E-5	1,841	2,252	92	114	589	729
F-9, E-9	2,062	2,459	92	106	115	133
G.6-3, C-3	-----	-----	-----	-----	-----	-----
G.8-4, C-4	434	447	173	184	410	436
F-3, E-3	-----	-----	-----	-----	-----	-----
H-5, C-5	1,151	1,217	173	190	410	450
H-6, H-7, H-8, F.7-9.2, C-9, C-8, C-7, C-6	1,370	1,621	173	203	78	86
F.1-10, D.8-10.8	1,463	1,511	73	77	589	621
C.3-11	896	1,038	173	205	410	483
F-8.1, F-7, F-6.1, E-6.1, E-7, E-8.1	2,156	2,459	92	100	115	125

***NOTE:** P_u values include column self weight and shearwall boundary element loads (where applicable).

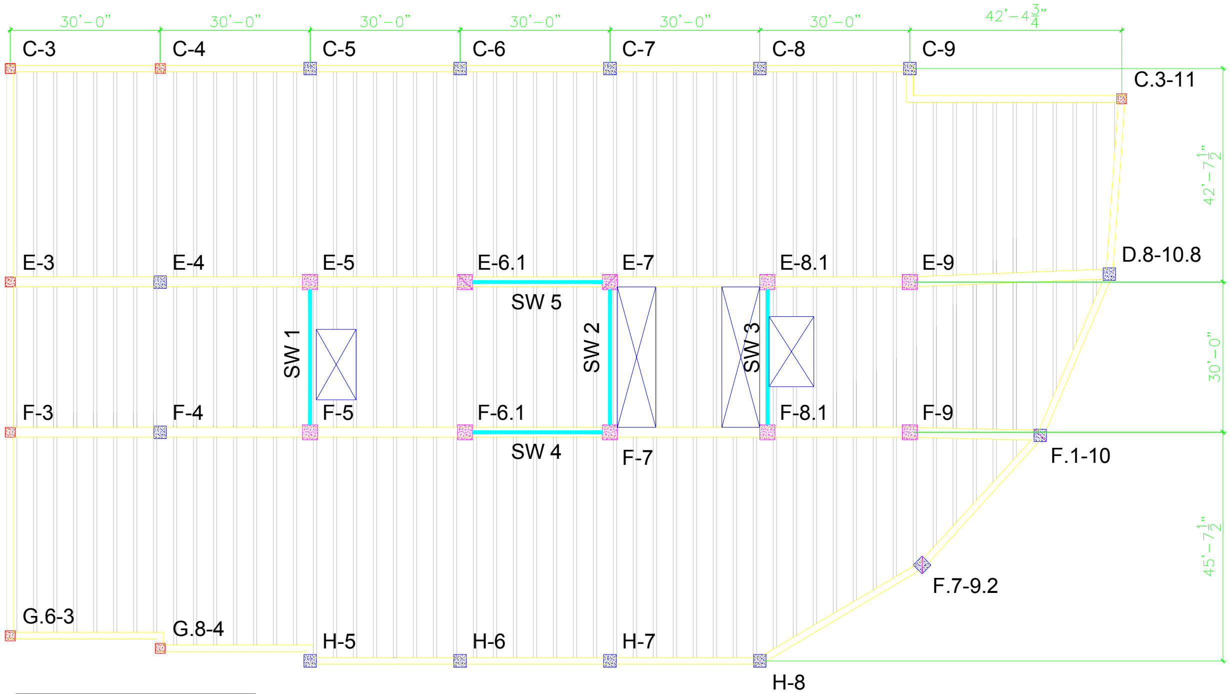
Column Loadings and Capacities (Level 10-12)

Column	$P_u(k)$*	$\phi P_n(k)$	$M_{ux}(ft-k)$	$\phi M_x(ft-k)$	$M_{uy}(ft-k)$	$\phi M_y(ft-k)$
F-4, E-4	-----	-----	-----	-----	-----	-----
F-5, E-5	521	551	92	100	589	638
F-9, E-9	774	826	92	100	115	124
G.6-3, C-3	-----	-----	-----	-----	-----	-----
G.8-4, C-4	-----	-----	-----	-----	-----	-----
F-3, E-3	-----	-----	-----	-----	-----	-----
H-5, C-5	323	343	173	186	410	442
H-6, H-7, H-8, F.7-9.2, C-9, C-8, C-7, C-6	519	585	173	199	78	90
F.1-10, D.8-10.8	572	580	73	76	589	619
C.3-11	357	388	173	191	410	454
F-8.1, F-7, F-6.1, E-6.1, E-7, E-8.1	770	896	92	100	115	125

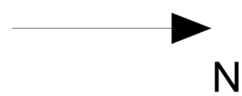
***NOTE:** P_u values include column self weight and shearwall boundary element loads (where applicable).

In conclusion, all of the columns are adequately designed.

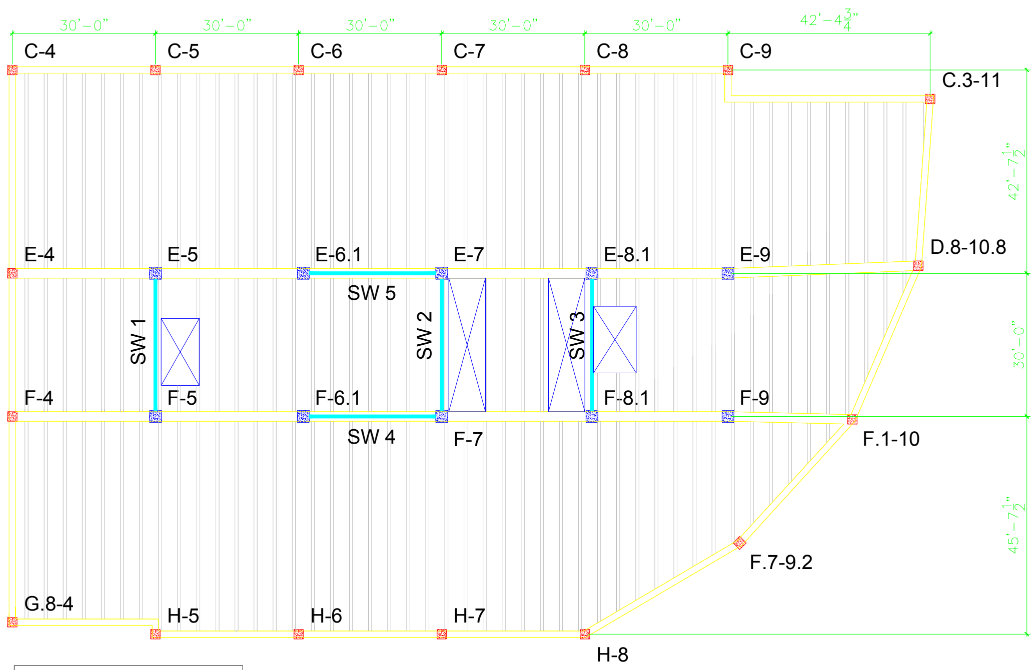
Column Design and Location Plan (Levels 1-5)



COLUMN KEY	
18" x 18"	
24" x 24"	
30" x 30"	
36" x 36"	

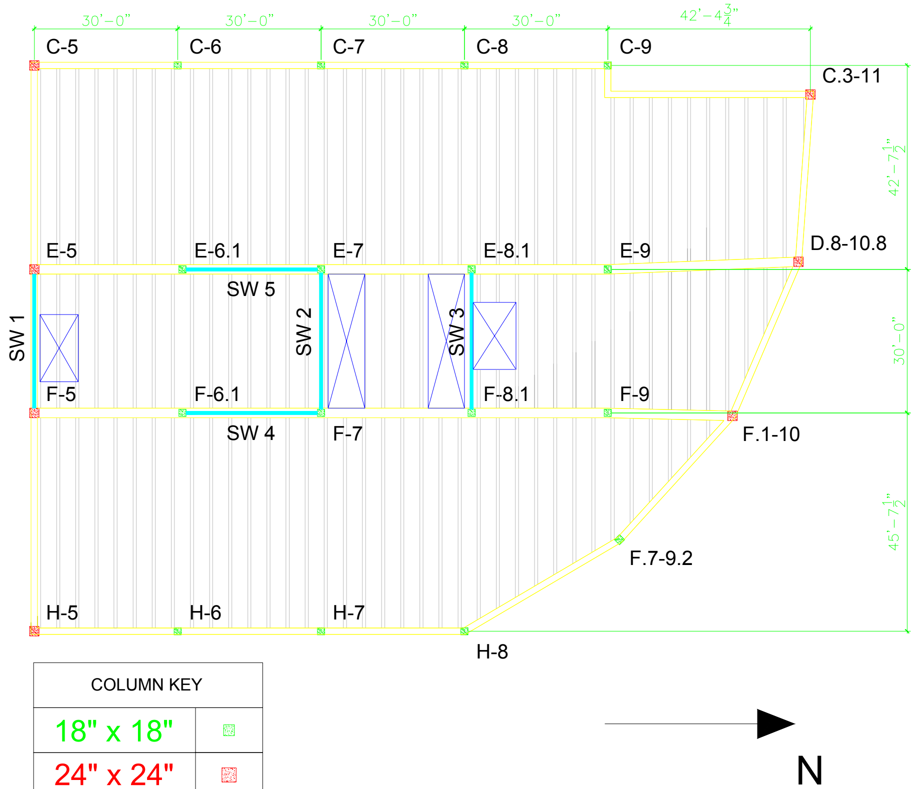


Column Design and Location Plan (Levels 6-9)



COLUMN KEY	
18" x 18"	
24" x 24"	
30" x 30"	
36" x 36"	

Column Design and Location Plan (Levels 10-12)



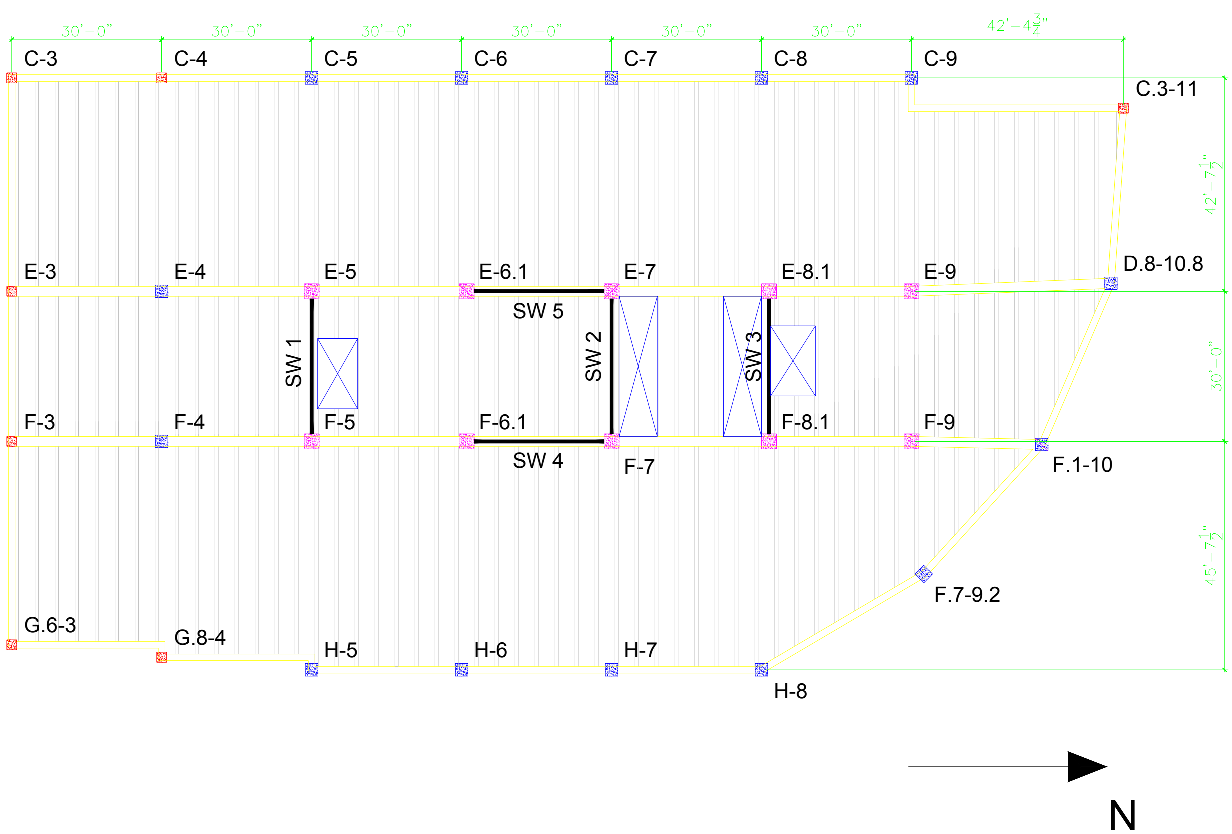
COLUMN KEY	
18" x 18"	
24" x 24"	
30" x 30"	
36" x 36"	

CIP Shearwall Designs

The steel system uses a series of 5 braced frames; two spanning in the North/South direction and three spanning in the East/West Direction. Alternative locations for the shearwall locations were considered including the concrete stairwell walls and the elevator shaft walls. After some preliminary calculations, it was determined that the best place to put the shearwalls was in the exact locations of the braced frames. The size of the stairwell walls were very small, and did not allow for adequate wall sizes to carry the lateral loads. Keeping the shearwalls in the same location as the braced frames, eliminated the need to introduce more interruptions in the floor system elsewhere in floorplan. In addition, by keeping the shearwalls bound between existing columns eliminated the need to introduce additional boundary elements. The existing columns double as the boundary elements for the shearwalls.

Keeping the shearwalls in the same locations as the braced frames resulted in 5 shearwalls, 30' long each, which are centrally located. The shearwalls will run the full height of the building. Shearwalls 1, 2, and 3 span East/West and resist the lateral forces in the East/West direction. Shearwalls 4 and 5 span in the North/South direction and resist the lateral forces in the North/South direction.

Shearwall Location Plan



The new seismic and wind lateral loads were calculated for the concrete system. The wind loads remained the same as the steel system, however the seismic lateral loads calculated for the concrete system were smaller than the seismic loads calculated for the steel system since the weight of the structure increased. Detailed calculations of the seismic loads for both the steel system and the concrete system, as well as the wind loads, are included in Appendix I.

The following table summarizes the lateral loads considered for the design of the shearwalls.

Load Cases and Controlling Lateral Forces (Concrete System)

Load Combinations Involving Wind Loads (W) and Seismic Loads (E)

ASCE 7-02 (Sec. 2.3.2)

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

Check 1.6W vs. 1.0E

Red = Controlling E-W Lateral Force, Blue = Controlling N-S Lateral Force

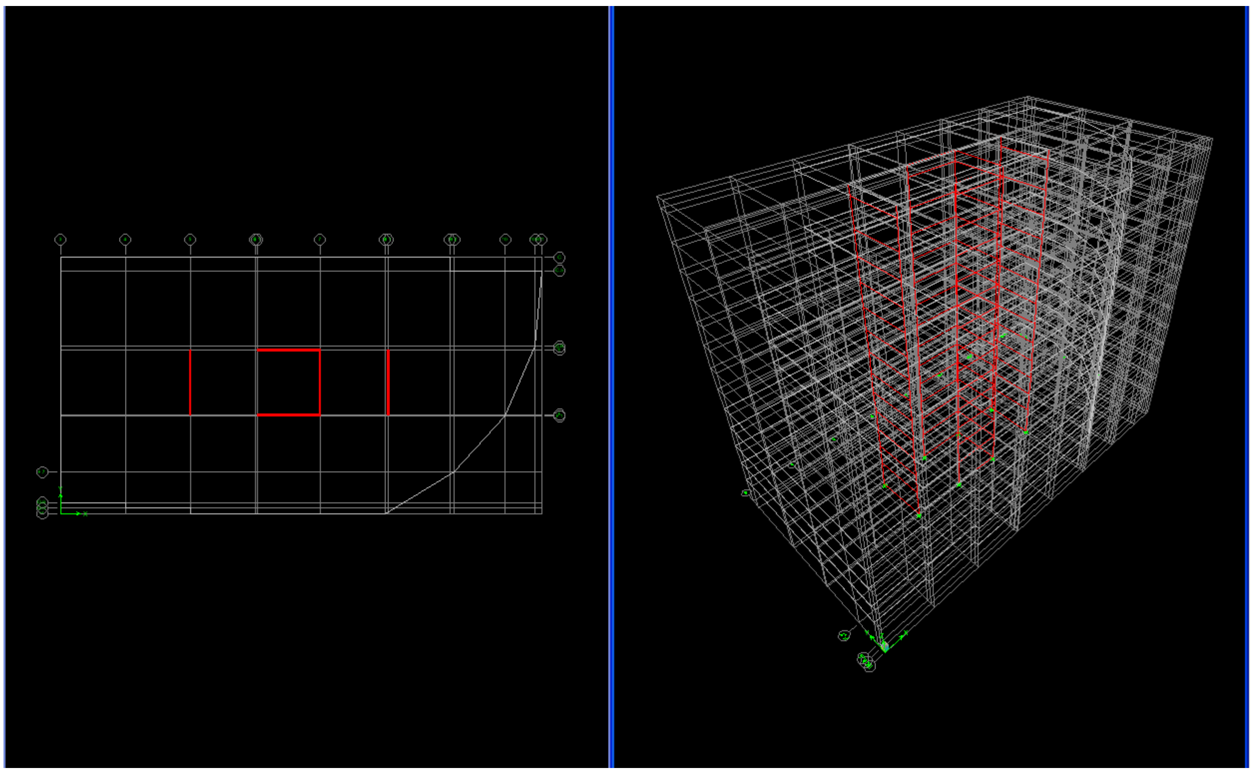
	1.6W (N-S)	1.6W (E-W)	1.0E (N-S/E-W)
Roof	**60.16**	**93.72**	30.61
12	**82.32**	**128.64**	66.53
11	45.55	**74.59**	**54.51**
10	44.91	**83.57**	**55.63**
9	43.95	**82.05**	**46.40**
8	**42.77**	**80.14**	37.82
7	**41.42**	**77.98**	29.92
6	**40.19**	**87.89**	26.55
5	**38.78**	**107.92**	19.08
4	**37.07**	**82.13**	12.57
3	**35.06**	**78.43**	7.15
2	**37.64**	**85.79**	3.01

Wind was the controlling lateral for the East/West direction and for most of the floors in the North/South direction.

An initial shearwall size of 8" was selected for all 5 shearwalls. An ETABS model was created in order to check the adequacy of the 8" shearwalls. A floor mesh was created in order for the computer analysis to take into account the changing center of mass as the floors reduced in size up the building. The shearwalls were designed to take 100% of the controlling lateral load.

The allowable total building deflection at the roof is l/400 or 5.42”. The results of the ETABS analysis concluded that the building deflection with 8” shearwalls would be approximately 2.0” in the North/South direction and approximately 1.5” in the East/West direction, which are both less than the allowable 5.42”.

ETABS Model



The following table summarizes the deflections of each shearwall.

Shearwall Deflections

Wall	Max Δx		Max Δy		Max Δz	
	E/S	W/N	E/S	W/N	E/S	W/N
1	2.053982”	2.032261”	1.503888”	1.503888”	0.164363”	-0.161891”
2	2.053982”	2.032261”	1.547330”	1.547330”	0.012486”	-0.289300”
3	2.053982”	2.032261”	1.570137”	1.570137”	0.148668”	-0.150556”
4	2.053982”	2.053982”	1.526333”	1.547330”	0.424625”	0.012286”
5	2.032261”	2.032261”	1.526333”	1.547330”	0.137942”	-0.289300”

$$\Delta_{allow} = \frac{h}{400} = \frac{180.75'(12''/ft)}{400} = 5.42''$$

All shearwall deflections are less than the allowable 5.42” $\therefore$ OK

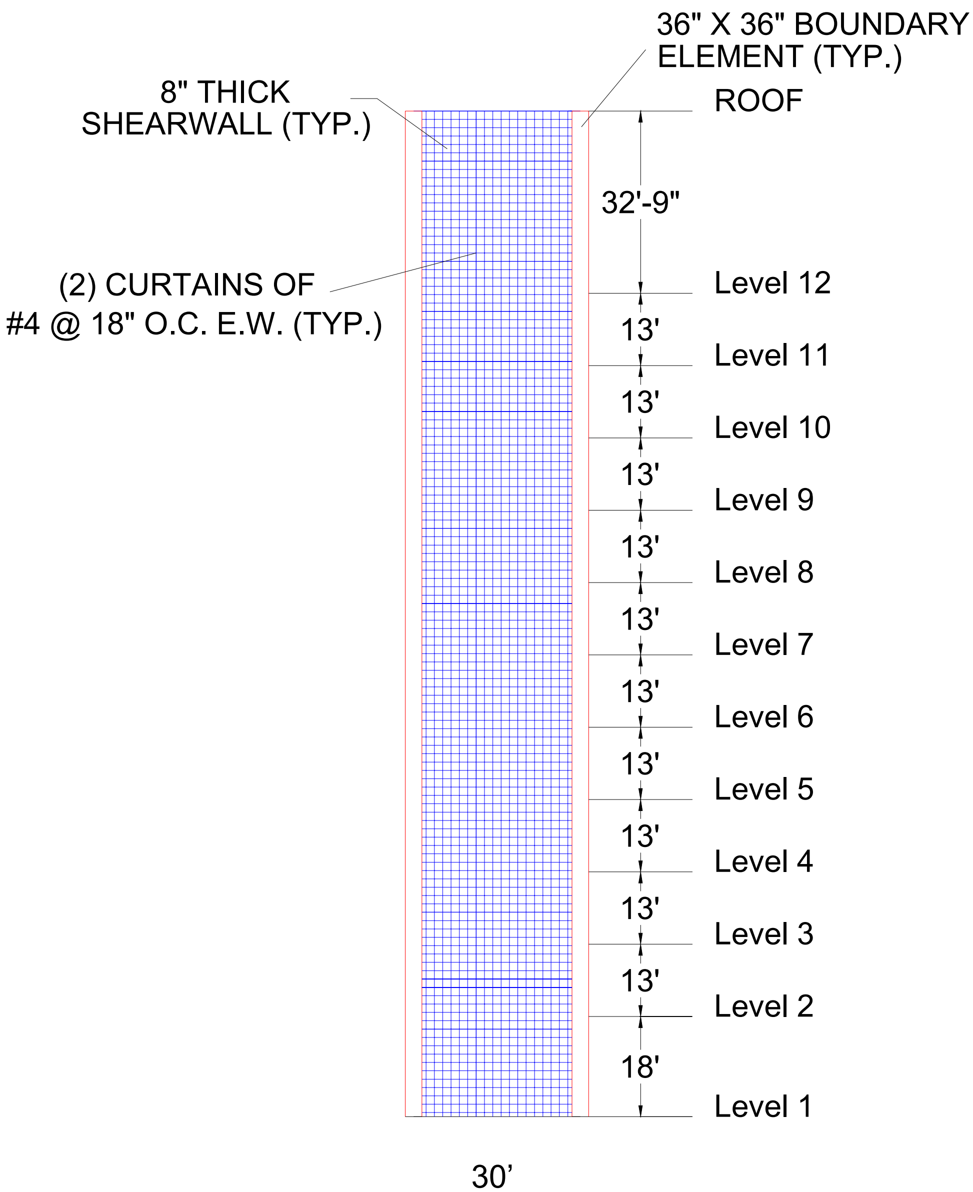
The shearwall calculations concluded that boundary elements were needed at the ends of each shearwall. These shearwall boundary element calculations can be found in Appendix D. The boundary elements are the columns at the ends of all of the shearwalls. They were designed to take the additional axial load caused by the force couple created by the overturning moment of the shearwall.

The shearwalls have a strength of f’c = 4000 psi and the reinforcing steel is fy = 60 ksi. The reinforcement was determined for the worst case wall at Level 2 and then used in the remaining four walls in order to keep all the shearwalls the same for construction efficiency. The worst case shearwall was SW 3. Detailed calculations for the shearwall reinforcement are included in Appendix D.

The results of the hand calculations concluded that (2) curtains of #4 bars spaced at a maximum of 18” o.c. in both horizontal and vertical directions would be adequate to carry the design lateral loads.

In conclusion, each shearwall will run the entire height of the building (approximately 181') and will be 30' in length and 8" thick. The concrete strength is f'c = 4000 psi and the reinforcing steel is fy = 60 ksi. The boundary elements for each shearwall are the columns at both end of each shearwall and they were designed to take the additional axial forced due to the lateral loads. Two curtains, one in each face of the shearwall, will consists of #4 bars spaced at 18" o.c. each way. According to ACI 7.7.1, the cover requirement for CIP walls not exposed to weather or ground with No. 4 bars is ¾".

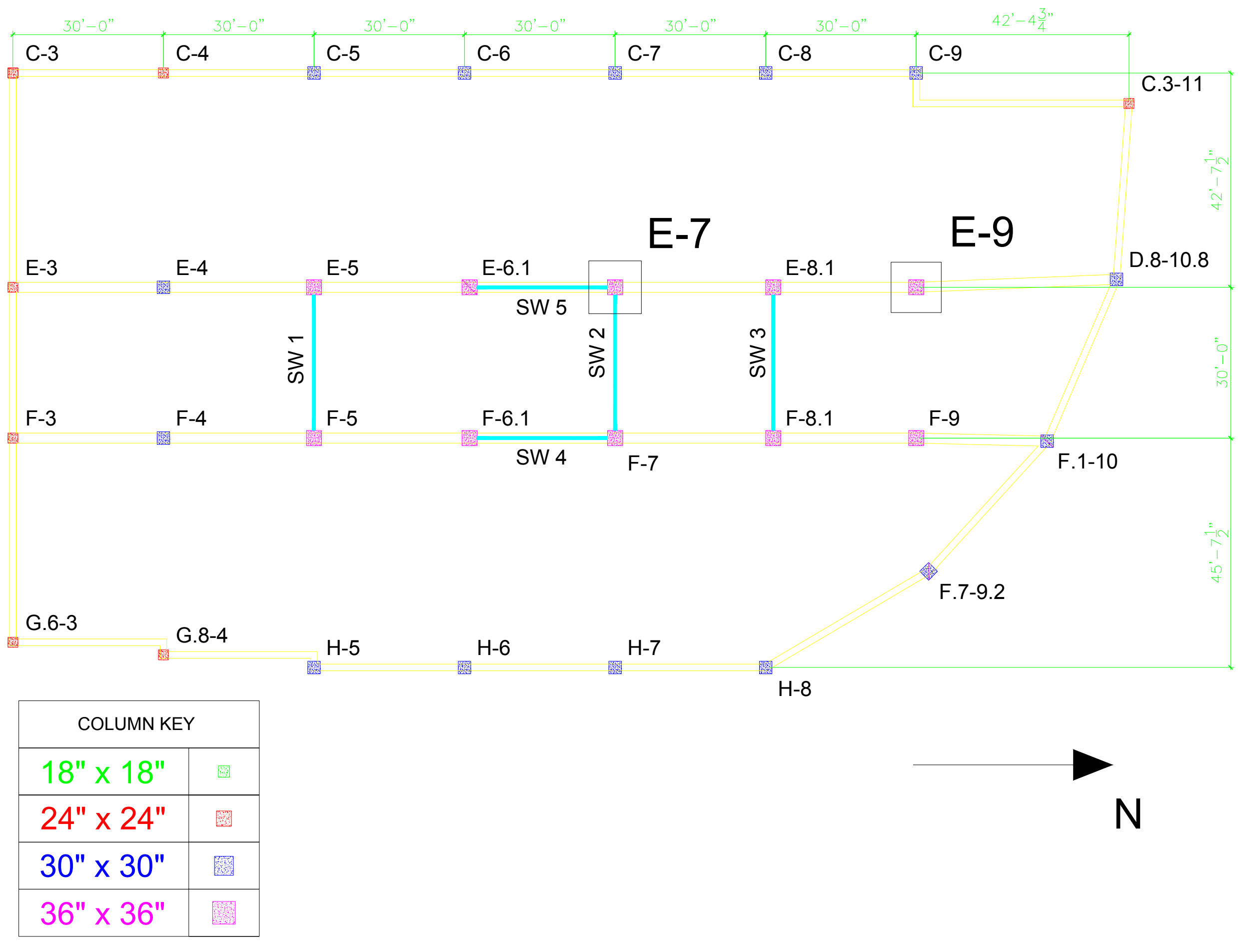
Final Shearwall Design



Representative Spread Footing Designs

In order to see the impacts on the foundations by utilizing a concrete system for this building, two representative spread footings were designed and compared to the corresponding spread footings of the steel system. The two spread footings selected for design are the square footings for columns E-7 and E-9.

Location Plan for Spread Footings E-7 and E-9

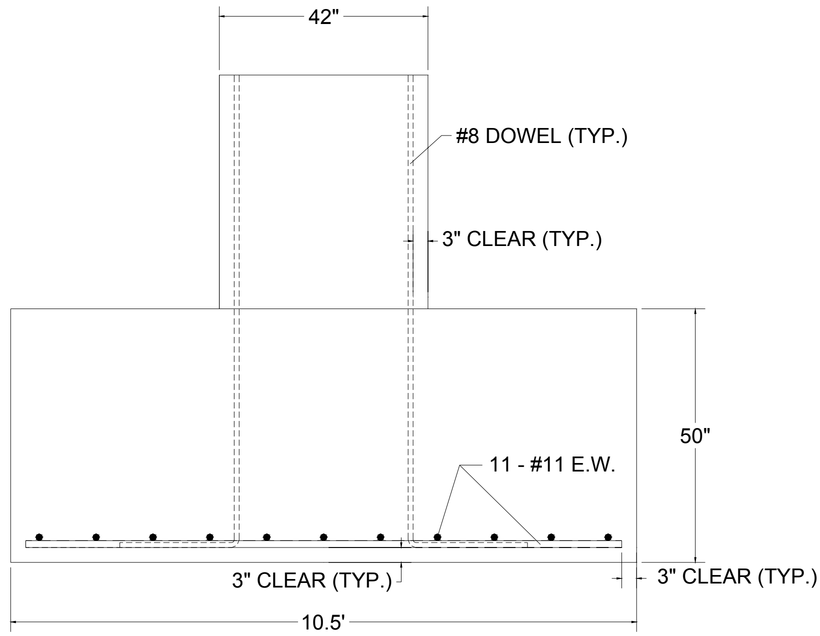


Column E-7 is an interior column that also acts a boundary element for adjoining shearwalls. In the steel system, Column E-7 was part of the lateral load resisting braced frame. Column E-9 is one of the most heavily loaded interior columns that is not a boundary element for any shearwalls or braced frames.

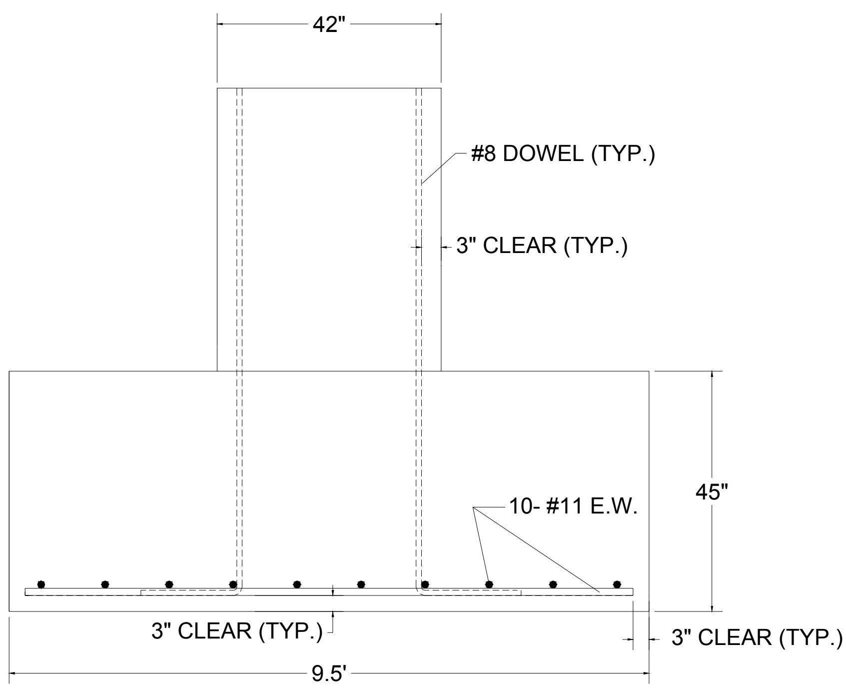
The allowable bearing pressure for this site is 40 KSF. The concrete strength is 3000 psi and the reinforcing steel is 60 ksi.

Although the scope of this report focuses on the superstructure above grade, the below-grade garage loads were calculated for Column E-7 and Column E-9. It was anticipated that both of these columns would be approximately 42” x 42” at the point of the spread footing. Detailed calculations, design loads, and design assumptions for the design of the spread footings are included in Appendix E. The flexural reinforcement had to meet the minimum reinforcement ration of 0.0018 in order to meet shrinkage and temperature requirements.

Square Footing Design for Column E-7



Square Footing Design for Column E-9



The following table summarizes the designs of the spread footings with the concrete system and also includes the design of the corresponding footings for the steel system, which were taken from the structural plans.

Spread Footing Schedule				
Concrete System Footing Schedule				
Footing	Allowable Bearing Pressure (KSF)	Size (Square)	Depth	Bottom Reinforcement
E-7	40 KSF	10.5' x 10.5'	50"	(11) #11 e.w.
E-9	40 KSF	9.5' x 9.5'	45"	(10) #11 e.w.
Steel System Footing Schedule				
Footing	Allowable Bearing Pressure (KSF)	Size (Square)	Depth	Bottom Reinforcement
E-7	40 KSF	9' x9'	50"	(9) #10 e.w.
E-9	40 KSF	8' x 8'	38"	(12) #9 e.w.

It can be concluded that the concrete system requires larger foundations.

For the square footings for Column E-7, which are lateral load supporting column footing, the footing sizes are significantly different in plan by 29 SF and are the same depth.

For the square footings for Column E-9, which are non-lateral load supporting column footings, the footing sizes are significantly different by 26 SF in plan and a 7” in depth.

Since both concrete system footing sizes are significantly larger than the corresponding steel system footing sizes, it can therefore be concluded that the concrete system significantly affects the size of the square footings as compared to the steel system.

Roof Design

In order to help minimize the weight of the structure, a steel joist with metal roof deck system was designed instead of a concrete roof system. The steel joists will span in the East/West direction across the 46' and 30' bays. The metal deck will then span in the North/South direction across the steel joists.

Roof Dead Loads	
SDL	15 PSF
Insulation	1.5 PSF
Built-up Roof (5-ply felt and gravel)	6.5 PSF
Metal Deck	22 gage – 2 PSF 18 gage – 3 PSF
Steel Joists	26K12 – 16.6 KLF 26K8 – 21.1 KLF
Roof Live Loads	
Snow	30 PSF
Mechanical	150 PSF

The roof joists were designed using The New Columbia Joist Company design guide. Detailed calculations for the design of the roof joists are included in Appendix F. The following table summarizes the designs of the roof joists.

Roof Joist Schedule				
	Size			
Span	Joist Designation	Depth	Spacing	Approx. Wt.
46/43'	26K12	26"	4' o.c.	16.6 klf
30'	26K8	26"	1.5' o.c.	12.1 klf

The roof deck was designed using the United Steel Deck Design Manual and Catalog of Products. Detailed calculations for the design of the roof joists are included in Appendix F. The following table summarizes the design of the roof deck.

Roof Deck Schedule				
	Size			
Deck Span	Type	Gage	Span Condition	Weight
4'	F, Intermediate Rib Deck	22	Triple	1.6 PSF
1.5'	F, Intermediate Rib Deck	18	Triple	2.6 PSF

Depth Study Conclusions

In conclusion, the cast-in-place concrete system is not considered the most efficient design for The Regent compared to the steel system. All of the CIP concrete members are very large in size in order to accommodate the large spans. As a result, the weight of the concrete structure is significantly heavier than the weight of the steel structure, which increases the size of the foundations. The depth of the concrete flooring system for the exterior bays exceeds the depth of the steel flooring system. The girders are also deeper than the steel beams. The concrete columns are significantly larger in area than the steel system columns and will interrupt more floor space. The shearwalls are also very large; however, one advantage is that they keep the building deflections to less than 50% of the allowable deflections.

Breadth Study:

Construction Management

Construction Management Breath Study Overview

Minimal building costs and a quick schedule were part of the design team's goals. After exploring alternative floor system designs, it was initially predicted that the steel system would be cheaper and quicker to erect than the concrete system. In order to make a comparison between the two systems on the basis of cost and schedule to see if this initial prediction was true, a cost and schedule analysis was done as a Construction Management breadth study. The scope of the cost and schedule analysis include a cost and schedule analysis for a typical lower level floor for each system, as well as a cost and schedule analysis of the representative spread footings for each system.

RS Means Building Construction Cost Data for 2006 was used for both the schedule and cost analysis.

This depth study is broken down into the following two sections, each with a corresponding Appendix, which contain the necessary calculations and spreadsheets used for analysis.

1. Cost Analysis — Appendix G
2. Schedule Analysis — Appendix H

Cost Analysis

In order to compare the cost of the steel system and the concrete system, a cost analysis of a typical lower level floor was done for each system. In addition, a cost analysis was done comparing the cost of the footings for each system. It was predicted in the proposal that the steel system would be cheaper than the concrete system.

The cost analysis was done using RS Means Building Construction Data 2006.

The scope of the cost analysis for a typical lower level floor is summarized in the following table.

Scope of Cost Analysis for each System for a Typical Lower Level Floor

Concrete System	**Steel System**
Concrete	Concrete Slab
Reinforcement	WWF
Formwork	Formwork
Placement (pumped)	Placement
Finishing	Metal Deck
Shoring/Reshoring	Steel Members
	Beams
	Columns
	Braced Frame Members

Detailed quantity take-offs and cost analysis calculations for a typical lower level floor for each system can be found in Appendix G. The following tables summarize the results of the cost estimates for a typical lower level floor for each system.

Concrete System Cost Analysis for a Typical Lower Level Floor

Total Cost	**Concrete System**		
	Material	Labor	Equipment
Joists/Slab	$225,435	$130,625	$4,543
Girders	$48,707	$58,946	$965
Columns	$24,756	$27,330	$625
Shearwalls	$8,534	$11,484	$365
Shoring/Reshoring	$149,865	$9,943	$0
	$457,297	$238,328	$6,498
	$702,123		

Steel System Cost Analysis for a Typical Lower Level Floor

Total Cost	**Steel**		
	Material	Labor	Equipment
Slab on Deck	$41,814	$10,153	$1,881
Metal Deck	$41,468	$10,428	$728
Beams	$160,851	$9,998	$4,937
Columns	$74,396	$964	$631
Braced Members	$22,447	$1,149	$659
	$340,976	$32,692	$8,836
	$382,504		

The cost of a typical floor for the concrete system is significantly higher than the cost of a typical floor for the steel system.

Even though steel usually has with higher material costs, the concrete system exceeds the steel material costs due to the large amounts of concrete, reinforcement, formwork, and shoring required for the concrete system. The steel system was designed as an unshored system, eliminating the extra costs for shoring.

The labor costs for the concrete system are significantly higher than the labor costs for the steel system. The cast-in-place concrete system requires a lot of labor in order to set up the formwork and the shoring, place the rebar, place the concrete, and pour the concrete. The steel system labor costs were mostly due to the labor required for the slab on deck. The steel is placed with a crane and requires less labor.

The concrete equipment costs were lower than the steel equipment costs. The steel system requires a crane rental which accounts for a large portion of the equipment costs.

Scope of Cost Analysis for the Concrete System and Steel System Spread Footings

Concrete and Steel Systems
Concrete
Reinforcement
Formwork
Placement

Cost Analysis for the Concrete System and Steel System Spread Footings

Footing	Cost			Total Cost
	Material	Labor	Equip.	
E-7 (Concrete)	$2,052	$863	$6	**$2,921**
E-7 (Steel)	$1,592	$722	$5	**$2,319**
E-9 (Concrete)	$1,583	$701	$5	**$2,289**
E-9 (Steel)	$966	$464	$3	**$1,433**

The cost of the concrete system footings is larger than the cost for the steel system footings. The concrete system footing sizes are significantly larger than the steel system footing sizes resulting in higher material and labor costs.

In conclusion, the cost of the concrete system footings is significantly larger than the steel system footings.

Schedule Analysis

In order to compare the difference in schedules of the steel system and the concrete system, a schedule analysis of a typical lower level floor was done for each system. In addition, a schedule analysis was done comparing the schedules of representative spread footings for each system. Overlap in schedule tasks were not accounted for. It was predicted in the proposal that the steel system would be quicker to erect than the concrete system.

The schedule analysis was done using RS Means Building Construction Data 2006 based off of the recommended crew size and their daily output each item.

The scope of the schedule analysis for a typical lower level floor is the same as for the cost analysis and is also summarized in the following table.

Scope of Schedule Analysis for each System for a Typical Lower Level Floor

Concrete System	**Steel System**
Concrete	Concrete Slab
Reinforcement	WWF
Formwork	Formwork
Placement	Placement
Finishing	Metal Deck
Shoring/Reshoring	Steel Members
	Beams
	Columns
	Braced Frame Members

Detailed quantity take-offs and schedule analysis calculations for a typical lower level floor for each system can be found in Appendix H. The following tables summarize the results of the estimated schedules for a typical lower level floor for each system.

Final Schedule	**Concrete**
	# of Days
Joists/Slab	30.31
Girders	11.03
Columns	6.36
Shearwalls	3.38
Shoring/Reshoring	6.93
	58.01

58 days

Final Schedule	Steel
	# of Days
Slab on Deck	10.36
Metal Deck	8.08
Beams	3.30
Columns	1.33
Braced Members	0.45
	23.52
	24 days

The initial concrete system schedule analysis yielded that a typical concrete floor would take 183 days. The long schedule was mostly due to the crew sizes being too small for the amount of rebar that needs to be placed and the amount of formwork that needs to be constructed since the entire system is cast-in-place concrete. In order to shorten the schedule to a more reasonable number of days, the number of rodmen was increased from the recommended 4 to 12 and the number C-2 crews were increased from the recommended 1 crew to 5 crews. Increasing the number of these two crew sizes decreased the concrete schedule from 183 days down to 58 days. The concrete system is very labor intensive and requires a larger than recommended workforce in order to complete the structure for a typical floor in a relatively reasonable amount of time.

The calculated number of days to complete the steel system was approximately 24 days. The actual schedule proposed for this project anticipated 12 days to complete a typical lower level floor.

The steel system has a much shorter number of days per floor as compared to the concrete system. The concrete system schedule is much longer than the steel system because of the large quantities or reinforcement, formwork, shoring and reshoring needing to be placed for each floor as well as the taking into account the longer curing time. The steel system was designed as an unshored system, eliminating shoring time and therefore minimizing the steel system schedule time. The steel system also has the advantages of a crane and minimal amount of concrete that needs to be placed.

In addition to a schedule analysis for a typical lower level floor, a schedule analysis was completed for representative spread footings for each system in order to compare the schedule impacts for the footings resulting from switching to a concrete system.

The scope of the schedule analysis for representative spread footings is the same as for the cost analysis and is also summarized in the following table.

Scope of Schedule Analysis for each System for Representative Spread Footings

Concrete and Steel Systems
Concrete
Reinforcement
Formwork
Placement

The results of the schedule analysis for the representative spread footings are summarized below.

Schedule Analysis for the Concrete System and Steel System Spread Footings

Footing	Schedule
E-7 (Concrete)	0.74 days
E-7 (Steel)	0.62 days
E-9 (Concrete)	0.66 days
E-9 (Steel)	0.40 days

The footings for the concrete system take longer to construct than the footings for the steel system because they are larger and require more formwork, concrete, and rebar to be placed.

In conclusion, the steel system is a significantly quicker system to erect in comparison to the concrete system.

Breadth Study:

Mechanical

Mechanical Breadth Study Overview

Since the concrete system has a different depth and layout than the steel system, a mechanical layout impact analysis was done as a Mechanical breadth study.

Mechanical Layout Impact Analysis

Mechanical Layout Impact Analysis

The concrete floor system has different depths than the steel floor system. In order to see the impacts on the layout of the mechanical ductwork, by using the concrete system, a mechanical system layout impact analysis was done.

The mechanical notes on the plans for the existing steel system state the following:

- "All ductwork shall be tight to the bottom of the structure unless otherwise indicated".
- "The bottom elevation of the main trunk duct including angle bracing and external insulation shall not be less than 9'-8" above the finished floor".

The main trunk duct starts at the core of each floor in the mechanical room and circles around the floor's exterior bays and returns to the mechanical room on the opposite side of the building.

The typical floor to floor height for floors 2-12 is 13' or 156" and the floor to ceiling height is 9' or 108". The remaining 8" of plenum space from the bottom of the ductwork insulation is for the ceiling panels, lighting, and electrical.

The finished floor was assumed to have a depth of 0.5". The thickness of the external insulation for the rectangular sheet metal ductwork, found in the Mechanical section of the specifications, is 1.5". The depth of the main trunk duct across the entire floor is 12".

The depth of the floor system for both systems changes between the exterior (46') bays and interior (30') bay in the East/West direction.

The following table summarizes the depth of floor system, ductwork and insulation, and flooring for each system for both the exterior bays and the interior bay.

Concrete Floor System		
	Exterior Bays	Interior Bay
Slab Thicknesss (in)	4.5	4.5
Joist Depth (in)	24	16
Ductwork (in)	12	12
Flooring Thickness	0.5	0.5
Ductwork Insulation (in)	2(1.5)	2(1.5)
	44"	**36"**
Steel Floor System		
	Exterior Bay	Interior Bay
Slab and Deck (in)	6.25	6.25
I-beam Depth (in"	18	16
Ductwork (in)	12	12
Flooring Thickness (in)	0.5	0.5
Ductwork Insulation (in)	2(1.5)	1(1.5)
	39.75"	**37.75"**

In order to meet the 9'8" requirement between the bottom of the ductwork insulation and the finished floor, the depth of the floor system, ductwork and insulation, and flooring should not exceed 3'-4" or 40".

The following tables summarize the actual and allowable depths of the floor system, ductwork and insulation, and flooring.

Concrete Floor System			
	Actual Depth	Allowable Depth	
Exterior Bay	44"	40"	NOT OK
Interior Bay	36"	40"	OK

Steel Floor System			
	Actual Depth	Allowable Depth	
Exterior Bay	39.75"	40"	OK
Interior Bay	37.75"	40"	OK

The depth of the concrete floor system in the exterior bay, where the main trunk duct runs, exceeds the allowable by 4". This reduces the floor to ceiling height to 8'-8", if this system was to be used as designed. Since the main trunk duct runs perpendicular to the joists, there is no option for the ductwork to run through the floor structure. The joists could be turned to span in the North/South direction, however, the concrete system would be an even less efficient design.

Options:

- Redesign the Joists to a 20” depth
- Resize the ductwork to an 8” depth
- Increase the floor to floor height by 4” to get a floor to ceiling height of 9’
- Keep a floor to ceiling height of 8’-8”

Resize the Joists

After reviewing the 20” depth wide module joists sizes and capacities in the CRSI Design Handbook, it was determined that there is no 20” depth joist that would accommodate a 46’ span with specified design loads.

The following table summarizes the results of some of the 20” joists sizes from the CRSI Design Handbook.

Factored superimposed load using the 1.4D + 1.7L load combinations = 764 PLF

40 + 8 + 20	435 PLF	NOT OK
40 + 9 + 20	495 PLF	NOT OK
40 + 10 + 20	555 PLF	NOT OK
53 + 10 + 20 (44’ span)	589 PLF	NOT OK
66 + 9 + 20 (42’ span)	558 PLF	NOT OK

Therefore, no 20” joists would work.

Resize the Ductwork

Resizing the ductwork to an 8” depth and a wider width would make the ductwork more inefficient because the cross-section would become even less “square”.

Increasing the Floor to Floor Height

If the floor to floor height were to be increased in order to meet the 9’ floor to ceiling height, the building would have to be less than 12 stories tall. The building is already designed to its maximum allowable height.

Keeping the 8’-8” Floor to Ceiling Height

Since The Regent is a spec office building in the D.C. area, an 8’-8” ceiling height is less desirable than a 9’ ceiling height.

In conclusion, the steel system is able to accommodate the architectural design intentions and spatial layouts throughout the entire floor and is therefore preferred over the concrete system.

Conclusions

The following chart contains a summary comparison of the steel system and the concrete system.

System Comparison Chart

	Steel System	**CIP Concrete System**
Floor System Depth	**24.5" (46' Span)** 22.5" (30' Span)	28.5" (46' Span) **20.5" (30' Span)**
Floor to Floor Height	18' (1st Floor) 13'	18' (1st Floor) 13'
Floor to Ceiling Height	**Interior Bay - 9'** **Exterior Bay – 9'**	**Interior Bay – 9'** Exterior Bay – 8'-8"
Cost of Typical Floor	**$382,504**	$702,123
Material	**$340,976**	$457,297
Labor	**$32,692**	$238,328
Equipment	$8,836	**$6,498**
Typical Floor Schedule	**24 days**	58 days
Cost of Foundation for Lateral Resisting and Gravity Member	**$2,319**	$2,921
Cost of Foundation for Gravity Only Member	**$1,433**	$2,289
Foundation Size for Lateral Resisting and Gravity Member	**9' x 9' x 50"** **(9) #10 e.w.**	10.5' x 10.5' x 50" (11) #11 e.w.
Foundation Size for Gravity Only Member	**8' x 8' x 38"** **(12) #9 e.w.**	9.5' x 9.5' x 45" (10) #11 e.w.
Allowable Depth for Mechanical System	Interior Bay – 14.25" **Exterior Bay – 12.25"**	**Interior Bay – 16"** Exterior Bay – 8"
Typical Floor Weight	**46 PSF + 10 PSF** **(concrete ponding)**	119 PSF 95 PSF

In reviewing the results of the system comparison chart, it is clear that the steel system has more advantages over the concrete system as was originally predicted in the proposal.

The steel system is significantly cheaper and quicker to erect than the concrete system. The steel system can better accommodate the spatial requirements of the original mechanical layout design and requires smaller foundations than the concrete system saving both time and money on the foundations.

Overall, the steel system, as compared to the CIP concrete system, is the most appropriate structural system to accommodate the design goals with a cheaper overall cost, a quicker and more practical schedule, and structure that maximizes plenum space and can accommodate the mechanical layout and architectural design intentions for The Regent.

One of the main purposes for completing CIP structural system design depth study was to gain experience designing a structure using a concrete system. By completing this thesis, a better understanding of the design processes, code requirements, and structural analysis and design programs for CIP concrete design was learned.

By completing the depth studies, a better understanding was learned of just how important it is to select the most appropriate structure for a building in order to meet its design goals whether they are cost, schedule, mechanical layout, or architecture.

Overall, this thesis was a very valuable learning experience.

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Loads

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Credits and Acknowledgements

Credits and Acknowledgements

Project Team

Steve Sanko – Structural Design Group, Ltd.

Structural Plans, Soils Report, SSK's, CAD Drawings

Katie Peterschmidt – Cooper Carry Architects
Lauren Schlather – Cooper Carry Architects

CAD Files, Specifications, Renderings

Kevin D. Gunthert – JBG Owner Representative

Permission to study The Regent

AE Faculty

Dr. Schneider – Faculty Consultant, Fall 2005
Dr. Memari – Faculty Consultant, Spring 2006
M. Kevin Parfitt – Senior Thesis Professor
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Paul Bowers

My fiancé Dan for his love and support

My family for their love and support

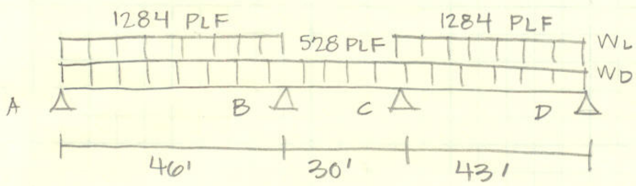
Appendices

Appendix A

CIP Joist Design Calculations

Joist Positive Moment – Moment Distribution

Joist Positive Moment



Dead Loads

24" Joists

Self: 119 PSF (4') = 476 PLF
SDL: 15 PSF (4') = 60 PLF
536 PLF

1.2(536) = 644 PLF

16" Joists

Self: 95 PSF (4') = 380 PLF
SDL: 15 PSF (4') = 60 PLF
440 PLF

1.2(440) = 528 PLF

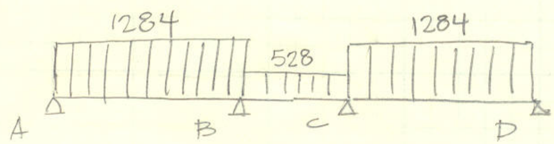
Live Loads

LL_{office} = 100 PSF

LL_{office} = 100 PSF (4') = 400 PLF   1.6(400) = 640 PLF

2

$I_{16''} = 12{,}128\ in^4$
$I_{24''} = 32{,}297\ in^4$



$K_{AB} = \frac{E(32297)}{46(12)} = 58.51\,E$ $\quad 1.74K$

$K_{BC} = \frac{E(12128)}{30(12)} = 33.69\,E$ $\quad K$

$K_{CD} = \frac{E(32297)}{43(12)} = 62.59\,E$ $\quad 1.86K$

$FEM_{AB} = -\frac{1.284(46)^2}{12} = -226.41$

$FEM_{BA} = +226.41$

$FEM_{BC} = -\frac{0.528(30)^2}{12} = -39.6$

$FEM_{CB} = +39.6$

$FEM_{CD} = -\frac{1.284(43)^2}{12} = -197.84$

$FEM_{DC} = +197.84$

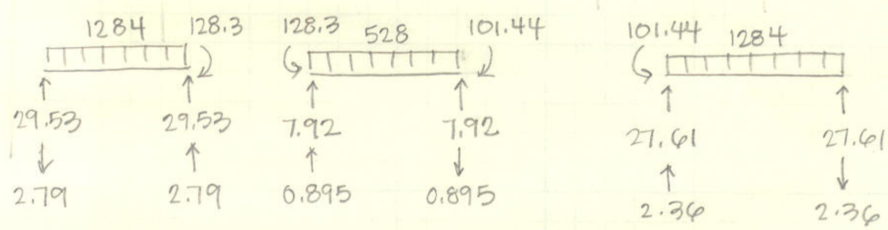
$K_{AB}^{m} = 0.75(1.74K) = 1.31\ K$

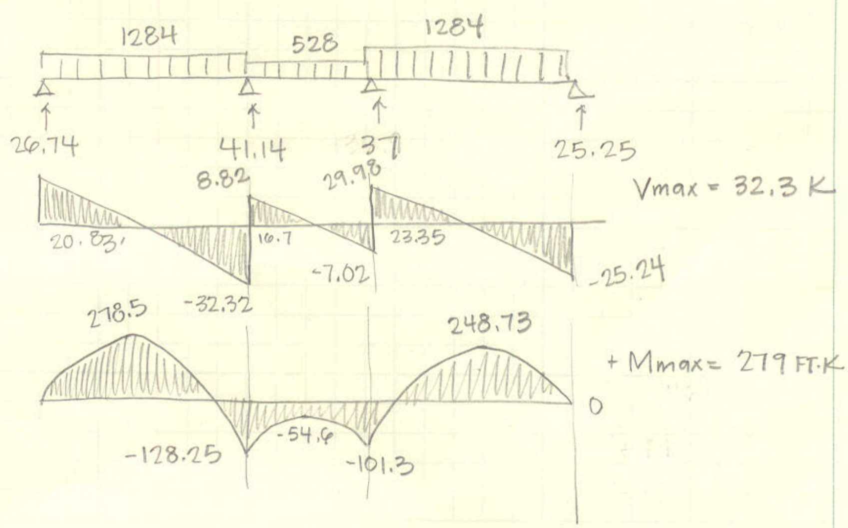
$K_{CD}^{m} = 0.75(1.86K) = 1.40K$

3

	A	B		C		D
	AB	BA	CB	BC	CD	DC
K	1.74K	1.74K	K	K	1.86K	1.86K
K^m		1.31K			1.40K	
DF	1	0.57	0.43	0.42	0.58	1
FEM	-226.4	226.41	-39.6	39.6	-197.84	197.84
	+226.4	+113.2			-98.92	-197.84
	0	-171	-129	-64.5		0
			+67.55	+135.1	+186.56	
		-38.5	-29.05	-14.53		
			+3.05	+6.1	+ 8.43	
		-1.74	-1.31	-0.55		
			+ 0.115	+0.23	0.32	
		-0.066	-0.049	-0.025		
				+0.01	+0.015	
	0	128.3	-128.3	101.44	-101.44	0

Max positive moment = 128.3 FT·K



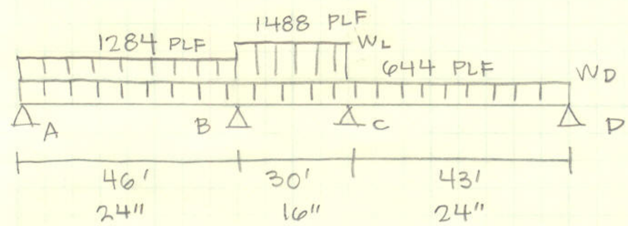


Joist Negative Moment – Moment Distribution

1

Joist Negative Moment

Joists 48" o.c.



Dead Loads

24" Joists:

self: 119 PSF (4') = 476 PLF
SDL: 15 PSF (4') = 60 PLF
536 PLF

1.2 (536) = 644 PLF

16" Joists:

self: 95 PSF (4') = 380 PLF
SDL: 15 PSF (4') = 60 PLF
440 PLF

1.2 (440) = 528 PLF

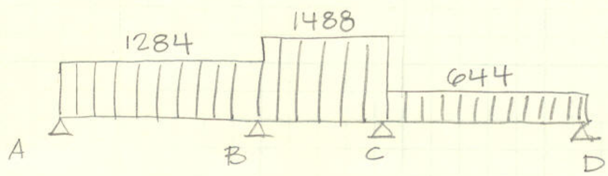
Live Loads

LLmech = 150 PSF
LLoffice = 100 PSF

LLmech = 150 PSF (4') = 600 PLF 1.6 (600) = 960 PLF
LLoffice = 100 PSF (4') = 400 PLF 1.6 (400) = 640 PLF

2

$I_{16''} = 12128$
$I_{24''} = 32,297$



$K_{AB} = \dfrac{E(32,297)}{46'(12''/FT)} = 58.51\ in^3E$ 1.74K

$K_{BC} = \dfrac{E(12,128)}{30(12)} = 33.69\ in^3E$ K

$K_{CD} = \dfrac{E(32,297)}{43(12)} = 62.59\ in^3E$ 1.86K

$FEM_{AB} = -\dfrac{wl^2}{12} = -\dfrac{1.284(46)^2}{12} = -226.41$ FT·K

$FEM_{BA} = 226.41$ FT·K

$FEM_{BC} = -\dfrac{1.488(30)^2}{12} = -111.6$ FT·K

$FEM_{CB} = 111.6$ FT·K

$FEM_{CD} = -\dfrac{0.644(43)^2}{12} = -99.23$ FT·K

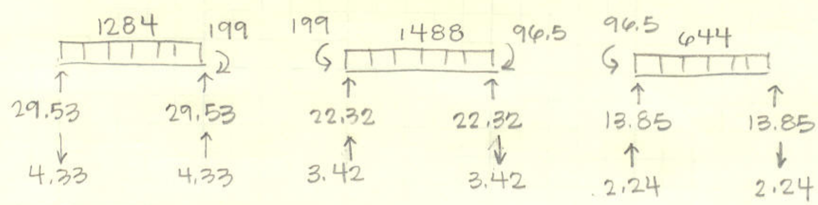
$FEM_{DC} = 99.23$ FT·K

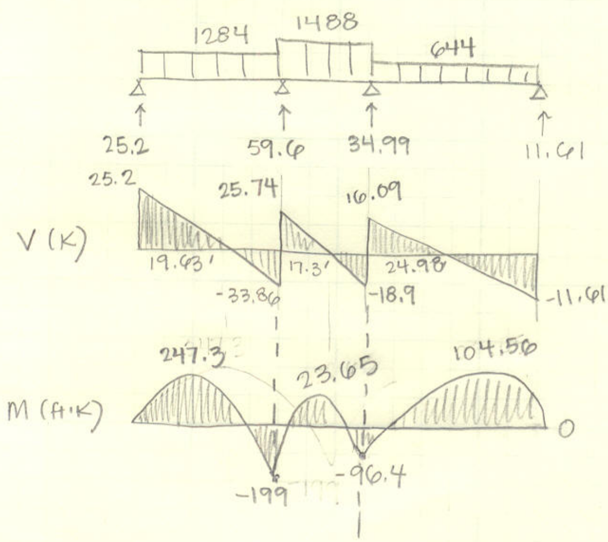
$K_{ab}^m = 0.75(1.74K) = 1.31K$
$K_{cd}^m = 0.75(1.86K) = 1.40K$

3

	A	B		C		D
	AB	BA	BC	CB	CD	DC
K	1.74K	1.74K	K	K	1.86K	1.86K
Km		1.31K			1.40K	
DF	1	0.57	0.43	0.42	0.58	1
FEM	-226.41	+226.41	-111.6	+111.6	-99.23	+99.23
	+226.41	+113.21			-49.62	-99.23
	0	-129.97	-98.05	-49.03		0
			+18.12	+36.24	+50.04	
		-10.33	-7.79	-3.90		
			+0.82	+1.64	+2.26	
		-0.47	-0.35	-0.175		
			+0.037	+0.074	+0.102	
		-0.021	-0.016			
	0	198.83	-199.1	96.45	-96.45	0

Max Negative Moment = 199.1 FT·K





Vmax = 33.9 K

-Mmax = -199 FT·K

+M16" = 23.65 FT·K

Kristin Ruth
Structural Option

Joist Positive Reinforcement [46’/43’ Span (24”)Joists]

Mu+ = 279 ft-k
fy = 60 ksi
f’c = 4 ksi
#3 Stirrups

Try (2) #10

d = 28.5”-1.5”-0.375”-0.5(1.27”)
d = 25.99”

As = 2(1.27in^2) = 2.54in^2

$$a = \frac{A_s fy}{0.85 f'cb}$$
$$a = \frac{2.54in^2(60ksi)}{0.85(4ksi)(48'')}$$
$$a = 0.934''$$

$$c = \frac{a}{\beta}$$
$$c = \frac{0.934''}{0.85}$$
$$c = 1.099'' \qquad \leq 0.375(25.99'') = 9.75'' \therefore \phi = 0.9$$

$$\phi M_n = \phi A_s f_y (d - a/2)$$
$$\phi M_n = 0.9(2.54in^2)(60ksi)\left(25.99'' - \frac{0.934''}{2}\right)$$
$$\phi M_n = 3500.7in - k$$
$$\phi M_n = 291.73 ft - k$$

$$\phi M_n = 291.73 ft - k \geq M_u = 279 ft - k \therefore OK$$

$$b_{\min} = 2(1.5'') + 2(1.27'') + (1)1.27'' + (0.375'')$$
$$b_{\min} = 7.2'' \leq 8'' \therefore OK$$

Use (2) #10 bottom bars

Joist Positive Reinforcement [30' Span (16") Joists]

Mu+ = 23.65 ft-k
fy = 60 ksi
f'c = 4 ksi
#3 Stirrups

Try (1) #6

d = 20.5"-1.5"-0.375"-0.5(0.75")
d = 18.25"

As = 1(0.44in^2) = 0.44in^2

$$a = \frac{A_s fy}{0.85 f'cb}$$
$$a = \frac{0.44in^2(60ksi)}{0.85(4ksi)(48")}$$
$$a = 0.162"$$

$$c = \frac{a}{\beta}$$
$$c = \frac{0.162"}{0.85}$$
$$c = 0.191" \qquad \le 0.375(18.25") = 6.84" \therefore \phi = 0.9$$

$$\phi M_n = \phi A_s f_y (d - a/2)$$
$$\phi M_n = 0.9(0.44in^2)(60ksi)\left(18.25" - \frac{0.162"}{2}\right)$$
$$\phi M_n = 431.7 in - k$$
$$\phi M_n = 35.97 ft - k$$

$$\phi M_n = 35.97 ft - k \ge M_u = 23.65 ft - k \therefore OK$$

$$b_{\min} = 2(1.5") + 1(0.75") + 1(0.375")$$
$$b_{\min} = 4.12" \le 8" \therefore OK$$

Use (1) #6 bottom bar

Joist Negative Reinforcement [46'/43' Span (24") Joists and 30' Span (16") Joists]

Mu- = -199 ft-k
fy = 60 ksi
f'c = 4 ksi
#3 Stirrups

Try (9) #5

$d_{16"}$ = 20.5"-0.75"-0.375"-0.5(0.625")
$d_{16"}$ = 19.06" ***CONTROLS**

$d_{24"}$ = 28.5"-0.75"-0.375"-0.5(0.625")
$d_{24"}$ = 27.06"

As = 9(0.31in^2) = 2.79 in^2

$$a = \frac{A_s fy}{0.85 f'cb}$$
$$a = \frac{2.79in^2(60ksi)}{0.85(4ksi)(8")}$$
$$a = 6.15"$$

$$c = \frac{a}{\beta}$$
$$c = \frac{6.15"}{0.85}$$
$$c = 7.24" \qquad \le 0.375(19.06") = 7.14" \therefore \phi \ne 0.9$$

$$\varepsilon_t = \frac{0.003(d-c)}{c}$$
$$\varepsilon_t = \frac{0.003(19.06"-7.24")}{7.24"}$$
$$\varepsilon_t = 0.004898$$

$$\varepsilon_y = \frac{f_y}{E_s}$$
$$\varepsilon_y = \frac{60ksi}{29{,}000ksi}$$
$$\varepsilon_y = 0.002069$$

$$\phi = 0.65 + 0.25\left(\frac{\varepsilon_t - \varepsilon_y}{0.005 - \varepsilon_y}\right)$$
$$\phi = 0.65 + 0.25\left(\frac{0.004898 - 0.002069}{0.005 - 0.002069}\right)$$
$$\phi = 0.891$$

$$\phi M_n = \phi A_s f_y (d - a/2)$$
$$\phi M_n = 0.891(2.79in^2)(60ksi)\left(19.06'' - \frac{6.15''}{2}\right)$$
$$\phi M_n = 2384..2in - k$$
$$\phi M_n = 199\,ft - k$$

$$\phi M_n = 199\,ft - k \geq M_u = 199\,ft - k \therefore OK$$

$$b_{\min} = 9(0.625'') + 8(1'') + 1(0.375'')$$
$$b_{\min} = 14'' \leq 48'' \therefore OK$$

Use (9) #5 top bars

Joist Shear Calculations

Joist Shear

$V_u = 33.9\text{ K}$

24" Joists $d = 25.99"$

$\phi V_n = 0.5(0.75)(2)\sqrt{f'_c}\, b_w d$
$= 0.5(0.75)(2)\sqrt{4000}\,(8")(25.99")$
$= 9.9\text{K} < 33.9\text{K}$ ∴ stirrups required

$V_s = \dfrac{A_s f_y d}{s}$ $S_{max} = \dfrac{d}{2} = \dfrac{25.99"}{2} = 13"$

$V_s = \dfrac{0.11(60)(25.99")}{12"}$

$V_s = 14.3\text{K}$

$V_c = (2)\sqrt{4000}\,(8")(25.99")$
$V_c = 26.3\text{K}$

$\phi V_n = V_c + V_s = 26.3\text{K} + 14.3\text{K} = 40.6\text{K} > 33.9\text{K}$ ∴ OK

$\boxed{\text{use single leg \#3 stirrups @ 12" o.c.}}$

16" Joists $d = 18.25"$

$\phi V_n = 0.5(0.75)(2)\sqrt{4000}\,(8)(18.25)$
$= 6.9\text{K}$

$V_s = \dfrac{0.11(60)(18.25)}{6"}$
$V_s = 20.1\text{ K}$

$V_c = 2\sqrt{4000}\,(8)(18.25) = 18.5\text{K}$

$\phi V_n = 20.1\text{K} + 18.5\text{K} = 38.6\text{K} > 33.9\text{K}$ ∴ OK

$\boxed{\text{use single leg \#3 stirrups @ 6" o.c.}}$

Joist Stirrups

46'/43' Span (24") Joists	#3 stirrups @12" o.c.
30' Span (16") Joists	#3 stirrups @6" o.c.

Joist Deflection Calculations

46'/43' Span (24") Joists

L = 46'
D = 536 PLF
L = 400 PLF
f'c = 4,000 psi
I = 32,297 in^4

$$E = w_c^{1.5}(33)\sqrt{f'c}$$
$$E = (150PCF)^{1.5}(33)(\sqrt{4000}psi$$
$$E = 3{,}834{,}254\,psi$$

$$\Delta_{TL,allow} = \frac{l}{360}$$
$$\Delta_{TL,allow} = \frac{46'(12''/ft)}{360}$$
$$\Delta_{TL,allow} = 1.5''$$

$$\Delta_{LL,allow} = \frac{l}{480}$$
$$\Delta_{LL,allow} = \frac{46'(12''/ft)}{480}$$
$$\Delta_{LL,allow} = 1.15''$$

$$\Delta_{TL} = \frac{5wl^4}{384EI}$$
$$\Delta_{TL} = \frac{5(78lb/in)(552in)^4}{384(3{,}834{,}254\,psi)(32{,}297in^4)}$$
$$\Delta_{TL} = 0.76'' < 1.5'' \therefore OK$$

$$\Delta_{LL} = \frac{5wl^4}{384EI}$$

$$\Delta_{LL} = \frac{5(33.3lb/in)(552in)^4}{384(3{,}834{,}254\,psi)(32{,}297in^4)}$$

$$\Delta_{LL} = 0.325" < 1.15" \therefore OK$$

30’ Span (16”) Joists

L = 30’
D = 440 PLF
L = 600 PLF
f’c = 4,000 psi
I = 12,128 in^4

$$E = w_c^{1.5}(33)\sqrt{f'c}$$

$$E = (150PCF)^{1.5}(33)(\sqrt{4000}psi$$

$$E = 3{,}834{,}254\,psi$$

$$\Delta_{TL,allow} = \frac{l}{360}$$

$$\Delta_{TL,allow} = \frac{30'(12"/ft)}{360}$$

$$\Delta_{TL,allow} = 1"$$

$$\Delta_{LL,allow} = \frac{l}{480}$$

$$\Delta_{LL,allow} = \frac{30'(12"/ft)}{480}$$

$$\Delta_{LL,allow} = 0.75"$$

$$\Delta_{TL} = \frac{5wl^4}{384EI}$$

$$\Delta_{TL} = \frac{5(86.7lb/in)(360in)^4}{384(3{,}834{,}254\,psi)(12{,}128in^4)}$$

$$\Delta_{TL} = 0.41" < 1" \therefore OK$$

$$\Delta_{LL} = \frac{5wl^4}{384EI}$$

$$\Delta_{LL} = \frac{5(50lb/in)(360in)^4}{384(3{,}834{,}254\,psi)(12{,}128in^4)}$$

$$\Delta_{LL} = 0.24" < 0.75" \therefore OK$$

Summary of Actual and Allowable Loads

46'/43' Span (24") Joists   24 + 8 + 40 Joists

M_u^+	279 ft-k	ϕM_n^+	292 ft-k	OK
M_u^-	199 ft-k	ϕM_n^-	199 ft-k	OK
V_u	33.9 k	ϕV_n	40.6 k	OK
Δ_{TL}	0.75"	$\Delta_{TL,allow}$ (l/360)	1.5"	OK
Δ_{LL}	0.325"	$\Delta_{TL,allow}$ (l/480)	1.15"	OK

30' Span (16") Joists   16 + 8 + 40 Joists

M_u^+	23.65 ft-k	ϕM_n^+	36 ft-k	OK
M_u^-	199 ft-k	ϕM_n^-	199 ft-k	OK
V_u	33.9 k	ϕV_n	38.6	OK
Δ_{TL}	0.41"	$\Delta_{TL,allow}$ (l/360)	1"	OK
Δ_{LL}	0.24"	$\Delta_{TL,allow}$ (l/480)	0.75"	OK

Appendix B

CIP Girder Design Calculations

Interior Girder Design

Loads

Dead

Self Weight: $\left(\frac{24''}{12''/ft}\right)\left(\frac{24''}{12''/ft}\right)(150PCF) = 600PLF$

SDL: $15PSF\left(\frac{46'+30'}{2}\right) = 570PLF$

Joists: 95PSF(15’) + 119PSF(23’) = 4162 PLF

Live: ~~100PSF(23’)~~ + 150PSF(15’) = ~~4550 PLF~~
56PSF(23’) + 150PSF(15’) = 3538 PLF

$$A_T = 38'(30') = 1140SF$$

$$L = L_o\left(0.25 + \frac{15}{\sqrt{2x1140SF}}\right)$$

$$L = 0.56L_o$$

w_D = 600 PLF + 570 PLF + 4162 PLF = 5332 PLF = 5.33 KLF
w_L = 3538 PLF = 3.6 KLF

Moments

l_n = 30’
w_D = 5.33 KLF
w_L = 3.6 KLF

ACI 8.3.3 Moments	Dead	Live	Seismic
$\pm\frac{wl^2}{16}$	300 ft-k	203 ft-k	33.5 ft-k
$+\frac{wl^2}{14}$	343 ft-k	231 ft-k	33.5 ft-k
$-\frac{wl^2}{10}$	480 ft-k	324 ft-k	33.5 ft-k
$-\frac{wl^2}{11}$	436 ft-k	295 ft-k	33.5 ft-k

Red = Worst Case (+) Moment
Blue= Worst Case (-) Moment

Load Combinations

1. 1.2D + 1.6L
2. 1.23D + L + E
3. 0.93D + E

$$E = \rho Q_E + 0.2 S_{DS} D$$
$$E = (1) Q_E + 0.2(0.153) D$$
$$E = Q_E + 0.03 D$$

(+) Moment = 782 ft-k	
Load Case 1	**782 ft-k**
Load Case 2	686 ft-k
Load Case 3	352 ft-k

(-) Moment = -1094 ft-k	
Load Case 1	**-1094 ft-k**
Load Case 2	-948 ft-k
Load Case 3	-480 ft-k

Determine B_{eff}

$16h_f + b_w = 16(4.5") + 24"$ = 96"

$\frac{L}{4} = \frac{30'(12"/ft)}{4}$ = 90" *CONTROLS

Web spacing = 28'(12"/ft)/2 = 168"

Flexural Reinforcement

(+)M

+M = 782 ft-k
f'c = 4 ksi
fy = 60 ksi
#5 stirrups
1.5" cover – bottom
0.75" cover – top

Try (8) #9

d = 28.5" – 1.5" – 0.625" – 0.5(1.25")
d = 25.81"

$A_s = 8(1\ in^2)$
$A_s = 8\ in^2$

$$a = \frac{A_s fy}{0.85 f'cb}$$
$$a = \frac{8in^2(60ksi)}{0.85(4ksi)(90")}$$
$$a = 1.57"$$

$$c = \frac{a}{\beta}$$
$$c = \frac{1.57"}{0.85}$$
$$c = 1.85" \qquad \le 0.375(25.81") = 9.68" \therefore \phi = 0.9$$

$$\phi M_n = \phi A_s f_y (d - a/2)$$
$$\phi M_n = 0.9(8in^2)(60ksi)\left(25.81" - \frac{1.57"}{2}\right)$$
$$\phi M_n = 10{,}810 in - k$$
$$\phi M_n = 901 ft - k$$

$$\phi M_n = 901 ft - k \ge M_u = 782 ft - k \therefore OK$$

$$b_{\min} = 2(1.5") + 2(0.625") + 8(1.125") + 7(1.125")$$
$$b_{\min} = 21.13" \le 24" \therefore OK$$

$$A_{s,\min} = \frac{3\sqrt{f'c} b_w d}{f_y}$$
$$A_{s,\min} = \frac{3\sqrt{4000} psi(24")(25.81")}{60000 psi}$$
$$A_{s,\min} = 1.96in^2 < 8in^2 \therefore OK$$

$$A_{s,\min} = \frac{200 b_w d}{f_y}$$
$$A_{s,\min} = \frac{200(24")(25.81")}{60000 psi}$$
$$A_{s,\min} = 2.06in^2 < 8in^2 \therefore OK$$

Use (8) #9 bottom bars

(-)M

-M = 1094 ft-k
f'c = 4 ksi
fy = 60 ksi
#5 stirrups
1.5" cover – bottom
0.75" cover – top

Try (12) #9

d = 28.5" – 0.75" – 0.625" - 0.625" – 0.5(1.125")
d = 25.94"

A_s = 12(1 in^2)
A_s = 12 in^2

$$a = \frac{A_s fy}{0.85 f'cb}$$
$$a = \frac{12in^2(60ksi)}{0.85(4ksi)(24'')}$$
$$a = 8.82''$$

$$c = \frac{a}{\beta}$$
$$c = \frac{8.82''}{0.85}$$
$$c = 10.38'' \qquad \leq 0.375(26.56'') = 9.73'' \therefore \phi \neq 0.9$$

$$\varepsilon_t = \frac{0.003(d-c)}{c}$$
$$\varepsilon_t = \frac{0.003(25.94''-10.38'')}{10.38''}$$
$$\varepsilon_t = 0.004497$$

$$\varepsilon_y = \frac{f_y}{E_s}$$
$$\varepsilon_y = \frac{60ksi}{29,000ksi}$$
$$\varepsilon_y = 0.002069$$

$$\phi = 0.65 + 0.25\left(\frac{\varepsilon_t - \varepsilon_y}{0.005 - \varepsilon_y}\right)$$

$$\phi = 0.65 + 0.25\left(\frac{0.004497 - 0.002069}{0.005 - 0.002069}\right)$$

$$\phi = 0.86$$

$$\phi M_n = \phi A_s f_y (d - a/2)$$

$$\phi M_n = 0.8(12in^2)(60ksi)\left(25.94'' - \frac{8.82''}{2}\right)$$

$$\phi M_n = 13{,}332 in - k$$

$$\phi M_n = 1{,}111 ft - k$$

$$\phi M_n = 1{,}111 ft - k \geq M_u = 1{,}094 ft - k \therefore OK$$

$$b_{\min} = 2(1.5'') + 2(0.625'') + 12(1.125'') + 11(1.125'')$$

$$b_{\min} = 30.125'' \leq 90'' \therefore OK$$

$$A_{s,\min} = \frac{3\sqrt{f'c}\, b_w d}{f_y}$$

$$A_{s,\min} = \frac{3\sqrt{4000}\, psi(24'')(25.94'')}{60000\, psi}$$

$$A_{s,\min} = 1.97 in^2 < 12 in^2 \therefore OK$$

$$A_{s,\min} = \frac{200 b_w d}{f_y}$$

$$A_{s,\min} = \frac{200(24'')(25.94'')}{60000\, psi}$$

$$A_{s,\min} = 2.08 in^2 < 12 in^2 \therefore OK$$

Use (12) #9 top bars

Interior Girder Shear/Torsion Reinforcement

w_u = 1.2D + 1.6L
w_u = 1.2(5.33 klf) + 1.6(3.6 klf)
w_u = 12.16 klf

Stirrups

Shear

$$V_u = \frac{1.15 w_u l_n}{2} = \frac{1.15(12.2klf)(28.5')}{2} = 200k * CONTROLS$$

$$V_u = \frac{12.2klf(28.5')}{2} = 174k$$

$$V_c = 2\sqrt{f'c} b_w d$$

$$V_c = 2\sqrt{4000} psi(24")(25.81")$$

$$V_c = 78.4k$$

$$0.5\phi V = 0.5(0.75)V_c$$

$$0.5\phi V_n = 0.5(0.75)(78.4k)$$

$$0.5\phi V_n = 29.4k < 200k \therefore Stirrups\ \mathrm{Re} quired$$

$$V_s = \frac{V_u}{\phi} - V_c$$

$$V_s = \frac{200k}{0.75} - 78.4k$$

$$V_s = 188.3k$$

Try #5 Stirrups with two legs $A_v = 2(0.31in^2) = 0.62in^2$

$$V_s = \frac{A_v f_y d}{s}$$

$$s = \frac{A_v f_y d}{V_s}$$

$$s = \frac{0.62in^2(60ksi)(25.81")}{188.3k}$$

$$s = 10.2" \rightarrow 4"$$

$$\Rightarrow A_V = 0.49in^2$$

Torsion

M_{joists} = 227 ft-k – 87.6 ft-k = 139.4 ft-k

$$T_u = \frac{139.4\,ft-k}{2} = 69.7\,ft-k$$

$$T_{n,req} = \frac{69.7\,ft-k}{0.75} = 92.9\,ft-k$$

$$T_n = \frac{2A_oA_t f_{yv}\cot\theta}{s}$$

θ = 45°

f_{yv} = 60 ksi

p_h = 2[28.5” – 2(1.5”) – 2(0.625”)] + 2[24” – 2(1.5”) – 2(0.625”)] = 88”

A_{oh} = 24.25”(19.75”) = 479 in^2

A_o = 0.83A_{oh} = 0.85(479 in^2) = 407 in^2

$$A_T = \frac{T_{n,req}s}{2A_o f_{yv}\cot\theta}$$

$$A_T = \frac{92.9\,ft-k(4")}{2(407in^2)(60ksi)\cot(45)} = 0.0076in^2$$

$$A = 2A_T + A_V$$

$$A = 2(0.0076in^2) + 0.49in^2$$

$$A = 0.51in^2 < 0.62in^2 \therefore OK$$

$$A_{s,\min} = \frac{50 b_w s}{f_y}$$

$$A_{s,\min} = \frac{50(24")(4")}{60000\,psi}$$

$$A_{s,\min} = 0.08 in^2 \leq 0.62 in^2 \;\therefore OK$$

Use #5 closed stirrups with two legs at 4" o.c.

Longitudinal Reinforcement

s = 12"

$$A_l = \frac{p_h \cot^2 \theta A_T}{s}$$

$$A_T = \frac{88" \cot^2(45)(0.0076 in^2)}{12"}$$

$$A_T = 0.056 in^2 \leq A_{\#4} = 0.2 in^2$$

Use #4 longitudinal reinforcement at 12" spacing

Interior Girder Deflection Calculations

L = 30'
D = 5332 PLF
L = 3538 PLF
f'c = 4,000 psi
I = 46,299 in^4

$$E = w_c^{1.5}(33)\sqrt{f'c}$$

$$E = (150PCF)^{1.5}(33)(\sqrt{4000}\,psi$$

$$E = 3{,}834{,}254\,psi$$

$$\Delta_{TL,allow} = \frac{l}{360}$$

$$\Delta_{TL,allow} = \frac{30'(12''/\,ft)}{360}$$

$$\Delta_{TL,allow} = 1''$$

$$\Delta_{LL,allow} = \frac{l}{480}$$

$$\Delta_{LL,allow} = \frac{30'(12''/\,ft)}{480}$$

$$\Delta_{LL,allow} = 0.75''$$

$$\Delta_{TL} = \frac{5wl^4}{384EI}$$

$$\Delta_{TL} = \frac{5(739lb/in)(360in)^4}{384(3{,}834{,}254\,psi)(46{,}299in^4)}$$

$$\Delta_{TL} = 0.91'' < 1'' \therefore OK$$

$$\Delta_{LL} = \frac{5wl^4}{384EI}$$

$$\Delta_{LL} = \frac{5(295lb/in)(360in)^4}{384(3{,}834{,}254\,psi)(46{,}299in^4)}$$

$$\Delta_{LL} = 0.36'' < 0.75'' \therefore OK$$

Exterior Girder Design

Loads

Dead

Self Weight: $\left(\frac{24''}{12''/ft}\right)\left(\frac{16''}{12''/ft}\right)(150PCF) = 400PLF$

SDL: $15PSF\left(\frac{46'}{2}\right) = 345PLF$

Joists: 119 PSF(23’) = 2,737 PLF

Façade: $20PSF\left(\frac{18'+13'}{2}\right) = 310PLF$

Live: ~~100PSF(23’) = 2,300 PLF~~
65 PSF(23’) = 1,495 PLF

$$A_T = 38'(23') = 690SF$$

$$L = L_o\left(0.25 + \frac{15}{\sqrt{2x690SF}}\right)$$

$$L = 0.65L_o$$

w_D = 400 PLF + 345 PLF + 2737 PLF + 310 PLF = 3792 PLF = 3.8 KLF
w_L = 1495 PLF = 1.5 KLF

Moments

l_n = 30'
w_D = 3.8 KLF
w_L = 1.5 KLF

ACI 8.3.3 Moments	Dead	Live	Seismic
$\pm\dfrac{wl^2}{16}$	214 ft-k	85 ft-k	21 ft-k
$+\dfrac{wl^2}{14}$	244 ft-k	97 ft-k	21 ft-k
$-\dfrac{wl^2}{10}$	342 ft-k	135 ft-k	21 ft-k
$-\dfrac{wl^2}{11}$	311 ft-k	123 ft-k	21 ft-k

Red = Worst Case (+) Moment
Blue= Worst Case (-) Moment

Load Combinations

1. 1.2D + 1.6L
2. 1.23D + L + E
3. 0.93D + E

$$E = \rho Q_E + 0.2 S_{DS} D$$
$$E = (1)Q_E + 0.2(0.153)D$$
$$E = Q_E + 0.03D$$

(+) Moment = 448 ft-k	
Load Case 1	**448 ft-k**
Load Case 2	419 ft-k
Load Case 3	248 ft-k

(-) Moment = -627 ft-k	
Load Case 1	**-627 ft-k**
Load Case 2	-577 ft-k
Load Case 3	-339 ft-k

Determine B_{eff}

$6h_f$ = 6(4.5") = 27" *CONTROLS
½ clear span = 0.5(28')(12"/ft) = 168"
½ to next beam = 0.5(46')(12"/ft) = 276"

Flexural Reinforcement

(+)M

+M = 448 ft-k
f'c = 4 ksi
fy = 60 ksi
#4 stirrups
1.5" cover – bottom
0.75" cover – top

Try (6) #8

d = 28.5" – 1.5" – 0.5" – 0.5(1")
d = 26"

A_s = 6(0.79 in^2)
A_s = 4.74 in^2

$$a = \frac{A_s fy}{0.85 f'cb}$$
$$a = \frac{4.74in^2(60ksi)}{0.85(4ksi)(27'')}$$
$$a = 3.1''$$

$$c = \frac{a}{\beta}$$
$$c = \frac{3.1''}{0.85}$$
$$c = 3.64'' \qquad \leq 0.375(26'') = 9.75'' \therefore \phi = 0.9$$

$$\phi M_n = \phi A_s f_y (d - a/2)$$
$$\phi M_n = 0.9(4.74in^2)(60ksi)\left(26'' - \frac{3.1''}{2}\right)$$
$$\phi M_n = 6258 in - k$$
$$\phi M_n = 522 ft - k$$

$$\phi M_n = 522 ft - k \geq M_u = 448 ft - k \therefore OK$$

$$b_{\min} = 2(1.5'') + 2(0.5'') + 6(1'') + 5(1'')$$
$$b_{\min} = 15'' \leq 16'' \therefore OK$$

$$A_{s,\min} = \frac{3\sqrt{f'c}\,b_w d}{f_y}$$

$$A_{s,\min} = \frac{3\sqrt{4000}\,psi(16")(26")}{60000\,psi}$$

$$A_{s,\min} = 1.32in^2 < 4.74in^2 \;\therefore OK$$

$$A_{s,\min} = \frac{200 b_w d}{f_y}$$

$$A_{s,\min} = \frac{200(16")(26")}{60000\,psi}$$

$$A_{s,\min} = 1.39in^2 < 4.74in^2 \;\therefore OK$$

Use (6) #8 bottom bars

(-)M

-M = 627 ft-k
f'c = 4 ksi
fy = 60 ksi
#4 stirrups
1.5" cover – bottom
0.75" cover – top

Try (8) #8

d = 28.5" – 0.75" – 0.5" – 0.625" - 0.5(1")
d = 26.13"

A_s = 8(0.79 in^2)
A_s = 6.32 in^2

$$a = \frac{A_s fy}{0.85 f'cb}$$
$$a = \frac{6.32in^2(60ksi)}{0.85(4ksi)(16'')}$$
$$a = 6.97''$$

$$c = \frac{a}{\beta}$$
$$c = \frac{6.97''}{0.85}$$
$$c = 8.2'' \qquad \le 0.375(26.13'') = 9.8'' \therefore \phi = 0.9$$

$$\phi M_n = \phi A_s f_y (d - a/2)$$
$$\phi M_n = 0.9(6.32in^2)(60ksi)\left(26.13'' - \frac{6.97''}{2}\right)$$
$$\phi M_n = 7{,}728 in - k$$
$$\phi M_n = 644 ft - k$$

$$\phi M_n = 644 ft - k \ge M_u = 627 ft - k \therefore OK$$

$$b_{\min} = 2(1.5'') + 2(0.5'') + 8(1'') + 7(1'')$$
$$b_{\min} = 19'' \le 27'' \therefore OK$$

$$A_{s,\min} = \frac{3\sqrt{f'c}b_w d}{f_y}$$

$$A_{s,\min} = \frac{3\sqrt{4000}psi(16")(26.13")}{60000psi}$$

$$A_{s,\min} = 1.32in^2 < 6.32in^2 \therefore OK$$

$$A_{s,\min} = \frac{200b_w d}{f_y}$$

$$A_{s,\min} = \frac{200(16")(26.13")}{60000psi}$$

$$A_{s,\min} = 1.39in^2 > 6.32in^2 \therefore OK$$

Use (8) #8 top bars

Exterior Girder Shear/Torsion Reinforcement

w_u = 1.2D + 1.6L
w_u = 1.2(3.8 klf) + 1.6(1.5 klf) = 7 klf

Stirrups

Shear

$$V_u = \frac{1.15 w_u l_n}{2} = \frac{1.15(7klf)(29')}{2} = 115k * CONTROLS$$
$$V_u = \frac{7klf(29')}{2} = 101k$$

$$V_c = 2\sqrt{f'c}b_w d$$
$$V_c = 2\sqrt{4000}psi(16")(26")$$
$$V_c = 52.6k$$

$$0.5\phi V = 0.5(0.75)V_c$$
$$0.5\phi V_n = 0.5(0.75)(52.6k)$$
$$0.5\phi V_n = 19.73k < 115k \therefore Stirrups \text{ Re} quired$$

$$V_s = \frac{V_u}{\phi} - V_c$$
$$V_s = \frac{115k}{0.75} - 52.6k$$
$$V_s = 100.73k$$

Try #4 Stirrups with two legs $\quad$ $A_v = 2(0.2in^2) = 0.4in^2$

$$V_s = \frac{A_v f_y d}{s}$$
$$s = \frac{A_v f_y d}{V_s}$$
$$s = \frac{0.4in^2(60ksi)(26")}{100.73k}$$
$$s = 6.19" \rightarrow 5"$$

Use 5" spacing $\quad \Rightarrow A_v = 0.321in^2$

Torsion

M_{joists} = 227 ft-k

$$T_u = \frac{227\,ft-k}{2} = 113.5\,ft-k$$

$$T_{n,req} = \frac{113.5\,ft-k}{0.75} = 151.3\,ft-k$$

$$T_n = \frac{2A_oA_tf_{yv}\cot\theta}{s}$$

θ = 45°

f_{yv} = 60 ksi

p_h = 2[28.5” – 2(1.5”) – 2(0.5”)] + 2[20” – 2(1.5”) – 2(0.5”)] = 73”

A_{oh} = 24.5”(12”) = 294 in^2

A_o = 0.83A_{oh} = 0.85(294) = 250 in^2

$$A_T = \frac{T_{n,req}s}{2A_of_{yv}\cot\theta}$$

$$A_T = \frac{151.3\,ft-k(5'')}{2(250in^2)(60ksi)\cot(45)} = 0.025in^2$$

$$A = 2A_T + A_V$$

$$A = 2(0.025in^2) + 0.321in^2$$

$$A = 0.37in^2 < 0.4in^2 \;\therefore OK$$

$$A_{s,\min} = \frac{50b_ws}{f_y}$$

$$A_{s,\min} = \frac{50(16'')(5'')}{60000\,psi}$$

$$A_{s,\min} = 0.067in^2 \leq 0.4in^2 \;\therefore OK$$

Use #4 closed stirrups with two legs at 5” o.c.

Longitudinal Reinforcement

s = 12”

$$A_l = \frac{p_h \cot^2 \theta A_T}{s}$$

$$A_T = \frac{73'' \cot^2(45)(0.025in^2)}{12''}$$

$$A_T = 0.152in^2 \leq A_{\#4} = 0.2in^2$$

Use #4 longitudinal reinforcement at 12” spacing

Exterior Girder Deflection Calculations

L = 30’
D = 3792 PLF
L = 1495 PLF
f’c = 4,000 psi
I = 30,866 in^4

$$E = w_c^{1.5}(33)\sqrt{f'c}$$
$$E = (150PCF)^{1.5}(33)(\sqrt{4000}psi$$
$$E = 3{,}834{,}254\,psi$$

$$\Delta_{TL,allow} = \frac{l}{360}$$
$$\Delta_{TL,allow} = \frac{30'(12''/ft)}{360}$$
$$\Delta_{TL,allow} = 1''$$

$$\Delta_{LL,allow} = \frac{l}{480}$$
$$\Delta_{LL,allow} = \frac{30'(12''/ft)}{480}$$
$$\Delta_{LL,allow} = 0.75''$$

$$\Delta_{TL} = \frac{5wl^4}{384EI}$$
$$\Delta_{TL} = \frac{5(441lb/in)(360in)^4}{384(3{,}834{,}254\,psi)(30{,}866in^4)}$$
$$\Delta_{TL} = 0.81'' < 1'' \therefore OK$$

$$\Delta_{LL} = \frac{5wl^4}{384EI}$$
$$\Delta_{LL} = \frac{5(125lb/in)(360in)^4}{384(3{,}834{,}254\,psi)(30{,}866in^4)}$$
$$\Delta_{LL} = 0.23'' < 0.75'' \therefore OK$$

Summary of Actual and Allowable Loads

Interior Girders

M_u^+	782 ft-k	ϕM_n^+	900 ft-k	OK
M_u^-	1094 ft-k	ϕM_n^-	1111 ft-k	OK
V_u	200 k	ϕV_n	318 k	OK
T_u	69.7 ft-k	ϕT_n	92.8 ft-k	OK
Δ_{TL}	0.91”	**$\Delta_{TL,allow}$ (l/360)**	1”	OK
Δ_{LL}	0.36”	**$\Delta_{TL,allow}$ (l/480)**	0.75”	OK

Exterior Girders

M_u^+	448 ft-k	ϕM_n^+	522 ft-k	OK
M_u^-	627 ft-k	ϕM_n^-	644 ft-k	OK
V_u	115 k	ϕV_n	177 k	OK
T_u	114 ft-k	ϕT_n	150 ft-k	OK
Δ_{TL}	0.81”	**$\Delta_{TL,allow}$ (l/360)**	1”	OK
Δ_{LL}	0.23”	**$\Delta_{TL,allow}$ (l/480)**	0.75”	OK

Appendix C

CIP Column Design Calculations

Column Loads

Dead

Façade	
Glass Curtain Wall	15 PSF
Precast	20 PSF
Roof	38 PSF
Typical Floor	
Joists (24" Joists)	119 PSF
Joists (16" Joists)	95 PSF
SDL	15 PSF
Girders	600 PLF

Live

Roof	
Snow	30 PSF
Mechanical	150 PSF
Typical Floor	
Office	100 PSF
Mechanical	150 PSF

Load Combinations

1.2D + 1.6L *CONTROLS
1.23D + L + E
0.93D + E

Example of a Column Axial Load Spreadsheet

Level	DL (k)	LL (k)	RedLL (k)	1.2DL (k)	1.6RedLL (k)
Roof	51.8	20.5	11.0	62.2	17.6
12	117.5	68.4	36.7	141.0	58.7
11	117.5	68.4	36.7	141.0	58.7
10	117.5	68.4	36.7	141.0	58.7
9	117.5	68.4	36.7	141.0	58.7
8	117.5	68.4	36.7	141.0	58.7
7	117.5	68.4	36.7	141.0	58.7
6	117.5	68.4	36.7	141.0	58.7
5	117.5	68.4	36.7	141.0	58.7
4	117.5	68.4	36.7	141.0	58.7
3	117.5	68.4	36.7	141.0	58.7
2	120.5	68.4	36.7	144.6	58.7
	1347	773	415	1617	664

Column 5,4,28,27,26		
A_T =	684	SF
K_{LL}=	4	
L=	0.54	Lo

Max Pu	2281	k

	1.2DL (k)	1.6LL (k)	Pu (k)
10-12	344	135	479
6-12	908	370	1278
1-12	1617	664	2281

Appendix D

CIP Shearwall Design Calculations

Shearwall Reinforcement Calculations

1

Shearwall Reinforcement

$f'c = 4000$ psi

8" thick
30' long (horizontally)

$Mu = 34{,}501$ FT·K

$I_{n.a.} = \frac{(0.667')(30')^3}{12} = 1500 \text{ FT}^4$

$Ag = 0.667'(30') = 20 \text{FT}^2$

$fc = \frac{Pu}{Ag} + \frac{Mu\,hw/2}{I_{n.a.}}$

$= \frac{4661 \text{K}}{20 \text{ FT}^2} + \frac{34{,}501 \text{ FT·K}(30')/2}{1500 \text{ FT}^4}$

$= 4.01 \text{ KSi} > 0.2(4 \text{ KSi}) = 0.8 \text{ KSi}$

∴ boundary elements needed

$Vu = 370$ K

$2 Acv\sqrt{f'c} = \frac{2(8")(30' \times 12"/ft)\sqrt{4000}}{1000} = 364\text{K} < 370\text{K}$

∴ Two curtains reinf. req'd

Req'd longitudinal and transverse reinforcement in wall

$\rho v = \frac{Asv}{Acv} = \rho n > 0.0025$ max

$Acv = 8(12) = 96 \text{ in}^2/\text{ft}$ of wall

$0.0025(96) = 0.24 \text{ in}^2/\text{ft}$

use #4 bars in two curtains
$As = 0.2(2) = 0.4 \text{ in}^2 > 0.24 \text{ in}^2$ ∴ OK

$S_{req'd} = \frac{2(0.2 \text{ in}^2)(12"/ft)}{0.24} = 20" \rightarrow 18"$

$S = \frac{lw}{5} = 72"$ or 18" * controls.

2

Determine reinforcement for shear

Assume #4 bars @ 18" e.w.

$$\frac{h_w}{l_w} = \frac{180.75'}{30'} = 6.03 > 2$$

$$\rho_n = \frac{2(0.2)}{8(12)} = 0.004167$$

$$\phi = 0.6$$

$$A_{cv} = 8(30 \cdot 12) = 2880 \text{ in}^2$$

$$\phi V_n = \phi A_{cv}\left(2\sqrt{f'_c} + \rho_n f_y\right)$$
$$= 0.6(8)(30 \cdot 12)\left(2\sqrt{4000} + 0.004167 \cdot 60000\right)$$
$$= 651 \text{ k} > V_u = 370 \text{K}$$

Use (2) curtains of #4 bars spaced at 18" o.c. in both horizontal and vertical directions

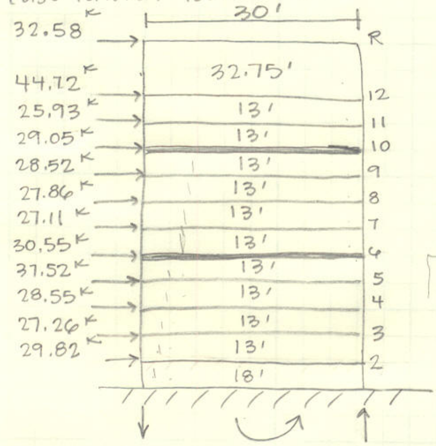
$$b_{min} = 0.75(2) + 0.5(2) + 0.5(2) + 1(1) = 4.5"$$

Shearwall Boundary Element Design Calculations

1

Shearwall Boundary Element Design

worst case lateral load - wall 3



$h_{1-12} = 180.75'$

$h_{6-12} = 110.75'$

$h_{10-12} = 58.75'$

$M_{1-12} = 29.82(18') + 27.26(31') + 28.55(44') + 37.52(57')$
$+ 30.55(70') + 27.11(83') + 27.86(96') + 28.52(109')$
$+ 29.05(122') + 25.93(135') + 44.72(148')$
$+ 32.58(180.75')$

$M_{1-12} = 34{,}501$ FT·K (factored already)

$M_{6-12} = 27.11(13') + 27.86(26') + 28.52(39') +$
$29.05(52') + 25.93(65') + 44.72(78')$
$+ 32.58(110.75')$
$= 12{,}482$ FT·K (factored already)

$M_{10-12} = 25.93(13') + 44.72(26') + 32.58(58.75')$
$= 3{,}414$ FT·K

$P_{w,u,1-12} = \dfrac{34{,}501}{30'} = 1{,}151$ K

$P_{wu,6-12} = \dfrac{12{,}482 \text{ FT·K}}{30'} = 417$ K

$P_{wu,10-12} = \dfrac{3{,}414}{30'} = 114$ K

2

col. 10, 11, 12, 19, 20, 21

	DL	LLR	1.6W	Mx	My
10-12	376.4	180.3	114	92	115
6-12	1005.2	452.7	417	92	115
1-12	1791.2	793.2	1151	92	115

Load combinations:

	1-12	6-12	10-12
1.4D	2508		
1.2D + 1.6L	3419		
1.2D + 0.8W	2725		
1.2D + 1.6W + L	4094	2076	746

col. 10, 11, 12, 19, 20, 21

Top
$P_u = 746$ K
$M_x = 92$ FT·K
$M_y = 115$ FT·K
$P_u = 770$ K

self = 19.9 K 1.2self = 24 K

$\boxed{18 \times 18 \quad (4)\ \#9}$

Mid
$P_u = 2100$
$M_x = 92$
$M_y = 115$
$P_u = 2159$

self = 49 1.2self = 59

$\boxed{30 \times 30 \quad (8)\ \#10}$

Bot.
$P_u = 4177$
$M_x = 92$
$M_y = 115$
$P_u = 4332$

self = 95 1.2self = 114

$\boxed{42 \times 42 \quad (20)\ \#9}$

$\phi P_n = 4448$
$\phi M_{nx} = 93$
$\phi M_{ny} = 116$

Hand calc. capacity check

$A_g = (36)(36) = 1296\ in^2$

$A_{st} = 43.68\ in^2$

$\rho_s = \frac{43.68}{1296} = 0.033704$ > 0.01, < 0.06

$\phi P_n = 0.8(0.7)[0.85(5\ ksi)(1296 - 43.68) + 60(43.68)]$

$\phi P_n = 4{,}448$ K $= 4{,}448$ K ∴ OK

3

col. 13,18

	DL	LLR	1.6W	Mx	My
10-12	218	102	114	92	589
6-12	800	362	417	92	589
1-12	1585	714	1151	92	589

Load combinations	1-12	6-12	10-12
1.4D	2219		
1.2D+1.6L	3045		
1.2D + 0.8W	2478		
1.2D+1.6W + L	3767	1739	478

col. 13,18

TOP

$P_u = 478$
$M_x = 92$
$M_y = 589$
$P_u = 521$

self - 35.4 1.2 self = 42.5

24 x 24
12-#9

Mid

$P_u = 1739 + 43 = 1782$
$M_x = 92$
$M_y = 589$
$P_u = 1841$

self 1.2 = 59

30x 30
12- #8

Bot.

$P_u = 3767 + 43 + 59 = 3869$
$M_x = 92$
$M_y = 589$
$P_u = 3983$

self- 1.2 = 114

36x36
24-#10

$\phi P_n = 4036$ K
$\phi M_{nx} = 92$ FT·K
$\phi M_{ny} = 587$ FT·K

Hand calc: capacity check:

$A_g = 36(36) = 1296 \text{ in}^2$
$A_{st} = 30.48$
$\rho = \frac{30.48}{1296} = 0.0235 \geq 0.01$
≤ 0.06

$\phi P_n = 0.8(0.7)[0.85(5)(1296-30.48) + 60(30.48)]$
$= 4036 \text{ K} > 3983 \therefore$ OK
$= 4{,}036 \text{ K} \therefore$ OK

Shearwall Deflections

Wall	Max Δx		Max Δy		Max Δz	
	E/S	W/N	E/S	W/N	E/S	W/N
1	2.053982”	2.032261”	1.503888”	1.503888”	0.164363”	-0.161891”
2	2.053982”	2.032261”	1.547330”	1.547330”	0.012486”	-0.289300”
3	2.053982”	2.032261”	1.570137”	1.570137”	0.148668”	-0.150556”
4	2.053982”	2.053982”	1.526333”	1.547330”	0.424625”	0.012286”
5	2.032261”	2.032261”	1.526333”	1.547330”	0.137942”	-0.289300”

$$\Delta_{allow} = \frac{h}{400} = \frac{180.75'(12''/ft)}{400} = 5.42''$$

All shearwall deflections are less than the allowable 5.42” ∴ OK

Appendix E

Representative Spread Footing Design Calculations

Spread Footing Design – E-7

Loads

Axial Loads from Tower
D = 1956 k
LLr = 794 k
W = 720 k

Garage Loads

Level	LL	SDL	Slab Thickness	Drop Size
1	100 PSF	15 PSF	9”	10’x10’x5.5”
G1	50 PSF	15 PSF	8”	10’x10’x5.5”
G2	50 PSF	15 PSF	8”	10’x10’x5.5”
G4	50 PSF	15 PSF	4”	------------

Level 1

LL: 100(0.5) PSF(900 SF) = 45,000 LB
SDL: 15 PSF(900 SF) = 13,500 LB
Drops: 10’(10’)(5.5”/12)(150 PCF) = 6,875 LB
Slab: 9”/12(900 SF)(150 PCF) = 101,250 LB
Column: (42”x42”)/144in^2(10’)(150 PCF) = 18,375 LB

Level G1 and G2

LL: 25 PSF(900 SF) = 22,500 LB
SDL: 15 PSF(900 SF) = 13,500 LB
Drops: 10’(10’)(5.5”/12)(150 PCF) = 6,875 LB
Slab: (8”/12)(900 SF)(150 PCF) = 90,000 LB
Column: (42”x42”)/144in^2(10’)(150 PCF) = 18,375 LB

Level G3

LL: 25 PSF(900 SF) = 22,500 LB
SDL: 15 PSF(900 SF) = 13,500 LB
Slab: (4”/12)(900 SF)(150 PCF) = 45,000 LB
Column: (42”x42”)/144in^2(3’)(150 PCF) = 5,513 LB

D = 462 k

L = 113 k

Service: 4,045 k
Factored: 1.2D + 1.6W + L = 4,961 k
Moments are negligible.

Footing Size and Reinforcement

$q_a = 40KSF$ (soils report)

$$q_a \geq \frac{P}{A}$$
$$40KSF \geq \frac{4045k}{B^2}$$
$$B \geq 10.1'$$
$$B = 10.5'$$
$$q = \frac{P_u}{A}$$
$$q = \frac{4961k}{(10.5')^2}$$
$$q = 45KSF$$
$$q = 312.5PSI$$

f'c = 3000 psi (Structural Notes)
42" x 42" column
Df = 36"

Try #11 bars

$$v_c = \phi 4\sqrt{f'c}$$
$$v_c = 0.75(4)\sqrt{3000}\,psi$$
$$v_c = 164\,psi$$

$$d^2\left(v_c + \frac{q}{\beta_c}\right) + d\left(v_c + \frac{q}{2}\right)w = \frac{q}{4}\left(BL - w^2\right)$$
$$d^2\left(164\,psi + \frac{312.5\,psi}{4}\right) + d\left(164\,psi + \frac{312.5\,psi}{2}\right)42'' = \frac{312.5\,psi}{4}\left(126''^2 - 42''^2\right)$$
$$d = 45.2''$$

h = 45.2" + 3" + 1.41"
h = 49.61"

h = 50”

d = 50” – 3” – 1.41” = 45.6”

$$l = \frac{10.5'-3.5'}{2}$$
$$l = 3.5'$$

$$M_u = \frac{ql^2}{2}$$
$$M_u = \frac{45KSF(3.5')^2}{2}$$
$$M_u = 275.6\,ft-k$$

$$a = \frac{A_s f_y}{0.85 f'cb}$$
$$a = \frac{A_s(60ksi)}{0.85(3ksi)(12'')}$$
$$a = 1.96A_s$$

$$M_u = \phi M_n = \phi A_s f_y\left(d - \frac{a}{2}\right)$$
$$275.6\,ft-k(12in/ft) = 0.9A_s(60ksi)\left(45.6''-\frac{1.96A_s}{2}\right)$$
$$A_s \geq 1.33in^2 < 1.56in^2 \therefore OK$$

$$\rho = \frac{A_s}{bh}$$
$$\rho = \frac{1.56in^2}{12''(51'')}$$
$$\rho = 0.002549 \geq 0.0018 \therefore OK$$

a = 1.96(1.56in^2) = 3.06in

c = 3.06in/0.85 = 3.6” ≤0.375(45.6”) = 17.1” $\phi = 0.9$

$$\phi B_n = \phi(0.85) f'cA_1$$
$$\phi B_n = 0.65(0.85)(3ksi)(42")^2$$
$$\phi B_n = 2{,}924k(2) = 5{,}848k > 4{,}961k \therefore OK$$

ACI 22.5.5

$$\sqrt{\frac{A_1}{A_2}} = \sqrt{\frac{126^2}{42^2}} = 3 \rightarrow 2$$

$$A_{s,\min} = 0.005A_{col}$$
$$A_{s,\min} = 0.005(42in)^2$$
$$A_{s,\min} = 8.82in^2 < 9.48in^2 = A_{(12)\#8}$$

Use 10.5’ x 10.5’ x 50” square footing with (11) #11 each way and (12) #8 dowels

Spread Footing Design – E-9

Loads

Axial Loads from Tower
D = 2141 k
LLr = 715 k

Garage Loads

Level	LL	SDL	Slab Thickness	Drop Size
1	100 PSF	15 PSF	9"	10'x10'x5.5"
G1	50 PSF	15 PSF	8"	10'x10'x5.5"
G2	50 PSF	15 PSF	8"	10'x10'x5.5"
G4	50 PSF	15 PSF	4"	------------

Level 1

LL: 100(0.5) PSF(900 SF) = 45,000 LB
SDL: 15 PSF(900 SF) = 13,500 LB
Drops: 10'(10')(5.5"/12)(150 PCF) = 6,875 LB
Slab: 9"/12(900 SF)(150 PCF) = 101,250 LB
Column: (42"x42")/144in^2(10')(150 PCF) = 18,375 LB

Level G1 and G2

LL: 25 PSF(900 SF) = 22,500 LB
SDL: 15 PSF(900 SF) = 13,500 LB
Drops: 10'(10')(5.5"/12)(150 PCF) = 6,875 LB
Slab: (8"/12)(900 SF)(150 PCF) = 90,000 LB
Column: (42"x42")/144in^2(10')(150 PCF) = 18,375 LB

Level G3

LL: 25 PSF(900 SF) = 22,500 LB
SDL: 15 PSF(900 SF) = 13,500 LB
Slab: (4"/12)(900 SF)(150 PCF) = 45,000 LB
Column: (42"x42")/144in^2(3')(150 PCF) = 5,513 LB

D = 462 k
L = 113 k

Service: 3,431 k
Factored: 1.2D + 1.6L = 4,448 k
Moment are negligible.

Footing Reinforcement

$q_a = 40KSF$ (soils report)

$$q_a \geq \frac{P}{A}$$

$$40KSF \geq \frac{3{,}434k}{B^2}$$

$$B \geq 9.26'$$

$$B = 9.5'$$

$$q = \frac{P_u}{A}$$

$$q = \frac{4448k}{(9.5')^2}$$

$$q = 49.3ksf$$

$$q = 342.3psi$$

f'c = 3000 psi (Structural Notes)
42" x 42" column

Try #11 bars

$$v_c = \phi 4\sqrt{f'c}$$

$$v_c = 0.75(4)\sqrt{3000}\,psi$$

$$v_c = 164\,psi$$

$$d^2\left(v_c + \frac{q}{\beta_c}\right) + d\left(v_c + \frac{q}{2}\right)w = \frac{q}{4}\left(BL - w^2\right)$$

$$d^2\left(164\,psi + \frac{342.3\,psi}{4}\right) + d\left(164\,psi + \frac{342.3\,psi}{2}\right)42'' = \frac{342.3\,psi}{4}\left(114''^2 - 42''^2\right)$$

$$d = 39.96''$$

h = 39.96" + 3" + 1.41"
h = 44.41"
h = 45"

d = 45" – 3" – 1.41" = 40.6"

$$l = \frac{10' - 3.5'}{2}$$
$$l = 3.25'$$

$$M_u = \frac{ql^2}{2}$$
$$M_u = \frac{49.3KSF(3.25')^2}{2}$$
$$M_u = 260.4\, ft - k$$

$$a = \frac{A_s f_y}{0.85 f'cb}$$
$$a = \frac{A_s(60ksi)}{0.85(3ksi)(12'')}$$
$$a = 1.96A_s$$

$$M_u = \phi M_n = \phi A_s f_y\left(d - \frac{a}{2}\right)$$
$$260.4\, ft - k(12in/ft) = 0.9A_s(60ksi)\left(40.6'' - \frac{1.96A_s}{2}\right)$$
$$A_s \geq 1.38in^2 < 1.56in^2 \therefore OK$$

$$\rho = \frac{A_s}{bh}$$
$$\rho = \frac{1.56in^2}{12''(50'')}$$
$$\rho = 0.0026 \geq 0.0018 \therefore OK$$

a = 1.96(1.56in^2) = 3.06 in

c = 3.06 in/0.85 = 3.6” ≤0.375(40.6”) = 15.2” $\phi = 0.9$

$$\phi B_n = \phi(0.85) f'cA_1$$
$$\phi B_n = 0.65(0.85)(3ksi)(42'')^2$$
$$\phi B_n = 2{,}924k(2) = 5{,}848k > 4{,}448k \therefore OK$$

ACI 22.5.5

$$\sqrt{\frac{A_1}{A_2}} = \sqrt{\frac{120^2}{42^2}} = 2.85 \rightarrow 2$$

$$A_{s,\min} = 0.005A_{col}$$
$$A_{s,\min} = 0.005(42in)^2$$
$$A_{s,\min} = 8.82in^2 < 9.48in^2 = A_{(12)\#8}$$

Use 9.5' x 9.5' x 45" square footing with (10) #11 each way and (12) #8 dowels

Appendix F

Roof Design Calculations

Roof Joist Calculations and Design – 46' Span

Roof Joists - snow only

span = 46'

Loads:

Dead

SDL	= 15 PSF	
Insulation	= 1.5 PSF	
Built-up roof	= 6.5 PSF	
Metal Deck	= 2 PSF	
Joists	= 16.6	4.2 PSF
	25 PSF	

$\boxed{30 \text{ PSF}}$

Live

snow - = 30 PSF

D: 25 PSF (4') = 100 PLF + 16.6 PLF = 117 PLF

L: 30 PSF (4') = 120 PLF

Wu = 1.2(117) + 1.6 (120)
Wu = 332.4 plf

use 26K12
d = 26"
TL = 380 > 332.4 ∴ OK
LL = 203 > 192 ∴ OK   l/360 deflection

Roof Joist Calculations and Design – 30' Span

Roof Joists - Mechanical & snow

span = 30'

Loads:

Dead

SDL	= 15 PSF	
Insulation	= 1.5 PSF	
Built-up roof	= 6.5 PSF	
Metal Deck	= 3 PSF	
Joists	= 12.1	8.1 PSF

$\boxed{35 \text{ PSF}}$

D: 26 PSF(1.5') + 12.1 = 51

L: 30(1.5) = 45 plf
150(1.5) = 225 plf

1. 1.2D + 1.6L + 0.5S
2. 1.2D + 1.6S + L

1. 1.2(51) + 1.6(225) + 0.5(45) = 444 PLF

use 26 K8
d = 26"
TL = 544 > 444 ∴ OK
LL = 457 > 432 ∴ OK   l/360 deflection

Roof Deck Calculations and Design – 46' Span

Roof Deck - snow only

Dead :

SDL:	= 15 PSF
Insulation (Rigid)	= 1.5 PSF
Built-up Roof (5 ply felt & gravel)	= 6.5 PSF
Metal Deck	= 2 PSF
	25 PSF

Live:

snow = 30 PSF

span = 4'

D = 15 + 1.5 + 6.5 + 2 = 25 PSF
L = 30 PSF

Load combinations

1.4D
1.2D + 1.6L + 0.5S
* 1.2D + 1.6S + L

Wu = 1.2(25) + 1.6(30) = 78 PSF < 224 PSF ∴ OK

$\boxed{\text{use 22 gage Type F, Intermediate Rib Deck}}$

Roof Deck Calculations and Design – 30' Span

Roof Deck - Mechanical

Dead

SDL	= 15 PSF
Insulation	= 1.5 PSF
Built up roof	= 6.5 PSF
Metal Deck	= 3 PSF
	26 PSF

Live

Snow = 30 PSF
Mechanical = 150 PSF

span 2'

Load combos

1. 1.2D + 1.6L + 0.5S
2. 1.2D + 1.6S + L

1. $W_u = 1.2(26) + 1.6(150) + 0.5(30) = 287$ k < 367
2. $W_u = 1.2(26) + 1.6(30) + 150 = 230$

$\boxed{\text{use 18 gage, type F Intermediate Rib Deck}}$

Appendix G

Cost Analysis Calculations

Concrete System Cost Analysis – Typical Floor

	Cost	**Joists/Slab**							
				Unit Cost			Total Cost		
		Quantity	Unit	Material	Labor	Equip.	Material	Labor	Equip.
03310-220-0300	Concrete	1100	CY	$91.00	$0.00	$0.00	$100,100	$0	$0
03210-600-0400	Reinf.	64	Ton	$905.00	$435.00	$0.00	$57,920	$27,840	$0
03110-420-3760	Formwork	24250	SF	$2.78	$3.45	$0.00	$67,415	$83,663	$0
03310-700-1600	Placement	1100	CY	$0.00	$10.55	$4.13	$0	$11,605	$4,543
03310-300-0010	Finishing	24250	SF	$0.00	$0.31	$0.00	$0	$7,518	$0
							$225,435	**$130,625**	**$4,543**

	Cost	**Girders**							
				Unit Cost			Total Cost		
		Quantity	Unit	Material	Labor	Equip.	Material	Labor	Equip.
03310-220-0300	Concrete	117	CY	$91.00	$0.00	$0.00	$10,647	$0	$0
	Reinf.						$0	$0	$0
03210-600-0100	#3-#7	11.33	Ton	$855.00	$790.00	$0.00	$9,687	$8,951	$0
03210-600-0150	#8-#18	24.53	Ton	$855.00	$470.00	$0.00	$20,973	$11,529	$0
	Formwork						$0	$0	$0
03110-405-2650	Interior	3464	SFCA	$0.97	$4.20	$0.00	$3,360	$14,549	$0
03110-405-1650	Exterior	4208	SFCA	$0.96	$5.10	$0.00	$4,040	$21,461	$0
03110-700-0200	Placement	117	CY	$0.00	$21.00	$8.25	$0	$2,457	$965
							$48,707	**$58,946**	**$965**

	Cost	**Columns**							
				Unit Cost			Total Cost		
		Quantity	Unit	Material	Labor	Equip.	Material	Labor	Equip.
03310-220-0400	Concrete	99	CY	$96.00	$0.00	$0.00	$9,504	$0	$0
03210-600-0250	Reinf.	14.1	Ton	$855.00	$550.00	$0.00	$12,056	$7,755	$0
	Formwork								
03110-410-6650	24"x24"	728	SFCA	$0.83	$4.51	$0.00	$604	$3,283	$0
03110-410	30"x30"	1820	SFCA	$0.79	$4.40	$0.00	$1,438	$8,008	$0
03110-410-7150	36"x36"	1560	SFCA	$0.74	$4.29	$0.00	$1,154	$6,692	$0
	Placement								
03310-700-0800	24"x24"	13.5	CY	$0.00	$20.50	$8.10	$0	$277	$109
03310-700	30"x30"	42.2	CY	$0.00	$17.03	$6.70	$0	$719	$283
03310-700-1000	36"x36"	44	CY	$0.00	$13.55	$5.30	$0	$596	$233
							$24,756	**$27,330**	**$625**

	Cost	**Shearwalls**		Unit Cost			Total Cost		
		Quantity	Unit	Material	Labor	Equip.	Material	Labor	Equip.
03310-220-0300	Concrete	49	CY	$91.00	$0.00	$0.00	$4,459	$0	$0
03210-600-0700	Reinforcement	0.89	Ton	$810.00	$420.00	$0.00	$721	$374	$0
03110-455-8060	Formwork	3900	SFCA	$0.83	$2.10	$0.00	$3,237	$8,190	$0
03310-700-4950	Placement	49	CY	$0.00	$19.00	$7.45	$0	$931	$365
03350-350-0020	Finishing	3900	SF	$0.03	$0.51	$0.00	$117	$1,989	$0
							$8,534	**$11,484**	**$365**

	Cost	**Shoring**		Unit Cost			Total Cost		
		Quantity	Unit	Material	Labor	Equip.	Material	Labor	Equip.
03150-600-1500	Reshoring	24,250	SF	$0.42	$0.41	$0.00	$10,185	$9,943	$0
03150-600-3050	Shores (12 mon.)	24,250	SF	$5.76	$0.00	$0.00	$139,680	$0	$0
							$149,865	**$9,943**	**$0**

Total Cost	**Concrete**		
	Material	Labor	Equipment
Joists/Slab	$225,435	$130,625	$4,543
Girders	$48,707	$58,946	$965
Columns	$24,756	$27,330	$625
Shearwalls	$8,534	$11,484	$365
Shoring/Reshoring	$149,865	$9,943	$0
	$457,297	$238,328	$6,498
	$702,123		

Concrete System Cost Analysis – Spread Footings

Cost: Square Footing E-7 Concrete System									
				Unit Cost			Total Cost		
	Item	Quantity	Unit	Material	Labor	Equip.	Material	Labor	Equip.
03310-220-0150	Concrete	17	CY	$87.00	$0.00	$0.00	$1,479	$0	$0
03210-600-0550	Reinforcement	0.61	ton	$770.00	$350.00	$0.00	$470	$214	$0
03110-430-5150	Formwork	175	SFCA	$0.59	$2.59	$0.00	$103	$453	$0
03310-700-2600	Placement (direct chute)	17	CY	$0.00	$11.55	$0.36	$0	$196	$6
							$2,052	$863	$6
							$2,921		

Cost: Square Footing E-9 Concrete System									
				Unit Cost			Total Cost		
	Item	Quantity	Unit	Material	Labor	Equip.	Material	Labor	Equip.
03310-220-0150	Concrete	12.53	CY	$87.00	$0.00	$0.00	$1,090	$0	$0
03210-600-0550	Reinforcement	0.53	ton	$770.00	$350.00	$0.00	$408	$186	$0
03110-430-5150	Formwork	143	SFCA	$0.59	$2.59	$0.00	$84	$370	$0
03310-700-2600	Placement (direct chute)	12.53	CY	$0.00	$11.55	$0.36	$0	$145	$5
							$1,583	$701	$5
							$2,288		

Steel System Cost Analysis – Typical Floor

Cost	**Slab**								
				Unit Cost			Total Cost		
		Quantity	Unit	Material	Labor	Equip.	Material	Labor	Equip.
03310-220-1010	Concrete	356	CY	$109.00	$0.00	$0.00	$38,804	$0	$0
03210-200-0100	WWF	242.5	CSF	$12.00	$18.05	$0.00	$2,910	$4,377	$0
03110-420-6650	Edge Form	199.5	SFCA	$0.50	$4.77	$0.00	$100	$952	$0
03310-700-1400	Placement	356	CY	$0.00	$13.55	$5.30	$0	$4,824	$1,887
							$41,814	**$10,153**	**$1,887**

Cost	**Metal Deck**							
			Unit Cost			Total Cost		
		Area (SF)	Material	Labor	Equip.	Material	Labor	Equip.
05310-300-5800	3"-20 gauge	24,250	$1.71	$0.43	$0.03	$41,468	$10,428	$728
						$41,468	**$10,428**	**$728**

Cost	**Beams**							
			Unit Cost			Total Cost		
		Length (ft)	Material	Labor	Equip.	Material	Labor	Equip.
05120-640-0300	W8x10	6.5	$10.45	$3.63	$2.38	$68	$24	$15
05120-640-1100	W12x14	40	$14.65	$2.48	$1.62	$586	$99	$65
05120-640-2700	W16x26	447	$27.00	$2.18	$1.43	$12,069	$974	$639
05120-640-2900	W16x31	157	$32.50	$2.42	$1.59	$5,103	$380	$250
05120-640-3300	W18x35	148	$36.50	$3.28	$1.58	$5,402	$485	$234
05120-640-3500	W18x40	122	$42.00	$3.28	$1.58	$5,124	$400	$193
05120-640-3520	W18x46	823	$48.00	$3.28	$1.58	$39,504	$2,699	$1,300
05120-640-3700	W18x50	322	$52.50	$3.46	$1.66	$16,905	$1,114	$535
05120-640-3900	W18x55	92	$57.50	$3.46	$1.66	$5,290	$318	$153
05120-640	W18x60	76	$62.75	$3.48	$1.67	$4,769	$264	$127
05120-640-3920	W18x65	136	$68.00	$5.50	$1.68	$9,248	$748	$228
05120-640-4100	W21x44	330	$46.00	$2.96	$1.42	$15,180	$977	$469
05120-640-4900	W24x55	189	$57.50	$2.84	$1.37	$10,868	$537	$259
05120-640-5100	W24x62	103	$65.00	$2.84	$1.37	$6,695	$293	$141
05120-640-5300	W24x68	37	$71.00	$2.84	$1.37	$2,627	$105	$51
05120-640-5500	W24x76	43	$79.50	$2.84	$1.37	$3,419	$122	$59
05120-640-5800	W27x84	86	$88.00	$2.65	$1.27	$7,568	$228	$109
05120-640-6300	W30x108	43	$113.00	$2.63	$1.26	$4,859	$113	$54
05120-640	W30x124	43	$129.50	$2.72	$1.31	$5,569	$117	$56
		3243.5				**$160,851**	**$9,998**	**$4,937**

Cost	**Columns**							
			Unit Cost			Total Cost		
		Length (ft)	Material	Labor	Equip.	Material	Labor	Equip.
05120-260-7200	W12x87	26	$91.00	$2.21	$1.45	$2,366	$57	$38
05120-260	W14x90	13	$102.00	$2.24	$1.47	$1,326	$29	$19
05120-260	W14x99	13	$102.00	$2.24	$1.47	$1,326	$29	$19
05120-260	W14x132	39	$139.75	$2.30	$1.51	$5,450	$90	$59
05120-260	W14x145	39	$154.50	$2.33	$1.53	$6,026	$91	$60
05120-260	W14x159	13	$169.25	$2.36	$1.54	$2,200	$31	$20
05120-260-7450	W14x176	78	$184.00	$2.39	$1.56	$14,352	$186	$122
05120-260	W14x193	39	$198.75	$2.42	$1.58	$7,751	$94	$62
05120-260	W14x211	26	$213.50	$2.45	$1.60	$5,551	$64	$42
05120-260	W14x233	26	$228.25	$2.48	$1.62	$5,935	$64	$42
05120-260	W14x257	91	$243.00	$2.51	$1.64	$22,113	$228	$149
		403				**$74,396**	**$964**	**$631**

Cost	**Braced Frame Members**							
			Unit Cost			Total Cost		
		Length (ft)	Material	Labor	Equip.	Material	Labor	Equip.
05120-640-3700	W18x50	60	$52.50	$3.46	$1.66	$3,150	$208	$100
05120-640-3920	W18x65	30	$68.00	$3.50	$1.68	$2,040	$105	$50
05120-640	W18x71	30	$73.75	$3.50	$1.68	$2,213	$105	$50
05120-640	W18x97	30	$100.50	$3.50	$1.68	$3,015	$105	$50
05120-260	HSS 8x8x3/8	48	$39.64	$3.11	$2.04	$1,903	$149	$98
05120-260	HSS 8x8x5/8	48	$39.64	$3.11	$2.04	$1,903	$149	$98
05120-260	HSS 10x10x1/2	48	$64.06	$2.84	$1.84	$3,075	$136	$88
05120-260	HSS 10x10x5/8	35	$64.06	$2.84	$1.84	$2,242	$99	$64
05120-260	HSS 12x12x1/2	35	$83.06	$2.64	$1.69	$2,907	$92	$59
		364				**$22,447**	**$1,149**	**$659**

Total Cost	Steel		
	Material	Labor	Equipment
Slab on Deck	$41,814	$10,153	$1,881
Metal Deck	$41,468	$10,428	$728
Beams	$160,851	$9,998	$4,937
Columns	$74,396	$964	$631
Braced Members	$22,447	$1,149	$659
	$340,976	$32,692	$8,836
	$382,504		

Steel System Cost Analysis – Spread Footings

Cost: Square Footing E-7 Steel System									
				Unit Cost			Total Cost		
	Item	Quantity	Unit	Material	Labor	Equip.	Material	Labor	Equip.
03310-220-0150	Concrete	12.5	CY	$87.00	$0.00	$0.00	$1,088	$0	$0
03210-600-0550	Reinforcement	0.54	ton	$770.00	$350.00	$0.00	$416	$189	$0
03110-430-5150	Formwork	150	SFCA	$0.59	$2.59	$0.00	$89	$389	$0
03310-700-2600	Placement (direct chute)	12.5	CY	$0.00	$11.55	$0.36	$0	$144	$5
							$1,592	$722	$5
							$2,318		

Cost: Square Footing E-9 Steel System									
				Unit Cost			Total Cost		
	Item	Quantity	Unit	Material	Labor	Equip.	Material	Labor	Equip.
03310-220-0150	Concrete	7.5	CY	$87.00	$0.00	$0.00	$653	$0	$0
03210-600-0550	Reinforcement	0.33	ton	$770.00	$350.00	$0.00	$254	$116	$0
03110-430-5150	Formwork	101.3	SFCA	$0.59	$2.59	$0.00	$60	$262	$0
03310-700-2600	Placement (direct chute)	7.5	CY	$0.00	$11.55	$0.36	$0	$87	$3
							$966	$464	$3
							$1,434		

Appendix H

Schedule Analysis Calculations

Concrete System Schedule Analysis – Typical Floor

	Schedule	**Joists/Slab**					
		Quantity	Unit	Crew	Daily Output	# of Days	
03310-220-0300	Concrete	1100	CY				
03210-600-0400	Reinforcement	64	Ton	12 Rodm	8.7	7.356	
03110-420-3760	Formwork	24250	SF	(5) C-2	2400	10.104	
03310-700-1600	Placement	1100	CY	C-20	180	6.111	
03310-300-0010	Finishing	24250	SF	4 Cefi	3600	6.736	
						30.308	
						31	**days**

	Schedule	**Girders**					
		Quantity	Unit	Crew	Daily Output	# of Days	
03310-220-0300	Concrete	117	CY				
	Reinforcement						
03210-600-0100	#3-#7	11.33	Ton	12 Rodm	4.8	2.360	
03210-600-0150	#8-#18	24.53	Ton	12 Rodm	8.1	3.028	
	Formwork						
03110-405-2650	Interior	3464	SFCA	(5) C-2	1977	1.752	
03110-405-1650	Exterior	4208	SFCA	(5) C-2	1625	2.590	
03110-700-0200	Placement	117	CY	C-20	90	1.300	
						11.030	
						12	**days**

	Schedule	**Columns**				
		Quantity	Unit	Crew	Daily Output	# of Days
03310-220-0400	Concrete					
03210-600-0250	Reinforcement	14.1	Ton	12 Rodm	6.9	2.043
	Formwork					
03110-410-6650	24"x24"	728	SFCA	(5) C-1	1190	0.612
03110-410	30"x30"	1820	SFCA	(5) C-1	1120	1.625
03110-410-7150	36"x36"	1560	SFCA	(5) C-1	1250	1.248
	Placement					
03310-700-0800	24"x24"	13.5	CY	C-20	92	0.147
03310-700	30"x30"	44	CY	C-20	116	0.379
03310-700-1000	36"x36"	42.2	CY	C-20	140	0.301
						6.356
						7 days

	Schedule	**Shearwalls**				
		Quantity	Unit	Crew	Daily Output	# of Days
03310-220-0300	Concrete	49	CY			
03210-600-0700	Reinforcement	0.89	Ton	12 Rodm	9	0.099
03110-455-8060	Formwork	3900	SFCA	(5) C-2	3950	0.987
03310-700-4950	Placement	49	CY	C-20	100	0.490
03350-350-0020	Finishing	3900	SF	4 Cefi	2160	1.806
						3.382
						4 days

	Schedule	**Shoring**				
		Quantity	Unit	Crew	Daily Output	# of Days
03150-600-1500	Reshoring	24,250	SF	5 carp.	7000	3.464
03150-600-3050	Shores (12 mon.)	24,250	SF	5 carp.	7000	3.464
						6.93
						7 days

Final Schedule	Concrete
	# of Days
Joists/Slab	30.31
Girders	11.03
Columns	6.36
Shearwalls	3.38
Shoring/Reshoring	6.93
	58.01
	58 days

Concrete System Schedule Analysis – Spread Footings

Schedule: Square Footing E-7 Concrete System							
	Item	Quantity	Unit	Crew	Daily Output	# of Days	
03310-220-0150	Concrete	17	CY				
03210-600-0550	Reinforcement	0.61	ton	4 Rodm	3.60	0.17	
03110-430-5150	Formwork	179	SFCA	C-1	414.00	0.43	
03310-700-2600	Placement (direct chute)	17	CY	C-6	120.00	0.14	
						0.74	**day(s)**

Schedule: Square Footing E-9 Concrete System							
	Item	Quantity	Unit	Crew	Daily Output	# of Days	
03310-220-0150	Concrete	12.53	CY				
03210-600-0550	Reinforcement	0.53	ton	4 Rodm	3.60	0.15	
03110-430-5150	Formwork	167	SFCA	C-1	414.00	0.40	
03310-700-2600	Placement (direct chute)	12.53	CY	C-6	120.00	0.10	
						0.66	**day(s)**

Steel System Schedule Analysis – Typical Floor

	Schedule	**Slab**				
		Quantity	Unit	Crew	Daily Output	# of Days
03310-220-1010	Concrete	356	CY			
03210-200-0100	WWF	242.5	CSF	2 Rodm	35	6.929
03110-420-6650	Edge Form	199.5	SFCA	C-1	225	0.887
03310-700-1400	Placement	356	CY	C-20	140	2.543
						10.358
						11 days

Schedule	**Metal Deck**				
		Area (SF)	Crew	Daily Output	# of Days
05310-300-5800	3"-20 gauge	24,250	E-4	3000	8.083
					8.083
					9 days

Schedule	**Beams**				
		Length (ft)	Crew	Daily Output	# of Days
05120-640-0300	W8x10	6.5	E-2	600	0.011
05120-640-1100	W12x14	40	E-2	880	0.045
05120-640-2700	W16x26	447	E-2	1000	0.447
05120-640-2900	W16x31	157	E-2	900	0.174
05120-640-3300	W18x35	148	E-5	960	0.154
05120-640-3500	W18x40	122	E-5	960	0.127
05120-640-3520	W18x46	823	E-5	960	0.857
05120-640-3700	W18x50	322	E-5	912	0.353
05120-640-3900	W18x55	92	E-5	912	0.101
05120-640	W18x60	76	E-5	906	0.084
05120-640-3920	W18x65	136	E-5	900	0.151
05120-640-4100	W21x44	330	E-5	1064	0.310
05120-640-4900	W24x55	189	E-5	1110	0.170
05120-640-5100	W24x62	103	E-5	1110	0.093
05120-640-5300	W24x68	37	E-5	1110	0.033
05120-640-5500	W24x76	43	E-5	1110	0.039
05120-640-5800	W27x84	86	E-5	1190	0.072
05120-640-6300	W30x108	43	E-5	1200	0.036
05120-640	W30x124	43	E-5	1160	0.037
		3244			3.296
					4 days

Schedule	Columns	Length (ft)	Crew	Daily Output	# of Days
05120-260-7200	W12x87	78	E-2	984	0.079
05120-260	W14x90	39	E-2	972	0.040
05120-260	W14x99	39	E-2	972	0.040
05120-260	W14x132	117	E-2	948	0.123
05120-260	W14x145	117	E-2	936	0.125
05120-260	W14x159	39	E-2	924	0.042
05120-260-7450	W14x176	234	E-2	912	0.257
05120-260	W14x193	117	E-2	900	0.130
05120-260	W14x211	78	E-2	888	0.088
05120-260	W14x233	78	E-2	876	0.089
05120-260	W14x257	273	E-2	864	0.316
		1209			1.330
					2 days

Schedule	Braced Frame Members	Length (ft)	Crew	Daily Output	# of Days
05120-640-3700	W18x50	60	E-5	912	0.066
05120-640-3920	W18x65	30	E-5	900	0.033
05120-640	W18x71	30	E-5	900	0.033
05120-640	W18x97	30	E-5	900	0.033
05120-260	HSS 8x8x3/8	48	E-2	700	0.069
05120-260	HSS 8x8x5/8	48	E-2	700	0.069
05120-260	HSS 10x10x1/2	48	E-2	768	0.063
05120-260	HSS 10x10x5/8	35	E-2	768	0.046
05120-260	HSS 12x12x1/2	35	E-2	828	0.042
		364			0.453
					1 day

Final Schedule	**Steel**
	# of Days
Slab on Deck	10.36
Metal Deck	8.08
Beams	3.30
Columns	1.33
Braced Members	0.45
	23.52

24 days

Steel System Schedule Analysis – Spread Footings

Schedule: Square Footing E-7 Steel System							
	Item	Quantity	Unit	Crew	Daily Output	# of Days	
03310-220-0150	Concrete	12.5	CY				
03210-600-0550	Reinforcement	0.54	ton	4 Rodm	3.60	0.15	
03110-430-5150	Formwork	150	SFCA	C-1	414.00	0.36	
03310-700-2600	Placement (direct chute)	12.5	CY	C-6	120.00	0.10	
						0.62	**day(s)**

Schedule: Square Footing E-9 Steel System							
	Item	Quantity	Unit	Crew	Daily Output	# of Days	
03310-220-0150	Concrete	7.5	CY				
03210-600-0550	Reinforcement	0.33	ton	4 Rodm	3.60	0.09	
03110-430-5150	Formwork	101.3	SFCA	C-1	414.00	0.24	
03310-700-2600	Placement (direct chute)	7.5	CY	C-6	120.00	0.06	
						0.40	**day(s)**

Appendix I

Design Load Calculations

Wind Loads (Steel and Concrete Systems)

Assumptions

- Assumed fixed at ground level even though there is a 3-level parking garage below grade
- Building shape, in plan and elevation, was assumed rectangular with the dimensions being 222.5' in the North / South direction and 119' in the East / West direction and a height of 180.75', which is the tallest height measurement for the building. See framing plans and elevations for actual building shape and dimensions.

  NOTE: These assumed building shapes and dimensions were used to calculate the pressure profiles along the height of the building for a conservative approach. When the actual forces to each floor were calculated, actual building dimensions and shapes were used.

- The wind load calculation procedures were taken from ASCE 7-02, Chapter 6. Method 2: Analytical Procedure (Sec. 6.5) was used for this building.

Building Information

- N-S direction – Shearwalls/Braced Frames
- E-W direction – Shearwalls/Braced Frames
- Location: Arlington, VA
- Exposure B
- Building Use: Office (Primary), Retail (1^{st} Level), Parking (Below Grade)

Velocity Pressure

○ K_{zt} = 1.0	(Fig. 6-4)	area is flat
○ K_d = 0.85	(Table 6-4)	Building MWFRS
○ V = 90 mph	(Fig. 6-1)	
○ Use Group II	(Table 1-1)	
○ I = 1.0	(Table 6-1)	

From Table 6-3 (Exposure B, Case 2)

z (ft)	Kz
0-15	0.57
20	0.62
25	0.66
30	0.70
40	0.76
50	0.81
60	0.85
70	0.89
80	0.93
90	0.96
100	0.99
120	1.04
140	1.09
160	1.13
180	1.17
200	1.20

$$q_z = 0.00256 K_{zt} K_d V^2 I K_z$$

$$q_z = 0.00256(1.0)(0.85)(90)^2(1.0)K_z$$

$q_z = 17.63 K_z$ PSF

$q_h = 17.63(1.17^*)$ *linear interpolation

$q_h = 20.65$ PSF

External Pressure Coefficients (Fig. 6-6)

Windward Wall: Cp = 0.8
Leeward Wall:
N-S: L/B = 222.5'/119' = 1.87 Cp = -0.326* *linear interpolation
E-W: L/B = 119'/222.5' = 0.53 Cp = -0.5

Internal Pressure Coefficients (6.5.11.1)

GC_{pi} = +0.18
= -0.18
$q_i = q_h$ = 20.65 PSF ($q_i = q_h$ for windward and leeward walls of enclosed buildings)

Internal Pressure = $qiGC_{pi} = \pm 20.65 PSF(0.18) = \pm 3.72 PSF$

Gust Factor (N-S Direction)

N-S Direction: B = 119’, L = 222.5’

Estimate Frequency (C_t = 0.02, x = 0.75 – Table 9.5.5.3.2)

$$f = \frac{1}{C_t h_n^x} = \frac{1}{0.02(180.75)^{0.75}} = 1.01Hz > 1.0 \therefore Rigid$$ (Inverse of Eq. 9.5.5.3.2-1)

G = 0.85 or

Calculate G

From Table 6-2 (Exposure B)

$$\bar{z}_{\min} = 30\,ft$$
$$c = 0.3$$
$$l = 320\,ft$$
$$\bar{\varepsilon} = 1/3$$

$$g_Q = 3.4$$
$$g_V = 3.4$$
(6.5.8.1)

$$\bar{z} = 0.6h = 0.6(180.75) = 108.45' > 30' \therefore \bar{z} = 108.45'$$ (6.5.8.1)

$$L_z = l(\bar{z}/33)^{\bar{\varepsilon}} = 320(108.45/33)^{1/3} = 475.76$$ (Eq. 6-7)

$$I_z = c(33/\bar{z})^{1/6} = 0.3(33/108.45)^{1/6} = 0.246$$ (Eq. 6-5)

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_z}\right)^{0.63}}} = \sqrt{\frac{1}{1+0.63\left(\frac{119+180.75}{475.76}\right)^{0.63}}} = 0.82$$ (Eq. 6-6)

$$G = 0.925\left(\frac{1+1.7g_Q I_z Q}{1+1.7g_V I_z}\right) = 0.925\left(\frac{1+1.7(3.4)(0.246)(0.82)}{1+1.7(3.4)(0.246)}\right) = 0.83$$ (Eq. 6-4)

Since 0.83 < 0.85, use G=0.83

Gust Factor (E-W Direction)

E-W Direction: B = 222.5', L = 119'

Estimate Frequency (C_t = 0.02, x = 0.75 – Table 9.5.5.3.2)

$$f = \frac{1}{C_t h_n^x} = \frac{1}{0.02(180.75)^{0.75}} = 1.01 Hz > 1.0 \therefore Rigid$$ (Inverse of Eq. 9.5.5.3.2-1)

G = 0.85 or

Calculate G

From Table 6-2 (Exposure B)

$$\bar{z}_{\min} = 30\,ft$$
$$c = 0.3$$
$$l = 320\,ft$$
$$\bar{\varepsilon} = 1/3$$

$$g_Q = 3.4$$
$$g_V = 3.4$$
(6.5.8.1)

$$\bar{z} = 0.6h = 0.6(180.75) = 108.45' > 30' \therefore \bar{z} = 108.45'$$ (6.5.8.1)

$$L_z = l(\bar{z}/33)^{\bar{\varepsilon}} = 320(108.45/33)^{1/3} = 475.76$$ (Eq. 6-7)

$$I_z = c(33/\bar{z})^{1/6} = 0.3(33/108.45)^{1/6} = 0.246$$ (Eq. 6-5)

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_z}\right)^{0.63}}} = \sqrt{\frac{1}{1 + 0.63\left(\frac{222.5 + 180.75}{475.76}\right)^{0.63}}} = 0.799$$ (Eq. 6-6)

$$G = 0.925\left(\frac{1 + 1.7 g_Q I_z Q}{1 + 1.7 g_V I_z}\right) = 0.925\left(\frac{1 + 1.7(3.4)(0.246)(0.799)}{1 + 1.7(3.4)(0.246)}\right) = 0.82$$ (Eq. 6-4)

Since 0.82 < 0.85, use G=0.82

N-S Windward Pressure

$$P_{wz} = q_z C_p G = q_z 0.8(0.83) = 0.664 q_z \text{ PSF}$$

N-S Leeward Pressure

$$P_{lh} = q_h C_p G = 20.65(-0.326)(0.83) = -5.59 \text{ PSF}$$

E-W Windward Pressure

$$P_{wz} = q_z C_p G = q_z 0.8(0.82) = 0.656 q_z \text{ PSF}$$

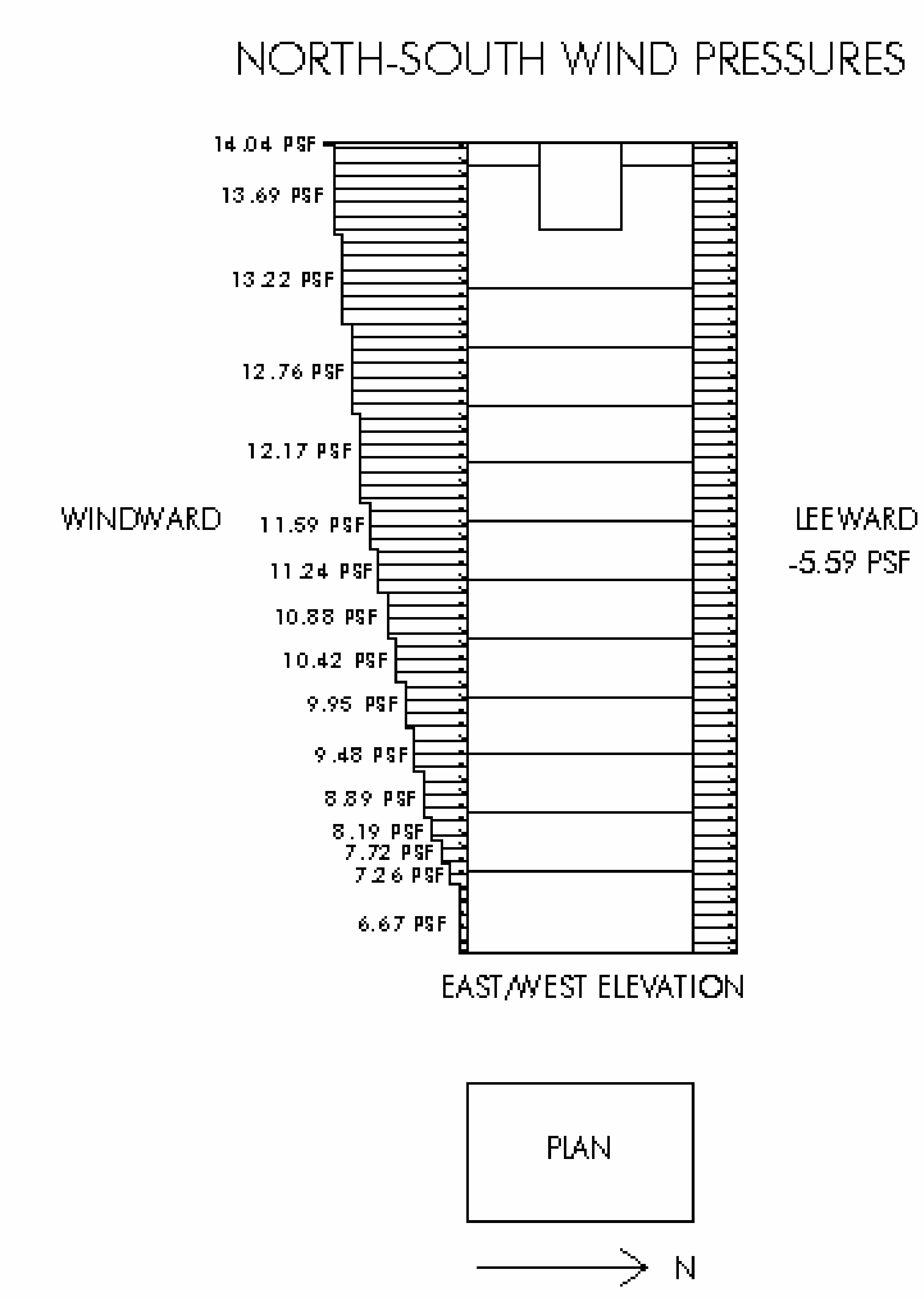
E-W Leeward Pressure

$$P_{lh} = q_h C_p G = 20.65(-0.5)(0.82) = -8.47 \text{ PSF}$$

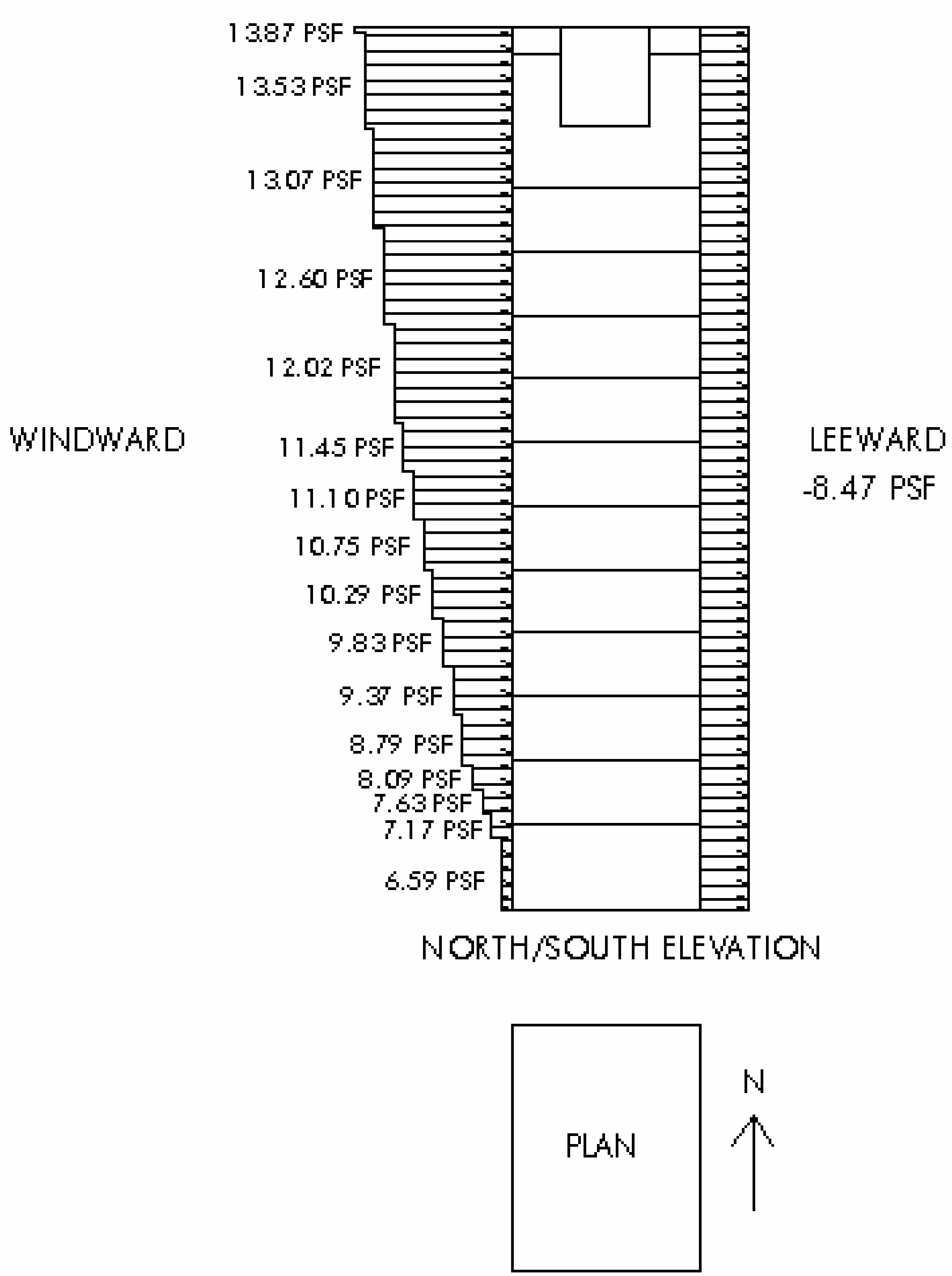
Total Pressures

z	Kz	qz	N-S Windward Pressure (PSF)	E-W Windward Pressure (PSF)	N-S Leeward Pressure (PSF)	E-W Leeward Pressure (PSF)	P_{total} (N-S) (PSF)	P_{total} (E-W) (PSF)
0-15	0.57	10.05	6.67	6.59	-5.59	-8.47	12.26	15.06
20	0.62	10.93	7.26	7.17	-5.59	-8.47	12.85	15.64
25	0.66	11.63	7.72	7.63	-5.59	-8.47	13.31	16.10
30	0.70	12.34	8.19	8.09	-5.59	-8.47	13.78	16.56
40	0.76	13.40	8.89	8.79	-5.59	-8.47	14.48	17.26
50	0.81	14.28	9.48	9.37	-5.59	-8.47	15.07	17.84
60	0.85	14.98	9.95	9.83	-5.59	-8.47	15.54	18.30
70	0.89	15.69	10.42	10.29	-5.59	-8.47	16.01	18.76
80	0.93	16.39	10.88	10.75	-5.59	-8.47	16.47	19.22
90	0.96	16.92	11.24	11.10	-5.59	-8.47	16.83	19.57
100	0.99	17.45	11.59	11.45	-5.59	-8.47	17.18	19.92
120	1.04	18.33	12.17	12.02	-5.59	-8.47	17.76	20.49
140	1.09	19.21	12.76	12.60	-5.59	-8.47	18.35	21.07
160	1.13	19.92	13.22	13.07	-5.59	-8.47	18.81	21.54
180	1.17	20.62	13.69	13.53	-5.59	-8.47	19.28	22.00
200	1.20	21.15	14.04	13.87	-5.59	-8.47	19.63	22.34

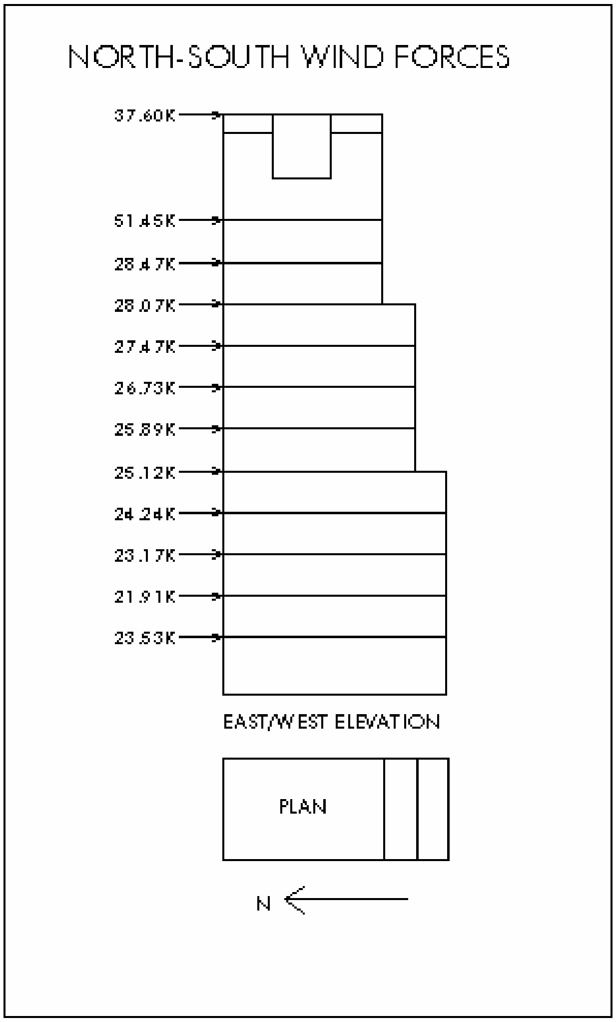
Wind Pressure Diagrams

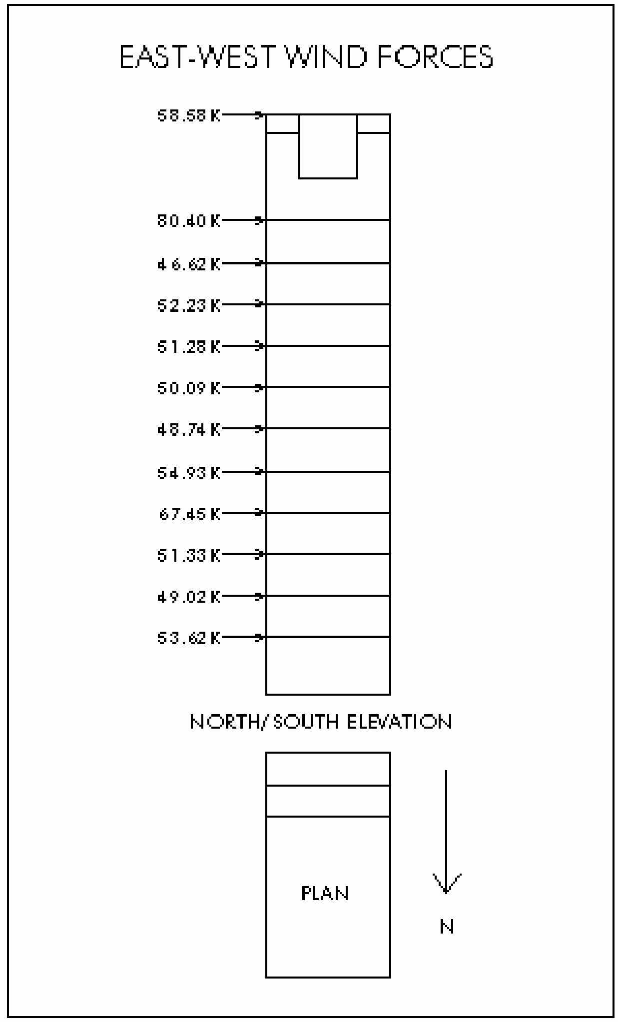


EAST-WEST WIND PRESSURES



Wind Force Diagrams





Seismic Loads (Concrete System)

Assumptions

- ASCE 7-02, Chapter 9 was used to calculate the seismic loads for this building.

Building Information

- N-S Direction: Reinforced Concrete Shearwalls
- E-W Direction: Reinforced Concrete Shearwalls
- Location: Arlington, VA
- Building Use: Office (Primary), Retail (1st Level), Parking (Below Grade)

Seismic Design Category

Occupancy Category - II	(Table 1-1)
Seismic Use Group: 1	(Table 9.1.3)
Site Class C:	(Structural Notes)
Acceleration from Maps:	
$S_s = 0.190$	(Fig. 9.4.1.1a)
$S_1 = 0.070$	(Fig. 9.4.1.1b)
Adjust for Site Class:	
$F_a = 1.2$	(Table 9.4.1.2.4a)
$F_V = 1.7$	(Table 9.4.1.2.4b)
$S_{ms} = F_aS_s = 1.2(0.19) = 0.228$	(Eq. 9.4.1.2.4-1)
$S_{m1} = F_vS_1 = 1.7(0.07) = 0.119$	(Eq. 9.4.1.2.4-2)

Design Spectral Response Acceleration Parameters

$S_{DS} = 2/3\ S_{ms} = 2/3(0.228) = 0.152$	(Eq. 9.4.1.2.5-1)
$S_{D1} = 2/3\ S_{m1} = 2/3(0.119) = 0.0793$	(Eq. 9.4.1.2.5-2)

Seismic Design Category

(Table 9.4.2.1a)
S.D.C. based on short period response acceleration = S.D.C.-A

(Table 9.4.2.1b)
S.D.C. based on 1-sec. period response acceleration = **S.D.C.-B*   worst case**

NOTE: Building does not meet any plan or vertical irregularities as specified in Tables 1616.5.1.1 or 1616.5.1.2 of the IBC 2000, therefore it is still S.D.C.-B.
Equivalent Lateral Force Procedure can be used.

Seismic Base Shear (V=C$_S$W)

R = 4 (Table 9.5.2.2) Reinforced Concrete Shearwalls
I = 1.0 (Table 9.1.4)

$T = C_t h_n^x$ (Eq. 9.5.5.3.2-1)

N-S: $T = C_t h_n^x = 0.016(180.75)^{0.9} = 1.72\sec$ (Table 9.5.5.3.2)

E-W: $T = C_t h_n^x = 0.016(180.75)^{0.9} = 1.72\sec$ (Table 9.5.5.3.2)

$$C_s = \frac{S_{DS}}{R/I} = \frac{0.152}{4/1} = 0.038$$

$$C_{S,\max}(N-S) = \frac{S_{D1}}{T\left(\frac{R}{I}\right)} = \frac{0.0793}{1.72\left(\frac{4}{1}\right)} = 0.011526$$ *Controls

$$C_{S,\max}(E-W) = \frac{S_{D1}}{T\left(\frac{R}{I}\right)} = \frac{0.0793}{1.72\left(\frac{4}{1}\right)} = 0.011526$$ *Controls

$$C_{S,\min} = 0.044 I S_{DS} = 0.044(1.0)(0.152) = 0.006688 < 0.011526 \therefore OK$$

Dead Loads

Roof Dead Load

Joists	9 PSF
Metal Deck	3 PSF
Insulation	1.5 PSF
SDL	15 PSF
Built-up Roof (5-ply felt and gravel)	6.5 PSF
	35 PSF

Snow Load 30 PSF (See Snow Load Calculations)

Typical Floor Load

Joists	119 PSF
Misc. DL	15 PSF
mech. ducts, plumbing, sprinklers, ceiling, etc.	134 PSF

Exterior Wall Loads

Glass Curtain Wall (N façade)	15 PSF
Precast/Windows (S,E,W facades)	20 PSF

$$w_{roof} = 834.2k$$
$$w_{11} = 2{,}500.5k$$
$$w_{10} = 2{,}375.7k$$
$$w_{9-6} = 2{,}854k$$
$$w_{5-2} = 3{,}332.4k$$
$$w_1 = 3364k$$

$$W = w_{roof} + w_{11} + w_{10} + 4w_{9-6} + 4w_{5-2} + w_1$$
$$W = 834.2k + 2{,}500.5k + 2{,}375.7k + 4(2{,}854k) + 4(3332.4k) + 3{,}364k$$
$$W = 33{,}820k$$

$V_{N\text{-}S}$ = 0.011526(33,820 k) = 390 k
$V_{E\text{-}W}$ = 0.011526(33,820 k) = 390 k

$$F_x = C_{vx}V$$
$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$

$$k(N-S) = 1 + \frac{1.72 - 0.5}{2} = 1.61^{*}$$ *linear interpolation

$$k(E-W) = 1 + \frac{1.72 - 0.5}{2} = 1.61^{*}$$ *linear interpolation

Seismic Base Shear and Overturning Moment

Level	w_x	h_x	$w_x h_x^{1.61}$	$w_x h_x^{1.61}$	C_{vx} (N-S)	C_{vx} (E-W)	F_x (N-S)	F_x (E-W)
Roof	834.1	180.75	3590163	3590163	0.079	0.079	30.61	30.61
12	2501	148	7802523	7802523	0.171	0.171	66.53	66.53
11	2376	135	6392692	6392692	0.140	0.140	54.51	54.51
10	2854	122	6523690	6523690	0.143	0.143	55.63	55.63
9	2854	109	5441400	5441400	0.119	0.119	46.40	46.40
8	2854	96	4435174	4435174	0.097	0.097	37.82	37.82
7	2854	83	3508890	3508890	0.077	0.077	29.92	29.92
6	3332	70	3113959	3113959	0.068	0.068	26.55	26.55
5	3332	57	2236987	2236987	0.049	0.049	19.08	19.08
4	3332	44	1474565	1474565	0.032	0.032	12.57	12.57
3	3332	31	839068	839068	0.018	0.018	7.15	7.15
2	3364	18	353057	353057	0.008	0.008	3.01	3.01
	33819.1		45712167	45712167	1.000	1.000	390	390

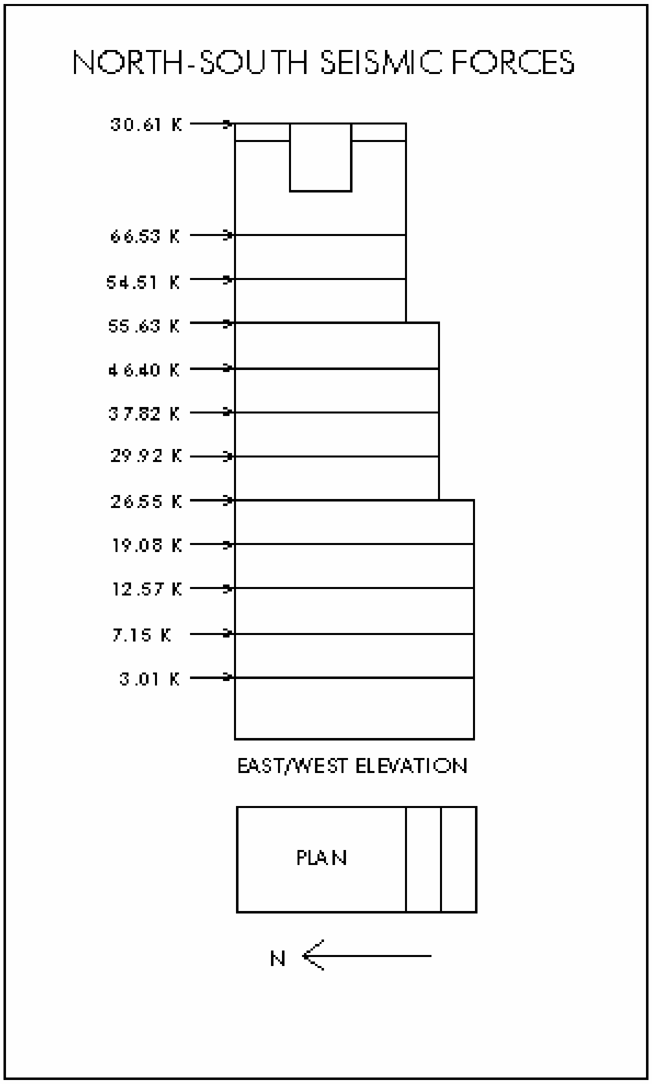
k (N-S)	1.61	
k (E-W)	1.61	
V (N-S)	390	k
V (E-W)	390	k

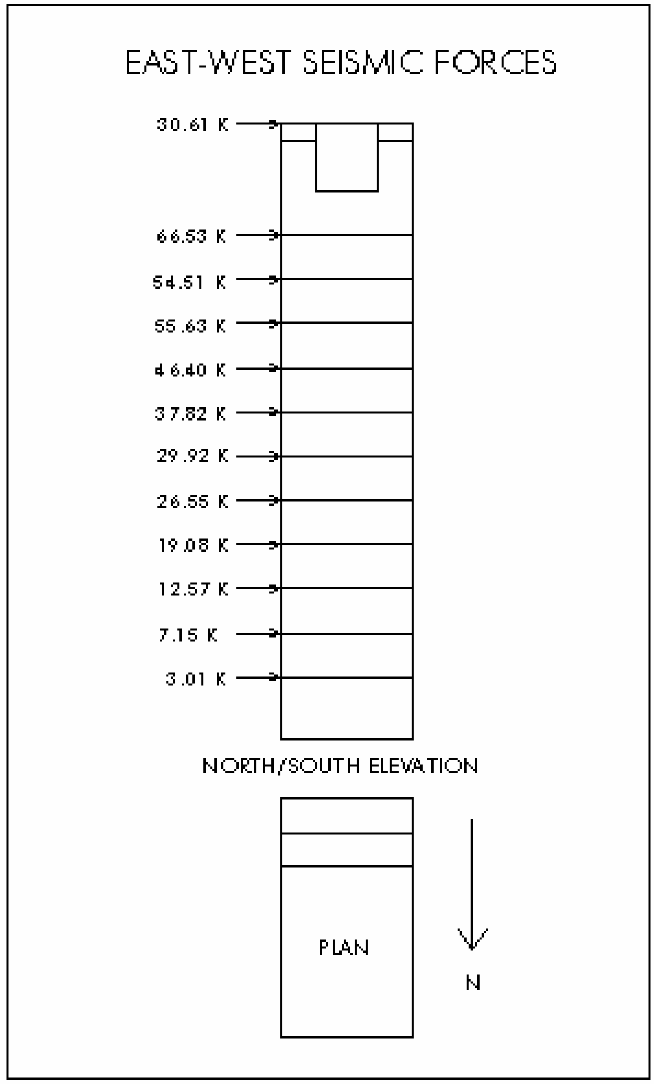
C_s (N-S)	0.011526
C_s (E-W)	0.011526

Base Shear		
N-S	390	k
E-W	390	k

Overturning Moment		
Overturning Moment (N-S)	44473.5034	ft-k
Overturning Moment (E-W)	44473.5034	ft-k

Seismic Force Diagrams





Load Cases and Controlling Lateral Forces (Concrete System)

Load Combinations Involving Wind Loads (W) and Seismic Loads (E)

ASCE 7-02 (Sec. 2.3.2)

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

Check 1.6W vs. 1.0E

Red = Controlling E-W Lateral Force, Blue = Controlling N-S Lateral Force

	1.6W (N-S)	1.6W (E-W)	1.0E (N-S/E-W)
Roof	**60.16**	**93.72**	30.61
12	**82.32**	**128.64**	66.53
11	45.55	**74.59**	**54.51**
10	44.91	**83.57**	**55.63**
9	43.95	**82.05**	**46.40**
8	**42.77**	**80.14**	37.82
7	**41.42**	**77.98**	29.92
6	**40.19**	**87.89**	26.55
5	**38.78**	**107.92**	19.08
4	**37.07**	**82.13**	12.57
3	**35.06**	**78.43**	7.15
2	**37.64**	**85.79**	3.01

Seismic Loads (Steel System)

Assumptions

- ASCE 7-02, Chapter 9 was used to calculate the seismic loads for this building.

Building Information

- N-S Direction: Steel Braced Frames
- E-W Direction: Steel Braced Frames
- Location: Arlington, VA
- Building Use: Office (Primary), Retail (1st Level), Parking (Below Grade)

Seismic Design Category

Occupancy Category - II	(Table 1-1)
Seismic Use Group: 1	(Table 9.1.3)
Site Class C:	(Structural Notes)
Acceleration from Maps:	
$S_s = 0.190$	(Fig. 9.4.1.1a)
$S_1 = 0.070$	(Fig. 9.4.1.1b)
Adjust for Site Class:	
$F_a = 1.2$	(Table 9.4.1.2.4a)
$F_V = 1.7$	(Table 9.4.1.2.4b)
$S_{ms} = F_aS_s = 1.2(0.19) = 0.228$	(Eq. 9.4.1.2.4-1)
$S_{m1} = F_vS_1 = 1.7(0.07) = 0.119$	(Eq. 9.4.1.2.4-2)

Design Spectral Response Acceleration Parameters

$S_{DS} = 2/3\ S_{ms} = 2/3(0.228) = 0.152$	(Eq. 9.4.1.2.5-1)
$S_{D1} = 2/3\ S_{m1} = 2/3(0.119) = 0.0793$	(Eq. 9.4.1.2.5-2)

Seismic Design Category

(Table 9.4.2.1a)
S.D.C. based on short period response acceleration = S.D.C.-A

(Table 9.4.2.1b)
S.D.C. based on 1-sec. period response acceleration = **S.D.C.-B*   worst case**

NOTE: Building does not meet any plan or vertical irregularities as specified in Tables 1616.5.1.1 or 1616.5.1.2 of the IBC 2000, therefore it is still S.D.C.-B.
Equivalent Lateral Force Procedure can be used.

Seismic Base Shear ($V=C_SW$)

R = 3 (Table 9.5.2.2) Structural steel systems not specifically detailed for seismic resistance.

I = 1.0 (Table 9.1.4)

$T = C_t h_n^x$ (Eq. 9.5.5.3.2-1)

N-S: $T = C_t h_n^x = 0.02(180.75)^{0.75} = 0.986$ (Table 9.5.5.3.2)

E-W: $T = C_t h_n^x = 0.02(180.75)^{0.75} = 0.986$ (Table 9.5.5.3.2)

$$C_s = \frac{S_{DS}}{R/I} = \frac{0.152}{3/1} = 0.050667$$

$$C_{S,\max}(N-S) = \frac{S_{D1}}{T\left(\frac{R}{I}\right)} = \frac{0.0793}{0.986\left(\frac{3}{1}\right)} = 0.02681$$ *Controls

$$C_{S,\max}(E-W) = \frac{S_{D1}}{T\left(\frac{R}{I}\right)} = \frac{0.0793}{0.986\left(\frac{3}{1}\right)} = 0.02681$$ *Controls

$$C_{S,\min} = 0.044IS_{DS} = 0.044(1.0)(0.152) = 0.006688 < 0.02681 \therefore OK$$

Dead Loads

Roof Dead Load

Metal Deck	5 PSF
Insulation	3 PSF
Misc. DL	10 PSF
Roofing	20 PSF
	38 PSF

Snow Load 30 PSF (See Snow Load Calculations)

NOTE: Since Snow Load is not greater than 30 PSF, 20% of the Snow Load does not need to be considered in the weight calculations.

Typical Floor Load

3 ¼" lt. wt. slab on 3" metal deck	46 PSF
Ponding of Concrete	10 PSF
Misc. DL	15 PSF
mech. ducts, plumbing, sprinklers, ceiling, etc.	71 PSF

Exterior Wall Loads

Glass Curtain Wall (N façade)	15 PSF
Precast/Windows (S,E,W facades)	20 PSF

$w_{roof} = 909k$

$w_{11} = 1617k$

$w_{10} = 1512k$

$w_{9-6} = 1781k$

$w_{5-2} = 2050k$

$w_{1} = 2083k$

$$W = w_{roof} + w_{11} + w_{10} + 4w_{9-6} + 4w_{5-2} + w_{1}$$
$$W = 909k + 1617k = 1512k + 4(1781k) + 4(2050k) + 2083k$$
$$W = 21{,}445k$$

V_{N-S} = 0.02681(21,445k) = 574.94k
V_{E-W} = 0.02681(21,445k) = 574.94k

$$F_x = C_{vx}V$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$

$$k(N-S) = 1 + \frac{0.986 - 0.5}{2} = 1.243^*$$ *linear interpolation

$$k(E-W) = 1 + \frac{0.986 - 0.5}{2} = 1.243^*$$ *linear interpolation

Seismic Base Shear and Overturning Moment

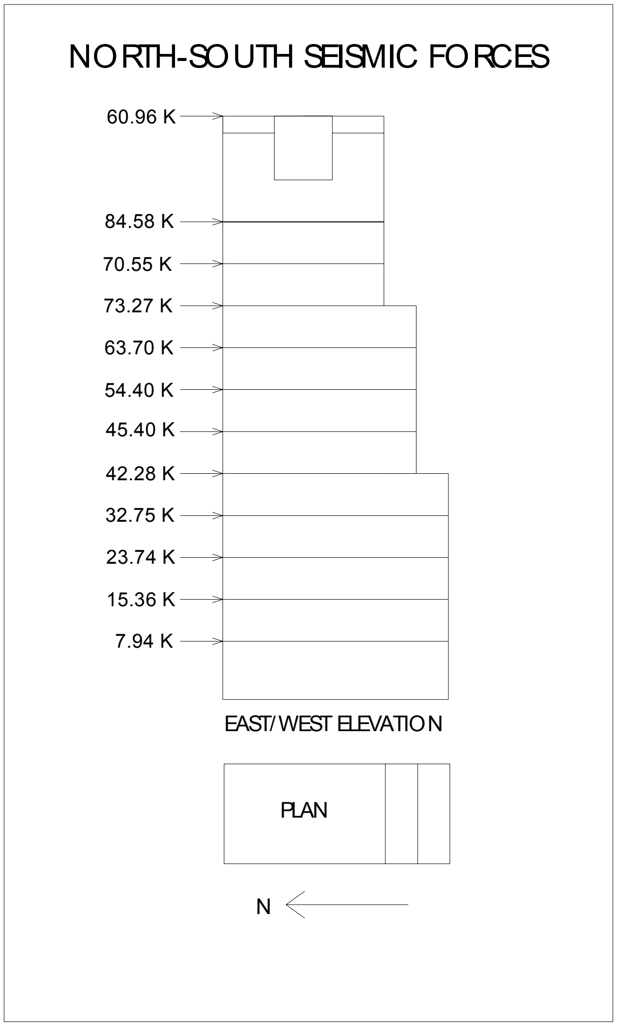
Level	w_x	h_x	$w_x h_x^{1.243}$	$w_x h_x^{1.243}$	C_{vx} (N-S)	C_{vx} (E-W)	F_x (N-S)	F_x (E-W)
12 (roof)	909	180.75	580915	580915	0.106	0.106	60.96	60.96
11	1617	148	806019	806019	0.147	0.147	84.58	84.58
10	1512	135	672290	672290	0.123	0.123	70.55	70.55
9	1781	122	698247	698247	0.127	0.127	73.27	73.27
8	1781	109	606995	606995	0.111	0.111	63.70	63.70
7	1781	96	518355	518355	0.095	0.095	54.40	54.40
6	1781	83	432592	432592	0.079	0.079	45.40	45.40
5	2050	70	402912	402912	0.074	0.074	42.28	42.28
4	2050	57	312109	312109	0.057	0.057	32.75	32.75
3	2050	44	226238	226238	0.041	0.041	23.74	23.74
2	2050	31	146392	146392	0.027	0.027	15.36	15.36
1	2083	18	75682	75682	0.014	0.014	7.94	7.94
			5478745	5478745	1.000	1.000	574.94	574.94

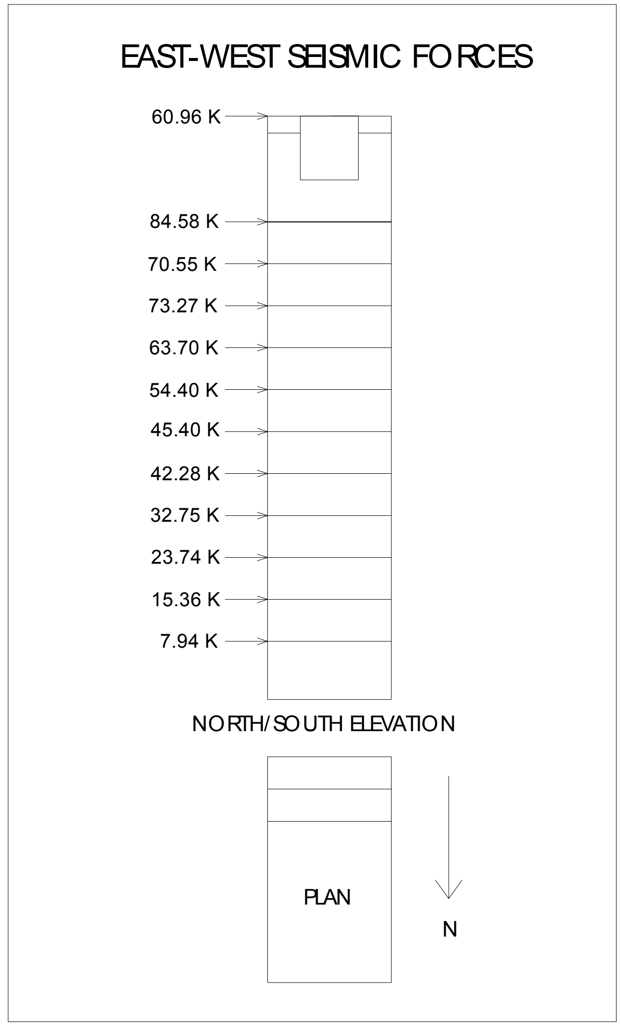
k (N-S)	1.243	
k (E-W)	1.243	
V (N-S)	574.94	k
V (E-W)	574.94	k

Base Shear		
N-S	574.94	k
E-W	574.94	k

Overturning Moment		
Overturning Moment (N-S)	64424.2942	ft-k
Overturning Moment (E-W)	64424.2942	ft-k

Seismic Force Diagrams





Load Cases and Controlling Lateral Forces (Steel System)

Load Combinations Involving Wind Loads (W) and Seismic Loads (E)

ASCE 7-02 (Sec. 2.3.2)

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

Check 1.6W vs. 1.0E

Red = Controlling E-W Lateral Force, Blue = Controlling N-S Lateral Force

	1.6W (N-S)	1.6W (E-W)	1.0E (N-S/E-W)
Roof	60.16	**93.72**	**60.96**
12	82.32	**128.64**	**84.58**
11	45.55	**74.59**	**70.55**
10	44.91	**83.57**	**73.27**
9	43.95	**82.05**	**63.70**
8	42.77	**80.14**	**54.40**
7	41.42	**77.98**	**45.40**
6	40.19	**87.89**	**42.28**
5	**38.78**	**107.92**	32.75
4	**37.07**	**82.13**	23.74
3	**35.06**	**78.43**	15.36
2	**37.64**	**85.79**	7.94

Snow Load (Steel and Concrete Systems)

Assumptions

- ASCE 7-02, Chapter 7 was used to calculate the snow loads for this building.

Building Information

- Location: Arlington, VA
- Max. Roof Slope = 4.55% or 2.62º*

*Since the maximum roof slope is less than 5º, then ASCE 7-02, Chapter 7, Section 7-3 can be used.

$p_f = 0.7 C_e C_t I p_g$ (Eq. 7-1)

$C_e = 0.9$	Surface roughness B (6.5.6.2) Fully exposed (Table 7-2)
$C_t = 1.0$	(Table 7-3)
$I = 1.0$	Category II (Table 7-4)
$p_g = 25$ PSF	(Fig. 7-1)

$p_f = 0.7(0.9)(1.0)(1.0)(25PSF) = 15.75PSF$

$p_{f,\min} = 20PSF \cdot I$ $p_g > 20$ PSF (Sec. 7-3)

$p_{f,\min} = 20PSF \cdot 1 = 20PSF$ 20 PSF > 15.75 PSF, therefore use 20 PSF

NOTE: Structural Notes specify a snow load value of 30 PSF.

Roof Live Load (Steel and Concrete Systems)

Assumptions

- ASCE 7-02, Chapter 4 was used to check the minimum roof live load.

$L_r = 20R_1R_2$ (Eq. 4-2)

R2 = 1 F<4 (4.9.1) Max. roof slope = 2.61º

R1= 0.6 A_t > 600 SF (4.9.1) At(roof col.) = 30'[(46'+30')/2]
At(roof col.) = 1140 SF > 600 SF

$L_r = 20(1)(0.6)$
$L_r = 12$ PSF < 30 PSF*

***Therefore snow load controls with a roof live load of 30 PSF.**