The Regent

950 N. Glebe Road
Arlington, VA

Architect: Cooper Carry Architects

Proposal

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Option: Structural
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Executive Summary

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. The Regent is currently under construction, and since the building was not pre-leased before construction began, the occupants or tenants are not known at this point.

Based off of the study, research, analysis, and designs of the existing system (steel framing with composite slab) and the four alternative systems (hollowcore planks with steel framing, precast double tees with precast framing, one-way wide module joists with CIP framing, and a two-way flat slab with drop panels and CIP framing), it was determined that the existing system is the most efficient design to meet the needs of the building, the project team, the schedule, and the site.

Having studied the existing steel structure all semester, I wanted to challenge myself next semester by proposing to do a redesign 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 want to do a redesign of The Regent using a concrete system in order to make comparisons between the two systems.

A concrete system design shall be selected that meets as many of the initial design team's 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
- Constructibility
- Labor
- Floor to floor height
- Floor to ceiling height
- Lateral system performance (braced frames vs. shearwalls)
- Weight
- Impact on the foundations

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

The existing structure utilizes a series of 5 braced frames; 2 spanning in the north / south direction and 3 spanning in the east / west direction. Since the redesign will be an all concrete system, a series of concrete shearwalls will be used as the lateral force resisting system. These shearwalls will ideally be placed around the elevator core and/or around the stairwells. Both the elevators and the stairwells are located in the central core of the building.

The loads considered for the existing design of The Regent were research, analyzed and checked throughout all of the Technical Reports. In some cases, the loads determined corresponded to the loads used in the existing design, in other cases they did not. In reviewing the loads considered for the existing design, some of the loads seemed to be very conservative such as the floor live load and the snow load, 100 PSF and 30 PSF, respectively. These conservative loadings may have been minimum requirements set forth by the structural engineer on this project. In the concrete redesign of The Regent, the loads considered will be optimized and will based off of IBC 2000, which was the model code used in the existing design. Although a direct comparison cannot be completed between the existing design and the redesign, the optimized loads will yield a more efficient design for the new concrete design.

The design of the concrete structure will be based off of ACI 318-05: *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: ADOSS, SAP, and PCACOL. Analysis of lateral loads will be completed by hand calculations and/or through the structural analysis software SAP2000. Trial sizes based off of the preliminary designs, determined through the CRSI Handbook and hand calculations, will be inputted into the computer programs along with the newly determined, optimized gravity and lateral loads. Live loading patterns will be considered and used to properly design the concrete gravity system.

Scope of Structure to be Designed

- Floor System - One-way Joists, Wide Module
- Cast-In-Place Beams
- Cast-In-Place Columns
- Lateral Load Resisting Shearwalls
- Foundations (representative redesign)

As part of the breadth analysis requirements, the following breadth areas have chosen to be studied in order to help compare the two systems.

- Construction Management
    - Cost
    - Schedule

- Mechanical
    - Impact on mechanical layout
    - Possible redesign of mechanical layout of necessary
- Fire Protection
    - Comparison in fire rating between the existing steel floor system and the new concrete floor system
- Acoustics
    - Comparison between the resistance to noise penetrations between the existing steel floor system and the new concrete floor system.

A schedule has been prepared describing what tasks will be completed and when throughout the semester.

Background

Building Overview

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. The Regent is currently under construction, and since the building was not pre-leased before construction began, the occupants or tenants are not known at this point.

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 1st floor.

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 6th and 10th levels.

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

Since The Regent is built to its maximum height allowance, its penthouse is sunken into the 12th story and as a result the 12th 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 1st level is 18' and the floor to floor height in parking garage is 10'.

Existing 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.

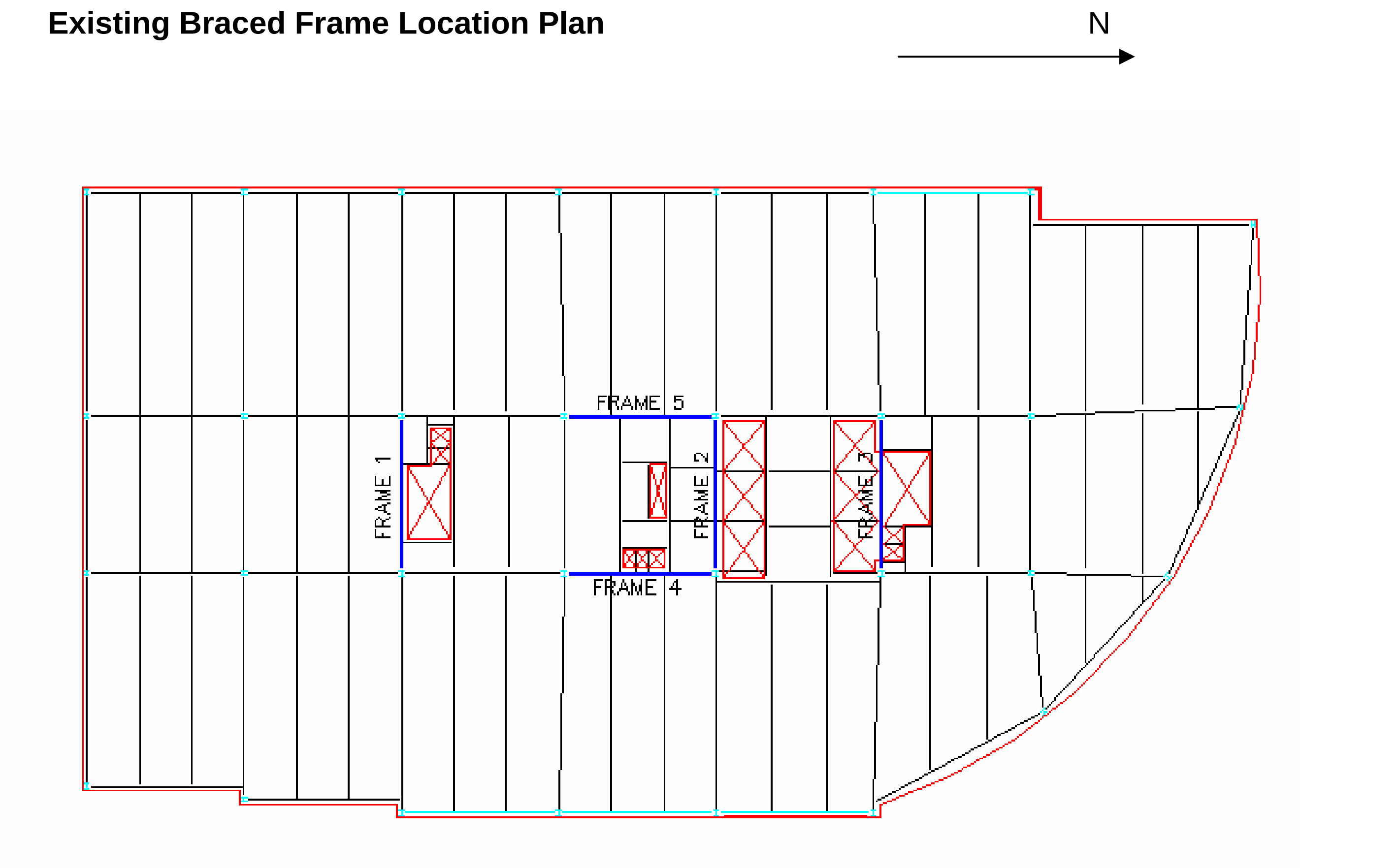
There is an elevator core running up the center of the building and through the center of each floor. The roof deck construction is 3” x 22 gage, deep rib, type N, painted roof deck.

Existing 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.

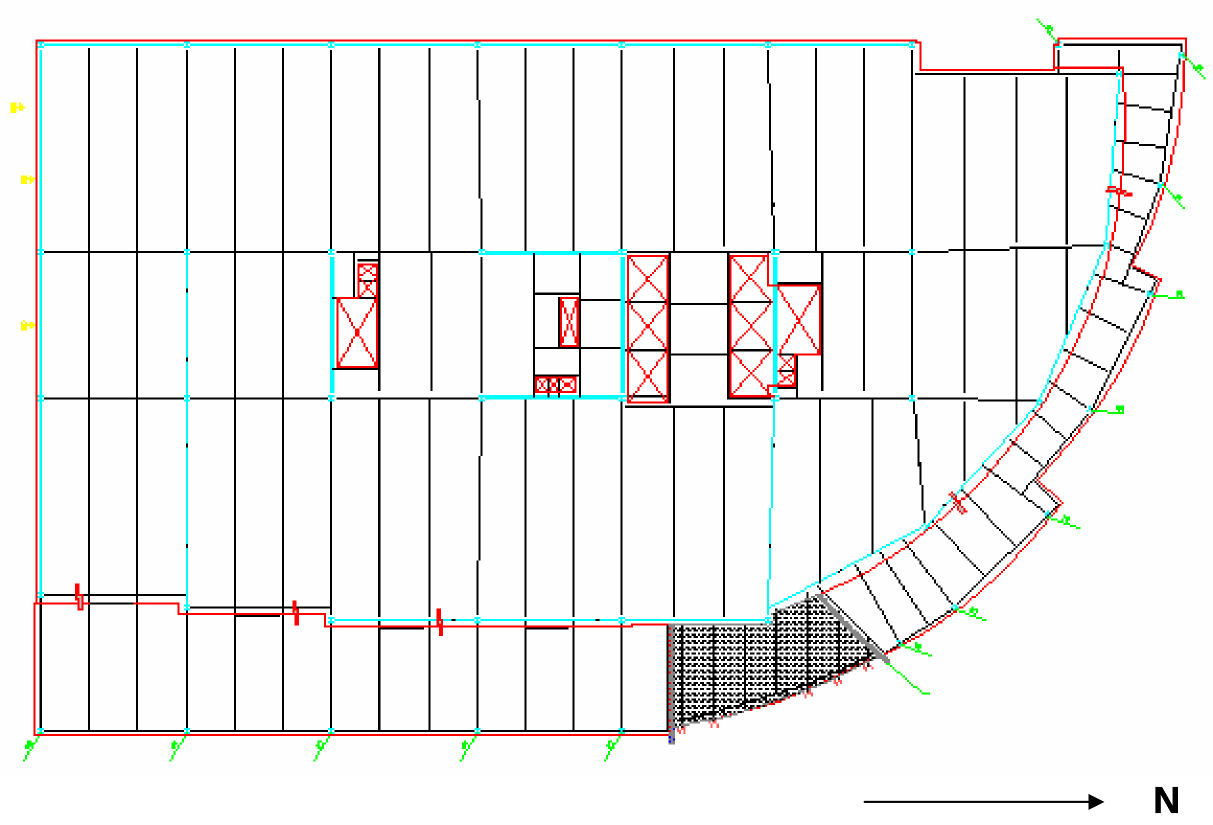
The typical diagonal steel members used in the braced frames are HSS 8” x 8”’s, 10” x 10”’s, and 12” x 12”’s with thicknesses ranging from 3/8” to 5/8”. The columns in the braced frames are all 14” wide flange members ranging in size from W14 x 233’s and W14 x 257’s near the base to W14 x 53’s to W14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

Existing Braced Frame Location Plan

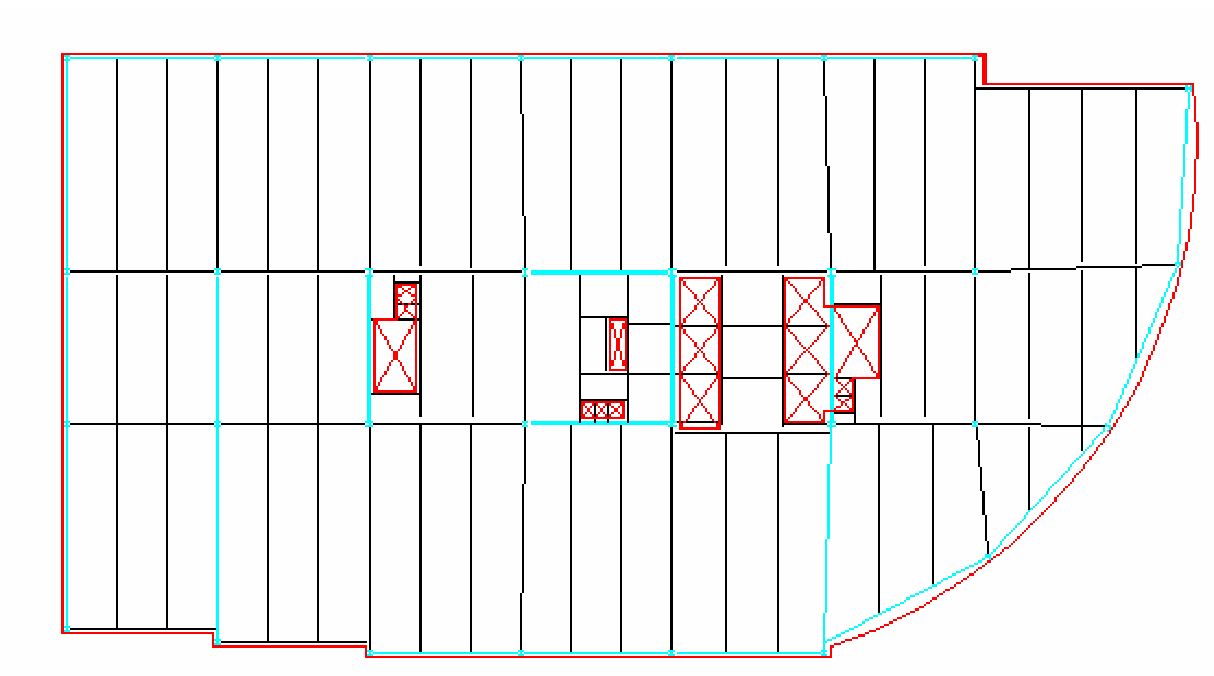


Existing Typical Framing Plans and Elevations

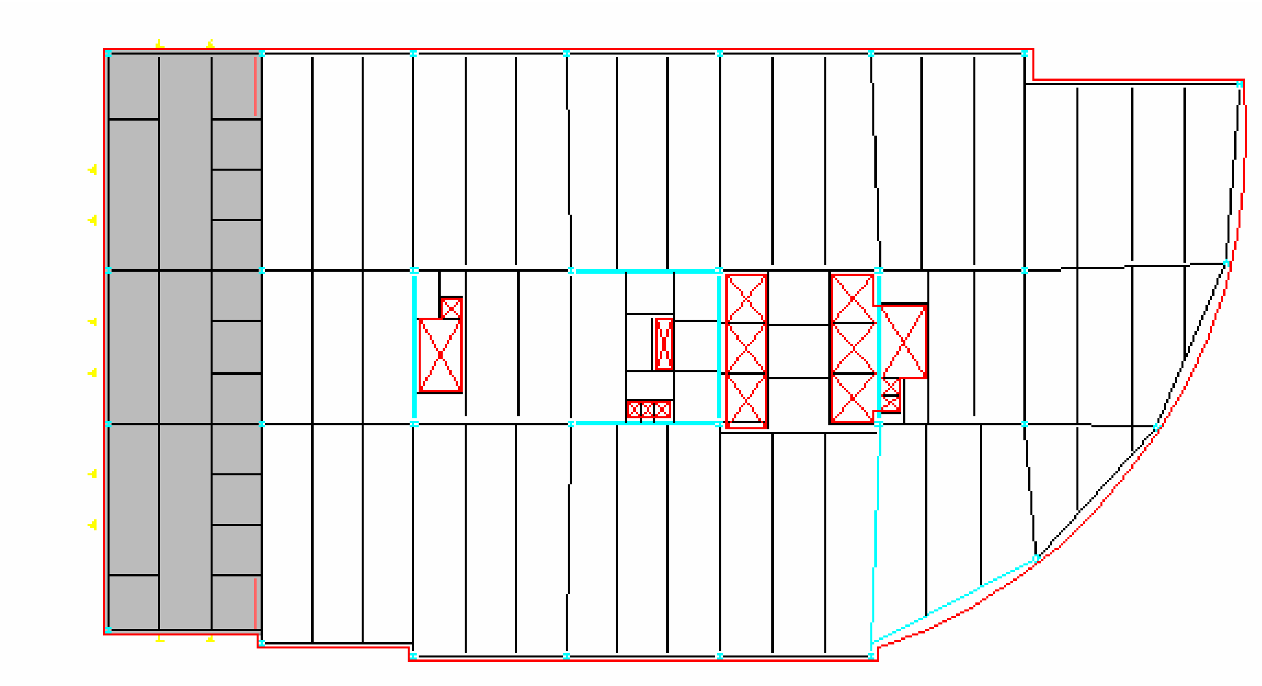
2nd Floor Faming Plan



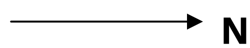
3rd – 5th Floor Framing Plan



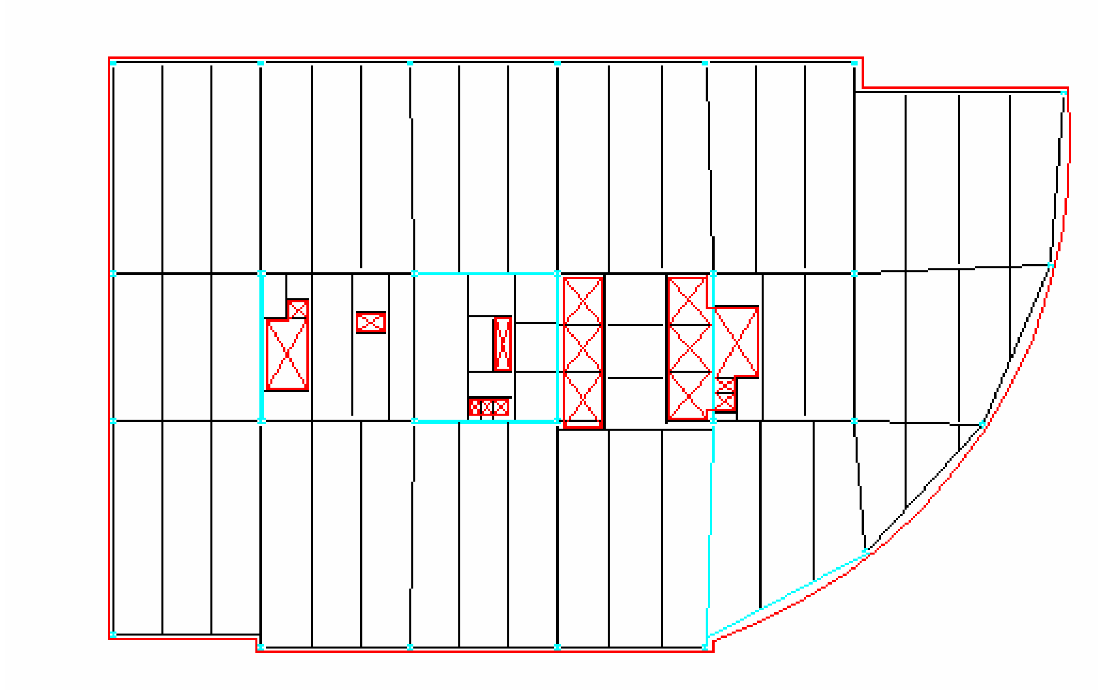
6th Floor Framing Plan



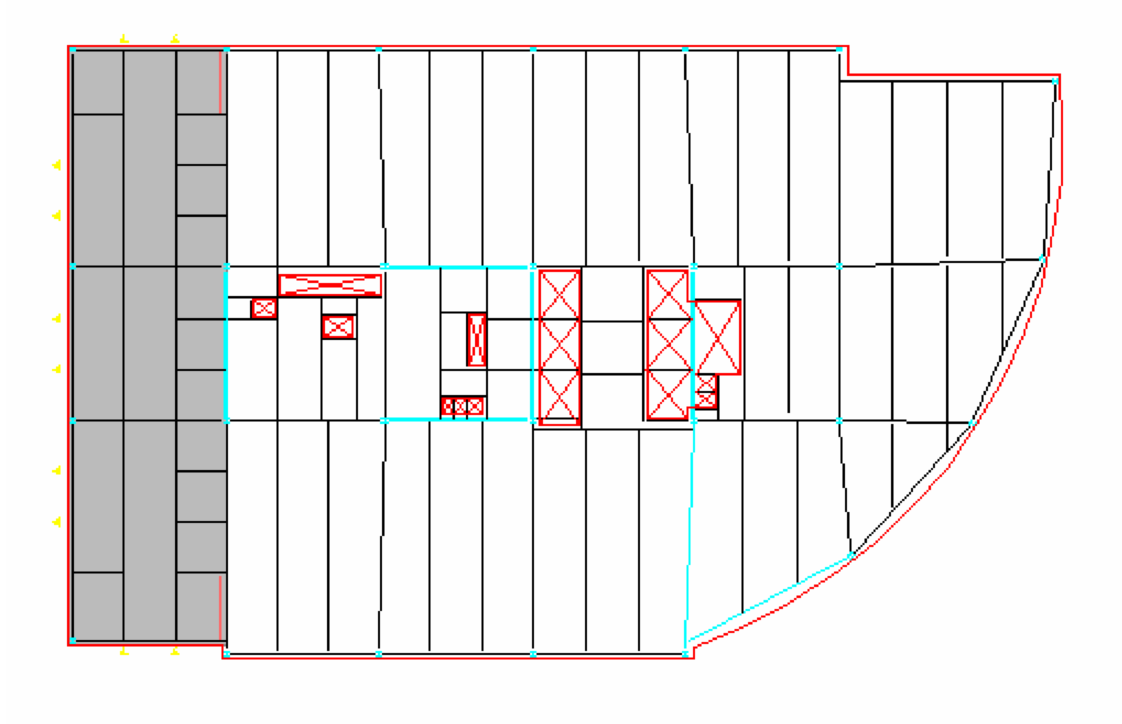
Note: Shaded area is roof construction



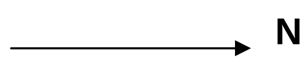
7-9th Floor Framing Plan



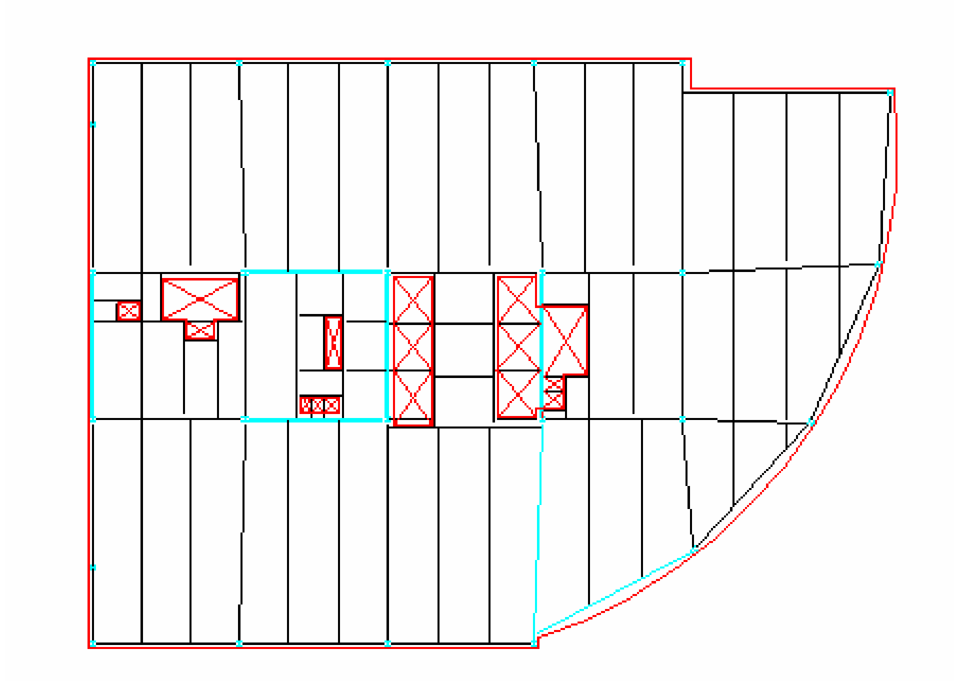
10[th] Floor Framing Plan



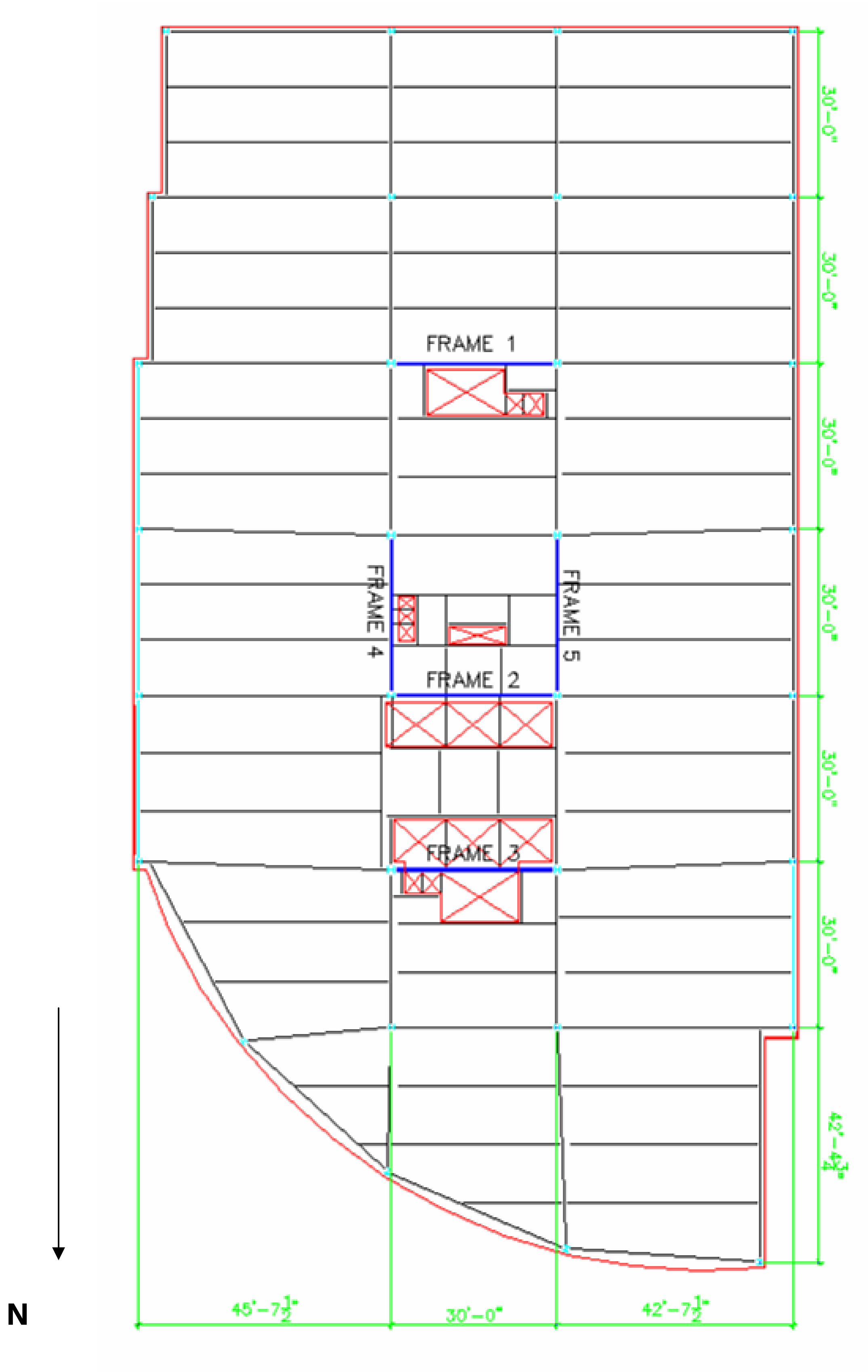
Note: Shaded area is roof construction



11[th] and 12[th] Floor Framing Plan



Enlarged Typical Framing Plan with Dimensions



Existing 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

Existing Typical Floor System Design

Levels 2-12 are intended to be used as rentable office space. The loads considered for the existing floor system design were researched, studied, and verified in Technical Report 1: Structural Concepts/Structural Existing Conditions Report, and are summarized below.

Loads:

Dead:

3 ¼" lt. wt. slab on 3" - 20 gage metal deck	46 PSF
Concrete Ponding	10 PSF
Misc. DL	15 PSF
Façade	15 PSF
Construction DL	56 PSF

Live:

Office	100 PSF (reducible)
Construction LL	20 PSF

The existing typical office floor system design consists of a concrete slab on metal deck supported by composite steel beams. The slab is 3 ¼" light weight concrete with an f'c = 3000 psi and is reinforced with 6 x 6 10/10 WWF. The metal deck is 3" – 20 gage composite steel deck bringing the total slab thickness to 6 ¼". The composite action between the slab on metal deck and the steel beams is provided by ¾" diameter, 5" headed shear studs.

There are three typical bay sizes for the steel superstructure above grade; 30' x 30', approximately 46' x 30', and approximately 43' x 30'. From North to South the columns are at a 30' spacing. From East to West the columns spacings are approximately 46', 30' and 43' respectively.

All of the columns are W14's.

The most common beam sizes are W18 x 50 for the 46' x 30' bays, W18 x 46 for the 43' x 30' bays, and W16 x 26 for the 30' x 30' bays 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, W24 x 55 and W21 x 44 around the perimeter.

Alternative Structural 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 below.

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 ≈ 176' (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.

Statement of the Problem

Based off of the study, research, analysis, and designs of the existing system and the four alternative systems, it was determined that the existing system is the most efficient design to meet the needs of the building, the project team, the schedule, 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 all semester, I want to challenge myself next semester by proposing to do a redesign 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 want to do a redesign 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
- Constructibility
- Labor
- Floor to floor height
- Floor to ceiling height
- Lateral system performance (braced frames vs. shearwalls)
- 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.

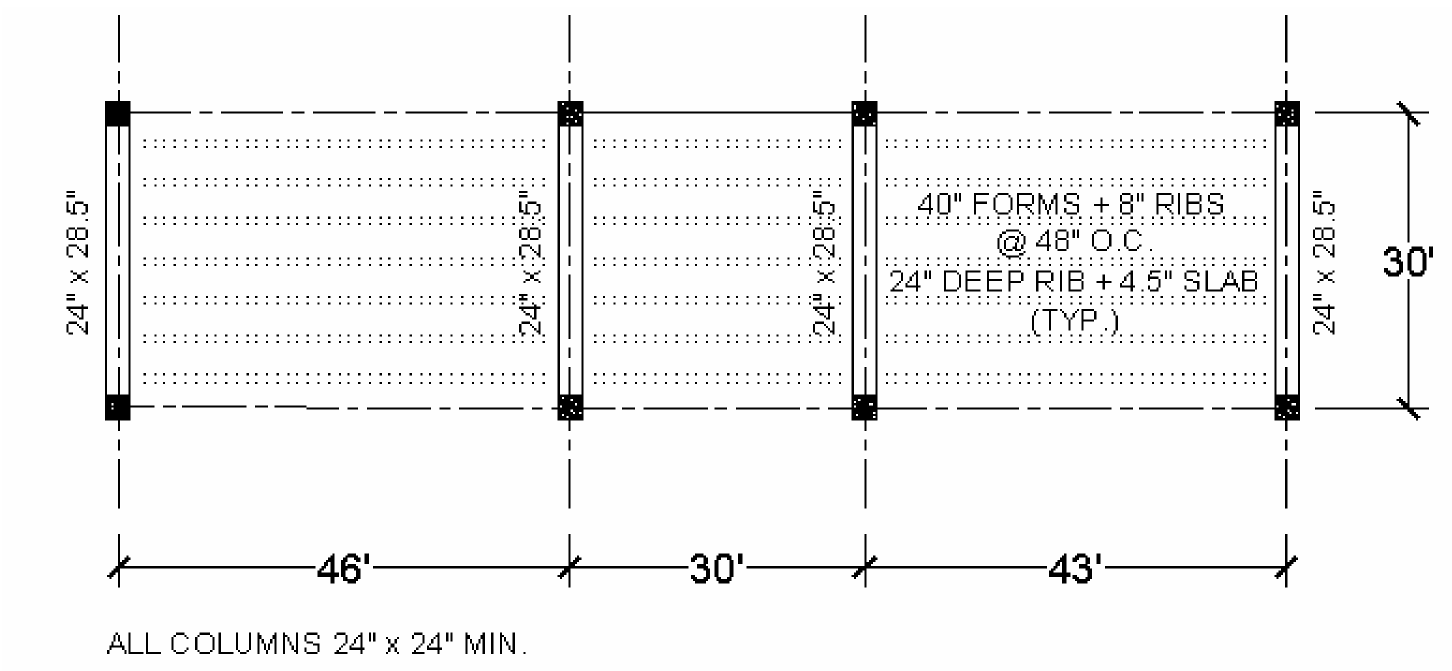
The goal is to keep the same column layout as the existing steel system in order to keep the original design intention of an open floor plan.

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

The One-way Joists with CIP Framing system was preliminarily designed in Technical Report 2 using the CRSI Handbook. The preliminary design is sketched below.

Please refer to CRSI, pages 8-67, 12-93, and 12-107, which can be found in the Appendix, for dimensions, reinforcing details, and properties of members. Also included in the Appendix are the calculations and loads considered for design.

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 3 spanning in the east / west direction. Since the redesign will be an all concrete system, a series of concrete shearwalls will be used as the lateral force resisting system. These shearwalls will ideally be placed around the elevator core and/or around the stairwells. Both the elevators and the stairwells are located in the central core of the building.

Loads

The loads considered for the existing design of The Regent were research, analyzed and checked throughout all of the Technical Reports. In some cases, the loads determined corresponded to the loads used in the existing design, in other cases they did not. In reviewing the loads considered for the existing design, some of the loads seemed to be very conservative such as the floor live load and the snow load, 100 PSF and 30 PSF, respectively. These conservative loadings may have been minimum requirements set forth by the structural engineer on this project. In the concrete redesign of The Regent, the loads considered will be optimized and will based off of IBC 2000, which was the model code used in the existing design. Although a direct comparison cannot be completed between the existing design and the redesign, the optimized loads will yield a more efficient design for the new concrete design.

Solution Method

The design of the concrete structure will be based off of ACI 318-05: *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: ADOSS, SAP, and PCACOL. Analysis of lateral loads will be completed by hand calculations and/or through the use of structural analysis software SAP2000. Trial sizes based off of the preliminary designs, determined through the CRSI Handbook and hand calculations, will be inputted into the computer programs along with the newly determined, optimized gravity and lateral loads. Live loading patterns will be considered and used to properly design the concrete gravity system.

Scope of Structure to be Designed

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

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 system and the new concrete system. Since it already has been initially pre-determined 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 new concrete system 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 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, a new mechanical system layout will be proposed.

Fire Protection

Since the new concrete structure is a new material, layout, and thickness than the existing steel structure, it will have a different fire rating. The fire rating of the new concrete system will be compared with the fire rating of the existing steel system. Also, any impacts on cost by utilizing the concrete system will be determined.

Acoustics

Since the new concrete system is significantly different than the existing steel system, it will have different acoustical values and effects. The Regent is primarily a spec office building, which has the potential to have several different tenants. An acoustical study will be performed on each system to see which performs better in preventing noise from penetrating through the floor system.

Tasks and Tools

Cast-In-Place Concrete Redesign

Task 1: Establish Trial Floor Plan and Member Sizes

- Determine preliminary floor plan (keeping existing column locations, determine joist span, shearwall locations, column and beam placement, coordinate with architectural drawings, etc.).
- Determine slab, joist, and beam limitations based off of ceiling height requirements and floor to floor height requirements.
- Determine economical balance between beam and floor system thickness.
- Determine trial pan size and depth and joist stem width and slab thickness.
- Determine of trial beam and column sizes using CRSI Handbook and ACI 318-05.

Task 2: Determine Optimized Loads, Gravity and Lateral

- Based on the IBC 2000, determine code required lateral and gravity loads and revise previously determined loads used in the Technical Reports.
- Determine the superimposed dead loads based off of building plans.
- Determine the live loads based off of IBC 2000 Table 1607.1.
- Determine the roof live load and snow load based off of ASCE 7-02, Chapters 4 and 7, respectively.
- Determine the wind loads based off of ASCE 7-02, Chapter 6, Method 2: Analytical Procedure.
- Determine seismic loads based off of ASCE 7-02, Chapter 9.
- Determine the self weight of trial members.
- Make a comparison chart of existing design loads and optimized design loads.
- Determine construction live loads and dead loads.

Task 3: Complete Initial Structural Analysis of Floor Framing System

- Determine factored shear and moment requirements and deflection limits in a typical bay based off of the newly determined loads.
- Check initial joist and beam size members by calculating joist and beam capacities based off of ACI 318-05 and compare to factored shear and moments.

Task 4: Complete Initial Structural Analysis of Shearwalls

- Complete initial lateral analysis of shearwalls by inputting initial shearwall sizes and locations and lateral loads into SAP2000 and run analysis
- Check computer results with hand calculations.

Task 5: Revise Trial Members based off of Initial Analysis

- Revise trial joist and beam sizes based off of initial analysis (repeat until system design is adequate to carry the applied gravity loads).
- Revise trial shearwall sizes based off of the initial computer analysis (repeat until system design is adequate to resist the applied lateral loads).
- Check results with hand calculated spot checks.

Task 6: Determine Column Loadings

- Determine column loadings throughout the structure.

Task 7: Complete Column Analysis and Design

- Use PCACOL to check the adequacy of the trial column sizes and design the columns for the determined loadings.
- Check PCACOL results with hand calculated spot checks.

Task 8: Complete a 3-D Structural Model of the Entire Building Using SAP2000

- Run an analysis of the structure as designed to this point for gravity and lateral loads.
- Revise any members that are not adequate.

Task 9: Preliminarily Redesign Foundations Based off of New Concrete Design and Loads

Task 10: Complete Construction Management Breath Study

- Complete cost analysis of the existing steel system and the new concrete system.
- Complete a schedule analysis of the existing steel system and the new concrete system.

Task 11: Other Breadth Studies: Mechanical, Fire Protection, Acoustical

- Determine the effects on the mechanical system layout due to the change in floor structure.
- The effect of the new structure with respect to spatial requirements.
- Determine the fire rating differences between the existing steel system and the new concrete system.
- Determine the acoustical differences between the existing steel floor system and the new proposed concrete floor system.

Task 12: Prepare Report

Task 13: Prepare Presentation

Schedule

Week	Description
Week of 1/9/06	• Meet with consultant to review proposal comments • Revise proposal • Post revised proposal • Post January work schedule
Week of 1/16/06	• Determine preliminary floor plan (keeping existing column locations, determine joist span, shearwall locations, column and beam placement, coordinate with architectural drawings, etc.) • Determine slab, joist, and beam limitations based off of ceiling height requirements and floor to floor height requirements • Determine economical balance between beam and floor system thickness • Determine trial pan size, depth, joist stem width and slab thickness • Determine trial beam and column sizes using CRSI Handbook and ACI 318-05 • Determine the superimposed dead loads based off of the building plans • Determine the self weight dead loads based off of the preliminary floor system design • Determine the live loads based off of IBC 2000 Table 1607.1 • Determine the roof live load and snow load based off of ASCE 7-02, Chapters 4 and 7 respectively • Determine the wind loads based off of ASCE 7-02, Chapter 6, Method 2: Analytical Procedure • Determine the seismic loads based off of ASCE 7-02, Chapter 9 • Determine construction live loads and dead loads
Week of 1/23/06	• Determine factored shear and moment requirements and deflection limits in a typical bay based off of the newly determined loads • Check initial joist and beam size members by calculating joist and beam capacities based off of ACI 318-05 and compare to factored shear and moments • Revise trial joist and beam sizes based off of the initial analysis (repeat until system design is adequate to carry the applied gravity loads)
Week of 1/30/06	• Complete initial lateral analysis of shearwalls by inputting initial shearwall sizes and locations and lateral loads into SAP 2000 and run analysis • Check computer results with quick hand calculations

	• Revise trial shearwall sizes and locations based off of the initial computer analysis (repeat until the system design is adequate to resist the applied lateral loads)
Week of 2/6/06	• Determine column loadings throughout the structure • Use PCACOL to check the adequacy of the trial column sizes and design the columns for the determined loadings • Check PCACOL results with hand calculated spot checks • Preliminarily redesign a spread footing based off of the new concrete design and loadings
Week of 2/13/06	• Complete cost analysis of the existing steel system and the new concrete system • Complete a schedule analysis of the existing steel system and the new concrete system
Week of 2/20/06	• Determine the effects on the mechanical system layout due to the change in floor structure • Determine the effect of the new structure with respect to spatial requirements • Determine the fire rating differences between the existing steel system and the new concrete system • Determine the acoustical differences between the existing steel floor system and the new concrete system
Week of 2/27/06	• System comparisons in cost, schedule, constructability, labor, floor to floor height, floor to ceiling height, lateral system performance (braced frames vs. shearwalls), self weight, and impact on the foundations
Week of 3/6/06	• SPRING BREAK • Make-up week or move onto report and presentation if on schedule
Week of 3/13/06	• Work on final thesis report
Week of 3/20/06	• Work on final thesis report
Week of 3/27/06	• Work on presentation
Week of 4/3/06	• **Final thesis report due posted to CPEP website on 4/3/06** • Print thesis report and get copies bound • **Final thesis report due by 12:00 PM on 4/7/06** • Finalize presentation
Week of 4/10/06	• Thesis presentations **PRESENTATION WEDNESDAY, APRIL 12, 2006** **TIME: 10:00 AM** **LOCATION: 107 ENG UNIT B**

Conclusion

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. The Regent is currently under construction, and since the building was not pre-leased before construction began, the occupants or tenants are not known at this point.

Based off of the study, research, analysis, and designs of the existing system (steel framing with composite slab) and the four alternative systems (hollowcore planks with steel framing, precast double tees with precast framing, one-way wide module joists with CIP framing, and a two-way flat slab with drop panels and CIP framing), it was determined that the existing system is the most efficient design to meet the needs of the building, the project team, the schedule, and the site.

Having studied the existing steel structure all semester, I wanted to challenge myself next semester by proposing to do a redesign 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 want to do a redesign of The Regent using a concrete system in order to make comparisons between the two systems.

A concrete system design shall be selected that meets as many of the initial design team's 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
- Constructibility
- Labor
- Floor to floor height
- Floor to ceiling height
- Lateral system performance (braced frames vs. shearwalls)
- Weight
- Impact on the foundations

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

The existing structure utilizes a series of 5 braced frames; 2 spanning in the north / south direction and 3 spanning in the east / west direction. Since the redesign will be an all concrete system, a series of concrete shearwalls will be used as the lateral force resisting system. These shearwalls will ideally be placed around the elevator core and/or around the stairwells. Both the elevators and the stairwells are located in the central core of the building.

The loads considered for the existing design of The Regent were research, analyzed and checked throughout all of the Technical Reports. In some cases, the loads determined corresponded to the loads used in the existing design, in other cases they did not. In reviewing the loads considered for the existing design, some of the loads seemed to be very conservative such as the floor live load and the snow load, 100 PSF and 30 PSF, respectively. These conservative loadings may have been minimum requirements set forth by the structural engineer on this project. In the concrete redesign of The Regent, the loads considered will be optimized and will based off of IBC 2000, which was the model code used in the existing design. Although a direct comparison cannot be completed between the existing design and the redesign, the optimized loads will yield a more efficient design for the new concrete design.

The design of the concrete structure will be based off of ACI 318-05: *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: ADOSS, SAP, and PCACOL. Analysis of lateral loads will be completed by hand calculations and/or through the use of structural analysis software SAP2000. Trial sizes based off of the preliminary designs, determined through the CRSI Handbook and hand calculations, will be inputted into the computer programs along with the newly determined, optimized gravity and lateral loads. Live loading patterns will be considered and used to properly design the concrete gravity system.

Scope of Structure to be Designed

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

As part of the breadth analysis requirements, the following breadth areas have chosen to be studied in order to help compare the two systems.

- Construction Management
    - Cost
    - Schedule

- Mechanical
    - Impact on mechanical layout
    - Possible redesign of mechanical layout of necessary
- Fire Protection
    - Comparison in fire rating between the existing steel floor system and the new concrete floor system
- Acoustics
    - Comparison between the resistance to noise penetrations between the existing steel floor system and the new concrete floor system.

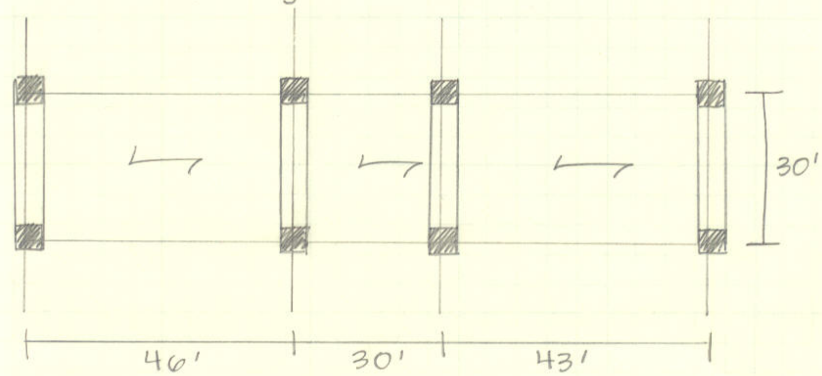
A schedule has been prepared describing what tasks will be completed and when throughout the semester.

Appendix

Wide Module One-Way Joists, Multiple Spans with CIP Framing System

Wide Module One-way Joists, Multiple Spans

Joists spanning Long Direction



worst case span = 46'

Loads:

Dead Load

Misc. DL = 15 PSF

Live Load

Office = 100 PSF

$A_T = 4'(46') = 184\ FT^2 < 400\ FT^2$ ∴ Not Reducible

Factored Superimposed Load

1.4 (15 PSF) + 1.7 (100 PSF) = 191 PSF

Use 1.4D + 1.7L in order to use CRSI charts

191 PSF (4') = 764 PLF

Wide Module One-Way Joists Spanning the Long Direction

Possible Joist Systems Take from CRSI

Option	Form Widths (IN)	Rib Widths (IN)	C-C Width (IN)	Rib Depth (IN)	Slab Depth (IN)	End Span Capacity (PLF)	Interior Span Capacity (PLF)	Self Weight (PLF)
1	40	8	48	24	4.5	873	926	475
2	40	9	49	24	4.5	987	1066	505
3	40	10	50	24	4.5	791	844	534
4	53	8	61	24	4.5	794	845	536
5	53	9	62	24	4.5	908	985	566
6	53	10	63	24	4.5	883	1110	595
7	66	9	75	24	4.5	827	903	627

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”

This wide-module one-way joist system was selected because it was the lightest design and because it had a modular width of exactly 4’. All of the possible systems had the same total depth.

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**

Interior Joist Band Beams

max. span = 30'

Loads:

Dead:
Misc. DL = 15 PSF
Joists = 119 PSF

Live:
Office = 100 PSF → 57 PSF

Live Load Reduction
$A_T = 38'(30') = 1140\ FT^2$
$K_{LL} = 2$
$L_o = 100$ PSF

$L = 100\left(0.25 + \frac{15}{\sqrt{2 \cdot 1140}}\right)$

L = 57 PSF

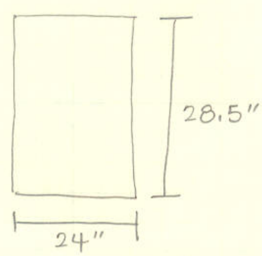
$W_u = 1.4(15 + 119) + 1.7(57)$
= 285 PSF

$W_u = 285\ PSF\left(\frac{46' + 30'}{2}\right)$
= 10,830 PLF
= 10.83 KLF

d = 28.5' to match total joist depth

Selection:

f'c = 4000 psi
fy = 60,000 psi



span = 30'

Bottom Bars:
l_n + 12" = (2) #14
0.875l_n = (1) #14

Top Bars: (5) #14

load capacity = 12.5 K/FT > 10.83 K/FT ∴ OK

Stirrups: 265E closed stirrups
26 - #5: 1 @ 2"
25 @ 7"

Exterior Joist Band Beams

max. span = 30'

Loads:

Dead:

Misc. DL = 15 PSF

Joists = 119 PSF

Live:

Office = 100 PSF → 66 PSF

Live Load Reduction

$A_T = 23'(30') = 690 \text{ FT}^2$

$K_{LL} = 2$

$L_o = 100 \text{ PSF}$

$$L = 100\left(0.25 + \frac{15}{\sqrt{2 \cdot 690}}\right)$$

$L = 66$ PSF

$W_u = 1.4(15+119) + 1.7(66)$

$= 300$ PSF

$$W_u = 300 \text{ PSF}\left(\frac{46'}{2}\right)$$

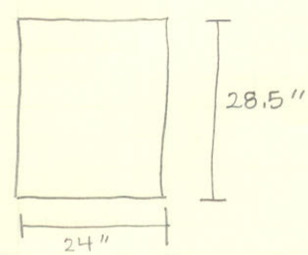
$= 6900$ PLF

$= 6.9$ KLF

Selection

f'c = 4000 psi

fy = 60,000 psi



span =

Bottom Bars:

$l_n + 12'' = 2\ \#14$

$0.875 l_n = 2\ \#14$

Stirrups: 235 F

- closed stirrups
- 23 #5: 1 @ 2"
  22 @ 8"

Top Bars: (4) #14

Load capacity: 10.1 K/FT > 6.9 K/FT

WIDE MODULE (1) ONE-WAY JOISTS MULTIPLE SPANS

40″ Forms + 8″ Ribs @ 48″ c.-c.

24″ Deep Rib + 4.5″ Top Slab = 28.5″ Total Depth

FACTORED USABLE SUPERIMPOSED LOAD (PLF)

f'_c = 4,000 psi, f_y = 60,000 psi

	End Span					End Span Defl. Coeff. (2)	Interior Span					Int. Span Defl. Coeff. (2)
TOP BARS NO	# 4	# 5	# 6	# 6	# 7		# 5	# 6	# 6	# 7	# 8	
AT	9.0	8.0	10.0	8.0	9.0		10.0	8.0	7.0	7.5	8.5	
BOTTOM BARS NO	2# 5	1# 8	1# 8	1# 7	1#10		2# 5	1# 8	1# 8	1# 7	1#10	
BARS NO	1# 5	1# 8	1# 9	2# 8	1#10		1# 5	1# 8	1# 9	2# 8	1#10	
STEEL (PSF)	.92	1.64	1.85	2.19	2.70		.89	.89	.89	.89	.89	
CLEAR SPAN												
37′-0″ (3)	226	832	1026	1393	1713	7.149	631	1512	1795	2329	2794	4.399
STIR	#3- 59	#3-139	#3-153	#3-153	#4-185		#3- 87	#3-143	#3-129	#4-167	#5-174	
38′-0″	180	754	938	1286	1589	7.953	564	1399	1667	2173	2614	4.894
STIR	#3- 55	#3-139	#3-154	#3-162	#4-188		#3- 86	#3-144	#3-136	#4-169	#4-178	
39′-0″	137	682	857	1187	1475	8.824	502	1295	1549	2030	2448	5.430
STIR	#3- 50	#3-140	#3-155	#3-170	#3-190		#3- 84	#3-146	#3-143	#4-172	#4-181	
40′-0″	97	616	782	1096	1370	9.765	444	1198	1440	1897	2295	6.009
STIR	#3- 46	#3-140	#3-156	#3-177	#3-193		#3- 83	#3-147	#3-150	#4-175	#4-184	
41′-0″	61	554	712	1011	1272	10.778	391	1108	1338	1773	2152	6.633
STIR	#3- 41	#3-140	#3-157	#3-179	#3-196		#3- 81	#3-148	#3-158	#4-177	#4-187	
42′-0″		497	647	932	1180	11.869	341	1025	1244	1658	2019	7.304
STIR		#3-140	#3-157	#3-181	#3-199		#3- 78	#3-149	#3-161	#3-180	#4-190	
43′-0″		443	587	859	1096	13.040	295	947	1156	1552	1896	8.025
STIR		#3-139	#3-158	#3-183	#3-179		#3- 76	#3-150	#3-163	#3-182	#4-193	
44′-0″		393	531	790	1016	14.296	252	875	1074	1452	1781	8.798
STIR		#3-139	#3-158	#3-184	#3-187		#3- 73	#3-151	#3-164	#3-185	#4-196	
45′-0″		347	478	726	942	15.641	211	807	998	1359	1673	9.625
STIR		#3-138	#3-158	#3-186	#3-196		#3- 71	#3-152	#3-166	#3-187	#4-198	
46′-0″		303	429	666	873	17.078	174	744	926	1272	1573	10.510
STIR		#3-137	#3-158	#3-187	#3-204		#3- 67	#3-153	#3-167	#3-163	#4-201	
47′-0″		262	383	610	809	18.613	138	684	859	1190	1479	11.454
STIR		#3-136	#3-158	#3-188	#3-207		#3- 64	#3-153	#3-168	#3-170	#4-204	
48′-0″		224	340	558	748	20.248	105	629	796	1114	1390	12.460
STIR		#3-135	#3-158	#3-189	#3-209		#3- 61	#3-154	#3-169	#3-177	#4-206	
49′-0″		188	299	508	691	21.989	74	576	737	1042	1307	13.532
STIR		#3-133	#3-157	#3-190	#3-210		#3- 57	#3-154	#3-170	#3-184	#4-208	
PROPERTIES FOR DESIGN (CONCRETE .79 CF/SF)												
NEGATIVE MOMENT												
STEEL AREA (SQ. IN.)	1.07	1.86	2.11	2.64	3.20		1.49	2.64	3.02	3.84	4.46	
ACTUAL STEEL %	.390	.682	.776	.970	1.180		.545	.970	1.109	1.415	1.649	
EFF. DEPTH, IN.	26.75	26.69	26.63	26.63	26.56		26.69	26.63	26.63	26.56	26.50	
– ICR/IGR	.126	.196	.215	.255	.292		.164	.255	.281	.333	.367	
POSITIVE MOMENT												
STEEL AREA (SQ. IN.)	.93	1.58	1.79	2.18	2.54		.93	1.58	1.79	2.18	2.54	
ACTUAL STEEL %	.331	.566	.641	.780	.913		.331	.566	.641	.780	.913	
EFF. DEPTH, IN.	26.69	26.50	26.46	26.52	26.36		26.69	26.50	26.46	26.52	26.36	
+ICR/IGR	.135	.217	.242	.291	.329		.135	.217	.242	.291	.329	

SINGLE LEG STIRRUP AT 13 IN. CONSTANT SPACING-DISTANCE (IN.)

(1) For gross section properties, see Table 8-3.
(2) Computation of deflection is not required above horizontal line (thickness ≥ ℓ_n/18.5 for end spans, ℓ_n/21 for interior spans).
(3) Single leg stirrup size space at X in. c.-c. Distance over which stirrups must extend from face of support at each end (in.).

WIDE MODULE (1) ONE-WAY JOISTS MULTIPLE SPANS

40″ Forms + 9″ Ribs @ 49″ c.-c.

24″ Deep Rib + 4.5″ Top Slab = 28.5″ Total Depth

FACTORED USABLE SUPERIMPOSED LOAD (PLF)

f'_c = 4,000 psi, f_y = 60,000 psi

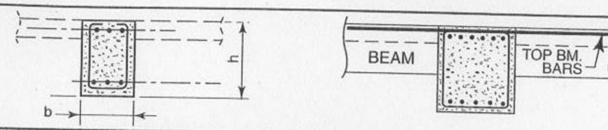
	End Span					End Span Defl. Coeff. (2)	Interior Span					Int. Span Defl. Coeff. (2)
TOP BARS NO	# 5	# 6	# 6	# 7	# 8		# 5	# 6	# 7	# 8	# 9	
AT	12.0	11.0	9.0	10.0	11.0		9.5	8.0	9.0	9.5	10.0	
BOTTOM BARS NO	2# 4	1# 6	2# 7	2# 8	1# 8		2# 4	1# 6	2# 7	2# 8	1# 8	
BARS NO	2# 5	2# 7	1# 8	1# 8	2# 9		2# 5	2# 7	1# 8	1# 8	2# 9	
STEEL (PSF)	1.02	1.61	2.01	2.37	2.78		.91	.91	.91	.91	.91	
CLEAR SPAN												
37′-0″ (3)	271	852	1177	1527	1911	7.149	716	1560	2033	2543	3101	4.399
STIR	#3- 59	#3-133	#3-154	#3-155	#4-185		#3- 87	#3-138	#3-154	#4-166	#5-175	
38′-0″	221	771	1079	1411	1775	7.953	642	1442	1891	2374	2903	4.894
STIR	#3- 55	#3-133	#3-155	#3-163	#4-188		#3- 86	#3-139	#3-133	#4-169	#5-178	
39′-0″	174	696	988	1304	1649	8.824	574	1333	1759	2218	2720	5.430
STIR	#3- 51	#3-133	#3-157	#3-172	#4-190		#3- 84	#3-141	#3-140	#4-171	#5-182	
40′-0″	130	627	905	1205	1533	9.765	511	1233	1637	2074	2551	6.009
STIR	#3- 46	#3-133	#3-158	#3-177	#4-194		#3- 82	#3-142	#3-148	#4-174	#5-185	
41′-0″	90	562	827	1113	1425	10.778	452	1139	1524	1940	2394	6.633
STIR	#3- 42	#3-132	#3-158	#3-178	#3-196		#3- 80	#3-143	#3-155	#4-176	#4-187	
42′-0″	52	503	755	1027	1325	11.869	397	1052	1419	1815	2248	7.304
STIR	#3- 37	#3-132	#3-159	#3-180	#3-199		#3- 78	#3-144	#3-163	#4-179	#4-191	
43′-0″		447	688	947	1231	13.040	347	971	1322	1699	2112	8.025
STIR		#3-131	#3-160	#3-182	#3-178		#3- 76	#3-144	#3-164	#4-181	#4-193	
44′-0″		395	625	873	1144	14.296	299	896	1230	1591	1986	8.798
STIR		#3-130	#3-160	#3-183	#3-186		#3- 73	#3-145	#3-166	#3-184	#4-196	
45′-0″		347	566	803	1063	15.641	255	826	1145	1490	1867	9.625
STIR		#3-129	#3-160	#3-184	#3-195		#3- 70	#3-145	#3-167	#3-186	#4-199	
46′-0″		301	512	738	987	17.078	214	760	1066	1396	1757	10.510
STIR		#3-128	#3-160	#3-186	#3-203		#3- 67	#3-146	#3-169	#3-164	#4-202	
47′-0″		259	460	678	915	18.613	175	698	991	1307	1653	11.454
STIR		#3-126	#3-160	#3-187	#3-207		#3- 64	#3-146	#3-170	#3-171	#4-204	
48′-0″		219	412	621	849	20.248	139	640	921	1224	1556	12.460
STIR		#3-124	#3-160	#3-188	#3-209		#3- 61	#3-146	#3-171	#3-178	#4-207	
49′-0″		182	367	567	786	21.989	104	586	855	1146	1464	13.532
STIR		#3-122	#3-160	#3-188	#3-211		#3- 57	#3-146	#3-172	#3-186	#4-209	
PROPERTIES FOR DESIGN (CONCRETE .82 CF/SF)												
NEGATIVE MOMENT												
STEEL AREA (SQ. IN.)	1.27	1.96	2.40	2.94	3.52		1.60	2.69	3.27	4.07	4.90	
ACTUAL STEEL %	.423	.656	.802	.987	1.185		.534	.902	1.097	1.372	1.654	
EFF. DEPTH, IN.	26.69	26.63	26.63	26.56	26.50		26.69	26.63	26.56	26.50	26.44	
– ICR/IGR	.136	.193	.226	.262	.297		.165	.247	.283	.330	.374	
POSITIVE MOMENT												
STEEL AREA (SQ. IN.)	1.02	1.64	1.99	2.37	2.79		1.02	1.64	1.99	2.37	2.79	
ACTUAL STEEL %	.331	.535	.650	.775	.913		.331	.535	.650	.775	.913	
EFF. DEPTH, IN.	26.71	26.58	26.54	26.5	26.45		26.71	26.58	26.54	26.50	26.45	
+ICR/IGR	.137	.209	.249	.290	.334		.137	.209	.249	.290	.334	

SINGLE LEG STIRRUP AT 13 IN. CONSTANT SPACING-DISTANCE (IN.)

(1) For gross section properties, see Table 8-3.
(2) Computation of deflection is not required above horizontal line (thickness ≥ ℓ_n/18.5 for end spans, ℓ_n/21 for interior spans).
(3) Single leg stirrup size space at X in. c.-c. Distance over which stirrups must extend from face of support at each end (in.).

$f'_c = 4{,}000$ psi
$f_y = 60{,}000$ psi

JOIST-BAND BEAMS, INTERIOR SPANS



STEM h in.	STEM b in.	BARS(1) BOTTOM ℓ_n + 12 in.	BARS(1) BOTTOM 0.875 ℓ_n	Layers (2)	TOP	SPAN, ℓ_n = 30 ft LOAD (4) k/ft	STIR. TIES (5)	ϕT_n ft-kips	$A\ell$ sq. in.	STEEL WGT lb.	SPAN, ℓ_n = 32 ft LOAD (4) k/ft	STIR. TIES (5)	ϕT_n ft-kips	$A\ell$ sq. in.	STEEL WGT lb.	SPAN, ℓ_n = 34 ft LOAD (4) k/ft	STIR. TIES (5)	ϕT_n ft-kips	$A\ell$ sq. in.	STEEL WGT lb.	SPAN, ℓ_n = 36 ft LOAD (4) k/ft	STIR. TIES (5)	ϕT_n ft-kips	$A\ell$ sq. in.	STEEL WGT lb.	$+\phi M_n$ / $-\phi M_n$ (6) ft-kip	DEFL (C) (7) $\times 10^{-9}$ in.
	24	2#10	1#10	1	4#10	6.7	113J	23	-	814	5.9	123J	23	-	870	5.2	123J	23	-	917	4.7	123J	23	-	963	419	122
				1			175I	93	2.2	1331		195H	92	2.2	1451		215H	91	2.2	1571		225H	91	2.1	1655	548	
		2#11	1#11	1	4#11	8.1	123J	23	-	990	7.1	133J	23	-	1057	6.3	133J	23	-	1115	5.6	133J	23	-	1172	507	121
				1			235F	93	2.2	1718		245F	92	2.2	1812		195I	91	2.1	1686		205I	91	2.1	1780	659	
		2#14	1#14	1	5#14	12.5	145J	23	-	1825	11.0	144J	23	-	1765	9.7	154J	23	-	1876	8.7	164J	23	-	1988	703	96
				1			265E	93	2.2	2365		285E	92	2.2	2532		265F	91	2.1	2551		275F	91	2.1	2681	1089	
		2#14	2#14	1	6#14	15.4	145J	23	-	2150	13.5	155J	23	-	2295	12.0	165J	23	-	2439	10.7	165J	23	-	2554	904	83
				1			305D	93	2.2	2837		325D	92	2.2	3025		295E	91	2.1	3030		315E	91	2.1	3218	1256	
	36	2#10	2#10	1	5#11	10.0	113J	43	-	1109	8.8	123J	42	-	1185	7.8	123J	42	-	1251	6.9	123J	42	-	1316	562	83
				1			265E	171	3.2	2072		195H	169	3.2	1847		215H	167	3.2	1995		723A	166	3.2	2158	838	
		2#11	2#11	1	5#14	12.1	124J	43	-	1578	10.6	133J	42	-	1560	9.4	133J	42	-	1648	8.4	133J	42	-	1737	681	78
				1			265E	171	3.2	2414		285E	169	3.2	2585		295E	167	3.2	2715		315E	166	3.1	2886	1159	
28.5		2#14	2#14	1	7#14	16.9	135J	43	-	2310	14.8	145J	42	-	2467	13.1	155J	42	-	2624	11.7	154J	41	-	2571	949	67
				1			305D	171	3.2	3136		325D	169	3.2	3344		345D	167	3.1	3552		315E	166	3.1	3552	1544	
		3#14	2#14	1	8#14	20.6	155GeJ	43	-	2765	18.1	165GdJ	42	-	2947	16.0	165GcJ	42	-	3098	14.3	165J	41	-	3248	1159	60
				1			365C	171	3.2	3775		385C	169	3.2	4009		415C	167	3.1	4284		365D	166	3.1	4227	1720	
	48	3#10	3#10	1	6#11	12.4	114J	64	-	1573	10.9	114J	63	-	1661	9.6	114J	63	-	1748	8.6	124J	62	-	1858	838	58
				1			364C	256	4.3	2381		**	253	4.5	1416		**	251	4.4	1504		**	249	4.3	1591	1014	
		3#11	3#11	1	6#14	17.2	125J	64	-	2192	15.1	134J	63	-	2177	13.4	134J	63	-	2294	11.9	144J	62	-	2433	1014	60
				1			455B	256	4.3	3859		285E	253	4.2	3187		295E	251	4.2	3351		315E	249	4.1	3561	1407	
		3#14	3#14	1	9#14	24.4	165FfJ	64	-	3167	21.5	155GeJ	63	-	3304	19.0	165GdJ	63	-	3511	17.0	165GcJ	62	-	3683	1407	49
				1			455B	256	4.3	4693		485B	253	4.2	5005		515B	251	4.2	5316		545B	249	4.1	5628	1998	
		4#14	3#14	1	11#14	28.7	205DIJ	64	-	3850	25.2	185EiJ	63	-	3988	22.3	185FhJ	63	-	4196	19.9	185FgJ	62	-	4404	1612	44
				1			455B	256	4.3	5237		485B	253	4.2	5584		515B	251	4.2	5931		545B	249	4.1	6278	2350	

TOTAL CAPACITY $U = 1.4D + 1.7L^{(3)}$

(1) See "Recommended Bar Details", Fig. 12-1. For girders, use tabulated beam depth — 2 inches (b — 2").
(2) In "Layers" column, first line is number of layers for bottom bars, second line is for number of layers for top bars.
(3) For superimposed factored load capacity, deduct 1.4 x stem weight.
(4) Total capacities tabulated causing deflection in excess of $\ell_n/360$ are designated thus: * $-\ell_n/360 <$ deflection $< \ell_n/240$
X $-\ell_n/240 <$ deflection $< \ell_n/180$
Y — deflection $> \ell_n/180$

(5) For each beam design, first line is for open stirrups, secondline is for closed ties. See Fig. 12-4. At free ends, use stirrups tabulated for "Interior Spans". For b > 24 in., provide 4 legs (two stirrups) of size and spacing tabulated. For stirrup nomenclature, see page 12-13.

Other notation: N/A — STIRRUPS ARE NOT REQUIRED
** — MAXIMUM SPACING IS LESS THAN 3 INCHES. NOT RECOMMENDED
*** — SHEAR STRESS IS GREATER THAN $10\sqrt{f'_c}$
**** — TORSION STRESS EXCEEDS ALLOWABLE

(6) $+\phi M_n$ and $-\phi M_n$ are design moment strength capacities for rectangular section b × h.
(7) Midspan elastic deflection (in.) = C x (w/1.6) x ℓ_n^4, where w = tabulated load (k/ft.), ℓ_n in ft.
"Average service load" is taken as w/1.6.