Project No. LDA-1302

Concrete vs. Steel Design Comparison

MQP Report

Justin Furst

Josh Raposo

Carl Haroian

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Concrete vs. Steel Design Comparison

A Major Qualifying Project Report
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by

Justin Furst

Carl Haroian

Joshua Raposo

Report Submitted to:

Professor Leonard Albano

Professor Aaron Sakulich

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Abstract

The objective of this MQP is to explore an effective design for a three- story office building. The proposed structure is situated on a section of land previously occupied by the Galleria Mall in downtown Worcester, Massachusetts. After exploring multiple options for structural steel and reinforced concrete, the two most economical designs for each material were further evaluated to determine the best approach in terms of cost. The body of work done takes various cost and schedule implications into account.

Authorship Page

Justin Furst____________________ Justin Furst

- Steel Structure Designer
    - Designed one steel option
- Foundation Designer
    - Designed foundations for two steel designs
- Co-author and Associate Editor

Carl Haroian_______________ Carl Haroian

- Concrete Structure Designer
    - Designed one concrete option
- Co-author and Principal Editor
    - Principally organized project report
- Cost Estimator
    - Estimated costs of each alternative
- Site Researcher/Planner
    - Researched the site and zoning regulations

Joshua Raposo_______________ Joshua Raposo

- Steel Structure Designer
    - Designed one steel options
- Concrete Structure Designer
    - Designed one concrete options
- Foundation Designer
    - Designed foundations for both concrete options
- Co-author and Associate Editor

Capstone Design

The objective of the project was to determine the more economical building design approach for a three-story office building. To determine feasibility, structural alternatives were developed in both structural steel and reinforced concrete framing and subjected to an array of cost analyses. The cost analyses included labor and material procurement costs, foundation costs, and interior construction costs. Initially, four structural systems were explored; two for each construction material. The costs associated with each system were compared in order to select the better option in each material for further design. Further cost evaluations were made in order select one of the two final alternatives as the most economical option out of all four; the analysis being done in terms of cost as well.

There are several constraints addressed by this project. The economic constraints for this project are primarily associated with the costs of the structure discussed above. Sustainability will be increased by utilizing a more eco-friendly concrete mix such as one with high volumes of fly ash to lower the carbon footprint of concrete construction. In terms of constructability, the plans and specifications for each structure contain parameters, such as accurate member sizes and calculated values for loading that demonstrated sufficient knowledge of the required technical information needed to construct each building. Each design was done according to the loading requirements presented in ASCE-7 for dead and live loads as well as structural building requirements addressed in the *Massachusetts State Building Code* to address health and safety concerns.

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Chapter 1: Introduction

Structural design is an important aspect of infrastructure development. There are many materials that may be used in the design of a building, whether it is residential, commercial, or industrial. Commercial buildings, such as office buildings and shopping centers, are primarily designed and built with steel or concrete because these materials are non-combustible. The City of Worcester, Massachusetts is currently exhibiting a substantial amount of infrastructure development, namely in building design and construction. One area located in the center of Worcester, known as the Galleria Mall, was recently demolished, and several new office buildings have since been erected.

As an alternative option to one of the buildings currently under construction, a three-story office building was investigated. Four alternative structural design options, two each in structural steel and reinforced concrete, were proposed and evaluated. One scheme in each material was selected for further design development and evaluation. The objective of this project was to determine the more economical approach for the three-story office building, steel or concrete.

The feasibility was addressed in terms of construction cost analysis, which included labor costs, interior construction costs, material procurement and costs for the framing and foundation. This project also addresses other real-world constraints such as health and safety, and sustainability.

Chapter 2: Background

Before the design of the proposed structured was initiated, other considerations were contemplated, such as the history of the site itself, the zoning regulations, and the dimensional and floor layouts of the proposed office building. The labor and material costs for steel and reinforced concrete construction as well as ancillary costs associated with each material, such as architectural finishes, were also considered.

2.1: History of the Galleria Mall Site/Current Development of Site

The mall, shown in Figure 2.1, was once built to try and save the City of Worcester, Massachusetts, but is now considered to be “A classic example of an urban renewal project gone wrong, built on an inhuman scale and unkind to its surroundings (Caldor).” The circled building shown in Figure 2.2 refers to the site for the proposed structure.

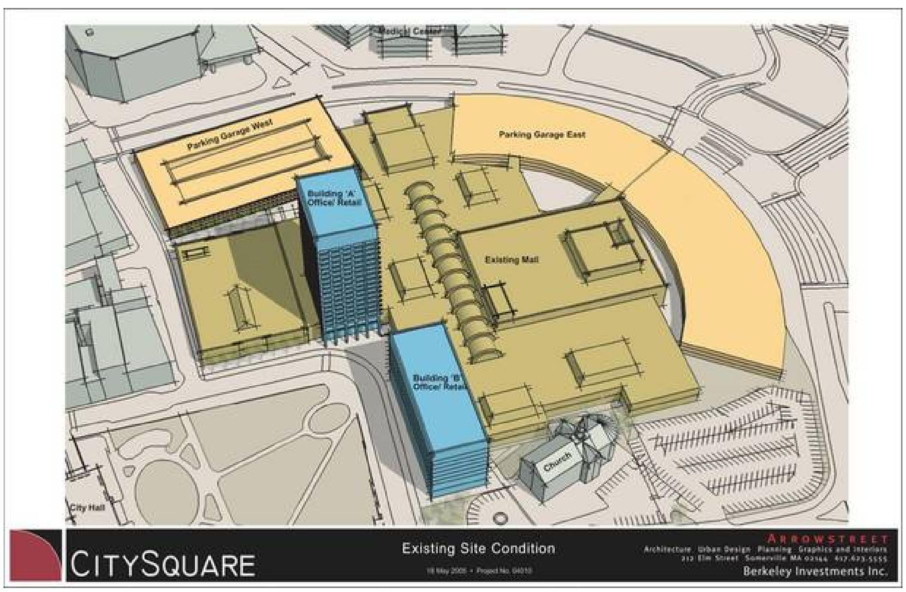

Figure 2.1: Mall and Parking Garage (Caldor).

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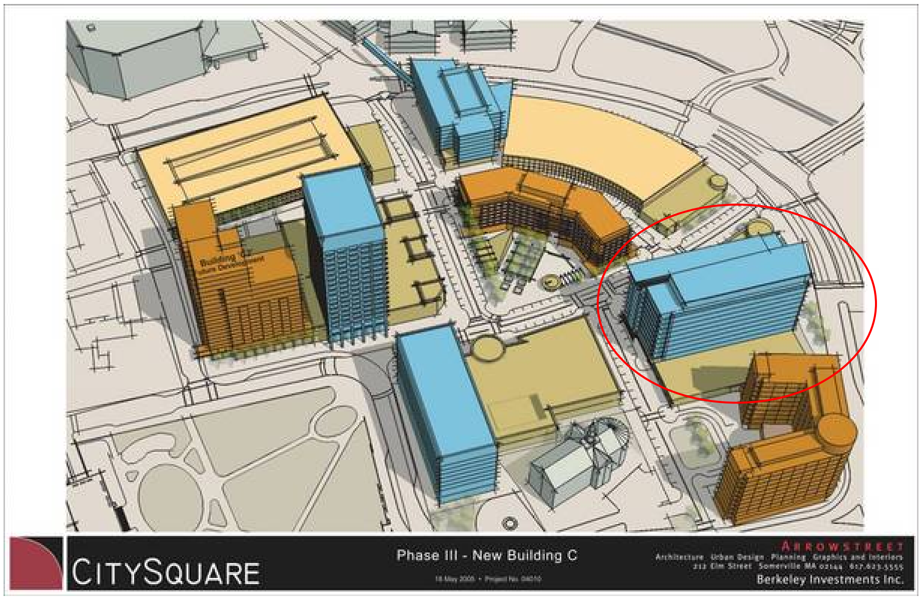


Figure 2.2: Proposed Layout of Area Replacing Mall (Caldor).

The mall was originally known as the Worcester Center Galleria, opened in 1971, and encompassed 1,000,000 square feet of floor space with a 4,300 car parking garage. At the time, it was the largest parking garage in the world. Within two years, the mall began to decline because the urban location of the mall deterred shoppers. People from the city itself disdained the fact it replaced historical areas of Worcester and blocked most routes to get from downtown to the east side. By the 1990's, the mall lost most of its patronage to suburban malls. It was subsequently purchased and converted into a high-end, more expensive shopping mall, known as the Worcester Common Fashion Outlets. The upscale shopping mall then endured its second major decline, and did not rebound from it (Caldor).

In 2004, Berkley Investments of Boston purchased the lot and razed the mall in order to create a "City Square," as illustrated in Figure 2.2. The plan was to bring urbanization to the center of the city, open up the city and allow easier commuting routes across the city. Two new office buildings are being constructed in the area; one of which was on the site selected for this project (Caldor). Based on the current uses of the former shopping area, a three-story office building was proposed to fit in with the new surrounding buildings.

2.2: Current Zoning Restrictions

The City of Worcester has many mandatory zoning and dimensional requirements for the buildings constructed within its boundaries. Zoning deals with how the building is situated on the site as well as how large the structure may be relative to surrounding buildings in a designated area. This area is known as a zoning district. Each zoning district has its own set of regulations based on the primary uses of the buildings that characterize it. In order to promote the health and safety of building occupants, the State of Massachusetts has composed a book of building codes that contain layout regulations in regards to fire safety and hazard control. The zoning regulations as well as the regulations denoted in the *Massachusetts State Building Code* for structural safety are discussed further in Chapter 4.

2.3: Layout of Proposed Design

The architectural layout of the building is of principal importance for the long term success of the project. This layout deals with the form and function of spatial relationships. The desired spaces must be situated in a way that allows easy access from one space to another. The architectural program denotes the areas and placements of various rooms throughout the building, including the office spaces, restrooms, mechanical rooms, and storage rooms, as well the placement of doors and windows. For this project, the architectural layout chosen was inspired by a layout already implemented by a building on the campus of Worcester Polytechnic Institute. The layout is further elaborated on in Chapter 4. *AutoCAD*, a computer drafting software, helps convey the building layout.

2.4: Structural Analysis

The structural layout of the building is largely influenced by the structural behavior of each construction material. Structural design consists of four major components: gravity loads, lateral loads, connections, foundation design. All of these components are imperative in achieving a structurally-sound office building.

Gravity loads involve the selections of beams, girders, and columns to support various types of vertical loads based on weight, including dead loads, live loads, and other gravity loads. The moment capacities of each member are also checked to make sure that they can sustain the loads acting on the structure. Typical load values for each aspect of the design are denoted in *ASCE (American Society of Civil Engineers)-7-10,* which has values for different types of loading for each region of the country as well as each area of a structure. The deflections of each member are also critical. If a member undergoes excessive deflection, this could result in a potential failure within the structural frame. Specifications for the structural steel designs were based on the *AISC (American Institute of Steel Construction-11) Specification*, which contains member sizes, member properties, and moment and deflection equations. The specifications for designing with concrete were from the *ACI (American Concrete Institute) 318-11 Specification,* which contains moment coefficients, deflection equations, and recommendations for member and slab thicknesses and depth of reinforcement.

The subsequent step is designing for various types of lateral loads, including wind loads and seismic loads. Lateral load design for structural frames consists of further column analysis for the effects of combined axial forces and bending moments. The structural analysis program *RISA* allows the user to analyze structural steel and reinforced concrete frames both for gravity loads and lateral loads as well as variations to the structure such as choosing the type of supports, member connection, and loading conditions. This software also allows the user to see the results graphically on a diagram of the effects that the axial forces, shear forces, and moment have on the structure.

The third aspect of structural analysis is the design of connections. The connections hold the components of the structural frame together. There are two types of connections. A simple shear connection only transfers lateral forces between members and is relatively low-priced, while moment connections transfer moment between the connected elements and cost more. Typical connections include beam-to-girder connections and beam-to-column connections. The engineer has the option of designing a given connection using high performance bolts or fillet welds to fasten structural members together. Design codes for connections were referred to in the *AISC Manual*.

Connections for concrete designs are performed by extending the rebar and tying it with the rebar of the connecting elements. Cross-ties and stirrups are used to tie the rebar of each member together. The two most common types of connections are gravity force-resisting and moment-resisting connections. Gravity force-resisting connections only consider gravity forces whereas moment-resisting connections resist lateral loads acting on the structure. Each type of connection has its own set of factors developed by ACI Committee 352 that account for such design (Nilson, 363).

The final aspect is the foundation design, which is what the transfers load onto the supporting soil. The foundation design involves soil analysis of the site of the proposed building. A composite soil profile is developed from boring log data. The two common types of foundations are shallow foundations and deep foundations. A shallow foundation is usually designed for firm soils or for supporting light loads, while a deep foundation is designed for weak soils or for supporting heavy loads.

Bearing capacity analysis of the soil deals with both the compressive stresses and shear stresses induced by structural loads. If the bearing pressure from the structure exceeds the shear strength of the soil, this could result in failure due to bearing capacity. The bearing capacity of a foundation uses two approaches: one approach utilizes the Terzaghi's method formulas, and the other utilizes Vesic's method formulas. Both sets of formulas involve using soil parameters and factors specific to each method to calculate bearing capacity.

Terzaghi's method for computing bearing capacity of soils consists of three key assumptions: the depth of the foundation is less than or equal to its width; the bottom of the foundation is rough enough that no sliding occurs between the foundation and the soil and the soil beneath the foundation extends to a great depth and the soil properties are considered uniform throughout.

Vesic's method, on the other hand, produces more accurate bearing values and applies to more loading and geometry ranges than Terzaghi's method. The only challenge to the use of Vesic's method is the increased complexity due to the variety of load and geometry conditions it considers in formulas (Coduto).

The settlement analysis of structures on a given foundation is comprised of two types of settlement patterns. The first is total settlement, which is uniform settlement from the structure. The second is differential settlement, which is tilting involved with the settlement, either with or without distortion of the structure. The settlement analysis of a structure also incorporates several approaches; one being the classical method based on Terzaghi's theory of consolidation. This theory assumes the settlement is a one-dimensional process. A one-dimensional process is a plane strain model with only vertical strain. The second approach is Schmertmann's method, which is based on a physical model of settlement. The physical model is calibrated using empirical data from laboratory tests. This method is generally used with cone penetration test (CPT) results and footings on sandy soils but can be adapted to accommodate other soil test results (Coduto). Generally, the serviceability requirement for foundation design sets an allowable total settlement of 1 inch for a typical office building.

2.5: Cost Analysis

Every aspect of the structural design has an impact on the total cost of the building. The number of beams and girders utilized as well as their sizes has a conspicuous impact on the final building cost. The cost of construction, including the delivery of materials and the labor required for installation is directly proportional to member sizes.

Architectural details and finishes, including exterior and interior masonry and walls, ceilings, lighting fixtures, door and window treatments, stairs, elevators, and interior carpentry have a considerable impact on the total cost in terms of the quality of each item. HVAC (heating, ventilation, and air conditioning), mechanical, electrical, and plumbing (MEP) installation contribute to the final cost as well. The exterior walls are influenced by the structural systems, whereas the other items to consider for cost analysis are not dependent on the structural design. The interior costs are primarily influenced by the final structural layout of each material and one architectural layout suited for both materials. The layout of the MEP equipment is determined by the final architectural layout. Prices for each component of the design may be found in various *RSMeans Cost Data* books that contain relevant costs.

Chapter 3: Methodology/Scope of Work

The flow chart shows the overall sequence of events that the project followed to determine the most economical construction methods for the proposed building structure. After the layout and site placement were completed, each structural design method was explored. Methods one and two refer to structural steel construction and utilized slab on metal decking; one option with rolled steel beams and another option with open web steel joists. Methods three and four dealt with reinforced concrete construction and were one-way slab-and-beam and one-way concrete joists, respectively. When analyzing costs of both frames supporting gravity loads, lateral load stability, and interior costs, cost data was taken from *RSMean Building Construction Cost Data: 70th Annual Edition*. Table 3.1 denotes each process in more technical detail.

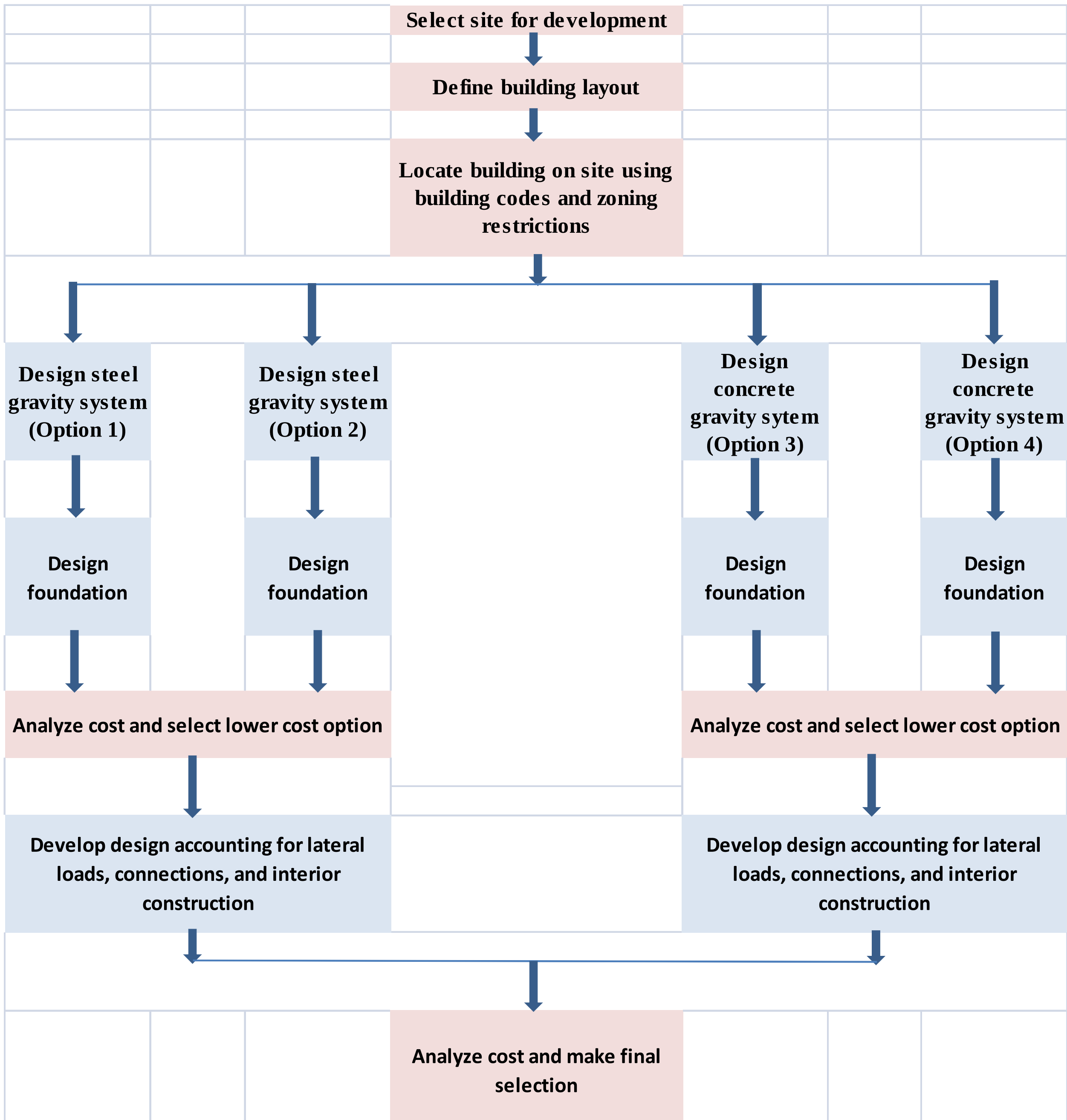


Figure 3.1: Methodology Flow Chart

Table 3.1: Description of Activities

Activities	Process	References
Background information of Galleria Mall site	Researched previous development, land use, possible future use of site that is currently under development	• Article by Caldor (9/14/06) regarding the history of the Galleria Mall & future developments
Layout of proposed office building	The office building layout was decided based on an existing, functioning building (Kaven Hall) with a double-loaded, center corridor, two wings of offices, and a set of staircases at each end	• Kaven Hall layout
Locate office building on Site	Locate the proposed office building on chosen site based on: • Local zoning regulations • Property setbacks	• Worcester, MA zoning regulations
Steel design of office Building	Structural analysis of the steel structure: • Two base designs: a) Steel structure using rolled steel beams b) Steel Structure using open web steel joist • design components for gravity loads include: beams→ girders → columns→ relevant AISC specification design checks	• *AISC Steel Construction Manual 14th Edition* • *Structural Steel Design 5th Edition (McCormac, 2012)* • *RISA* analysis program • *ASCE 7 2010 Edition* • *Steel Joist Institute 43rd Edition Standard* Specifications

<table>
<tr><td>Reinforced Concrete design of office building</td><td>Structural analysis of the reinforced concrete structure:
Two base designs:
a) Concrete structure using one- way slab and beam
b) Concrete structure using one-way concrete joists
design components for gravity loads include: slabs→ columns→ relevant ACI design checks
Look at alternative concrete mix design</td><td>ACI 318:Building Code Requirements for Structural Concrete and Commentary
Design of Concrete Structures 14th Edition (Nilson, 2010)
ASCE 7 2010 Edition
Design and Control of Concrete Mixtures 14th Edition
RISA analysis program</td></tr>
<tr><td>Foundations for concrete and Steel Designs</td><td>Foundation/footing design components include:
Obtained a composite soil profile with soil parameters.
Footing sizes determined based on analysis of bearing capacity and settlement.
Relevant design checks for reinforced concrete elements</td><td>Bearing capacity for shallow foundation spreadsheet
Settlement analysis of shallow foundation (classical/Schmertmann method)
Foundation Design 2nd Edition (Coduto, 2000)</td></tr>
<tr><td>Connections</td><td>Perform connection analysis on the two selected designs for structural steel and reinforced concrete:
Chosen steel design using either bolted or welded connection:
a) Simple shear connection/ moment connection
Chosen concrete design connection</td><td>AISC Steel Construction Manual 14th Edition
Structural Steel Design5th Edition (McCormac, 2012)
ACI 318:Building Code Requirements for Structural Concrete and Commentary
Design of Concrete Structures 14th Edition (Nilson, 2010)</td></tr>
</table>

		• *ASCE 7 2010 Edition*
Compared and Contrasted the chosen concrete design vs. chosen steel design	Evaluated both methods by taking the following into account: labor, cost analysis, duration of construction, procurement of materials, design process.	• *RSMean Building Construction Cost Data: 70th Annual Edition* • *RSMean Building Square Foot Cost Data* • Various other sources that contain pertinent information regarding each topic
Deliverables	• Design of footing • *AutoCAD* drawing of office building • Engineering drawings (steel bay, beam→ girder connections, etc.)	• *AutoCAD* • *REVIT Architecture* • Worcester, MA zoning regulations
Other impacts of project:	• Sustainability addressed through research of various types of concrete mixtures (fly ash and silica fume). • Health and safety impacts addressed through the structural adherence to the codes denoted in the Worcester, MA zoning regulations and *ASCE-7*.	• Worcester, MA Zoning Regulations • *ASCE-7 2010 edition.*

3.1: Overview of Structural Analysis and Design

For steel construction, the proposed three-story office building was designed using Load and Resistance Factor Design (LRFD), an acceptable AISC specification method for designing steel structural members and their connections. The steel design for the office building consisted of two base designs: one is a steel bay with rolled steel beams, and the second relies on open-web steel joists to support the roof and floor deck. For reinforced concrete construction, there were also two base designs: one being one-way concrete joists and the other one-way slab-and-beam. The LRFD approach was also used for these concrete designs. When performing structural analysis on the structure, both steel designs and both concrete designs took into account gravity loads and lateral loads for the proposed office building.

3.1.1: Gravity Loads

The gravity load analysis of a given structure takes into account all the forces acting in the vertical plane of the structure. The considered loads are anticipated dead loads, design live loads, and other applicable loads for a structure located in the New England area. The relevant LRFD load combinations used with steel design, based on *AISC* specification, were considered to apply uniform loading to a simply supported beam analysis. With concrete, factors of safety were obtained from *ACI 318-11* and adjusted for the same anticipated loads.

The gravity load analysis started with the design of a typical steel beam. A design capacity load was obtained based on the factored loads, where a reasonable beam size was determined. To check acceptability the beam chosen should have a capacity load that is greater than target load. Then the design of a typical steel girder is performed in a similar manner taking into account the dead loads due to the beams that the girders support. Once the beam and girder sizes were obtained, the structural design for columns was completed by considering the load converged by the girders, beams, and floor and roof decks to the column. To check acceptability the beam, girder, and column size had a capacity load that is greater than target load. The design took into account shored and unshored construction. Shored construction involves using temporary supports to support steel beam and concrete until acceptable curing, while unshored relies solely on the steel beam to sustain the construction loads.

With concrete, beam depths based on *ACI 318-11* specifications were chosen and the amount of required reinforced steel was calculated to support the load. The beams were also subjected to a simply-supported analysis. The floor-to-ceiling height was accounted for when deciding a slab thickness. A similar beam-to-girder-to-column approach as in steel design was used. The gravity load analysis takes into account shear and moment capacity checks.These checks determined how much rebar was put into the beam to help carry loads.

3.1.2: Lateral Loads

The lateral load analysis for structural design takes into account all the horizontal forces acting on the structure including wind and seismic loads. The lateral load analysis includes lateral deflection (drift) checks for each floor and combined flexural and axial force checks based on *AISC* interaction equation for steel and *ACI* specifications for concrete.

RISA was also used to examine the lateral loads and the effects that they have on the concrete structure similar to its applications to the steel design. With the frame of the structure created on the program, the modulus of elasticity and the moments of inertia values need to be adjusted accordingly for each slab and column section that was entered into the software. The corresponding story forces due to wind and seismic loads were applied to each model and results were obtained from a comprehensive analysis.

3.1.3: Connections

The analysis of connections for the structural steel option looked at the various types of structural connections available to connect the building components. Some common connections that were considered are simple shear connections, moment connections, and other connections that are applicable to the structure. For this project, simple shear connections were used. The design of the framing connections were done using both bolted and welded connections between the structural elements, each of which consist of multiple design checks based on AISC specifications.

The analysis of connections for concrete structures was done using a typical connection between an interior girder and outer column on each floor. To complete this connection, the interior girders included steel reinforcing on the top of the section to resist tensile forces due to negative moments acting at the connection area. The hook development length of the rebar was used to complete shear and moment checks for the connection. The connections only considered gravity loads because the connected members were not part of the lateral load-resisting system. These factors are important in checking whether the shear forces generated within the connection are smaller than the allowable shear stress of the reinforcing steel. Stirrups were also included to tie the reinforcing steel of the girder into that of the column. Interior connections are more complicated compared to exterior connections in that there are two sets of girders intersecting at a column, which makes shear and moment checks more difficult (Nilson).

3.1.4: Foundations Design

The foundation design began with analyzing the boring log obtained from a local geotechnical engineering firm. When designing the foundation for each structure, the geotechnical design took into account the governing soil layer's bearing capacity and settlement. The geotechnical design was done using a shallow foundation based on Allowable Stress Design

(ASD), an approved ASCE method. The bearing capacity of the soil considered the shear forces and compressive forces that the structure transmits to the soil. The footings of the foundation were placed on the soil layer that was strong enough to resist the forces acting on the soil. The settlement analysis dealt with how much the soil settles into the ground when the foundation extends beyond placement of building usage. The total settlement was less than the allowable settlement. The calculations for each foundation design were done using spreadsheets that used the specifications of the soil denoted on the boring log as well as other assumed parameters discussed in Chapter 5.

3.2: Overview of Cost Analysis

The *RSMeans Building Construction Cost Analysis* books were used to estimate the cost of structural frame and foundation. Estimates for structural steel were made by linear foot of steel, while concrete estimates were made by volume per cubic yard. Cost data for material fabrication, erection and the price of the material itself are denoted in the books. The cost of fabrication for steel, mixing for concrete, and the labor required for each material, are also of considerable importance. The initial cost analyses only considered the structural frames and foundations for each of the four designs while the final cost analyses also included enclosure and interior construction costs. Investigating the differences between each cost component for each material led to the identification of the most economical structural design and construction approach for the office building.

3.2.1: Enclosure Costs

The enclosure costs are predominantly related to the exterior material surrounding the structural frame of the building as well as doors and windows. Different exterior materials were evaluated in terms of aesthetic and functional value for the office building. The costs for each material were taken from the *RSMean Square Foot Cost* book. The estimates for each material were given in terms of square foot of floor area. The costs for doors and windows were given in terms of the cost for each door and window separately.

3.2.2: Interior Construction Costs

The interior construction costs are primarily associated with the architectural carpentry and finishes. These encompass lighting, and other architectural finishes. Other important interior costs are the MEP equipment. The cost estimates for all interior components (excluding interior walls) were provided in terms of the square foot of floor area. The interior wall prices were based on each square foot of wall area.

Chapter 4: Consideration of Zoning and Building Layout

The City of Worcester, Massachusetts has a multitude of zoning regulations to which each building is subject to. Thus, the proposed structure must conform to a variety of rules for the building placement on the site as well as the overall size and uses of the building.

4.1: Zoning Regulations

The lot chosen was 137,553 square feet, or 3.16 acres (Tax Map N25). It abuts Foster Street on the east side. The lot has an irregular shape with substantial curvature on the Foster Street side. The land is also flat and is not near any bodies of water that may have an influence on the foundation design. The soil is sufficient for construction, based on the boring log used. Because of the relatively large lot size, on-site parking may be accommodated. Table 4.1 shows the various regulations to which the proposed structure must adhere.

Table 4.1: Pertinent Zoning Regulations

Zoning District	BG 6.0 (Underlying Business District)	Mixed-Use (Overlay)
Minimum Lot Area (SF)	5,000	Same
Minimum Frontage (Linear Feet)	40 per du* (no more than 200)	Same
Front Yard Setbacks Minimum Depth (Linear Feet)	N/A	N/A
Side Yard Setbacks Minimum Depth (Linear Feet)	N/A	N/A
Rear Yard Setbacks Minimum Depth (Linear Feet)	10	Same
Maximum Height (Stories)	N/A	N/A
Maximum Height (Feet)	N/A	May exceed by 20%
Floor to Area Ratio (Maximum)	6 to 1	Same

*du = dwelling unit

The proposed three-story office building is in a general business district, BG-6.As shown in the table, the only mandated setback on the property is a ten-foot rear setback. The frontage must be at least 40 feet from the road but no more than 200 feet to avoid undue restrictions on larger buildings with more than five dwelling units. The floor-to-area ratio, or the ratio of the total square footage of the building to the square footage of the lot, has a maximum of 6:1. The floor space aspect, however, may include parking spaces on-site or in a designated area 1,000 feet away from the facility. Each parking space contributes 600 square feet to the floor space (Zoning Regulations).

The site is also located in the mixed-use overlay district, thus, the structure must abide by additional regulations. Because the three-story building is in this district, it may exceed the height outlined in the underlying district by 20%; however, this regulation does not affect the proposed structure because there is no building height maximum set forth in the district. The additional regulations for building uses do not have considerable effect on the building because the proposed structure is designed for non-residential mixed use spaces (Zoning Regulations).

4.2: Architectural Layout

The footprint of the building is a 50 x 80 ft. rectangle. The interior consists of a hallway flanked by office spaces on each side, with the exception being the first floor, which has a 'greeting' area, or lobby upon entering the front door. The interior layout was inspired by Kaven Hall, a building on the WPI campus, where a central hallway is flanked by classrooms and offices, creating equal allocations of space for each office. The first floor greeting area contains public bathrooms as well as an elevator that accesses all three floors. The mechanical equipment that runs the building and the elevator equipment are housed in a discrete area on the first floor as well. The stairs are located at west end of the hallway. The architectural layout for each design option was done using AutoCAD. Each layout is displayed in Figures 4.1-4.4

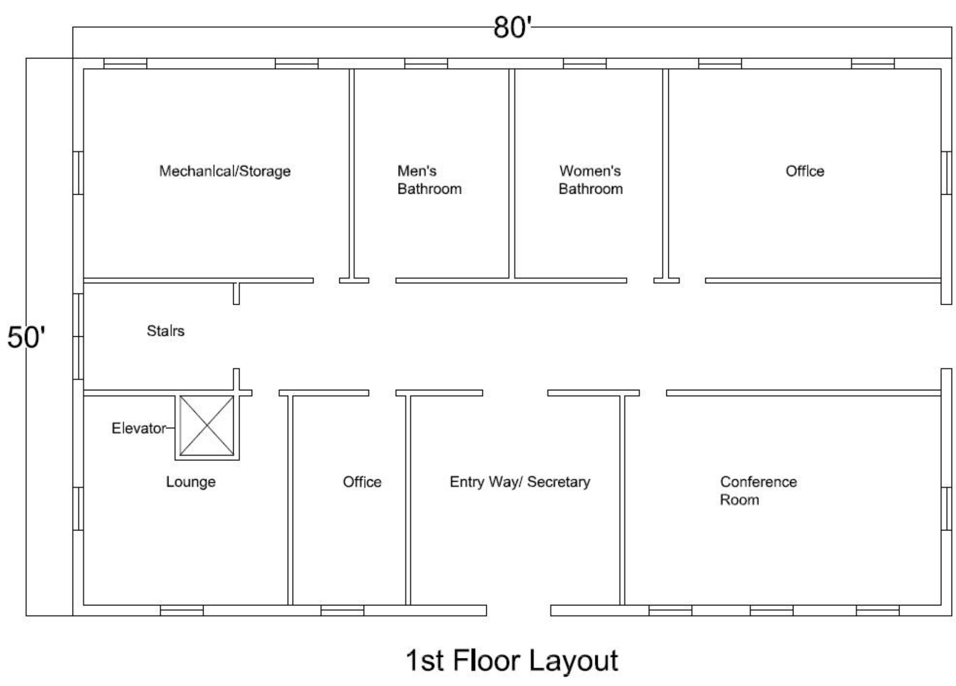


Figure 4.1: Steel Architectural Layout 1

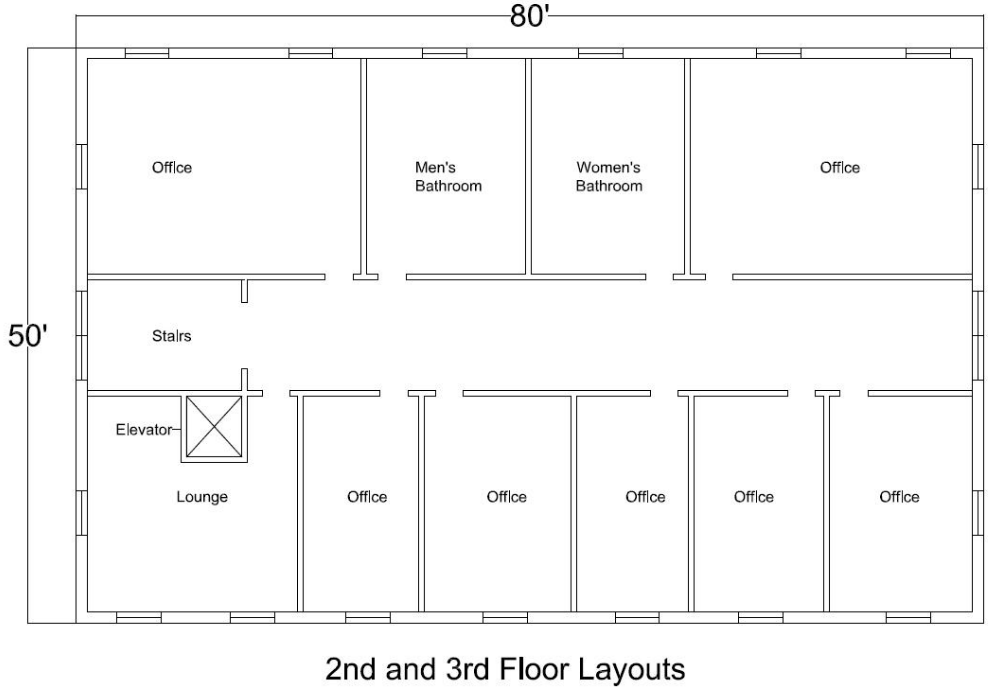


Figure 4.2: Steel Architectural Layout 2

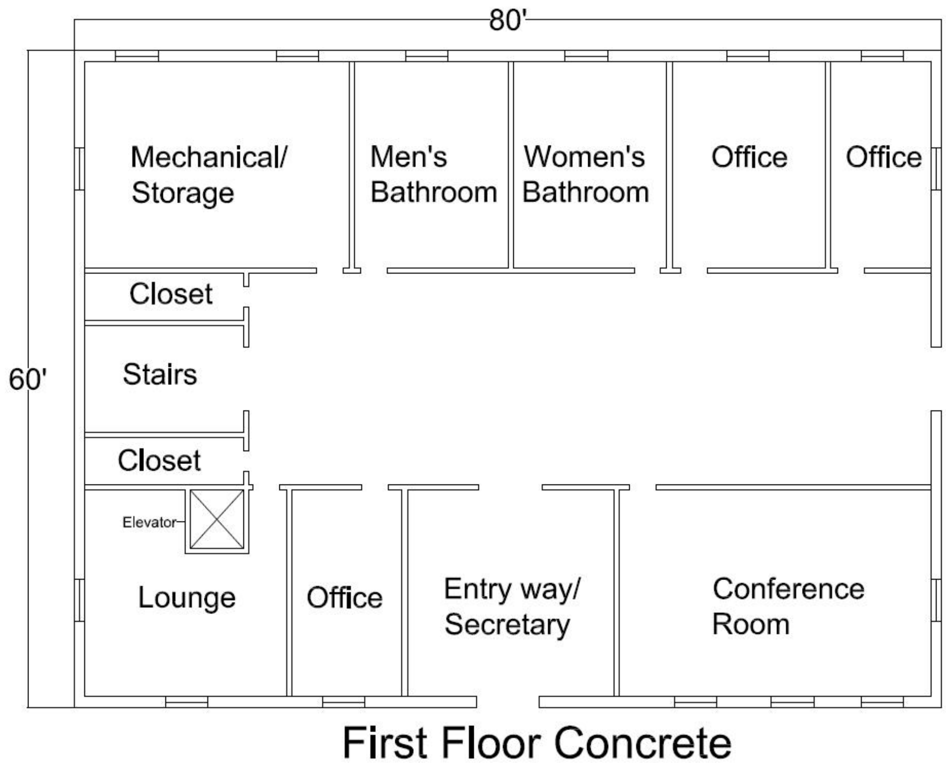


Figure 4.3: Concrete Architectural Layout 1

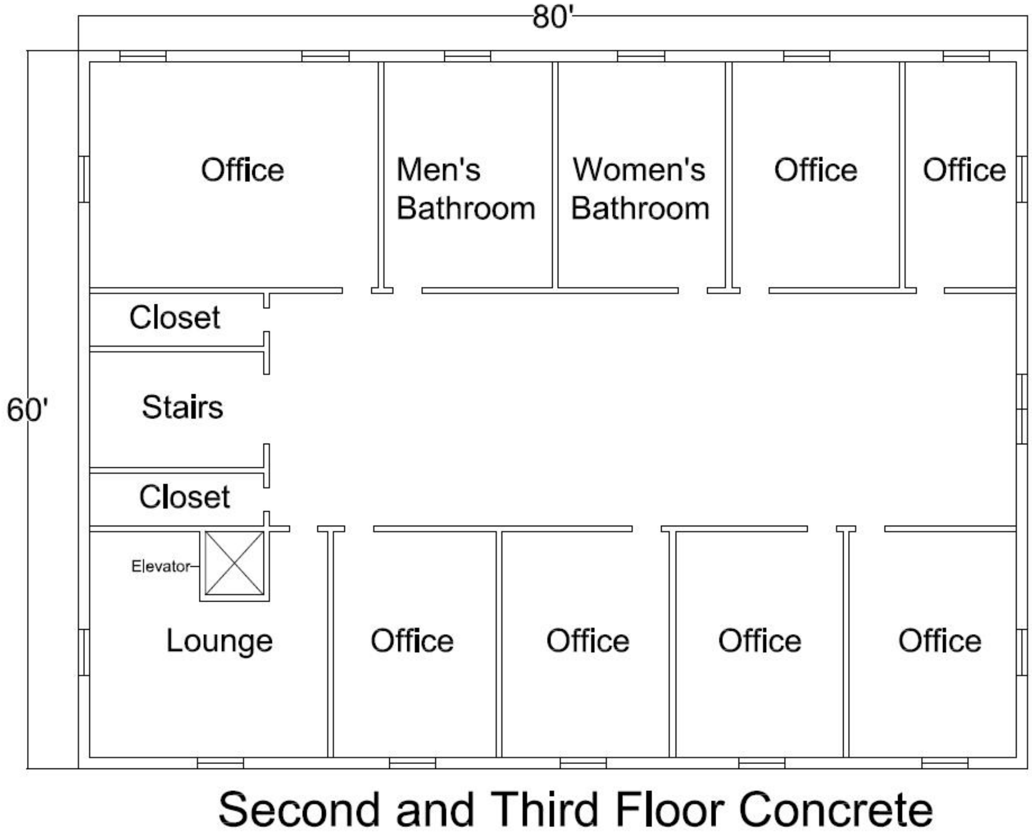


Figure 4.4: Concrete Architectural Layout 1

As shown above, the layout for the reinforced concrete design option has a wider hallway than that of the structural steel design, due to the large size of the building frame. This extra space generated from the larger hallway width creates room for closets that flank the stairs on each floor. The office spaces in the reinforced concrete layout are larger than those of steel, albeit there are fewer offices for this design option.

In terms of the use of office space, it was decided that one company would buy into the building and fit the offices to suit their business needs. This would create a single source of rental income versus multiple sources if the offices were leased to several businesses instead of one.

According to the 7th Edition of the *Massachusetts State Building Code*, the proposed structure is classified in Business Group B (Mass. Building Code, 49). The proposed office building is of Type II, Class A construction, which means the structural frame and roofing materials are non-combustible and the structural frame contains supplementary fire proofing through sprays or covering (Mass. Building Code, 126; Korel). The building is not affected story or height limitations for a Type II, Class A building (Mass. Building Code, 126).

Chapter 5: Consideration of Initial Structural Design Alternatives

The preparation of each structural design alternative involved several key assumptions. Table 5.1 indicates the assumptions made for structural steel and reinforced concrete, respectively.

Table 5.1: Design Properties

Structural Steel Assumptions	Reinforced Concrete Assumptions
Phi Factor (ϕ) = 0.9	Compressive Strength (f'c) = 6 ksi
Yielding Strength F_y = 50 ksi	Unit Weight = 150 lb/ft^3
Modulus of elasticity = 29,000 ksi	Rebar Yielding Strength f_y = 60 ksi

5.1: Structural Steel Design Alternatives

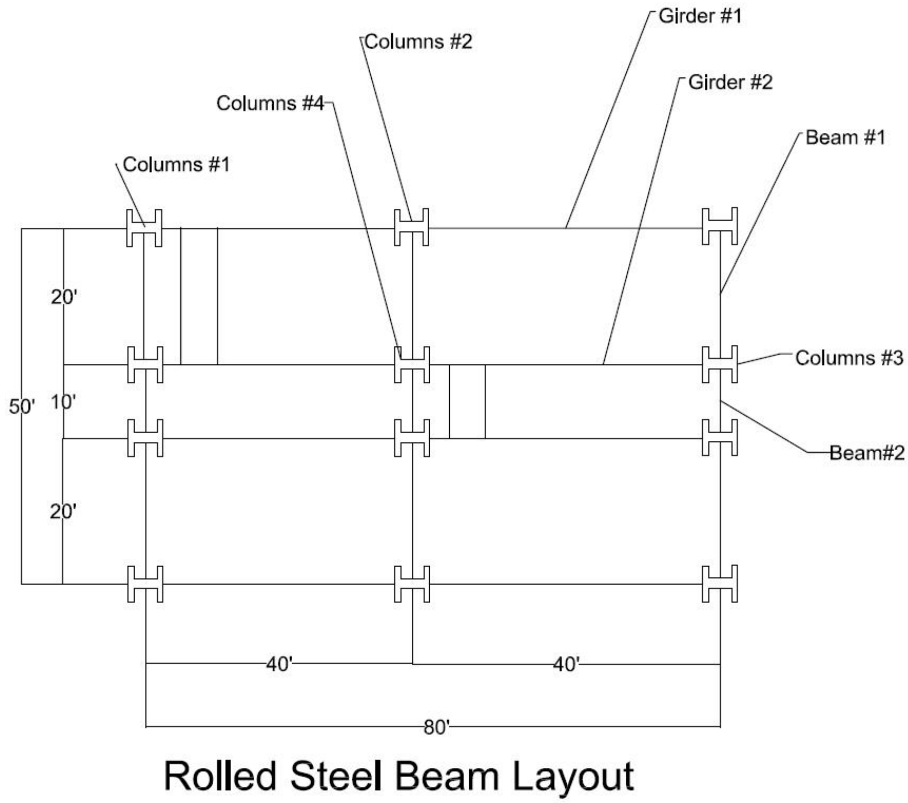
The two structural steel systems explored were a design with rolled steel beams and a design using open-web steel joists. Each design contained a slab on metal decking. Both designs were subjected to a gravity load analysis. The calculations for each design are in Appendix IV.

5.1.1: Rolled Steel Beams

The advantage of using rolled steel beams as a design for this building is the ability to use composite construction. In composite construction, the concrete slab takes a majority of the compressive strength and the steel beams below contribute tensile strength. The slab and beam then work together in making the beam much stronger; allowing the possibility for lighter steel beams as they carry fewer loads. Shear studs must be added to keep the two together, but the cost of these studs is much less then the added price of a significantly heavier steel beam.

After exploring numerous structural layouts, the design in Figure 5.1 was chosen for the floor and roof layouts since it corresponded well with the architectural layout. The corridor contained column bays that were 10 ft. by 40 ft. as well as office bays of 20 ft. by 40 ft. This framing layout provided the ability to split up the live loads of the corridor (80psf) and the office space

(50psf) into separate areas of the design and separate beams. It also provides flexibility in the office construction since tenants can fit out their spaces based on their functional needs. The layout for the rolled steel beam layout is shown in Figure 5.1. The beams within each bay were spaced every 16 ft. The numbers in parentheses are the number of shear studs in each beam. A section detail for the rolled steel beams design is shown in Figure 5.2



Member	Size	Member	Size	Member	Size
Column #1	W10 x 33	Beam #1	Floors: W14 x 22 (32)	Girder #1	Floors: W21 x 48
Column #2	W10 x 33		Roof: W12 x 26 (36)		Roof: W21 x 44
Column #3	W10 x 33	Beam #2	Floors: W10 x 12 (18)	Girder #2	Floors: W21 x 68
Column #4	W 12 x 39		Roof: W10 x 12 (18)		Roof: W21 x 55

Figure 5.1: Layouts of Rolled Steel Beams

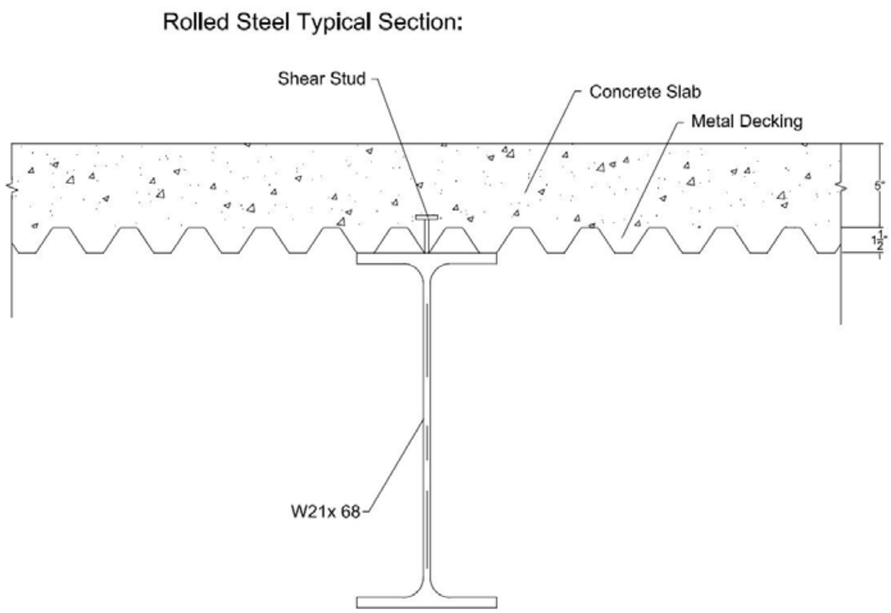


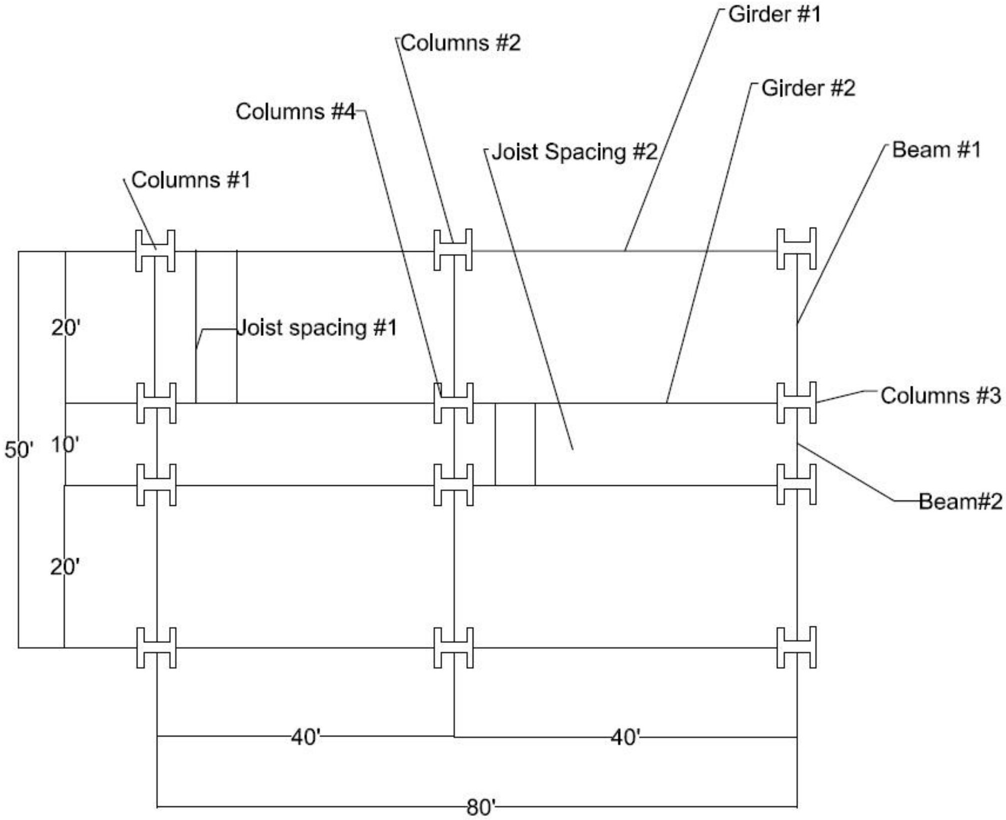
Figure 5.2: Rolled Steel Beams Section

When using the load tables in the *AISC Manual*, an un-braced length of 12 ft. was chosen to be the height of each floor. For the gravity load analysis, a K value of 1.0 was used; thus KL is 12 ft. Specifications for all beams, girders, and columns for this design are shown in Appendix II.

5.1.2: Open-Web Steel Joists

The Steel Joist Institute (SJI) has established three main types of open- web steel joists that a designer can use. Open-web steel joists come in K-Series, LH-Series, and DLH-Series. Each series of joists has advantages and disadvantages based on application, layout of structure, and loading condition. Generally, the K-Series are standard parallel chord trusses with standard depths and used for shorter spans. The LH-Series is tailored towards long spanning joists. The last series DLH-Series is used primarily for deeper joists with significant spans.

After exploring multiple design options, the layout in Figure 5.3 was chosen because the addition of inner girders supporting the hallway and the addition of more columns makes for a more practical structural frame, given the loading conditions. The joists within each bay were spaced every five ft. A typical section for the open-web steel joist design is shown in Figure 5.4.



Open-Web Steel Joist Layout

Member	Size	Member	Size	Member	Size	Member	Size
Column #1	W10x 33	Joist #1	Floors: 14 K 4	Beam #1	Floors: W10x15	Girder #1	Floors: W18x 35
Column #2	W10x 33		Roof:12 K 3		Roof: W10x 15		Roof: W18x 35
Column #3	W10x 33	Joist #2	Floors: 10 K 1	Beam #2	Floors: W10x 12	Girder #2	Floors: W21x 62
Column #4	W 12x 40		Roof:10 K 1		Roof: W10x 12		Roof: W21x 44

Figure 5.3: Steel Joist Layout

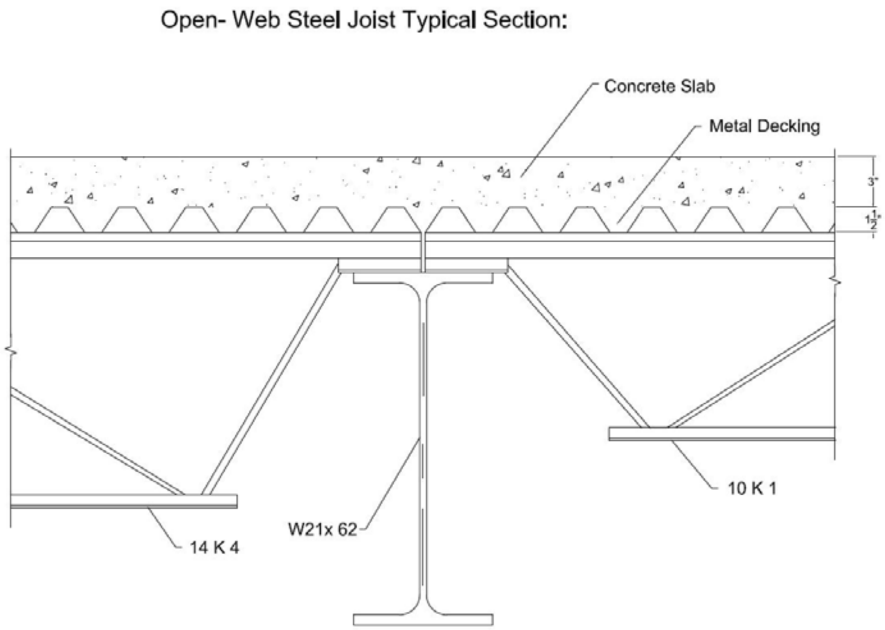


Figure 5.4: Steel Joist Section

Overall, the sizes of K-Series joists and W-shape columns in all aspects of the design are light compared to the associated girder sizes. The girders are the heaviest members in the design, due to large loads they must carry over their long spans. Rolled W-shape sections are placed along the column lines of the design instead of joists to enhance the diaphragm stiffness at each floor level as well as the frame stability during construction. Specifications for all joists, girders, and columns for this design are shown in Appendix II.

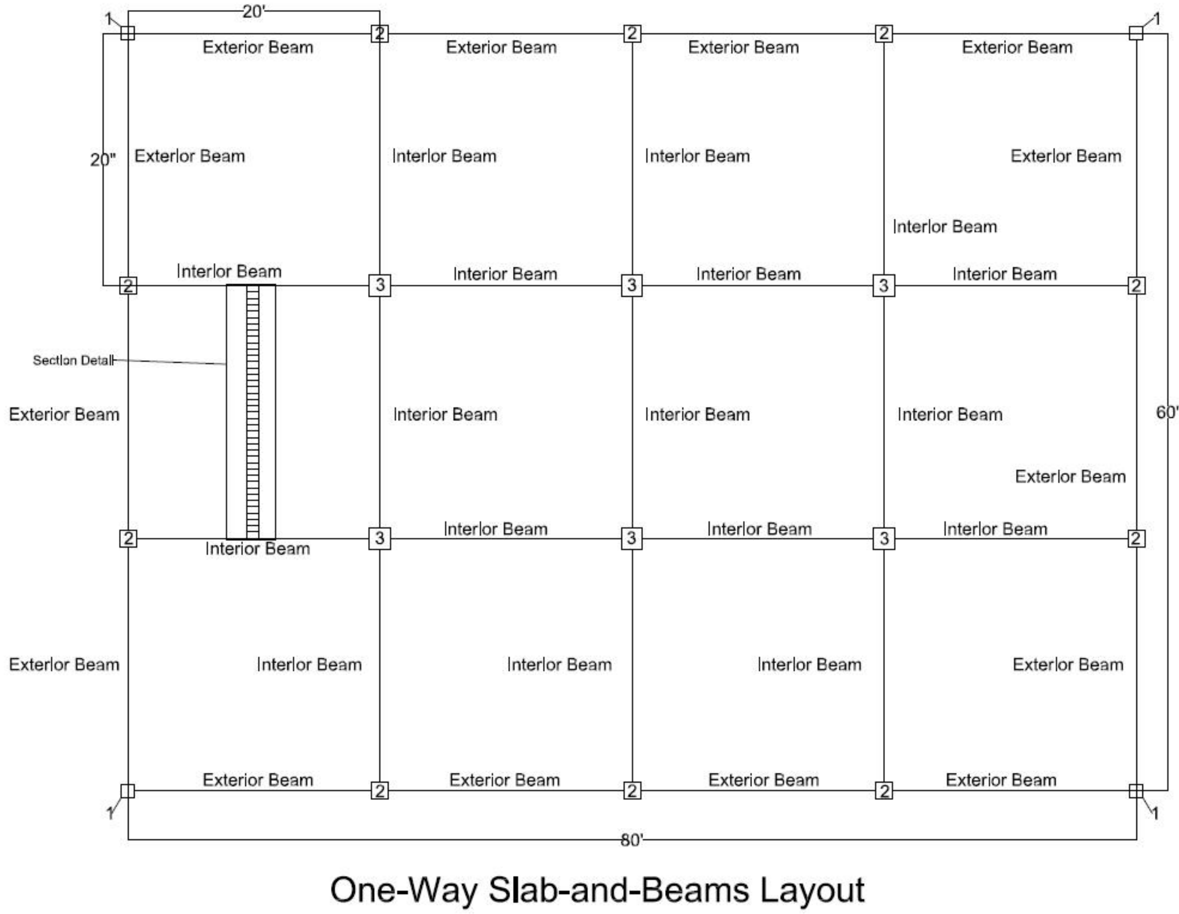
5.2: Reinforced Concrete Design Alternatives

The two reinforced concrete systems explored were a design with one-way slab-and-beams and a design using concrete joists. Each design was done with a one-way slab. Both designs were subjected to a gravity load analysis. The calculations for each design are in Appendix IV.

5.2.1: One-Way Slab-and-Beams

The first concrete method involved the design of a one-way slab with support beams. A one-way slab carries load perpendicular to the direction of the support beams. In this case, the slab includes extra reinforcing beams to carry high loads (Nilson, 424). The slab contained rebar, or steel reinforcement, for tensile forces acting on it.

Multiple one-way slab designs were explored. The one-way slab design in Figure 5.5 proved to be sufficient. These 12 one-way slabs were different from conventional one-way slabs in that each featured beams spanning both directions instead of one. This was done to further reinforce the structure as well as provide continuity in the structural frame. An extra 10 ft. were added to the width of the building to form a slab with one end continuous and uniform bay sizes of 20 ft. by 20 ft. throughout the building. The addition of 10 ft. would make construction of the slab easier as well as provide the added bonus of more office space. The full-size drawings are in Appendix V.



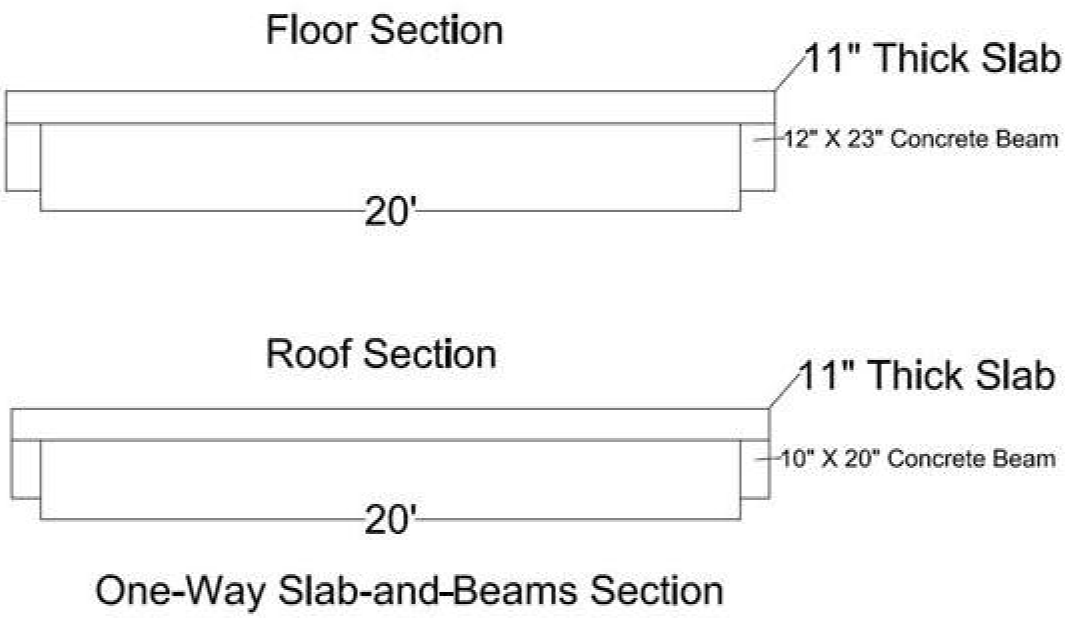
Columns	Size (in)
1	12 X 12
2	16 X 16
3	20 X 20

Floor Beams	Size (in)
Exterior	10 X 20
Interior	12 X 23

Roof Beams	Size (in)
Exterior	9 X 18
Interior	10 X 20

Figure 5.5: Final One-Way Slab Layout

Two slab designs were created: one encompassing the loads of the second floor and the other supporting the roof loads. Because the loading scenarios on the second and third floors of the building are identical, only one typical slab design was required for the floors. A section detail of one 20 ft. span of slab is shown in Figure 5.6.



Slab Rebar Detail:

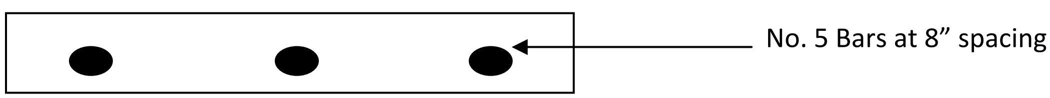
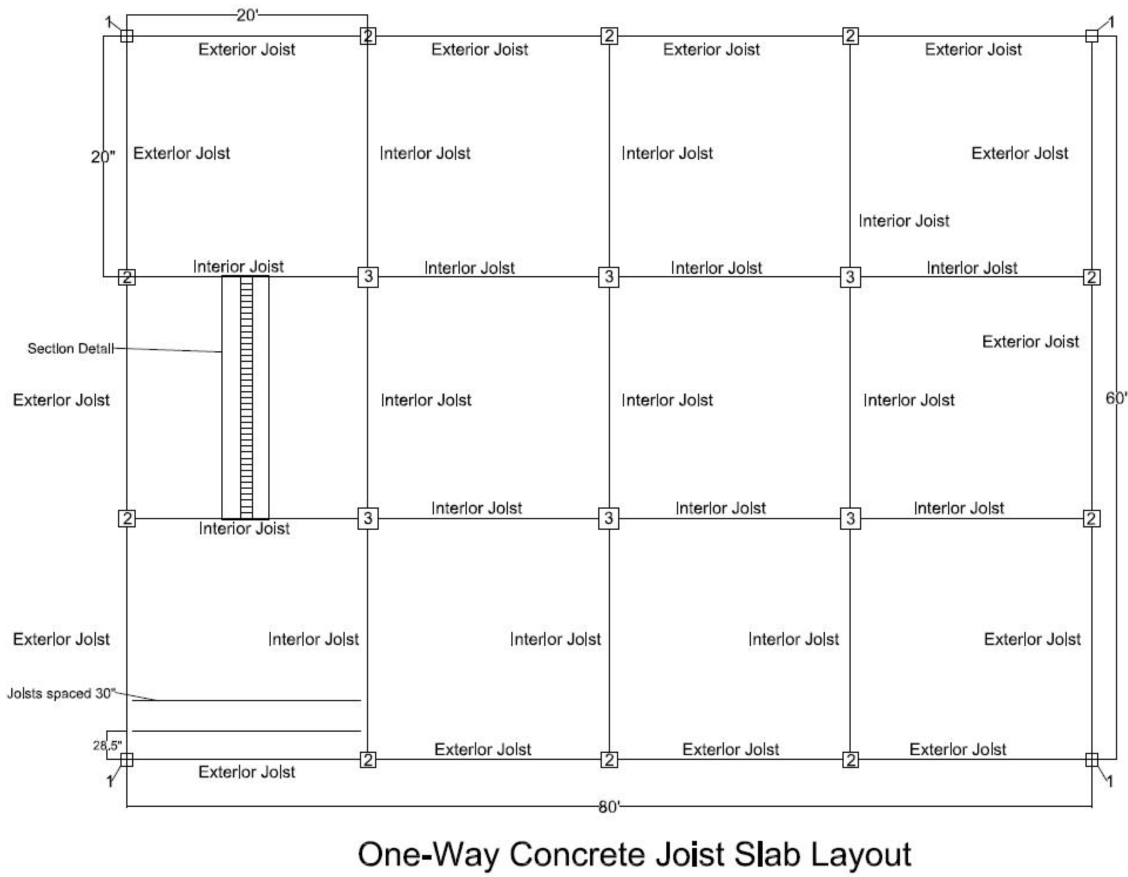


Figure 5.6: Final Slab Section

The slab section above refers to an interior bay, thus the exterior support beams are not shown. The continuous slab has a uniform thickness of 11 inches based on an ACI standard for the minimum thickness of nonprestressed concrete slabs, which makes beam connections to the slab and slab construction easier for contractors. The specifications for each slab, beam, and column are in Appendix II.

5.2.2: Concrete Joist Slab

Prestressed Concrete Inc. (PCI) offers the designer a variety of options for concrete joist span and depth of a particular precast member. The precast concrete allows for less self-weight and uses less reinforcement. The concrete joist standard sizes make it possible to also reuse some of the form work, thus limiting the amount of formwork needed. The joists also allow for thinner slabs, given the typical spacing of joists. The same layout for the one-way slab-and-beams was also chosen for the concrete joist design after various design options were explored. It is shown in Figure 5.7



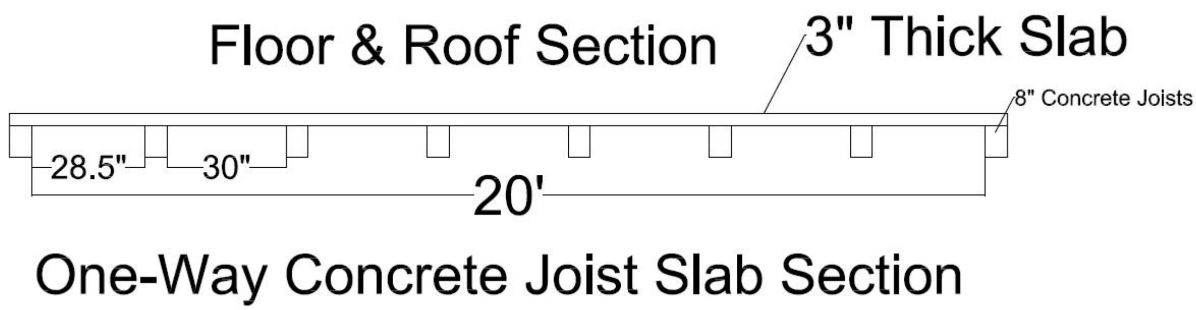
Columns	Size (in)
1	12 X 12
2	14 X 14
3	17 X 17

Joist Size (in)
8 X 5.5 (All)

Figure 5.7: Alternative Layout

This new slab design, shown above, proved to be more sufficient in construction since the uniform thickness makes formwork and slab-to-beam connections easier. The new floor layout design allowed the members to be analyzed using continuous supported analysis. The new dimensions allowed for all slabs spaced 20 ft. apart, which made them continuous. Assuming continuous slabs allowed for a thinner slab thickness with smaller moments.

The new alternative layout makes use of 12, one-way slabs spanning 20 ft. The preliminary calculations for the slab were done by analyzing a one foot continuous section. The analyzed slab section detail is shown in Figure 5.8.



Slab Rebar Detail:

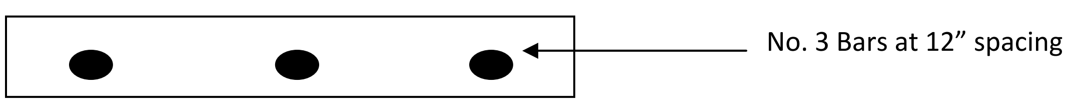


Figure 5.8: Alternative Slab Section

The analysis on the one foot slab section was assumed to be one end continuous for ease of construction, thus designed on the conservative side. The design started by assuming an initial slab thickness based on ACI recommendation for slab thickness as used in the previous design method. The critical moment sections for the slab were analyzed using ACI moment coefficients. Other checks in the design include checking minimum reinforcement requirements for shrinkage and temperature cracking, slab shear capacity, etc. The specifications for each slab, beam, joist, and column are in Appendix II.

5.3: Structural Steel Design Alternatives

Foundations for the office building were designed using a shallow foundation footing based on ASD. A typical foundation design is based on the soil profile from boring logs of the site. Once the soil profile of the site was determined, representative soil parameters were assumed based on the type of soil. The foundation properties gathered from outside sources for design are summarized in Table 5.2. The preliminary foundation designs analyzed the bearing capacity, settlement, and the acceptable column load of the footings.

Table 5.2: Assumed Foundation Properties

Unit weight of bearing layer soil	$\gamma = 115$ lb/ft^3
Undrained shear strength	$S_u = 2000$ lb/ft^2
Effective friction angle	$\phi' = 0^o$
Consolidation properties	$C_c / (1+e_o) = C_r / (1+e_o) = 0.011$

The footings were analyzed based on the similar column groups, given that some columns had the same configuration in both concrete layouts. The preliminary design calculations were done for square footings. The foundation analysis initiated by selecting proper base and length dimensions for the footing based on the loading condition and soil parameters. These parameters were established from studies of the bearing capacity of the soil. The bearing capacity was analyzed using two approaches, Terzaghi's method and Vesic's method.

From the preliminary calculations, the Vesic bearing capacity generally was more conservative due to the nature and number of factors this method considers compared to the Terzaghi bearing capacity. Terzaghi's method assumes a level footing placed on the soil layer, where Vesic's method takes into account footings placed on a slope in the soil. The settlement was then analyzed based on the more conservative bearing capacity. The foundation design assumed a factor of safety of 3.5, which is in the higher range of typical values between 2.5 and 3.5 (Coduto). The value of 3.5 was used given the limited soil information obtained. These factors are based on soil classifications and available site data. The specifications for each foundation design are in Appendix II.

5.4: Initial Cost Analysis

The preliminary cost analysis looked at each material and its cost based on certain parameters specific to it, such as size and quantity. The costs for each material were found in the *RSMeans Construction Cost Data* book. The factors considered for the cost analysis for each material are as follows:

Unit Costs: Steel

- Member costs (beams, girders columns) were based on cost per linear foot all members.
- Open-web Joist costs were based on the depth and size of the joists and were denoted in terms of cost per linear foot.
- Slab costs were based on the price of raw concrete (4000 psi) as well as the labor costs associated with casting. The dimensions of the slab are also included in the final slab cost.

Unit Costs: Concrete

- Member costs (beams, columns, and joists) were based on the price of raw concrete (6000 psi) as well as the labor costs associated with casting.
- Slab costs were based on the price of raw concrete (6000 psi) as well as the labor costs associated with casting. The dimensions of the slab are also included in the final slab cost.

Unit Costs: Foundations

- Footing costs were based on the price of raw concrete (4000 psi) as well as the labor costs associated with casting. The dimensions of the footing and number of footings are also included in the final slab cost.

Shear studs and rebar were not included in the preliminary analysis.

Table 5.4 shows cost estimates for each design method. The total cost for each steel design includes the beams, girders, joists (when applicable), columns, slabs, and footings. The total cost for each concrete design includes the costs of each concrete slab as well as beams, columns, and footings.

Table 5.4: Cost Estimation Data

Method	Rolled Steel Beams	Open-Web Steel Joists	One-Way Slab w/ Beams	Concrete Joist Slab
Structure (S)	$150,153	$125,204	$129,766	$377,452
Foundations ($)	$5,495	$4,288	$10,579	$8,487
Total ($)	$155,648	$129,492	$140,345	385,939
$/SF Floor Area	$13	$11	$12	$32

The more economical steel option is the open-web steel joists while the better concrete option is the one-way slab-and-beams system. The open-web joist system provides open space for the placing of conduits, electrical, piping, etc. for the building, which may have a positive impact on the final cost (McCormac 647). However, it would take longer to build the open-web steel joist design because the steel must be fabricated in a separate facility then shipped to the site. Though one-way slab design option is more expensive, construction could proceed more qickly than that of the open-web steel joist design since the concrete may be cast-in-place on site. The prices shown are marked up 10% to include potential profit for the suppliers and fabricators of each aspect of the structural design. An example of a cost estimation spreadsheet is in Appendix III.

Chapter 6: Further Design Development

6.1: Lateral Load Design

The lateral load analyses for the final designs were done using the program *RISA* to analyze the two frames for the design gravity and lateral loads applied. The analysis for both designs considered a rigid floor diaphragm within both structural bay layouts, which assumes the building moves as one and each structural member experiences the same lateral impact. The wind and seismic loads from each design are shown in Figures 6.1 and 6.2.

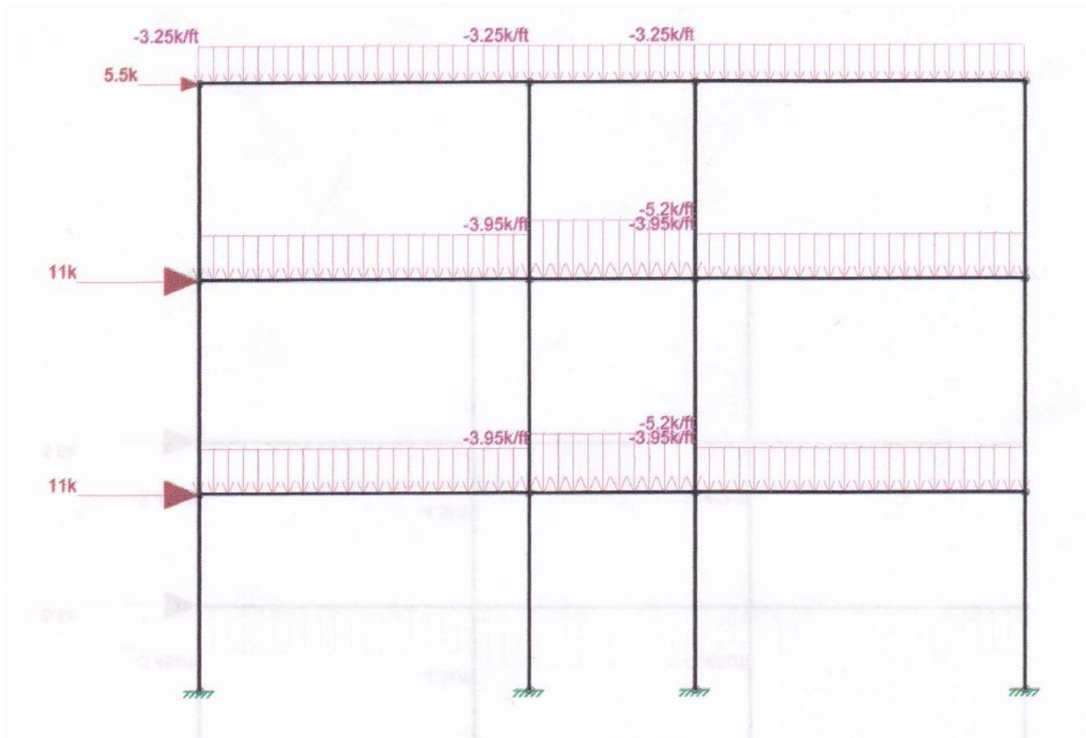


Figure 6.1: Open-Web Steel Joists Lateral Loading

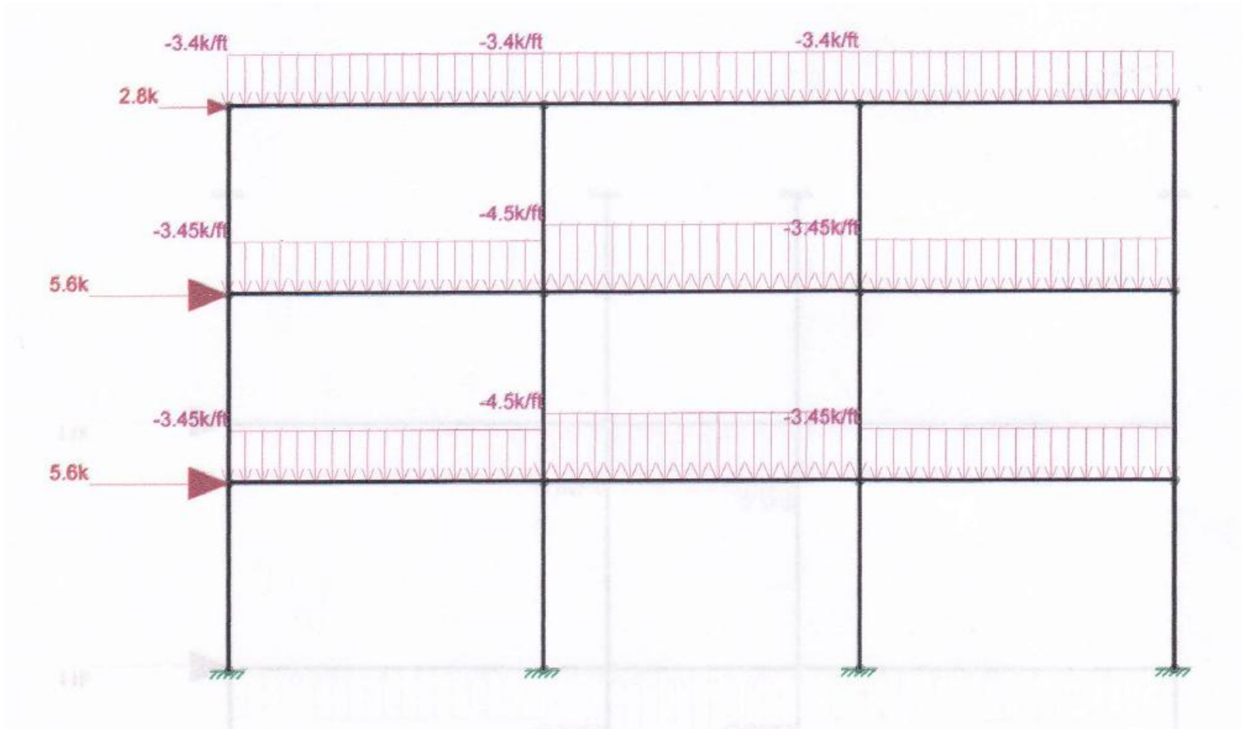


Figure 6.2: One-Way Slab-and-Beams Lateral Loading

Each design was analyzed using the story stiffness method an approximate method for second- order analysis. This second- order analysis attempts to capture the impact of member deformations and frame drift on the forces and moments within the column. The key values associated with each frame, and a few resulting values after they were analyzed are displayed in Table 6.2.

Table 6.2: Lateral Load Analyses Properties

Key Values	Steel Joist Frame	Concrete Frame
Distance Between Frames	40 ft	20 ft
Floor Height	12 ft	12 ft
Axial force (Pr)	16.7 kips	94.78 kips
Flexural force (Mr)	63.74 Kip x ft	37.9 kip x ft
Lateral deflection	2.4 in	0.070 in
P_{story}	58.25 kips	660 kips
Lateral Loads sum	55.1 kips	14.0 kips

The axial and bending forces acting on the concrete frame, as calculated by *RISA*, were then plotted on an interaction diagram for a concrete column. The reason for the high P_{story} value for the concrete design was because the concrete weighed much more than the steel. The layout of the concrete design has larger dimensions than that of the steel design. The interaction diagram indicated that the steel ratio obtained for the columns in the gravity analysis was sufficient given that the new steel ratio falls in the envelope of the original steel ratio. The calculations as well as the interaction diagram are in Appendix IV. The original steel ratio of 3% steel is suitable given the existing loading conditions. In general, the steel ratio for columns is between 2% and 4%, thus the decision to not add more reinforcing steel to the columns was made.

The story stiffness method was used for the steel frame using the results of the *RISA* analysis. The effective length method is being phased out and not used anymore, so the story stiffness method was sufficient. The results for the story stiffness method showed that the W10x33 columns would be satisfactory against the lateral wind loads along with the gravity loads. Calculations for this can be found in Appendix IV.

6.2: Connections

The connections for each structure were done using typical connection details.The open-web steel joists connection looked at welded and bolted connections, while the concrete connection incorporated methodologies focused on the development length of the beam and extending the rebar of each beam to form a stable connection.

6.2.1: Open-Web Steel Joists Connections

A typical connection design for steel joist- to- girder connection uses fillet welds to fasten these members together, whereas a typical column-to- girder connection uses bolts with a single angle or double angle connection. The connection properties assumed for bolt and weld design are summarized in Table 6.2.

Table 6.2: Structural Steel Connection Properties

Fillet welds		Bolts	
Use E70 series electrode	F_{exx} = 70 ksi	Use type A325-N in standard holes	
A36 base metal	F_y =36 ksi F_u = 58 ksi	¾” ϕ bolts A36 base metal	F_y =36 ksi F_u = 58 ksi

The first preliminary design calculations were done for a typical steel joist-to-girder connection using a weld connection, as recommended by SJI. The steel joists would be welded to the top flange of the girder using a fillet weld, which joins two surfaces at a right angle. The connection specifications for a typical steel joist-to-girder connection are displayed in Table 6.3.

Table 6.3: Typical Steel Joist- To- Girder Connection

Location	Connection Designation	Type of Connection	Connection Specification (thickness x required length)
2^{nd} & 3^{rd} Floor	Outer Girder:W18x35 Outer Joist: 14K4	Fillet weld	(2) 1/8” x 2” long weld
2^{nd} & 3^{rd} Floors	Inner Girder: W21x62 Outer Joist: 14K4 Inner Joist: 10K1	Fillet weld	(2) 1/8” x 3” long weld per joist
Roof	Outer Girder: W18x35 Outer Joist: 12K3	Fillet weld	(2) 1/8” x 2” long weld
Roof	Inner Girder: W21x44 Outer Joist: 12K3 Inner Joist: 10K1	Fillet weld	(2) 1/8” x 2” long weld per joist

(n), n represents the number of welds.

A typical steel joist-to-girder connection was analyzed for two cases: one when the girder had to support one steel joist, and second when the girder had to support two steel joists. Both connection cases were considered for the floors and roof of the structure and yielded similar connection specifications. The most effective and lowest cost welding method was to connect the joists to the girders. This is a result of there being many joists and the use of a bolt on each of these connections can be costly. Figures 6.3-6.6 display the bolt and welded connections.

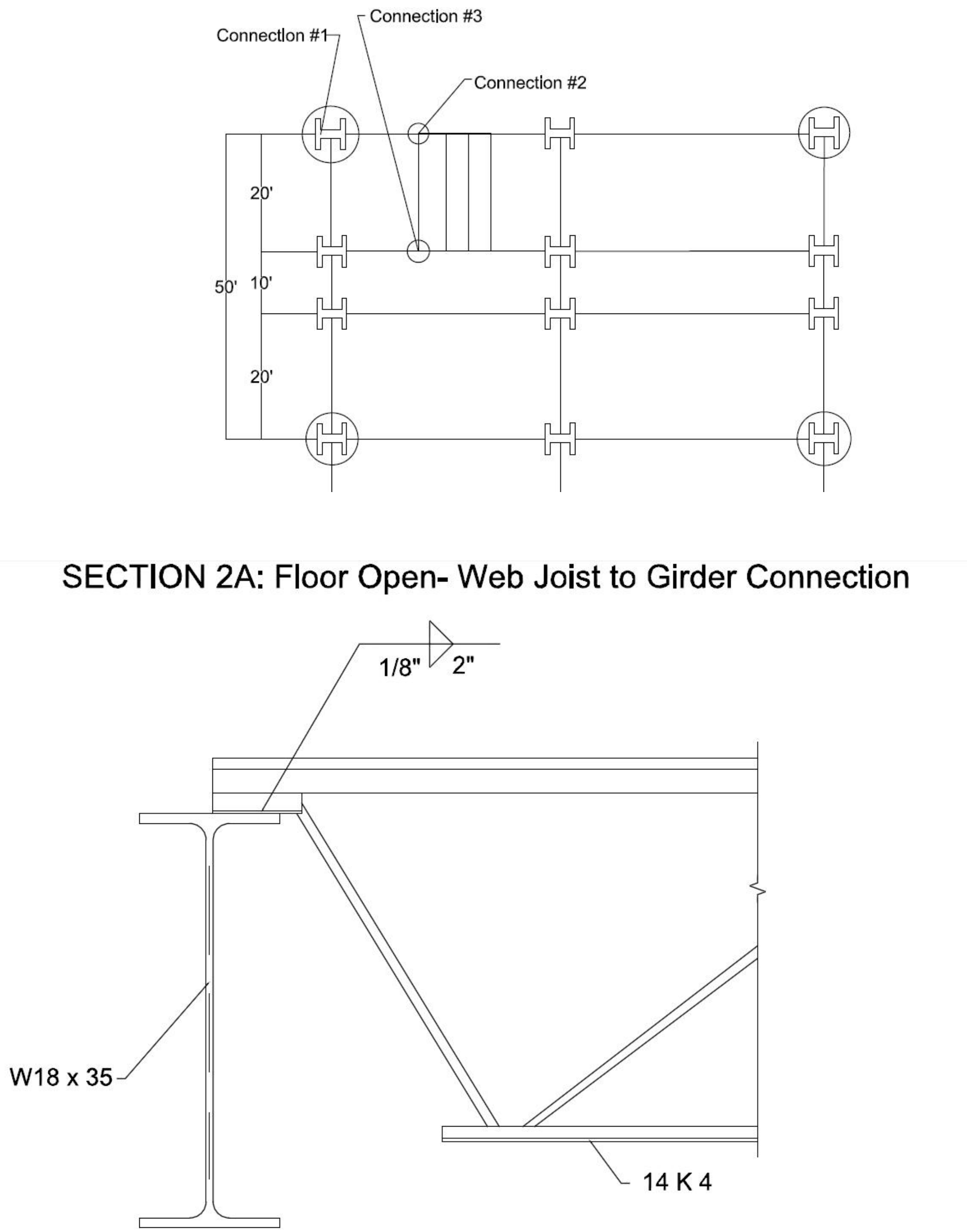


Figure 6.3: Typical Floor Office Connection

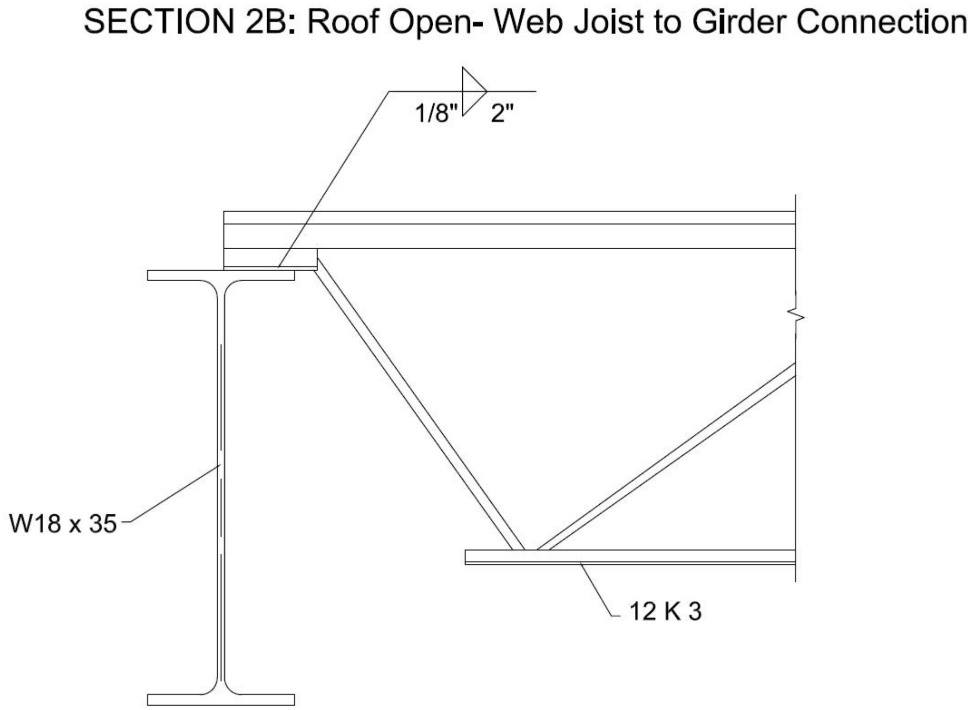


Figure 6.4: Typical Roof Office Connection

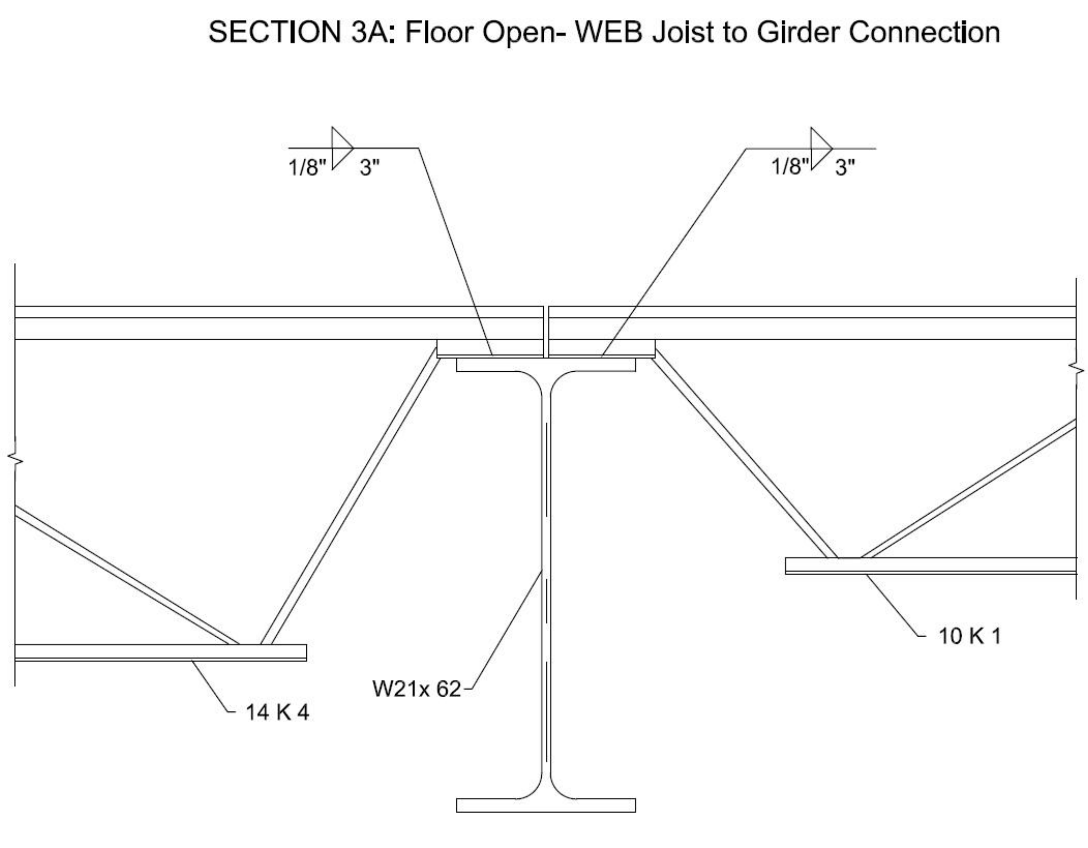


Figure 6.5: Typical Roof Corridor Connection

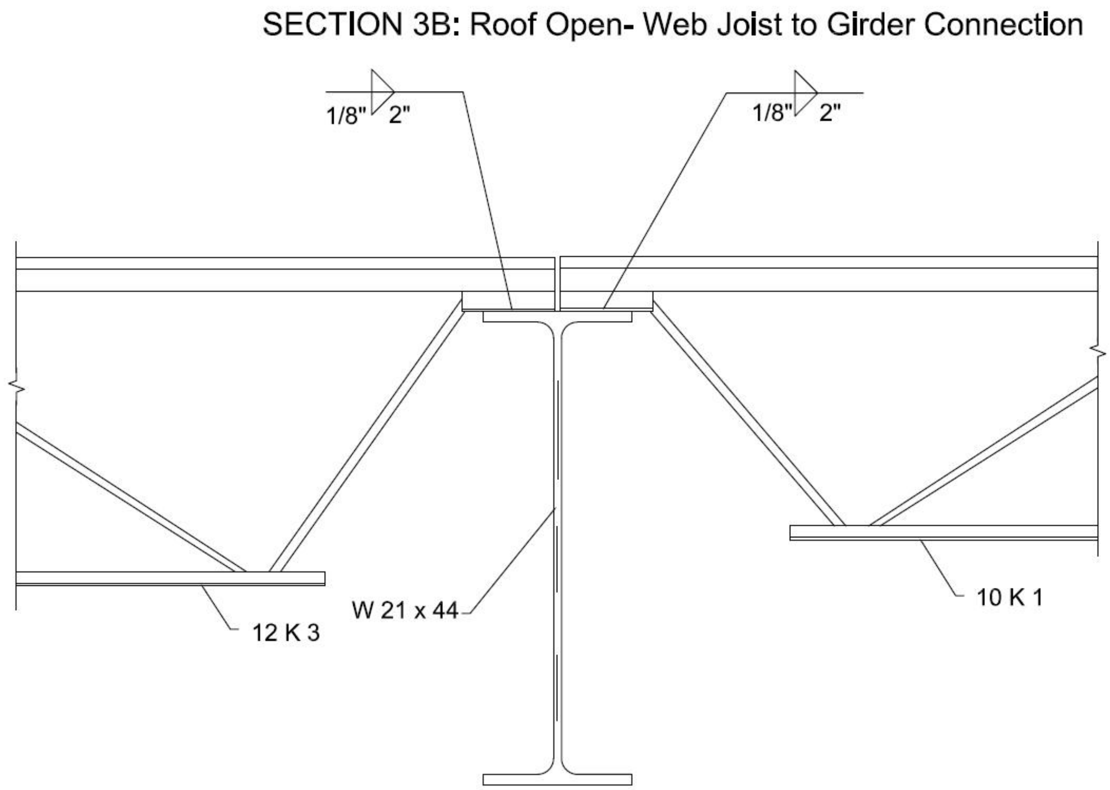


Figure 6.6: Typical Roof Corridor Connection

The interior column connections were not performed due to their complexity in that both two girders and two joists would be connected. These connections would require the use of a gusset plate to secure all the parts and transfer the loads.

For the corner connections, angles were used to connect the girders and the end beams to the column. The final decision was to connect the girder to the flange of the columns because the flanges of the columns are thicker than the web; thus more loads can be supported. It was important to make sure that the width of the angles, which in this case were 3.5”, was not longer than half of the flange width. Diagrams of the connection geometries are shown in Figures 6.7 and 6.8.

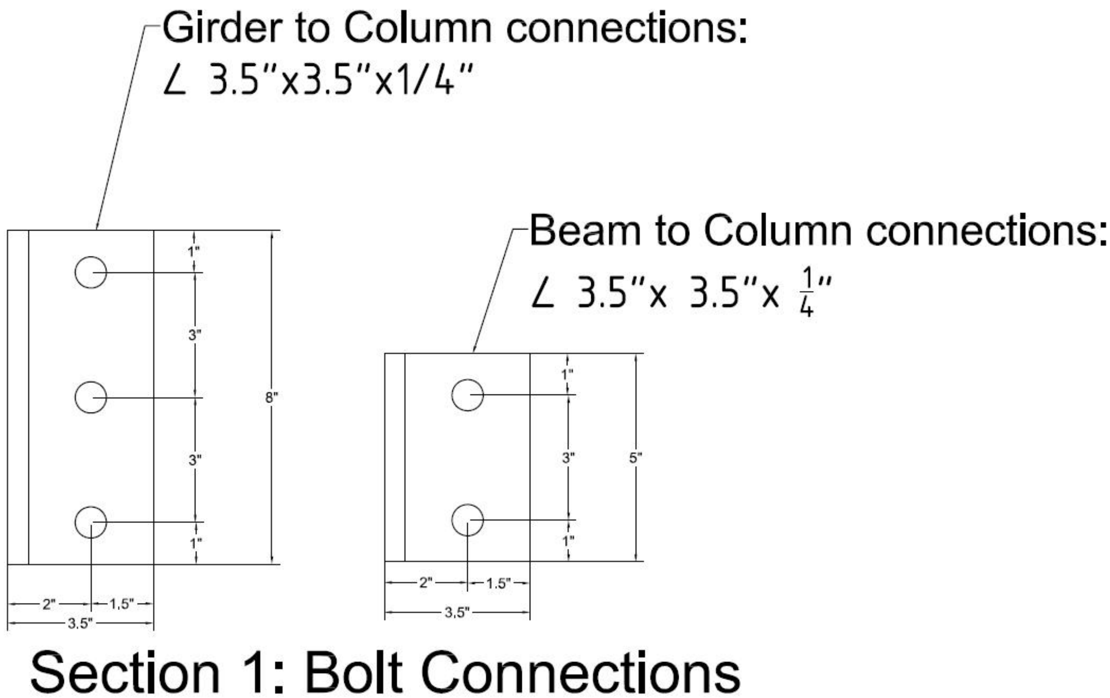


Figure 6.7: Bolt Connections

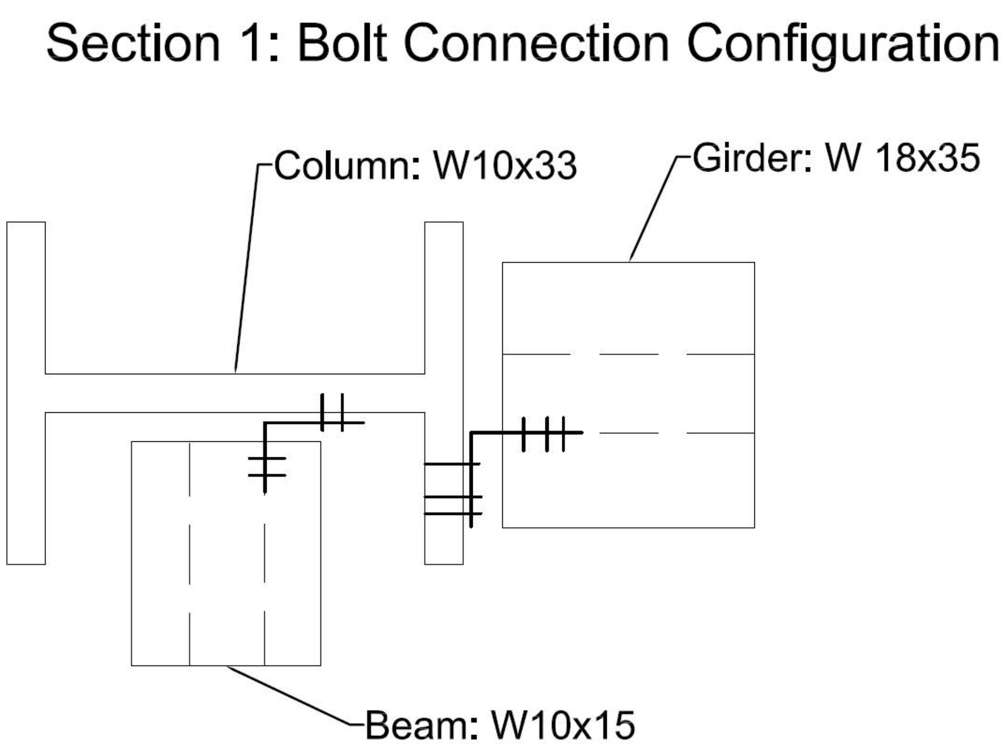


Figure 6.8: Bolt Configuration

The angle sizes and number of bolts for the bolted connections that were designed can be found in Table 6.4.

Table 6.4: Typical Corner Connections

Location	Connection Designation	Connection Size	Number of bolts	Length of Angle
2nd and 3rd Floors	Girder: W18x35 Column: W 10x33	∠ 3.5"x3.5"x0.25"	3	8"
	Beam: W10x15 Column: W 10x33	∠ 3.5"x3.5"x0.25"	2	5"
Roof	Girder: W18x35 Column: W 10x33	∠ 3.5"x3.5"x0.25"	3	8"
	Beam: W10x15 Column: W 10x33	∠ 3.5"x3.5"x0.25"	2	5"

The girder to column connections would have been strong enough using two bolts and the beam to column connections would have been sufficient with one bolt. Bolts were added to guarantee the length of the connection would be within the range of T/2 and T.

6.2.2: One-Way Slab-and-Beams Connections

The connections for the one-way slab-and-beams design only studied gravity loads. Each interior beam incorporated tensile steel to resist negative moments from the columns. Because the width of the interior beam on the roof was less than three-fourths of the width of the column, the joint connection was assumed to be a corner joint while the interior beams had widths that were equal to three-fourths of the column width. These joint connections were classified as exterior joints (Nilson, 358-369).Figure 6.9 displays these connections.

With the addition of tensile reinforcing bars, each beam passed all of the shear and moment checks required for the joint connection. The tensile reinforcing bars of the beams are denoted in the Table 6.5. Detailed sketches of each connection are in Appendix V.

Table 6.5: Concrete Beam-Column Connection Specifications

Connection	Column Sizes (b x h)	Steel Reinforcement	Rebar Location	Floor Location
1	16"x 16"	3 No. 7 Bars	Top	2^{nd},3^{rd}
2	16"x 16"	2 No. 10 Bars	Top	Roof

For corner connections, the edge beams in this design must have reinforcing steel that aids the beam in overcoming the negative moment at each edge column of the structure. The concrete connections are shown in Figure 6.9.

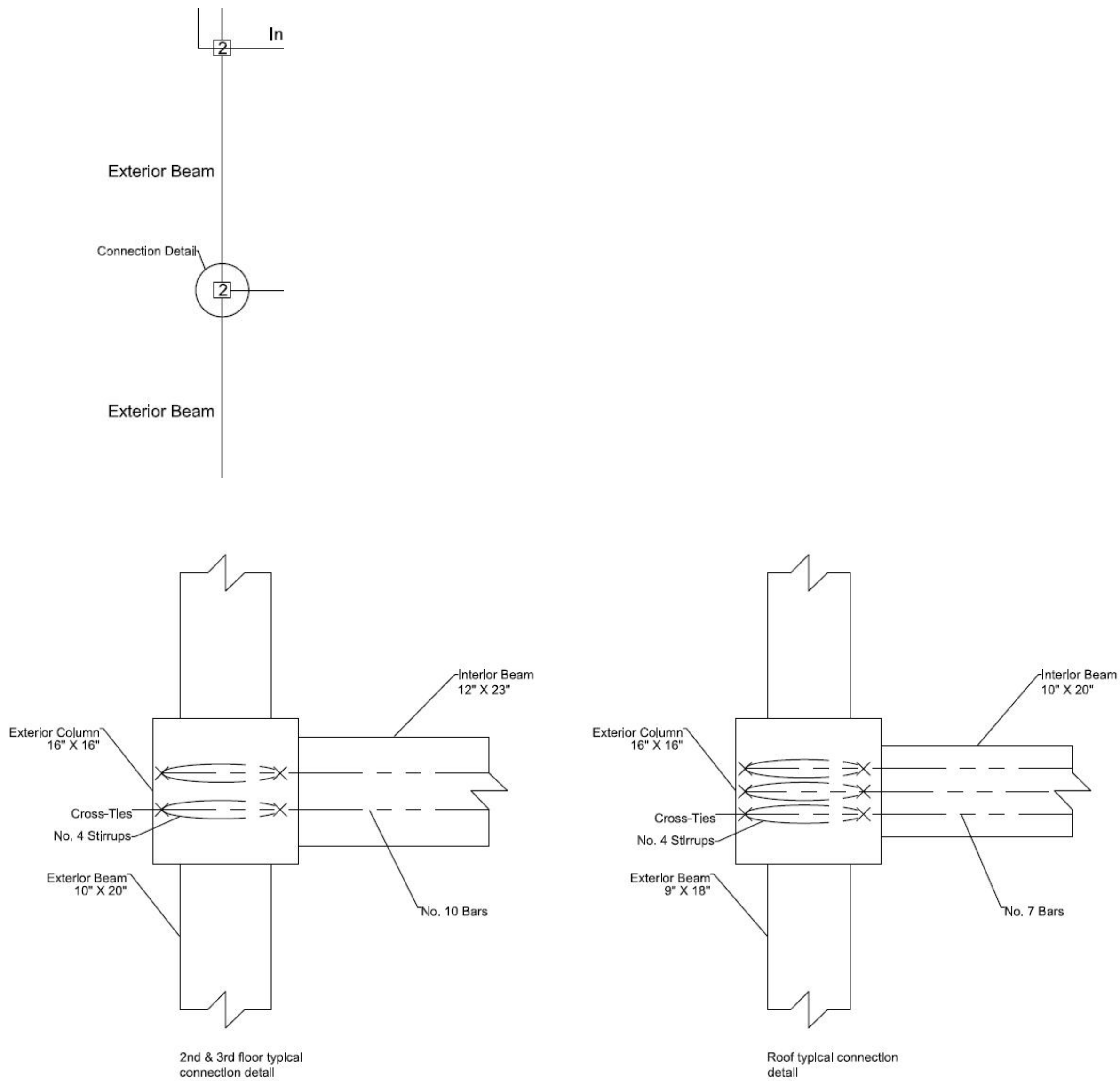


Figure 6.9: Typical Concrete Connection

6.3: Sustainable Concrete Mixes

The incorporation of high volumes of fly ash, a post-industrial waste product, reduces the carbon footprint of the concrete mixture due to the increase in the use of recycled materials. In recent years, as much as 51% of the cement content in concrete mixtures for buildings has been replaced with fly ash. In terms of strength gain and curing, concrete with high volumes of fly ash has been shown to achieve or even exceed the desired strength in 28 days, provided that sufficient vigilance is given to weather and curing conditions (Szecsy). The proposed structure was designed with a conventional concrete mix, however, this alternative HVFA mixture was also analyzed in terms of cost.

According to a study done at the University of California Berkeley, in order for a mixture to be classified as a "High Volume Fly Ash" mix, a minimum of 50% of the cementitious material must be fly ash. If the mix design surpasses the 28-day strength of 30 MPa (around 4,350 psi) and is subjected to freezing and thawing environments, the mixture must include admixtures for water reduction and air entrainment. In terms of mix proportions, the cement content in an HVFA mix is 200 kg/m^3, or 576 lbs (7 bags of cement material) per cubic yard (Mehta).

An alternative concrete mix design for this project has 40% of the cement replaced with fly ash, which is more realistic due to limited testing data available for concrete mixes with more than 40% fly ash. This amount of fly ash would be the equivalent of about three 94-lb bags of cement out of the seven bags typically used (Waier). According to the Portland Cement Association, cement makes up between 10 to 15% of the concrete mix design by volume (PCA). Thus, the 40% HVFA mix design contains about four to six percent fly ash by volume.

If fly ash was incorporated into each mix, the cost per cubic yard of the 4,000 psi concrete would be equal to the initial estimate of $103 per cubic yard and the cost for the 6,000 psi would be reduced from $124 per cubic yard to $123 per cubic yard. Each concrete mix requires an air-entraining admixture, however, according to the study conducted by the University of California Berkeley, the HVFA mix requires high-range water reducing admixture, which would further increase the price.

Furthermore, HVFA concrete may be used as an alternative to reduce the carbon footprint of the building, but the savings, if any, are negligible. This is primarily due to the additional costs of the necessary admixtures needed to improve the workability of the concrete. HVFA concrete has been used in 2005 for the mat foundation slab for a 31-story condominium tower in Dallas, Texas with steel and blue-glass exterior. This building is known as *Azure*. There were no cost increases when the HVFA mix was used on the foundation slab (Szecsy). Based on this case study, a concrete mix with high volumes of fly ash may be a suitable mix for future construction projects

Chapter 7: Final Cost and Scheduling Analysis

The final cost analysis includes the costs of the structural frame and foundations as well as the exterior enclosure and interior construction costs. The interior construction costs include HVAC, mechanical, electrical, and plumbing. The costs of doors and windows are factored into both the exterior enclosure cost and interior construction cost. The exterior and interior costs were obtained from the *RSMeans Square Footage Cost Data* book that contains unit costs for exterior and interior details. The unit cost is based on the average cost per square foot of a component and is applicable to any building of the caliber denoted in that particular section of the book.

7.1: Final Costs

Table 7.1 summarizes the final costs of each component for both structural systems:

Table 7.1: Final Costs

Open-Web Steel Joists		One-Way Slab & Beams	
Structure (S)	145,815	Structure (S)	120,842
Foundations ($)	$3,621	Foundations ($)	$8,122
Exterior (S)	$102,555	Exterior (S)	$106,251
Interior (S)	$929,984	Interior (S)	$1,097,499
Total ($)	$1,181,975	Total ($)	$1,332,714
$/SF Floor Area	$99	$/SF Floor Area	$93

Unlike the cost data in Chapter 5, these prices do not include a 10% markup. This was done to create more realistic construction cost in terms of each component alone. The cost data for the one-way slab-and-beams also includes rebar, unlike the figures in the preliminary analysis. Table 7.2 breaks down the exterior and interior costs for both structural design options.

Table 7.2: Final Exterior and Interior Costs

Open-Web Steel Joists		One-Way Slab-and-Beams	
Exterior Enclosure Costs		Exterior Enclosure Costs	
Brick on w/steel joists	$24,077	Brick on w/concrete frame	$25,901
Exterior Doors	13,264	Exterior Doors	13,264
Exterior Windows	$38,934	Exterior Windows	$35,550
Roofing	$26,280	Roofing	$31,536
Total	$102,555	Total	$106,251
Interior Costs		Interior Costs	
Partition Walls	$4,448	Partition Walls	$5,118
Interior Doors	$36,820	Interior Doors	39,976
Stairs	$25,050	Stairs	$25,050
Floor Finishes	$96,120	Floor Finishes	$115,344
Wall Finishes	$37,706	Wall Finishes	49,123
Ceiling Finishes	$78,840	Ceiling Finishes	$94,608
Elevator	$114,600	Elevator	$114,600
Plumbing	$26,760	Plumbing	$32,112
HVAC	$193,800	HVAC	$232,560
Fire Protection	$51,840	Fire Protection	$62,208
Electrical	$264,000	Electrical	$326,800
Total	$929,984	Total	$1,097,499

As shown above, the cost for the steel design option was determined to be \$99/ft.^{2} while that of the concrete option was \$93/ft.^{2} According to the *RSMeans Square Foot Cost* book, a building of this size for steel should run about \$210/ft.^{2} while that of concrete is \$217/ft.^{2}The reason for the difference in prices is that the cost estimate in this project only covered a limited range of variables whereas the cost estimate in the *RSMeans Square Foot Cost* included more components to the estimate. For example, the foundation costs for this project only included the costs of the footings for a shallow foundation. The true foundation costs include much more than the cost of footings. Another reason for such differences is due to the fact *RSMeans* also considers the contractors fees and architect's fees in their estimates. If the estimates in this project also include such fees, the structural steel option would be \$131/ft.^{2} and that of the concrete option would be \$123/ft.^{2} The distributions of costs are shown in Figures 7.1-7.2 for each design.

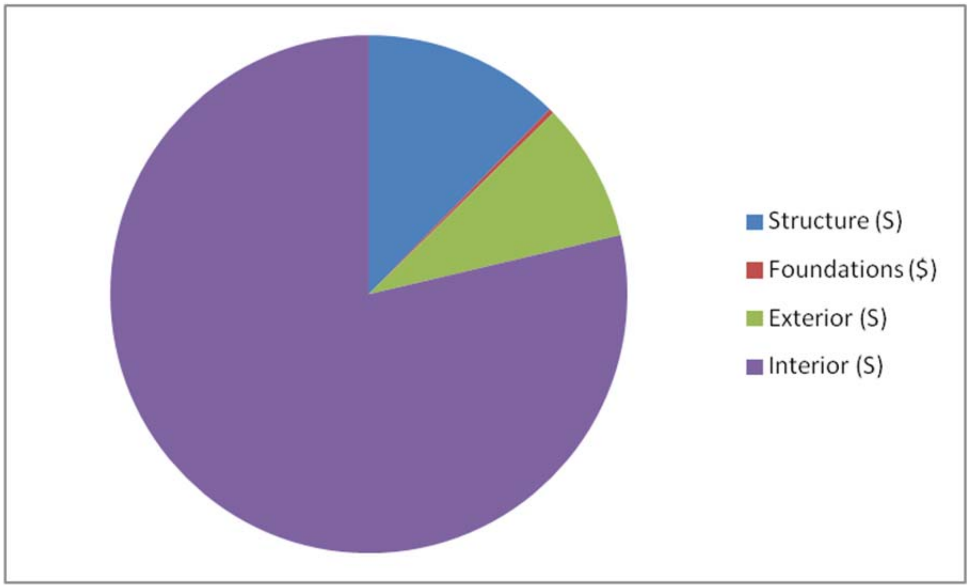


Figure 7.1: Open-Web Steel Joist Cost Distribution

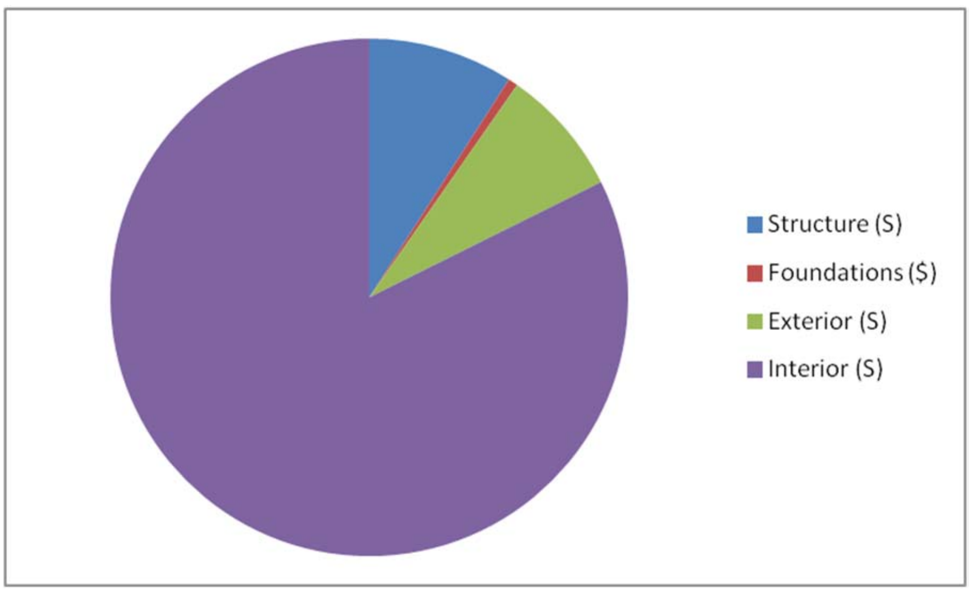


Figure 7.2: One-Way Slab-and-Beams Cost Distribution

As shown in the pie charts, the interior costs account for more than 75% of the cost estimations for each design. The foundation costs have the smallest impact, though design was limited in this regard.

7.2: Scheduling Implications

When deciding between the steel and the concrete building there are aspects to the scheduling that are of considerable importance as well. For example, the construction of cast-in-place concrete buildings may begin immediately once the design has been approved for building. In steel buildings, the steel W-shapes and other steel structural elements need to be fabricated and shipped to the site. The concrete building would be started much sooner than the steel building. However, once the steel structural elements arrive on site, they do not take time to cure as the concrete does. In this instance, the welded connections of joists to girders will already be done.

Another factor to take into account is the weather or season in which the construction will be taking place. The winters in New England make steel construction less costly than that of the concrete structure. The concrete must be heated when it is being poured. It would also have to be monitored so that thermal cracking does not occur during the curing process (Nilson). Because of this, it would be better to delay construction until spring. This negates the early start that concrete would get over the steel building, had it been done in the winter.

7.3: Final Recommendations

Based on the estimates for each design, the open-web steel joist option would be the best option overall in terms of total cost and scheduling implications. Even though the concrete design option has a lower $/ft.^{2} for each aspect, the scheduling and construction implications create difficulties before and during construction. Also, regardless of when construction starts, the steel members would be ordered far in advance to eliminate gaps in construction duration.

Chapter 8: Conclusion

The design of the three-story office building solidified structural design skills in the theoretical and practical realms as well as provided a real-world application where knowledge of multiple aspects of civil engineering, including structural and foundation design and analysis, site planning, zoning regulation abidance, and project management, was cultivated. In terms of the achieving the desired objectives set at the beginning of the project, each objective from the structural design to final cost analysis was met through consistent and persistent effort. The project also assisted in the development of effective collaboration between engineers. Effective collaboration may be defined by cohesive and thoughtful scheduling of activities and approximated deadlines for submittals, appropriate contributions of work by each group member, and cultivation of professionalism within the deliverables produced and the interactions between group members and advisors. The final product was a structurally sound office building that demonstrates effectiveness in form and function. Overall, the project upheld the motto of WPI: “Theory and Practice.”

Initially, this project took on a broader approach by considering four structural design options. Future MQP’s of this scope could take the final design chosen and expand upon them in multiple dimensions. These dimensions include an expanded structural foundation analyses both in terms of structural design and cost as well as enhance the project management aspect by expanding the cost and scheduling analyses. This expansion could include a CPM network diagram for the scheduling aspect and research not only the costs of each component of the design, but also contractor and subcontractor costs. Perhaps the end result could be a bid proposal for the construction of this project.

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Appendices

I: Proposal
II: Tables of Beam, Girder, and Column Specifications for Design Alternatives
III: Foundation and Cost Estimation Spreadsheets
IV: Calculations for Structural Design Alternatives
V: AutoCAD Layouts
VI: Miscellaneous

Project No. LDA-1302

Concrete vs. Steel Design Comparison

MQP Proposal

Justin Furst

Josh Raposo

Carl Haroian

November 12th, 2012

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Abstract

The objective of this MQP is to explore effective designs for a three- story office building. The proposed structure is situated on a section of the land previously occupied by the Galleria Mall. After exploring multiple approaches for each material; steel and concrete, the two most feasible designs for each material are given further evaluation to determine the best approach. The body of work done takes various cost and schedule implications into account.

Introduction

Structural design is an important factor in infrastructure development. There are many materials that may be used in the design of a building, whether it is residential, commercial, or industrial. Commercial buildings, such as office buildings and shopping centers, are primarily designed and built with steel or concrete. The city of Worcester, Massachusetts is currently exhibiting a substantial amount of infrastructure development, namely in building design and construction. One area located in the center of Worcester, known as the Galleria Mall, was recently demolished, and several new office buildings have since been erected. As an alternative option to one of the buildings currently under construction, a three-story office building is proposed. Alternative structural schemes in steel and reinforced concrete are proposed and evaluated. One scheme in each material is selected for further design development and evaluation. The objective of this project is to determine the more feasible approach for the three-story mixed-use building; steel or concrete. The feasibility is addressed in terms of construction cost analysis, which includes labor costs, material procurement and costs for the framing and foundation as well as construction approaches. The interior costs are conducive to the overall cost of the most feasible structural design approach. This proposal also addresses other constraints in other disciplines such as health and safety, sustainability, and environmental impacts.

Background

Before the design of the proposed structured is initiated, a certain number of considerations are contemplated such as the history of the site itself, the zoning regulations, and the dimensional and floor layouts of the proposed office building. The labor and material costs for steel and reinforced concrete as well as supplementary costs, such as architectural finishes, are also considered.

1.1: History of the Galleria Mall Site/Current Development of Site:

The site that was chosen to build the three-story office building is the circled building shown in Figure 1. This is one of the buildings going up to replace the former Worcester Outlet mall also known as the Galleria mall.

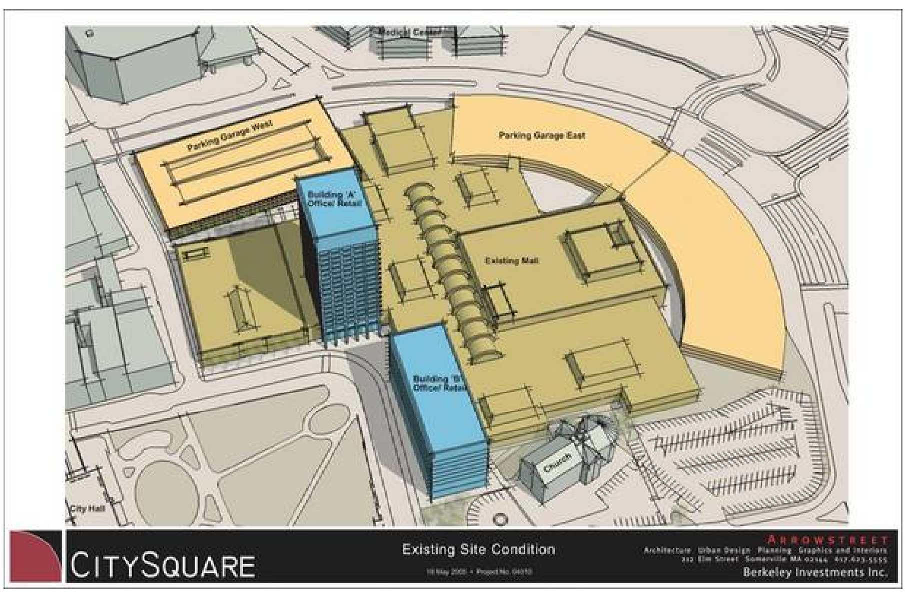

Figure 1: Mall and Parking Garage.http://www.labelscar.com/massachusetts/worcester-common

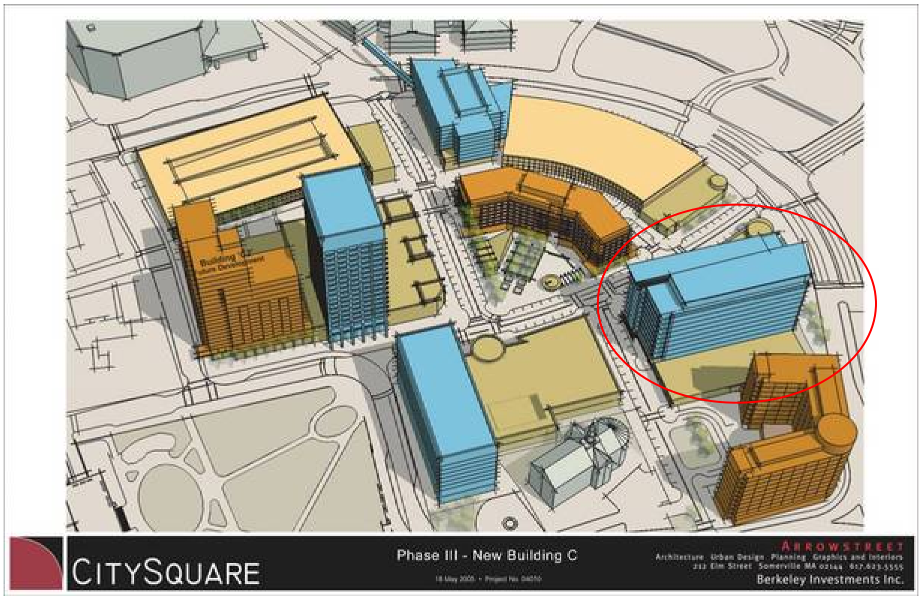

Figure 2: Proposed Layout of Area Replacing Mall.http://www.labelscar.com/massachusetts/worcester-common

The mall, shown in Figure 1, was once built to try and save the failing city of Worcester, Massachusetts,, but is now considered to be "A classic example of an urban renewal project gone wrong, built on an inhuman scale and unkind to its surroundings (Caldor)."

The mall was originally known as the Worcester Center Galleria. It opened in 1971, encompassing 1,000,000 square feet of floor space with a 4,300 car parking garage. At its time, this was the largest parking garage in the world. Within two years, the mall began its decline because the urban element of the mall deterred shoppers and people from the city itself disdained the fact it replaced historical areas of Worcester and blocked most routes to get from downtown to the east side. By the 1990's, the mall lost most of its patronage to suburban malls. It was subsequently purchased and converted into a high-end, more expensive shopping mall, known as the Worcester Common Fashion Outlets. The upscale shopping mall then endured its second major decline, and did not rebound from it (Caldor).

In 2004, Berkley Investments of Boston purchased the lot and razed the mall in order to create a "City Square," as illustrated in Figure 2. The plan was to bring urbanization to the center of the city and also open up the city to allow shorter interurban commutes. Two new office buildings are being constructed in the area; one of which is on the site used for this project (Caldor). Based on the current uses of the shopping area, a three-story office building is being proposed to fit in with the new surrounding buildings.

1.2: Current Zoning Restrictions and Site Dimensions:

The city of Worcester has many mandatory zoning and dimensional requirements for the buildings constructed within its bounds. Thus, the proposed structure must conform to a variety of rules for the building placement on the site as well as the overall size and uses of the building .The table below denotes all relevant rules and regulations.

Table 1: Pertinent Zoning Regulations

Zoning District	BG 6.0 (Underlying Business District)	Mixed-Use (Overlay)
Minimum Lot Area (SF)	5,000	Same
Minimum Frontage (Linear Feet)	40 per du* (no more than 200)	Same
Front Yard Setbacks Minimum Depth (Linear Feet)	N/A	N/A
Side Yard Setbacks Minimum Depth (Linear Feet)	N/A	N/A
Rear Yard Setbacks Minimum Depth (Linear Feet)	10	Same
Maximum Height (Stories)	N/A	N/A
Maximum Height (Feet)	N/A	May exceed by 20%
Floor to Area Ratio (Maximum)	6 to 1	Same

*du = dwelling unit

The proposed three-story office building is in a general business district, BG-6. As shown in the table, the only mandated setback on the property is a ten-foot rear setback. The frontage must be at least forty feet from the road but no more than two hundred feet to avoid undue restrictions on larger buildings with more than five dwelling units. The floor-to-area ratio, or the ratio of the square footage of the building to the square footage of the lot, has a maximum of six to one. The floor space aspect, however, may comprise parking spaces on-site or in a designated area one-thousand feet away from the facility. Each parking space contributes 600 square feet to the floor space (Zoning Regulations).

The site is also located in the mixed-use overlay district, thus, there are some extenuating regulations. Because the three-story building is in this district, it may exceed the height outlined in the underlying district by twenty percent; however, this regulation does not affect the proposed structure because there is no building height maximum set forth in the district. The additional regulations for building uses do not have considerable effect on the building because the proposed structure is designed for non-residential mixed use spaces (Zoning Regulations).

The lot chosen was 137,553 square feet, or 3.1578 acres (Tax Map N25). It abuts Foster Street on the east side. The lot has an irregular shape with substantial curvature on the Foster street side. The land is also flat and is not near any bodies of water that may have an influence on the foundation design. The soil is assumed to be sufficient for construction. Because of the relatively large lot size, on-site parking in a designated area may be accommodated with ease.The orientation of the building is that of the circled building in Figure 2. The proposed structure is three stories high in order to bring the suburban vibe to a densely populated area that initially detracted patronage from shopping at the former Galleria Mall.

1.3: Layout of Proposed Design:

As previously mentioned, the intention of the building is to showcase office spaces available for business purposes. The footprint of the building is a 50-foot by 80-foot rectangle. The interior consists of a hallway flanked by office spaces on each side, with the exception being the first floor, which has a “greeting area” through accessing the front door. The interior layout was inspired by Kaven Hall a building on the WPI campus, where a central hallway is flanked by classrooms and offices, creating equal allocations of space for each office. The first floor consists of a greeting area accessed through the front door as previously mentioned. The greeting area contains public bathrooms for each gender as well as an elevator that accesses all three floors. The mechanical equipment that runs the building and the elevator equipment are housed in a discrete area on the first floor as well. The stairs are located at each end of the hallway. According to the Seventh Edition of the Massachusetts Building Code, the proposed structure is classified in Business Group B (Mass. Building Code, 49).The proposed office building is of Type 1 construction, which means the structural frame and roofing materials are non-combustible (Mass. Building Code, 126; Korel). It is further designated in Class A, denoting the structural frame contains supplementary fire proofing through sprays or covering (Mass. Building Code, 126; Korel). There are no story or height limitations for a Type 1, Class A building (Mass. Building Code, 126).AutoCAD, a computer drafting software, is used to convey the building layout.

1.4: Structural Analysis

The layout of the building is largely influenced by the structural designs for each material. The structural design for each material consists of four major components: gravity loads, lateral loads, connections, and foundation design. Gravity loads involve the selections of beams, girders, and columns to support various types of loads, including dead loads, live loads, and other gravity loads. The subsequent step is designing for various types of lateral loads, including wind loads and seismic loads. Lateral load design consists of further column analysis, which influences the height of the building. The third aspect of structural analysis is the design of connections. The connections hold the components of the structural frame together. There are beam-to-girder connections, beam-to-column connections, and footings for columns. The final aspect is the foundation design, which is what the frame is situated on. All of these components are imperative in achieving a structurally-sound office building.

Steel and concrete each employ different design methodologies for each of the four components. The fundamental differences in each design sequence reveal differences in many facets of cost, which are discussed later. They also reveal differences in multiple aspects of construction, which include labor, material procurement, and scheduling. The most effective design solution is selected base on these factors.

Foundation design begins with selecting the soil profile. As mentioned above, the soil is assumed to be sufficient for construction. Once the soil profile is chosen, the foundation is designed based on the bearing capacity and settlement analysis of the soil. The bearing capacity of the soil takes into accounts the shear forces and compressive forces the structure places onto the soil. The bearing capacity of the soil must be larger than the stresses acting on it or failure will result. Because of this, the footings of the foundation must be placed on the soil layer that is able to resist the forces acting on the soil. Settlement analysis deals with how much the soil settles into the ground when the foundation is poured. The total settlement must be greater than the allowable settlement. The total settlement for an office building is once inch (Coduto). All of these factors affect the total cost of the foundation.

1.5: Cost Analysis:

A cost analysis is done both on steel and concrete designs to determine the more feasible approach for this particular site. The size of each member is directly proportional to the cost of its fabrication, delivery, and labor required for installation. Other obstacles affecting costs during and after construction are also considered. The RSMeans Building construction cost analysis books are used to estimate cost on structures by means of each ton of steel, while concrete estimates are made by volume per cubic yard. Prices for material fabrication, erection and the price of the material itself are denoted in the books.

Architectural details and finishes, including exterior and interior masonry and walls, ceilings, lighting fixtures, door and window treatments, stairs, elevators, and interior carpentry have a considerable impact on the total cost in terms of the quality of each item. HVAC (heating, ventilation, and air conditioning), mechanical, electrical, and plumbing installation are also considered. The exterior walls are influenced by the structural systems, whereas the other items to consider for cost analysis are not dependent on the structural design.

1.6: Sustainable Concrete Mixes

Improved sustainability of the proposed structure may be obtained through the use of alternative concrete mix designs. The incorporation of fly ash, which is essentially a post-industrial waste product, increases the sustainability of the concrete mixture due to the inherent increase in recycled materials. In recent years, as much as 51% of the cement content in concrete mixtures for buildings has been fly ash. In terms of strength gain and curing, Fly Ash concrete has been shown to achieve or even exceed a desired strength at twenty-eight days, provided that sufficient vigilance is given to weather conditions (Szecsy). The proposed structure is designed with a conventional concrete mix, however, this alternative HVFA mixture is also analyzed in terms of cost. The alternative mixture has 40 % Fly Ash due to the price of the Fly Ash.

Capstone Design

The objective of the project is to determine the more feasible building design approach for a three-story office building between steel and concrete. To determine feasibility, the structures are subjected to an array of cost analyses. The cost analyses include labor and material procurement costs, foundation costs, interior costs, and the costs covering the construction process. Initially, four methods of building design are explored; two for each building material. The costs associated with each material are compared in order to select the best option in each material for further design. One of the two final selected designs is then selected as the more feasible option out of all four; the analysis being done in terms of cost as well.

There are several constraints addressed by the proposal. The economic constraints for this proposal are primarily associated with the costs of the structure mentioned above. Sustainability may be increased by utilizing a more eco-friendly concrete mix as well as eco-friendly construction techniques. In terms of constructability, the design of the each structure through each material should be buildable according to the plans and specifications derived from the design work. Each design is done according to the structural building codes presented in ASCE-7, which addresses health and safety concerns.

Methodology/Scope of Work

The flow chart below shows the overall sequence of events that the project will follow to determine the most feasible construction methods for the proposed building structure. Method one with steel for the gravity system will study slab on metal decking. Method two, also using steel will involve open web steel joist. Methods three and four will deal with concrete and are two-way slab and beam and concrete joists, respectively. When analyzing costs of both frames supporting gravity loads, lateral load stability, and interior of building cost, the RSMeans book values will be used. The following table denotes each process in more technical detail.

Figure 3: Methodology Flow Chart.

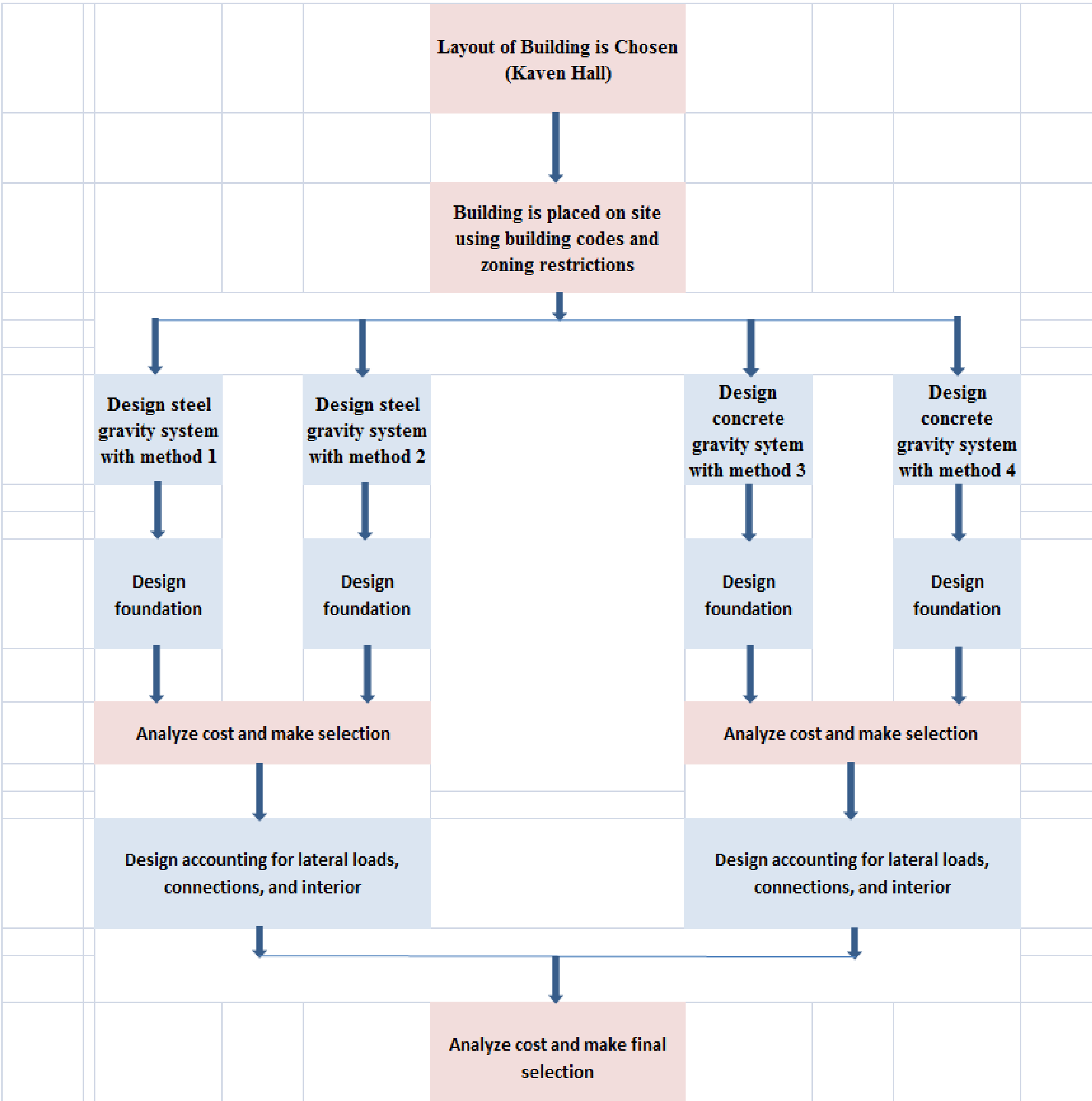

Table 2: Description of Activities

Activities	Process	References
Background information of Galleria Mall site	Research previous development, land use, possible future use of site that current developers are designing	• Article by Caldor regarding the history of the Galleria Mall & future developments
Layout of proposed office building	The office building layout was decided based on a existing functioning building (Kaven Hall) as a; • One hallway down the center with two wings of offices and two staircases at the end	• Kaven Hall layout
Locate office building on Site	Locate the proposed office building on chosen site based on: • Local zoning regulations • Property setbacks	• Worcester, MA zoning regulations
Steel design of office Building	Structural analysis of the steel structure: • Two base designs: c) Steel structure using slab on metal decking d) Steel Structure using open web steel joist • design components include: beams→ girders → columns→ connections→ relevant AISC specification design checks	• Steel Construction Manual 14th Edition • Structural Steel Design textbook 5th Edition • RISA analysis program • ASCE 7 2010 Edition • Steel Joist Institute 43rd Edition Standard Specifications

Reinforced Concrete design of office building	Structural analysis of the reinforced concrete structure: • Two base designs: c) Concrete structure using two- way slab and beam d) Concrete structure using concrete joist • design components include: slabs→ columns→ relevant ACI design checks • Look at alternative concrete mix design	• ACI 318:Building Code Requirements for Structural Concrete and Commentary • Design of ConcreteStructurestextbook 14th Edition • ASCE 7 2010 Edition • Design and Control of Concrete Mixtures text book 14th Edition • RISA analysis program
Foundations for concrete and Steel Designs	Foundation/footing design components include: • Obtaining a composite soil profile with soil parameters. • Footing sizes determined based on analysis of bearing capacity and settlement. • Relevant design checks for reinforced concrete elements	• Bearing Capacity for Shallow Foundation spreadsheet • Settlement Analysis of shallow foundation (classical/Schmertmann method) • Foundation Design textbook 2nd Edition
Connections	Perform connection analysis on both designs: • Chosen steel design using either bolted or welded connection: b) Simple shear connection/ moment connection • Chosen concrete design connection	• Steel Construction Manual 14th Edition • Structural Steel Design textbook 5th Edition • ACI 318:Building Code Requirements for Structural Concrete and Commentary • Design of ConcreteStructurestextbook 14th Edition • ASCE 7 2010 Edition
Compare and Contrast the chosen concrete design vs. chosen steel design	Evaluate both methods by taking the following into account:labor, cost analysis, duration of construction,	• RSMeanscost reference book • Various other books that contain pertinent

	procurement of materials, design process.	information regarding each topic
Deliverables	• Site plan • Foundation Plan • Design of footing • CAD drawing of office building • Engineering drawings (steel bay, beam→ girder connections, etc.)	• AutoCAD • REVIT Architecture • Worcester, MA zoning regulations
Other impacts of project:	• Environmental impacts addressed through research of various types of concrete mixtures (fly ash and silica fume). • Health and safety impacts addressed through the structural adherence to the codes denoted in the Worcester, MA zoning regulations and ASCE-7.	• Worcester, MA Zoning Regulations • ASCE-7 2010 edition.

2.1: Overview of Structural Analysis and Design

For steel design, the proposed three-story office building will be designed using Load and Resistance Factor Design (LRFD), an acceptable AISC (American Institute of Steel Construction) specification method for designing steel structural members and their connections. The steel design for the office building will consist of two base designs; one is a steel bay with a concrete slab on metal decking, and the second is open-web steel joists. For concrete design, there will also be two base designs: one being concrete joists and the other two-way slab and beam. Specifications for designing with concrete are from the ACI (American Concrete Institute) 318 design specification. The LRFD approach will also be used for these concrete designs. When performing structural analysis on the structure, both steel designs and both concrete designs take into account gravity loads and lateral loads for the proposed office building.

2.1.1: Gravity Loads

The gravity load analysis on a given structure takes into account all the forces acting in the vertical plane of the structure. The considered loads are anticipated dead loads, design live loads, and other applicable loads for a structure located in the New England area. The relevant LRFD load combinations used with steel design, based on AISC Specification, will be considered to apply uniform loading as a simple supported beam analysis. The steel members in this building will be designed using steel with the following properties: 50 ksi yield strength and 29,000 ksi modulus of elasticity. With concrete, factors of safety are obtained in ACI 318-11 and adjusted for the same anticipated loads. Properties assumed for the concrete will consist of a compressive strength of 6 ksi and a modulus of elasticity is calculated using ACI equation for modulus based on compressive strength.

The gravity load analysis starts with the design of a typical steel beam. A design capacity load will be obtained based on the factored loads, where a reasonable beam size is determined. To check acceptability the beam chosen should have a capacity load that is greater than target load. Then the design of a typical steel girder is performed in a similar manner taking into account the beams that the girders support. Once the beam and girder sizes are obtained, the structural column design will be performed taking the girders, beams, and floors the column supports. To check acceptability the beam, girder, and column size chosen should each have a capacity load that is greater than target load. The design will take into account shored and unshored construction. Shored construction involves temporarily using supports to support steel beam and concrete until acceptable curing, while unshored relies on the steel beam to sustain construction loads.

With concrete, gravity loads with a beam depth based on ACI 318-11 specifications are chosen based on a range and the amount of required reinforced steel is calculated to support the load. A trial-and-error approach will be used until the structure is at an acceptable use of the concrete versus steel and is not over-designed. The floor-to-ceiling height should also be accounted for when deciding a slab thickness. Similar beam to girder to column approach as in steel design will be performed. The gravity load analysis takes into account the following checks; one is deflection checks due to applicable loads, and other relevant checks that apply.

2.1.2: Lateral Loads:

The lateral load analysis on a given steel and concrete structure takes into account all the horizontal forces acting on the structure including wind, seismic, and other applicable loads. The lateral load analysis takes into account lateral deflection (drift) checks for each floor, combined flexural and axial forces check based on AISC interaction equation for steel and ACI specifications for concrete, and other relevant checks that apply.

The structural analysis program RISA will be used to analyze a steel frame both for gravity loads and lateral loads. The program allows for variations to the steel structure such as choosing the type of supports, connection, and loading conditions acting on the steel structure. This software also allows the user to see the results graphically on a diagram of the effects the axial forces, shear forces, and moment have on the steel structure.

RISA can be used to examine the lateral loads and the effects that they have on the concrete structure similar to how they were used on the steel design. With the frame of the structure created on the program the modulus of elasticity and the moment of inertia needs to be adjusted accordingly for each slab and column as they were for the beams, girders, and columns as in the steel design. With those adjustments, the program works the same way as it does with steel frame

2.1.3: Connections:

The analysis of connections for a steel structure looks at the various types of structural connections available to connect the building together. Some common connections that are considered are; simple shear connections, moment connections, and other connections that are applicable to the structure. A simple shear connection only transfers lateral forces between members and are reasonably priced, while moment connections transfer moment between the connected elements and cost more to use. The design of the framing connections will be done using either bolted connections or welded connections between the structural elements, each of which consist of multiple design checks based on AISC specifications.

The analysis of connections for concrete structures will be done using typical connection details. Those details are further explored throughout the project duration.

2.1.4: Foundations Design:

The foundation design will involve soil analysis of the site of the proposed building and from the data a composite soil profile is developed. The two common types of foundations are shallow foundations and deep foundations. A shallow foundation is usually designed for firm soils or supporting light loads, while a deep foundation is designed for weak soils or supporting heavy loads. Both foundations are considered in the analysis of the soil profile until a definitive choice can be made based on the soil parameters and determined loads acting on the structure. When designing the foundation for a structure, the geotechnical design takes into account the governing soil layer’s bearing capacity and settlement. The geotechnical design is done using Allowable Stress Design (ASD), an approved ASCE method.

Bearing capacity analysis of the soil deals with both the induced compressive stresses and shear stresses from the applied structural loads. If the bearing pressure from the structure exceeds the shear strength of soil, this could result in failure due to bearing capacity. The bearing capacity of a foundation uses two accepted approaches; one approach utilizes the Terzaghi's method formulas, and the other utilizes Vesic' method formulas. Both methods involve using soil parameters and factors specific to each method to calculate bearing capacity.

Terzaghi's method for computing bearing capacity of soils consists of three key assumptions; the depth of the foundation is less than or equal to its width, the bottom of the foundation is rough enough that no sliding occurs between the foundation and the soil, and the soil beneath the foundation is a homogeneous semi-infinite mass (meaning the soil below the foundation extends to a great depth and the soil properties are considered uniform throughout). This method is considered simple and familiar to work with, but the drawback is the method does not consider special cases such as large depth: width ratios of footings or inclined loads.

TheVesic' method, on the other hand, produces more accurate bearing values and applies to more loading and geometry ranges than the Terzaghi's method. The only challenge to the Vesic' method is the increased complexity due to variety of load and geometry conditions it considers in formulas (Coduto).

The settlement analysis of structures on a given foundation is comprised of two types of settlement tests. The first one is total settlement, which is uniform settlement from the structure. The second is differential settlement, which is tilting involved with the settlement, either with or without distortion of the structure. The settlement analysis of a structure also incorporates several approaches; one being the classical method based on Terzaghi's theory of consolidation. This theory assumes the settlement is a one-dimensional process. A one- dimensional process is a plane strain model with only vertical strain. The second approach is the Schmertmann's method, which is based on a physical model of settlement. The physical model is calibrated using empirical data from laboratory tests. This method is generally used with cone penetration test (CPT) results and footings on sandy soils but can be adapted to accommodate other soil test results (Coduto).

2.2: Overview of Cost Analysis

Every aspect of the structural design has a considerable impact on the total cost of the building. The number of beams and girders utilized has a conspicuous impact on the final building cost. As mentioned in the background, estimates for the costs of the structural steel are determined per ton of steel, while estimates for concrete are determined by volume per cubic yard of concrete. The cost of fabrication for steel, mixing for concrete, the transportation of both materials to the site and the labor require for each material, is also of considerable importance. The differences in each cost component for each material lead to the most feasible structural design and construction approach for the office building.

2.2.1: Exterior Costs

The exterior costs are predominantly related to the structural frame of the building. The beams, girders, and columns are all part of the exterior costs. Thus, the layout was chosen to minimize the amount of required beams and girders, thus reducing the overall cost of the building. The exterior building material used over the structural frame also drives up the total building cost, depending on which material is chosen. The prices chosen for each component are based on flat rates that are applicable to many different areas of the United States. Other features such as doors, windows, and other architectural finishes also impact the total cost of the building.

2.2.2: Interior Costs

The interior costs are primarily associated with the architectural carpentry and finishes previously mentioned in the background. Though the exterior costs also encompass doors, windows, lighting, and other architectural finishes, those costs have a much lower total price than do interior costs regarding the same components. The interior costs are primarily influenced by the final structural layout of each material and one architectural layout suited for both materials. The most important interior costs are the mechanical, electrical, and plumbing equipment. The layout of this equipment is determined by the final architectural layout.

The architectural layout of Kaven Hall was chosen because the layout for the mechanical, electrical, and plumbing equipment is less complex than that of other buildings of this caliber. Therefore, installation costs are lower as a result.

Conclusion

The design of the three-story office building solidifies structural design skills in the theoretical and practical realms as well as provides a real-world application where knowledge of multiple aspects of civil engineering, including structural design and analysis, site planning, zoning regulation abidance, and project management, is facilitated. The proposal also assists in the development of effective collaboration between engineers. Effective collaboration may be defined by cohesive and thoughtful scheduling of activities and approximated deadlines for submittals, appropriate contributions of work by each group member, and cultivation of professionalism within the deliverables produced and the interactions between group members and advisors.

The final product will be a well-designed, structurally sound office building that demonstrates effectiveness in form and function. The deliverables will be professional, pragmatic, and proficient. Overall, the project upholds the motto of WPI: “Theory and Practice.”

Table 3: Schedule for B and C terms

Key Time period	What should be done	Assigned Responsibility
Thanksgiving break	• The gravity load designs for all four methods of design finished. • Foundations will be designed or near finished. • Begin Cost analysis with RSMeans.	• Steel: Josh • Concrete: Justin • Foundations: Josh and Justin • Cost analysis: Carl
End of B term	• Cost analysis will be completed and the two cheapest gravity loads will be chosen. • The lateral load systems will be configured in RISA and near completion. • Typical connections for steel design will be designed. • Research done on concrete connections and design for those begun.	• Cost analysis: Carl • Lateral load systems steel with RISA: Justin • Steel Connections: Josh • Research on concrete connections: Carl
Two weeks into C term	• Lateral loads systems finished. • Connections for concrete design finished. • Interior of buildings started.	• Lateral loads for both systems: Justin • Concrete connections: Carl • Interior: Josh
Two weeks left with C term	• Interior buildings finished and deliverables worked on including: REVIT drawing, AutoCAD drawings. • Cost analysis started	• Interior: Josh • REVIT: Justin • AutoCAD: Carl • Final cost analysis: Josh
One week left with C term	• Cost analysis done. • Finalizing Paper	• Final cost analysis: Josh • Paper finalization: All of us

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Rolled Steel Beams Tables:

Beam Specifications

Beam	Beam Size	#studs	Stud spacing
Floor Corridor Beams	W10x12	18	16"
Floor Office Beams	W14x22	32	7.27"
Roof Corridor Beams	W10x12	18	16"
Roof Office Beams	W12x26	36	6.5"

Girder Specifications

Girder	Size
Exterior Girders Floor	W21x48
Interior Girders Floor	W21x68
Exterior Girders Roof	W21x44
Interior Girders Roof	W21x55

Column Specifications

Columns Group	Column size
A	W10x33
B	W10x33
C	W10x33
D	W10x39

Open-Web Steel Joists Tables:

Steel Joist Specifications

Open- Web Steel Joist	Joist Size	Spacing	Span
Floor Corridor Joists	10K1	5’	10’
Floor Office Joists	14K4	5’	20’
Roof Corridor Joists	10K1	5’	10’
Roof Office Joists	12K3	5’	20’

Girder Specifications

Girder	Girder Sizes	Spacing (ft.)	Span
Exterior Floor Girders	W18x35	20’	40’
Interior Floor Girders	W21x62	10’	40’
Exterior Roof Girders	W18x35	20’	40’
Interior Roof Girders	W21x44	10’	40’

Column Specifications

Column Groups	Column Size	KL value (K=1 for Gravity loads)	Location
1	W10x33	12’	Corner
2	W10x33	12’	Mid-End
3	W10x33	12’	Mid-End
4	W12x40	12’	Interior

One-Way Slab-and-Beams Tables:

One-Way Slab Specifications

Slab	Type	Floor (s)	Depth (in)	Tension Steel Bars	Spacing (In)	Tension Steel Area (in^2)
1	One-Way	2nd, 3rd	11	No. 5	8	0.46
2	One-Way	Roof	11	No. 5	8	0.46

Beam Specifications

	Type	Floor (s)	Width (in)	Depth (in)	Tension Rebar	Quantity	Tension Steel Area (in^2)	Shear Steel Area (in^2)	Spacing (in)	Span (ft)
1	Secondary	2nd, 3rd	10	20	No. 7	7	4.2	0.4 (2 stirrups)	4	20
2	Main	2nd, 3rd	12	23	No. 9	6	6	0.4 (2 stirrups)	5	20
3	Secondary	Roof	9	18	No. 7	6	3.6	0.4 (2 stirrups)	4	20
4	Main	Roof	10	20	No. 7	7	4.2	0.4 (2 stirrups)	4	20

Column Specifications

Column Groups	Column Sizes (b x h)	Steel Reinforcement	Location	Design Capacity (K)
1	12”x 12”	6 No. 8 Bars	Corner	139.2
2	16”x 16”	8 No. 9 Bars	Edge	301.9
3	20”x 20”	8 No. 11 Bars	Interior	510.1

Concrete Joist Slab Tables:

Concrete Joist Slab Specifications

One-Way concrete slab	Slab thickness	Steel reinforcement
Floor corridor slab	3''	Bar No.3 spaced 12''
Floor office slab	3''	Bar No.3 spaced 12''
Roof corridor slab	3''	Bar N.3 spaced 12''
Roof Office slab	3''	Bar No.3 spaced 12''

Concrete Joist Specifications

Concrete Joists	Joist Depth	Spacing	Span
Exterior Floor Joists	8" joist	2'-6"	20'
Interior Floor Joists	8" joist	2'-6"	20'
Exterior Roof Joists	8" joist	2'-6"	20'
Interior Roof Joists	8" joist	2'-6"	20'

Table 6.7: Column Design Specifications

Column Groups	Column Sizes (b x h)	Steel Reinforcement	Location	Design Capacity (K)
1	12"x 12"	4 No. 10 Bars	Corner	100.7
2	14"x 14"	4 No. 11 Bars	Edge	163.4
3	17"x 17"	4 No. 14 Bars	Interior	274.5

Foundation for Rolled Steel Beams

Foundation Group	Base Size	Load (Kips)	Bearing capactiy (Terzaghi)	Bearing capacity (Vesic)	Allowable Load kips (Terzaghi)	Allowable Load kips (Vesic)	Settkement (max 1")
A	7'x7'	168	4400 lb/ft^2	4675 lb/ft^2	216	229	0.81"
B	9'x9'	334.6	4400 lb/ft^2	4453 lb/ft^2	356	361	0.94"
C	8'x8'	260.2	4400 lb/ft^2	4550 lb/ft^2	282	291	0.88"
D	11'x11'	518.5	4400 lb/ft^2	4311 lb/ft^2	532	522	1.07"

Foundation for Open-Web Steel Joists

Foundation Group	Base Size	Load (Kips)	Bearing capactiy (Terzaghi)	Bearing capacity (Vesic)	Allowable Load kips (Terzaghi)	Allowable Load kips (Vesic)	Settkement (max 1")
A	6'x6'	136.7	4400 lb/ft^2	4842 lb/ft^2	158	174	0.74"
B	7'x7'	204.5	4400 lb/ft^2	4675 lb/ft^2	216	229	0.81"
C	8'x8'	272	4400 lb/ft^2	4550 lb/ft^2	282	291	0.88"
D	10'x10'	408	4400 lb/ft^2	4375 lb/ft^2	440	437	1.01"

Note:The settlements in red is above the one inch limit, however, the parameters for the calculations, including the factor of safety, are more conservative than other similar analyses.

Foundation for One-Way Concrete Joist Slab

Column Groups	Footing Size (B x L)	Load (lbs)	Allowable Bearing Capacity (Terzaghi) (lb/ft^2)	Allowable Bearing Capacity (Vesic) (lb/ft^2)	Allowable Load (Terzaghi) (lbs)	Allowable Load (Vesic) (lbs)	Settlement (in)
1	6'x 6'	85,906	4,399	4,842	158,000	174,000	0.71
2	7'x 7'	167,672	4,399	4,675	216,000	229,000	0.78
3	9'x 9'	324,158	4,399	4,453	356,000	361,000	0.89

Foundation for One- Way Slab with Beams

Column Groups	Footing Size (B x L)	Load (lbs)	Allowable Bearing Capacity (Terzaghi) (lb/ft^2)	Allowable Bearing Capacity (Vesic) (lb/ft^2)	Allowable Load (Terzaghi) (lbs)	Allowable Load (Vesic) (lbs)	Settlement (in)
1	6'x 6'	131,000	4,399	4,842	158,000	174,000	0.71
2	8' x 8'	265,800	4,399	4,550	282,000	291,000	0.84
3	12'x 12'	534,600	4,399	4,258	633,000	613,000	1.05

Foundation Example: Bearing Capacity for Group 1 Columns of One-Way Concrete Joist

BEARING CAPACITY OF SHALLOW FOUNDATIONS
Terzaghi and Vesic Methods

Date December 11, 2012
Identification Example 6.4

Input

Units of Measurement
E SI or E

Foundation Information

Shape	SQ	SQ, CI, CO, or RE
B =	6	ft
L =	6	ft
D =	5	ft

Soil Information

c =	2000	lb/ft^2
phi =	0	deg
gamma =	115	lb/ft^3
Dw =	10	ft

Factor of Safety
F = 3.5

Results

	Terzaghi	Vesic
Bearing Capacity		
q ult =	15,395 lb/ft^2	16,948 lb/ft^2
q a =	4,399 lb/ft^2	4,842 lb/ft^2
Allowable Column Load		
P =	158 k	174 k

Unit conve	1000
Gamma w	62.4
phi (radian	0
Terzaghi Computation	
a theta =	1
Nc =	5.70
Nq =	1.00
N gamma	0.00
gamma' =	104.6
coefficient	1.3
coefficient	0.4
sigma zD'	575
Vesic Computation	
Nc =	5.14
sc =	1.19
dc =	1.33
Nq =	1.00
sq =	1.00
dq =	1.00
N gamma	0.00
s gamma	0.60
d gamma	1.00
B/L =	1
k =	0.833333
W sub f	0

Foundation Example: Settlement for Group 1 Columns of One-Way Concrete Joist

SETTLEMENT ANALYSIS OF SHALLOW FOUNDATIONS
Classical Method

Date December 11, 2012
Identification Example 7.4

Input

Units	E	E or SI
Shape	SQ	SQ, CI, CO, or RE
B =	6	ft
L =	6	ft
D =	5	ft
P =	174	k
Dw =	10	ft
r =	0.85	

Results

q =	5583 lb/ft^2
delta =	0.71 in

Initial Cost Estimations: Rolled Steel Beams:

Rolled Steel Beams						
Beam	Beam Size	Cost ($/per ft)	Quantity	Length of one beam (ft)	Cost ($)	
Floor Corridor	W10x12	28.5	10	10	2850	
Floor Office	W14x22	40	20	20	16000	
Roof Corridor	W10x12	28.5	5	10	1425	
Roof Office	W12x26	46.5	10	20	9300	
Total:					29575	
Girder	Girder Size	Cost ($/per ft)	Quantity	Length of one girder (ft)	Cost ($)	
Exterior Girders Floor	W21x48	80.5	8	40	25760	
Interior Girders Floor	W21x68	111	4	40	17760	
Exterior Girders Roof	W21x44	74.5	4	40	11920	
Interior Girders Roof	W21x55	87	4	40	13920	
Total:					69360	
Column	Column Size	Cost ($/per ft)	Quantity	Length of one column (ft)	Cost ($)	
A	W10x33	56	4	36	8064	
B	W10x33	56	2	36	4032	
C	W10x33	56	4	36	8064	
D	W10x39	60	2	36	4320	
Total:					24480	
Footing	Depth (ft)	Width (ft)	concrete price ($/per cubic yard)	Foundation Labor Cost ($/per cubic yard)	Quantity	Cost ($)
1	7	7	113	60.5	4.00	1,258.22
2	8	8	113	60.5	4.00	1,643.39
3	9	9	113	60.5	2	1,040
4	11	11	113	60.5	2	1,553.52
Total:						5,495.13
Slab	Depth (in)	Area (ft^3)	Price ($/per cy)	Cost w/Labor ($/per cy)	Quantity	Cost ($)
1	5	1667	113	31.5	3	26,737.85

Initial Cost Estimations: Open-Web Steel Joists:

Open-Web Steel Joists						
Joists	Joist Size	Cost ($/per ft)	Quantity	Length (ft)	Cost ($)	
Floor Corridor Joists	10K1	11.45	32	10	3664	
Floor Office Joists	14K4	10.85	64	20	13888	
Roof Corridor Joists	10K1	10.85	16	10	1736	
Roof Office Joists	12K3	10.6	32	20	6784	
Total:					26072	
Girder	Girder Size	Cost ($/per ft)	Quantity	Length (ft)	Cost ($)	
Exterior Girders Floor	W18x35	62	8	40	19840	
Interior Girders Floor	W21x62	102	4	40	16320	
Exterior Girders Roof	W18x35	62	4	40	9920	
Interior Girders Roof	W21x44	74.5	4	40	11920	
Total:					58000	
Column	Column Size	Cost ($/per ft)	Quantity	Length (ft)	Cost ($)	
A	W10x33	56	4	36	8064	
B	W10x33	56	2	36	4032	
C	W10x33	56	4	36	8064	
D	W12X40	68.5	2	36	4932	
Total:					25092	
Footing	Depth (ft)	Width (ft)	Price ($/per cy)	Cost w/Labor ($/per cy)	Quantity	Cost ($)
1	6	6	113	60.5	4	924
2	7	7	113	60.5	4	1,258.22
3	8	8	113	60.5	2	821.70
4	10	10	113	60.5	2	1,283.90
Total:						4,288
Slab	Depth (in)	Area (ft^3)	Price ($/per cy)	Cost w/Labor ($/per cy)	Quantity	cost ($)
1	3	1000	113	31.5	3	16,039.50

Initial Cost Estimations: One-Way Slab-and-Beams:

One-Way Slab-and-Beams								
Beam	Area (ft^3)	Concrete Price ($/per cy)	Cost w/ Labor ($/per cy)	Quantity	Weight of Rebar (lb/ft)	Quantity	Cost ($/per ton)	Cost ($)
Floor Support	28	139	260	28	2.044	7	2550	7280
Floor Main	38	139	260	34	3.4	6	2550	8840
Roof Support	31	139	260	14	2.044	6	2550	3640
Roof Main	28	139	260	17	2.044	7	2550	4420
Total:								24180
Slab	Area (ft^3)	Price ($/per cy)	Cost w/ Labor ($/per cubic yard)	Quantity	Weight of Rebar (lb/ft)	Quantity	Cost ($/per ton)	Cost ($)
Floor	4400	139	181	2	1.043	120	2000	58,933.60
Roof	4400	139	181	1	1.043	120	2000	29,466.80
Total:								88400.4
Columns	Area (ft^3)	Price ($/per cy)	Cost w/Labor ($/per cy)	Quantity	Weight of Rebar (lb/ft)	Quantity	Cost ($/per ton)	Cost ($)
Corner	36	139	136	4	4.303	6	2,075	1,465
Edge	65	139	112	10	3.4	8	2,075	6,037
Middle	101	139	86	6	5.313	8	2,075	5,045
Total:								12,547
Footing	Depth (ft)	Width (ft)	Price (per cy)	Cost w/Labor (per cy)	Quantity	Cost ($)		
1	6	6	113	60.5	4	924		
2	8	8	113	60.5	10	4,108.48		
3	12	12	113	60.5	6	5,546		
Total						10,579		

Initial Cost Estimations: One-Way Concrete Joist Slab:

One-Way Concrete Joist Slab								
Joist	Type	Quantity	Length (ft)	Price incl. fabrication ($/per ft.)	Cost ($)			
Exterior Floor	8"	160	20	51.25	164,000.00			
Interior Floor	8"	32	20	51.25	32,800.00			
Exterior Roof	8"	80	20	51.25	82,000.00			
Interior Roof	8"	16	20	51.25	16,400.00			
Edge	8"	48	20	51.25	49,200.00			
Total					344,400.00			
Slab	Area (ft^3)	Price ($/per cy)	Cost w/ Labor ($/per cy)	Quantity	Weight of Rebar (lb/ft)	Quantity	cost ($/per ton)	Cost ($)
Floor	1200	139	170.5	2	0.11	96	2000	15,140
Roof	1200	139	170.5	1	0.11	96	2000	7,570
Total:								22710.6
Columns	Area (ft^3)	Price ($/per cy)	Cost w/Labor ($/per cy)	Quantity	Weight of Rebar (lb/ft)	Quantity	cost ($/per ton)	Cost ($)
Corner	36	139	136	4	4.303	4	2,075	1,465
Edge	49	139	124	10	5.313	4	2,075	4,768
Middle	72	139	118	6	7.65	4	2,075	4,108
Total:								10,341
w/fly ash		83.4						
Footing	Depth (ft)	Width (ft)	Price (per cy)	Cost w/Labor (per cy)	Quantity	Cost ($)		
1	7	7	113	60.5	4	1,258.22		
2	8	8	113	60.5	10	4,108.48		
3	9	9	113	60.5	6	3,119.88		
Total:						8,486.58		

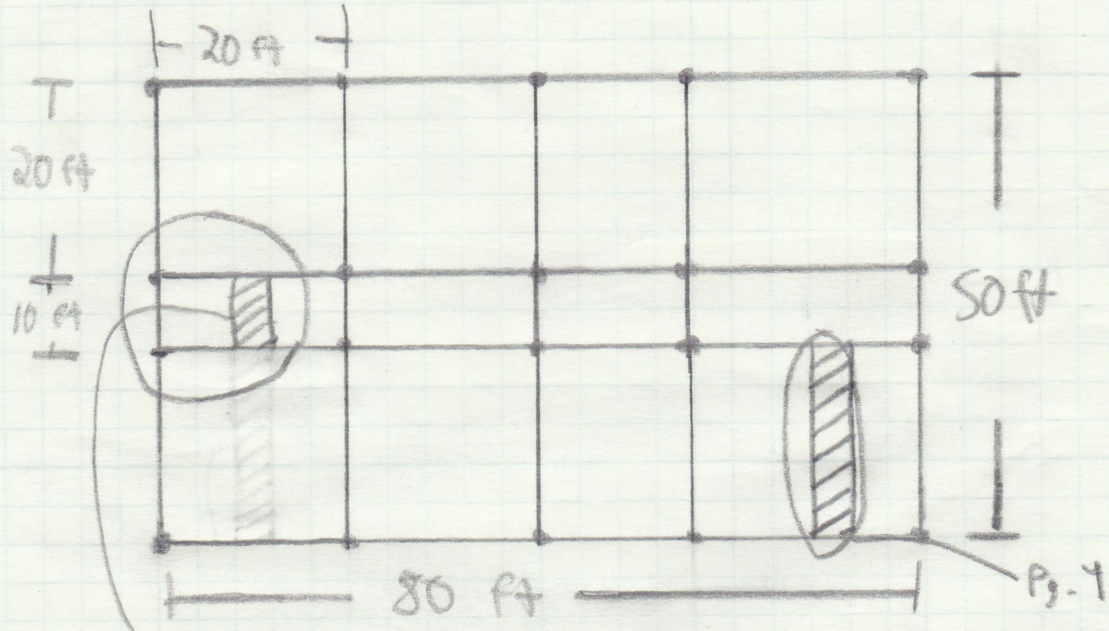
Final Cost Estimations: Open-Web Steel Joists:

Open-Web Steel Joists							
	Joists	Size	Cost ($/ft)	Quantity	Length (ft)	Cost ($)	
	Floor Corridor	10K1	8.53	28	10	2388.4	
	Floor Office	14K4	8.35	24	20	4008	
	Roof Corridor	10K1	8.53	14	10	1194.2	
	Roof Office	12K3	8.12	28	20	4547.2	
	Total:					12137.8	
	Girder	Size	Cost ($/ft)	Quantity	Length (ft)	Cost ($)	
	Ext. Floor	W18x35	53.76	8	40	17203.2	
	Int. Floor	W21x62	90.83	4	40	14532.8	
	Ext. Roof	W18x35	53.76	4	40	8601.6	
	Int. Roof	W21x44	65.69	4	40	10510.4	
	Edge	W10X12	$23.49	8	10	1879.2	
	Edge	W10X15	27.49	4	20	2199.2	
	Total:					54926.4	
	Column	Size	Cost ($/ft)	Quantity	Length (ft)	Cost ($)	
	A	W10x33	50.22	4	36	7231.68	
	B	W10x33	50.22	2	36	3615.84	
	C	W10x33	50.22	4	36	7231.68	
	D	W12X40	56	2	36	4032	
	Total:					22111.2	
Footing	Depth (ft)	Width (ft)	Area (cy)	Conc. ($/ cy)	Cost w/labor ($/cu yard)	Quantity	Cost ($)
1	6	6	1.33	103	146.6	4	779.912
2	7	7	1.81	103	146.6	4	1061.384
3	8	8	2.37	103	146.6	2	694.884
4	10	10	3.7	103	146.6	2	1084.84
Total:							3,621
	Slab w/deck	Depth (in)	Area (sf)	Conc. ($/sf)	Quantity	cost ($)	
	1	3	4,000	4.72	3	56640	

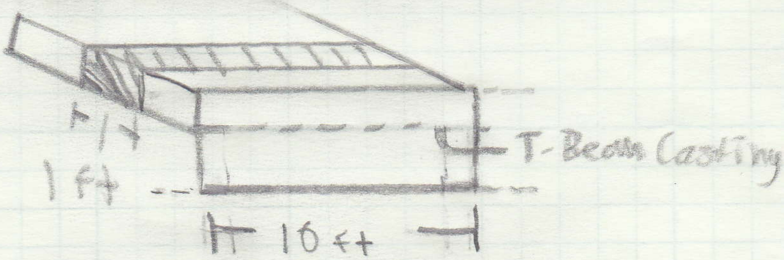
Final Cost Estimations: One-Way Slab-and-Beams:

One-Way Slab with Beams									
Beam	Area (cy)	Price ($/per cy)	Cost w/ Labor ($/per cy)	Quantity	Cost ($)	Weight of Rebar (lb/ft)	Quantity	Cost ($/ton)	Cost ($)
Floor Support	1.03	124	184.1	28	5309.444	2.044	7	$1,960	14.308
Floor Main	1.42	124	184.1	34	8888.348	3.4	6	$1,560	20.4
Roof Support	0.833	124	184.1	14	2146.974	2.044	6	$1,960	12.264
Roof Main	1.03	124	184.1	17	3223.591	2.044	7	$1,960	14.308
Total:					19568.36				61.28
w/Rebar					19629.64				
Slab	Area (cy)	Price ($/per cy)	Cost w/ Labor ($/per cy)	Quantity	Cost ($)	Weight of Rebar (lb/ft)	Quantity	Cost ($/ton)	Cost ($)
Floor	163	124	154.05	2	50220.3	1.043	120	$1,590	125.16
Roof	163	124	154.05	1	25110.15	1.043	120	$1,590	125.16
Total:					75330.45				250.32
w/Rebar					75580.77				
Columns	Area (cy)	Price ($/per cy)	Cost w/labor ($/per cy)	Quantity	Cost ($)	Weight of Rebar (lb/ft)	Quantity	Cost ($/ton)	Cost ($)
Comer	1.33	124	222	4	1181.04	2.67	6	1,665	16.02
Edge	2.37	124	204.32	10	4842.384	3.4	8	1,665	27.2
Middle	3.7	124	190.26	6	4223.772	5.313	8	1,665	42.504
Total:					10247.2				86
w/Rebar					10,333				
Footing	Size (ft)	Area (cy)	Price (per cy)	Cost w/labor (per cy)	Quantity	Cost ($)			
1	6 x 6	1.33	103	146.6	4	779.912			
2	8 x 8	1.81	103	146.6	10	2653.46			
3	12 x 12	5.33	103	146.6	6	4688.268			
Total						8,122			

Building frame:



Slab Section for 10' Span



Thickness of the slab = $L/20$ (both ends continuous)

$= 10 \times 12/20 = 6.00$ in

Use 6.00 in as trial thickness

Assumptions for all slabs:

$f_y = 60{,}000$ psi $w_c = 150$ pcf

$f'_c = 6{,}000$ psi $b = 12$ in

Dead load for the slab $= 150 \text{ pcf} \times 6/12 = 75$ psf

other Dead Loads and Live loads

DL — Ceiling = 3 psf
MEP = 5 psf
Insulation = 2 psf

LL — Corridors = 80 psf

2nd & 3rd Story One-Way Hallway Slab

$$W_u = 1.2D + 1.6L = 1.2(3 + 5 + 2 + 75) + 1.6(80)$$
$$= 102 + 128 = \boxed{230 \text{ psf}}$$

Max moment $M_u = \frac{WL^2}{8} = \frac{(230)(1)(10)^2}{8} / 1000 = 2.90$ k·ft (1 ft for b)

Assume $\phi = 0.9$, $M_n = \frac{M_u}{\phi} = \frac{2.90 \text{ k·ft}}{0.9} = \boxed{3.22 \text{ k·ft}}$

$$\rho_{0.005} = 0.85\beta_1 \frac{f'c}{f_y} \frac{0.003}{0.003 + 0.005}$$

$$\beta_1 = 0.85 - 0.05\left(\frac{f'c - 4{,}000}{1{,}000}\right) = 0.85 - 0.05\left(\frac{6{,}000 - 4{,}000}{1{,}000}\right) = 0.75$$

For all Slabs

$$\rho_{0.005} = (0.85)(0.75)\frac{6{,}000}{60{,}000}\frac{0.003}{0.003 + 0.005} = 0.024$$

= Maximum Practical Reinforcement

The effective depth d,

$$d^2 = \frac{M_n}{\rho f_y b(1 - 0.59\rho f_y / f'_c)} = \frac{3.22 \times 12}{0.024(60)(12)[1 - 0.59 \times 0.024 \times \frac{60}{6})}$$

$d = 1.6$ in, $1.6 \text{ in} < 6 \text{ in} - 1 \text{ in}$,

$d = 5$ inches (governing d)

Area of Steel:

$A_s = \frac{M_n}{f_y(d - a/2)}$, assume $a = 1.00$, $\frac{3.22 \times 12}{60(6 - 1/2)} = \boxed{0.14 \text{ in}^2}$

check a

$$a = \frac{A_s f_y}{0.85 f'c b} = \frac{0.14 \times 60}{0.85(6)(12)} = 0.14 \text{ in}^2$$

Steel Area for temperature shrinkage

$$A_s = 0.0018 \times 12 \times 6 = \boxed{0.13 \text{ in}^2}$$

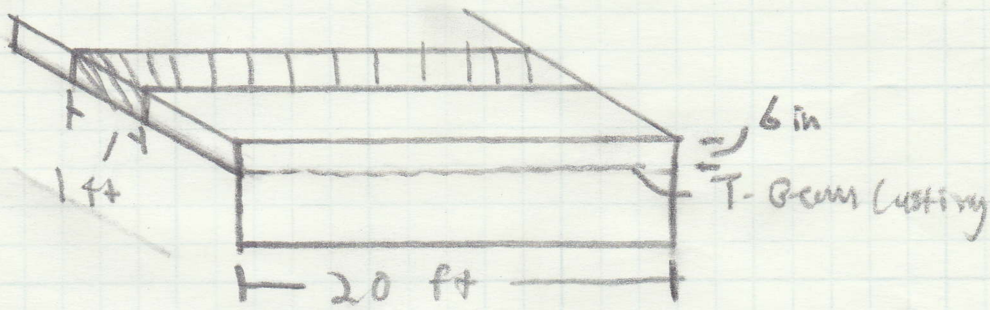
per foot in strip

factored shear V_u = Formula not known

$V_c = V_n = 2\lambda\sqrt{f'_c}\,bd$ = $\lambda = 1$ for all cases, $2(1)\sqrt{6{,}000}\,(12)(5)/1{,}000$

$V_c = 9.30\,k$

Slab Section for 20' Span



Dead Load of Slab = 75 psf (same)

Other Dead Loads & Live Loads

DL – { Ceiling = 3 psf
MEP = 5 psf
Insulation = 2 psf }
LL – Offices = 50 psf

2nd & 3rd Floor Office Slab

$W_u = 1.2(3+5+2+75) + 1.6(50) =$
$= 102 + 80 = \boxed{182 \text{ psf}}$

Max Moment $M_u = \frac{WL^2}{8} = \frac{(182)(1)(20)^2}{8} = 9.1 \text{ k}\cdot\text{ft}$

Assume $\phi = 0.9$, $M_n = \frac{9.1 \text{ k}\cdot\text{ft}}{0.9} = 10.11 \text{ k}\cdot\text{ft}$

$\rho_{0.005} = 0.024$ (same max steel reinforcement)

The effective Depth d

$d^2 = \frac{M_n}{bf_y\rho(1-0.59\rho f_y/f'c)} = \frac{10.11 \times 12}{(12)(60)(0.024)(1-0.59(0.024)(\frac{60}{6})}$

$d^2 = 8.18$ in, $d = 2.86$ in $< 6-1$ in $= 5$ in

$d = 5$ in

Area of the Steel:

$$A_s = \frac{M_n}{f_y(d - a/2)} = \text{assume } a = 1.00, \quad \frac{3.22 \times 12}{60(6 - 1/2)} = \boxed{0.12 \text{ in}^2}$$

check a

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(0.12)(60)}{0.85(6)(12)} = 0.12$$

Steel Area for temperature shrinkage:

$$A_s = 0.0018 \times 12 \times 6 = \boxed{0.13 \text{ in}^2}$$ per foot in strip

V_u = Unknown - Design halted

V_u = Unknown - Design halted

V_c = 9.3 k (same),

Roof Slabs

Same Assumptions & Sections Apply

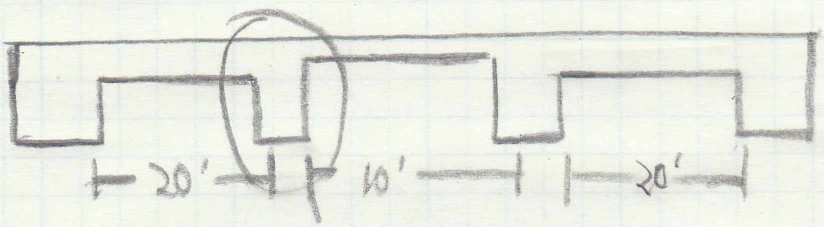
One-Way 10 ft slab

Dead Load of slab = 75 psf

Other Dead Loads are Live Loads

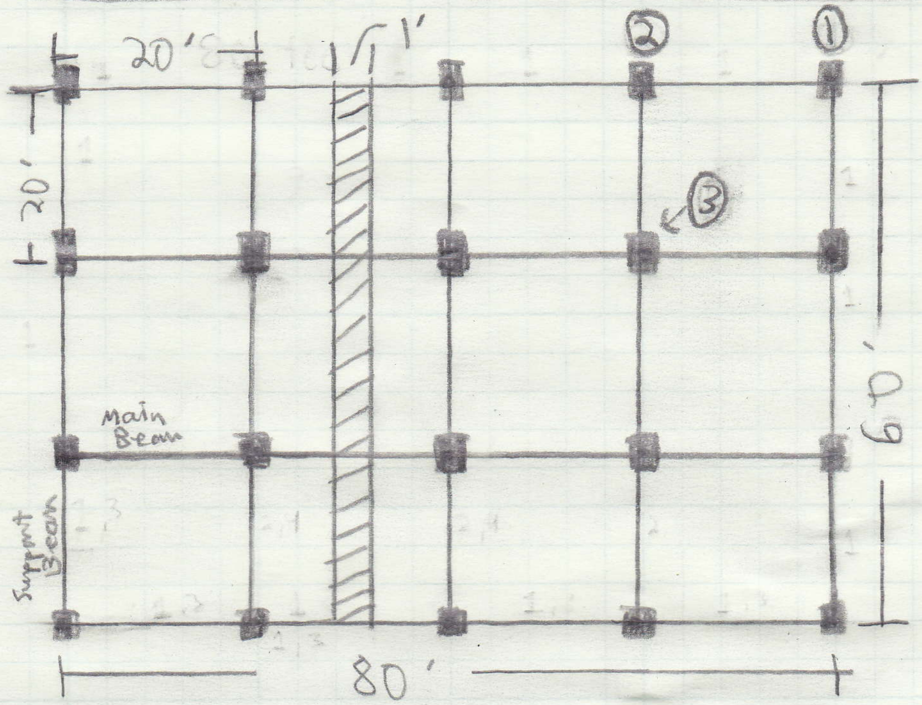
Ceiling = 3 psf
MEP = 5 psf
Insulation = 2 psf

Slab Design Not sufficient:



Not sufficient for beam design due to varying thicknesses within the slab

NEW 2nd & 3rd Floor One-Way Slab Design



Thickness of slab = $l_n/24 = 20 \times 12/24 = 10$ in

Use 11.00 in as trial thickness (1 in clearance included)

- Assumptions for previous slab apply

Dead Load of concrete slab = 150 pcf × 11/12 = 137.5 psf

Other Dead Loads and Live Loads

DL: Ceiling = 3 psf; MEP = 5 psf; Insulation = 2 psf

LL: Corridors = 80 psf; Offices = 50 psf

$W_u = 1.2D + 1.6L = 1.2(3+5+2+137.5) + 1.6(80+50)$

$W_u = 385$ psf or 0.385 ksf × 1ft = 0.385 k/ft

Moments for Each Part of Slab:

Interior support: $-M = \frac{W_u l_n^2}{10} = \frac{(0.385)(20^2)}{10}$ 15.4 k-ft

At Midspan: $+M = \frac{W_u l_n^2}{14} = \frac{(0.385)(20^2)}{14} = 11.0$ k-ft

Interior Span: $+M = \frac{W_u l_n^2}{16} = \frac{(0.385)(20^2)}{16} = 9.63$ k-ft

faces of Interior supports: $-M = \frac{W_u l_n^2}{11} = \frac{(0.385)(20^2)}{11} = 14$ k-ft

Interior faces of Exterior supports: $-M = \frac{W_u l_n^2}{24} = \frac{(0.355)(20^2)}{24} = 6.42$ k-ft

from previous slab

Reinforcement Ratio:

$$\rho_{0.005} = (0.85)(0.75)\left(\frac{6}{60}\right)\frac{0.003}{0.003+0.005} = 0.024$$

Effective Depth D:

$$d^2 = \frac{M_u}{\phi \rho f_y b\,(1-0.59\rho f_y/f'_c)} = \frac{15.4 \times 12}{0.9 \times 0.024 \times 60 \times 12\,[1-0.59\times 0.024 \times 60/6]}$$

$$d^2 = 13.84\ in^2,\ d = 3.72\ in$$

$3.72 \leq 11-1 = 10$ in, $d = 10$

Steel Area: Trial for $a = 1.00$

$$A_s = \frac{M_u}{\phi f_y (d - a/2)} = \frac{15.4 \times 12}{0.9 \times 60\,(10 - 1/2)} = 0.36\ in^2$$

Check a value

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{0.36 \times 60}{0.85 \times 6 \times 12} = 0.35\ in^2$$

$a = 0.35$ in

Use No. 4 bars with $6\frac{1}{2}$ in spacing → $A_s = 0.36\ in^2$

$$A_s = \frac{15.4 \times 12}{0.9 \times 60 \times (10 - 0.35/2)} = 0.35\ in^2,\ a = 0.35 \times \frac{0.35}{0.36} = 0.34$$

Steel Areas for other critical Sections:

Midspan: $A_s = \frac{11 \times 12}{0.9 \times 60 \times (10 - 0.34/2)} = 0.25\ in^2$ → Use No. 3 bars with 5 in spacing → $A_s = 0.26\ in^2$

Interior Span: $A_s = \frac{9.63 \times 12}{0.9 \times 60 \times 9.83} = 0.22\ in^2$ → Use No. 3 Bars with 6 in spacing

Face of interior Supports: $A_s = \frac{14 \times 12}{0.9 \times 60 \times 9.83} = 0.32\ in^2$ → Use No. 3 bars with 4 in spacing → $A_s = 0.33\ in^2$

Interior face of exterior support: $A_s = \frac{6.42 \times 12}{0.9 \times 60 \times 9.83} = 0.15\ in^2$ → Use No 3 bars with 9 in spacing

Steel Area for shrinkage and temperature cracking

$A_s = 0.0018h = 0.0018 \times 12 \times 11 = 0.24\ in^2$ – Use No. 4 bars with 10 in spacing

Factored Shear

$$V_u = 1.15\frac{W_u l_n}{2} - W_u d = 1.15 \times \frac{0.385 \times 20}{2} - 0.385 \times \frac{10}{12}$$

$$V_u = 4.11\ k$$

Nominal Shear Strength

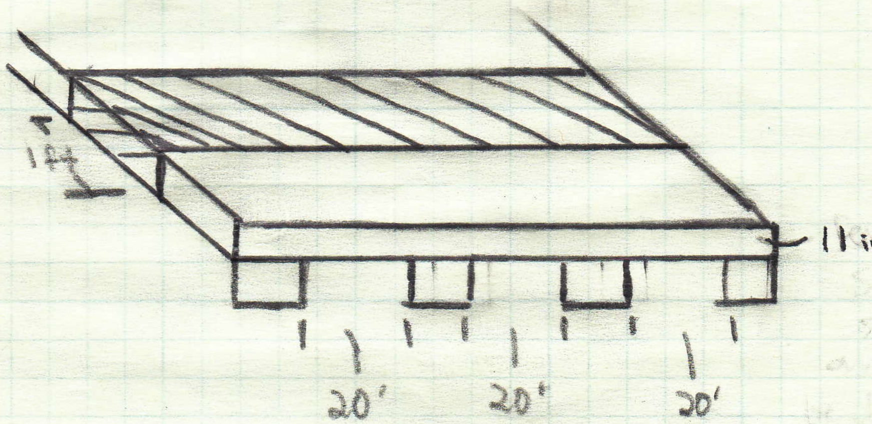
$$V_n = V_c = 2\lambda\sqrt{f'c}\, b d = 2(1)\sqrt{6{,}000} \times 12 \times 10 = 20.45\ k$$

$$\tfrac{1}{2}\phi V_c = 0.5 \times 0.75 \times 20.45 = 7.67\ k \geq 4.11\ k,$$

No shear reinforcement

Slab Section:

Applies to both slabs



Check Steel Strain for each A_s

Midspan: $c = \frac{(0.26)(60)}{0.85(6)(12)} / 0.75 = 0.33\bar{3}$, $\varepsilon_s = \frac{0.003}{\left(\frac{c}{d}\right)} - 0.003 = \frac{0.003}{\left(\frac{0.333}{10}\right)} - 0.003 = 0.087$
$0.087 \geq 0.005$ ✓

Int. Span: $c = \frac{(0.22)(60)}{0.85(6)(12)} / 0.75 = 0.293\bar{3}$, $\varepsilon_s = \frac{0.003}{\left(\frac{0.2933}{10}\right)} - 0.003 = 0.1 \geq 0.005$ ✓

Int. Support: $c = \frac{(0.36)(60)}{0.85(6)(12)} / 0.75 = 0.47$, $\varepsilon_s = \frac{0.003}{\left(\frac{0.47}{10}\right)} - 0.003 = 0.061 \geq 0.005$ ✓

Face of Int. 2 Support: $c = \frac{(0.33)(60)}{0.85(6)(12)} / 0.75 = 0.43$, $\varepsilon_s = \frac{0.003}{\left(\frac{0.43}{10}\right)} - 0.003 = 0.067 \geq 0.005$ ✓

Int. face of Ext. Support: $c = \frac{(0.15)(60)}{0.85(6)(12)} / 0.75 = 0.20$, $\varepsilon_s = \frac{0.003}{\left(\frac{0.20}{10}\right)} - 0.003 = 0.147 \geq 0.005$ ✓

Moment Check, Use $\phi = 0.9$

Interior Support: $\phi M_n = A_s f_y \left(d - \frac{a}{2}\right)\phi = 0.36(60)\left(10 - \frac{0.34}{2}\right)/12 \times 0.9$

$\phi M_n = 15.9\ k\text{-}ft > 15.4\ k\text{-}ft$ ✓

Midspan: $\phi M_n = 0.26(60)(9.83)/12 \times 0.9$

$\phi M_n = 11.5\ k\text{-}ft > 11\ k\text{-}ft$ ✓

Int. Span: $\phi M_n = 0.22(60)(9.83)/12 \times 0.9$

$\phi M_n = 9.73\ k\text{-}ft > 9.63\ k\text{-}ft$ ✓

Face of Int. Span: $\phi M_n = 0.33(60)(9.83)/12 \times 0.9$

$\phi M_n = 14.6\ k\text{-}ft > 14\ k\text{-}ft$ ✓

Int. Face of Ext. Support: $\phi M_n = 0.15(60)(9.83)/12 \times 0.9$

$\phi M_n = 6.6\ k\text{-}ft > 6.42\ k\text{-}ft$ ✓

$A_{s\,min}$ Checks

$$A_{s\,min} = \frac{3\sqrt{f'_c}\,bd}{f_y} = \frac{3\sqrt{6{,}000}(12)(10)}{60{,}000} = 0.46\ in^2$$

$$A_{s\,min} = \frac{200bd}{f_y} = \frac{200(12)(10)}{60{,}000} = 0.4\ in^2$$

$0.46\ in^2$ governs,

Use No. 5 bars $= 0.46\ in^2$
with 8 in spacing for all parts of slab

$a = \frac{(0.46)(60)}{0.85 \times 6 \times 12} = 0.45$, $c = \frac{0.45}{0.75} = 0.6$, $\varepsilon_s = \frac{0.003}{\left(\frac{0.46}{10}\right)} - 0.003 = 0.062$

$0.062 \geq 0.005$ ✓

$$\phi M_n = (0.46)(60)\left(10 - \frac{0.45}{2}\right)/12 \times 0.9$$

$\phi M_n = 20.23\ k\text{-}ft$ – Governing Moment Capacity

NEW Roof One-Way Slab

Loads:

DL:
- Dead-weight of concrete = 137.5 psf
- Ceiling = 3 psf
- MEP = 5 psf
- Insulation = 2 psf
- Roof Load = 10 psf

SL — Snow Load = 25 psf

WL — Wind Load = 23 psf

$W_u = 1.2D + 1.6S + 0.5W$ (governing equation)

$= 1.2(3+5+2+10+137.5) + 1.6(25) + 0.5(23)/1{,}000$

$W_u = 0.241 \text{ k/sf} \times 1\text{ft} = 0.241 \text{ k/ft}$

Moments For Each Part of Slab

Interior Support: $-M = \dfrac{(0.241)(20)^2}{10} = 9.64 \text{ k-ft}$

Midspan: $+M = \dfrac{(0.241)(20)^2}{14} = 6.89 \text{ k-ft}$

Interior Span $= \dfrac{(0.241)(20)^2}{16} = 6.03 \text{ k-ft} = +M$

Face of Interior Support $= \dfrac{(0.241)(20)^2}{11} = 8.76 \text{ k-ft} = -M$

Interior face of Exterior support $= \dfrac{(0.241)(20)^2}{24} = 4.02 \text{ k-ft} = -M$

Effective Depth D: Use $\rho = 0.024$ from previous slab

$$d^2 = \frac{9.64 \times 12}{0.9 \times 60 \times 0.024 \times 12\left[1 - 0.59 \times 0.024 \times 60/6\right]}$$

$d^2 = 8.67 \text{ in}^2, \quad d = 2.94 \text{ in}$

$2.94 \leq 11 - 1 = 10 \text{ in}, \quad d = 10 \text{ in}$

Steel Area: Trial $a = 1.00$

$$A_s = \frac{9.64 \times 12}{0.9 \times 60\,(10 - \frac{1}{2})} = 0.23\ in^2$$

$A_{s\,min} = 0.46\ in^2 > 0.23\ in^2$, Use No. 5 bars with 8 in spacing for whole slab

$-\phi M_n = 20.23\ k \cdot ft$ ← governing capacity from previous slab

$\varepsilon_s \geq 0.005$ [from previous slab]

$$V_c = 1.15\,\frac{(0.241) \times 20}{2} - (0.241)\left(\frac{10}{12}\right) = 2.57\ k$$

$V_c = 20.45\ k$, $\frac{1}{2}\phi V_c = 7.67\ k \geq 2.57\ k$, no shear reinforcement

2nd and 3rd Floor Support Beam:

$$Span = L = 20', \; d = \frac{20 \times 12}{16} = 15 \text{ in}, \text{ use } 17 \text{ in}$$

$h = 20$ in (including 3 inches of clear cover)

$$b = 0.6d = 0.6(15) = 10.2 \text{ in}, \text{ Use } 10 \text{ in}$$

$$W_{beam} = 150 \text{ pcf} \times \frac{20}{12} \times \frac{10}{12} = 208.316/\text{ft}$$

Other Dead Loads and Live Loads

DL:
- Ceiling - 3 psf
- MEP - 5 psf
- Insulation - 2 psf
- Weight of Slab - 137.5 psf

Included in Slab

LL:
- Corridors - 80 psf
- Offices - 50 psf

$$W_u = 1.2D + 1.6L = 1.2\left((208.316/\text{ft}) + \left(\overset{385+2(10)}{137.5}\right)(10)\right) + 1.6(80+50)(10)/1{,}000$$

$$W_u = 4.10 \text{ k/ft}$$

$$M_u = \frac{wL^2}{8} = \frac{4.10(20)^2}{8} = \boxed{205 \text{ k-ft}}$$

Area of Steel:

$$\rho_{0.005} = 0.024 \text{ (from previous slabs)}. \quad A_s = \rho b d = (0.024)(10)(17)$$

$$A_s = 4.08 \text{ in}^2$$

Use 7 No. 7 bars, $A_s = 4.20 \text{ in}^2$

$\beta_1 = 0.75$ from previous slabs

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{(4.20)(60)}{0.85 \times 6 \times 9} = 4.94, \quad c = \frac{a}{\beta_1} = \frac{4.94}{0.75} = 6.59$$

$$\varepsilon_s = \frac{0.003}{\left(\frac{5.95}{15}\right)} - 0.003 = 0.0047 < 0.005, \quad \phi = 0.88$$

$$\phi = 0.65 + (0.0047 - 0.002)\left(\frac{250}{3}\right)$$

$$M_n = A_s f_y \left(d - \frac{a}{2}\right) = (4.20)(60)\left(17 - \frac{4.94}{2}\right)/12 = 305.13 \text{ k/ft}$$

$$\phi M_n = (0.88)(305.13) = \boxed{268.5 \text{ k-ft} > 205 \text{ k-ft}}$$

Check for shear reinforcement:

$$V_u = \frac{w-d}{w}(V_{end}),\quad V_{end} = \frac{wL}{2} = \frac{(4.10)(20)}{2} = 41\ \text{k-ft}$$

$$V_u = \left(\frac{\overset{10\times12}{120} - 17}{120}\right) 41 = \boxed{35.2k}$$

$$V_c = 2\lambda\sqrt{f'c}\,b\,d = 2(1)\sqrt{6{,}000}(10)(17)/1{,}000 = 26.3k$$

Shear Inequality

$$V_u > V_c > \phi V_c > \tfrac{1}{2}V_c$$

Shear Steel Spacing (minimum Reinforcement)

$$S_{max} = d/2 = 17/2 \approx 9\ \text{in}$$

Pick No. 4 stirrups, $A_s = 0.2(2) = 0.4\ in^2$ (2 stirrups)

$$S_1 = \frac{A_{v\,min} f_y}{0.75\sqrt{f'c}\,b} = \frac{(0.4)(60{,}000)}{0.75\sqrt{6{,}000}(10)} = 25.8\ \text{in}$$

$$S_2 = \frac{A_{v\,min} f_y}{50\,b} = \frac{(0.4)(60{,}000)}{50(10)} = 30\ \text{in}$$

$S_{max} = 9$ in governs

Shear Steel Spacing (Design Reinforcement)

$$V_s = V_u - \phi V_c/\phi = 35.2 - (0.75)(26.3)/0.75 = 20.6$$

$$S = \frac{A_{v\,min}\,d\,f_y}{V_s} = \frac{(0.4)(17)(60{,}000)}{20{,}600} = 19.8\ \text{in spacing}$$

$$V_s \le 8\lambda\sqrt{f'c}\,b\,d = 8(1)(\sqrt{6{,}000})(10)(17)/_{1{,}000} = 105.3k,\ 20.6k < 105.3k$$

$$V_s > 4\lambda\sqrt{f'c}\,b\,d = 4(1)\sqrt{6{,}000}(170)/_{1{,}000} = 52.7k > 20.6k,\ S_{max} = \frac{d}{4}$$

$$S_{max} = d/4 = 17/4 \approx \boxed{4\ \text{in spacing}} \leftarrow \text{governs}$$

☆ Because the cross sectional area for the beam ($10\times17 = 170\ in^2$) and the slab ($120 \times 10 = 1200$) with respect to the effective depth d for both are very close, $4.20\ in^2$ will be far larger than the $A_{s\,min}$ checks

2nd & 3rd Floor Main-Beams

$d = 20\text{ in}$ $b = (0.6 \times 20) = 12\text{ in}$

$h = 23\text{ in}$

$W_{beam} = 150 \times \left(\frac{23}{12}\right) \times \left(\frac{12}{12}\right) = 287.5\text{ lb/ft}$

$W_u = 1.2(287.5 + 147.5(20)) + 1.6(130 \times 20) = 8.05\text{ k/ft}$

$M_u = \frac{(8.05)(20^2)}{8} = \boxed{402.5\text{ k}\cdot\text{ft}}$

$A_s = \rho bd = (0.024)(12)(20) = 5.76\text{ in}^2$, Use 6 No. 9 bars, $A_s = 6.00\text{ in}^2$

$a = \frac{(6)(60)}{0.85(6 \times 12)} = 5.88$, $c = \frac{5.88}{0.75} = 7.84$, $\varepsilon_s = \frac{0.003}{\left(\frac{7.84}{20}\right)} - 0.003 = 0.0047 < 0.005$

$\phi = 0.65 + (0.0047 - 0.002)\left(\frac{250}{3}\right) = 0.88$

$M_n = (6)(60)(20 - \frac{5.88}{2})/12 = 511.8\text{ k}\cdot\text{ft}$

$\phi M_n = (0.88)(511.8) = \boxed{450.4\text{ k ft} > 402.5\text{ k ft}}$

Check Shear Reinforcement:

$V_{end} = \left(\frac{8.05 \times 20}{2}\right) = 80.5\text{ k}$, $V_u = \left(\frac{120 - 20}{120}\right)80.5 = 67.1\text{ K}$

$V_c = 2(1)\sqrt{6{,}000}(12)(20)/1{,}000 = 37.2\text{ k}$

$67.1 > V_c > \phi V_c > \frac{1}{2}V_c$, Design Shear Reinforcement

$S_{max} = \frac{20}{2} = 10\text{ in}$ spacing, $S_1 = \frac{(0.4)(60{,}000)}{0.75\sqrt{6{,}000}(12)} = 21.52\text{ in}$

Use No. 4 stirrups, $A_s = 0.2\text{ in}^2$, $S_2 = \frac{(0.4)(60{,}000)}{50(12)} = 25\text{ in}$

$V_s = V_u - \phi V_c/\phi = 67.1 - (0.75 \times 37.2)/0.75 = 52.3\text{ k}$

$S = \frac{(0.4)(20)(60{,}000)}{52{,}300} = 9.2\text{ in}$ Spacing

$V_s \leq 8(1)\sqrt{6{,}000}(12)(20) = 148.7\text{ k}$, $52.3\text{ k} < 148.7\text{ k}$

$V_s > 4(1)\sqrt{6{,}000}(12)(20) = 74.4\text{ k} > 52.3\text{ k}$,

$S_{max} = 20/4 = \boxed{5\text{ in spacing}}$

Roof Beams: Support Beam

$d = 15$ in, $h = 18$ in

$b = 9$ in

$W_{beam} = 168.8$ lb/ft

- Other Dead Loads and Live Loads

DL
- Ceiling = 3 psf
- MEP = 5 psf
- Insulation = 2 psf
- Slab weight = 137.5 psf
- Roof Load = 10 psf

Included in slab weight

LL
- Snow Load = 25 psf
- Wind Load = 23 psf

$W_u = 1.2D + 1.6S + 0.5W$

$= 1.2(168.8 + (10 + 3 + 5 + 2 + 137.5(10)) + 1.6(25 \times 10) + 0.5(23 \times 10)/1{,}000$

$= 2.61$ k/ft $\quad M_u = \dfrac{2.61(20^2)}{8} = \boxed{130.5 \text{ k-ft}}$

Roof Beams: Main Beam:

$W_u = 1.2(168.8 + (20 \times 20) + (137.5 \times 20) + 1.6(25 \times 20) + 0.5(23 \times 20) / 1{,}000$

$= 5.01$ k/ft

$M_u = \dfrac{5.01(20^2)}{8} = \boxed{250.5 \text{ k-ft}}$

Use same beam as 2nd & 3rd floor support beam

$A_s = \rho b d = (0.024)(9)(15) = 3.24$ in², Use $A_s = 3.60$ in² for 6 No. 7 bars

$a = \dfrac{(3.60)(60)}{0.85(6)(9)} = 4.71$, $c = \dfrac{4.71}{0.75} = 6.28$ $\varepsilon_s = \dfrac{0.003}{\left(\frac{6.28}{15}\right)} - 0.003 = 0.0042 < 0.005$,

$\phi = 0.65 + (0.0022)\left(\frac{250}{3}\right)$
$= 0.83$

$\phi M_n = (0.83)(3.60)(60)\left(15 - \frac{4.71}{2}\right)/12 = \boxed{188.9 \text{ k-ft}}$ → 188.9 k-ft > 130.5 k-ft

$V_{end} = \left(\frac{2.61 \times 20}{2}\right) = 26.1$ k, $V_u = \left(\frac{120 - 15}{120}\right) 26.1 = 22.8$ k

$V_c = 2(1)\sqrt{4{,}000}(9)(15) = 20.9$ k

21.6 k > $V_c > \phi V_c > \frac{1}{2} V_c$, Design Shear Reinforcement

$S_{max} = \frac{15}{2} \approx 8$ in spacing, $S_1 = \dfrac{(0.4)(60{,}000)}{0.75\sqrt{4{,}000}(9)} = 28.7$ in $\quad S_2 = \dfrac{(0.4)(60{,}000)}{50(9)} = 33.\overline{3}$ in

Use No. 3 Stirrups
$A_v = 0.2$ in²

$$V_s = V_u - \phi V_c / \phi$$

$$= 26.1 - (0.75 \times 20.9) / 0.75 = 13.9\,k$$

$$S = \frac{0.4(15)(60{,}000)}{13{,}900} = 25.9\ in$$

$$V_s < 8(1)\sqrt{6{,}000}\,(9)(15)/1{,}000 = 83.7\,k,\ 13.9\,k\ 83.7\,k$$

$$V_s > 4(1)\sqrt{6{,}000}\,(9)(15)/1{,}000 = 41.8\,k > 13.9\,k$$

Use d/4 for spacing

$$S = \frac{15}{4} \approx \boxed{4\ \text{in spacing}}$$

Beams Used:

1. B
2.
3.

Corner Column Design

Loads:

DL:
- Ceiling – 3 psf
- MEP – 5 psf
- Insulation – 2 psf
- Roof Load – 10 psf

LL:
- Offices – 50 psf
- Corridors – 80 psf
- Roof Live Load – 20 psf

S – Snow Load – 25 psf

$W_{column} = 150 \times \left(\frac{12}{12}\right)^2 \times (36) = 5400$ lbs

$W_{slab} = 137.5$ psf

$W_{beam,1} = 208.3$ lb/ft (2nd & 3rd floor edge beam)

$W_{beam,2} = 168.8$ lb/ft (Roof edge beam)

$W_{beam\,3} = 287.5$ lb/ft (2nd & 3rd floor mid-beam)

2nd & 3rd Floor Loads

$DL = (3+5+2+137.5)(10)^2 + (2 \times 208.3 \times 10) = 18{,}916$ lbs per floor

$LL = (80+50)(10^2) = 13{,}000$ lbs per floor

Roof Loads

$DL = (3+5+2+10+137.5)(10^2) + (2 \times 168.8 \times 10) = 19{,}126$ lbs

$SL = (25 \times 100) = 2500$ lbs – governs over roof live load

Combined Factored Loads

$P_u = 1.2D + 1.6L + 0.5S$

$= 1.2(18{,}916 \times 2 + 19{,}126 + 5{,}400) + 1.6(13{,}000 \times 2) + 0.5(2500) / 1{,}000 = 116.0\text{K}$

$P_n = \frac{P_u}{\phi} = \frac{116.0}{0.65} = 178.5\text{k}$

assumed (0.65)

3 percent of A_g is the steel area, A_s

$P_n = 0.8(0.85f'_c A_g + A_s f_y) = 0.8(0.85f'_c A_g + 0.03 A_g f_y)$

$178.9 = 0.8(0.85(6)A_g + 0.03(60)A_g)$

$6.9A_g = 223.6, \quad A_g = 32.4\ \text{in}^2$ – Too small because of beam widths

Try $b = 10$, $h = 10$, $A_g = 100\ \text{in}^2$

Calculate Steel Area, A_s

$A_s = 0.03A_g = 100(0.03) = 3.00\ \text{in}^2 \rightarrow$ Use Try 4 No. 8 bars, $A_s = 3.16\ \text{in}^2$

$A_s' \approx A_s$

Calculate P_{nb}:

$C = \frac{0.003}{0.003+0.002}\overbrace{(10-3)}^{d-d'} = 4.2$, $a = \beta_1 c = (0.75)(4.2) = 3.15$ in

$f'_s = \varepsilon'_s E_s = 0.003\left(\frac{c-d'}{c}\right)E_s = 0.003\left(\frac{4.2-3}{4.2}\right)(29{,}000) = 24.9$ ksi

$f_s \approx f_y = 60$ ksi

$P_{nb} = 0.85 f'_c a b + A'_s f'_s - A_s f_y$ — 2 #8 bars

$= \underbrace{0.85(6)(3.15)(10)}_{C_1} + \underbrace{\left(\frac{3.16}{2}\right)(24.9)}_{C_2} - \underbrace{\left(\frac{3.16}{2}\right)(60)}_{T} = 105.17\text{ k} < 111.2\text{ k}$

Try $b = 12$ in, $h = 12$ in, $A_g = 144$ in^2

$A_s = 0.03A_g = (144 \times 0.03) = 4.32\text{ in}^2 \rightarrow$ Try 6 #8 bars, $A_s = 4.74\text{ in}^2$

$C = \frac{0.003}{0.005}(12-3) = 5.4$ in, $a = 0.75(5.4) = 4.05$

$f'_s = 0.003\left(\frac{5.4-3}{5.4}\right)(29{,}000) = 38.7$ ksi, $f_s \approx f_y = 60$ ksi

$P_{nb} = (0.85)(6)(4.05)(12) + \left(\frac{4.74}{3}\right)(38.7) - \left(\frac{4.74}{3}\right)(60) = 214.2\text{ k}$

$\phi P_n = 0.65(214.2) = 139.2\text{ k} > 116.0\text{ k}$ ✓

assumed tie reinforcement

Mid-Edge Column Design

$W_{column} = 150 \times \left(\frac{16}{12}\right) \times 36 = 7{,}200$ lbs

2nd & 3rd Floor Loads:

$DL = (117.5(10 \times 20)) + (2(208.3 \times 10) + (287.5 \times 20)) = 39{,}416$ lbs per floor

$LL = (130 \times (20 \times 10)) = 26{,}000$ lbs per floor

Roof Loads:

$DL = (152.5)(20 \times 10) + (2 \times 168.8 \times 10) + (208.3 \times 20) = 39{,}042$ lbs

$SL = (25 \times (20 \times 10)) = 5{,}000$ lbs

Combined Factored Loads:

$P_u = 1.2(39{,}416 \times 2) + 39{,}042^{+7{,}200}) + 1.6(26{,}000 \times 2) + 0.5(5{,}000) / 1{,}000$

$= 235.8k$

$A_c \approx A_g$

Try $b = 16$ in, $h = 16$ in, $A_g = 256$ in^2

$A_s = 0.03 A_g = 0.03(256) = 7.68$ in^2, Use 8 #9 bars, $A_s = 8$ in^2

$A_s' \approx A_s$

Check P_{nb}:

$c = \frac{0.003}{0.005}(16-3) = 7.8$ in, $a = 0.75(7.8) = 5.85$

$f_s' = 0.003\left(\frac{7.8-3}{7.8}\right)(29{,}000) = 53.5$ ksi

$f_s \approx f_y = 60$ ksi

$P_{nb} = 0.85(6)(5.85)(16) + (8/4)(53.5) - (8/4)(60) = 464.4k$

$\phi P_n = (0.65)(464.4) = 301.9k > 235.8k$ ✓

tie reinforcement

For Connections:

$M_n = 0.85(6)(5.85)(16)\left(8 - \frac{5.85}{2}\right) + (2)(53.5)(8-3)$

$+ (2)(60)(13-8) =$

$e = \frac{M_n}{P_n} = \frac{3{,}557.6}{464.4} = 7.7$

Middle Column Design

$W_{column} = 150 \times \left(\frac{20}{12}\right) \times (36)$
$= 9{,}000 \text{ lbs}$

2nd & 3rd Floor Loads

$DL = (147.5 \times (20^2)) + (4 \times 287.5 \times 20) = 82{,}000$ lbs per floor
$LL = (130)(20^2) = 52{,}000$ per floor

Roof Loads:

$DL = (157.5(20^2)) + (4 \times 208.3 \times 20) = 79{,}664$ lbs
$SL = (25 \times (20^2)) = 10{,}000$ lbs

Combined factored Loads:

$P_u = 1.2(82{,}000 \times 2 + 79{,}664 + 9{,}000) + 1.6(52{,}000 \times 2) + 0.5(10{,}000) / 1{,}000$
$= 474.8\text{ k}$

Try $b = 20$ in, $h = 20$ in, $A_g = 400 \text{ in}^2$

$A_s = 0.03(400) = 12 \text{ in}^2 \rightarrow$ Use 8 #11 bars, $A_s = 12.48 \text{ in}^2$

$A'_s \approx A_s$

Check P_{nb}

$c = \frac{0.003}{0.005}(20-3) = 10.2$ in, $a = (0.75)(10.2) = 7.65$ in

$f'_s = 0.003\left(\frac{10.2-3}{10.2}\right)29{,}000 = 61.4$ ksi

$f_s \approx f_y = 60$ ksi

$P_{nb} = 0.85(6)(7.65)(20) + \left(\frac{12.48}{4}\right)(61.4) - \left(\frac{12.48}{4}\right)(60) = 784.7\text{ k}$

$\phi P_n = 0.65(784.7) = 510.1\text{ k} > 474.6\text{ k}$ ✓

Negative Moment: 2nd & 3rd Floor Main Beams

$\frac{1}{16} wL^2 = \frac{1}{16}(8.05)(20)^2 = 201 \text{ k}\cdot\text{ft} = M_u$

$M_n = \frac{201.25}{0.9} = 223.6\overline{1} \text{ k}\cdot\text{ft}$ or $2{,}683{,}333.\overline{33} \text{ lb}\cdot\text{in}$

$M_n = A_s f_y \left(d - \frac{\frac{A_s f_y}{0.85 f'_c b}}{2}\right)$

$= 60{,}000 A_s (20 - 0.49 A_s) = 2{,}683{,}333.\overline{33}$

$= -29{,}400 A_s^2 + 1{,}200{,}000 A_s - 2{,}683{,}333.\overline{33}$

$$\frac{-1{,}200{,}000 \pm \sqrt{(1{,}200{,}000)^2 - 4(29{,}400)(2{,}683{,}333.\overline{33})}}{2(-29{,}400)}$$

$A_s = 2.37 \text{ in}^2$, Use 2 No. 10 bars, $A_s = 2.40 \text{ in}^2$

$a = \frac{2.40(60{,}000)}{0.85(6000)(12)} = 2.35$, $c = \frac{2.35}{0.75} = 3.13$

$M_n = 60{,}000(2.37)\left(20 - \frac{2.35}{2}\right) = 2{,}868{,}930 \text{ lb}\cdot\text{in}$ or $239.08 \text{ k}\cdot\text{ft}$

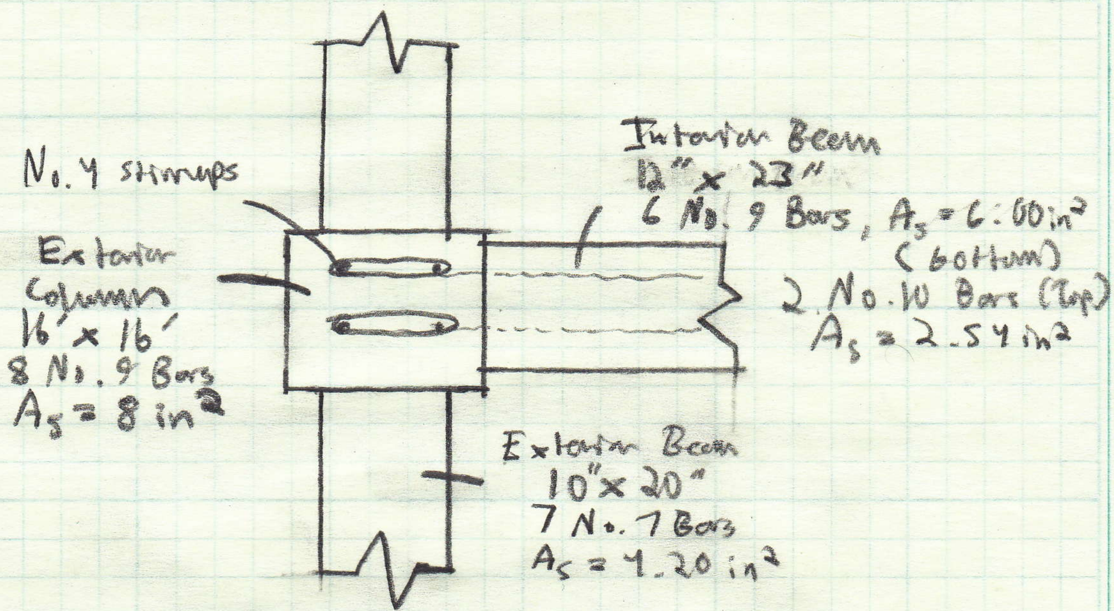
$\varepsilon_s = \frac{0.003}{\left(\frac{3.13}{20}\right)} - 0.005 = 0.016 \geq 0.005$, $\phi = 0.9$

$\phi M_n = 0.9(239.08) = 215.17 \text{ k}\cdot\text{ft} > 201.2 \text{ k}\cdot\text{ft}$ ✓

Add 2 No. 10 Bars to this beam for tensile reinforcement against negative moment.

Concrete Connections:

Exterior Type 2 Joint for 2nd and 3rd Floor Connections:



$$l_{dh} = \left(\frac{0.02\,\psi_e f_y}{\lambda\sqrt{f'_c}}\right) d_b = \left(\frac{0.02 \times 1 \times 60{,}000}{1 \times \sqrt{6{,}000}}\right) 1.25 = 15.49 \text{ in}$$

$l_{dh} = 15.49 \times 0.7 = 10.843$, $23-2 = 21 \text{ in} > 10.843 \text{ in}$ ✓

Shear strength of Joint

$$A_s f_y = 2.54 \times 60{,}000 = 152{,}400 \text{ lbs}$$

$$d = 23 - 2 - 1.25/2 = 20.375, \quad a = 2.35$$

$$M_u = M_n = 2.54(60{,}000)\left(20.375 - \frac{2.35}{2}\right) = 245{,}300.5 \text{ ft}\cdot\text{lbs}$$

Column Shear In Joint

$$V_{col} = \frac{245{,}300.5}{12} = 20{,}441.7 \text{ lbs}$$

$$V_u = 152{,}400 - 20{,}441.7$$
$$= 131{,}958.3 \text{ lbs}$$
or 132 kips

$\gamma = 20$ (exterior joint)

$$b_j = \frac{12+20}{2} = 16 \text{ in}$$

$$b_j = 12 + 2\left(\frac{0.3 \times 20}{2}\right) = 18 \text{ in}$$

$$V_n = 20\sqrt{6{,}000}\; 16^2/1{,}000 = 396.6 \text{ k}$$

$$\phi V_n = 0.75(396.6) = 297.45 \text{ k} \geq 132 \text{ k} \checkmark$$

Negative Moment: Roof Main Beam

$$\frac{1}{16}WL^2 = \frac{1}{16}(5.01)(20)^2 = 125.25\ \text{k-ft} = M_u$$

$$A_s = \frac{0.85 f'c\,a\,b}{f_y} \qquad M_n = \frac{M_u}{\phi} = \frac{125.25}{0.9} = 139.2\ \text{k}\cdot\text{ft}$$

$\phi = 0.9$ — or 1,670,400 lb·in

$$M_n = A_s f_y \left(d - \frac{\frac{A_s f_y}{0.85 f'c\,b}}{2}\right)$$

$$= (60,000)(A_s)\left(17 - \frac{0.59 A_s}{2}\right) = 1,670,400\ \text{lb}\cdot\text{in}$$

$$= -35,400 A_s^2 + 1,020,000 A_s - 1,670,400$$

$$\frac{-1,020,000 +/- \sqrt{(+1,020,000)^2 + 4(35,400)(1,670,400)}}{-35,400(2)}$$

$A_s = 1.74\ \text{in}^2 \rightarrow$ Use 3 No. 7 Bars, $A_s = 1.80\ \text{in}^2$

$$a = \frac{(1.8)(60,000)}{0.85(6000)(10)} = 2.12 \qquad c = \frac{2.12}{0.75} = 2.83$$

$$M_n = (1.8)(60,000)\left(17 - \frac{2.12}{2}\right) = 143.46\ \text{k}\cdot\text{ft}$$

$$\varepsilon_s = \frac{0.003}{\left(\frac{2.83}{17}\right)} - 0.005 = 0.013 \geq 0.005,\ \phi = 0.9$$

$$\phi M_n = 0.9(143.46) = 129.11\ \text{k-ft} > 125.25\ \text{k-ft}\ \checkmark$$

Add 3 No. 7 Bars to this Beam for tensile reinforcement against negative moment

Loads: obtained from ASCE 7

Dead load: Ceiling = 3psf
root deck = 10psf
MEP = 5psf
Insulation = 2psf

Total Dead load = 20psf

Live Loads: 100psf (Lobbies + 1st floor corridors)
50psf (offices)
80psf (corridors above 1st floor)

total live load (due to occupancy) = 230psf

roof live Load (L_r) = 20psf

Snow load: $P_f = 0.7\, C_e C_t I_s P_g$

$P_g = 50\ lb/ft^2$ (From Fig 7-1 ASCE 7)
$C_e = 0.9$
$C_t = 1.0$
$I_s = 0.80$ (Risk category I, table 1.5-2)

$P_f = 0.7(0.9)(1)(0.80)(50\ lb/ft^2)$

$P_f = 25.2\ lb/ft$ ⇒ Snow load = 25 psf

Wind load:
V = 54 m/s or 120 mph (figure 26.5-1c category I)
$K_d = 0.85$ (table 26.6-1)
Exposure B
$K_{zt} = 1.0$ (No hills)
$G = 1.0$
$K_z = 0.74$ (table 27.3-1)
$GC_{pi} = {+0.18 \atop -0.18}$ (table 26.11-1)

Z = 12 ft 0.70 0.76
$K_z = 0.76 + \frac{36-30}{40-30}(0.76-0.70)$

$q_h = 0.00256\ K_z K_{zt} K_d V^2$

$q_h = 0.00256(0.74)(1.0)(0.85)(120\ mph)^2 = 23.2\ lb/ft^2$
$C_p = 0.8$ (Figure 27.4-1)

$P = qGC_p - q_i GC_{pi}$

$P = (23.2\ lb/ft^2)(1.0)(0.8) - (23.2\ lb/ft^2)(1.0)(\pm 0.18)$

$P = 22.7\ lb/ft^2$ or $14.4\ lb/ft^2$ ⇒ wind load = 23 psf

Load Combinations for OFFice building (LRFD Design)

$D_L = 20\ psf$
$L_L = 230\ psf$
$L_r = 20\ psf$
$S = 25\ psf$
$W = 23\ psf$

1) $1.4D = 1.4(20psf) = 28psf$

2) $1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$
$1.2(20psf) + 1.6(230psf) + 0.5(25psf) = \underline{\underline{404.5}}\ psf$

3) $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.5W)$
$1.2(20psf) + 1.6(25psf) + 230psf = 294\ psf$

4) $1.2D + 1.0W + L + 0.5(L_r \text{ or } S \text{ or } R)$
$1.2(20psf) + 1.0(23psf) + 230psf + 0.5(25psf) = 289.5psf$

5) $1.2D + 1.0E + L + 0.2S$
$1.2(20psf) + 230psf + 0.2(25psf) = 259\ psf$

6) $0.9D + 1.0W = 0.9(20psf) + 1.0(23psf) = 41\ psf$

$\Rightarrow$ governing load combination $1.2D + 1.6L + 0.5S$

Determining Spacing of open web Steel joists
$\Rightarrow$ based on load obtained from load Combination compare to total safe factored uniformly distributed load - carrying Capacities in load tables.

layout

joist Span = 50 ft

try 8 ft joist spacing
$1.2(20psf \times 8ft) + 1.6(230psf \times 8ft) + 0.5(25psf \times 8ft) = 3,236\ lb/ft$
too big

try 5ft joist spacing
$1.2(20psf \times 5ft) + 1.6(230psf \times 5ft) + 0.5(25psf \times 5ft) = 2,022.5\ lb/ft$
too big

try 2.5ft joist spacing
$1.2(20psf \times 2.5ft) + 1.6(230psf \times 2.5ft) + 0.5(25psf \times 2.5ft) = 1,011.3\ lb/ft$
$\underline{\underline{\text{Ok}}}$

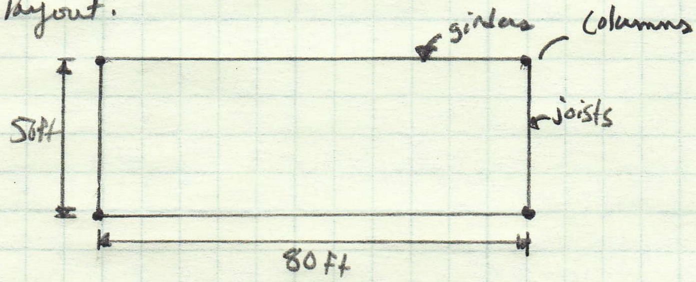
try 2.5ft spacing

2nd Floor Design: governing load combination $1.2D + 1.6L$

$D_L = 20 psf$
$L_L = 150 psf$

Determine spacing of open web steel joist
$\Rightarrow$ based on load obtained from load combinations compare to total safe factored uniformly distributed load carrying capacities in load tables

layout:



joists span 50 ft

try 8 ft spacing

$1.2(20 psf \times 8 ft) + 1.6(150 psf \times 8 ft) = 2,112 lb/ft$ too big

try 5 ft spacing

$1.2(20 psf \times 5 ft) + 1.6(150 psf \times 5 ft) = 1,320 lb/ft$

try 2.5 ft spacing

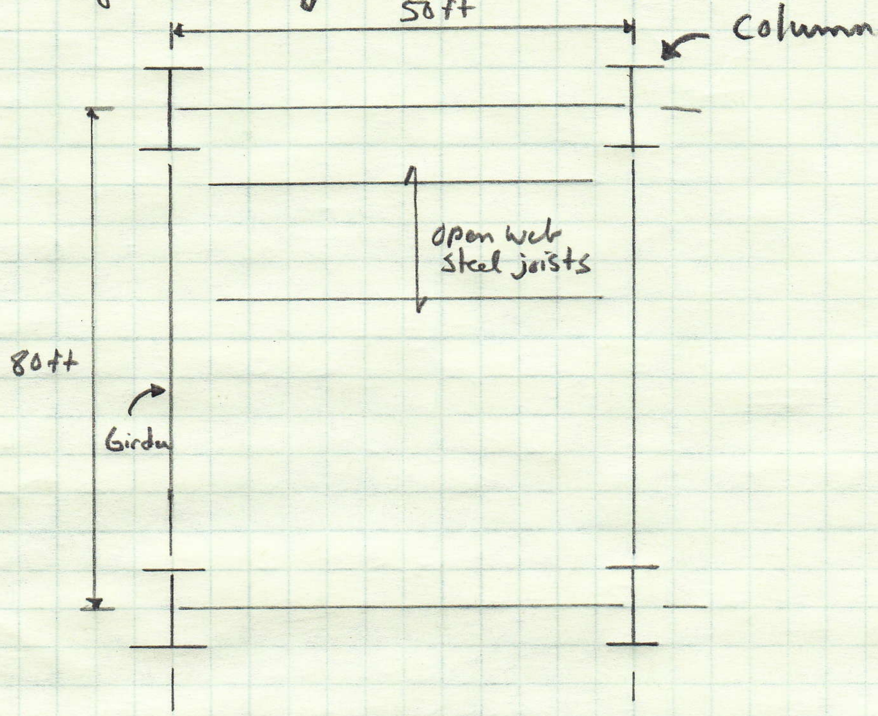
$1.2(20 psf \times 2.5 ft) + 1.6(150 psf \times 2.5 ft) = 660 lb/ft$

ok

try 2.5 spacing

layout of Floor bay:

layout of Floor bay:



Bay size 50 x 80

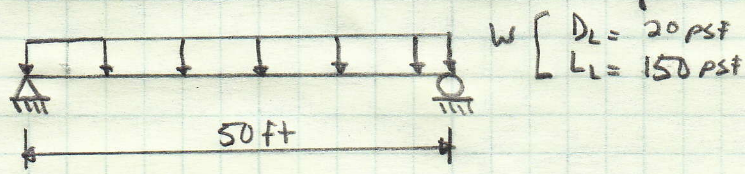
open web Steel joist span = 50 ft

open web Steel joist Spacing = 2.5 ft

$D_L = 20\,psf$

$L_L = 150\,psf$

Finding open web steel joist weight: from Standard LRFD Load Table for LH-Series Long Span Steel joists



Factored Load: using governing load comb. $1.2D + 1.6L + 0.5S$

$W_{TL} = 1.2(20\,psf \times 2.5\,ft) + 1.6(150\,psf \times 2.5\,ft)$

$W_{TL} = 660\ lb/ft$ use $W_{TL} = 660\ lb/ft$

target load-carrying capacity $W_{TL} = 660\ lb/ft$

target load-carrying capacity $\quad W_{TL} = 660$ lb/ft

from Standard LRFD Load table for LH-Series Long Span Steel Joists

$W_{TL} = 660$ lb/ft $\quad$ Span $= 50$ ft

try 32LH09 $\quad$ wt = 21 lb/ft

allow $W_u = 774$ lb/ft

live load $W_u = 319$ lb/ft to cause to have a deflection approx. equal to 1/360th of the span

$W_u = 1.2(20\,psf \times 2.5\,ft + 21\,lb/ft) + 1.6(150\,psf \times 2.5\,ft)$

$W_{TL} = 685.2$ lb/ft

ok, since allowable $W_u = 774$ lb/ft $> W_{TL} = 685.2$ lb/ft

Check deflection due to the unfactored design live load
$\Rightarrow$ Should not exceed 1/360th of span $\quad \Delta_{LL,allow} \le \frac{L}{360}$

$W_{LL} = 150\,psf \times 2.5\,ft = 375$ lb/ft

Since, from the table live load $W_u = 319$ lb/ft $< W_{LL} = 375$ lb/ft

this joist is not acceptable because $W_u = 319$ lb/ft will cause joist to have a deflection equal to 1/360th of span, so in this case $W_{LL} = 375$ lb/ft will create a large deflection.

Choose a joist size with a live load $W_u > 375$ lb/ft

try 32LH11 $\quad$ with live load $W_u = 385$ lb/ft target load-carrying capacity $W_{TL} = 660\ lb/ft$

From Standard LRFD Load table for LH-Series LongSpan Steel Joists

$W_{TL} = 660\ lb/ft$ span = 50 ft

try 32LH11 wt = 24 lb/ft

allowable W_u = 937 lb/ft

live load W_u = 385 lb/ft to cause jist to have a deflection approx. equal to 1/360th of the Span

$W_u = 1.2(20psf \times 2.5ft + 24\ lb/ft) + 1.6(150\ psf \times 2.5ft)$

$W_{TL} = 688.8\ lb/ft$

ok, since allowable $W_u = 937 lb/ft > W_{TL} = 689\ lb/ft$

Check deflection due to the design live load Should not exceed 1/360th of Span

$\Delta_{LL, Allow} \le \frac{L}{360}$

W_{LL} : unfactored

$W_{LL} = 150\ psf \times 2.5\ ft = 375\ lb/ft$

Since, from the table $W_u = 385\ lb/ft > W_{LL} = 375\ lb/ft$

this joist size is acceptable because $W_u = 385 lb/ft$ will cause joist to have a deflection equal to 1/360th of span, so $W_{LL} = 375\ lb/ft$ will create a smaller deflection.

joist Deflection check

$$\Delta = \frac{1.15(5)\,w\,L^4}{384\,E\,I}$$

$L = (\text{span} - 0.33\ ft)$

$$I = 26.767\ wL^3\ 10^{-6}$$

$$I = 26.767(385 lb/ft)(50-0.33)^3\ 10^{-6} = 1263\ in^4$$

$$\Delta_{LL, allow} = \frac{1.15(5)(385\ lb/ft)(50-0.33ft)^4}{384(29{,}000\times10^3 psi)(1263 in^4)}\left(\frac{12 in}{ft}\right)^3 = 1.66\ in$$

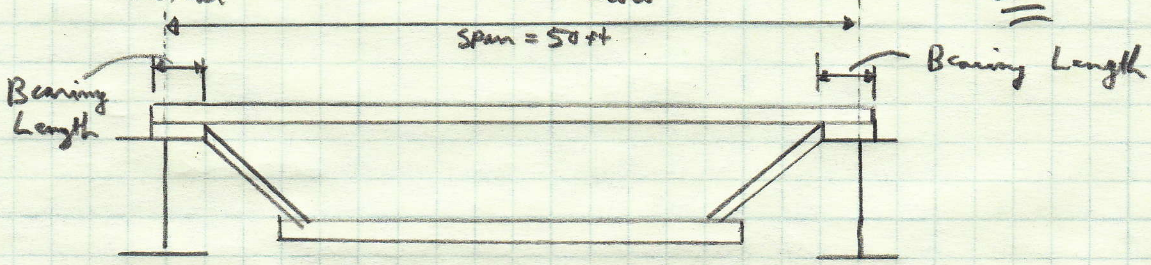
or

$$\Delta_{LL, allow} = \frac{L}{360} = \frac{(50-0.33ft)}{360} \times \left(\frac{12''}{ft}\right) = 1.66\ in$$

$$\Delta_{LL, actual} = \frac{1.15(5)(375\ lb/ft)(50-0.33ft)^4}{384(29{,}000\times10^3 psi)(1263\ in^4)}\left(\frac{12in}{ft}\right)^3 = 1.62\ in$$

Since,

$\Delta_{Actual} = 1.62\text{ in} < \Delta_{allow} = 1.66\text{ in}$ ok



Open web steel joist 32 LH11

Design Length = Span − 0.33 ft

From table 104.4-1, steel joist 32 LH 11
Minimum bearing Length is 4"

- The span of a long span joist shall not exceed 24 times its depth

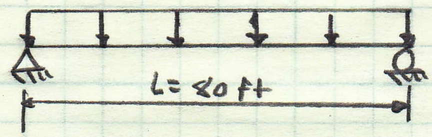
depth = 32 in
Span = 50 ft

$50\,ft\left(\frac{12''}{ft}\right) < 24 \times 32\,in$

$600\,in < 768\,in$ ok

Finding Girder weight:

$$W_u = \begin{cases} D = \\ LL = \end{cases}$$



Girder span = 80 ft
Girder spacing = 50 ft

Girder load Approx.

$$W_{DL} = \left[\text{Superimposed DL} + \frac{\text{joist wt.}}{\text{joist spacing}}\right] \times (\text{Girder spacing})$$

$$W_{DL} = \left[20\,psf + \frac{24\,lb/ft}{2.5\,ft}\right] \times (50\,ft) = 1{,}480\ lb/ft$$

$$W_{LL} = 150\,PSF \times \text{Girder spacing}$$

$$W_{LL} = (150\,psf)(50\,ft) = 7{,}500\ lb/ft$$

Factored loads:

$$W_u = 1.4D = 1.4(1{,}480\,lb/ft) = 2{,}072\,lb/ft$$

$$W_u = 1.2D + 1.6L = 1.2(1{,}480\,lb/ft) + 1.6(7{,}500\,lb/ft) = 13{,}776\ lb/ft$$

Target Capacity:
$$M_u = \frac{W_u L^2}{8} = \frac{(13{,}776\,lb/ft)(80\,ft)^2}{8} = 11{,}021\ k\cdot ft$$

$\phi = 0.9$
$F_y = 50ksi$

Design Equation:
$$Z_x \geq \frac{M_u}{\phi F_y} = \frac{11{,}021\,k\cdot ft}{(0.9)(50ksi)}\left(\frac{12in}{ft}\right) \qquad Z_x \geq 2939\,in^3$$

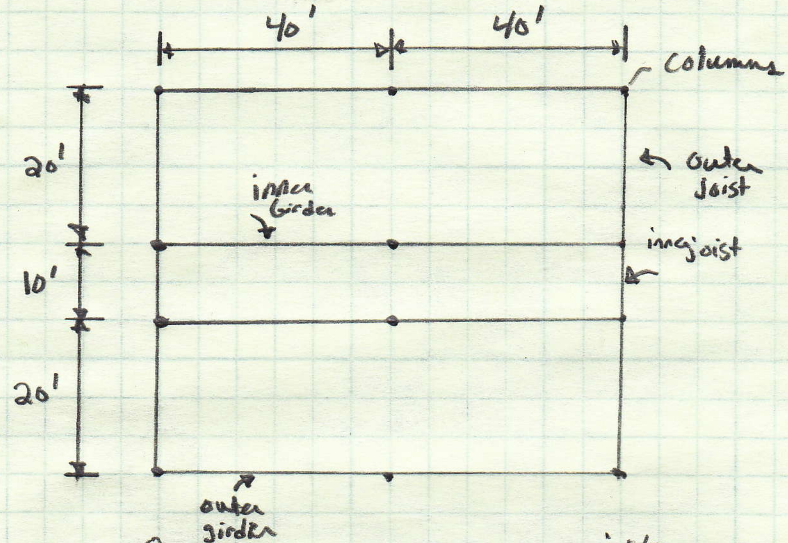
table 3-2

Girder is too big!

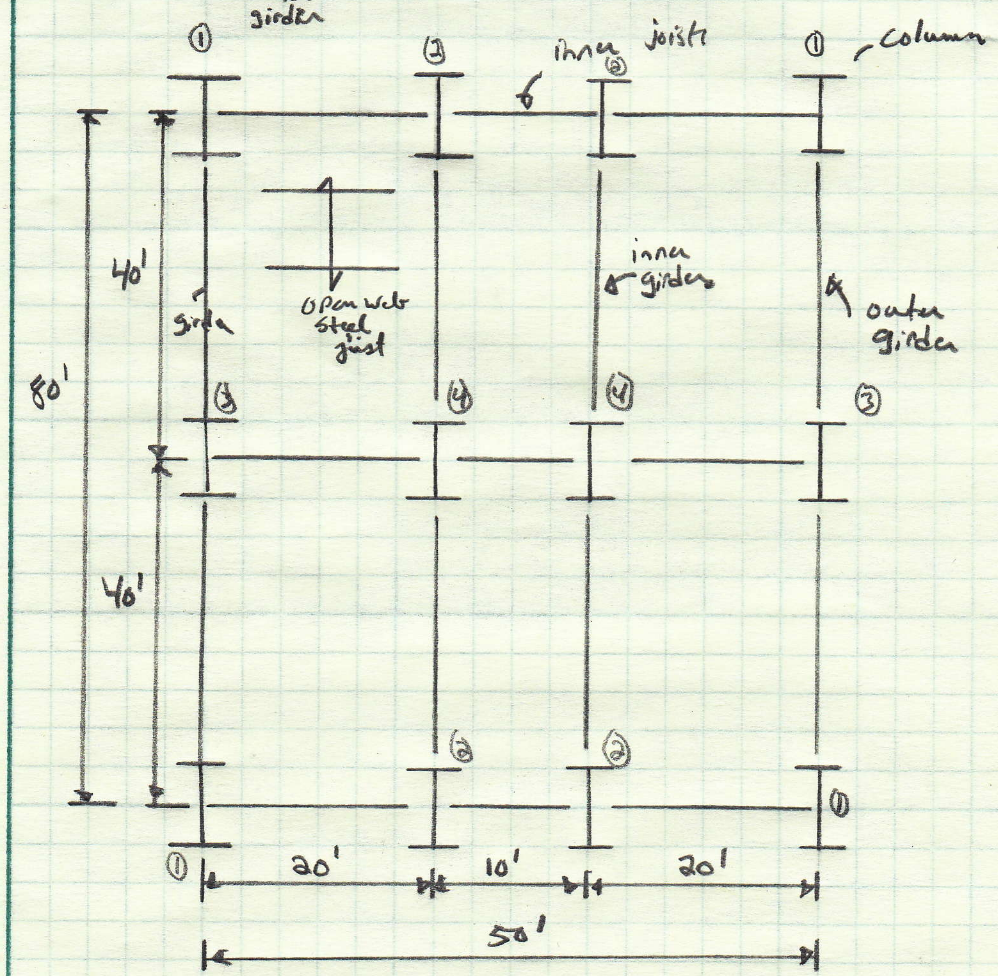
Consider changing layout!

2nd & 3rd Floor Design: governing load combination $1.2D + 1.6L$

$D_L = 20$ psf
$L_L = 50$ psf (offices)
$L_L = 80$ psf (corridors above 1st Floor)



outer joist span = 20'
inner joist span = 10'



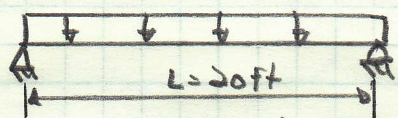
$D_L = 20$ psf
$L_L = 50$ psf (offices)
$L_L = 80$ psf (Corridors above 1st Floor)

try Spacing 2.5 ft   outer joists span 20 ft   use LL = 50 psf

$1.2D + 1.6 = 1.2(20\text{psf} \times 2.5\text{ft}) + 1.6(50\text{psf} \times 2.5\text{ft}) = 260\ \text{lb/ft}$
ok

outer web steel joist span = 20 ft
open steel web joist Spacing = 5.0 ft

Finding open web steel joist weight for outer joists:



W { DL = 20 psf
LL = 51 psf

Factored Load

$W_{TL} = 1.2(20\text{psf} \times 5.0\text{ft}) + 1.6(50\text{psf} \times 5.0\text{ft}) = 520\ \text{lb/ft}$

target load carrying capacity $W_{TL} = 520\ \text{lb/ft}$

- Use Stand. LRFD Load Tables for K-Series joists

$W_{TL} = 520\ \text{lb/ft}$ Span 20 ft

try 14K4   wt = 6.7 lb/ft
allow W_u = 642 lb/ft
live load W_u = 287 lb/ft to cause a deflection approx. equal to 1/360th of span

$W_u = 1.2(20\text{psf} \times 5\text{ft} + 6.7\ \text{lb/ft}) + 1.6(50\text{psf} \times 5\text{ft})$
$W_{TL} = 528\ \text{lb/ft}$

ok, since allow $W_u = 642\ \text{lb/ft} > W_{TL} = 528\ \text{lb/ft}$

Check deflection due to the unfactored design live load:

$W_{LL} = 50\text{psf} \times 5\text{ft} = 250\ \text{lb/ft}$

Since, from the table $W_u = 287\ \text{lb/ft} > W_{LL} = 250\ \text{lb/ft}$
the joist size is acceptable because $W_u = 287\ \text{lb/ft}$ will cause joist to have deflection approx. equal to 1/360th of span, so $W_{LL} = 250\ \text{lb/ft}$ will create a smaller deflection.

(11)

joist Deflection check:

$L = (\text{Span} - 0.33\,ft)$

$$\Delta = \frac{1.15(5)\,wL^4}{384\,EI}$$

$$I = 26.767\,wL^3\,10^{-6}$$

$$I = 26.767\,(287\,lb/ft)(20\,ft - 0.33)^3\,10^{-6} = 58.5\,in^4$$

$$\Delta_{LL,\,allow} = \frac{1.15(5)(287\,lb/ft)(20-0.33\,ft)^4}{384(29{,}000\times10^3\,psi)(58.5\,in^4)}\left(\frac{12\,in}{ft}\right)^3 = 0.66\,in$$

or

$$\Delta_{LL,\,allow} = \frac{L}{360} = \frac{(20-0.33\,ft)}{360}\times\left(\frac{12''}{ft}\right) = 0.66\,in$$

$$\Delta_{LL,\,actual} = \frac{1.15(5)(250\,lb/ft)(20-0.33\,ft)^4}{384(29{,}000\times10^3\,psi)(58.5\,in^4)}\left(\frac{12\,in}{ft}\right)^3 = 0.57\,in$$

Since $\Delta_{LL,\,actual} = 0.57\,in < \Delta_{LL,\,allow} = 0.66\,in$ ok

- The span of a K-series joist shall not exceed 24 times its depth

depth = 14 in
Span = 20 ft

$$20\,ft\left(\frac{12\,in}{ft}\right) < 24\times14\,in$$

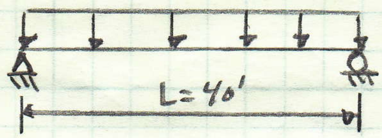
$$240\,in < 360\,in \quad \text{ok}$$

Open Web Steel joist 14K4

Design Length = Span - 0.33 ft

Minimum bearing Length is $2\frac{1}{2}''$

Finding outer Girder Weight:



$W_u = \begin{cases} D_L = \\ L_L = \end{cases}$

Girder Span = 40'

Girder spacing $= \dfrac{20'}{2} = 10'$

Girder load Approx.

$$W_{DL} = \left[\text{superimposed DL} + \frac{\text{Joist weight}}{\text{joist spacing}} \right] \times \left(\text{Girder Spacing} \right)$$

$$W_{DL} = \left[20\,psf + \frac{6.7\,lb/ft}{5\,ft} \right] \times (10') = 213.4\ lb/ft$$

$W_{LL} =$ 50 psf × Girder Spacing
$W_{LL} =$ 50 psf × 10 ft = 500 lb/ft

Factored Loads:

$W_u = 1.4\,D = 1.4(213.4\,lb/ft) = 324\ lb/ft$

$W_u = 1.2D + 1.6L = 1.2(213.4\,lb/ft) + 1.6(500\,lb/ft) = 1078\ lb/ft$

Target Capacity $\quad M_u = \dfrac{W_u L^2}{8} = \dfrac{(1078\,lb/ft)(40')^2}{8} = 216\ k{\cdot}ft$

$\phi = 0.9$
$F_y = 50\ ksi$

Design equation
$$Z_x \geq \frac{M_u}{\phi F_y} = \frac{216\ k{\cdot}ft}{(0.9)(50\,ksi)}\left(\frac{12\,in}{ft}\right) \qquad Z_x \geq 58\ in^3$$

table 3-2 $\quad$ trial: W18×35 $\quad Z_x = 66.5\,in^3 \quad 35\,lb/ft$

$W_u = 1.2(213.4\,lb/ft + 35\,lb/ft) + 1.6(500\,lb/ft)$
$W_u = 1{,}098\ lb/ft$

$$M_u = \frac{W_u L^2}{8} = \frac{(1{,}098\,lb/ft)(40')^2}{8} = 220\ k{\cdot}ft$$

W18×35 Capacity:

$\phi M_p = \phi Z_x F_y = (0.9)(66.5\,in^3)(50\,ksi)\left(\frac{ft}{12''}\right) = 249\ k{\cdot}ft$

$\phi M_p = 249\ k{\cdot}ft > 220\ k{\cdot}ft$ ok

Check Compact Section Criteria: $\Rightarrow$ Outer girder Design W18×35

$\dfrac{b_f}{2t_f} = 7.06 \qquad \dfrac{h}{t_w} = 53.5$ (From table 1-1)

$\dfrac{b_f}{2t_f} = 7.06 < 0.38\sqrt{E/F_y} = 9.2$] Flange compact $\quad$ ok $\quad \phi M_n = \phi M_p$

$\dfrac{h}{t_w} = 53.5 < 3.76\sqrt{E/F_y} = 90.5$] Web is compact

Girder Deflection: W 18 x 35

From table 3-3
$I_x = 510 \text{ in}^4$
$E = 29{,}000 \text{ ksi}$

W_{LL} at 50% $= \frac{50 \text{ psf} \times 10 \text{ ft}}{2} = 250 \text{ lb/ft}$

$$\Delta_{LL} = \frac{5WL^4}{384 EI_x} = \frac{5(250 \text{ lb/ft})(40 \text{ ft})^4 \left(\frac{12 \text{ in}}{\text{ft}}\right)^3}{384(29{,}000 \times 10^3 \text{ psi})(510 \text{ in}^4)} = 0.97 \text{ in}$$

Check: $\Delta_{LL} \le \frac{L}{360}$ $\quad \Delta_{LL} \le \frac{40 \text{ ft}}{360}\left(\frac{12''}{\text{ft}}\right)$ $\quad \Delta_{LL} \le 1.33 \text{ in}$

$\Delta_{LL} = 0.97 \text{ in} \le 1.33 \text{ in}$ ok

$W_D = 213.4 \text{ lb/ft} + 35 \text{ lb/ft} = 248.4 \text{ lb/ft}$

$W_{LL} + W_D = 250 \text{ lb/ft} + 248.4 \text{ lb/ft} = 498.4 \text{ lb/ft}$

$$\Delta_{D+L} = \frac{5WL^4}{384 EI_x} = \frac{5(498.4 \text{ lb/ft})(40 \text{ ft})^4 \left(\frac{12''}{\text{ft}}\right)^3}{384(29{,}000 \times 10^3 \text{ psi})(510 \text{ in}^4)} = 1.94 \text{ in}$$

Check:

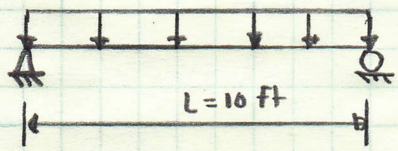
$\Delta_{D+L} \le \frac{L}{240}$ $\quad \Delta_{D+L} \le \frac{40 \text{ ft}}{240}\left(\frac{12''}{\text{ft}}\right)$ $\quad \Delta_{D+L} \le 2 \text{ in}$

$\Delta_{D+L} = 1.94 \text{ in} \le 2 \text{ in}$ ok

Inner joists: use $D_L = 20\,psf$ $L_L = 80\,psf$

Inner web steel joist span = 10'
open steel web joist spacing = 5'

Finding open web steel joist weight for Inner joists



$W\left[\begin{array}{l} D_L = 20\,psf \\ L_L = 80\,psf \end{array}\right.$

Check: Span of K-series joist shall not exceed 24 times depth

depth = 10 in
span = 10 ft

$10\,ft\left(\frac{12\,in}{ft}\right) < 24 \times 10\,in$

$120\,in < 240\,in$ ok

<u>Factored Load:</u>

$W_{TL} = 1.2(20\,psf \times 5\,ft) + 1.6(80\,psf \times 5\,ft) = 760\,lb/ft$

target load capacity $W_{TL} = 760\,lb/ft$

- use Stand. LRFD Load Tables for K-series joists

$W_{TL} = 760\,lb/ft$ Span = 10 ft

Try 10 K 1

wt = 5.0 lb/ft
allow $W_u = 825\,lb/ft$
live load $W_u = 550\,lb/ft$ to cause deflection equal to 1/360th of span

$W_u = 1.2(20\,psf \times 5\,ft + 5.0\,lb/ft) + 1.6(80\,psf \times 5.0\,ft)$

$W_{TL} = 766\,lb/ft$

ok, since allow $W_u = 825\,lb/ft > W_{TL} = 766\,lb/ft$

Check deflection due to the unfactored design live load:

$W_{LL} = 80\,psf \times 5\,ft = 400\,lb/ft$

Since, from table $W_u = 550\,lb/ft > W_{LL} = 400\,lb/ft$ the size is acceptable

Joist Deflection check: $\Delta = \dfrac{1.15(5)\,wL^4}{384\,EI}$ $L = (\text{span} - 0.33\,ft)$

$I = 26.767\,wL^3\,10^{-6}$

$I = 26.767(550\,lb/ft)(10-0.33\,ft)^3\,10^{-6} = 13.3\,in^4$

$\Delta_{LL,allow} = \dfrac{1.15(5)(550\,lb/ft)(10-0.33\,ft)^4}{384(29{,}000\times10^3\,psi)(13.3\,in^4)}\left(\dfrac{12\,in}{ft}\right)^3 = 0.32\,in$

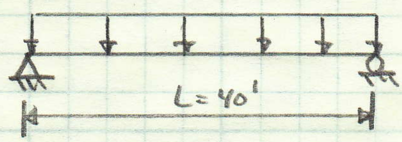
or

$\Delta_{LL,allow} = \dfrac{L}{360} = \dfrac{(10-0.33\,ft)\left(\frac{12''}{ft}\right)}{360} = 0.32\,in$

$\Delta_{LL,actual} = \dfrac{1.15(5)(400\,lb/ft)(10-0.33\,ft)^4}{384(29{,}000\times10^3\,psi)(13.3\,in^4)}\left(\dfrac{12\,in}{ft}\right)^3 = 0.235\,in$

Since $\Delta_{LL,actual} = 0.235\,in < \Delta_{LL,allow} = 0.32\,in$ <u>OK</u>

Finding Inner Girder Weight:



$W_u \begin{cases} D_L = \\ L_L = \end{cases}$

Girder span = 40'

Girder spacing $= \frac{20'}{2} + \frac{10'}{2} = 15'$

Girder load Approx.

$$W_{DL} = \left[\text{Superimposed DL} + \frac{\text{joist weight}}{\text{joist spacing}} \right] \times \left(\text{Girder tributary width spacing} \right)$$

$$W_{DL} = \left[20\,psf + \frac{5.0\,lb/ft + 6.7\,lb/ft}{5\,ft} \right] \times (15') = 335.1\ lb/ft$$

$W_{LL} =$ 80 PSF × Girder Spacing

$W_{LL} =$ 80 psf × 15' = 1,200 lb/ft

Factored Loads:

$W_u = 1.4D = 1.4(335.1\,lb/ft) = 469.2\ lb/ft$

$W_u = 1.2D + 1.6L = 1.2(335.1\,lb/ft) + 1.6(1200\,lb/ft) = 2{,}322.12\ lb/ft$

target capacity $\quad M_u = \frac{W_u L^2}{8} = \frac{(2{,}322.12\,lb/ft)(40\,ft)^2}{8} = 464.4\ k\cdot ft$

$\phi = 0.9$

$F_y = 50\,ksi$

Design Equation:

$$Z_x \geq \frac{M_u}{\phi F_y} = \frac{464.4\ k\cdot ft}{(0.9)(50\,ksi)} \left(\frac{12\,in}{ft}\right) \qquad Z_x \geq 123.8\ in^3$$

table 3-2 trial: W21×62 62 lb/ft $Z_x = 144\,in^3$

$W_u = 1.2(335.1\,lb/ft + 62\,lb/ft) + 1.6(1200\,lb/ft)$

$W_u = 2{,}396.5\ lb/ft$

$$M_u = \frac{W_u L^2}{8} = \frac{(2{,}396.5\,lb/ft)(40')^2}{8} = 479.3\ k\cdot ft$$

W21×62 Capacity

$$\phi M_p = \phi Z_x F_y = (0.9)(144\,in^3)(50\,ksi)\left(\frac{ft}{12''}\right) = 540\ k\cdot ft$$

$\phi M_p = 540\,k\cdot ft > 479.3\ k\cdot ft$ ok

Check Compact Section criteria:

$\frac{b_f}{2t_f} = 6.70 \qquad \frac{h}{t_w} = 46.9$ (from table 1-1) ⟹ Inner girder Design W21×62

$\frac{b_f}{2t_f} = 6.70 < 0.38\sqrt{E/F_y} = 9.2$] Flange compact

ok $\quad \phi M_n = \phi M_p$

$\frac{h}{t_w} = 46.9 < 3.76\sqrt{E/F_y} = 90.5$] Web compact

Girder Deflection: W21 x 62

From table 3-3
$I_x = 1330\ in^4$
$E = 29{,}000\ ksi$

W_{LL} at 50% $= \dfrac{80psf \times 15ft}{2} = 600\ lb/ft$

$$\Delta_{LL} = \frac{5wL^4}{384EI_x} = \frac{5(600\ lb/ft)(40\ ft)^4\left(\frac{12in}{ft}\right)^3}{384(29{,}000\times10^3 psi)(1330\ in^4)} = 0.896\ in$$

Check: $\Delta_{LL} \le \dfrac{L}{360}$ $\quad \Delta_{LL} \le \dfrac{40ft}{360}\left(\dfrac{12in}{ft}\right)$ $\quad \Delta_{LL} \le 1.33in$

$\Delta_{LL} = 0.896\ in < 1.33in$ ok

$W_D = 335.1\ lb/ft + 62\ lb/ft = 397.1\ lb/ft$

$W_{LL}+W_D = 600\ lb/ft + 397.1\ lb/ft = 997.1\ lb/ft$

$$\Delta_{D+L} = \frac{5wL^4}{384EI_x} = \frac{5(997.1\ lb/ft)(40ft)^4\left(\frac{12in}{ft}\right)^3}{384(29{,}000\times10^3 psi)(1330\ in^4)} = 1.50\ in$$

Check:

$\Delta_{D+L} \le \dfrac{L}{240}$ $\quad \Delta_{D+L} \le \dfrac{40ft}{240}\left(\dfrac{12in}{ft}\right)$ $\quad \Delta_{D+L} \le 2in$

$\Delta_{D+L} = 1.50in \le 2in$ ok

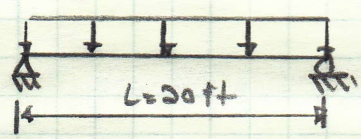
Roof Design:

$D_L = 30psf$ $L_r = 20psf$ $S = 25psf$ $W = 23psf$

$1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$
$1.2(30psf) + 0.5(25psf) = 48.5\ psf$
$1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.5W)$
$1.2(30psf) + 1.6(25psf) + 0.5(23psf) = \underline{\underline{87.5}}\ psf$ governing load comb.
$1.2D + 1.0W + L + 0.5(L_r \text{ or } S \text{ or } R)$
$1.2(30psf) + 23psf + 0.5(25psf) = 71.5psf$
$1.2D + 1.0E + L + 0.2S$
$1.2(30psf) + 0.2(25psf) = 41psf$
$0.9D + 1.0W$
$0.9(30psf) + (23psf) = 50psf$

Outer Web Steel joist:
Span = 20 ft
Spacing = 5.0 ft

Finding open web steel joist weight for outer joists:



$W \begin{cases} D_L = 30psf \\ S = 25psf \end{cases}$ $W = 23psf$

Factored Load:
$w_{TL} = 1.2D + 1.6S + 0.5W$
$w_{TL} = 1.2(30psf \times 5ft) + 1.6(25psf \times 5ft) + 0.5(23psf \times 5ft)$

Target load carrying capacity $w_{TL} = 437.5\ lb/ft$

- Use stand. LRFD Load Tables for K-series joists
$w_{TL} = 438\ lb/ft$ Span = 20ft

try 12K3 wt = 5.7 lb/ft
allow $w_u = 453\ lb/ft$
live load $w_u = 177\ lb/ft$ to cause deflection approx. equal to 1/360th of span

$w_u = 1.2(30psf \times 5ft + 5.7\,lb/ft) + 1.6(25psf \times 5ft) + 0.5(23psf \times 5ft)$
$w_{TL} = 444.3\ lb/ft$

ok, since allow $w_u = 453\ lb/ft > w_{TL} = 444.3\ lb/ft$

Check deflection due to the unfactored design live load

$w_{Lr} = 20psf \times 5ft = 100\ lb/ft$
Since, from the table $w_u = 177\ lb/ft > w_{Lr} = 100ft$ the joist size is acceptable

Joist Deflection Check:

$$\Delta = \frac{1.15(5)\,w\,L^4}{384\,E\,I}$$

$L = (\text{Span} - 0.33\,ft)$

$$I = 26.767\,w\,L^3\,10^{-6}$$

$$I = 26.767(177\,lb/ft)(20-0.33\,ft)^3\,10^{-6} = 36.1\ in^4$$

$$\Delta_{Lr,allow} = \frac{1.15(5)(177\,lb/ft)(20-0.33\,ft)^4}{384(29{,}000\times10^3\,psi)(36.1\,in^4)}\left(\frac{12\,in}{ft}\right)^3 = 0.66\ in$$

or

$$\Delta_{Lr,allow} = \frac{L}{360} = \frac{20-0.33\,ft}{360}\times\left(\frac{12''}{ft}\right) = 0.66\ in$$

$$\Delta_{Lr,actual} = \frac{1.15(5)(100\,lb/ft)(20-0.33\,ft)^4}{384(29{,}000\times10^3\,psi)(36.1\,in^4)}\left(\frac{12\,in}{ft}\right)^3 = 0.37\ in$$

Since $\Delta_{Lr,actual} = 0.37\,in < \Delta_{Lr,allow} = 0.66\,in$ ok

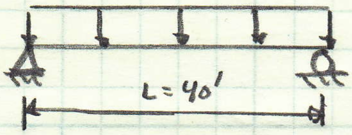
- The span of a K-series joist shall not exceed 24 times its depth

depth = 12 in
Span = 20 ft

$20\,ft \times \left(\frac{12\,in}{ft}\right) < 12\,in \times 24$

$240\,in < 288\,in$ ok

Finding outer Girder Weight:



$w_u = \begin{cases} D_L = 30\,psf \\ S = 25\,psf \end{cases}$ $w = 23\,psf$ $L_r = 20\,psf$

Girder span = 40'
Girder spacing = $\frac{20'}{2} = 10'$

Girder load Approx.

$$W_{DL} = \left[\text{Superimposed DL} + \frac{\text{joist weight}}{\text{joist spacing}} \right] \times (\text{Girder Spacing})$$

$$W_{DL} = \left[30\,psf + \frac{5.7\,lb/ft}{5\,ft} \right] \times (10') = 311.4\ lb/ft$$

$W_S = 25\,psf \times$ Girder spacing
$W_S = 25\,psf \times 10\,ft = 250\ lb/ft$

$W_W = 23\,psf \times$ girder spacing
$W_W = 23\,psf \times 10' = 230\ lb/ft$

Factored Loads:

$W_u = 1.4D = 1.4(311.4\ lb/ft) = 436\ lb/ft$

$W_u = 1.2D + 1.6S + 0.5W = 1.2(311.4\ lb/ft) + 1.6(250\ lb/ft) + 0.5(230\ lb/ft)$

$W_u = 889\ lb/ft$

target capacity $M_u = \frac{W_u L^2}{8} = \frac{(889\ lb/ft)(40')^2}{8} = 178\ k\text{-}ft$

$\phi = 0.9$
$F_y = 50\,ksi$

Design equation

$$Z_x \geq \frac{M_u}{\phi F_y} = \frac{178\ k\text{-}ft}{(0.9)(50\,ksi)}\left(\frac{12\,in}{ft}\right) \qquad Z_x \geq 48\ in^3$$

table 3-2 trial W18x35 $Z_x = 66.5\,in^3$

$W_u = 1.2(311.4\ lb/ft + 35\ lb/ft) + 1.6(250\ lb/ft) + 0.5(230\ lb/ft)$

$W_u = 931\ lb/ft$

$$M_u = \frac{W_u L^2}{8} = \frac{(931\ lb/ft)(40')^2}{8} = 186\ k\text{-}ft$$

W18x35 Capacity:

$$\phi M_p = \phi Z_x F_y = (0.9)(66.5\,in^3)(50\,ksi)\left(\frac{ft}{12''}\right) = 249\ k\text{-}ft$$

Check Compact Section Criteria:

$\phi M_p = 249\ k\text{-}ft > 186\ k\text{-}ft$ ok

$\frac{b_f}{2t_f} = 7.06$ $\frac{h}{t_w} = 53.5$ (from table 1-1)

$\frac{b_f}{2t_f} = 7.06 < 0.38\sqrt{E/F_y} = 9.2$] Flange Compact

$\frac{h}{t_w} = 53.5 < 3.76\sqrt{E/F_y} = 90.5$] Web is compact

outer girder Design
W18x35

Ok $\phi M_n = \phi M_p$

Girder Deflection: W18 × 35 from table 3-3 $I_x = 510 in^4$

$$w_{L_r} = \frac{20psf \times 10ft}{2} = 100 \text{ lb/ft}$$
at 50%

$$\Delta_{L_r} = \frac{5wL^4}{384EI_x} = \frac{5(100 \text{ lb/ft})(40')^4(\frac{12in}{ft})^3}{384(29{,}000 \times 10^3 psi)(510in^4)} = 0.39 \text{ in}$$

Check: $\Delta_{Lr} \leq \frac{L}{360}$ $\Delta_{Lr} \leq \frac{40ft}{360}(\frac{12''}{ft})$ $\Delta_{Lr} \leq 1.33in$

$\Delta_{Lr} = 0.39in < 1.33in$ ok

$$w_D = 311.4 \text{ lb/ft} + 35 \text{ lb/ft} = 346.4 \text{ lb/ft}$$
$$w_{L_r} + w_D = 100 \text{ lb/ft} + 346.4 \text{ lb/ft} = 446.4 \text{ lb/ft}$$

$$\Delta_{D+L_r} = \frac{5wL^4}{384EI_x} = \frac{5(446.4 \text{ lb/ft})(40ft)^4(\frac{12''}{ft})^3}{384(29{,}000 \times 10^3 psi)(510 in^4)} = 1.74 \text{ in}$$

Check:

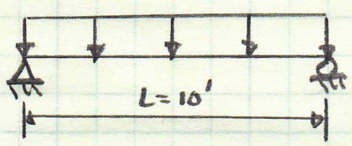
$\Delta_{D+L_r} \leq \frac{L}{240}$ $\Delta_{D+L_r} \leq \frac{40ft}{240}(\frac{12''}{ft})$ $\Delta_{D+L_r} \leq 2in$

$\Delta_{D+L_r} = 1.74in < 2in$ ok

Inner joists: use $D_L = 30psf$ $L_r = 20psf$ $S = 25psf$ $W = 23psf$

Inner Web Steel joist Span = 10'
Inner web steel joist Spacing = 5'

Finding open web steel joist weight for inner joists



$W \left[\begin{array}{l} D_L = 30psf \\ S = 25psf \end{array} \right.$ $W = 23psf$

Factored load:

$W_{TL} = 1.2D + 1.6S + 0.5W$

$W_{TL} = 1.2(30psf \times 5ft) + 1.6(25psf \times 5ft) + 0.5(23psf \times 5ft)$

target load carrying capacity $W_{TL} = 437.5\ lb/ft$

- use LRFD Load tables for K-series joists
$W_{TL} = 437.5\ lb/ft$ Span = 10 ft

Try 10K1 wt = 5.0 lb/ft
allow $W_u = 825\ lb/ft$
live load $W_u = 550\ lb/ft$

$W_u = 1.2(30psf \times 5ft + 5.0\ lb/ft) + 1.6(25psf \times 5ft) + 0.5(23psf \times 5ft)$

$W_{TL} = 443.5\ lb/ft$

ok, since allow $W_u = 825\ lb/ft > W_{TL} = 443.5\ lb/ft$

Check deflection due to unfactored designed live load

$W_{Lr} = 20psf \times 5ft = 100\ lb/ft$

Since, from table $W_u = 550\ lb/ft > W_{Lr} = 100\ lb/ft$ the size is acceptable

Joist Deflection check: $\Delta = \dfrac{1.15(5)wL^4}{384EI}$ $L = (Span - 0.33ft)$

$I = 26.767\, wL^3 10^{-6}$

$I = 26.767(550\ lb/ft)(10 - 0.33ft)^3 10^{-6} = 13.3\ in^4$

$\Delta_{Lr,allow} = \dfrac{1.15(5)(550\ lb/ft)(10-0.33ft)^4}{384(29{,}000 \times 10^3 psi)(13.3 in^4)} \left(\dfrac{12in}{ft}\right)^3 = 0.32\ in$

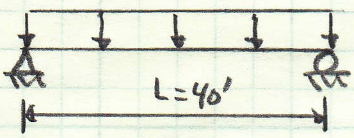
or

$\Delta_{Lr,allow} = \dfrac{L}{360} = \dfrac{(10-0.33ft)}{360}\left(\dfrac{12''}{ft}\right) = 0.32\ in$

$\Delta_{Lr,actual} = \dfrac{1.15(5)(100\ lb/ft)(10-0.33ft)^4}{384(29{,}000 \times 10^3 psi)(13.3 in^4)} \left(\dfrac{12in}{ft}\right)^3 = 0.10\ in$

Since, $\Delta_{Lr,actual} = 0.10\ in < \Delta_{Lr,allow} = 0.32\ in$ ok

Finding Inner Girder Weight:



$W_u \left[\begin{array}{l} D_L = 30\,psf \\ S = 25\,psf \end{array} \right.$ $W = 23\,psf$ $L_r = 20\,psf$

Girder Span = 40'

Girder Spacing $= \frac{20'}{2} + \frac{10'}{2} = 15'$

Girder load Approx.

$$W_{DL} = \left[\text{Superimposed DL} + \frac{\text{Joist Weight}}{\text{Joist Spacing}} \right] \times (\text{Girder tributary width spacing})$$

$$W_{DL} = \left[30\,psf + \frac{5\,lb/ft + 5.7\,lb/ft}{5\,ft} \right] \times (15') = 482.1\ lb/ft$$

$W_S = 25\,psf \times$ Girder Spacing

$W_S = 25\,psf \times 15' = 375\ lb/ft$

$W_W = 23\,psf \times$ Girder Spacing

$W_W = 23\,psf \times 15' = 345\ lb/ft$

Factored loads:

$W_u = 1.4D = 1.4(482.1\ lb/ft) = 674.9\ lb/ft$

$W_u = 1.2D + 1.6S + 0.5W = 1.2(482.1\ lb/ft) + 1.6(375\ lb/ft) + 0.5(345\ lb/ft)$

$W_u = 1351\ lb/ft$

target Capacity $M_u = \frac{W_u L^2}{8} = \frac{(1351\ lb/ft)(40')^2}{8} = 270.4\ k\cdot ft$

Design equation

$\phi = 0.9$

$F_y = 50\,ksi$

$Z_x \geq \frac{M_u}{\phi F_y} = \frac{270.4\ k\cdot ft}{(0.9)(50\,ksi)} \left(\frac{12''}{ft}\right)$ $Z_x \geq 72\,in^3$

table 3-2 trial: W18 x 40 $Z_x = 78.4\,in^3$

$W_u = 1.2(482.1\ lb/ft + 40\ lb/ft) + 1.6(375\ lb/ft) + 0.5(345\ lb/ft)$

$W_u = 1399\ lb/ft$

$M_u = \frac{W_u L^2}{8} = \frac{(1399\ lb/ft)(40')^2}{8} = 280\ k\cdot ft$

W 18 x 40 Capacity

$\phi M_p = \phi Z_x F_y = (0.9)(78.4\,in^3)(50\,ksi)\left(\frac{ft}{12''}\right) = 294\ k\cdot ft$

$\phi M_p = 294\ k\cdot ft > 280\ k\cdot ft$ ok

Check Compact Section criteria:

$\frac{b_f}{2t_f} = 5.73$ $\frac{h}{t_w} = 50.9$ (from table 1-1)

$\frac{b_f}{2t_f} = 5.73 < 0.38\sqrt{E/F_y} = 9.2$] Flange compact

$\frac{h}{t_w} = 50.9 < 3.76\sqrt{E/F_y} = 90.5$] Web compact

ok

Inner girder Design W 18 x 40

$\phi M_n = \phi M_p$

Girder Deflection: W 18X40

From table 3-3
$I_x = 612 in^4$

$W_{Lr} = \frac{20 psf \times 15 ft}{2} = 150\ lb/ft$
at 50%

$\Delta_{Lr} = \frac{5wL^4}{384EI_x} = \frac{5(150 lb/ft)(40')^4 \left(\frac{12''}{ft}\right)^3}{384(29{,}000\times10^3 psi)(612 in^4)} = 0.49 in$

Check:

$\Delta_{Lr} \le \frac{L}{360}$ $\Delta_{Lr} \le \frac{40 ft}{360}\left(\frac{12''}{ft}\right)$ $\Delta_{Lr} \le 1.33 in$

$\Delta_{Lr} = 0.49 in < 1.33 in$ ok

$W_D = 482.1 lb/ft + 40 lb/ft = 522.1\ lb/ft$
$W_{Lr} + W_D = 150 lb/ft + 522.1 lb/ft = 672.1\ lb/ft$ 676

$\Delta_{D+L_r} = \frac{5wL^4}{384EI_x} = \frac{5(672.1 lb/ft)(40 ft)^4\left(\frac{12''}{ft}\right)^3}{384(29{,}000\times10^3 psi)(612 in^4)} = 2.2\ in$

check

$\Delta_{D+L_r} \le \frac{L}{240}$ $\Delta_{D+L_r} \le \frac{40 ft}{240}\left(\frac{12 in}{ft}\right)$ $\Delta_{D+L_r} \le 2 in$

$\Delta_{D+L_r} = 2.2 in > 2 in$ NO

16x45
309
584
W21x44
843
358

trial: $Z_x > 78.4\ in^3$
table 3-2 W21x44 $Z_x = 95.4\ in^3$

$W_u = 1.2(482.1 lb/ft + 44 lb/ft) + 1.6(375 lb/ft) + 0.5(345 lb/ft)$
$W_u = 1404\ lb/ft$

$M_u = \frac{W_u L^2}{8} = \frac{(1404 lb/ft)(40 ft)^2}{8} = 281\ k\cdot ft$

W21x44 capacity:
$\phi M_p = \phi Z_x F_y = 0.9(95.4 in^3)(50 ksi)\left(\frac{ft}{12''}\right) = 358\ k\cdot ft$

$\phi M_p = 358 k\cdot ft > 281 k\cdot ft$ ok

Inner Girder Design
W21x44

Check compact Section Criteria:

$\frac{b_f}{2t_f} = 7.22$ $\frac{h}{t_w} = 53.6$ (from table 1-1)

$\frac{b_f}{2t_f} = 7.22 < 0.38\sqrt{E/F_y} = 9.2$] Flange Compact

Ok $\phi M_n = \phi M_p$

$\frac{h}{t_w} = 53.6 < 3.76\sqrt{E/F_y} = 90.5$) Web compact

Girder Deflection $W21 \times 44$

from table 3-3
$I_x = 843 \text{ in}^4$

W_{Lr} at 50% $= \dfrac{20 \text{ psf} \times 15 \text{ ft}}{2} = 150 \text{ lb/ft}$

$$\Delta_{Lr} = \frac{5wL^4}{384 EI_x} = \frac{5(150 \text{ lb/ft})(40')^4 \left(\frac{12 \text{ in}}{\text{ft}}\right)^3}{384(29{,}000 \times 10^3 \text{ psi})(843 \text{ in}^4)} = 0.35 \text{ in}$$

Check:

$$\Delta_{Lr} \leq \frac{L}{360} = \frac{40 \text{ ft}}{360}\left(\frac{12''}{\text{ft}}\right) \qquad \Delta_{Lr} \leq 1.33 \text{ in}$$

$$\Delta_{Lr} = 0.35 \text{ in} < 1.33 \text{ in}$$

$W_D = 482.1 \text{ lb/ft} + 44 \text{ lb/ft} = 526.1 \text{ lb/ft}$

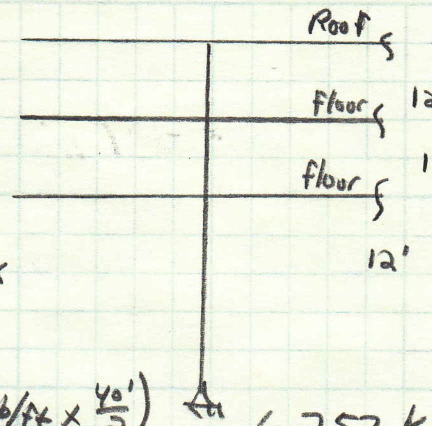
$W_{Lr} + W_D = 150 \text{ lb/ft} + 526.1 \text{ lb/ft} = 676.1 \text{ lb/ft}$

$$\Delta_{D+Lr} = \frac{5wL^4}{384 EI_x} = \frac{5(676.1 \text{ lb/ft})(40 \text{ ft})^4 \left(\frac{12 \text{ in}}{\text{ft}}\right)^3}{384(29{,}000 \times 10^3 \text{ psi})(843 \text{ in}^4)} = 1.6 \text{ in}$$

Check: $\Delta_{D+Lr} \leq \dfrac{L}{240} \quad \dfrac{40 \text{ ft}}{240}\left(\dfrac{12''}{\text{ft}}\right) \qquad \Delta_{D+Lr} \leq 2 \text{ in}$

$$\Delta_{D+Lr} = 1.6 \text{ in} < 2 \text{ in} \quad \text{ok}$$

Design of columns: (#1)



Loads:

Roof Deadload $= 30psf\left(\frac{20'}{2}\right)\left(\frac{40'}{2}\right) = 6.0k$

roof live load $= 20psf\left(\frac{20'}{2}\right)\left(\frac{40'}{2}\right) = 4.0k$

roof snow load $= 25psf\left(\frac{20'}{2}\right)\left(\frac{40'}{2}\right) = 5.0k$

Roof loads

$D_L = 6.0k + \left(5.7 \text{ lb/ft} \times \frac{20'}{2}\right) + \left(35 \text{ lb/ft} \times \frac{40'}{2}\right) = 6.757k$

$L_r = 4.0k$

$S = 5.0k$

Floor Loads: (2nd and 3rd Floor)

$D_L = 20psf\left(\frac{20'}{2}\right)\left(\frac{40'}{2}\right) + \left(6.7 \text{ lb/ft} \times \frac{20'}{2}\right) + \left(35 \text{ lb/ft} \times \frac{40'}{2}\right) = 4.767k$

$L_L = 130psf\left(\frac{20'}{2}\right)\left(\frac{40'}{2}\right) = 26k$

Total loads:

Dead load from 2 floors and roof $= 6.757k + 4.767k + 4.767k = 16.29k$

Floor Live Load $= 26k + 26k = 52k$

Roof snow load $= 5.0k$

Roof live load $= 4.0k$

load combinations

$1.4D = 1.4(16.29k) = 22.8k$

$1.2D + 1.6L + 0.5S = 1.2(16.29k) + 1.6(52k) + 0.5(5k) = 105.3k$

from table 4-1 : $KL = 12ft$ $K = 1$ for Gravity columns

use W10 x 33 $\phi P_n = 292k$

from table 1-1 : W10X33

$r_x = 4.19"$ $A_g = 9.71 in^2$

$r_y = 1.94"$

$\frac{K_x L}{r_x} = \frac{(1.0)(L=12')}{4.19"} \times \left(\frac{12"}{ft}\right) = 34.4"$

$\frac{K_y L}{r_y} = \frac{(1.0)(L=12')}{1.94"} \times \left(\frac{12"}{ft}\right) = 74.2"$ => Governs

$\frac{K_y L}{r_y} = 74.2" \leq 4.71\sqrt{\frac{E=29000}{Fy=50}} = 113.4'$ => use $F_{cr} = \left[0.658^{(F_y/F_e)}\right]F_y$

$F_e = \frac{\pi^2 E}{(KL/r_y)^2} = \frac{\pi^2(29000 ksi)}{(74.2")^2} = 51.99 ksi$

Ratio $F_y/F_e = 50/51.99 = 0.96$

$F_{cr} = \left[0.658^{(0.96)}\right](F_y = 50ksi) = 33.5 ksi$

Column design Capacity

$\phi P_n = (\phi = 0.9)(F_{cr} = 33.5 ksi)(A_g = 9.71 in^2)$

$\phi P_n = 293k$ ok

Check compact criteria ; W10X33 (table 1-1)

$\frac{b_f}{2t_f} = 9.15$ $\frac{h}{t_w} = 27.1$

$\frac{b_f}{2t_f} = 9.15 < 0.38\sqrt{E/F_y} = 9.2$] Flange compact

$\frac{h}{t_w} = 27.1 < 3.76\sqrt{E/F_y} = 90.5$] web compact

o/k

Design of Columns cont: (#2)

Roof loads

$D_L = 30psf\left(\frac{20'}{2}+\frac{10'}{2}\right)\left(\frac{40'}{2}\right) + \left(5.7\frac{lb}{ft} \times \frac{20'}{2}\right) + \left(5.0\, lb/ft \times \frac{10'}{2}\right) + \left(44\, lb/ft \times \frac{40'}{2}\right)$

$D_L = 9.96\, k$

$L_r = 20psf\left(\frac{20'}{2}+\frac{10'}{2}\right)\left(\frac{40'}{2}\right) = 6k$

$S = 25psf\left(\frac{20'}{2}+\frac{10'}{2}\right)\left(\frac{40'}{2}\right) = 7.5k$

Floor loads : (2nd + 3rd Floor)

$D_L = 20psf\left(\frac{20'}{2}+\frac{10'}{2}\right)\left(\frac{40'}{2}\right) + \left(6.7\, lb/ft \times \frac{20'}{2}\right) + \left(5.0\, lb/ft \times \frac{10'}{2}\right) + \left(62\, lb/ft \times \frac{40'}{2}\right)$

$D_L = 7.3k$

$L_L = 130psf\left(\frac{20'}{2}+\frac{10'}{2}\right)\left(\frac{40'}{2}\right) = 39k$

Total Loads:

D_L from 2 Floors and Roof = 9.96k + 7.3k + 7.3k = 24.56k

Floors Live load = 39k + 39k = 78k

Roof snow load = 7.5k

Roof L_r (live load = 6.0k

load combinations:

$1.4D = 1.4(24.56k) = 34.4\, k$

$1.2D + 1.6L + 0.5S = 1.2(24.56k) + 1.6(78k) + 0.5(7.5k) = 158\, k$

from table 4-1: KL = 12 ft k=1 for gravity columns

Use W10X33 $\phi P_n = 292\, k$

from table 1-1: W10X33

$r_x = 4.19''$ $A_g = 9.71\, in^2$

$r_y = 1.94''$

$\frac{K_x L}{r_x} = \frac{(1.0)(L=12')}{4.19''} \times \left(\frac{12''}{ft}\right) = 34.4''$

$\frac{K_y L}{r_y} = \frac{(1.0)(L=12')}{1.94''} \times \left(\frac{12''}{ft}\right) = 74.2''$ => Governs

$\frac{K_y L}{r_y} = 74.2'' \leq 4.71\sqrt{\frac{E=29000}{F_y=50}} = 113.4''$ => Use $F_{cr} = \left[0.658^{(F_y/F_e)}\right]F_y$

$F_e = \frac{\pi^2 E}{(KL/r_y)^2} = \frac{\pi^2(29000)}{(74.2'')^2} = 51.99\, ksi$

Ratio $F_y/F_e = 50/51.99 = 0.96$

$F_{cr} = \left[0.658^{(0.96)}\right](F_y = 50ksi) = 33.5\, ksi$

$\phi P_n = (\phi = 0.9)(F_{cr} = 33.5ksi)(A_g = 9.71 in^2) = 293k$

$\phi P_n = 293k > 158\, k$ ok

Design of Columns cont: (#3)

roof loads

$D_L = 30psf\left(\frac{40'}{2}+\frac{40'}{2}\right)\left(\frac{20'}{2}\right) + \left(5.7\,lb/ft \times \frac{20'}{2}\right) + \left(35\,lb/ft \times \frac{40'}{2}\right) + \left(35\,lb/ft \times \frac{40'}{2}\right)$

$D_L = 13.5\,k$

$L_r = 20psf\left(\frac{40'}{2}+\frac{40'}{2}\right)\left(\frac{20'}{2}\right) = 8.0\,k$

$S = 25psf\left(\frac{40'}{2}+\frac{40'}{2}\right)\left(\frac{20'}{2}\right) = 10.0\,k$

Floor loads: (2nd + 3rd Floor)

$D_L = 20psf\left(\frac{40'}{2}+\frac{40'}{2}\right)\left(\frac{20'}{2}\right) + \left(6.7\,lb/ft \times \frac{20'}{2}\right) + \left(35\,lb/ft \times \frac{40'}{2}\right) + \left(35\,lb/ft \times \frac{40'}{2}\right)$

$D_L = 9.5\,k$

$L_L = 130psf\left(\frac{40'}{2}+\frac{40'}{2}\right)\left(\frac{20'}{2}\right) = 52\,k$

Total loads

D_L from 2 floors and roof $= 13.5k + 9.5k + 9.5k = 32.5k$

Floors live load $= 52k + 52k = 104k$

Roof snow load $= 10.0k$

Roof Live load $= 8.0k$

load combinations:

$1.4D = 1.4(32.5k) = 45.5k$

$1.2D + 1.6L + 0.5S = 1.2(32.5k) + 1.6(104k) + 0.5(10k) = 210.5k$

From table 4-1: $KL = 12ft$ $K=1$ for Gravity columns

use 10×33 $\phi P_n = 292k$

from table 1-1: W10×33

$r_x = 4.19"$ $A_g = 9.71\,in^2$

$r_y = 1.94"$

$\frac{K_x L}{r_x} = \frac{(1.0)(L=12')}{4.19"} \times \left(\frac{12"}{ft}\right) = 34.4"$

$\frac{K_y L}{r_y} = \frac{(1.0)(L=12')}{1.94"} \times \left(\frac{12"}{ft}\right) = 74.2" \Rightarrow$ Governs

$\frac{K_y L}{r_y} = 74.2" \leq 4.71\sqrt{\frac{E=29000}{F_y=50}} = 113.4" \Rightarrow$ use $F_{cr} = \left[0.658^{(F_y/F_e)}\right]F_y$

$F_e = \frac{\pi^2 E}{(KL/r_y)^2} = \frac{\pi^2(29000\,ksi)}{(74.2")^2} = 51.99\,ksi$

Ratio $F_y/F_e = 50/51.99 = 0.96$

$F_{cr}\left[0.658^{(0.96)}\right](F_y = 50\,ksi) = 33.5\,ksi$

$\phi P_n = (\phi=0.9)(F_{cr}=33.5\,ksi)(A_g=9.71\,in^2) = 293k$

$\phi P_n = 293k > 210.5k$ ok

Design of columns cont. : (#4)

roof loads 40 15

$$D_L = 30\,psf\left(\frac{40'}{2}+\frac{40'}{2}\right)\left(\frac{20'}{2}+\frac{10'}{2}\right) + \left(5.7\,lb/ft \times \frac{20'}{2}\right) + \left(5.0\,lb/ft \times \frac{10'}{2}\right) + \left(44\,lb/ft \times \frac{40'}{2}\right) + \left(44\,lb/ft \times \frac{40'}{2}\right)$$

$D_L = 19.8\ k$

$L_r = 20\,psf\left(\frac{40'}{2}+\frac{40'}{2}\right)\left(\frac{20'}{2}+\frac{10'}{2}\right) = 12\,k$

$S = 25\,psf\left(\frac{40'}{2}+\frac{40'}{2}\right)\left(\frac{20'}{2}+\frac{10'}{2}\right) = 15\,k$

Floor Loads:

$$D_L = 20\,psf\left(\frac{40'}{2}+\frac{40'}{2}\right)\left(\frac{20'}{2}+\frac{10'}{2}\right) + \left(6.7\,lb/ft \times \frac{20'}{2}\right) + \left(5.0\,lb/ft \times \frac{10'}{2}\right) + \left(62\,lb/ft \times \frac{40'}{2}\right) + \left(62\,lb/ft \times \frac{40'}{2}\right)$$

$D_L = 14.6\ k$

$L_L = 130\,psf\left(\frac{40'}{2}+\frac{40'}{2}\right)\left(\frac{20'}{2}+\frac{10'}{2}\right) = 78k$

Total Loads:

D_L from 2 floors and roof = $19.8k + 14.6k + 14.6k = 49k$

Floors live load = $78k + 78k = 156k$

Roof snow load = $15k$

Roof live load = $12k$

load combinations

$1.4D = 1.4(49k) = 68.6\ k$

$1.2D + 1.6L + 0.5S = 1.2(49k) + 1.6(156k) + 0.5(15k) = 316\ k$

From table 4-1 : $KL = 12\,ft$ $\quad k=1$ for gravity column

use W12 x 40 $\quad \phi P_n = 352\ k$

from table 1-1 :

$r_x = 5.13''$ $\quad A_g = 11.7\ in^2$

$r_y = 1.94''$

$\frac{K_x L}{r_x} = \frac{(1.0)(L=12')}{5.13''} \times \left(\frac{12''}{ft}\right) = 28.1''$

$\frac{K_y L}{r_y} = \frac{1.0\,(L=12')}{1.94''} \times \left(\frac{12''}{ft}\right) = 74.2''$ ⇒ Governs

$\frac{K_y L}{r_y} = 74.2'' \le 4.71\sqrt{\frac{E=29000}{F_y=50}} = 113.4''$ ⇒ use $F_{cr} = \left[0.658^{(F_y/F_e)}\right] F_y$

$F_e = \frac{\pi^2 E}{(KL/r_y)^2} = \frac{\pi^2 (29000\,ksi)}{(74.2'')^2} = 51.99\ ksi$

Ratio $F_y/F_e = 50/51.99 = 0.96$

$F_{cr} = \left[0.658^{(0.96)}\right](F_y = 50\,ksi) = 33.5\,ksi$

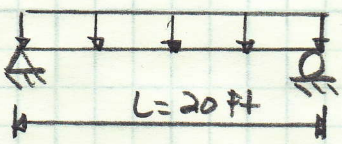
$\phi P_n = (\phi = 0.9)(F_{cr} = 33.5\,ksi)(A_g = 11.7\,in^2) = 353\ k$

$\phi P_n = 353\,k > 316\,k$ ok

W-Shape End beams on the end column lines:

<u>Office W-Shape End beam:</u> 2nd + 3rd floor

$W_u = \left[\begin{matrix} DL \\ LL \end{matrix}\right.$



$W_{DL} = \left[23psf + 145pcf\left(\frac{3''}{12''/ft}\right)\right] \times 5.0\ ft = 296.3\ lb/ft$

$W_{LL} = 50\ psf \times 5.0\ ft = 250\ lb/ft$

<u>Factored loads:</u>

$W_u = 1.4D = 1.4(296.3\ lb/ft) = 414.8\ lb/ft$

$W_u = 1.2D + 1.6L = 1.2(296.3\ lb/ft) + 1.6(250\ lb/ft) = 755.56\ lb/ft$

Target Capacity $M_u = \frac{W_u L^2}{8} = \frac{(755.56\ lb/ft)(20\ ft)^2}{8} = 37.8\ k\cdot ft$

$\phi = 0.9$
$F_y = 50ksi$

Design equation

$Z_x \geq \frac{M_u}{\phi F_y} = \frac{18.9\ k\cdot ft\left(\frac{12''}{ft}\right)}{(0.9)(50ksi)}$ $Z_x \geq 10.1 in^3$

table 3-2: trial: W10X15 $Z_x = 16\ in^3$

$W_u = 1.2(296.3\ lb/ft + 15\ lb/ft) + 1.6(250\ lb/ft)$

$W_u = 714.3\ lb/ft$

$M_u = \frac{W_u L^2}{8} = \frac{(714.3\ lb/ft)(20\ ft)^2}{8} = 35.7\ k\cdot ft$

W10 X 15 Capacity:

$\phi M_p = \phi Z_x F_y = (0.9)(16in^3)(50ksi)\left(\frac{ft}{12''}\right) = 60\ k\cdot ft$

Check compact Section Criteria: $\phi M_p = 60\ k\cdot ft > 35.7\ k\cdot ft$ ok

$\frac{b_f}{2t_f} = 7.41$ $\frac{h}{t_w} = 38.5$ from (table 1-1)

⇒ Outer end beam W10x15

$\left.\begin{matrix} \frac{b_f}{2t_f} = 7.41 < 9.2 \\ \frac{h}{t_w} = 38.5 < 90.5 \end{matrix}\right]$ Flange Compact, Web is compact

Ok $\phi M_n = \phi M_p$

beam deflection: W10X15 from table 3-3

$I_x = 68.9\ in^4$

$E = 29{,}000\ ksi$

$$W_{LL\ at\ 50\%} = \frac{50psf \times 5ft}{2} = 125\ lb/ft$$

$$\Delta_{LL} = \frac{5wL^4}{384EI_x} = \frac{5(125 lb/ft)(20ft)^4\left(\frac{12 in}{ft}\right)^3}{384(29{,}000\times10^3 psi)(68.9 in^4)} = 0.225\ in$$

Check $\Delta_{LL} \le \frac{L}{360} = \frac{20ft\left(\frac{12''}{ft}\right)}{360} = 0.67 in$

$\Delta_{LL} = 0.23 in < 0.67 in$ ok

$W_D = 296.3\ lb/ft + 15 lb/ft = 311.3 lb/ft$

$W_{LL+DL} = 125 lb/ft + 311.3 lb/ft = 436.3 lb/ft$

$$\Delta_{D+L} = \frac{5wL^4}{384EI_x} = \frac{5(436.3 lb/ft)(20ft)^4\left(\frac{12''}{ft}\right)^3}{384(29{,}000\times10^3 psi)(68.9 in^4)} = 0.79\ in$$

Check:

$\Delta_{D+L} \le \frac{L}{240} = \frac{20ft\left(\frac{12''}{ft}\right)}{240}$ $\Delta_{D+L} \le 1''$

$\Delta_{D+L} = 0.79 in < 1''$ ok

Corridor W-Shape End beam:

$$W_{DL} = \left[23psf + 145pcf\left(\frac{3''}{12''/ft}\right)\right] \times 2.5ft = 148.13 lb/ft$$

$W_{LL} = 80psf \times 2.5ft = 200 lb/ft$

$W_u = 1.2(148.13 lb/ft) + 1.6(200 lb/ft) = 497.8 lb/ft$

Target Capacity $M_u = \frac{W_u L^2}{8} = \frac{(497.8 lb/ft)(10ft)^2}{8} = 6.3\ k\cdot ft$

Design Equation:

$$Z_x \ge \frac{M_u}{\phi F_y} = \frac{6.3 k\cdot ft \times\left(\frac{12''}{ft}\right)}{0.9(50ksi)}$$ $Z_x \ge 2.0\ in^3$

table 3-2 trial W10X12 $Z_x = 12.6\ in^3$

$W_u = 1.2(148.13 lb/ft + 12 lb/ft) + 1.6(200 lb/ft)$

$W_u = 512.2\ lb/ft$

$M_u = \frac{W_u L^2}{8} = \frac{(512.2 lb/ft)(10ft)^2}{8} = 6.4\ k\cdot ft$

W10X12 Capacity

$$\phi M_p = (\phi = 0.9)(Z_x = 12.6 in^3)(F_y = 50ksi)\left(\frac{ft}{12''}\right) = 47.3\ k\cdot ft$$

$\phi M_p = 47.3 k\cdot ft > 6.4 k\cdot ft$ ok beam deflection : W10X12 from table 3-3

$I_x = 53.8 in^4$

$E = 29,000 ksi$

$$w_{LL\ at\ 5ft} = \frac{80psf \times 2.5ft}{2} = 100\ lb/ft$$

$$\Delta_{LL} = \frac{5wL^4}{384EI_x} = \frac{5(100lb/ft)(10ft)^4\left(\frac{12in}{ft}\right)^3}{384(29,000\times10^3psi)(53.8in^4)} = 0.015in$$

Check $\Delta_{LL} \leq \frac{L}{360} = \frac{10ft\left(\frac{12''}{ft}\right)}{360} = 0.33in$

$\Delta_{LL} = 0.015in < 0.33in$ ok

$W_D = 148.13\ lb/ft + 12lb/ft = 160.3\ lb/ft$

$W_{LL+DL} = 100lb/ft + 160.3\ lb/ft = 260.3lb/ft$

$$\Delta_{D+L} = \frac{5wL^4}{384EI_x} = \frac{5(260.3lb/ft)(10ft)^4\left(\frac{12''}{ft}\right)^3}{384(29,000\times10^3psi)(53.8in^4)} = 0.04in$$

Check:

$$\Delta_{D+L} \leq \frac{L}{240} = \frac{10ft\left(\frac{12''}{ft}\right)}{240} = 0.5in$$

$\Delta_{D+L} = 0.04in < 0.5in$ ok

Office W-Shape End beam: Roof

$$W_{DL} = \left[23psf + 10psf + 145pcf\left(\frac{3''}{12''/ft}\right)\right]\times 5ft = 346.25\ lb/ft$$

$W_S = 25psf \times 5ft = 125\ lb/ft$

$W_W = 23psf \times 5ft = 115lb/ft$

$W_u = 1.2(346.25lb/ft) + 1.6(125lb/ft) + 0.5(115lb/ft) = 673\ lb/ft$

target capacity: $M_u = \frac{W_uL^2}{8} = \frac{(673lb/ft)(20ft)^2}{8} = 33.7k\cdot ft$

$$Z_x \geq \frac{M_u}{\phi F_y} = \frac{33.7k\cdot ft\left(\frac{12''}{ft}\right)}{(0.9)(F_y=50ksi)} \qquad Z_x \geq 9.0in^3$$

table 3-2 trial: W10X15 $Z_x = 16.0in^3$

$W_u = 1.2(346.25lb/ft + 15lb/ft) + 1.6(125lb/ft) + 0.5(115lb/ft)$

$W_u = 691lb/ft$

$$M_u = \frac{W_uL^2}{8} = \frac{(691\ lb/ft)(20ft)^2}{8} = 35\ k\cdot ft$$

W8X15 Capacity

$\phi M_p = 60k\cdot ft > 35k$

$\phi M_p = (\phi=0.9)(Z_x=16.0in^3)(F_y=50ksi)\left(\frac{ft}{12''}\right) = 51k\cdot ft$

ok

beam deflection: W10X15

$I_x = 68.9\ in^4$
$E = 29{,}000\ ksi$
(table 3-3)

w_{Lr} at 50%: $= \frac{20psf \times 5ft}{2} = 50\ lb/ft$

$$\Delta_{Lr} = \frac{5wL^4}{384EI_x} = \frac{5(50\ lb/ft)(20')^4\left(\frac{12''}{ft}\right)^3}{384(29{,}000\times10^3 psi)(68.9 in^4)} = 0.10\ in$$

Check $\Delta_{Lr} \le \frac{L}{360} = \frac{20ft\left(\frac{12''}{ft}\right)}{360} = 0.67\ in$

$\Delta_{Lr} = 0.10\ in < 0.67\ in$ ok

$w_D = 346.25\ lb/ft + 15\ lb/ft = 361.25$

$w_{Lr+D} = 50\ lb/ft + 361.25\ lb/ft = 411.25\ lb/ft$

$$\Delta_{D+Lr} = \frac{5wL^4}{384EI_x} = \frac{5(411.25\ lb/ft)(20ft)^4\left(\frac{12''}{ft}\right)^3}{384(29{,}000\times10^3 psi)(68.9 in^4)} = 0.74\ in$$

Check $\Delta_{D+Lr} \le \frac{L}{240} = \frac{20'\times\left(\frac{12}{ft}\right)}{240} = 1 in$

$\Delta_{D+Lr} = 0.74 in < 1 in$ ok

Corridor W-Shape End beam: Roof

$$w_{DL} = \left[23psf + 10psf + 145pcf\left(\frac{3''}{12''/ft}\right)\right] \times 2.5ft = 173.25\ lb/ft$$

$w_S = 25psf \times 2.5ft = 62.5\ lb/ft$

$w_W = 23psf \times 2.5\ ft = 57.5\ lb/ft$

$$w_u = 1.2(173.25\ lb/ft) + 1.6(62.5\ lb/ft) + 0.5(57.5\ lb/ft) = 336.7\ lb/ft$$

target capacity: $M_u = \frac{w_u L^2}{8} = \frac{(336.7\ lb/ft)(10')^2}{8} = 4.2\ k\cdot ft$

$$Z_x \ge \frac{M_u}{\phi F_y} = \frac{4.2\ k\cdot ft\left(\frac{12''}{ft}\right)}{(0.9)(50ksi)} \qquad Z_x \ge 1.2\ in^3$$

table 3-2: trial: W10X12 $Z_x = 12.6\ in^3$

$$w_u = 1.2(173.25\ lb/ft + 12\ lb/ft) + 1.6(62.5\ lb/ft) + 0.5(57.5\ lb/ft)$$
$w_u = 352\ lb/ft$

$$M_u = \frac{w_u L^2}{8} = \frac{(352\ lb/ft)(10')^2}{8} = 4.4\ k\cdot ft$$

W10X12 capacity

$$\phi M_p = (\phi = 0.9)(Z_x = 12.6\ in^3)(F_y = 50ksi)\left(\frac{ft}{12''}\right) = 47.3\ k\cdot ft$$

$\phi M_p = 47.3\ k\cdot ft > 4.4\ k\cdot ft$
ok

beam deflection: $W10 \times 12$

from table 3-3
$I_x = 53.8 \, in^4$
$E = 29{,}000 \, ksi$

$$W_{Lr} \text{ at } 50\% = \frac{20 psf \times 2.5 ft}{2} = 25 \, lb/ft$$

$$\Delta_{Lr} = \frac{5wL^4}{384 EI_x} = \frac{5(25 lb/ft)(10')^4 \left(\frac{12''}{ft}\right)^3}{384 (29{,}000 \times 10^3 psi)(53.8 \, in^4)} = 0.004 \, in$$

Check: $\Delta_{Lr} \leq \frac{L}{360} = \frac{10'\left(\frac{12''}{ft}\right)}{360} = 0.33 \, in$

$$\Delta_{Lr} = 0.004 \, in < 0.33 \, in \quad \underline{\underline{ok}}$$

$$W_D = 173.25 \, lb/ft + 12 \, lb/ft = 185.25 \, lb/ft$$

$$W_{Lr+D} = 25 \, lb/ft + 185.25 \, lb/ft = 210.25 \, lb/ft$$

$$\Delta_{D+Lr} = \frac{5wL^4}{384 EI_x} = \frac{5(210.25 lb/ft)(10')^4 \left(\frac{12''}{ft}\right)^3}{384 (29{,}000 \times 10^3 psi)(53.8 \, in^4)} = 0.03 \, in$$

Check $\Delta_{D+Lr} \leq \frac{L}{240} = \frac{10 ft \left(\frac{12''}{ft}\right)}{240} = 0.5 \, in$

$$\Delta_{D+Lr} = 0.03 \, in < 0.5 \, in \quad \underline{\underline{ok}}$$

Steel Joist-to-Girder Connection — 2nd + 3rd floor

Outer Girder: W18X35 — $t_w = 0.300''$, $t_f = 0.425''$, $b_f = 6.0\,in$ (table 1.1 AISC Manual)

AISC J2.2B: $t < \frac{1}{4}''$ Minimum

Outer Joist: 14K4

Wt. 6.7 lb/ft

$a_{min} = t \Rightarrow$ Fillet weld $\frac{1}{8}''$

Joist end Reaction: (For outer Girder) $a_{min} = \frac{1}{8}''$

$W_u = [1.2D + 1.6L] \times$ tributary width

A36 base Metal
$F_y = 36\,ksi$
$F_u = 58\,ksi$

$$D_L = 23psf + 145pcf\left(\frac{3''}{12''/ft}\right) = 59.25psf$$

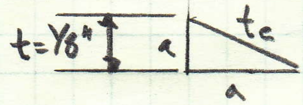
$$W_u = 1.2(59.25psf \times 5ft + 6.7lb/ft) + 1.6(50psf \times 5ft)$$

$$W_u = 0.764\ K/ft$$

$$V_u = \frac{W_u L}{2} = \frac{(0.764\,k/ft)(20ft)}{2} = 7.64\ K$$

Weld size

$$t_e = 0.707a = 0.707(\tfrac{1}{8}'') = 0.0884''$$



Use E70 series electrode $\Rightarrow F_{exx} = 70ksi$

$$\phi R_n = \phi\, 0.6\, F_{exx}\, t_e$$

$$\phi R_n = (\phi=0.75)(0.6)(70ksi)(0.0884'') = 2.78\ k/in$$

Check Stresses in Base Metal ($t = \frac{1}{8}''$ governs over t_w)

Shear yield: $\phi R_n = \phi\, 0.6\, F_y\, t$

$$(\phi=1.0)(0.6)(F_y=36ksi)(t=\tfrac{1}{8}'') = 2.7\ k/in \quad \underline{\underline{ok}}$$

Shear Rupture: $\phi R_n = \phi\, 0.6\, F_u\, t$

$$\phi R_n = (\phi=0.75)(0.6)(F_u=58ksi)(t=\tfrac{1}{8}'') = 3.26\ k/in \quad \underline{\underline{ok}}$$

Required weld Length: $L_w = \dfrac{7.64K}{\phi R_n = 2.7\,k/in} = 2.83'' \Rightarrow$ say 4" weld

$\phi R_n = (2.7\frac{k}{in})(4'') = 10.8k$

$\phi R_n = 10.8k > V_u = 7.64\,k$ $\underline{\underline{ok}}$

$\Rightarrow (2)\ \frac{1}{8}'' \times 2''$ long weld

Steel Joist - to - Girder Connection 2nd + 3rd Floor

Inner Girder: W21x62 $t_w = 0.400"$ $b_f = 8.24"$ (table 1.1 AISC Manual)

Outer joist: 14K4 wt. 6.7 lb/ft $t_f = 0.615"$

Inner joist: 10K1 wt: 5.0 lb/ft

AISC J2.2B
$t < 1/4"$
$a_{min} = t = 1/8" \Rightarrow$ Minimum Fillet weld = 1/8"

Joist end Reaction: (for Inner Girder)

$W_u = 1.2D + 1.6L$

$D_L = 23psf + 145pcf\left(\frac{3"}{12"/ft}\right) = 59.25\ psf$

$W_u = 1.2(59.25psf \times 5ft + 6.7lb/ft + 5lb/ft) + 1.6(130psf \times 5ft)$

$W_u = 1.41\ K/ft$

$V_u = \frac{W_u L}{2} = \frac{(1.41\ K/ft)(20\ ft)}{2} = 14.1\ K$

$t_e = 0.707a = 0.707(1/8") = 0.0884"$

Use E70 electrode $\Rightarrow F_{exx} = 70\ ksi$

$\phi R_n = \phi\, 0.6\, F_{exx}\, t_e = (\phi = 0.75)(0.6)(70ksi)(0.0884") = 2.78\ k/in$

Check Stresses in Base Metal ($t = 1/8"$ governs over t_w)

Shear yield: $\phi R_n = \phi\, 0.6\, F_y\, t$

$\phi R_n = (\phi = 1.0)(0.6)(F_y = 36ksi)(t = 1/8") = 2.7\ k/in$ ok

Shear Rupture: $\phi R_n = \phi(0.6)\, F_u\, t$ ok

$\phi R_n = (\phi = 0.75)(0.6)(F_u = 58ksi)(t = 1/8") = 3.26\ k/in$

Required weld Length: $L_w = \frac{14.1\ k}{2.7 k/in} = 5.22" \Rightarrow$ Say 6" weld

$\phi R_n = (2.7\frac{k}{in})(6") = 16.2k$
$\phi R_n = 16.2k > V_u = 14.1k$ ok $\Rightarrow$ (2) 1/8" x 3" long weld per joist

Steel Joist - to - Girder Connection : Roof

Outer Girder : W18X35 $t_w = 0.300''$ $b_f : 6.0$ in
table 1.1
Outer Joist : 12K3 wt. 5.7 lb/ft
$t_f = 0.425''$

$a_{min} = t = 1/8''$ ⇒ Min. Fillet Weld

Joist End Reaction: (for outer Girder)

$D_L = 23psf + 10psf + 145pcf\left(\frac{3''}{12''/ft}\right) = 69.25psf$

$W_u = 1.2D + 1.6S + 0.5W$ S: 25 psf
W: 23psf

$W_u = 1.2(69.25psf \times 5ft + 5.7 lb/ft) + 1.6(25psf \times 5ft) + 0.5(23psf \times 5ft)$

$W_u = 0.680\ k/ft$

$$V_u = \frac{W_u L}{2} = \frac{(0.680\ k/ft)(20ft)}{2} = 6.8\ k$$

$t_e = 0.707a = 0.707(1/8'') = 0.0884''$

$\phi R_n = (\phi = 0.75)(0.6)(70ksi)(0.0884'') = 2.78\ k/in$

Check stresses in Base Metal ($t = 1/8''$ governs over t_w)

Shear yield: $\phi R_n = (\phi = 1.0)(0.6)(F_y = 36ksi)(t = 1/8'') = 2.7\ k/in$

Shear Rupture: $\phi R_n = (\phi = 0.75)(0.6)(F_u = 58ksi)(t = 1/8'') = 3.26\ k/in$

Weld Length: $L_w = \frac{6.8K}{\phi R_n = 2.7\ k/in} = 2.52''$ ⇒ say 4" weld

⇒ (2) $\frac{1}{8}''$ x 2" long weld

$\phi R_n = (2.7\ k/in)(4'') = 10.8k$

$\phi R_n = 10.8\ K > V_u = 6.8k$
ok

Steel Joist - to - Girder Connection : Roof

Inner Girder : W21X44 $t_w = 0.350''$ $b_f = 6.5$ in (table 1.1)

Outer joist : 12K3 wt. 5.7 lb/ft $t_f = 0.450''$
inner joist 10K1 wt. 5.0 lb/ft

$a_{min} = t = 1/8''$ => min. Fillet Weld

Joist End Reaction : (for Inner Girder)

$D_L = 23\,psf + 10\,psf + 145\,pcf\,(3''/12''/ft) = 69.25\ psf$

$W_u = 1.2(69.25\,psf \times 5ft + 5.7\,lb/ft + 5.0\,lb/ft) + 1.6(25\,psf \times 5ft) + 0.5(23\,psf \times 5ft)$

$W_u = 0.686\ k/ft$

$$V_u = \frac{W_u L}{2} = \frac{(0.686\ k/ft)(20\,ft)}{2} = 6.86\ k$$

$t_e = 0.707a = 0.707(1/8'') = 0.0884''$

$\phi R_n = (\phi = 0.75)(0.6)(70\,ksi)(0.0884'') = 2.78\ k/in$

Check Stresses in Base Metal ($t = 1/8''$ governs over t_w)

Shear yield:
$\phi R_n = (\phi = 1.0)(0.6)(F_y = 36\,ksi)(t = 1/8'') = 2.7\ k/in$

Shear Rupture:

$\phi R_n = (\phi = 0.75)(0.6)(F_u = 58\,ksi)(t = 1/8'') = 3.26\ k/in$

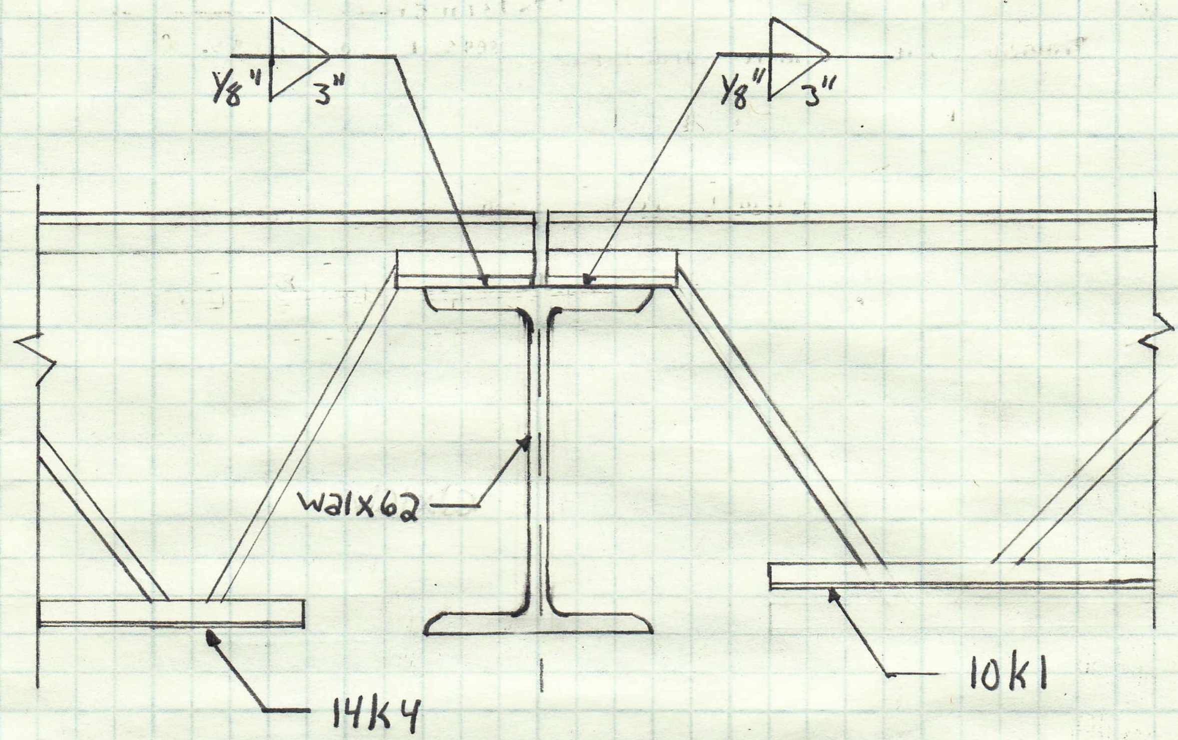
Required Weld Length : $L_w = \dfrac{6.86\,k}{\phi R_n = 2.7\,k/in} = 2.54''$ => Say 4'' Weld

=> (2) $\frac{1}{8}''$ x 2'' long per joist

$\phi R_n = (2.7\ \frac{k}{in})(4'') = 10.8k$

$\phi R_n = 10.8\,k > V_u = 6.86\,k$
ok

Typical Steel Joist-to-Girder Connection: 2nd + 3rd Floor



Lateral Design for one-way Slab w/ beams

P_{nt} (axial force from gravity loads) = 71.4 k

P_{lt} (axial force from lateral loads) = 13.3 k

M_{nt} (moment from gravity load) = 25.1 k·ft

M_{lt} (moment from lateral loads) = 12.5 k·ft

ΣH (total story Shear) = 2.8 k + 5.6 k + 5.6 k
= 14.0 k

lateral deflection (drift) for Story
$\Delta_H = 0.03\ in + 0.02\ in + 0.02\ in = 0.070\ in$

Amplifier B_2

$P_{story} = 3.4\ k/ft (60') + (2)(3.45\ k/ft)(40') + (2)(1.5\ k/ft)(20') = 660\ k$

$P_{mf} = 0$ (braced frame)

$P_{e\,story} = R_m \dfrac{\Sigma H L}{\Delta_H}$ $\qquad L = 12\ ft$

$R_m = (1 - 0.15(P_{mf}/P_{story}))$

$R_m = 1 - 0.15(0/660) = 1$

$P_{e\,story} = (1)\dfrac{(\Sigma H = 14.0k)(L = 12\,ft \times \frac{12''}{ft})}{\Delta_H = 0.070\ in} = 28{,}800$ kips

$B_2 = \dfrac{1}{1 - \dfrac{(\alpha = 1)(P_{story} = 660k)}{P_{e\,story} = 28{,}800k}} = 1.023 > 1.0$ ok

Amplifier B1

$M_1 = 10.3$ k·ft
$M_2 = 25.1$ k·ft

$\Rightarrow$ reverse curvature

$C_m = 0.6 \pm 0.4(M_1/M_2)$ use (-) for reverse curvature

$C_m = 0.6 - 0.4(10.3\,k\cdot ft / 25.1\,k\cdot ft)$

$C_m = 0.44$

$P_r = P_{nt} + B_2 P_{lt}$

$P_r = 91.4k + 1.023(3.3k) = 94.78$ kips

Elastic critical buckling load for column

$P_{el} = \pi^2 EI / (k_1 L)^2$ where $k_1 = 1.0$

$E_{con} = 57{,}000\sqrt{6{,}000psi} = 4{,}415{,}201$ psi

$I = \frac{1}{12} b h^3 = \frac{1}{12}(12")(12")^3 = 1728\ in^4$

$P_{el} = \pi^2 (4{,}415{,}201\,psi)(1728\,in^4) / (1.0 \cdot 12ft \cdot \frac{12"}{ft})^2$

$P_{el} = 3{,}631.4$ kips

$B_1 = \dfrac{C_m}{1 - \alpha P_r / P_{el}} \geq 1$

$B_1 = \dfrac{0.44}{1 - \left(\frac{94.78\,kips}{3{,}631.4\,kips}\right)} = 0.45$ use $B_1 = 1.0$

Required for Strength values

$P_r = P_{nt} + B_2 P_{lt} = 91.4k + (1.023)(3.3k) = 94.78$ kips

$M_r = B_1 M_{nt} + B_2 M_{lt}$

$M_r = (1.0)(25.1\,k\cdot ft) + (1.023)(12.5\,k\cdot ft)$

$M_r = 37.9\ k\cdot ft$

$M_r = 38.0\ k\cdot ft$

$$K_n = \frac{P_r/\phi}{f_c' A_g} = \frac{(94{,}780\ \text{lb})/\phi = 0.65}{(6000\ \text{psi})(12\ \text{in} \times 12\ \text{in})} = 0.21$$

$$R_n = \frac{M_r/\phi}{f_c' A_g h} = \frac{(456{,}000\ \text{lb-in})/\phi = 0.65}{(6{,}000\ \text{psi})(12'' \times 12'')(12'')} = 0.13$$

$A_g = bh$

$A_s = \rho bh$

$$\rho = \frac{A_s = 4.74\ \text{in}^2}{12'' \times 12''} = 0.03$$

$12\ \text{in} - 5\ \text{in} = \gamma h$

$$\gamma = \frac{12\ \text{in} - 5\ \text{in}}{12\ \text{in}} \simeq 0.7$$

From Interaction Diagram:

$f_c' = 6000$ psi $\quad f_y = 60$ ksi $\quad \gamma = 0.7$

$$\left.\begin{aligned} K_n &= 0.21 \\ R_n &= 0.13 \end{aligned}\right\} \qquad \frac{0.01}{0.10} = \frac{X}{0.13} \qquad X = 0.013$$

$\Rightarrow \rho = 0.013$

$A_s = \rho bh = (\rho = 0.013)(12'')(12'')$

$A_s = 1.87\ \text{in}^2$

$\Rightarrow$ design is ok since

$\rho = 0.013$ is in the envelope of the recommended $\rho = 0.03$

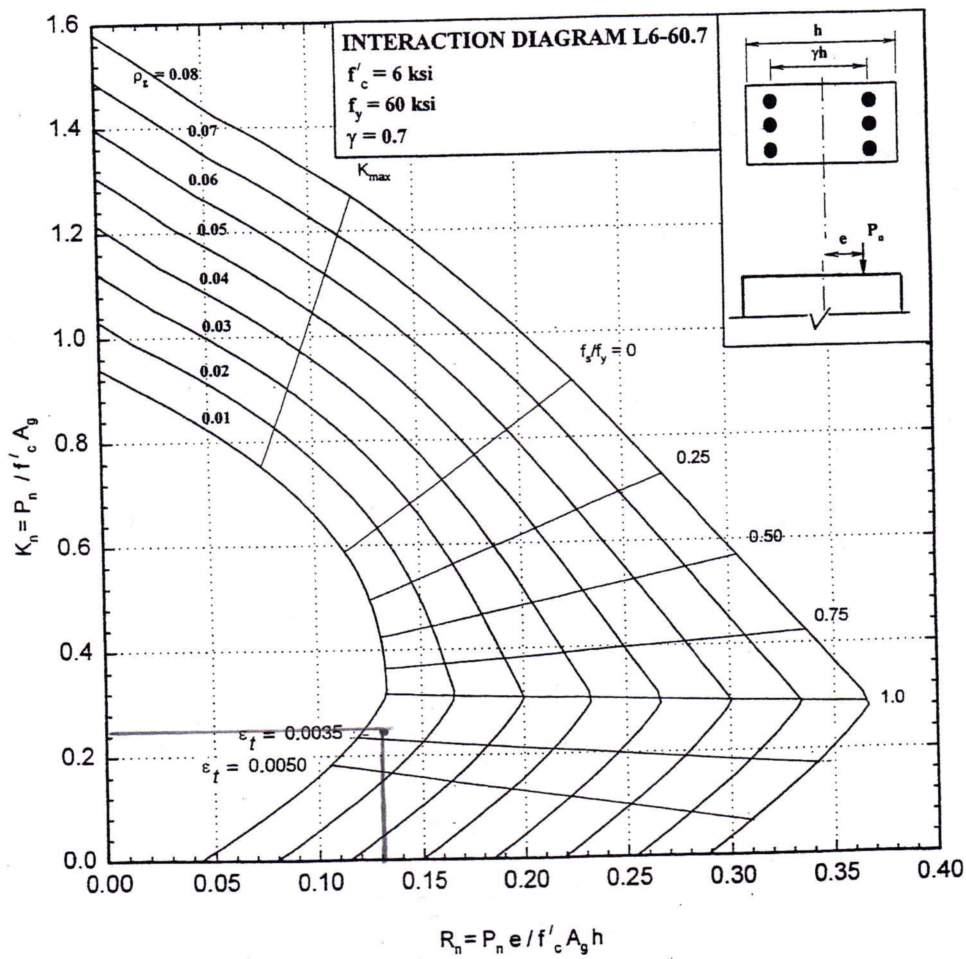
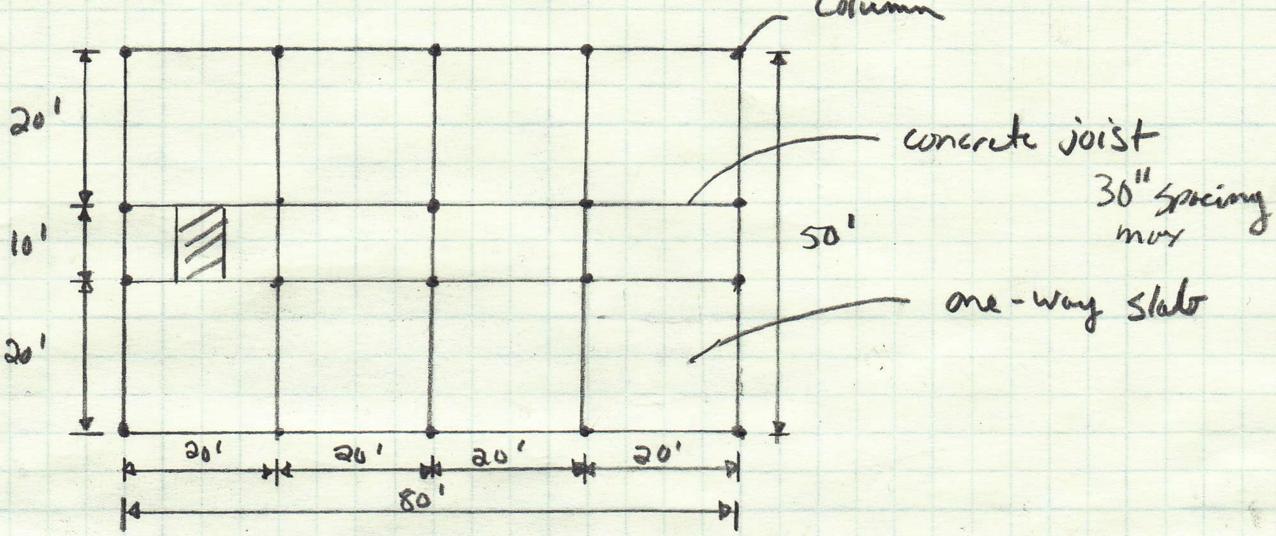
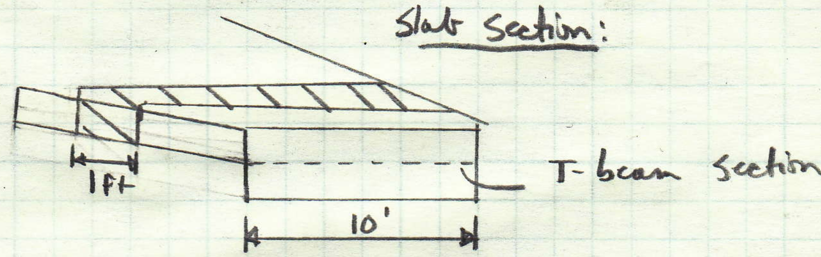


Figure A.19 Rectangular column nominal load-moment strength interaction diagram: $f_c' = 6000$ psi, $f_y = 60{,}000$ psi, $\gamma = 0.7$ (ACI-SP17 and Refs. 9.8, 9.10, 9.11).

Concrete Joist Design: Original Floor Design



Slab Section: $b = 12''$



Thickness of the Slab (h) = $L/20$ for simply supported

$$h = \frac{10\,ft\left(\frac{12''}{1\,ft}\right)}{20} = 6.0 \text{ in}$$

use 6.0 in as an intial trial thickness $h = 6.0$ in

$$W_{con} = (150\,pcf)(6\,in \times 12\,in)\left(\frac{ft}{12\,in}\right)^2 \quad 75 \ lb/ft$$

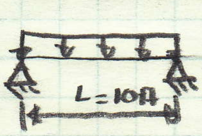
$$D_L = 75\,lb/ft + \left[3\,psf + 5\,psf + 2\,psf\right] \times 12''\left(\frac{ft}{12''}\right) = 85\ lb/ft$$

$$L_L = 80\,psf \times 12''\left(\frac{ft}{12''}\right) = 80\,lb/ft$$

$$W_u = 1.2D + 1.6L$$

$$W_u = 1.2(85\,lb/ft) + 1.6(80\,lb/ft) = 230\ lb/ft$$

Simple supported Beam



$$M_{max} = \frac{W_u L^2}{8}$$

$$M_u = \frac{(230\,lb/ft)(10\,ft)^2}{8} = 2.88 \ k\cdot ft$$

$$M_u = 2.88 \ k\cdot ft$$

design equation $\phi M_n > M_u$ $f_c' = 6000\,psi$

assume $\phi = 0.9$, and $\varepsilon_s > 0.005$ $F_y = 60{,}000\,psi$

$$M_n > \frac{M_u = 2.88\ k\cdot ft}{\phi = 0.9} \qquad M_n > 3.2\ k\cdot ft$$

a = 5.74
4 = 6.75

$$\beta_1 = 0.85 - 0.05\left(\frac{6000-4000}{1000}\right) = 0.75$$

$$\rho_{max} = 0.85\,\beta_1 \frac{f_c'}{f_y} \frac{0.003}{0.003+0.005}$$

$$\rho_{max} = 0.85(0.75)\frac{6000\,psi}{60{,}000\,psi}\left(\frac{0.003}{0.003+0.005}\right) = 0.0239$$

$$\rho = \frac{A_s}{bd}$$

$$\phi M_n = \phi \rho f_y b d^2 \left(1 - 0.59 \frac{\rho f_y}{f_c'}\right)$$

$$d^2 = \frac{M_n}{\rho f_y b \left(1 - 0.59 \frac{\rho f_y}{f_c'}\right)}$$

$$d = \sqrt{\frac{3.2\ k\cdot ft\ \left(\frac{12''}{ft}\right)}{(0.0239)(60\,ksi)(12'')\left(1 - 0.59\left(\frac{0.0239(60\,ksi)}{6\,ksi}\right)\right)}} = 1.60\ in$$

1.60

d = effective depth = (h = 6 in) − 1.0 in = 5.0 in

assume a = 1.0 in, the area of Steel required per foot of width in top of Slab

$$A_s = \frac{M_u}{\phi f_y (d - a/2)} = \frac{2.88\ k\cdot ft \times \left(\frac{12''}{ft}\right)}{0.9(60\,ksi)(5\,in - 1.0/2)} = 0.14\,in^2$$

check the assumed a

$$a = \frac{A_s f_y}{0.85 f_c' b} = \frac{(0.14\,in^2)(60\,ksi)}{0.85(6\,ksi)(12'')} = 0.14\ in$$

Second trial a = 0.14 in

$$A_s = \frac{2.88\ k\cdot ft \times \left(\frac{12''}{ft}\right)}{0.9(60\,ksi)(5 - 0.14\,in/2)} = 0.13\,in^2$$

$$a = 0.14\,in \times 0.13\,in^2 / 0.14\,in^2 = 0.13\ in$$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{A_s(60\,ksi)}{0.85(6\,ksi)(12")} = 0.98 A_s$$

$$M_n = A_s f_y \left(d - \frac{a}{2}\right)$$

$$3.2\,k\cdot ft\left(\frac{12"}{ft}\right) = A_s(60\,ksi)\left(5\,in - \frac{0.98 A_s}{2}\right)$$

$$0 = A_s(60\,ksi)\left(5\,in - \frac{0.98 A_s}{2}\right) - 38.4\,k\text{-}in$$

$$0 = 300 A_s - 29.4 A_s^2 - 38.4$$

$$A_s = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a}$$

$$A_s = \frac{-(300) \pm \sqrt{(300)^2 - 4(-29.4)(-38.4)}}{2(-29.4)}$$

$$A_s = \frac{-300 \pm 292.37}{2(-29.4)} \qquad A_s = 0.13\,in^2$$

Choose Bar No. 3 spaced 9"

$A_s = 0.15\,in^2$

$$a = \frac{(A_s = 0.15\,in^2)(60\,ksi)}{0.85(6\,ksi)(12")} = 0.15\,in$$

$$a = \beta_1 c \qquad c = \frac{a}{\beta_1} = \frac{0.15\,in}{0.75} = 0.2\,in$$

$$\frac{c}{d} = \frac{0.003}{0.003 + \varepsilon_s}$$

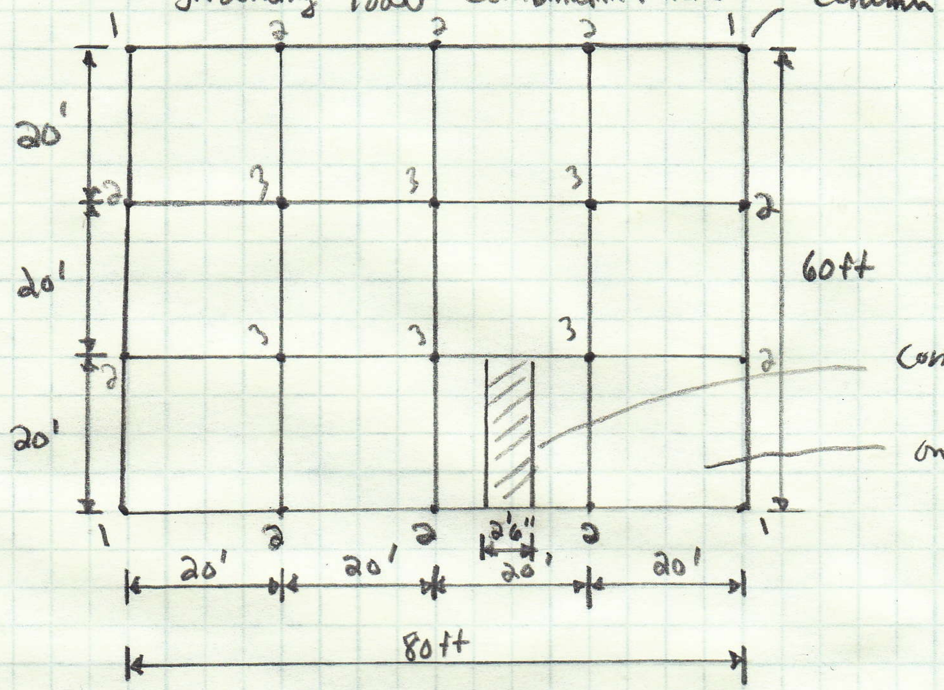
$$\frac{c = 0.2\,in}{d = 5\,in} = \frac{0.003}{0.003 + \varepsilon_s} \qquad \varepsilon_s = 0.072$$

$\varepsilon_s = 0.072 > 0.005$

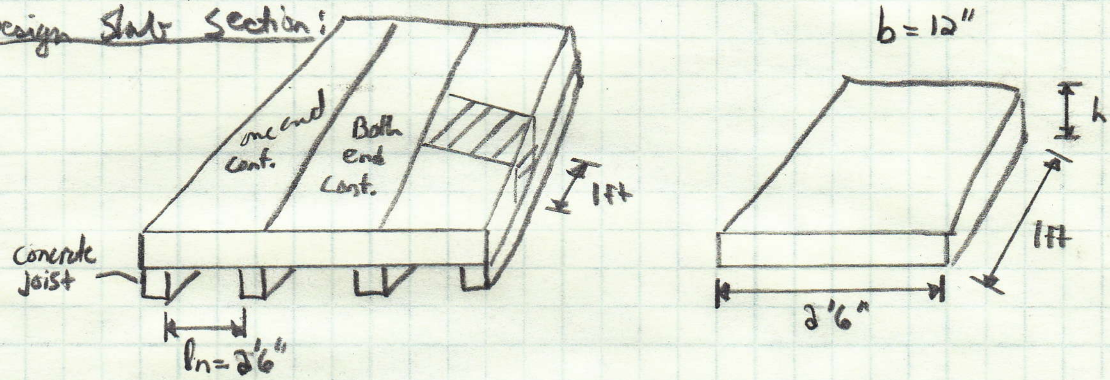
<u>ok</u>.

Concrete Joist Design: for 2nd and 3rd Floor

Governing load Combination: 1.2D+1.6L



Design Slab section:



- Use one end continuous for ease of construction (more conservative) assume whole slab is one end continuous

Thickness of Slab (h) = $l/24$ for one end continuous (according to ACI)

$$h = \frac{2.5\,ft\left(\frac{12"}{ft}\right)}{24} = 1.25\,in$$

- add at least 3/4" min. for slabs for cover, ACI

use h = 3in as an initial trial thickness

$$W_{con} = (150\,pcf)(3in \times 12in)\left(\frac{ft}{12"}\right)^2 = 37.5\ lb/ft$$

$$D_L = 37.5\ lb/ft + [3psf + 5psf + 2psf] \times 12"\ \frac{ft}{12"} = 47.5\ lb/ft$$

$$L_L = 130\,psf \times 12"\left(\frac{ft}{12"}\right) = 130\ lb/ft$$

$W_u = 1.2D + 1.6L$

$W_u = 1.2(47.5\ lb/ft) + 1.6(130\ lb/ft) = 265\ lb/ft$

Factored Moments at critical sections using ACI moment Coefficients (table 12.1)

exterior support: $-M = \frac{1}{24} W_u l_n^2 = \frac{1}{24}(265\ lb/ft)(2.5ft)^2 = 0.070\ k \cdot ft$

Midspan: $+M = \frac{1}{14} W_u l_n^2 = \frac{1}{14}(265\ lb/ft)(2.5ft)^2 = 0.118\ k \cdot ft$

interior support: $-M = \frac{1}{10} W_u l_n^2 = \frac{1}{10}(265\ lb/ft)(2.5ft)^2 = 0.166\ k \cdot ft$

interior support: $-M = \frac{1}{11} W_u l_n^2 = \frac{1}{11}(265\ lb/ft)(2.5ft)^2 = 0.151\ k \cdot ft$

Midspan: $+M = \frac{1}{16} W_u l_n^2 = \frac{1}{16}(265\ lb/ft)(2.5ft)^2 = 0.104\ k \cdot ft$

governing $M_u = 0.166\ k \cdot ft$

design equation $\phi M_n > M_u$

assume $\phi = 0.9$, and $\varepsilon_s > 0.005$

$f_c' = 6{,}000psi$

$f_y = 60{,}000psi$

$$\beta_1 = 0.85 - 0.05\left(\frac{6000-4000}{1000}\right) = 0.75$$

$$\rho_{max} = 0.85\beta_1 \frac{f_c'}{f_y} \frac{0.003}{0.003+0.005}$$

$$\rho_{max} = 0.85(0.75)\frac{6{,}000psi}{60{,}000psi}\left(\frac{0.003}{0.003+0.005}\right) = 0.0239$$

$$\phi M_n = \phi \rho f_y b d^2 \left(1 - 0.59\frac{\rho f_y}{f_c'}\right)$$

$$d^2 = \frac{M_u}{\phi \rho f_y b \left(1 - 0.59\frac{\rho f_y}{f_c'}\right)}$$

$$d = \sqrt{\frac{0.166\ k \cdot ft\left(\frac{12''}{ft}\right)}{(0.9)(0.0239)(60ksi)(12'')\left(1-0.59\left(\frac{0.0239(60ksi)}{6ksi}\right)\right)}} = 0.15\ in$$

d = effective depth $= (h = 3in) - 1.0in = 2in$

assume $a = 1.0in$, the area of steel required per foot of width in top of slab

$$A_s = \frac{M_u}{\phi f_y (d - \frac{a}{2})} = \frac{0.166\ k \cdot ft\left(\frac{12''}{ft}\right)}{0.9(60ksi)\left(2in - \frac{1.0}{2}\right)} = 0.03\ in^2$$

check the assumed a

$$a = \frac{A_s f_y}{0.85 f_c' b} = \frac{(0.03in^2)(60ksi)}{0.85(6ksi)(12'')} = 0.03\ in$$

Second trial $a = 0.03\text{ in}$

$$A_s = \frac{(0.166\text{ k·ft})\left(\frac{12''}{ft}\right)}{0.9(60\text{ ksi})\left(2\text{ in} - \frac{0.03\text{ in}}{2}\right)} = 0.02\text{ in}^2$$

$$a = 0.03\text{ in} \times 0.02\text{ in}^2 / 0.03\text{ in}^2 = 0.02\text{ in}$$

$$a = \frac{A_s f_y}{0.85 f_c' b} = \frac{A_s(60\text{ ksi})}{0.85(6\text{ ksi})(12'')} = 0.98 A_s$$

$$M_n = A_s f_y \left(d - \frac{a}{2}\right)$$

$$\phi M_n > M_u$$

$$M_n > \frac{M_u = 0.166\text{ k·ft}}{\phi = 0.9}$$

$$M_n > 0.184\text{ k·ft}$$

$$0.184\text{ k·ft}\left(\frac{12''}{ft}\right) = A_s(60\text{ ksi})\left(2\text{ in} - \frac{0.98A_s}{2}\right)$$

$$0 = A_s(60\text{ ksi})\left(2\text{ in} - \frac{0.98A_s}{2}\right) - 2.208\text{ k·in}$$

$$0 = 120A_s - 29.4A_s^2 - 2.208$$

$$A_s = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a}$$

$$A_s = \frac{-(120) \pm \sqrt{(120)^2 - 4(-29.4)(-2.208)}}{2(-29.4)}$$

$$A_s = \frac{-120 \pm 118.913}{2(-29.4)}$$

$$A_s = 0.02\text{ in}^2$$

(from table A.3) Choose Bar NO.3 spaced 12"

$A_s = 0.11\text{ in}^2$ per foot

$$a = \frac{(A_s = 0.11\text{ in}^2)(60\text{ ksi})}{0.85(6\text{ ksi})(12'')} = 0.11\text{ in}$$

$$a = \beta_1 c$$

$$c = \frac{a}{\beta_1} = \frac{0.11\text{ in}}{0.75} = 0.15\text{ in}$$

$$\frac{c = 0.15\text{ in}}{d = 2\text{ in}} = \frac{0.003}{0.003 + \varepsilon_s} \Rightarrow \varepsilon_s = 0.037$$

Since $\varepsilon_s = 0.037 > 0.005$

use $\phi = 0.9$ ok

$$M_n = A_s f_y\left(d - \frac{a}{2}\right)$$

$$M_n = (0.11\text{ in}^2)(60\text{ ksi})\left(2\text{ in} - \frac{0.11\text{ in}}{2}\right)$$

$$M_n = 1.07\text{ k·ft}$$

$$M_r = (\phi = 0.9)(1.07\text{ k·ft}) = 0.96\text{ k·ft}$$

$$M_r = 0.96\text{ k·ft} > M_u = 0.166\text{ k·ft}$$

ok

At interior Support:

$M_u = 0.151\ k\cdot ft$

design equation $\phi M_n > M_u$

assume $\phi = 0.9$, $\varepsilon_s > 0.005$

$$d = (h = 3in) - 1.0_{in} = 2.0\ in$$

$$a = \frac{A_s f_y}{0.85 f_c' b} = \frac{A_s(60ksi)}{0.85(6ksi)(12'')} = 0.98\ A_s$$

$$M_n = A_s f_y \left(d - \frac{a}{2}\right)$$

$$M_n > \frac{M_u = 0.151\ k\cdot ft}{\phi = 0.9}$$

$$M_n > 0.168\ k\cdot ft$$

$$0.168\ k\cdot ft \left(\frac{12''}{ft}\right) = A_s(60ksi)\left(2in - \frac{0.98A_s}{2}\right)$$

$$0 = A_s(60ksi)\left(2in - \frac{0.98A_s}{2}\right) - 2.016\ k\cdot in$$

$$0 = 120A_s - 29.4A_s^2 - 2.016\ k\cdot in$$

$$A_s = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a}$$

$$A_s = \frac{(-120) \pm \sqrt{(120)^2 - 4(-29.4)(2.016)}}{2(-29.4)}$$

$$A_s = \frac{-120 \pm 119.008}{2(-29.4)}$$

$\Rightarrow A_s = 0.02\ in^2$

At Midspan: $M_u = 0.104\ k\cdot ft$

$$A_s = \frac{(0.104\ k\cdot ft)\left(\frac{12''}{ft}\right)}{0.9(60ksi)\left(2in - \frac{0.02}{2}\right)} = 0.012\ in^2$$

At Midspan: $M_u = 0.118\ k\cdot ft$

$$A_s = \frac{0.118\ k\cdot ft\left(\frac{12''}{ft}\right)}{0.9(60ksi)\left(2in - \frac{0.02}{2}\right)} = 0.013\ in^2$$

At exterior Support: $M_u = 0.070\ k\cdot ft$

$$A_s = \frac{0.070\ k\cdot ft\left(\frac{12''}{ft}\right)}{0.9(60ksi)\left(2in - \frac{0.02}{2}\right)} = 0.008\ in^2$$

$\Rightarrow$ Use Bar NO.3 Spaced 12"

$A_s = 0.11 in^2$ per foot

for all critical Spans

Check $A_{s\,min}$:

$$A_{s\,min} \geq \frac{200 b_w d}{f_y} = \frac{(200)(12'')(2'')}{60{,}000psi} = 0.08 in^2$$

$$A_{s\,min} \geq \frac{3\sqrt{f_c'} b_w d}{f_y} = \frac{3\sqrt{6000psi}\,(12'')(2'')}{60{,}000psi} = 0.09 in^2 \text{ governs}$$

Since $A_s = 0.11 in^2 > A_{s\,min} = 0.09 in^2$

OK

The minimum reinforcement that is required for control of Shrinkage and temperature cracking

use temp. and Shringage rf in Slabs ratio 0.0018 for grade 60 bars (table 13.2)

$$A_{s_{min}} = 0.0018\ b\ h$$

$$A_{s_{min}} = 0.0018(12'')(3'') = 0.0648\,in^2$$

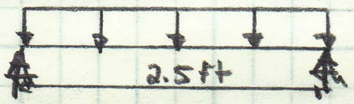
$$\Rightarrow A_{s_{min}} = 0.0648\,in^2$$

$$A_s = 0.11\,in^2 > A_{s_{min}} = 0.0648\,in^2$$

ok

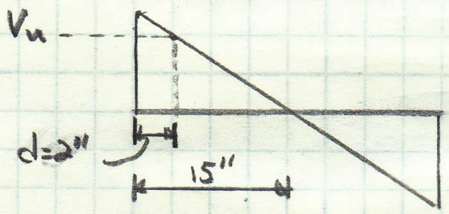
Shear Reinforcement:

$w_u = 265\ lb/ft$



$$V_{end} = \frac{w_u\ l_n}{2}$$

$$V_{end} = \frac{(265\,lb/ft)(2.5\,ft)}{2} = 331.25\ lb$$



Shear diagram

Similar triangles

$$\frac{V_{end} = 331.25\,lb}{15''} = \frac{V_u}{15'' - 2''}$$

$$\Rightarrow V_u = 287.08\ lb$$

(table 12.1) Shear Coefficient 1.15

$$1.15\,V_u = 1.15(287.08\,lb) = 330.2\,lb$$

$$\Rightarrow V_u = 330.2\ lb$$

Approx. Method : eq 4.12b

$$V_c = 2\lambda\sqrt{f_c'}\,b_w\,d$$

$$V_c = 2(1)\sqrt{6000\,psi}\,(12'')(2\,in)$$

$$V_c = 3{,}718.06\ lb$$

$$\phi V_c = (\phi = 0.75)(V_c = 3{,}718.06\,lb) = 2788.5\ lb$$

Since ϕV_c is well above V_u, No Shear reinforcement

$$\tfrac{1}{2}\phi V_c = \tfrac{1}{2}(\phi = 0.75)(V_c = 3{,}718.06\,lb)$$

$$\tfrac{1}{2}\phi V_c = 1{,}394.3\ lb$$

Since

$$V_u = 330.2\,lb < \tfrac{1}{2}\phi V_c = 1{,}394.3\,lb$$

No Shear Reinforcement needed

Design of concrete joist: for 2nd and 3rd floor

Design Span = 20 ft
Spacing = 2.5 ft or 30"

one-way Slab
h = 3 in (thickness)
d = 2 in

$W_{con} = (150\,pcf)(3in \times 30in)\left(\frac{ft}{12"}\right)^2 = 93.75\ lb/ft$

$D_L = 93.75\ lb/ft + [3psf + 5psf + 2psf] \times 30"\left(\frac{ft}{12"}\right) = 118.75\ lb/ft$

$L_L = 130\ psf \times 30"\left(\frac{ft}{12"}\right) = 325\ lb/ft$

$W_u = 1.2D + 1.6L$

$W_u = 1.2(118.75\ lb/ft) + 1.6(325\ lb/ft) = 662.5\ lb/ft$

$W_{TL} = 662.5\ lb/ft$ target load carry capacity

From PCI load tables, choose 8" joist w/ 3" slab

8" joist w/ 3in slab spaced 2'-6" : span = 20ft

Wt. 45.5 lb/ft
allow W_u = 290 psf

allow $W_u = (290\,psf) \times 2.5\,ft = 725\ lb/ft$

$W_u = 1.2(118.75\ lb/ft + 45.5\ lb/ft) + 1.6(325\ lb/ft)$

$W_{TL} = 717.1\ lb/ft$

OK, since allow $W_u = 725\ lb/ft > W_{TL} = 717.1\ lb/ft$

$\Rightarrow$ use 8" joist w/ 3" slab
Span = 20 ft
Spaced: 2'-6"
Wt. 45.5 lb/ft

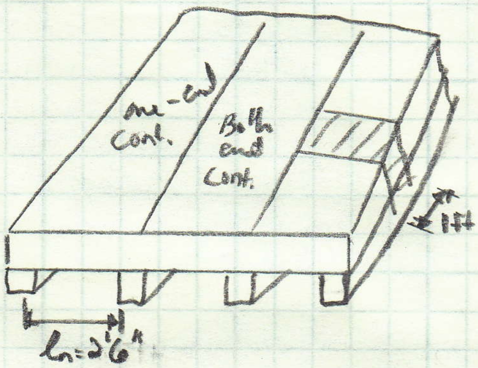
Concrete Joist Design: for Roof

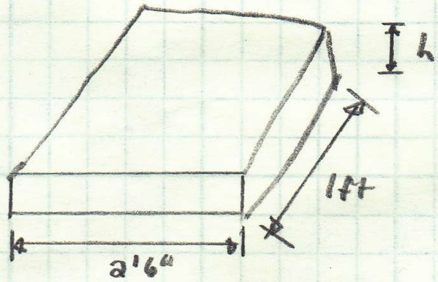
governing load combination: $1.2D + 1.6(L_r \text{ or } S) + (L \text{ or } 0.5W)$

$D_L = 20\,psf$ $L_r = 20\,psf$ $S = 25\,psf$ $W = 23\,psf$

Design slab section:

$b = 12"$





- use one end continuous for ease of construction (more conservative) assume whole slab is one end continuous

Thickness of slab $(h) = l/24$ for one end continuous (ACI)

$$h = \frac{2.5ft\left(\frac{12"}{ft}\right)}{24} = 1.25\,ft$$

(add at least 3/4" for cover for slab, ACI)

use $h = 3in$ as an initial trial thickness

$$W_{con} = (150\,pcf)(3in \times 12in)\left(\frac{ft}{12"}\right)^2 = 37.5\,lb/ft$$

$$D_L = 37.5\,lb/ft + \left[20psf \times 12"\left(\frac{ft}{12"}\right)\right] = 57.5\,lb/ft$$

$$S = 25psf \times 12"\left(\frac{ft}{12"}\right) = 25\,lb/ft$$

$$W = 23psf \times 12"\left(\frac{ft}{12"}\right) = 23\,lb/ft$$

$$W_u = 1.2D + 1.6S + 0.5W$$

$$W_u = 1.2(57.5\,lb/ft) + 1.6(25\,lb/ft) + 0.5(23\,lb/ft) = 120.5\,lb/ft$$

Factored Moments at critical sections using ACI Moment Coefficients (table 12.1)

exterior support: $-M = \frac{1}{24} W_u l_n^2 = 1/24(120.5\,lb/ft)(2.5ft)^2 = 0.031\,k\cdot ft$

Midspan: $+M = 1/14\, W_u l_n^2 = 1/14(120.5\,lb/ft)(2.5ft)^2 = 0.054\,k\cdot ft$

interior support: $-M = \frac{1}{10} W_u l_n^2 = 1/10(120.5\,lb/ft)(2.5ft)^2 = 0.075\,k\cdot ft$

interior support: $-M = 1/11\, W_u l_n^2 = 1/11(120.5\,lb/ft)(2.5ft)^2 = 0.068\,k\cdot ft$

Midspan: $+M = \frac{1}{16} W_u l_n^2 = 1/16(120.5\,lb/ft)(2.5ft)^2 = 0.047\,k\cdot ft$

governing $M_u = 0.075$ k-ft
design equation $\phi M_n > M_u$
assume $\phi = 0.9$, and $\varepsilon_s > 0.005$

$f_c' = 6,000psi$
$f_y = 60,000psi$

$\beta_1 = 0.75$
$\rho_{max} = 0.0239$

$$\phi M_n = \phi \rho f_y b d^2 \left(1 - 0.59 \frac{\rho f_y}{f_c'}\right)$$

$$d^2 = \frac{M_u}{\phi \rho f_y b \left(1 - 0.59 \frac{\rho f_y}{f_c'}\right)}$$

$$d = \sqrt{\frac{0.075\,k\text{-}ft\left(\frac{12''}{ft}\right)}{(0.9)(0.0239)(60ksi)(12'')\left(1 - 0.59\left(\frac{0.0239(60ksi)}{6ksi}\right)\right)}} = 0.10\,in$$

d = effective depth = $(h = 3in) - 1.0in = 2.0in$

assume $a = 1.0in$, the area of steel required per foot of width in top of slab

$$A_s = \frac{M_u}{\phi f_y (d - \frac{a}{2})} = \frac{0.075\,k\text{-}ft\left(\frac{12''}{ft}\right)}{0.9(60ksi)(2in - \frac{1.0}{2})} = 0.01\,in^2$$

Check the assumed a

$$a = \frac{A_s f_y}{0.85 f_c' b} = \frac{(0.01in^2)(60ksi)}{0.85(6ksi)(12'')} = 0.01\,in$$

Second trial $a = 0.01in$

$$A_s = \frac{0.075\,k\text{-}ft\left(\frac{12''}{ft}\right)}{0.9(60ksi)(2in - \frac{0.01in}{2})} = 0.01\,in^2$$

$$a = 0.01in \times 0.01in^2 / 0.01in^2 = 0.01in$$

$$a = \frac{A_s f_y}{0.85 f_c' b} = \frac{A_s(60ksi)}{0.85(6ksi)(12'')} = 0.98 A_s$$

$$M_n = A_s f_y \left(d - \frac{a}{2}\right)$$

$\phi M_n > M_u$

$$M_n \geq \frac{M_u = 0.075\,k\text{-}ft}{\phi = 0.9}$$

$M_n > 0.083$ k-ft

$$0.083\,k\text{-}ft\left(\frac{12''}{ft}\right) = A_s(60ksi)\left(2in - \frac{0.98 A_s}{2}\right)$$

$$0 = A_s(60ksi)\left(2in - \frac{0.98A_s}{2}\right) - 0.996\ k\text{-}in$$

$$0 = 120 A_s - 29.4 A_s^2 - 0.996$$

$$A_s = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a}$$

$$A_s = \frac{-(120) \pm \sqrt{(120)^2 - 4(-29.4)(-0.996)}}{2(-29.4)}$$

$$A_s = \frac{-120 \pm 119.51}{2(-29.4)}$$

$A_s = 0.01in^2$

(from table A.3) Choose Bar NO. 3 spaced 12"
$A_s = 0.11\ in^2$ per foot

$$a = \frac{(A_s = 0.11in^2)(60ksi)}{0.85(6ksi)(12")} = 0.11in$$

$a = \beta_1 c$ $\quad c = \frac{a}{\beta_1} = \frac{0.11in}{0.75} = 0.15in$

$$\frac{c = 0.15in}{d = 2in} = \frac{0.003}{0.003 + \varepsilon_s} \Rightarrow \varepsilon_s = 0.037$$

Since $\varepsilon_s\ \varepsilon_s = 0.037 > 0.005$
use $\phi = 0.9$

$M_n = A_s f_y (d - \frac{a}{2})$
$M_n = (0.11in^2)(60ksi)(2in - \frac{0.11in}{2})$

$M_n = 1.07$ k.ft
$M_r = (\phi = 0.9)(M_n = 1.07k.ft) = 0.96$ k.ft

$M_r = 0.96$ k.ft $> M_u = 0.075$ k.ft
ok

-0.076
0.907

At interior support : $M_u = 0.068$ k.ft

$$A_s = \frac{0.068\ k.ft\ (\frac{12"}{ft})}{0.9(60ksi)(2in - \frac{0.01in}{2})} = 0.008\ in^2$$

at midspan: $M_u = 0.047$ k.ft

$$A_s = \frac{0.047\ k.ft\ (\frac{12"}{ft})}{0.9(60ksi)(2in - \frac{0.01in}{2})} = 0.005\ in^2$$

At Midspan: $M_u = 0.054$ k.ft

$$A_s = \frac{0.054\ k.ft\ (\frac{12"}{ft})}{0.9(60ksi)(2in - \frac{0.01in}{2})} = 0.006\ in^2$$

At exterior support : $M_u = 0.031$ k.ft

$$A_s = \frac{0.031\ k.ft\ (\frac{12"}{ft})}{0.9(60ksi)(2in - \frac{0.01in}{2})} = 0.003\ in^2$$

$\Rightarrow$ Use Bar No.3 spaced 12"
$A_s = 0.11in^2$ per foot
for all critical spans

Check A_{smin} :

$$A_{smin} \geq \frac{200 b_w d}{f_y} = \frac{200(12")(2")}{60{,}000psi} = 0.08in^2$$

$$A_{smin} \geq \frac{3\sqrt{f'_c}\, b_w d}{f_y} = \frac{3\sqrt{6000psi}\,(12")(2")}{60{,}000psi} = 0.09in^2$$

Since,
$A_s = 0.11in^2 > A_{smin} = 0.09in^2$
ok

The minimum reinforcement that is required for control of Shrinkage and temperature cracking:

Use Temp. and Shrinkage r/f ratio 0.0018 for grade 60 bars (table 13.2)

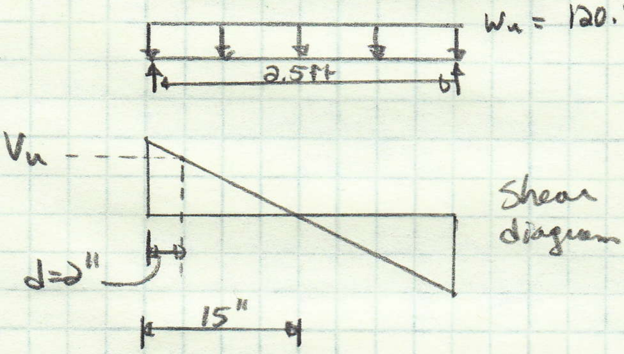
$A_{s\,min} = 0.0018\, b\, h$

$A_{s\,min} = 0.0018(12'')(3'') = 0.0648\ in^2$

$\Rightarrow A_{s\,min} = 0.065 in^2$

$A_s = 0.11 in^2 > A_{s\,min} = 0.065 in^2$ ok

Shear Reinforcement:



$V_{end} = \dfrac{w_u l_n}{2}$

$V_{end} = \dfrac{(120.5 lb/ft)(2.5ft)}{2} = 150.6\ lb$

similar triangles

$\dfrac{V_{end} = 150.6\ lb}{15''} = \dfrac{V_u}{15'' - 2''}$

$\Rightarrow V_u = 130.5\ lb$

(table 12.1) Shear coefficient 1.15

$1.15 V_u = 1.15(130.5 lb) = 150.1\ lb$

$\Rightarrow V_u = 150.1 lb$

approx. Method: eq 4.12b

$V_c = 2\lambda\sqrt{f_c'}\, b_w d$

$V_c = 2(1)\sqrt{6000 psi}(12'')(2 in)$

$V_c = 3{,}718.06\ lb$

$\phi V_c = (\phi = 0.75)(V_c = 3{,}718.06 lb) = 2{,}788.5\ lb$

Since ϕV_c is well above V_u, No Shear reinforcement

$\frac{1}{2}\phi V_c = \frac{1}{2}(\phi = 0.75)(V_c = 3{,}718.06\ lb)$

$\frac{1}{2}\phi V_c = 1{,}394.3\ lb$

Since, $V_u = 150.1\ lb < \frac{1}{2}\phi V_c = 1{,}394.3\ lb$

No Shear Reinforcement needed

Design of concrete joist: For Roof

governing load combination: $1.2D + 1.6(L_r \text{ or } S) + (L \text{ or } 0.5W)$

$D_L = 20 psf$ $L_r = 20 psf$ $S = 25 psf$ $W = 23 psf$

Design Span = 20 ft
Spacing = 2.5 ft or 30"

one-way Slab
$h = 3in$ (thickness)
$d = 2in$

$W_{con} = (150 pcf)(3in \times 30in)\left(\frac{ft}{12"}\right)^2 = 93.75 \ lb/ft$

$D_L = 93.75 \ lb/ft + \left[20 psf \times 30"\left(\frac{ft}{12"}\right)\right] = 143.75 \ lb/ft$

$S = 25 psf \times 30"\left(\frac{ft}{12"}\right) = 62.5 \ lb/ft$

$W = 23 psf \times 30"\left(\frac{ft}{12"}\right) = 57.5 \ lb/ft$

$W_u = 1.2D + 1.6S + 0.5W$

$W_u = 1.2(143.75 \ lb/ft) + 1.6(62.5 \ lb/ft) + 0.5(57.5 \ lb/ft) = 301.25 \ lb/ft$

$W_{TL} = 301.25 \ lb/ft$

from PCI load tables, choose 8" joist w/ 3" slab

8" joist w/ 3in slab spaced 2'-6" ; Span = 20 ft

wt. = 45.5 lb/ft
allow W_u = 290 psf

allow $W_u = 290 psf \times 2.5 ft = 725 \ lb/ft$

$W_u = 1.2(143.75 \ lb/ft + 45.5 \ lb/ft) + 1.6(62.5 \ lb/ft) + 0.5(57.5 \ lb/ft)$

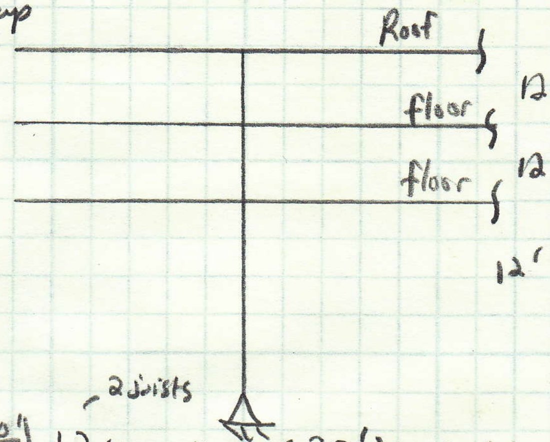
$W_{TL} = 355.85 \ lb/ft$

ok, since allow $W_u = 725 \ lb/ft > W_{TL} = 355.85 \ lb/ft$

use 8" joist w/ 3" Slab

$\Rightarrow$ Span = 20 ft
Spaced : 2'-6"
Wt. 45.5 lb/ft

Design of columns: (#1) group



Roof Loads: , tributary area

$D_L = 20\,psf\left(\frac{20'}{2}\right)\left(\frac{20'}{2}\right) = 2000\ lb$

$L_r = 20\,psf\left(\frac{20'}{2}\right)\left(\frac{20'}{2}\right) = 2000\ lb$

$S = 25\,psf\left(\frac{20'}{2}\right)\left(\frac{20'}{2}\right) = 2500\ lb$

2 joists

$D_L = 2000\ lb + \underbrace{(150\,pcf)(3'')\left(\frac{ft}{12''}\right)\left(\frac{20'}{2}\right)\left(\frac{20'}{2}\right)}_{Slab} + 2\ 45.5\ lb/ft\left(\frac{20'}{2}\right) = 6,660\ lb$

$L_r = 2000\ lb$

$S = 2500\ lb$

Floor Loads: (2nd and 3rd Floor)

$D_L = 10\,psf\left(\frac{20'}{2}\right)\left(\frac{20}{2}\right) + 150\,pcf(3'')\left(\frac{ft}{12''}\right)\left(\frac{20'}{2}\right)\left(\frac{20'}{2}\right) + 2\ 45.5\ lb/ft\left(\frac{20'}{2}\right) = 5,660\ lb$

$L_L = 130\,psf\left(\frac{20'}{2}\right)\left(\frac{20'}{2}\right) = 13,000\ lb$

Total Loads:

Dead load from 2 floors and roof = 6,660 lb + 5,660 lb + 5,660 lb = 17,980 lb

Floor Live load = 13,000 lb + 13,000 lb = 26,000 lb

Roof snow load = 2,500 lb

Roof live load = 2,000 lb

load combination

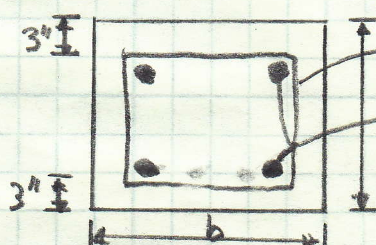
$1.4D = 1.4(17,980\ lb) = 25,172\ lb$

$P_u = 1.2D + 1.6L + 0.5S = 1.2(17,980\ lb) + 1.6(26,000\ lb) + 0.5(2,500\ lb) = 64,426\ lb$

$P_u = 64,426\ lb$

Square Column Design

Assume clear cover 3"



tie r/s ⟹ Use 0.80 for accidental eccentricity reduction factor.

longitudinal bars

– column under Axial load only

Use ratio: $\frac{A_s}{A_g} = 0.03$ (3% of steel in concrete)

Assume $A_g \sim A_{con}$

$A_s = 0.03 A_g$

$P_n = \phi 0.80 [0.85 f_c' A_g + A_s f_y]$

$A_s = 0.03\ A_g$

$P_n = \phi 0.80 [0.85 f_c' A_g + 0.03 A_g f_y]$

$P_n = (0.65) 0.80 A_g [0.85 f_c' + 0.03 f_y]$

Solve A_g

$A_g = bh$

$\phi = 0.65$ (compression failure region)

$\phi P_n \geq P_u$

$P_n = \dfrac{P_u = 64{,}426\ lb}{\phi = 0.65}$

$P_n = 99{,}117\ lb$

$99{,}117\ lb = \cancel{(0.65)(0.80)} A_g [0.85(6{,}000\ psi) + 0.03(60{,}000\ psi)]$

$A_g = \dfrac{99{,}117}{3588} = 27.6\ in^2$

$A_g = bh$

Choose a b and h, that gives you a greater gross Area than one calculated

$b = 9\ in$ $h = 9\ in$, since the 8" joist has a $b = 5\text{-}\frac{7}{16}"$

$A_g = (b = 9\ in)(h = 9\ in) = 81\ in^2$

$A_s = 0.03(A_g = 81\ in^2) = 2.43\ in^2$

Assume $A_s' = A_s$, so $A_s = 1.215\ in^2$, $A_s' = 1.215\ in^2$

axial + bending: P_{nb}, M_{nb}

$\varepsilon_s = 0.002$

$\varepsilon_{con} = 0.003$

$c_b = \dfrac{0.003}{0.003 + 0.002}(d = 9" - 3") = 3.6\ in$

$a_b = (\beta_1 = 0.75)(c_b = 3.6\ in) = 2.70\ in$

$P_n = 0.85 f_c' ab + A_s' f_s' - A_s f_s$

$f_s' = \varepsilon_s' E = 0.003\left(\dfrac{3.6\ in - 3"}{3.6\ in}\right)(29{,}000{,}000\ psi) = 14{,}500\ psi$

$P_n = 0.85(6000\ psi)(2.70\ in)(9") + (1.215\ in^2)(14{,}500\ psi) - (1.215\ in^2)(60{,}000\ psi)$

$P_n = 68{,}648\ lb$

$\phi P_n = (0.65)(P_n = 68{,}648)$

$\phi P_n = 44{,}621\ lb$

$\phi P_n = 44{,}621\ lb < P_u\ 64{,}426\ lb$

Not <u>ok</u>

try $b = 12in$ $h = 12in$

$A_g = (b = 12in)(h = 12in) = 144 in^2$

$A_s = 0.03(A_g = 144 in^2) = 4.32 in^2$

Assume $A_s' = A_s$, so $A_s = 2.16 in^2$, $A_s' = 2.16 in^2$

P_{nb}, M_{nb}

$\varepsilon_s = 0.002$

$\varepsilon_{con} = 0.003$

$c_b = \frac{0.003}{0.003 + 0.002}(d = 12'' - 3'') = 5.4 in$

$a_b = (\beta_1 = 0.75)(c_b = 5.4 in) = 4.05 in$

$P_n = 0.85 f_c' a b + A_s' f_s' - A_s f_s$

$f_s' = \varepsilon_s' E = 0.003\left(\frac{5.4in - 3''}{5.4in}\right)(29{,}000{,}000 psi) = 38{,}667 psi$

$P_n = 0.85(6000psi)(4.05in)(12'') + (2.16in^2)(38{,}667psi) - (2.16in^2)(60{,}000psi)$

$P_n = 201{,}781 lb$

accidental eccentricity

$\phi P_n = (0.80)(\phi = 0.65)(P_n = 201{,}781 lb)$

$\phi P_n = 104{,}926 lb$

$\phi P_n = 104{,}926 lb > P_u = 64{,}426 lb$ ok

$\Rightarrow$ Select 4 No.16 bars
$A_s = 5.08 in^2$
(from table A.2)

$\frac{A_s}{A_g} = \frac{5.08 in^2}{144 in^2} = 0.035$

$A_s' = A_s$, so $A_s = 2.54 in^2$, $A_s' = 2.54 in^2$

$P_n = 0.85(6000psi)(4.05in)(12'') + (2.54in^2)(38{,}667psi) - (2.54in^2)(60{,}000psi)$

$P_n = 193{,}674 lb$

$\phi P_n = (0.80)(0.65)(P_n = 193{,}674 lb)$

$\phi P_n = 100{,}711 lb$

$\phi P_n = 100{,}711 lb > P_u = 64{,}426 lb$ ok

Check column capacity w/ added column self weight:

$W_{con} = 150 pcf (b = 12in \times h = 12'')\left(\frac{1ft}{12''}\right)^2 (12' \times 3 \text{ stories})$

$W_{con} = 5{,}400 lb$

$P_u = 1.2(17{,}980 lb + 5{,}400 lb) + 1.6(26{,}000 lb) + 0.5(2{,}580 lb)$

$P_u = 70{,}906 lb$

$\phi P_n = 100{,}711 lb > P_u = 70{,}906 lb$ ok

Design of Columns : (#2) group

Roof loads : (tributary Area)

$D_L = (20\,psf)\left(\frac{20'}{2}+\frac{20'}{2}\right)\left(\frac{20'}{2}\right) = 4{,}000\,lb$

$L_r = (20\,psf)\left(\frac{20'}{2}+\frac{20'}{2}\right)\left(\frac{20'}{2}\right) = 4{,}000\,lb$

$S = (25\,psf)\left(\frac{20'}{2}+\frac{20'}{2}\right)\left(\frac{20'}{2}\right) = 5{,}000\,lb$

$D_L = 4000\,lb + \underbrace{(150\,pcf)(3'')\left(\frac{ft}{12''}\right)\left(\frac{20'}{2}+\frac{20'}{2}\right)\left(\frac{20'}{2}\right)}_{Slab} + 2\ 45.5\,lb/ft\left(\frac{20'}{2}+\frac{20'}{2}\right)$

$D_L = 13{,}320\,lb$

$L_r = 4000\,lb$

$S = 5{,}000\,lb$

Floor loads : (2nd and 3rd Floor)

$D_L = (10\,psf)\left(\frac{20'}{2}+\frac{20'}{2}\right)\left(\frac{20'}{2}\right) + (150\,pcf)(3'')\left(\frac{ft}{12''}\right)\left(\frac{20'}{2}+\frac{20'}{2}\right)\left(\frac{20'}{2}\right) + 2(45.5\,lb/ft)\left(\frac{20'}{2}+\frac{20'}{2}\right)$

$D_L = 11{,}320\,lb$

$L_L = (130\,psf)\left(\frac{20'}{2}+\frac{20'}{2}\right)\left(\frac{20'}{2}\right) = 26{,}000\,lb$

Total loads

Dead load from 2 floors and roof = 13,320 lb + 11,320 + 11,320 = 35,960 lb

Floor live load = 26,000 lb + 26,000 lb = 52,000 lb

Roof snow load = 5,000 lb

Roof live load = 4,000 lb

load combination:

$P_u = 1.2D + 1.6L + 0.5S = 1.2(35{,}960\,lb) + 1.6(52{,}000\,lb) + 0.5(5{,}000\,lb)$

$P_u = 128{,}852\,lb$

SQuare Column Design

Use ratio $\frac{A_s}{A_g} = 0.03$

$\phi = 0.65$

$\phi P_n > P_u$

$P_n = \frac{P_u = 128{,}852\,lb}{\phi = 0.65}$

$P_n = 198{,}234\,lb$

$P_n = \phi 0.80\left[0.85\,f_c'\,A_g + A_s\,f_y\right]$

$P_n = \phi 0.80\left[0.85\,f_c'\,A_g + 0.03\,A_g\,f_y\right]$

$P_n = \phi 0.80\,A_g\left[0.85\,f_c' + 0.03\,f_y\right]$

$198{,}234\,lb = (0.65)(0.80)\,A_g\left[0.85(6000\,psi) + (0.03)(60{,}000\,psi)\right]$

$A_g = \frac{198{,}234}{3588} = 55\,in^2$

try $b = 14in$ $h = 14in$

$A_g = (b = 14in)(h = 14'') = 196 in^2$

$A_s = 0.03\,(A_g = 196 in^2) = 5.88 in^2$

Assume $A_s' = A_s$, so $A_s = 2.94 in^2$
$A_s' = 2.94 in^2$

P_{nb}, M_{nb}
$\varepsilon_s = 0.002$
$\varepsilon_{con} = 0.003$

$c_b = \frac{0.003}{0.003 + 0.002}\,(d = 14'' - 3'') = 6.6 in$

$a_b = (\beta_1 = 0.75)(c_b = 6.6in) = 4.95 in$

$P_n = 0.85 f_c' a b + A_s' f_s' - A_s f_s$

$f_s' = \varepsilon_s' E = 0.003\left(\frac{6.6in - 3''}{6.6''}\right)(29{,}000{,}000 psi) = 47{,}455 psi$

$P_n = 0.85(6000psi)(4.95in)(14'') + (2.94in^2)(47{,}455psi) - (2.94in^2)(60{,}000p$

$P_n = 316{,}548\ lb$

$\phi P_n = (0.80)(0.65)(P_n = 316{,}548 lb)$

$\phi P_n = 164{,}605\ lb$

$\phi P_n = 164{,}605\ lb > P_u = 128{,}852\ lb$
ok

$\Rightarrow$ Select 4 NO. 11 bars
$A_s = 6.24 in^2$
(from table A.2)

$\frac{A_s}{A_g} = \frac{6.24 in^2}{196 in^2} = 0.032$

assume $A_s = A_s'$, so $A_s = 3.12 in^2$
$A_s' = 3.12 in^2$

$P_n = 0.85(6000psi)(4.95)(14'') + (3.12in^2)(47{,}455psi) - (3.12in^2)(60{,}000psi)$

$P_n = 314{,}290\ lb$

$\phi P_n = 0.80(0.65)(P_n = 314{,}290 lb)$

$\phi P_n = 163{,}431\ lb$

$\phi P_n = 163{,}431 lb > P_u = 128{,}852 lb$
ok

Check column capacity w/ added self weight:
$w_{con} = 150 pcf\,(b = 14'' \times h = 14'')\left(\frac{1ft}{12''}\right)^2 (12' \times 3 \text{ stories})$
$w_{con} = 7{,}350\ lb$

$P_u = 1.2(35{,}960 lb + 7{,}350 lb) + 1.6(52{,}000 lb) + 0.5(5{,}000 lb)$

$P_u = 137{,}672\ lb$

$\phi P_n = 163{,}431 lb > P_u = 137{,}672 lb$
ok

Design of columns: (#3) group

Roof loads:

$D_L = (20\,psf)\left(\frac{20'}{2}+\frac{20'}{2}\right)\left(\frac{20'}{2}+\frac{20'}{2}\right) = 8{,}000\ lb$

$L_r = 20\,psf\left(\frac{20'}{2}+\frac{20'}{2}\right)\left(\frac{20'}{2}+\frac{20'}{2}\right) = 8{,}000\ lb$

$S = 25\,psf\left(\frac{20'}{2}+\frac{20'}{2}\right)\left(\frac{20'}{2}+\frac{20'}{2}\right) = 10{,}000\ lb$

$D_L = 8{,}000\ lb + (150\,pcf)(3'')\left(\frac{ft}{12''}\right)\left(\frac{20'}{2}+\frac{20'}{2}\right)\left(\frac{20'}{2}+\frac{20'}{2}\right) + 2(45.5\ lb/ft)\left(\frac{20'}{2}+\frac{20'}{2}\right)$

$D_L = 24{,}820\ lb$
$L_r = 8{,}000\ lb$
$S = 10{,}000\ lb$

Floor loads: (2nd and 3rd Floor)

$D_L = 10\,psf\left(\frac{20'}{2}+\frac{20'}{2}\right)\left(\frac{20'}{2}+\frac{20'}{2}\right) + (150\,pcf)(3'')\left(\frac{ft}{12''}\right)\left(\frac{20'}{2}+\frac{20'}{2}\right)\left(\frac{20'}{2}+\frac{20'}{2}\right) + 2(45.5\ lb/ft)\left(\frac{20'}{2}+\frac{20'}{2}\right)$

$D_L = 20{,}820\ lb$

$L_L = 130\,psf\left(\frac{20'}{2}+\frac{20'}{2}\right)\left(\frac{20'}{2}+\frac{20'}{2}\right) = 52{,}000\ lb$

Total Loads:

Dead load from 2 floors and roof = 24,820 lb + 20,820 + 20,820 = 66,460 lb

Floor Live load = 52,000 + 52,000 = 104,000 lb
Roof Snow = 10,000 lb
Roof live load = 8,000 lb

load combination

$P_u = 1.2D + 1.6L + 0.5S = 1.2(66{,}460\ lb) + 1.6(104{,}000\ lb) + 0.5(10{,}000\ lb)$

$P_u = 251{,}152\ lb$

Square column Design

$\phi = 0.65$

$\phi P_n > P_u$

$P_n = \frac{P_u = 251{,}152}{\phi = 0.65}$

$P_n = 386{,}388\ lb$

use ratio $\frac{A_s}{A_g} = 0.03$

$P_n = \phi\, 0.80 [0.85 f_c' A_g + 0.03 A_g f_y]$

$386{,}388\ lb = (0.65)(0.80) A_g \{0.85(6000\,psi) + 0.03(60{,}000\,psi)\}$

$A_g = \frac{386{,}388}{3588} = 107.7\ in^2$

try $b = 16in$ $h = 16in$

$A_g = (b=16in)(h=16in) = 256in^2$

$A_s = 0.03\,(A_g = 256in^2) = 7.68in^2$

Assume $A_s' = A_s$, so $A_s = 3.84in^2$
$A_s' = 3.84in^2$

P_{nb}, M_{nb}
$\varepsilon_s = 0.002$
$\varepsilon_{con} = 0.003$

$C_b = \frac{0.003}{0.003+0.002}(d = 16'' - 3'') = 7.8in$

$a_b = (\beta_1 = 0.75)(C_b = 7.8in) = 5.85in$

$P_n = 0.85(6000psi)(5.85'')(16'') + (3.84in^2)(53{,}539psi) - (3.84in^2)(60{,}000psi)$

$f_s' = \varepsilon_s' E = 0.003\left(\frac{7.8in - 3''}{7.8in}\right)(29{,}000{,}000psi) = 53{,}539\ Psi$

$P_n = 452{,}550\ lb$

$\phi P_n = (0.80)(0.65)(P_n = 452{,}550\ lb)$

$\phi P_n = 235{,}326\ lb$

$\phi P_n = 235{,}326\ lb < P_u = 251{,}152\ lb$
Not ok

try $b = 17in$ $h = 17in$

$A_g = (b=17in)(h=17in) = 289in^2$

$A_s = 0.03\,(A_g = 289in^2) = 8.67in^2$

$A_s = 4.335in^2$
$A_s' = 4.335in^2$

$C_b = \frac{0.003}{0.003+0.002}(d = 17'' - 3'') = 8.4in$

$a_b = (\beta_1 = 0.75)(C_b = 8.4in) = 6.3in$

$f_s' = \varepsilon_s' E = 0.003\left(\frac{8.4 - 3''}{8.4''}\right)(29{,}000{,}000psi) = 55{,}929\ Psi$

$P_n = 1.85(6000psi)(6.3in)(17'') + (4.335in^2)(55{,}929psi) - (4.335in^2)(60{,}000psi)$

$P_n = 528{,}562\ lb$

$\phi P_n = (0.80)(0.65)(P_n = 528{,}562\ lb) = 274{,}852\ lb$

$\phi P_n = 274{,}852\ lb > P_u = 251{,}152\ lb$
ok

$\Rightarrow$ Select 4 No. 14 bars
$A_s = 9.00in^2$ $A_s = 4.5in^2$
$A_s' = 4.5in^2$

$P_n = 0.85(6000psi)(6.3in)(17'') + (4.5in^2)(55{,}929psi) - (4.5in^2)(60{,}000psi)$

$P_n = 527{,}891\ lb$

$\phi P_n = (0.80)(0.65)(P_n = 527{,}891\ lb) = 274{,}503\ lb > P_u = 251{,}152\ lb$ ok

Check column capacity w/ added self weight:

$W_{con} = 150\,pcf\,(b = 17'' \times h = 17'')\left(\frac{1\,ft}{12''}\right)^2 (12' \times 3 \text{ stories})$

$W_{con} = 10,838\ lb$

$P_u = 1.2(66,460\,lb + 10,838\,lb) + 1.6(104,000\,lb) + 0.5(10,000\,lb)$

$P_u = 264,158\ lb$

$\phi P_n = 274,503\,lb > P_u = 264,158\ lb$

OK

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10/24/12

Design Loads and moments:

1.0 During construction:

wet concrete: $\left[\left(\frac{5''}{12} \times 145\ ^{lb}/_{ft} \times 8'\right) \times 1.1\right] = 531.67\ ^{lb}/_{ft}$ 91.67 lb/ft

Construction LL $= 20psf \times 8' = 160\ ^{lb}/_{ft}$

$S = 25psf \times 8' = 200\ ^{lb}/_{ft}$

$w_u = 1.2D + 1.6L + .5S$ decking $M_c = \frac{wl^2}{8}$ $l = 50'$

$w_u = 1.2(0) + 1.6(160\ ^{lb}/_{ft} + 531.67\ ^{lb}/_{ft} + 16\ ^{lb}/_{ft}) + .5(200\ ^{lb}/_{ft})$

$w_c = 1232.3\ ^{lb}/_{ft}$

After construction:

Dead load $= \left[\left(\frac{5''}{12} \times 145\ ^{lb}/_{ft} \times 8'\right) \times 1.1 + (20 \times 8')\right] = 691.7\ ^{lb}/_{ft}$

LL: $230psf \times 8' = 1840\ ^{lb}/_{ft}$

Decking $= 2psf \times 8' = 16\ ^{lb}/_{ft}$

$S = 25 \times 8' = 200\ ^{lb}/_{ft}$

$w_u = 1.2D + 1.6L + .5S$

$w_u = 1.2(16\ ^{lb}/_{ft} + 691.7\ ^{lb}/_{ft}) + 1.6(1840\ ^{lb}/_{ft}) + .5(200\ ^{lb}/_{ft})$

$w_A = 846\ ^{lb}/_{ft} + 2944\ ^{lb}/_{ft} + 100\ ^{lb}/_{ft}$

$w_A = 3890\ ^{lb}/_{ft}$

$l = 50'$

$M_c = \frac{wl^2}{8}$ $M_A = \frac{w_A l^2}{8}$

$M_c = \frac{1232.3\ ^{lb}/_{ft}(50')^2}{8}$ $M_A = \frac{3890\ ^{lb}/_{ft}(50')^2}{8}$

$M_c = 385.1^{\,'K}$ $M_A = 1215.6^{\,'K}$

Full composite

$\phi M_p = 385.1^{1K}$ $a = 1''$ initially

$M_u = 121526^{1K}$ $y_2 = 5'' - a/2 = 4.5'' = y_2$

W 24x68 → $\phi M_p = 574^{1K}$ M_u @ $4.5''$ $y_2 = 1120^{1K}$

$W_c = 1.2D + 1.6L + .5S$

$W_c = 1.2(68) + 1.6(160^{lb/ft} + 531.67 + 16) + .5(200\ lb/ft)$

$W_c = 1313.9\ ^{lb}/_{ft}$

$$M_c = \frac{wl^2}{8} = \frac{(1313.9)(50^2)}{8} = 410.6^{1K} = M_c$$

$W_A = 1.2(68 + 16 + 691.74) + 1.6(1840) + .5(200)$

$W_A = 930.9 + 2944 + 100$

$W_A = 3974.9\ ^{lb}/_{ft}$

$$M_A = \frac{wl^2}{8} = \frac{3974.9(50^2)}{8} = 1242.2^{1K} = M_A$$

$M_c \leq \phi M_p$ $410.6^{1K} \leq 664^{1K}$ ✓

$M_A < M_A @ 4.5''$ $1242.2^{1K} \leq 1230^{1K}$ ✗

W 24x76 $\phi M_p = 750^{1K}$ M_u @ $4.5''$ $y_2 = 1380^{1K}$

$W_c = 1.2D + 1.6L + .5S$

$W_c = 1.2(76) + 1.6(160 + 531.7 + 16) + .5(200)$

$W_c = 1323.5\ ^{lb}/_{ft}$

$$M_c = \frac{wl^2}{8} = \frac{(1323.5)(50^2)}{8} = 413.6^{1K} = M_c$$

$W_A = 1.2(76 + 16 + 691.74) + 16(1840) + .5(200)$

$W_A = 940.4 + 2944 + 100$

$W_A' = 3984.4\ ^{lb}/_{ft}$

$$M_A = \frac{wl^2}{8} = \frac{(3984.4)(50^2)}{8} = 1245^{1K} = M_A$$

$M_c \leq \phi M_p$ $413.6^{1K} \leq 750^{1K}$ ✓

$M_A \leq M_A @ 4.5''$ $1245^{1K} \leq 1380^{1K}$ ✓

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$a = \frac{\Sigma Q_n}{.85 F'_c b_E}$

$\Sigma Q_n = 1120$ kips

$f'_c = 4$ ksi

$b_E = 2 \times \frac{L}{8} = 2 \times \frac{50 \times 12}{8} \leq 150''$

$b_E = 2 \times \frac{8' \times 12}{2} \leq \underline{96''} \rightarrow$ governs

$a = \frac{1120 \text{ kips}}{.85 (4 ksi)(96'')}$

$a = 3.43'' < 5$

$Y_2 = 5'' - \frac{3.43}{2} = 3.28'' \rightarrow 3''$

$M_A \leq M_n @ 3''$ $\quad 1245^{1k} \leq 1260^{1k}$ ✓

Deflection checks:

max: $L/360 = \frac{50' \times 12''/'}{360} = 1.67''$

$1.67'' \geq 1''$

$1.67'' =$ max construction

$1'' =$ max in service

In service-

$\Delta = \frac{5 w l^4}{384 E I} = \frac{5 (1120 \text{ lb/ft}) (50')^4}{384 (29000000) (4600 in^4)} \times 1728 \frac{in^3}{ft^3}$

$\Delta = .97'' < 1''$ ✓

$w = \frac{\cancel{...} + 1840 \text{ lb/ft}}{2} = 920 \text{ lb/ft}$

$l = 50'$

$E = 29{,}000{,}000$ psi

table 3-20 $\rightarrow I_x = 4600 \text{ in}^4$

Construction:

$\Delta = \frac{5 w l^4}{384 E I} = \frac{5 (611.7 \text{ lb/ft}) (50')^4}{384 (29000000) (2100 in^4)} \times 1728 \frac{in^3}{ft^3}$

$\Delta = 1.41'' < 1.5''$ ✓

$w = 531.67 \text{ /ft} + \frac{20 \times 8}{2} = 611.7 \text{ lb/ft}$

$l = 50'$

$E = 29{,}000{,}000$

$I = 2100 \text{ in}^4$

Studs:

$Q_n = .5 A_{sa} \sqrt{F'_c E_c} \leq A_{sa} F_u R_g R_p$

$Q_{n_1} = .5 (.4418) \sqrt{(4)(3492)} = 26.1$ kip/stud

$Q_{n_2} = .4418 (65)(1.0)(.75) = \underline{21.5 \text{ k/stud}}$

$A_{sa} = .4418 \text{ in}^2$

$F_u = 65$ ksi

$F'_c = 4$ ksi

$E_c = (w^{1.5}) \sqrt{F'_c} = 3492$ ksi

$R_g = 1.0$

$R_p = .75$

$n = \frac{\Sigma Q_n}{Q_n} = \frac{1120}{21.5} = 53 \text{ studs} \times 2 = 106 \text{ studs}$

spacing $= \frac{50' \times 12''/'}{106+1} = \underline{5.6'' \text{ spacing}}$

$S_{min} = 6 \times d_s = 4.5''$

$S_{max} = 8 t_s = 36''$

$4.5'' \leq 5.6'' \leq 36''$ ✓

$\boxed{W\ 24 \times 76\ (106)}$

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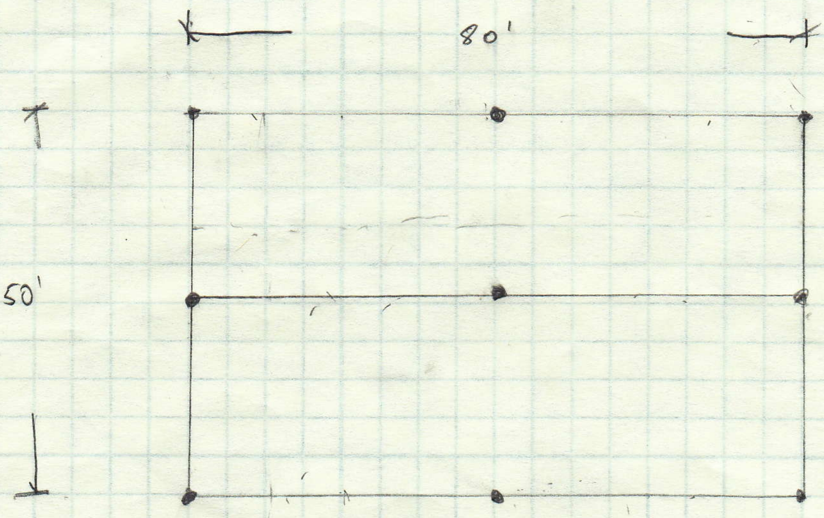
10/25/12

Girders:

$$W_{DL} = \left[691.7\ ^{lb}/_{ft} + \frac{76\ ^{lb}/_{ft}}{8'} \right] \times 80'$$

$W_{DL} = 56096$ psf ⇒ too big

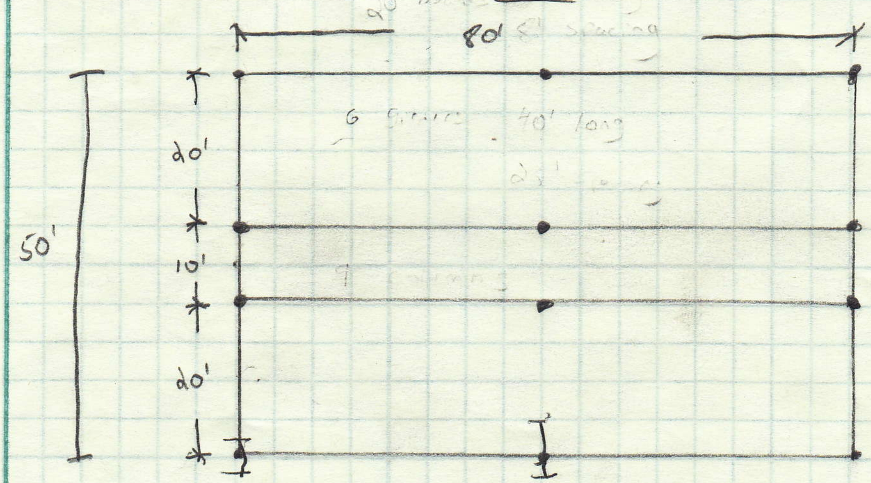
- 20 beams: 25' long, 8' spacing
- 6 girders: 40' long, 25' spacing
- 9 columns



4000 ft²
× 3 stories
12000 ft²

1 acre = 43560 ft²

or



Use this One

- 20 beams: 20' long, 8' spacing and 10 beams: 10' long, 8' spacing
- 8 girders: 40' long,
- 12 columns

Design loads: corridor beams; 2nd and 3rd floor.

During construction:

wet concrete $-\left[\left(\frac{5''}{12} \times 145\ ^{lb}/_{ft} \times 8'\right) \times 1.1\right] = 531.67\ ^{lb}/_{ft}$

Construction L_L = 20 psf $\times 8' = 160\ ^{lb}/_{ft}$

decking = 2psf $\times 8' = 16\ ^{lb}/_{ft}$

$W_u = 1.2D + 1.6L + .5S$

$W_u = 1.\cancel{2}(0) + 1.6(160\ ^{lb}/_{ft} + 531.67\ ^{lb}/_{ft} + 16\ ^{lb}/_{ft}) + .\cancel{5}(0)$

$W_c = 1132.3\ ^{lb}/_{ft}$

After construction:

Dead load: $= \left[\left(\frac{5''}{12} \times 145\ ^{lb}/_{ft} \times 8'\right) \times 1.1 + \left[\left(\underset{\text{ceiling}}{3\text{ psf}} + \underset{\text{MEP}}{5\text{ psf}} + \underset{\text{insulation}}{2\text{ psf}}\right) 8'\right]\right] = 611.7\ ^{lb}/_{ft}$

LL = 80 psf $\times 8 = 640\ ^{lb}/_{ft}$

Decking = 2psf $\times 8' = 16\ ^{lb}/_{ft}$

$W_u = 1.2D + 1.6L$

$W_u = 1.2(16 + 611.7) + 1.6(640)$

$W_A = 1777.2\ ^{lb}/_{ft}$

$\ell = 10'$

$M_c = \frac{w_c \ell^2}{8}$

$M_c = \frac{(1132.3\ ^{lb}/_{ft})(10')^2}{8}$

$M_c = 14.15\ ^{1k}$

$M_A = \frac{w_A \ell^2}{8}$

$M_A = \frac{(1777.2\ ^{lb}/_{ft})(10')^2}{8}$

$M_A = 22.2\ ^{1k}$

Full composite

$a = 1''$ initially

$\varphi M_p = 14.15'^k$

$Y_2 = 5'' - a/2 = 4.5'' = Y_2$

$M_u = 22.2'^k$

$W10 \times 12 \rightarrow \varphi M_p = 46.9'^k$ $M_u @ 4.5'' = 125'^k$

$W_c = 1.2D + 1.6L$
$W_c = 1.2(12) + 1.6(160\ \#/ft + 531.67 + 16)$
$W_c = 1146.7\ lb/ft$

$$M_c = \frac{wl^2}{8} = \frac{1146.7(10^2)}{8} = 14.3'^k$$

$W_A = 1.2(12 + 16 + 611.7) + 1.6(640)$

$W_A = 1791.6\ lb/ft$

$$M_A = \frac{wl^2}{8} = \frac{1791.6(10^2)}{8} = 22.4'^k$$

$M_c \le \varphi M_p \rightarrow 14.3'^k \le 46.9'^k$ ✓

$M_A \le M_u \rightarrow 22.4'^k \le 125'^k$ ✓

$a = \frac{\Sigma Q_n}{.85 f'_c b_E}$ $\Sigma Q_n = 177$ kips
$f'_c = 4ksi$
$b_E = 2 \times \frac{L}{8} = 2 \times \frac{10 \times 12}{8} \le \underline{30''}$ governs
$a = \frac{177 \text{ kips}}{.85(4ksi)(30'')}$ $b_E = 2 \times \frac{8' \times 12}{2} \le 96''$

$a = 1.74'' < 5$

$Y_2 = 5'' - \frac{1.74''}{2} = 4.1 \rightarrow 4''$

$M_A < M_n @ 4''$ $22.4'^k < 119'^k$ ✓

Deflection checks

max ; $L/360 = \frac{10' \times 12''}{360} = .33''$

In service

$w = \frac{640\ lb/ft}{2} = 320\ lb/ft$

$$\Delta = \frac{5wl^4}{384EI} = \frac{5(320\ lb/ft)(10')^4}{384(29000000)(195 in^4)} \times 1728 \frac{in^3}{ft^3}$$

$l = 10'$
$E = 29{,}000{,}000$ psi
table 3-20 $I_x = 195 in^4$

$\Delta = .013'' < .33''$ ✓

Construction

$w = 531.6 + \frac{160}{2} = 611.7\ lb/ft$

$$\Delta = \frac{5wl^4}{384EI} = \frac{5(611.7)(10')^4}{384(29000000)(53.8)} \times 1728 \frac{in^3}{ft^3}$$ table 1-1 $I_x = 53.8 in^4$

$\Delta = .09'' < .33''$ ✓

Studs:

$Q_n = .5 A_{sa} \sqrt{f'_c E_c} \leq A_{sa} F_u R_g R_p$

$Q_{n1} = .5(.4418)\sqrt{4(3492)} = 26.1\ ^{k}/stud$

$Q_{n2} = .4418(65)(1.0)(.75) = \underline{21.5\ ^{k}/stud}$

$n = \frac{\Sigma Q_n}{Q_n} = \frac{177 \text{ kips}}{21.5\ ^{k}/stud} = 9 \text{ studs} \times 2 = 18 \text{ studs}$

$\text{spacing} = \frac{10' \times 12''}{18+1} = 6.3'' \text{ spacing}$

$S_{min} = 6 \times d_s = 4.5''$

$4.5'' \leq 6.3'' \leq 36''$ ✓

$S_{max} = 8 t_s = 36''$

$\boxed{W\,10 \times 12\,(18)}$ → very over designed

Change spacing to 16'

During construction

$\text{wet conc} = \left[\left(\frac{5}{12} \times 145 \times 16'\right)1.1\right) = 1063.3\ ^{lb}/ft$

$w_u = 1.2D + 1.6L$

$\text{construction } L_c = 20 \times 16' = 320\ ^{lb}/ft$

$w_u = 1.2(0) + 1.6(1063.3 + 320 + 32) = 2264.5\ ^{lb}/ft$

$\text{decking} = 2 psf \times 16' = 32\ ^{lb}/ft$

$M_c = \frac{w_c l^2}{8} = \frac{(2264.5\ ^{lb}/ft)(10')^2}{8}$

$\underline{M_c = 28.3\ ^{1k}}$

After construction:

$\text{Dead load} = \left[\left(\frac{5}{12} \times 145' \times 16'\right)1.1\right) + \left[(3 psf + 5 psf + 2 psf)16'\right] = 1223.3\ ^{lb}/ft$

$LL = 80 psf \times 16' = 1280\ ^{lb}/ft$

$\text{Decking} = 32\ ^{lb}/f$

$w_u = 1.2D + 1.6L$

$w_u = 1.2(1223.3 + 32) + 1.6(1280) = 3554.4\ ^{lb}/ft$

$M_A = \frac{wl^2}{8}$

$M_A = \frac{(3554.4\ ^{lb}/ft)(10')^2}{8}$

$\underline{M_A = 44.4\ ^{1k}}$

$\phi M_p = 28.3'^{K}$

$M_u = 44.4'^{K}$

W10 x 12

$W_c = 1.2(12) + 1.6(1063.3 + 320 + 32) = 2279\ ^{lb}/_{ft}$

$$M_c = \frac{wl^2}{8} = \frac{2279(10')^2}{8}$$

$$M_c = 28.5'^{K}$$

$W_A = 1.2(12 + 1223.3 + 32) + 1.6(1280) = 3569\ ^{lb}/_{ft}$

$$M_A = \frac{wl^2}{8} = \frac{3569(10')^2}{8}$$

$$M_A = 44.6'^{K}$$

$M_c \leq \phi M_p \rightarrow 28.5'^{K} \leq 46.9'^{K}$

$M_A \leq M_u \rightarrow 44.6'^{K} \leq 125'^{K}$

Deflection checks:

max = .33"

In service:

only change in w

$w = \frac{1280}{2} = 640\ ^{lb}/_{ft}$

$$\Delta = \frac{5wl^4}{384EI} = \frac{5(640\ ^{lb}/_{ft})(10')^4}{384(29000000)(195 in^4)} \times 1728$$

$$\Delta = .025'' < .33'' \checkmark$$

Construction:

$w = \frac{1063.3}{2} + \frac{320}{2} = 1223.3\ ^{lb}/_{ft}$

$$\Delta = \frac{5wl^4}{384EI} = \frac{5(1223.3)(10')^4}{384(29000000)(53.8)} \times 1728$$

$$\Delta = .18'' < .33'' \checkmark$$

same $\boxed{W10 \times 12\ (18)}$

works with 16' spacing.

Design loads: office beams ; 2nd and 3rd floor

During construction –

Wet concrete – $\left[\left(\frac{5''}{12} \times 145\ ^{lb}/_{ft} \times 16'\right) \times 1.1\right] = 1063.3\ ^{lb}/_{ft}$

Construction $L_L = 20 \times 16' = 320\ ^{lb}/_{ft}$

decking $= 2psf \times 16' = 32\ ^{lb}/_{ft}$

$w_u = 1.2D + 1.6L$

$w_u = 1.2(0) + 1.6(1063.3 + 320 + 32)$

$w_c = 2264.5\ ^{lb}/_{ft}$

$M_c = \frac{w_c l^2}{8} = \frac{(2264.5)(20')^2}{8}$

$M_c = 113.2^{1k}$

After construction

Dead Load $= \left[\left(\frac{5}{12} \times 145 \times 16'\right)1.1\right] + \left[(3+5+2)16\right] = 1223.3\ ^{lb}/_{ft}$

$L_L = 50 \times 16' = 800\ ^{lb}/_{ft}$

Decking $= 32\ ^{lb}/_{ft}$

$w_u = 1.2D + 1.6L$

$w_u = 1.2(32 + 1223.3) + 1.6(800)$

$w_u = 2786.4\ ^{lb}/_{ft}$

$M_A = \frac{w_u l^2}{8} = \frac{(2786.4)(20')^2}{8}$

$M_A = 139.4^{1k}$

Full composite

$\phi M_p = 113.2^{1k}$

$M_u = 139.4^{1k}$

$W14 \times 22 \rightarrow \phi M_p = 125^{1k}$ $\quad M_u @ 4.5'' = 276^{1k}$

$w_c = 1.2(22) + 1.6(1063.3 + 320 + 32)$

$w_c = 2291\ ^{lb}/_{ft}$

$M_c = \frac{2291(20')^2}{8} = M_c = 114.5^{1k}$

$w_A = 1.2(22 + 32 + 1223.3) + 1.6(800)$

$w_A = 2813\ ^{lb}/_{ft}$

$M_A = \frac{2813(20')^2}{8} = M_A = 140.6^{1k}$

$M_c \le \phi M_p \rightarrow 114.5^{1k} \le 125^{1k}$ ✓

$M_A \le M_u \rightarrow 140.6^{1k} \le 276^{1k}$ ✓

$a = \frac{\Sigma Q_n}{.85 f'_c b_E}$

$b_E = 2 \times \frac{L}{8} = 2 \times \frac{20' \times 12}{8} = \cancel{60''}$

$b_E = 2 \times \frac{s}{2} = 2 \times \frac{16' \times 12}{2} = 192''$

$a = \frac{325}{.85(4)(60'')}$

$a \approx 1.6 < 5$

$Y_2 = 5'' - \frac{1.6}{2} = 4.2 \rightarrow 4''$

$M_u = 264'' > 140.6^{k}$ ✓

Justin Furst

10/31/12

Deflection checks:

$$\text{Max} \rightarrow L/360 = \frac{20' \times 12}{360} = .66''$$

In Service

$$\Delta = \frac{5wl^4}{384EI} = \frac{5(400\,\text{lb/ft})(20')^4}{384(29000000)(581\,\text{in}^4)} \times 1728\,\frac{\text{in}^3}{\text{ft}^3}$$

$w = \frac{800}{2} = 400\,\text{lb/ft}$

$l = 20'$

table 3-20 $I_x = 581\,\text{in}^4$

$$\Delta = .085'' < .66'' \checkmark$$

Construction

$$\Delta = \frac{5wl^4}{384EI} = \frac{5(1223.3)(20')^4}{384(29000000)(199\,\text{in}^4)} \times 1728\,\frac{\text{in}^3}{\text{ft}^3}$$

$w = 1063.3 + \frac{320}{2} = 1223.3\,\text{lb/ft}$

$l = 20'$

table 1-1 $I_x = 199\,\text{in}^4$

$$\Delta = .76'' < .66'' \times$$

shored • fine still works

Studs:

$Q_n = 21.5\,\text{k/stud}$

$$n = \frac{\Sigma Q_n}{Q_n} = \frac{325}{21.5} = 15.1 = 16\text{ studs} \times 2 = 32\text{ studs.}$$

$$\text{spacing} = \frac{20' \times 12''/'}{32+1} = 7.27''\text{ spacing}$$

$S_{min} = 4.5''$

$S_{max} = 36''$

$$4.5'' \leq 7.27'' \leq 36'' \checkmark$$

$$\boxed{W14 \times 22\ (32)}$$

Justin Furst

11/5/12

Exterior girders:

During construction:

$$W_{DL} = \left[2psf + \left(\tfrac{5}{12} \times 145^{lb}/ft\right)1.1 + \frac{22\ lbs/ft}{16'} \right](10') = 698\ ^{lb}/ft$$

$$W_{LL} = [20\ psf]\ 10' = 200\ ^{lb}/ft$$

$$W_C = 1.2D + 1.6L$$

$$W_C = 1.2(698\ ^{lb}/ft) + 1.6(200\ ^{lb}/ft) = 1157.6\ ^{lb}/ft$$

$$M_C = \frac{w l^2}{8} = \frac{(1157.6\ ^{lb}/ft)(40')^2}{8} = \underline{231.5\ ^{1k} = M_C}$$

After construction:

$$W_{DL} = \left[12psf + \left(\tfrac{5}{12} \times 145\ ^{lb}/ft\right)1.1 + \frac{22\ lbs/ft}{16'} \right](10') = 798.3\ ^{lb}/ft$$

$$W_{LL} = [50\ psf]\ 10' = 500\ ^{lb}/ft$$

$$W_A = 1.2D + 1.6L$$

$$W_A = 1.2(798.3\ ^{lb}/ft) + 1.6(500\ ^{lb}/ft) = 1758\ ^{lb}/ft$$

$$M_A = \frac{w l^2}{8} = \frac{(1758\ ^{lb}/ft)(40')^2}{8} = \underline{351.6\ ^{1k} = M_A}$$

Full composite:

$\phi M_p = 231.5^{1k}$

$M_u = 351.6^{1k}$

$$W18 \times 35 \rightarrow \phi M_p = 249^{1k} \rightarrow M_u @ 4.5'' = 516^{1k}$$

$$W_C = 1.2(698\ ^{lb}/ft + 35) + 1.6(200) = 1199.6\ ^{lb}/ft$$

$$M_C = \frac{1199.6(40')^2}{8} = \underline{240^{1k} = M_C}$$

$$W_A = 1.2(798.3 + 35) + 1.6(500) = 1800\ ^{lb}/ft$$

$$M_A = \frac{1800(40')^2}{8} = \underline{360^{1k} = M_A}$$

$$M_C \leq \phi M_p \rightarrow 240^{1k} \leq 249^{1k}\ \checkmark$$

$$M_A \leq M_u \rightarrow 360^{1k} \leq 516^{1k}\ \checkmark$$

$$a = \frac{\Sigma Q_n}{.85 f'_c B_e} \qquad B_E = 2 \times \frac{L}{8} = 2 \times \frac{40' \times 12}{8} = 120''$$

$$b_E = 2 \times \frac{10' \times 12}{2} = 120''$$

$$a = \frac{515}{.85(4)(120)} = 1.26 \qquad Y_2 = 4'' \qquad M_u = 496^{1k} \geq 360^{1k}\ \checkmark$$

Deflection checks:

$$\max \rightarrow L/360 = \frac{40' \times 12''/'}{360} = 1.33''$$

In service

$$\Delta = \frac{5 w l^4}{384 E I} = \frac{5(250^{lb}/ft)(40')^4}{384(29000000)(1300 in^4)} \times 1728 \frac{in^3}{ft^3}$$

$$w = \frac{500\ lb/ft}{2} = 250^{lb}/ft$$

table 3-2 $I_x = 1300\ in^4$

$\Delta = .4'' < 1.33''$ ✓

construction:

$$\Delta = \frac{5 w l^4}{384 E I} = \frac{5(798 \frac{lb}{ft})(40')^4}{384(29000000)(510 in^4)} \times 1728 \frac{in^3}{ft^3}$$

$$w = 698 + \frac{200}{2} = 798\ ^{lb}/ft$$

table 1-1 $I_x = 510\ in^4$

$\Delta = 3.1'' < 1.33''$ ✗ problem

studs:

$Q_n = 21.5\ ^k/stud$

$$n = \frac{515\ kips}{21.5\ k/stud} = 24\ studs \times 2 = 48\ studs$$

$$\text{spacing} = \frac{40' \times 12''/'}{48+1} = 9.8''$$

$4.5'' \le 9.8'' \le 36''$ ✓

$\boxed{W18\times35\ (48)}$

Interior Girders

During construction

$$W_{DL} = \left[2psf + \left(\tfrac{5}{12} \times 145\right)1.1 + \frac{22}{16}\right](10') + \left[2psf + \left(\tfrac{5}{12} \times 145\right)1.1 + \frac{12\ lb/ft}{16'}\right](5') = 1014\ \tfrac{lb}{ft}$$

$$W_{LL} = [20psf](15) = 300\ lb/ft$$

$$W_c = 1.2D + 1.6L$$

$$W_c = 1.2(1014\ lb/ft) + 1.6(300\ lb/ft) = 1697\ lb/ft$$

$$M_c = \frac{w l^2}{8} = \frac{(1697\ lb/ft)(40')^2}{8} = \underline{339.4\ 'k = M_c}$$

After construction

$$W_{DL} = \left[12psf + \left(\tfrac{5}{12} \times 145\right)1.1 + \frac{22\ lb/ft}{16'}\right]10' + \left[12psf + \left(\tfrac{5}{12} \times 145\right)1.1 + \frac{12\ lb}{ft}\right](5') = 1195\ lb/ft$$

$$W_{LL} = [50psf]10' + [80psf]5' = 900\ lb/ft$$

$$W_A = 1.2D + 1.6L$$

$$W_A = 1.2(1195\ lb/ft) + 1.6(900\ lb/ft) = 2874\ lb/ft$$

$$M_A = \frac{w l^2}{8} = \frac{(2874\ lb/ft)(40')^2}{8} = \underline{575\ 'k = M_A}$$

Full composite:

$\varphi M_p = 339.4\ 'k$

$M_u = 575\ 'k$

$$W18x50 \rightarrow \varphi M_p = 379\ 'k, \quad M_u = 744\ 'k$$

$$W_c = 1.2(1014 + 50) + 1.6(300) = 1757\ lb/ft$$

$$M_c = \frac{wl^2}{8} = \frac{(1757)(40')^2}{8} = \underline{351.4\ 'k = M_c}$$

$$M_A = 1.2(1195 + 50) + 1.6(900) = 2934\ lb/ft$$

$$M_A = \frac{2934(40)^2}{8} = \underline{586.8\ 'k = M_A}$$

$$M_c \le \varphi M_p \longrightarrow 351.4\ 'k \le 379\ 'k \checkmark$$

$$M_A \le M_u \longrightarrow 586.8\ 'k \le 744\ 'k \checkmark$$

$$a = \frac{\Sigma Q_n}{.85(4)120}$$

$$a = \frac{735}{.85(4)(120)} = 1.8$$

$$Y_2 = 4.5 - \frac{a}{2} = 4''$$

$$M_u = 717\ 'k > 586.8\ 'k \checkmark$$

Deflection checks:

$$\max \rightarrow L/360 = \frac{40' \times 12''/'}{360} = 1.33''$$

In service

$$\Delta = \frac{5 w L^4}{384 E I} = \frac{5(450\ ^{lb}/_{ft})(40')^4}{384(29000000)(2040)} \times 1728 \frac{in^3}{ft^3}$$

$w = \frac{900}{2} = 450\ ^{lb}/_{ft}$

$I_x = 2040\ in^4$

$\Delta = .44'' < 1.33''$ ✓

Construction:

$$\Delta = \frac{5 w L^4}{384 E I} = \frac{5(1164\ ^{lb}/_{ft})(40')^4}{384(29000000)(800 in^4)} \times 1728 \frac{in^3}{ft^3}$$

$w = 1014 + \frac{300}{2} = 1164''$

$I_x = 800\ in^4$

$\Delta = 2.9'' < 1.33''$ ✗

Studs:

$n = \frac{735}{21.5} = 35$ studs $\times 2 = 70$ studs

Spacing $= \frac{40' \times 12''/'}{70+1} = 6.8''$ $4.5'' \le 6.8'' \le 36''$

$\boxed{W18 \times 50\ (70)}$

During construction deflection problems – Try girders not being composite.

Exterior girders:

$$W_{DL} = \left[12 psf + \left(\tfrac{5}{12} \times 145\ ^{lb}/_{ft}\right)1.1 + \frac{22\ ^{lb}/_{ft}}{16}\right]10' = 798.3\ ^{lb}/_{ft}$$

$$W_{LL} = [50 psf]\ 10' = 500\ ^{lb}/_{ft}$$

$$W_A = 1.2D + 1.6L = 1758\ ^{lb}/_{ft}$$

$$M_u = \frac{wL^2}{8} = \frac{(1758\ ^{lb}/_{ft})(40')^2}{8} = 351.6^{k'} = M_u$$

$$Z_x \ge \frac{M_u}{\phi F_y} = \frac{351.6k \times 12}{.9(50 ksi)} \rightarrow Z_x \ge 93.76 in^3$$

table 3-2

$W21 \times 44 \quad Z_x = 95.4 in^3$

$W_u = 1.2(798.3 + 44) + 1.6(500)$

$W_u = 1810.8\ ^{lb}/_{ft}$

$M_u = 362.2^{k'}$

$\phi M_p = \phi Z_x F_y$

$\phi M_p = .9(95.4)(50) \times \frac{1}{12}$

$\phi M_p = 357.75^{k}$

$357.75^{k'} \ge 362.2^{k'}$ ✗

$W21\times48 \rightarrow Z_x = 107\ in^3$

$w_u = 1.2(798.3+48) + 1.6(500)$
$w_u = 1815.6\ lb/ft$

$M_u = 363.12^{\prime k}$

$\phi M_p = \phi Z_x F_y$

$\phi M_p = .9(107 in^3)(50 ksi) \times \frac{1}{12}$

$\phi M_p = 401.25^{\prime k}$

$401.25^{\prime k} \geq 363.12^{\prime k}$ ✓

$\boxed{W21\times48}$

~~In~~ Deflection ~~c~~ check

~~$W_{DL} = [\ 12$~~

$\Delta_{max} = l/360 = \frac{40' \times 12''/'}{360} = 1.33''$

$w = \frac{500}{2} = 250\ lb/ft$

table 1-1 $I_x = 959 in^3$

$\Delta = \frac{5wl^4}{384EI} \times 1728 \frac{in^3}{ft^3}$

$\Delta = \frac{5(250)(40')^4}{384(29000000)(959 in^3)} \times 1728 \frac{in^3}{ft^3}$

$\Delta = .51'' < 1.33''$ ✓

Interior Girders

$w_{DL} = [12 psf + (5/12 \times 145)1.1 + \frac{22}{16}](10') + [12psf + (5/12 \times 145)1.1 + \frac{12}{16}](5') = 1195\ lb/ft$

$W_{LL} = [50 psf]10' + [80 psf]5' = 900\ lb/ft$

$w_A = 1.2D + 1.6L$
$w_A = 1.2(1195) + 1.6(900) = 2874\ lb/ft$

$M_u = \frac{wl^2}{8} = \frac{2874(40')^2}{8} = 575^{\prime k} = M_u$

$Z_x \geq \frac{M_u}{\phi F_y} = \frac{575^{\prime k} \times 12''/'}{.9(50 ksi)} \Rightarrow \underline{Z_x \geq 153.3\ in^3}$

table 3-2
$W21\times68 \quad Z_x = 160 in^3$

$w_u = 1.2(1195+68) + 1.6(900) =$

$W_u = 2956\ lb/ft$

$M_u = 591.1^{\prime k}$

$\phi M_p = \phi Z_x F_y$

$\phi M_p = .9(160 in^3)(50 ksi) \times \frac{1}{12}$

$\phi M_p = 600^{\prime k} \geq 591.1^{\prime k}$ ✓

Deflection check

$\Delta_{max} = 1.33''$

$$\Delta = \frac{5w\,l^4}{384EI} \times 1728$$

$w = \frac{900}{2} = 450$ lb/ft

$$\Delta = \frac{5(450)(40')^4}{384(29000000)(1480in^3)} \times 1728\frac{in^3}{ft^3}$$

$I_x = 1480in^3$

$\Delta = .6'' < 1.33''$ ✓

$\boxed{W21 \times 68}$

Design loads   Roof beams: Short beams

During construction

wet concrete $-\left[\left(\frac{5}{12} \times 145\,^{lb}/ft \times 16'\right)1.1\right] = 1063.3\,^{lb}/ft$

construction $L_L = 20 \times 16' = 320\,^{lb}/ft$

decking $= 2psf \times 16' = 32\,^{lb}/ft$, Snow $= 25 \times 16 = 400\,^{lb}/ft$

$W_U = 1.2D + 1.6L + .5S$

$W_U = 1.2(0) + 1.6(1063.3 + 320 + 32) + .5(400\,^{lb}/ft) = 2464.5$

$M_C = \frac{w l^2}{8} = \frac{2464.5(10')^2}{8}$

$M_C = 30.8\,^{1K}$

After construction

$DL = \left[\left(\frac{5}{12} \times 145\,^{lb}/ft \times 16'\right)1.1\right] + \left[20psf \times 16'\right] = 1383.3\,^{lb}/ft$

$L_L = 20psf \times 16 = 320\,^{lb}/ft$

$S = 25 \times 16 = 400\,lb/ft$, Decking $= 32\,^{lb}/ft$

$W_U = 1.2D + 1.6L + .5S$

$W_U = 1.2(1383.3) + 1.6(320) + .5(400)$

$W_A = 2410\,^{lb}/ft$

$M_A = \frac{w l^2}{8} = \frac{2410(10')^2}{8}$

$M_A = 30.1\,^{1K}$

W10x12 →

$W_C = 2464.5\,^{lb}/ft + 1.2(12) = 2479\,^{lb}/ft$

$M_C = \frac{w l^2}{8} = 31\,^{1K} = M_C$

$M_C \leq \phi M_p \rightarrow 31\,^{1K} \leq 46.9\,^{1K}$ ✓

$M_A \leq M_J \rightarrow 44.6\,^{1K} \leq 125\,^{1K}$ ✓

$W_A = 2410 + 1.2(12) = 2424.4$

$M_A = 30.3\,^{1K}$

Deflection checks: max = .33"

In service, $I = 195\,in^4$, $l = 10'$, $w = 160\,^{lb}/ft$

$\Delta = \frac{5 w l^4}{384 E I} \times 1728 =$ I = 195

$\Delta = .01" < .33"$ ✓

construction, $I = 53.8\,in^4$, $1223.3 = w$

$\Delta = \frac{5 w l^4}{384 E I} = \frac{5(1223.3)(10')^4}{384(29000000)(53.8)} \times 1728$

$\Delta = .18" < .33"$ ✓

$\boxed{W10 \times 12 (18)}$

Design loads — Roof beams: Short beams

During construction

wet concrete $- \left[\left(\frac{5}{12} \times 145^{lb/ft} \times 16'\right)1.1\right] = 1063.3^{lb/ft}$

construction $L_L = 20 \times 16' = 320^{lb/ft}$

decking $= 2psf \times 16' = 32^{lb/ft}$, snow $= 25 \times 16 = 400^{lb/ft}$

$W_u = 1.2D + 1.6L + .5S$

$W_u = 1.2(0) + 1.6(1063.3 + 320 + 32) + .5(400^{lb/ft}) = 2464.5$

$M_c = \frac{w\ell^2}{8} = \frac{2464.5(10')^2}{8}$

$M_c = 30.8^{1k}$

After construction

$DL = \left[\left(\frac{5}{12} \times 145^{lb/ft} \times 16'\right)1.1\right] + \left[20psf \times 16'\right] = 1383.3^{lb/ft}$

$L_L = 20psf \times 16 = 320^{lb/ft}$

$S = 25 \times 16 = 400 lb/ft$, Decking $= 32^{lb/ft}$

$W_u = 1.2D + 1.6L + .5S$

$W_u = 1.2(1383.3) + 1.6(320) + .5(400)$

$W_A = 2410^{lb/ft}$

$M_A = \frac{w\ell^2}{8} = \frac{2410(10')^2}{8}$

$M_A = 30.1^{1k}$

W10x12 →

$W_c = 2464.5^{lb/ft} + 1.2(12) = 2479^{lb/ft}$

$M_c = \frac{w\ell^2}{8} = 31^{1k} = M_c$

$M_c \le \phi M_p \rightarrow 31^{1k} \le 46.9^{1k}$ ✓

$M_A \le M_u \rightarrow 44.6^{1k} \le 125^{1k}$ ✓

$W_A = 2410 + 1.2(12) = 2424.4$

$M_A = 30.3^{1k}$

Deflection checks: $max = .33''$

In service, $I = 195 in^4$, $\ell = 10'$, $w = 160^{lb/ft}$

$\Delta = \frac{5w\ell^4}{384EI} \times 1728 =$ $I = 195$

$\Delta = .01'' < .33''$ ✓

construction, $I = 53.8 in^4$, $1223.3 = w$

$\Delta = \frac{5w\ell^4}{384EI} = \frac{5(1223.3)(10')^4}{384(29000000)(53.8)} \times 1728$

$\Delta = .18'' < .33''$ ✓

$\boxed{W10 \times 12 (18)}$

Exterior Girders for roof:

$$W_{DL} = \left[20psf + \left(\tfrac{5}{12} \times 145\right)1.1 + \frac{26\ lb/ft}{16}\right]10' = 881.8\ lb/ft$$

$$W_{LL} = [20\ psf]\ 10' = 200\ lb/ft$$

$$W_S = [25\ psf]\ 10' = 250\ lb/ft$$

$$W_u = 1.2D + 1.6L + .5S = 1502.2\ lb/ft$$

$$M_u = \frac{W_u l^2}{8} = \frac{(1502.2)(40')^2}{8} = 300.4\ ^{k} = M_u$$

$$Z_x \geq \frac{M_u}{\varphi F_y} = \frac{300.4\ ^{k} \times 12}{.9\ (50ksi)} = Z_x \geq 80.1\ in^3$$

table 3-2

$$W_u = 1.2(881 + 44) + 1.6(200) + .5(250) \quad W21 \times 44 \quad Z_x \geq 95.4\ in^3$$

$$W_u = 1555\ lb/ft$$

$$M_A = 311\ ^{k}$$

$$\varphi M_p = \varphi Z_x F_y$$

$$\varphi M_p = .9(95.4\ in^3)(50) \times \frac{1}{12}$$

$$\varphi M_p = 357.75\ ^{k}$$

$$311\ ^{k} < 357.75\ ^{k} \checkmark$$

Deflection check

$$max = 1.33''$$

$$\Delta = \frac{5wl^4}{384EI} \qquad w = \frac{200\ lb/ft}{2} = 100\ lb/ft$$

$$I = 843\ in^4$$

$$\Delta = \frac{5(100)(40')^4}{384(29000000)(843)} \times 1728$$

$$\Delta = .24'' < 1.33'' \checkmark \qquad \boxed{W21 \times 44}$$

Interior Girders:

$$W_{DL} = \left[20 + \left(\tfrac{5}{12} \times 145\right)1.1 + \frac{26}{16}\right]10' + \left[20 + \left(\tfrac{5}{12} \times 145\right)1.1 + \frac{12}{16}\right]5' = 1226.2\ lb/ft$$

$$W_{LL} = [20psf] \times 15' = 300\ lb/ft$$

$$W_S = [25\ psf] \times 15' = 375\ lb/ft$$

$$W_u = 1.2D + 1.6L + .5S = 2140\ lb/ft$$

$$M_u = \frac{wl^2}{8} = 428\ ^{k} = M_u$$

$$Z_x \geq \frac{M_u}{\phi F_y} = \frac{428^{1k} \times 12}{.9(50\,ksi)} = Z_x \geq 114.2\,in^3$$

$W21 \times 55 \rightarrow Z_x = 126\,in^3$

$W_u = 1.2(1226.2 + 55) + 1.6(300) + 0.5(375)$

$W_u = 2206\ ^{lb}/ft \rightarrow M_u = 441.2^{1k}$

$\phi M_p = \phi Z_x F_y$

$\phi M_p = .9(126\,in^3)(50) \times \frac{1}{12}$

$\phi M_p = 472.5^{1k} > 441.2^{1k}$ ✓

Deflection check:

$$\Delta = \frac{5 w l^4}{384 E I} \times 1728$$

$w = \frac{300}{2} = 150\ ^{lb}/ft$

$I = 1140\ in^4$

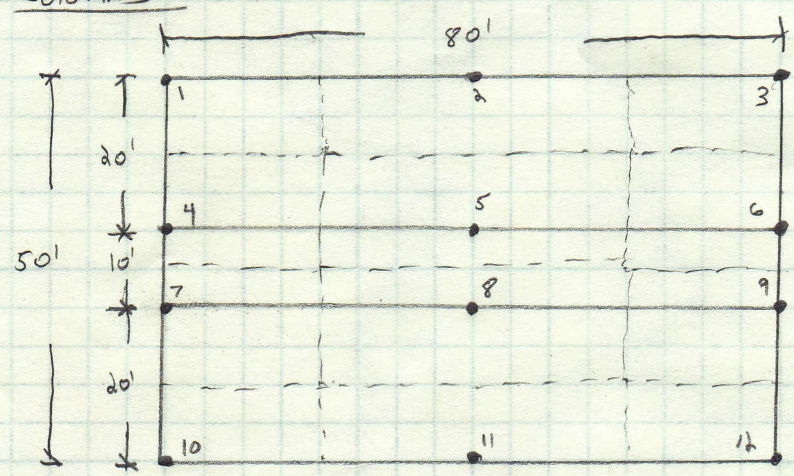
$max = 1.33''$

$$\Delta = \frac{5(150\ ^{lb}/ft)(40')^4}{384(29000000)(1140\,in^4)} \times 1728$$

$\Delta = .26'' < 1.33''$ ✓

$\boxed{W21 \times 55}$

Columns:



Column Groups

Group A = 1, 3, 10, 12 - W10x33

Group B = 2, 11 - W10x33

Group C = 4, 7, 6, 9 - W10x33

Group D = 5, 8 - W10x39

columns A: $TA = 10' \times 20' = 200 ft^2$

Roof level

$DL = \left[\left(\frac{5}{12} \times 145\right)1.1 + 20 psf\right] \times TA + \left(26 psf \times \frac{20'}{2}\right) + \left(44 psf \times \frac{40'}{2}\right) = 18.5 kips$

$LL = [20 psf] \times TA = 4 kips$

$S = [25 psf] \times TA = 5 kips$

Floor level

$DL = \left[\left(\frac{5}{12} \times 145\right)1.1 + 10 psf\right] \times TA + \left(22 psf \times \frac{20'}{2}\right) + \left(48 psf \times \frac{40'}{2}\right) = 16.5 kips$

$LL = [50 psf] \times TA = 10 kips$

$P_u = 1.2D + 1.6L + .5S$

$P_u = 1.2(2(16.5 kips) + 18.5') + 1.6(4 + 2(10 kips)) + .5(5 kips)$

$P_u = 102.7 kips$

braced $KL = 1.0 \rightarrow KL = 12 ft$ table 4-1

$\boxed{W10 \times 33}$ $P_u @ KL = 12' = 292 kips$

$\frac{K_x L}{r_x} < 4.71\sqrt{\frac{E}{F_y}} \Rightarrow 34.3 < 113.4$ ✓ $r_x = 4.19 in \times \frac{1 ft}{12} = .35$

$r_y = 1.94 in \times \frac{1}{12} = .16$

$F_e = \frac{\pi^2 E}{(KL/r_y)^2} = 50803.3$

$\frac{K_y L}{r_y} < 4.71\sqrt{\frac{E}{F_y}} \Rightarrow 75 < 113.4$ ✓

$F_{cr} = \left[.658^{(F_y/F_e)} F_y\right] = 33.14 kis$

columns B: $TA = 10' \times 40' = 400 ft^2$

Roof

$DL = \left[\left(\frac{5}{12} \times 145\right)1.1 + 20 psf\right] \times TA + \left(26 \times \frac{20'}{2}\right) + 2\left(44 psf \times \frac{40'}{2}\right) = 36.6 kips$

$LL = [20 psf] \times TA = 8 kips$

$S = [25 psf] \times TA = 10 kips$

Floor level

$DL = \left[\left(\frac{5}{12} \times 145\right)1.1 + 10 psf\right] \times TA + \left(22 + \frac{20'}{2}\right) 2\left(48 psf + \frac{40'}{2}\right) = 33 kips$

$LL = [50 psf] \times TA = 20 kips.$

$P_u = 1.2D + 1.6L + .5S$

$P_u = 1.2\left[36.6 + 2(33)\right] + 1.6\left[8 + 2(20)\right] + .5(10)$

$P_u = 205$ kips

table 4-1 @ $KL = 12$ ft $\boxed{W10\times33}$ $\phi P_n = 292$ kips

Column C: $TA = 15' \times 20' = 300\,ft^2$

Roof

$DL = \left[\left(\frac{5}{12}\times145\right)1.1 + 20\right]\times TA + \left(22\times\frac{20'}{2}\right) + \left(12\times\frac{10}{2}\right) + \left(55\,psf\times\frac{40'}{2}\right) = 27.3$ kips

$LL = [20\,psf]\times TA = 6$ kips

$S = [25\,psf]\times TA = 7.5$ kips

Floor level:

$DL = \left[\left(\frac{5}{12}\times145\right)1.1 + 10\right]\times TA + \left(22\times\frac{20}{2}\right) + \left(12\times\frac{10}{2}\right) + \left(68\,psf\times\frac{40'}{2}\right) = 24.6$ kips

$LL = [50\,psf]\times(10'\times20') + [80\,psf]\times(5'\times20') = 18$ kips

$P_u = 1.2D + 1.6L + .5S$

$P_u = 1.2\left[27.3 + 2(24.6)\right] + 1.6\left[6 + 2(18)\right] + .5(7.5)$

$P_u = 162.75$ kips

table 4-1 @ $KL = 12$ ft $\boxed{W10\times33}$ $\phi P_n = 292$ kips.

Column D: $TA = 15' \times 40' = 600\,ft^2$

Roof

$DL = \left[\left(\frac{5}{12}\times145\right)1.1 + 20\right]\times TA + \left(22\times\frac{20}{2}\right) + \left(12\times\frac{10}{2}\right) + \left[2\left(55\times\frac{40}{2}\right)\right] = 54.4$ kips

$LL = [20\,psf]\times TA = 12$ kips

$S = [25\,psf]\times TA = 15$ kips

Floor level:

$DL = \left[\left(\frac{5}{12}\times145\right)1.1 + 10\right]\times TA + \left(22\times\frac{20}{2}\right) + \left(12\times\frac{10}{2}\right) + 2\left(68\times\frac{40}{2}\right) = 49$ kips

$LL = [50]\times(10\times40) + [80\,psf]\times(5\times40) = 36$ kips

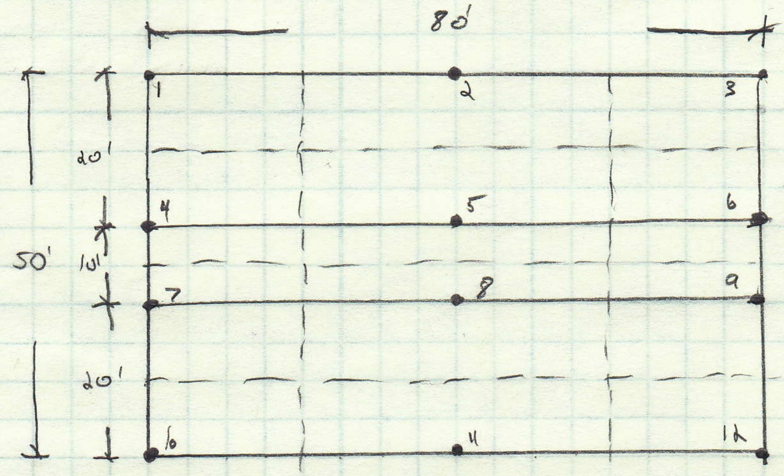
$P_u = 1.2D + 1.6L + .5S$

$P_u = 1.2\left[54.4 + 2(49)\right] + 1.6\left[12 + 2(36)\right] + .5(15)$

$P_u = 325$ kips.

table 4-1 @ $kL = 12$ ft $\boxed{W10\times39}$ $\phi P_n = 352$ kips

Foundations:



Groups	columns	column size	P
A	1, 3, 10, 12	W10x33	102.7k
B	2, 11	W10x33	205 k
C	4, 7, 6, 9	W10x33	162.7k
D	5, 8	W10x39	325 K

Bottom Floor loads: TA = 200 ft² W10x33

A:)

$DL = \left[(5/12 \times 145)1.1\right] \times TA + 33\text{lbs}(36') = 14.5$ kips

$LL = \left[150 \text{ psf}\right] \times TA = 30$ kips

$P_u = 1.2D + 1.6L = 65.4\text{ kips} + 102.7\text{ k} = \boxed{168\text{ K}}$

B:) $TA = 400\text{ ft}^2$

DL = (same above) = 28 K

$LL = \left[150\text{ psf}\right] \times TA = 60$ kips

$P_u = 1.2D + 1.6L = 129.6^k + 205 = \boxed{334.6^K}$

C.) $TA = 300\text{ ft}^2$

DL = (same diff TA) = 21.2 K

$LL = \left[150\text{ psf}\right] \times TA = 45$ K

$P_u = 1.2D + 1.6L = 97.5^k + 162.7^k = \boxed{260.2^K}$

D:) $TA = 600$ ft

$DL = \left[(5/12 \times 145)\,1.1\right] \times TA + 39\ \text{lbs}(36') = 41.2\,K$

$LL = [150\ psf] \times TA = 90\,K$

$P_u = 1.2D + 1.6L = 193.4^k + 325^k = \boxed{518.5^k}$

Data for bearing capacity:

$D_w = 10'$
$D = 5'$
$\gamma' = 115\ psf \rightarrow$ Fine sand
$\phi = 0°$

Steel Girder-to-Column connection: 2nd + 3rd floor

Girder end reaction:

Girder: W18x35

Column: W10 x 30

$W_U = [1.2D + 1.6L] \times TA$

$D_L = 23\ psf + 145\ pcf\left(\frac{3''}{12''/1}\right) = 59.25\ psf$

$W_U = 1.2(59.25psf \times 10' + 35\ ^{lb}/ft) + 1.6(50psf \times 10')$

$W_U = 1.55\ ^{k}/ft$

$V_U = \frac{W_U L}{2} = \frac{(1.55\ ^{k}/ft)(40')}{2} = 31k = V_U$

number bolts:

3/4" ϕ bolts, A325-N in standard bolts: table J3.2 $F_v = 54ksi$

$\phi R_n = \phi F_v A_b = (.75)(54ksi)\left(\frac{\pi}{4}\left(\frac{3}{4}\right)^2\right) = \phi R_n = 17.9\ ^{k}/bolt$

$n = \frac{V_U}{\phi R_n} = \frac{31k}{17.9k} = 1.73\ bolts = \underline{2\ bolts}$

Girder dimensions

$t_f = 0.425''$

$t_w = 0.300''$

$A = 10.3\ in^2$

$d = 17.7''$

$T = 15\frac{1}{2}''$

$b_f = 6''$

Column dimensions

$t_f = 0.510''$

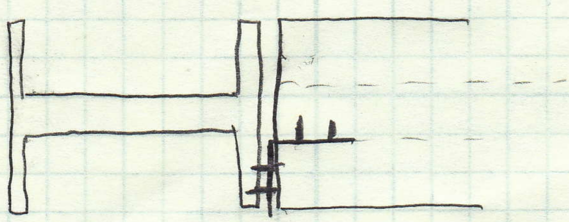
$t_w = 0.300''$

$A = 8.84\ in^2$

$d = 10.5in$

$T = 8\frac{1}{4}''$

$b_f = 5.81''$



Girder t_w < Column t_f

bearing on Girder web:

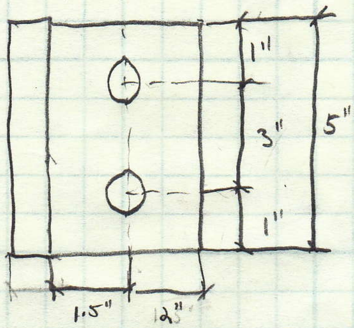
$t_w = 0.300''$

$\phi R_n = \phi\, 2.4\, d_b\, t\, F_u$

$\phi R_n = 2(.75)(2.4)(0.300'')(3/4'')(65ksi)$

$\phi R_n = 52.65k > 31^k$ ✓

Angle geometry:



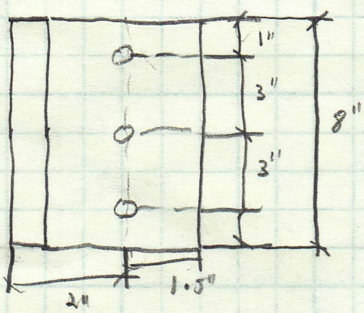
Girder

$T/2 \le$ angle length $\le T \rightarrow T = 15\frac{1}{2}$"

$7\frac{3}{4}" < 5" \le 15\frac{1}{2}"$ ✗

TOO short

add a bolt make it 8"



$7\frac{3}{4}" < 8" < 15\frac{1}{2}"$ ✓

$\phi R_n = \phi\, 2.4\, d_b\, t F_u = 3(.75)(2.4)(.300')\left(\frac{3}{4}\right)(65)$

$\phi R_n = 79k > 31k$ ✓

1.5" gap + half hole $= \frac{3}{4}/2 = \frac{3}{8} + \frac{1}{16} + \frac{1}{16} = \frac{1}{2} + 1.5 = 2"$

$L\,3.5" \times 3.5" \times 8" \times t$

Check half flange of column and width angle

$3.5" < \frac{b_f}{2} = \frac{5.81"}{2} \rightarrow 3.5" > 2.9"$ ✗

need bigger column

W10x33

$\frac{b_f}{2} = \frac{7.96}{2} = 3.98" > 3.5"$ ✓

Angle steel: A36

$F_y = 36ksi$, $F_u = 58ksi$

1) Bolt tear-out

$\phi R_n = \phi 1.2 L_c t F_u \le \phi 2.4 d_b t F_u$

-Bottom two by bearing

$\phi\, 2.4\, d_b\, t F_u = (.75)(2.4)\left(\frac{3"}{4}\right) t\, (58ksi) = 78.3\, t$

-Top bolt tearout

$\phi\, 1.2 L_c t F_u = (.75)(1.2)\left(1 - \frac{1}{2}\left(\frac{3}{4} + \frac{2}{16}\right)\right) t\, (58ksi) = 29.4t$

$2(78.3t) + 29.4t \ge 31k$

$186t \ge 31k$

$t = .166"$

2.) Angle shear rupture

$L - nd_e = 8'' - 3\left(\frac{3}{4} + \frac{1}{8}\right)$
$= 5.375''$

$\phi R_n = \phi(.6F_u)(L - nd_e)\,t \geq 31^k$

$\phi R_n = (.75)(.6 \times 58)(5.375)\,t = 140.3t \geq 31^k$

$\underline{t \geq .221''}$

3.) Angle shear yield

$\phi R_n = \phi(.6F_y)\,Lt \geq 31^k$

$(1.0)(.6 \times 36)(8t) = 172.8t \geq 31^k$

$\underline{t \geq .18''}$

Largest $t = .221''$

$\boxed{\text{Angle } \angle 3\tfrac{1}{2} \times 3\tfrac{1}{2} \times \tfrac{1}{4}''}$

<u>Beam to column connection - Floors:</u>

$W_u = [1.2D + 1.6L] \times TA$

Beam: W10x15
Column: W10x33

$D_L = 23\,psf + 145\,pcf\left(\frac{3''}{12}\right) = 59.25\,psf$

$W_u = 1.2(59.25\,psf \times 2.5' + 15\,^{lb}/ft) + 1.6(50 \times 2.5')$

$W_u = .395\,^k/ft$

$V_u = \frac{w_u L}{2} = \frac{(.395\,^k/ft)(20')}{2} = 3.95^k = V_u$

$n = \frac{V_u}{\phi R_n} = \frac{3.95^k}{17.9\,^k/bolt} = .22\,\text{bolts} \rightarrow \underline{1\,\text{bolt}}$

<u>beam dimensions</u>

$t_f = 0.27''$
$t_w = .230''$
$A = 4.44\,in^2$
$d = 9.99''$
$T = 8\,3/8''$
$b_f = 4''$

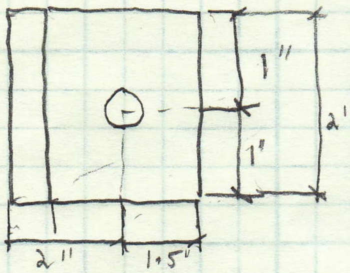
beam t_w < column t_w

$\phi R_n = \phi\, 2.4\, d_b\, t\, F_u$

$\phi R_n = (.75)(2.4)(.230)(3/4'')(65\,ksi)$

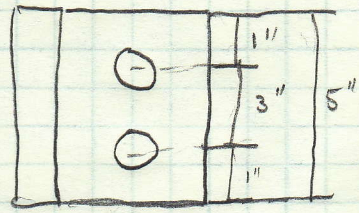
$\underline{\phi R_n = 20.2^k > 3.95^k}$ ✓

Angle geometry:



$T/2 \le$ angle length $\le T \rightarrow$ $8\frac{3}{8}" = T$

$4\frac{3}{16} \le 2" \le 8\frac{3}{8}$ × add bolt



✓

$\angle 3.5" \times 3.5" \times 5" \times t$

1.) Bolt tear-out

$$\phi R_n = \phi 1.2 L_c t F_u \le \phi 2.4 d_b t F_u$$

-top by bearing

$$\phi 2.4 d_b t F_u = (.75)(2.4)(3/4")t(58\,ksi) = 78.3\,t$$

-top tearout

$$\phi 1.2 L_c t F_u = (.75)(1.2)\left(1-\tfrac{1}{2}\left(\tfrac{3}{4}+\tfrac{2}{16}\right)\right)t(58\,ksi) = 29.4t$$

$$(78.3t) + 29.4t \ge 3.95\,k$$

$$107.7t \ge 3.95^k$$

$$\underline{t = .04"}$$

2.) Angle shear rupture

$$\phi R_n = \phi(.6F_u)(L - nd_e)t \ge 3.95\,k$$

$$\phi R_n = (.75)(.6 \times 58)(2.375")t \ge 3.95k$$

$$62\,k\,t \ge 3.95\,k$$

$$\underline{t = .06"}$$

$5" - 3\left(\tfrac{3}{4}+\tfrac{1}{8}\right)$

$(L - nd_e) = 2.375"$

3.) Angle shear yield

$$\phi R_n = \phi(.6F_y)Lt \ge 3.95\,k$$

$$(1.0)(.6 \times 36)(5t) = 108t \ge 3.95^k$$

$$\underline{t = .04"}$$

Angle $\angle 3\frac{1}{2} \times 3\frac{1}{2} \times 5" \times \frac{1}{4}"$

Steel Girder-to-Column connections: Roof

Girder end reaction

W18x35 → girders

W10x33 → columns

$W_u = [1.2D + 1.6L + .5S]$

$W_u = 1.2\left[\left(23\,psf + 145\left(\frac{3}{12}\right) \times 10'\right) + 35\,lb/ft\right] + 1.6(25 \times 10') + 0.5(23 \times 10')$

$W_u = 1.14\ k/ft$

$V_u = \frac{W_u L}{2} = \frac{(1.14\,k/ft)(40')}{2} = V_u = 22.85\ K$

Number bolts:

$n = \frac{V_u}{\phi R_n} = \frac{22.85\,K}{17.9\,k/bolt} = 1.27\ bolts \rightarrow 2\ bolts$

Girder t_w < column t_f

Column dimensions

$t_f = .435''$
$t_w = .290''$
$A = 9.71\,in^2$
$d = 9.73\,in$
$T = 7\tfrac{1}{2}''$
$b_f = 7.96$

Girder T_w < column t_f

$t_w = .3''$

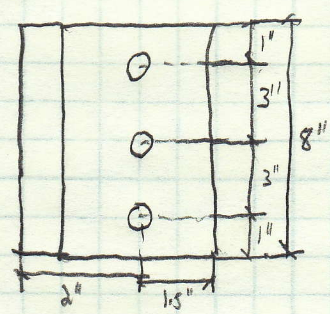
$\phi R_n = \phi\, 2.4\, d_b\, t\, F_u$

$\phi R_n = 2(.75)(2.4)(.300'')(\tfrac{3}{4}'')(65\,ksi)$

$\phi R_n = 52.65\ K > 22.85\ k$ ✓

$T/2 \leq$ angle length $\leq T \rightarrow 15\tfrac{1}{2}''$

$7\tfrac{3}{4} \leq 8'' \leq 15\tfrac{1}{2}''$



$\angle\ 3.5'' \times 3.5'' \times 8'' \times t$

1.) Bolt Tear-Out

$$\phi R_n = \phi 1.2 L_c t F_u \leq \phi 2.4 d_b t F_u$$

- Bottom two by bearing

$$\phi 2.4 d_b t F_u = (.75)(2.4)(3/4'')t(58ksi) = 78.3t$$

- Top bot tearout

$$\phi 1.2 L_c t F_u = (.75)(1.2)(1 - \tfrac{1}{2}(3/4 + 1/16))t(58ksi) = 29.4t$$

$$2(78.3t) + 29.4t \geq 22.85^k$$

$$\underline{t = .123''}$$

2.) Angle shear rupture

$$\phi R_n = \phi(.6F_u)(L - nd_e)t \geq 22.85^k \qquad (L - nd_e) = 8'' - 3(3/4 + \tfrac{1}{8}) = 5.375$$

$$\phi R_n = (.75)(.6 \times 58)(5.375'')t \geq 22.85^k$$

$$\underline{t = .163''}$$

3.) Angle shear yield

$$\phi R_n = \phi(.6F_y)L\,t \geq 22.85^k$$

$$(1.0)(.6 \times 36)(8t) \geq 22.85$$

$$\underline{t = .132''}$$

$$\boxed{\text{Angle } \angle\, 3\tfrac{1}{2}'' \times 3\tfrac{1}{2}'' \times \tfrac{1}{4}''}$$

Beam to column connections - roof:

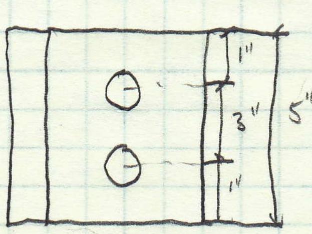
$$W_u = [1.2D + 1.6L + .5S] \times TA$$

$$W_u = 1.2\left[\left(23 + 145\left(\tfrac{3}{12}\right) \times 2.5\right) + 15\right) + 1.6(25 \times 2.5) + .5(23 \times 2.5)$$

$$W_u = 0.325^k/ft$$

$$V_u = \frac{W_u L}{2} = \frac{(0.325^k/ft)(20')}{2} = 3.25^k = V_u$$

$$n = \frac{V_u}{\phi R_n} = \frac{3.25^k}{17.9^k/bolt} = 0.18 \text{ bolts} \rightarrow \underline{1 \text{ bolt}} \rightarrow \text{use } \underline{\text{two bolts}} \text{ to be right size}$$



$$\angle\, 3.5'' \times 3.5'' \times 5'' \times t$$

1.) Bolt tearout

$$\phi R_n = \phi 1.2 L_c t F_u \leq \phi 2.4 d_b t F_u$$

- TOP bearing

$$\phi 2.4 d_b t F_u = (.75)(2.4)(3/4) t (58 ksi) = 78.3 t$$

- bottom bearing

$$\phi 1.2 L_c t F_u = (.75)(1.2)(1 - \tfrac{1}{2}(3/4 + 1/8)) t (58 ksi) = 29.4 t$$

$$78.3 t + 29.4 t \geq 3.25^k$$

$$\underline{t = .03''}$$

2.) Angle shear rupture

$$\phi R_n = \phi(.6 F_u)(L - n d_e) t \geq 3.25^k$$

$$\phi R_n = (.75)(.6 \times 58)(2.375) t \geq 3.25^k$$

$$\underline{t = .052''}$$

3.) Angle shear yield

$$\phi R_n = \phi(.6 F_y) L t \geq 3.25^k$$

$$(1.0)(.6 \times 36)(5t) \geq 3.25$$

$$\underline{t = .03''}$$

$$\boxed{\text{Angle } \angle\ 3\tfrac{1}{2}'' \times 3\tfrac{1}{2}'' \times 5'' \times 1/4''}$$

$P_{nt} = 11.1\ K \rightarrow$ axial from gravity

$P_{LT} = 5.5K \rightarrow$ axial from lateral

$M_{nt} = 5.6\ k \cdot ft \rightarrow$ moment from gravity

$M_{LT} = 57.6\ k\,ft \rightarrow$ moment from lateral

Reverse curvature

$\Sigma H = 11.02k + 22.04k + 22.04k = \underline{55.1\ K}$

lateral deflection:

$.807'' + .8'' + .8'' = 2.407''$

lower moment $= 12.3\ k \cdot ft$

$P_{story} = 4(.395\ ^{k}/_{ft} \times 20') + 2(.52\ ^{k}/_{ft} \times 10') + (.325\ ^{k}/_{ft} \times 50') = 58.25^{k}$

$P_{mf} = P_{story}$

$$P_{estory} = R_M \frac{\Sigma HL}{\Delta H} = (.85)\frac{55.1k \times (12 \times 12)}{2.407''} = \underline{2801.9\ K}$$

$R_M = 1 - .15(58.25/58.25) = .85$

$$B_2 = \frac{1}{1 - \frac{\alpha P_{story}}{P_{estory}}} = \frac{1}{1-(1)\frac{58.25}{2801.9}} = \underline{1.02 = B_2}$$

$$C_m = 0.6 - 0.4\left(\frac{M_1}{M_2}\right) = 0.6 - 0.4\left(\frac{12.3k \cdot ft}{57}\right) = .514 = C_m$$

$P_r = P_{nt} + B_2 P_{LT} = 11.1k + 1.02(5.5) = 16.71K$

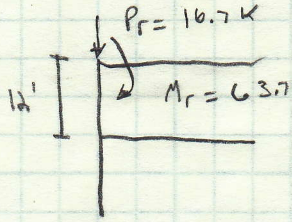
$$P_{e1} = \frac{\pi^2 EI}{(K_1 L)^2} = \frac{(\pi^2)(29000)(171\ in^4)}{[(1.0) \times (12 \times 12)]^2} = 2360.3\ K$$

$$B_1 = \frac{C_m}{1 - \alpha P_r/P_{e1}} = \frac{.514}{1-(1)\left(\frac{16.71}{2360.3}\right)} = .517 \leq 1.0 \rightarrow 1.0 = B_1$$

$P_r = P_{nt} + B_2 P_{LT} = 11.1k + 1.02(5.5k) = \boxed{16.71K = P_r}$

$M_r = B_1 M_{nt} + B_2 M_{LT} = 1.0(5.6) + 1.02(57) = \boxed{63.74\ k \cdot ft = M_r}$

Story stiffness | pg 2/2 | Justin Furst 2/17/13



$K_x = 1.25$

$K_y = 1.0$

$$\frac{K_x L}{r_x} = \frac{1.25(12' \times 12''/')}{4.19''} = 42.96''$$

$$\frac{K_y L}{r_y} = \frac{1.0(12' \times 12''/')}{1.94''} = \underline{74.23''} \rightarrow \text{governs}$$

table 4-1

$$KL = 1.0(12') = 12'$$

$$\Longrightarrow P_c = \phi P_n = 292\,K$$

$$\text{ratio} = \frac{P_r}{P_c} = \frac{16.7\,K}{292\,K} = \underline{.06} < 0.2 \rightarrow \quad \text{use} \quad \frac{P_r}{P_c} + \left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \le 1.0$$

$M_{rx} = 63.74\,k\cdot ft$

$M_{cx} = ?$

$\underline{L_b = 12'}$, $L_p = 1.76 r_y \sqrt{E/F_y} = \underline{6.85\,ft = L_p}$

$$L_r = 1.95\, r_{ts} \frac{E}{0.7F_y} \sqrt{\frac{J_c}{S_x h_o} + \sqrt{\left(\frac{J_c}{S_x(h_o)}\right)^2 + 6.76\left(\frac{0.7F_y}{E}\right)^2}}$$

$c = 1$
$r_{ts} = 2.2$
$J_c = .583\,in^4$
$S_x = 35\,in^3$
$h_o = 9.3\,in$
$Z_x = 38.8$

$$L_r = 1.95(2.2)\left(\frac{29000}{0.7(50)}\right)\sqrt{\left(\frac{.583}{35(9.3)}\right) + \sqrt{\left(\frac{.583}{35(9.3)}\right)^2 + 6.76\left(\frac{0.7(50)}{29000}\right)^2}}$$

$$L_r = 3554.57\sqrt{(.0018) + \sqrt{3.2\times10^{-6} + 9.85\times10^{-6}}}$$

$\underline{L_r = 21.8\ ft}$

$$M_p = Z_x F_y = 38.8(50)\left(\frac{1}{12}\right) = 162\,k\cdot ft$$

$$M_n = M_p - (M_p - 0.7 S_x F_y)\left(\frac{L_b - L_p}{L_r - L_p}\right)$$

$$M_n = 162\,k\cdot ft - \left(162 - \frac{[.7(35)(50)]}{12}\right)\left(\frac{12 - 6.85}{21.8 - 6.85}\right) = M_n = 141.36\,k\cdot ft$$

$$M_{cx} = \phi M_n = 0.9(141.36) = \underline{127.2\,k\cdot ft}$$

$$\frac{16.7}{292} + \left(\frac{63.74\,k\cdot ft}{127.2\,k\cdot ft} + 0\right) = \boxed{.56 \le 1.0} \checkmark \ \underline{OK}$$

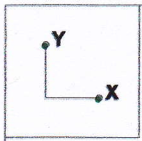
Deflections Concrete:

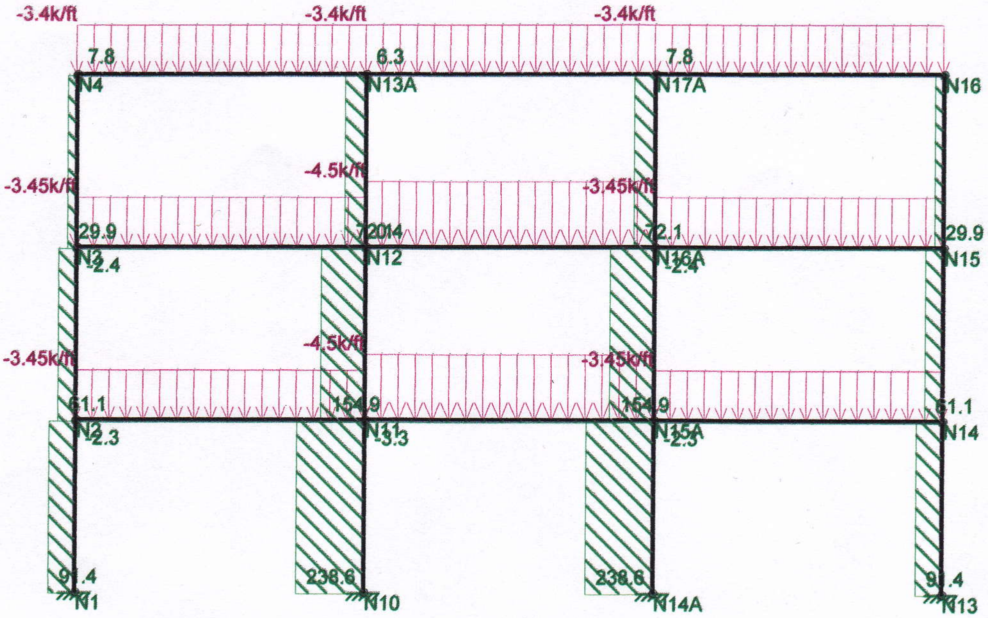
Gravity only:

Joints	X	Y	Rotation
N1	0	0	0
N2	0	0	-4.46E-05
N3	0	-0.001	-3.40E-05
N4	0	-0.001	-8.00E-05
N13	0	0	0
N14	0	0	4.46E-05
N15	0	-0.001	3.40E-05
N16	0	-0.001	8.00E-05
N10	0	0	0
N11	0	0	-7.86E-07
N12	0	-0.002	-4.99E-06
N13A	0	-0.002	1.14E-05
N14A	0	0	0
N15A	0	0	7.86E-07
N16A	0	-0.002	4.99E-06
N17A	0	-0.002	-1.14E-05

Lateral Only:

Joint	**X**	**Y**	**Rotation**
N1	0	0	0
N2	0.003	0	-7.99E-06
N3	0.005	0	-4.92E-06
N4	0.006	0	-2.27E-06
N13	0	0	0
N14	0.002	0	-7.83E-06
N15	0.005	0	-4.99E-06
N16	0.006	0	-2.36E-06
N10	0	0	0
N11	0.002	0	-1.13E-05
N12	0.005	0	-6.47E-06
N13A	0.006	0	-3.02E-06
N14A	0	0	0
N15A	0.002	0	-1.12E-05
N16A	0.005	0	-6.50E-06
N17A	0.006	0	-3.06E-06



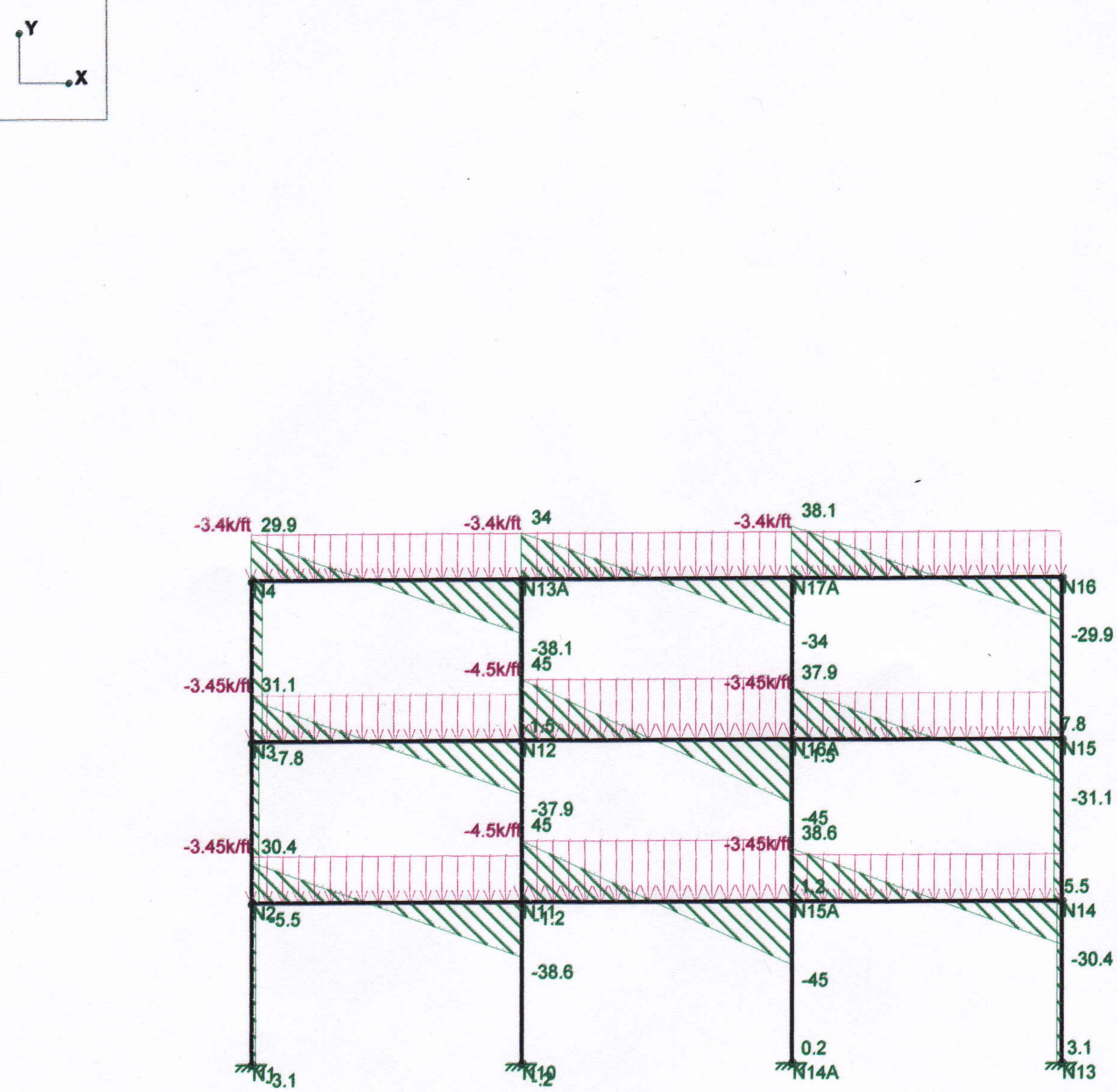


Member Axial Forces (k)

February 9, 2013

6:10 PM

Concrete Gravity Loads.r2e

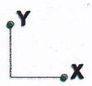


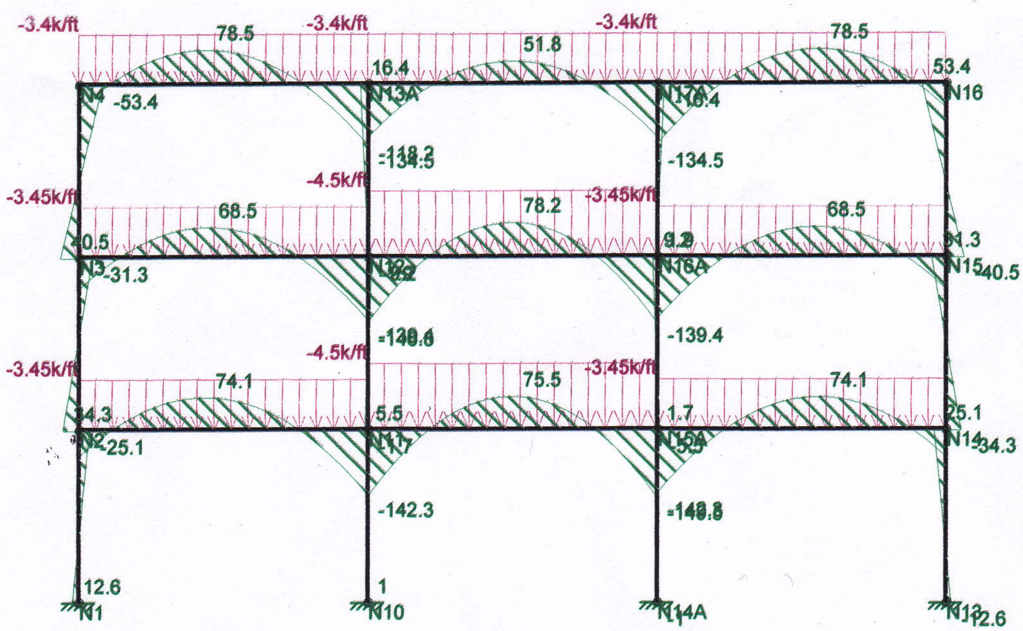
Member Shear Forces (k)

February 9, 2013

6:11 PM

Concrete Gravity Loads.r2e



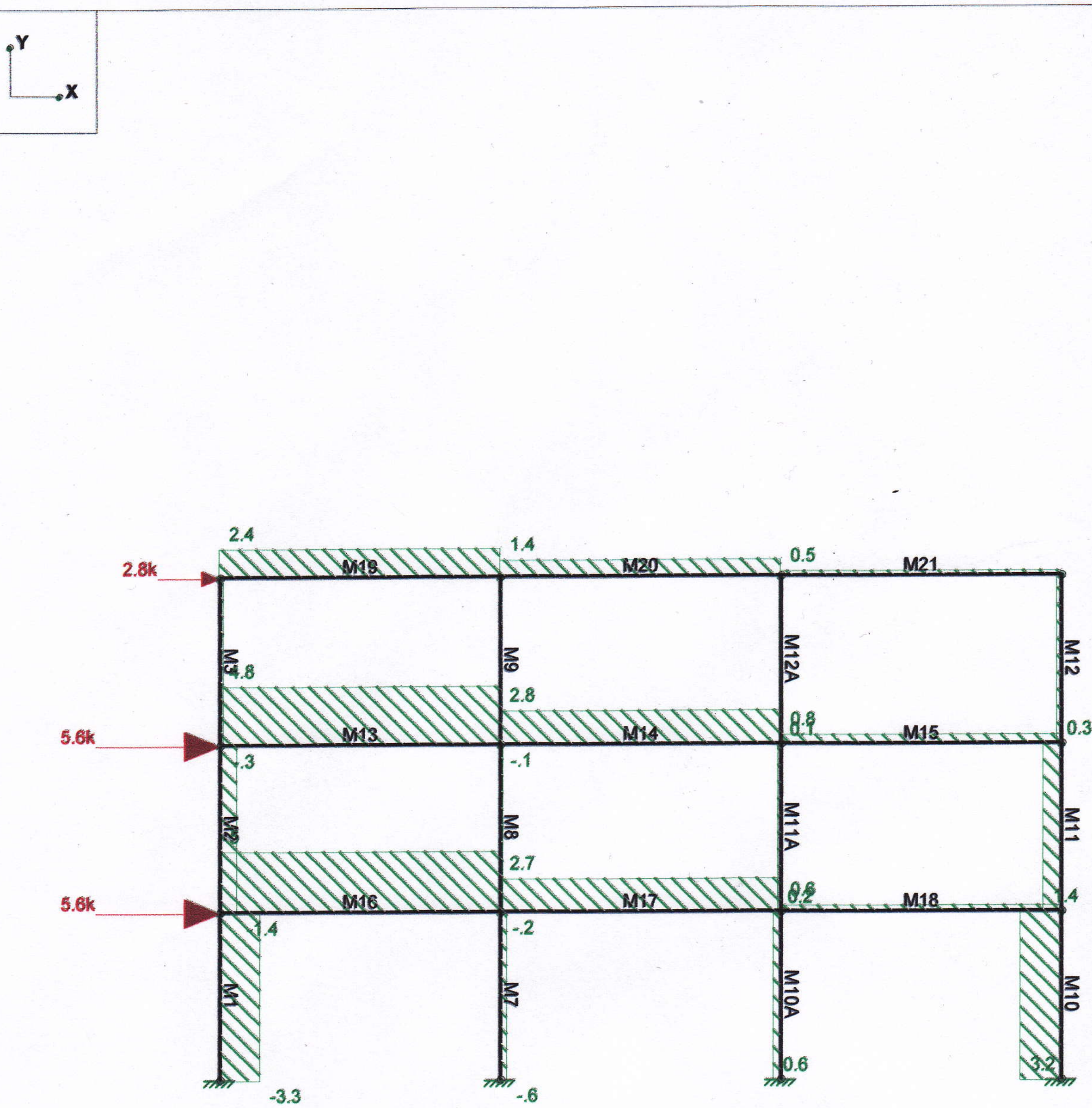


Member Bending Moments (k-ft)

February 9, 2013

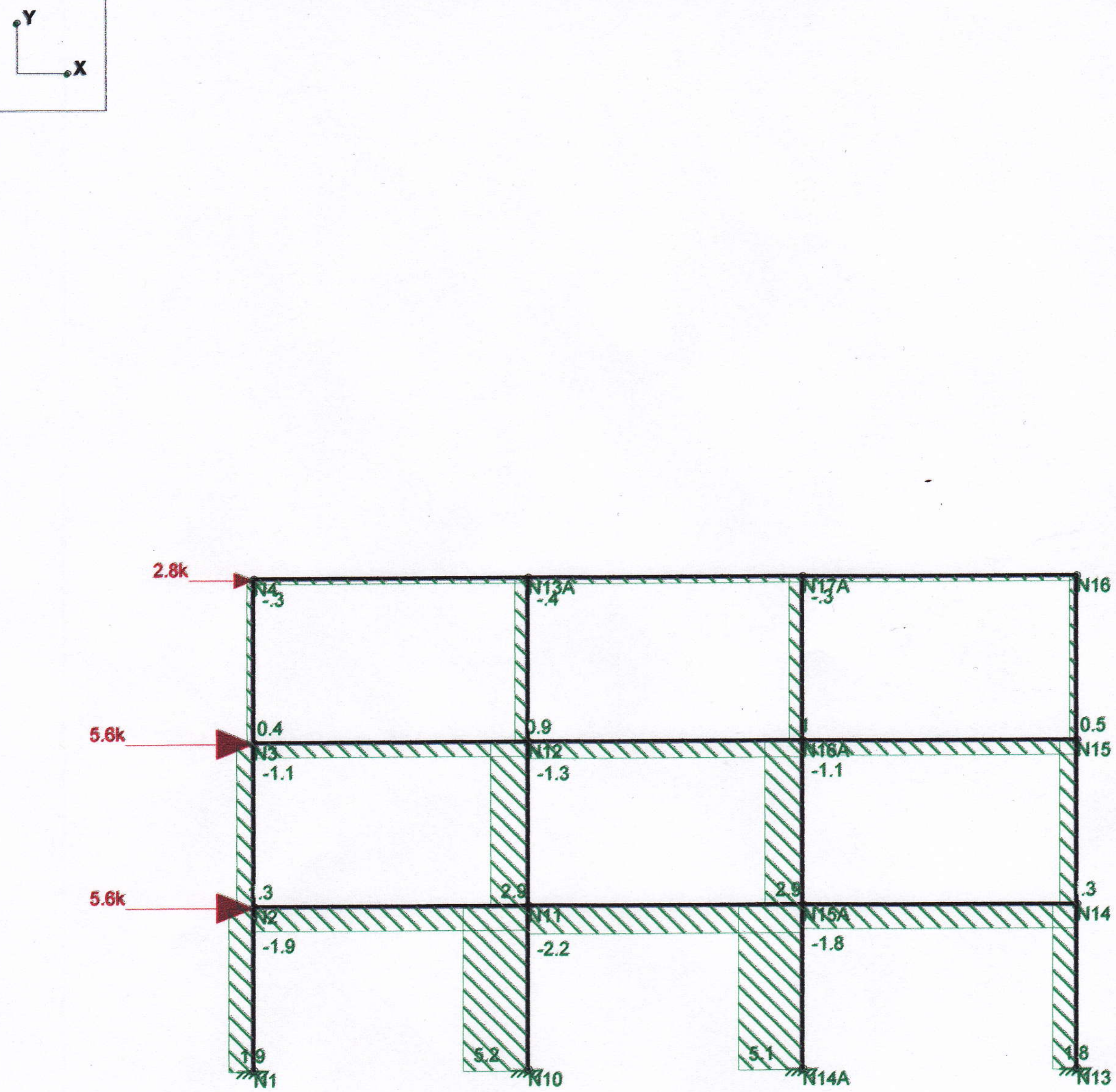
6:13 PM

Concrete Gravity Loads.r2e



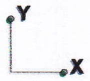
Member Axial Forces (k)

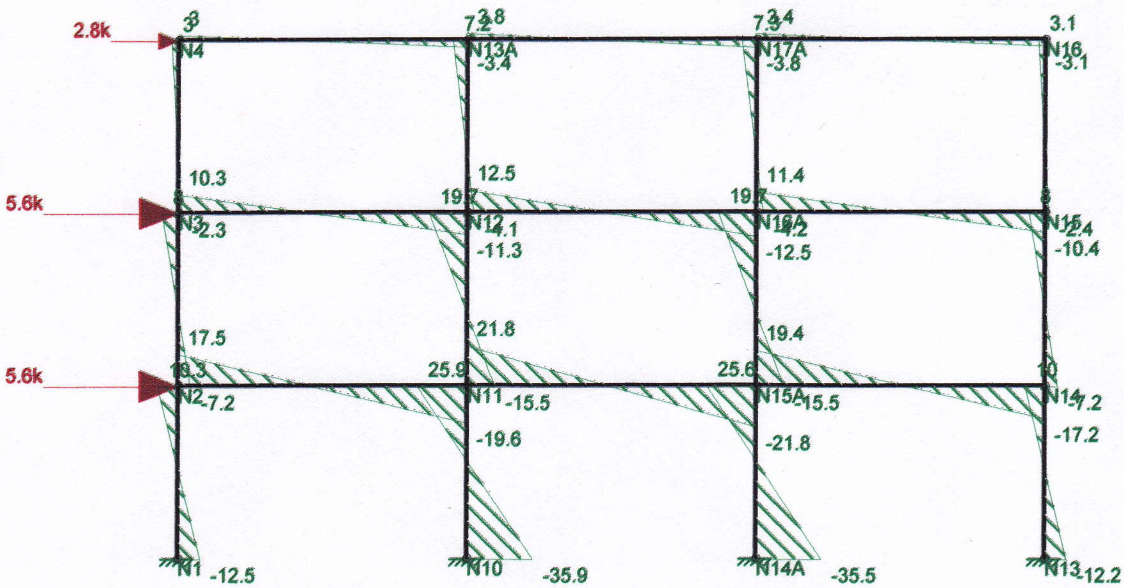
February 12, 2013	
7:42 PM	
Concrete RISA.r2e	



Member Shear Forces (k)

February 12, 2013	
8:04 PM	
Concrete RISA.r2e	





Member Bending Moments (k-ft)

	February 12, 2013
	8:05 PM
	Concrete RISA.r2e

February 17, 2013
8:44 PM
Concrete Gravity Loads.r2e

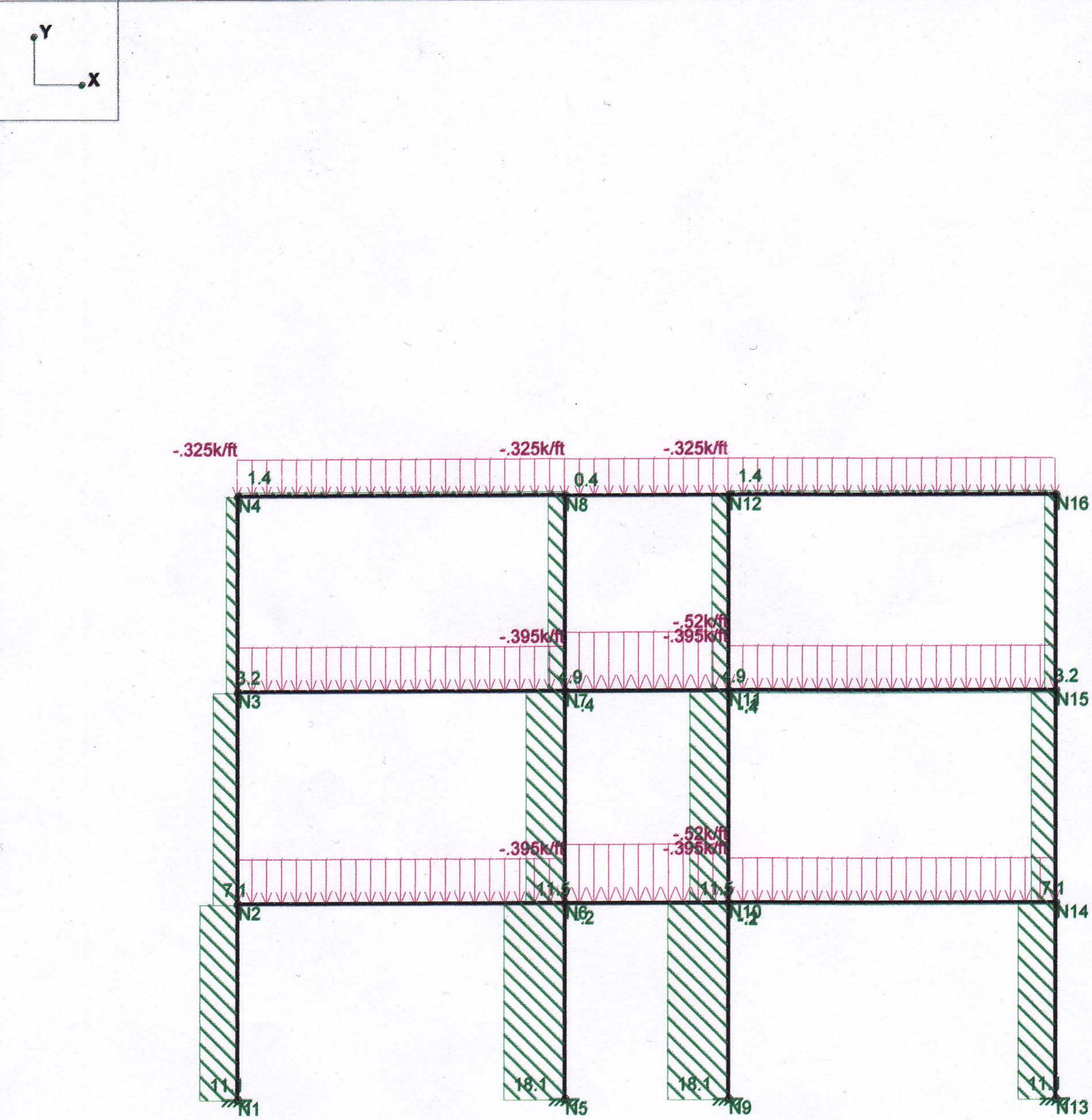
Deflections Steel:

Gravity Only:

Joints	X	Y	Rotation
N1	0	0	0
N2	0	-0.006	-4.85E-04
N3	0	-0.01	-2.88E-04
N4	0.003	-0.011	-7.31E-04
N5	0	0	0
N6	0	-0.009	3.16E-04
N7	0	-0.016	1.71E-04
N8	0	-0.018	4.87E-04
N9	0	0	0
N10	0	-0.009	-3.16E-04
N11	0	-0.016	-1.71E-04
N12	0	-0.018	-4.87E-04
N13	0	0	0
N14	0	-0.006	4.85E-04
N15	0	-0.01	2.88E-04
N16	-0.003	-0.011	7.31E-04

Lateral Only:

Joint	X	Y	Rotation
N1	0	0	0
N2	0.822	0.003	-7.20E-03
N3	1.947	0.004	-4.94E-03
N4	2.502	0.005	-2.48E-03
N5	0	0	0
N6	0.807	0.005	-5.71E-03
N7	1.931	0.008	-4.28E-03
N8	2.493	0.008	-1.93E-03
N9	0	0	0
N10	0.801	-0.005	-5.68E-03
N11	1.925	-0.008	-4.30E-03
N12	2.49	-0.008	-1.94E-03
N13	0	0	0
N14	0.796	-0.003	-7.05E-03
N15	1.92	-0.004	-5.01E-03
N16	2.489	-0.005	-2.56E-03

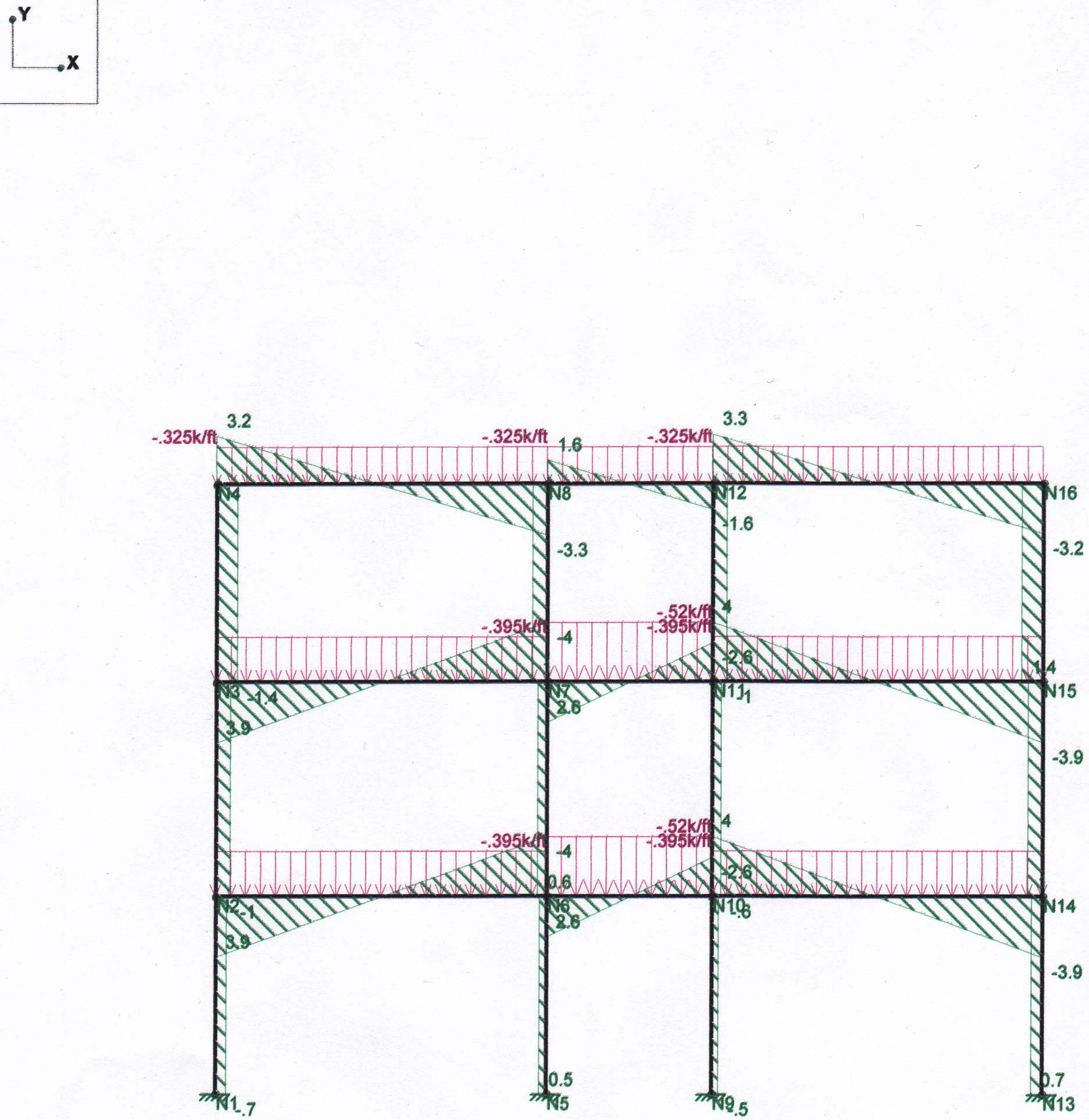


Member Axial Forces (k)

February 9, 2013

6:40 PM

Steel RISA.r2e

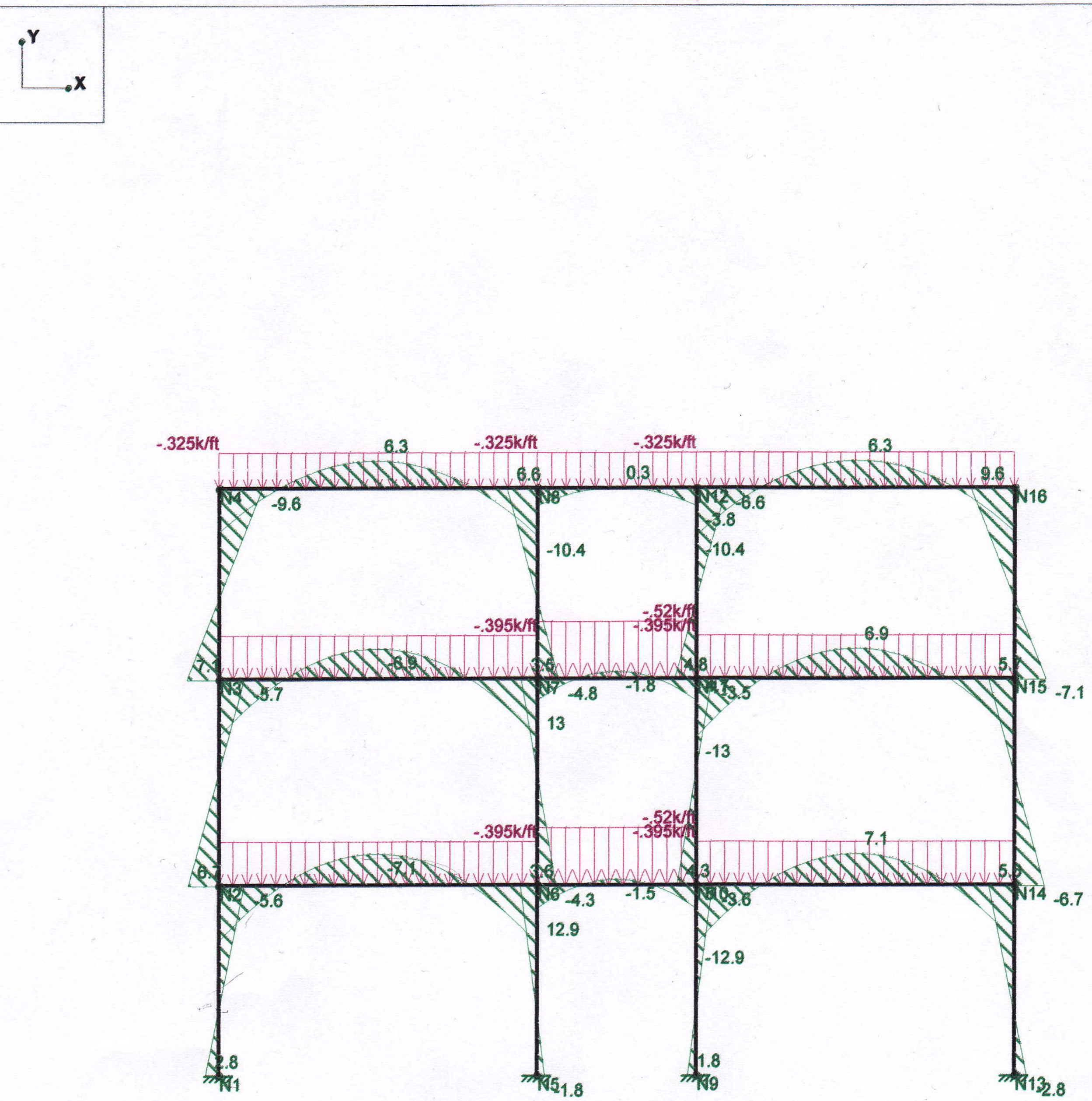


Member Shear Forces (k)

February 9, 2013

6:41 PM

Steel RISA.r2e

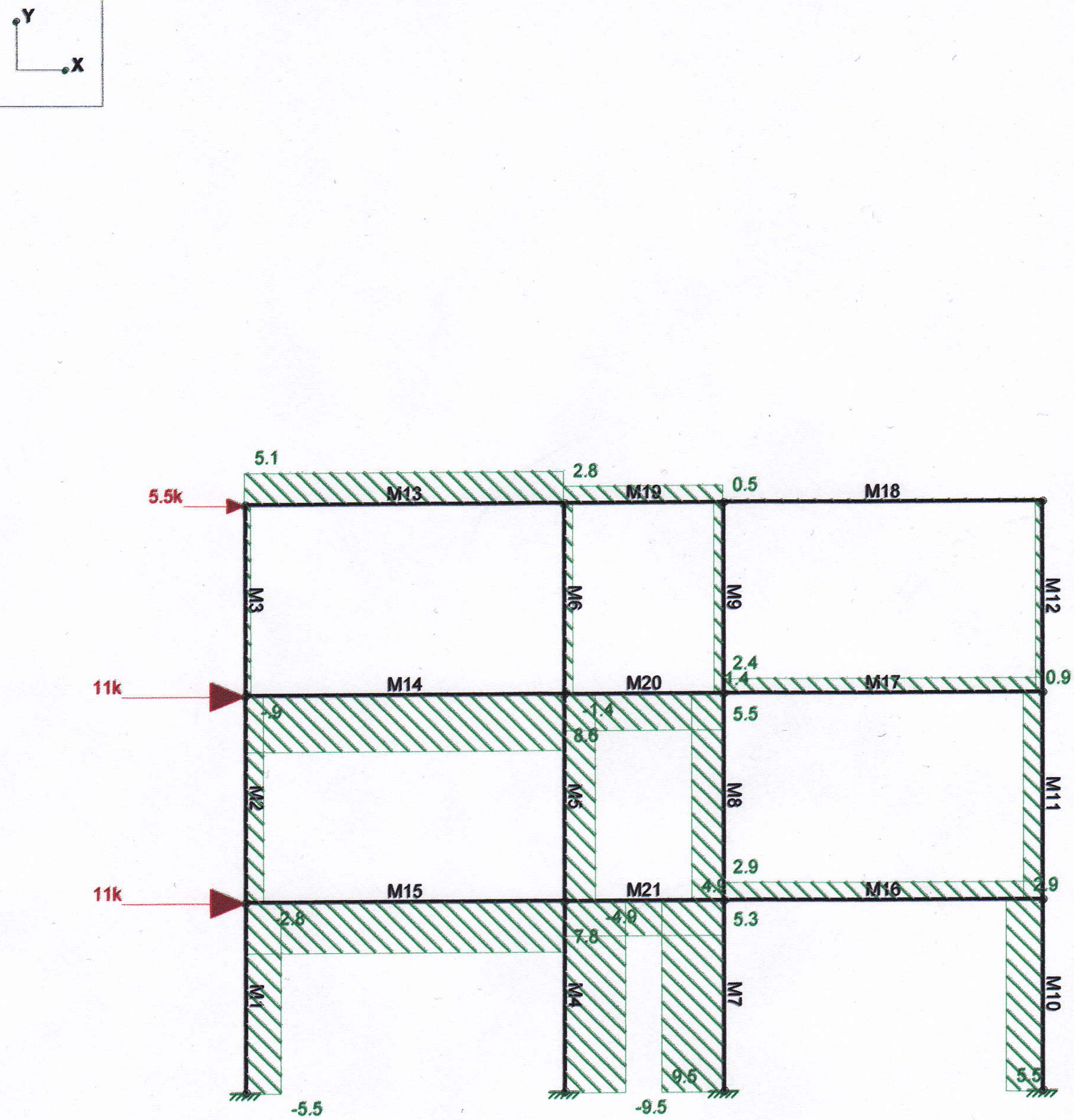


Member Bending Moments (k-ft)

February 9, 2013

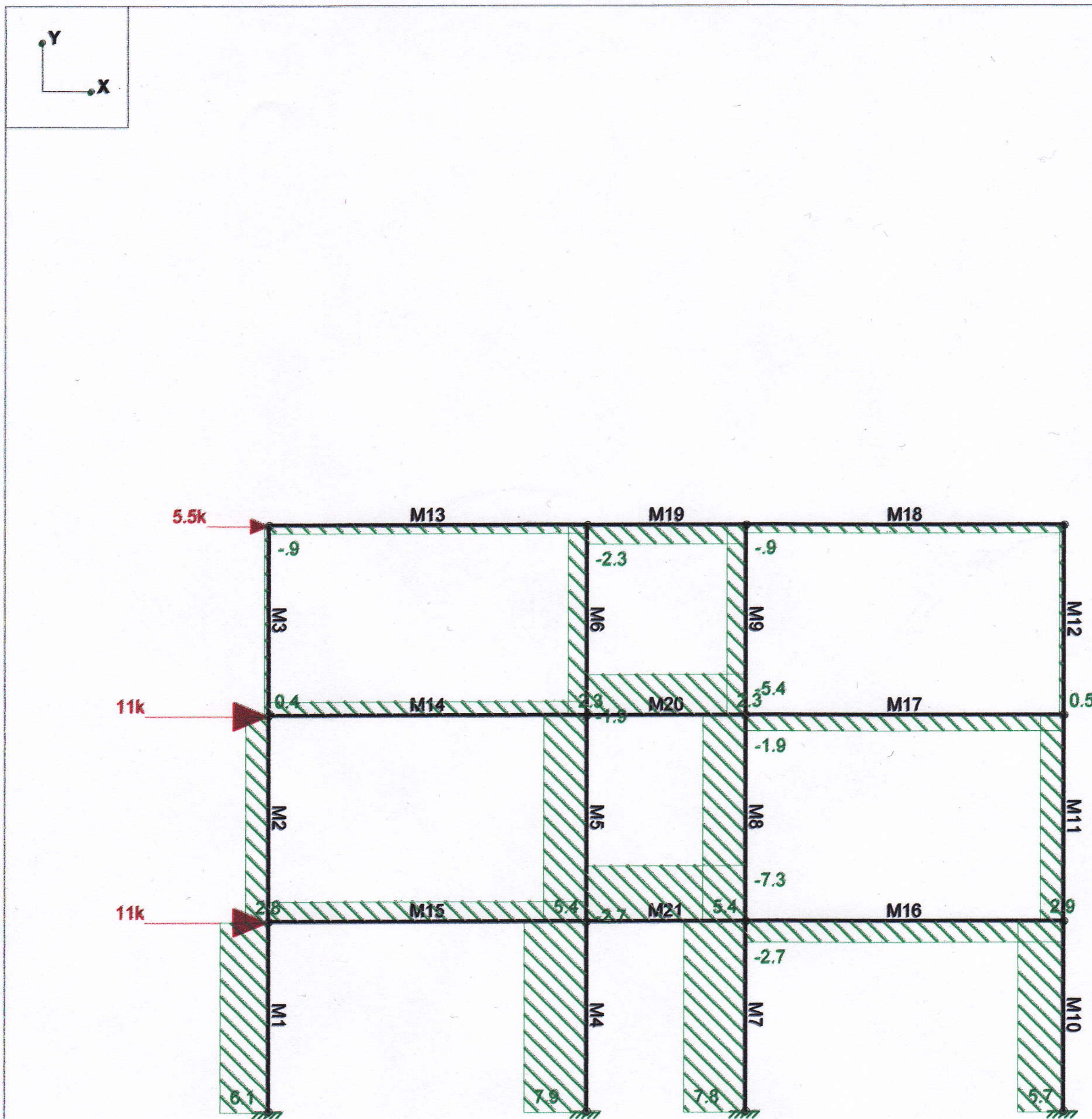
6:43 PM

Steel RISA.r2e



Member Axial Forces (k)

	February 12, 2013
	8:16 PM
	Steel RISA.r2e



Member Shear Forces (k)

	February 12, 2013
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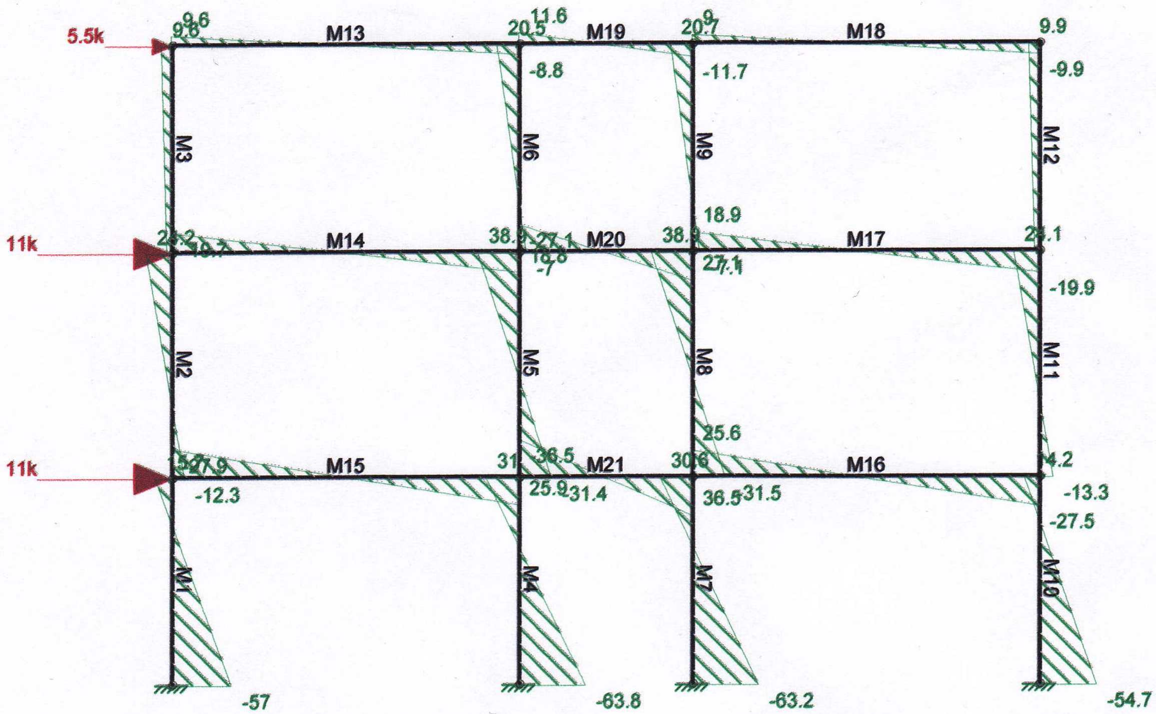
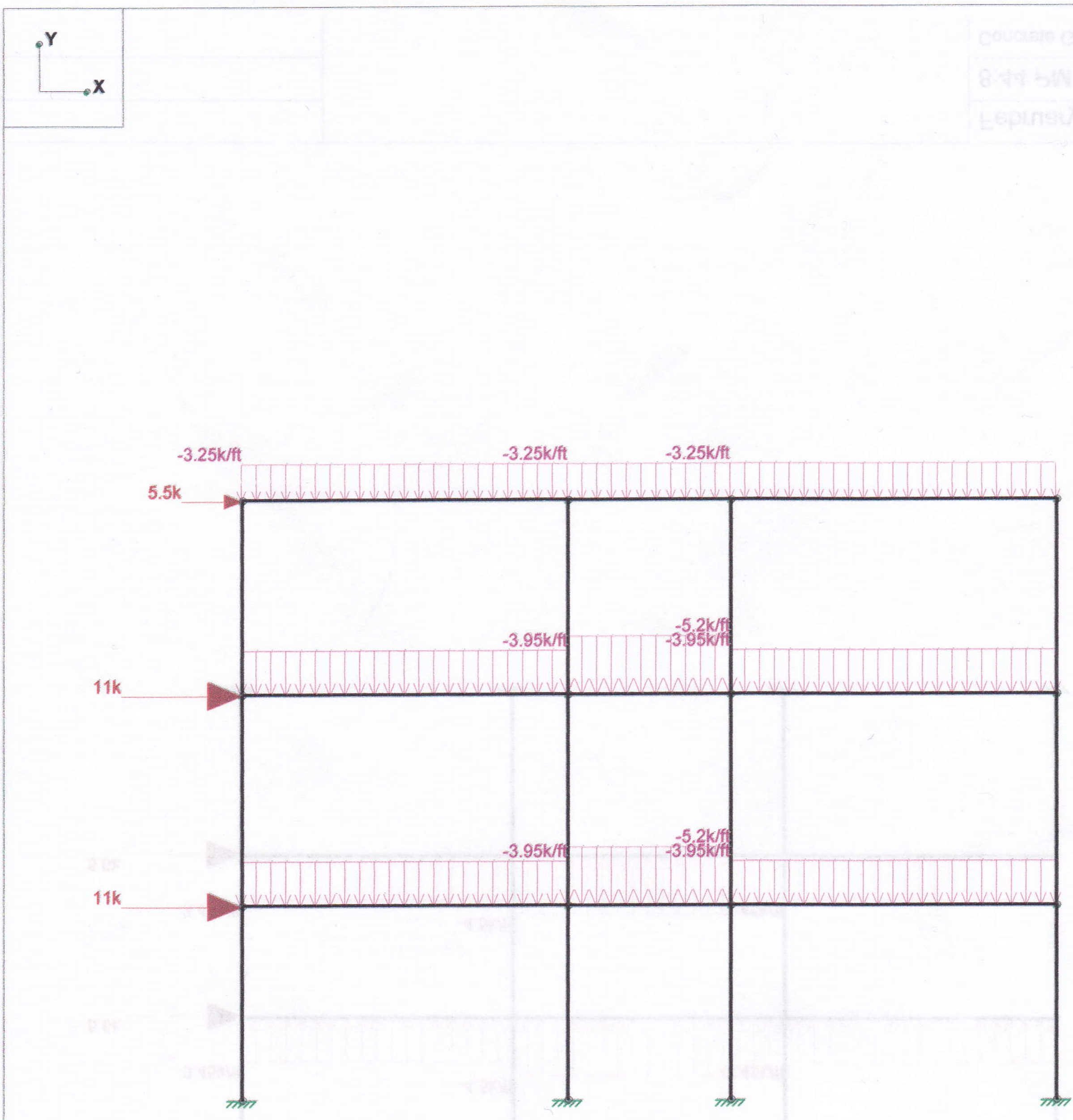


Member Bending Moments (k-ft)

February 12, 2013

8:18 PM

Steel RISA.r2e



February 17, 2013

8:47 PM

Steel RISA.r2e

CARR-DEE CORP.

37 LINDEN STREET P.O. BOX 67 MEDFORD, MA 02155-0001 Telephone (617) 391-4500

To: McPHAIL ASSOCIATES, INC. 30 NORFOLK ST., CAMBRIDGE, MA Date: ______ Job No.: 2005-129

Location: WORCESTER CENTER MALL, WORCESTER, MA Scale: 1 in. = 6 ft.

B0RING 10

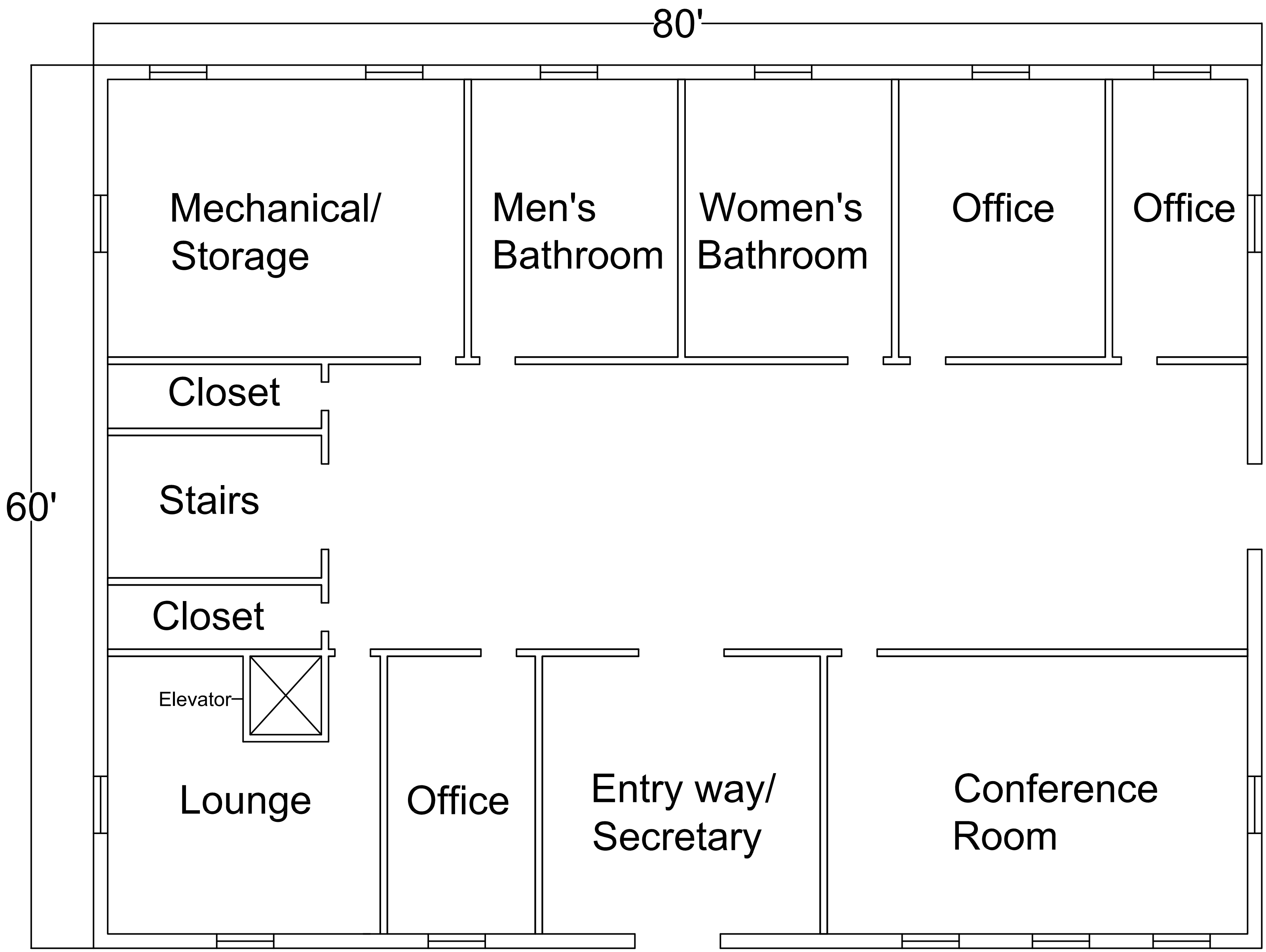
GROUND SURFACE Elev. +462.0

6"	CONCRETE	S#1, 6" to 2'6" (3-7-5-4) RECOVERED 15 in.
7'6"	FILL FINE MEDIUM SAND	S#2, 5' to 7' (2-2-1-3) RECOVERED 12 in.
14'	MEDIUM DENSE FINE SAND, TRACE INORGANIC SILT	S#3, 10' to 12' (7-10-11-13) RECOVERED 18 in.
18'	MEDIUM DENSE FINE SAND, TRACE INORGANIC SILT	S#4, 15' to 17' (3-4-9-12) RECOVERED 18 in.
22'	MEDIUM DENSE SILTY FINE SAND	S#5, 20' to 22' (9-8-14-21) RECOVERED 22 in.
32'	DENSE FINE SAND, TRACE INORGANIC SILT	S#6, 25' to 27' (11-15-16-13) RECOVERED 15 in. S#7, 30' to 32' (7-12-18-14) RECOVERED 14 in.

WATER LEVEL 10'
SIZE OF CASING BW LENGTH 30'0"
DRILLER: JOSEPH DESIMONE, INSPECTOR: ROB COLLINS
DATE STARTED & COMPLETED 7-14-15-2005

NOTE: USED CONCRETE CORE MACHINE

All samples have been visually classified by DRILLER. Unless otherwise specified, water levels noted were observed at completion of borings, and do not necessarily represent permanent ground water levels. Figures in parenthesis indicate the number of blows required to drive Two-inch Split Sampler 6 inches using 140 lb. weight falling 30 inches(±). Figures in column to left (if noted) indicate number of blows to drive casing one foot, using 300 lb. weight falling 24 inches (±).



First Floor Concrete

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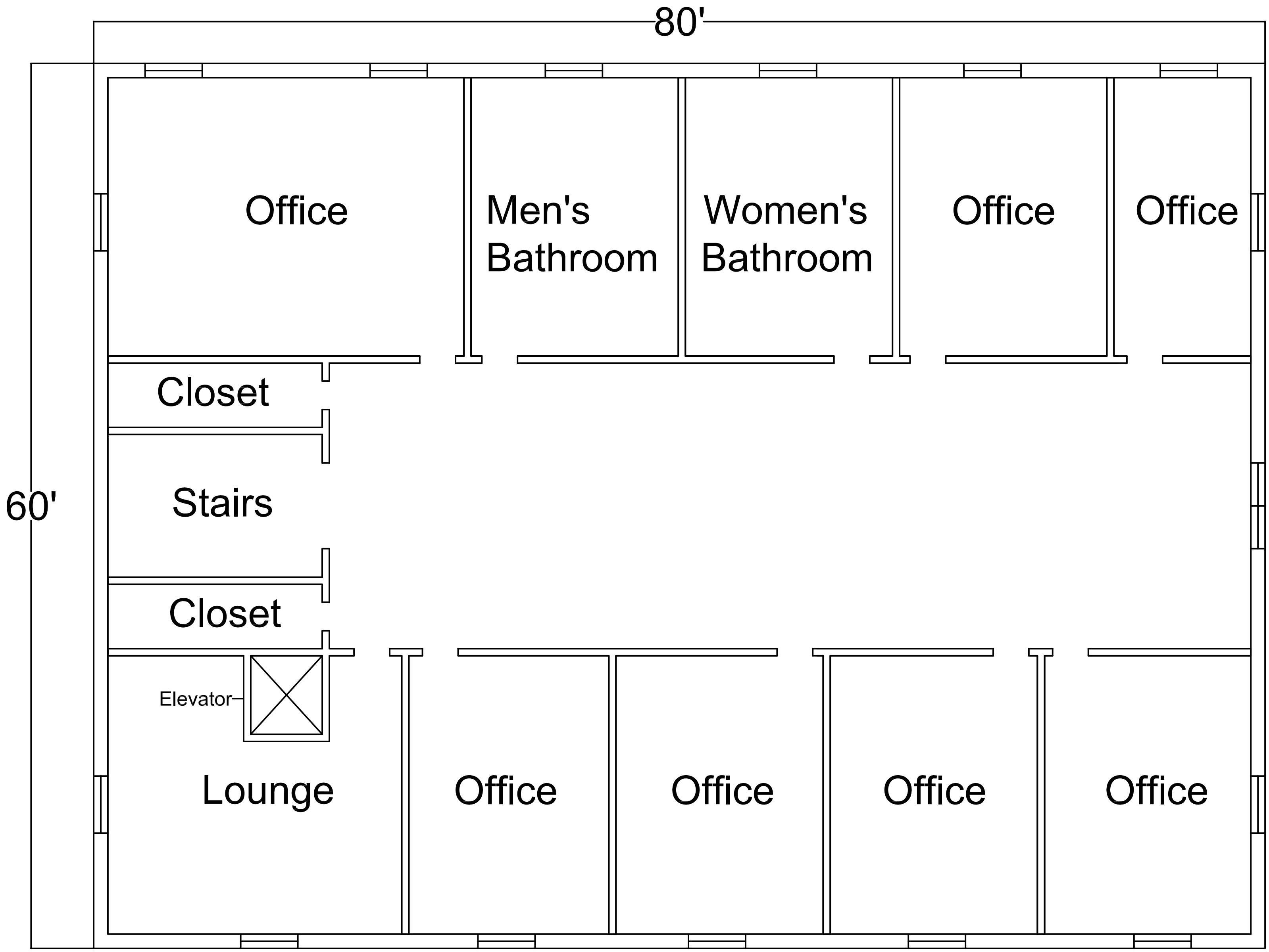
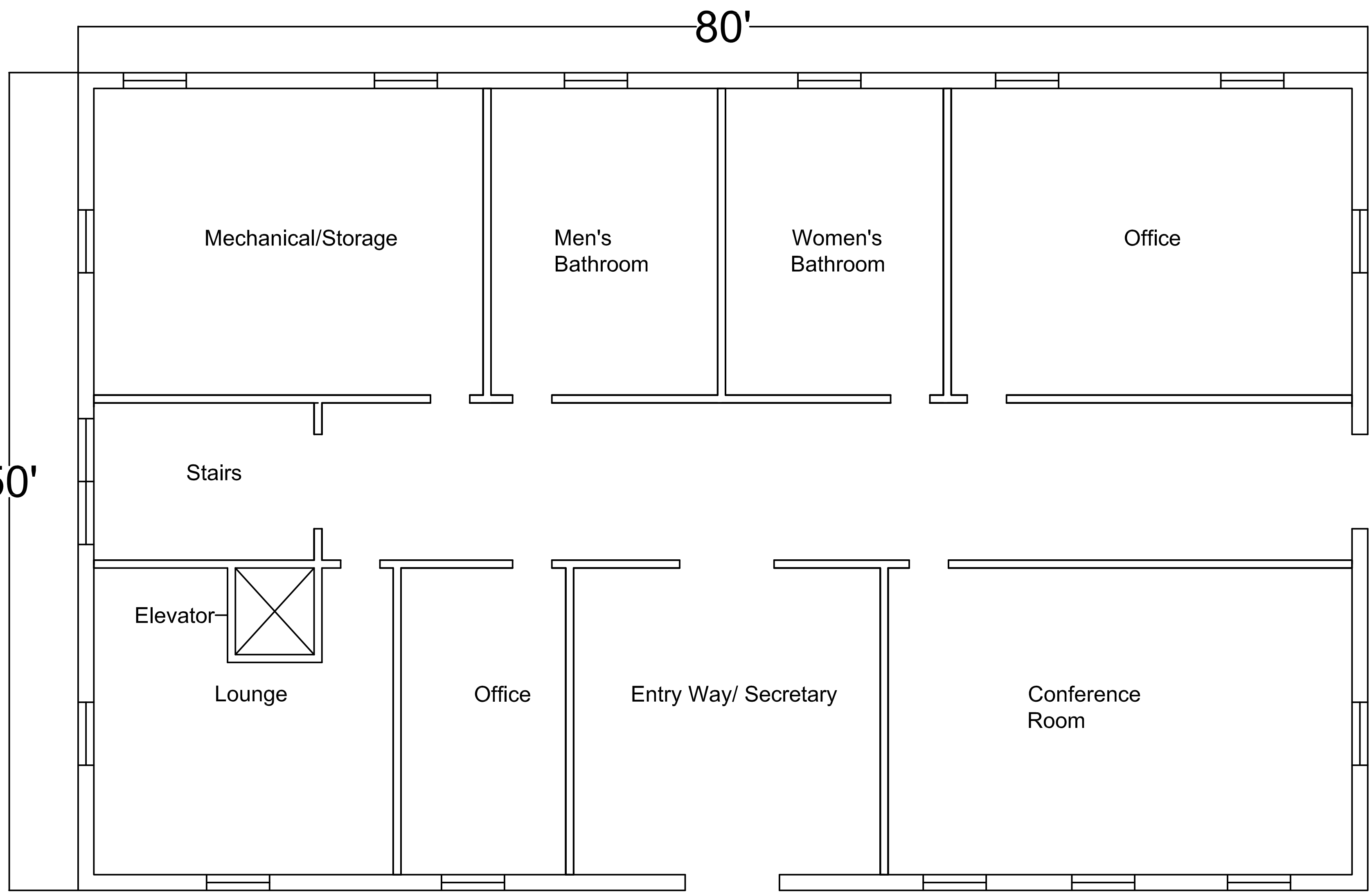
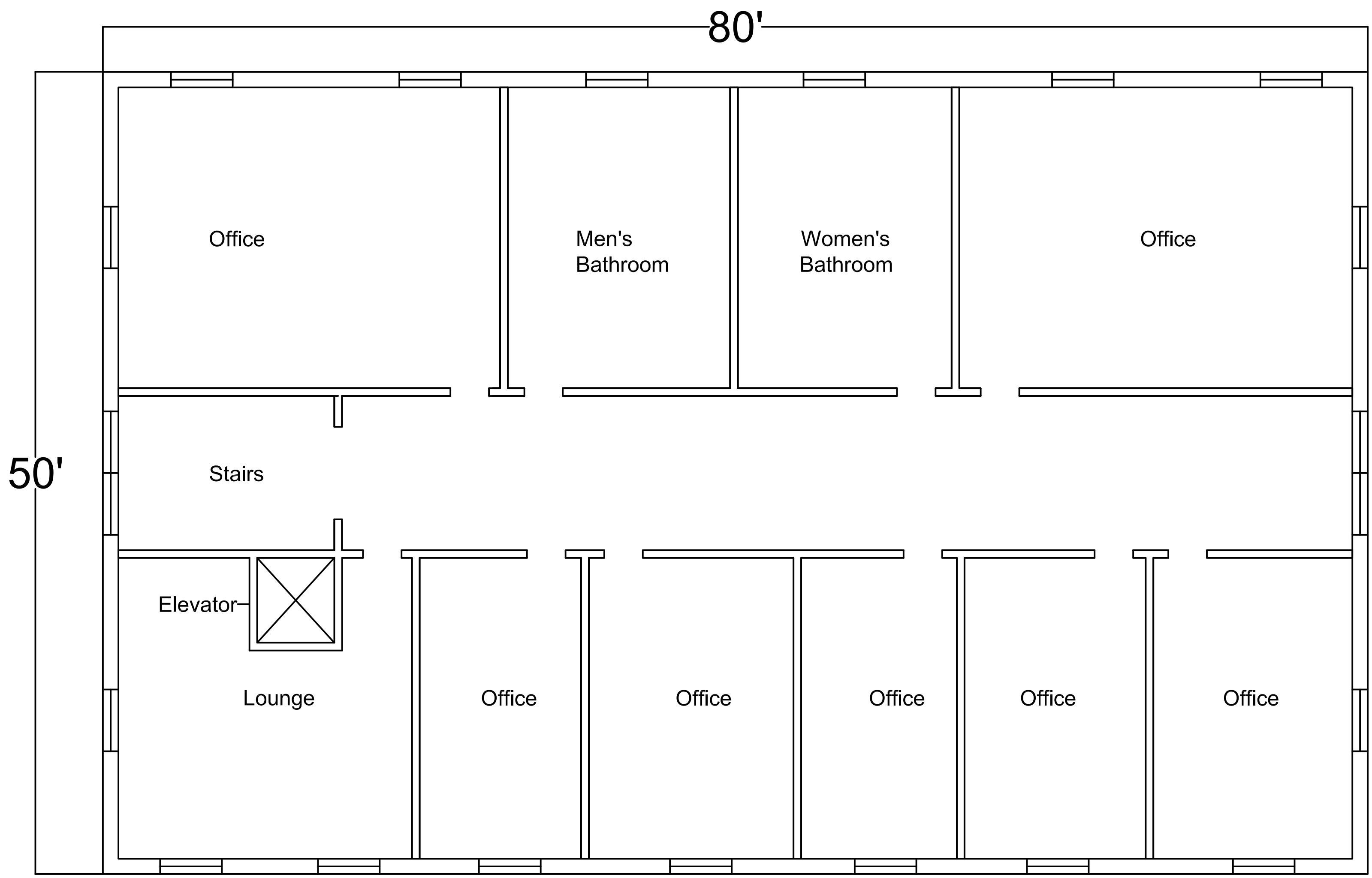
PRODUCED BY AN AUTODESK EDUCATIONAL PRODUCT



Second and Third Floor Concrete



1st Floor Layout



2nd and 3rd Floor Layouts

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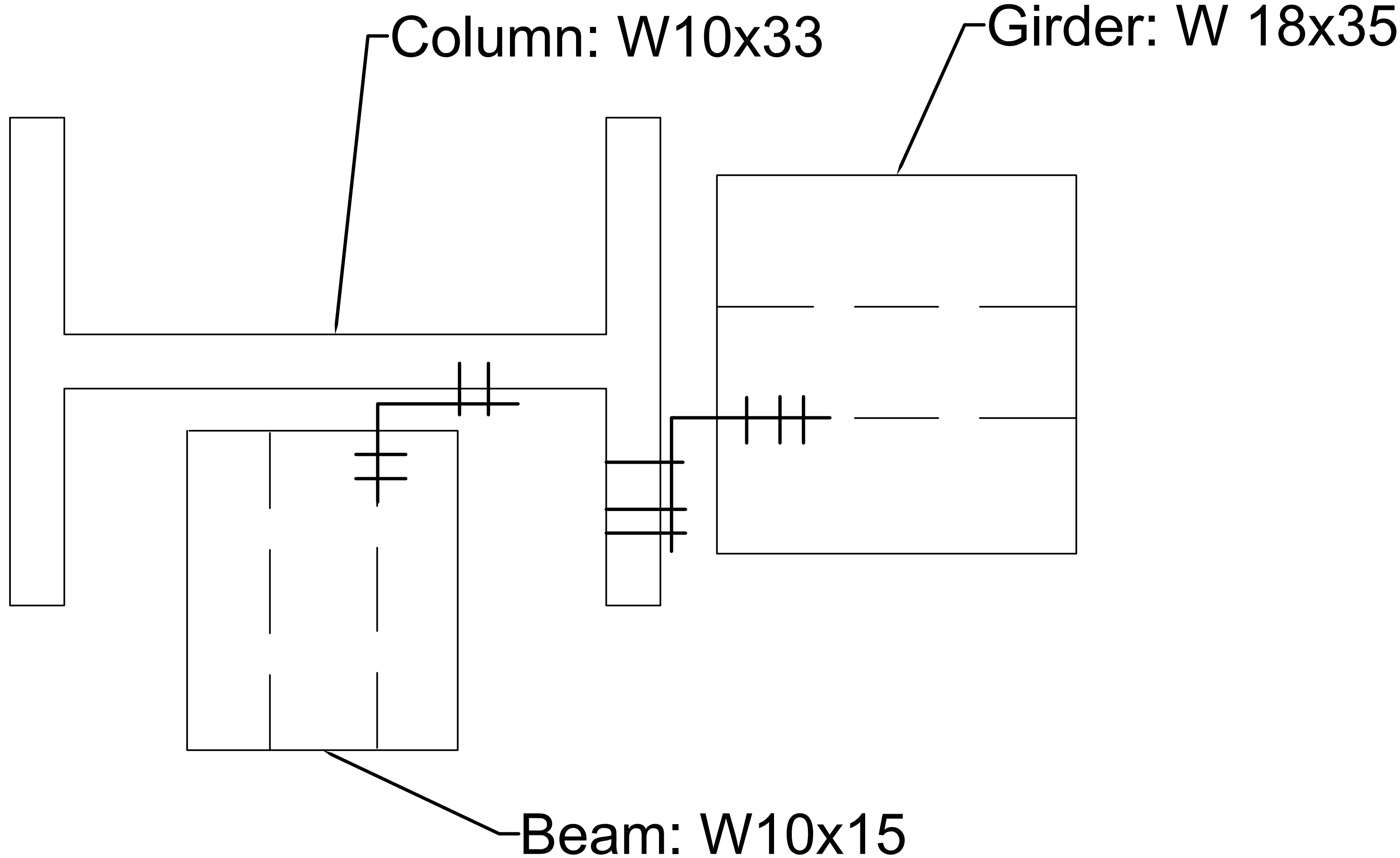
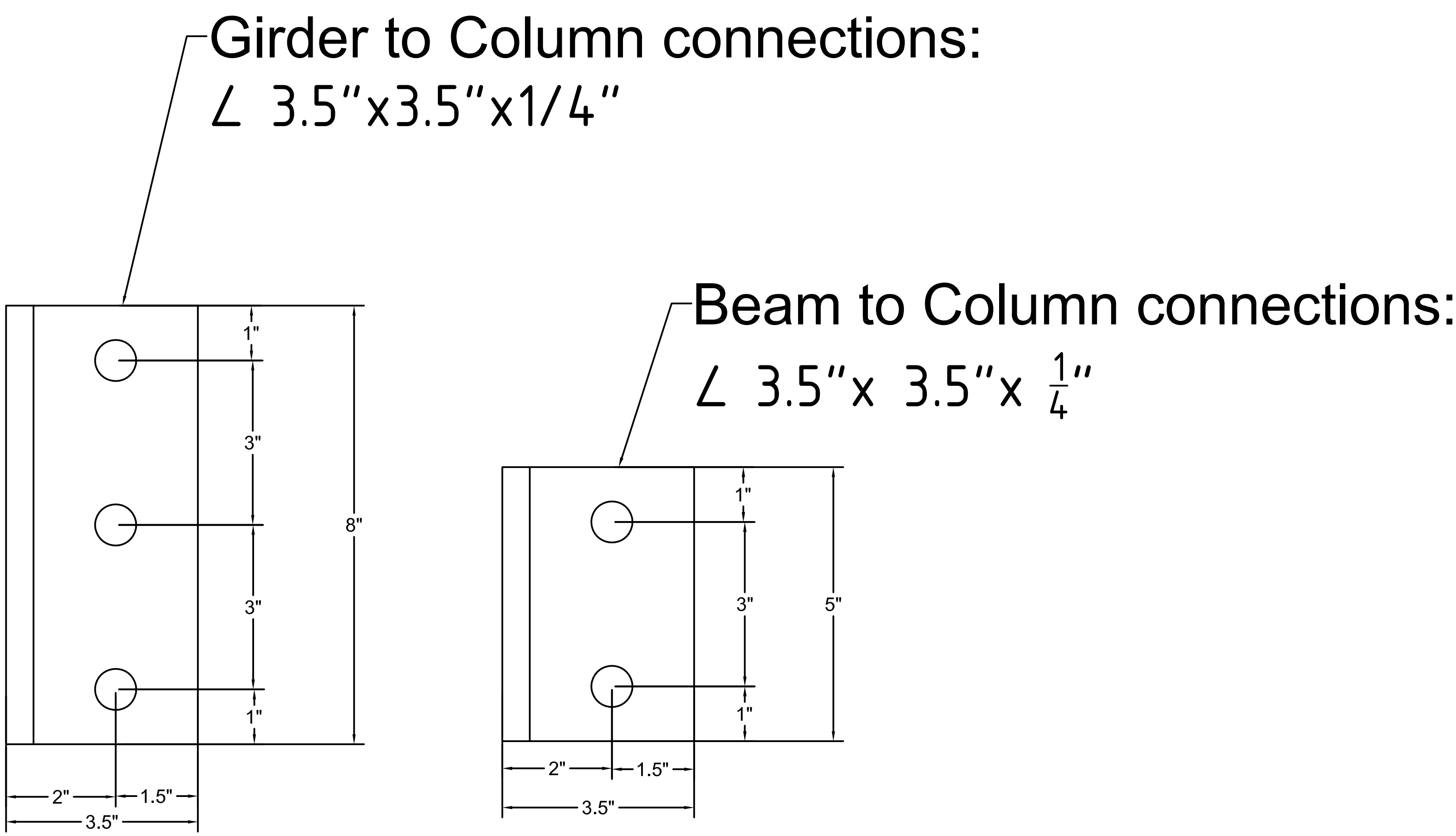
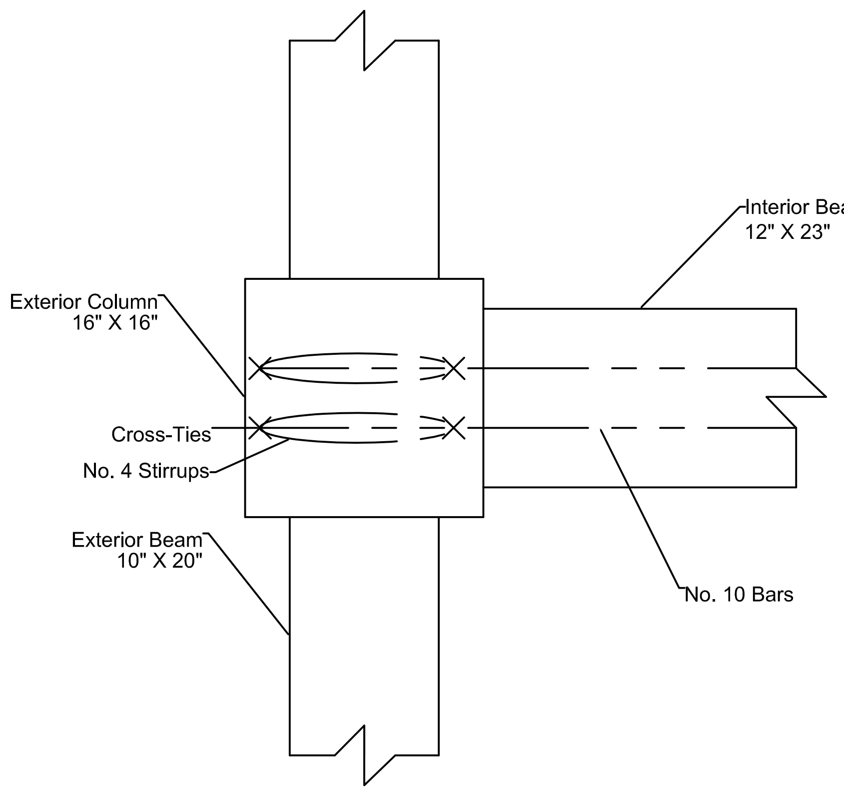
Section 1: Bolt Connection Configuration

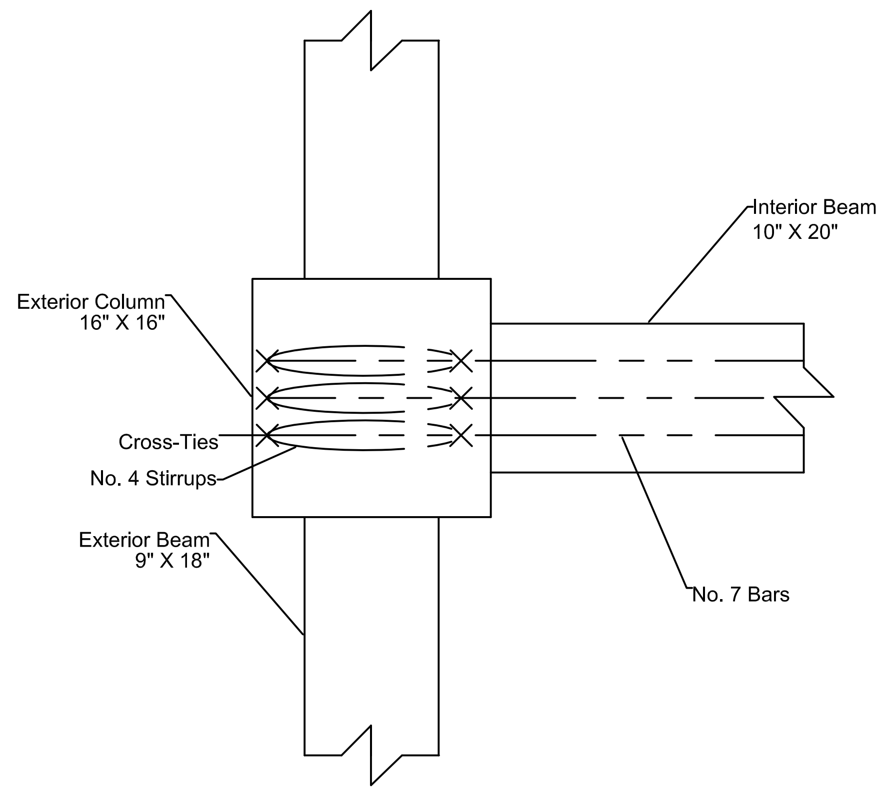




Section 1: Bolt Connections

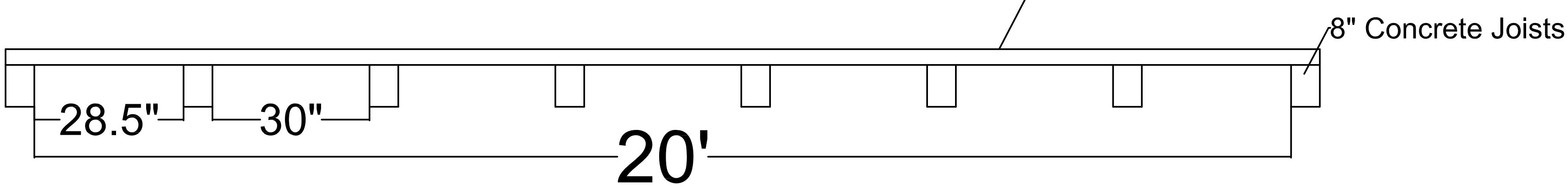


2nd & 3rd floor typical connection detail

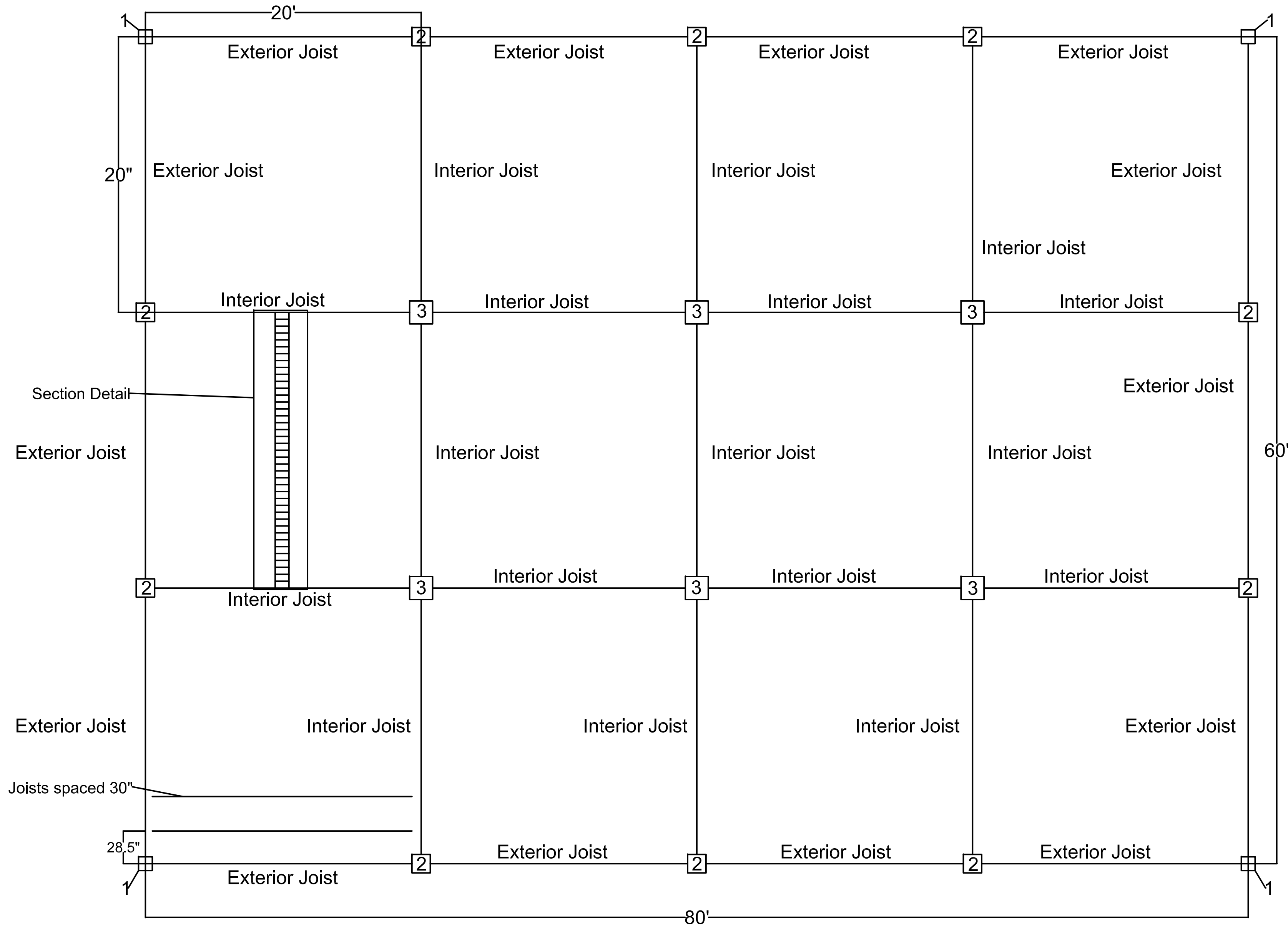


Roof typical connection detail

Floor & Roof Section

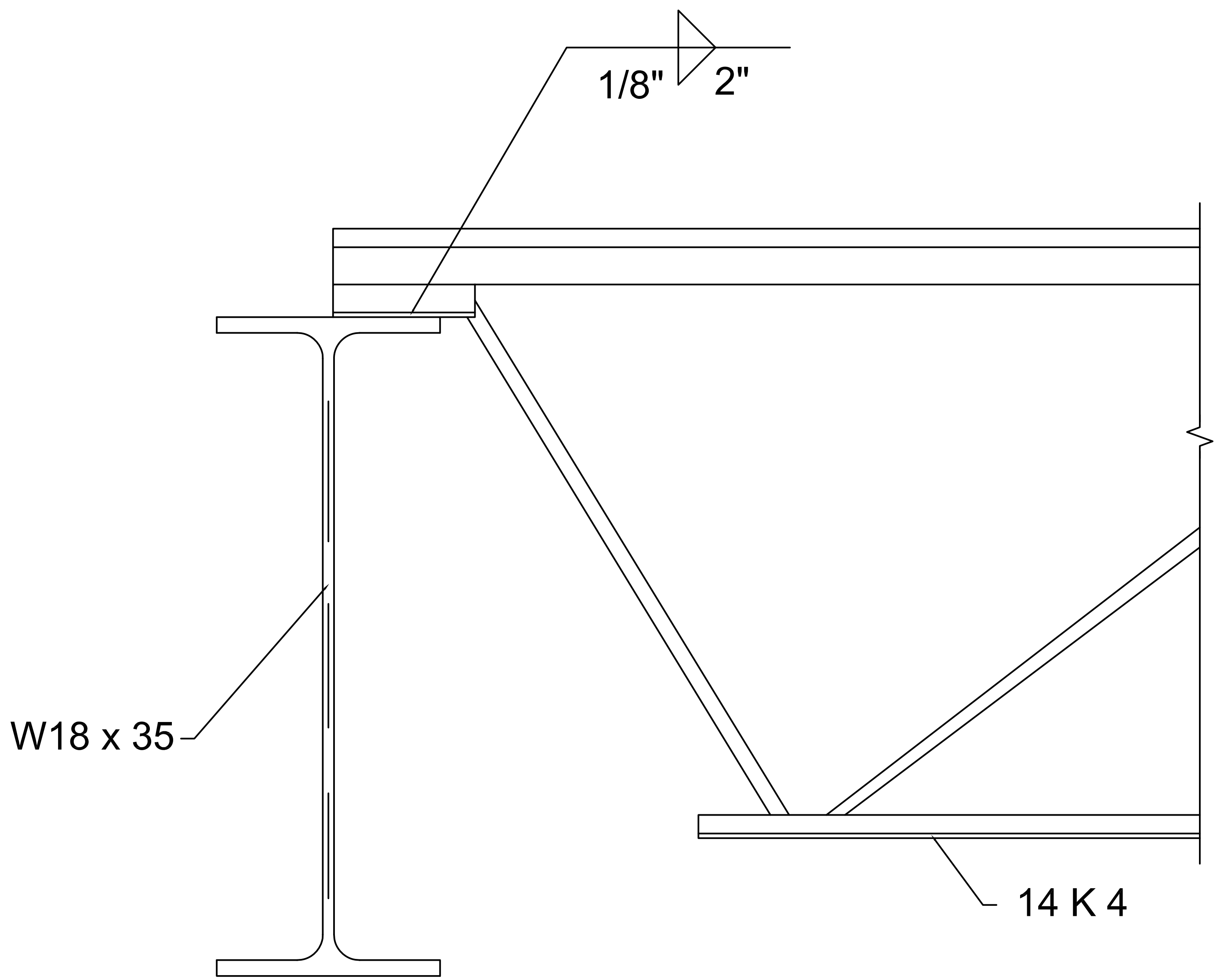


One-Way Concrete Joist Slab Section

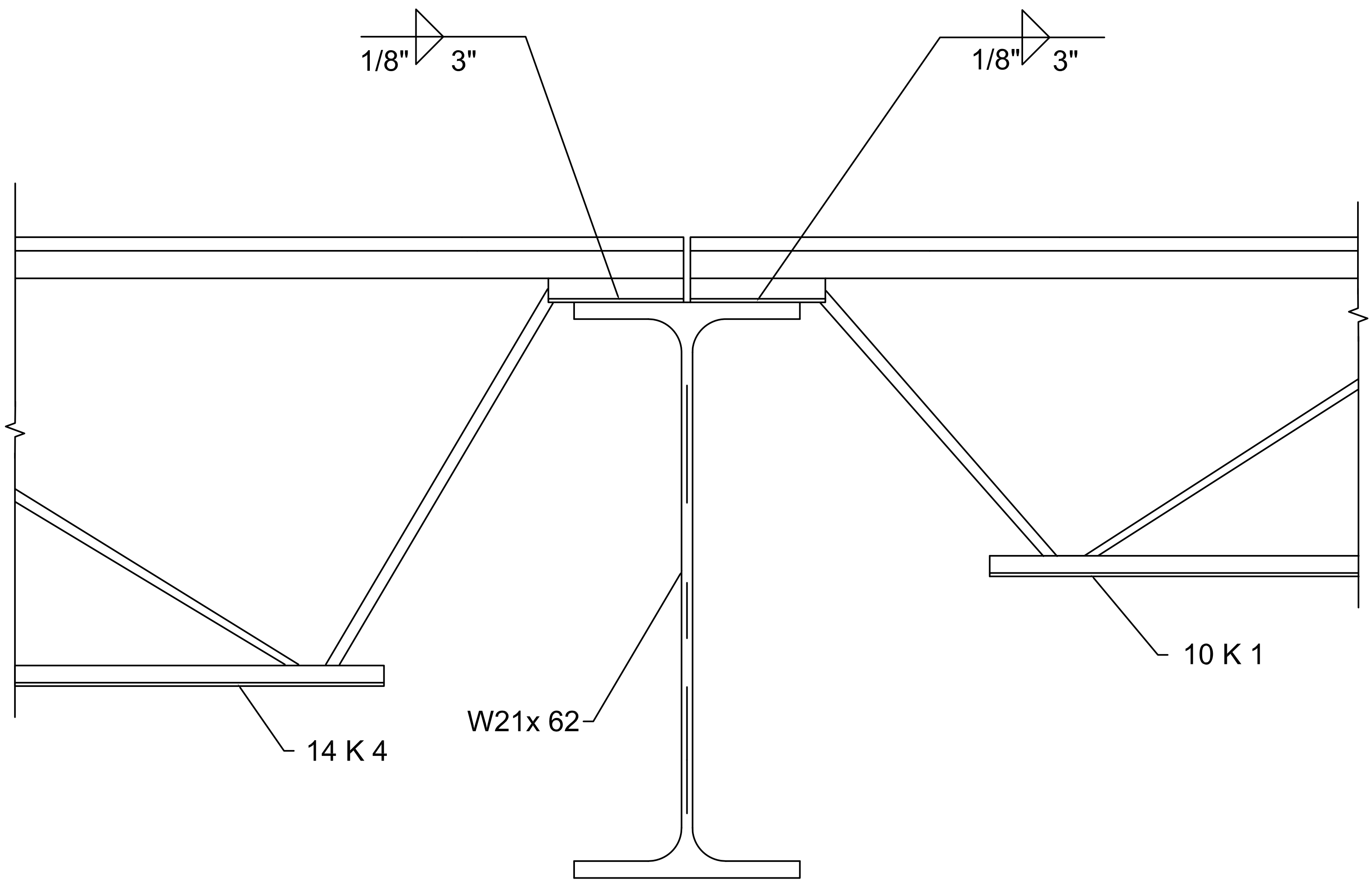


One-Way Concrete Joist Slab Layout

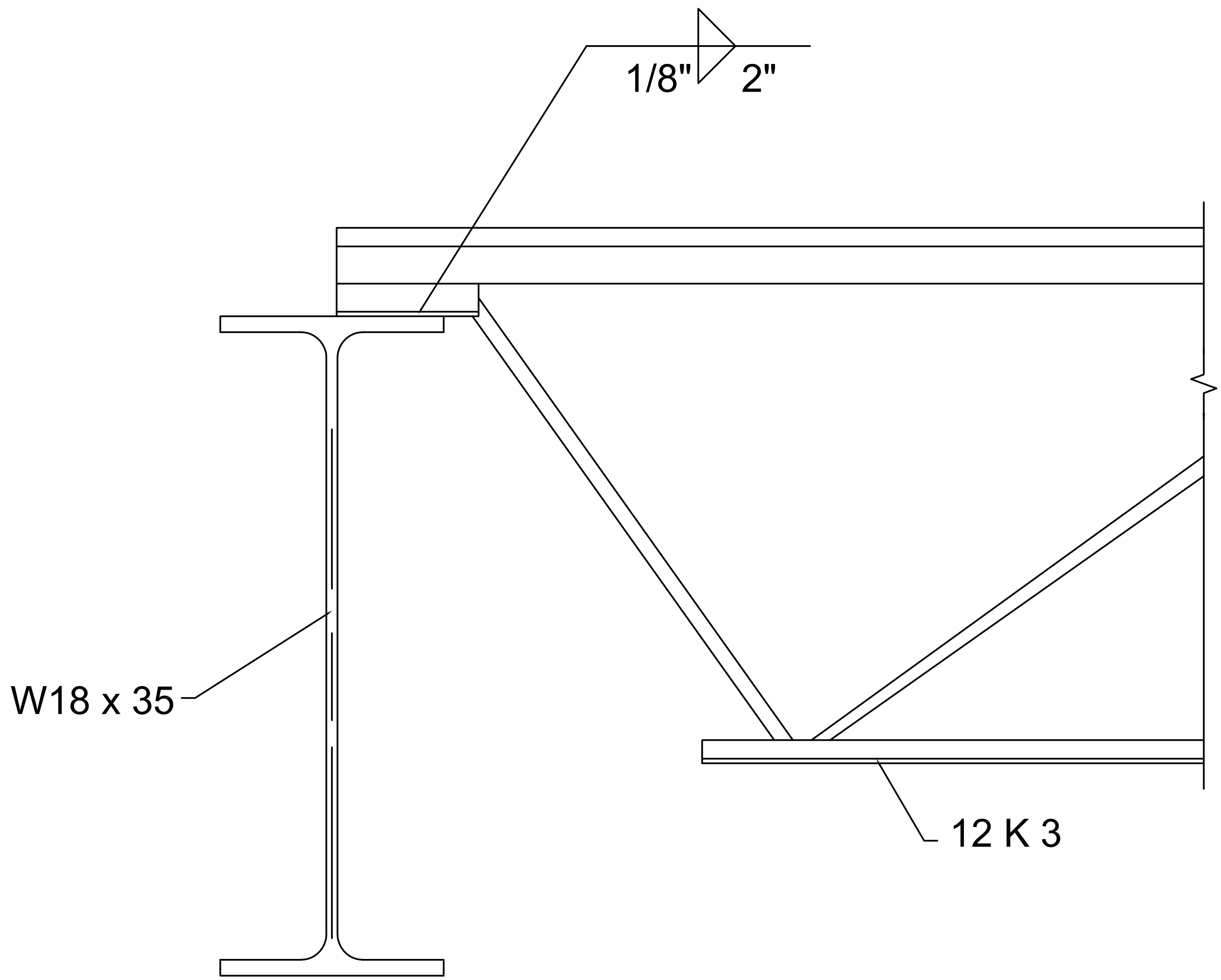
SECTION 2A: Floor Open- Web Joist to Girder Connection



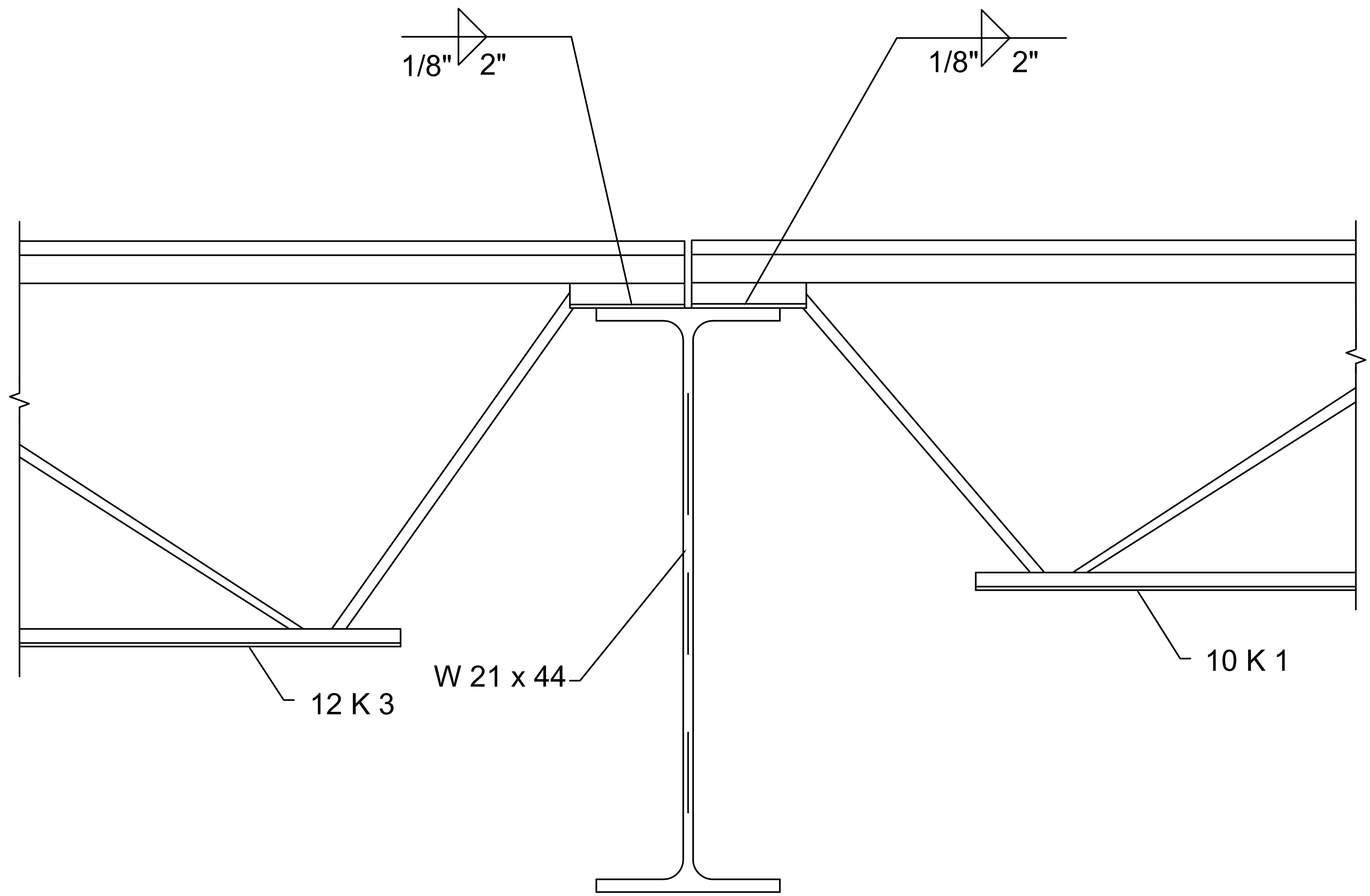
SECTION 3A: Floor Open- WEB Joist to Girder Connection

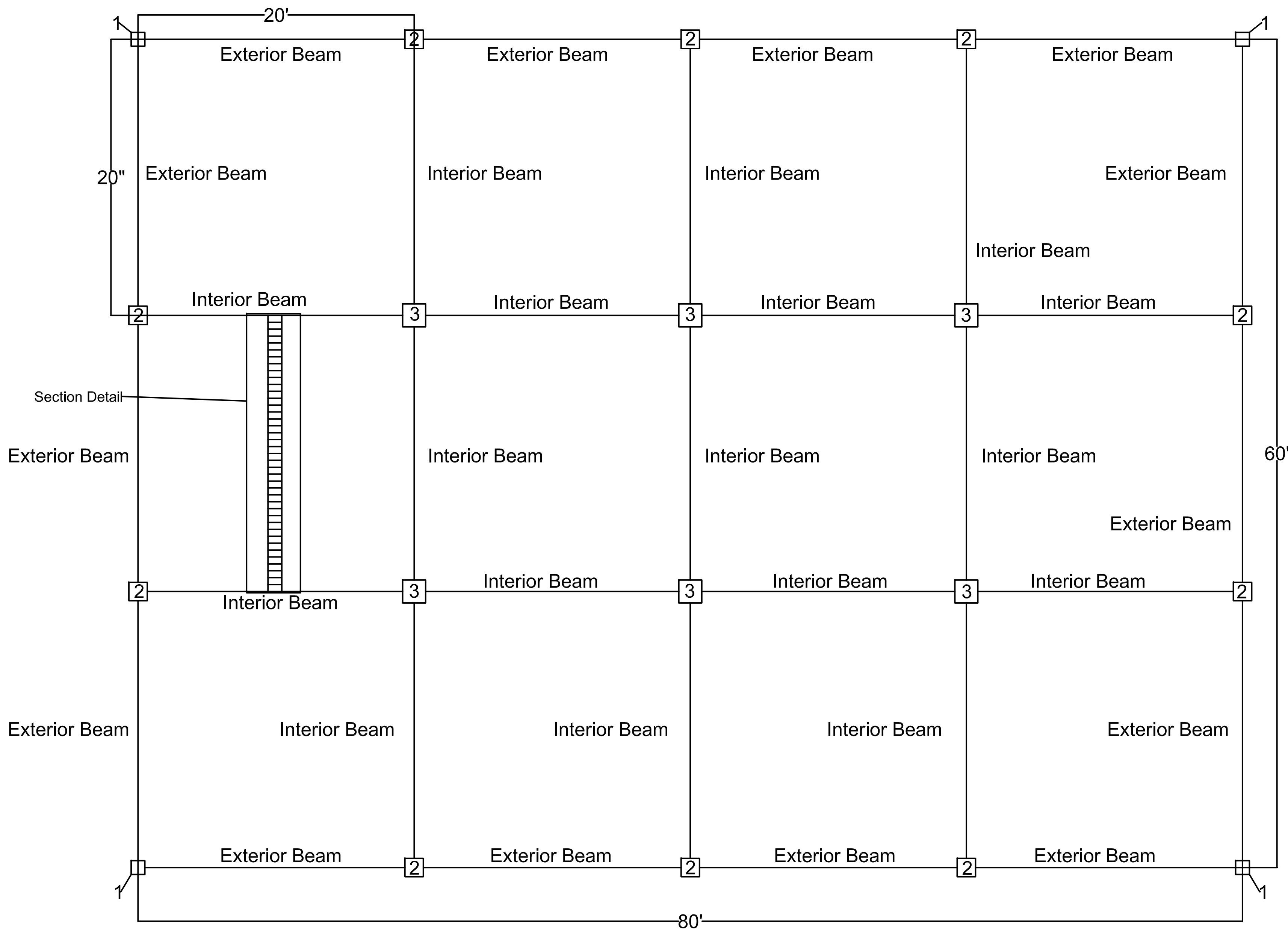


SECTION 2B: Roof Open- Web Joist to Girder Connection

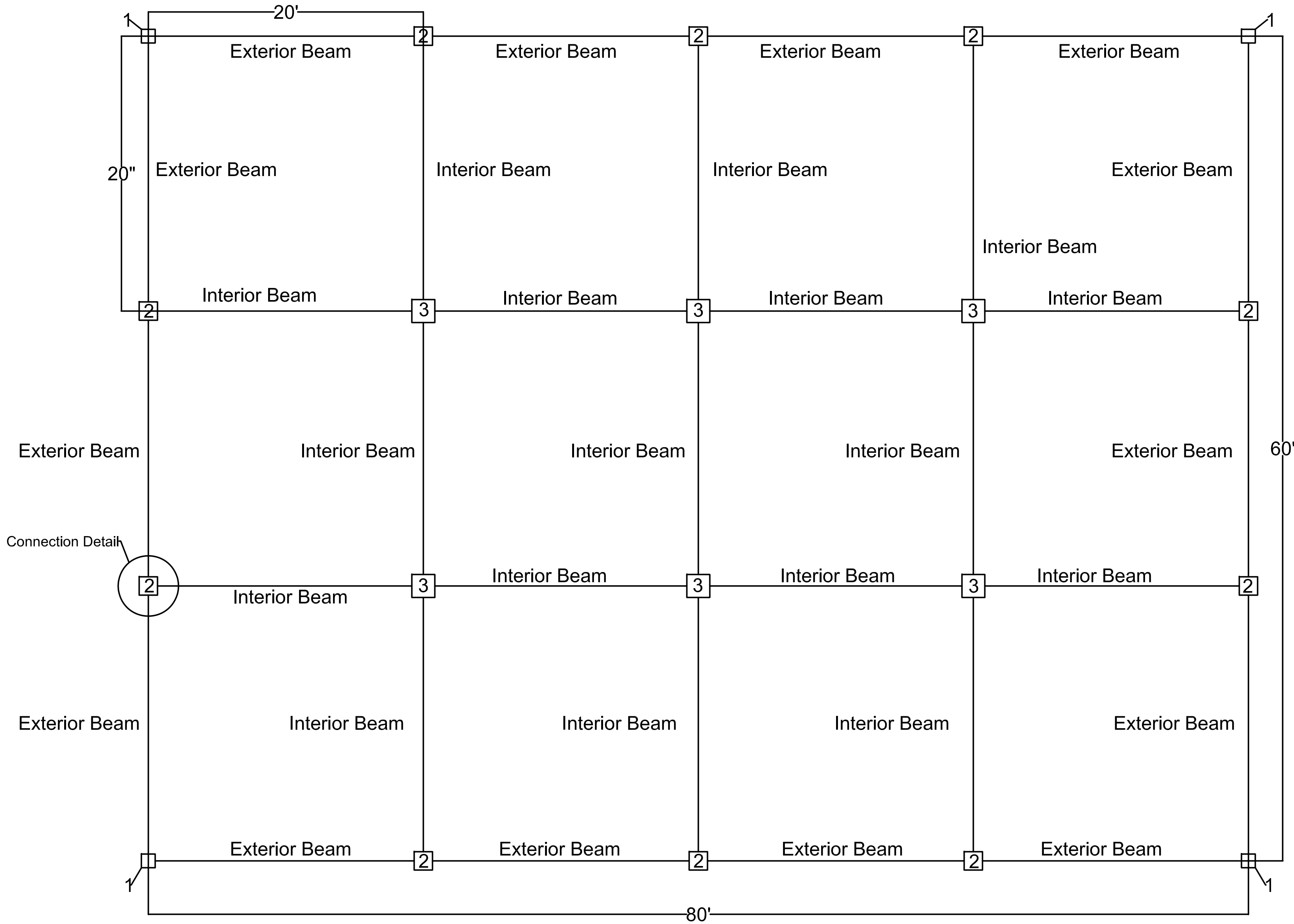


SECTION 3B: Roof Open- Web Joist to Girder Connection



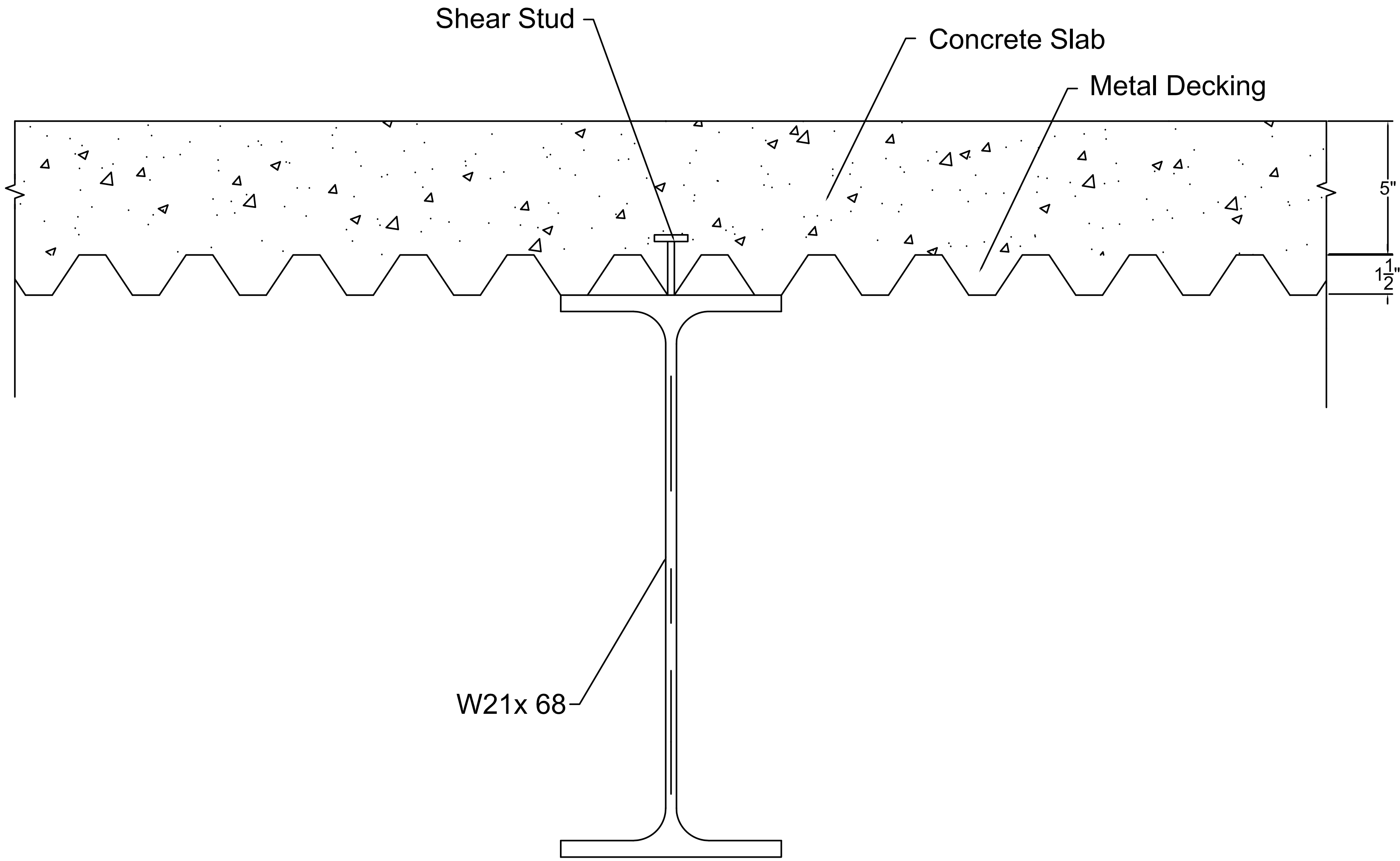


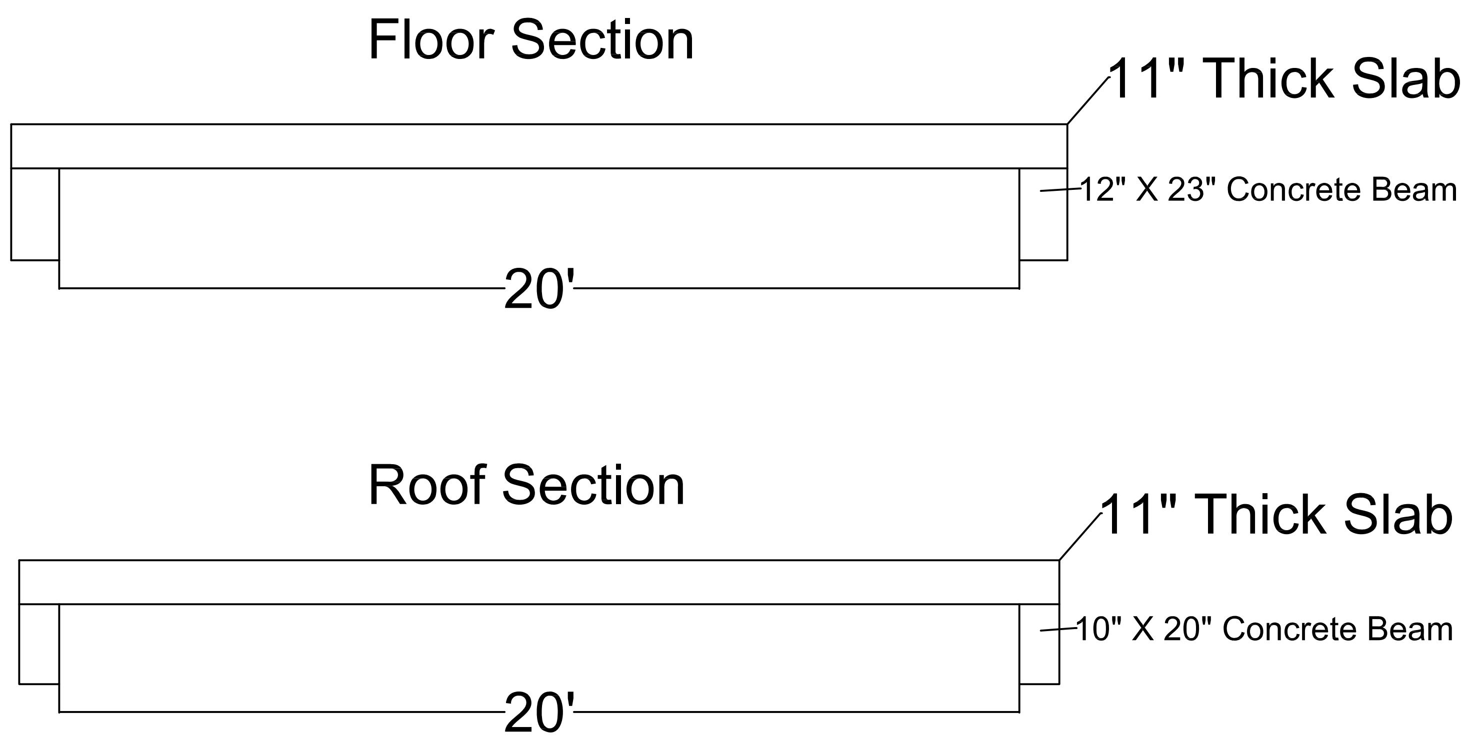
One-Way Slab-and-Beams Layout



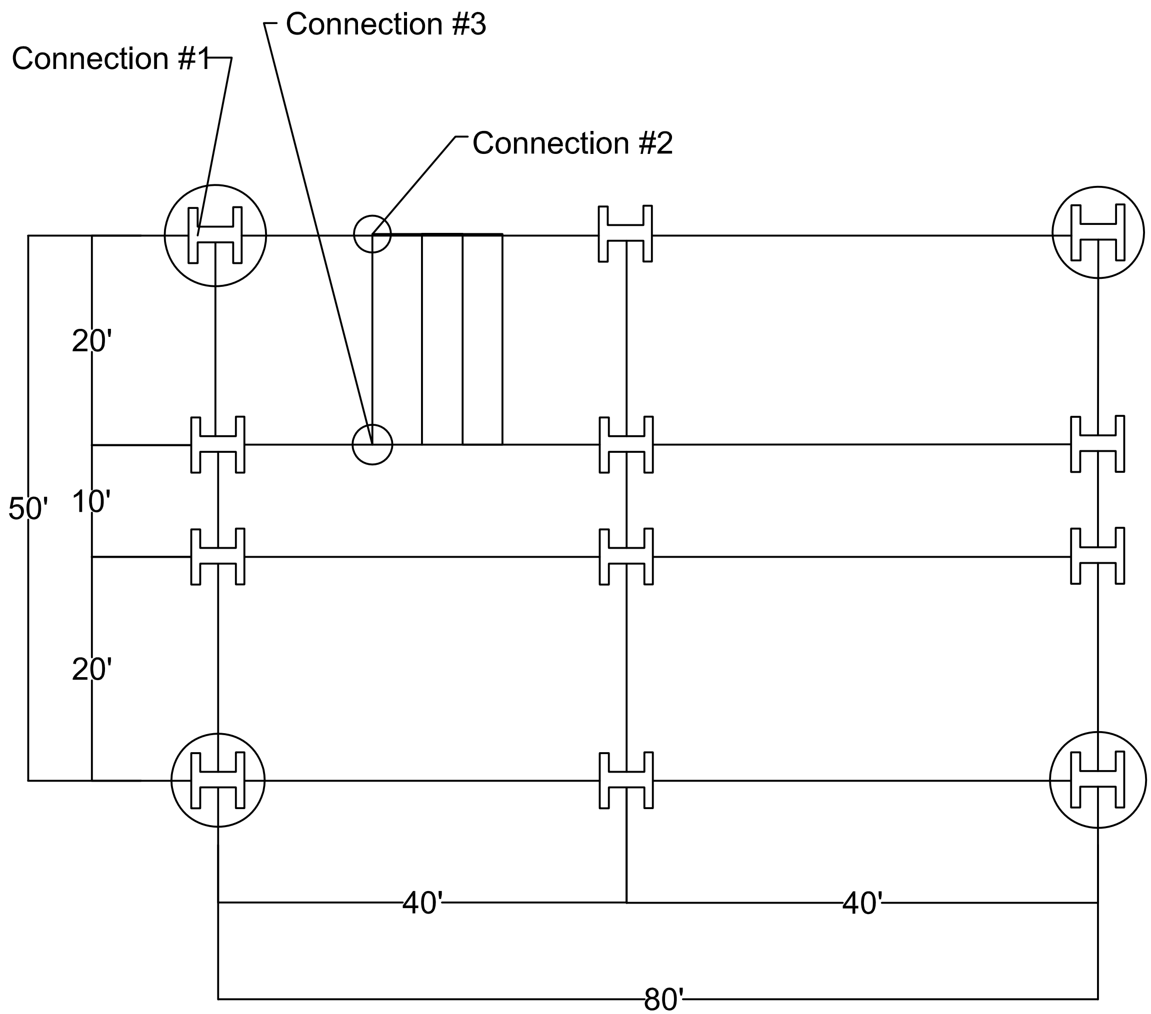
One-Way Slab-and-Beams Layout

Rolled Steel Typical Section:

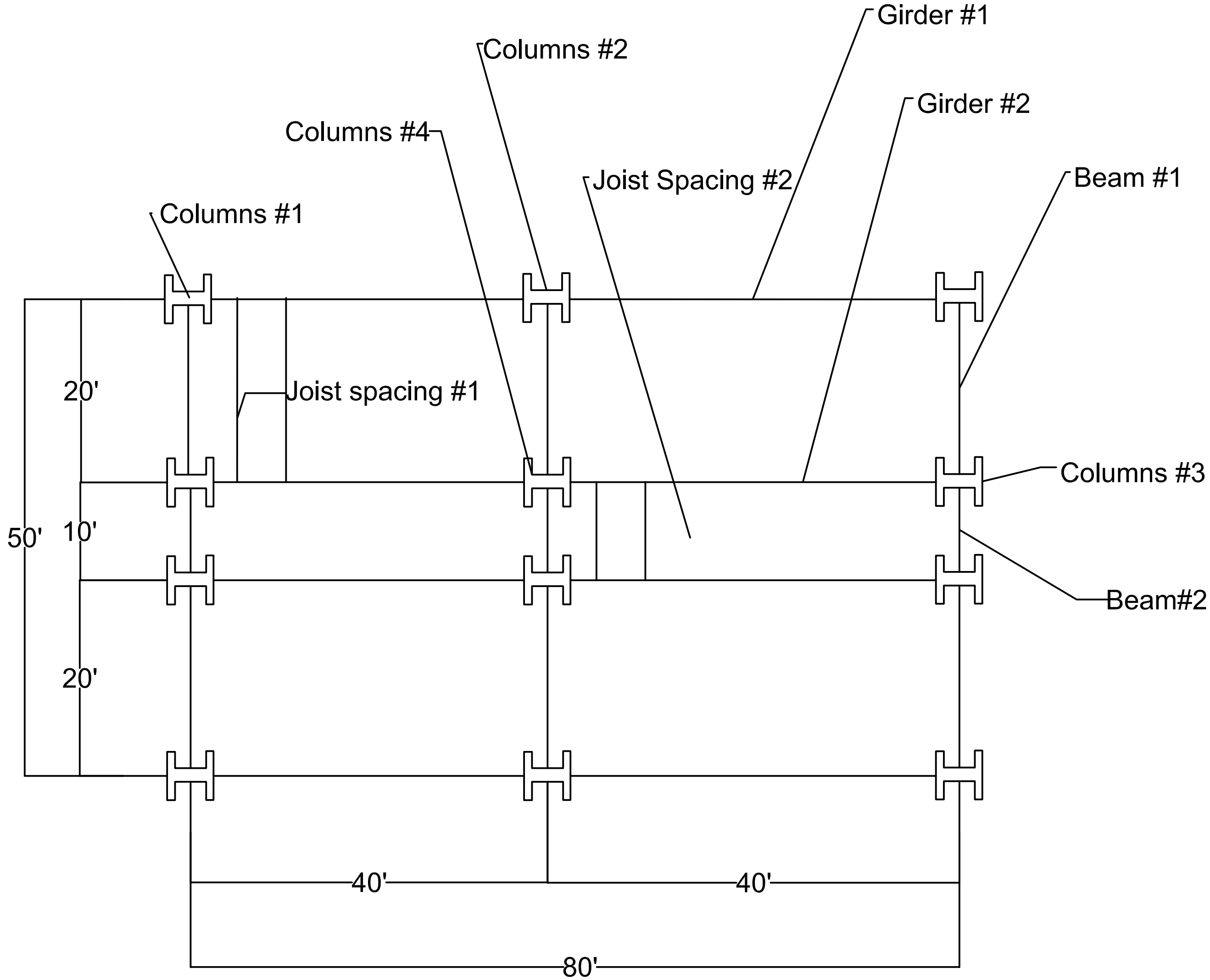




One-Way Slab-and Beams Section



Open-Web Steel Joist Layout



Open-Web Steel Joist Layout

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RSMeans Costs

Item	Cost		Unit
Floor: 3" LW Concrete on 2" Metal Deck	$	4.72	SF
Floor: 6" Concrete	$	7.47	SF
Floor: 1 1/2" Metal Roof Deck	$	2.09	SF
Concrete-Rectangular-Column: 24 x 24	$	146.39	LF
W-Wide Flange-Column: W10X49	$	59.35	LF
Footing-Rectangular: 72" x 48" x 12	$	504.00	EA
Wall Foundation: Bearing Footing - 36" x 12"	$	37.83	LF
HSS-Hollow Structural Section: HSS6X6X.500	$	27.82	LF
W-Wide Flange: W12X26	$	43.12	LF
W-Wide Flange: W16X36	$	65.12	LF
W-Wide Flange: W21X83	$	125.46	LF
Basic Wall: Foundation - 12" Concrete	$	16.44	SF