Steel Structural and Fire Safety Design for a Four-Story Office Building

A Major Qualifying Project Report

Submittal to the Faculty of

WORCESTER POLYTECHNIC INSTIUTE

In partial fulfillment of the requirements for the

Degree of Bachelor of Science

By

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Approved:

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Abstract

The purpose of this project is to develop the design of a four-story steel building. Using different types of steel framing and construction, we recommended a specific plan of construction that is both economical and structurally sound. We also investigated different building fire safety applications in order to design and recommend a fire protection system fit for the purpose and construction of our building based on economic and fire safety factors.

Acknowledgements

The whole team of this MQP would like to acknowledge the faculty advisor who helped us throughout the process and provided the team numerous valuable suggestions. Without his help, this MQP would not be a successful project. Also we would like to acknowledge faculty member from Department of Fire Protection Engineering, Professor Nicholas Alex Dembsey. He brought us many valuable suggestions to facilitate our analysis.

Project Advisor: Professor Leonard D. Albano (Associate Professor at WPI)

Authorship

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Capstone Design

In this Major Qualifying Project, the three of us were acting as an engineering design team responsible for the design of a four-story office building with fire protection systems. As engineers, we must design a structurally adequate building with respect to costs and structural integrity. Using the *AISC Steel Manual*, the *ACI Manual* and the *International Building Code*, we developed several different construction schemes. These construction schemes were evaluated by cost using the *RS Means Building Construction Cost Data*. Once a structural scheme was selected, the fire safety aspects of the building were analyzed and implemented using the ICC and NFPA documents.

Economy

- Cost-efficiency was one of the most important aspects of any design proposal.
- Using the *RS Means Building Construction Cost Data*, we evaluated our design scenarios for our building in order to rank them form least expensive to most expensive.

Constructability

- Our team developed several design scenarios that include composite and non composite construction, various bay sizes, various beam spacings and open web joist construction. Investigations on each scenario were conducted to examine the constructability, such as the repetition of structural members, the application of fireproofing spray, and site excavation for footings.
- We also investigated different lateral loading systems for our structure. Both rigid and braced frames were investigated in order to determine which is best for our structure based on cost.

Safety

- Structural integrity was addressed by using engineering method to design the structural steel frame.
- Using the *International Building Code* and NFPA publications, our team designed the necessary health and safety requirements, including the concepts and principles of egress, smoke control, fire partitions, fire department access, sprinkler systems and structural performance in fire. All of these aspects were investigated and then implemented in order to design a structurally sound building that is safe from fire and other disasters.

Ethics

- We designed a quality building with a quality fire protection system all while demonstrating good design and construction practices.
- Our design complied with the most up to date codes and provisions of the *International Building Code* and NFPA publications.
- We were not concerned only with cost, but also with the overall performance of our building.

- We performed this entire project in compliance with Code of Ethics for Engineers from NSPE (National Society of Professional Engineers) – 1. The safety, health, and welfare of the public are our highest priority; 2. We only practice our knowledge in the area we are trained and competent; 3. We avoided any deceptive act; and 4. We treat our reputation singularly depending on our responsibility.

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Chapter 1: Introduction

In this project, the major focuses were design and selection of steel structural system and fire protection system for a four story office building. Each team member acted as an engineering consultant for a hypothetical architectural firm, regarding the design of a four-story office building located in Worcester, Massachusetts. The major tasks of the team included alternative designs of steel structural frames, the design of fire protection systems and determination the cost-value for each design. By looking at alternative steel design and protection systems, the team is able to select the design or system that is most beneficial to the client.

The building floor plan designs were developed in compliance with the *International Building Code 2009*, while the design of sprinkler systems satisfied NFPA 13, *Standard for the Installation of Sprinkler Systems.* Both prescriptive and performance-based designs and analyses were conducted by the team to investigate the overall cost and performance of structural systems and fire protection systems. The performance-based designs and analysis were based on various documents from National Fire Protection (NFPA) which includes but not limited to NFPA 72, *National Fire Alarm and Signaling Code, 2010 Edition;* NFPA 101, *Life Safety Code, 2009 Edition;* and NFPA 5000, *Building Construction and Safety Code, 2009 Edition.* The design of the floor plan from the architectural firm was a base for design of the structural steel frame.

When designing a steel building, there are several different routes that an engineer can follow to design numerous functional structural schemes. These structural schemes vary in detail: steel member types, member sizes, member spacing, member layouts, etc. The team investigated alternative structural bay scenarios with various combinations of construction systems and different steel structure members to compare the cost, the ease of construction and the performance. The investigations of structural members included W-shape beams and open web joists; the construction included non-composite and composite construction; and the bay size varied from 20 ft X 20 ft to 25 ft X 25 ft with beam spacing varied from 4 ft to 10 ft. The variations in the structural schemes offer different constructional and architectural benefits. The schemes also vary in cost. By investigating the advantages and disadvantages of different scheme details, our team selected and designed a steel scheme which was determined to offer the most beneficial combination of architectural freedom, ease of construction and cost.

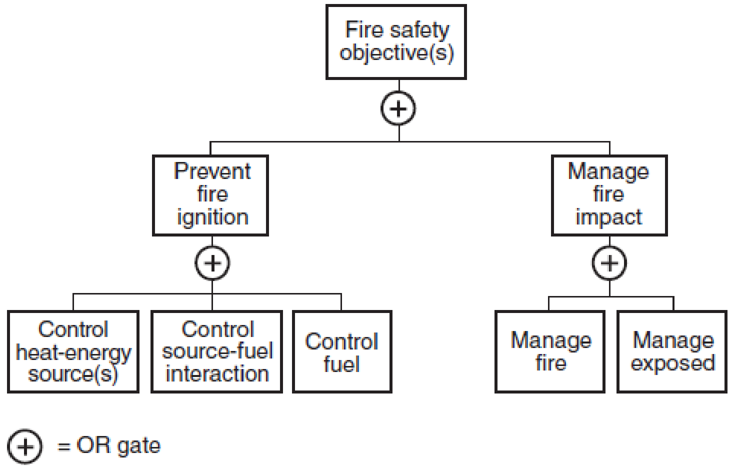
Alternative designs for the fire protection systems were investigated through two scenarios: one with automatic sprinkler systems and one without. In order to conform to the fire safety requirements of the Authority Having Jurisdiction, the team conducted a code review to examine the preliminary design from the architectural firm. Based on the code review, modifications to the existing design were proposed for both scenarios. Fire protection systems with and without automatic sprinkler systems were designed to compare differences based on cost and performance. Recommendations to the architectural firm were concluded by the team based on the comparison. Performance-based analyses were conducted in terms of the performance of structural members under fire conditions and performance of the fire protection system under design fire scenarios.

Chapter 2: Background

There are an endless number of steel buildings, towers, bridges and other structures in United States. However, if we think about all the advantages of steel, there would be no doubt why steel is one of the most predominating materials in construction. Steel has really high strength per unit weight. The property of steel does not change with time and it will last “indefinitely”. Additionally, the behaviors of steel can be calculated accurately. Furthermore, the ductility of steel ensures that a large amount of elongation will occur at the point of failure before the actual fracture occurs. Addition to all these merits, steel can be made into any shape while the integrity can be perfectly preserved. This feature enhanced the design options for architects and structural engineers. Modern steel construction has so many options for the structural elements. One of the classical structural steel members is the wide-flange beam. The I shape cross-section of wide-flange beam is a sophisticate design which gives the best performance while using less material. Steel open web joist has been developed to save even more material. The performance of steel structures can also further been enhanced by using composite construction. These merits of steel construction make the steel construction as one of the most popular construction in the world.

However, steel is not perfect. If corrosion was addressed and protection against it was not applied, steel structure will not last long. Furthermore, the strength of steel will decrease if it is exposed to a high temperature. It is believed that the steel lose lots of its strength at approximately 550 ℃. (Drysdale, 1998) Therefore, building fire issues need to be addressed in every building. Moreover, building fire does not only have bad impact on structure steel frame, it is also a big threat to many other aspects.

Fire is one of the most devastating events. The direct property damage is about 10 million dollars each year in United States (John, 2008). And if includes the indirect losses (such as factory shuts down, or company discontinues operation), the economic losses are much more. Fire is also one of the biggest threats to people’s lives. Although, the number of people killed in fire is decreasing each year, there were still more than 3900 death due to fire in 2004. (John, 2008) Hence, fire safety should be addressed in each building. There are many model codes require the building designers and engineers to ensure the safety of people and property. The strategies behind all of the codes related to fire safety, and engineering methods can be concluded in fire safety concepts tree. The principle branches are shown on next page.



Based on the fire safety concept tree, in order to achieve certain fire safety objectives, such as life safety and property protection, either preventing fire ignition or managing fire impact is needed. Unfortunately, no one can predict when and where the fire occurs. Therefore, how to manage fire impact is extremely important. Most building fire deaths are directly related to these products of combustion. In order to control the combustion process to ensure life safety, the interior finish materials and furniture can be carefully selected. Moreover, fire spread can be controlled by contain it within enclosure bounded by fire barriers. Furthermore, fire can be suppressed by either manually or automatic suppression systems. "Manage exposed" primarily means to limit the number of individuals and amount of property that are exposed in fire. Therefore, in any fire incidents, people shall be able to safely move out from the building. In order to this, exit system shall be carefully designed to ensure efficient evacuation. However, if fire goes out of control, the heat cumulated can jeopardize the structural integrity, especially for steel structures. If the structure fails, the magnitude of loss increases dramatically. Traditionally, there is no overlap between structural engineers and fire protection engineers. However some incidents, such as the structure collapse of World Trade Center, show people that it is important that structural engineers and fire protection engineers can understand each other to prevent such devastate accident.

Chapter 3: Methodology

The starting point for our project began with the architectural floor plan and layouts that guided our design for structural and fire protection systems. Office buildings typically have very simple and straight forward designs since there purpose is to maximize usable floor space and work areas for employees. The *Architectural Graphic Standards 11th Edition* provides typical work station dimensions and recommends that the floor plan be symmetrical about the X and Y axes. The floor plan of any building also depends very much on safety. Building codes, such as the *International Building Code 2009*, outlines requirements for egress and occupancy. Lastly, the floor plan must accommodate the structural frame. An initial look into column spacing and location is important to maintain usable area and reduce conflicts with member locations and interior furnishings.

As the building design process continues, we moved from the design of floor plans to structural members. Typical beams, girders and columns were designed following the *Structural Steel Design, 4th Edition* and AISC *Steel Construction Manual*. Our focus, however, was on the selection of the best structural scheme with multiple types of steel construction. We designed structural bays using composite, non-composite and open web joist construction. Composite and non-composite constructions differ greatly in strength. In composite construction the steel member works in conjunction with the steel slab to resist load. This increase in strength allows for smaller steel sections to be used than in non-composite construction and lower material costs. The use of open web joists was also explored for cost and performance considerations. These members have open cross sections and are much smaller and lighter than W-shape beams. They are spaced at much closer distances, to meet strength requirements; however the cost is expected to be lower because of the overall material cost.

The structural scheme selection process was one of the more difficult tasks to complete. Unlike the design of structural members, there is not set process for selecting a final layout and construction method for our structural frame. Generally, designs are selected based on cost, as the owner does not want to pay more than necessary for a building. For our office building the different scenarios require many more criteria to effectively select the best one. To do this we came up with our own selection process with seven criteria. A score was determined for each of the seven criteria for every scenario. The scores were then added up and normalized using the score value for cost. We decided to use the cost as the most critical factor since it is the deciding issue between two designs that perform equally well. Our other considerations included architectural and construction concerns. The architectural scores were based on two criteria; net usable area and floor depth. Construction scores were determined for member uniformity, cooperation with the floor plan, number of footings and fire resistance.

After the structural frame is design for gravity loads, the effect of lateral loads must be considered and appropriately resisting with some form of lateral resistance. Lateral forces on structures are typically encountered in three forms; seismic, wind and soil pressures. Our main focus was on the effects of wind. Once the nature of lateral loading is determined a method to resist theses forces must be developed. There are two ways of resisting lateral forces, through vertical bracing or with moment resistive connections. Wind loads were determined using the provisions of

ASCE 7-05 Main Wind Force Resisting Systems. Once, the wind loads were determined we used the frame analysis program RISA 2D to determine the forces on girders and columns from both lateral and gravity loads. New members were design for a braced and rigid frame using the interaction equation from the *AISC Steel Construction Manual*. A cost analysis was performed on both frames and the lateral drift performance of the rigid frame to select the most appropriate style.

Once, the architectural and structural design for our office building were complete we moved on to the fire protection aspects of our projects. Initially, we performed a code analysis for a fully sprinklered vs. non-sprinklered office building. We determined the general, passive fire protection and active fire protection requirements for both scenarios following the *International Building Code 2009*. The conditions discovered in the code analyses led to the design of smoke detection and sprinkler systems. Smoke detectors are required by the *IBC* and the locations were determined in accordance with *NFPA 72: National Fire Alarm and Signaling Code*. A performance analysis of the smoke detection system to determine activation time was also completed using fire dynamics calculations and a design fire scenario typical of an office environment.

The sprinkler system was designed in accordance with *NFPA 13: Standard for the Installation of Sprinkler Systems* for light hazard occupancy. The piping system was sized originally using the pipe schedule method and was later revised using hydraulic calculations. A cost analysis of the sprinkler system vs. a building protected with only passive protection systems and the associated risks was performed to evaluate the advantages and disadvantages. The performance of the sprinkler system was checked using fire dynamics equations and the same design fire scenario used in the smoke detection performance analysis. A total time for sprinkler activation was found and the effectiveness of the system concluded.

The final aspect of our report includes the performance of structural members under fire conditions. The performance of structural elements was investigated under standard fire ASTM E-119. Utilizing numerical calculation method, the performance of unprotected steel girders and open web joists, steel girders and open web joist protected with suspended ceiling, and also the composite column were evaluated. This study reveals steel structure is vulnerable under fire but with sufficient protection, it can perform very well under elevated temperature.

Chapter 4: Architectural Floor Plans

4.1 Introduction

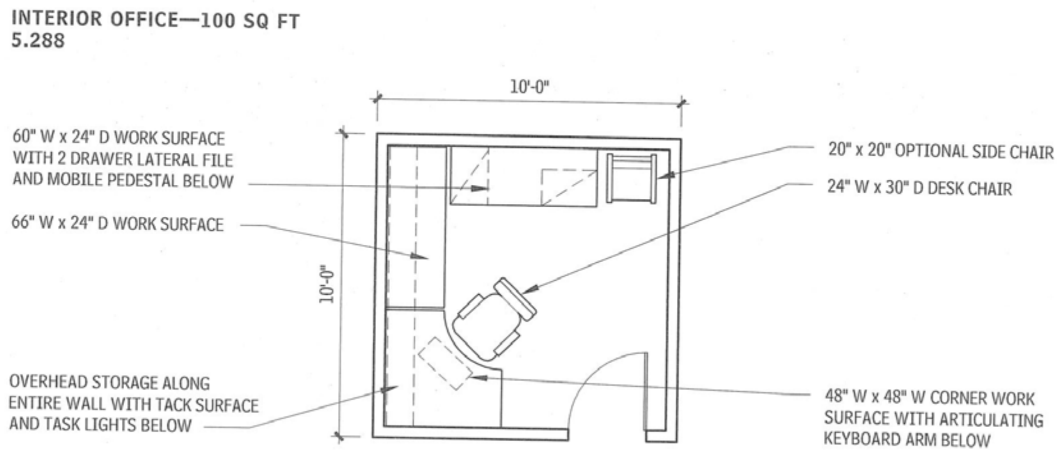
The designs of the floor plans were all based on *International Building Code 2009* (IBC 2009) and *Architectural Graphic Standards 11th Edition*. The floor plans provided user-friendly and safe environment for the clients. These floor plans also served as the basis of the structural steel designs, fire protection systems designs and code analyses. It was assumed that the owner wants a four-story 80,000 ft^2 office building. The team designed the floor plans with 200 ft X 100 ft footprint and 12 ft story height for the first floor and 10 ft story height for the second, third and fourth floors. It should be noted that according to section 903.2.11.3 of IBC, this office building was not required to have automatic sprinkler systems since the total height is below 55 ft. Therefore, the team considered alternative fire protection systems, specifically one without a sprinkler system and one with a sprinkler system. Since the team had only limited knowledge of architectural designs of floor plans and was not familiar with building codes, there were two attempts to finish the designs of the floor plans. The first attempt touched on some basic principals about floor plan design such as the number of exits per floor and minimum width of doors and corridors. The second attempt was conducted while the team was more familiar with IBC 2009, and revisions were made based on the first attempts and preliminary code analyses.

4.2 First Attempt

The team had referenced *Alternative Structural and Fire Protection designs of a 4-Story Office Building*, another MQP report, for designs of floor plans. In order to simplify the designs of floor plan, the general principle was to keep the building symmetric about both X and Y axes. Same components (such as office room, work stations) were repeated to reduce the complexity.

4.2.1 Office Compartments

While most employees work in their cubicles, higher ranking executives and managers have individual offices sometimes with a reception area. Typical dimensions of offices and these individual rooms were also referenced which is shown in figure below. The working cubicles were based on the 100 ft^2 office.



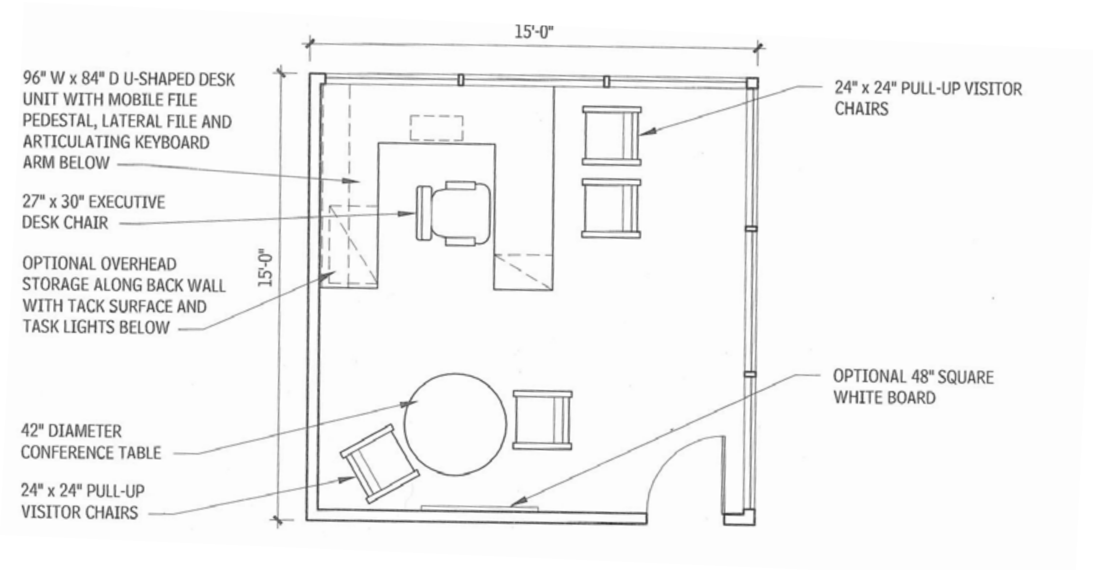


Figure 1 Sample Office and Conference Room Dimension (*Architectural Graphic Standards, pp568-570)*

4.2.2 Number of Exits

According to section 1003.7 in IBC 2009, elevators in most situations are not used as means of egress. Only in a few cases can the elevator be treated as the exit when there is a fire in the building. Extensive performance-based designs and analyses are needed if elevator needs to be treated as a means of egress. The team decided not to consider the elevator as an accessible means of egress in the floor plan designs. Therefore, each floor needed to have enough number of stairways to serve as exits.

Occupancy Load was the first thing that needed to determine the number of exits. The team decided that the second to fourth floors were all classified as business occupancy (the conference rooms are small enough to be considered as an accessory space which were classified as the predominate classification), the gross area per occupant is 100 ft^2. And the first floor had some spaces for gathering people, such as a cafeteria, entrance lobby and conference room which are indicated in Figure 2. Those areas were classified as assembly with area of 9600 ft^2 and the team decided that net floor area per occupant is 15ft^2. A portion of Table 1004.1.1 Maximum Floor Area Allowance per Occupant from IBC 2009 is shown below.

Table 1 Maximum Floor Area Allowance per Occupant (adopted from Table 1004.1.1 of IBC)

FUNCTION OF SPACE	**FLOOR AREA IN SQ. FT. PER OCCUPANT**
Accessory storage areas, mechanical equipment room	300 gross
Agricultural building	300 gross
Aircraft hangars	500 gross
Airport terminal Baggage claim Baggage handling Concourse Waiting areas	20 gross 300 gross 100 gross 15 gross
Assembly Gaming floors (keno, slots, etc.)	11 gross
Assembly with fixed seats	See Section 1004.7
Assembly without fixed seats Concentrated (chairs only-not fixed) Standing space Unconcentrated (tables and chairs)	7 net 5 net 15 net
Bowling centers, allow 5 persons for each lane including 15 feet of runway, and for additional areas	7 net
Business areas	100 gross
Courtrooms-other than fixed seating areas	40 net
Day care	35 net

Since the area of each floor is 20,000 ft^2, the approximate number of occupants is 20,000/100= 200 for 2nd to 4th floor while there are about 10400/100+9600/15=744. Based on Table 1021.1

from IBC 2009, the minimum number of exits shall be 2 when the occupant load is below 500 and 3 for 500 to 1000. Therefore, in our design, there were 3 exits for each floor.

4.2.3 The Minimum Width for Egress

The requirements for width for egress were also considered in the first design. According to section 1009.1 of IBC 2009, the stairway width shall not be less than 44in. And for corridors the minimum width is also 44in (section 1018.2). And based on section 1005 of IBC 2009, all other egress components shall not be less than the total occupant load served by the means of egress multiplied by 0.2 inches per occupant. Therefore a typical 32 inch single swing door can serve up to 160 occupants. The team decided to consistently use 32 inch doors in most applications.

4.2.4 The First Floor Plan Designs

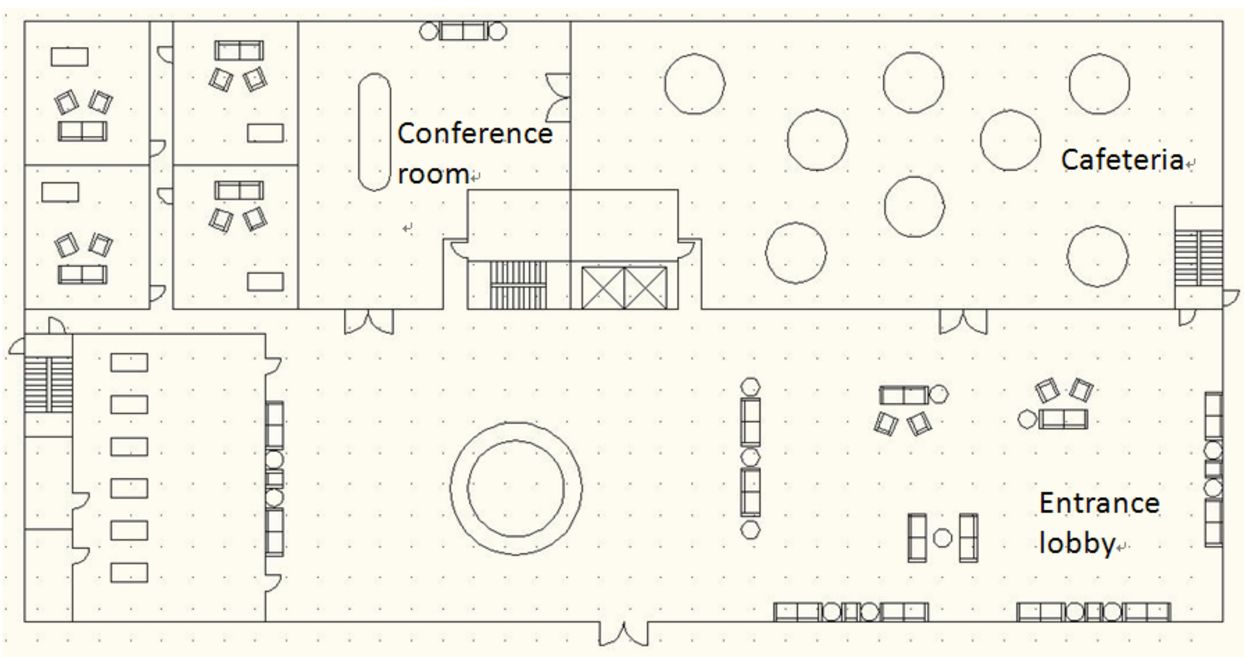


Figure 2 The First Floor Plan

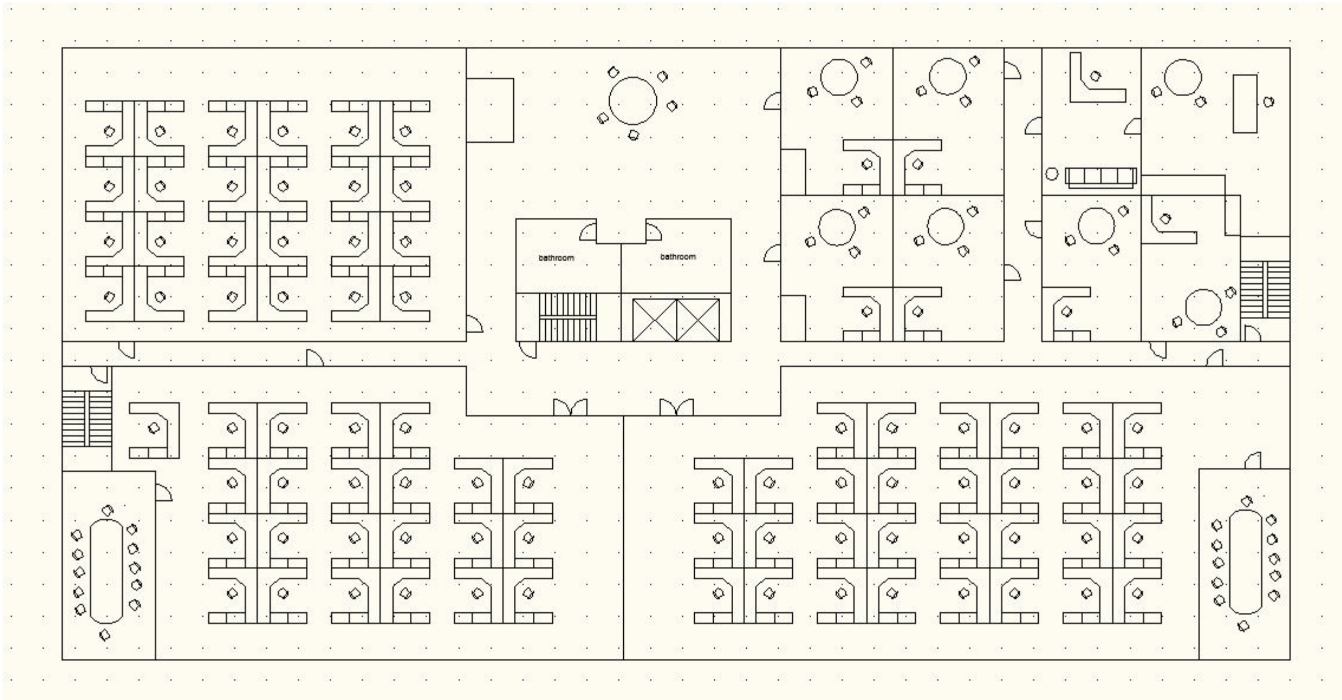


Figure 3 The 2nd to 4th Floor Plan

4.3 Floor Plan Revisions

4.3.1 Introduction

The first floor plan created a base for the team to design the steel structural system. However, it is not a sophistic design. In fact, there was a lot needed to be changed based on IBC. The revisions were to modify the floor plans to make them be compliant with the means of egress requirements in IBC for both sprinklered and non-sprinklered building. Furthermore, first floor was modified based on the 2nd to 4th floor plans to make all the floor plans had higher degree of uniformity. This uniformity reduced the work load in code analyses and sprinkler system designs in future sections. Since the team developed the structural system layouts or the typical bay sizes based on the footprint of this building, the locations of columns became problematical in the first sets of floor plans. With this in mind, the team also focused on how well floor plans would interact with structural systems in floor plan revisions. (i.e., the modification of locations of columns).

4.3.2 Means of Egress

With As shown in the Figure 3 for 2nd to 4th floor from previous section, the doors beside the center corridor are big obstructions and this also violated the IBC specifications about door encroachment. Hence, all doors had been modified to minimize the obstruction. Furthermore, the dead end corridor violated the length limit of 20 ft for a non-sprinklered building. Therefore, a door was added at the conjunction of "old" dead end corridor and the center corridor and thus there was no dead end corridor in the second attempt floor plans. In terms of doors, the distance between two exit doors of a room was not considered in the first attempt. The locations of two exit doors of a room also were modified to meet the remoteness requirements with a distance of ½ of maximum diagonal distance of the room.

4.3.3 Structural Systems and Floor Plans

It had been noticed that the column locations of structural systems did not cooperate well with the floor plans which means there would be a lot of change in structural design if many column location has be changed. For example, the 25’ X 25’ bay size structural systems had a row of columns located at the center corridor with only 4ft width. And in both 20’ X 20’ and 25’ X 25’ bay size structural systems, there were too many columns were exposed without being hidden in the interior wall. This reduced the net useable area a lot. Therefore, in order to make sure the floor plans and structural systems can both works well, the center corridor had been widened into 10ft.

The final floor plans are shown in Figure 4 and Figure 5.

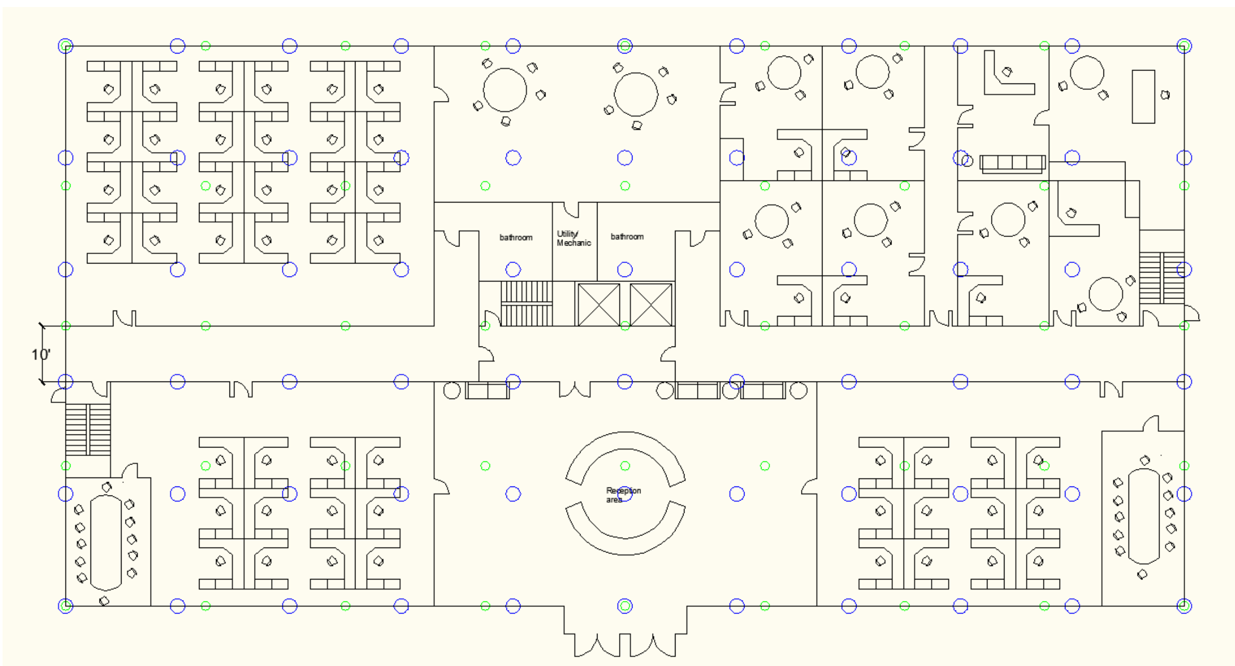


Figure 4 First Floor Plans (Blue circle represent column locations of 20'X20' bay size system; Green circle represent column locations of 25'X25' bay size system)

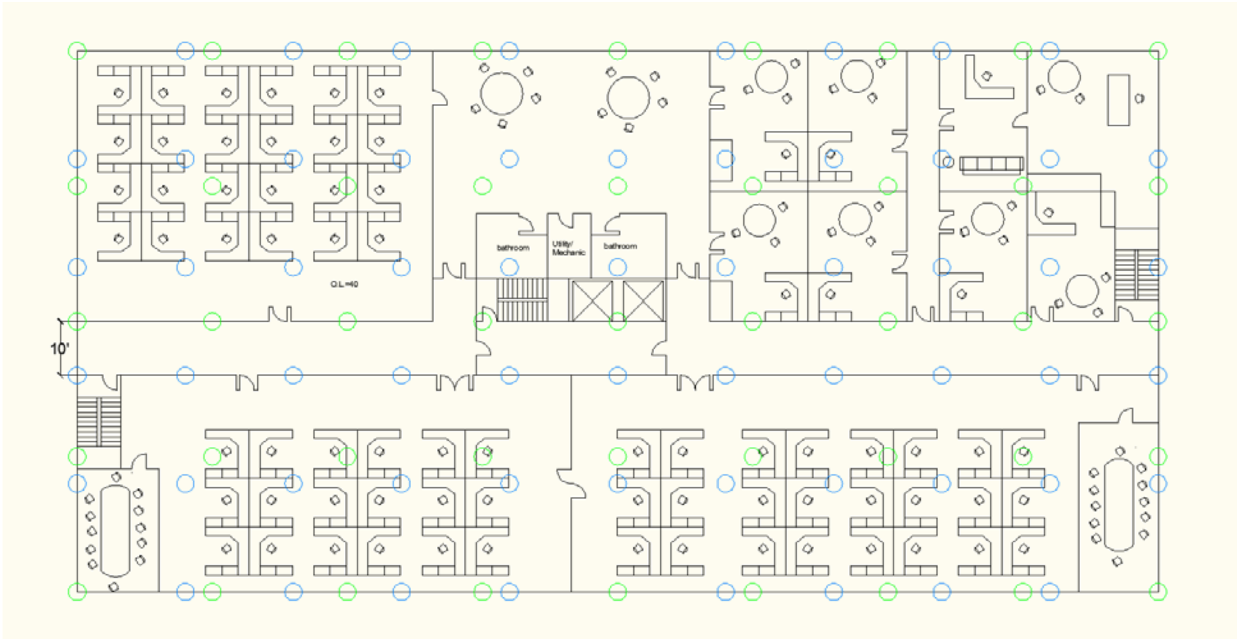


Figure 5 Second to Fourth Floor Plans (Blue circle represent column locations of 20'X20' bay size system; Green circle represent column locations of 25'X25' bay size system)

Chapter 5: Steel Structural Design

5.1 Introduction

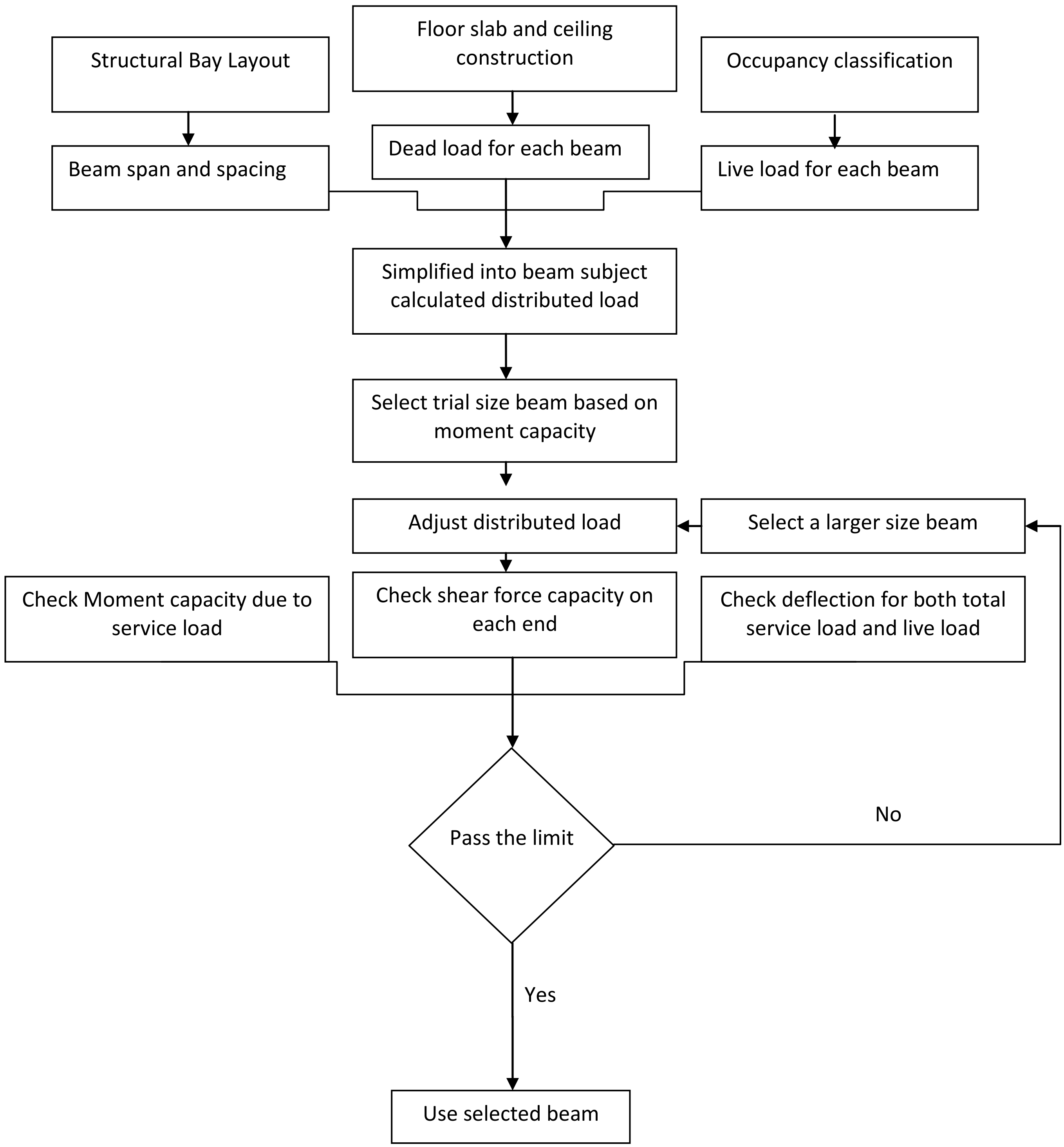
In this section, investigation of various structural systems are presented including W-shape sections and open web joists (OWJ) as infill beams, composite beam-and-slab construction and non-composite beam-and-slab construction, and composite columns (W shape steel member enclosed in concrete) and non-composite columns (exposed W shape steel member). The team examined the differences between different spans and different spacing. In order to minimize the bay size changes for an entire floor, repetitive bay sizes were selected. The designs of a typical bay size were 20 ft X 20 ft and 25 ft X 25 ft, which are evenly distributed across the footprint of the office building. Furthermore, for a W-shape beam, 3 different filler beam spacings (4 ft, 5 ft, and 10 ft) were assigned for the 20 ft X 20 ft bay while one filler beam spacing of 5 ft on center was selected for the 25 ft X 25 ft bay size. The different spacing in the 20 ft X 20 ft bay helped the team to identify the difference in spacing and the 5 ft spacing for 25 ft X 25 ft would examine the different spans. However, for open web joist (OWJ) construction, the team could not follow the same spacings as for the W-shape beams due to the load capacity. Therefore, 2 ft spacing for the 20 ft X 20ft bay and 2.5 ft spacing for the 25 ft X 25 ft was selected while girders were kept as W shape steel members. The composite construction and non-composite construction were investigated based on W-shape beam-slab construction and column construction.

All of the different structural system designs were compared based on construction cost, constructability and other various architectural concerns, such as floor depth, and net usable area. Based on the comparisons, an overall evaluation for each structural system design was conducted and the recommended structural system was selected. Since columns shall not be located in stairways and elevators, the typical bay could not satisfy all design requirements. Changes of the structural bay around the elevator and stairways needed to be made to conform to actual situation. Based on the selected structural construction and changes around those special areas, the team finalized the structural system for the whole building.

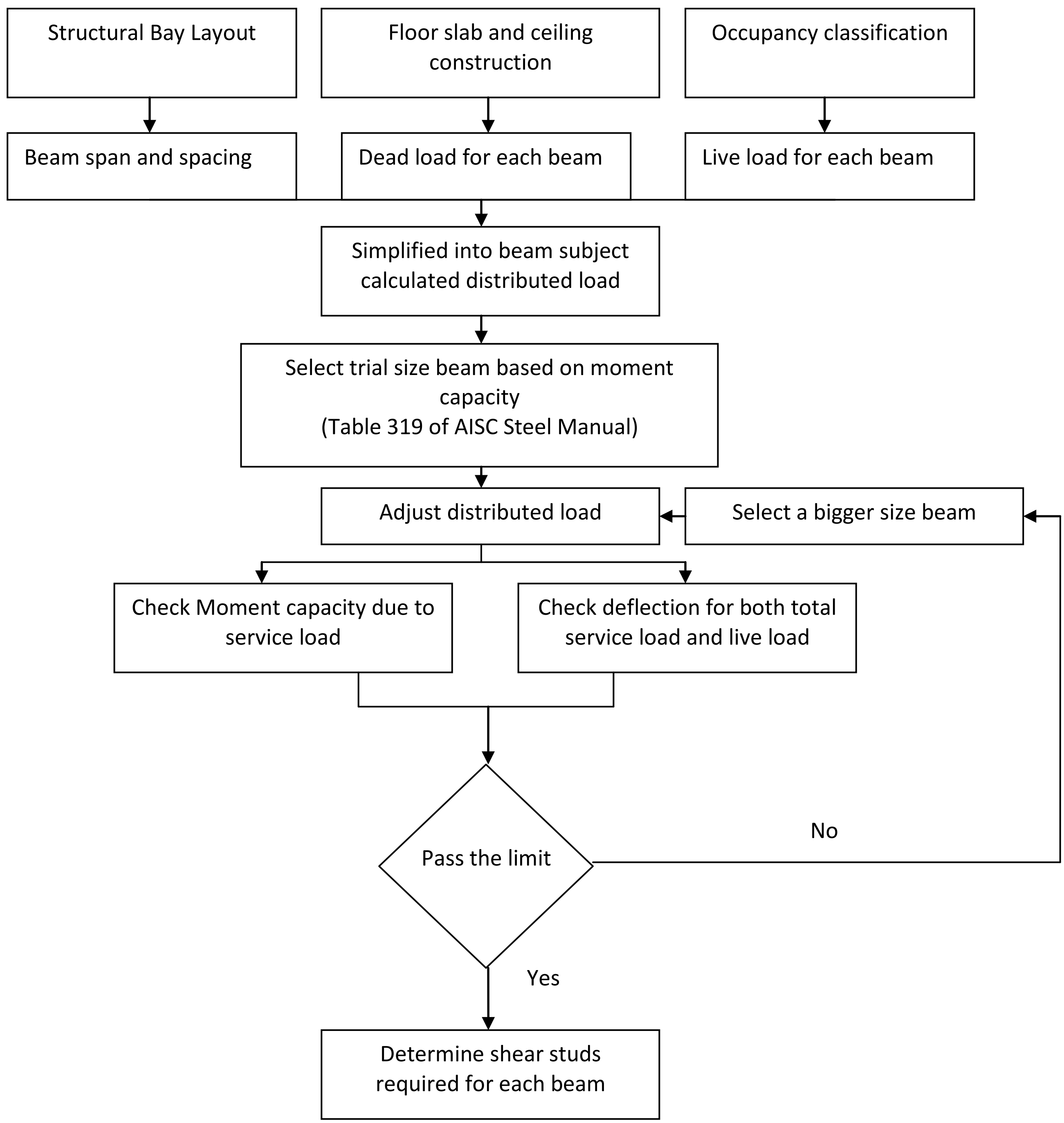
5.2 Steel Structural Design methodologies

In this section, a design flow-chart is provided for each structural design process. They are: W-shape non-composite beam and girder design; W-shape composite beam and girder design; W-shape column design; W-shape composite column design (concrete enclosed); Open web joist design; and one way slab design. A detailed written summary about the design process can be found in Appendix B.

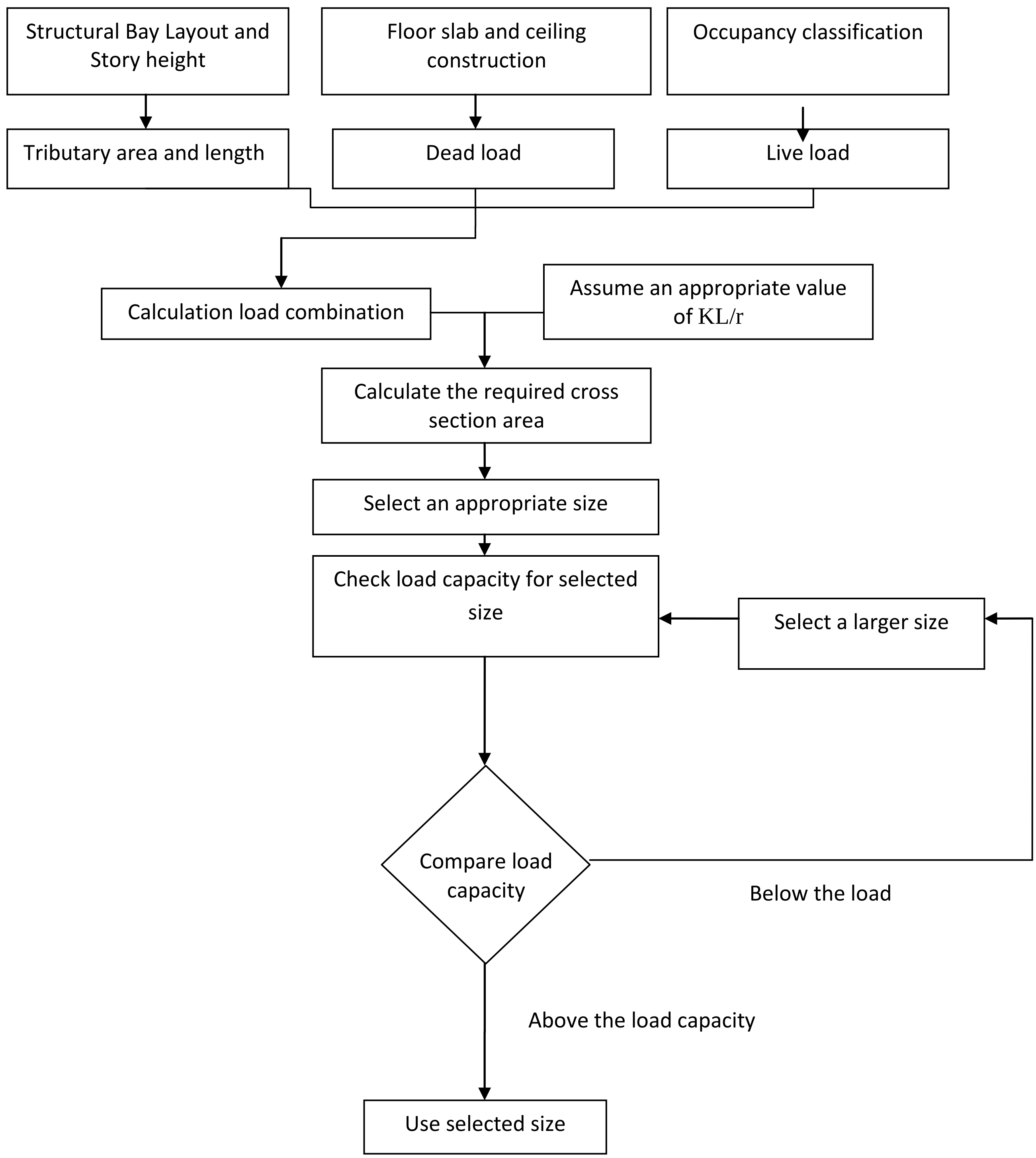
5.2.1 W-shape Non-Composite Beam and Girder Design



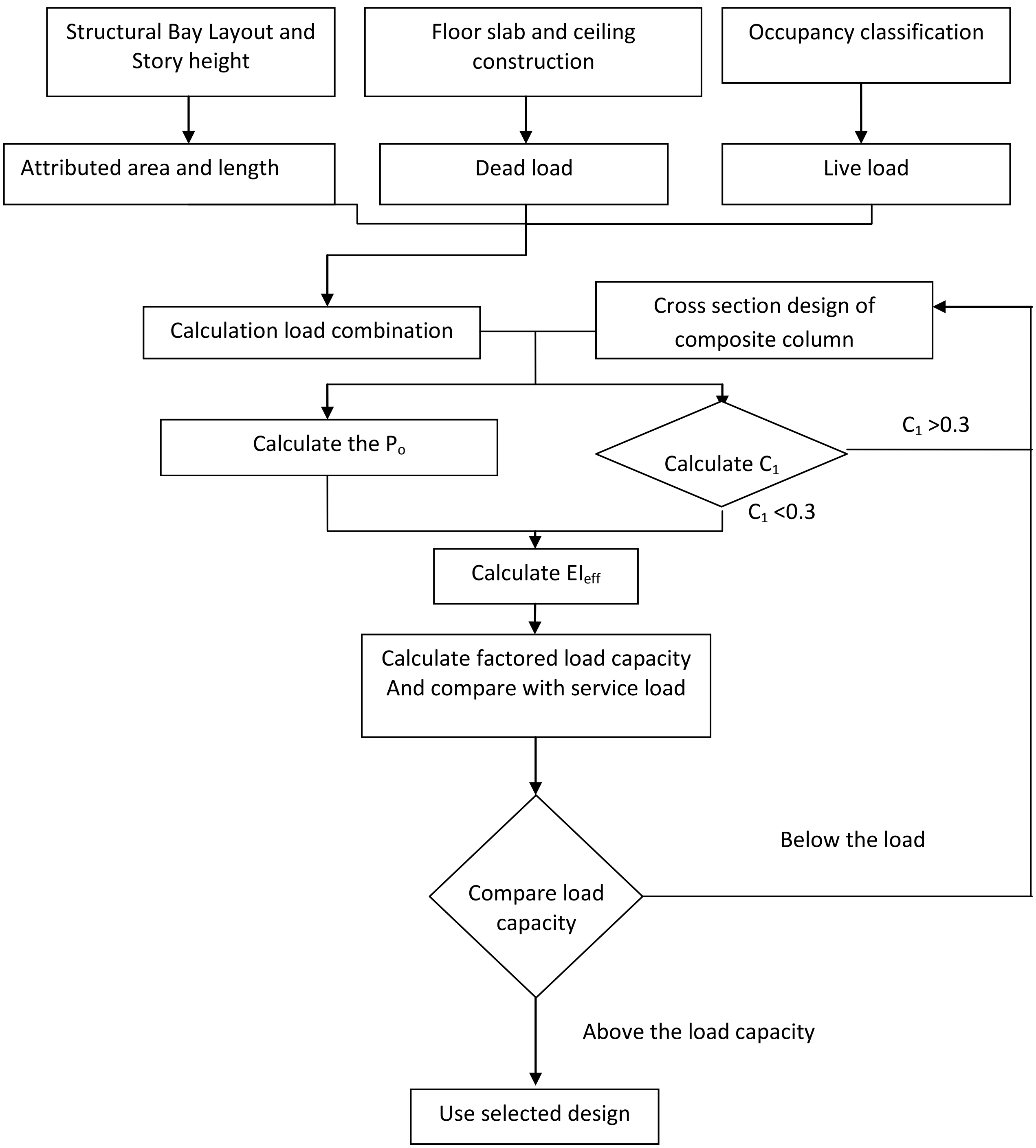
5.2.2 W-shape Composite Beam and Girder Design



5.2.3 W-shape Column Design

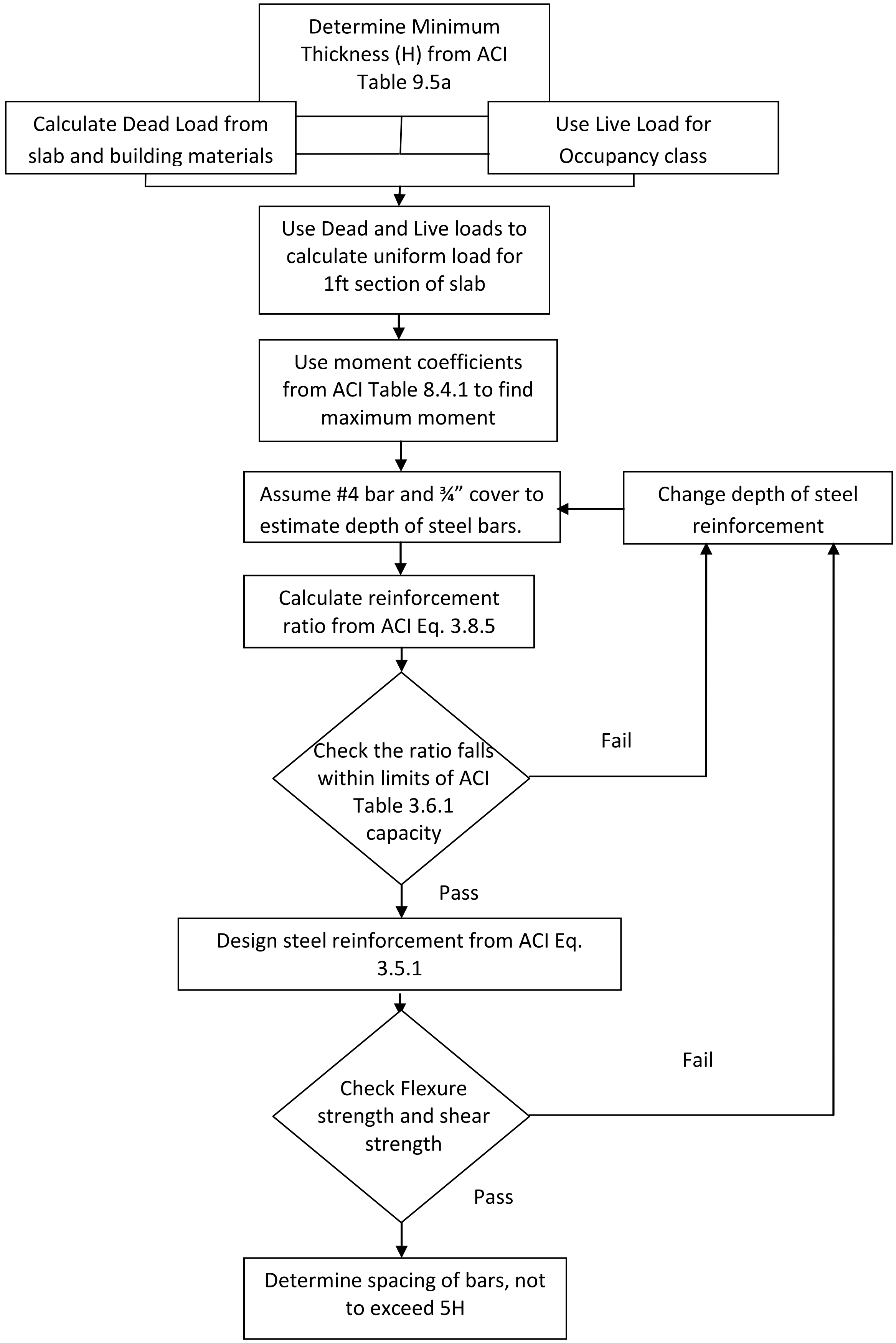


5.2.4 W-shape Composite Column Design (Concrete Enclosed)

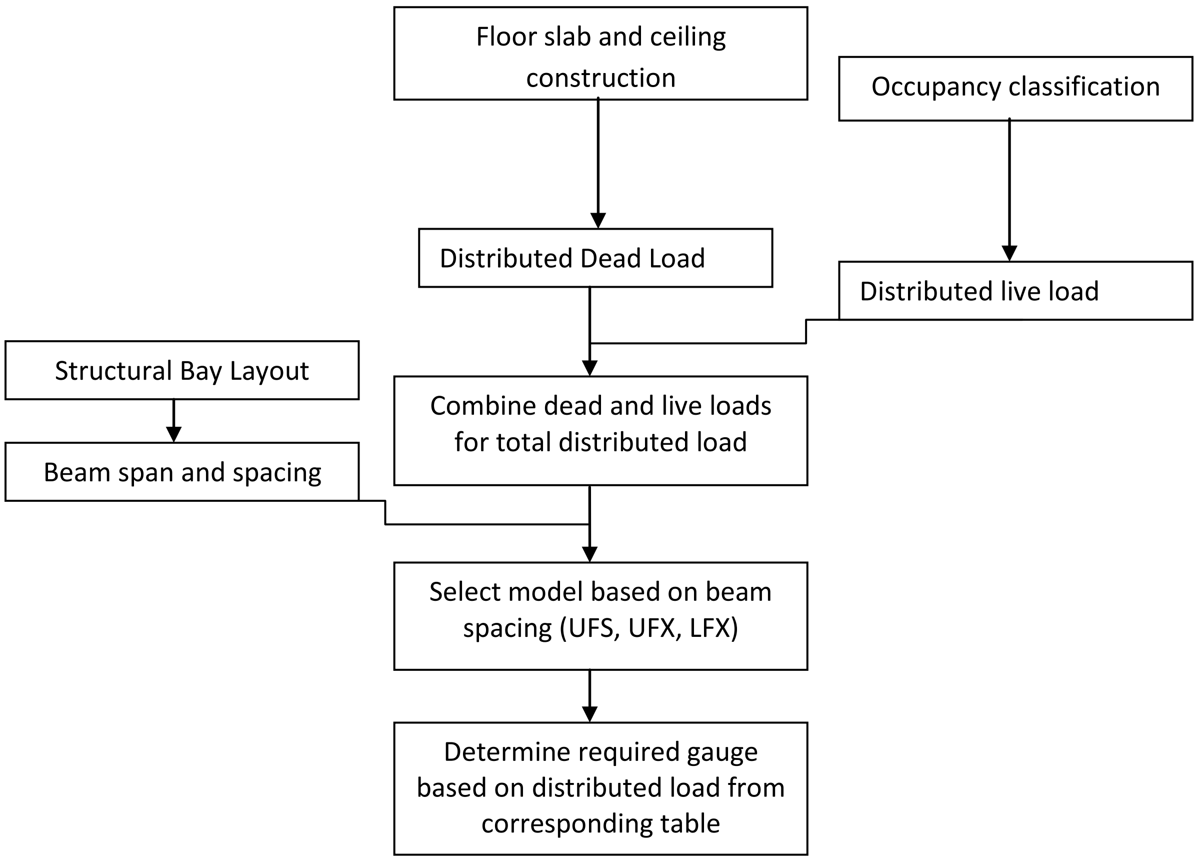


5.2.5 Open Web Joist (OWJ) Design

5.2. 6 One-way Slabs Design



5.2.7 Metal Decking Design



5.3 Cost Estimation and Evaluation Summary

After designing all possible structural members, we created a matrix which contains all of our possible design scenarios. This matrix is shown in Table 2.

Table 2 Construction Scenario Summary

Scenario ID #	OWJ/W Section	Comp/NonComp Columns	Comp/NonComp Beams and Girders	Bay Size (ft)	Beam Spacing (ft)
1A	W Section	Comp	Comp	20x20	4
2A	W Section	Comp	Comp	20x20	5
3A	W Section	Comp	Comp	20x20	10
4A	W Section	Comp	Comp	25x25	5
1B	W Section	Comp	NonComp	20x20	4
2B	W Section	Comp	NonComp	20x20	5
3B	W Section	Comp	NonComp	20x20	10
4B	W Section	Comp	NonComp	25x25	5
1C	W Section	NonComp	Comp	20x20	4
2C	W Section	NonComp	Comp	20x20	5
3C	W Section	NonComp	Comp	20x20	10
4C	W Section	NonComp	Comp	25x25	5
1D	W Section	NonComp	NonComp	20x20	4
2D	W Section	NonComp	NonComp	20x20	5
3D	W Section	NonComp	NonComp	20x20	10
4D	W Section	NonComp	NonComp	25x25	5
1E	OWJ	Comp	N/A	20x20	2
2E	OWJ	NonComp	N/A	20x20	2
3E	OWJ	Comp	N/A	25x25	2.5
4E	OWJ	NonComp	N/A	25x25	2.5

Each scenario is assigned an identification number which consists of two characters: first, a number (1 through 4) and second, a letter (A through E).

The cost of materials and construction needs to be determined for each scenario in order to make a design recommendation. Each structural member for each scenario needs to be identified and assigned a cost. The subtotal of costs for the members within a particular scenario are then summed and a final cost of materials and construction is determined. Below are the methods used to determine the costs for I Beam construction and open web joist construction.

5.3.1W Section and Open Web Joist Construction Cost Estimation

5.3.1.1 Floor Slab

For the slab designs depicted in Appendix C.5, the amount of concrete and metal decking determines. The amount of these materials is driven by the decking gauge which determines the slab thickness. Costs for these decks are specified in the *RS Means Building Construction Cost Data, 2009* and is determined by beam spacing, slab thickness and decking gauges. Below is a table taken from Appendix E that shows us the total cost of materials and construction for the slabs of all twenty design scenarios.

Slab and Metal decking cost 3 floors

20x20(Bay) Beam Spacing	Slab Depth	decking gauge	total cost
4'	3"	22	$395000
5'	3"	22	$395000
10'	5"	19	$462000
2'	2"	26	$266000
25x25(Bay) Beam Spacing	**Slab Depth**	**decking gauge**	**total cost**
5'	3"	22	$396000
2.5'	2"	22	$107000

5.3.2W Section Construction – Cost Estimation

5.3.2.1 Structural Bays

First we must identify the members used in our non-composite structural bays for both the 20’x20’ bay and the 25’x25’ bay for the three floors and the roof. For the 25’x25’ bay, there is only one set of members because there is only one beam spacing distance for that bay (5 ft). The 20’x20’ bay, on the other hand, has three distances for beam spacing (4 ft, 5 ft, 10 ft) and therefore has three different sets of structural bay members. Once, the total number of each structural member was determined for each of the four sets of members (one 25’x25’ member set and three 20’x20’ member sets), the costs of these members was determined with the *RS Means Building Construction Cost Data, 2009* using data sheet *05 12 23.75*, which gave us the cost of each member per linear foot. So by determining how many linear feet of each member is needed for a particular bay size and beam spacing, the cost of each type of member can easily be determined. By adding the costs of these members together, the cost of materials and construction for a particular bay size and beam spacing is calculated.

The same procedure is done for determining the costs of materials and construction for composite structural bays. Again, we have four different member sets: one 25’x25’ member set and three 20’x20’ member sets. After determining the lengths of each type of composite member used in the composite member sets, the costs for materials and construction of these members can be determined again by using the *RS Means Building Construction Cost Data, 2009*, data sheet *05 12 23.75*. Because shear studs are needed to attach the steel members to the metal decking, the costs of all shear studs must also be determined in order to calculate the complete cost of materials and construction for the composite structural bays.

Below is a table taken from Appendix E that shows the costs (to the nearest whole dollar) of materials and constructions for the above mentioned structural bays for the various bay sizes and beam spacing for both composite and non-composite construction.

Cost for 3 Floors and Roof

Bay Size=20'x20'	Composite	Non Composite
4' spacing	$1239600	$1911500
5' spacing	$1057600	$1452900
10' spacing	$772300	$1120600
Bay Size=25x25		
5' spacing	$1443000	$1613000

5.3.2.2 Columns

The methods used for determining costs for composite and non-composite columns is similar but different from the methods used to determine the costs of the beams and girders. First we estimated the costs for **non-composite columns**. But because the structural bay had a direct effect on the sizes and number of columns, we must investigate the costs for non-composite columns for the following scenarios: 20’x20’ composite bays, 20’x20’ non-composite bays, 25’x25’ composite bays and 25’x25’ non-composite bays. For each scenario, the number of each size of column is determined and then a total length of that column is calculated. For example, for a 20’x20’ non-composite bay, there are a total of 22 W8x24 columns; 18 of which are second floor exterior columns and 4 of which are first floor corner columns. Columns located on the second, third and fourth floor have lengths of ten feet whereas columns located on the first floor have lengths of 12 feet. So for the W8x24 columns, we have a total length of steel of 228 feet (4 x 12’ + 18 x 10’ = 228’). The total lengths of the non-composite columns are then multiplied by the price per linear foot of those members. This price data is found in the *RS Means Building Construction Cost Data, 2009* on data sheet number *05 12 23.75*. By adding these costs together, we calculate the cost of construction and materials for all of the columns of a particular scenario. Below is a table taken from Appendix E, which shows the costs (to the nearest whole dollar) of materials and construction for columns for each of the four non-composite column scenarios. As you can see, a 25’x25’ non-composite bay is the most cost efficient bay design for non-composite columns.

Cost for Non Composite Columns

Baysize	20'x20'	25'x25'
Composite	$170000	$140000
Non Composite	$169000	$135000

The methods used for calculating costs for **composite columns** are almost the exact same as calculating costs for non-composite columns. Like non-composite columns cost, we must investigate the costs of composite columns for four different scenarios: 20’x20’ composite bays, 20’x20’ non-composite bays, 25’x25’ composite bays and 25’x25’ non-composite bays. For each scenario, the number of each size of column is determined and then a total length of that column is determined; just like in the example given in the previous paragraph. The total lengths of each member type need to be multiplied by the price per linear foot in order to calculate the cost of steel for that particular member for that particular scenario. However, unlike the cost estimation of non-composite members, we are not done here. We must also take into account the cost of the concrete encasings. The total lengths of each member for each of the four scenarios needs to be multiplied by the cross-sectional area of the concrete for that particular member in order to determine the volume of concrete needed for each type of column. The price per cubic yard of concrete (found from *RS Means Building Construction Cost Data*, *2009*) is then multiplied by the volume of concrete for each total column length for each scenario, which gave us the price of concrete of each composite column for each of the four scenarios. For example, for a 25'x25' composite bay, there are 20 W8x18 columns needed for the first floor. Because the columns on the first floor are 12 feet long, we have a total length of 240 feet for the 20 W8x18 columns. By multiplying the 240 feet of column by the price per linear foot (in this case $42.50), we calculate the cost of the steel for that particular member for that particular scenario. By multiplying the cross-sectional area of concrete for the composite W8x18 column (which in this case is 90.74 sq in.) by the total length of the column (240 feet), we calculate the volume of concrete for that column of that scenario. When we add up all of our necessary composite column concrete volumes, we can multiply that total volume of concrete by the price of concrete per volume. This gave us our total cost of concrete which we can add to our total cost of steel for the columns; this sum equaled our total cost of construction and materials for composite columns in a 25'x25' composite bay. This process is will be repeated for the other three composite column scenarios. Below is a table taken from Appendix E; the table shows the costs (to the nearest whole dollar) of materials and construction for columns for each of the four composite column scenarios. As you can see, a 25'x25' non-composite bay is the most cost efficient bay design for non-composite columns.

Cost for Composite Columns

Baysize	20'x20'	25'x25'
Composite	$148000	$126000
Non Composite	$148000	$120000

5.3.2.3 Complete Cost

Once we have determined the costs of the slab and metal decking, the composite and non-composite beams and girders, and the columns (for each of the bay scenarios), we can determine the costs for each of the 16 W Section design scenarios: 1A, 2A, 3A, 4A, 1B, 2B, 3B, 4B, 1C, 2C, 3C, 4C, 1D, 2D, 3D and 4D. Below are two charts taken from Appendix E that show the total costs of materials and construction for the complete structural frame (bay and columns) and slab for the W Section designs. One is for composite columns and the other is for non-composite columns.

Total Structural Framing and Slab Cost (Non Composite Columns)

Bay Size=20'x20'	Composite	Non Composite
4' spacing	$1805000	$2476000
5' spacing	$1623000	$2017000
10' spacing	$1405000	$1752000
Bay Size=25x25		
5' spacing	$1979000	$2144000

Total Structural Framing and Slab Cost (Composite Columns)

Bay Size=20'x20'	Composite	Non Composite
4' spacing	$1783000	$1783000
5' spacing	$1601000	$1601000
10' spacing	$1383000	$1731000
Bay Size=25x25		
5' spacing	$1966000	$2128000

5.3.3 Open Web Joist Construction – Cost Estimation

The same fundamental methods are used to quantify the costs of open web joist construction that were used to quantify the costs of W-Section construction.

5.3.3.1 Structural Bays

First we must identify the members used in our **open web joist structural bays** for both the 20’x20’ bay and the 25’x25’ bay for the three floors and the roof. For the 25’x25’ bay, there is only one set of members because there is only one beam spacing distance for that bay: 2.5 ft. The same goes for the 20’x20’ bay, which has a beam spacing of 2’.

Once, the total number of each structural member was determined for each set of joists (one 25’x25’ member set and one 20’x20’ member set), the costs of these members was determined with the *RS Means Building Construction Cost Data, 2009* using data sheet *05 21 19.10* which gave us the cost of each member per linear foot. So by determining how many linear feet of each joist is needed for a particular bay size, the cost of each type of member can easily be determined.

By adding the costs of materials and construction of these members together, the total cost of materials and construction for a particular bay size is calculated.

5.3.3.2 Columns

The columns used for the open web joist construction are the same as the columns used for the W Section construction. However, composite decking was not designed for open web joist construction like it was for the W Sections, only non-composite bays would be implemented for open web joists. Determining the costs of the columns for the open web joist, non-composite floor system is done the same way that it is done for W Section, non-composite floor systems. Below is a table depicting the costs for composite and non-composite columns for both non-composite bays (20’x20’ and 25’x25’).

Cost for Non Composite & Composite Columns (Open Web Joists)

Non-Composite Bay Size	20'x20'	25'x25'
Composite Columns	$148000	$120000
Non Composite Columns	$169000	$135000

This table shows that a 25’x25’ structural bay provided less costly column materials and column construction for the building than the 20’x20’ structural bay would provide.

5.3.3.3 Complete Cost

Once we have determined the costs of the slab and metal decking, the joist beams and girders, and the columns (for each of the bay scenarios), we can determine the costs for each of the 4 open web joist design scenarios: 1E, 1F, 4E and 4F. Below is a chart that shows the total costs of materials and construction for the complete structural frame (bay and columns) and slab for the four open web joist scenarios.

Complete Costs for Open Web Joists Construction

Non-Composite Bay Size	20'x20'	25'x25'
Composite Columns	$1391000	$1239000
Non Composite Columns	$1412000	$1255000

As one can clearly tell, open web joist bays with composite columns are more cost efficient than structural bays with non-composite columns.

5.3.4 Cost Evaluation – All W Section and Open Web Joist Design Scenarios

Table below shows us again all of our twenty design scenarios and now with their respective complete costs of material and construction and their cost's rank amongst the other design scenarios.

Table 3 Cost for Each Structural System Scenario

Scenario	OWJ/Normal	Comp/NonComp Columns	Comp/NonComp Beams	Bay Size (ft)	Beam Spacing (ft)	Cost	Cost Rank
1A	Normal	Comp	Comp	20x20	4	$ 1,782,951.80	12
2A	Normal	Comp	Comp	20x20	5	$ 1,600,951.80	7
3A	Normal	Comp	Comp	20x20	10	$ 1,382,520.00	3
4A	Normal	Comp	Comp	25x25	5	$ 1,965,608.20	15
1B	Normal	Comp	NonComp	20x20	4	$ 1,782,951.80	12
2B	Normal	Comp	NonComp	20x20	5	$ 1,600,951.80	7
3B	Normal	Comp	NonComp	20x20	10	$ 1,730,819.70	10
4B	Normal	Comp	NonComp	25x25	5	$ 2,128,239.20	18
1C	Normal	NonComp	Comp	20x20	4	$ 1,805,398.80	14
2C	Normal	NonComp	Comp	20x20	5	$ 1,623,398.80	9
3C	Normal	NonComp	Comp	20x20	10	$ 1,404,966.70	5
4C	Normal	NonComp	Comp	25x25	5	$ 1,979,220.20	16
1D	Normal	NonComp	NonComp	20x20	4	$ 2,475,794.80	20
2D	Normal	NonComp	NonComp	20x20	5	$ 2,017,194.80	17
3D	Normal	NonComp	NonComp	20x20	10	$ 1,751,762.70	11
4D	Normal	NonComp	NonComp	25x25	5	$ 2,143,755.20	19
1E	OWJ	Comp	N/A	20x20	2	$ 1,390,906.22	4
2E	OWJ	NonComp	N/A	20x20	2	$ 1,411,849.22	6
3E	OWJ	Comp	N/A	25x25	2.5	$ 1,239,350.50	1
4E	OWJ	NonComp	N/A	25x25	2.5	$ 1,254,866.50	2

If we were to select a design scenario based on cost alone, then we would have the information necessary to so. But before making a design recommendation, we investigated constructability and net usable area of each of the design scenarios.

5.4 Construction Summaries

5.4.1 Composite Beams and Girders

Composite beam-slab systems offer many advantages that cannot be achieved with non-composite construction. Beams and slabs act as one unit when resisting loads and can support 33 to 50 percent more load than the same non-composite section. The bond between concrete and steel can be achieved in three ways. The steel beam can be encased in concrete to create a bond and friction between the two; however this technique is rarely used due to its cost. A more economical method is through the use of shear connectors that span the beam to provide sufficient shear transfer between the concrete and steel beam. However, formed steel deck is used in almost all composite construction nowadays.

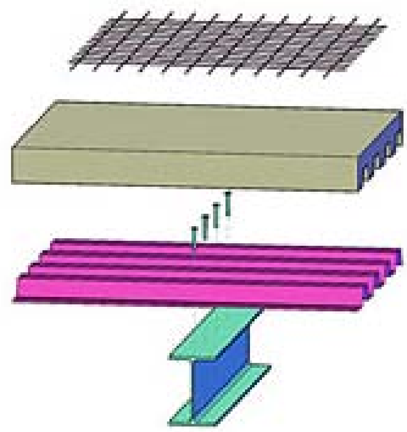


Figure 6 Composite beam construction

Not all composite sections are the same. They may be either full composite or partial composite sections. A full composite section develops the full strength of the composite beam. The bond between the concrete and steel is sufficient to provide the full shear transfer. This places the plastic neutral axis in the slab and allows the entire steel section to yield during bending. A partial composite section cannot develop the full strength of the composite beam. It only has enough shear connectors to transfer some of the shear to the beam. This will result in a plastic neutral axis that is in the steel section, the steel now acquires some of the compressive forces, therefore only a portion of the steel section can yield during bending.

In terms of constructability of composite beams and girders, it is efficient in high rise structures.

1. Lighter steel sections can be used to provide sufficient strength.
2. Steel decking eliminates the need for shoring of the concrete slab.
3. The use of steel decking and lighter section reduces overall cost of materials and labor.
4. Time is saved during erection and concrete prep work due to the lighter sections and elimination of shoring.

5.4.2 Composite Columns

Composite columns are categorized into two types: 1) rolled or built-up steel structural sections encased in concrete; 2) concrete placed in HSS or pipe sections. Examples of cross section are shown below in figure below.

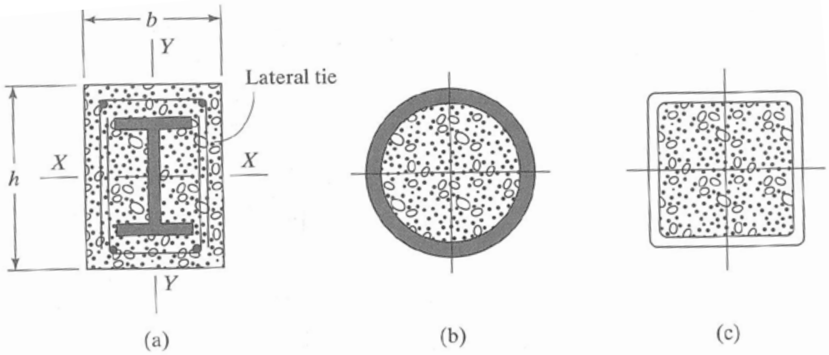


FIGURE 17.1
Composite columns.

Figure 7 Cross-section view of some sample composite columns (McCormac, 2007)

Since fire protection is one of the major concerns in this project, the team decided to investigate the W-shape embedded in concrete composite columns. The concrete enclosed the steel section can enhance the fire proof ability. Since concrete is essentially a complex chemical compound with water, the fire is less likely to affect the steel section inside the concrete due to the water vapor formed by high temperature. Comparing to the expensive fire proof spray and other temperature inert materials to protect steel member, concrete would be a batter choice. Despites the fire proof property provide by concrete, there are many other advantages of composite columns. In high rise building, the sizes of composite columns often are smaller than reinforced-concrete columns which can support the same load. And it will save a lot valuable floor area. In low rise building, such as warehouses and parking garages, concrete can protect the steel section from corrosion and vehicles (garages). Since concrete is a very strong material for compression, the composite columns usually need smaller steel member which can be observed in our designs. Furthermore, the composite columns can easily limit the swaying or lateral deflections and hence improve the stability o f whole structural system from lateral forces.

In terms of constructability of composite column, it is efficient in high rise structures. Following are the brief descriptions and a diagram below is a good illustration:

1. One group of workers is erecting the steel beams and columns for one or two stories on top of the frame.
2. Two or three stories below, another group can set the metal decking for the floors at the same time.
3. A few stories below, another group of workers can place the concrete for the floor slab.
4. Same circle can continue as we go down the building, with one group of workers are working on the column reinforcing bars in cages, while below them another group is

placing the column forms, and placing the column concrete.

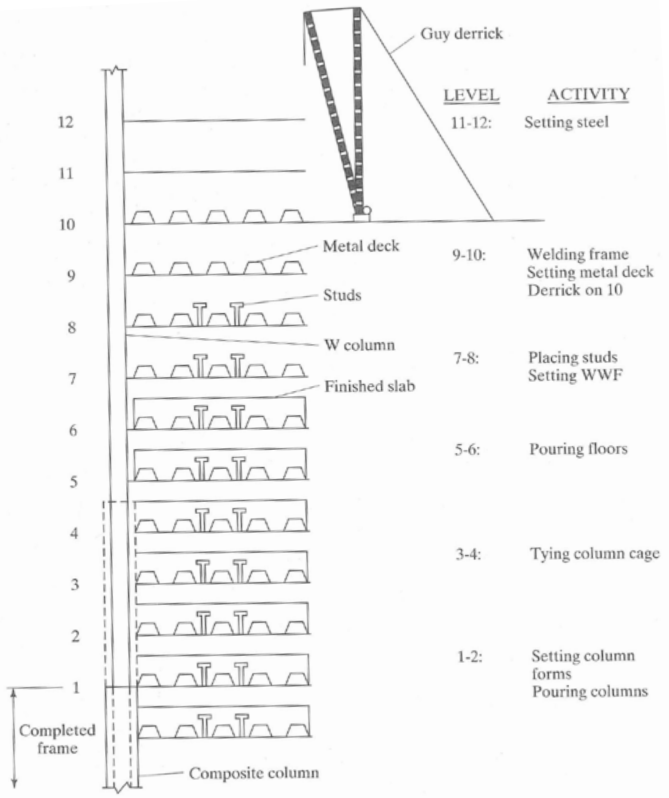


FIGURE 17.2
Sequence of construction operations for a composite building frame. (Courtesy of AISC.)

Figure 8 Composite column construction (McCormac, 2007)

Although composite columns have many advantages, there are also some disadvantages. For instance, we are still lack of knowledge on the mechanical bond between the concrete and the steel sections. And for high rising structures, it is very difficult to control the rates and amount of shortening due to the load, in relation to shear walls and adjacent plain steel columns. Another problem would be the creep which may cause unlevel concrete floor. (McCormac, 2008)

5.4.3 Open web joist

Open web joists are well suited for relatively light weight loads and structures without too much vibration. The common applications are schools, apartment houses, hotels, office buildings, and restaurant buildings. While open web joists can carry the distribute load well, it is not suitable for supporting concentrated loads, unless it is especially detailed to carry such loads.

There are some merits of open web joist construction. First of all, they are easy to handle and they are quickly erected. Another advantage is that open web joist construction has much less tonnage compared to W-shape steel structures. This means less cost. (McCormac, 2008)

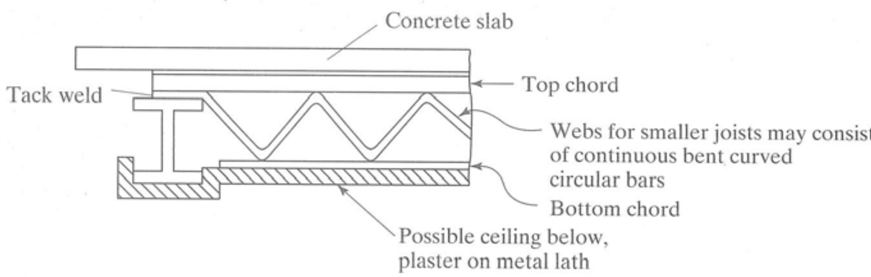


FIGURE 19.5

Open-web joists.

Figure 9 Side view of OWJ construction (McCormac, 2007)

Chapter 6: Structural Scheme Selection

6.1 Introduction

The purpose of this chapter is to show how our team made a final decision as to which structural scheme to implement in our structural design. There are several different scheme scenarios all with their own specific pros and cons. By analyzing and weighing these pros and cons, we can determine the scheme best suited for the needs of the owner.

6.2 Scenario Identification

We have designed twenty different structural schemes for the four-story office building. The structural scenarios vary in type of construction, materials, girder size, bay size, beam size and beam spacing. The table below depicts each scenario with all of its corresponding design specifications. The scenarios are generally very similar to one another, however, each is unique. Each scenario is labeled with a two-character identification number consisting of a number, “1” through “4”, and a letter, “A” through “E”.

Table 4 Design Scenarios

Scenario	OWJ/Normal	Comp/NonComp Columns	Comp/NonComp Beams	Bay Size (ft)	Beam Spacing (ft)
1A	Normal	Comp	Comp	20x20	4
2A	Normal	Comp	Comp	20x20	5
3A	Normal	Comp	Comp	20x20	10
4A	Normal	Comp	Comp	25x25	5
1B	Normal	Comp	NonComp	20x20	4
2B	Normal	Comp	NonComp	20x20	5
3B	Normal	Comp	NonComp	20x20	10
4B	Normal	Comp	NonComp	25x25	5
1C	Normal	NonComp	Comp	20x20	4
2C	Normal	NonComp	Comp	20x20	5
3C	Normal	NonComp	Comp	20x20	10
4C	Normal	NonComp	Comp	25x25	5
1D	Normal	NonComp	NonComp	20x20	4
2D	Normal	NonComp	NonComp	20x20	5
3D	Normal	NonComp	NonComp	20x20	10
4D	Normal	NonComp	NonComp	25x25	5
1E	OWJ	Comp	N/A	20x20	2
2E	OWJ	NonComp	N/A	20x20	2
3E	OWJ	Comp	N/A	25x25	2.5
4E	OWJ	NonComp	N/A	25x25	2.5

6.3 Scheme Selection Criteria

In order to select the structural scheme best suited for the building, we analyzed the constructional, architectural and cost benefits of each of the scenarios. The construction criterion includes member uniformity, structural layout, footing design and structural fire protection. The architectural criteria are net usable area and floor depth. All of these criteria will affect the ease of construction, cost of construction and other architectural aspects. These criteria were evaluated for each scenario and assigned a particular score. The scores of each of the four criteria were combined to give a final score for each design scenario. These scores will then be normalized by dividing them by scores assigned to their particular costs. The scenario with the best final normalized score provides the best for the four-story office building.

Listed below are the descriptions of each criterion and the methods used for scoring the data.

6.3.1 Cost

The cost of construction materials and labor is a critical design selection criterion. Our structural scheme was decided mainly upon the complete cost of the building's construction. This cost varies for each of the structural scenarios. The cost of each scenario is shown in Chapter 3.3, *Cost Estimation and Evaluation Summary*. The cost of construction for each scenario is somewhere between 1 and 2.5 million dollars.

Scoring these costs was done by assigning each cost scenario a number between 0 and 100. A zero was assigned to any scenario that cost $2,500,000.00 or more; a 100 was assigned to any scenario that cost $1,000,000.00 or less. Scenarios were scored with respect to their percentile rank between these two boundaries. These scores were used to normalize the scores calculated from the architectural and constructional concerns.

6.3.2 Architectural Concerns

- <u>Usable Area</u>

Usable area refers to the total amount of area of all four stories that may be used by the occupants of the building. Each scenario's net usable area varied based on the size, location and types of columns used. Buildings with larger floor space can turn a larger profit than those with less floor space.

The total amount of floor area that columns took up was calculated for each scenario. External columns or other columns that were flush within the faces of a wall were ignored. This left us with varying amounts of internal columns of varying sizes. The floor areas of non-composite columns were calculated with the notion that each non-composite column would be encased with ½" gypsum board in order to create a smooth and square column.

The total areas of exposed internal columns were added up; totals were calculated and ranged from 30.8 to 64.1 square feet. These totals (in square feet) were used to give scores for this particular criterion. The team developed a score calculating formula for the net usable area for each scenario.

$$\text{Score} = 100 - A$$

Where:

A: Total area of exposed internal columns

- <u>Floor Depth</u>

  The depth of floors for each scenario also has an impact on the structural scheme selection. Floor depths vary for each scenario based on girder and slab depth. These floor spaces house the automatic sprinkler system, the HVAC system and several other mechanical, electrical and plumbing components. As floor depth increases, building height increases, which then requires more expenses for vertical walls, pipes, cables and other office building elements.

  Total floor depths for each scenario were determined by adding the slab thickness with the largest girder for each of the four floors. The total depths of four floors for each of the design scenarios ranged from 67.2 to 97 inches. We calculated scores for each scenario by assuming that each floor should be approximately 12 to 30 inches deep. So for a four story building, the total floor depth should be approximately 48 to 120 inches. Based on this, the team developed a score calculation formula for the total depth of all four floors for each scenario:

$$\text{Score} = (120 - d)/(120\text{-}48)*100$$

Where:

d: total floor depth in inches

This function of the total floor depth provides us with a score varying between 1 and 100 for the given floor depths. Naturally, the smaller the floor depth, the larger the score.

6.3.3 Construction Concerns

- Member Uniformity

  We believed that a less diverse set of members was better for the ease of the construction for the office building. Less variation in member sizes means increased standardization in fabrication and erection activities. Also, because of their uniform shape and size, identical members are easier to deliver to the construction site and easier to store at the construction site.

For each structural system scenario, structural members were identified to inventory all of the different steel member sizes. The percentage of a steel member to the total number of steel members was calculated. Pie charts that show the amount and percentage for each scenario are found later in this chapter. Since less variation in member size is more beneficial, a higher number of member sizes influences a lesser score. The higher percentage of one certain member to the total number of members influences a higher score. Based on these two aspects, the team developed a score calculation formula:

$$\text{Score} = (1\text{-}0.01*(N\text{-}1))*\{(P_1*100)+(0.1*[P_2/(P_1 - P_2)]*100)\}$$

Where:

N: Number of Member Size Variation

P_1: Highest Percentage

P_2: Second Highest Percentage

(Note: 1. the full score is 100 where structural members are in uniform size. 2. The fact that multiple members may be dominating members had to also be considered. In this case, the 1^{st} and 2^{nd} most used members were included.)

This function gives a score for each design scenario. The member uniformity scores for each scenario, with respect to the scores of the other scenarios, represent an accurate level of member uniformity. From the formula, one can see that as the number of different member sizes, "N", increases, the overall score will decrease. Also, as the first and second highest percentages of specific member sizes (P_1 and P_2) increase, the entire score will increase, per the formula.

- Structural Layout

This criterion is an evaluation of how well the structural plan cooperates with the floor plan. Since the atypical bay sizes were determined by the footprint of this building, no considerations or adjustments were made for the column locations around vertical shafts. It was necessary to establish the atypical bay sizes to minimize the design calculations for the steel structural members, and the repetition of same bay size will be beneficial for construction. However, additional structural design efforts were needed to conform the areas where vertical shafts were located.

In order to be compliant with the structural needs in the areas around the vertical shafts, many columns were modified (moved, removed and added). And due to the changes in column locations, additional girders or beams were designed in accordance with these changes. The adjusted structural layouts for each bay size can be seen below in Figures 10 and 11.

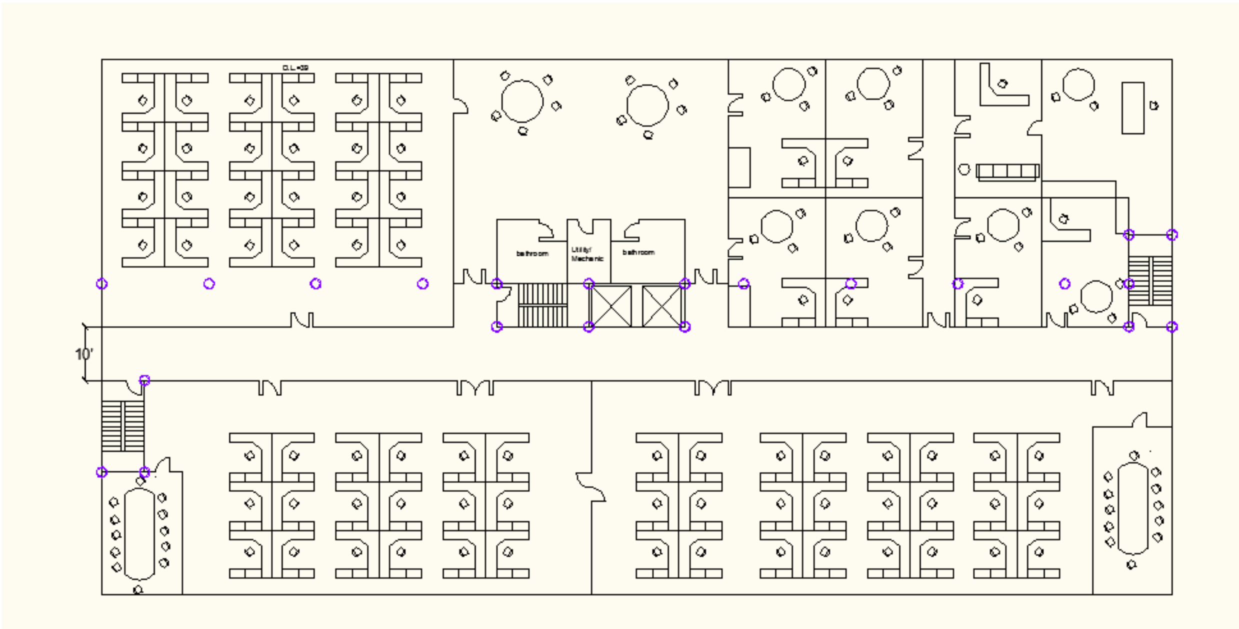


Figure 10 Column Modification for 20'X20' (columns are represented as purple circle)

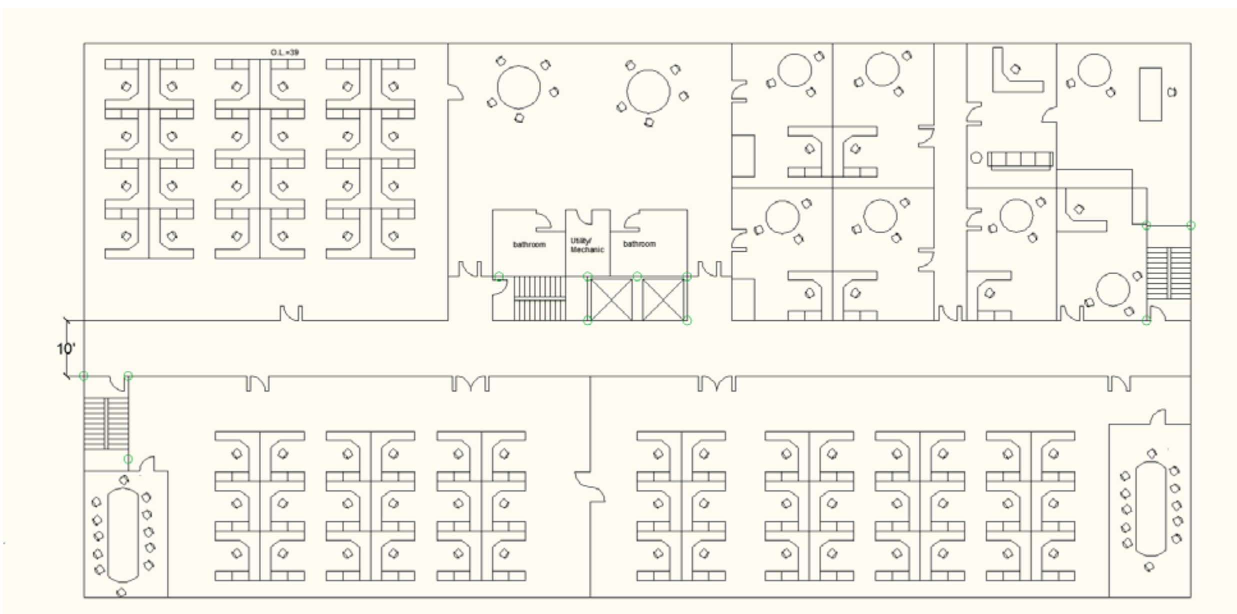


Figure 11 Column Modification for 25'X25' (columns are represented as green circle)

The team counted the number of columns needed to be modified per floor for both the 20’X20’ and 25’X25’ bay sizes. Less modification referred to a higher score. And the detailed information for scoring and results is shown below in the table below.

Table 5 Scoring for Structural Layout

Bay Size	Modified Column Number	Percentage of Modified Columns (Modified/Total)	Score Calculation	Score
20’X20’	22	22/66=33.3%	100-33.3=66.7	66.7
25’X25’	12	12/45=26.7%	100-26.7=73.3	73.3

- Footing Construction

Although this project did not include any footing design, in terms of construction, the number of footings is an important factor. The construction of footings involves site work, excavation and thus uncertainty. In this project, the number of footings was simplified as the number of columns on first floor.

The scoring was simply determined by the number of columns. As mentioned above, fewer columns means fewer footings and therefore less excavation and uncertainty. Thus fewer columns had a higher score. The score was simply assigned as 100 minus the number of columns for each scenario.

Table 6 Scoring for Footing

Bay Size	Column Number	Score Calculation	Score
20’X20’	66	100-66=34	34
25’X25’	45	100-45=55	55

- Structural Fire Protection

Except for composite columns, the steel members require fireproofing material due to their level of exposure. Steel members not encased in concrete are more susceptible to fire damage, which can cause structural failure and collapse under serious fire conditions. Some degree of protection is needed (i.e. fireproofing spray) and the area of fireproofing spray was used to evaluate this criterion for each of the design scenarios.

The total area of fireproofing spray was evaluated for the whole building. For beams and girders, the fireproofing spray shall cover the web and bottom flange, and the bottom surfaces of the metal floor decking as shown in the figure below. To calculate the total area required for fireproofing spray in the ceiling, the area of the exposed metal decking is added to the gross web and bottom flange areas for all beams and girders. For a wide-flange steel member, the area can be estimated by the cross-section perimeter (web depth, and flange width) and the member length which depends on the bay size. However, for an open web joist, it was difficult to obtain the exact cross-section perimeter. Therefore, the area was estimated by treating the web as solid web. And the second step was to multiply the area of the “solid web” by 20% considering the web has an 80% opening.

For columns, fireproofing spray shall cover the entire steel member unless the column is a composite column (enclosed by concrete). For an unexposed column, the total area of fireproofing spray needed can be calculated the same way area was calculated for a wide flange beam or girder.

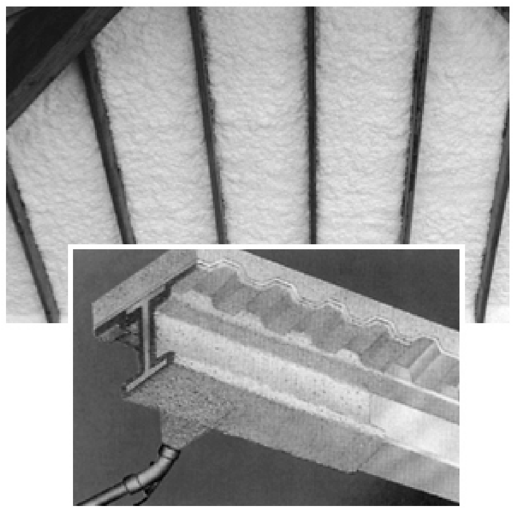


Figure 12 Fireproofing spray (Puchovsky, 2009)

The tables detailing the area of fire proofing spray for each of the design scenarios is shown in Appendix J.4. As shown in the appendix, the total areas of coverage were in the range of 80000 ft^2 to 140000 ft^2. Therefore the team defined the score criteria as a linear interpolation between these two values: at 80000 ft^2, the score is 100; and at 140000 ft^2, the score is 0. The formula to calculate the score is shown below:

$$Score = (140000-A)/(140000-80000)*100$$

Where:

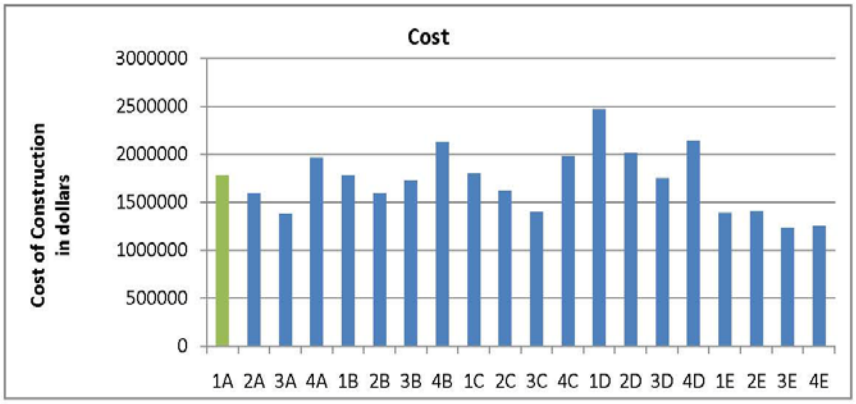
A is the total area of fireproofing spray for a given scenario.

6.4 Scenario Score Results

Score sheets for each of the design scenarios are presented on the following pages. These sheets contain the data analyzed for each of the criteria as well as the numerical scores given for each of these design criteria. Each sheet details the elements of the total score for a particular design scenario. On each sheet, there are three figures. The figures on the top of the score sheets are bar charts that help compare the total cost of construction for a particular design scheme with that for the other design schemes. The figures on the bottom left of the sheets are tables which summarize the types of members used for each scenario. The figures on the bottom left are pie charts that show the uniformity of member sizes throughout particular structural schemes. Also on these sheets are final normalized scores for each of the design scheme scenarios.

Scenario ID : **1A**

Cost Concerns: Total Cost of Construction: $ 1,782,951.80 **Score (used for normalization):** 53



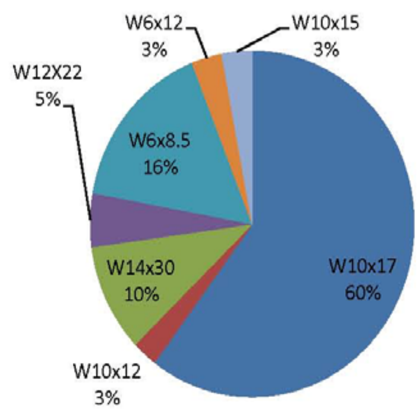
Architectural Concerns:

- Usable Area — Total Area of Exposed Internal Columns: 69.17 sq ft — **Score:** 30.8
- Floor Depth — Total Floor Depth of four floor systems: 67.2 in — **Score:** 73.3

Construction Concerns:

- Member Uniformity — **Score:** 59.8

Beams(floors & roof)		Interior	W10x17
		Exterior	W10x12
Gider(floors & roof)		Interior	W14x30
		Exterior	W12X22
Column	4th	Interior	8"x8" W6x8.5
		Exterior	8"x8" W6x8.5
		Corner	8"x8" W6x8.5
	3rd	Interior	8"x8" W6x8.5
		Exterior	8"x8" W6x8.5
		Corner	8"x8" W6x8.5
	2nd	Interior	10"x10" W6X12
		Exterior	8"x8" W6x8.5
		Corner	8"x8" W6x8.5
	1st	Interior	12"x12" W10x15
		Exterior	8"x8" W6x8.5
		Corner	8"x8" W6x8.5



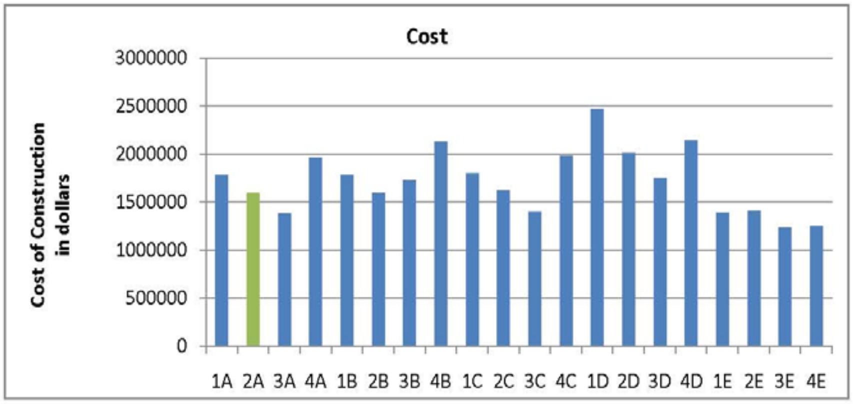
- Structural Layout — Bay Size: 20'x20' — **Score:** 66.7
- Footing Construction — Bay Size: 20'x20' — **Score:** 34
- Structural Fire Protection — Total Area of Exposed Steel Members: 106755.5 ft^2 — **Score:** 55.4

Total Score: 320

Normalized Total Score: **6.04**

Scenario ID : **2A**

Cost Concerns: Total Cost of Construction: $ 1,600,951.80 **Score (used for normalization):** 41



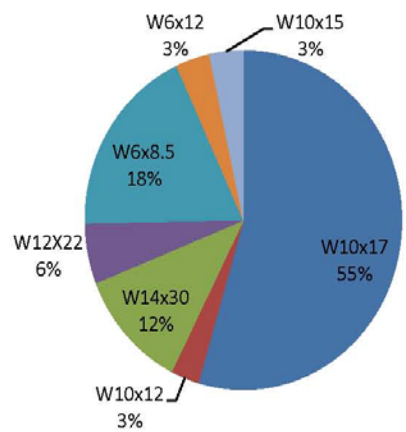
Architectural Concerns:

- Usable Area — Total Area of Exposed Internal Columns: 69.17 sq ft — **Score:** 30.8
- Floor Depth — Total Floor Depth of four floor systems: 67.2 in — **Score:** 73.3

Construction Concerns:

- Member Uniformity — **Score:** 56.3

Beams(floors & roof)		Interior	W10x17
		Exterior	W10x12
Gider(floors & roof)		Interior	W14x30
		Exterior	W12X22
Column	4th	Interior	8"x8" W6x8.5
		Exterior	8"x8" W6x8.5
		Corner	8"x8" W6x8.5
	3rd	Interior	8"x8" W6x8.5
		Exterior	8"x8" W6x8.5
		Corner	8"x8" W6x8.5
	2nd	Interior	10"x10" W6X12
		Exterior	8"x8" W6x8.5
		Corner	8"x8" W6x8.5
	1st	Interior	12"x12" W10x15
		Exterior	8"x8" W6x8.5
		Corner	8"x8" W6x8.5



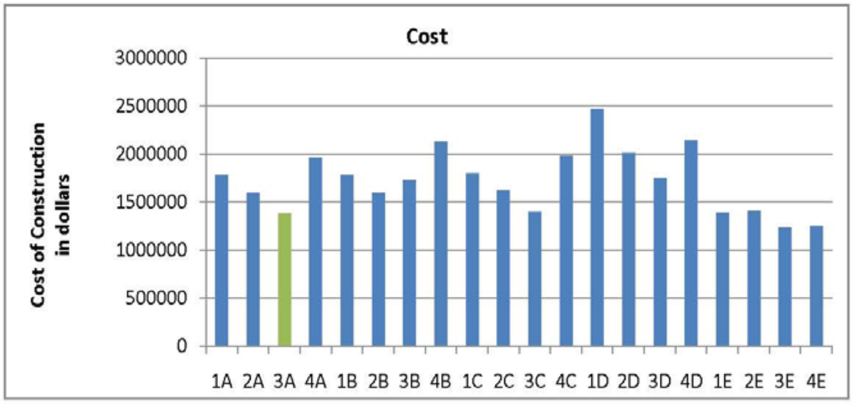
- Structural Layout — Bay Size: 20'x20' — **Score:** 66.7
- Footing Construction — Bay Size: 20'x20' — **Score:** 34
- Structural Fire Protection — Total Area of Exposed Steel Members: 99695.5 ft^2 — **Score:** 67.2

Total Score: 328.3

Normalized Total Score: **8.01**

Scenario ID : **3A**

Cost Concerns: Total Cost of Construction: $ 1,382,520.00 **Score (used for normalization):** 26

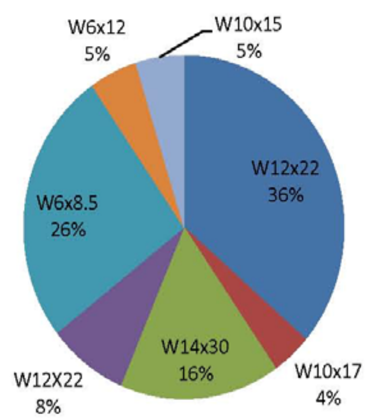


Architectural Concerns:

- Usable Area — Total Area of Exposed Internal Columns: 69.17 sq ft — **Score:** 30.8
- Floor Depth — Total Floor Depth of four floor systems: 75.2 in — **Score:** 62.2

- Member Uniformity — **Score:** 58.3

Beams(floors & roof)		Interior	W12x22
		Exterior	W10x17
Gider(floors & roof)		Interior	W14x30
		Exterior	W12X22
Column	4th	Interior	8"x8" W6x8.5
		Exterior	8"x8" W6x8.5
		Corner	8"x8" W6x8.5
	3rd	Interior	8"x8" W6x8.5
		Exterior	8"x8" W6x8.5
		Corner	8"x8" W6x8.5
	2nd	Interior	10"x10" W6X12
		Exterior	8"x8" W6x8.5
		Corner	8"x8" W6x8.5
	1st	Interior	12"x12" W10x15
		Exterior	8"x8" W6x8.5
		Corner	8"x8" W6x8.5



Construction Concerns:

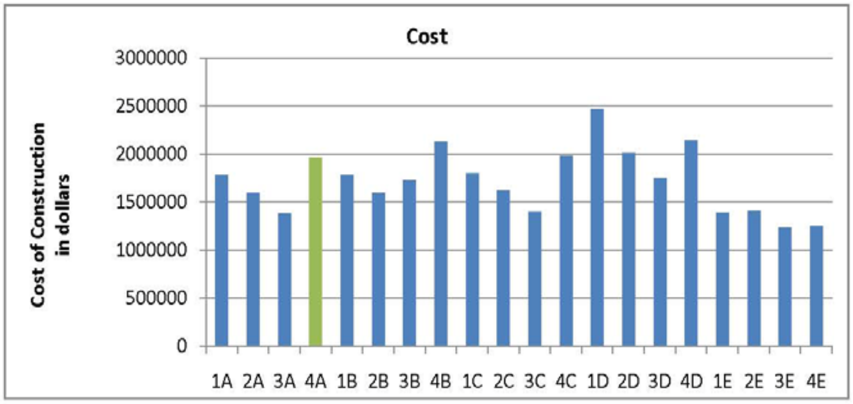
- Structural Layout — Bay Size: 20'x20' — **Score:** 66.7
- Footing Construction — Bay Size: 20'x20' — **Score:** 34
- Structural Fire Protection — Total Area of Exposed Steel Members: 87587 ft^2 — **Score:** 87.4

Total Score: 339.4

Normalized Total Score: 13.05

Scenario ID : **4A**

Cost Concerns: Total Cost of Construction: $ 1,965,608.20 **Score (used for normalization):** 65



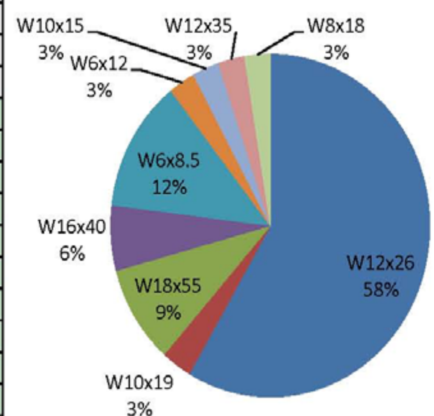
Architectural Concerns:

- Usable Area — Total Area of Exposed Internal Columns: 50.36 sq ft — **Score:** 49.6
- Floor Depth — Total Floor Depth of four floor systems: 84.4 in — **Score:** 49.4

Construction Concerns:

- Member Uniformity — **Score:** 56.4

Beams(floors & roof)		Interior	W12x26
		Exterior	W10x19
Gider(floors & roof)		Interior	W18x55
		Exterior	W16x40
Column	4th	Interior	8"x8" W6x8.5
		Exterior	8"x8" W6x8.5
		Corner	8"x8" W6x8.5
	3rd	Interior	10"x10" W6x12
		Exterior	8"x8" W6x8.5
		Corner	8"x8" W6x8.5
	2nd	Interior	12"X12" W10x15
		Exterior	8"x8" W6x8.5
		Corner	8"x8" W6x8.5
	1st	Interior	14"x14" w12x35
		Exterior	10"x10" W8X18
		Corner	10"x10" W8X18



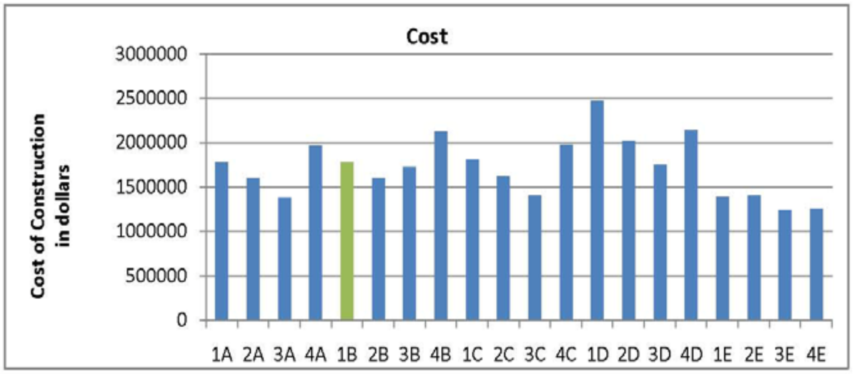
- Structural Layout — Bay Size: 25'x25' — **Score:** 73.3
- Footing Construction — Bay Size: 25'x25' — **Score:** 55
- Structural Fire Protection — Total Area of Exposed Steel Members: 109222 ft^2 — **Score:** 51.3

Total Score: 335

Normalized Total Score: **5.15**

Scenario ID : **1B**

Cost Concerns: Total Cost of Construction: $ 1,782,951.80 **Score (used for normalization):** 53



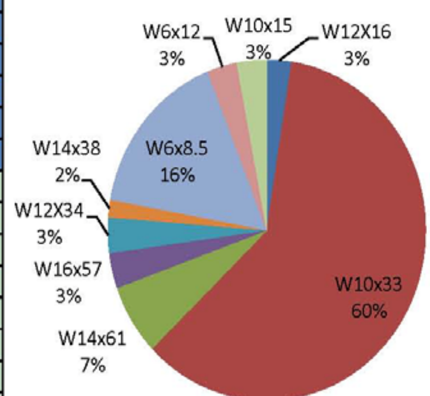
Architectual Concerns:

- Usable Area — Total Area of Exposed Internal Columns: 69.17 sq ft — **Score:** 30.8
- Floor Depth — Total Floor Depth of four floor systems: 70.1 in — **Score:** 69.3

Construction Concerns:

- Member Uniformity — **Score:** 59.2

Beams	Roof	Interior	W10x33
		Exterior	W12x16
	Floors	Interior	W10x33
		Exterior	W12X16
Gider	Roof	Interior	W16x57
		Exterior	W14x38
	Floors	Interior	W14x61
		Exterior	W12X34
Column	4th	Interior	8"x8" W6x8.5
		Exterior	8"x8" W6x8.5
		Corner	8"x8" W6x8.5
	3rd	Interior	8"x8" W6x8.5
		Exterior	8"x8" W6x8.5
		Corner	8"x8" W6x8.5
	2nd	Interior	10"x10" W6X12
		Exterior	8"x8" W6x8.5
		Corner	8"x8" W6x8.5
	1st	Interior	12"x12" W10x15
		Exterior	8"x8" W6x8.5
		Corner	8"x8" W6x8.5



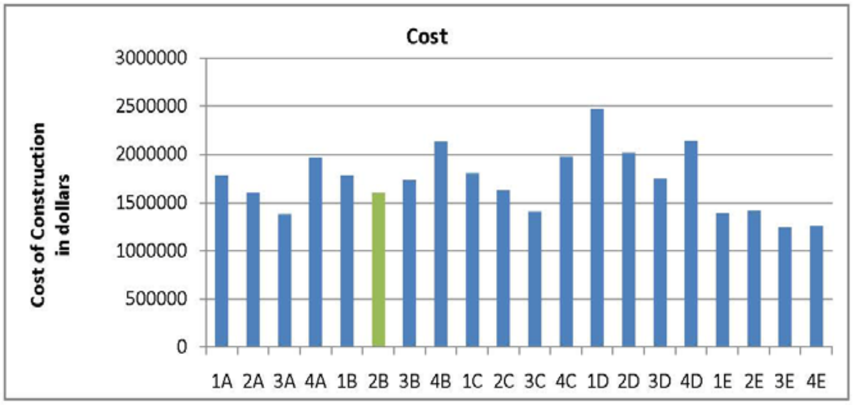
- Structural Layout — Bay Size: 20'x20' — **Score:** 66.7
- Footing Construction — Bay Size: 20'x20' — **Score:** 34
- Structural Fire Protection — Total Area of Exposed Steel Members: 117773 ft^2 — **Score:** 37

Total Score: 297

Normalized Total Score: **5.60**

Scenario ID : **2B**

Cost Concerns: Total Cost of Construction: $ 1,600,951.80 **Score (used for normalization):** 41



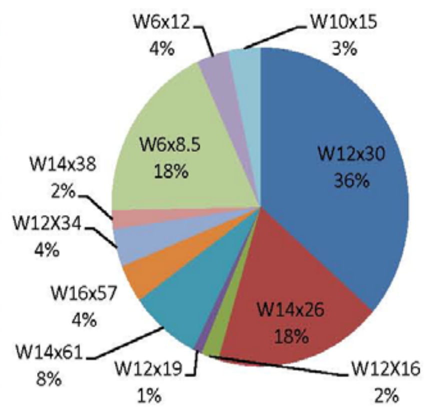
Architectural Concerns:

- Usable Area — Total Area of Exposed Internal Columns: 69.17 sq ft — **Score:** 30.8
- Floor Depth — Total Floor Depth of four floor systems: 70.1 in — **Score:** 69.3

Construction Concerns:

- Member Uniformity — **Score:** 41.4

Beams	Roof	Interior	W10x33
		Exterior	W12x16
	Floors	Interior	W10x33
		Exterior	W12X16
Gider	Roof	Interior	W16x57
		Exterior	W14x38
	Floors	Interior	W14x61
		Exterior	W12X34
Column	4th	Interior	8"x8" W6x8.5
		Exterior	8"x8" W6x8.5
		Corner	8"x8" W6x8.5
	3rd	Interior	8"x8" W6x8.5
		Exterior	8"x8" W6x8.5
		Corner	8"x8" W6x8.5
	2nd	Interior	10"x10" W6X12
		Exterior	8"x8" W6x8.5
		Corner	8"x8" W6x8.5
	1st	Interior	12"x12" W10x15
		Exterior	8"x8" W6x8.5
		Corner	8"x8" W6x8.5



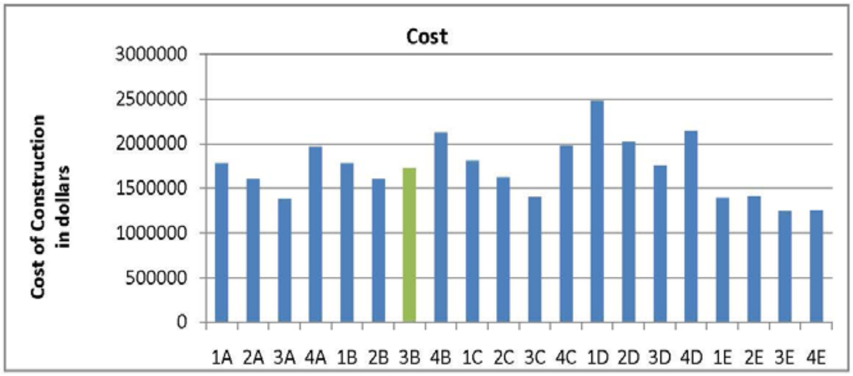
- Structural Layout — Bay Size: 20'x20' — **Score:** 66.7
- Footing Construction — Bay Size: 20'x20' — **Score:** 34
- Structural Fire Protection — Total Area of Exposed Steel Members: 106571.7 ft^2 — **Score:** 55.7

Total Score: 297.9

Normalized Total Score: **7.27**

Scenario ID : **3B**

Cost Concerns: Total Cost of Construction: $ 1,730,819.70 **Score (used for normalization):** 49



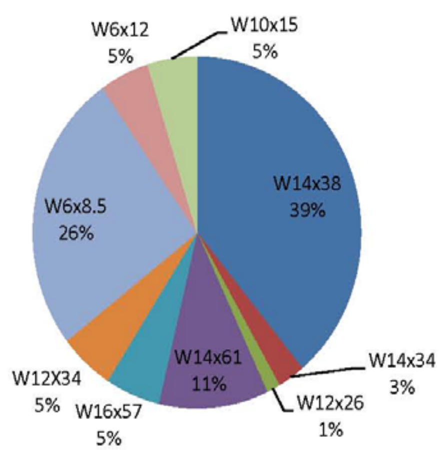
Architectual Concerns:

- Usable Area — Total Area of Exposed Internal Columns: 69.17 sq ft — **Score:** 30.8
- Floor Depth — Total Floor Depth of four floor systems: 78.1 in — **Score:** 58.2

Construction Concerns:

- Member Uniformity — **Score:** 54.3

Beams	Roof	Interior	W10x33
		Exterior	W12x16
	Floors	Interior	W10x33
		Exterior	W12X16
Gider	Roof	Interior	W16x57
		Exterior	W14x38
	Floors	Interior	W14x61
		Exterior	W12X34
Column	4th	Interior	8"x8" W6x8.5
		Exterior	8"x8" W6x8.5
		Corner	8"x8" W6x8.5
	3rd	Interior	8"x8" W6x8.5
		Exterior	8"x8" W6x8.5
		Corner	8"x8" W6x8.5
	2nd	Interior	10"x10" W6X12
		Exterior	8"x8" W6x8.5
		Corner	8"x8" W6x8.5
	1st	Interior	12"x12" W10x15
		Exterior	8"x8" W6x8.5
		Corner	8"x8" W6x8.5



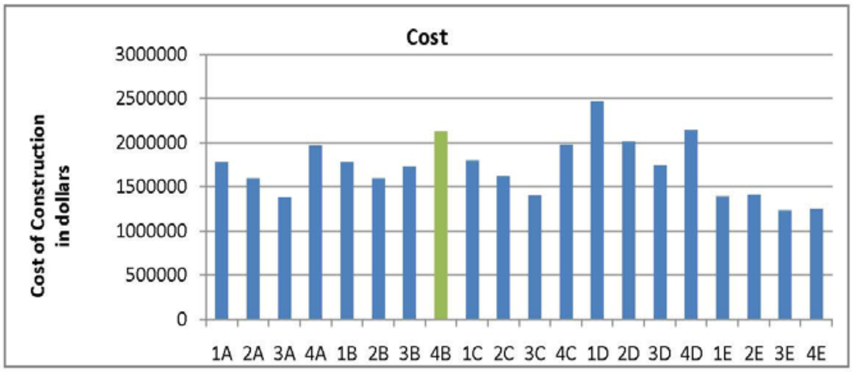
- Structural Layout — Bay Size: 20'x20' — **Score:** 66.7
- Footing Construction — Bay Size: 20'x20' — **Score:** 34
- Structural Fire Protection — Total Area of Exposed Steel Members: 91009.8 ft^2 — **Score:** 81.7

Total Score: 325.7

Normalized Total Score: **6.65**

Scenario ID : **4B**

Cost Concerns: Total Cost of Construction: $ 2,128,239.20 **Score (used for normalization):** 79



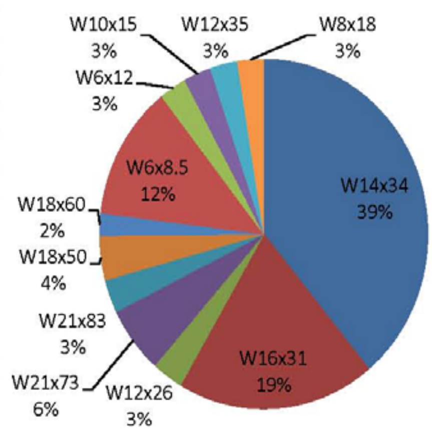
Architectural Concerns:

- Usable Area — Total Area of Exposed Internal Columns: 50.36 sq ft — **Score:** 49.6
- Floor Depth — Total Floor Depth of four floor systems: 97 in — **Score:** 31.9

Construction Concerns:

- Member Uniformity — **Score:** 43.2

Beams	Roof	Interior	W16x31
		Exterior	W12x26
	Floors	Interior	W14x34
		Exterior	W12x26
Gider	Roof	Interior	W21x83
		Exterior	W18x60
	Floors	Interior	W21x73
		Exterior	W18x50
Column	4th	Interior	8"x8" W6x8.5
		Exterior	8"x8" W6x8.5
		Corner	8"x8" W6x8.5
	3rd	Interior	10"x10" W6x12
		Exterior	8"x8" W6x8.5
		Corner	8"x8" W6x8.5
	2nd	Interior	12"X12" W10x15
		Exterior	8"x8" W6x8.5
		Corner	8"x8" W6x8.5
	1st	Interior	14"x14" W12X26
		Exterior	10"x10" W8x13
		Corner	10"x10" W8x10



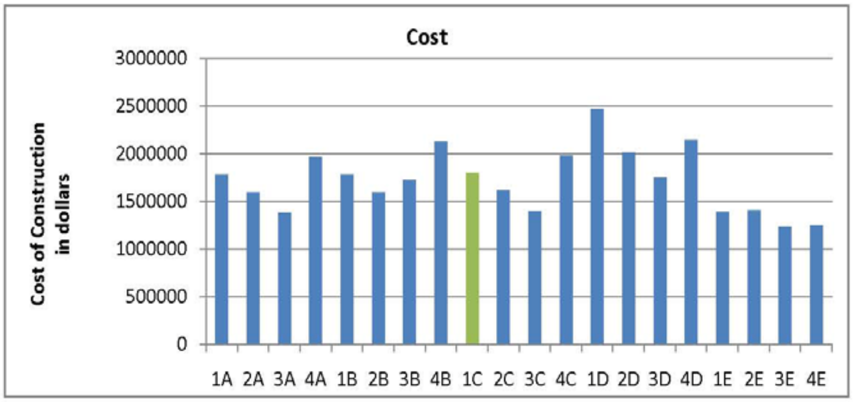
- Structural Layout — Bay Size: 25'x25' — **Score:** 73.3
- Footing Construction — Bay Size: 25'x25' — **Score:** 55
- Structural Fire Protection — Total Area of Exposed Steel Members: 115704.5 ft^2 — **Score:** 40.5

Total Score: 293.5

Normalized Total Score: **3.72**

Scenario ID : **1C**

Cost Concerns: Total Cost of Construction: $ 1,805,398.80 **Score (used for normalization):** 54



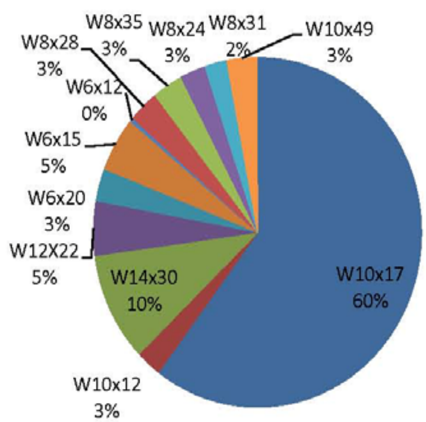
Architectural Concerns:

- Usable Area — Total Area of Exposed Internal Columns: 59.31 sq ft — **Score:** 40.7
- Floor Depth — Total Floor Depth of four floor systems: 67.2 in — **Score:** 73.3

Construction Concerns:

- Member Uniformity — **Score:** 54.6

Beams(floors & roof)		Interior	W10x17
		Exterior	W10x12
Gider(floors & roof)		Interior	W14x30
		Exterior	W12X22
Column	4th	Interior	W6X20
		Exterior	W6X15
		Corner	W6X12
	3rd	Interior	W8X28
		Exterior	W6X15
		Corner	W6X15
	2nd	Interior	W8X35
		Exterior	W8X24
		Corner	W6X15
	1st	Interior	W10X49
		Exterior	W8X31
		Corner	W8x24



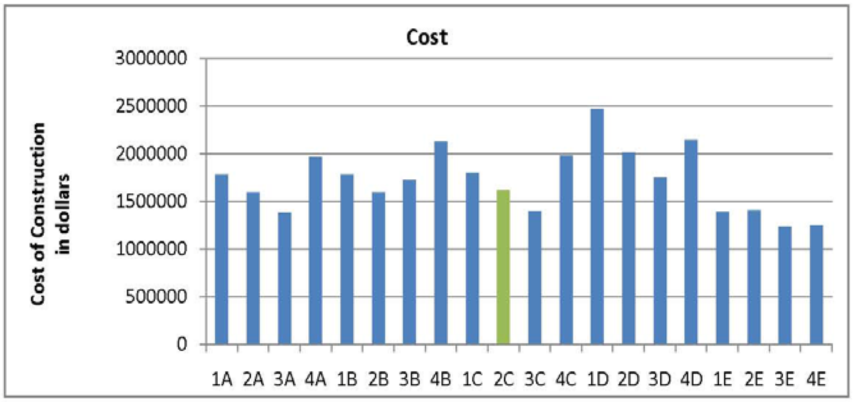
- Structural Layout — Bay Size: 20'x20' — **Score:** 66.7
- Footing Construction — Bay Size: 20'x20' — **Score:** 34
- Structural Fire Protection — Total Area of Exposed Steel Members: 116343.7 ft^2 — **Score:** 39.4

Total Score: 308.7

Normalized Total Score: **5.72**

Scenario ID : **2C**

Cost Concerns: Total Cost of Construction: $ 1,623,398.80 **Score (used for normalization):** 41



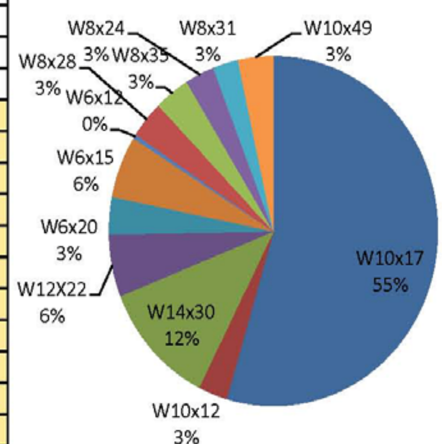
Architectural Concerns:

- Usable Area — Total Area of Exposed Internal Columns: 59.31 sq ft — **Score:** 40.7
- Floor Depth — Total Floor Depth of four floor systems: 67.2 in — **Score:** 73.3

Construction Concerns:

- Member Uniformity — **Score:** 50.9

Beams(floors & roof)		Interior	W10x17
		Exterior	W10x12
Gider(floors & roof)		Interior	W14x30
		Exterior	W12X22
Column	4th	Interior	W6X20
		Exterior	W6X15
		Corner	W6X12
	3rd	Interior	W8X28
		Exterior	W6X15
		Corner	W6X15
	2nd	Interior	W8X35
		Exterior	W8X24
		Corner	W6X15
	1st	Interior	W10X49
		Exterior	W8X31
		Corner	W8x24



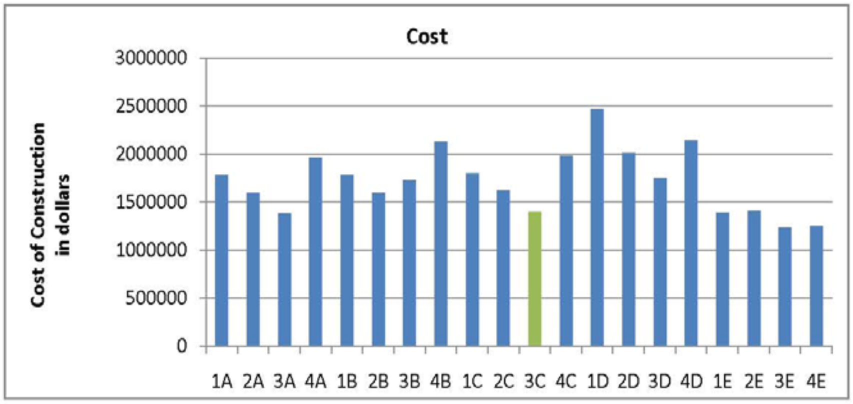
- Structural Layout — Bay Size: 20'x20' — **Score:** 66.7
- Footing Construction — Bay Size: 20'x20' — **Score:** 34
- Structural Fire Protection — Total Area of Exposed Steel Members: 109283.7 ft^2 — **Score:** 51.2

Total Score: 316.8

Normalized Total Score: **7.73**

Scenario ID : **3C**

Cost Concerns: Total Cost of Construction: $ 1,404,966.70 **Score (used for normalization):** 27



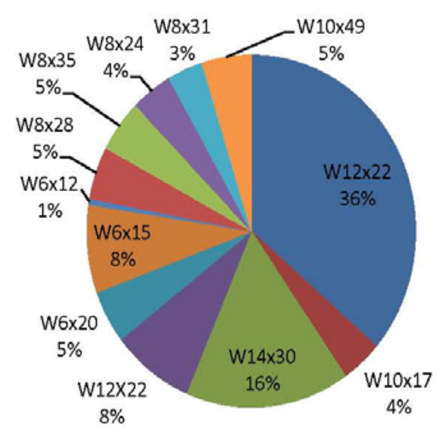
Architectural Concerns:

- Usable Area Total Area of Exposed Internal Columns: 59.31 sq ft **Score:** 40.7
- Floor Depth Total Floor Depth of four floor systems: 75.2 in **Score:** 62.2

Construction Concerns:

- Member Uniformity **Score:** 38.7

Beams(floors & roof)		Interior	W12x22
		Exterior	W10x17
Gider(floors & roof)		Interior	W14x30
		Exterior	W12X22
Column	4th	Interior	W6X20
		Exterior	W6X15
		Corner	W6X12
	3rd	Interior	W8X28
		Exterior	W6X15
		Corner	W6X15
	2nd	Interior	W8X35
		Exterior	W8X24
		Corner	W6X15
	1st	Interior	W10X49
		Exterior	W8X31
		Corner	W8x24



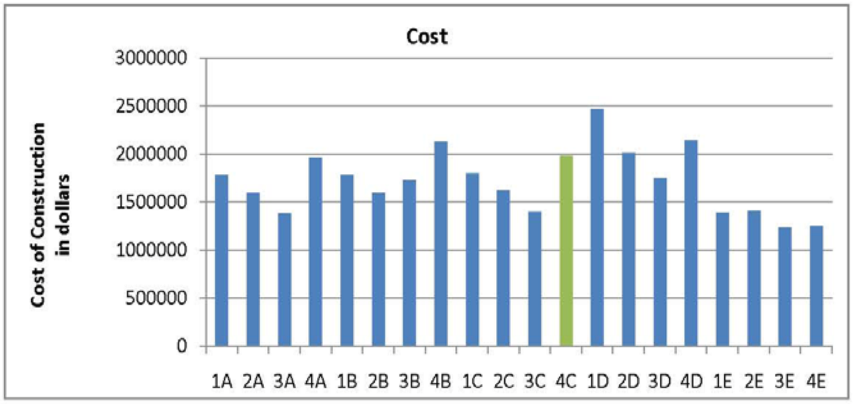
- Structural Layout Bay Size: 20'x20' **Score:** 66.7
- Footing Construction Bay Size: 20'x20' **Score:** 34
- Structural Fire Protection Total Area of Exposed Steel Members: 95163.7 ft^2 **Score:** 74.7

Total Score: 317

Normalized Total Score: **11.74**

Scenario ID : **4C**

Cost Concerns: Total Cost of Construction: $ 1,979,220.20 **Score (used for normalization):** 66



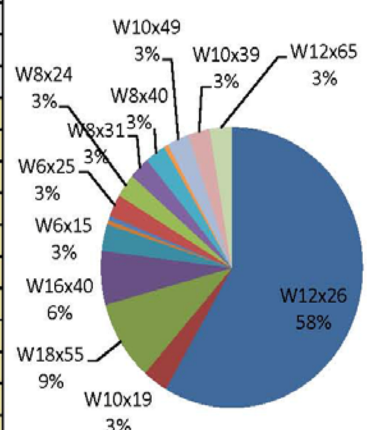
Architectural Concerns:

- Usable Area — Total Area of Exposed Internal Columns: 35.93 sq ft — **Score:** 64.1
- Floor Depth — Total Floor Depth of four floor systems: 84.4 in — **Score:** 49.4

Construction Concerns:

- Member Uniformity — **Score:** 50.9

Beams(floors & roof)		Interior	W12x26
		Exterior	W10x19
Gider(floors & roof)		Interior	W18x55
		Exterior	W16x40
Column	4th	Interior	W8X24
		Exterior	W6X15
		Corner	W6X12
	3rd	Interior	W10x39
		Exterior	W6X25
		Corner	W6X15
	2nd	Interior	W10X49
		Exterior	W8X31
		Corner	W6X20
	1st	Interior	W12x65
		Exterior	W8X40
		Corner	W8X28



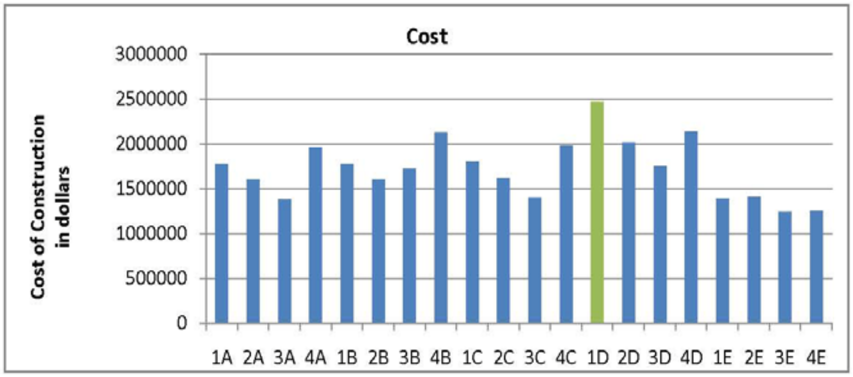
- Structural Layout — Bay Size: 25'x25' — **Score:** 73.3
- Footing Construction — Bay Size: 25'x25' — **Score:** 55
- Structural Fire Protection — Total Area of Exposed Steel Members: 116260.7 ft^2 — **Score:** 39.4

Total Score: 332.1

Normalized Total Score: **5.03**

Scenario ID : **1D**

Cost Concerns: Total Cost of Construction: $ 2,475,794.80 **Score (used for normalization):** 98



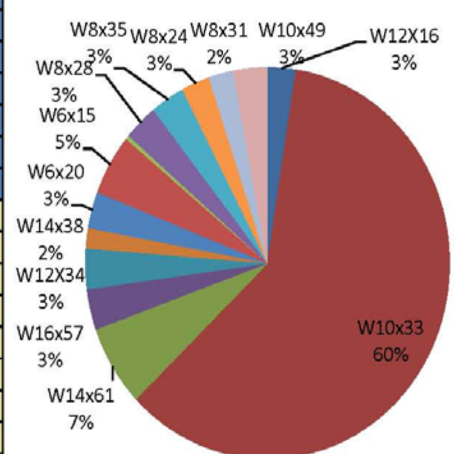
Architectural Concerns:

- Usable Area — Total Area of Exposed Internal Columns: 52.61 sq ft — **Score:** 47.4
- Floor Depth — Total Floor Depth of four floor systems: 70.1 in — **Score:** 69.3

Construction Concerns:

- Member Uniformity — **Score:** 52.7

Beams	Roof	Interior	W10x33
		Exterior	W12x16
	Floors	Interior	W10x33
		Exterior	W12X16
Gider	Roof	Interior	W16x57
		Exterior	W14x38
	Floors	Interior	W14x61
		Exterior	W12X34
Column	4th	Interior	W6X20
		Exterior	W6X15
		Corner	W6X12
	3rd	Interior	W8X28
		Exterior	W6X15
		Corner	W6X15
	2nd	Interior	W8X35
		Exterior	W8X24
		Corner	W6X15
	1st	Interior	W10X49
		Exterior	W8X31
		Corner	W8x24



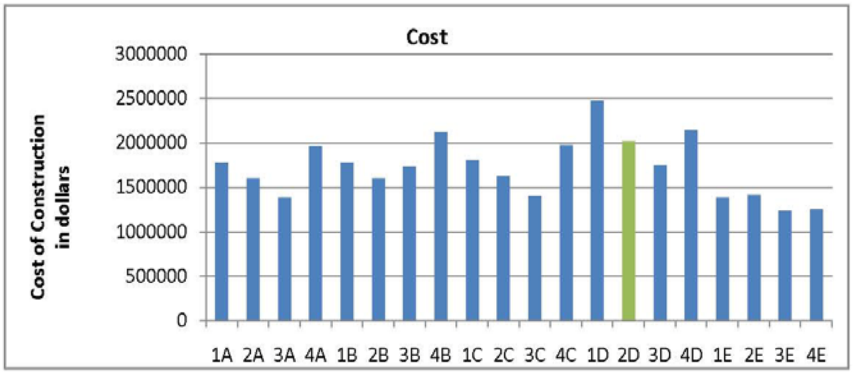
- Structural Layout — Bay Size: 20'x20' — **Score:** 66.7
- Footing Construction — Bay Size: 20'x20' — **Score:** 34
- Structural Fire Protection — Total Area of Exposed Steel Members: 1273612.7 ft^2 — **Score:** 21.1

Total Score: 291.2

Normalized Total Score: **2.97**

Scenario ID : **2D**

Cost Concerns: Total Cost of Construction: $2,017,194.80 **Score (used for normalization):** 68



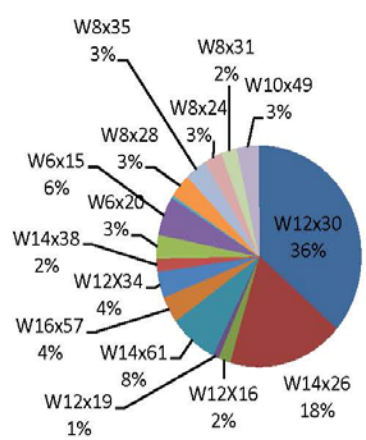
Architectural Concerns:

- Usable Area — Total Area of Exposed Internal Columns: 52.61 sq ft — **Score:** 47.4
- Floor Depth — Total Floor Depth of four floor systems: 70.1 in — **Score:** 69.3

Construction Concerns:

- Member Uniformity — **Score:** 38.6

Beams	Roof	Interior	W14x26
		Exterior	W12x19
	Floors	Interior	W12x30
		Exterior	W12X16
Gider	Roof	Interior	W16x57
		Exterior	W14x38
	Floors	Interior	W14x61
		Exterior	W12X34
Column	4th	Interior	W6X20
		Exterior	W6X15
		Corner	W6X12
	3rd	Interior	W8X28
		Exterior	W6X15
		Corner	W6X15
	2nd	Interior	W8X35
		Exterior	W8X24
		Corner	W6X15
	1st	Interior	W10X49
		Exterior	W8X31
		Corner	W8x24



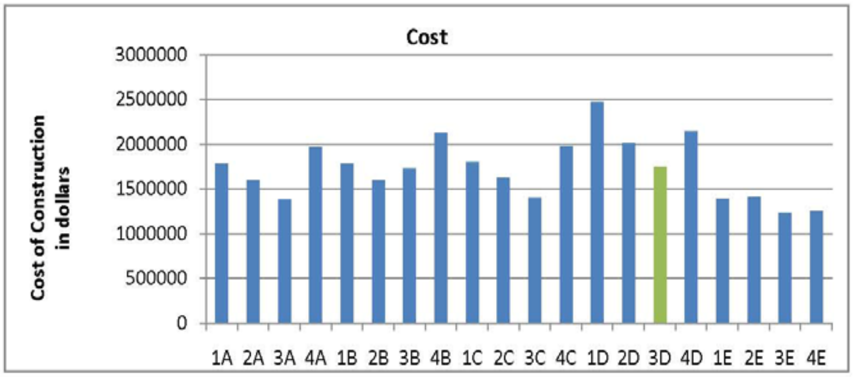
- Structural Layout — Bay Size: 20'x20' — **Score:** 66.7
- Footing Construction — Bay Size: 20'x20' — **Score:** 34
- Structural Fire Protection — Total Area of Exposed Steel Members: 116159.9 ft^2 — **Score:** 39.7

Total Score: 295.7

Normalized Total Score: **4.35**

Scenario ID : **3D**

Cost Concerns: Total Cost of Construction: $ 1,751,762.70 **Score (used for normalization):** 50



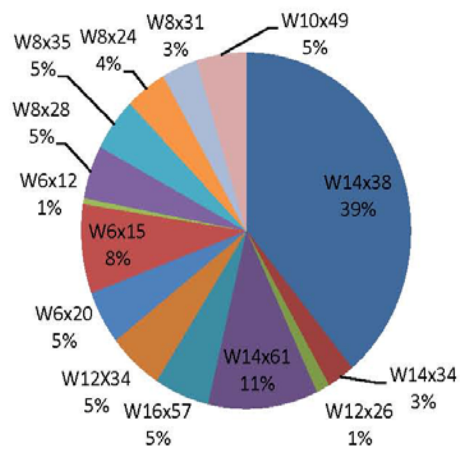
Architectural Concerns:

- Usable Area — Total Area of Exposed Internal Columns: 52.61 sq ft — **Score:** 47.4
- Floor Depth — Total Floor Depth of four floor systems: 78.1 in — **Score:** 58.2

Construction Concerns:

- Member Uniformity — **Score:** 36.9

Beams	Roof	Interior	W14x38
		Exterior	W12x26
	Floors	Interior	W14x38
		Exterior	W14x34
Gider	Roof	Interior	W16x57
		Exterior	W14x38
	Floors	Interior	W14x61
		Exterior	W12X34
Column	4th	Interior	W6X20
		Exterior	W6X15
		Corner	W6X12
	3rd	Interior	W8X28
		Exterior	W6X15
		Corner	W6X15
	2nd	Interior	W8X35
		Exterior	W8X24
		Corner	W6X15
	1st	Interior	W10X49
		Exterior	W8X31
		Corner	W8x24



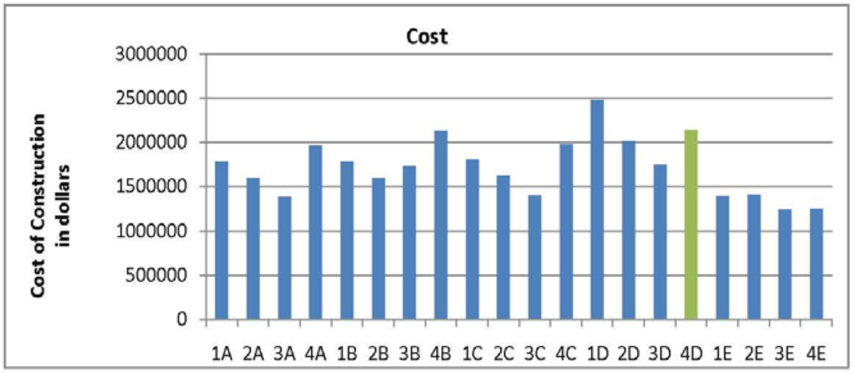
- Structural Layout — Bay Size: 20'x20' — **Score:** 66.7
- Footing Construction — Bay Size: 20'x20' — **Score:** 34
- Structural Fire Protection — Total Area of Exposed Steel Members: 100598.1 ft^2 — **Score:** 65.7

Total Score: 308.9

Normalized Total Score: **6.18**

Scenario ID : **4D**

Cost Concerns: Total Cost of Construction: $ 2,143,755.20 **Score (used for normalization):** 77



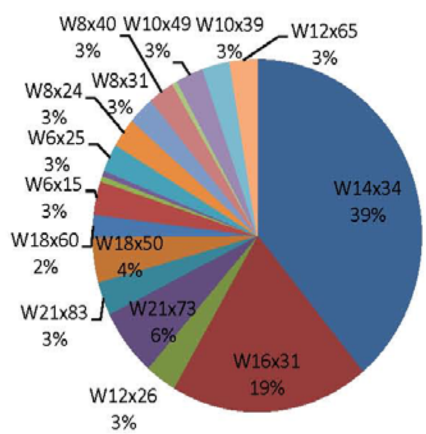
Architectural Concerns:

- Usable Area Total Area of Exposed Internal Columns: 43.82 sq ft **Score:** 56.2
- Floor Depth Total Floor Depth of four floor systems: 97 in **Score:** 31.9

Construction Concerns:

- Member Uniformity **Score:** 39.8

Beams	Roof	Interior	W16x31
		Exterior	W12x26
	Floors	Interior	W14x34
		Exterior	W12x26
Gider	Roof	Interior	W21x83
		Exterior	W18x60
	Floors	Interior	W21x73
		Exterior	W18x50
Column	4th	Interior	W8X24
		Exterior	W6X15
		Corner	W6X12
	3rd	Interior	W10x39
		Exterior	W6X25
		Corner	W6X15
	2nd	Interior	W10X49
		Exterior	W8X31
		Corner	W6X20
	1st	Interior	W12x65
		Exterior	W8X40
		Corner	W8X28



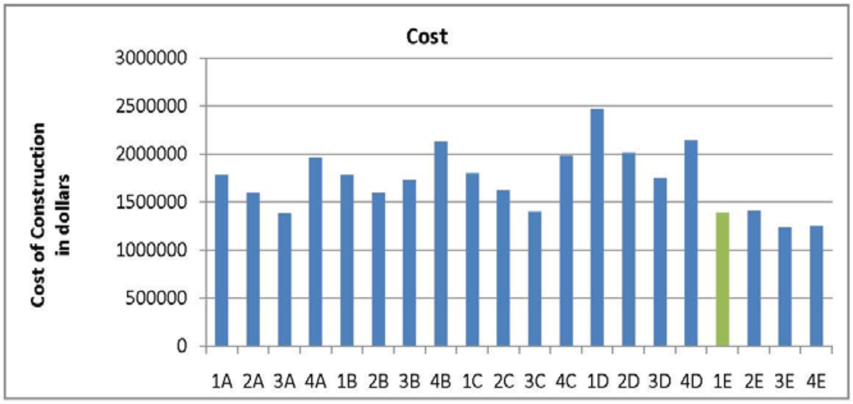
- Structural Layout Bay Size: 25'x25' **Score:** 73.3
- Footing Construction Bay Size: 25'x25' **Score:** 55
- Structural Fire Protection Total Area of Exposed Steel Members: 122743.2 ft^2 **Score:** 28.8

Total Score: 285

Normalized Total Score: **3.70**

Scenario ID : **1E**

Cost Concerns: Total Cost of Construction: $ 1,390,906.22 **Score (used for normalization):** 27



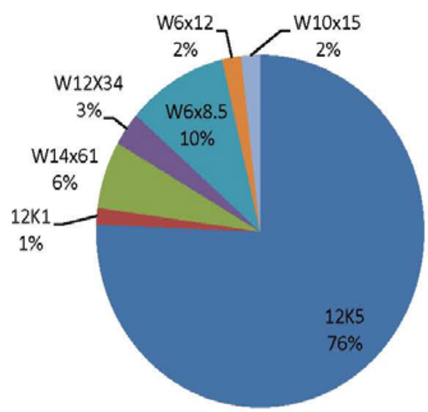
Architectural Concerns:

- Usable Area — Total Area of Exposed Internal Columns: 69.17 sq ft — **Score:** 30.8
- Floor Depth — Total Floor Depth of four floor systems: 67.2 in — **Score:** 73.3

Construction Concerns:

- Member Uniformity — **Score:** 72.1

Beams(floors & roof)		Interior	12K5
		Exterior	12K1
Gider(floors & roof)		Interior	W14x30
		Exterior	W12X22
Column	4th	Interior	8"x8" W6x8.5
		Exterior	8"x8" W6x8.5
		Corner	8"x8" W6x8.5
	3rd	Interior	8"x8" W6x8.5
		Exterior	8"x8" W6x8.5
		Corner	8"x8" W6x8.5
	2nd	Interior	10"x10" W6X12
		Exterior	8"x8" W6x8.5
		Corner	8"x8" W6x8.5
	1st	Interior	12"x12" W10x15
		Exterior	8"x8" W6x8.5
		Corner	8"x8" W6x8.5



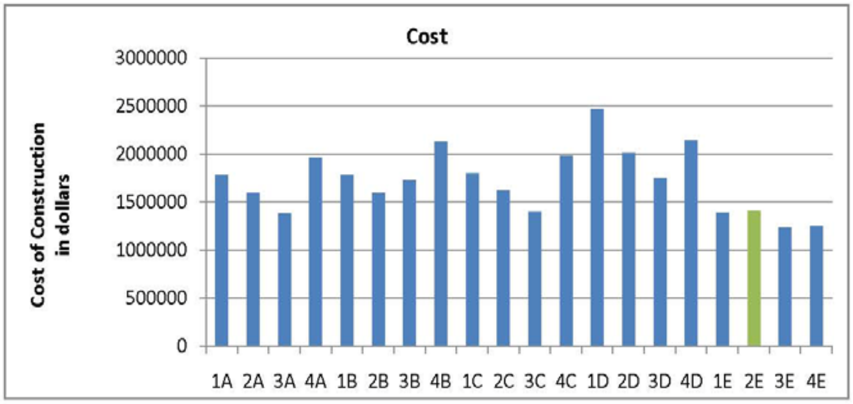
- Structural Layout — Bay Size: 20'x20' — **Score:** 66.7
- Footing Construction — Bay Size: 20'x20' — **Score:** 34
- Structural Fire Protection — Total Area of Exposed Steel Members: 95706.0 ft^2 — **Score:** 73.8

Total Score: 350.7

Normalized Total Score: **12.99**

Scenario ID : **2E**

Cost Concerns: Total Cost of Construction: $ 1,411,849.22 **Score (used for normalization):** 28



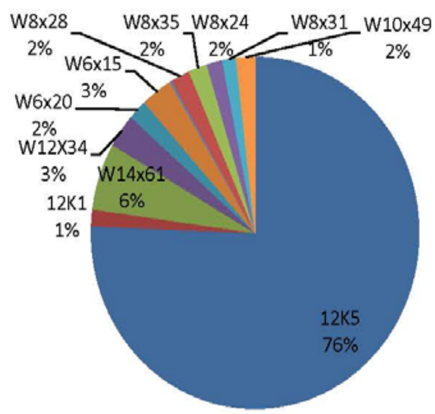
Architectural Concerns:

- Usable Area — Total Area of Exposed Internal Columns: 59.31 sq ft — **Score:** 40.7
- Floor Depth — Total Floor Depth of four floor systems: 67.2 in — **Score:** 73.3

Construction Concerns:

- Member Uniformity — **Score:** 67.6

Beams(floors & roof)		Interior	12K5
		Exterior	12K1
Gider(floors & roof)		Interior	W14x30
		Exterior	W12X22
Column	4th	Interior	W6X20
		Exterior	W6X15
		Corner	W6X12
	3rd	Interior	W8X28
		Exterior	W6X15
		Corner	W6X15
	2nd	Interior	W8X35
		Exterior	W8X24
		Corner	W6X15
	1st	Interior	W10X49
		Exterior	W8X31
		Corner	W8x24



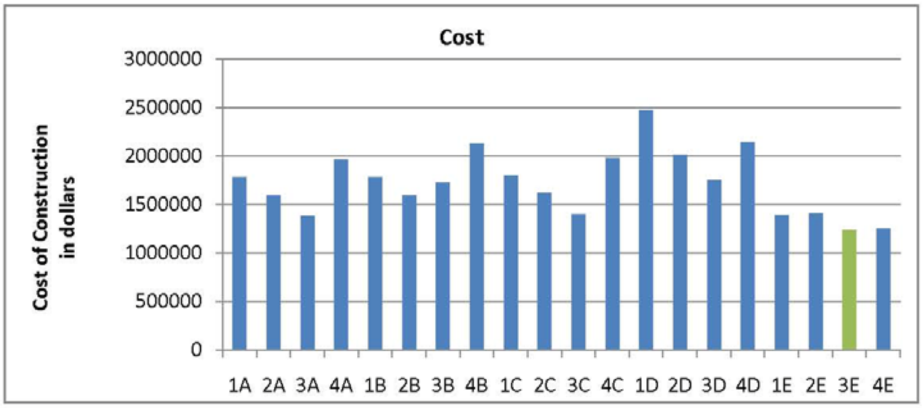
- Structural Layout — Bay Size: 20'x20' — **Score:** 66.7
- Footing Construction — Bay Size: 20'x20' — **Score:** 34
- Structural Fire Protection — Total Area of Exposed Steel Members: 105294.2 ft^2 — **Score:** 57.8

Total Score: 340.1

Normalized Total Score: 12.15

Scenario ID : **3E**

Cost Concerns: Total Cost of Construction: $ 1,239,350.50 **Score (used for normalization):** 16



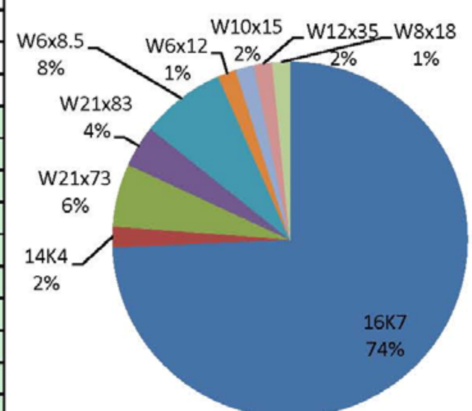
Architectural Concerns:

- Usable Area — Total Area of Exposed Internal Columns: 50.36 sq ft — **Score:** 49.6
- Floor Depth — Total Floor Depth of four floor systems: 84.4 in — **Score:** 49.4

Construction Concerns:

- Member Uniformity — **Score:** 68.4

Beams(floors & roof)		Interior	16K7
		Exterior	14K4
Gider(floors & roof)		Interior	W14x30
		Exterior	W12X22
Column	4th	Interior	8"x8" W6x8.5
		Exterior	8"x8" W6x8.5
		Corner	8"x8" W6x8.5
	3rd	Interior	10"x10" W6x12
		Exterior	8"x8" W6x8.5
		Corner	8"x8" W6x8.5
	2nd	Interior	12"X12" W10x15
		Exterior	8"x8" W6x8.5
		Corner	8"x8" W6x8.5
	1st	Interior	14"x14" W12X26
		Exterior	10"x10" W8x13
		Corner	8"x8" W6x8.5



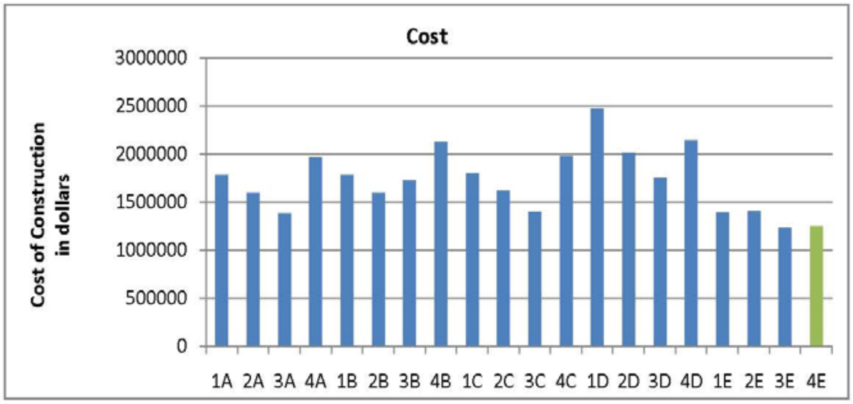
- Structural Layout — Bay Size: 25'x25' — **Score:** 73.3
- Footing Construction — Bay Size: 25'x25' — **Score:** 55
- Structural Fire Protection — Total Area of Exposed Steel Members: 95535.0 ft^2 — **Score:** 74.1

Total Score: 369.8

Normalized Total Score: 23.11

Scenario ID : **4E**

Cost Concerns: Total Cost of Construction: $ 1,254,866.50 **Score (used for normalization):** 17



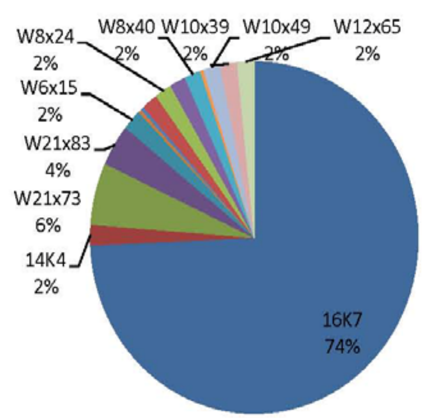
Architectural Concerns:

- Usable Area Total Area of Exposed Internal Columns: 35.93 sq ft **Score:** 64.1
- Floor Depth Total Floor Depth of four floor systems: 84.4 in **Score:** 49.4

Construction Concerns:

- Member Uniformity **Score:** 63.7

Beams(floors & roof)		Interior	16K7
		Exterior	14K4
Gider(floors & roof)		Interior	W14x30
		Exterior	W12X22
Column	4th	Interior	W8X24
		Exterior	W6X15
		Corner	W6X12
	3rd	Interior	W10x39
		Exterior	W6X25
		Corner	W6X15
	2nd	Interior	W10X49
		Exterior	W8X31
		Corner	W6X20
	1st	Interior	W12x65
		Exterior	W8X40
		Corner	W8X28



- Structural Layout Bay Size: 25'x25' **Score:** 73.3
- Footing Construction Bay Size: 25'x25' **Score:** 55
- Structural Fire Protection Total Area of Exposed Steel Members: 102573.7 ft^2 **Score:** 62.4

Total Score: 367.9

Normalized Total Score: 21.64

6.5 Final Structural Scheme Decision

The table below is a summary of all the scores for each of the design criteria for each structural scenario. In this table, it is apparent that Scenario 3E is the best suited structural scheme for the four-story office building. Scenario 3E had the highest combined architectural and construction score as well as the lowest cost, making it the best candidate for the building.

Table 7 Scoring Summary

Score	Total Cost	Architecture		Construction				Total Score (Architecture and Construction)	Normalized Score	Rank
Scenario	Cost	Floor Depth	Net Useable Area	Member Diversity	Structural Layout	Footing	Structural Fire Protection			
1A	53	73.3	30.8	59.8	66.7	34	55.4	320.03	6.04	12
2A	41	73.3	30.8	56.3	66.7	34	67.2	328.24	8.01	7
3A	26	62.2	30.8	58.3	66.7	34	87.4	339.34	13.05	5
4A	65	49.4	49.6	56.4	73.3	55	51.3	334.97	5.15	15
1B	53	69.3	30.8	59.2	66.7	34	37.0	296.99	5.60	13
2B	41	69.3	30.8	41.4	66.7	34	55.7	297.9	7.27	9
3B	49	58.2	30.8	54.3	66.7	34	81.7	325.63	6.65	10
4B	79	31.9	49.6	43.2	73.3	55	40.5	293.46	3.71	18
1C	54	73.3	40.7	54.6	66.7	34	39.4	308.69	5.72	14
2C	41	73.3	40.7	50.9	66.7	34	51.2	316.75	7.73	8
3C	27	62.2	40.7	38.7	66.7	34	74.7	317.05	11.74	6
4C	66	49.4	64.1	50.9	73.3	55	39.6	332.23	5.03	16
1D	98	69.3	47.4	52.7	66.7	34	21.1	291.2	2.97	20
2D	68	69.3	47.4	38.6	66.7	34	39.7	295.77	4.35	17
3D	50	58.2	47.4	36.9	66.7	34	65.7	308.89	6.18	11
4D	77	31.9	56.2	39.8	73.3	55	28.8	284.93	3.70	19
1E	27	73.3	30.8	72.1	66.7	34	73.8	350.71	12.99	3
2E	28	73.3	40.7	67.6	66.7	34	57.8	340.17	12.15	4
3E	16	49.4	49.6	68.4	73.3	55	74.1	369.85	23.12	1
4E	17	49.4	64.1	63.7	73.3	55	62.4	367.83	21.64	2

Scenario 3E consists of 25’x25’ structural bays with open web joists seated on composite girders. The joists within the structural bay are spaced 2.5’, and the columns are composite concrete encased. Scenario 4E came in at a close second behind scenario 3E. These scenarios are quite similar, but because scenario 3E has better member uniformity as well as less fire protection spray coverage, scenario 3E is the better structural scheme. This scenario offers a low cost and structurally sound system that is sufficient for the office building.

Chapter 7: Lateral Loading

7.1 Introduction

Structural analysis of buildings generally begins with the design of the gravity system for dead and live loads that must be transferred vertically from the roof and floors of the building to the foundation through a system of beams, girders, and columns. This process was demonstrated earlier in Chapter 3; however, buildings are not only affected by vertical loads. Horizontal forces, or lateral loads, must also be included in the design process. Most people understand that buildings collapse from vertical loads too large for the structural members to support. Lateral loads can also cause buildings to fail through sideways collapse. Buildings must be designed to resist this sideways force, which is typically achieved by some form of bracing or moment-resistive connections. This chapter will investigate the causes of lateral loads and lateral load resisting techniques based on James Ambrose's book *Design for Lateral Loading*, and the most suitable method for supporting our building against lateral loads will be chosen.

7.2 Sources of Lateral Loads

There are three main sources of lateral loads, and these are wind, seismic and soil pressure. The magnitude of each form of lateral loading must be determined, and the structure must be designed to sustain the governing load accordingly.

7.2.1 Wind

All buildings are susceptible to some amount of lateral force due to wind. Some regions have exceptionally high winds, while others have virtually none compared to other types of lateral forces and can be dismissed. Wind loads primarily affect the lateral bracing system via drag and positive and negative pressures. The majority of the wind load is on surfaces facing and receiving the wind directly, creating a high positive pressure. The leeward side of a building experiences a negative pressure or suction effect. Adding to the positive pressure is the drag on the building in the direction of the wind. After the wind strikes the building it continues to the leeward side creating the drag force. (Ambrose, 1987)

7.2.2 Seismic

Seismic activity or earthquakes occur throughout the world. Some areas have very destructive earthquakes that result in the loss of lives and damage to infrastructure. The effect of seismic activity on buildings is mostly due to the immense vibrations in the soil and the effects of inertia. Since buildings are attached to the ground by foundations, as the ground moves the building follows and shakes. The inertia created by the mass of the buildings generates a swaying effect as the ground shakes the building back and forth, causing immense stress in the structural frame. The effects of seismic forces can be analyzed by equivalent static forces as well as the dynamic motion of the building, creating a complicated analysis. (Ambrose, 1987)

7.2.3 Soil Pressure

Soil pressure creates two types of horizontal forces on foundations, active and passive. Active pressure is due to the soil load itself acting on a resisting component, such as a retaining or basement wall. Passive pressure is produced when the soil must resist a horizontal force placed on a foundation. The horizontal effects of these pressures are dependent on the type and characteristics of the soil at the building site. The forces created by soil pressure are typically resisted by retaining walls constructed of reinforced concrete; however, very high soil pressure can require improved moment resisting walls. (Ambrose, 1987)

7.2.4 Focus

Our building is located in the northeast United States where there is very little seismic activity compared to states on the west coast, however new buildings must be designed to meet minimum requirements for earthquakes (MBC, 7th Edition). In addition, we are focusing on a steel frame system to resist the lateral loads. The affects of soil pressure are neglected as none of our building will be below grade. We will assume that wind loads are the governing lateral force and focus on the effects of high winds on our structure to choose the most appropriate method for bracing our steel frame.

7.3 Load Resisting Systems

Typical framing configurations utilizing post and beam construction are unstable under lateral loading. Member connections cannot resist the full moments that can develop under loading conditions, requiring some form of bracing. Steel members are strong and ductile, allowing them to resist very high stresses in multiple directions and absorb high level of energy during plastic failure. This inherent strength and toughness allows steel framed structures to be made to resist lateral loads very easily and is usually achieved through bracing (trussing) of the steel frame, or by specially designed moment resistive connections. (Ambrose, 1987)

7.3.1 Braced Frames

A braced frame refers to the use of trussing as the bracing technique for resisting lateral loads. There are several ways to brace a frame against lateral loading, and trussing is generally used for the vertical bracing system in conjunction with a system of horizontal diaphragms. It is possible to use trussing for the horizontal system; however, the roof and floor decks in our building will be considered rigid to provide the necessary capacity for diaphragm action in combination with the vertical bracing. (Ambrose, 1987)

Trussing of steel frames is often achieved by the placement of diagonal steel members in the rectangular bays of the building. There are four bracing configurations that are most commonly used in steel construction as shown in Figure 13. The simplest and most obvious arrangements are dual functioning and x-bracing systems. Dual functioning bracing only uses a single diagonal member that must serve as a tension and compression element to resist lateral loading in both directions. X-bracing systems incorporate two diagonal members that criss-cross to form an X in the bays. When loading is applied the diagonals only act in tension, eliminating the need for compression members. (Ambrose, 1987)

Two methods that employ a combination of truss and rigid-frame actions are knee-bracing and K-bracing. Knee braces are often used in roof construction to resist lateral loads; however, they can be utilized in the entire framing system to achieve lateral load resistance. Simple pin-type joints can be used for knee-bracing connections, eliminating the need for complicated moment-resistant connections that may be difficult to develop on certain members. K-bracing requires two diagonal members similar to X-bracing but they do not criss-cross, they only connect to one corner of the bay. The advantage of K-bracing is the energy capacity that can be developed through plastic hinging in the frame. This allows for the ultimate plastic capacity of steel to be used which can be up to 58ksi for A36 steel (Ambrose, 1987)

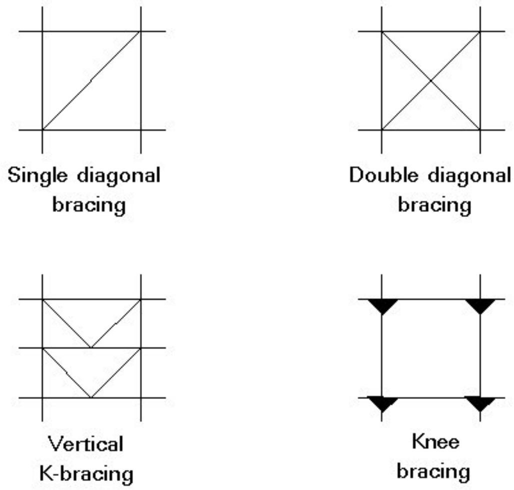


Figure 13 Braced Frame Designs

The bracing of buildings is also important from an architectural point of view. Bracing must be placed with consideration of interior circulation and the problems that arise with openings for doors and windows. Thus limited bracing is utilized to reduce the number of bays that contain diagonal members. Single-story buildings often have bracing on both ends with none in the central bays. Continuity in the horizontal framing allows the rest of the bays to follow under lateral loading. The enormous height of high-rise building results in much higher lateral loads than smaller buildings and more bracing is required. Utilizing the central bays for bracing allows for architectural freedom and sufficient lateral resistance. (Ambrose, 1987)

7.3.2 Rigid Frames

Rigid frames use the connections between members to transfer moments and deform under lateral loading. The term “rigid”, however, was not used to refer to the frame but the connections that are fixed rather than pinned. Rigid frames actually deform or sway much more under lateral loads than braced frames. These types of frames have since been called moment-resisting frames to describe their true function; however, the use of “rigid frames” is much easier and will be used for discussion in this section.

In rigid frames both the gravity and lateral loads induce moments between the members. For seismic loading can abruptly create very high forces requiring rigid frames to be ductile, however this characteristic is inherent in steel. The use of moment resistive connections and the ductility of steel produce very flexible frames. This allows the frame to absorb energy through deformation and its impressive strength. Deformation of the frame softens the loading and allows the frame to work less hard under force resistance, creating a very efficient system under seismic loading. This efficiency is important because of the impact forces that could otherwise fracture members if significant deformation did not occur (Ambrose, 1987)

Rigid steel frames use either bolted or welded connections to transfer the moments between members as shown in Figures 14 and 15.

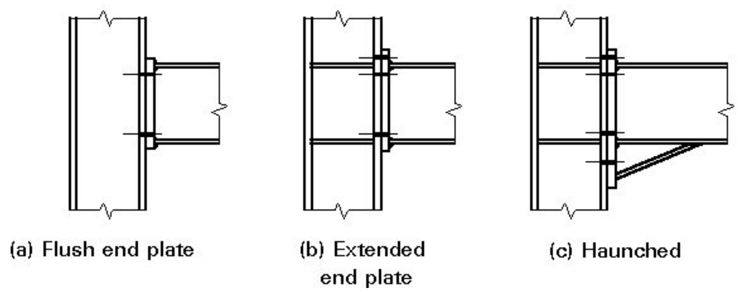


Figure 14 Bolted Moment Connections
(http://www.fgg.unilj.si/kmk/esdep/media/wg11/f0600002.jpg)

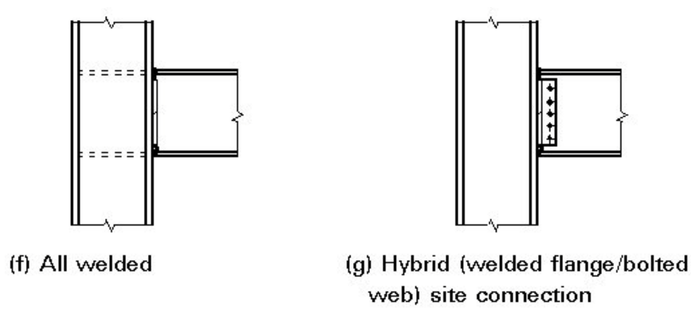


Figure 15 Welded Moment Connections
(http://www.fgg.unilj.si/kmk/esdep/media/wg11/f0600002.jpg)

Unlike lateral bracing, there is no requirement for vertical diaphragms or diagonal members, leaving the wall and interior spaces open. This allows for a high degree of architectural freedom and ease of building planning during the design phase. The load factor for rigid frames compared to braced frames is also much lower because of the inelastic behavior of the frame. The *IBC 2009* allows a K-factor of 0.67 for rigid frames and a much higher value of 1.33 for braced frames. Even with the combination of rigid frame and bracing the K-factor is only reduced to 0.80 or 40% less than a braced system alone. (Ambrose, 1987)

Analysis of rigid frames is very different from braced frames. Since the structure incorporates deformation to reduce the effects of loading, there must be limits to reduce damages to non-structural members and the degree to which occupants notice these movements. Limiting deformations often leads to choosing members based on stiffness requirements rather than for stress limits. Deformation of rigid frames involves the actions of both gravity and lateral loads. Therefore, many loading combinations are possible and must be evaluated. Rigid frames are also indeterminate and require the use of dynamics to determine the true behavior of the structure. Phenomena such as whiplash and resonance must also be studied to ensure the building will be stable under all conditions. (Ambrose, 1987)

7.4 Determination of Lateral Loading

Our building was designed for a location in Worcester, MA. This area and New England, in general, has much higher design forces for wind that may affect building performance, versus the seismic design forces that are considered. Therefore, our primary concern for lateral loading was wind. Design for wind loads followed the requirements of the *International Building Code, 2009 Edition*. The *IBC 2009* states that the determination of wind loads on any and all structures can be completed using either Chapter 6 of ASCE 7 or by the alternate all-heights method exhibited in section 1609.6 (the *IBC 2009*: 1609.1.1). The design methods of Chapter 6 of ASCE 7 were applied to determine the design wind loads.

7.4.1 Wind Pressures

The wind loads were determined using the provisions of ASCE 7-05 Main Wind Force Resisting System Method 1 (Simplified Procedure). ASCE 7-05 provides factors for location, height and exposure of the structure that adjust the effects of wind for specific areas. The wind pressures were determined using values for these factors for a building located in Worcester, MA; the value and reasoning for each factor are presented below.

Importance Factor – This factor is based on the occupancy of the building. Our building is considered to be category II since it is not a significant threat to the public if the building fails.

$I=1.0$ (Table 6-1)

Topographic Factor- This factor depends on the geography of the area surrounding the structure. It was assumed that our building is located in a flat area of Worcester with many surrounding buildings.

$K_{zt}=1.0$

Height and Exposure Factor- This factor depends on the mean height of the building and its exposure to wind. Our building has a flat roof so that the mean height is equal to the overall height. It was considered to be in Exposure Category B because the City of Worcester is an urban area which designates the area to have a category B surface roughness which continues for more than 2600ft from the location of our building.

$h_{mean}=42ft$

Exposure B

λ=1.12 (Figure 6-2)

Average Wind Speed- The average wind speed depends on the location of the structure and is based on data from years of monitoring wind speeds.

V=100mph (Figure 6-1)

Net Wind Pressure- This value is based on the average wind speed, mean roof height and exposure category. It is also dependent on the location of the wind and varies for interior and end zones on walls and roofs.

ps_{30} – given in ASCE 7-05Table 6-1

Once the value of the magnification factors have been determined the final wind pressure for each zone is determined using the following equation:

$Ps=\lambda K_{zt} I ps_{30}$

7.4.2 Beam-Column Design

Frames that are designed to resist the lateral loads that are applied to a structure must be designed to support both horizontal and vertical loads. For the columns, this requires an analysis of the compressive and flexural forces from the gravity loads in combination with the axial and the flexural forces produced by lateral loads. To complete this analysis we first had to determine the axial forces and moments on critical girders and columns produced due to lateral loading. We used the frame analysis program Risa 2-D to complete a first-order analysis of the forces on our frames. The vertical loads determined in the design of our gravity system and the wind loads from ASCE 7-05 were entered into the program, and the computed member forces and joint displacements were used to design adequate sections for girders and columns. Before we could design the sections we determined the critical LRFD load combination.

To design each section we had to check the effects of the combined loading. This was done by following the methods of AISC Specification Chapter H. Chapter H involves the use of an interaction equation to check the adequacy of a section for combined bending and axial loads. To use the correct equation we used the Risa outputs to check the ratio of Pr/Pc .

If Pr/Pc >0.2 use AISC equation H1-1a

If Pr/Pc<0.2 use AISC equation H1-1b

Where Pr is the axial load on the member and Pc is the strength of the section from AISC Table 4-1. In order to apply the interaction equation we had to first determine the magnification factors B1 and B2. These factors were used in place of a lengthy and rigorous second-order analysis and are given by AISC equations C2-2 and C2-3.

$B_1=Cm/(1-\alpha Pr/P_{e1})$ AISC equation C2-2

$$B_2=1/(1-\sum P_{nt}/\sum P_{e2})\text{AISC equation C2-3}$$

The magnification factors involve the Euler buckling strength of the section for no translation P_{e1} and lateral translation P_{e2}. These were determined by AISC equations C2-5 and C2-6a.

$$P_{e1}=\pi^2 EI/(K_1L)^2 \text{ AISC equation C2-5}$$

$$P_{e2}=\pi^2 EI/(K_2L)^2 \text{ AISC equation C2-6a}$$

The main difference between the two Euler buckling values is in the K-factor. We use a value of 1.0 for the K-factor in finding P_{e1} since we assume zero translation of the frame. However for P_{e2} we assume there will be translation of the frame, and find the value of the K-factor was determined using the AISC nomograph and the values for G_A and G_B.

$$G=\sum(I_c/L_c)/ \sum(I_g/L_g)$$

The values for G were determined using the moments of inertia and lengths of girders and columns at joints of the member where g and c denote girders and columns respectively.

We then found the value of Cm using AISC equation C2-4 and the values for B_1 and B_2. Before we used the interaction equation we still had to determine the flexural strength of the section. To do this we investigated lateral torsional buckling, web local buckling and flange local buckling.

$$\text{WLB: } \lambda<\lambda_p=\sqrt{(E/Fy)}$$

$$\text{FLB: } b_f/2t_f<90.5$$

All sections met the above requirements for web and flange local buckling and the governing mode of failure was lateral torsional buckling. We determined the values of L_p, L_r and L_B and interpolated to find the flexural strength of the section from AISC Table 3-2. Finally we entered the values into the required interaction equation and checked to see that the section was adequate.

$$Pr/Pc+8/9(M_{rx}/M_{cx}+ M_{ry}/M_{cy})<1.0 \text{ AISC equation H1-1a}$$

$$Pr/2Pc+(M_{rx}/M_{cx}+ M_{ry}/M_{cy})<1.0 \text{ AISC equation H1-1b}$$

If the section was inadequate we used the equivalent axial load method to select an appropriate section.

$$P_{ueq}=Pu +M_{ux}m+M_{uy}n$$

Where Pu is the factored axial load and the value for the moments M_{ux} and M_{uy} are multiplied by a factor “m” from Table 11.2 (McCormac, 2008) to select a member from AISC Table 4-1 with an axial strength greater than P_{ueq}. The new section was checked following the steps above until an adequate member was selected.

7.5 Design Results

Table 8 below shows both horizontal and vertical wind pressures for each zone. Positive values (+) are towards the surface while negative values (-) are suction. Values are shown for wind pressures for h=30ft and I=1.0 (p_{s30}) and adjusted for our building with λ=1.12 (**p**). Table 9 shows the distributed loads for the interior and exterior zones of the building and the total story forces caused by wind.

Table 8 Average and Final Wind Pressures

Wind Pressures

No Overhangs

	Horizontal Pressures				**Vertical Pressures**				**Overhangs**	
Zone	A	B	C	D	E	F	G	H	E_{OH}	G_{OH}
p_{s30}=	15.9	~~-8.2~~	10.5	~~-4.9~~	-19.1	-10.8	-13.3	-8.4	~~-26.7~~	~~-20.9~~
p=	**17.81**	~~**-9.18**~~	**11.76**	~~**-5.49**~~	**-21.39**	**-12.09**	**-14.9**	**-9.41**	~~**-29.9**~~	~~**-23.41**~~

Building has flat roof so there are no horizontal pressures B and D

Table 9 Linear Loads and Shear Forces in Transverse and Longitudinal Directions

	Distributed Wind loads(plf)		**Story Forces(k)**	
Level	Interior	Exterior	Transverse	Longitudinal
Floor 2	135.24	204.82	44	22.75
Floor 3	129.36	195.91	27.5	14.3
Floor 4	129.36	195.91	27.5	14.3
Roof	64.68	97.96	14	7.2

7.6 FRAME and RISA 2-D Analysis

Two frame designs were completed to determine the most appropriate type for our building. The first was a simple braced frame with one diagonal brace for each story as shown below in Figure 16.

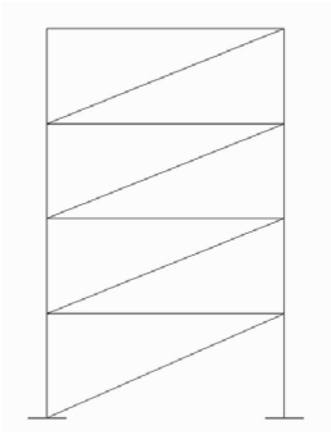


Figure 16 Braced Frame

The second was a rigid frame with moment- resistive connections to provide stability against the wind loads as shown below in Figure 17.

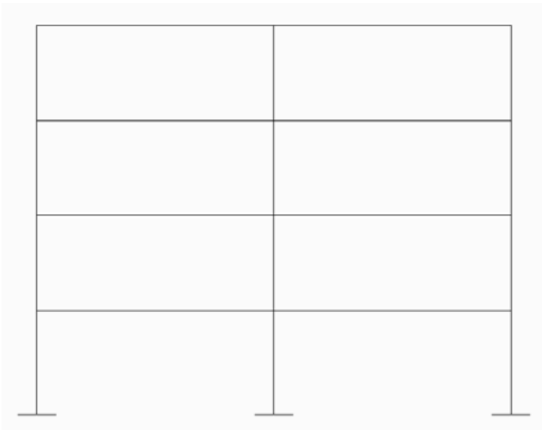


Figure 17 Rigid Frame

Both frames were analyzed using the RISA-2D computer program. The frames in Figures 18 and 19 below are shown with the appropriate wind loads that were determined in accordance with ASCE 7-05.

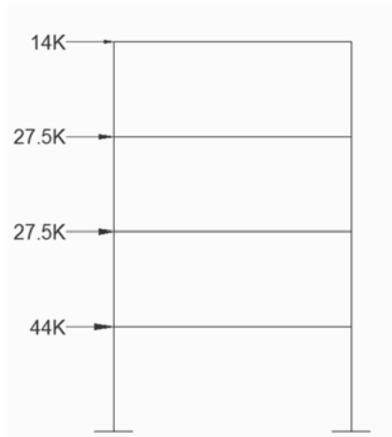


Figure 18 Transverse Wind Loads

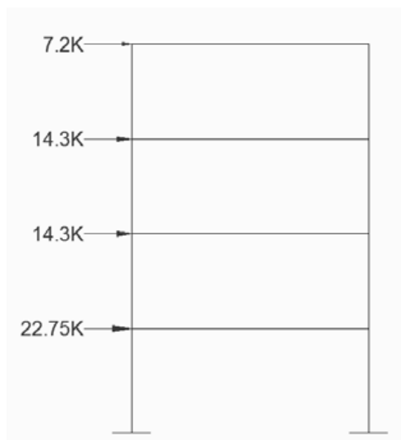


Figure 19 Longitudinal Wind Loads

The gravity loads applied for the frame are listed below with the governing LRFD load combination entered into the computer program for analysis.

Dead Load- 1.3klf

Live Load- 1.4klf

Snow load- .55klf

LRFD load combination- 1.2D+1.6W+.5L+.5S (LRFD L.C. #4)

The frame analysis was completed with the loads applied to both rigid and braced frames. The RISA-2D program produced the following outputs for columns and girders, shown in Table 10 and 11 below, that must be designed for combined loading of moments and axial forces.

Table 10 Braced Frame Outputs

Braced Frame

Columns(level)	1	2	3	4
Axial(kips)	221	168	91	35
Moment(ft-kips)	70	62	50	54
Girders(level)	1	2	3	4
Axial(kips)	168	106	62	30
Moment(ft-kips)	262	262	262	284

Table 11 Rigid Frame Outputs

Rigid Frame

Colums(level)	1	2	3	4
Mlt(ft-kips)	381	183	116	3
Mnt(ft-kips)	25	37	36	42
Plt(kips)	63	32	13	23
Pnt(kips)	241	178	116	56
Girders(level)	1	2	3	4
Mlt(ft-kips)	328	214	108	42
Mnt(ft-kips)	262	262	262	284
Plt(kips)	44	33	31	17
Pnt(kips)	3	0	1	7

The braced frame design only required the total axial loads and moments for the combined wind and gravity loads, since the expression B_2M_{lt} is assumed to be negligible. The rigid frame however required an analysis with gravity and wind loads each applied separately because the second-order effect B_2M_{lt} may be significant. The analysis produced the values Pnt and Mnt for no translation of the frame with only gravity loads applied, and Plt and Mlt for translation of the frame when wind loads are applied. Additionally, the total lateral sway of the building had to be determined for the rigid frame design. Without braces the rigid frame has more flexibility and too much lateral drift is unacceptable. Table 12 below shows the Risa outputs and drift at each story.

Table 12 Rigid Frame Sway

Story	X-deflection(in)	Drift(in)	Height(ft)	Maximum Sway=H/360
1	0.263	0.263	11	0.36
2	0.707	0.444	21	0.73
3	0.846	0.139	31	1.03
4	1.03	0.184	41	1.37

The maximum sway at any story cannot exceed H/360, where “H” is the height of the story. As shown in Table 11 above the drifts for all stories are less than the maximum, therefore our rigid frame design is acceptable.

With the appropriate loads determined for columns and girders in both frames, design calculations were completed to select adequate members to support the combined loading. The interaction equations from AISC specification H were applied and our final frame designs are summarized in Tables 13 and 14 and shown in Figures 20 and 21 below, with member sizes for girders and columns at each level. As you can see from our results, the rigid frame required much larger sections for columns and girders since the lateral loading cannot be distributed to any bracing members.

Table 13 Braced Frame Member Sizes

Braced Frame	Level 1	Level 2	Level 3	Level 4
Columns	W10x39	W10x33	W10x26	W10x22
Girders	W12x87	W12x72	W12x72	W12x72
Braces	W10x45	W10x45	W10x45	W10x45

Table 14 Rigid Frame Member Sizes

Rigid Frame	Level 1	Level 2	Level 3	Level 4
Columns	W10x112	W10x68	W10x49	W10x33
Girders	W12x120	W12x96	W12x79	W12x72

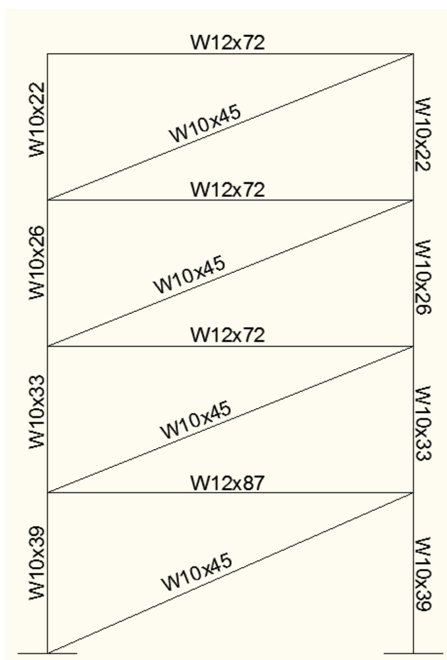


Figure 20 Braced Frame with Member Designations

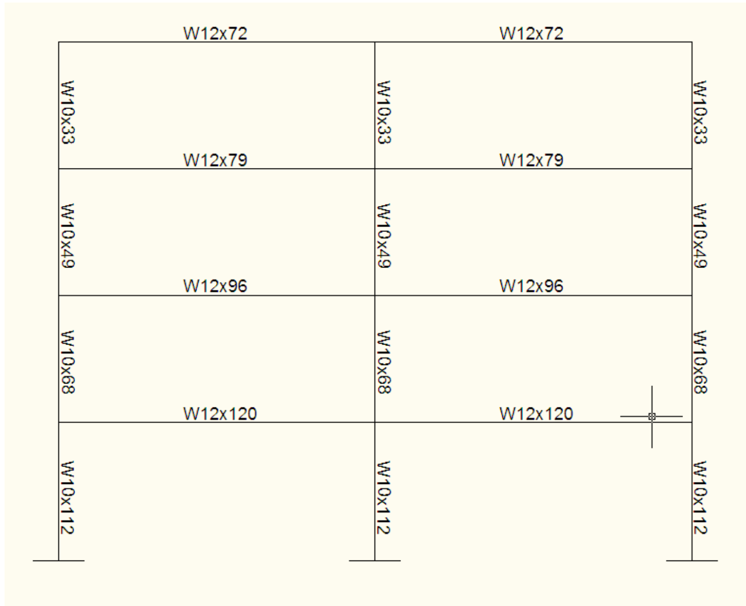


Figure 21 Rigid Frame with Member Designations

7.7 Frame Selection

Selection of the type of frame to use for a building generally involves consideration of cost, architectural flexibility, and performance. Since both frames are suitable to resist the combined effects of gravity and lateral loading, we began our selection based on construction cost. To determine the cost of each frame we used the estimates for steel framing from the *RS Means Building Construction Cost Data, 2009* book for construction cost estimating.

The two tables below present the material cost for each frame. A unit price from RS Means was used for each member size and multiplied by the quantity and length of the girder, column or brace. The unit price accounts for the bare material cost, labor, equipment and profit of the contractor. The pricing represents the cost of moment frames and braced frames for one side of the building with two braced frames and one rigid frame to resist against lateral loads.

Table 15 Cost of Rigid Frame

Rigid Frame

Member Size	Quantity	Length	Unit Price	Net Price
W10x112	3	11	$209.00	$6,897.00
W10x68	3	10	$129.00	$3,870.00
W10x49	3	10	$100.00	$3,000.00
W10x33	3	10	$71.00	$2,130.00
W12x120	2	25	$224.00	$11,200.00
W12x96	2	25	$171.00	$8,550.00
W12x79	2	25	$153.00	$7,650.00
W12x72	2	25	$140.00	$7,000.00
			Total	$50,297.00

Table 16 Cost of Braced Frame

Braced Frame

Member Size	Quantity	Length	Unit Price	Net Price
W10x39	3	11	$63.00	$2,079.00
W10x33	3	10	$55.00	$1,650.00
W10x26	3	10	$44.50	$1,335.00
W10x22	3	10	$39.50	$1,185.00
W12x87	2	25	$167.00	$8,350.00
W12x72	6	25	$140.00	$21,000.00
W10x45	8	27.75	$87.50	$19,425.00
			Total	$55,024.00

The total cost of each frame is over $50,000, but the rigid frame is $5,000 less than the braced frame. However, the estimated cost of each frame is for simple bolted connections, and the rigid frame requires special moment resistant connections. These connections can require more material and labor to install adding to the total cost of the rigid frame. If the connections are welded, less material may be used compared to bolting, and the cost of simple and moment connections is about equal. However, for continuous construction the cost of connections is determined per linear foot of weld (RS Means). The design of the connections was outside of the scope of this project making it difficult to determine the actual cost of bolted connections versus moment connections. Since the gap between the estimated costs of the frames was only a few thousand dollars we chose the frame based on the building layout.

We chose to utilize a rigid frame for our building since it allows the structural bays to remain completely open and enables flexible use of the interior space. With a braced frame design the location of windows and walls can be compromised by the location of the vertical braces.

Chapter 8: Code Analyses

8.1 Introduction

In order to ensure the building will provide a safe environment for the occupants, the local Authority Having Jurisdiction (AHJ) enforces every building to a set of standards according to the adopted model codes (i.e. the *IBC 2009*, *NFPA 101:Life Safety Code*). Fire safety, as one of the most important concerns in many building codes, was the major focus in the code analyses. The code analyses were conducted in two alternative scenarios: 1) the building was not protected by sprinkler systems; 2) the building is fully sprinklered. While the *International Building Codes*, 2009 edition (the *IBC 2009*) acted as the enforced document, there were three major aspects covered in each scenario: first, the general requirements, such as the occupancy classification, building construction type, limitation of building area, and means of egress; second, the passive fire protection features, such as the location of fire barriers, and the fireproof spray construction for steel members; and third, the active fire protection system, such as the fire alarm, the standpipe, and portable fire extinguisher. The detailed codes referenced for each of the three aspects are not listed in this section; however, the sections of the *IBC 2009* provisions used are available in Appendix F for checking purpose.

8.2 Building without Sprinkler Systems

8.2.1 General Requirements

Occupancy Classification

Every floor of this building will be used as office space. Although the building code provisions required classifying each area separately, all the small rooms which are not used as office space are all accessory use area. The codes require that, all the accessory use area shall be classified as the predominate classification. Therefore, all the areas are classified as Group B (business).

Building Area and Height Limitation

For a Group B building, the *IBC 2009* has specific limitations on building area, height and the number of stories. Since this building is 42 ft tall with 4 stories where each story footprint is 20,000 ft^2, the construction type has to be Type II A.

Construction Type

As defined by the *IBC 2009*, all the exposed steel structural members shall be classified as Type II B with 0 hour fire-resistance rating. Therefore, all the open web joists and W-shape girders are classified as Type IIB construction. However, due to the building area and height limitation, only Type IIA construction is permitted. Therefore, the current Type II B steel construction needs to be upgraded to Type II A. As specified in the *IBC 2009*, Type II A construction requires all the steel structural members to have a fire-resistance rating of 1 hour. Spray-applied fireproofing is an efficient way to increase the fire rating for all the exposed steel structural members, or upgrade all Type II B to Type II A. The appropriate fireproof spray applications for various fire-resistance rating have been tested by Underwriters Laboratories. Therefore, the team referenced the designs from *Fire Resistance-Volume 1* to determine the appropriate fireproof spray thickness for the exposed steel members. Based on Design No. N715 and Design No. N789 as shown in Figure 22, the fireproof spray thickness applied on W-shape girders shall be 7/16 inch while it shall be 1-1/16 inch for K series open web joists.

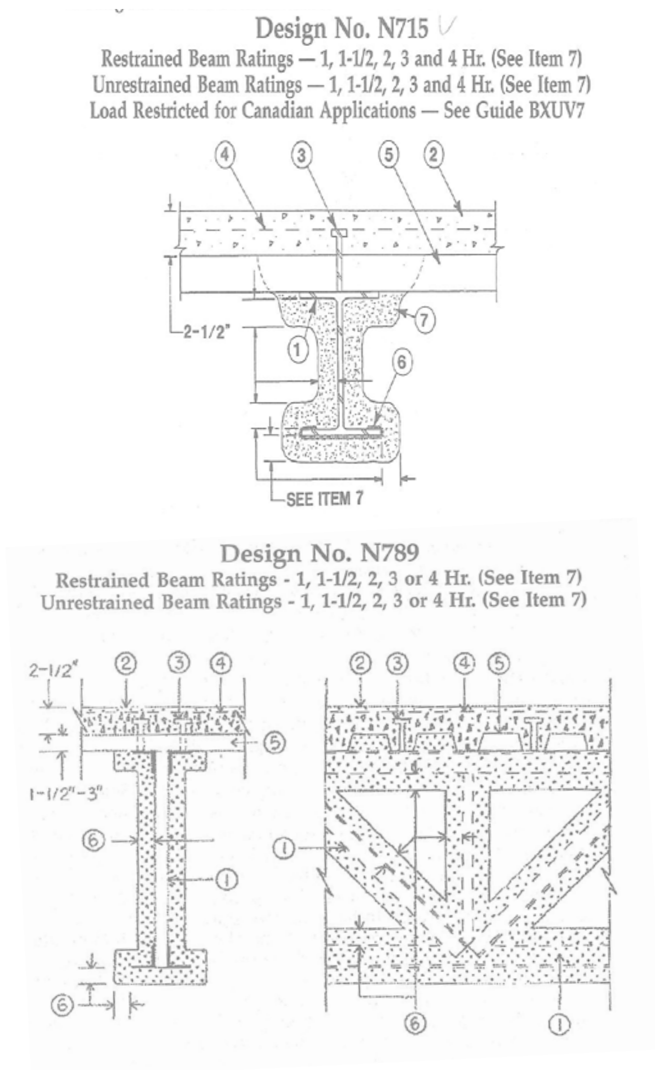


Figure 22 Fireproof Spray Applications (from Fire Resistance-Volume 1 of Underwriters Laboratories)

Since the steel members in composite columns are protected by concrete where the minimum thickness is 1 inch, all the composite columns are rated with 1 hour fire-resistance based on Table 721.5.1(7) of the *IBC 2009*. Therefore no additional protection is needed.

Means of Egress

Number of Exits

Based on the estimated occupant load 200 persons per floor, a minimum of two exits are required for each floor. In this building three exits are provided at each floor.

Exit Capacity

The exits shall have certain width to ensure the flow of people during an evacuation can easily be accommodated. As a component of the exit system, the corridor is as wide as 10 ft which is much greater than the required minimum width of 44 inches. Furthermore, all doors in this building have uniform clear width of 32 inches, while the stairs are 44 inches in width. With three stairs for 2^{nd} to 4^{th} floor and three exits doors in which one is 4-leaf door, the exits provided adequate capacity.

Exit Access Travel distance

For a non-sprinklered building, the maximum exit access travel distance is 200 ft. However, for each floor of this building, the maximum distance is 130 ft which is less than the limitation and shown in Figure 17 and 18.

Common Path of Egress

Common path of egress refers to portion of the exit which the occupants are required to travel before two separate and distinct paths of egress to two exits doors are available. Usually, it is a corridor. The length of common path of egress is limited to a distance not greater than 75 ft. In this office building, the longest common path of egress is 50 ft which is shown in Figure 17 and 18.

Exits Remoteness

For a non-sprinklered building, the *IBC 2009* requires the distance of two exits for an area shall be at least one half of the maximum diagonal length. This requirement had been fulfilled while the team revised the floor plans.

Corridors

The center corridor is a major exit principal in every floor of this building, playing a critical role in means of egress. As mentioned before, the width of corridor is 10 ft which is far greater than what is required. Therefore, the capacity of the corridor is adequate. Furthermore, as mentioned in the section of floor plan revision, the corridor does not have any obstructions due to open doors installed on the corridor walls. However, the 50 feet long dead end corridor shown in Figure 23 and 24 was violate the code provision. Therefore, project team suggested architects should redesign the floor layout to eliminate the 50 feet long dead end corridor and make sure that the length of dead end corridor is not more than 20 feet for an office building without a sprinkler system.

Elevator Lobby

Since the elevator connected more than three stories, it is required to have an elevator lobby at each floor by the *IBC 2009*. And a direct means of egress shall be provided in elevator lobby. The team had applied these requirements while in floor plan revision process. It should be noted that, although it is not required by the codes, the elevator lobby can be utilized as an area of refuge if the doors close automatically and the enclosure walls can perform as smoke partitions.

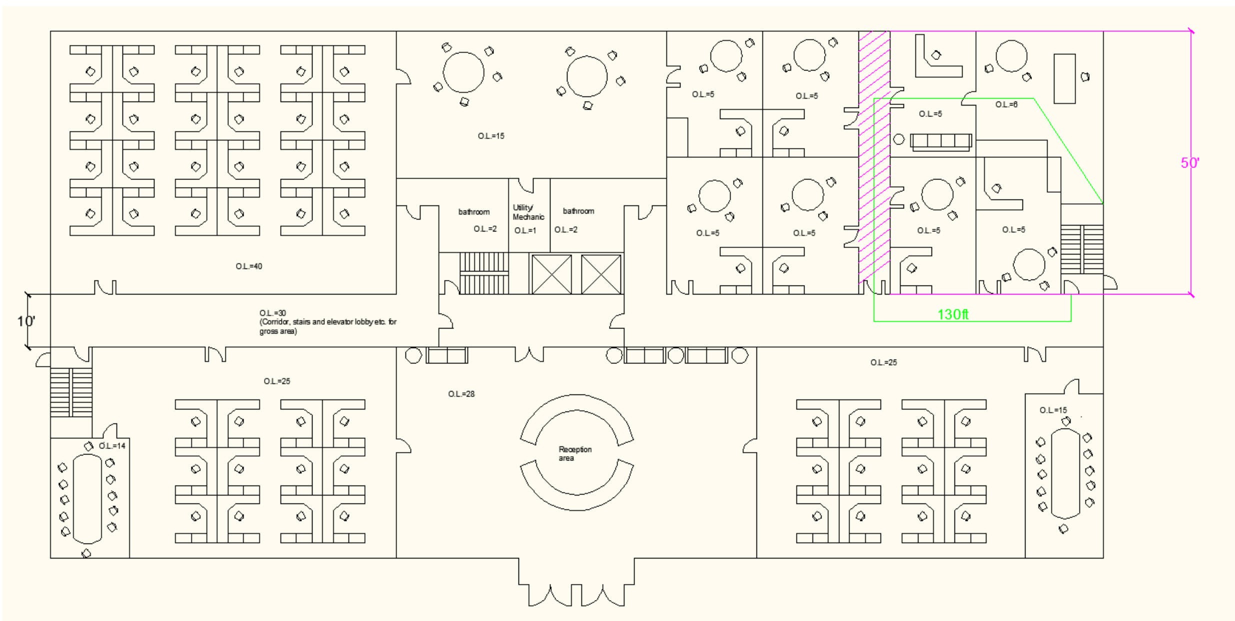


Figure 23 1st Floor Plan (Green line stands for maximum travel distance; pink hatch stands for the maximum common path of travel; O.L. is occupant load)

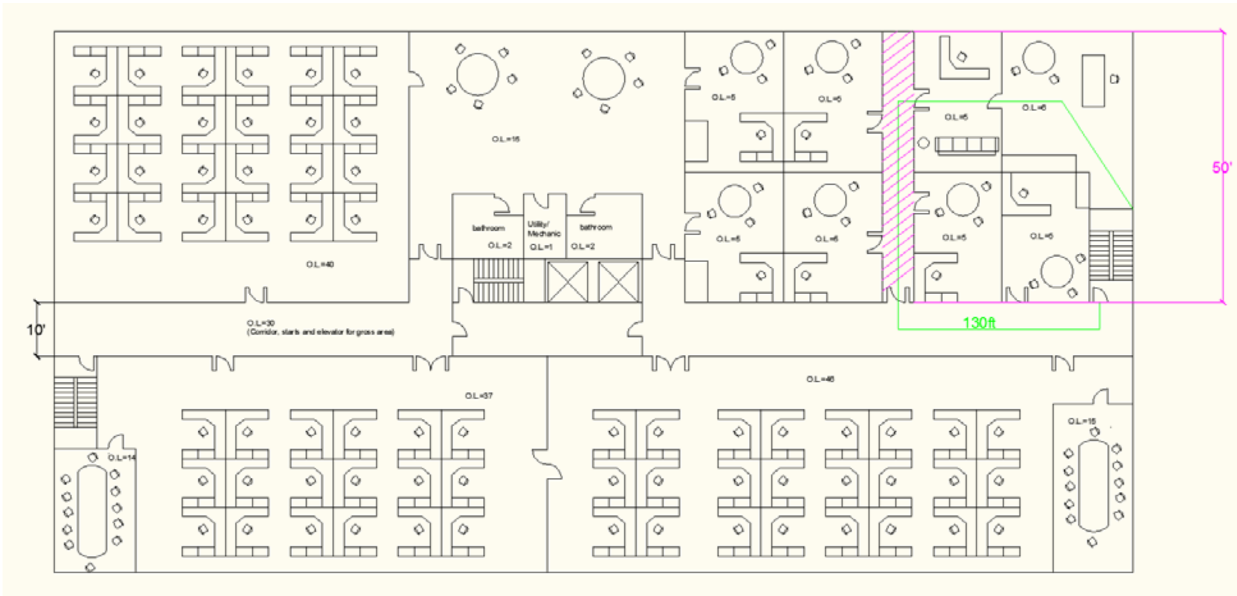


Figure 24 2nd to 4th Floor Plan (Green line stands for maximum travel distance; pink hatch stands for the maximum common path of travel; O.L. is occupant load)

Illumination of Means of Egress

The *IBC 2009* provisions require that the means of egress is illuminated with 11 lux at all times. Therefore, light shall be provided all times at stair ways, corridors and entrance hall at first floor. In order to ensure the power supply, secondary power such as a power generator within the building or batteries storage within the illuminate units is necessary. Using power generator will be easier to maintain. However, renewing of batteries is usually easy to omit and thus the system is less reliable. Therefore, the project team recommended a power generator for this building. And the illuminate units can directly connect with the power generator.

Exit Signs

While the people working in this building will be familiar with the locations of exits, they will not be that obvious for visitors. Therefore, exit signs are essential for the people unfamiliar with the building. The signs are required to be illuminated all times and connected to the emergency power generator to ensure the 90 minutes illumination duration. The location of exits sign is regulated by *NFPA 101*, and typical sign placement criteria are shown in Figure 25.

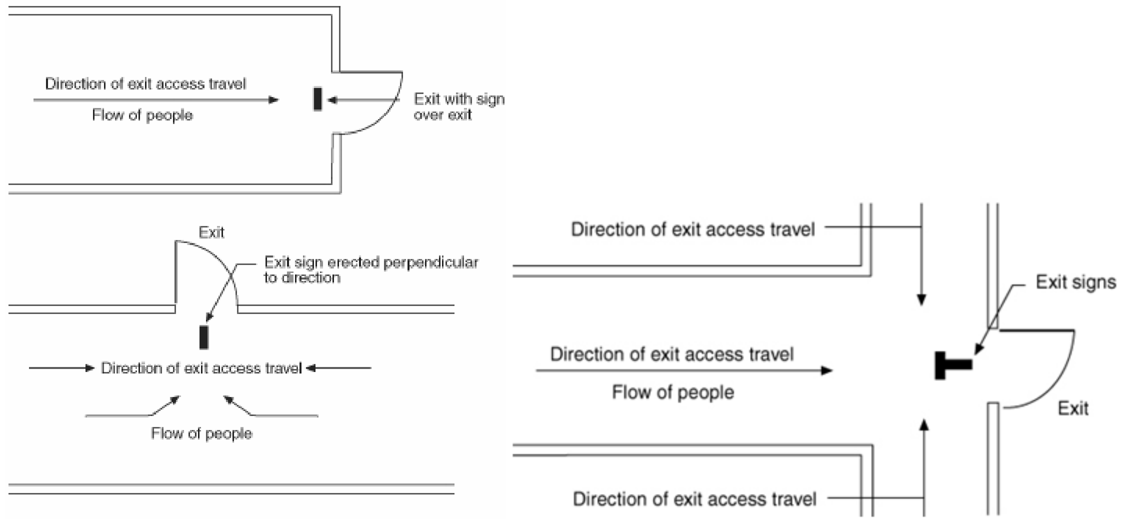


Figure 25 Location of Exit Signs (taken from Figure A.7.10.1.2.1, NFPA 101)

8.2.2 Passive Fire Protection

Fire Barriers

The *IBC 2009* specifies the location of fire barrier and the fire-resistance rating of these fire barriers. As required, the elevator shaft and the stair ways shall be constructed and enclosed by at least 2-hour rating fire barriers. Three samples of two-hour fire rated wall construction are shown in Figure 27. As an important component of means of egress, the center corridor shall be enclosed by fire barriers with at least 1-hour rating, and two samples of the 1hour rating wall construction are shown in Figure 26. The locations of each fire rated wall are displayed in Figure 29 and 30.

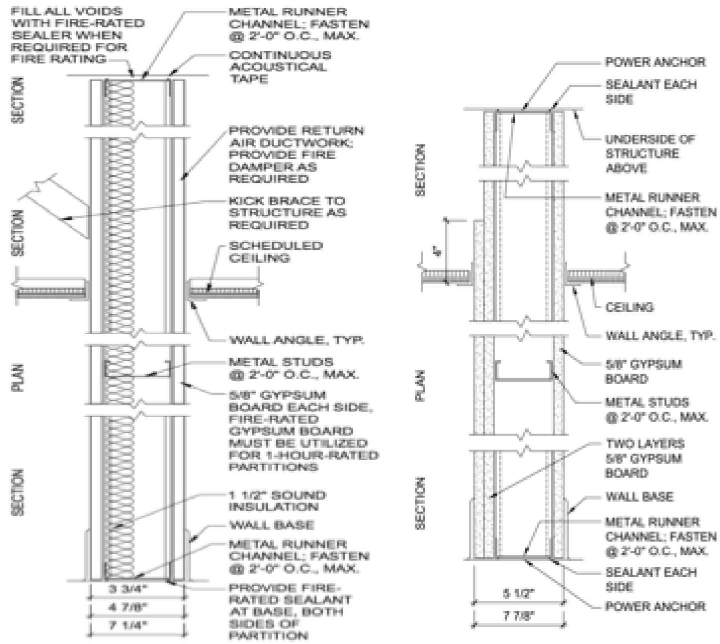


Figure 26 Samples of One-Hour Fire Rated Wall Construction (Puchovsky, 2009)

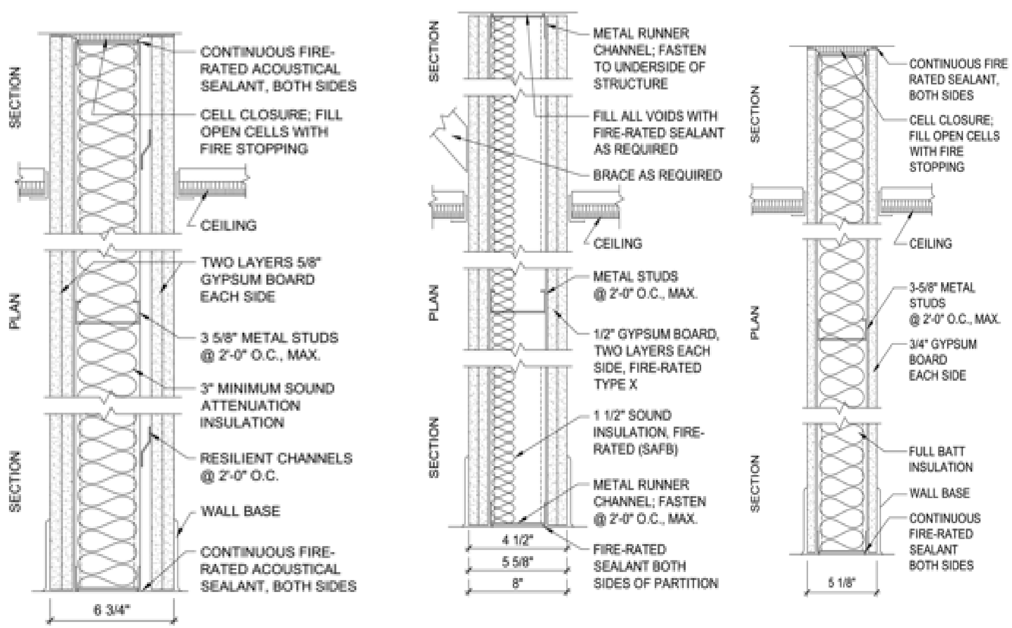


Figure 27 Samples of Two-Hour Fire-Rated Wall Construction (Puchovsky, 2009)

Furthermore, all doors on the fire barriers shall be fire doors with a corresponding fire-protection rating. As required by the *IBC 2009*, fire doors installed on the walls of center corridors shall have a 20 minutes fire-protection rating, and fire doors enclosing stairways shall have at least a 1 ½ hour rating. Besides the fire-protection rating, all fire doors shall be self or automatic-closing doors. Moreover the fire doors located at elevator lobby and the door at the end of the 50 ft center corridor shall be smoke-activated door. All fire doors are indicated in Figure 29 and 30. In order to keep the integrity of the barriers any penetrations, ducts and air transfer openings through fire barriers shall be protected appropriately by rated seals and fire or smoke dumpers. In order to ensure the continuity of fire barriers, appropriate seals shall be applied at the bottom of floor assembly to cover the gap.

Figure 28 Illustration of Vertical Continuity of Fire Barriers (Puchovsky, 2009)

Interior Wall and Ceiling Finish

Based on the occupancy, the absence of sprinkler system and the area of the floor, wall and ceiling finish materials are restricted by the *IBC 2009* to control the fuel load. As required by the *IBC 2009*, the finish shall be Class A at all stair enclosures. For both corridors in each floor, Class B or Class A finish can be applied. For the rest of the floor, the minimum requirement is a Class C finish.

Interior Floor Finish

The floor finish at corridors and stair enclosure is required to be not less than Class II in accordance with *NFPA 253: Standard Method of Test for Critical Radiant Flux of Floor Covering Systems Using a Radiant Heat Energy Source*.

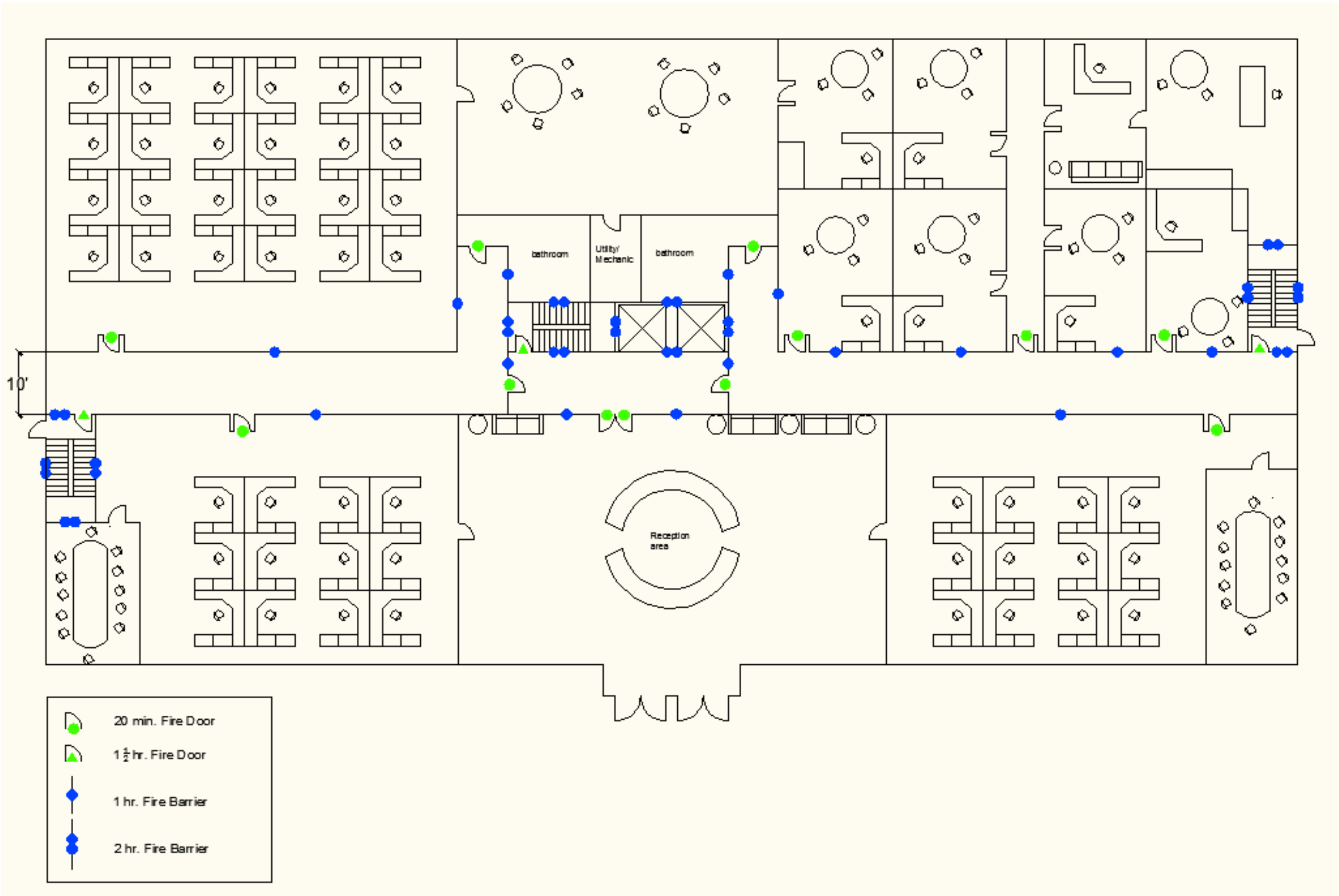


Figure 29 Location of Fire Doors and Fire Rated Walls for First Floor

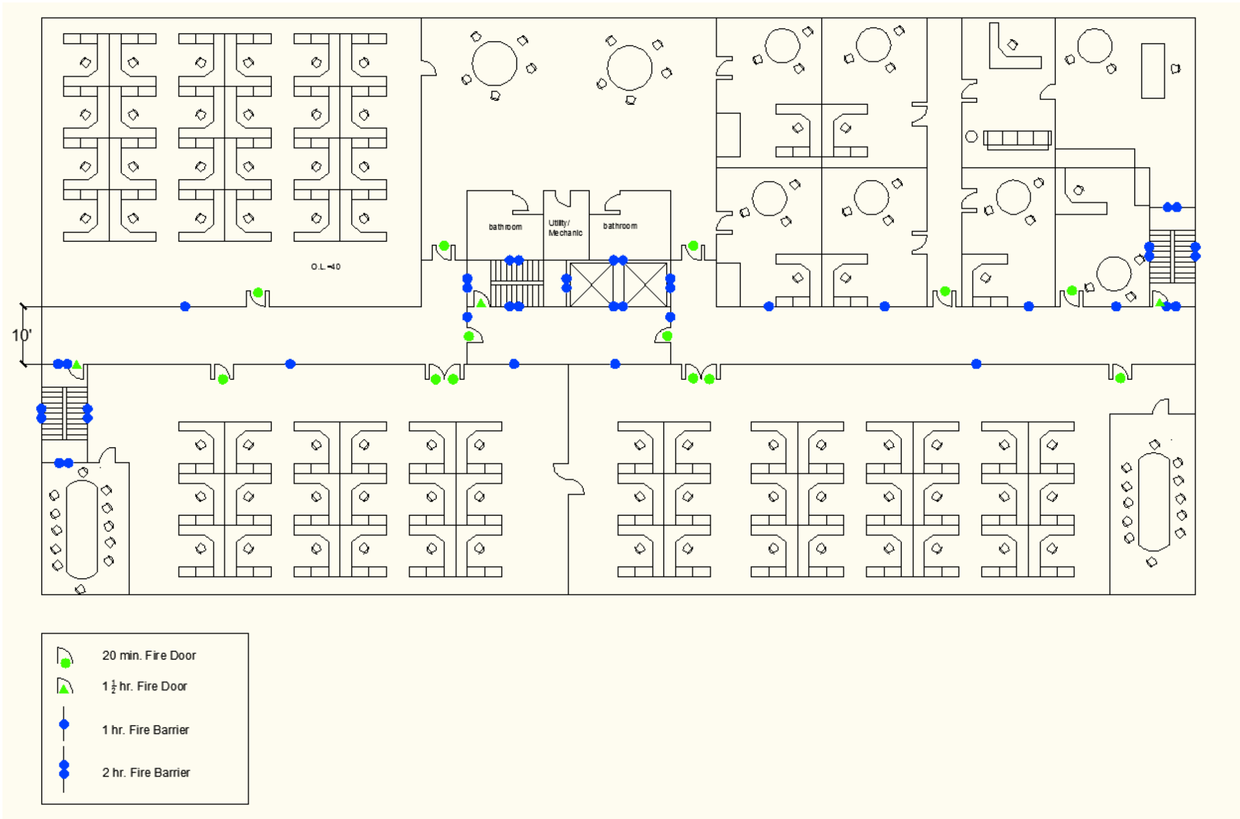


Figure 30 Location of Fire Doors and Fire Rated Walls for 2nd to 4th Floor

8.2.3 Active Fire Protection

Fire Detection and Alarm System

For a Group B building, only a manual fire alarm system is required to be installed according to the *IBC 2009*. However, based on the fact that the required smoke-activated doors need smoke detectors to activate them, smoke detectors are necessary for each floor. Detailed information will be covered in a later section.

Figure 31 A Manual Fire Alarm Box or Manual Pull Alarm Box (taken from http://www.garrett-smarthome.com/firelite/bg-12.htm)

The locations of manual fire alarm boxes are also regulated by the *IBC 2009*: 1) maximum distance from the entrance of an exit is 5 ft; 2) maximum distance between two manual fire alarm boxes is 200ft; 3) vertical distance from the floor level shall be in the range of 42 in to 48 in; 4) it shall be provided in public areas and common areas that are visible. An illustration of the location of a manual fire alarm box is shown in the figure below. Furthermore, all manual fire alarm boxes are required to be red in color with a red or transparent protective cover, and produce a sound pressure 15 dBA above the average ambient sound level or 5 dBA above the maximum sound level having duration of 60 seconds, but not to exceed 110 dBA. In order to make sure the manual fire alarm boxes function well, they should also be connected to the emergency power generator.

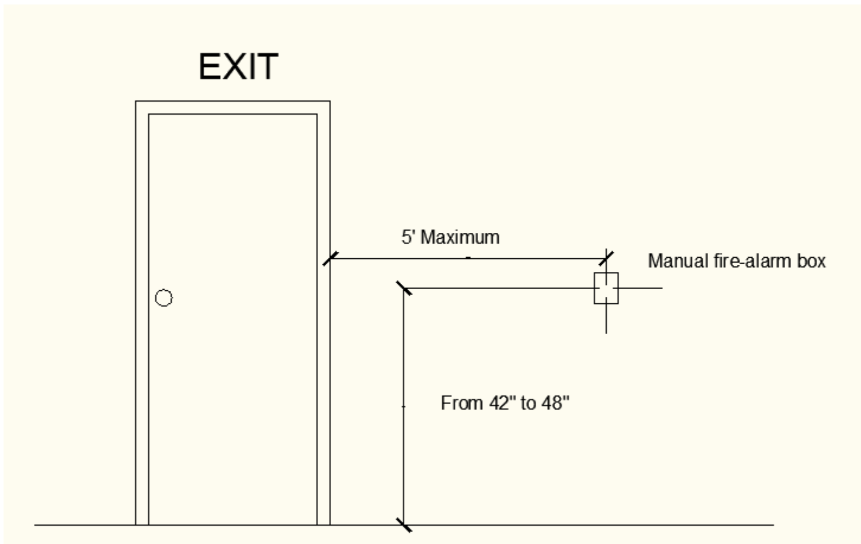


Figure 32 Location of Manual Fire Alarm Box

Fire Department Connection

When fire starts, water is crucial to fire extinguishing, and fire department connection is one of the safe factors that ensures sufficient water. The fire department connection allows the responding fire fighters to boost the sprinkler system pressure and deliver more water to operating sprinklers by pumping water from fire hydrant on street. A fire department connection is required for this building. It is required to be located on the street side of the building where there are no obstructions for fire apparatus and hoses.

Standpipe System

According to section 905.3.1 of the *IBC 2009*, Class III standpipe systems must be installed throughout the building. A Class III system is essentially a hose connection for fire fighters in case of fire as shown in Figure 33. In order to provide a safer environment for fire fighters, all standpipes are located in stairway, at an intermediate floor level landing as shown in Figure 28. Furthermore, in order to make sure the hoses connected to standpipes can cover the entire floor, it is required that the location of standpipe shall ensure that all portions of the building floors are within 30 ft of a muzzle attached to 100 ft of hose. According to the locations of standpipe in

Figure 34, with 130 ft extension, three standpipes are able to cover the entire floor area. Therefore, three standpipes at each stairway are sufficient.

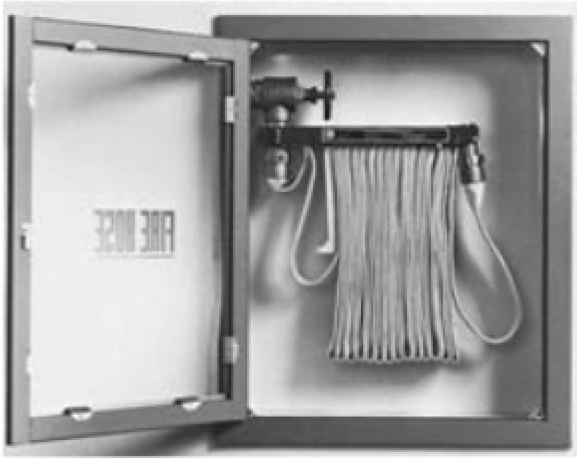
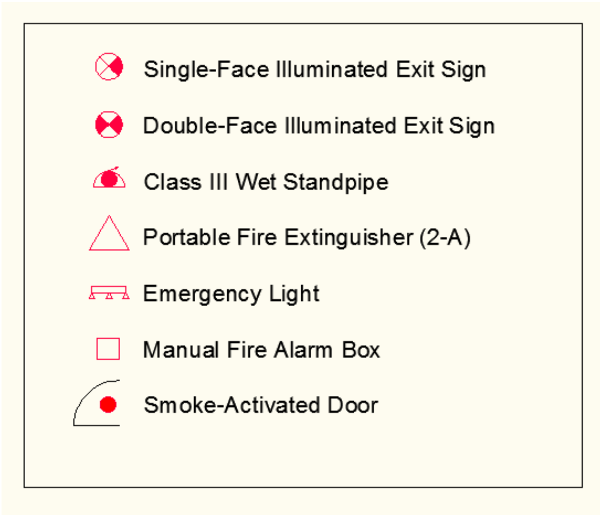


Figure 33 A Sample of Class III Standpipe System (Puchovsky, 2009)

Portable Fire Extinguisher

For office building without a sprinkler system, portable fire extinguishers are required to be provided in each floor per section 906.1 of the *IBC 2009*. Base on the classification of potential fire and classification of hazard from *NPFA 10: Standard for Portable Fire Extinguishers*, 2010 edition, the potential fire was classified as Class A fire while the classification of hazards is light/low hazards. Therefore all portable fire extinguishers shall be 2-A extinguishers. It has a 3000ft^2 as the maximum coverage area and the maximum travel distance to another extinguisher shall not greater than 75 ft. All extinguishers shall be located in locations where they will be readily accessible and immediately available for use without any obstruction from view. Based these requirements, the selected locations of extinguishers are shown in both Figure 34 and 35. If cabinets are used to store the extinguishers, they shall not be closed. The required location of an extinguisher depends with weight. Based on the *IBC 2009*, an extinguisher weighs 40 pound or less, it shall be installed so that their tops are not more than 5 ft above the floor, while that for heavier extinguisher, their tops cannot be more than 3.5 ft above the floor. And both have a floor clearance of not less than 4 in.



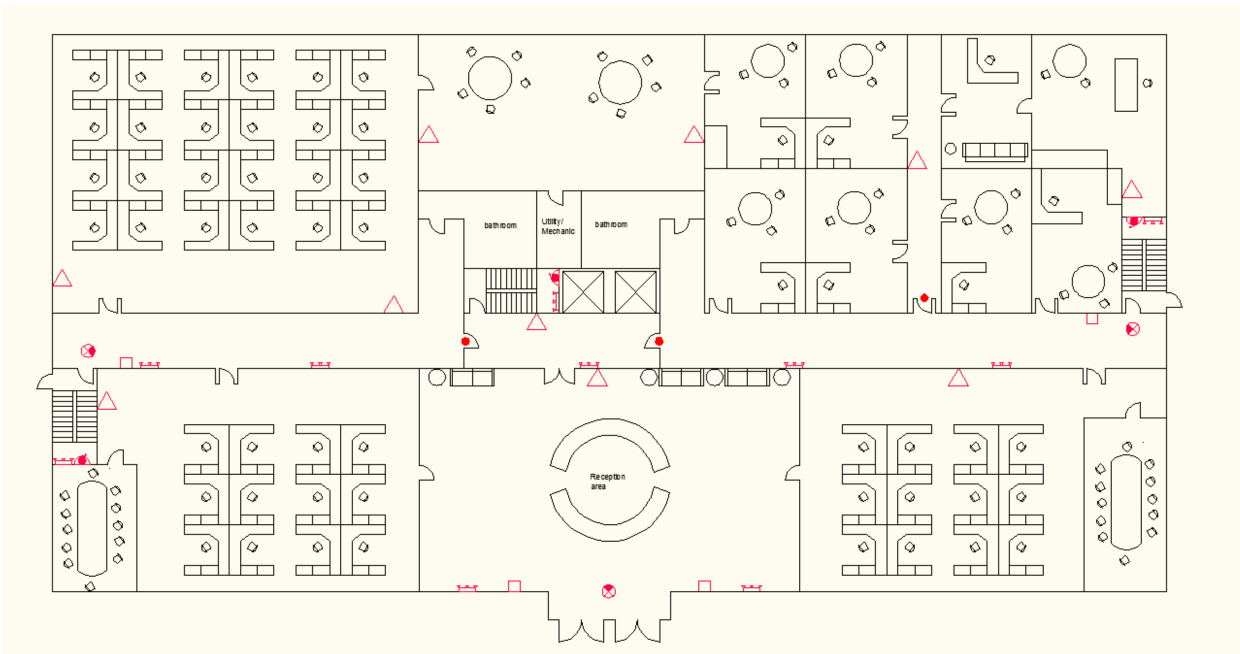


Figure 34 Active Fire Protection Systems for 1st Floor

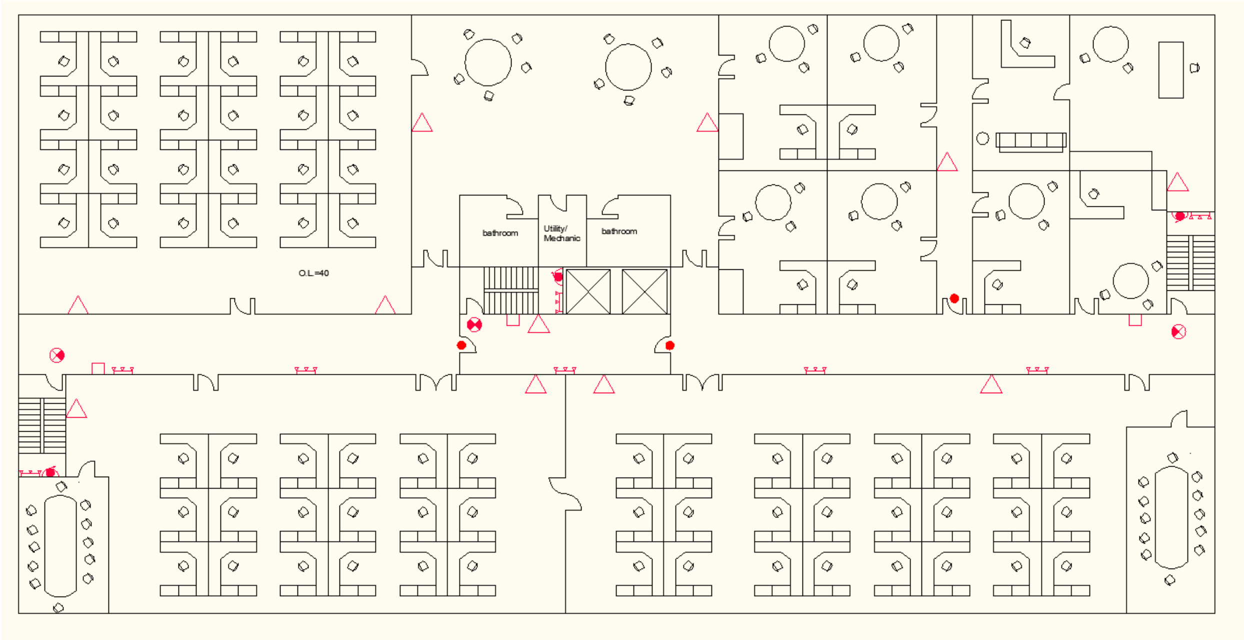


Figure 35 Active Fire Protection Systems for 2nd to 4th Floor

8.3 Building with Sprinkler Systems

8.3.1 General Requirements

Occupancy Classification

Every floor of this building will be used as office space. Although the building code provisions required classifying each area separately, all the small rooms which are not used as office space are all accessory use area. The codes require that all the accessory use area shall be classified as the predominate classification. Therefore, all the areas are classified as Group B (business).

Building Area and Height Limitation

For a Group B building, the *IBC 2009* has specific limitations on building area, height and the number of story. Since this building is 42 ft with 4 stories where each story footprint is 20,000 ft^2, the construction type must be at least Type II B. For this four story office building, the construction is required to be at least Type II B.

Construction Type

As defined by the *IBC 2009*, all the exposed steel structural members shall be classified as Type IIB with 0 hour fire-resistance rating. Therefore, all the open web joists and W-shape girders are classified as Type IIB construction. It should be note that, with sprinklers, Type II B construction is permitted for this building while as shown in the non-sprinklered building code analysis, at least Type II A construction is permitted. However, since all columns are composite columns (concrete enclosed), their fire resistance rating are still with 1 hour fire-resistance rating based on Table 721.5.1(7) of the *IBC 2009*.

Means of Egress

Number of Exits

Based on the occupant load estimation of 200 persons per floor, a minimum two exits are required for each floor. In this building three exits are provided at each floor.

Exit Capacity

The exits shall have a certain width to ensure the flow of people can during an evacuation easily be accommodated. As a component of the exit system, the corridor is as wide as 10 ft which is much greater than the required minimum width of 44 in. Furthermore, all doors in this building have uniform clear width of 32 in, while the stairs are 44 in in width. With three stairs for 2^{nd} to 4^{th} floor and three exits doors in which one is 4-leaf door, exits provided adequate capacity.

Exit Access Travel distance

For a fully sprinklered building, the maximum exit access travel distance is 300 ft. However, for this building at each floor, the maximum distance is 130 ft which is less than the limitation as shown in Figure 30 and 31.

Common Path of Egress

The length of common path of egress is limited to not greater than 100 ft. In this office building, the longest common path of egress is 50 ft and they are shown in Figure 36 and Figure 37.

Exits Remoteness

For a fully sprinklered building, the *IBC 2009* requires the distance of two exits for an area shall be at least one third of the maximum diagonal length. This requirement had been fulfilled while the team revised the floor plans.

Corridors

The center corridor is a principle exit access in every floor of this building, playing a critical role in means of egress. As mentioned before, the width of corridor is 10 ft which is far greater than what is required. Therefore, the capacity of the corridor is adequate. Furthermore, as mentioned in the section of floor plan revision, the corridor does not have any obstructions due to open doors installed on the corridor walls.

Elevator Lobby

Since the elevator serves more than three stories, as an elevator lobby is required at each floor the *IBC 2009*. And a direct means of egress shall be provided in elevator lobby. The team had applied these requirements while in floor plan revision process. It should be noted that, although it is not required by the codes, the elevator lobby can be utilized as an area of refuge if the doors close automatically, and the enclosure walls can perform as smoke partitions.

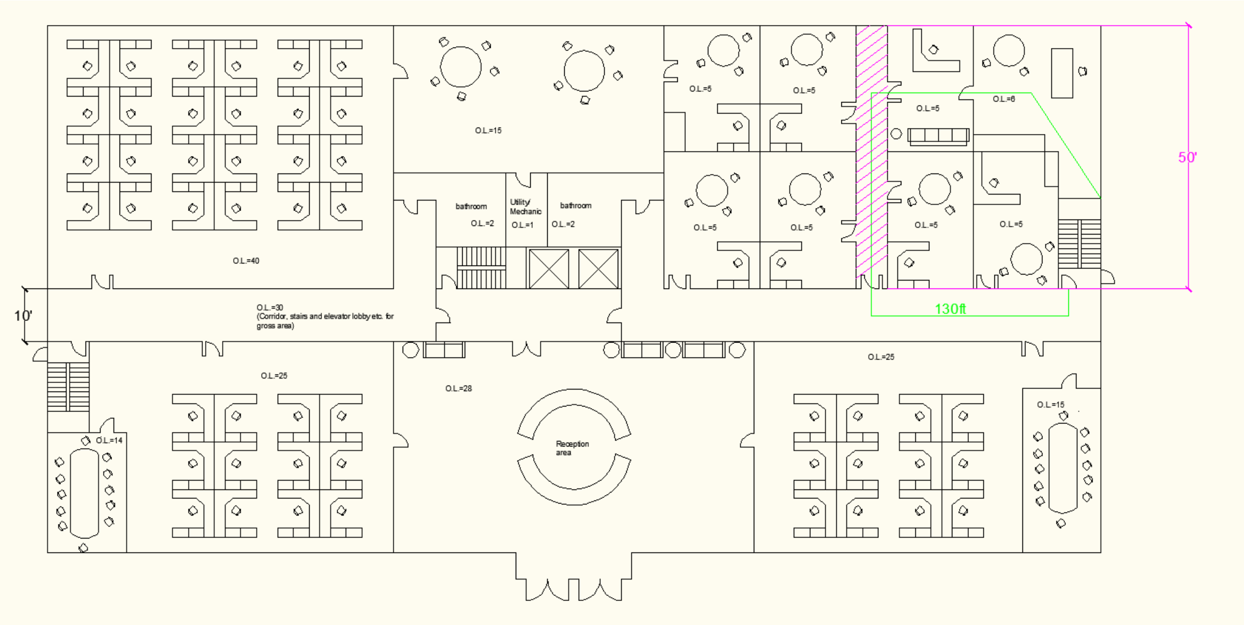


Figure 36 First Floor Plan (Green line stands for maximum travel distance; pink hatch stands for the maximum common path of travel; O.L. is occupant load)

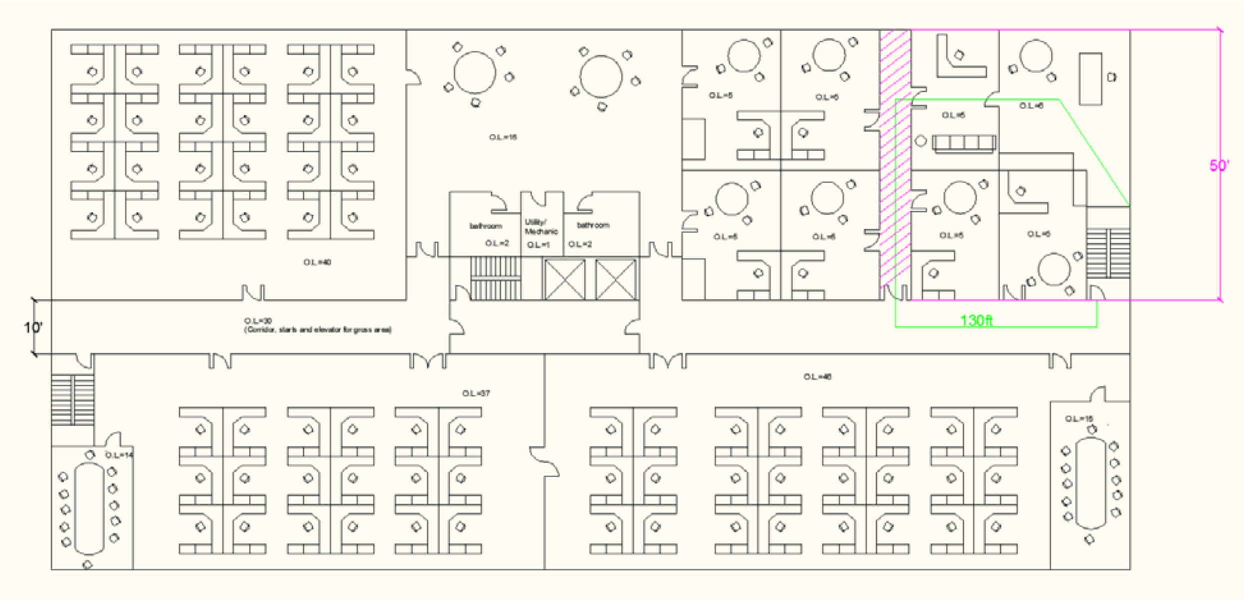


Figure 37 Second to fourth Floor Plan (Green line stands for maximum travel distance; pink hatch stands for the maximum common path of travel; O.L. is occupant load)

Illumination of Means of Egress

The *IBC 2009* provisions require that the means of egress is illuminated with 11 lux at all times. Therefore, light shall be provided all times at stair ways, corridors and entrance hall at first floor. As discussed in previous section, the team suggested emergency power generator for all general use, including illumination.

Exit Signs

While the people working in this building will be familiar with the locations of exits, they will not be that obvious for the visitors. Therefore, exit signs are essential for the people way-finding. The signs must be illuminated at all times, and connected to the emergency power generator to ensure at least of 90 minutes illumination during a power outage. The location of exits sign is regulated by *NFPA 101*, and three typical placements criteria are shown in the figure below.

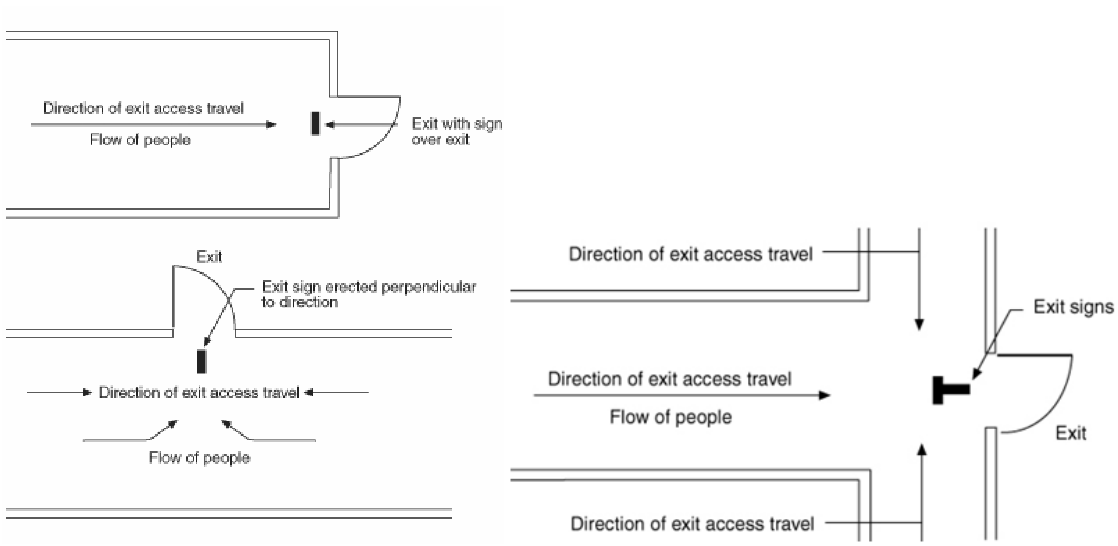


Figure 38 Locations of Exit Signs (taken from Figure A.7.10.1.2.1, NFPA 101)

8.3.2 Passive Fire Protection

Fire Barriers

The *IBC 2009* specifies the location of fire barriers and the fire-resistance rating of these fire barriers. As required, the elevator shaft and the stair ways shall be constructed and enclosed by at least 2-hour rating fire barriers, and the center corridor is not required to be protected by fire barriers any more. Therefore, the fire-resistance rating for the corridor walls can be 0 hour. In Figure 39 and 40, all the fire rated walls are indicated with blue dots while one blue dot means 1 hour rating and two blue dots mean 2 hour rating. Furthermore, all doors on the fire barriers shall be fire doors with a corresponding fire-protection rating. As required by the *IBC 2009*, fire doors enclosing stair ways shall be 1 ½ hour rating. Besides the fire-protection rating, all fire doors shall be self or automatic-closing doors. And the fire doors located at elevator lobby and the door at the end of the 50 ft corridor shall be smoke-activated doors. In Figure 39 and 40, 20 min rated doors are indicated with green dots and 1 ½ hour rating doors are indicated with green triangles. And in Figure 43 and 44, self-closing doors or smoke activating doors are indicated with red dots. Moreover, any penetrations, ducts and air transfer openings through fire barriers shall be protected appropriately by rated seals and fire or smoke dumpers. In order to ensure the continuity of fire barriers, appropriate seals shall be applied at the bottom of floor assembly to cover the gap.

Interior Wall and Ceiling Finish

Based on the occupancy, the presence of sprinkler system and the area of the floor, wall and ceiling finish are restricted by the *IBC 2009* to control the fuel load. As required by the *IBC 2009*, the finish shall be at least Class C.

Interior Floor Finish

For building with a sprinkler system, the requirements to floor covering are lower. According to the *IBC 2009,* the floor finish at corridors and stair enclosure is required to be complying with the Federal standard DOCFF-1-70 (Surface Flammability of Carpets and Rugs – Methenamine Pill Test).

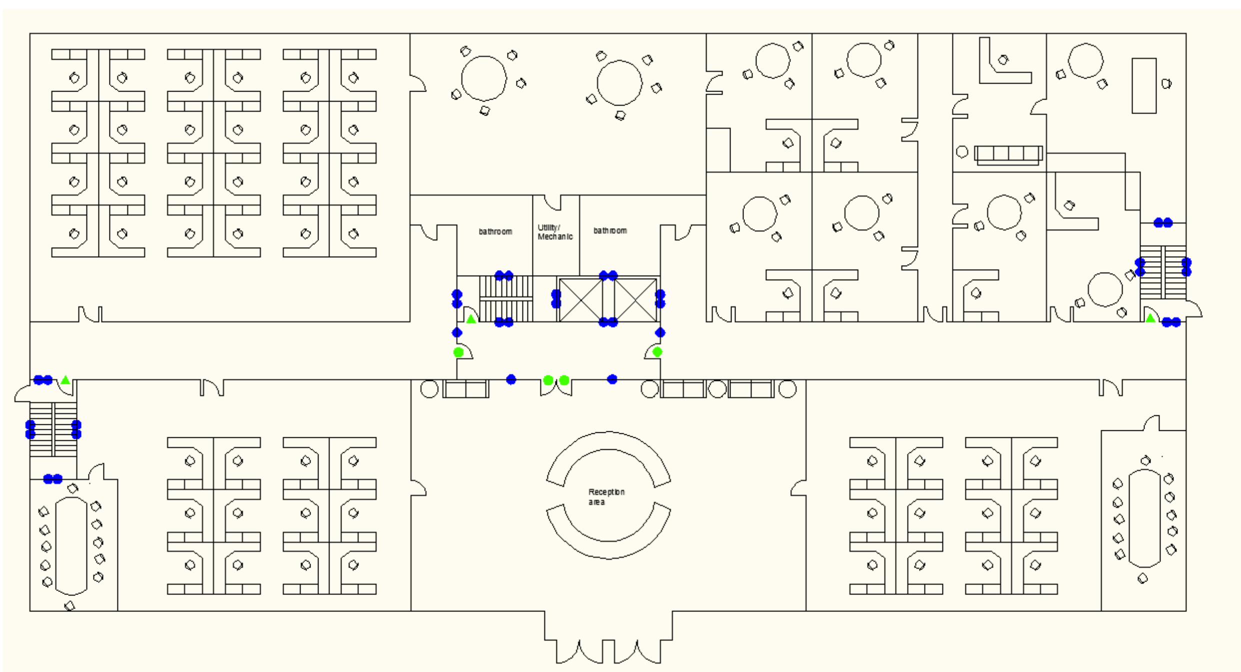


Figure 39 Location of Fire Doors and Fire Rated Walls for First Floor

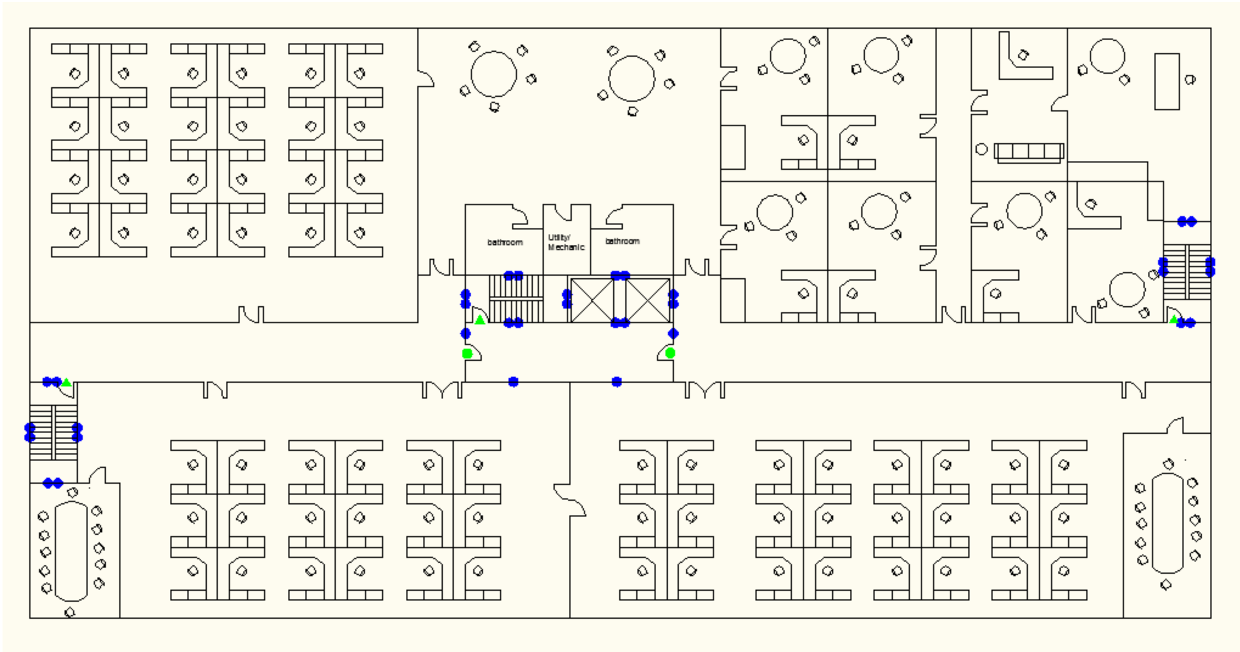


Figure 40 Location of Fire Doors and Fire Rated Walls for Second to Fourth Floor

8.3.3 Active Fire Protection

Sprinkler System

The sprinkler system shall be designed and installed in according to *NFPA 13: Standard for the Installation of Sprinkler Systems*. The detailed design and installation requirements will be covered in a later section.

Fire Detection and Alarm System

For a Group B building which is equipped with a sprinkler system, manual fire alarm systems are not required to be installed. However, at least one manual fire alarm box shall be installed to send signal to fire alarm system. The locations of manual fire alarm boxes are also regulated by the *IBC 2009*: 1) maximum distance from the entrance of an exit is 5 ft; 2) vertically distance from the floor level shall be in the range of 42 in to 48 in; 3) it shall be provided in public areas and common areas that it is visible. An illustration of location of a manual fire alarm box is shown in Figure 41, Figure 43, and Figure 44. Furthermore, all manual fire alarm boxes are required to be red in color with a red or transparent protective cover, and produce a sound pressure 15 dBA above the average ambient sound level or 5 dBA above the maximum sound level having duration of 60 seconds, but not exceed 110 dBA. In order to make sure the manual fire alarm boxes function well, they shall be connected to the emergency power generator.

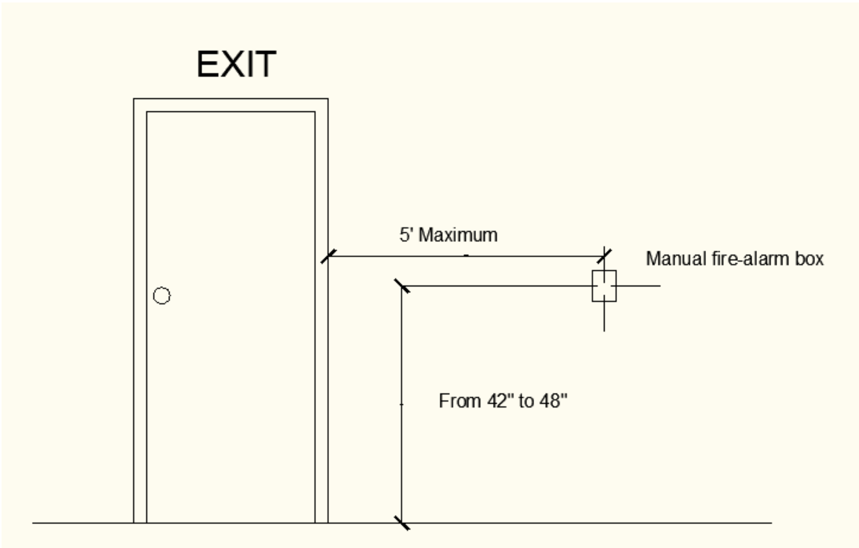


Figure 41 Location of Manual Fire Alarm Box

Fire Department Connection

The fire department connection allows the responding fire fighters to boost sprinkler system pressure and deliver more water to operating sprinklers by pumping the water from the fire hydrants on the street. It is required to have a fire department connection for this building. It is required to be located on the street side of the building, and there cannot be any obstruction for fire apparatus and hose.

Standpipe System

Class I standpipe systems shall be installed throughout the building, and they are allowed to be combined with automatic sprinkler systems. Not like Class III system, Class I standpipe is merely a 2 1/2-inch hose connection to supply water for use by fire fighters, and it is shown in Figure 42. In order to provide a safer environment for fire fighters, all standpipes are located in stairway, at an intermediate floor level landing and all locations are shown in Figure 43 and 44.

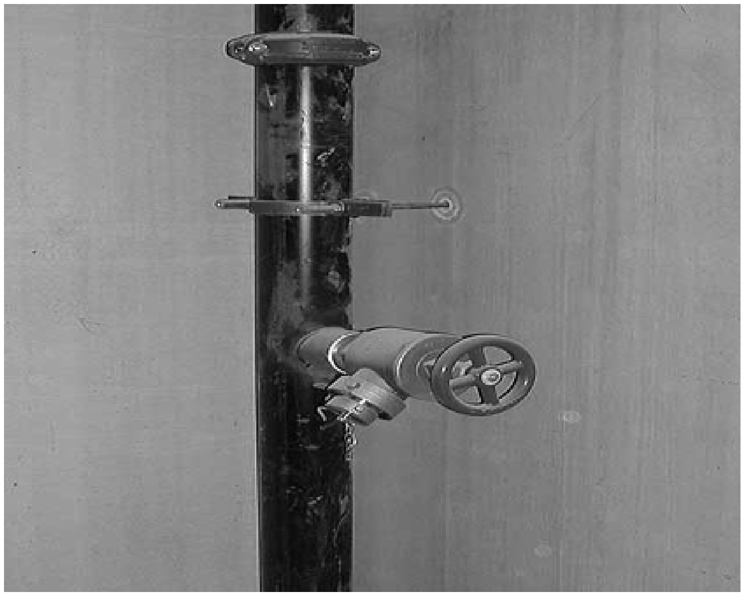
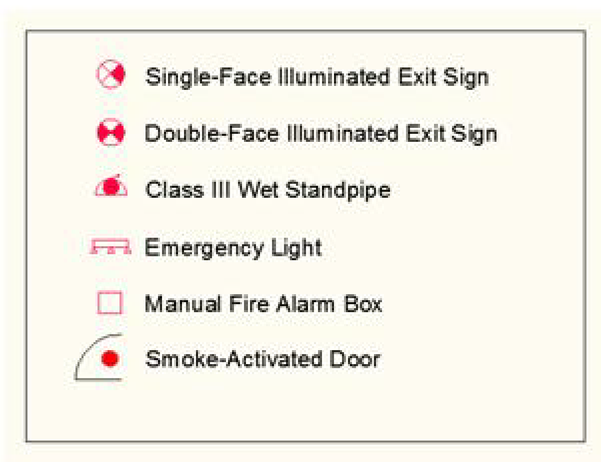


Figure 42 Class I Stand Pipe

Portable Fire Extinguisher

Since the office building is equipped with sprinkler system, portable fire extinguishers are not required anymore.



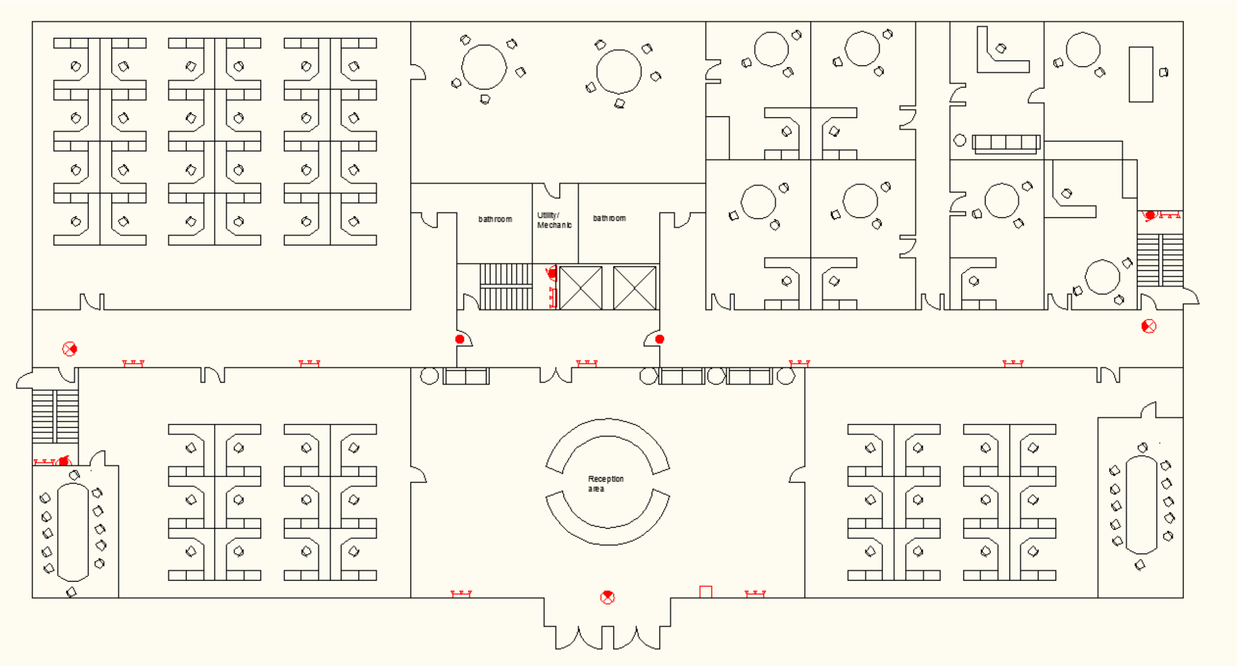


Figure 43 Active Fire Protection Systems for 1st Floor

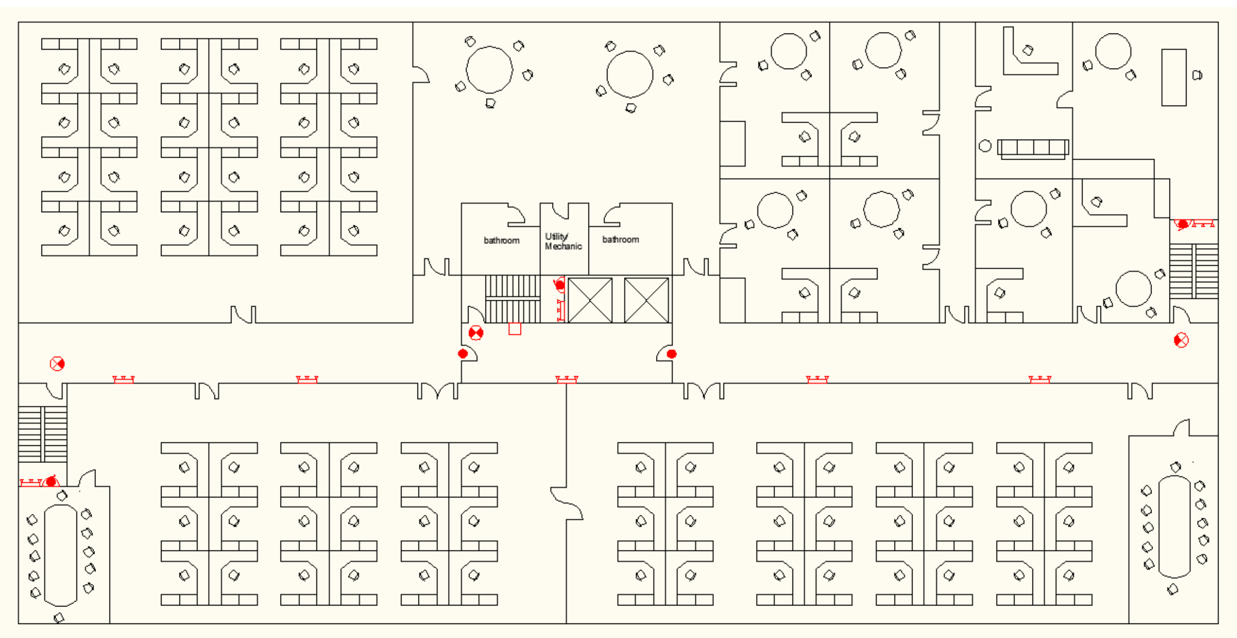


Figure 44 Active Fire Protection Systems for 2nd to 4th Floor

8.4 The Selection of Alternative Fire Protection System

There are many advantages that can be gained by including a sprinkler system within a building's fire protection system. First of all, automatic sprinklers are highly effective elements for achieving fire safety. For example, when sprinkler systems are installed, the chances of dying in a fire and property loss per fire are cut by one-half to two-thirds, compared to the number of fire deaths reported by the fire departments where sprinkler systems are not present. (John R. Hall, 2008). Actually, the data for this comparison were collected regardless of type, coverage, or operational status of the sprinkler. When sprinklers are properly installed and maintained at good operational status, the effectiveness should be more impressive. (John R. Hall, 2008) Second, by installing a sprinkler system, owners and architects have more flexibility in design and layout while achieving the code compliance.

On the other hand, building codes usually have more strict requirements when sprinkler systems are not present. For instance, building codes permit building materials with reduced fire-resistance rating, such as interior finish, to be used in sprinklered buildings. Architects usually are restrained by strict code provisions when sprinklers are not going to be installed. For example, the lengths of dead corridors are much longer if they are protected by sprinklers; travel distance and common path of travel are also reduced if it is not protected by sprinklers. Furthermore, aside from this specific office building, with sprinkler systems, building designs with sprinkler systems are allowed greater flexibility in maximizing the building size which is attractive for the owner since usually more profit can be gained from more rentable space. Third, from an economic point of view, a building with sprinkler systems is much beneficial. The team conducted cost estimation for both fire protection systems (with sprinkler systems, without sprinkler systems) for this four-story office building. As shown in the tables below, it turned out that with sprinkler system, owner can save about $23,000. Table 17 and Table 18 revealed that the key saving is fireproofing spray, fire rated wall constructions, and fire rated doors. In other words, as more money is invested in active fire protection, or sprinkler systems, the code provisions require less in passive fire protection. And the net result is saving more money. However, this cost estimation was only a rough estimation (detailed information is available in Appendix G)

Table 17 Cost Estimation for Non-Sprinklered Building

Non-sprinklered Building					
Components		Unit Price	Applied Quantity	Cost	Total Cost
Fire Rated Walls	1-hour walls	$2.10/sqft	15990spft	$33600	$256000 Or $3.20/ft^2
	2-hour walls	$3.36/sqft	8484 sqft	$28500	
Fire Doors	20-minutes doors	$413 each	51	$21100	
	1 1/2-hour doors	$460 each	12	$5520	
Standpipes	Class III	$739each	9	$6650	
Manual Fire Alarm boxes		$29 each	13	$377	
Fireproofing Spray	Girders & OWJ	$1.95/ sqft	35535sqft	$69300	
	decking	$1.48/ sqft	60000sqft	$88800	
Portable Fire Extinguisher		$54 each	43	$2320	

Table 18 Cost Estimation for Sprinklered Building

Spinklered Building					
Components		Unit Price	Applied Quantity	Cost	Total Cost
Fire Rated Walls	1-hour walls	$2.10/sqft	3780spft	$7940	$233000 Or $2.90/ft^2
	2-hour walls	$3.36/sqft	8484 sqft	$28500	
Fire Doors	20-minutes doors	$413 each	10	$4130	
	1 1/2-hour doors	$460 each	12	$5520	
Standpipes	Class I	$345 each	9	$3110	
Manual Fire Alarm boxes		$29 each	4	$116	
Fireproofing Spray	Girders & OWJ	$1.95/ sqft	0 sqft	$0	
	decking	$1.48/ sqft	0 sqft	$0	
Portable Fire Extinguisher		$54 each	0	$0	
Sprinkler System	First floor	$2.77/sqft	20000 sqft	$55400	
	Other floors	$2.14/sqft	60000 sqft	$128000	

In some cases, by installing sprinkler, the required construction type can be changed. And significant amount of money can be saved for construction materials and corresponding labor cost. An example of estimated construction costs for both sprinklered and non-sprinklered 40,000 ft^2 one-story office building can be found in *Designer's Guide to Automatic Sprinkler Systems* by Robert M. Gagnon (2005). In this example, the construction was required to be Type II A without sprinkler protection while the construction can be any type when sprinkler systems are installed. It shows that by installing sprinkler system and with Type III construction, an owner can save about $ 715,600.

Sprinkler systems will be even more attractive considering about the tax and insurance. The cost of sprinkler system is a deductible item on the federal income tax at the corporate rate. And the interest on a building improvement loan to install a sprinkler system is completely deductible from federal taxes. Furthermore, some state governments offer a straight percentage property tax rebate for buildings equipped with sprinkler systems. Property insurance for building owner and tenants is usually lower with sprinklers, and contents insurance for occupants is usually lower. (Gagnon, 2005) Therefore, given these construction cost saving and financial incentives, the team recommended installing sprinkler systems for the entire building.

Chapter 9: Smoke Detector System Design and Performance Analysis

9.1 Introduction

Recall from the code analyses, smoke detectors are needed to initiate closure of the smoke activation doors. Furthermore, smoke detectors are usually activated before sprinklers. Or in other words, the alarm from smoke detectors can provide occupants more time to exit the building. In this chapter, the design of smoke detector system focused on the locations for their installation. Utilizing fire dynamics to simulate the fire environment such as plume temperature, ceiling jet temperature and velocities, and smoke concentrations, calculations for the response time of smoke detectors under design fire conditions were performed by numerical method. The response time values were compared with sprinkler activation time, and these comparisons are presented in later chapters. The comparison can tell if the smoke detectors work adequately, in that they will activate before sprinklers in the presence of fire.

9.2 Smoke Detector Installation

The locations of smoke detectors were selected by the requirements in *NFPA 72: National Fire Alarm and Signaling Code*. The *Model 4W-B* photoelectric smoke detector from *System Sensor* was selected for this building. *Model 4W-B* is a commonly used smoke detector for office building, and it is also listed or approved by *Underwriters Laboratories* and *Factory Mutual* who are the major certificate companies for fire protection products. Not like installation requirements for sprinklers, the spacing of smoke detector is not regulated in related codes. However, a 30 ft spacing is recommended in Annex B of *NFPA 72*. Therefore, the team laid out the detectors with a 30 ft X 30 ft grid system. Furthermore, modifications had been made to the grid system to ensure that all accessible areas with combustible materials, as well as elevator shaft and stair shafts, are all covered by detectors. Moreover, it should be noted that smoke detectors on ceilings cannot be installed within 4 inches of a wall. The results of smoke detector system layout are shown in Figure 39 and 40, where the red dots are the smoke detectors.

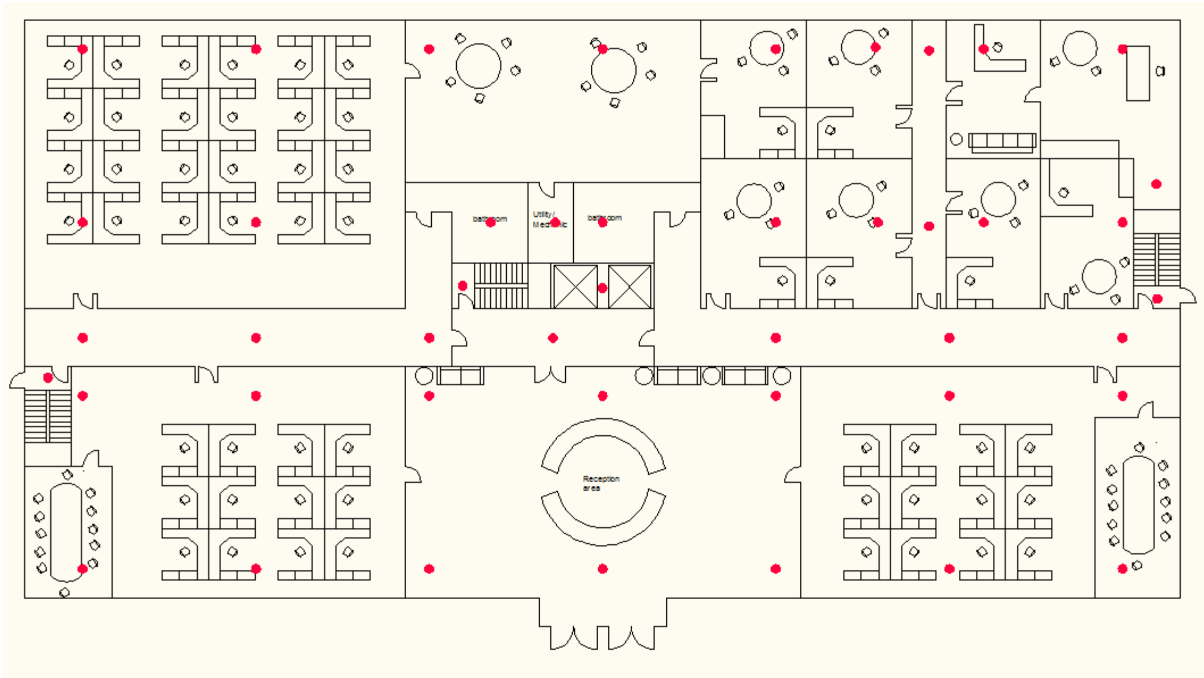


Figure 45 Smoke Detector Layout for 1st Floor

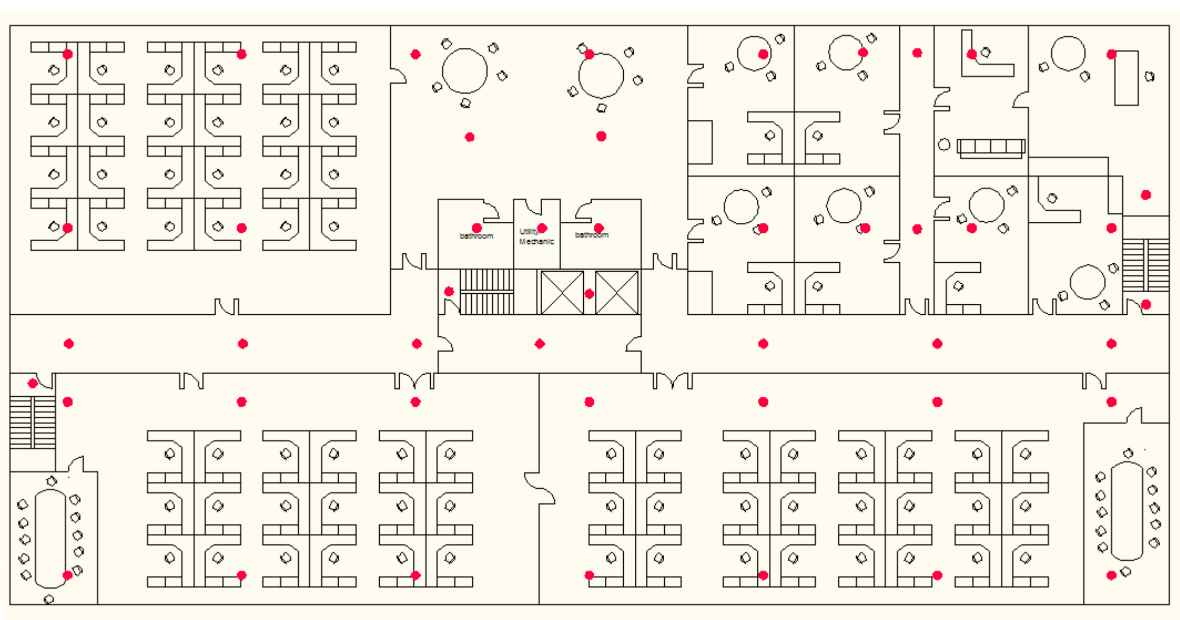


Figure 46 Smoke Detector Layout for 2nd to 4th Floor

9.3 Smoke Detector Performance Analysis

The response time of a smoke detector largely depends on the quantity of soot produced by fire, the ceiling height (3.11m for first floor and 2.5m for the upper floors), and the distance from the smoke detector to fire. The ceiling jet from the rising fire plume plays an important role in transporting the smoke from where the plume hits the ceiling to other place along the ceiling. And the plume depends on the size of fire. Therefore, it is very important to select an appropriate design fire for this analysis.

Considering the potential fire severity in different area of this building, it is obvious that the work station poses the biggest fire hazard. And in the workstation area, the furthest distance from fire to detector is 21.2 ft. Therefore, it is appropriate to analyze the smoke detector performance in the workstation area. A schematic drawing is shown below, where the left is an overhead view and the right is a profile view.

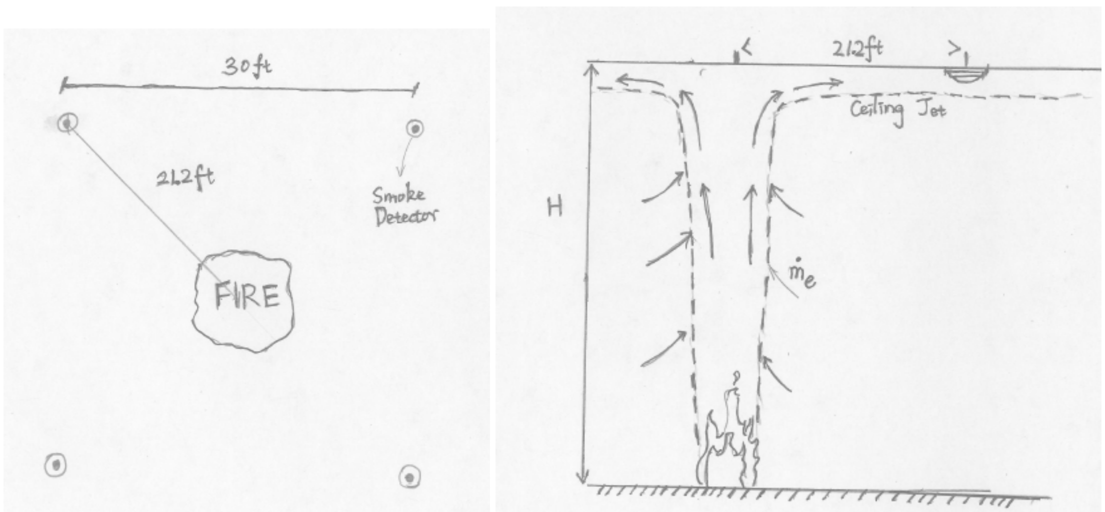
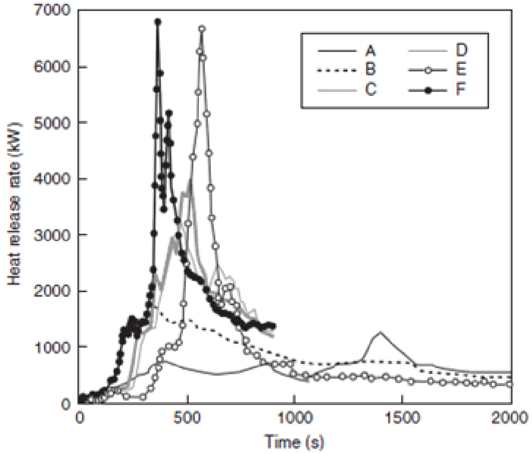


Figure 47 Schematic Drawings

For the characteristics of workstation fire, several tests had been conduct by NIST (National Institute of Standards and Technology). The *SFPE Handbook* collected some of the tests results which are shown below in Figure 48.



Code	Combustible Mass (kg)	Description	Number of Sides with Acoustic Panels	Reference
A	291	Mostly old-style wood furniture	0	110
B	291	Semi-modern furniture	1	110
C	335	Modern furniture	2	112
D	—	Modern furniture	3	112
E	291	Modern furniture	4	110
F	—	Modern furniture	4	112

Figure 48 Office Workstation Fire Test (taken from the SFPE Handbook, Figure 3-1.38 and Table 3-1.11)

In this case, scenario F which is the most severe fire hazard was selected for performance analysis. By applying fire growth power law, a growth rate of 0.0425 kW/sec^2 was obtained for scenario F. According to the *Three Panel Workstation Fire Test* conducted by NIST (available at http://www.fire.nist.gov/fire/fires/work3/work3.html), at the beginning stage of this fire, the burning materials are the finish on the chairs and panels. As fire grows, entire chair, panels, computer monitor, trash bin were involved in this fire.

For this analysis, the team assumed that the detector would be activated at the early stage of the fire. Therefore, the burning materials can be simplified as the finish materials on the chair and panels. Furthermore, the team assumed the finish material is nylon for both chair and panels. The burning properties of nylon are available from Table 3-4.14 of the *SFPE Handbook.*

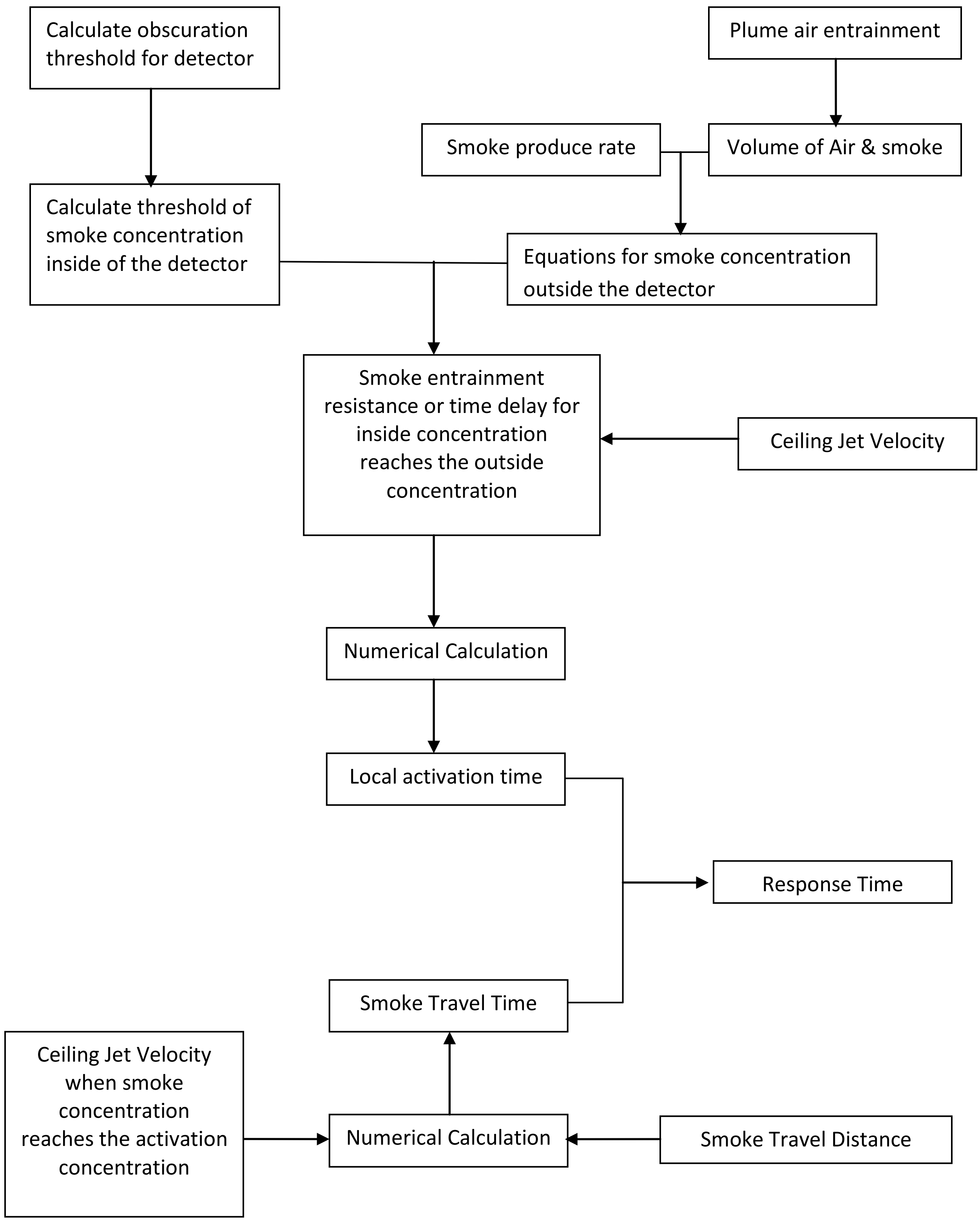
The response time can be studied in two different parts. The first part is the smoke travel time. It accounts for the time for smoke which reaches the activation concentration to travel from the point where the plume hits the ceiling to the specific detector. The second part is the local activation time. The local activation time refers to elapsed time for the smoke concentration inside the detector to reach the outside concentration which is sufficient to activate the detector. With the characteristic of the design fire and the geometry layout of detector, numerical calculations were conducted for solving the response time. The major equations for the calculations are listed in Table 19.

Table 19 Governing Equations

$\dot{m}_{ent} = C_e\left(C_p T_\infty\right)^{-\frac{1}{3}}(\rho_\infty\sqrt{g})^{2/3}\, \dot{Q_{con}}^{1/3} H^{5/3}$	<u>Entrainment</u>, taken from Chapter 10 of *Fundamentals of Fire Phenomena* by James G. Quintiere
$u = 0.195\frac{(\frac{\dot{Q}}{H})^{1/3}}{(\frac{r}{H})^{5/6}}$	<u>Ceiling jet velocity</u>, taken from Section 2 Chapter 2 of *SFPE Handbook*
$C_{i,n} = \frac{1}{\tau}\left(C_{o,n} - C_{i,n-1}\right)\,(s^{-1}m^{-1})$	<u>Smoke concentration inside detector at time step n</u>, taken from Annex B of *NFPA 72*

The calculation flow charts are also provided in following page while the detailed equations and calculations are in Appendix H. Based on the calculation, the smoke detector response time for first floor is 65.5 seconds while for second to fourth floor, the time is 42.2 seconds. The difference is primarily caused by difference in ceiling height, where the ceiling height for the first floor is 12 ft while 10 ft for second to fourth floors. It requires more time for fire plume reaches the ceiling when ceiling is higher. Furthermore, the ceiling jet velocity is smaller where the ceiling is higher. Therefore, with same travel distance, smoke needs more time to the specific detector for a higher ceiling.

Activation Time



Chapter 10: Automatic Sprinkler System Design

10.1 Introduction

According to the *International Building Code 2009 Edition*, sprinklers systems are not specifically required for office buildings, although they are required for buildings with stories more than 55ft above the lowest level of fire department vehicle access (the *IBC 2009*: 903.2.11.3, 2009 Ed.). The maximum height of our building is 42ft; therefore a sprinklers system is not required. In Chapter 6 code analyses for an office building not protected by a sprinkler system and fully sprinklered were completed. In order to appreciate the cost and level of protection a sprinkler system can provide; compared to passive protection measures, we designed a system specific to our building.

10.2 System Design and Layout

We designed two wet-pipe tree systems; one with the cross mains running along the East and North walls of the office and a second with the cross main running directly along the central corridor. This end result of these designs reveals how the layout of a sprinkler system affects cost and performance. The design of our systems are in accordance with the 2009 edition of *NFPA 13, Standard for the Installation of Sprinkler Systems*, as it is acceptable for use by the *International Building Code* (the *IBC 2009*: 903.3.1.1, 2009 Ed.).

The first and most important concept in sprinkler design is occupancy classification. This classification recognizes the type of environment present in the building and level of protection that must be provided by the sprinkler system. If the designer selects the incorrect occupancy the system is very likely to fail and the goal of either suppression or control will not be achieved. When the correct occupancy class is chosen, the correct sprinkler system and components can be used to match the hazards present. In addition to the system and components the occupancy class determines the size and required density of the design area (Futrell, 75).

Once the correct occupancy was chosen and a defined design area and density we determined the size of the piping and evaluate our water supply. First, we used the pipe schedule method to achieve preliminary pipe sizes. We then applied the use of hydraulic calculations to resize our piping system and determine the total flow and pressure requirements for the water supply. By evaluating the water supply we confirmed that adequate water and pressure can be delivered to our sprinkler system and fire control can be achieved. The procedure for our sprinkler system design is outlined by the flow chart below. The major components of the design are shown from classification of occupancy to sprinkler piping layout. The design for area and density along with the hydraulic calculations are detailed in upcoming sections.

10.2.1 Sprinkler System Design Flow Chart

Classify Occupancy and Commodity Hazards
(Chapter 5)

↓

Select Material for Above Ground Pipe
(Section 6.3)

↓

Wet Pipe System Requirements
(Section 7.1)

↓

Determine Installation Requirements (Chapter 8)

Basic Requirements (Section 8.1)

Area Limitations (Section 8.2)

Temperature Ratings (Section 8.3.2)

Thermal Sensitivity (Section 8.3.3)

Sprinkler Type Application (Section 8.4.1)

Position, Location and Spacing (Section 8.5)

10.3 System Design Results

The design of this sprinkler system was based on the minimum requirements from *NFPA 13* for light hazard occupancy. We decided to follow the minimum requirements as closely as possible because of the dispute between the effectiveness of light hazard systems for offices. It has been debated that the light hazard requirements are too lenient for offices, seeing as the fuel loads can be quite high. Thus the required density would be too low to for effective protection. Using the minimum requirements allowed us to see how well the system performs under these requirements (Futrell, 79).

The requirements for light hazard occupancies allow the designer to make some decisions for certain criteria. We chose to use ordinary temperature sprinklers because the ambient temperature in offices generally does not exceed 100F. Quick response sprinklers were chosen because test results comparing quick response and standard response sprinkler in office environments show some evidence that quick response sprinklers can improve life safety and property protection (Walton and Budnick, 24). The sprinkler office size we chose was K-5.6 because the flow and pressure requirements are not very high for light hazard occupancies. Additionally, with quick response sprinklers, water will be applied shortly after a fire begins which allows for the sprinklers to control the fire with less water.

For the above ground piping we decided to use steel pipe. It is the most durable of all sprinkler piping materials and has the highest strength. The pipe would be much less likely to be damaged during construction and allow for quick and easy installation. Furthermore, steel pipe has fewer restrictions than other materials such as copper and plastic types. We would recommend that the steel pipe be coated with zinc, to prevent corrosion from occurring after system flushes, allowing for a protective covering from scale build up of the zinc. Steel piping will also perform well when exposed to fire; it will not melt or produce any toxic gasses, while PB and CPVC materials may. Steel pipe also has the least restrictive requirements for support spacing. This would be advantageous since long stretches of pipe will be needed for the office building. Steel is also relatively inexpensive and will not cost significantly more than other materials.

The requirements for light hazard occupancies and design we selected can be seen in Table 20. Figure 49 shows the requirements for pipe sizes using the pipe schedule method. The layouts of our sprinkler systems can be seen in Figures 50 and 51. Figures 52 and 53 show the pipe sizes from the pipe schedule method, also used in the initial hydraulic calculations.

Table 20 Sprinkler Design Criteria

Design Criteria	**Allowed Per NFPA 13**	**Used**	**Reference Section**
Occupancy Class	Light Hazard	Light Hazard	5.2
System Protection Area/Floor	52,000ft^2	20,000ft^2	8.2.1
Aboveground Pipe	Steel/Copper/CPVC	Steel	6.3
Thermal Sensitivity	Ordinary/Intermediate	Ordinary	8.3.2
Temperature Rating	Quick/Standard Response	Quick Response	8.3.3
Orifice Size	K5.6 or Greater	K5.6	8.3.4
Sprinkler Orientation	Upright or Pendent	Pendent	8.4.1.1
Max. Area Per Sprinkler	225ft^2	225ft^2	Table 8.6.2.2.1(a)
Max. Sprinkler Spacing	15ft	15ft	Table 8.6.2.2.1(a)
Max Distance From Walls	7.5ft	7.5ft	8.6.3.2.1
Min. Distance From Walls	4in	4in	8.6.3.3
Min. Sprinkler Spacing	6ft	10ft	8.6.3.4.1
Obstructions	None	None	8.6.5
Deflector Position	1in. Min/12in. Max	1in.	8.6.4.1.1.1
Design Area and Density	1500ft^2 and 0.1 GPM/ft^2	1500ft^2 and 0.1GPM/ft^2	Figure 11.2.3.1.1

TABLE 3.1

Light Hazard Pipe Schedules

Steel		Copper	
1 in.	2 sprinklers	1 in.	2 sprinklers
1¼ in.	3 sprinklers	1¼ in.	3 sprinklers
1½ in.	5 sprinklers	1½ in.	5 sprinklers
2 in.	10 sprinklers	2 in.	12 sprinklers
2½ in.	30 sprinklers	2½ in.	40 sprinklers
3 in.	60 sprinklers	3 in.	65 sprinklers
3½ in.	100 sprinklers	3½ in.	115 sprinklers
4 in.	See Section 8.2 of NFPA 13	4 in.	See Section 8.2 of NFPA 13

Figure 49 Pipe Schedule Method
(From *Designer's Guide to Automatic Sprinkler System*)

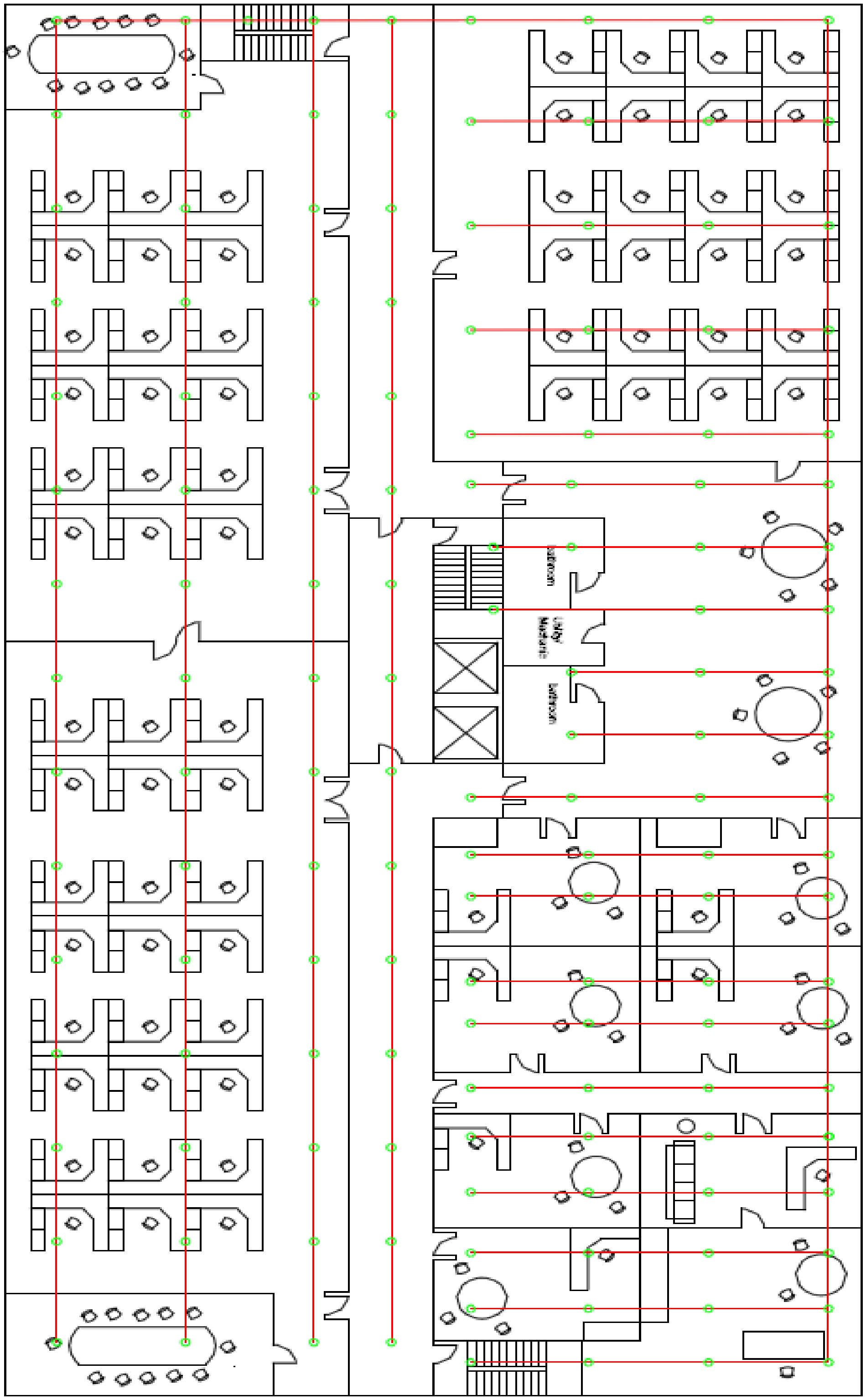


Figure 50 Sprinkler System 1 Layout

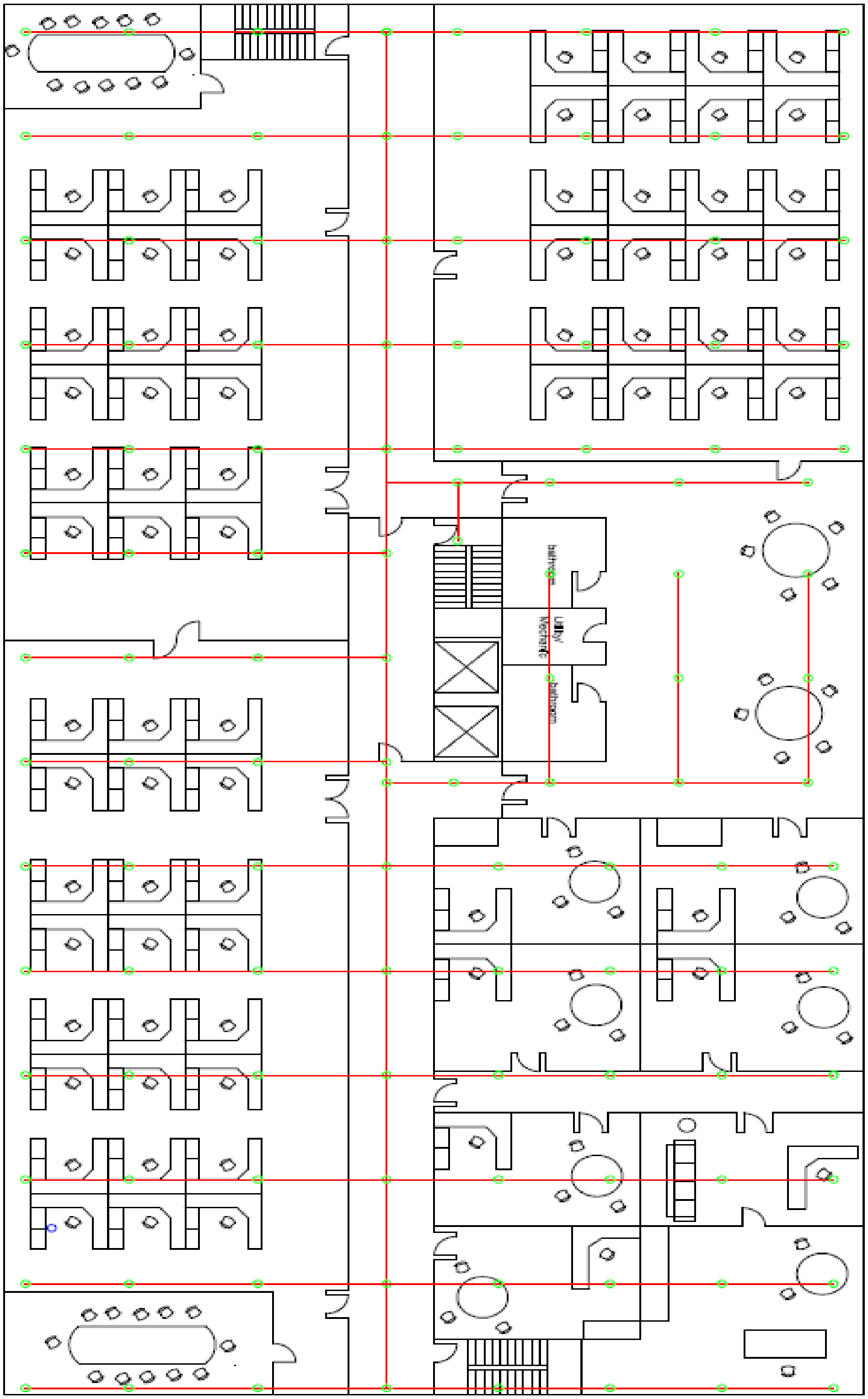


Figure 51 Sprinkler System 2

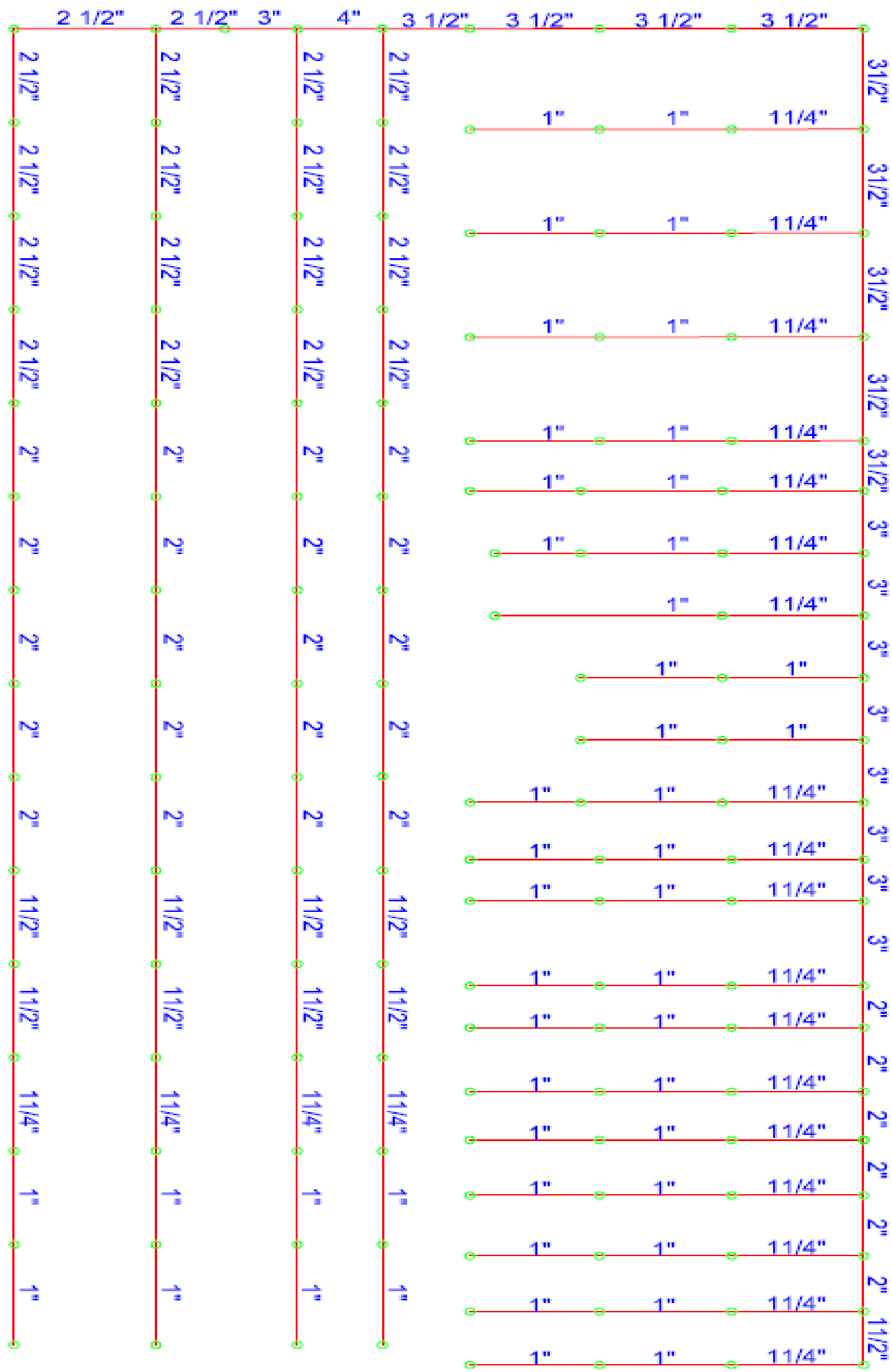


Figure 52 Sprinkler System 1- Pipe Schedule Method

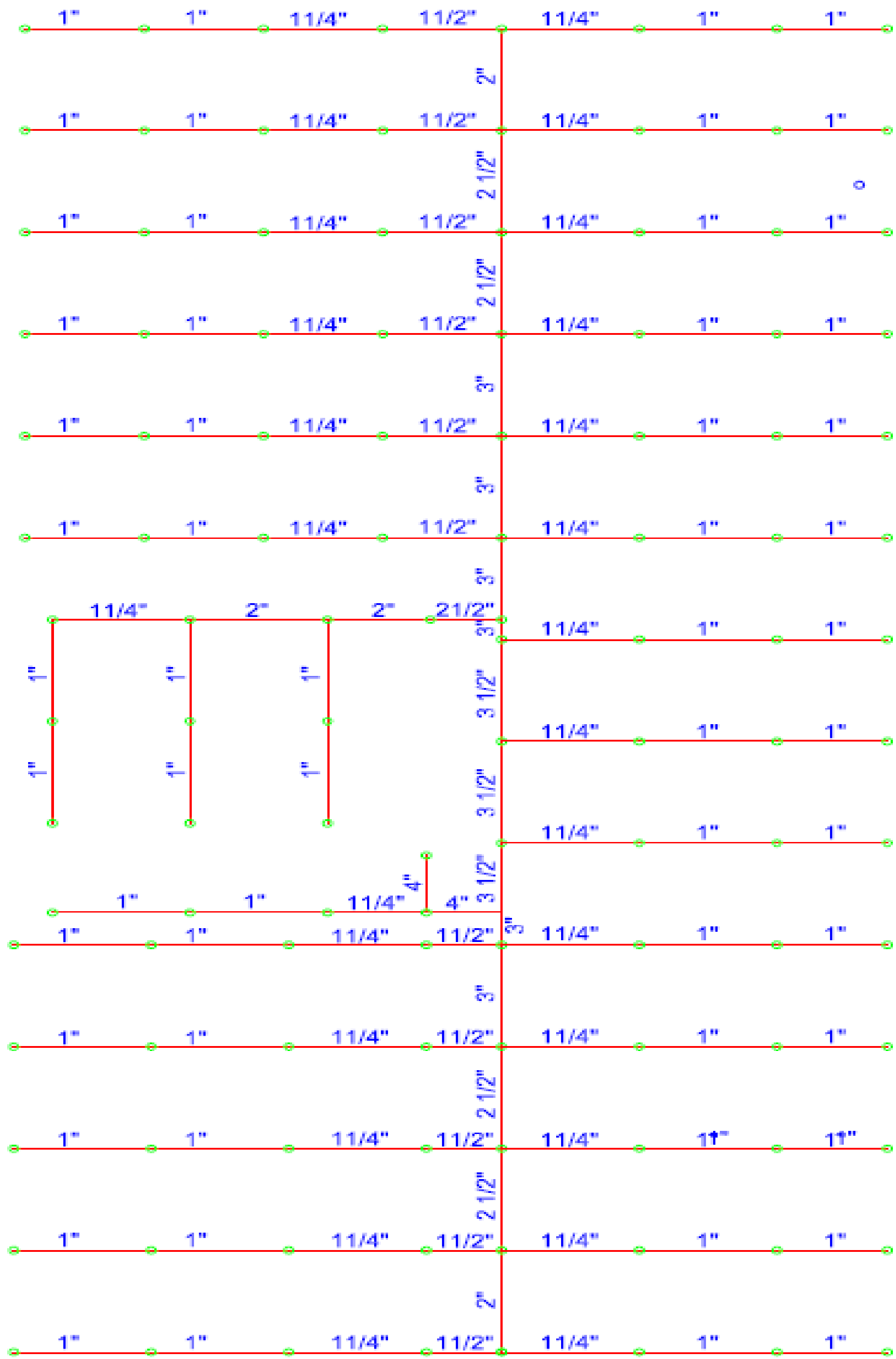


Figure 53 Sprinkler System 2- Pipe Schedule Method

10.4 Hydraulic Design Procedure

After the hazards present in the building have been identified and the basic layout of the sprinkler system was completed, we used the hydraulic calculations procedure from Chapter 22 of *NFPA 13* to resize the aboveground piping and determine the flow and pressure requirements for the system. The end result of the sprinkler calculations establishes the total demand of the system and provided a base to ensure that the piping and water supply can deliver this demand.

We decided to use the density/area approach to design our system. The method begins with the required density which is determined from Figure 11.2.3.1.1 for light hazard occupancy. The density gives the minimum flow that must be delivered from the most remote sprinkler in the system over a designated area. With the density and area known we can use equation 10.2:

$$\text{Equation 10.2- } (d)(A)=Q_m$$

Where "d" is the density in gpm/ft^2 and "A" is the area the sprinkler must cover. Multiplying these two variables gives the minimum flow "Q_m" from the most remote sprinkler in gallons per minute. The minimum pressure from the most remote sprinkler is determined using equation 10.3:

$$\text{Equation 10.3- } Q=K\sqrt{P}$$

Where "K" is the K-factor assigned to the sprinkler for the nominal orifice size and "Q" and "P" are the flow and pressure respectively. By solving for "P" we determined the minimum pressure required at the most remote sprinkler.

After we had the minimum flow and pressure required, we determined the design area of the system. This is the area that is hydraulically the most remote from the sprinkler riser. The number of sprinklers to be calculated is then determined by the design area divided by the area of coverage per sprinkler which is found in Table 8.6.2.2.1(a). We then determined the layout of the design area based on the number of sprinklers per branch line from the relationship:

$$1.2\sqrt{A}/S$$

Where "A" is the design area and S is the distance between sprinklers on branch lines. The design area is generally rectangular unless there is an odd number of sprinklers in which one additional sprinkler is included on another branchline.

Once we had determined the design area and minimum flow and pressure requirements we began the calculations for each sprinkler in the design area from the most remote sprinkler back to the water supply. The hydraulic calculation procedure requires that pressure and flows must be determined at each node and balanced at junction points. The first step is to determine the pressure loss from the most remote sprinkler to the next one on the branch line. Pressure losses were determined using the Hazen Williams equation.

$$\text{Equation 10.5- } P_f=(4.52)(Q)^{1.85}/(C)^{1.85}(D)4.87$$

“Q” is the flow rate from the sprinkler, “C” is the Hazen Williams coefficient based on the roughness of the pipe and “D” is the internal pipe diameter. This pressure loss is per foot of pipe from the first node to the second. To get the actual pressure loss along this section of the pipe we multiplied “P_f” by the equivalent length of pipe. This includes lengths for fittings and devices that may be between the nodes and the actual length of pipe. The pressure loss or gain due to elevation changes “Pe” must also be accounted for and added to the friction loss. Finally we must add the pressure losses and gains from friction and elevation changes to the pressure that was determined for the first node P_t. The new P_t is then the pressure that must be supplied at node 2. We repeated this process for flowing and non-flowing nodes until we reached the water supply. Once we completed the process we had the total pressure and flow that must be supplied by the water supply.

The above summary of the hydraulic calculation procedure was performed for our sprinkler systems can be seen in the form of an excel spreadsheet in Appendix J.

10.5 Hydraulic Design Results

We decided to use two different K-factors for both systems to see how the sprinkler thread size affects system demand in addition to piping layout and sizes. The design criteria for both systems can be seen in Table 21 below for light hazard occupancy.

Table 21 Light Hazard Design Criteria

Hydraulic Design Criteria	Layout 1		Layout 2	
	K 5.6	K 8.0	K 5.6	K 8.0
Density	0.10 gpm/ft^2	0.10 gpm/ft^2	0.10 gpm/ft^2	0.10 gpm/ft^2
Design Area	1500 ft^2	1500 ft^2	1500 ft^2	1500 ft^2
Distance Between Sprinklers	12 ft	12 ft	13 ft	13 ft
Distance Between Branchlines	10 ft	10 ft	15 ft	15 ft
Area per Sprinkler	120 ft^2	120 ft^2	195 ft^2	195 ft^2
# of sprinklers in Design Area	13	13	8	8
# of Sprinklers per Branchline	4	4	4	4
K-factor	5.6	8	5.6	8
Flow Required from HMR Sprinkler	12 gpm	12 gpm	19.5 gpm	19.5 gpm
Pressure Required from HMR Sprinkler	4.59 psi	2.25	12.12 psi	5.94 psi
Hazen-Williams Coefficient	120	120	120	120
Hose Stream Allowance	100 gpm	100	100 gpm	100

We used the data from Table 21 above to perform the hydraulic calculations on each system and determine the total demand at the riser. The results from our calculations can be seen in Table 22 below with flow and pressures for K5.6 sprinklers.

Table 22 System Demand for K5.6 Sprinklers

Sprinkler System Demand at Riser	Layout 1		Layout 2	
	Flow	Pressure	Flow	Pressure
Pipe Schedule-K5.6	201 gpm	46 psi	188 gpm	43 psi
Resized Branchlines-K5.6	179 gpm	36 psi	169 gpm	30 psi
Resized Mains-K5.6	195 gpm	24 psi	188 gpm	34 psi
Resized Mains and Branchlines-K5.6	173 gpm	18 psi	169 gpm	22 psi

System 1 required a flow of 201 gpm and a pressure of 46 psi, while system two required 188 gpm and 43 psi. These initial demands are not very high and public water supplies would most likely be adequate without the use of a fire pump. However, our system is designed for light hazard occupancy and the required flow at the hydraulically most remote sprinkler is only 12 gpm. Sprinkler systems designed for higher hazard occupancies require much greater demands, and it is important to appreciate how piping layouts and sizes alter total flows and pressures.

The first alteration to the sprinkler systems we made was to only resize the branchlines. Pipe sizes were increased from the original size up to next nominal diameter for schedule 40 steel pipe. With larger branchlines we were able to reduce the total flow and pressure required significantly. The demand for layout one was reduced to 179 gpm and 36 psi. For layout 2, flow and pressure requirements were reduced to 169 gpm and 30 psi.

In our second attempt we only resized the main lines that supply the branchlines. Pipe sizes were again increased from the original size to the next largest nominal diameter for schedule 40 steel pipe. For System one the flow was reduced slightly to 195 gpm, however the pressure was reduced significantly to 24 psi. There was no reduction in flow for System two, but the pressure was reduced to 34 psi. Flow demands were not significantly decreased by resizing the mains because flowing sprinklers are not required in the design area. The pressure however is significantly reduced because of the great distance the water must travel from the riser to reach the branchlines.

Our final alteration combined resizing the branchlines and mains. This obviously resulted in the greatest reduction in system demand. The larger branchlines allowed for a great reduction in flow since pressure is reduced and the flow required from subsequent sprinklers is dependent on the pressure. Increased pipe sizes for the mains allowed for a great reduction in pressure because the friction loss reduction per foot was much greater. Visual graphs of the flow and pressure differences can be seen in Figures 54 and 55 respectively.

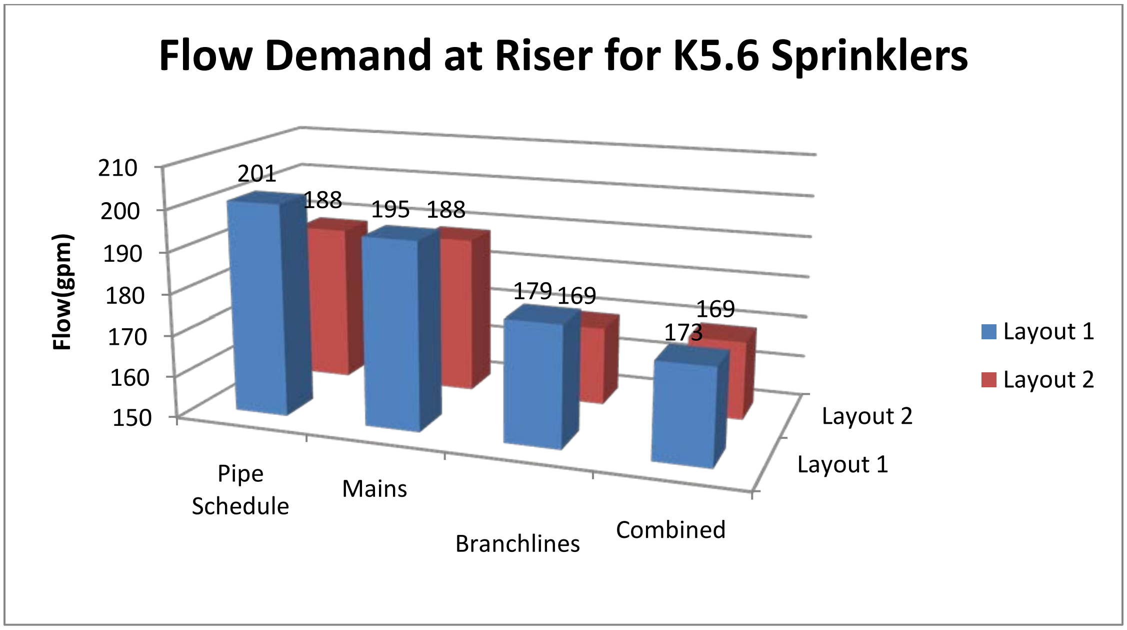


Figure 54 Flows for K5.6 Sprinklers

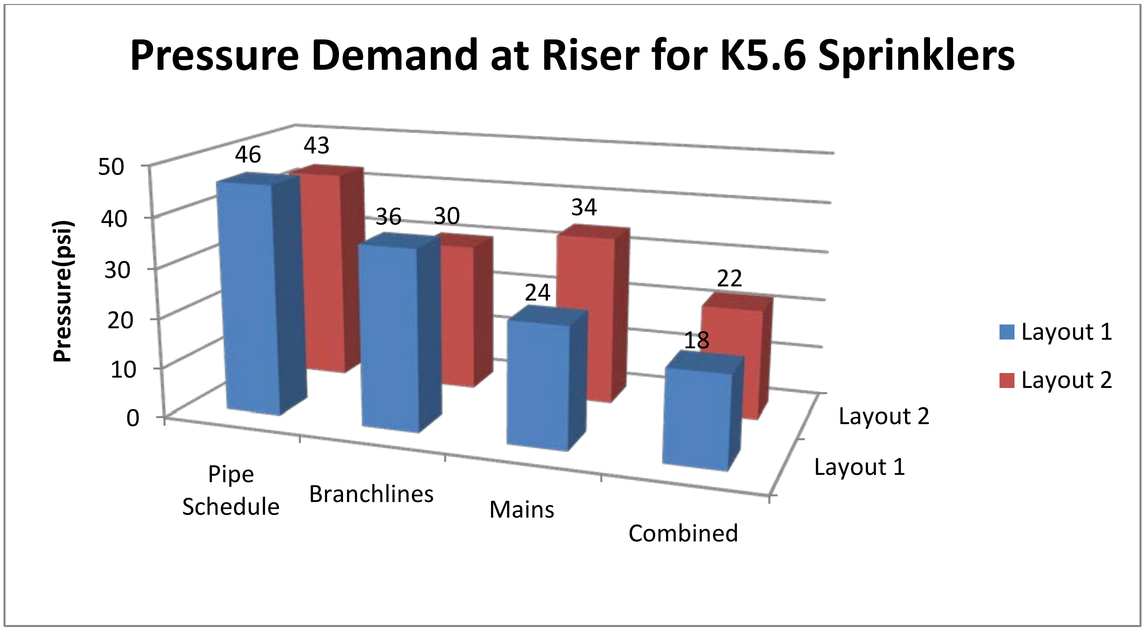


Figure 55 Pressures for K5.6 Sprinklers

By altering layouts and pipe sizes of branchlines and cross mains we have shown how the required system pressures and flows fluctuate. Naturally, with an increase in pipe sizes the pressure loss due to friction is decreased and thus the total flow and pressures are reduced as well. However, through modification of pipe sizes for branchlines and cross mains separately we see which one reduces flow and pressure more and why. In Figure 49 above we can see that increasing the size of the branchlines has the most significant affect on total flow. This is because sprinklers on the branchlines are designed to activate and when the friction loss is reduced so is the flow from the following node. For pressure, however, the greatest reduction is different for layouts one and two. This is due to the fact that there are flowing sprinklers on the cross main for layout 1 but not layout 2. Therefore increasing the size of the pipes on the mains for layout 1 has a much greater affect on pressure.

Although changing the layout and size of pipes has shown to have a great effect on the presssure and flow requirements of the system, there is still one more important factor the designer must select . Choosing the K-factor of the sprinklers is very important, as this factor relates to the thread size of the sprinkler. A low K-factor has a small thread size compared to a high K-factor which correlates to a large diamter thread. Generally sprinklers with high K-factors can produce more flow, and require less pressure than smaller diameter sprinklers. For our project we looked at two sizes, K5.6 and K8.0, since the demand on a light hazard sprinkler system is not very high and very unlikely that sprinklers larger than K8.0 would be used.

We performed the same hyrdaulic calculation for both layouts with K8.0 sprinklers and made the same alterations to view the change in pressure and flow requirements. Figures 56 and 57 below show the system demand for layouts one and two and each modification.

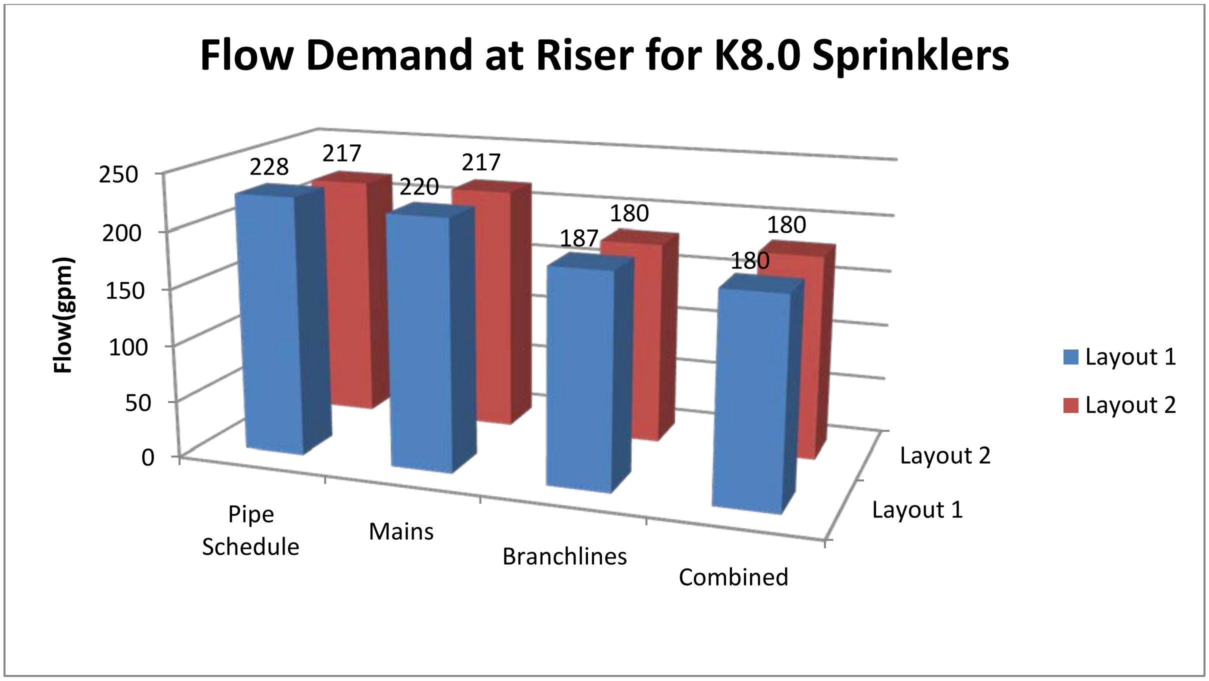


Figure 56 Flows for K8.0 Sprinklers

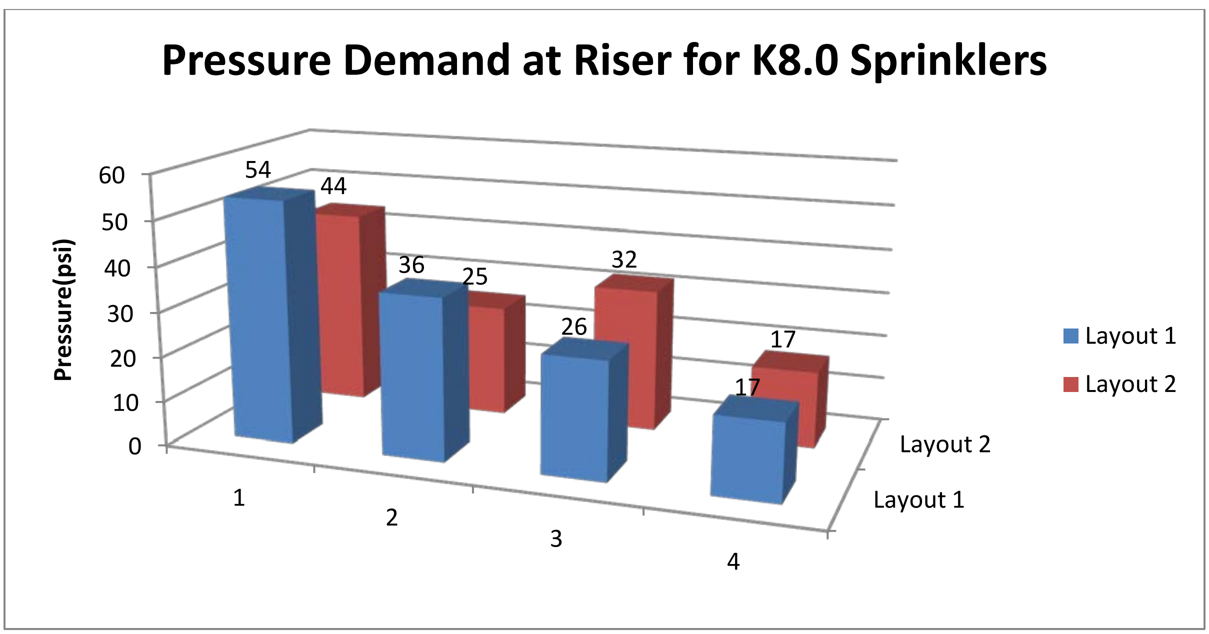


Figure 57 Pressures for K8.0 Sprinklers

The changes in pressure and flow requirements for K8.0 sprinklers mirror that for the K5.6 sprinklers when pipe sizes are increased. However, because of the larger thread size the total flow is higher for K8.0 sprinklers than K5.6 sprinklers. The initial pressure for the pipe schedule system is also higher, but when both the size of the mains and branchlines are increased the pressure is lower for the K8.0 sprinklers. As, expected the K8.0 sprinklers provide a greater flow but at a lower pressure than the K5.6 sprinklers. Figures 58 and 59 below present the differences in flow and pressure between K5.6 and K8.0 sprinklers.

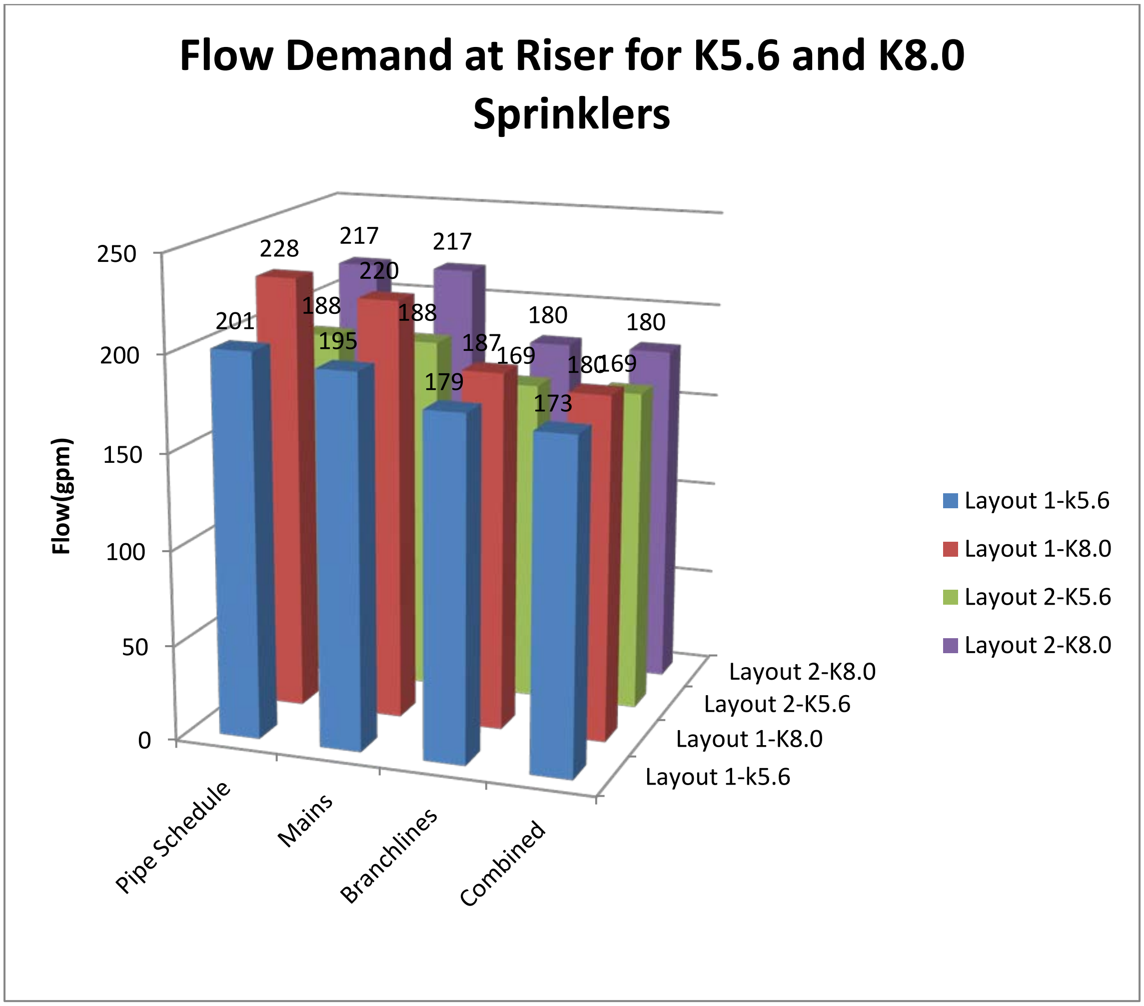


Figure 58 Flows for K5.6 vs. K8.0 Sprinklers

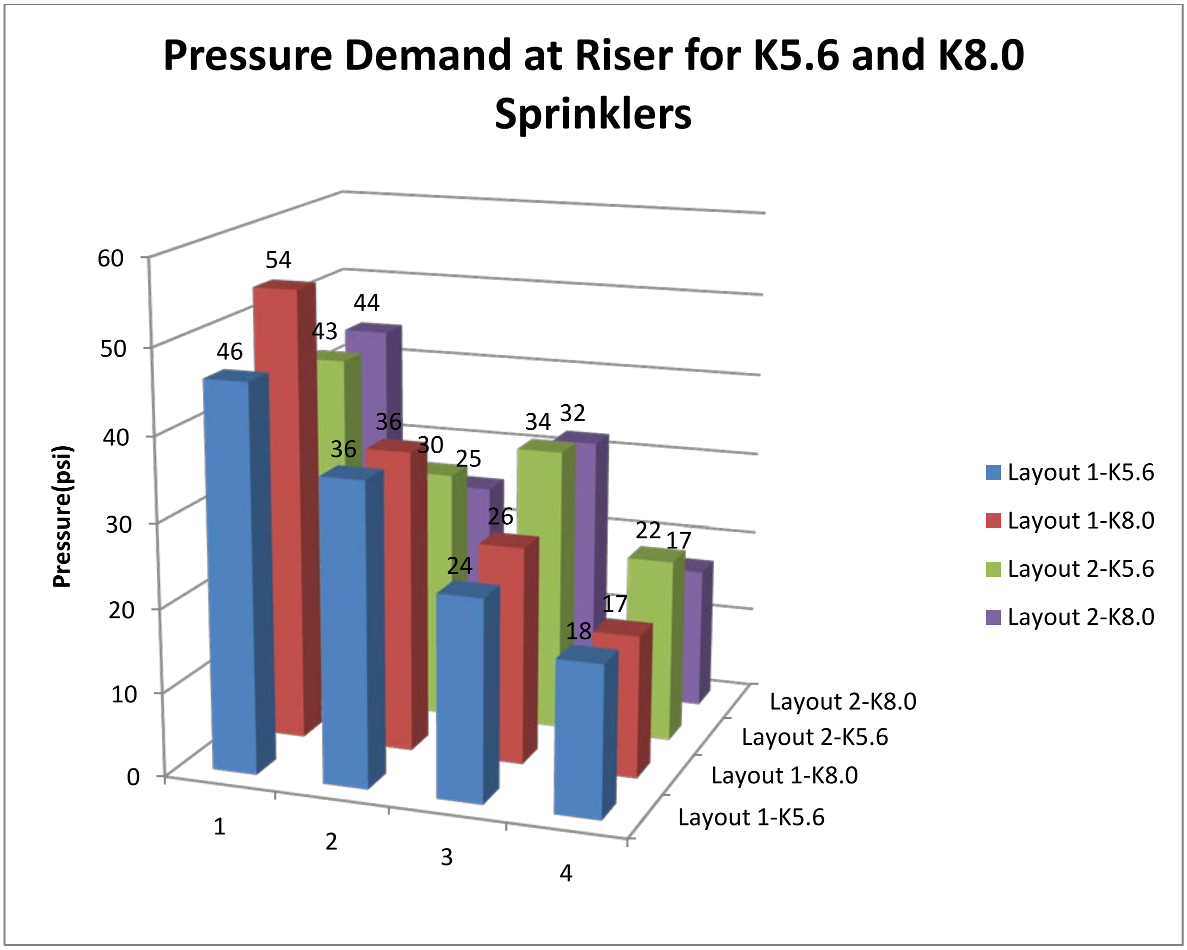


Figure 59 Pressures for K5.6 vs. K8.0 Sprinklers

10.6 Cost

10.6.1 Methodology

To determine how the layout and pipe sizes affect sprinkler system cost, we determined the overall material and labor costs for both systems. We included a cost analysis for the initial pipe schedule sizes and the combination of resizing both the mains and branchlines using hydraulic calculations. Material costs and labor hours for the piping were taken from the *RS Means Construction Costs Data*. To determine the cost of sprinklers we used the price of a Victaulic, standard spray, quick response sprinkler compatible with our design requirements. We referenced the labor hour requirements for sprinkler installation from a previous Major Qualifying Project, *Alternative Structural and Fire Protection Design of a 4-Story Office Building*. In addition to piping and sprinklers, complete systems require several more components such as hangers, fittings and valves which add to the total cost. To account for these items we increased the total material cost and labor hours by 30 percent. Using a 30 percent increase was validated by comparing the cost of our system to one from the *RS Means Square Foot Costs, 2009 Edition*. The cost of our sprinkler system was \$2.40/sqft and a 20,000ft^2, three story office was $2.96/sqft. Our price was lower, however, this is normal as square foot costs generally decrease with building size (RS Means). We then compared the total cost and labor hours for the four scenarios and the affect of a fire pump.

10.6.2 Results

In our analysis we used four scenarios for cost comparisons.

Scenario 1-System 1 using pipe schedule method

Scenario 2-System 1 with resized mains and branchlines

Scenario 3-System 2 using pipe schedule method

Scenario 4-System 2 with resized mains and branchlines

The total material cost and labor hours can be seen for each scenario in Table 23 below.

Table 23 Sprinkler System Material and Labor Costs

	Scenario 1		Scenario 2		Scenario 3		Scenario 4	
Item	Mat. Cost	Labor Hrs.	Mat. Cost	Labor Hrs.	Mat. Cost	Labor Hrs.	Mat. Cost	Labor Hrs.
Schedule 40 Pipe	$14,452	413.28	$18,313	486.3	$9,197	292.25	$11,572	339.26
Sprinklers	$3,189	14.3	$3,189	14.3	$2,609	11.7	$2,609	11.7
Total	$17,641	427.58	$21,502	500.6	$11,806	303.95	$14,181	350.96
30% Adjustment	$22,933	555.854	$27,952	650.78	$15,348	395.135	$18,436	456.248

Scenario two has the highest material cost and labor hours at $27,952 and 651 hours, compared to the lowest material cost and least labor intensive at $15,348 and 395 hours for Scenario three. This difference in cost and labor hours demonstrates how a well planned and efficient layout of the aboveground piping is essential to installing a cost efficient system. There is an 82% increase in material costs and a 65% increase in labor hours between Scenarios 2 and 3. These differences, displayed in Figures 60 and 61, are staggering and prove that a competent sprinkler designer must be aware of the cost of a poorly laid out system.

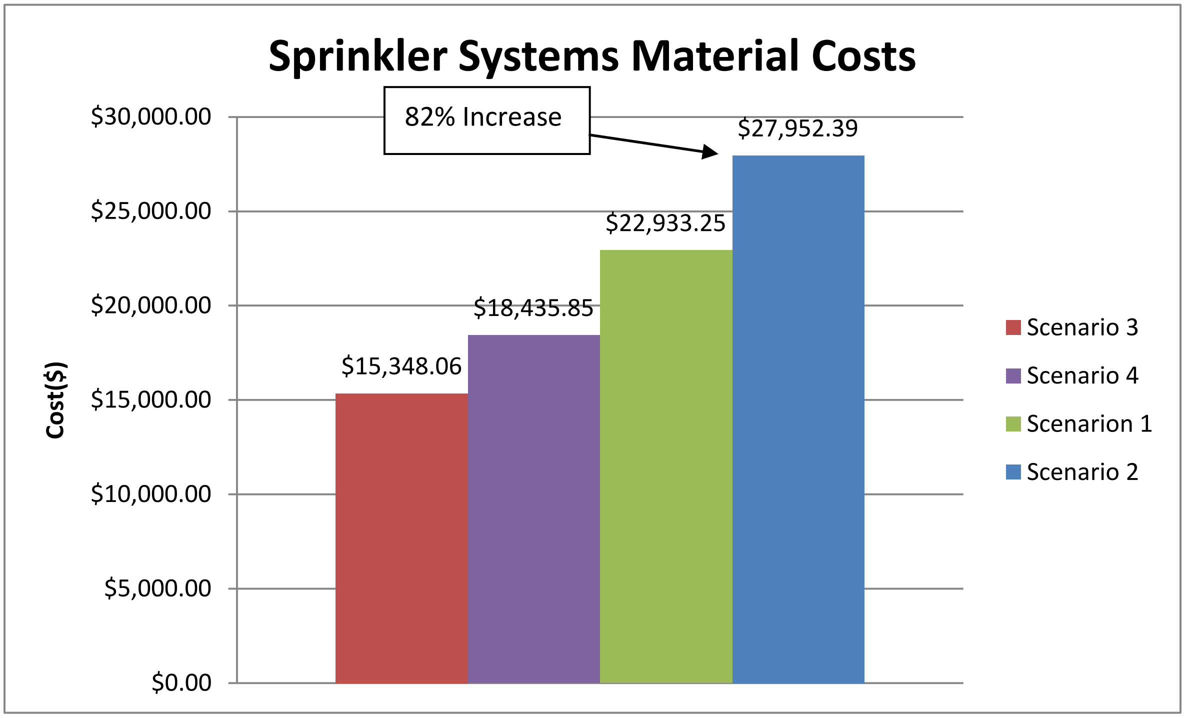


Figure 60 Material Cost Comparison

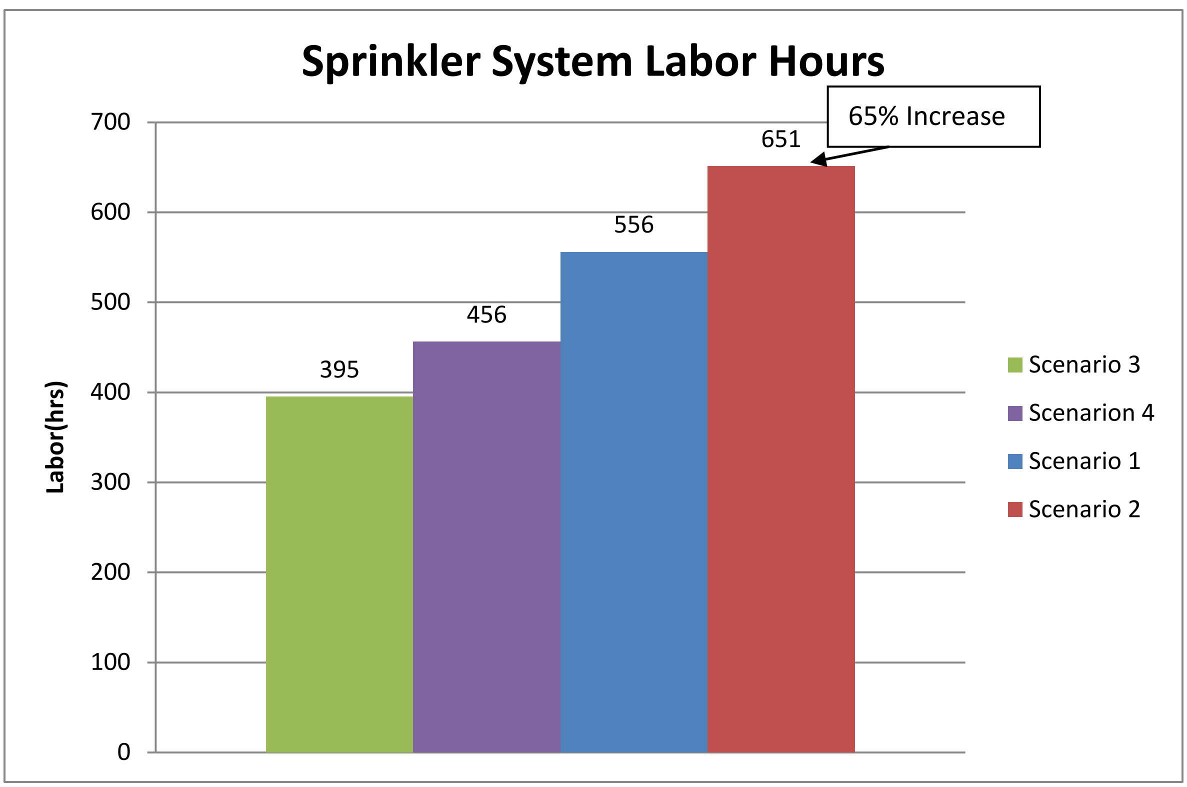


Figure 61 Labor Hours Comparison

Part of the reason there is such a great disparity in the cost and labor hours between Scenarios two and three is because of the difference in pipe sizes. The pipe sizes for branchlines and mains were enlarged in Scenario two to reduce pressure and flow requirements compared to that of Scenario three where pipe sizes were obtained from the pipe schedule method. It is important to realize that although material costs are increased as pipe sizes are enlarged, it is much more cost effective than having to install a fire pump. Fire pumps are installed in a sprinkler system when the water supply cannot meet the demand for pressure and flow. Resizing pipes can lessen the demand and prevent the installation of a fire pump. The pump alone can range anywhere from $10,000 to $40,000 depending on the size. Since the flow and pressure requirements for scenario 1 and 3 are higher we will assume a 20,000 fire pump is required.

Initially the material cost for Scenario three was only $15,348 and the materials for Scenario four cost $18,436. Without a fire pump the sprinkler system for Scenario three cost 20% less, shown in Figure 62.

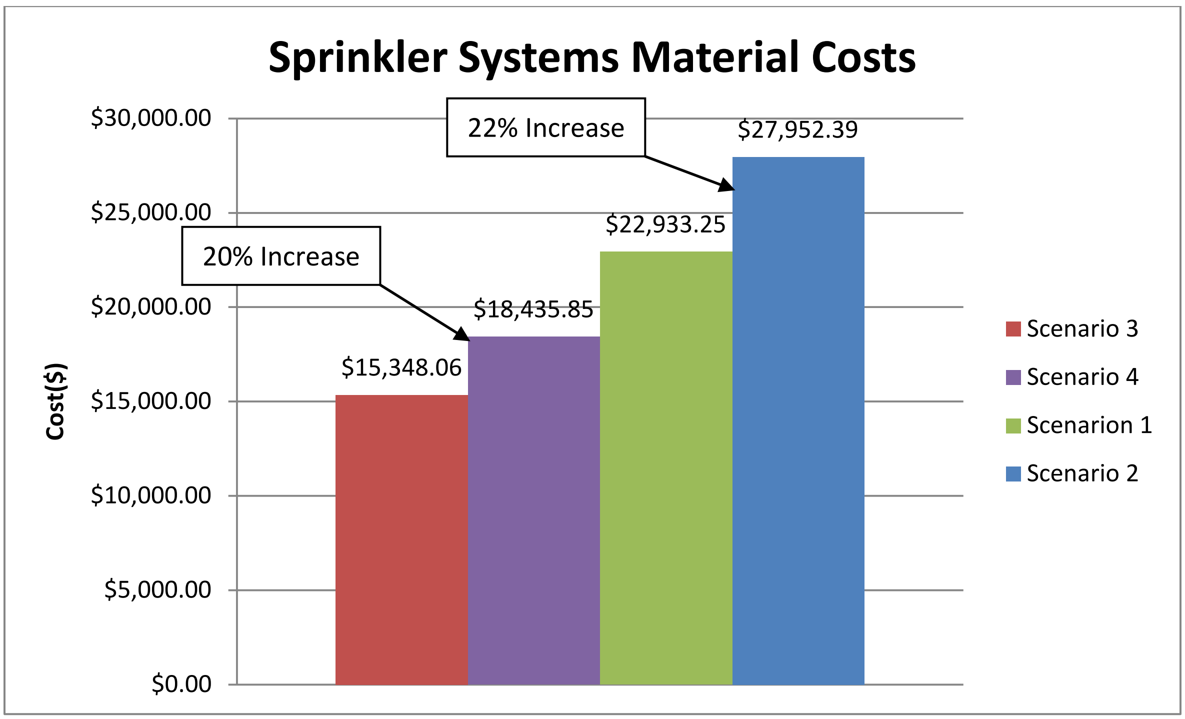


Figure 62 Material Cost Comparison Without Fire Pump

However, with the addition of a fire pump, the cost of Scenario three jumped up to $35,438. Now Scenario four material costs are 92% less, shown in Figure 63. Comparing the cost of a system that requires fire pump to one that does not demonstrates the importance of hydraulically designed systems for reducing flow and pressure and relieving the need for a fire pump.

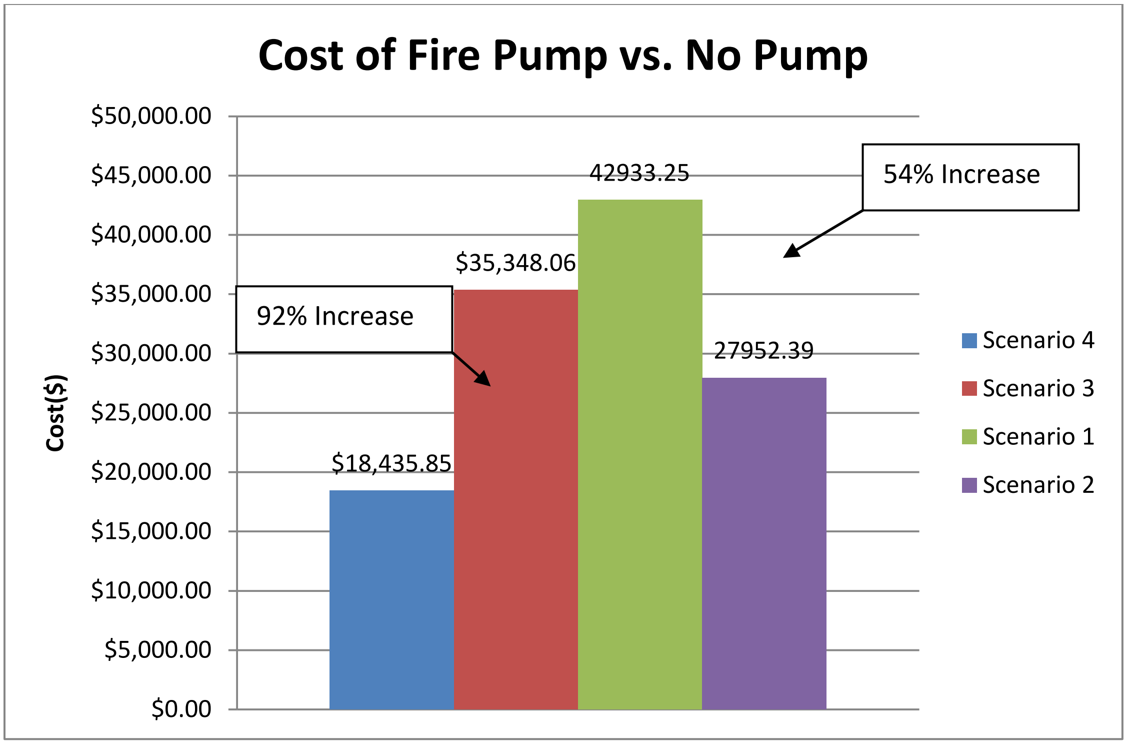


Figure 63 Material Cost Comparison with Fire Pump

10.7 Sprinkler System Performance

10.7.1 Introduction

System performance is dependent on many factors, such as correct operation of activated sprinklers and the delivery of water, however we will assume that once activated the sprinkler system will perform as designed. To establish how well our sprinkler systems perform we will determine the activation time. This is an approximation of the elapsed time before a sprinkler actuates and delivers water once a fire has ignited. For any building it is important know maximum activation time. This will occur when a fire is in a location farthest from any single sprinkler, typically in a corner or the center of a group of sprinklers.

In order to achieve a valid approximation it is important to have an accurate design fire for use in the performance calculations. We used the same design as for the performance of smoke detectors in Chapter 7. The design fire consists of a workstation constructed of modern furniture and four sides of acoustic panels. To simplify the analysis for a performance based calculation we assumed the ceilings are flat and continuous, without any obstructions. Also, the fire is assumed to start on the floor of the workstation and the distance to the sprinkler is measured from the finished floor to the ceiling. Finally, we assumed that the ambient temperature in the office is at an average of 68°F.

10.7.2 Calculation Parameters

Based on the layouts of both sprinkler systems we determined that the worst case scenario for System 1 was 10.17ft and 10.75ft for System 2. The difference in maximum distances for each system is due to the difference in layout. Sprinklers are typically evenly spaced; however, obstructions and corner fires create longer distances that must be accounted for in the performance analysis. Many of the important variables in the calculations for activation time come from the type of sprinkler used. Both of our systems were fitted with standard temperature, quick response sprinklers with 3mm bulbs. Based on the dimensions of the building floors and properties of the sprinklers installed we have the following values for our calculations:

Response Time Index- RTI= $50ms^{1/2}$

Activation Temperature- T_{act}= 80°C (175°F)

Ambient Temperature- T_a= 20° (68°F)

Ceiling Height- H= 2.44m (8ft)

Radial Distance to Sprinkler- R_1= 3.1m (10.17ft), R_2= 3.28m (10.75ft)

Fuel Growth Constant- α=0.0425kW/sec^2

Convection Fraction- Xconv=0.7

Density of Air- ρ=1.205kg/m^3

Specific Heat of Air- Cp=1.005kJ/kgK

10.7.3 Activation Time Calculations

Activation times for sprinklers are determined in the same ways as for heat detectors. Essentially an automatic sprinkler operates as a heat detector as well because when the heat sensitive link reaches its activation temperature the sprinkler actuates and delivers water. Three different processes can be used to determine activation times with each one having potential errors. To determine the activation times for our sprinklers we decided to use two processes from the *SFPE Handbook of Fire Protection Engineering 4th Edition*; the first with a quasi steady-state growing fire and equations developed by Ronald Alpert. This process is quite simple because the growing fire is modeled as a series of increasing steady heat release rates. The second process we used was for power or t^2 fires that grow according to the power-law. The equations for this process were developed by the Hekestad and Delichatsios correlations and involve a more detailed approach (SFPE, 4th Edition).

In determining the activation times following Alpert's process the heat release rate curve for the design fire is divided up into time intervals with corresponding values for the heat release rate. Once these values our determined, the next step is to calculate the change in temperature of the gas and corresponding ceiling jet velocity.

Change in Gas Temperature - $\Delta Tg= [5.38(Q/r)^{2/3}]/H$

Ceiling Jet Velocity – $u= (20Q^{1/3}H^{1/2})/r^{5/6}$

With the gas temperature and ceiling velocity known we can calculate the change in temperature of the detector or in the case of a sprinkler, the bulb. This term is also dependent on the RTI value for the sprinkler, demonstrating how the response time directly correlates to activation.

Change in Bulb Temperature - $\Delta Td = [u^{1/2}(Tg-Td)/RTI]\Delta t$

Finally the detector temperature is determined by adding ΔTd to the previous temperature; which initially begins at the ambient temperature of the room. This process is continued until the detector temperature is equal to or greater than the activation temperature and the corresponding time is when the sprinkler will actuate (*SFPE Handbook*). Figures 64 and 65 below show the time temperature changes following Alpert's method for estimating sprinkler activation times. The activation time for both systems is at 140 seconds or 2.33 minutes. Although the radial distance is slightly higher for system two, both systems have the same activation times. This is likely due to the ten second time step. To check the sensitivity of the calculations we also used a 5 second time step. The resulting activation times were almost exactly the same, however, for system 2 the time increased to 145 seconds, shown in Figure 66.

Time step	time(t)	Q(kW)	ΔTg	Tg	u	ΔTd	Td
0	0	0	0	0	0	0	20
1	10	4	2.613312	22.61331	0.193172	0.229717	20.22972
2	20	18	7.123055	27.12306	0.318919	0.778574	21.00829
3	30	40	12.12992	32.12992	0.416176	1.43495	22.44324
4	40	72	17.94897	37.94897	0.506253	2.206511	24.64975
5	50	113	24.24024	44.24024	0.588324	3.00527	27.65502
6	60	162	30.81967	50.81967	0.663379	3.773433	31.42846
7	70	221	37.90934	57.90934	0.735734	4.542792	35.97125
8	80	289	45.33322	65.33322	0.804556	5.267363	41.23861
9	90	366	53.06466	73.06466	0.870463	5.938653	47.17726
10	100	452	61.08158	81.08158	0.933906	6.552945	53.73021
11	110	546	69.28083	89.28083	0.994614	7.090949	60.82116
12	120	650	77.82042	97.82042	1.054131	7.597494	68.41865
13	130	763	86.59663	106.5966	1.111984	8.051785	76.47044
14	140	885	95.59757	115.5976	1.168345	8.458505	84.92894

Figure 64 Alpert Activation Time for System 1

Time step	time(t)	Q(kW)	ΔTg	Tg	u	ΔTd	Td
0	0	0	0	0	0	0	20
1	10	4	2.516807	22.51681	0.184296	0.216092	20.21609
2	20	18	6.860012	26.86001	0.304267	0.732962	20.94905
3	30	40	11.68198	31.68198	0.397054	1.352612	22.30167
4	40	72	17.28615	37.28615	0.482993	2.082774	24.38444
5	50	113	23.34509	43.34509	0.561293	2.841044	27.22548
6	60	162	29.68155	49.68155	0.6329	3.572985	30.79847
7	70	221	36.5094	56.5094	0.701931	4.308191	35.10666
8	80	289	43.65913	63.65913	0.76759	5.003091	40.10975
9	90	366	51.10506	71.10506	0.83047	5.649212	45.75896
10	100	452	58.82594	78.82594	0.890997	6.242557	52.00152
11	110	546	66.7224	86.7224	0.948916	6.764481	58.766
12	120	650	74.94663	94.94663	1.005699	7.256715	66.02272
13	130	763	83.39876	103.3988	1.060893	7.699439	73.72215
14	140	885	92.0673	112.0673	1.114665	8.096785	81.81894

Figure 65 Alpert Activation Time for System 2

Time step	time(t)	Q(kW)	ΔTg	Tg	u	ΔTd	Td
0	0	0	0	0	0	0	20
1	5	0.791	0.854268	20.85427	0.107371	0.055985	20.05598
2	10	3.164	2.152621	22.15262	0.170442	0.173117	20.2291
3	15	7.119	3.696204	23.6962	0.223342	0.327704	20.55681
4	20	12.656	5.424265	25.42426	0.270559	0.506365	21.06317
5	25	19.775	7.30389	27.30389	0.313956	0.699357	21.76253
6	30	28.476	9.313849	29.31385	0.354533	0.899251	22.66178
7	35	38.759	11.43909	31.43909	0.392905	1.100362	23.76214
8	40	50.624	13.66829	33.66829	0.429486	1.298402	25.06054
9	45	64.071	15.99254	35.99254	0.464569	1.490236	26.55078
10	50	79.1	18.40465	38.40465	0.498374	1.673663	28.22444
11	55	95.711	20.89863	40.89863	0.531069	1.847249	30.07169
12	60	113.904	23.46943	43.46943	0.562786	2.010171	32.08186
13	65	133.679	26.11271	46.11271	0.593633	2.162086	34.24395
14	70	155.036	28.82471	48.82471	0.623698	2.303018	36.54697
15	75	177.975	31.6021	51.6021	0.653055	2.433265	38.98023
16	80	202.496	34.44193	54.44193	0.681766	2.553319	41.53355
17	85	228.599	37.34159	57.34159	0.709885	2.663803	44.19735
18	90	256.284	40.29869	60.29869	0.737458	2.765416	46.96277
19	95	285.551	43.31108	63.31108	0.764524	2.858897	49.82166
20	100	316.4	46.37681	66.37681	0.79112	2.944992	52.76666
21	105	348.831	49.49408	69.49408	0.817275	3.024428	55.79108
22	110	382.844	52.66124	72.66124	0.843019	3.097903	58.88899
23	115	418.439	55.87677	75.87677	0.868375	3.16607	62.05506
24	120	455.616	59.13926	79.13926	0.893367	3.229531	65.28459
25	125	494.375	62.44738	82.44738	0.918013	3.288837	68.57343
26	130	534.716	65.79992	85.79992	0.942333	3.344483	71.91791
27	135	576.639	69.19572	89.19572	0.966343	3.396913	75.31482
28	140	620.144	72.63371	92.63371	0.990059	3.446518	78.76134
29	145	665.231	76.11289	96.11289	1.013493	3.493644	82.25498

Figure 66 Alpert Method with 5 Second Time Step

We also decided to determine the activation times of our sprinklers systems using the Hekestad and Delichatsios correlations. This method assumes that the fire grows according to the power law relationship. Instead of predicting gas temperatures and ceiling jet velocities with a heat release rate, this method uses a fuel growth constant α. The same fuel constant used in the performance of the smoke detectors was used in the analysis of the sprinkler systems. We also had to assume a convective fraction used to find α_c since heat transfer to the sprinkler is assumed to be through convection only (SFPE).

The Hekestad and Delichatsios method also accounts for the amount of time it takes for the heat front to reach the detector. This time t^*_{2f} is based on the radial distance r and ceiling height H.

$$t^*_{2f}= 0.813(1+r/H)$$

Next we have to determine the constant A, which will be used to determine t^*_2. If t^*_2 is not greater than t^*_{2f} the heat front has not yet reached the sprinkler and the temperature of the bulb does not increase.

$$A= g/CpTa\rho_0$$

$$t^*_2= t/A^{-1/5}\ \alpha_c^{-1/5}H^{4/5}$$

Once there has been enough fire growth for the heat front to reach the sprinkler and t^*_2 is greater than t^*_{2f} we calculated the ratio u/u^*_2 and $\Delta T/\ \Delta T^*_2$ for the non-dimensional ceiling jet velocity and temperature relationships.

$$u/u^*_2= A^{1/5}\ \alpha_c^{1/5}H^{1/5}$$

$$\Delta T/\ \Delta T^*_2= A^{2/5}(Ta/g)\ \alpha_c^{2/5}H^{-3/5}$$

We then determined two constants D and $u^*_2/(\ \Delta T^*_2)^{1/2}$ and ΔT^*_2 which are used to determine the change in the sprinkler temperature.

$$D= 0.146+0.242(r/H)$$

$$u^*_2/(\ \Delta T^*_2)^{1/2}= 0.59(r/H)^{-0.63}$$

$$\Delta T^*_2= (t^*_2- t^*_{2f})/D$$

Finally the change in temperature of the sprinkler bulb was determined using the following equations and dimensionless relationships found above.

$$Y=(3/4)\surd\ (u/u^*_2)\surd(\ u^*_2/(\ \Delta T^*_2)^{1/2})(\ \Delta T^*_2/RTI)D$$

$$\Delta Td= (\Delta T/\ \Delta T^*_2)\ \Delta T^*_2[1-(1-e^{-Y})/Y]D$$

Based on the above calculations we determined the activation time for both sprinkler systems to be 170 seconds or 2.83 minutes shown in Figures 67 and 68. The longer time compared to Alpert's method is likely due to the term t^*_{2f} which accounts for the time it takes for the hot gasses to reach the sprinkler. However, the sensitivity of this method is much greater than for Alpert's method. By changing the time step to 5 seconds, activation times were reduced to 120 seconds, shown in Figures 69 and 70. Overall these activation times are quite close and are still only estimates as both methods have inherent errors due to uncertainties in the exact gas temperatures and ceiling jet velocities.

Time step	time(t)	t*2	ΔT*2	Y	ΔTd	Td
0	0	0	0	0	0	20
1	10	1.214668	0	0	0	20
2	20	2.429337	1.694657	0.031878	0.157709	20.15771
3	30	3.644005	7.60064	0.142974	0.681985	20.83969
4	40	4.858674	15.12586	0.284529	1.296795	22.13649
5	50	6.073342	23.76122	0.446968	1.935984	24.07247
6	60	7.28801	33.2753	0.625935	2.567298	26.63977
7	70	8.502679	43.52958	0.818826	3.172531	29.8123
8	80	9.717347	54.43003	1.023872	3.74142	33.55372
9	90	10.93202	65.90781	1.239779	4.268721	37.82244
10	100	12.14668	77.90997	1.465549	4.752486	42.57493
11	110	13.36135	90.39423	1.700388	5.192936	47.76787
12	120	14.57602	103.326	1.943644	5.591689	53.35956
13	130	15.79069	116.6761	2.194772	5.951214	59.31077
14	140	17.00536	130.4201	2.453306	6.27445	65.58522
15	150	18.22003	144.5363	2.718844	6.564536	72.14976
16	160	19.43469	159.0063	2.991036	6.82463	78.97439
17	170	20.64936	173.8134	3.26957	7.057787	86.03217

Figure 67 Hekestad and Delichatsios Activation Time for System 1

Time step	time(t)	t*2	ΔT*2	Y	ΔTd	Td
0	0	0	0	0	0	20
1	10	1.214668	0	0	0	20
2	20	2.429337	1.392739	0.026752	0.132575	20.13257
3	30	3.644005	6.899339	0.132523	0.634291	20.76687
4	40	4.858674	13.98549	0.268635	1.230563	21.99743
5	50	6.073342	22.14093	0.425286	1.854489	23.85192
6	60	7.28801	31.13911	0.598124	2.473838	26.32575
7	70	8.502679	40.84557	0.784567	3.070282	29.39604
8	80	9.717347	51.16947	0.98287	3.633292	33.02933
9	90	10.93202	62.0445	1.191759	4.157262	37.18659
10	100	12.14668	73.41975	1.410256	4.639844	41.82643
11	110	13.36135	85.25468	1.637583	5.080856	46.90729
12	120	14.57602	97.51606	1.873101	5.481538	52.38883
13	130	15.79069	110.1761	2.116277	5.844019	58.23285
14	140	17.00536	123.2112	2.366656	6.170943	64.40379
15	150	18.22003	136.6009	2.623846	6.465202	70.86899
16	160	19.43469	150.3272	2.887505	6.729754	77.59875
17	170	20.64936	164.3746	3.157328	6.967497	84.56624

Figure 68 Hekestad and Delichatsios Activation Time for System

Time step	time(t)	t*2f	t*2	u/u*2	ΔT/ΔT*2	ΔT*2	u*2/ΔT*2	Y	ΔTd	Td
0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	20.00
1	5	1.85	0.60	0.30	1.08	0.00	0.51	0.00	0.00	20.00
2	10	1.85	1.20	0.29	1.05	0.00	0.51	0.00	0.00	20.00
3	15	1.85	1.80	0.29	1.05	0.00	0.51	0.00	0.00	20.00
4	20	1.85	2.40	0.29	1.05	1.57	0.51	0.03	0.15	20.15
5	25	1.85	3.00	0.29	1.05	4.18	0.51	0.08	0.39	20.53
6	30	1.85	3.59	0.29	1.05	7.32	0.51	0.14	0.66	21.20
7	35	1.85	4.19	0.29	1.05	10.84	0.51	0.21	0.96	22.16
8	40	1.85	4.79	0.29	1.05	14.68	0.51	0.28	1.27	23.43
9	45	1.85	5.39	0.29	1.05	18.79	0.51	0.36	1.59	25.01
10	50	1.85	5.99	0.29	1.05	23.14	0.51	0.44	1.90	26.91
11	55	1.85	6.59	0.29	1.05	27.70	0.51	0.52	2.22	29.13
12	60	1.85	7.19	0.29	1.05	32.46	0.51	0.61	2.53	31.66
13	65	1.85	7.79	0.29	1.05	37.41	0.51	0.71	2.84	34.50
14	70	1.85	8.39	0.29	1.05	42.52	0.51	0.81	3.13	37.63
15	75	1.85	8.99	0.29	1.05	47.79	0.51	0.91	3.42	41.05
16	80	1.85	9.58	0.29	1.05	53.21	0.51	1.01	3.70	44.75
17	85	1.85	10.18	0.29	1.05	58.77	0.51	1.11	3.97	48.72
18	90	1.85	10.78	0.29	1.05	64.46	0.51	1.22	4.23	52.95
19	95	1.85	11.38	0.29	1.05	70.29	0.51	1.33	4.47	57.42
20	100	1.85	11.98	0.29	1.05	76.24	0.51	1.44	4.71	62.13
21	105	1.85	12.58	0.29	1.05	82.30	0.51	1.56	4.93	67.06
22	110	1.85	13.18	0.29	1.05	88.48	0.51	1.68	5.15	72.21
23	115	1.85	13.78	0.29	1.05	94.77	0.51	1.80	5.35	77.57
24	120	1.85	14.38	0.29	1.05	101.17	0.51	1.92	5.55	83.12

Figure 69 Hekestad and Delichatsios with 5 Second Time Step for System 1

Time step	time(t)	t*2f	t*2	u/u*2	ΔT/ΔT*2	ΔT*2	u*2/ΔT*2	Y	ΔTd	Td
0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	20.00
1	5	1.91	0.60	0.30	1.08	0.00	0.51	0.00	0.00	20.00
2	10	1.91	1.20	0.29	1.05	0.00	0.49	0.00	0.00	20.00
3	15	1.91	1.80	0.29	1.05	0.00	0.49	0.00	0.00	20.00
4	20	1.91	2.40	0.29	1.05	1.28	0.49	0.02	0.12	20.12
5	25	1.91	3.00	0.29	1.05	3.70	0.49	0.07	0.35	20.47
6	30	1.91	3.59	0.29	1.05	6.64	0.49	0.13	0.62	21.09
7	35	1.91	4.19	0.29	1.05	9.95	0.49	0.19	0.90	21.99
8	40	1.91	4.79	0.29	1.05	13.57	0.49	0.26	1.20	23.19
9	45	1.91	5.39	0.29	1.05	17.45	0.49	0.34	1.51	24.71
10	50	1.91	5.99	0.29	1.05	21.55	0.49	0.42	1.82	26.53
11	55	1.91	6.59	0.29	1.05	25.87	0.49	0.50	2.13	28.66
12	60	1.91	7.19	0.29	1.05	30.37	0.49	0.59	2.44	31.10
13	65	1.91	7.79	0.29	1.05	35.05	0.49	0.68	2.74	33.84
14	70	1.91	8.39	0.29	1.05	39.89	0.49	0.77	3.03	36.87
15	75	1.91	8.99	0.29	1.05	44.88	0.49	0.87	3.32	40.18
16	80	1.91	9.58	0.29	1.05	50.01	0.49	0.97	3.59	43.77
17	85	1.91	10.18	0.29	1.05	55.28	0.49	1.07	3.86	47.63
18	90	1.91	10.78	0.29	1.05	60.68	0.49	1.17	4.11	51.75
19	95	1.91	11.38	0.29	1.05	66.20	0.49	1.28	4.36	56.11
20	100	1.91	11.98	0.29	1.05	71.83	0.49	1.39	4.60	60.70
21	105	1.91	12.58	0.29	1.05	77.58	0.49	1.50	4.82	65.52
22	110	1.91	13.18	0.29	1.05	83.44	0.49	1.61	5.04	70.56
23	115	1.91	13.78	0.29	1.05	89.41	0.49	1.73	5.24	75.80
24	120	1.91	14.38	0.29	1.05	95.47	0.49	1.85	5.44	81.24

Figure 70 Hekestad and Delichatsios with 5 Second Time Step for System 2

10.8 Conclusions

Automatic sprinkler systems are relied upon for protection against loss of life and property; therefore, proper design and performance of the systems is imperative. This burden is placed on the engineer, who is responsible for having the knowledge and experience to design an effective sprinkler system. The engineer is responsible for selecting the type of system, occupancy hazard, layout and several more components. The affects of these decisions directly influence the cost and performance of the system. The owner expects an efficient system that is cost effective and provides the required level of safety. The same sprinkler system with different layouts can provide equal degrees of protection; however, one may be much less expensive. Furthermore, classifying the hazard of the building can result in a sprinkler system that is unable to prevent a full burnout. These consequences are very possible, but with careful consideration of all the design factors, an appropriate system can be selected.

Chapter 11 Structural Integrity during Fire Conditions

11.1 Introduction

Modern steel construction has several merits, such as cost efficiency, and high ratio of load bearing capacity to structure weight. However, steel is vulnerable to high temperature. Although building fires can hardly melt steel members, the elevated temperature can reduce the elastic modulus and the yield strength of the steel structure member to a dangerous level. Elevated temperatures also cause steel members to undergo thermal expansion which can cause structural failure in several applications. One important example of steel failure at elevated temperature is the collapse of the World Trade Center twin towers. The structural system was designed to have sufficient redundant columns to ensure structural stability in the event of a loss of column; however, the twin towers collapsed due to elevated temperatures. The fuel from the jet plane did not melt the columns or open web joists, rather the fire heated up the columns and caused buckling. Due to the buckling, the floor slab sagged excessively and eventually fell, causing the dynamic domino effect which resulted in a total collapse. (Eagar & Musso, 2001)

Steel has high value of stiffness. This merit of steel construction can ensure the steel structural system can carry the same load but use much less material compared to reinforced concrete and heavy timber construction. However, this merit makes unprotected steel structures tend to perform poorly in fires compared with reinforced concrete or heavy timber structures. This is because steel members are quite thin and have a much higher thermal conductivity and lower heat capacity. (Buchanan, 2002)

In this chapter, we will investigate how fire affects the structural mechanics of the designed four-story steel office building as well as show the development and results of a heat transfer analysis used to provide us with structural performance members at elevated temperatures. By analyzing the structural members in a fire environment simulation, we can determine how particular structural members will respond mechanically to elevated temperatures. The heat transfer analysis gives us the information we need to make particular recommendations that could potentially save the building from structural collapse in a fire.

11.2 Overview of Structural Failure due to Fire

In order to analyze the structural stability of the building's members in fire conditions, it is necessary to understand how these members fail or attribute to the failure of other members. It is important to recognize that in most building fire scenarios, multiple structural members are exposed to elevated temperatures at the same time. Elevated temperatures cause certain structural elements to behave in a particular manner, which can cause minor or major failures throughout the structure. The ways heat from fire affects structural members are through the following:

- Induced strain caused by thermal growth of steel and concrete members (function of temperature)
- Reduction in modulus of elasticity of steel and concrete (function of temperature)
- Reduction in yield strength of steel and concrete (function of temperature)

- Induced gravitational loads or moments due to the failure or deformation of other structural elements

Generally, steel and concrete members undergo thermal expansion at elevated temperatures. This expansion of a structural member can create an induced strain on itself or an adjacent member. These induced strains create additional axial loads for columns, beams and girders which are taken into account when evaluating the stability of members at elevated temperatures. The temperature induced strains of steel and concrete are given by a series of functions of temperature shown below:

Thermal Expansion of Steel Strain (Wang, 2002):

For $20°C < T \leq 750°C$,

$$\varepsilon_{th} = -2.416 \times 10^{-4} + 1.2 \times 10^{-5}T + 0.4 \times 10^{-8}T^2$$

For $750°C < T \leq 860°C$,

$$\varepsilon_{th} = 0.011$$

For $860°C < T$,

$$\varepsilon_{th} = -0.0062 + 2 \times 10^{-5}T$$

Thermal Expansion of Concrete Strain (Wang, 2002):

For $20°C < T \leq 700°C$,

$$\varepsilon_{th} = -1.8 \times 10^{-4} + 9 \times 10^{-6}T + 2.3 \times 10^{-11}T^3$$

For$T > 700°C$,

$$\varepsilon_{th} = 0.0014$$

Elevated temperatures from fires also cause structural properties like elastic modulus and yield strength to decay. The rates at which these decay are logarithmic functions of temperature. As temperature from fire increases, the load bearing capacity for the member decreases. Once the bearing capacity is less than the actual load, the member is susceptible to collapse. Below are the functions of temperature for the yield strength (F_y) and elastic modulus (E) of steel as shown in section 4-9 of *SFPE Handbook:*

For $0 < T \leq 600°C$,

$$F_y = \left[1 + \frac{T}{900 \ln\left(\frac{T}{1750}\right)}\right] F_{y0} \qquad E = \left[1 + \frac{T}{2000 \ln\left(\frac{T}{1150}\right)}\right] E_0$$

For $T > 600°C$,

$$F_y = \frac{340 - 0.34T}{T - 240} F_{y0} \qquad E = \frac{690 - 0.69T}{T - 53.5} E_0$$

For concrete, the equations of stress, strain, Young's modulus due to thermal expansion are provided in Chapter 9 of *Structural Design for Fire Safety* by Andrew H. Buchanan and are also listed below:

Compressive strength:

For$0 < T \leq 500°C$

$$f'_c = 4\ ksi$$

For $T > 500°C$

$$f'_c = \frac{4(1000 - T)}{500} ksi$$

Young's Elastic Modulus:

For $0 < T \leq 150°C$

$$E_{c,th} = 4350\ ksi$$

For $T > 150°C$

$$E_{c,th} = \frac{700 - T}{550} \times 4350\ ksi$$

As individual structural members fail during a fire, gravity loads will be redistributed to neighboring beams, girders and columns. These induced loads can cause the load compensating members to collapse themselves.

By understanding the failure modes of structural members and the relationships between temperature and structural properties, we are capable of predicting failure times and temperatures. This next section will detail the likely ways in which the designed office building's members will fail.

11.2.1 Composite Girders and Open Web Joists

Beams and girders, because of their location with respect to the floor slab, are very susceptible to damage caused by elevated temperature. Hot gas accumulates beneath the slab within the cavity of the ceiling; heat is transferred into these members through radiation from the slab and ceiling membrane (if any) and convection from the heated space within the cavity. Enough transferred heat into the steel member can cause thermal elongation, loss of yield strength and loss of elastic modulus.

The composite girders used in the design typically span of 25 feet and are connected to composite columns at either end; they are connected to the decking and slab along the top flange with steel bolts. Because of the composite connection of the girder to the floor slab, the steel member is assumed to be in pure tension. As temperature of the steel increases, the girder will want to grow axially depending on the temperature and length of exposure. Due to axial restraints of the connections at either end of the girder, compressive axial loads will grow as a function of temperature. This axial compressive force will counteract the tensile force due to bending, which will offer the steel member strain relief. However, depending upon the strength of the end connection and the rigidity of the adjacent column, the restrained thermal elongation of the composite girder can cause failure at an adjacent column. On other hand, the open web joists usually does not act as Wide-flange beam. The end connections of open web joists are usually simple pin connections which are connected to the top chord. Due to the simple connection and smaller cross-section area, the effects of thermal expansion of the top chord to the composite columns are neglect able. The nature of this composite column failure will be developed in the next section.

During the initial stages of fire, thrust will develop at the end of each girder or joist within the bottom flange. This will cause the girder to bow upwards pushing into the concrete slab above. In the later stages of fire, when temperature of the steel member has increased, the stiffness of the beam will decrease and the girder will bow downwards.

The girders and open web joists are susceptible to deformation and failure caused by overstress, lateral instability, excessive torsional loading and excessive deflection (*SFPE*, p141). Thermal bowing, yield strength and elastic modulus, which are functions of temperature, are the contributors to the types of beam and girder failures listed. Girders are also susceptible to these failures by increased loads during fires. Open web joists are especially susceptible to these failures because of the joists' high surface area to volume ratio and small cross-sectional area of the top, bottom and diagonal chords. Heat can transfer into the joists easily and cause decay in yield strength and elastic modulus at a fast rate.

11.2.2 Composite Columns

The columns that we have chosen to carry the loads vertically throughout our designed structure are all composite. The columns are comprised of wide flange steel members of various sizes (ranging from W6x8.5 to W12x26) which are completely encased in lightweight concrete. The lightweight concrete offers great fire protection: the concrete acts as a buffer zone for the transferred heat and prevents local buckling. However, the concrete encased columns are still vulnerable to failure when exposed to elevated temperatures.

Global buckling can occur in a composite column when the load is too large for the load capacity of the concrete encased steel member. The load capacity of a column can become insufficient during a fire due to surrounding member failure (load compensation) or decreased yield strength and elastic modulus. Buckling can also occur because of an increase in column axial load due to thermal expansion and axial restraint. As the column grows axially due to elevated temperatures, the restraints on either end of the column prevent the column from growing freely. This restraint causes more strain to occur within the steel and concrete vertical member. Figure 71 shows how axial growth and restraint of a column can cause axial compressive load to grow within the member.

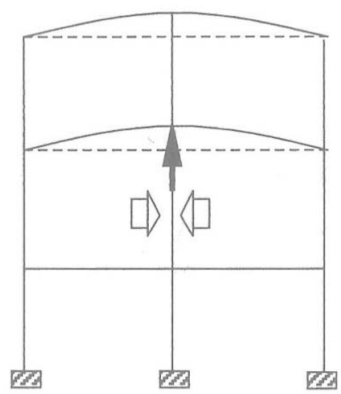


Figure 71 Axial Restraint of Column (p123, Wang)

Columns can also fail due to the thermal expansion of adjacent composite girders. As girders grow axially due to elevated temperatures, they can displace a column from its designed position at one end. This displacement can cause a change in the column's bending moment which directly affects the load capacity of the column. Figure 72 shows the effects of girder thermal expansion on adjacent columns.

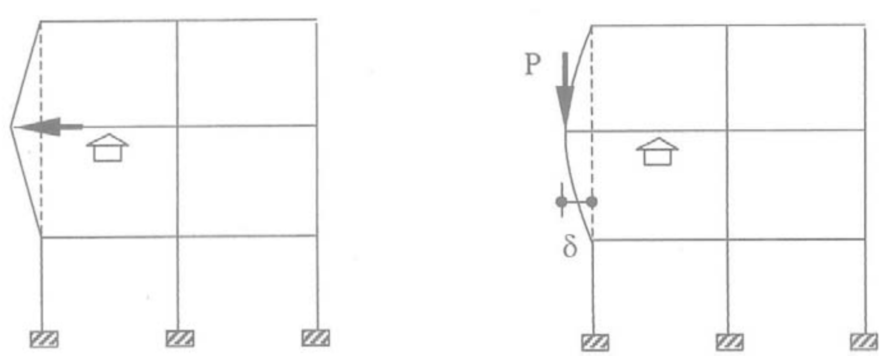


Figure 72 Thermal Expansion of Girders (p124, Wang)

11.2.3 Reinforced Concrete Slab and Metal Decking

Both the slab and decking, in a fire condition, can contribute to the failure or protection of other structural members. The thick concrete slab acts as a fire barrier from one floor to the next. Heat is generally locked in on one side of the slab as long as the slab is without any gaps or holes. As heat is transferred into the decking and concrete slab, the steel rebar within the concrete can begin to distort and grow due to the thermal expansion of steel at elevated temperatures. This can cause cracking and fractures to form within the concrete. If gaps are created in the slab, then heat and flames are able to move through the floor system into the next floor or roof. Also, if joists and/or girders were to fail, the slab would become unsupported and collapse atop the area below. The partial collapse of the floor slab would lead to extreme fire spread throughout multiple stories of the office building.

Most likely, the decking and slab will contribute to mechanical failure of girders or open web joists. As mentioned in the previous section, hot gas accumulates beneath the slab within the cavity of the ceiling; heat is transferred into the steel members through radiation from the slab and ceiling membrane (if any) and convection from the heated space within the cavity. The heat transfer analysis that is detailed in the next section does not contain an analysis of the concrete floor slab and metal decking. Rather, we focused our attention on the failures of the girders, beams and columns.

11.3 Failure Mechanisms to be examined by Heat Transfer Analysis

Composite girders: concrete slab is 2" for 3E structural system scenario (detail see the one-page structural scenario summary sheets in previous chapter). The initial neutral plane is located around the middle of the slab. Therefore, the slab is in compression while the steel member is in tension. However, the location of neutral plane is a function of steel yield strength and concrete compressive strength. And both steel yield strength and concrete compressive strength vary with temperature. Therefore, in order to simply the analysis, the project team assumed that the neutral plane remains at the center of slab and does not move with temperature change. Therefore, as temperature goes up, the composite girders will slag and the steel member will yield due to tension when the yield strength decreases to a certain point.

Open web joists: Although the open web joist is essentially a truss, it acts globally like a wide flange web: the top and bottom chord as flanges to resist moment, the diagonal rod like web to resist the shear force. Or in other words, due to the bending effects the bottom chord is subjected to tension while the top chord is subjected to compression, and the maximum bending moment is located at the center of the span. Since the maximum shear stress is at the end of the span, the diagonal rods at the ends are subjected to the maximum shear stress, and the vertical component of the diagonal rod force carries the shear stress. For this analysis, the cross section dimensions of 16K7 and 14K4 for 3E structural design scenario are given in Table 24.

Table 24 Cross section dimensions for open web joists

	Top/Bottom Chords (double angle)	Diagonal Rod (diameter)
16K7	1 ¾ X 1 ¾ X 3/16	11/16
14K4	1 ½ X 1 ½ X 3/16	5/8

These dimensions were obtained from a manufacture. Based on the information provided by this manufacture, the dimensions may not be the exactly dimensions of open web joists used in construction field since all the dimensions varies with manufactures. Since the designs vary from different manufactures, the name of this manufacture is kept as confidential information.

Composite Columns: It is hard to isolate behavior of steel core under fire condition because the composite acts as a whole to support the load. Therefore, composite column was examined by its load capacity, and the equation is shown below, which is from section 17.6 of *Structural Steel Design* by Jack C. McCormac.

$$P_n = A_s F_y + A_{sr} F_{yr} + 0.85 f'_c A_c$$

Where,

$$A_s = area\ of\ steel\ section$$

$$A_{sr} = total\ area\ of\ continuous\ reinforcing\ steel\ bars\ (4\ No.9\ Grade\ 60\ steel\ bars\ in\ the\ design)$$

$$F_{yr} = specified\ minimum\ yield\ strenght\ of\ steel\ bars\ (60ksi)$$

$$A_c = area\ of\ concrete$$

$$f'_c = compressive\ strenght\ of\ concrete\ (4ksi\ is\ the\ average\ value)$$

Since the yield strength of steel and concrete decreases with elevated temperature, P_n will decrease with increasing fire exposure. Once the gravity load is larger than P_n the concrete may be crushed and the steel member starts to yield and thus the column fails. If the column is restrained, under elevated temperature it is subject to both gravity load and force induced by thermal expansion. Based on the Young's modulus and thermal expansion strain, the thermal expansion stress can be calculated by $\sigma_{th} = \varepsilon_{th} E_{th}$.

Figure 69 below details the total load requirements for the beams, girders and columns for our designed structure in a fire scenario. It should be noted that the load values for columns are the cumulated values which is the sum of the loads from the upper floor(s) and roof.

Member Location			Member Type	Total Load (lb) or (lb/ft)			
				Floor	Roof		
Beams(floors & roof)		Interior	16K7	374	269	lb/ft	
		Exterior	14K4	187	134.5	lb/ft	
Gider(floors & roof)		Interior	W14x30	3740	2690	lb/ft	
		Exterior	W12X22	1870	1345	lb/ft	
Column	4th	Interior	8"x8" W6x8.5		2690	lb	
		Exterior	8"x8" W6x8.5	N/A	33625	lb	
		Corner	8"x8" W6x8.5		16812.5	lb	
	3rd	Interior	10"x10" W6x12	96190		lb	
		Exterior	8"x8" W6x8.5	80375		lb	
		Corner	8"x8" W6x8.5	40187.5		lb	
	2nd	Interior	12"X12" W10x15	189690		lb	
		Exterior	8"x8" W6x8.5	127125	N/A	lb	
		Corner	8"x8" W6x8.5	63562.5		lb	
	1st	Interior	14"x14" W12X26	283190		lb	
		Exterior	10"X10"W8X13	173875		lb	
		Corner	8"x8" W6x8.5	86937.5		lb	

Figure 73 Load Requirements

The "total load" field reflects the dead load and live load on the particular member during a fire condition. Appendix 4 of the *AISC Steel Construction Manual* provides load combinations for members during fire conditions. These load combinations account for significantly less live load than normal LRFD load combination. This is because during a fire, the majority of live load (people) will have exited the office building. These minimum load requirements will be used to determine points of failure of each structural member. The members highlighted in pink in Figure 73 have the largest total loads for that particular member size, thus the loads associated with those members will be used in the heat transfer and strength analysis.

11.4 Heat Transfer Analyses

As discussed in pervious section, steel structural members are vulnerable when exposed to high temperature. Although the building in this projected is fully protected with sprinklers, the performance of steel structural member should be investigated to ensure that in case of the failure of sprinkler systems the steel structural system can survive in the fire.

It is known that yield strength is decreasing with elevated temperature. And this property of steel has been captured by the formulas present in section 4-9 of *SFPE Handbook.*

For $0 < T \leq 600°C$,

$$F_y = \left[1 + \frac{T}{900 \ln\left(\frac{T}{1750}\right)}\right] F_{y0} \qquad (1)$$

For $T > 600°C$,

$$F_y = \frac{340 - 0.34T}{T - 240} F_{y0} \qquad (2)$$

Since the yield strength is a function of temperature of steel member, it is essential to capture the temperature change of steel member under elevated surrounding temperatures. Therefore, the team conducted the heat transfer analysis to figure out the temperature of steel structural members: W shape girders, open web joist, and the composite columns.

11.4.1 Girders and Open Web Joists

W shape girders and open web joist as the infill beams are Type IIB construction and not required to be protected by the *IBC 2009*. Therefore, there is no applied fire proofing spray or insulation for them. However, for atheistic reason, structural members shall be separated from room by suspended ceiling. Therefore, the team has two options: 1).utilizing suspended ceiling system to create a membrane protection for steel member, 2) using ceiling system which has no effect to delay the temperature elevation in steel. Both cases were analyzed in this project, and the results are in this section. In addition, recommendation was made based on these results.

11.4.1.1 Unprotected Steel Member

A step-by-step heat transfer analysis for temperature elevation of unprotected steel member is presented in Chapter 8 Steel Structures of *Structural Design for Fire Safety*. Therefore, the step-by-step method will not be reproduced again this project. However, it is worth mentioning the key ideas and factors used in such analysis.

Essentially, this analysis was trying to establish simple numerical model, with known surrounding temperature and principles of heat transfer, to solve for the temperature change in steel. The governing equation is based on energy conservation: the energy stored in the steel member is equal to the energy transferred into the steel member. And the steel member is treated as lumped mass or temperature is assumed uniformly distributed because of steel's high thermal conductivity. Or, mathematically, for Δt:

$$\Delta Q_{steel} = \Delta Q_{transfer} \qquad (3)$$

Or,

$$c_{p,steel}\, \Delta T_{steel}\, V_{steel}\, \rho_{steel} = \bar{h} A_{steel} \left(T_{gas} - T_{steel}\right) \Delta t \qquad (4)$$

Thus,

$$\Delta T_{steel} = \frac{\bar{h}}{c_{p,steel}\, \rho_{steel}} \frac{A_{steel}}{V_{steel}} \left(T_{gas} - T_{steel}\right) \Delta t \qquad (5)$$

Where,

$$\bar{h} = h_c + h_r = h_c + \frac{\sigma \varepsilon_r}{(T_{gas} - T_{steel})} \left(T_{gas}{}^4 - T_{steel}{}^4\right) \qquad (6)$$

h_c is a constant which is 0.025 kW/m^2°C suggested by *SFPE Handbook*

$c_{p,steel}$ is a constant which is 0.52 kJ/kg˚C suggested by ECCS Technical Committee 3 – Fire Safety of Steel Structures

$\varepsilon_r = 0.5$ according to Table 4-9.7 of *SFPE Handbook*

It should be noted that the dimensions and the cross-section shape vary with different steel member. Therefore, the value of the section factor $\frac{A_{steel}}{V_{steel}}$ is unique for each steel member. Figure 74 below shows the recommended calculation methods of $\frac{A_{steel}}{V_{steel}}$ for different types of steel member.

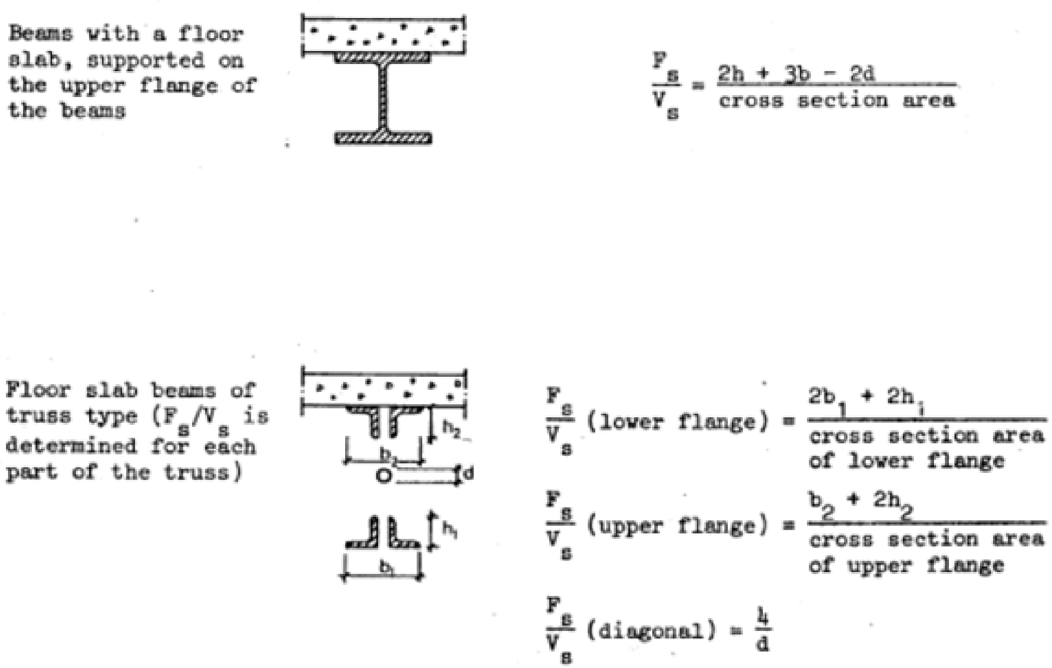


Figure 74 The Shape Factor for W-shape beam and Open Web Joist (Pettersson & Magnusson, 1976)

11.4.1.2 Steel Structural Member Protected Suspended Ceiling

The fundamental concepts for heat transfer analysis are the same as those for unprotected structural member. However, steel members are not directly exposed to the hot gas. Instead, they are speared from the hot gas of the room by the suspended ceiling. And the air in the concealed space between the ceiling and slab will first be heated by the heat from the room through the ceiling. And then the steel member is heated by the air in the concealed space by convection and radiation. Furthermore, the ceiling tile will radiate heat to the bottom flange of steel member. For analysis, the team referenced the suspended ceiling design from *Standard Details for Fire-Resistive Building Construction* as the design for our structural system which is shown in Figure 75. Therefore, in this project, the ½" gypsum board ceiling system is installed 1" below the deepest beam or girder member. For example, in the selected structural system 3E, the interior infill beam is 16K7 with nominal depth of 16" while the interior girder is W14X30 with nominal depth of 14". Therefore, the ceiling is installed 1" below the bottom 16K7 or (16"+1"-14")=3" below the bottom flange of W14X30.

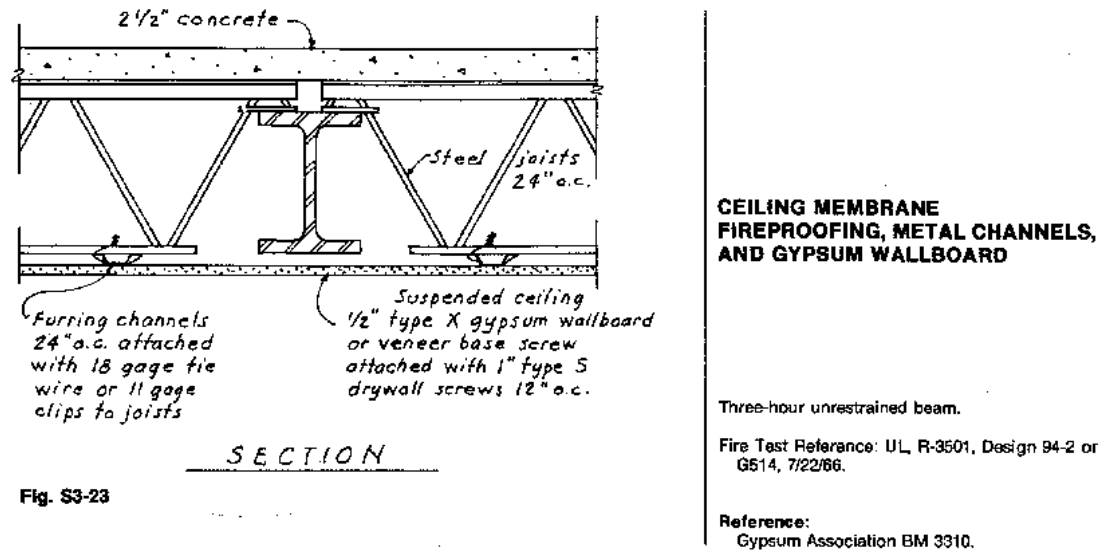


Figure 75 Suspended Ceiling (Przetak, 1977)

With this in mind, the first step becomes to establish the model to capture the temperature change of the air in the concealed space. Once the air temperature of concealed space is available, the same equation for unprotected steel member can be used to determine the temperature change in steel. Therefore, a method to establish a numerical model to simulate the temperature change of the air in concealed space is presented in this section. This method can be used for both W shape beams and open web joists. The following example analysis is based on W14X30 girder and the geometry is shown in Figure 76. The size of the gap, s, depends on the depth of 16K7 open web joist as mentioned before. In this case it is 3".

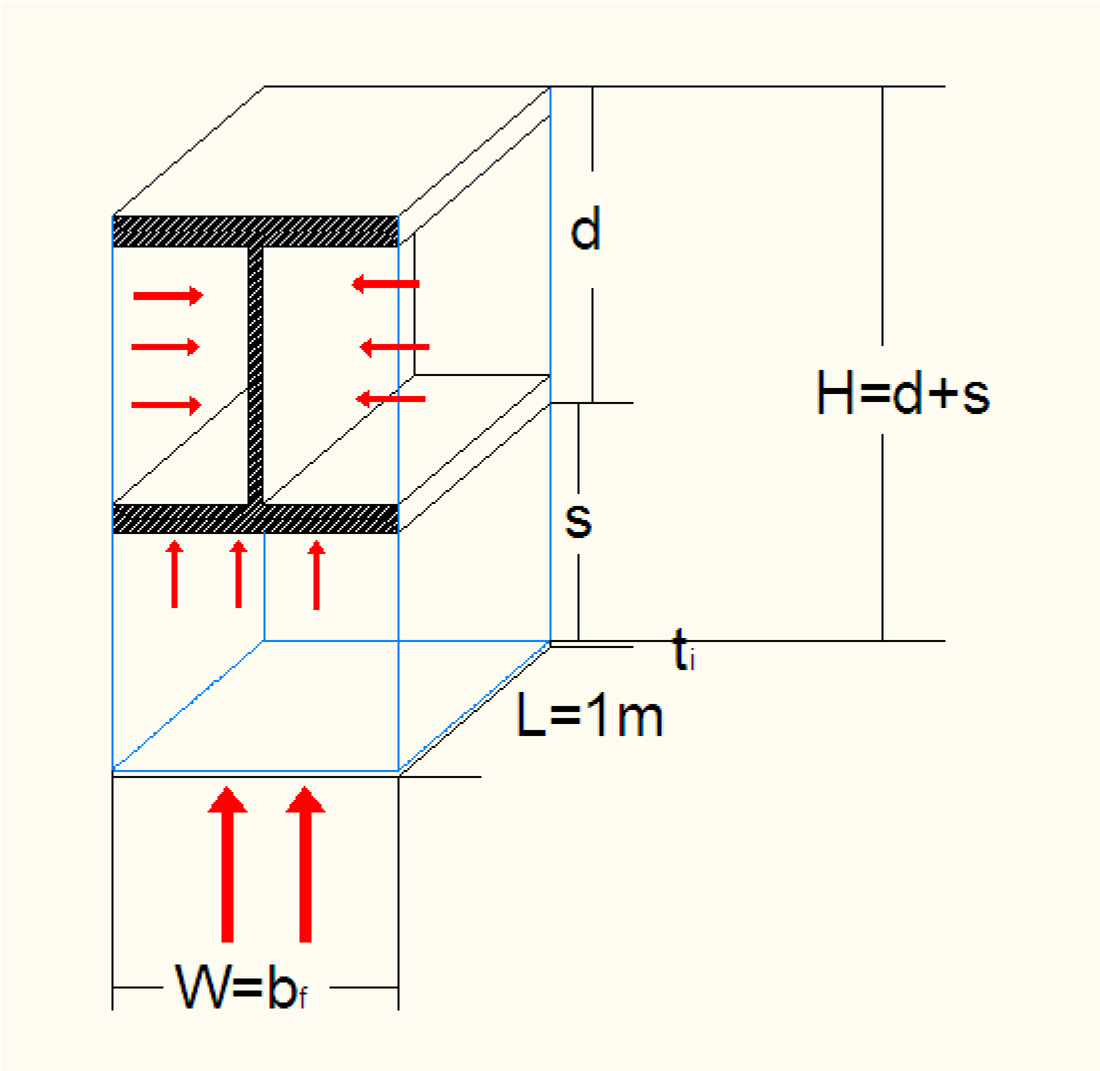


Figure 76 Control Volume Analyses

Since the air in concealed space is heated, it will move upward due to the buoyancy force. Therefore, the team assumed that air in concealed space has even distributed temperature. Based on this assumption, the team did control volume analysis for the temperature change of air in concealed space (The control volume is defined as shown in Figure 76, which is the blue box). In other words, the team analyzed the air temperature change in the control volume when a heat flux is applied on area A=WL.

The convection and radiation heat transfer are involved in both sides of the ceiling. And the Bi number of ½" gypsum board is much larger than 0.1. Thus the ceiling cannot be analyzed by lumped sum method. The temperature gradient for both sides of the ceiling board is illustrated in Figure 77.

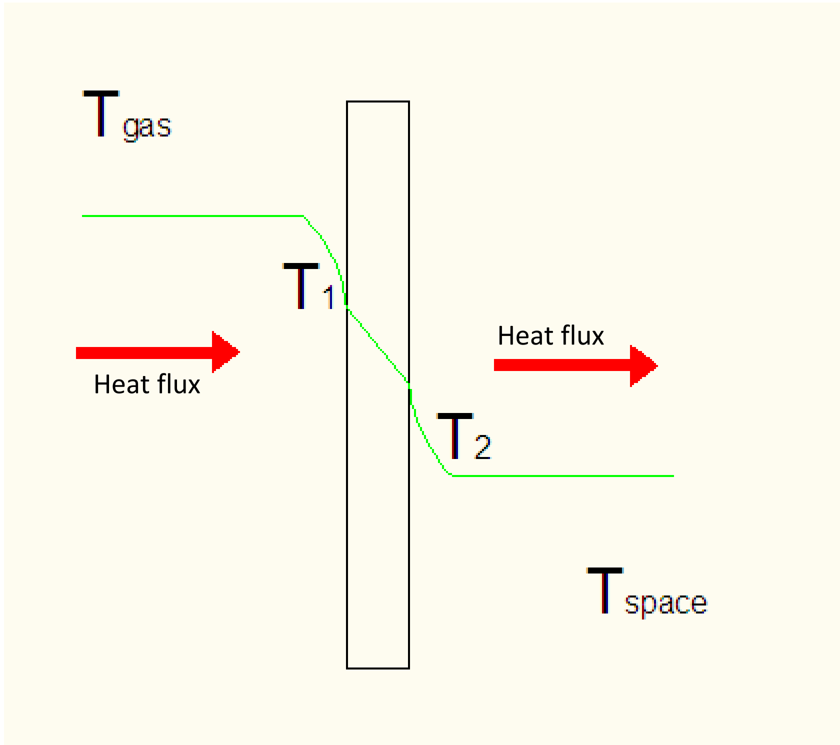


Figure 77 Heat Transfer through the ceiling

By conducting one-dimensional heat transfer analysis, we got:

$$q = \frac{T_{gas} - T_1}{\bar{h}_1 A} = \frac{T_2 - T_{space}}{\bar{h}_2 A} = \frac{T_1 - T_2}{\frac{t_i}{k_i A}} \qquad (7)$$

Where:

$$A = LW \qquad (8)$$

$$\bar{h}_1 = h_c + h_{r,1} = h_c + \frac{\sigma \varepsilon_r}{(T_{gas} - T_1)} ({T_{gas}}^4 - {T_1}^4) \qquad (9)$$

$$\bar{h}_2 = h_c + h_{r,2} = h_c + \frac{\sigma \varepsilon_r}{(T_2 - T_{space})} ({T_2}^4 - {T_{space}}^4) \qquad (10)$$

Rearranging the equation above, we got:

$$q = \frac{T_{gas} - T_{space}}{\frac{t_i}{k_i A} + \bar{h}_2 A + \bar{h}_1 A} \qquad (11)$$

For gypsum board, the internal resistance to heat transfer is much larger than that on the surface. Therefore, $\frac{t_i}{k_i A} \gg \bar{h}_2 A + \bar{h}_1 A$. And thus,

$$q = \frac{T_{gas} - T_{space}}{\frac{t_i}{k_i A}} \qquad (12)$$

The governing equation is still the energy conservation. Or in other words, the energy stored in steel, air in concealed space and the ceiling gypsum board is equal to the heat transferred into them.

$$\Delta Q_{steel} + \Delta Q_{space} + \Delta Q_{ceiling} = \Delta Q_{transfer} \qquad (13)$$

Expanded from the equation above, we have:

$$c_{p,steel}\,\Delta T_{steel}\,V_{steel}\,\rho_{steel} + c_{p,air}\,\Delta T_{space}\,V_{space}\,\rho_{air} + c_{pi}\,\Delta T_i V_i \rho_i = q\Delta t$$

$$= \frac{(T_{gas} - T_{space})}{t_i/k_i A}\Delta t \qquad (14)$$

Where,

$$\Delta T_i = \frac{1}{2}(\Delta T_1 + \Delta T_2) = \frac{1}{2}[\left(T_{1,n+1} - T_{1,n}\right) + \left(T_{2,n+1} - T_{2,n}\right) \qquad (14)$$

And $T_{1,n} = T_{gas} - \frac{q}{h_1 A}$ (15); $T_{2,n} = \frac{q}{h_2 A} - T_{space}$ (16)

Recalled from the heat transfer analysis, the total heat transfer coefficients for both sides of the ceiling are:

$$\bar{h}_1 = h_c + h_{r,1} = h_c + \frac{\sigma\varepsilon_r}{(T_{gas} - T_1)}\left(T_{gas}{}^4 - T_1{}^4\right) \qquad (9)$$

$$\bar{h}_2 = h_c + h_{r,2} = h_c + \frac{\sigma\varepsilon_r}{(T_2 - T_{space})}\left(T_2{}^4 - T_{space}{}^4\right) \qquad (10)$$

Unlike the unprotected steel member analysis whose governing equation has only one unknown - T_{steel}, there are too many unknown parameters in the governing equation (T_1, T_2, T_{space}, T_{steel}), and it is impossible to solve for T_{space}. Therefore, the team modified the method based on the analysis above to reduce the unknown parameters into on and thus made an approximation for T_{space}.

Modified Method

In order to eliminate the unknown parameters, we ignored the radiation effects from hot gas to ceiling and from ceiling to concealed space. Therefore, the total heat transfer coefficients for both sides of the ceiling are all convection heat transfer coefficient which is a constant. Thus,

$T_{1,n} = T_{gas} - \frac{q}{h_c A}$ (17); $T_{2,n} = \frac{q}{h_c A} - T_{space}$ (18)

And $\Delta T_1 = T_{1,n+1} - T_{1,n}$ (19); $\Delta T_2 = T_{2,n+1} - T_{2,n}$ (20)

Rearranging the equations above, we had:

$$\Delta T_1 = \Delta T_g - \frac{\Delta T_g - \Delta T_{space}}{t_i/k_i A} \qquad (21)$$

$$\Delta T_2 = \frac{\Delta T_g - \Delta T_{space}}{t_i/k_i A} - \Delta T_{space} \qquad (22)$$

Therefore, T_1 and T_2 are not unknown parameters anymore.

Furthermore, we assumed the bulk temperature of steel members and air in concealed space is the same. Therefore, the energy stored in steel member can be estimated. More specifically, for same volume of steel and air, if the temperature change is the same, the difference of energy stored in steel and air is depend on their density and heat capacity. Therefore,

$$\frac{\Delta Q_{steel}}{\Delta Q_{air}} = \frac{c_{p,steel}\,\Delta T_{steel}\,V_{steel}\,\rho_{steel}}{c_p\,\Delta T_{air}\,V_{air}\,\rho_{air}} = \frac{0.54\,\frac{kJ}{kgK}\,7850 kg/m^3}{1\,\frac{kJ}{kgK}\,1 kg/m^3} = 3925 \qquad (23)$$

Therefore, with same temperature change, the energy stored in steel member can be estimated by multiply 3925 to the energy stored in air with same volume. In other words, we can rewrite a portion of the left hand side of the governing equation in this way:

$$\begin{aligned} &c_{p,steel}\,\Delta T_{steel}\,V_{steel}\,\rho_{steel} + c_{p,air}\,\Delta T_{space}\,V_{space}\,\rho_{air} \\ &\qquad = 3925\,c_{p,air}\,\Delta T_{space}\,V_{steel}\,\rho_{air} + c_{p,air}\,\Delta T_{space}\,V_{space}\,\rho_{air} \qquad (24) \end{aligned}$$

Where,

$$V_{steel} = A_{cross-section}\,L \qquad (25); \quad V_{space} = WLH - V_{steel} \qquad (26)$$

Substitute equation 21, 22, and 24 in equation 14 and rearrange the terms we got:

$$\Delta T_{space} = \frac{\left(\frac{T_g - T_{space}}{\frac{t_i}{Ak_i}}\,\Delta t - \frac{1}{2}c_{pi}At_i\rho_i\Delta T_{gas}\right)}{\left[c_{p,air}\,\rho_{air}\,\left(V_{space} + 3925 V_{steel}\right) - \frac{1}{2}c_{pi}At_i\right]} \qquad (27)$$

Where,

$$\Delta T_{gas} = T_{gas,n+1} - T_{gas,n} \qquad (28)$$

$$\Delta T_{space} = T_{space,n+1} - T_{space,n} \qquad (29)$$

With equations 27 to 29, excel can be utilized to calculat the temperature change of the air in concealed space. Based on the temperature of air in concealed space, using equation 5 from unprotected steel member analysis, the team simulated the temperature change in steel member with temperature change of the air in concealed space.

11.4.2 Composite Columns

The composite design can utilize the properties of concrete to protect the steel from fire. Like steel, concrete is non-combustible, however, it has lower thermal conductivity and higher specific heat capacity than steel. Furthermore, the cement paste in concrete undergoes an endothermic reaction when heated, which can reduce the temperature rising of the structural member. The lightweight concrete has been shown to have excellent fire resistance, due to the low thermal conductivity compared with normal weight concrete. (Buchanan, 2002) Therefore for the steel member encased in the concrete, the temperature elevation in steel is much slower than for the unprotected steel members. In this particular case, the team conducted the heat transfer analysis based on "boxed column" model which is conservative and simplified version of encased steel composite column. The boxed column is actually a steel I beam encased by 4 boards and the material of these boards are concrete as shown in Figure 78.

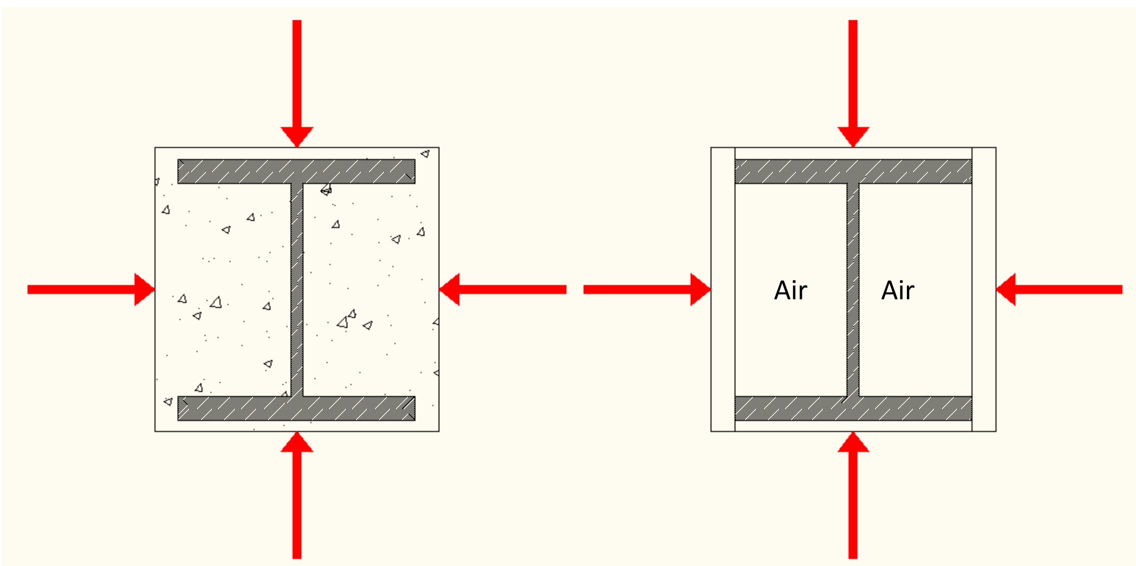


Figure 78 Left: Encased steel composite column; Right: Boxed column model (red arrows represents the heat flux)

There are several reasons for using this method. First of all the primary heat transfer mode for heating up the steel is conduction. Therefore, the heat needs to travel through the concrete. The thickness of concrete from the surface to the steel core is different and heat travels less distance to reach the steel flange than web as shown in the Figure 75 below where the red arrows represents the heat flux. Second, once the flanges are heated, due to the high thermal conductivity, the heat will transfer though the web and heat it up while the portion of concrete beside the web, which is highlight in blue in Figure79, has not been heated up to the temperature of the web by the heat flux from the surface. Therefore, the highlighted portion of concrete becomes a sink absorbing the heat from the web and thus reduces the temperature of the web.

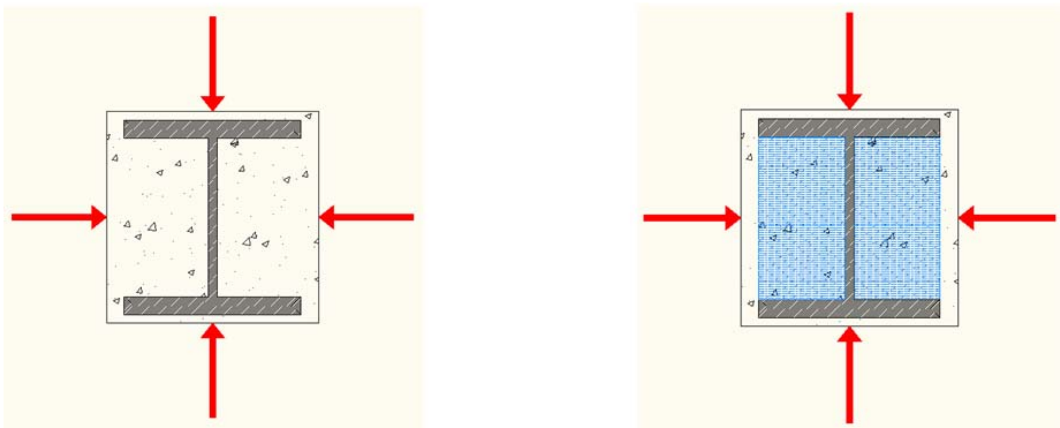


Figure 79 Heat sink

The boxed column model captured the fact that heat transfer to steel member is primarily from the flanges and then the web. Furthermore, in the boxed column model, the air replaced the heat sink – the highlighted part of concrete. Different from the concrete, air does not act like heat sink as concrete due to the density and heat capacity of air. In other words, the web cannot be cooled by the air in the boxed column. Therefore the air can be viewed as a conservation factor to the heat transfer and temperature elevation analysis.

A step-by-step heat transfer analysis for temperature calculation of protected steel member is covered in Chapter 8 of *Structural Design for Fire Safety*. Hence, the details are not going to be shown in this project. However, it is worth mentioning the key ideas and factors used in such analysis. The governing equation is based on energy conservation: the energy stored in steel member is equal to the energy transferred into steel member. And steel member is assumed as lumped mass or temperature is uniformly distributed due to the high thermal conductivity of steel. Or, mathematically, for Δt:

$$\Delta Q_{steel} + \Delta Q_i = \Delta Q_{transfer} \qquad (30)$$

Or,

$$c_{pi} \Delta T_i A t_i \rho_i + c_{p,steel} \Delta T_{steel} V_{steel} \rho_{steel} = \frac{k_i}{t_i} A_i \left(T_{gas} - T_{steel}\right) \Delta t \qquad (31)$$

Where,

$$\Delta T_i = \frac{1}{2}\left(\Delta T_g + \Delta T_s\right) \qquad (32)$$

Thus,

$$\Delta T_{steel} = \frac{\frac{k_i}{t_i} A_i (T_{gas} - T_{steel}) \Delta t - \frac{1}{2} c_{pi} A t_i \rho_i \Delta T_{gas}}{c_{ps} \rho_s \frac{V_s}{A_i} + \frac{1}{2} c_{pi} t_i \rho_i} \tag{33}$$

Where,

$c_{p,steel}$ is a constant which is 0.52 kJ/kg˚C suggested by ECCS Technical Committee 3 – Fire Safety of steel Structures

ε_r is 0.7 according to Table 4-9.7 of *SFPE Handbook*

c_{pi} is the heat capacity of concrete which is 0.96 kJ/kgK and ρ_i is the density of concrete which is 1750kg/m^3 (Note: in reality, c_{pi} varies with temperature, in this calculation, it is assumed to be a constant.)

Based on *Swedish Steel Design Manual* the recommended calculation method for $\frac{V_s}{A_i}$ of boxed column is shown below in Figure 80.

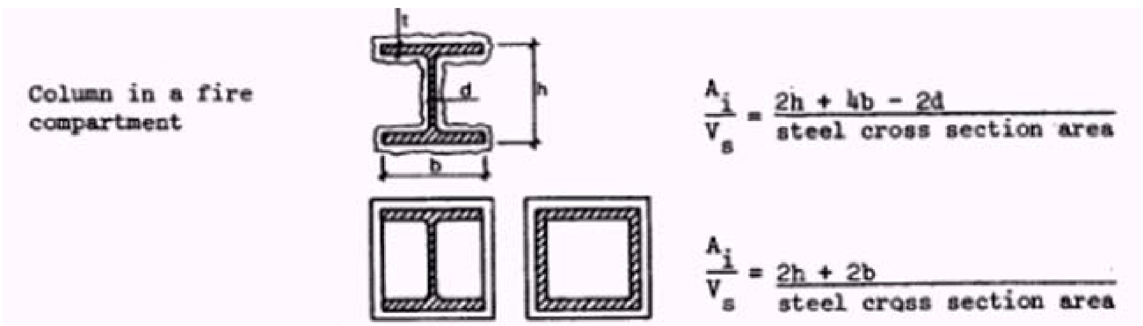


Figure 80 Shape Factor (Pettersson & Magnusson, 1976)

11.4.3 Design Fire

Full-size fire-resistance tests are used to assess the fire performance of building materials and structural elements. The time-temperature curve used in fire-resistance tests is called the "standard curve". ASTM E119 and ISO 834 curves are widely used in the world (Buchannan, 2002). In this project, the team analyzed the structural element or the structural assembly (girder, open web joist with suspended ceiling protection) exposed to the standard fire that matches ASTM E 119. The ASTM E 119 curve is defined by a number of discrete points, which are shown in Table 25.

Table 25 Definition point for ASTM E 119

Time (minutes)	ASTM E 119 Temperature (˚C)
0	20
5	538
10	704
30	843
60	927
120	1010
240	1093
480	1260

Additionally, an equation approximating the ASTM E 119 curve is given in Chapter 4-8 of *SFPE Handbook*.

$$T = 750\left[1 - e^{-3.79533\sqrt{t_h}}\right] + 170341\sqrt{t_h} + T_0$$

Where, T is in˚C, and t_h is the time in hour.

With the equation shown above, the team could plot ASTM E 119 as shown below.

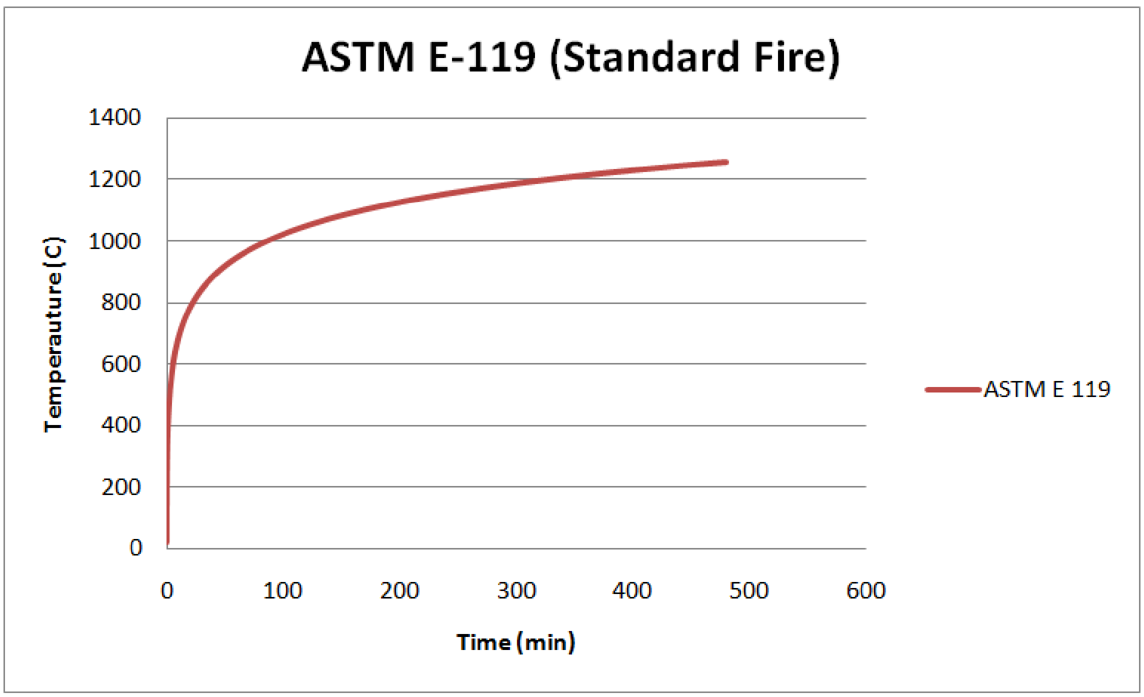


Figure 81 ASTM E-119 standard fire curve

Although the standard fire is widely used to assess the structural element performance under fire conditions, it should be noted that a natural fire behave different from the standard fire. For example, a plot for several different workstations shown in Figure 82 below is taken from Chapter 3-1 of *SFPE Handbook*. It is clear that a natural fire has four different realms: growth, peak, steady burning, and decay. Therefore, it is also necessary to use actual room fires to assess the performance of structural elements.

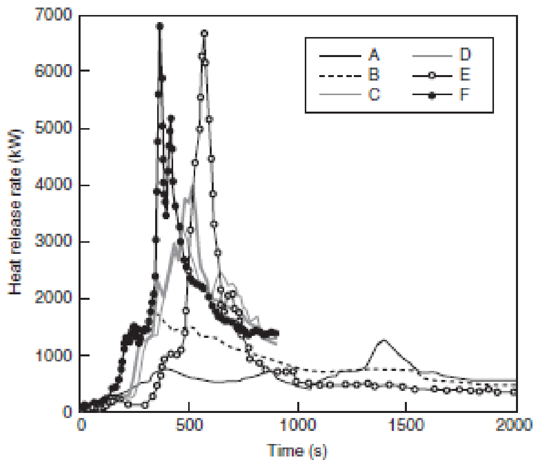


Table 3-1.11 ***Identification of Workstations in Figure 3.1.38***

Code	Combustible Mass (kg)	Description	Number of Sides with Acoustic Panels	Reference
A	291	Mostly old-style wood furniture	0	110
B	291	Semi-modern furniture	1	110
C	335	Modern furniture	2	112
D	—	Modern furniture	3	112
E	291	Modern furniture	4	110
F	—	Modern furniture	4	112

Figure 82 Workstation fire

However, to model the fire in any room of this office building is a very complex problem. Essentially, there are two ways to model the fire, either by zone modeling or field models. Zone modeling is the most identifiable lumped-mass model. In this type of modeling, a compartment is idealized as containing one or more fire sources, and the fire environment is modeled as one or two discrete control volumes or zones. It is relatively simple technique. The core idea of zone model is to treat the fire as a point source. Furthermore, the growth and spread of fire in compartment needs to be defined for zone modeling. (Quintiere, 2002)

In this office building, the most hazardous items are the workstations, and therefore, they are the main fuel for fire. If they ignited, and fire starts to spread, the fire cannot be treated as point source anymore. Furthermore, it is hard to define the room fire as it is multiple workstation fire. The single workstation fire has been studied by National Institute of Standard and Technology (NIST) and the results of heat release rate curves are presented in *SFPE Handbook* which is shown in Figure 78 above. The heat release curve is the most important data to define the fire growth. However, the fire growth and spread for any room in this office building involves the process that fire spread from one workstation to others. Thus the heat release rate for this process is quite different from single fire for workstation and simply superposing heat release curves into one cannot reveal the actual physics of the fire growth. And there is no sophisticate study on how to convert multiple single fire heat release rate curve into one equivalent curve. In other words, the fire, its growth and heat release rate cannot be defined without full scale tests. Therefore, without well defined fire, zone modeling cannot simulate the temperature changes of a compartment.

In order to simulate the room fire, field models, such as computational fluid dynamics or Fire Dynamic Simulator (FDS), are needed. These program are the computational finite-element solver. Therefore, it is capable solve the issues mentioned before. However, it requires time devotion to master FDS and other same type of softwares. Due to the time constrain, FDS was not utilized in this project. Therefore, in this project, the team only investigated the performance of structural elements and assemblies under standard fire – ASTM E 119.

11.5 Results and Discussion

11.5.1 Temperature Change

11.5.1.1 Unprotected Girders and Open Web Joists

The temperature changes of different girders and open web joists were plotted in Figure 83. The temperature changes of unprotected structural steel members are rapid. No matter what size of the steel member, all temperature curves converge with ASTM E-119 curve within 30 minutes. The temperature difference is more obvious within the initial 5 minutes where the heating just started. The Excel application showed that with larger member size, the heating process is slower. The girders were heated slowest comparing with open web joists. Therefore, the wide-flange girders have better thermal resistance to the open web joists. Furthermore, within an open web joist, the bottom chord is the most vulnerable part. As shown in Figure 83, the curves of the bottom chords of 16K7 and 14K4 are near the ASTM E-119 curve. The curve of bottom chord of 14K4 is slightly higher than 16K7 is because member size of 14K4 is smaller than the one of 16K7. Therefore, based on thermal resistance or the temperature change, wide-flange steel members are better than open web joist. However, within 5 to 10 minutes, all steel members were heated to above 500˚C which is a critical temperature for steel structural member. (Drysdale, The Psot-Flashover Compartment Fire, 1998) Therefore, in general, steel structural elements are vulnerable to fire.

11.5.1.2 Girders and Open Web Joists with Membrane Protection

With membrane protection (suspended ceiling protection), the temperature changes in the structural steel were delayed to significant extend. As shown in Figure 84, the time-temperature curves of girders do not converge with ASTM E-119. And for the temperature curves of elements of open web joists, the convergence does not happen until around 150 minutes. The Excel application shows the membrane protection is a good application to protect the floor system. However, it should be noted that in reality, whether the ceiling can efficient block the heat from steel structures is complex problem. In reality, ceiling assembly may collapse and expose the structural system to fire, or the penetrations in ceiling (such as ducts for air ventilation, and electric lights and cables) are not well sealed or protected, the heat or the fire may go though the penetration to heat up the structural member. In our analysis, we assumed the ceiling assembly is perfected installed. Any situation that may cause fire penetrate into the cavity space between slab and ceiling was excluded from the analysis and the excel application. Nevertheless, the performance of ceiling assembly as a membrane protection to structural steel elements is a good study for future work.

11.5.1.3 Composite Columns

Due to the concrete, the steel core inside the composite columns has some degree of protection. In the incident of fire, it requires certain time for the heat to penetrate the concrete and then heat up the steel core. Therefore before the heat penetrates concrete, the steel member should keep the same temperature. This fact was revealed in this analysis. As shown in Figure 85, for the initial 15 minutes, the temperature of the steel core does not rise. However it should be noted that the

10”X10” composite column with steel member of W6X12 is quite unique, comparing to others. Although, the size of column and the size of the steel members vary, the temperature curves for other columns converge into one curve, while temperature curve of 10”X10” composite column with W6X12 is much lower than the rests. The key difference between 10”X10” composite column with W6X12 and others is that, it has 2” thick concrete between the flange and the exposed surface while others have only 1” thick concrete. Therefore, the size of the wide-flange section in the composite column is only the secondary factor for temperature change. The thickness of the concrete between the flanges to the exposed surface is critical to the temperature change in the steel section for composite columns.

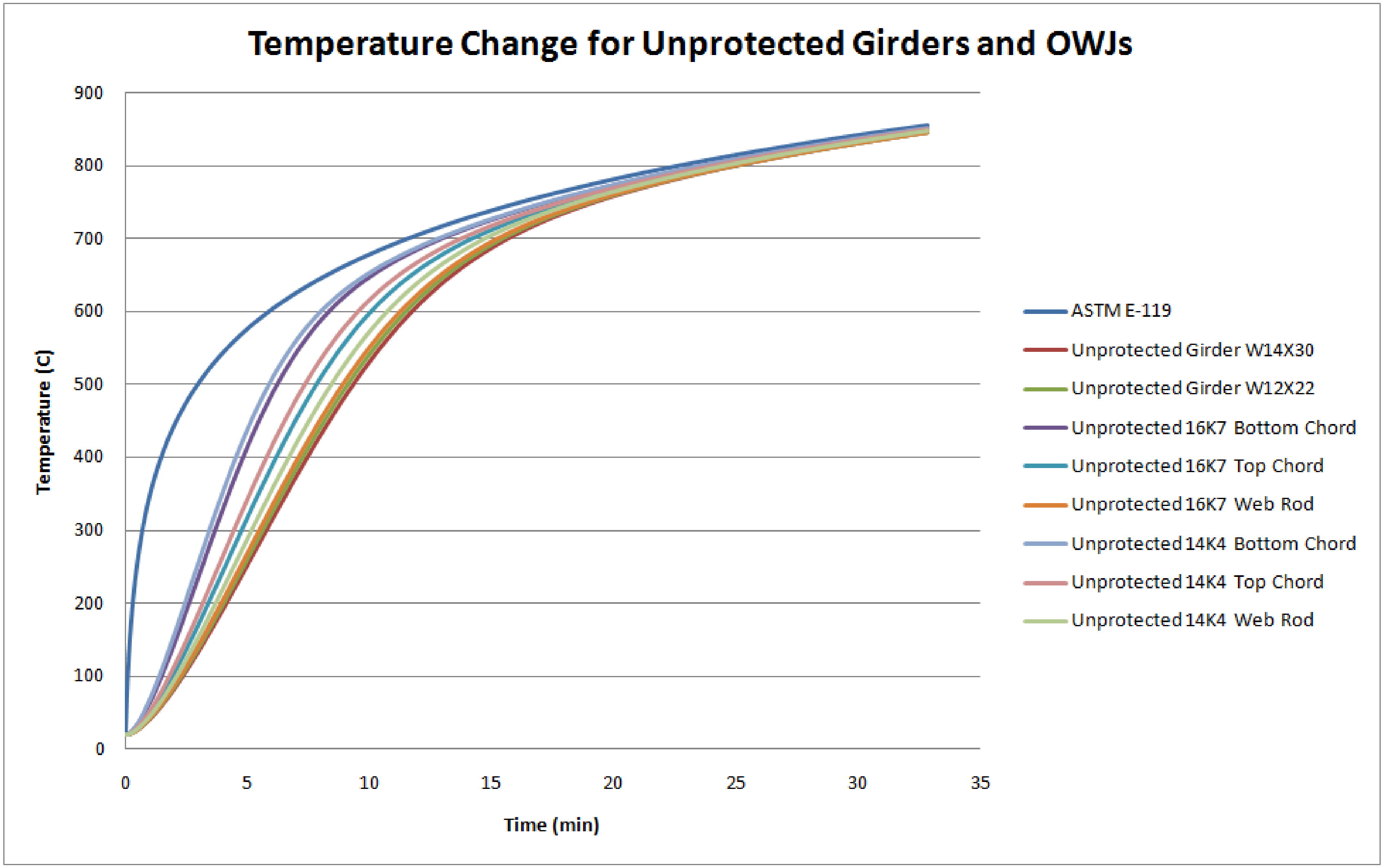


Figure 83 Temperature change for Unprotected Girders and OWJs

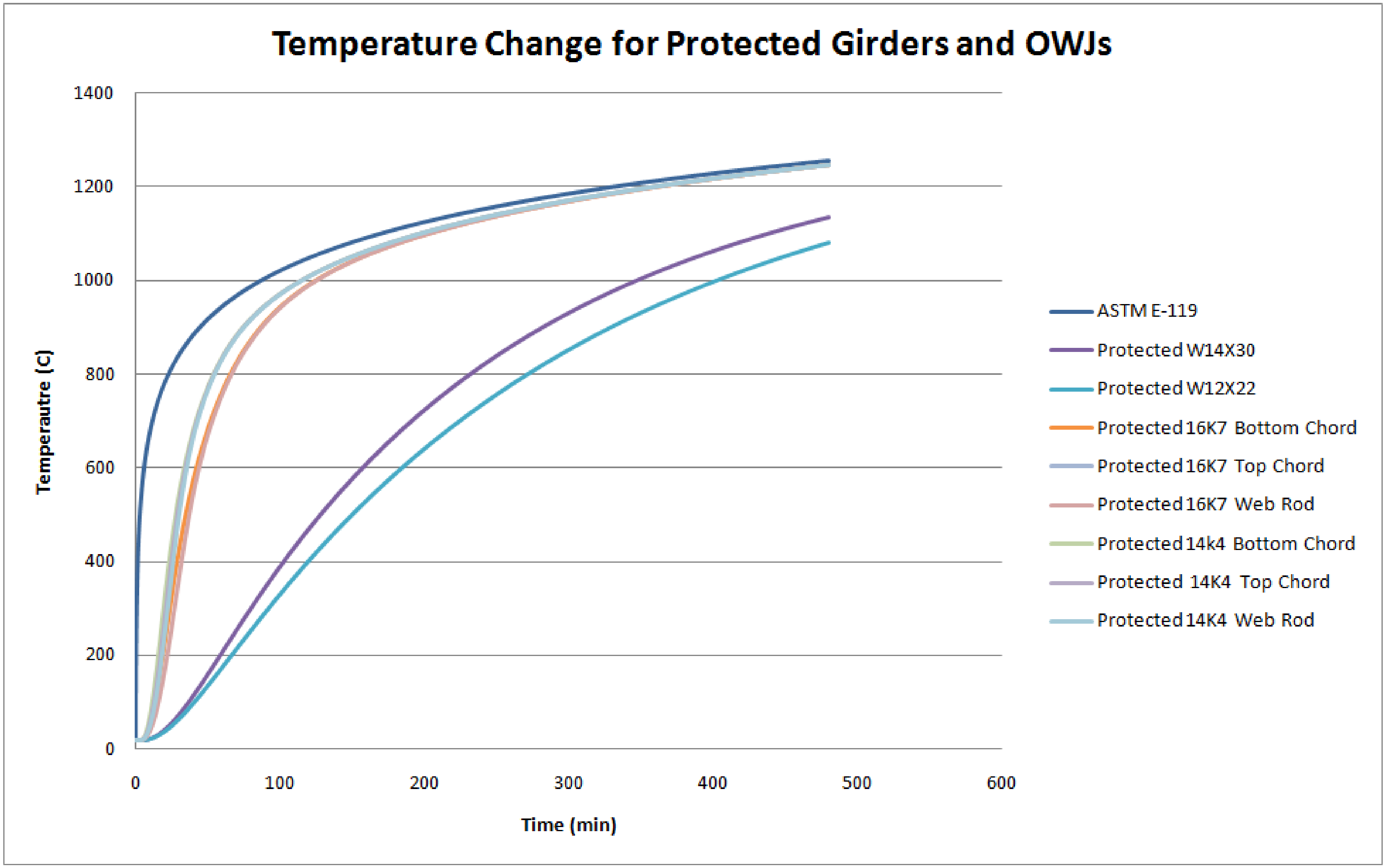


Figure 84 Temperature change for Protected Girders and OWJs

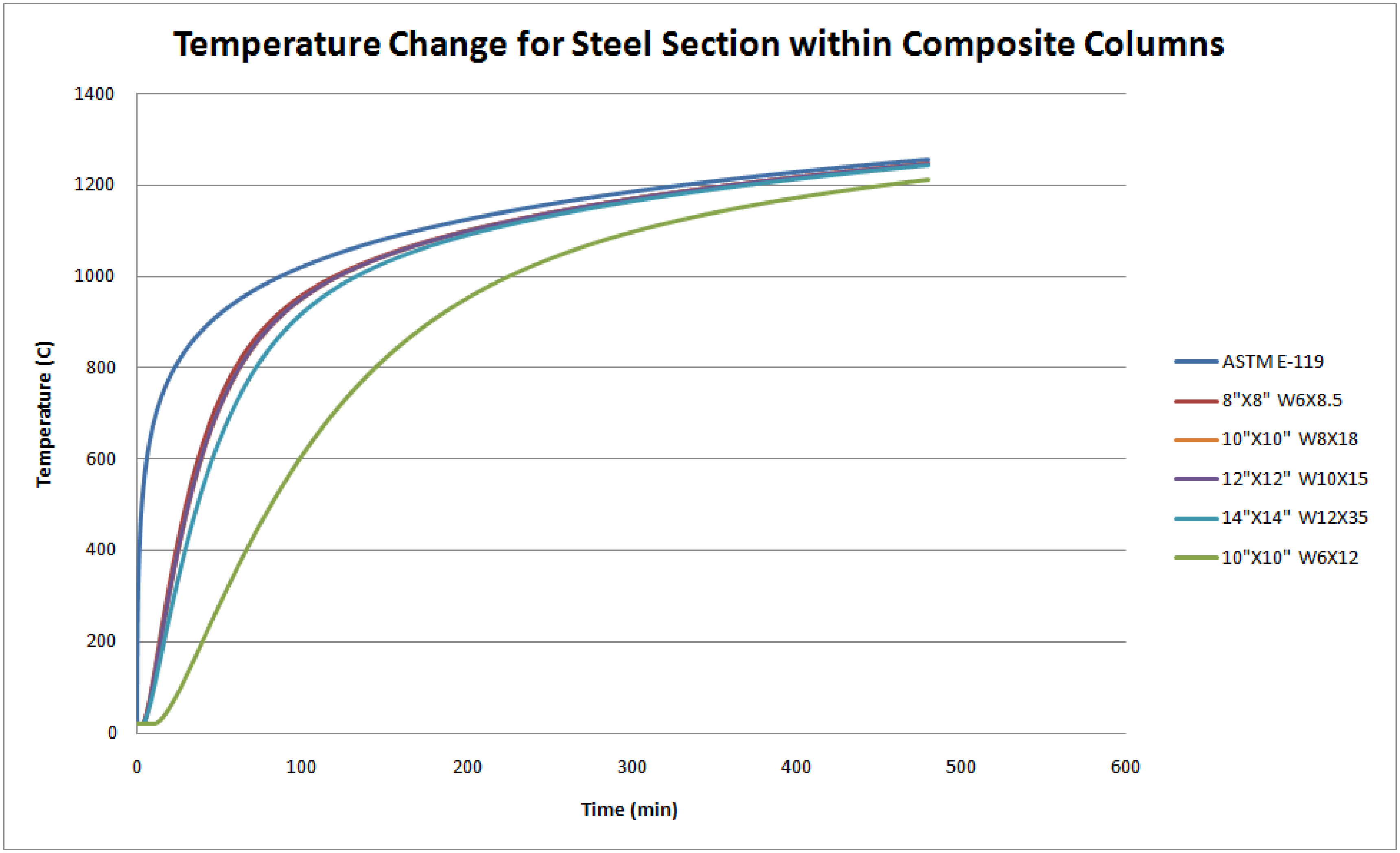


Figure 85 Temperature Change for Steel Section within Composite Columns

11.5.2 Structural Failure

For girders and open web joists, the criterion to determine the failure of each structural element is the yield stress of steel member: once the stress produced by the load exceeds the yield strength, the structural element fails. In other words, the yield strength and Young's Modulus or the load capacity decreases as temperature increasing, once the load capacity is smaller than load it subject to, the structural element is failed. For composite column, the failure criterion is the load capacity: once the load capacity of composite column (including the wide flange steel member, the steel rebar and the concrete) is lower than the applied load, the column fails. It should be noted another widely used failure criterion for column is buckling which depends on slenderness of the column. However, the force to create buckling calculated by Euler's equation is much larger than the load to make structural member reach the yield point for the composite columns of this project. An similar example calculation is shown in section 17.6 of *Structural Steel Design* by Jack C. McCormac, where the buckling load is 8650 kips while load to reach the yield point is 2410 kips. Therefore, the buckling effect was not considered in the analysis. The results of failure of structural elements are summarized in Table 26.

In reality, the failure due to insufficient yield strength usually does not cause the structure collapse. The steel members go through a large deformation once it reaches the yield point. However, steel members still have strength even at elevated temperature. Therefore, the criteria are conservative.

Table 26 Structural Failure Summary

	Steel Member		Time to Failure (min)	Temperature (C)	
				ASTM E-119	Steel Section
Unprotected (Girders & OWJs)	W14X30		3.8	537	183
	W12X22		7.34	632.9	407.7
	16K7	Bottom Chord	8	645.7	589
		Top Chord	9.73	674	589
		Web Rod	11.57	699.6	611.6
	14K4	Bottom Chord	16.4	751.5	741.8
		Top Chord	17.07	757.4	741.8
		Web Rod	15	738.5	705.3
Protected (Girders & OWJs with Membrane Protection)	W14X30		54.68	931	182.6
	W12X22		120.9	1049	406.5
	16K7	Bottom Chord	41.1	888.5	588.5
		Top Chord	42.2	892.3	588.7
		Web Rod	44.8	901.4	611.3
	14K4	Bottom Chord	46	905.3	741.9
		Top Chord	46.6	907	741.8
		Web Rod	43.3	896.4	705.3
Composite Columns	8"X8" W6X8.5	Restrained	8.1	646.9	81
		Non-restrained	48.1	913.2	718.1
	10"X10" W6X12	Restrained	21.8	793.8	67
		Non-restrained	97.45	1017.5	594
	10"X10" W8X13	Restrained	10.3	678.5	112.9
		Non-restrained	45.73	904.4	671.2
	12"X12" W10X15	Restrained	10.47	684.8	122
		Non-restrained	40	884.4	613.4
	14"X14" W12X26	Restrained	13.55	723.1	150.7
		Non-restrained	50.5	919.7	645.2

11.5.2.1 Unprotected Girders and Open Web Joist

Although it has been discussed in previous section that due to larger size, girders should perform better than open web joist under fire condition, the failures of girders actually occur before open web joist. As shown in Table 26, it takes only 3.8 minutes to cause W14X30 (interior girder) to fail and 7.34 minutes to cause W12X22 (exterior girder) to fail. This result shows that the critical members or the ones support more load are more sensitive to temperature. The analysis showed failure time for each part of open web joist, including bottom chord, top chord and web rod. As shown, different elements of the open web joist fail at different times. However, it should be noted that once one element of open web joist fails the load distribution changes dramatically. Therefore, once one element fails the entire open web joist fails. The effects of thermal expansion of girders and open web joist are more likely to be the compressive strength if there is any restrained condition. However, the major failure mode of horizontal members is moment, while compression is a minor effect. Therefore the effects due to thermal expansion were not included in the analysis.

11.5.2.2 Girders and Open Web Joist with Membrane Protection

With membrane protection, the fire resistance is enhanced significantly as shown in Table 26. With the same load condition, the failure occurs at the same temperature of steel member, respectively. The membrane system successfully delays the temperature rising within the structural elements. The shortest time is 41.1 minutes for membrane protection while for the unprotected scenario it is only 3.8 minutes. This large time difference shows how much time membrane protection can save the structure from failure. Although ASTM E-119 does not represent real fire, any fire with relevant intensity can cause significant damage to the structure. The amount of time save by membrane protection will be essential for occupancies to leave the building or for fire department to put off the fire and avoid structure collapse.

11.5.2.3 Composite Columns

Two situations were investigated for each column – restrained and non-restrained conditions. Restrained condition is much more critical since thermal expansion induces large compressive stress to the member. Therefore, under fire conditions, column is under influences of both deceasing strength and increasing thermal stress. On the other hand, columns under non-restrained condition are only affected by decreasing strength. This has been included in the analysis and the Excel application and the results shown in Table 26. As shown, the lengths of time before failure of non-restrained columns are about 4 times than the restrained conditions. This reveals that thermal stress plays a very important role in failure of column. In reality, the behaviors of columns under fire condition should fall into the zone in between restrained and non-restrained condition. Because the both ends of column are not ideal restrained, however due to rigid connection, there is some degree of resistance to free thermal expansion. Another important type of failure of column is due to the thermal expansion of girders. The horizontal force due to girder thermal expansion will push the end of column and induce dislocation. The dislocation will create a moment in the column which can accelerate the column failure. Since the focus of the project is at single structural element not the entire structural system, this interaction between girders or beams and columns was not covered in this project.

11.6 Summary

By conducting heat transfer analysis and utilizing fundamental equation of conservation of energy, the temperature change within structural elements was studied. The performance of structural member with elevated temperature was also investigated. The failure time and temperature of each member was identified. The result showed that without protection, steel structural members are extremely vulnerable to fire. However, with appropriate protection method, the temperature rising within the structural member can be delayed and thus avoid structural failure. For example, the membrane protection was provided an excellent method in this project. If suspended ceiling is proposed by architects, the project team recommends upgrading the suspended ceiling into membrane protection to enhance the fire resistance. It was found that the size has effect on the temperature rising, however when the structural element fails does not merely depend on how quick the temperature change but also the loading conditions. Furthermore, it has been identified that the restrained condition is more critical for structural member under fire condition. Another important finding is that size of member does not play an important role in fire resistance for composite column. Only the thickness of concrete from the surface to steel core is the governing factor. This study also showed the importance of sprinkler system. Without sprinkler system, structural failure is more likely to occur if the building fire reaches certain intensity.

Furthermore, the team suggested several future studies that can be conducted based on the work in this chapter: the design fire can be modified use FDS to reveal the reality; the interaction between difference structural elements, and the performance of the entire system can be further investigated; last, it will be an interesting study to investigate the performance of the suspended ceiling as the membrane protection under fire.

Chapter 12 Recommendations and Conclusion

12.1 Structural Design Recommendations

The team investigated several different structural scenarios consisting of different sized structural bays, various beam spacings, open web joists versus wide flange beams, and composite versus noncomposite construction. We identified twenty different possible combinations of these variables which each had their own constructional, architectural and cost benefits. For this project, these different combinations are defined as structural scenarios or structural schemes. We created a scoring method for these scenarios that is weighed by cost, net usable area, floor depth, member uniformity, structural modification, footing construction, and structural fire protection. The scoring system is detailed in section 6.3. After tabulating the scores, it was determined that Scenario 3E is the best suited structural scheme for the four-story office building. This scenario received a score of 23.12, and the average score of all the scenarios was 8.64. Structural Scenario 3E is the recommended design for the office building.

Scenario 3E consists of 25’x25’ structural bays with open web joists seated on composite girders. The joists within the structural bay are spaced 2.5’, and the columns are composite concrete encased. This scenario offers a low cost of construction and has other construction and architectural benefits.

By designing and presenting an analytical method of choosing a proper structural scheme, we are able to support our recommendations with actual relevant data. And with our experience and research of steel building design, we are able to design the recommended scenario so that its members are suitable for the loads associated with the building.

12.2 Framing Recommendations

Choosing a framing system is an important decision in the process of building design. A braced frame can provide adequate stability against lateral loading. However, because of the additional diagonal members that cross bays, architectural conflicts may arise. This problem is removed if a rigid frame is used to resist lateral loading. Moment-resistive connections are used and bracing members are not required. Yet, additional costs are commonly added because of the material and labor required to assemble and design these connections.

Selection of lateral steel framing for an office building is primarily dependent upon resistance performance of lateral forces that the location of the building site offers. Wind, seismic activity and soil pressure are mainly the factors to consider. Since both types of frames, rigid and braced, are capable of resisting the lateral loads for the area of construction, we investigated other considerations.

First, using *RS Means Building Construction Cost Data, 2009*, it was determined that the costs for each system were not very far from one another; both were within $50,000 to $60,000. Because the costs were so close, we recommend that a rigid frame be utilized for the office building. The rigid frame offers a great architectural advantage. The open vertical bays of the rigid frame give the architect much freedom to design large interior space. Whereas the braced frame confines the architect with load bearing walls on each vertical bay.

12.3 Sprinkler System Recommendations

The design process for an automatic sprinkler system is seemingly easy because one can technically follow the guidelines set by *NFPA 13* and come up with a design. However, as demonstrated in Chapter 8 it is imperative that the designer be experienced in sprinkler design and fully understand the implications of his or her decisions. There are many factors for which the designer is responsible for choosing, and a wrong decision may very well cause the system to fail. The selection of a system does not only affect performance but cost as well. In any area of engineering there is a delicate balance between performance, i.e. safety and cost. For example, classifying the space intended for a sprinkler system incorrectly can result in a system that is unable to control a fire. If the aboveground piping is poorly laid out, or sized, the cost will be much higher than for a better designed system.

Sprinkler system performance is also of very high importance. Our systems were hydraulically designed to determine the most efficient and cost effective system. Knowledge of how piping sizes and layouts affect system demand helps the designer to produce a system that does not require additional equipment to increase or boost the water supply. If the water demand is too high for the available water supply, a fire pump would be required, increasing the price significantly. Another factor in performance is the time at which the sprinklers will activate after a fire has ignited. Based on our considered design fire involving an office workstation, both layouts will activate between 140 and 170 seconds. This is for ideal conditions, however, it allows for a good estimate of possibility for fire control or suppression.

12.4 Detection and Alarm Recommendations

The *International Building Code, 2009* requires the designed four-story office building to have smoke-detectors for the purpose of activating smoke activated doors. These smoke detectors also offer protection to occupants of the building from fire by alerting all when smoke is in the building. The recommended type of smoke detector is a *Model 4W-B* photoelectric smoke detector from *System Sensor*. This smoke detector is typically used in office buildings and has been approved by both *Underwriters Laboratories* and *Factory Mutual*. Annex B of *NFPA 72* recommends that the smoke detectors be placed every 30 feet in a gird formation. Modifications were made to the grid system to ensure that all accessible areas with combustible materials, as well as the elevator shaft and stair shafts, are covered by detectors. Furthermore, smoke detectors are not to be placed within four inches of any wall. Below are graphical representations of the recommended locations for smoke detectors for the first floor and the locations typical to the second, third and fourth floors.

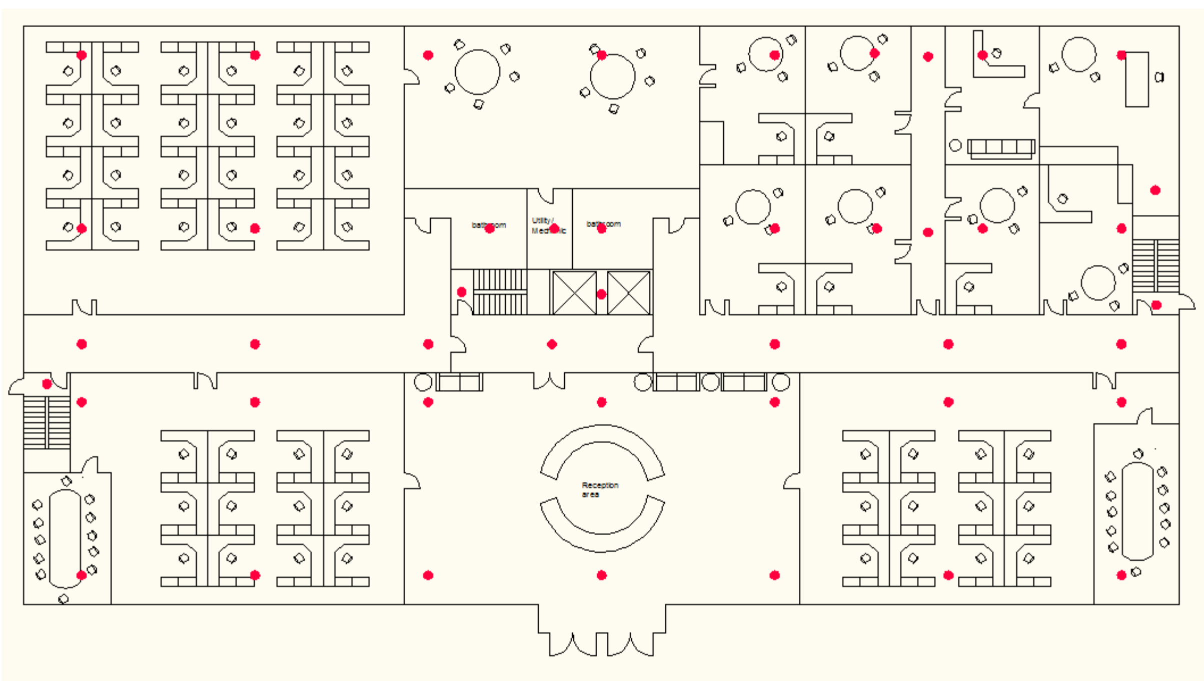


Figure 86 Smoke Detector Layout for 1st Floor

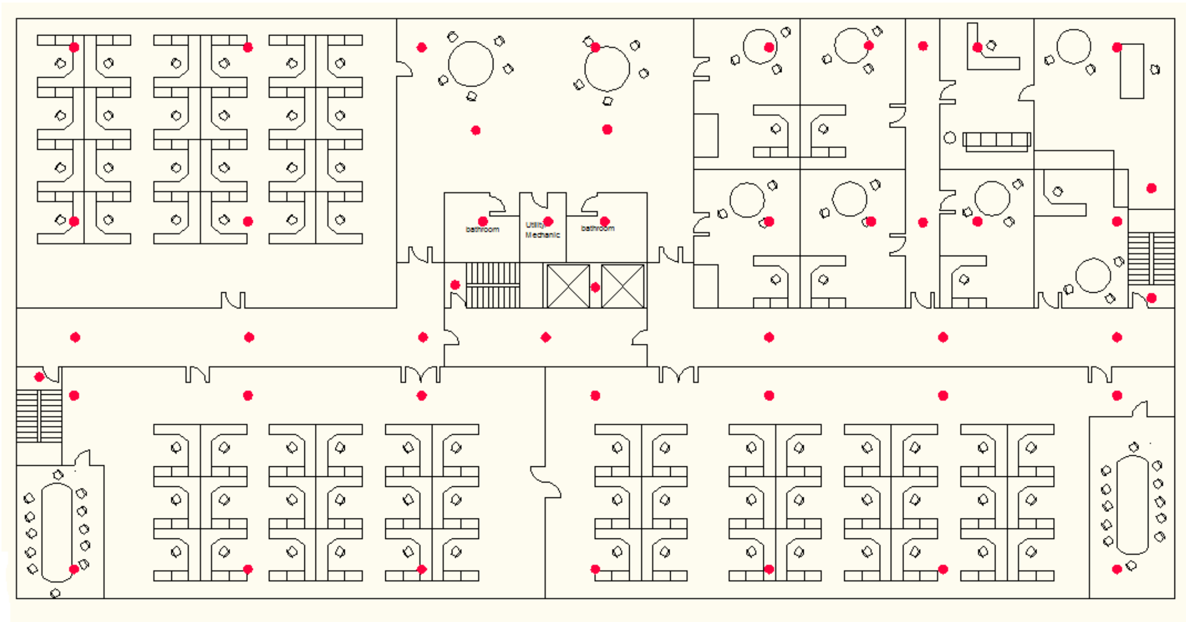


Figure 87 Smoke Detector Layout for 2nd to 4th Floor

By recognizing the requirements of the *IBC 2009* and the architectural boundaries of the designed office layout, we have provided the client with a smoke detector layout plan that complies with the code and provides the office occupants with a quick notification in a fire emergency.

12.5 Structural Member Fire Protection Recommendations

By performing heat transfer and strength analyses upon the structural members used in our design, we were able to determine points of failure of structural members for a typical ASTM Standard E-119 fire. In these analyses we accounted for ceiling systems with and without membrane protection. We wanted to investigate approximately how much more time a membrane type ceiling offered structural member stability during a typical fire. Our calculated results showed that the critical member within an unprotected ceiling failed due to elevated temperatures from fire within approximately 3.8 minutes. The critical member of the membrane-protected ceiling failed in approximately 41.1 minutes. The amount of time that the ceiling membrane offers is crucial to allow occupants sufficient time to egress from the building in the event of a fire. The ceiling membrane will also help prevent costly reconstruction due to thermal deformation, failure or collapse of structural elements.

The recommended ceiling membrane consists of ½" gypsum board to be installed directly beneath the girders and open web joists. The ½" gypsum board will provide sufficient fire protection to the structural members if installed according to design. The gypsum board should cover the entire ceiling leaving zero gaps for hot gas from fires to pour through. Also, in order to provide space for convection, it is recommended that the ceiling be installed exactly 1” inch below the bottom flange of the deepest member. The deepest members in the designed structural bays are the typical 16K7 open web joists. The tracking system for the gypsum board ceiling can be attached to the bottom chords of the open web joists with 18 gage tie wire or 11 gage clips so that the ceiling sits one inch below the bottom chords of the 16K7 typical joists.

By investigating the approximate times of failure for structural elements in fire conditions using our heat transfer and strength analyses, we have provided the client with real data that would allow him to make decisions regarding the desired level of fire protection. An area of study that is recommended by the team but has not been studied is the heat transfer and failure analysis of the reinforced concrete slab. By investigating the failure times of the slab in a fire condition with various levels of fire protection, a recommendation can be made regarding the proper level of slab protection needed to minimize human injury and building rehabilitation costs.

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Appendices

Appendix A: Proposal

Steel Structural and Fire Safety Design for a Four-Story Office Building

A Proposal for a Major Qualifying Project Report

Submittal to the Faculty of

WORCESTER POLYTECHNIC INSTIUTE

In partial fulfillment of the requirements for the

Degree of Bachelor of Science

By

Nicholas Kozlowski

Jeffrey Root

Yanxuan Xie

Date: October 15, 2009

Approved:

Professor Leonard D. Albano, Faculty Advisor

Abstract

The purpose of this Major Qualifying Project is to develop a design of a four-story steel building. Using different types of steel framing and construction, we will recommend a specific and detailed plan of construction that is both economical and structurally sound. We shall also investigate different building fire safety applications in order to design and recommend a fire protection system fit for the purpose and construction of our building. Through this project, we hope to not only advance our knowledge in the fields of structural steel design and fire protection, but also to apply this knowledge in a professional manner.

Introduction

In this project, each member will act as an engineering consultant for an imaginary architectural firm, regarding the design of a four-story office building located in Worcester, Massachusetts. The major tasks of the team will include alternative designs of steel structural frames, the design of fire protection systems and determination the cost-value for each design. Recommendations will be based on construction cost, performance, and construction comparisons of alternative designs.

The building designs will be in compliance with the *International Building Code 2009*, while the design of sprinkler systems will satisfy NFPA 13, *Standard for the Installation of Sprinkler Systems.* Both prescriptive and performance-based designs and analysis will be conducted by the team to investigate the overall cost and performance of structural systems and fire protection systems. The performance-based designs and analysis will be based on various documents from National Fire Protection (NFPA) which includes but not limited to NFPA 72, *National Fire Alarm and Signaling Code, 2010 Edition,* NFPA 101, *Life Safety Code, 2009 Edition*, NFPA 5000, *Building Construction and Safety Code, 2009 Edition.*

The design of the floor plan from the architecture firm will be a base for steel structural frame design. The team will investigate alternative structural bay scenarios with various combinations of construction methods and different steel structure members to compare the cost, the ease of construction and the performance. The investigations of structural members will include W-shape beams and Open Web Joists; the construction will includes non-composite and composite construction; and the bay size will varies from 20 ft X20 ft to 25 ft X 25 ft with beam spacing varies from 4 ft to 10 ft.

The design of fire protection systems of this four-story office building will be investigated through two scenarios: one with automatic sprinkler systems and one without. In order to conform to the fire safety requirements of Authority Having Jurisdiction, the team will conduct a code review to examine the preliminary design from the architectural firm. Based on the code review, modifications to the existing design will be proposed for both categories. Fire protection systems with and without automatic sprinkler systems will be designed to compare differences based on cost and performance. Recommendations to the architectural firm will be concluded by the team based on the comparison. Furthermore, revisions of the steel structural design will be provided in compliance with the recommended fire protection systems. Performance-based analyses will be conducted in terms of the performance of structural members under fire conditions and performance of the fire protection system under designed fire scenarios.

Scope of Work

This project will cover extensive topics in structural design and fire protection systems. The team will act as consultants to the architectural firm to present the preliminary floor plan based on the limited knowledge of codes restrictions. The preliminary floor plan will be treated as a base for all structural system and fire protection system designs. The team will primarily focus on steel structural system design. Variations of steel structure systems, however, will also be investigated. The cost will be the major concern of structural design while other concerns related to architectural layout, such as net usable area, will also be investigated. Furthermore, the team will design alternative fire protection systems for this building in compliance with related building codes and requirements from authorities. The performance-based design will also be conducted for fire protection systems to identify the level of flexibilities and cost efficiency it may offer. Based on all the cost and performance comparisons of structure design and fire protection design, recommendations will be made to the architecture firm.

Capstone Design

In this Major Qualifying Project, the three of us are acting as an engineering design team responsible for the design of a four story office building with fire protection systems. As engineers, we must design a structurally adequate building with respect to costs and health and safety. Using the *AISC Steel Manual*, the *ACI Manual* and the *International Building Code*, we will develop several different construction schemes. These construction schemes will be evaluated by cost using the *RS Means Manual*. Once a structural scheme is selected, the health and safety aspects of the building will be analyzed and implemented using the *IBC 2009* and NFPA documents.

Economy

- Cost-efficiency is one of the most important aspects of any design proposal.
- Using the RS Means Manual, we will evaluate our design scenarios for our building in order to rank them form least expensive to most expensive.

Constructability

- Our team will develop several design scenarios that include composite and non composite construction, various bay sizes, various beam spacing and Open Web Joist construction.
- We shall also investigate different lateral loading systems for our structure. Both rigid and braced frames will be investigated in order to determine which is best for our structure.

Health and Safety

- Using the *International Building Code* and NFPA publications, our team will design the necessary health and safety requirements for our building. The necessary requirements are outlined in the publications previously mentioned.
- The concepts and principles of egress, smoke control, fire partitions, fire department access, sprinkler systems and structural performance in fire will all be investigated and then implemented in order to design a structurally sound building that is safe from fire and other disasters.

Ethics

- We will design a quality building with a quality fire protection system all while demonstrating good building practice.
- Our design will comply with the most up to date codes and provisions of the International Building Code and NFPA publications.
- We are not concerned only with cost, but also with the overall performance of our building.

Methodology

In order to fulfill the scope of our project, there are several questions that we must investigate. Those questions and their corresponding methods of completion are listed below.

Structural Aspects:

What are the benefits of a particular length of span in terms of net usable area; Cost; Construability?

How does the spacing of beams affect the cost? And what is the most cost efficient spacing?

What advantages can be gained from composite beam construction in terms of the usability? What types of disadvantages does this type of construction introduce?

Can composite columns increase the net useable floor area? Will composite columns increase or decrease the cost of the structure? What is the usability of this type of columns?

What are the differences between Open web joist and W shape beams in terms of structural stability, the construction cost and duration of construction?

Methods: Our team will compare multiple structural bay scenarios utilizing composite and non composite construction techniques. Floor systems consisting of W-Shape beams and open web joist systems will also be compared for cost and constructability. Constructability will be assessed mostly by the length of the construction period and also by the difficulty of installation. Based on these comparisons, a recommendation will be given to the architect for use in this office building.

What should be done to resist the lateral force?

Methods: Rigid and braced frames will be investigated for various building heights and lateral loading. By utilizing a literary review and a computer analysis program, lateral loads and size of bracing members can be determined. A recommendation will be made based on cost and constructability.

Fire Protection Aspect:

How safe is the floor plan from Fire protection point of view? And if necessary, what changes need to be made?

> *How is the egress design, what improvement is needed? Where are the fire partition barriers? How is the accessibility for the fire department? Where are emergency signs? How can we*

control the fire movement by construction? How to control the smoke or what is the smoke management?

In terms of fire suppression system, which is best for this building: non-sprinklered or sprinklered?

What are the restrictions if building is non-sprinklered?

What benefit, in terms of building or structure, will sprinkler system offer?

Methods: Our team will use the *International Building Code* and *NFPA 13* to conduct a code review in order to design the necessary provisions. Our original design will be reviewed for code compliance and modifications to our design will be made where necessary.

Which sprinklers system suit for this building?

What should be done to protect structural members? And what is the cost?

Methods: A literary review of NFPA publications for various types of sprinkler systems and fire proofing materials will be made to address the merits of each one. A recommendation will be made based on the performance, code requirements, applications and costs.

How will the designed sprinkler system work under certain fire conditions?

What will different structural members of our design perform in certain fire conditions?

For a proposed fire protection plan, what changes need to be made for structural members?

Methods: Based on theories of fire modeling and zone modeling, potential fire scenarios will be analyzed. Our team will utilize FDS or another computer program to predict the performance of our designed fire protection system. The performance of the structural members will be investigated through analytical methods for determining fire resistance of steel members. Modifications to the fire protection system and structural members will be made where necessary.

Deliverables

Architectural

- Floor Plan

Structural

- Design methods and flow charts

    Composite W-shape beams and girders

    Non Composite W-shape beams and girder

    Composite columns

    Non composite columns

    Steel open web joists

    Concrete slab

    Metal Decking

Table 27 Construction Scenario Summary

Scenario ID #	OWJ/W Section	Comp/NonComp Columns	Comp/NonComp Beams and Girders	Bay Size (ft)	Beam Spacing (ft)
1A	W Section	Comp	Comp	20x20	4
2A	W Section	Comp	Comp	20x20	5
3A	W Section	Comp	Comp	20x20	10
4A	W Section	Comp	Comp	25x25	5
1B	W Section	Comp	NonComp	20x20	4
2B	W Section	Comp	NonComp	20x20	5
3B	W Section	Comp	NonComp	20x20	10
4B	W Section	Comp	NonComp	25x25	5
1C	W Section	NonComp	Comp	20x20	4
2C	W Section	NonComp	Comp	20x20	5
3C	W Section	NonComp	Comp	20x20	10
4C	W Section	NonComp	Comp	25x25	5
1D	W Section	NonComp	NonComp	20x20	4
2D	W Section	NonComp	NonComp	20x20	5

3D	W Section	NonComp	NonComp	20x20	10
4D	W Section	NonComp	NonComp	25x25	5
1E	OWJ	Comp	N/A	20x20	2
1F	OWJ	NonComp	N/A	20x20	2
4E	OWJ	Comp	N/A	25x25	2.5
4F	OWJ	NonComp	N/A	25x25	2.5

- Construction Summaries
    - Composite vs. non composite
    - W-shapes vs. open web joist
    - Cost Analysis
- Appendices with design calculations
    - Composite beams and girders
    - Non composite beams and girders
    - Composite columns
    - Non composite columns
    - Open web joists
    - Concrete slab
    - Cost

Lateral Frame

- Summary of techniques
- Computer analysis for selection of framing system
- Cost and Constructability evaluation

Code Analysis

- Spinklered building restrictions and requirements
- Unsprinklered building restrictions and requirements
- Egress compliance
- Fire rated construction compliance

Fire Protection

- Design of sprinkler system

  Flow Chart

  Summary

  Piping layout, sizes and materials

  Sprinkler types and product facts sheet from manufactures

  Sprinkler locations

  Valves, components and trim specifications and their product facts sheet from manufactures

- Smoke management system(HVAC)

  Type of smoke management systems to be designed for the building.

- Detection system

  Types of detection systems

  Location of detection systems throughout building

- Cost

Performance Based Analyses

- Fire modeling

  Sprinkler performance

  Activation Time

  Effects on designed fire

  Structural performance

  Strength change in structural members

  Structural deformation

- Egress Efficiency

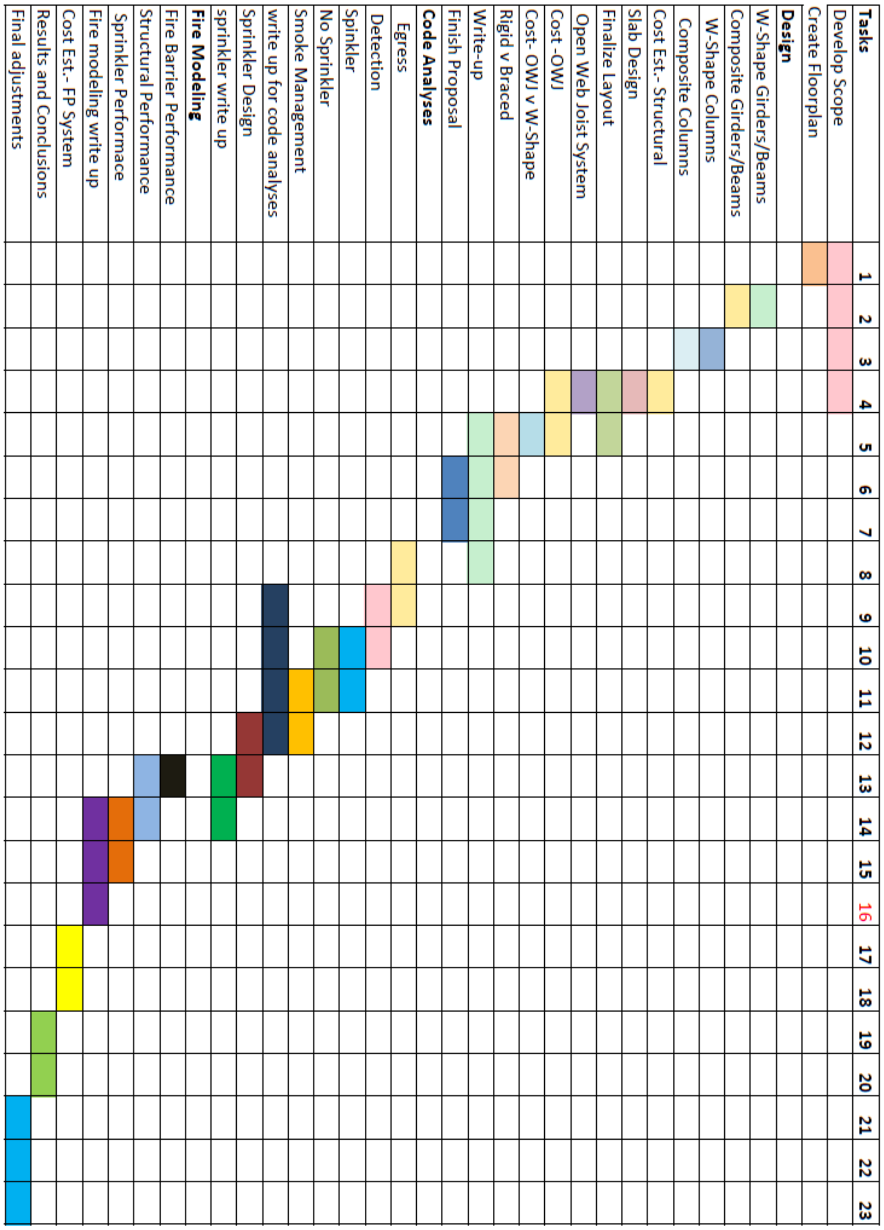
  Exit routes and corresponding travel time

Conclusions

This project is designed to simulate the projects in real word applications. By doing this project, each team member team will practice how to collaborate with each other, how to working as a team, how to conduct an efficient communication and how to plan and fulfill an engineering design task. The experience gained will certainly be valuable in future work. Furthermore, the project is also designed to be an educational experience for each of us. The team will practice skills learned from course study, as well as study and apply new topics which were not covered in our curriculum. The research areas and practice in this project will provide each team member a deeper understanding of structural engineering and fire protection engineering.

Schedule

Table 28 Planned Schedule



Appendix B: Steel Structural Design Summaries

B.1 Non-composite Beams and Girders Design

The designs followed the procedures provided in *Structural Steel Design, 4th Edition.* All values of design loads were based on the *IBC 2009* and *Minimum Design Loads for Building and Other Structure*, while other design values of selected sections were referenced from *AICS Steel Construction Manual*.

<u>Basic Information</u>

- Steel Properties: yield strength (F_y) is 50 ksi; modulus of elasticity (E) is 29,000ksi
- It is assumed that it is an unshored construction (no decking, the wet concrete will be supported by steel beams only)
- Layouts: 20'X20' Bay: Beam spacing: 4' O.C.; 5' O.C.; 10' O.C.
  25'X25' Bay: Beam spacing: 5' O.C.
- All designs followed LRFD method

<u>Step by Step Design Process</u>

1. Determine the Dead Load and Live Load or other loads may apply.

   Dead Loads: 63 lb/ft^2 for the slab; 4 lb/ft^2 for the mechanics devices; 2 lb/ft^2 for the ceiling construction; 6 lb/ft^2 for the floor finishing.8 lb/ft^2 for the exterior wall.

   Live Loads: 100 lb/ft^2 for office building

   Other Loads: for roof design, the team assumed the governing load is snow load (SL) which is 40 lb/ft^2

   The service load combinations were followed as below:

   Wu=1.2 DL + 1.6 LL; $Wu,_{roof}$=1.2 DL + 1,6 SL

2. Select a trail section from Design Chart or Table 3-11 of *AISC Steel Construction Manual*.
3. Check the moment capacity of selected section
   a) Determine which zone the beam falls in by using AISC Equation F2-5 and A1-7
   b) Calculate the Capacity by using AICS Equation F1-1 and F2-2 to F2-4

The design criteria is : $\phi_b M_n = C_b[\phi_b M_{px} - BF(L_b - L_p)] < \phi_b M_{px}$

4. Check the shear capacity at the ends

   Using AISC Equation G2-1 to G2-5 to determine ϕV_n while the design criteria is $\phi V_n > V_u$ (Where Vu is the actual shear force)

5. Check the deflection:

   The limits are: LL: Span/360; DL+LL: Span/240

   The actual deflection: $\Delta = 5wL^4/384EI$

6. The selected section failed at any previous procedure, should select another larger member size.

Hand Calculations are in Appendix C and Design Spread Sheets are in Appendix D.

B.2 Composite Beams and Girders Design

The designs followed the procedures provided in *Structural Steel Design, 4th Edition.* All values of design loads were based on the *IBC* and *Minimum Design Loads for Building and Other Structure*, while other design values of selected sections were referenced from *AICS Steel Construction Manual.*

Basic Information

- Steel Properties: yield strength (F_y) is 50 ksi; modulus of elasticity (E) is 29,000ksi
- It is assumed that it is an shored construction (steel decking, the wet concrete will be supported)
- Layouts: 20'X20' Bay: Beam spacing: 4' O.C.; 5' O.C.; 10' O.C.

  25'X25' Bay: Beam spacing: 5' O.C.
- All designs followed LRFD method

Step by Step Design Process

7. Determine the Dead Load and Live Load or other loads may apply.

   Dead Loads: 63 lb/ft^2 for the slab; 4 lb/ft^2 for the mechanics devices; 2 lb/ft^2 for the ceiling construction; 6 lb/ft^2 for the floor finishing.8 lb/ft^2 for the exterior wall.

   Live Loads: 100 lb/ft^2 for office building

Other Loads: for roof design, the team assumed the governing load is snow load (SL) which is 40 lb/ft^2

The service load combinations were followed as below:

Wu=1.2 DL + 1.6 LL; $Wu,_{roof}$=1.2 DL + 1,6 SL

8. Select a trail section from Design Chart or Table 3-19 of *AISC Steel Construction Manual.*
9. Check the moment capacity of selected section
   c) Determine effective width of concrete from AISC spec. section I3.1a
   d) Determine distance Y2 from AISC Figure 3-3
   e) Calculate the Capacity by using the lower and upper bound values for the member from Table 3-19 *AISC Steel Construction Manual.*
10. Check the deflection:
    Use lower bound moment of inertia from AISC Table 3-20
    The limits are: LL: Span/360; DL+LL: Span/240
    The actual deflection: $\Delta=5wL^4/384EI$
11. Design shear studs
    Stud strength of 21kips
    Determine required number of studs from horizontal shear force (∑Qn) from ACI Table 3-19 for the selected member.
12. The selected section failed at any previous procedure, should select another larger member size.

Hand Calculations are in Appendix C and Design Spread Sheets are in Appendix D.

B.3 Non-composite Column Design

The designs followed the procedures provided in *Structural Steel Design, 4th Edition.* All values of design loads were based on the *IBC* and *Minimum Design Loads for Building and Other Structure*, while other design values of selected section were referenced from *AICS Steel Construction Manual.*

Basic Information

- Steel Properties: yield strength (F_y) is 50 ksi; modulus of elasticity (E) is 29,000ksi

- Layouts: 20’X20’ Bay; 25’X25’ Bay
- Story height: 1st floor: 12 ft; 2nd to 4th floor: 10ft.
- All design followed LRFD methods
- End connections were assumed to be pinned connections to simplify the calculations.

Step by Step Design Process

1. Determine the Dead Load and Live Load and other loads that may apply.

   Dead Loads: 63 lb/ft^2 for the slab; 4 lb/ft^2 for the mechanics devices; 2 lb/ft^2 for the ceiling construction; 6 lb/ft^2 for the floor finishing.8 lb/ft^2 for the exterior wall.

   Live Loads: 100 lb/ft^2 for office building

   Other Loads: for roof design, the team assumed the governing load is snow load (SL) which is 40 lb/ft^2; for every floor, steel structural members were considered based on each layout scenarios.

   The service load combinations were followed as below:

   P_u=1.2 DL + 1.6 LL; $P_{u,roof}$=1.2 DL + 1,6 SL

2. Calculate the required cross-section area while assume the value of KL/r = 50.
3. Select a trial section with sufficient area.
4. Check the capacity from Table 4-22 of *AISC Steel Construction Manual.*
5. If failed at step 4, try a larger section.

Hand Calculations are in Appendix C and Design Spread Sheets are in Appendix D.

B.4 Composite Column (Concrete Enclosed) Design

The designs followed the procedures provided in *Structural Steel Design, 4th Edition.* All values of design loads were based on the *IBC* and *Minimum Design Loads for Building and Other Structure*, while other design values of selected section were referenced from *AICS Steel Construction Manual.* The design procedures and cross-section design were based on the information provided in *Structural Steel Deign.*

Basic Information

- Steel Properties: yield strength (F_y) is 50 ksi; modulus of elasticity (E) is 29,000ksi
- Layouts: 20'X20' Bay; 25'X25' Bay: Beam spacing
- Story height: 1st floor: 12 ft; 2nd to 4th floor: 10ft.
- All design followed LRFD methods
- Ends connections were assumed to be pinned connections to simplify the calculations.

Step by Step Design Process

1. Determine the Dead Load and Live Load or other loads may apply.

   Dead Loads: 63 lb/ft^2 for the slab; 4 lb/ft^2 for the mechanics devices; 2 lb/ft^2 for the ceiling construction; 6 lb/ft^2 for the floor finishing.8 lb/ft^2 for the exterior wall.

   Live Loads: 100 lb/ft^2 for office building

   Other Loads: for roof design, the team assumed the governing load is snow load (SL) which is 40 lb/ft^2; for every floor, steel structural members were considered based on each layout scenarios.

   The service load combinations were followed as below:

   P_u=1.2 DL + 1.6 LL; $P_{u,roof}$=1.2 DL + 1,6 SL

2. Design a cross-section for composite column while the area of W-shape steel section is less than 10% of total cross-section area.
3. Use AISC Equation I2-1 to I2-7 to determine the capacity of this composite column.
4. If the factored capacity is less than the service load, redesign the cross-section (either select a larger W-shape beam or increase the concrete cross-section area.

B.5 Open Web Joist (OWJ) Design

The designs followed the procedures provided in *Structural Steel Design, 4th Edition.* All values of design loads were based on the *IBC* and *Minimum Design Loads for Building and Other Structure*, while other design values and procedures were based on *Standard Specifications for Open Web Steel Joists, K-Series* from *Steel Joist Institute (SJI).*

Basic Information

- Steel Properties: yield strength (F_y) is 50 ksi; modulus of elasticity (E) is 29,000ksi
- It is assumed that it is unshored construction (no decking, the wet concrete will be support by steel beams only)
- Layouts: 20'X20' Bay: Joists spacing: 2' O.C.
  25'X25' Bay: Beam spacing: 2.5' O.C
- All design followed LRFD methods

Step by Step Design Process

1. Determine the Dead Load and Live Load and other loads that may apply.

   Dead Loads: 63 lb/ft^2 for the slab; 4 lb/ft^2 for the mechanics devices; 2 lb/ft^2 for the ceiling construction; 6 lb/ft^2 for the floor finishing.8 lb/ft^2 for the exterior wall.

   Live Loads: 100 lb/ft^2 for office building

   Other Loads: for roof design, the team assumed the governing load is snow load (SL) which is 40 lb/ft^2

   The service load combinations were followed as below:

   Wu=1.2 DL + 1.6 LL; $Wu,_{roof}$=1.2 DL + 1,6 SL

Determine the size of K-series open web joist based on Standard LFRD Load Table in *Standard Specifications for Open Web Steel Joists, K-Series.*

B.6 Reinforced Concrete Slab Design

The designs followed the procedures provided in *Reinforced Concrete Design, 7th Edition.* All values of design loads were based on the *IBC* and *Minimum Design Loads for Building and Other Structure*, while other design values of selected sections were referenced from *ACI Reinforced Concrete Design Manual.*

Basic Information

- Concrete Properties: Compressive strength (f'c) is 4000 psi, yield strength of reinforcing steel (fy) is 60,000 psi, unit weight of concrete is 150 pcf, effective width (b) is 12"

- Layouts: Layouts: 20'X20' Bay: Beam spacing: 2'O.C.; 4' O.C.; 5' O.C.; 10' O.C.

  25'X25' Bay: Beam spacing: 2.5' O.C ; 5' O.C.
- All design followed LRFD methods

Step by Step Design Process

6. Determine minimum slab thickness (h) from ACI Table 9.5a
7. Dead Loads: thickness(h) times the unit weight of concrete; 4 lb/ft^2 for the mechanics devices; 2 lb/ft^2 for the ceiling construction; 6 lb/ft^2 for the floor finishing.8 lb/ft^2 for the exterior wall.

   Live Loads: 100 lb/ft^2 for office building

   Other Loads: for roof design, the team assumed the governing load is snow load (SL) which is 40 lb/ft^2; for every floor, steel structural members were considered based on each layout scenarios.

   The service load combinations were followed as below: P_u=1.2 DL + 1.6 LL; $P_{u,roof}$=1.2 DL + 1,6 SL Calculate uniform load for one foot section of slab
8. Use moment coefficients from ACI Table 8.4.1 to determine maximum moment
9. Assume #4 bars and ¾" cover to estimate depth of reinforcement (d)
10. Check reinforcement ratio (ρ)
    a.) Calculate reinforcement ratio (ρ) from ACI equation 3.8.5
    b.) Check limits

       Minimum ρ=200/fy

       Maximum ρ from ACI Table 3.6.1
11. Design reinforcing steel
    a.) Determine minimum steel are required (As) from ACI equation 3.5.1 As= ρbd
    b.) Pick bar from ACI Table 1.12.1 to meet area requirement
12. Check flexure strength using Whitney stress block accepted by ACI 10.2.7
13. Check shear strength using ACI equation 5.11.3
14. Determine minimum spacing requirements for temperature and shrinkage steel
    a.) ACI 7.12.2.3 governs minimum reinforcement ratio
    b.) Calculate minimum steel area using ACI equation 3.5.1

Hand Calculations are in Appendix C and Design Spread Sheets are in Appendix D.

Appendix C: Steel Structural Design Hand Calculation Samples

C.1 Non-Composite Beams and Girders Design

Thus larger member is selected

Try W8X31

Moment: Include Beam weight

$w_u = 1.25\ k/ft + 31\ lb/ft \cdot 1.6 = 1.3\ k/ft$

$\therefore M_u = 65\ k\text{-}ft$, $\phi M_u = 58.5\ k\text{-}ft$

From Table 3-2 : $L_p = 7.18\ ft$, $L_r = 24.8\ ft$

$L_p < L_b < L_r \Rightarrow$ Zone 2 (inelastic behavior)

$M_p = F_y \cdot Z = 50ksi \cdot 30.4\ in^3 = 126.67\ k\text{-}ft$

$\phi M_n = C_b[\phi_b M_{px} - BF(L_b - L_p)]$

$= 1.0[0.9 \cdot 126.67 - 2.37(24.8 - 7.18)] = 72.24\ k\text{-}ft$

$\leq \phi_b M_{px} = 114\ kip\text{-}ft$ $\boxed{OK}$

Shear: From AISC Table 1-1 W8X31 ($A = 9.12$, $d = 8$, $t_w = 0.285$, $k_{des} = 0.829$)

$\frac{h}{t_w} = \frac{8 - 0.829 \times 2}{0.285} = 22.25 \leq 1.10\sqrt{\frac{k_v E}{F_y}} = 1.10 \cdot \sqrt{\frac{5.29 \cdot 29000}{50}} = 24.12$

$\therefore C_v = 1$

$\because w_u = 1.29\ k/ft \Rightarrow V_u = 10ft \cdot 1.29\ k/ft = 12.9\ k/ft$

AISC Table 3-2 : $\phi_v V_n = 68.4\ kip > 12.9\ k/ft$ $\boxed{OK}$

Deflection:

$\Delta_{DL} = \frac{65 \cdot 20^2}{161 \cdot 110} = 1.47\ in > \text{limit } 1\ in$ $\boxed{NG}$

Try W12X30

Moment: $W_u = 1.3\ k/ft$ $\phi M_u = 58.5\ k\text{-}ft$

$L_p = 5.37\ ft$ $L_r = 15.6\ ft < L_b \Rightarrow$ Zone 3 (elastic buckling)

$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_{ts}}\right)^2 C} \cdot \sqrt{1 + 0.078 \frac{Jc}{S_x h_o}\left(\frac{L_b}{r_{ts}}\right)^2}$ ($c = 1$ for I beam, $C_b = 1$)

From Table 1-1 $r_{ts} = 1.77\ in$ $J = 0.457\ in^4$, $h_o = 11.9$ $S_x = 38.6\ in^3$

$\therefore F_{cr} = 24.3\ ksi$

$M_{nx} = F_{cr} S_x = 936.1\ kip\text{-}in = 78\ k\text{-}ft < M_p = \frac{50 \cdot 43.1}{12} = 179.6$

$\boxed{Ok}$

$\therefore \phi_b M_{nx} = 0.9 \cdot 78\ k\text{-}ft = 70.2\ k\text{-}ft < \phi_b M_{px} = 162\ k\text{-}ft$

$\boxed{OK}$

Thus larger Member is selected

Try W8x31

Moment: Include Beam weight

$w_u = 1.25\,k/ft + 31\,lb/ft \cdot 1.6 = 1.3\,k/ft$

$\therefore M_u = 65\,k\text{-}ft \quad \phi M_u = 58.5\,k\text{-}ft$

From Table 3-2: $L_p = 7.18\,ft$, $L_r = 24.8\,ft$

$L_p < L_b < L_r \Rightarrow$ Zone 2 (inelastic behavior)

$M_p = F_y \cdot Z = 50\,ksi \cdot 30.4\,in^3 = 126.67\,k\text{-}ft$

$\phi M_n = C_b[\phi_b M_{px} - BF(L_b - L_p)]$

$= 1.0[0.9 \cdot 126.67 - 2.37(24.8 - 7.18)] = 72.24\,k\text{-}ft$

$\leq \phi_b M_{px} = 114\,kip\text{-}ft$ $\boxed{OK}$

Shear: From AISC Table 1-1 W8x31 ($A = 9.12$, $d = 8$, $t_w = 0.285$, $k_{des} = 0.829$)

$\frac{h}{t_w} = \frac{8 - 0.829 \times 2}{0.285} = 22.25 \leq 1.10\sqrt{\frac{k_v E}{F_y}} = 1.10 \cdot \sqrt{\frac{0.529 \cdot 29000}{50}} = 24.17$

$\therefore C_v = 1$

$\because w_u = 1.29\,k/ft \Rightarrow V_u = 10\,ft \cdot 1.29\,k/ft = 12.9\,k/ft$

AISC Table 3-2: $\phi_v V_n = 68.4\,kip > 12.9\,k/ft$ $\boxed{OK}$

Deflection:

$\Delta_{DL} = \frac{65 \cdot 20^2}{161 \cdot 110} = 1.47\,in >$ limit $1\,in$ $\boxed{NG}$

Try W12x30

Moment: $w_u = 1.3\,k/ft \quad \phi M_u = 58.5\,k\text{-}ft$

$L_p = 3.37\,ft \quad L_r = 15.6\,ft < L_b \Rightarrow$ Zone 3 (elastic buckling)

$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_{ts}}\right)^2 C} \cdot \sqrt{1 + 0.078 \frac{J_c}{S_x h_o}\left(\frac{L_b}{r_{ts}}\right)^2}$ $\left(\begin{matrix} C = 1 \text{ for I beam} \\ C_b = 1 \end{matrix}\right)$

From Table 1-1 $r_{ts} = 1.77\,in$ $J = 0.457\,in^4$, $h_o = 11.9$ $S_x = 38.6\,in^3$

$\therefore F_{cr} = 24.3\,ksi$

$M_{nx} = F_{cr} S_x = 936.1\,kip\text{-}in = 78\,k\text{-}ft < M_p = 50 \cdot 43.1 = 17$

Shear. AISC Tab 1-1 W12x30 $d = 12.3$, $t_w = 0.26$ $k_{des} = 0.74$ $A = 8.76$

$$\frac{h}{t_w} = \frac{12.3 - 2 \cdot 0.74}{0.26} = 41.62 > 1.37\sqrt{\frac{k_v E}{F_y}} = 28.4$$

$$\therefore C_v = \frac{1.51\, E k_v}{\left(\frac{h}{t_w}\right)^2 F_y} = 0.374$$

$$V_n = 0.6 \cdot 50 \cdot (12.3 \cdot 0.26) \cdot 0.374 = 35.8 \text{ kips}$$

$$\phi_v V_n = 1 \cdot V_n = 35.8 \text{ kips} \geq V_u = 13 \text{ kips} \quad \boxed{\text{ok}}$$

Deflection:

$$\Delta_{DL} = \frac{58.5 \cdot 20^2}{161 \cdot 238} = 0.61 \text{ in}$$

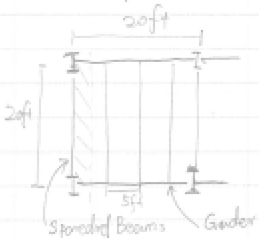
$$\text{limit } \Delta_{DL} = \frac{L}{240} = 1 \text{ in} \quad \boxed{\text{ok}}$$

$$\Delta_L = \frac{8 \cdot 20^2}{161 \cdot 238} = 0.052 \text{ in}$$

$$\text{limit } \Delta_L = \frac{L}{360} = 0.67 \text{ in} \quad \boxed{\text{ok}}$$

Use $\boxed{W12 \times 30}$ for interior beam.

For Spandrel Beam : Attribute width : $\frac{5ft}{2} = 2.5ft$



$DL = (63psf + 4psf + 6psf + 2psf) \cdot 2.5ft$

$+ \underbrace{12ft}_{\text{floor height}} \cdot \underbrace{8psf}_{\text{potential curtain wall}} = 283.5\ lb/ft$

$LL = 100\ psf \cdot 2.5ft = 250\ lb/ft$

$W_u = 1.2DL + 1.6LL = 740.2\ lb/ft = 0.74\ k/ft$

$M_u = \frac{0.74 \cdot 20^2}{8} = 37\ k\text{-}ft$

• From AISC Table 3-16 Try W8X28

moment : with beam wieght : $W_u = 0.74\ k/ft + 28\ lb/ft \cdot 1.6 = 0.785\ k/ft$

$M_u = \frac{0.785 \cdot 20^2}{8} = 39.25\ kip\text{-}ft$

Table 3-2 $L_p = 5.72$ $L_r = 21$. $L_p < L_b < L_r$

$M_p = F_y \cdot Z_x = 113.3\ kft$

$\phi M_n = C_b [\phi_b M_p - BF(L_b - L_p)]$

$= 66.27\ k\text{-}ft < \phi_b M_p = 102\ ft\text{-}k$ $\boxed{ok}$

Shear . $\frac{h}{t_w} = \frac{d - 2k_{des}}{t_w} = \frac{8.06 - 2 \cdot 0.859}{0.285} = 22.3 < 1.1\sqrt{\frac{K_v E}{F_y}} = 24.6$

$\therefore C_v = 1$

$W_u = 0.785\ k/ft$

$V_n = 0.6 \cdot 50 \cdot (0.285 \cdot 8.06) \cdot 1 = 68.9\ kip$

$\phi V_n = 1 \cdot V_n = 68.9\ kip > V_u = 0.785 \cdot 10 = 7.85\ K$ $\boxed{ok}$

Deflection

limit $\Delta_L = \frac{L}{360} = 0.667\ in$

$\Delta_L = \frac{M_L \cdot 20^2}{161 \cdot 98} = \frac{\frac{1.2 \cdot 0.25 \cdot 20^2}{8} \cdot 20^2}{161 \cdot 98} = 0.38\ in < 0.667\ in$

limit $\Delta_{DL} = \frac{L}{240} = 1\ in$

$\Delta_{DL} = \frac{39.25 \cdot 20^2}{161 \cdot 98} = 0.995 < 1\ in$ $\boxed{OK}$

Use $\boxed{W8 \times 28}$ for spandrel beam

C.2 Composite Beams and Girders Design

Beam Design

Bay size 20'x20' , Beam spacing = 5' O.C. w/ composite beams

Dead load

slab = 5" , $\frac{5}{12}\times 150pcf$ = 62.5 psf round to 63psf

metal decking = 2psf

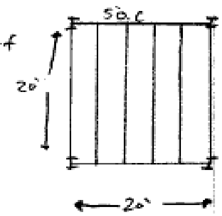
Floor covering = 4psf

MEP = 6 psf

int. wall = 8psf

ext. wall = 8psf

Total DL = 91psf



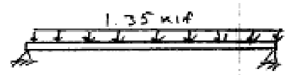
LL max office = 100psf

Load combination

$w_u = 1.2D + 1.6L = 1.2(91) + 1.6(100) = 269.2$ psf

Tributary Area = 5' $\quad w_u = 269.2(5) = 1346$ plf = 1.35 klf

L = 20 ft



$$M_u = \frac{(w_u)(L)^2}{8} = \frac{(1.35)(20)^2}{8} = 67.5 \text{ ft}\cdot\text{k}$$

$b_e = (2)\left(\frac{1}{8}\times 20 \times 12\right) = 60.in \qquad b_e = 60.in$

$b_e = (2)(2.5\times 12) = 60 in$

select W section

5in slab, 3" deep decking

$Y_{con} = 5+2 = 7in \qquad$ assume $a = 2"$

$Y_1 = 0in$

$Y_2 = Y_{con} - a/2 = 7 - 2/2 = 6"$

Try W 10x12 , $A = 3.54 in^2$, $I_x = 53.8 in^4$

check deflection during construction

Dead load - wet concrete = $\left(\frac{5}{12}\right)(150)(5)(1.1) = 344$ plf

beam = 12 plf

Live load - (20psf)(5ft) = 100plf

$w = 1.2(344+12) + 1.6(100) = 587.2 = .587$ klf

$$M = \frac{wL^2}{8} = \frac{(.587)(20)^2}{8} = 29.35 \text{ ft}\cdot\text{k}$$

$$\Delta_c = \frac{5 w_c L^4}{384 E I_x} = \frac{(5)(.587)(20)^4}{(384)(29000)(53.8)}(1728) = 1.35$$

Try W10×19 $A = 5.62\ in^2$, $I_x = 96.3\ in^4$ $\Sigma Q_n = 281^k$

beam weight = 19 plf   new $w_c = 1.2(19+344) + 1.6(100) = 595.6\ plf = .596\ klf$

$$\Delta_c = \frac{5 w_c L^4}{384 E I_x} = \frac{(5)(.596)(20)^4}{(384)(29{,}000)(96.3)}(1728) = 0.77 < \frac{L}{240} = 1.0 \checkmark$$

Check strength

$$a_{req} = \frac{\Sigma Q_n}{.85 f'_c b_e} = \frac{281}{(.85)(4.5)(60)} = 1.22$$

$$Y_2 = Y_{con} - \frac{a}{2} = 7 - \frac{1.22}{2} = 6.39$$

$\Phi_b M_n$ by interpolation $= 245 + \left(\frac{0.39}{0.50}\right)(255-245) = 252.8 > 67.5 \checkmark$

Check service ~~EI~~ deflection

$L_L = 100\ psf \times 5\ ft = 500\ plf$ service $= \frac{1}{2}LL = 250\ plf = .25\ klf$

$$\Delta_c = \frac{5 w_{sL} L^4}{384 E I} = \frac{(5)(.25)(20)^4}{(384)(29000)(96.3)}(1728) = 0.33$$

$$Limit = \frac{L}{360} = \frac{20\times 12}{360} = 0.67 > 0.33 \checkmark$$

Design studs

$Q_N = 21^k$

$$\#\ studs = \frac{2\Sigma Q_n}{Q_n} = \frac{2(281)}{21} = 27 \times 2 = 54 \text{ studs per beam}$$

Check shear

$$V_u = \frac{(20)(1.35)}{2} = 13.5^k \qquad \Phi V_n = 76.8 \text{ from T. 3-6} \checkmark$$

GIRDER DESIGN NPK 9.9.09

TRIBUTARY AREA = 20'

UNIFORM LOAD = $20' \times (1.2(91 psf) + 1.6(100 psf)) = 5384$ plf

5.384 klf

$M_U = \frac{(W_U)(L)^2}{8} = \frac{(5.384)(20)^2}{8} = 269.2$ ft·K

$b_e = 2(1/8 \times 20 \times 12) = 60$ IN ← governs

$b_e = 2(10 \times 12) = 240$ IN

SELECT W SECTION

5" SLAB, 3" deep decking, assume $a = 2$"

$M_U = 269.2$ ft·K

TRY W14×22, $A = 6.49$ IN², $I_x = 199$ IN⁴

CHECK DEFLECTION DURING CONSTRUCTION

DEAD LOAD - WET CONC. = $(\frac{5}{12})(150)(20)(1.1) = 1375$

beam = 22 plf

LIVE LOAD = (20 psf)(20 ft) = 400 plf

$W = 1.2(1375 + 22) + 1.6(400) =$
$= 1676.4 + 640$
$= 2316.4$
$= 2.3164$ klf

$M = \frac{(2.3164)(20)^2}{8} = 115.82$ ft·K

$\Delta_c = \frac{5 w_c L^4}{384 E I_x} = \frac{(5)(2.3164)(20)^4}{384 \cdot 29000 \cdot 199} \times 1728 = 1.44$

$\text{LIMIT} = \frac{L}{240} = \frac{20 \times 12}{240} = 1 < 1.44$

TRY 14x26 $I_x = 245$, $A = 7.69$

$$W_c = 1.2(1375 + 26) + 1.6(400)$$
$$= 2321.2$$
$$= 2.3212 \text{ klf}$$

$$M = \frac{(2.3212)(20)^2}{8} = 116.06 \text{ ft}\cdot\text{K}$$

$$\Delta_c = \frac{(5)(2.3212)(20)^4}{384 \cdot 29000 \cdot 245} \cdot 1728 = 1.18$$

$$1.18 > 1.00 \; \times$$

TRY $\boxed{14 \times 30}$ $I_x = 291$, $A = 8.85$

$$W_c = 1.2(1375 + 30) + 1.6(400)$$
$$= 1686 + 640$$
$$= 2326 = 2.326 \text{ KlF}$$

$$M = \frac{(2.326)(20)^2}{8} = 116.3$$

$$\Delta_c = \frac{5 W_c L^4}{384 E I_x} = \frac{(5)(2.326)(20^4)}{384 \cdot 29000 \cdot 291} \cdot 1728 = .99 \text{ IN}$$

$$\text{LIMIT} = \frac{L}{240} = \frac{20 \times 12}{240} = 1.00 > .99 \text{ IN} \; \checkmark$$

CHECK STRENGTH

W14x30 $\Sigma Q_N = 442$ $\quad a_{REQ} = \frac{\Sigma Q_N}{.85 f'_c b_e} = \frac{442}{.85(4.5)(60)}$

$$= 1.926$$

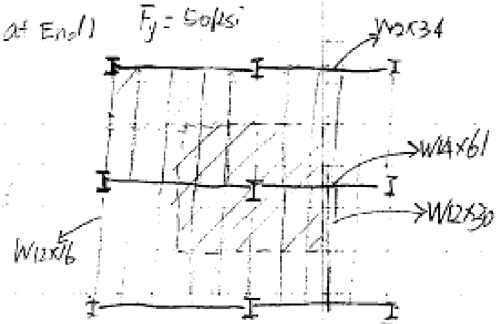
$$Y_2 = Y_{CON} - \frac{a}{2} = 7 - \frac{1.926}{2} = 6.04$$

$$\phi_b M_n = 429 + \left(\frac{0.04}{0.5}\right)(445 - 429) = 430.28 > M = 267.2 \; \checkmark$$

C.3 Non-composite Column Design

Sample Hand- Calculation

Column Design (Assume K=1, Simple connection at End) $F_y = 50ksi$



Interior Columns

Bay size 20x20

non-composite 4" C. slab

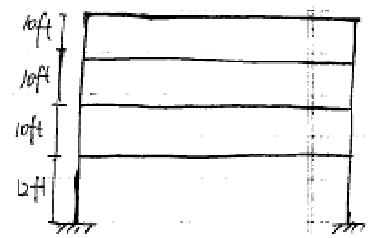
$P_u = 286.4 \cdot 20 \cdot 20 + 1.6\ (1 \times 61 + 5 \times 30)$

$= 114.9$ kips

Assume $\frac{kL}{r} = 50$

AISC Table 4-22 $\phi_c F_{cr} = 37.5$ ksi

$A_{Req.} = \frac{P_u}{\phi_c F_{cr}} = 3.064\ in^2$



Try W6x12 (A=3.55 r_y=0.918)

$\left(\frac{KL}{r}\right)_y = \frac{12 \cdot 10}{0.918} = 130.72$

$\Rightarrow$ Table 4-22 $\phi_c F_{cr} = 13.2$ ksi

$\phi_c P_n = 13.2\ ksi \cdot 3.55\ in^2 = 46.86$ kips < 114.9 kips N.G.

Try $\boxed{W6x20}$ (A=5.87 r_y = 1.5)

$\left(\frac{kL}{r}\right)_y = \frac{120}{1.5} = 80 \Rightarrow$ Table 4-22 $\phi_c F_{cr} = 28.2$ksi

$\phi_c P_n = 28.2ksi \cdot 5.87 in^2 = 165.534$ kips > 114.9 kips $\boxed{OK}$

Exterior Columns — (Girder aside)

$P_u = 286.4 \cdot 20 \cdot 10 + 1.6(34 + \frac{1}{2} \cdot 5 \times 30) + (8 \times 20 \times 10)\ 1.6 = 60.65$ kips

Try $\boxed{W6x16}$ (A=4.74 in^2 r_y = 0.967)

$\left(\frac{kL}{r}\right)_y = \frac{12 \cdot 10}{0.967} = 124.1 \Rightarrow \phi_c F_{cr} = 14.7$ ksi

$\phi_c P_n = 14.7 \cdot 4.74 = 69.68$ kips > 60.65 kips $\boxed{ok}$

Exterior Column — (beams aside)

$P_u = 286.4 \cdot 20 \cdot 10 + 1.6\ (0.5 \times 61 + 3 \times 30) + (8 \times 20 \times 10)\ 1.6 = 62.54$kips

From above Use $\boxed{W6x16}$

Corner Columns

$P_u = 286.4 \times 10 \times 10 + 1.6\ (0.5 \times 34 + 0.5 \cdot 3 \cdot 30) + 8 \times 20 \times 10 = 32.34$ kips

Try W6X12. , based on above $\boxed{W6X12}$ $\boxed{OK}$

non-composite : S. O.C.

Interior columns

$P_u = 286.4 \times 20 \times 20 + 1.6(61 + 4 \times 30) = 114.85$ kips

Based on information above Choose $\boxed{W6 \times 20}$

Exterior columns — Gir. Aside

$P_u = 286.4 \times 20 \times 10 + 8 \times 20 \times 10 \times 1.6 + 1.6(34 + \frac{4}{2} \times 30) = 60.8$ kips

Based on above Choose $\boxed{W6 \times 16}$

Exterior Columns — Beam. Aside

$P_u = 2864 \times 20 \times 10 + 8 \times 20 \times 10 \times 1.6 + 1.6(0.5 \times 61 + 2.5 \times 30) = 60.0$ kips

Also choose $\boxed{W6 \times 16}$

Corner Columns

$P_u = 286.4 \times 10 \times 10 + 8 \times 20 \times 10 \times 1.6 + 1.6(0.5 \times 34 + 1.25 \times 30) = 31.29$ kips

Based 4.O.C. choose $\boxed{W6 \times 12}$

Column design samples

2000 4' O.C.

Interior Column

$P_u = 286.4 \times 20 \times 20 + 1.6 \times 20 \times (57 + 5 \times 33) = 121.66 \text{ kips}$

Assume $KL/r_y = 50$

$\phi F_{cr} = 37.5 \text{ ksi}$

Try $\boxed{W6 \times 20}$ $(A = 5.87 \text{ in}^2 \quad r_y = 1.5 \text{ in}) \Rightarrow \phi_c F_{cr} = 13.2 \text{ ksi}$

$\phi_c P_n = 13.2 \text{ ksi} \cdot 5.87 \text{ in}^2 = 165.5 \text{ kips} > 121.66 \text{ kips} \quad \boxed{ok}$

Exterior Column Gir. aside

$P_u = 286.4 \times 20 \cdot 10 + 1.6 \times 20 (38 + 2.5 \cdot 33) + 8 \times 10 \times 20 \times 1.6 = 63.7 \text{ kips}$

Try $\boxed{W6 \times 15}$ $(A = 4.43 \quad r_y = 1.45)$

$\left(\frac{KL}{r_y}\right) = \frac{12 \times 10}{1.45} = 82.75$

$\phi_c P_n = 120.5 \text{ kips} > P_u = 63.7 \text{ kips} \quad \boxed{ok}$

Exterior Column beam aside

$P_u = 59.776 \text{ kips}$

Try $\boxed{W6 \times 15}$ $(A = 4.43 \quad r_y = 1.45)$

$\left(\frac{KL}{r_y}\right) = \frac{12 \times 10}{1.45} = 82.75 \Rightarrow \phi_c F_{cr} = 27.2 \text{ ksi}$

$\phi_c P_n = 4.43 \cdot 27.2 = 120.5 \text{ kips} > 59.776 \text{ kips} \quad \boxed{ok}$

Corner Column

$P_u = 30.38 \text{ kips}$

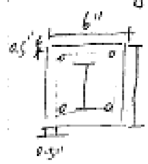
Try $\boxed{W6 \times 12}$ $(A = 3.55 \quad r_y = 0.918)$

$\left(\frac{KL}{r_y}\right) = 130.72 \Rightarrow \phi_c F_{cr} = 13.2 \text{ ksi}$

$\phi_c P_n = 13.2 \text{ ksi} \cdot 3.55 \text{ in}^2 = 46.86 \text{ kips} > P_u = 30.38 \text{ kips} \quad \boxed{ok}$

C.4 Composite Column Design (Concrete Enclosed)

Try a smaller size of cross-section



$A_c = 6\times6 - 3.83 - 4.1 = 28.17\ in^2$

$P_o = 3.83\cdot50 + 4.60 + 0.85\cdot28.17\cdot3.5$

$= 279.9 = 280\ kips$

$C_1 = 0.1 + 2\left(\frac{3.83}{28.17+3.83}\right) = 0.339 > 0.3$ N.G.

Try $7''\times7''$

$A_c = 7\times7 - 3.83 - 4 = 41.17\ in^2$

$P_o = 554\ kips$

$C_1 = 0.1 + 2\cdot\frac{3.83}{41.17+3.83} = 0.27 < 0.3$ ok

$I_c = \frac{1}{12}\cdot7\cdot7^3 - 3.86 = 196.22$

$EI_{eff} = 29000\cdot3.86 + 0.5\cdot29000\ (4\times1\times3^2)$
$+ 0.27\cdot3.410\times10^3\cdot196.22 = 0.8146\times10^6\ k\text{-}in^2$

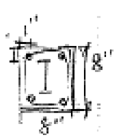
$P_e = \frac{\pi^2 EI_{eff}}{(KL)^2} = 558.318\ kips > 0.44P_o = 249.9\ kips$

$P_n = P_o\left[0.658^{\frac{P_o}{P_e}}\right] = 380.6\ kips$

$\phi P_n = 285.45\ kips > 121.64\ kips$ $\boxed{OK}$ [13 + 3.45 = 16.45 lb/ft]

Thus choose $7''\times7''$ cross-section with W4x13 ($\phi P_n = 285.45\ kips$)

Try a lighter W shape beam W6x8.5 For $8''\times8''$ ($A_s = 2.52$, $I_{sx} = 14.9$, $I_{sy} = 5.73$)



$A_c = 8\times8 - 2.52 - 4 = 57.48$

$P_o = 2.52\cdot50 + 4.60 + 0.85\cdot57.48\cdot3.5$

$= 537\ kips$

$C_1 = 0.1 + 2\left(\frac{2.52}{57.48+2.52}\right) = 0.184 < 0.3$ ok

$I_c = \left(\frac{1}{12}\right)8\cdot8^3 - 5.73 = 335.6\ in^4$

$EI_{eff} = (29000\cdot5.73) + 0.5\cdot29000\cdot4\cdot1\cdot3^2 + 0.184\cdot3410\times10^3\cdot335.6$

$= 0.898\times10^6\ kips$

$P_e = \frac{\pi^2 EI_{eff}}{(KL)^2} = 615.99\ k > 0.44P_o = 236.28\ k$

$P_n = P_o\left[0.658^{\frac{P_o}{P_e}}\right] = 372.83\ kips \Rightarrow \phi_c P_n = \boxed{279.62\ kips}$

[8.5 + 4.82 = 13.3 lb/ft]

C.5 Slab Design

Concrete slab design

$F'_c = 4000\,psi$, $B_1 = 0.85$, $F_y = 50{,}000\,psi$, 2ft section

DL - slab 5", 4psf floor covering, 8psf (int. wall)

slab $\left(\frac{5"}{12}\right)(150\,pcf) = 62.5\,psf + 4 + 8 = 74.5\,psf$

$LL = 100\,psf$

$w_D = (74.5)(1.2) = 89.4\,plf \rightarrow 90\,plf$
$w_L = (100)(1.6) = 160\,plf$

beam spacing = 10 ft → Clear Span = $10' - \frac{12}{12} = 9\,ft$

$M_u = \frac{1}{10}(90 + 160)(9)^2 = 2025\,\text{lb} = 2.03\,k\text{-}ft/ft\ \text{of width}$

ratio ρ choose $.35\rho_b$

$\rho_{max} = 0.675\rho_b = 0.675(0.0367) = 0.0248$

$d = h - \frac{1}{2}d_b - cvr - d_b(temp) = 5 - \frac{1}{2} - 1.5 - .5 = 2.5$

positive steel

$M_u \le \phi M_n$ $\phi = .9$ $M_n = \frac{M_u}{\phi} = \frac{2.03}{.9} = 2.25 \rightarrow 2.3\,ft\text{-}k$

$\frac{M_n}{bd^2} = R_n = \frac{(2.3)(12)}{(12)(2.5)^2} = 0.368\,ksi$

$R = \rho f_y\left(1 - \frac{1}{2}\rho m\right)$, $m = \frac{f_y}{.85f'_c} = \frac{50{,}000}{(.85)(4000)} = 14.71$

$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2mR_n}{f_y}}\right) = \frac{1}{14.71}\left(1 - \sqrt{1 - \frac{2(14.71)(368)}{50{,}000}}\right) = 0.0078$

Check ρ_{max}, ρ_{min} from T. 3.6.1 $\rho_{max} = 0.0248$

$\rho_{min} = \frac{200}{f_y} = \frac{200}{50{,}000} = 0.004$

$\underset{\rho_{min}}{0.004} < \underset{\rho}{0.0078} < \underset{\rho_{max}}{0.0248}$ ✓ use $\rho = 0.008$

pick steel - $A_s = \rho bd = (0.008)(12)(2.5) = 0.24\,in^2$

use 1-#5 bar $A_s = 0.31\,in^2$

check strength

$C = .85 f'_c\, ba = (.85)(4)(12)a = 40.8a$

$T = a_s f_y = (.31)(50) = 15.5 \quad C = T,\ a = \frac{15.5}{40.8} = 0.38$

$M_n = T\left(d - \frac{a}{2}\right) = 15.5\left(2.5 - \frac{0.38}{2}\right) = 35.885/12 = 2.98 > 2.03$ ✓

check strain

$x = \frac{a}{\beta_1} = \frac{0.38}{.85} = 0.45 \quad \epsilon_t = 0.003\,\frac{d_t - x}{x} = 0.003\left(\frac{8 - .38}{.38}\right) = 0.06$

$\epsilon_t = 0.06 > 0.005$ min ✓

Spacing

#5 bar, max s = 3h or 18.in

3h = 3x5 = 15" max s = 15"

$\frac{A_{req'd}}{b} = \frac{A_b}{s}$ @ $A_{req'd} = \rho b d = (0.008)(12)(2.5) = 0.24$

$A_b = 0.31\,in$, $b = 12\,in$ $\quad s = \frac{1 \times 12}{0.24} = 50"$ use s = 15"

use #5 bars @ 15" spacing

check shear – $V_u \le \phi V_c$, $\phi = .85$

$\phi V_c = \phi 2\sqrt{f'_c}\, bd = (.85)\,2\sqrt{4000}\,(12)(2.5) = 1.3^k$

$V_{u\,max} = \frac{1}{2} wL = \frac{1}{2}(.25)(10) = 1.25^k$

$\phi V_c = 1.3 > 1.25$ ✓

temp + shrinkage steel

$A_s = \rho b h = (.0018)(12)(5) = 0.108$

max s = 5(h) or 18"

5h = 5(5) = 25", max s = 18"

use #3 bar, $A_s = .11\ in^2$

$A_s = 0.11(12)/11 = 0.12$ sq in /per ft of width

spacing #3 bars at 11"

Appendix D: Structural Design Spreadsheet

D.1 Non-Composite Beam and Girder Design

20X20space4' interior

DL	300	lb/ft	Beam choose	W10X33	Beam weight:	33	
LL	400	lb/ft			Wu	1.0528	lb/ft
Wu	1	kip/ft			Mn	52.64	
Mu	50	kip-ft					
Span	20	ft	Attributed width			4	ft

Moment

Beam Properties

Fy	50						
Zx	38.8	Sx	35				
Mp	161.6667	Jc	0.583				
Lp	6.85	ho	9.3				
Lr	21.8	rts	2.2				
Lb	20						
BF	3.59			require	φMp or Mp	Mu or φMn	
		zone 1	Lb<Lp	φMp>Mu	145.5	52.64	
		zone 2	Lp<Lb<Lr	φMn<φMp	145.5	98.2915	
		zone 3	Lr<Lb	Mn<Mp	161.6666667	114.4618748	OK

Shear

h	7.86					Cv
tw	0.29	a	h/tw	27.10345	a<b	1
d	9.73	b	1.1*sqrt(k*E/Fy)	25.61607	b<a<c	0.945122
kdes	0.935	c	1.37*sqrt(k*E/Fy)	31.90365	a>c	1.040901

Ix	171					Cv choose	0.945122	
		Requires						
Vu	21.056	φVn>Vu						
φVn	80.00553					ok		

Deflection

LL limit	0.666667	in	act. LL def.	0.348698		Ix require	89.44099	
DL+LL limit	1	in	act. D+L del.	0.764811		Ix require	130.7826	ok

20X20space4' spandrel

DL	198	lb/ft	Beam choose	W12X16	Beam weight:	16	
LL	200	lb/ft			Wu	0.5832	lb/ft
Wu	0.5576	kip			Mn	29.16	
Mu	27.88	kip-ft					
Span	20	ft	Attributed width		2	ft	

Moment

Beam Properties

Fy	50						
Zx	20.1	Sx	17.1				
Mp	83.75	Jc	0.103				
Lp	2.73	ho	11.7				
Lr	8.03	rts	0.982				
Lb	20						
BF	5.75			require	φMp or Mp	Mu or φMn	
		zone 1	Lb<Lp	φMp>Mu	75.375	29.16	

zone 2	Lp<Lb<Lr	φMn<φMp	75.375	-23.9275	
zone 3	Lr<Lb	Mn<Mp	83.75	12.5881	OK

Shear

h	10.87					Cv	
tw	0.22	a	h/tw	49.40909	a<b	1	
d	12	b	1.1*sqrt(k*E/Fy)	19.91273	b<a<c	0.403018	
kdes	0.565	c	1.37*sqrt(k*E/Fy)	24.80041	a>c	0.18927	
Ix	103				Cv choose	0.18927	
		Requires					
Vu	11.664	φVn>Vu					
φVn	14.99019		ok				

Deflection

LL limit	0.666667	in	act. LL def.	0.289453	Ix require	44.7205	
DL+LL limit	1	in	act. D+L del.	0.703371	Ix require	72.4472	ok

20X20space10' interior

DL	750	lb/ft	Beam choose	W14X38	Beam weight:	38	
LL	1000	lb/ft			Wu	2.5608	lb/ft
Wu	2.5	kip			Mn	128.04	
Mu	125	kip-ft					
Span	20	ft	Attributed width		10	ft	

Moment

Beam Properties

Fy	50

Zx	61.5	Sx	54.6
Mp	256.25	Jc	0.798
Lp	5.47	ho	13.6
Lr	16.2	rts	1.82
Lb	20		
BF	8.1		

		require	φMp or Mp	Mu or φMn	
zone 1	Lb<Lp	φMp>Mu	230.625	128.04	
zone 2	Lp<Lb<Lr	φMn<φMp	230.625	112.932	
zone 3	Lr<Lb	Mn<Mp	256.25	117.4052	OK

Shear

							Cv
h	12.27						
tw	0.31	a	h/tw	39.58065		a<b	1
d	14.1	b	1.1*sqrt(k*E/Fy)	25.34062		b<a<c	0.640228
kdes	0.915	c	1.37*sqrt(k*E/Fy)	31.56059		a>c	0.477642
Ix	385					Cv choose	0.477642

		Requires	
Vu	51.216	φVn>Vu	
φVn	62.63319		ok

Deflection

LL limit	0.666667	in	act. LL def.	0.38719	Ix require	223.6025	
DL+LL limit	1	in	act. D+L del.	0.826264	Ix require	318.1118	ok

20X20space10' spandrel

DL	423	lb/ft	Beam choose	W10X33	Beam weight:	33
LL	500	lb/ft			Wu	1.3604	lb/ft
Wu	1.3076	kip			Mn	68.02
Mu	65.38	kip-ft				
Span	20	ft	Attributed width	5	ft	

Moment

Beam Properties

Fy	50		
Zx	38.8	Sx	35
Mp	161.6667	Jc	0.583
Lp	6.85	ho	9.3
Lr	21.8	rts	2.2
Lb	20		
BF	3.59		

		require	φMp or Mp	Mu or φMn	
zone 1	Lb<Lp	φMp>Mu	145.5	68.02	
zone 2	Lp<Lb<Lr	φMn<φMp	145.5	98.2915	OK
zone 3	Lr<Lb	Mn<Mp	161.6666667	114.4619	

Shear

h	7.86					Cv
tw	0.29	a	h/tw	27.10345	a<b	1
d	9.73	b	1.1*sqrt(k*E/Fy)	25.61607	b<a<c	0.945122
kdes	0.935	c	1.37*sqrt(k*E/Fy)	31.90365	a>c	1.040901
Ix	171				Cv choose	0.945122

Requires

Vu	27.208	φVn>Vu
φVn	80.00553	

ok

Deflection

LL limit	0.666667	in	act. LL def.	0.435872	Ix require	111.8012
DL+LL limit	1	in	act. D+L del.	0.988268	Ix require	168.9938

ok

25X25space5' interior

DL	375	lb/ft
LL	500	lb/ft
Wu	1.25	kip
Mu	97.65625	kip-ft
Span	25	ft

Beam choose	W14X34	Beam weight:	34

Wu	1.3044	lb/ft
Mn	101.9063	

Attributed width	5	ft

Moment

Beam Properties

Fy	50
Zx	54.6
Mp	227.5
Lp	5.4
Lr	15.6
Lb	25
BF	7.59

Sx	48.6
Jc	0.569
ho	13.5
rts	1.8

		require	φMp or Mp	Mu or φMn	
zone 1	Lb<Lp	φMp>Mu	204.75	101.9063	
zone 2	Lp<Lb<Lr	φMn<φMp	204.75	55.986	
zone 3	Lr<Lb	Mn<Mp	227.5	70.80739	OK

Shear

h	12.29					Cv
tw	0.285	a	h/tw	43.12281	a<b	1
d	14	b	1.1*sqrt(k*E/Fy)	24.49569	b<a<c	0.568045
kdes	0.855	c	1.37*sqrt(k*E/Fy)	30.50827	a>c	0.37601
Ix	340				Cv choose	0.37601

		Requires	
Vu	26.088	φVn>Vu	
φVn	45.00837		ok

Deflection

LL limit	0.833333	in	act. LL def.	0.5352	Ix require	218.3618	
DL+LL limit	1.25	in	act. D+L del.	1.163526	Ix require	316.479	ok

25X25space5' spandrel

DL	235.5	lb/ft	Beam choose	W12X26	Beam weight:	26	
LL	250	lb/ft			Wu	0.7242	lb/ft
Wu	0.6826	kip			Mn	56.57813	
Mu	34.13	kip-ft					
Span	25	ft	Attributed width			2.5	ft

Moment

Beam Properties

Fy	50		
Zx	37.2	Sx	33.4
Mp	155	Jc	0.3

Lp	3.81	ho	11.8
Lr	14.9	rts	1.75
Lb	25		
BF	5.42		

		require	φMp or Mp	Mu or φMn	
zone 1	Lb<Lp	φMp>Mu	139.5	56.57813	
zone 2	Lp<Lb<Lr	φMn<φMp	139.5	24.6502	
zone 3	Lr<Lb	Mn<Mp	155	44.91118	OK

Shear

						Cv
h	10.84					
tw	0.23	a	h/tw	47.13043	a<b	1
d	12.2	b	1.1*sqrt(k*E/Fy)	21.84546	b<a<c	0.463511
kdes	0.68	c	1.37*sqrt(k*E/Fy)	27.20752	a>c	0.250353
Ix	204				Cv choose	0.463511

		Requires	
Vu	14.484	φVn>Vu	
φVn	39.01832		ok

Deflection

LL limit	0.833333	in	act. LL def.	0.446	Ix require	109.1809	
DL+LL limit	1.25	in	act. D+L del.	1.076645	Ix require	175.7085	ok

20X20 Girder interior

DL	1500	lb/ft	Beam choose	W14X61	Beam weight:	61	
LL	2000	lb/ft			Wu	5.0976	lb/ft

Wu	5	kip		Mn	254.88
Mu	250	kip-ft			
Span	20	ft	Attributed width	20	ft

Moment

Beam Properties

Fy	50		
Zx	102	Sx	92.1
Mp	425	Jc	3.01
Lp	8.65	ho	13.3
Lr	27.5	rts	2.8
Lb	20		
BF	7.46		

		require	φMp or Mp	Mu or φMn	
zone 1	Lb<Lp	φMp>Mu	382.5	254.88	
zone 2	Lp<Lb<Lr	φMn<φMp	382.5	297.829	OK
zone 3	Lr<Lb	Mn<Mp	425	463.9953	

Shear

						Cv
h	11.42					
tw	0.375	a	h/tw	30.45333	a<b	1
d	13.9	b	1.1*sqrt(k*E/Fy)	29.49969	b<a<c	0.968685
kdes	1.24	c	1.37*sqrt(k*E/Fy)	36.74053	a>c	1.09345
Ix	640				Cv choose	1.09345

		Requires	
Vu	101.952	φVn>Vu	
φVn	170.9883		ok

Deflection

LL limit	0.666667	in	act. LL def.	0.465839	Ix require	447.205	
DL+LL limit	1	in	act. D+L del.	0.989441	Ix require	633.2422	ok

20X20 Girder spandrel

DL	798	lb/ft
LL	1000	lb/ft
Wu	2.5576	kip
Mu	127.88	kip-ft
Span	20	ft

Beam choose	W14X34	Beam weight:	34

Wu	2.612	lb/ft
Mn	130.6	

Attributed width	10	ft

Moment

Beam Properties

Fy	50
Zx	54.6
Mp	227.5
Lp	5.4
Lr	15.6
Lb	20
BF	7.59

Sx	48.6
Jc	0.569
ho	13.5
rts	1.8

		require	φMp or Mp	Mu or φMn	
zone 1	Lb<Lp	φMp>Mu	204.75	130.6	
zone 2	Lp<Lb<Lr	φMn<φMp	204.75	93.936	
zone 3	Lr<Lb	Mn<Mp	227.5	96.77011	OK

Shearr

h	12.29	Cv

tw	0.285	a	h/tw	43.12281	a<b	1
d	14	b	1.1*sqrt(k*E/Fy)	24.49569	b<a<c	0.568045
kdes	0.855	c	1.37*sqrt(k*E/Fy)	30.50827	a>c	0.37601
Ix	340				Cv choose	0.568045

		Requires	
Vu	52.24	φVn>Vu	
φVn	67.99498		ok

Deflection

LL limit	0.666667	in	act. LL def.	0.438436	Ix require	223.6025	
DL+LL limit	1	in	act. D+L del.	0.95433	Ix require	324.472	ok

25X25 Girder interior

DL	1875	lb/ft
LL	2500	lb/ft
Wu	6.25	kip
Mu	488.2813	kip-ft
Span	25	ft

Beam choose	W21X73	Beam weight:	73

Wu	6.3668	lb/ft
Mn	497.4063	

Attributed width	25	ft

Moment

Beam Properties

Fy	50		
Zx	172	Sx	151
Mp	716.6667	Jc	3.02
Lp	6.39	ho	20.5
Lr	19.2	rts	2.19

Lb	25
BF	19.4

		require	φMp or Mp	Mu or φMn	
zone 1	Lb<Lp	φMp>Mu	645	497.4063	
zone 2	Lp<Lb<Lr	φMn<φMp	645	283.966	
zone 3	Lr<Lb	Mn<Mp	716.6666667	299.0629	OK

Shear

h	18.72					Cv
tw	0.455	a	h/tw	41.14286	a<b	1
d	21.2	b	1.1*sqrt(k*E/Fy)	29.49969	b<a<c	0.717006
kdes	1.24	c	1.37*sqrt(k*E/Fy)	36.74053	a>c	0.599073
Ix	1600				Cv choose	0.599073

		Requires	
Vu	127.336	φVn>Vu	
φVn	173.3598		ok

Deflection

LL limit	0.833333	in	act. LL def.	0.568651	Ix require	1091.809	
DL+LL limit	1.25	in	act. D+L del.	1.206828	Ix require	1544.74	ok

25X25 Girder spandrel

DL	985.5	lb/ft
LL	1250	lb/ft
Wu	3.1826	kip
Mu	248.6406	kip-ft

Beam choose	W18X50	Beam weight:	50

Wu	3.2626	lb/ft
Mn	254.8906	

Span	25	ft	Attributed width		12.5	ft

Moment

Beam Properties

Fy	50					
Zx	101	Sx	88.9			
Mp	420.8333	Jc	1.24			
Lp	5.83	ho	17.4			
Lr	17	rts	1.98			
Lb	25					
BF	13.1			require	φMp or Mp	Mu or φMn
		zone 1	Lb<Lp	φMp>Mu	378.75	254.8906
		zone 2	Lp<Lb<Lr	φMn<φMp	378.75	127.623
		zone 3	Lr<Lb	Mn<Mp	420.8333333	144.1427	OK

Shear

h	16.056					Cv
tw	0.355	a	h/tw	45.22817	a<b	1
d	18	b	1.1*sqrt(k*E/Fy)	26.11799	b<a<c	0.577472
kdes	0.972	c	1.37*sqrt(k*E/Fy)	32.52877	a>c	0.388593
Ix	800				Cv choose	0.388593

		Requires	
Vu	65.252	φVn>Vu	
φVn	74.49334		ok

Deflection

LL limit	0.833333	in	act. LL def.	0.568651	Ix require	545.9045

DL+LL limit	1.25	in	act. D+L del.	1.236853	Ix require	791.5858	ok

Roof 20X20 4OC interior

DL	348	lb/ft	Beam choose	W10X33	Beam weight:	33	
LL	80	lb/ft			Wu	1.1984	lb/ft
Wu	1.1456	kip			Mn	59.92	
Mu	57.28	kip-ft					
Span	20	ft	Attributed width			4	ft

Moment

Beam Properties

Fy	50		
Zx	38.8	Sx	35
Mp	161.6667	Jc	0.583
Lp	6.85	ho	9.3
Lr	21.8	rts	2.2
Lb	20		
BF	3.59		

		require	φMp or Mp	Mu or φMn	
zone 1	Lb<Lp	φMp>Mu	145.5	59.92	
zone 2	Lp<Lb<Lr	φMn<φMp	145.5	98.2915	OK
zone 3	Lr<Lb	Mn<Mp	161.6666667	114.4619	

Shear

h	7.86					Cv
tw	0.29	a	h/tw	27.10345	a<b	1
d	9.73	b	1.1*sqrt(k*E/Fy)	25.61607	b<a<c	0.945122

kdes	0.935	c	1.37*sqrt(k*E/Fy)	31.90365		a>c	1.040901
Ix	171					Cv choose	0.945122

		Requires	
Vu	23.968	φVn>Vu	
φVn	80.00553		ok

Deflection

LL limit	0.666667	in	act. LL def.	0.06974		Ix require	17.8882	
DL+LL limit	1	in	act. D+L del.	0.870582		Ix require	148.8696	ok

Roof 20X20 4OC exterior

DL	158	lb/ft	Beam choose	W12X16	Beam weight:	16	
LL	200	lb/ft			Wu	0.752	lb/ft
Wu	0.7264	kip			Mn	37.6	
Mu	36.32	kip-ft					
Span	20	ft	Attributed width			2	ft

Moment

Beam Properties

Fy	50		
Zx	20.1	Sx	17.1
Mp	83.75	Jc	0.103
Lp	2.73	ho	11.7
Lr	8.03	rts	0.962
Lb	20		
BF	5.75		

require φMp or Mp Mu or

				φMn
zone 1	Lb<Lp	φMp>Mu	75.375	37.6
zone 2	Lp<Lb<Lr	φMn<φMp	75.375	-23.9275
zone 3	Lr<Lb	Mn<Mp	83.75	12.25836	OK

Shear

h	10.87					Cv
tw	0.22	a	h/tw	49.40909	a<b	1
d	12	b	1.1*sqrt(k*E/Fy)	19.91273	b<a<c	0.403018
kdes	0.565	c	1.37*sqrt(k*E/Fy)	24.80041	a>c	0.18927
Ix	103				Cv choose	0.18927

		Requires	
Vu	15.04	φVn>Vu	
φVn	14.99019		ok

Deflection

LL limit	0.666667	in	act. LL def.	0.289453	Ix require	44.7205	
DL+LL limit	1	in	act. D+L del.	0.906953	Ix require	93.41615	ok

Roof 20X20 5O.C. interior

DL	375	lb/ft
LL	500	lb/ft
Wu	1.432	kip
Mu	71.6	kip-ft
Span	20	ft

Beam choose	W14X26	Beam weight:	26

Wu	1.4736	lb/ft
Mn	73.68	

Attributed width	5	ft

Moment

Beam Properties

Fy	50		
Zx	40.2	Sx	35.3
Mp	167.5	Jc	0.358
Lp	3.81	ho	13.5
Lr	11.1	rts	1.31
Lb	20		
BF	7.99		

		require	φMp or Mp	Mu or φMn	
zone 1	Lb<Lp	φMp>Mu	150.75	73.68	
zone 2	Lp<Lb<Lr	φMn<φMp	150.75	21.3919	
zone 3	Lr<Lb	Mn<Mp	167.5	43.20676	OK

Shear

h	12.26					Cv
tw	0.255	a	h/tw	48.07843	a<b	1
d	13.9	b	1.1*sqrt(k*E/Fy)	23.98908	b<a<c	0.498957
kdes	0.82	c	1.37*sqrt(k*E/Fy)	29.87731	a>c	0.290108
Ix	245				Cv choose	0.290108

		Requires	
Vu	29.472	φVn>Vu	
φVn	30.84868		ok

Deflection

LL limit	0.666667	in	act. LL def.	0.304221	Ix require	111.8012	
DL+LL limit	1	in	act. D+L del.	0.747167	Ix require	183.0559	ok

Roof 20X20 5O.C exterior

DL	195.5	lb/ft	Beam choose	W12X19	Beam weight:	19	
LL	250	lb/ft			Wu	0.9384	lb/ft
Wu	0.908	kip			Mn	46.92	
Mu	45.4	kip-ft					
Span	20	ft	Attributed width		2.5	ft	

Moment

Beam Properties

Fy	50						
Zx	24.7	Sx		21.3			
Mp	102.9167	Jc		0.18			
Lp	2.9	ho		11.8			
Lr	8.62	rts		1.02			
Lb	20						
BF	6.43			require	φMp or Mp	Mu or φMn	
		zone 1	Lb<Lp	φMp>Mu	92.625	46.92	
		zone 2	Lp<Lb<Lr	φMn<φMp	92.625	-17.328	
		zone 3	Lr<Lb	Mn<Mp	102.9166667	18.56415	OK

Shear

h	10.9					Cv
tw	0.235	a	h/tw	46.38298	a<b	1
d	12.2	b	1.1*sqrt(k*E/Fy)	21.35814	b<a<c	0.460474
kdes	0.65	c	1.37*sqrt(k*E/Fy)	26.60059	a>c	0.247083
Ix	130				Cv choose	0.247083

		Requires
Vu	18.768	φVn>Vu
φVn	21.25162	

ok

Deflection

LL limit	0.666667	in	act. LL def.	0.28667	Ix require	55.90062	
DL+LL limit	1	in	act. D+L del.	0.896703	Ix require	116.5714	ok

Roof 20X20 10 O.C interior

DL	750	lb/ft
LL	1000	lb/ft
Wu	2.864	kip
Mu	143.2	kip-ft
Span	20	ft

Beam choose	W14X38	Beam weight:	38

Wu	2.9248	lb/ft
Mn	146.24	

Attributed width	10	ft

Moment

Beam Properties

Fy	50		
Zx	61.5	Sx	54.6
Mp	256.25	Jc	0.798
Lp	5.47	ho	13.6
Lr	16.2	rts	1.82
Lb	20		
BF	8.1		

		require	φMp or Mp	Mu or φMn
zone 1	Lb<Lp	φMp>Mu	230.625	146.24
zone 2	Lp<Lb<Lr	φMn<φMp	230.625	112.932

zone 3	Lr<Lb	Mn<Mp		256.25	117.4052	OK

Shear

h	12.27					Cv
tw	0.31	a	h/tw	39.58065	a<b	1
d	14.1	b	1.1*sqrt(k*E/Fy)	25.34062	b<a<c	0.640228
kdes	0.915	c	1.37*sqrt(k*E/Fy)	31.56059	a>c	0.477642
Ix	385				Cv choose	0.477642

		Requires
Vu	58.496	φVn>Vu
φVn	62.63319	ok

Deflection

LL limit	0.666667	in	act. LL def.	0.38719	Ix require	223.6025	
DL+LL limit	1	in	act. D+L del.	0.943712	Ix require	363.3292	ok

Roof 20X20 10O.C exterior

DL	383	lb/ft	Beam choose	W12X26	Beam weight:	26	
LL	500	lb/ft			Wu	1.5504	lb/ft
Wu	1.5088	kip			Mn	77.52	
Mu	75.44	kip-ft					
Span	20	ft	Attributed width		5	ft	

Moment

Beam Properties

Fy	50		
Zx	37.2	Sx	33.4

Mp	155	Jc	0.3
Lp	5.33	ho	11.8
Lr	14.9	rts	1.75
Lb	20		
BF	5.89		

		require	φMp or Mp	Mu or φMn	
zone 1	Lb<Lp	φMp>Mu	139.5	77.52	
zone 2	Lp<Lb<Lr	φMn<φMp	139.5	53.0937	
zone 3	Lr<Lb	Mn<Mp	155	61.62338	OK

Shear

h	10.84					Cv
tw	0.23	a	h/tw	47.13043	a<b	1
d	12.2	b	1.1*sqrt(k*E/Fy)	21.84546	b<a<c	0.463511
kdes	0.68	c	1.37*sqrt(k*E/Fy)	27.20752	a>c	0.250353
Ix	204				Cv choose	0.463511

		Requires	
Vu	31.008	φVn>Vu	
φVn	39.01832		ok

Deflection

LL limit	0.666667	in	act. LL def.	0.365364	Ix require	111.8012	
DL+LL limit	1	in	act. D+L del.	0.944099	Ix require	192.5963	ok

Roof 25X25 5O.C interior

DL	375	lb/ft	Beam choose	W16X31	Beam weight:	31

LL	500	lb/ft		Wu	1.4816	lb/ft
Wu	1.432	kip		Mn	115.75	
Mu	71.6	kip-ft				
Span	25	ft	Attributed width	5	ft	

Moment

Beam Properties

Fy	50						
Zx	54	Sx	47.2				
Mp	225	Jc	0.461				
Lp	4.13	ho	15.4				
Lr	11.9	rts	1.42				
Lb	25						
BF	10.2			require	φMp or Mp	Mu or φMn	
		zone 1	Lb<Lp	φMp>Mu	202.5	115.75	
		zone 2	Lp<Lb<Lr	φMn<φMp	202.5	-10.374	
		zone 3	Lr<Lb	Mn<Mp	225	45.1762	OK

Shear

h	14.216					Cv
tw	0.275	a	h/tw	51.69455	a<b	1
d	15.9	b	1.1*sqrt(k*E/Fy)	24.30876	b<a<c	0.470238
kdes	0.842	c	1.37*sqrt(k*E/Fy)	30.27545	a>c	0.257673
Ix	375				Cv choose	0.257673
		Requires				
Vu	29.632	φVn>Vu				

φVn	33.80032		ok

Deflection

LL limit	0.833333	in	act. LL def.	0.485248	Ix require	218.3618	
DL+LL limit	1.25	in	act. D+L del.	1.19824	Ix require	359.472	ok

Roof 25X25 5 O.C ex.

DL	195.5	lb/ft
LL	250	lb/ft
Wu	0.7928	kip
Mu	39.64	kip-ft
Span	25	ft

Beam choose	W12X26	Beam weight:	26

Wu	0.8344	lb/ft
Mn	65.1875	

Attributed width	2.5	ft

Moment

Beam Properties

Fy	50		
Zx	37.2	Sx	33.4
Mp	155	Jc	0.3
Lp	5.33	ho	11.8
Lr	14.9	rts	1.75
Lb	25		
BF	5.89		

		require	φMp or Mp	Mu or φMn	
zone 1	Lb<Lp	φMp>Mu	139.5	65.1875	
zone 2	Lp<Lb<Lr	φMn<φMp	139.5	23.6437	
zone 3	Lr<Lb	Mn<Mp	155	44.91118	OK

Shear

h	10.84					Cv
tw	0.23	a	h/tw	47.13043	a<b	1
d	12.2	b	1.1*sqrt(k*E/Fy)	21.84546	b<a<c	0.463511
kdes	0.68	c	1.37*sqrt(k*E/Fy)	27.20752	a>c	0.250353
Ix	204				Cv choose	0.463511

		Requires	
Vu	16.688	φVn>Vu	
φVn	39.01832		ok

Deflection

LL limit	0.833333	in	act. LL def.	0.446	Ix require	109.1809	
DL+LL limit	1.25	in	act. D+L del.	1.240476	Ix require	202.4457	ok

Roof Gir. 20X20 in.

DL	1500	lb/ft	Beam choose	W16x57	Beam weight:	57	
LL	2000	lb/ft			Wu	5.8192	lb/ft
Wu	5.728	kip			Mn	290.96	
Mu	286.4	kip-ft					
Span	20	ft	Attributed width			20	ft

Moment

Beam Properties

Fy	50		
Zx	105	Sx	92.2
Mp	437.5	Jc	2.22
Lp	5.65	ho	15.7

Lr	18.3	rts	1.92
Lb	20		
BF	12		

		require	φMp or Mp	Mu or φMn	
zone 1	Lb<Lp	φMp>Mu	393.75	290.96	
zone 2	Lp<Lb<Lr	φMn<φMp	393.75	221.55	
zone 3	Lr<Lb	Mn<Mp	437.5	238.3975	OK

Shear

h	14.16					Cv
tw	0.43	a	h/tw	32.93023	a<b	1
d	16.4	b	1.1*sqrt(k*E/Fy)	28.03598	b<a<c	0.851375
kdes	1.12	c	1.37*sqrt(k*E/Fy)	34.91753	a>c	0.844648
Ix	758				Cv choose	0.851375

		Requires	
Vu	116.384	φVn>Vu	
φVn	180.1169		ok

Deflection

LL limit	0.666667	in	act. LL def.	0.39332	Ix require	447.205	
DL+LL limit	1	in	act. D+L del.	0.95367	Ix require	722.882	ok

Roof Gir. 20X20 ex.

DL	798	lb/ft
LL	1000	lb/ft
Wu	2.9408	kip

Beam choose	W14X38	Beam weight:	38

Wu	3.0016	lb/ft
Mn	150.08	

Mu	147.04	kip-ft
Span	20	ft

Attributed width	10	ft

Moment

Beam Properties

Fy	50
Zx	61.5
Mp	256.25
Lp	5.47
Lr	16.2
Lb	20
BF	8.1

Sx	54.6
Jc	0.798
ho	13.6
rts	1.82

		require	φMp or Mp	Mu or φMn	
zone 1	Lb<Lp	φMp>Mu	230.625	150.08	
zone 2	Lp<Lb<Lr	φMn<φMp	230.625	112.932	
zone 3	Lr<Lb	Mn<Mp	256.25	117.4052	OK

Shear

h	12.27
tw	0.31
d	14.1
kdes	0.915
Ix	385

				Cv
a	h/tw	39.58065	a<b	1
b	1.1*sqrt(k*E/Fy)	25.34062	b<a<c	0.640228
c	1.37*sqrt(k*E/Fy)	31.56059	a>c	0.477642
			Cv choose	0.640228

		Requires	
Vu	60.032	φVn>Vu	
φVn	83.95304		ok

Deflection

LL limit	0.666667	in	act. LL def.	0.38719	Ix require	223.6025	
DL+LL limit	1	in	act. D+L del.	0.968492	Ix require	372.8696	ok

Roof Gir. 25X25 in.

DL	1875	lb/ft	Beam choose	W21X83	Beam weight:	83	
LL	2500	lb/ft			Wu	7.2928	lb/ft
Wu	7.16	kip			Mn	569.75	
Mu	559.375	kip-ft					
Span	25	ft	Attributed width			25	ft

Moment

Beam Properties

Fy	50							
Zx	196	Sx		171				
Mp	816.6667	Jc		4.34				
Lp	6.46	ho		20.6				
Lr	20.2	rts		2.21				
Lb	25							
BF	20.8			require	φMp or Mp		Mu or φMn	
		zone 1	Lb<Lp	φMp>Mu		735	569.75	
		zone 2	Lp<Lb<Lr	φMn<φMp		735	349.368	
		zone 3	Lr<Lb	Mn<Mp		816.6666667	368.4345	OK

Shear

h	18.72					Cv
tw	0.515	a	h/tw	36.34951	a<b	1

d	21.4	b	1.1*sqrt(k*E/Fy)	30.66614		b<a<c	0.843646
kdes	1.34	c	1.37*sqrt(k*E/Fy)	38.19328		a>c	0.829382
Ix	1830					Cv choose	0.829382

		Requires	
Vu	145.856	φVn>Vu	
φVn	274.2186		ok

Deflection

LL limit	0.833333	in	act. LL def.	0.497181	Ix require	1091.809	
DL+LL limit	1.25	in	act. D+L del.	1.208613	Ix require	1769.41	ok

Roof Gir. 25X25 ex.

DL	985.5	lb/ft	Beam choose	W18X60	Beam weight:	60	
LL	1250	lb/ft			Wu	3.7528	lb/ft
Wu	3.6568	kip			Mn	293.1875	
Mu	285.6875	kip-ft					
Span	25	ft	Attributed width	12.5	ft		

Moment

Beam Properties

Fy	50		
Zx	123	Sx	108
Mp	512.5	Jc	2.17
Lp	5.93	ho	17.5
Lr	18.2	rts	2.02
Lb	25		

BF	14.5			require	φMp or Mp	Mu or φMn	
		zone 1	Lb<Lp	φMp>Mu	461.25	293.1875	
		zone 2	Lp<Lb<Lr	φMn<φMp	461.25	184.735	
		zone 3	Lr<Lb	Mn<Mp	512.5	201.4492	OK

Shear

h	16					Cv
tw	0.415	a	h/tw	38.55422	a<b	1
d	18.2	b	1.1*sqrt(k*E/Fy)	27.78453	b<a<c	0.720661
kdes	1.1	c	1.37*sqrt(k*E/Fy)	34.60437	a>c	0.605196
Ix	984				Cv choose	0.605196

		Requires	
Vu	75.056	φVn>Vu	
φVn	137.1314		ok

Deflection

LL limit	0.833333	in	act. LL def.	0.462317	Ix require	545.9045	
DL+LL limit	1.25	in	act. D+L del.	1.156657	Ix require	910.5202	ok

D.2 Composite Beam and Girder Design

Scenario 2A	Bay Size=20x20		Beam spacing=5ft	
Design Values>	f'c(ksi)	Fy(ksi)	be(in)	slab(in)
	4	50	60	5
Design Loads	DL(psf)	LL(psf)	tributary	L(ft)

			width(ft)	
Values-	91	100	5	20
Dead Load(plf)	455			
Live load(plf)	500			
Uniform Load=1.2D+1.6L(klf)	1.35			
Mu(ft*k)	67.3			
Choose Beam	**W10x17**			
Construction Deflection	beam weight	concrete(pcf)	E(ksi)	Ix(in^4)
Values-	17	150	29000	81.9
Wet concrete(plf)	343.75			
Uniform Load(klf)	0.59			
Moment	29.65			
deflection(in)	0.89			
Δc<limit	**TRUE**			
Check strength	ΦMn lower(ft*k)	ΦMn upper(ft*k)	Y2 change(in)	
Values-	216	226	0.018	
a(in)	0.96			
Y2(in)	4.52			
ΦMn(ft*k)	216.36			
ΦMn>Mu	**TRUE**			
Service deflection				
Live Load(klf)	0.25			
Moment	12.5			
deflection(in)	0.378			

ΔL<limit	TRUE			
Design studs	Qn(k)	ΣQn(k)		
Values-	21	221		
# studs	22			
studs per beam	44			

Slab Design						
Design Values-	L(ft)	h(in)	conc.(pcf)	f'c	fy	B1
	10	5	150	4000	60000	0.85
min thickness						
h=L/24	5					
min h=	5					
Loads	DL slab	DL other	LL			
Design Values-	62.5	12	100			
uniform load(plf)	249.4					
Mu(k-ft)	2.494					
Design Steel	pmax	Pb	Φ	b	d	
Design Values-	0.0248	0.0367	0.9	12	2.5	
p=.35pb	0.012845					
Mn=	2.771111					
Rn=	683.3502					
m=	17.64706					
required bd^2=	48.66221					
d=	2.013749					

Rn=	443.3778					
p=	0.007947					
pmin=	0.003333					
pmin<p<pmax	TRUE					
As=	0.238406					
pick bar						
bar size	#5					
As=	0.31					
check strength	C	T				
Design Values-	40.8	18.6				
a=	0.455882					
Mn=	3.521691					
ΦMN>MU	TRUE					
Check shear						
Vu=	1.247					
ΦVc=	2.84605					
ΦVc>Vu	TRUE					
Spacing						
As=	0.238406					
smax=	15					
s required=	9.228609					
s=	9					

D.3 Non- Composite Column Design

Column Designs for Non-composite Beams and Girders

	Roof								
Bay Size=20'x20'			Column Size		Column Size		Column Size		Column Size
Beam Spacing		interior		Exterior (gri. aside)		Exterior(be. Aside)		conner	
4'	Pu	121.664	W6X20	63.696	W6X15	59.776	W6X15	30.38	W6X12
5'	Pu	120.544	W6X20	62.72	W6X15	59.232	W6X15	29.996	W6X12
10'	Pu	122.464	W6X20	62.272	W6X15	58.496	W6X15	29.788	W6X12
Bay Size=25'x25'									
Beam Spacing		interior		Exterior (gri. aside)		Exterior(be. Aside)		conner	
5'	Pu	188.52	W8X24	95.964	W6X15	92.068	W6X15	46.58875	W6X12

	Floor 3								
Bay Size=20'x20'			Column Size		Column Size		Column Size		Column Size
Beam Spacing		interior		Exterior (gri. aside)		Exterior(be. Aside)		conner	
4'	Pu	224.896	W8X28	117.984	W6X15	110.336	W6X15	56.236	W6X15
5'	Pu	223.296	W8X28	116.288	W6X15	109.408	W6X15	55.516	W6X15
10'	Pu	226.496	W8X28	115.136	W6X15	107.776	W6X15	55.18	W6X15
Bay Size=25'x25'									
Beam Spacing		interior		Exterior (gri. aside)		Exterior(be. Aside)		conner	
5'	Pu	348.24	W8X35	177.3	W6X15	169.596	W6X15	86.12075	W6X15

	Floor 2								
Bay Size=20'x20'			Column Size		Column Size		Column Size		Column Size
Beam Spacing		interior		Exterior (gri. aside)		Exterior(be. Aside)		conner	
4'	Pu	328.408	W8X35	172.422	W8X24	161.046	W6X20	82.242	W6X15
5'	Pu	326.328	W8X35	170.006	W8X24	159.734	W6X20	81.186	W6X15
10'	Pu	330.808	W8X35	168.15	W8X24	157.206	W6X20	80.722	W6X15
Bay Size=25'x25'									
Beam Spacing		interior		Exterior (gri. aside)		Exterior(be. Aside)		conner	
5'	Pu	508.31	W10X49	258.786	W8X31	247.274	W8X31	125.8028	W6X20

	Floor 1								
Bay Size=20'x20'			Column Size		Column Size		Column Size		Column Size
Beam Spacing		interior		Exterior (gri. aside)		Exterior(be. Aside)		conner	
4'	Pu	431.99	W8X48	226.95	W8X31	211.806	W8x31	108.248	W8x24
5'	Pu	429.43	W8X48	223.814	W8X31	210.11	W8x31	106.856	W8x24
10'	Pu	435.19	W8X48	221.254	W8X31	206.686	W8x31	106.264	W8x24
Bay Size=25'x25'									
Beam Spacing		interior		Exterior (gri. aside)		Exterior(be. Aside)		conner	

5'	Pu	668.52	W12x65	340.432	W10X39	325.112	W10X39	165.5448	W8X24

Column for Composite Beams and Girder									
	Roof								
Bay Size=20'x20'			Column Size		Column Size		Column Size		Column Size
Beam Spacing		interior		Exterior (gri. aside)		Exterior(be. Aside)		conner	
4'	Pu	118.24	W6X20	61.904	W6X15	58.576	W6X15	29.548	W6X12
5'	Pu	118.24	W6X20	61.632	W6X15	58.44	W6X15	29.412	W6X12
10'	Pu	119.04	W6X20	61.248	W6X15	57.936	W6X15	29.26	W6X12
Bay Size=25'x25'									
Beam Spacing		interior		Exterior (gri. aside)		Exterior(be. Aside)		conner	
5'	Pu	186.4	W8X24	95.004	W6X15	91.42	W6X15	46.03675	W6X12
	Floor 3								
Bay Size=20'x20'			Column Size		Column Size		Column Size		Column Size
Beam Spacing		interior		Exterior (gri. aside)		Exterior(be. Aside)		conner	
4'	Pu	229.52	W8X28	120.328	W6X15	113.672	W6X15	57.536	W6X15
5'	Pu	229.52	W8X28	119.784	W6X15	113.4	W6X15	57.264	W6X15
10'	Pu	231.12	W8X28	119.016	W6X15	112.392	W6X15	56.96	W6X15
Bay									

Size=25'x25'									
Beam Spacing		interior		Exterior (gri. aside)		Exterior(be. Aside)		conner	
5'	Pu	361.925	W10x39	184.5705	W6X25	177.4025	W6X25	89.636	W6X15
	Floor 2								
Bay Size=20'x20'			Column Size		Column Size		Column Size		Column Size
Beam Spacing		interior		Exterior (gri. aside)		Exterior(be. Aside)		conner	
4'	Pu	341.28	W8X35	179.052	W8X24	169.068	W8X24	85.794	W6X15
5'	Pu	341.28	W8X35	178.236	W8X24	168.66	W8X24	85.386	W6X15
10'	Pu	343.68	W8X35	177.084	W8X24	167.148	W8X24	99.78	W6X15
Bay Size=25'x25'									
Beam Spacing		interior		Exterior (gri. aside)		Exterior(be. Aside)		conner	
5'	Pu	538.04	W10X49	274.537	W8X31	263.785	W8X31	133.50525	W6X20
	Floor 1								
Bay Size=20'x20'			Column Size		Column Size		Column Size		Column Size
Beam Spacing		interior		Exterior (gri. aside)		Exterior(be. Aside)		conner	
4'	Pu	452.91	W10X49	237.716	W8X31	224.404	W8X31	113.932	W8x24
5'	Pu	452.91	W10X49	236.628	W8X31	223.86	W8X31	113.388	W8x24
10'	Pu	456.11	W10X49	235.092	W8X31	221.844	W8X31	127.63	W8x24

Bay Size=25'x25'									
Beam Spacing		interior		Exterior (gri. aside)		Exterior(be. Aside)		conner	
5'	Pu	714.055	W12x65	364.4135	W8X40	350.0775	W8X40	177.3045	W8X28

D.4 Composite Column Design (Concrete Enclosed)

Composite Columns Load Capacity (Concrete Enclosed)

W beam	**section size**	64	Seleted beam	6X12			
As	3.55						
Is	2.99						
Es	29000						
Fy	50		Po	585.4388			
Steel Bar			C1	0.159167	<0.3		
Asr	4		EIeff	808588.7			
d	3						
Esr	29000		Pe	554.1979	>0.44*Po	257.5931	
Fy	60		ϕPn	**282.1771**			
Concrete							
Ac	56.45						
Ic	338.3433						
Ec	3410						
f'c	3.5						
W beam	**section size**	100	Seleted beam	W8x15			

W beam	**section size**	100	Seleted beam	W8x28	
As	8.24				
Is	21.7				
Es	29000				
Fy	50		Po	913.086	
Steel Bar			C1	0.185833	<0.3
Asr	4		EIeff	1681285	
d	4				
Esr	29000		Pe	800.2325	>0.44*P
Fy	60		ϕPn	**424.7799**	
Concrete					
Ac	87.76				
Ic	811.6333				
Ec	3410				
f'c	3.5				
W beam	**section size**	100	Seleted beam	W8x18	

As	4.44				
Is	3.41				
Es	29000				
Fy	50	Po	734.391		
Steel Bar		C1	0.14625	<0.3	
Asr	4	EIeff	1470653		
d	4				
Esr	29000	Pe	699.979	>0.44*Po	323.132
Fy	60	ϕPn	**355.0408**		
Concrete					
Ac	91.56				
Ic	829.9233				
Ec	3410				
f'c	3.5				

As	5.26			
Is	7.97			
Es	29000			
Fy	50	Po	772.9515	
Steel Bar		C1	0.154792	<0.3
Asr	4	EIeff	1516199	
d	4			
Esr	29000	Pe	721.6574	>0.44*P
Fy	60	ϕPn	**370.2706**	
Concrete				
Ac	90.74			
Ic	825.3633			
Ec	3410			
f'c	3.5			

W beam	**section size**	100	Seleted beam	W8x21		
As	6.16					
Is	9.77					
Es	29000					
Fy	50		Po	815.274		
Steel Bar			C1	0.164167	<0.3	
Asr	4		EIeff	1567678		
d	4					
Esr	29000		Pe	746.1592	>0.44*Po	358.7206
Fy	60		ϕPn	**387.0379**		

W beam	**section size**	100	Seleted beam	W8x13	
As	3.84				
Is	2.73				
Es	29000				
Fy	50		Po	706.176	
Steel Bar			C1	0.14	<0.3
Asr	4		EIeff	1435890	
d	4				
Esr	29000		Pe	683.433	>0.44*P
Fy	60		ϕPn	**343.6775**	

Concrete	
Ac	89.84
Ic	823.5633
Ec	3410
f'c	3.5

Concrete	
Ac	92.16
Ic	830.6033
Ec	3410
f'c	3.5

W beam			Seleted beam	w10x30		
As	8.84					
Is	16.7					
Es	29000					
Fy	50		Po	1072.201		
Steel Bar			C1	0.163143	<0.3	
Asr	4	d	EIeff	2658386		
Esr	29000		Pe	1265.298	>0.44*Po	471.7684
Fy	60		ϕPn	**564.0321**		
Concrete						
Ac	#REF!					
Ic	#REF!					
Ec	3410					
f'c	3.5					

W beam	**section size**	144	Seleted beam	w10x26	
As	7.61				
Is	14.1				
Es	29000				
Fy	50		Po	1014.36	
Steel Bar			C1	0.154357	<0.3
Asr	4		EIeff	2572815	
d	5				
Esr	29000		Pe	1224.569	>0.44*Po
Fy	60		ϕPn	**537.8766**	
Concrete					
Ac	132.39				
Ic	1713.9				
Ec	3410				
f'c	3.5				

W beam	**section size**	144	Seleted beam	w10x39
As	11.5			
Is	42.1			
Es	29000			

W beam	**section size**	144	Seleted beam	w10x15
As	4.41			
Is	2.89			
Es	29000			

Fy	50		Po	1197.288		
Steel Bar			C1	0.182143	<0.3	
Asr	4		EIeff	2830625		
d	5					
Esr	29000		Pe	1347.277	>0.44*Po	526.8065
Fy	60		ϕPn	**619.0449**		
Concrete						
Ac	128.5					
Ic	1685.9					
Ec	3410					
f'c	3.5					

Fy	50		Po	863.8803	
Steel Bar			C1	0.1315	<0.3
Asr	4		EIeff	2351455	
d	5				
Esr	29000		Pe	1119.21	>0.44*P
Fy	60		ϕPn	**469.0396**	
Concrete					
Ac	135.59				
Ic	1725.11				
Ec	3410				
f'c	3.5				

W beam	**section size**	144	Seleted beam	w10x12		
As	3.54					
Is	2.18					
Es	29000					
Fy	50		Po	822.9685		
Steel Bar			C1	0.125286	<0.3	
Asr	4		EIeff	2289972		
d	5					
Esr	29000		Pe	1569.522	>0.44*Po	362.1061
Fy	60		ϕPn	**495.6013**		
Concrete						
Ac	136.46					
Ic	1725.82					

W beam	**section size**	196	Seleted beam	w12x30	
As	8.79				
Is	20.3				
Es	29000				
Fy	50		Po	1224.55	
Steel Bar			C1	0.145781	<0.3
Asr	4		EIeff	3924246	
d	6				
Esr	29000		Pe	1867.803	>0.44*P
Fy	60		ϕPn	**698.0151**	
Concrete					
Ac	183.21				
Ic	3181.033				

Ec	3410					
f'c	3.5					
W beam	**section size**	196	Seleted beam	w12x26		
As	7.65					
Is	17.3					
Es	29000					
Fy	50		Po	1170.941		
Steel Bar			C1	0.139844	<0.3	
Asr	4		EIeff	3828211		
d	6					
Esr	29000		Pe	1822.093	>0.44*Po	515.2142
Fy	60		ϕPn	**671.0918**		
Concrete						
Ac	184.35					
Ic	3184.033					
Ec	3410					
f'c	3.5					

Ec	3410				
f'c	3.5				
W beam	**section size**	196	Seleted beam	w12x35	
As	10.3				
Is	24.5				
Es	29000				
Fy	50		Po	1295.558	
Steel Bar			C1	0.153646	<0.3
Asr	4		EIeff	4051146	
d	6				
Esr	29000		Pe	1928.202	>0.44*Po
Fy	60		ϕPn	**733.4729**	
Concrete					
Ac	181.7				
Ic	3176.833				
Ec	3410				
f'c	3.5				

Appendix E: Cost Analysis Spreadsheet

E.1 Cost for Composite Beams and Girders

Full Composite Floor Beams and Girders

Floor Beams and Girders

Bay Size=20'x20'

Beam Spacing	Interior Beam	Exterior Beam	Interior Girder	Exterior Girder	# of Interi	# of Exterior Beams	# of Interior Girders	# of Exterior Girders	# of int. columns	# of ext. columns	# of corner columns
4'	W10x17	W10x12	W14x30	W12X22	245	10	40	20	36	26	4
5'	W10x17	W10x12	W14x30	W12X22	195	10	40	20	36	26	4
10'	W12x22	W10x17	W14x30	W12X22	95	10	40	20	36	26	4

Floor Beams and Girders

Bay Size=25'x25'

Beam Spacing	Interior Beam	Exterior Beam	Interior Girder	Exterior Girder	# of Interi	# of Exterior Beams	# of Interior Girders	# of Exterior Girders	# of int. columns	# of ext. columns	# of corner columns
5'	W12x26	W10x19	W18x55	W16x40	156	8	24	16	21	20	4

Roof Beams and Girders

Bay Size=20'x20'

Beam Spacing	Interior Beam	Exterior Beam	Interior Girder	Exterior Girder	# of Interi	# of Exterior Beams	# of Interior Girders	# of Exterior Girders	# of int. columns	# of ext. columns	# of corner columns
4'	W10x17	W10x12	W14x30	W12X22	245	10	40	20	36	26	4
5'	W10x17	W10x12	W14x30	W12X22	195	10	40	20	36	26	4
10'	W12x22	W10x17	W14x30	W12X22	95	10	40	20	36	26	4

Roof Beams and Girders

Bay Size=25'x25'

Beam Spacing	Interior Beam	Exterior Beam	Interior Girder	Exterior Girder	# of Interi	# of Exterior Beams	# of Interior Girders	# of Exterior Girders	# of int. columns	# of ext. columns	# of corner columns
5'	W12x26	W10x19	W18x55	W16x40	156	8	24	16	21	20	4

E.2 Total Cost for W-shape Beams

Cost per floor(beams and girders)

Bay Size=20'x20'	Composite	Non Composite
4' spacing	309,900	478,800
5' spacing	264,400	372,600
10' spacing	193,075	281,800
Bay Size=25x25		
5' spacing	360,860	403,900

Cost for Non Composite Columns

Baysize	20'x20'	25'x25'
Composite	170406	139959
Non Composite	168902	135184

Cost per roof(beams and girders)

Bay Size=20'x20'	Composite	Non Composite
4' spacing	309,900	475,100
5' spacing	264,400	335,100
10' spacing	193,075	275,200
Bay Size=25x25		
5' spacing	360,860	401,050

Total Structural Framing Cost(Non Composite Columns)

Bay Size=20'x20'	Composite	Non Composite
4' spacing	1410006	2080402
5' spacing	1228006	1621802
10' spacing	942706	1289502
Bay Size=25x25		
5' spacing	1583399	1747934

Total Structural Framing and slabs Cost(Non Composite Columns)

Bay Size=20'x20'	Composite	Non Composite
4' spacing	1805398.8	2475794.8
5' spacing	1623398.8	2017194.8
10' spacing	1404966.7	1751762.7
Bay Size=25x25		
5' spacing	1979220.2	2143755.2

Cost for 3 floors and roof

Bay Size=20'x20'	Composite	Non Composite
4' spacing	1239600	1911500
5' spacing	1057600	1452900
10' spacing	772300	1120600
Bay Size=25x25		
5' spacing	1443440	1612750

Cost for Composite Columns

Baysize	20'x20'	25'x25'
Composite	147959	126347
Non Composite	147959	119668

Slab and Metal decking cost 3 floors

20x20(Bay)	Slab Depth	decking gauge	total cost
4'	3"	22	395392.8
5'	3"	22	395392.8
10'	5"	19	462260.7
2'	2"	26	265727.2
25x25(Bay)	**Slab Depth**	**decking gauge**	**total cost**
5'	3"	22	395821.2
2.5'	2"	22	106522.5

Total Structural Framing Cost (Composite Columns)

Bay Size=20'x20'	Composite	Non Composite
4' spacing	1387559	1387559
5' spacing	1205559	1205559
10' spacing	920,259	1268559
Bay Size=25x25		
5' spacing	1569787	1732418

Total Structural Framing and slabs Cost (Composite Columns)

Bay Size=20'x20'	Composite	Non Composite
4' spacing	1782951.8	1782951.8
5' spacing	1600951.8	1600951.8
10' spacing	1,382,520	1730819.7
Bay Size=25x25		
5' spacing	1965608.2	2128239.2

E.3 Cost of Open Web Joist

Open Web Joist Floor Beams and Girders									
Bay size=20x20	beam spacing=2'								
int. beams	size	plf	span(ft)	qty.		L.F PER MEMBER		UNIT COST(per l	Total Cost
	12K5	7.1	20	495		9900		11.95	118305
weight(tons)=	35.145								
ext. beams	size	plf	span(ft)	qty.					
	12K1	5	20	10		200		12	2400
weight(tons)=	0.5								
TOTAL BEAMS	35.645								
int. girders	size	plf	span(ft)	qty.					
	W14x61	15	20	40		800		119	95200
weight(tons)=	6								
ext. girders	size	plf	span(ft)	qty.					
	W12x34	16	20	20		400		71	28400
								total=	244305
weight(tons)=	3.2								
total=	44.845					cost beams=	105152.8		
Bay size=25x25	beam spacing=2.5'								
int. beams	size	plf	span(ft)	qty.		L.F PER MEMBER		UNIT COST(per l	Total Cost
	16K7	8.6	25	316		7900		13.7	108230
weight(tons)=	33.97								
ext. beams	size	plf	span(ft)	qty.					
	14K4	6.7	25	8		200		12.3	2460
weight(tons)=	0.67								
TOTAL BEAMS	34.64								
int. girders	size	plf	span(ft)	qty.					
	W21x83	21	25	24		600		159	95400
weight(tons)=	6.3								
ext. girders	size	plf	span(ft)	qty.					
	W18x60	15	25	16		400		118	47200
								total=	253290
weight(tons)=	3								
total=	43.94					cost beams=	102188		

E.4 Cost of Slab

Slab and Decking Cost							
20x20(Bay)	Depth	sqft of slab	sqft of decking	decking gauge	decking co	slab cost(	total cost
4'	3"	19382	19382	22	4.3	2.5	131797.6
5'	3"	19382	19382	22	4.3	2.5	131797.6
10'	5"	19382	19382	19	4.65	3.3	154086.9
2'	2"	19382	19382	26	2.57	2	88575.74
25x25(Bay)	Depth	sqft of slab	sqft of decking	decking gauge	decking co	slab cost(	total cost
5'	3"	19403	19403	22	4.3	2.5	131940.4
2.5'	2"	19403	19403	22	3.49	2	106522.5

20x20(Bay)	Slab Depth	decking gauge	total cost
4'	3"	22	131797.6
5'	3"	22	131797.6
10'	5"	19	154086.9
2'	2"	26	88575.74
25x25(Bay)	Slab Depth	decking gauge	total cost
5'	3"	22	131940.4
2.5'	2"	22	106522.47

E.5 Total Cost for Beams and Girders

Cost per floor(beams and girders)

Bay Size=20'x20'	Composite	Non Composite
4' spacing	309,900	478,800
5' spacing	264,400	372,600
10' spacing	193,075	281,800
Bay Size=25x25		
5' spacing	360,860	403,900

Cost for Non Composite Columns

Baysize	20'x20'	25'x25'
Composite	170406	139959
Non Composite	168902	135184

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Baysize	Colums Noncomposite	Composite	
			244305
20x20	1411849.22	1390906.22	253290
25x25	1254866.5	1239350.5	

Cost per roof(beams and girders)

Bay Size=20'x20'	Composite	Non Composite
4' spacing	309,900	475,100
5' spacing	264,400	335,100
10' spacing	193,075	275,200
Bay Size=25x25		
5' spacing	360,860	401,050

Total Structural Framing Cost(Non Composite Columns)

Bay Size=20'x20'	Composite	Non Composite
4' spacing	1410006	2080402
5' spacing	1228006	1621802
10' spacing	942706	1289502
Bay Size=25x25		
5' spacing	1583399	1747934

Total Structural Framing and slabs Cost(Non Composite Columns)

Bay Size=20'x20'	Composite	Non Composite
4' spacing	1805398.8	2475794.8
5' spacing	1623398.8	2017194.8
10' spacing	1404966.7	1751762.7
Bay Size=25x25		
5' spacing	1979220.2	2143755.2

Cost for 3 floors and roof

Bay Size=20'x20'	Composite	Non Composite
4' spacing	1239600	1911500
5' spacing	1057600	1452900
10' spacing	772300	1120600
Bay Size=25x25		
5' spacing	1443440	1612750

Cost for Composite Columns

Baysize	20'x20'	25'x25'
Composite	147959	126347
Non Composite	147959	119668

Slab and Metal decking cost 3 floors

20x20(Bay)	Slab Depth	decking gauge	total cost
4'	3"	22	395392.8
5'	3"	22	395392.8
10'	5"	19	462260.7
2'	2"	26	265727.2
25x25(Bay)	Slab Depth	decking gauge	total cost
5'	3"	22	395821.2
2.5'	2"	22	106522.5

Total Structural Framing Cost (Composite Columns)

Bay Size=20'x20'	Composite	Non Composite
4' spacing	1387559	1387559
5' spacing	1205559	1205559
10' spacing	920,259	1268559
Bay Size=25x25		
5' spacing	1569787	1732418

Total Structural Framing and slabs Cost (Composite Columns)

Bay Size=20'x20'	Composite	Non Composite
4' spacing	1782951.8	1782951.8
5' spacing	1600951.8	1600951.8
10' spacing	1,382,520	1730819.7
Bay Size=25x25		
5' spacing	1965608.2	2128239.2

E.6 Cost of Columns

Columns Cost									
W6X8.5	W10X15	W6X8.5							
8.5	15	8.5							
60	210	40	Total Feet	1890		68604		Total Conc. Volume	506.5118519
								in cubic yards	
W6X8.5	W6X12	W6X8.5						Price Conc/cubic yards	114
8.5	12	8.5							
60	210	40						Cost Conc	57742.35111
								TOTAL COST CONC & STEEL	**126346.3511**
W6X8.5	W6X8.5	W6X8.5							
8.5	8.5	8.5							
60	210	40							
# Exterior (Beam Side) Columns	**# Interior Columns**	**# Corner Columns**	**Member Size**	**Length (ft)**	**Price/Linear FT**	**Price Steel**	**X-Sect Conc. Area**		**Volume Conc.**
8	36	4	W6X8.5	1980	26.5	52470	57.48		9484.2
			W10X15	432	37	15984	135.59		4881.24
			W6X12	360	32.5	11700	56.45		1693.5
W6X8.5	W10X15	W6X8.5							
8.5	15	8.5							
96	432	48							
W6X8.5	W6X12	W6X8.5							
8.5	12	8.5							
80	360	40	Total Feet	2772		80154		Total Conc. Volume	594.7755556
								in cubic yards	
W6X8.5	W6X8.5	W6X8.5						Price Conc/cubic yards	114
8.5	8.5	8.5							
80	360	40						Cost Conc	67804.41333
								TOTAL COST CONC & STEEL	**147958.4133**
W6X8.5	W6X8.5	W6X8.5							
8.5	8.5	8.5							
80	360	40							

Appendix F: Lateral Loading

F1: Design of Wind Loads

Wind loads on our building were determined using ASCE 7-02 Main Wind Force Resisting Systems Method 1or simplified procedure. Our building meets all the requirements of ASCE 7-02 section 6.4.4.1 for the use of the simplified method. This method was chosen from the International Building Code, 2009 Edition and all values are based on the IBC and ASCE 7-02 as referenced.

Basic Information

- Building located in Worcester, MA
- Four stories with flat roof and overall height of 45ft
- Transverse length of 200ft, Longitudinal length of 100ft
- Office with occupant load under 5000
- No congregation areas of more than 300 people

Step by Step Design Process

1.) Determine values of factors in ASCE equation 6-1, $p=\lambda K_{zt}Ip_{s30}$.
2.) Importance factor I=1.0 from Table 6-1 for wind since our building does not represent a substantial hazard to human life in the event of failure.
3.) Building Exposure Category B since Worcester is an urban area.
4.) Topographic factor K_{zt}=1.0, there are no considerable elevation changes.
5.) Adjustment factor λ=1.12 from Figure 6-2, for mean roof height of 45ft and Exposure B
6.) Average wind speed V=100mph from Figure 6-1 for Worcester
7.) Determine wind pressures on interior and exterior zones of the building using equation $p=\lambda K_{zt}Ip_{s30}$, with values for p_{s30} from figure 6-2.
8.) Determine length "2a" of interior zone
    a. a=0.4xmean roof height or 0.1xleast width of structure

       a=0.1x100=10ftx2=20ft
9.) Determine tributary height for each level
10.) Calculate linear load at each level for interior and exterior zones
    a. Multiply interior and exterior pressures by tributary heights

11.) Determine shear forces at each level by summing moments

F2: Frame Design

A first order analysis was performed to determine the effects of gravity and lateral loads on a rigid frame for our building. The wind loads calculated by following ASCE 7-05 are used for the lateral loads and gravity loads were taken from the design of beams and columns in Chapter 3. AISC Specification H was used in designing the columns for resistance against the factored loads obtained from Risa 2-D analyses.

Basic Information

- Steel Properties: yield strength (F_y) is 50 ksi; modulus of elasticity (E) is 29,000ksi
- Layout: 25'X25' Bay: Beam spacing: 5' O.C.
- All designs followed LRFD method

Step by Step Design Process

1. Determine maximum factored axial loads and moments on column using Risa 2-D software analysis for wind and gravity loads.
2. Determine magnification factor B2
    a. Must determine effective length factor K_2 to find B2
        i. Use AISC nomograph and values G_A and G_B to find K_2
    b. Determine column Euler Buckling strength P_{e2} from AISC equation C2-6a
    c. Calculate B2 using AISC equation C2-3
3. Determine magnification factor B1
    a. Determine column Euler Buckling strength P_{e1} from AISC equation C2-5
    b. Use value P_{e1} and axial load Pu to determine Cm factor from Table 11.1
    c. Calculate required tensile strength Pr using AISC equation C2-1b
    d. Calculate B1 using AISC equation C2-2
4. Determine design Flexural strength Mc
    a. Determine moment capacity of section Φ_bMn from AISC Table 3-2
    b. Check section for governing failure mode
        i. Flange Local Buckling
        ii. Lateral Torsional Buckling

iii. Web Local Buckling

c. Use lowest value for Φ_bMn

5. Determine required flexural strength Mr using AISC equation C2-1a
6. Determine design axial strength Pc from AISC Table 4-1
7. Select interaction equation
   a. If Pr/Pc>0.2 use AISC equation H1-1a
   b. If Pr/Pc<0.2 use AISC equation H1-1b
8. Solve interaction equation and check that value is less than 1.0
   a. If value is greater than 1.0 choose new section and repeat steps 2-8

F4: Braced Frame Spreadsheet

Braced Frame Design	Level 1					
Design Values	W-section	Area(in^2)	Ix(in^4)	ΦbMpx(ft-k)	L(ft)	Cm
	10x39	11.9	209	176	11	1
	Axial(k)	Moment(ft-k)	Lr(ft)	Lp(ft)	Lb(ft)	K
	221	70	35.2	7.35	11	1
Determine interaction Equation						
Pr=Pu	221					
KL=	11					
ΦcPn from AISC Table 4-1=	473					
Pr/Pc=	0.467230444					
Pr/Pc>.02	TRUE	Use eq.H1-1a				
Check section for adequacy						
Pe1=π^2EIx/(KL)^2=	3019.445914					
B1=Cm/(1-αPr/Pe1)=	1.078972404					

Mrx=B1Mr=	75.52806825					
Lp<Lb<Lr	TRUE					
ΦbMpx=	162.7935					
Pr/Pc+8/9(Mrx/Mcx)=	0.879630575					
Section Adequate	TRUE					

F5: Rigid Frame Spreadsheet

Rigid Frame Design	Level 1					
Risa Outputs	Pnt(k)	Plt(k)	Mnt(ft-k)	Mlt(ft-k)		
	241	63	42	376		
Section Properties	Ag(in^2)	Ix(in^4)	ΦMn(ft-k)	Sx(in^3)	L(ft)	ΦcPn(k)
W10x112	32.9	716	551	126	12	1200
Determine B2 and Pr						
K2=	1.3					
Pe2=Π^2EIx/KL^2=	5841.961					
B2=1/1-(Pnt/Pe2)	1.043028					
Pr=Pnt+B2Plt=	306.7108					
Determine B1						
Pe1=Π^2EIx/KL^2=	9872.914					
Cm=1-.4(Pu/Pe1)	0.987683					

B1=	1.01935					
Determine Mcx						
Lateral buckling limits		Lp	Lr	Lb	BF	
		9.47	64.3	12	4.02	
LTB:						
Lp<Lb<Lr	TRUE					
Interpolate for ΦMn						
ΦMn=	540.8294					
FLB:						
λ=	4.17					
λp=	9.15					
ΦMn=	551					
WLB:						
h/tw=	10.4					
h/tw=	90.5					
ΦMn=	551					
Mcx=least of LTB, WLB, FLB=	540.8294					
Determine interaction equation						
Pr/Pc=	0.255592					
Pr/Pc>0.2	TRUE					
if true use eq. H1-1a						
If false use eq. H1-1b						
Mux=B1Mnt+B2Mlt=	434.9914					
Pr/Pc+8/9(Mrx/Mux)=	0.970529					

Pr/Pc+8/9(Mrx/Mux)<1.0	TRUE					

F.6 Hand Calculations

Determine Wind Loads

ASCE 7-06 Simplified Procedure

Determine avg. wind speed - Worcester $V = 100$ mph (Fig. 6-1)

Importance Factor I - Occupancy category II $I = 1.0$ (Table 6-1)

Topographic Factor K_{zt} = assume to be 1

height + exposure Factor λ

mean roof height = 45 ft
Exposure B applies $\lambda = 1.12$ (Fig. 6-2)

roof slope = 0° (Flat roof)

$$P_s = \lambda K_{zt} I p_{s30} = (1.12)(1.0)(1.0) p_{s30} = 1.12 p_{s30}$$

$$\boxed{P_s = 1.12 p_{s30}}$$

Main Wind Force Resisting Systems (MWFRS)

4 Roof Zones, 4 wall Zones

Zone A: $P_{s30} = 15.9$ psf $P_s = (15.9)(1.12) = 17.81$ psf

Zone B: $P_{s30} = -8.2$ psf $P_s = (-8.2)(1.12) = -9.18$ psf

Zone C: $P_{s30} = 10.5$ psf $P_s = (10.5)(1.12) = 11.76$ psf

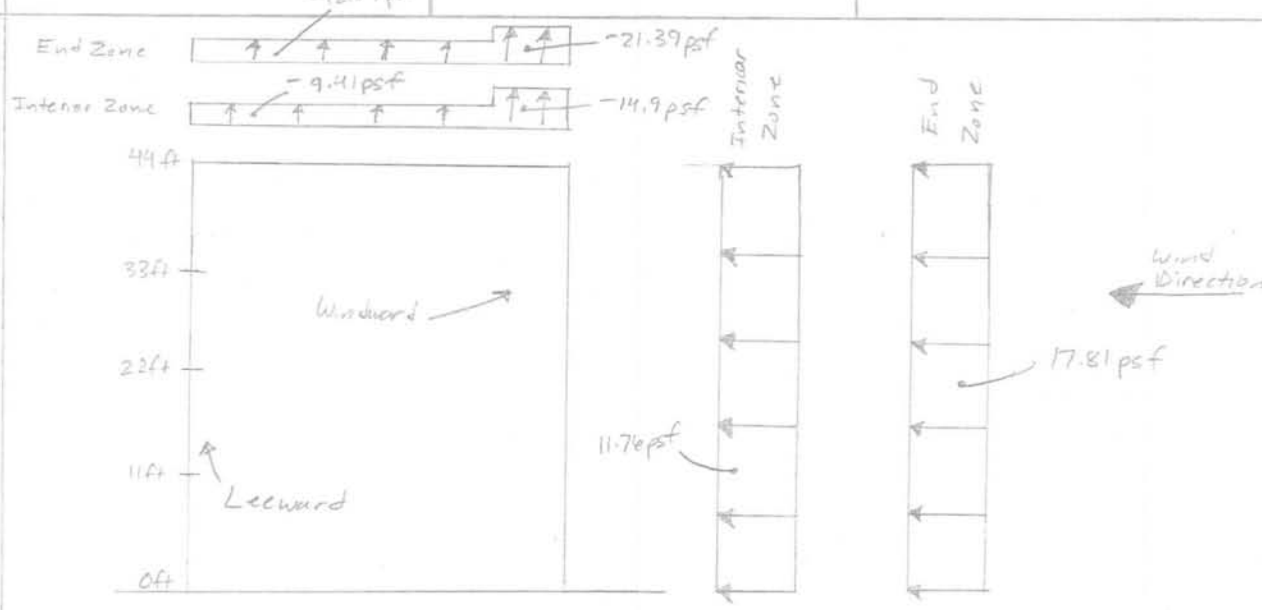
Zone D: $P_{s30} = -4.9$ psf $P_s = (-4.9)(1.12) = -5.49$ psf

Zone E: $P_{s30} = -19.1$ psf $P_s = (-19.1)(1.12) = -21.39$ psf

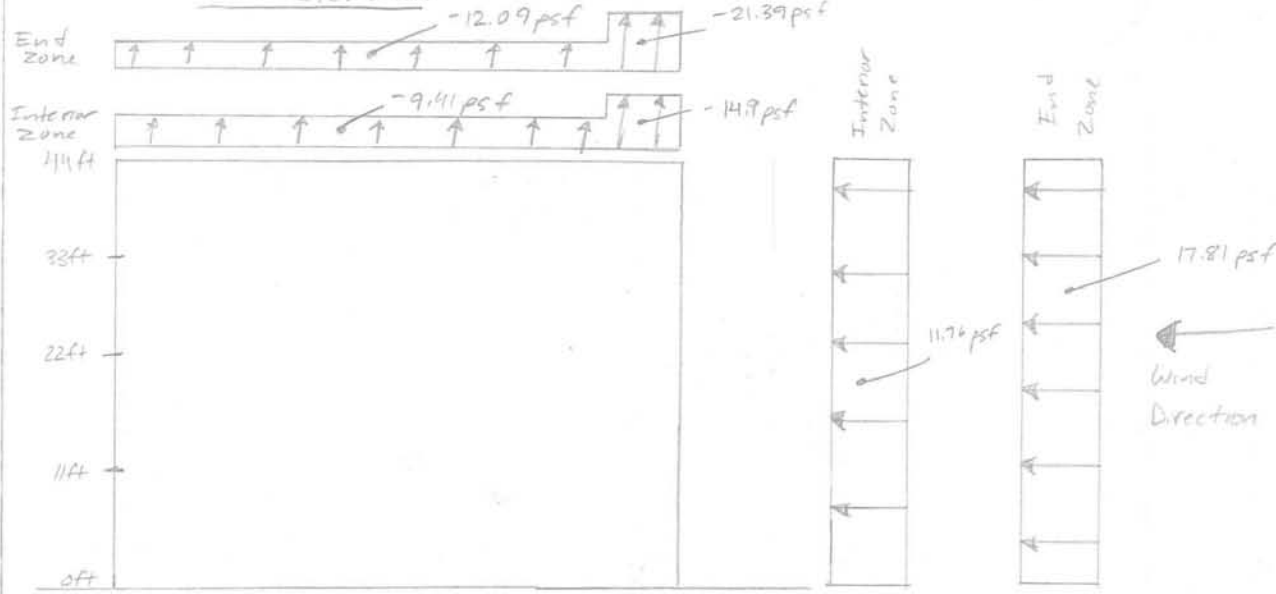
Zone F: $P_{s30} = -10.8$ psf $P_s = (-10.8)(1.12) = -12.09$ psf

Zone G: $P_{s30} = -13.3$ psf $P_s = (-13.3)(1.12) = -14.9$ psf

Zone H: $P_{s30} = -8.4$ psf $P_s = (-8.4)(1.12) = -9.41$ psf



End Elevation



Side Elevation

length of higher end zone pressure

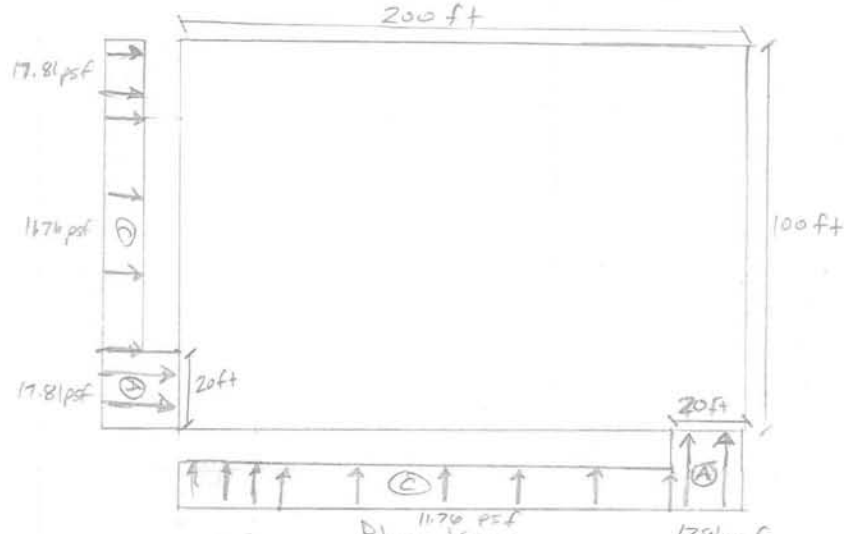
$0.4 h_{mean} = (0.4)(45) = 18 ft$

$0.1 (\text{least width}) = (0.1)(100) = 10 ft$

$a = \text{lesser of two} = 10 ft \qquad 2a = (2)(10) = 20 ft$

Determine shear forces at each level

Determine Shear Forces at each level



Plan View

Tributary Height

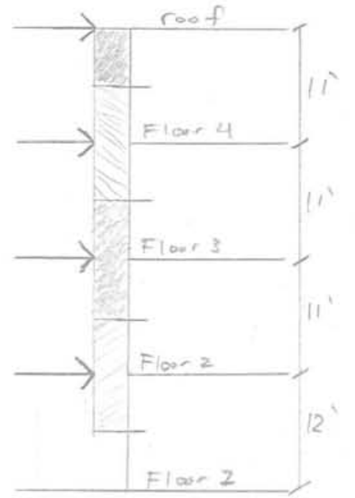
Floor 2 = 6 + 5.5' = 11.5'
Floor 3 = 11'
Floor 4 = 11'
roof = 5.5'

Linear Loads

Floor 2 = (17.81)(11.5) = 204.82 plf
= (11.76)(11.5) = 135.24 plf

Floor 3 = Floor 4
int. = (11.76)(11) = 129.36 plf
ext. = (17.81)(11) = 175.91 plf

Roof
int. = (11.76)(5.5) = 64.68 plf
ext. = (17.81)(5.5) = 97.96 plf



Tributary Wind Loads

Forces at each story

Longitudinal direction - endzone width = 20 ft, Interior zone = 100-20 = 80 ft

wind load = (20)(17.81) + (80)(11.76) = 1.3 klf

at each story = W_L x Tributary height

Floor 2 = (1.3)(12 + 5.5) = 22.75 K

Floor 3 = (1.3)(11) = 14.3 ^{K} Floor 4 = Floor 3

roof = (1.3)(5.5) = 7.2 ^{K}

Transverse direction - endzone width = 20 ft, Interior zone = 200-20 = 180 ft
wind load = (20)(17.81) + (180)(11.76) = 2.5 K

at each story = W_L x Tributary height

Floor 2 = (2.5)(12 + 5.5) = 44 K

Floor 3 = (2.5)(11) = 27.5 K Floor 3 = Floor 4

roof = (2.5)(5.5) = 14 K

Beam - Column Design

Colums -

int = W8x48

ext = W8x31

Corner = W8x24

Column Design for Combined loading (Longitudinal)

W8x48, L = 12 ft, $A = 14.1\,in^2$, $I_x = 184\,in^4$, $\phi_b M_{px} = 184$ ft-k

$\frac{M_{ax}}{\Omega_b} = 122$ ft-k

E = 29,000, Fy = 50,000, rts = 2.35 in, J = 1.96, $h_o = 7.82$, $S_x = 43.2\,in^3$, c = 1.0

$$L_r = (1.95 r_{ts})\left(\frac{E}{.7F_y}\right)\left(\sqrt{\frac{Jc}{S_x h_o}}\right)\sqrt{1+\sqrt{1+6.76(.7F_y/E)(E S_x h_o/Jc)^2}}$$

$$= (1.95)(2.35)\left(\frac{29{,}000}{(.7)(50{,}000)}\right)\left(\sqrt{\frac{(1.96)(1)}{(43.2)(7.82)}}\right)\left(\sqrt{1+\sqrt{1+6.76(.7(50/29)(29)(43.2)(7.82)/1.96)^2}}\right)$$

$L_p = 1.76 r_y \sqrt{E/F_y} = (1.76)(2.08)\sqrt{29/50} = 7.39$ $\quad L_r = 35.2$

Determine. Max moments and axial Loads

Tributary width = 12.5 ft floor, wall = 11 ft

DL = 75psf slab x12.5 = 0.94 klf
4psf floor covering x12.5 = 0.05 klf
5psf MEP x12.5 = 0.063 klf
8psf steel framing x12.5 = 0.1 klf
2psf ceiling x12.5 = 0.025 klf
8psf ext. wall x11 = 0.088 klf
40psf roof deck x12.5 = 0.50 klf

Floor Loads = 1.3 klf $\quad$ roof loads = 0.83 klf

LL = 100psf + 15psf partitions = 115 x 12.5 = 1.4 klf

S = 44psf x 12.5 = 0.55 klf

Wind loads - determined from ASCE 7-06

roof = 7.2^k
Floor 4 = 14.3^k
floor 3 = 14.3^k
floor 2 = 22.8^k

LRFD Load Combinations

① U = 1.2D + 1.6(Lr or S) + (.5L or .8W)

② U = 1.2D + 1.6W + .5L + .5S

③ U = 1.2D + 1.6L + .5 (Lr or S or R)

Determine critical load combination

(1) - $U = 1.2D + 1.6(Lr \text{ or } S) + (.5L \text{ or } .8W) = 1.2(1.3) + 1.6(.55) + .5(1.4) = 3.14$
$+ .8(22.8) = 20.68$

(2) - $U = 1.2D + 1.6W + .5L + .5S = 1.2(1.3) + 1.6(22.8) + .5(1.4) + .5(.55) = 39$

(3) - $U = 1.2D + 1.6L + .5(Lr, S \text{ or } R) = 1.2(1.3) + 1.6(1.4) + .5(.55) = 4$

Load combination (2) governs - Load factors are entered into Risa 2D with wind and distributed loads on Frame.

Max. Load values

Girder - axial = 85 Kips moment = 293 ft-k
Column - axial = 221 Kips moment = 52 ft-k
Brace - axial = 100 kips moment = 10 ft-k

Check column for combined loading

column - W8x31, $A = 9.12 in^2$, $I_x = 110 in^4$, $\phi_b M_{px} = 114$ ft·k
$\frac{Max}{52b} = 75.8$ ft·k $\phi_c P_n = 283$ kips, $L_r = 24.8$ ft, $L_p = 7.12$, $L_b = 12$ ft

$P_r = P_u = 221$ kips, $M_{rx} = M_u = 52$ ft-k

Braced Frame - $K = 1.0$
$KL_x = KL_y = (12)(1) = 12$ ft $P_c = \phi_c P_n = 283$ kips (AISC T4-1)

Check $\frac{P_r}{P_c} = \frac{221}{283} = 0.78 > 0.2$ must use AISC eq. H1-1a

$C_m = 1.0$ (conservative)

assume zero sidesway - $P_{e1} = \frac{\pi^2 E I_x}{(K_x L_x)^2}$

$P_{e1} = \frac{(\pi^2)(29,000)(110)}{(12 \times 12)^2} = 1516.8 = 1517$ kips

$B_1 = \frac{C_m}{1 - \frac{\alpha P_r}{P_{e1}}} = \frac{1.0}{1 - \frac{(1.0)(221)}{1517}} = 1.17$

$M_{rx} = B_1 M_r = (1.17)(52) = 60.84 = 61$ ft-k

$L_p < L_b < L_r$ and $BF = 5.81$ (LRFD)

$\phi_b M_{px} = 1.0[114 - 5.81(12 - 7.12)] = 85.65 = 86$ ft-k eq H1-1a $-\ \frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right)$, $M_{rx} = M_{cy} = 0$ (No translation)

$\frac{221}{283} + \frac{8}{9}\left(\frac{61}{86}\right) = 1.41 > 1.0$ ✗ must choose larger column

<u>use Equivalent axial load method</u>

$P_{ueq} = P_u + M_{ux}m + M_{uy}mu$

$P_u = 221$ kips , $M_{ux} = 52$ ft·k

From T41.2 $m = 1.8$ for $KL = 12ft$ (all shapes)

$P_{ueq} = 221 + (1.8)(52) + 0 = 314.6$

Try W 8×40 $\phi_c P_n = 367$ kips ($A = 11.7 in^2$, $I = 146 in^4$, $\phi_b M_{px} = 149$ ft·k

$L_p = 7.21$, $L_r = 29.9$

$\frac{P_r}{P_c} = \frac{221}{367} = 0.6 > 0.2$ use eq. H1-1a

$C_m = 1.0$

$P_{e1} = \frac{\pi^2 E I_x}{(KL)^2} = \frac{(\pi^2)(29{,}000)(146)}{(12\times12)^2} = 2013.2 = 2013 K$

$B_1 = \frac{C_m}{1 - \frac{\alpha P_r}{P_{e1}}} = \frac{1.0}{1 - \frac{(1.0)(221)}{2013}} = 1.12$

$M_{rx} = B_1 M_r = (1.12)(52) = 58.24$

$L_p < L_b < L_r$ $\phi_b M_{px} = 1.0\,[149 - (5.81)(12 - 7.21)] = 121.17$

$\frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) = \frac{221}{367} + \frac{8}{9}\left(\frac{58.24}{121.17}\right) = 1.03 \approx 1.0$

Use W8×40 since $C_m = 1.0$, still conservative

<u>Design Brace</u>

axial load = 100 kips design for tension and compression

Brace length $= \sqrt{25^2 + 12^2} = 27.75'$ $F_y = 50{,}000$ psi , $\phi_t = 0.9$

In tension - $P_c = P_r = \phi_t F_y A_g$ $A_g = \frac{P_r}{\phi_t F_y} = \frac{100}{(0.9)(50)} = 2.22 in^2$

use W10×12 $\phi_t P_n = 159 k > 100$ ✓

Check compression - $F_y = 50\text{ ksi}$, $E = 29{,}000\text{ ksi}$

$4.71\sqrt{\frac{E}{F_y}} = 113.4$, $L = 27.75'$ W10x12, $r_y = 0.785$, $A_g = 3.54\text{ in}^2$

$$\frac{KL}{r} = \frac{(12)(27.75)}{.785} = 424 > 113.4$$

use $F_{cr} = 0.877 F_e$

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{(\pi^2)(29{,}000)}{(424)^2} = 1.59$$

$F_{cr} = (0.877)(1.59) = 1.39$

$\phi_c P_n = .9 F_{cr} A_g = (0.9)(1.39)(3.54) = 4.42\text{K} < 100\text{K}$ X redesign

try W10x45, $A_g = 13.5\text{ in}^2$, $r_y = 2.15$

$$\frac{KL}{r} = \frac{(12)(27.75)}{2.15} = 155 > 113.4$$

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} = \frac{(\pi^2)(29{,}000)}{(155)^2} = 11.91$$

$F_{cr} = (0.877)(11.91) = 10.45$

$\phi_c P_n = (0.9)(10.45)(13.5) = 127\text{K} > 100\text{K}$ OK.

$\boxed{\text{Brace - use W10x45}}$

Rigid Frame - Transverse wind loading
wind loads from ASCE 7-06

roof = 14 K
Floor 3 and 4 = 27.5 K
Floor 2 = 44 K

From Braced Frame Design - D = 1.3 klf (Floors)
= .83 klf (roof)
LL = 1.4 klf
S = 0.55 klf

Governing Load combination - $U = 1.2D + 1.6W + .5L + .5S$

Loads entered into Risa to determine axial loads and moments (int. column)

$P_{nt} = 241\,K$ $\quad M_{nt} = 42$ ft.K

$P_{lt} = 63\,K$ $\quad M_{lt} = 376$ ft.K

Find B_1 and B_2 $\quad$ W8x48, $A_g = 14.1\,in^2$, $I_x = 184\,in^4$, $r_y = 2.10\,in$

$$C_m = 1 - 0.4\left(\frac{P_u}{P_{e1}}\right), \quad P_{e1} = \frac{\pi^2 EI}{(KL)^2} = \frac{(\pi^2)(29000)(184)}{(12\times12)^2} = 2537$$

$$C_m = 1 - 0.4\left(\frac{304}{2537}\right) = 0.95$$

$$B_1 = \frac{C_m}{1 - \frac{\alpha P_r}{P_{e1}}} = \frac{0.95}{1 - \frac{(1.0)(316)}{2537}} = 1.09$$

Find K_2 - $G_A = 1.0$ (Fixed support)

$$G_B = \frac{\Sigma I_c/L_c}{\Sigma I_g/L_g} = \frac{\frac{184}{12} + \frac{184}{11}}{\frac{984}{25} + \frac{984}{25}} = 0.42, \quad K = 1.3$$

$$P_{e2} = \frac{\pi^2 EI}{(K_2L)^2} = \frac{(\pi^2)(29{,}000)(184)}{(1.3\times12\times12)^2} = 1501$$

$$B_2 = \frac{1}{1 - \frac{\Sigma P_{nt}}{\Sigma P_{e2}}} = \frac{1}{1 - \frac{241}{1501}} = 1.19$$

$$P_r = P_{nt} + B_2 P_{lt} = 241 + 1.19(63) = 316\,K$$

Determine M_{cx}

Check FLB, LTB, WLB $\quad L_p = 7.35$, $L_r = 35.2$, $L_B = 12$

$L_p < L_B < L_r$ $\quad \phi M_p = 184$ ft.K, $F_y = 50$ksi, $S_x = 43.2\,in^3$

LTB: interpolate $\quad M_n = C_b\left[M_p - (M_p - .7F_yS_x)\left(\frac{L_B - L_p}{L_r - L_p}\right)\right]$

$$M_n = 1.0\left[184 - (184 - .7(50)(43.2))\left(\frac{12 - 7.35}{35.2 - 7.35}\right)\right] = 115.55$$

FLB: $\quad \lambda_p = .38\sqrt{\frac{E}{F_y}} = 9.15$, $\lambda = 5.92 \therefore \phi M_n = 184$ ft.K

WLB: $\quad \frac{h}{t_w} = 15.9 \le 90.5 \therefore \phi M_n = 184$ ft.K

$$\boxed{M_{cx} = 115.55 \text{ ft.K}}$$

$M_{ux} = B_1 M_{nt} + B_2 M_{Lt} = (1.09)(42) + (1.19)(376) = 493$ ft·k

$M_{uy} = 0$

$P_r = P_{nt} + B_2 P_{Lt} = (241) + (1.19)(63) = 316$ k

$P_c = \phi_c P_n = 447$ k

$\frac{P_r}{P_c} = \frac{316}{447} = 0.71 > 0.2$ use eq. H1-1a

$\frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_{rx}}{M_{cx}} + \cancel{\frac{M_{ry}}{M_{cy}}}\right) \le 1.0 \quad 0.71 + \frac{8}{9}\left(\frac{493}{115.55}\right) = 4.49 \not\le 1.0$

Choose new section

$P_{ueq} = P_u + M_{ux} m + M_{uy} m u = 304 + (1.6)(418) = 973$ k

try W 10 x 112, $I_x = 716$, $\phi M_n = 551$ ft-k, $L = 11$, $\phi_c P_n = 1200$ k

$L_p = 9.47$, $L_r = 64.3$

$$P_{e_1} = \frac{\pi^2 EI}{(KL)^2} = \frac{(\pi^2)(29{,}000)(716)}{(11 \times 11)^2} = 11749.58$$

$$C_m = 1 - 0.4\left(\frac{P_u}{P_{e_1}}\right) = 1 - 0.4\left(\frac{304}{11749.58}\right) = 0.98$$

$$B_1 = \frac{C_m}{1 - \frac{\alpha P_r}{P_{e_1}}} = \frac{0.98}{1 - \frac{(1.0)(306.26)}{11749.58}} = 1.02$$

$K_2 = 1.3$

$$P_{e_2} = \frac{\pi^2 EI}{(K_2 L)^2} = \frac{(\pi^2)(29{,}000)(716)}{(1.3)(12)(12)^2} = 6952.41$$

$$B_2 = \frac{1}{1 - \frac{\Sigma P_{nt}}{\Sigma P_{e_2}}} = \frac{1}{1 - \frac{241}{6952.41}} = 1.04$$

$P_r = P_{nt} + B_2 P_{Lt} = 241 + (1.04)(63) = 306.26$

Determine M_{cx} - check FLB, LTB, WLB

LTB: $L_p < L_B < L_r$ so interpolate ϕM_n

$M_n = \phi M_n - BF(L_B - L_p) = 551 - 4.02(12 - 9.47) = 540.82$

FLB: $\lambda_p = 9.15$, $\lambda = 4.17$ $\therefore$ $\phi M_n = 551$ ft·k

WLB: $\frac{h}{t_w} = 10.4 < 90.5$ $\therefore$ $\phi M_n = 551$ ft·k

Use lowest value $\phi M_n = 540.82$ ft·k

$M_{ux} = B_1 M_{nt} + B_2 M_{lt} = (1.02)(42) + (1.04)(376) = 432.17$ ft·k

$P_r = 306.26$k, $\phi_c P_n = 1200$k

$$\frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \le 1.0$$

$$\frac{306.26}{1200} + \frac{8}{9}\left(\frac{432.17}{540.82}\right) = 0.96 \le 1.0 \checkmark$$

$\boxed{\text{Use W10x112}}$

Appendix G: Code Analyses

G1: Code analysis (without sprinkler system)

General:

- Occupancy classification: Group B (section 304.1)
- Construction type: Type I and II

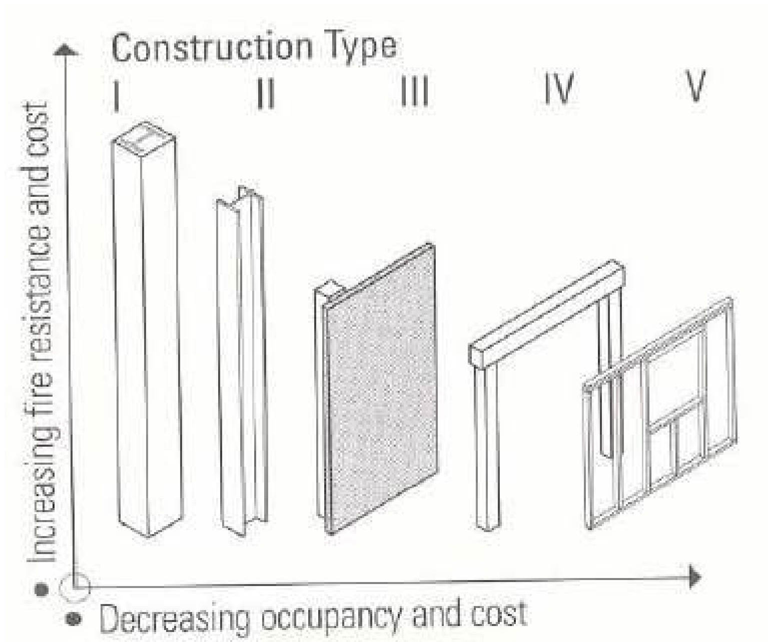


Figure 88 Construction Type

Figure 89 Type II with Protection (FP570-module 7)

Figure 90 Type II without Protection (FP570-module 7)

Table 29 Equivalent Construction Type between Different Codes (Table 19.12, NFPA Fire Protection Handbook, 2008)

TABLE 19.1.2 Comparison of Construction Types (Based on the MCSC National System)

NFPA 220	I (443)	I (332)	II (222)	II (111)	II (000)	III (211)	III (200)	IV (2HH)	V (111)	V (000)
UBC	—	I FR	II FR	II 1-hr	II N	III 1-hr	III N	IV HT	V 1-hr	V-N
BNBC	1A	1B	2A	2B	2C	3A	3B	4	5A	5B
SBC	I	II	—	IV 1-hr	IV unp	V 1-hr	V unp	III	VI 1-hr	VI unp
IBC	—	IA	IB	IIA	IIB	IIIA	IIIB	IVHT	VA	VB

(NFPA FPH pp19-4)

- Height / area limitation: For Group B, 5 stories above grade, 65 ft max. 37500 ft^2 per floor for Type II A construction. (Table 503) 3 stories above the grade with maximum 55 ft height and 23000 ft^2 per floor for Type II B construction. Since our building is 4-story, 42 ft high Group B. The structure members have to be at least Type II A.
- Fire Resistant Rating based on Construction Type. (Based on Table 601, and Table 602)

Table 30 Fire Resistant Rating in Hours (Table 601 and 602)

BUILDING ELEMENT	**TYPE I**		**TYPE II**		**TYPE III**	
	A	**B**	**A[d]**	**B**	**A[d]**	**B**
Primary structural frame (Columns, girders, trusses, etc.)	3[a]	2[a]	1	0	1	0
Bearing walls Exterior Interior	3 3[a]	2 2[a]	1 1	0 0	2 1	2 0
Nonbearing walls and partitions Exterior	0 (Assume the fire separation distance > 30ft)					
Nonbearing walls and partitions Interior	0	0	0	0	0	0
Floor construction and secondary members (supporting beams and joists)	2	2	1	0	1	0
Roof construction and secondary	$1^{1}/_{2}$[b]	1[b,c]	1[b,c]	0[c]	1[b,c]	0

members (supporting beams and joists)						

In order to meet the 1 hour rating requirements, several tables in IBC can guide us:

Use IBC Equation 7-12 in order to determine the fire-resistance rating of a

steel and gypsum board assembly. (IBC Section 721.5.1.2)

Use IBC Equation 7-13 in order to determine the fire-resistance rating of

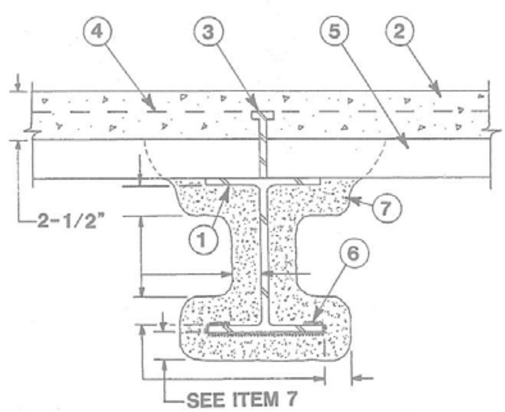
spray-applied fire resistance materials. (IBC Section 721.5.1.3)

Fire Resistance – Volume 1 (determine the thickness of fireproofing spray for 1 hour rating)

Based on Design No. N715, Spray-Applied Fire Resistive Materials shall be 7/16 in for girders

Design No. N715

Restrained Beam Ratings — 1, 1-1/2, 2, 3 and 4 Hr. (See Item 7)
Unrestrained Beam Ratings — 1, 1-1/2, 2, 3 and 4 Hr. (See Item 7)
Load Restricted for Canadian Applications — See Guide BXUV7

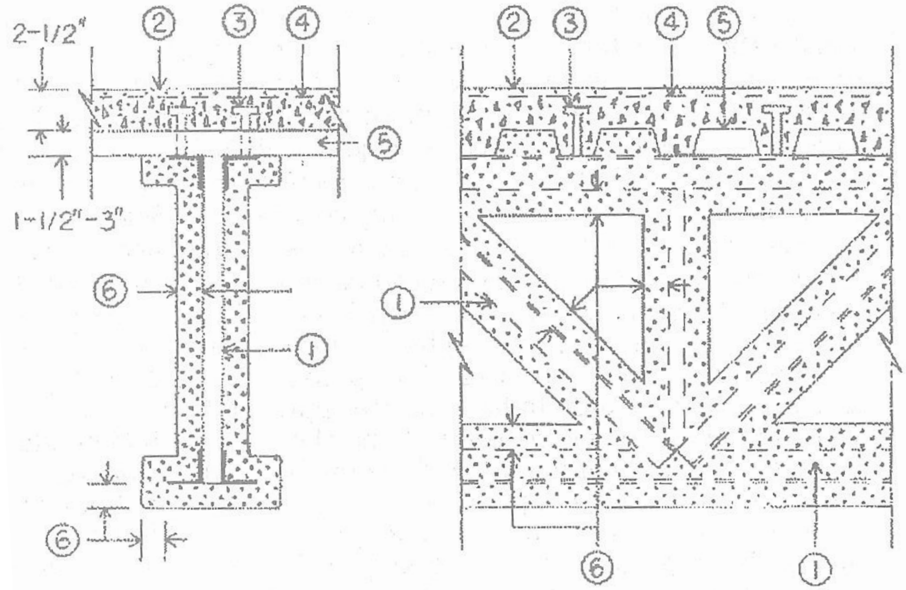


	Min Thkns In.	
Rating Hr	Restrained Beam Rating Hr	Unrestrained Beam Rating Hr
1	7/16	7/16
1-1/2	1/2	1/2
2	3/4	13/16
3	1-1/8	1-1/4
4	1-9/16	1-13/16

Based on Design No. N789, Spray-Applied Fire Resistive Materials shall be 1-1/16 in for K series Open web joists.

Design No. N789

Restrained Beam Ratings - 1, 1-1/2, 2, 3 or 4 Hr. (See Item 7)
Unrestrained Beam Ratings - 1, 1-1/2, 2, 3 or 4 Hr. (See Item 7)

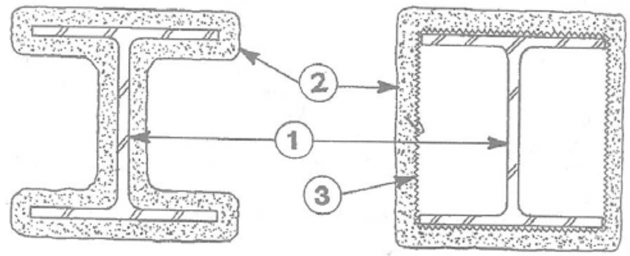


Restrained & Unrestrained Beam Rating, Hr	Min Thkns w/Fluted Deck, In.	Min Thkns w/ Cellular Deck, In.
1	1-1/16	1-1/16
1-1/2	1-3/8	1-3/8
2	1-5/8	1-5/8
3	2-1/4	2-7/16
4	2-13/16	3-1/4

Based on Design No. X789, Spray-Applied Fire Resistive materials shall be 1 in for columns

Design No. X798

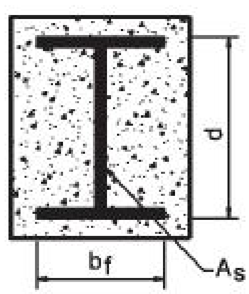
Ratings — 1, 1-1/2, 2, 3, and 4 Hr.



1. **Steel Column** — Wide flange steel column of min sizes as shown in the table below.
2. **Spray-Applied Fire Resistive Materials*** — Applied by mixing with water and spraying in more than one coat to the thickness shown below, to steel surfaces which are clean and free of dirt, loose scale and oil. Min avg and min ind density of 18/16 pcf for Type CP-2 and 23/21 pcf for Type P-20, respectively. For method of density determination, see Design Information Section, Sprayed Material.

Min Column Size	Min W/D	Min Thk In. 1 Hr	1-1/2 Hr	2 Hr	3 Hr	4 Hr
W6x9	0.33	1-1/16	1-3/8	1-11/16	2-1/4	2-7/8
W6x12	0.43	1	1-5/16	1-5/8	2-3/16	2-3/4
W6x16	0.57	1	1-1/4	1-9/16	2-1/8	2-3/4
W8x28	0.67	13/16	1-1/8	1-7/16	2-1/16	2-11/16
W10x49	0.83	5/8	7/8	1-3/16	1-13/16	2-7/16
W12x106	1.46	7/16	11/16	15/16	1-7/16	2
W14x233	2.52	1/4	7/16	5/8	15/16	1-1/4
W14x730	6.62	1/4	1/4	1/4	7/16	9/16

For composite columns (concrete encased steel W shape column)



(c)
CONCRETE
ENCASED
WIDE-FLANGE
SHAPE

Table 31 Minimum Cover (inch) for Steel Columns Encased in Normal-weight Concrete (Table 721.5.1(7))

STRUCTURAL SHAPE	FIRE-RESISTANCE RATING (hours)				
	1	$1^1/_2$	2	3	4

<table>
<tr><td>W14 × 233</td><td rowspan="7">1</td><td rowspan="5">1</td><td rowspan="3">1</td><td rowspan="2">$1\frac{1}{2}$</td><td>2</td></tr>
<tr><td>× 176</td><td rowspan="3">$2\frac{1}{2}$</td></tr>
<tr><td>× 132</td><td rowspan="3">2</td></tr>
<tr><td>× 90</td><td rowspan="4">$1\frac{1}{2}$</td></tr>
<tr><td>× 61</td><td rowspan="3">3</td></tr>
<tr><td>× 48</td><td rowspan="2">$1\frac{1}{2}$</td><td rowspan="2">$2\frac{1}{2}$</td></tr>
<tr><td>× 43</td></tr>
<tr><td>W12 × 152</td><td rowspan="5">1</td><td rowspan="2">1</td><td rowspan="2">1</td><td rowspan="3">2</td><td rowspan="2">$2\frac{1}{2}$</td></tr>
<tr><td>× 96</td></tr>
<tr><td>× 65</td><td rowspan="3">$1\frac{1}{2}$</td><td rowspan="3">$1\frac{1}{2}$</td><td rowspan="3">3</td></tr>
<tr><td>× 50</td><td rowspan="2">$2\frac{1}{2}$</td></tr>
<tr><td>× 40</td></tr>
<tr><td>W10 × 88</td><td>1</td><td rowspan="5">$1\frac{1}{2}$</td><td rowspan="4">$1\frac{1}{2}$</td><td>2</td><td rowspan="3">3</td></tr>
<tr><td>× 49</td><td rowspan="4">1</td><td rowspan="4">$2\frac{1}{2}$</td></tr>
<tr><td>× 45</td></tr>
<tr><td>× 39</td><td rowspan="2">$3\frac{1}{2}$</td></tr>
<tr><td>× 33</td><td>2</td></tr>
<tr><td>W8 × 67</td><td rowspan="6">1</td><td rowspan="2">1</td><td rowspan="3">$1\frac{1}{2}$</td><td rowspan="3">$2\frac{1}{2}$</td><td rowspan="2">3</td></tr>
<tr><td>× 58</td></tr>
<tr><td>× 48</td><td rowspan="4">$1\frac{1}{2}$</td><td rowspan="3">$3\frac{1}{2}$</td></tr>
<tr><td>× 31</td><td rowspan="3">2</td><td rowspan="3">3</td></tr>
<tr><td>× 21</td></tr>
<tr><td>× 18</td><td>4</td></tr>
<tr><td>W6 × 25</td><td rowspan="2">1</td><td>$1\frac{1}{2}$</td><td>2</td><td rowspan="2">3</td><td>$3\frac{1}{2}$</td></tr>
<tr><td>× 20</td><td>2</td><td>$2\frac{1}{2}$</td><td>4</td></tr>
</table>

× 16				$3^1/_2$	
× 15	$1^1/_2$				
× 9					

In our design, all composite columns had a minimum thickness of 1.5 in. Therefore, all columns had the fire-resistance rating of at least 1 hour.

Means of Egress

- Doors
  - Size of doors: clear width of 32in min (section 1008.1.1)
  - Opening force: other than fire door, not exceed 5 pounds 1008.1.3
  - Fire Doors: fire doors are required to have a minimum fire protection rating of 20 minutes where located in corridor walls or smoke barrier walls having a fire-resistance rating in accordance with Table 715.4 . Based on section 715.4.4, doors in exit enclosures and exit passageways shall have a maximum transmitted temperature end point of not more than 450 F above the ambient at the end of 30 minutes of standard fire test exposure. According to section 715.4.8, all fire doors in this office building shall be self- or automatic-closing.

Table 32 FIRE DOOR AND FIRE SHUTTER FIRE PROTECTION RATINGS (Table 715.4)

TYPE OF ASSEMBLY	REQUIRED ASSEMBLY RATING (hours)	MINIMUM FIRE DOOR AND FIRE SHUTTER ASSEMBLY RATING (hours)
Fire walls and fire barriers having a required fire-resistance rating greater than 1 hour	4 3 2 $1^1/_2$	3 3[a] $1^1/_2$ $1^1/_2$
Fire barriers having a required fire-resistance rating of 1 hour:	1	1

Shaft, exit enclosure and exit passageway walls Other fire barriers	1	$^{3}/_{4}$
Fire partitions: Corridor walls Other fire partitions	1 0.5 1 0.5	$^{1}/_{3}$ b $^{1}/_{3}$ b $^{3}/_{4}$ $^{1}/_{3}$
Exterior walls	3 2 1	$1^{1}/_{2}$ $1^{1}/_{2}$ $^{3}/_{4}$
Smoke barriers	1	$^{1}/_{3}$ b

- Stairs
    - Width: 44 inches min. 1009.1
    - Headroom clearance 80 inches (measured vertically from a line connecting gate edge of the nosing) 1009.2
    - Riser 4 to 7 inches. Tread depth:11 min. section 1009.4.2
    - Landings: 48in max. (same in floor plan) section 1009.5
- Occupant load: Based on Table 1004.1.1 where for Business, gross floor area per occupant is 100ft^2; Also based on section 1004.7 for fixed seating or in our project workstation and tables.
    - 1st floor: total occupant load is 238 in total instead of 200 due to the fixed seating (work station)
    - 2nd ~4th floor : total occupant load is 243 in total instead of 200 due to the fixed seating (work station)

- Number of Exits: Based on Table 1021.1, 2 exits are required for each story (1~500 occupant load). For 2nd to 4th floor, 3 stairs served as exits, and on the first floor 3 exits doors are provided (including the main entrance, 2 exit doors at stair shaft). Therefore, the design is in compliance with the code.

- Exit Access travel distance: based on section 1016.1, the maximum for non-sprinklered building is 200ft. In our floor plan, the maximum travel distance is 130 ft which is in compliance with the code.
- Common Path of Travel: maximum is 75 ft. in our design, it is 50ft. (section 1014.3)

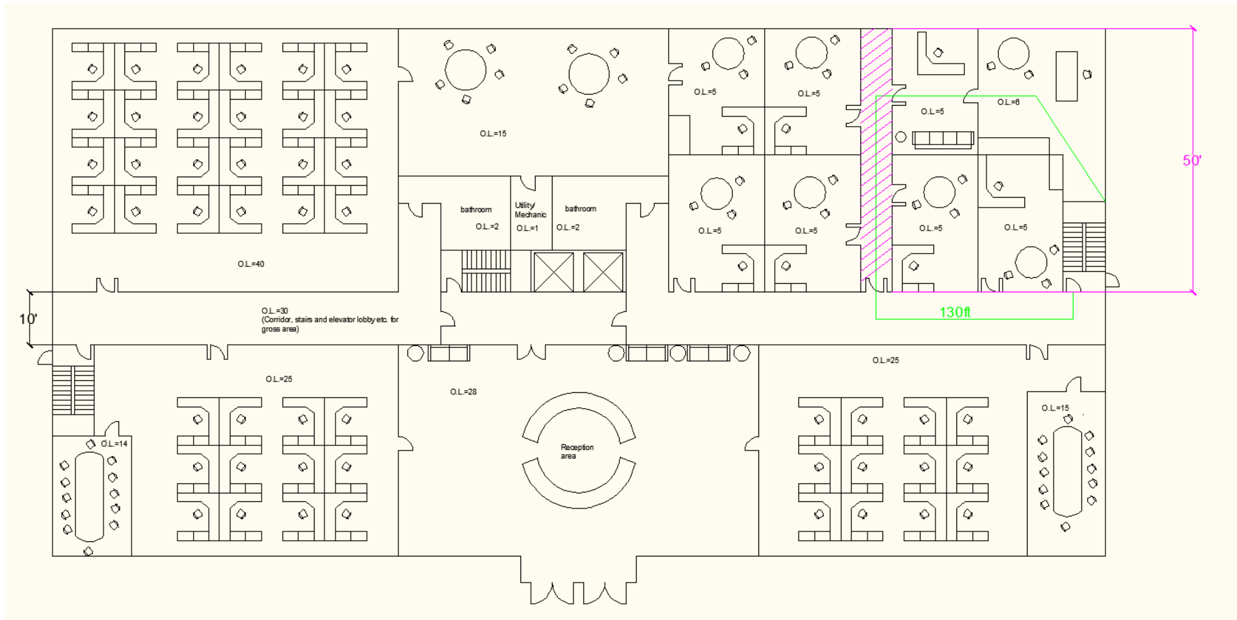


Figure 91. 1st Floor Plan (Green line stands for maximum travel distance; pink hatch stands for the maximum common path of travel; O.L. is occupant load)

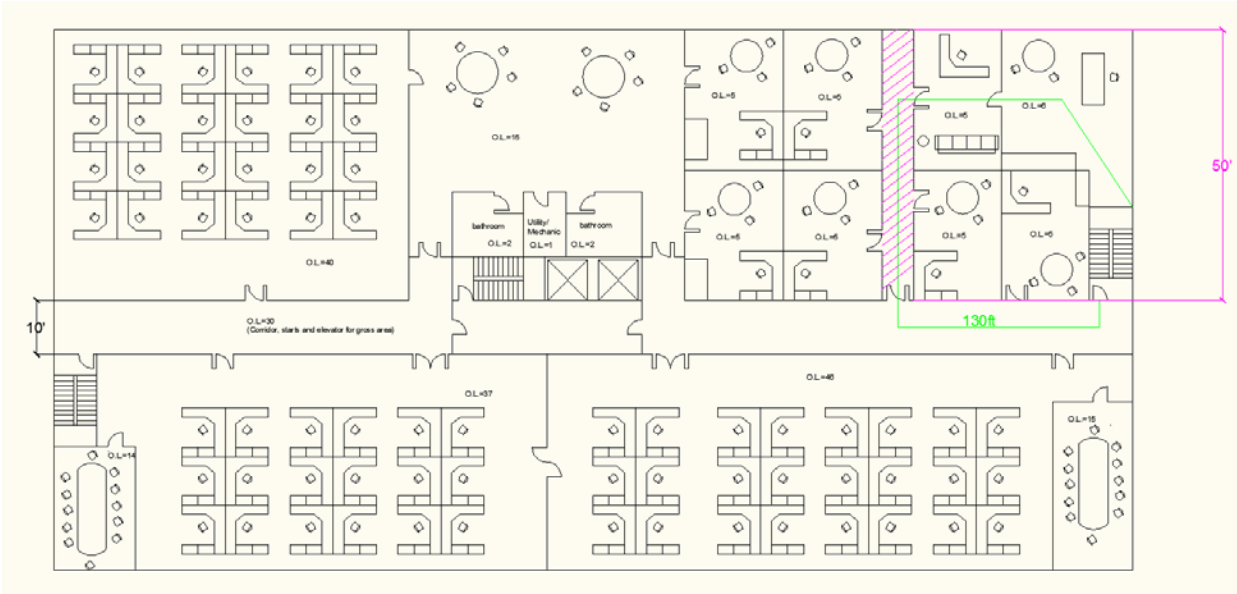


Figure 92. 2nd to 4th Floor Plan (Green line stands for maximum travel distance; pink hatch stands for the maximum common path of travel; O.L. is occupant load)

- Exits Remoteness: based on section 1015.2.1, the minimum distance is ½ of maximum diagonal distance of the room for non-sprinklered area, while it reduced to 1/3 for sprinklered building. And where exit enclosures are provided as a portion of the required exit and are interconnected by a 1-hour fire-resistance rated corridor, the required exit separation shall be measured along the shortest direct line of travel within the corridor.

  In our design, all of these requirements are met.
- Corridors:
  - Dead end corridor: section 1018.4 indicates that the maximum length is 20ft while if the building is protected with sprinkler, it can be 50 ft.

    In our second attempt, there is no dead end corridor.
  - Minimum width 44 in per section 1018.2. In our design, the width is 10 ft.
  - Obstruction: no obstruction at all per section 1018.3. Where in second attempt, doors were all fixed to meet this requirements.
  - The fire doors which are also automatic-closing doors installed across a corridor shall be automatic-closing by the actuation of smoke detectors installed accordance with section 907.3 or by loss of power to the smoke detector or hold-open device. Automatic closing

by smoke detection shall not have more than a 10-second delay before the door starts to close after the smoke detector is actuated. (section 715.4.8.3)

In the floor plan, the smoke-activated doors are the doors at elevator lobby.

- Elevator lobby: according to section 708.14.1, it is required to have enclosed elevator lobby at each floor since the elevator shaft connects more than three stories. And the lobby enclosure shall separate the elevator shaft enclosure doors from each floor by fire partitions. In addition to the requirements in Section 709 for *fire partitions*, doors protecting openings in the elevator lobby enclosure walls shall also comply with Section 715.4.3 as required for *corridor* walls and penetrations of the elevator lobby enclosure by ducts and air transfer openings shall be protected as required for *corridors* in accordance with Section 716.5.4.1. Elevator lobbies shall have at least one *means of egress* complying with Chapter 10 and other provisions within this code.
- Illumination of means of egress: The means of egress, shall be illuminated at all times (corridor, stair enclosure) with not less than 1 foot-candle (11 lux) at the walking surface (section 1006.1 and 1006.2)
- Exit signs:

  Exits are required be marked by an approved exit sign readily visible from any direction of egress travel. Exit sign placement shall be such that no point in an exit access corridor is more than 100 ft from the nearest visible exit sign. It is required to be illuminated at all times,and extra power source shall be provided to ensure continued illumination for a duration of not less than 90 minutes. (section 1011)

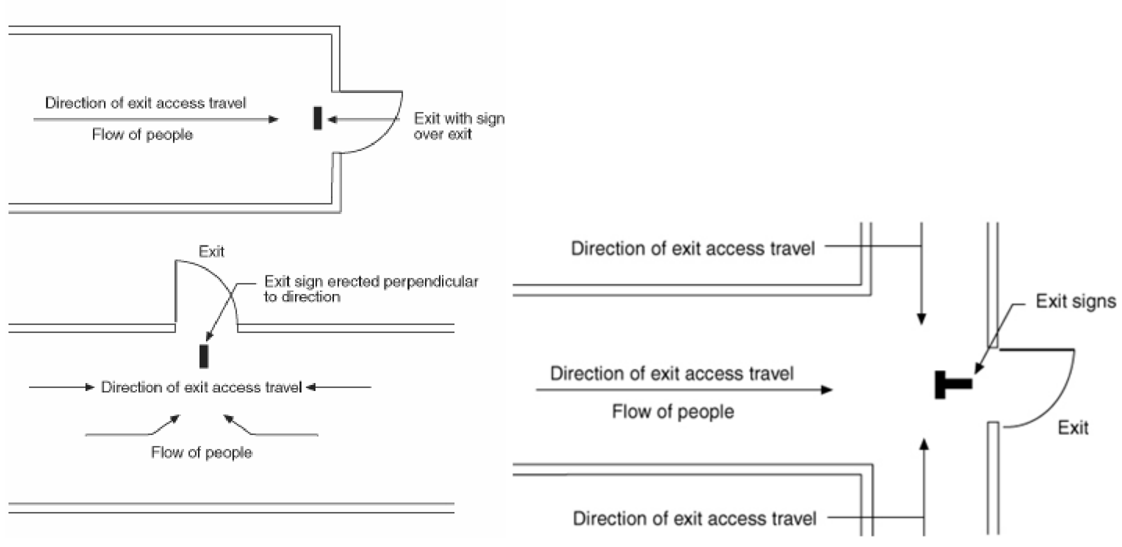


Figure 93 Location of Exit Signs (Figure A.7.10.1.2.1, NFPA 101)

- Area of Refuge: Where an elevator lobby is used as an *area of refuge*, the shaft and lobby shall comply with Section 1022.9 for *smokeproof enclosures* except where the elevators are in an *area of refuge* formed by a *horizontal exit* or *smoke barrier*.(section 1007.6) The elevator can be treated as a area of refuge. But it is not required.

Without sprinkler systems.

Passive Fire Protection: the fire-resistance of building elements gives occupants time to leave the building in the event of an emergency and allow fire fighters time to fight the fire by delaying structural failure

- Fire barriers:
  - Shaft enclosure (elevator shaft) section 708.4 : not less than 2 hours where connecting four stories or more.
  - Exit enclosures (stair enclosure) section 1022.1: not less than 2 hours where connecting four stories or more.
  - Corridor walls with 1hour rating (Table 1018.1)
  - Incidental accessory occupancies Table 508.4

- Continuity of fire barriers: shall extend from the top of the floor/ceiling assembly below to the underside of the floor or roof sheeting, slab or deck above and shall be securely attached there to. (Section 707.5)
- Opening protection: in accordance with section 715: 20min for corridor doors; openings in exit enclosures and shall comply with section 1022.3, 715.4:1 ½ hour; fire door assemblies in exit enclosure shall have an maximum transmitted temperature end point of not more than 450 ˚F . All fire doors shall be self or automatic-closing (section 715.4.8). doors are limited to a max. aggregate width of 25 percent of the length of the wall, and not exceed 156 ft^2(section 707.6) .
- Penetrations: comply with section 713.
- Ducts and air transfer openings : comply with section 716

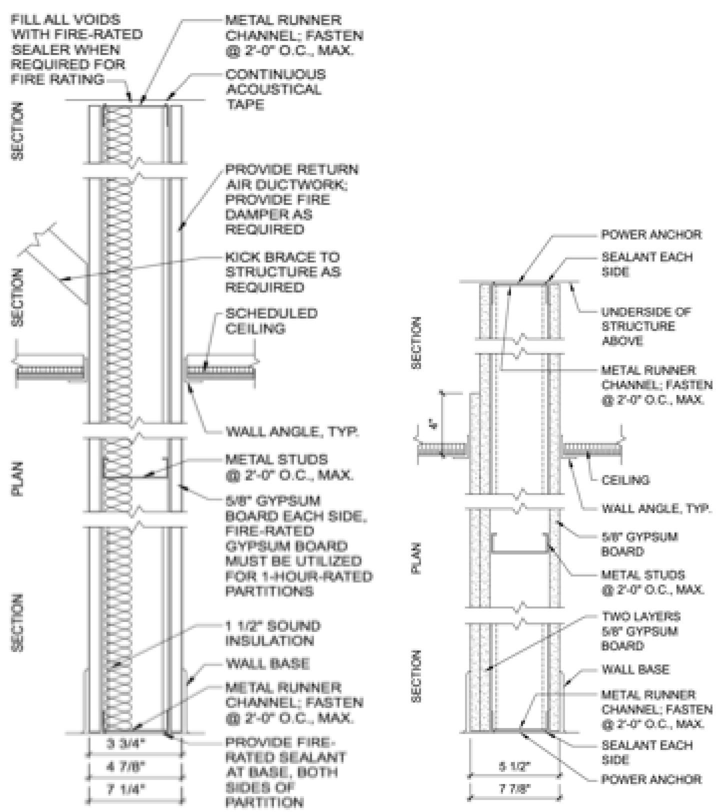


Figure 94 Samples of One-Hour Fire Rated Wall Construction (FPE 570, module 11)

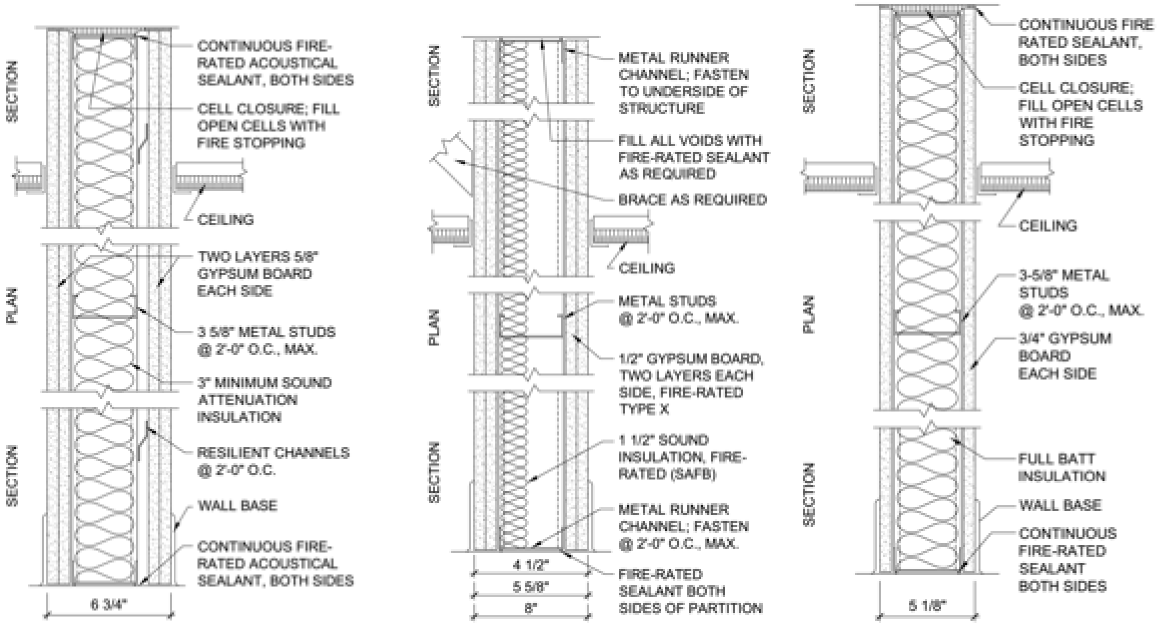


Figure 95 Samples of Two-Hour Fire-Rated Wall Construction (FPE 570, module 11)

Figure 96 Illustration of Vertical Continuity of Fire Barriers

- Smoke barriers:
  - o 1 hour fire-resistance is required (section 710.3)
  - o There is no required smoke barrier or partition for Group B building. However, it will be good to has exit corridor constructed as smoke barrier.
- Capacity of exits: In preliminary code analysis

- Arrangement of means of egress: In preliminary code analysis
- Protection for Floor or Roof Slab System: Table 720.1(3). 5.1 in thick lightweight concrete is equivalent to 4 hours fire-resistance rating and 3 hour for 4.4 in, 2 hour for 3.6 in. In our design the thick is 3 in (except 5" in 10'ft spacing)
- Interior wall and ceiling finish

Table 33 Interior Wall and Ceiling Finish Requirement by Occupancy (Table 803.9)

GROUP	SPRINKLERED[l]			NONSPRINKLERED		
	Exit enclosures and exit passageways[a, b]	**Corridors**	**Rooms and enclosed spaces[c]**	**Exit enclosures and exit passageways[a, b]**	**Corridors**	**Rooms and enclosed spaces[c]**
A-1 & A-2	B	B	C	A	A[d]	B[e]
A-3[f], A-4, A-5	B	B	C	A	A[d]	C
B, E, M, R-1	B	C	C	A	B	C
R-4	B	C	C	A	B	B
F	C	C	C	B	C	C
H	B	B	C[g]	A	A	B
I-1	B	C	C	A	B	B
I-2	B	B	B[h, i]	A	A	B
I-3	A	A[j]	C	A	A	B
I-4	B	B	B[h, i]	A	A	B
R-2	C	C	C	B	B	C
R-3	C	C	C	C	C	C
S	C	C	C	B	B	C
U	No restrictions			No restrictions		

Table 34 Classification of Wall and Ceiling Finish (Section 803.1.1)

Class A:	Flame spread index 0-25; smoke-developed index 0-450.
Class B:	Flame spread index 26-75; smoke-developed index 0-450.
Class C:	Flame spread index 76-200; smoke-developed index 0-450.

Therefore, based on Table 803.9, Corridors shall have Class B wall and ceiling finish, Other rooms shall have Class C wall and ceiling finish.

- Interior floor finish

  Interior floor finish and floor covering materials in exit passageways and corridors shall not be less than Class II . In all areas, floor covering materials shall comply with DOCFF-1 “pill test”. (section 804.4.1)

  804.2 Classification. *Interior floor finish* and floor covering materials required by Section 804.4.1 to be of Class I or II materials shall be classified in accordance with NFPA 253. The classification referred to herein corresponds to the classifications determined by NFPA 253 as follows: Class I, 0.45 watts/cm^2 or greater; Class II, 0.22 watts/cm^2 or greater.

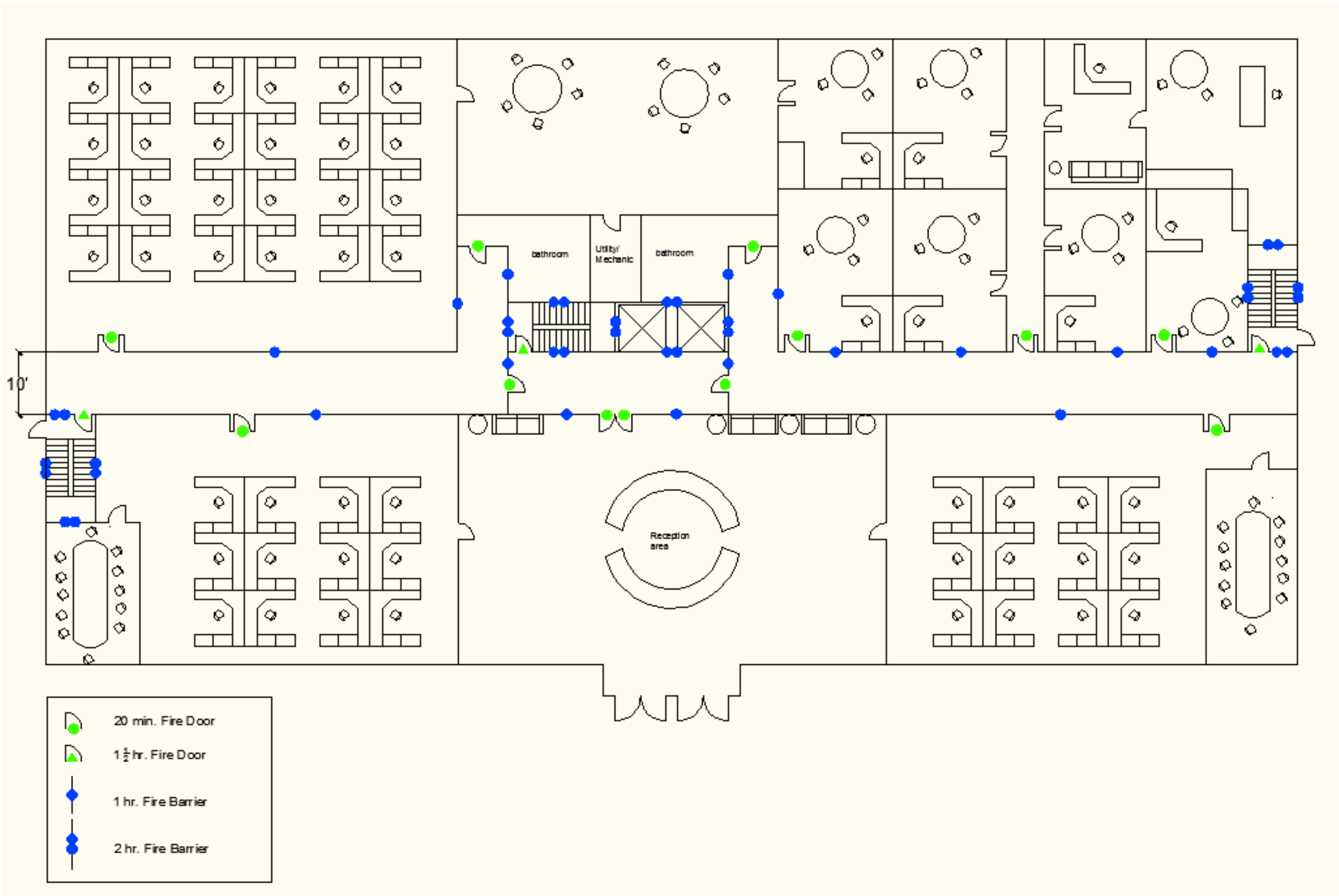


Figure 97 Location of Fire Doors and Fire Rated Walls for First Floor

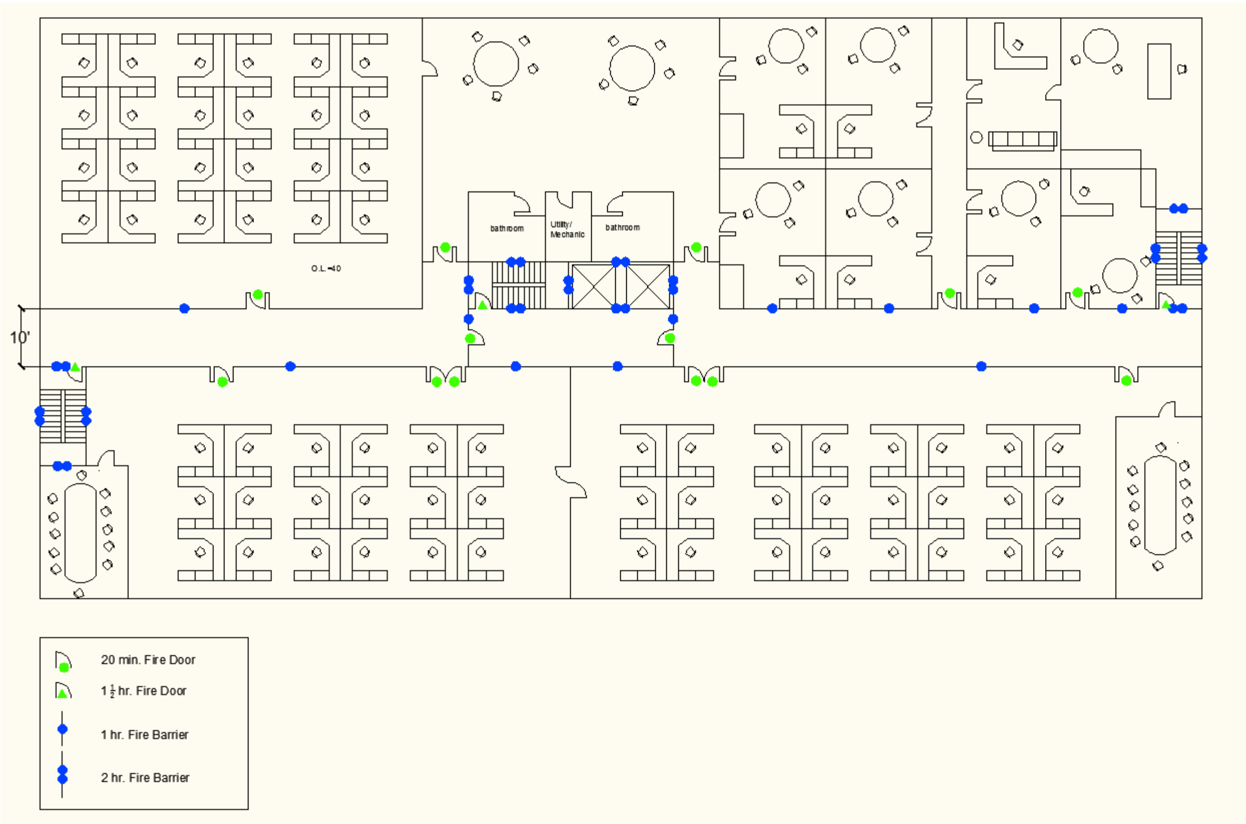


Figure 98 Location of Fire Doors and Fire Rated Walls for 2nd to 4th Floor

Active Protection

- Sprinkler system is not required for this building per section 903.2
- Fire Detection and Alarm system:
    - An approved fire alarm system installed in accordance with the provisions of IBC and NFPA 72 shall be provided. And for this building as Group B, only a manual fire alarm system shall be installed per section 907.2.2.
    - The smoke detector is not mandatory. However, since the smoke-activate doors (required by the codes, 715.4.8.3 and Table 715.4) needs the smoke detector to activate themselves. Smoke detectors are needed to be installed. The locations of smoke detectors are going to be covered in latter sections.
    - Manual fire alarm boxes: Where a manual fire alarm system is required by another section of this code, it shall be activated by fire alarm boxes (section 907.4.2)

        - Location: not more than 5 ft from the entrance to each exit. Additional manual fire alarm boxes shall be located so that travel distance to the nearest box does not exceed 200ft.
        - Height: shall be minimum of 42 in. and maximum of 48 in measured vertically, from the floor level to the activating handle or lever of the box.
        - Color: red
        - Signs: Where fire alarm systems are not monitored by a supervising station, an *approved* permanent sign shall be installed adjacent to each manual fire alarm box that reads: WHEN ALARM SOUNDS CALL FIRE DEPARTMENT
        - Protective Covers: the protective cover shall be transparent or red in color with a transparent face to permit visibility of the manual fire alarm box.
    - A fire alarm system shall annunciate at the panel and shall initiate occupant notification upon activation (section 907.5)
    - Alarm notification appliances shall be provided (section 907.5.2)
        - Average sound pressure: The audible alarm notification appliances shall provide a sound pressure level of 15 dBA above the average ambient sound level or 5 dBA above the maximum sound level having a duration of at least 60 seconds, whichever is greater, in every occupiable space with the building. And 60 dBA is the minimum in this building.
        - Maximum sound pressure: 110 dBA
        - Visible alarm notification appliances shall be provided in public areas and common areas. Where employee work areas have audible alarm coverage, the notification appliance circuits serving the employee work areas shall be initially designed with a minimum of 20-percent spare capacity to account for the potential of adding visible notification appliances in the future to accommodate hearing impaired employee(s) (section 907.5.2.3)
        -
    - Installation:
        - Power supply: the primary and secondary power supply for the fire alarm system shall be provided in accordance with NFPA 72.

        - Zones: each floor shall be zoned separately and a zone shall not exceed 22500 ft^2. The length of any zone shall not exceed 300 ft in any direction. In our floor plan, the footprint is 200ft X 100ft, therefore, it is in compliance with this code.
        - Access shall be provided to each fire alarm device and notification appliance for periodic inspection, maintenance and testing.

- Smoke Management/Control system: A smoke management would be required include covered mall buildings with floor openings connecting more than two stores (IBC:402.10), atriums as defined by each code (IBC: 404.5), smokeproof enclosures (IBC: 403.5.4, 909.20, 1022.9), underground structures (IBC: 405.5), underground structures in assembly occupancies, stages (IBC: 410.3.7, 101:12.4.5.5.1) and smoke-protected assembly seating in assembly occupancies (IBC: 1028.6.2.1, 101:12.4.2.1(2))

  Thus in this building, smoke management system is not required.
- Fire Department Connection: Fire department connections shall be installed in accordance with the NFPA standard applicable to the system design and shall comply with Sections 912.2 through 912.5.
    - Location: With respect to hydrants, driveways, buildings and landscaping, fire department connections shall be so located that fire apparatus and hose connected to supply the system will not obstruct access to the buildings for other fire apparatus. The location of fire department connections shall be *approved* by the fire chief. Fire department connections shall be located on the street side of buildings, fully visible and recognizable from the street or nearest point of fire department vehicle access or as otherwise approved by the fire chief.
    - Access: immediate access shall be maintained at all times and without obstruction by fences, bushes, trees, walls or any other fixed or moveable object.

- Stand Pipe System: Standpipe systems shall be installed per section 905.3.
    - Class III standpipe systems shall be installed throughout building (section 905.3.1)
        - Class III system: A system providing $1^1/_2$-inch (38 mm) hose stations to supply water for use by building occupants and $2^1/_2$-inch (64 mm) hose connections to supply a larger volume of water for use by fire departments and those trained in handling heavy fire streams (section 902.1)

- Location of Class III standpipe hose connections:
    - In every required stairway, a hose connection shall be provided for each floor level. Hose connections shall be located at an intermediate floor level landing between floors.
    - On each side of the wall adjacent to the exit opening of a horizontal exit (invalid for this office building due to no horizontal exit)
    - It shall be located so that all portions o the building are within 30 ft of a muzzle attached to 100 ft of hose.

- Portable Fire Extinguishers: For a new Group B occupancy, it is required to be installed unless building is equipped with quick response sprinklers. (section 906.1)
    - Portable fire extinguishers shall be selected, installed and maintained in accordance with this section and NFPA 10 (section 906.2)
        - Classifications of Fires: Class A Fires (section 5.2.1)
        - Classification of Hazards: light (low) hazards (section 5.4.1.1)

Table 6.2.1.1 Fire Extinguisher Size and Placement for Class A Hazards

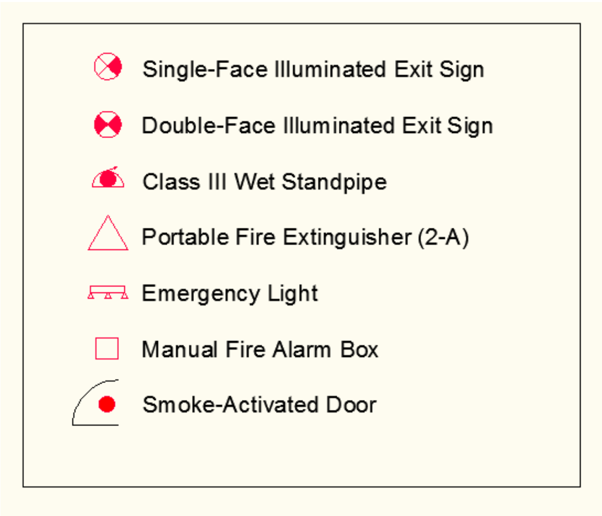
Criteria	Light (Low) Hazard Occupancy	Ordinary (Moderate) Hazard Occupancy	Extra (High) Hazard Occupancy
Minimum rated single extinguisher	2-A	2-A	4-A
Maximum floor area per unit of A	3000 ft^2	1500 ft^2	1000 ft^2
Maximum floor area for extinguisher	11,250 ft	11,250 ft	11,250 ft
Maximum travel distance to extinguisher	75 ft	75 ft	75 ft

For SI units, 1 ft = 0.305 m; 1 ft^2 = 0.0929 m^2.
Note: For maximum floor area explanations, see E.3.3.

Figure 99 Fire Extinguisher Size and Placement for Class A Hazards (Table 6.2.1.1, NFPA 10)

- Location: extinguisher shall be located in conspicuous locations where they will be readily accessible and immediately available for use. (section 906.5)
- Unobstructed: it shall not be obstructed or obscured from view. (section 906.6)

- Cabinets: cabinets used to house portable fire extinguishers shall not be locked. (section 906.8)
- Extinguisher Installation:
  - Extinguishers weighing 40 pounds or less: it shall be installed so that their tops are not more than 5 ft above the floor.
  - Extinguishers weighing more than 40 pounds: hand-held portable fire extinguishers shall be installed so that their tops are not more than 3.5 feet above the floor.
  - Floor clearance: The clearance between the floor and the bottom of installed hand-held portable fire extinguishers shall not be less than 4 in.



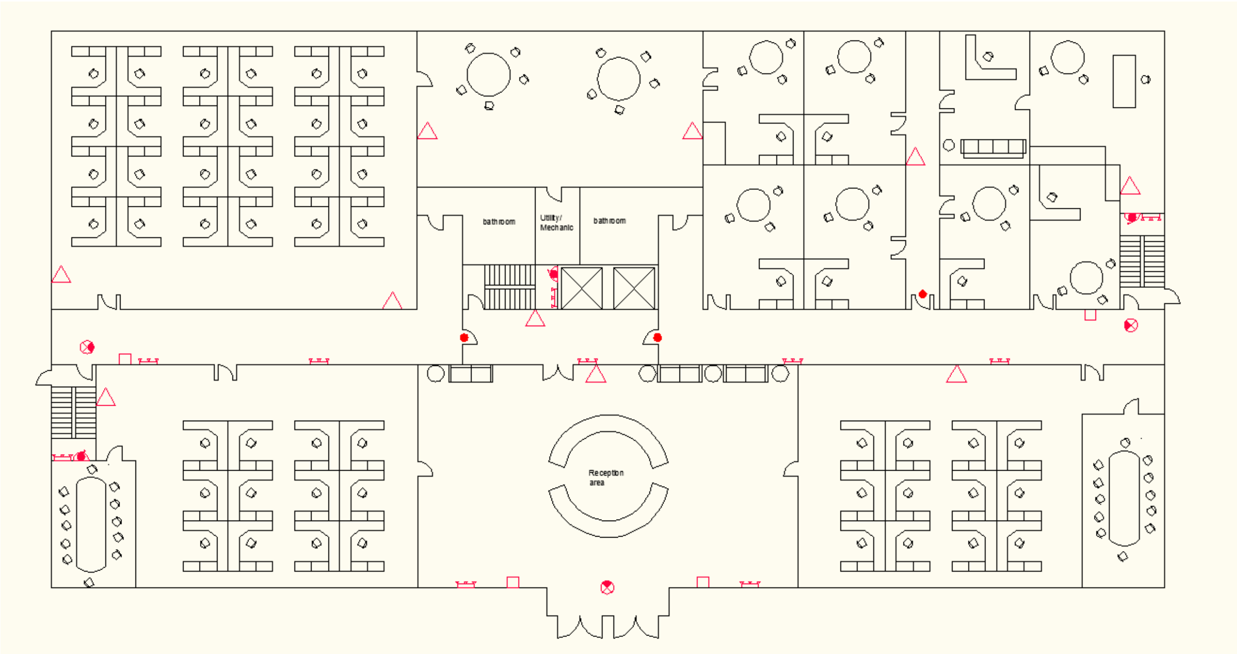


Figure 100 Activate Fire Protection System for 1st Floor

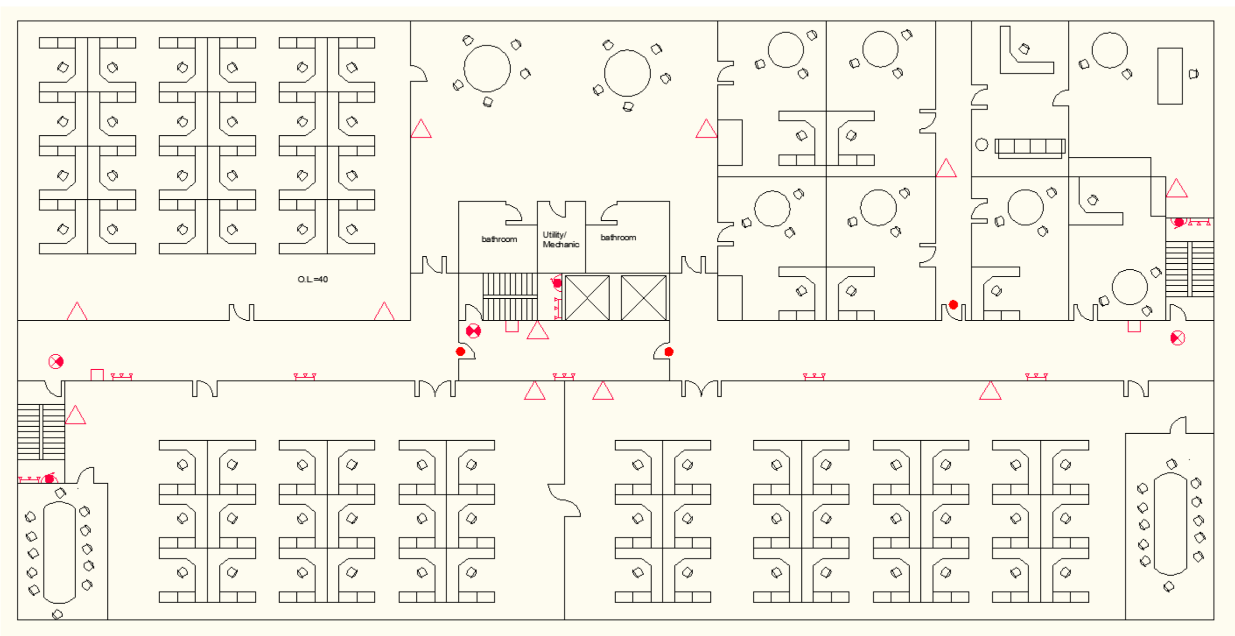


Figure 101 Activate Fire Protection System for 2nd to 4th Floor

G2: Code Analyses (with Sprinkler system)

(Note: the different requirements to non-sprinklered building are highlighted)

General:

- Occupancy classification: Group B (section 304.1)
- Construction type: Type I and II
- Height / area limitation: For Group B spinklered building, 6 stories above grade, 85 ft max. 37500 ft^2 per floor for Type II A construction. 4 stories above the grade with maximum 75 ft height and 23000 ft^2 per floor for Type II B construction. Since our building is 4-story, 42 ft high Group B. The structure members have to be at least Type II B. (0 hour rating for the structural members) (Table 503, section 504.2)

Means of Egress

- Doors
  - Size of doors: clear width of 32in min (section 1008.1.1)
  - Opening force: other than fire door, not exceed 5 pounds 1008.1.3
  - Fire Doors: fire doors are required to have a minimum fire protection rating of 20 minutes where located in corridor walls or smoke barrier walls having a fire-resistance rating in accordance with Table 715.4 . Based on section 715.4.4, doors in exit enclosures and exit passageways shall have a maximum transmitted temperature end point of not more than 450 F above the ambient at the end of 30 minutes of standard fire test exposure. According to section 715.4.8, all fire doors in this office building shall be self- or automatic-closing.
- Stairs
  - Width: 44 inches min. 1009.1
  - Headroom clearance 80 inches (measured vertically from a line connecting gate edge of the nosing) 1009.2
  - Riser 4 to 7 inches. Tread depth:11 min. section 1009.4.2

- Landings: 48in max. (same in floor plan) section 1009.5

- Occupant load: Based on Table 1004.1.1 where for Business, gross floor area per occupant is 100ft^2; Also based on section 1004.7 for fixed seating or in our project workstation and tables.
  - 1st floor: total occupant load is 238 in total instead of 200 due to the fixed seating (work station)
  - 2nd ~4th floor : total occupant load is 243 in total instead of 200 due to the fixed seating (work station)

- Number of Exits: Based on Table 1021.1, 2 exits are required for each story (1~500 occupant load). For 2nd to 4th floor, 3 stairs served as exits, and on the first floor 3 exits doors are provided (including the main entrance, 2 exit doors at stair shaft). Therefore, the design is in compliance with the code.
- Exit Access travel distance: based on section 1016.1, the maximum for sprinklered building is 300ft. In our floor plan, the maximum travel distance is 130 ft which is in compliance with the code.
- Common Path of Travel: maximum is 100 ft for sprinklered area. in our design, it is 50ft. (section 1014.3)

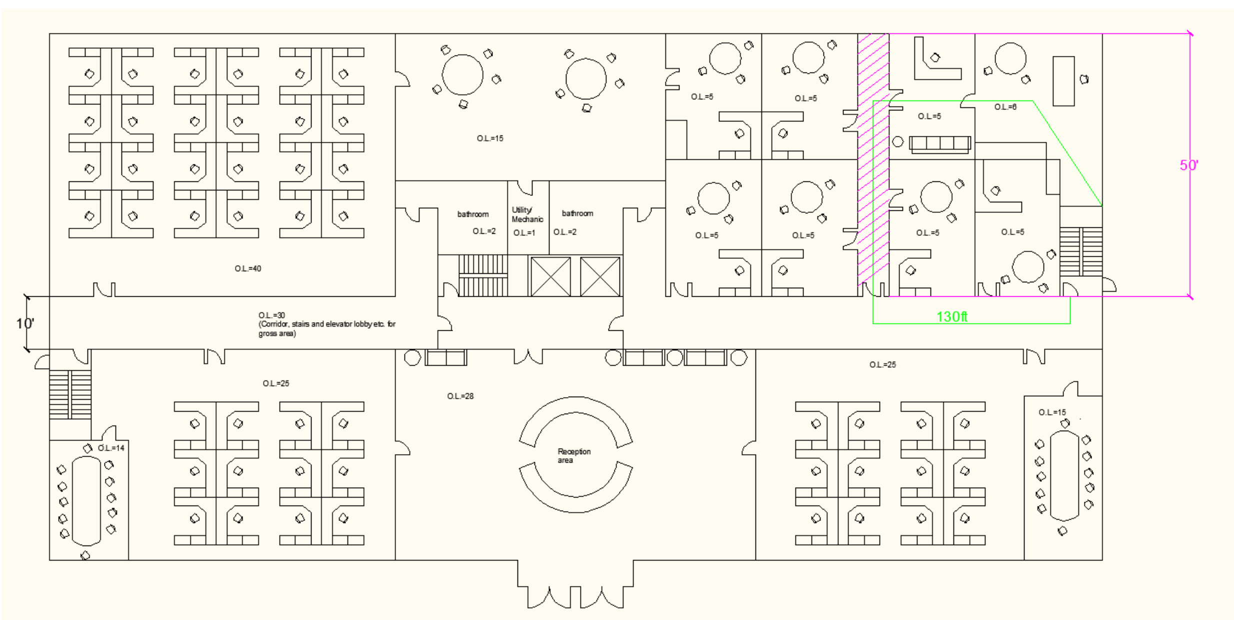


Figure 102. 1st Floor Plan (Green line stands for maximum travel distance; pink hatch stands for the maximum common path of travel; O.L. is occupant load)

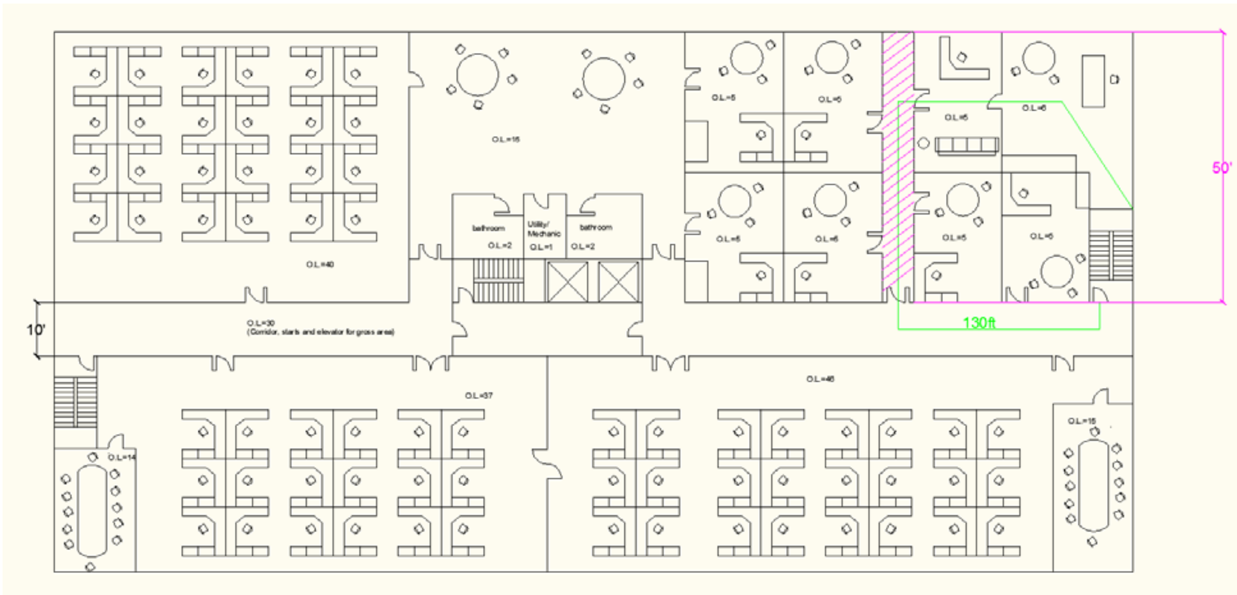


Figure 103. 2nd to 4th Floor Plan (Green line stands for maximum travel distance; pink hatch stands for the maximum common path of travel; O.L. is occupant load)

- Exits Remoteness: based on section 1015.2.1, the minimum distance is ½ of maximum diagonal distance of the room for non-sprinklered area, while it reduced to 1/3 for sprinklered building. And where exit enclosures are provided as a portion of the required exit and are interconnected by a 1-hour fire-resistance rated corridor, the required exit separation shall be measured along the shortest direct line of travel within the corridor.

  In our design, all of these requirements are met.
- Corridors:
  - Dead end corridor: section 1018.4 indicates that the maximum length is 20ft while if the building is protected with sprinkler, it can be 50 ft.

    In our second attempt, there is no dead end corridor.
  - Minimum width 44 in per section 1018.2. In our design, the width is 10 ft.
  - Obstruction: no obstruction at all per section 1018.3. Where in second attempt, doors were all fixed to meet this requirements.
  - The fire doors which are also automatic-closing doors installed across a corridor shall be automatic-closing by the actuation of smoke detectors installed accordance with section 907.3 or by loss of power to the smoke detector or hold-open device. Automatic closing

by smoke detection shall not have more than a 10-second delay before the door starts to close after the smoke detector is actuated. (section 715.4.8.3)

In the floor plan, the smoke-activated doors are the doors at elevator lobby.

- Elevator lobby: according to section 708.14.1, it is required to have enclosed elevator lobby at each floor since the elevator shaft connects more than three stories. And the lobby enclosure shall separate the elevator shaft enclosure doors from each floor by fire partitions. In addition to the requirements in Section 709 for *fire partitions* where the fire-resistance rating shall be not less than 1 hour, doors protecting openings in the elevator lobby enclosure walls shall also comply with Section 715.4.3 as required for *corridor* walls and penetrations of the elevator lobby enclosure by ducts and air transfer openings shall be protected as required for *corridors* in accordance with Section 716.5.4.1. Elevator lobbies shall have at least one *means of egress* complying with Chapter 10 and other provisions within this code.
- Illumination of means of egress: The means of egress, shall be illuminated at all times (corridor, stair enclosure) with not less than 1 foot-candle (11 lux) at the walking surface (section 1006.1 and 1006.2)
- Exit signs:

  Exits are required be marked by an approved exit sign readily visible from any direction of egress travel. Exit sign placement shall be such that no point in an exit access corridor is more than 100 ft from the nearest visible exit sign. It is required to be illuminated at all times,and extra power source shall be provided to ensure continued illumination for a duration of not less than 90 minutes. (section 1011)

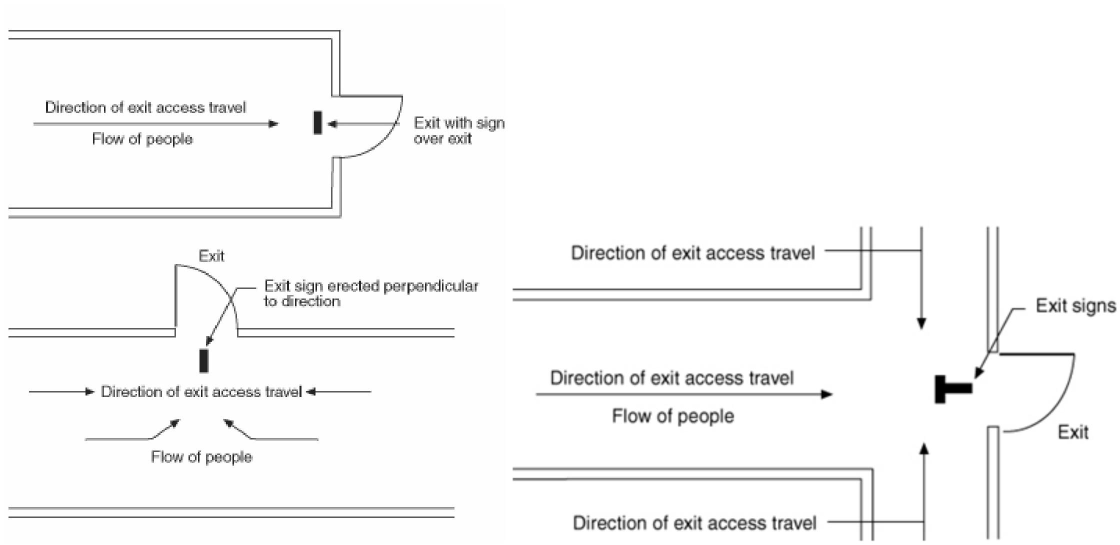


Figure 104 Location of Exit Signs (Figure A.7.10.1.2.1, NFPA 101)

- Area of Refuge: Where an elevator lobby is used as an *area of refuge*, the shaft and lobby shall comply with Section 1022.9 for *smokeproof enclosures* except where the elevators are in an *area of refuge* formed by a *horizontal exit* or *smoke barrier*.(section 1007.6) The elevator can be treated as a area of refuge. But it is not required.

Passive Fire Protection: the fire-resistance of building elements gives occupants time to leave the building in the event of an emergency and allow fire fighters time to fight the fire by delaying structural failure

- Fire barriers:
  - Shaft enclosure (elevator shaft) section 708.4 : not less than 2 hours where connecting four stories or more.
  - Exit enclosures (stair enclosure) section 1022.1: not less than 2 hours where connecting four stories or more.
  - Corridor walls do not required to be fire-resistance rated or 0 hour rating. (Table 1018.1)
  - Incidental accessory occupancies Table 508.4

- Continuity of fire barriers: shall extend from the top of the floor/ceiling assembly below to the underside of the floor or roof sheeting, slab or deck above and shall be securely attached there to. (Section 707.5)
- Opening protection: in accordance with section 715: 20min for corridor doors; openings in exit enclosures and shall comply with section 1022.3, 715.4:1 ½ hour; fire door assemblies in exit enclosure shall have an maximum transmitted temperature end point of not more than 450 ˚F . All fire doors shall be self or automatic-closing (section 715.4.8). doors are limited to a max. aggregate width of 25 percent of the length of the wall, and not exceed 156 ft^2(section 707.6) .
- Penetrations: comply with section 713.
- Ducts and air transfer openings : comply with section 716

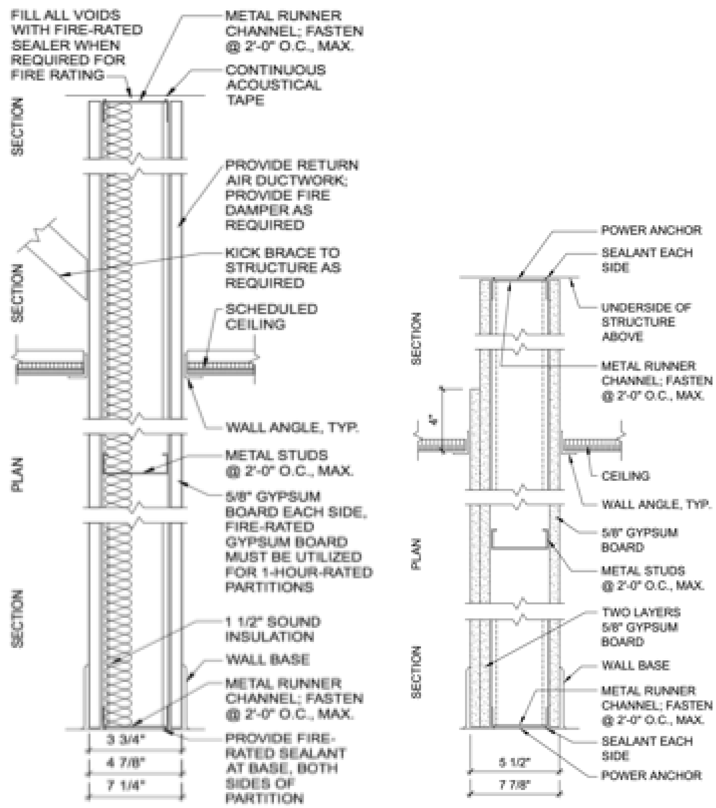


Figure 105 Samples of One-Hour Fire Rated Wall Construction (FPE 570, module 11)

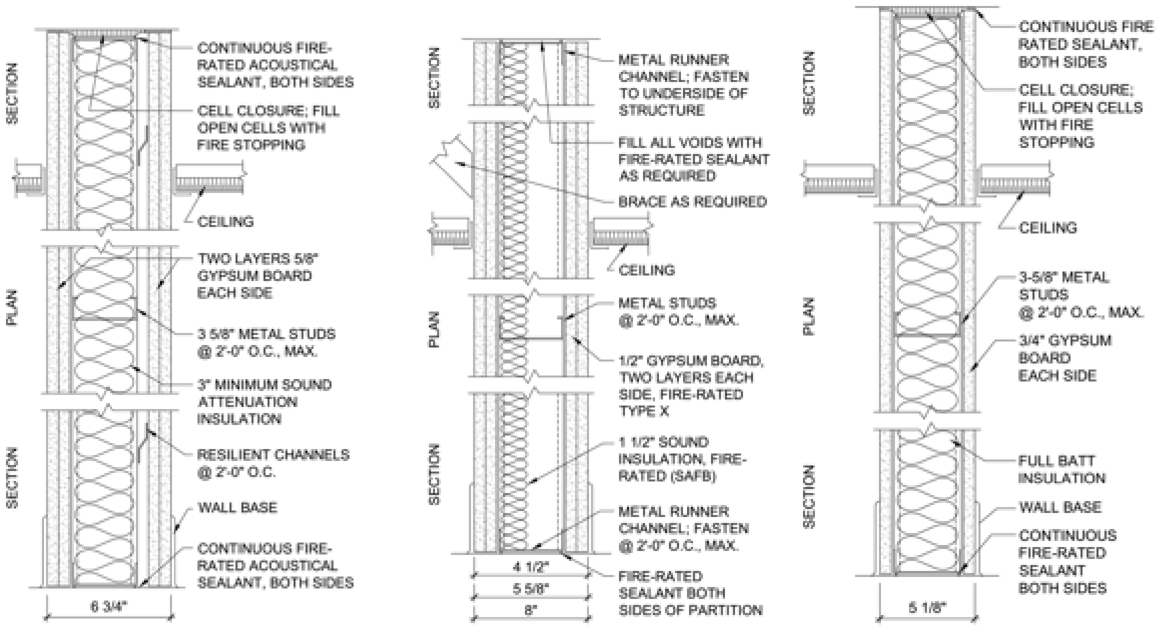


Figure 106 Samples of Two-Hour Fire-Rated Wall Construction (FPE 570, module 11)

Figure 107 Illustration of Vertical Continuity of Fire Barriers

- Smoke barriers:
  - 1 hour fire-resistance is required (section 710.3)
  - There is no required smoke barrier or partition for Group B building. However, it will be good to has exit corridor constructed as smoke barrier.
- Capacity of exits: In preliminary code analysis

- Arrangement of means of egress: In preliminary code analysis
- Protection for Floor or Roof Slab System: Table 720.1(3). 5.1 in thick lightweight concrete is equivalent to 4 hours fire-resistance rating and 3 hour for 4.4 in, 2 hour for 3.6 in. In our design the thick is 3 in (except 5" in 10'ft spacing)
- Interior wall and ceiling finish

Table 35 Interior Wall and Ceiling Finish Requirement by Occupancy (Table 803.9)

GROUP	SPRINKLERED[l]			NONSPRINKLERED		
	Exit enclosures and exit passageways[a, b]	**Corridors**	**Rooms and enclosed spaces[c]**	**Exit enclosures and exit passageways[a, b]**	**Corridors**	**Rooms and enclosed spaces[c]**
A-1 & A-2	B	B	C	A	A[d]	B[e]
A-3[f], A-4, A-5	B	B	C	A	A[d]	C
B, E, M, R-1	B	C	C	A	B	C
R-4	B	C	C	A	B	B
F	C	C	C	B	C	C
H	B	B	C[g]	A	A	B
I-1	B	C	C	A	B	B
I-2	B	B	B[h, i]	A	A	B
I-3	A	A[j]	C	A	A	B
I-4	B	B	B[h, i]	A	A	B
R-2	C	C	C	B	B	C
R-3	C	C	C	C	C	C
S	C	C	C	B	B	C
U	No restrictions			No restrictions		

Table 36 Classification of Wall and Ceiling Finish (Section 803.1.1)

Class A:	Flame spread index 0-25; smoke-developed index 0-450.
Class B:	Flame spread index 26-75; smoke-developed index 0-450.
Class C:	Flame spread index 76-200; smoke-developed index 0-450.

Therefore, based on Table 803.9, Corridors shall have Class C wall and ceiling finish, Other rooms shall have Class C wall and ceiling finish.

- Interior floor finish

Interior floor finish and floor covering materials shall be at least complying with the DOC FF-1 “pill test” in all area. (section 804.4.1)

804.2 Classification. *Interior floor finish* and floor covering materials required by Section 804.4.1 to be of Class I or II materials shall be classified in accordance with NFPA 253. The classification referred to herein corresponds to the classifications determined by NFPA 253 as follows: Class I, 0.45 watts/cm^2 or greater; Class II, 0.22 watts/cm^2 or greater.

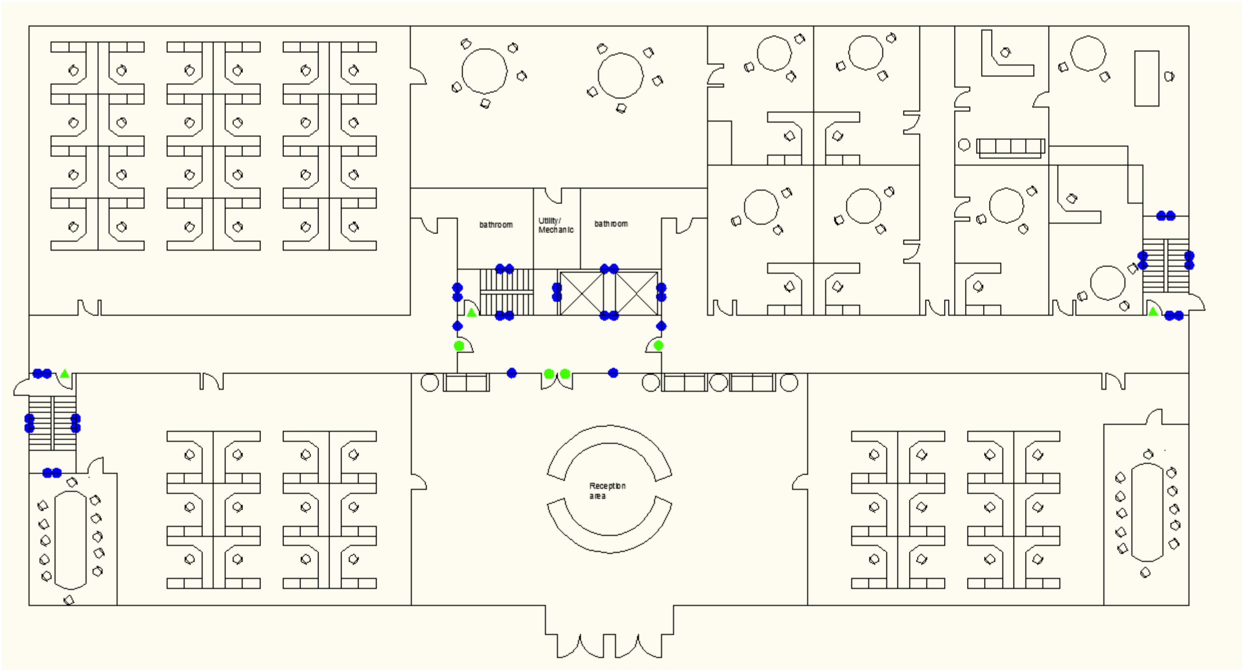


Figure 108 Location of Fire Doors and Fire Rated Walls for First Floor

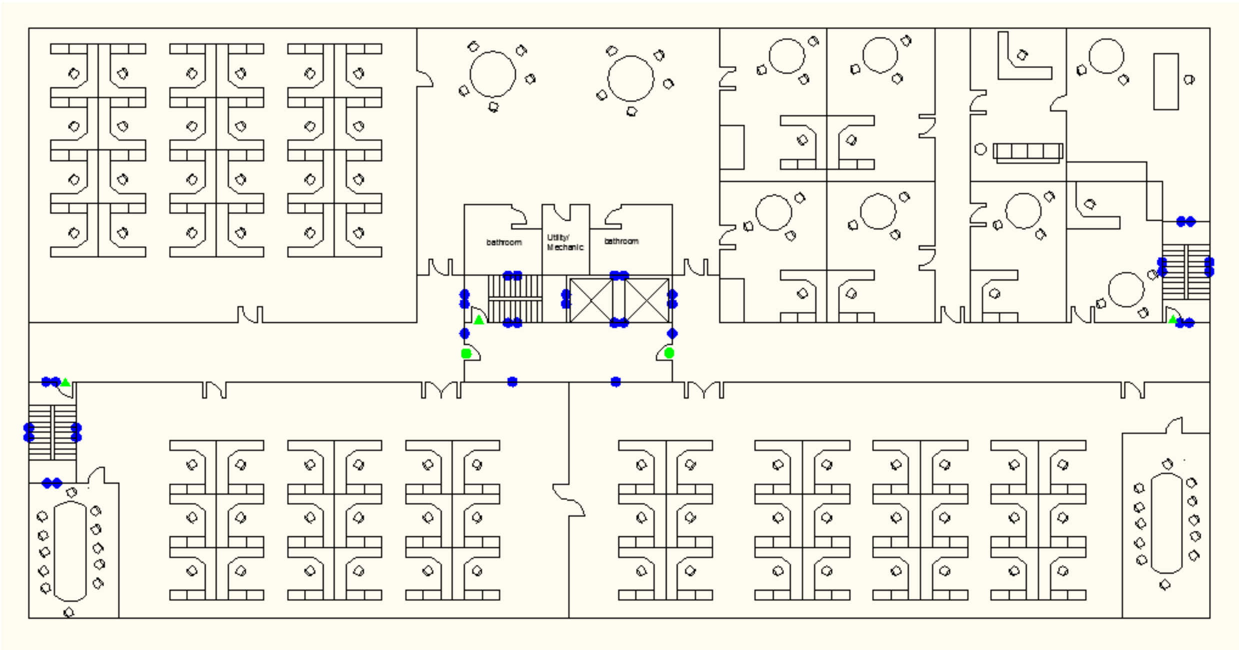


Figure 109 Location of Fire Doors and Fire Rated Walls for 2nd to 4th Floor

Active Protection

- Sprinkler system shall be installed based on NFPA 13 per section 903.3.1
- Fire Detection and Alarm system:
  - Manual fire alarm boxes are not required for sprinklered Group B building, , the occupant notification applicances will activated throughout the notification zones upon spinkler waterflow per section 907.2.2. However. Based on section 907.2, a minimum of one manual fire alarm box shall be installed unless fire alarm systems dedicated to elevator recall control and supervisory service. Therefore, in this office building, one manual fire alarm box shall be provided at each floor.
  - The smoke detector is not mandatory. However, since the smoke-activate doors (required by the codes, 715.4.8.3 and Table 715.4 needs the smoke detector to activate themselves. Smoke detectors are needed to be installed. The locations of smoke detectors are going to be covered in latter sections.
  - Manual fire alarm boxes: Where a manual fire alarm system is required by another section of this code, it shall be activated by fire alarm boxes (section 907.4.2)
    - Location: not more than 5 ft from the entrance to each exit.

        - Height: shall be minimum of 42 in. and maximum of 48 in measured vertically, from the floor level to the activating handle or lever of the box.
        - Color: red
        - Signs: Where fire alarm systems are not monitored by a supervising station, an *approved* permanent sign shall be installed adjacent to each manual fire alarm box that reads: WHEN ALARM SOUNDS CALL FIRE DEPARTMENT
        - Protective Covers: the protective cover shall be transparent or red in color with a transparent face to permit visibility of the manual fire alarm box.
    - A fire alarm system shall annunciate at the panel and shall initiate occupant notification upon activation from manual fire alarm boxes or automatic fire-extinguishing systems (section 907.5)
    - Alarm notification appliances shall be provided (section 907.5.2)
        - Average sound pressure: The audible alarm notification appliances shall provide a sound pressure level of 15 dBA above the average ambient sound level or 5 dBA above the maximum sound level having a duration of at least 60 seconds, whichever is greater, in every occupiable space with the building. And 60 dBA is the minimum in this building.
        - Maximum sound pressure: 110 dBA
        - Visible alarm notification appliances shall be provided in public areas and common areas. Where employee work areas have audible alarm coverage, the notification appliance circuits serving the employee work areas shall be initially designed with a minimum of 20-percent spare capacity to account for the potential of adding visible notification appliances in the future to accommodate hearing impaired employee(s) (section 907.5.2.3)
        -
    - Installation:
        - Power supply: the primary and secondary power supply for the fire alarm system shall be provided in accordance with NFPA 72.
        - Zones: each floor shall be zoned separately and a zone shall not exceed 22500 ft^2. The length of any zone shall not exceed 300 ft in any direction. In our floor plan, the footprint is 200ft X 100ft, therefore, it is in compliance with this code.

        - Access shall be provided to each fire alarm device and notification appliance for periodic inspection, maintenance and testing.

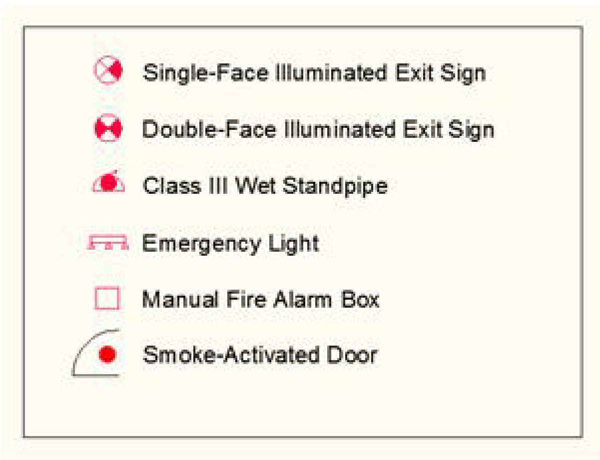
- Smoke Management/Control system: A smoke management would be required include covered mall buildings with floor openings connecting more than two stores (IBC:402.10), atriums as defined by each code (IBC: 404.5), smokeproof enclosures (IBC: 403.5.4, 909.20, 1022.9), underground structures (IBC: 405.5), underground structures in assembly occupancies, stages (IBC: 410.3.7, 101:12.4.5.5.1) and smoke-protected assembly seating in assembly occupancies (IBC: 1028.6.2.1, 101:12.4.2.1(2))

  Thus in this building, smoke management system is not required.
- Fire Department Connection: Fire department connections shall be installed in accordance with the NFPA standard applicable to the system design and shall comply with Sections 912.2 through 912.5.
    - Location: With respect to hydrants, driveways, buildings and landscaping, fire department connections shall be so located that fire apparatus and hose connected to supply the system will not obstruct access to the buildings for other fire apparatus. The location of fire department connections shall be *approved* by the fire chief. Fire department connections shall be located on the street side of buildings, fully visible and recognizable from the street or nearest point of fire department vehicle access or as otherwise approved by the fire chief.
    - Access: immediate access shall be maintained at all times and without obstruction by fences, bushes, trees, walls or any other fixed or moveable object.

- Stand Pipe System: Standpipe systems shall be installed per section 905.3.
    - Class I standpipe systems shall be installed throughout building, they are allowed to be combined with automatic sprinkler systems. (section 905.3.1)
        - Class I system: A system providing $2^1/_2$-inch (64 mm) hose connections to supply water for use by fire departments and those trained in handling heavy fire streams (section 902.1)
    - Location of Class I standpipe hose connections:

- In every required stairway, a hose connection shall be provided for each floor level. Hose connections shall be located at an intermediate floor level landing between floors.
- On each side of the wall adjacent to the exit opening of a horizontal exit (invalid for this office building due to no horizontal exit)

- Portable Fire Extinguisher: .it is not required to be installed if building is protected by sprinkler system with quick response sprinkler. (section 906.1)



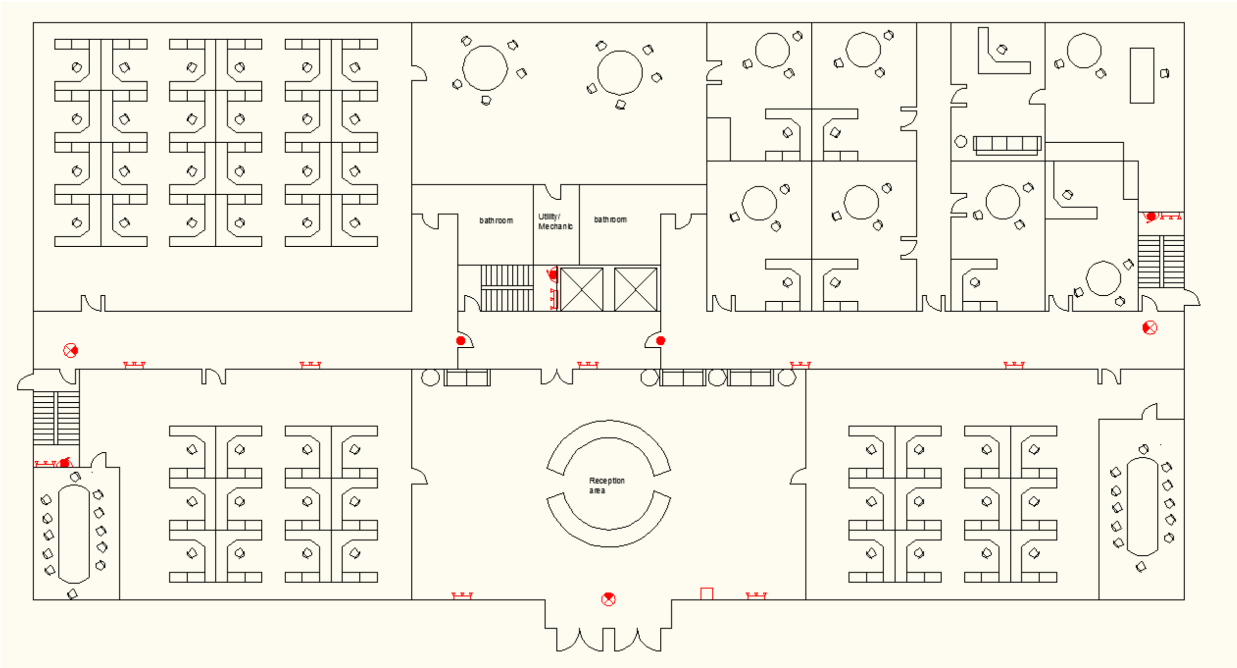


Figure 110 Activate Fire Protection System for 1st Floor

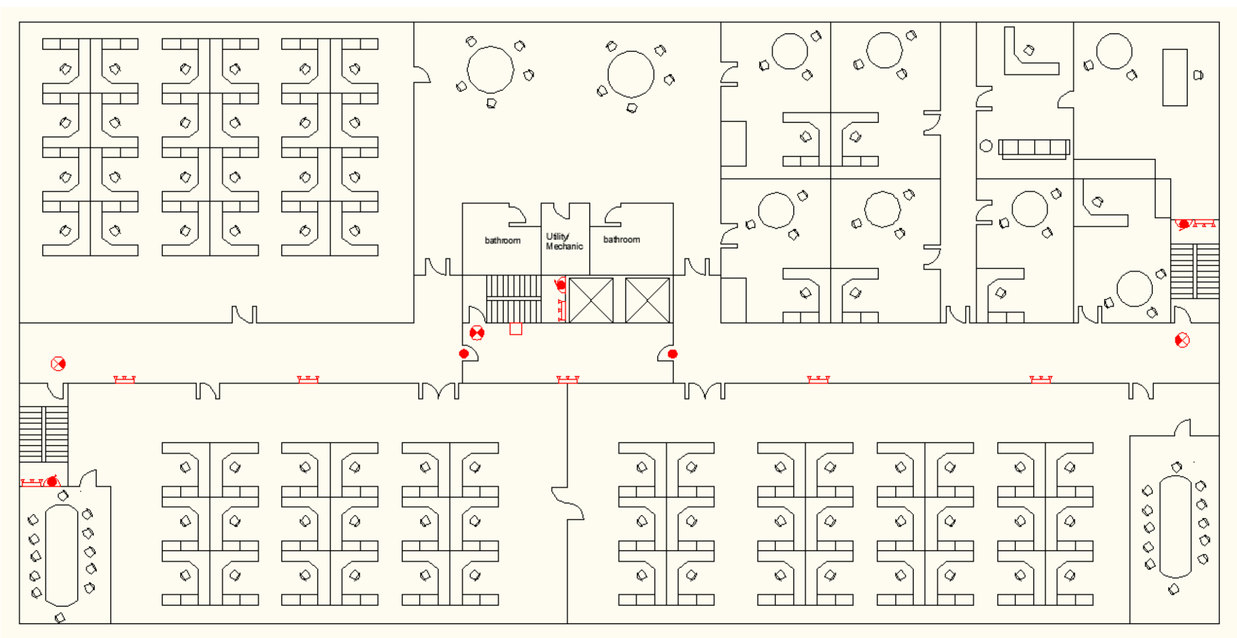


Figure 111 Activate Fire Protection System for 2nd to 4th Floor

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Appendix H: Cost Estimation for Alternative Fire Protection Systems (Based on RS Means Building Construction Cost Data, 67 edition)

Note: only the different components in two fire protection systems were selected for cost estimation. Same components (the number, the type of the component do not change when the building is sprinklered or not), such as exit signs and emergency lights, and something whose costs are hard to obtain, like different interior finish materials, are not included in this cost estimation.

Walls: (09 29 Gypsum Board and 07 21 16.20 Blanket insulation for Walls)

1-hour wall: based on the sample one-hour wall, 2 layer of 5/8" gypsum board applied: in total 5/4 in. thick, a 1/2" thick gypsum board on wall is $\$0.84/ft^2$, therefore, 5/4 in is $\$2.1/ft^2$

2-hour wall: Based on the sample of two-hour wall, 2 layer of ¾" gypsum board applied with 5" thick full batt insulation: in total 1.5 in thick gypsum board, therefore $\$2.52/ft^2$; the 6" batt insulation is $\$0.84/ft^2$. Therefore, the total is $\$3.36/ ft^2$

Doors:

Fire door: (08 14 16.20 Wood Fire Doors)

20-minute door: $440 each for 1 hour door, the team decided deduction based on the cost of 1 hour door and 1 ½ hour door. The cost for 20 minutes door is 440-(460-440)/30*(60-20)=$413 each

1 ½ -hour door: $460 each

Standpipe:

Class I: Standpipe connections, wall, w/ plugs & chains, single flush, brass, 2-1/2" x 2-1/2": $345 each (21 11 19 Fire-Department Connections)

Class III: Standpipe connections, wall, w/ plugs & chains, single flush, brass, 2-1/2” x 2-1/2”: $345 each (21 11 19 Fire-Department Connections, RS Means Building Construction Cost Data, 67 edition); 130 ft hose connection: $3.03 /ft (21 12 13 Fire-Suppression Hoses and Nozzles)

Fire Alarm:

Manual fire alarm boxes: alarm device: $29 each (28 31 Fire Detection and Alarm, RS)

Portable Fire Extinguisher: $54 each (10 44 16.13 Portable Fire Extinguishers)

Fireproof Spray: Decking: 1” thickness, $1.95/ft^2; Beams: 1” thickness, $1.48/ft^2 (07 81 Applied Fireproofing)

Sprinkler System: assume it is wet pipe system, $2.77/ft^2 and $2.14/ft^2 for each additional floor (D4010 410 Wet Pipe Sprinkler System, RS Means Assemblies Cost Data, 34 edition)

Table 37 Cost Estimation for Non-Sprinklered Building

Non-sprinklered Building					
Components		Unit Price	Applied Quatity	Cost	Total Cost
Walls	1-hour walls	2.1/sqft	15990spft	33579	
	2-hour walls	3.36/sqft	8484 sqft	28506.24	
Doors	20-minutes doors	413 each	51	21063	
	1 1/2-hour doors	460 each	12	5520	
Standpipes	Class III	739each	9	6651	256111.5
Manual Fire Alarm boxes:		29 each	13	377	
Fireproofing Spray	Girders & OWJ	1.95/ sqft	35535sqft	69293.25	
	decking	1.48/ sqft	60000sqft	88800	
Portable Fire Extinguisher		54 each	43	2322	

Table 38 Cost Estimation for Sprinklered Building

Spinklered Building					
Components		Unit Price	Applied Quatity	Cost	Total Cost
Walls	1-hour walls	2.1/sqft	3780spft	7938	233115.2
	2-hour walls	3.36/sqft	8484 sqft	28506.24	
Doors	20-minutes doors	413 each	10	4130	
	1 1/2-hour doors	460 each	12	5520	
Standpipes	Class I	345 each	9	3105	
Manual Fire Alarm boxes:		29 each	4	116	
Fireproofing Spray	Girders & OWJ	1.95/ sqft	0 sqft	0	
	decking	1.48/ sqft	0 sqft	0	
Portable Fire Extinguisher		54 each	0	0	
Sprinkler System	First floor	2.77/sqft	20000 sqft	55400	
	Other floors	2.14/sqft	60000 sqft	128400	

Therefore, by installing sprinkler, the owner will save $22996

Appendix I: Smoke Detector

I.1 Installation Requirements

Installation Requirements (NFPA 72, 2002)

It is recommended that smoke detectors space at 30 ft when they are installed on flat, smooth ceilings. (B.4.1.1)

5.5.2.1 Total (Complete) Coverage. If required and unless otherwise modified by 5.5.2.1.1 through 5.5.2.1.6, total coverage shall include all rooms, halls storage areas, basements, attics, lofts, spaces above suspended ceilings , and other subdivisions and accessible spaces as well as the inside of all closets, elevator shafts, enclosed stairways, dumbwaiter shafts, and chutes.

5.5.2.1.1 where inaccessible areas do not contain combustible materials, they shall not be required to be protected by detectors.

5.5.2.1.2 where inaccessible areas contain combustible materials, unless otherwise specified in 5.5.2.1.3, they shall be made accessible and shall be protected by a detector(s).

5.5.2.4.1 Where installed, detection that is not required by an applicable law, code, or standard, whither total (complete), partial, or selective coverage, shall conform to the requirements of this Code(NFPA 72)

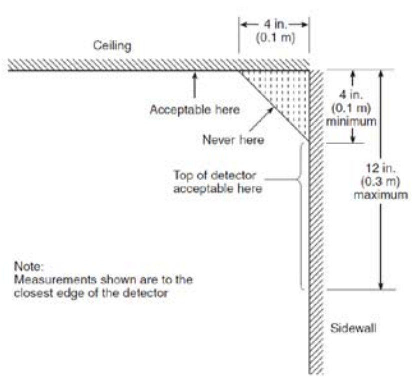
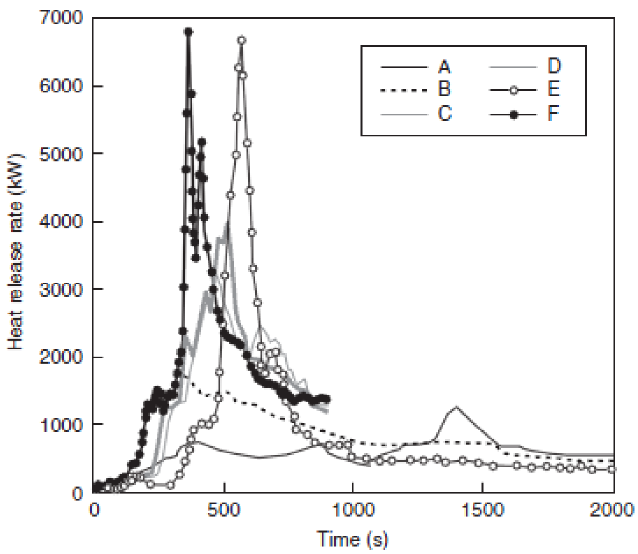


FIGURE 14.2.16 Mounting of Spot-Type Detectors

I.2 Design Fire

Code	Combustible Mass (kg)	Description	Number of Sides with Acoustic Panels	Reference
A	291	Mostly old-style wood furniture	0	110
B	291	Semi-modern furniture	1	110
C	335	Modern furniture	2	112
D	—	Modern furniture	3	112
E	291	Modern furniture	4	110
F	—	Modern furniture	4	112



$Q=\alpha t^p$, where p=2, and the worst one fire grows to about 6800kW (peak) at approximately 400sec. ➔ α=0.0425 kW/sec^2

Also similar fire behavior can be observed in NIST test for a three panel workstation (http://my.wpi.edu/webapps/portal/frameset.jsp?tab_id=_2_1&url=%2fwebapps%2fblackboard%2fexecute%2flauncher%3ftype%3dCourse%26id%3d_93254_1%26url%3d)

I.3 Burning Materials:

Major burning items are chair, panel, computer monitor, trash bin (paper) . The material of chair pad is assumed to be nylon, the textile on the panels is nylon underlay the textile is PMMA board. Further assume that, the smoke detectors will be activated at the very early stage of the fire. Or in other words, the majority of burning fuel is nylon. Based on Table 3-4.14 of SFPE HB,

For Nylon

ΔH_T=30.8 kJ/g; ΔH_{ch}=27.1 kJ/g; ΔH_{con}=16.3 kJ/g; ΔH_{rad}=10.8 kJ/g; y_s=0.075g/g

For PMMA

ΔH_T=25.2 kJ/g; ΔH_{ch}=24.2 kJ/g; ΔH_{con}=16.6 kJ/g; ΔH_{rad}=7.6 kJ/g; y_s=0.022g/g

The response time of smoke detector is the sum of the time for local activation time and smoke travel time.

I.4 Local Activation Time

First, we need to know what is the obscuration per unit length would activate the smoke detector.

$$OD = -\log_{10}(1 - O_u) \qquad (1)$$

OD is optical density and O_u is the obscuration per unit length which is given in product sheets as 2.5%/ft or, 8%/m

$$OD = -\log_{10}(1 - 8/100) = 0.036\, m^{-1}$$

However, information about obscuration per unit length is hard to obtain by doing fire modeling. Therefore, it will be helpful to convert obscuration per unit length into smoke concentration.

We know that:

$$OD = \frac{(1.5)kv_pN(0.43)}{d_p} \qquad (2)$$

k is the extinction codification, and the team assumed it is 1.5. And d_p is the diameter of smoke particles, which is assumed to be 2μm. N the number of smoke particles in one cubic meter. v_p is the volume. Manipulating equation above, we got the equation shows below.

$$N = \frac{(OD)d_p}{(1.5)k(0.43)} \qquad (3)$$

The concentration of smoke is equation 4, where ρ_p is the particle density which is assume to be carbon with a value of 2.3x10^6 g/m^3

$$C = \rho_p v_p N \qquad (4)$$

Substituting equation 3 into equation 4, we obtained equation 5 which is shown below.

$$C_{id} = \frac{(OD)\rho_p d_p}{(1.5)k(0.43)}$$

Therefore, the smoke concentration inside the detector that will activate the detector is:

$$C_{id} = \frac{(0.036\, m^{-1})2.3\text{x}10^6\ \text{g/m}^3 2\mu\text{m}}{(1.5)1.5(0.43)} = 0.017\ \text{g/m}^3$$

The next step is to figure out what is the smoke concentration outside the detector.

We have:

$$\dot{Q}_t = \Delta H_{ch}\dot{m}_f \qquad (5)$$

Where ΔH_{ch} is the chemical heat of combustion, and $\dot{m}_f$ is the fuel burning rate.

Manipulating equation 5, we got fuel burning rate:

$$\dot{m}_f = \frac{\dot{Q}_t}{\Delta H_{ch}} \qquad (6)$$

Based on soot yield fraction provided in SFPE Handbook, soot production can be calculated as follow:

$$\dot{m}_s = y_s\dot{m}_f \qquad (7)$$

By now, we know the mass change rate. In order to get concentration, we need to know what the volume change rate (shown in equation 8). In this case, it is appropriate to calculate the smoke concentration by volume of air entranced the plume. And the volume can be calculated by dividing the plume air entrainment by density of air (shown in equation 9, assumed to be 1.2kg/m^3).

$$C_o = \frac{\dot{m}_s}{\dot{V}} \qquad (8)$$

$$\dot{V} = \frac{\dot{m}_{ent}}{\rho_\infty} \qquad (9)$$

Therefore, we need to know what plume air entrainment is. Based on equation 10.43b *Fundamentals of Fire Phenomena* by James G. Quintiere, it can be calculated by equation 10. And Ce is suggested to be 0.17 by Quintiere.

$$\dot{m}_{ent} = C_e\left(C_p T_\infty\right)^{-\frac{1}{3}}(\rho_\infty\sqrt{g})^{2/3}\,\dot{Q_{con}}^{1/3}H^{5/3} \qquad (10)$$

It should be note that the smoke concentration in the detector is not the same with the concentration outside it. There is entrainment resistance. In order to calculate this time delay, based on B.4 *Smoke Detector Spacing for Flaming Fires* form Annex B of NFPA 72, differential equation 11 is developed.

$$\frac{dC_i}{dt} = \frac{1}{\tau}(C_o - C_i)\ (s^{-1}m^{-1}) \qquad (11)$$

$$\tau = \frac{L_c}{u}\ (s) \qquad (12)$$

Where L_c =5.3m for photoelectric detector (Tasble 4-1.14 of SFPE Handbook), u is the ceiling jet velocity.

In order to calculate ceiling, knowledge from SFPE Handbook, Section 2 Chapter 2 is applied.

In this case:

$$\frac{r}{H} = \frac{6.46m}{3.11m} = 2.077 > 0.15$$

Therefore, ceiling jet velocity is :

$$u = 0.195\frac{(\frac{\dot{Q}}{H})^{1/3}}{(\frac{r}{H})^{5/6}} \qquad (13)$$

Although, we obtained the differential equation 11, it will be hard to get an analytical solution. Therefore numerical method was applied. More specifically, the following shall be applied to simplify differential equation 11:

$$C_{i,n} = \frac{1}{\tau}\left(C_{o,n} - C_{i,n-1}\right)\left(s^{-1}m^{-1}\right) \qquad (14)$$

Use numerical method to solve for equation 5 to 14, the time when the smoke concentration inside the detector reaches the threshold can be calculated. The Spread sheet is provided below:

1st floor								
α=	0.0425	kW/sec^2						
ΔH_{ch}=	27.1	kJ/g	χ_{con}=	0.601476				
y_s=	0.075	g/g						
H=	3.11	m						
r=	6.46	m						
t (sec)	Q (kW)	m_f (g/s)	m_s (g/s)	m_e(g/s)	C_o (g/m^3)	u (m/s)	τ	C_i (g/m^3)
0	0	0	0	0	#DIV/0!	0	#DIV/0!	0
1	0.0425	0.001568	0.000118	0.120774	0.001169	0.025352	209.054	5.59024E-06
2	0.17	0.006273	0.00047	0.191717	0.002945	0.040244	131.6958	2.79087E-05
3	0.3825	0.014114	0.001059	0.25122	0.005057	0.052735	100.5027	7.79432E-05
4	0.68	0.025092	0.001882	0.304331	0.007421	0.063884	82.96314	0.000166448
5	1.0625	0.039207	0.00294	0.353145	0.009992	0.074131	71.49547	0.000303875

6	1.53	0.056458	0.004234	0.398787	0.012742	0.083711	63.31273	0.000500324
7	2.0825	0.076845	0.005763	0.441949	0.015649	0.092772	57.12942	0.000765488
8	2.72	0.100369	0.007528	0.483096	0.018699	0.101409	52.2635	0.001108617
9	3.4425	0.12703	0.009527	0.522559	0.021878	0.109693	48.31665	0.001538481
10	4.25	0.156827	0.011762	0.560583	0.025178	0.117675	45.03932	0.002063347
11	5.1425	0.18976	0.014232	0.597359	0.02859	0.125395	42.26654	0.002690948
12	6.12	0.22583	0.016937	0.633035	0.032107	0.132884	39.88452	0.003428474
13	7.1825	0.265037	0.019878	0.667732	0.035723	0.140167	37.812	0.004282553
14	8.33	0.30738	0.023054	0.70155	0.039433	0.147266	35.98928	0.005259244
15	9.5625	0.35286	0.026464	0.734572	0.043233	0.154198	34.37144	0.006364036
16	10.88	0.401476	0.030111	0.766867	0.047117	0.160977	32.92394	0.007601842
17	12.2825	0.453229	0.033992	0.798496	0.051084	0.167616	31.61981	0.008977007
18	13.77	0.508118	0.038109	0.82951	0.05513	0.174127	30.43758	0.010493313
19	15.3425	0.566144	0.042461	0.859955	0.059251	0.180518	29.36	0.012153987
20	17	0.627306	0.047048	0.88987	0.063445	0.186797	28.373	0.013961718
21	18.7425	0.691605	0.05187	0.919291	0.067709	0.192973	27.46496	0.015918667
22	20.57	0.759041	0.056928	0.948248	0.072042	0.199052	26.62625	0.018026486
23	22.4825	0.829613	0.062221	0.976769	0.076441	0.205039	25.84877	0.02028634
24	24.48	0.903321	0.067749	1.00488	0.080904	0.21094	25.12567	0.022698923
25	26.5625	0.980166	0.073512	1.032603	0.08543	0.216759	24.45111	0.025264482
26	28.73	1.060148	0.079511	1.059958	0.090016	0.222501	23.82007	0.027982845
27	30.9825	1.143266	0.085745	1.086966	0.094662	0.228171	23.22822	0.030853438
28	33.32	1.22952	0.092214	1.113641	0.099365	0.23377	22.67183	0.033875313
29	35.7425	1.318911	0.098918	1.140001	0.104124	0.239304	22.14759	0.037047178
30	38.25	1.411439	0.105858	1.16606	0.108939	0.244774	21.65265	0.040367416

31	40.8425	1.507103	0.113033	1.19183	0.113808	0.250183	21.18446	0.043834116
32	43.52	1.605904	0.120443	1.217325	0.118729	0.255535	20.74079	0.047445094
33	46.2825	1.707841	0.128088	1.242556	0.123701	0.260831	20.31963	0.051197926
34	49.13	1.812915	0.135969	1.267533	0.128724	0.266075	19.91923	0.055089965
35	52.0625	1.921125	0.144084	1.292266	0.133797	0.271266	19.53799	0.059118374
36	55.08	2.032472	0.152435	1.316765	0.138918	0.276409	19.17448	0.063280143
37	58.1825	2.146956	0.161022	1.341038	0.144087	0.281504	18.82742	0.067572116
38	61.37	2.264576	0.169843	1.365093	0.149302	0.286554	18.49564	0.071991014
39	64.6425	2.385332	0.1789	1.388938	0.154564	0.291559	18.17811	0.076533454
40	68	2.509225	0.188192	1.412581	0.159871	0.296522	17.87387	0.081195972
41	71.4425	2.636255	0.197719	1.436027	0.165222	0.301444	17.58204	0.085975044
42	74.97	2.766421	0.207482	1.459283	0.170617	0.306326	17.30184	0.090867098
43	78.5825	2.899723	0.217479	1.482355	0.176054	0.311169	17.03255	0.095868539
44	82.28	3.036162	0.227712	1.505249	0.181534	0.315975	16.77349	0.100975761
45	86.0625	3.175738	0.23818	1.52797	0.187056	0.320744	16.52406	0.106185163
46	89.93	3.31845	0.248884	1.550524	0.192619	0.325479	16.28371	0.111493163
47	93.8825	3.464299	0.259822	1.572915	0.198222	0.330179	16.05191	0.116896211
48	97.92	3.613284	0.270996	1.595147	0.203866	0.334846	15.82818	0.1223908
49	102.0425	3.765406	0.282405	1.617226	0.209548	0.33948	15.61209	0.127973476
50	106.25	3.920664	0.29405	1.639155	0.215269	0.344084	15.40323	0.13364085
51	110.5425	4.079059	0.305929	1.660938	0.221029	0.348656	15.20122	0.139389605
52	114.92	4.24059	0.318044	1.682579	0.226826	0.353199	15.0057	0.1452165
53	119.3825	4.405258	0.330394	1.704082	0.232661	0.357713	14.81635	0.151118384
54	123.93	4.573063	0.34298	1.72545	0.238532	0.362198	14.63286	0.157092192
55	128.5625	4.744004	0.3558	1.746687	0.24444	0.366656	14.45495	0.163134958

56	133.28	4.918081	0.368856	1.767795	0.250384	0.371087	14.28236	0.169243814
57	138.0825	5.095295	0.382147	1.788778	0.256363	0.375492	14.11482	0.175415993
58	142.97	5.275646	0.395673	1.809639	0.262377	0.379871	13.95211	0.181648834
59	147.9425	5.459133	0.409435	1.83038	0.268426	0.384225	13.79401	0.187939779
60	153	5.645756	0.423432	1.851004	0.274509	0.388554	13.64031	0.194286379

2nd to 4th floor								
α=	0.0425	kW/sec^2						
ΔH_{ch}=	27.1	kJ/g	χ_{con}=	0.601476				
y_s=	0.075	g/g						
H=	2.5	m						
r=	6.46	m						
t (sec)	Q (kW)	m_f (g/s)	m_s (g/s)	m_e(g/s)	C_o (g/m^3)	u (m/s)	τ	C_i (g/m^3)
0	0	0	0	0	#DIV/0!	0	#DIV/0!	0
1	0.0425	0.001568	0.000118	0.120774	0.001169	0.02273	233.1679	5.01211E-06
2	0.17	0.006273	0.00047	0.191717	0.002945	0.036082	146.8866	2.50264E-05
3	0.3825	0.014114	0.001059	0.25122	0.005057	0.047281	112.0954	6.99121E-05
4	0.68	0.025092	0.001882	0.304331	0.007421	0.057277	92.53273	0.00014935
5	1.0625	0.039207	0.00294	0.353145	0.009992	0.066464	79.74229	0.00027278
6	1.53	0.056458	0.004234	0.398787	0.012742	0.075054	70.61569	0.000449353
7	2.0825	0.076845	0.005763	0.441949	0.015649	0.083177	63.71916	0.000687894
8	2.72	0.100369	0.007528	0.483096	0.018699	0.090922	58.29197	0.000996868
9	3.4425	0.12703	0.009527	0.522559	0.021878	0.098349	53.88986	0.00138435
10	4.25	0.156827	0.011762	0.560583	0.025178	0.105505	50.2345	0.001858003

11	5.1425	0.18976	0.014232	0.597359	0.02859	0.112427	47.14188	0.002425055
12	6.12	0.22583	0.016937	0.633035	0.032107	0.119141	44.4851	0.003092284
13	7.1825	0.265037	0.019878	0.667732	0.035723	0.125671	42.17352	0.003866007
14	8.33	0.30738	0.023054	0.70155	0.039433	0.132036	40.14055	0.004752067
15	9.5625	0.35286	0.026464	0.734572	0.043233	0.138251	38.33609	0.005755833
16	10.88	0.401476	0.030111	0.766867	0.047117	0.144329	36.72164	0.00688219
17	12.2825	0.453229	0.033992	0.798496	0.051084	0.150282	35.26707	0.008135543
18	13.77	0.508118	0.038109	0.82951	0.05513	0.156119	33.94848	0.009519821
19	15.3425	0.566144	0.042461	0.859955	0.059251	0.161849	32.74661	0.01103848
20	17	0.627306	0.047048	0.88987	0.063445	0.167479	31.64575	0.012694508
21	18.7425	0.691605	0.05187	0.919291	0.067709	0.173016	30.63298	0.014490439
22	20.57	0.759041	0.056928	0.948248	0.072042	0.178466	29.69752	0.016428364
23	22.4825	0.829613	0.062221	0.976769	0.076441	0.183834	28.83037	0.018509939
24	24.48	0.903321	0.067749	1.00488	0.080904	0.189125	28.02386	0.020736405
25	26.5625	0.980166	0.073512	1.032603	0.08543	0.194342	27.27148	0.023108601
26	28.73	1.060148	0.079511	1.059958	0.090016	0.199491	26.56765	0.025626982
27	30.9825	1.143266	0.085745	1.086966	0.094662	0.204574	25.90754	0.028291636
28	33.32	1.22952	0.092214	1.113641	0.099365	0.209594	25.28696	0.031102303
29	35.7425	1.318911	0.098918	1.140001	0.104124	0.214555	24.70226	0.034058396
30	38.25	1.411439	0.105858	1.16606	0.108939	0.21946	24.15023	0.037159018

I.5 Smoke travel time

First of all, there is no method to calculate the time for smoke rising along plume. Therefore, assumption was made that the smoke hits the ceiling instantly. The next step is to figure out, how long does smoke travel from the center of plume where it hits the ceiling to the detector. In order to do that, numerical method was applied.

The basic idea is that $time = \frac{velocity}{distance}$. As shown in equation 13, the ceiling jet velocity is function of r. Or based on u =f(r), the velocity of ceiling jet at r from the center of the plume at ceiling level can be calculated. For distance, set Δr =0.1m, therefore, $r_{n+1} - r_n = \Delta r$. The time of smoke travel is $\frac{\Delta r}{u_{avg.}}$. Where $u_{avg.} = (u_{n+1} + u_n)/2$. Hence, the numerical equation to solve for smoke travel time is:

$$t_{tr,n+1} = t_{tr,n} + \frac{r_{n+1} - r_n}{\frac{u_{n+1} + u_n}{2}} \qquad (15)$$

The calculation is shown below in the excel spread sheet.

1st floor		
u (m/s)	r (m)	t (sec)
0.371087	6.46	9.491566
0.375943	6.36	9.22384
0.380941	6.26	8.959598
0.386088	6.16	8.698852
0.39139	6.06	8.44161
0.396854	5.96	8.187881
0.40249	5.86	7.937676
0.408305	5.76	7.691005
0.414307	5.66	7.447877
0.420508	5.56	7.208303
0.426916	5.46	6.972293

0.433543	5.36	6.73986
0.440401	5.26	6.511012
0.447502	5.16	6.285762
0.45486	5.06	6.064122
0.462489	4.96	5.846102
0.470406	4.86	5.631716
0.478627	4.76	5.420975
0.487171	4.66	5.213893
0.496058	4.56	5.010481
0.505309	4.46	4.810754
0.514949	4.36	4.614726
0.525003	4.26	4.422409
0.535499	4.16	4.233819
0.546468	4.06	4.048971
0.557944	3.96	3.867879
0.569964	3.86	3.690559
0.582568	3.76	3.517028
0.595802	3.66	3.347302
0.609717	3.56	3.181399
0.624367	3.46	3.019335
0.639814	3.36	2.86113
0.656128	3.26	2.706802
0.673386	3.16	2.556371
0.691675	3.06	2.409857
0.711093	2.96	2.267282

0.731753	2.86	2.128667
0.753782	2.76	1.994036
0.777323	2.66	1.863411
0.802546	2.56	1.736818
0.829642	2.46	1.614283
0.858835	2.36	1.495833
0.890388	2.26	1.381497
0.924609	2.16	1.271304
0.961864	2.06	1.165286
1.002589	1.96	1.063476
1.047311	1.86	0.965911
1.09667	1.76	0.872626
1.151453	1.66	0.783663
1.212642	1.56	0.699064
1.281472	1.46	0.618875
1.359526	1.36	0.543147
1.448864	1.26	0.471931
1.552225	1.16	0.405289
1.67333	1.06	0.343284
1.817372	0.96	0.285989
1.99184	0.86	0.233485
2.207963	0.76	0.185863
2.483418	0.66	0.143232
2.847824	0.56	0.105717
3.355096	0.46	0.073474

4.115454	0.36	0.046703
5.397492	0.26	0.025679
8.089155	0.16	0.010849
18.31796	0.06	0.003275

2nd to 4th floor		
u (m/s)	r (m)	t (sec)
0.183834	6.46	19.15968
0.18624	6.36	18.61924
0.188715	6.26	18.08585
0.191265	6.16	17.5595
0.193892	6.06	17.04024
0.196599	5.96	16.52806
0.199391	5.86	16.023
0.202271	5.76	15.52506
0.205245	5.66	15.03429
0.208317	5.56	14.55068
0.211491	5.46	14.07427
0.214774	5.36	13.60508
0.218172	5.26	13.14313
0.221689	5.16	12.68844
0.225334	5.06	12.24104
0.229114	4.96	11.80094
0.233036	4.86	11.36818
0.237108	4.76	10.94278

0.241341	4.66	10.52476
0.245744	4.56	10.11416
0.250327	4.46	9.71099
0.255102	4.36	9.315286
0.260083	4.26	8.927076
0.265282	4.16	8.546388
0.270716	4.06	8.173253
0.276401	3.96	7.807701
0.282356	3.86	7.449764
0.2886	3.76	7.099474
0.295156	3.66	6.756865
0.302049	3.56	6.421972
0.309307	3.46	6.09483
0.316959	3.36	5.775477
0.325041	3.26	5.463951
0.33359	3.16	5.160291
0.342651	3.06	4.864538
0.352271	2.96	4.576736
0.362505	2.86	4.296928
0.373418	2.76	4.025161
0.38508	2.66	3.761482
0.397575	2.56	3.505942
0.410998	2.46	3.258593
0.425461	2.36	3.019489
0.441092	2.26	2.78869

0.458045	2.16	2.566254
0.476501	2.06	2.352246
0.496676	1.96	2.146734
0.51883	1.86	1.949788
0.543282	1.76	1.761484
0.570422	1.66	1.581903
0.600734	1.56	1.411131
0.634832	1.46	1.249262
0.673499	1.36	1.096396
0.717757	1.26	0.952641
0.768961	1.16	0.818116
0.828956	1.06	0.692953
0.900313	0.96	0.577297
0.986743	0.86	0.471312
1.093809	0.76	0.375184
1.230267	0.66	0.289128
1.410791	0.56	0.213401
1.66209	0.46	0.148316
2.038766	0.36	0.094274
2.673879	0.26	0.051835
4.007309	0.16	0.0219
9.074585	0.06	0.006612

I.6 Smoke Detector Response Time

For first floor: $t_{resp.} = t_{local} + t_{travel} = 56 \sec + 9.5\, sec = 65.5\, sec$

For second to fourth floor: $t_{resp.} = t_{local} + t_{travel} = 23\,\mathrm{sec} + 19.2\ sec = 42.2\ sec$

Appendix J: Design of Automatic Sprinkler System

J1: Sprinkler System 1Hydraulic Calculations

System 1-Pipe Schedule Method-K5.6 Sprinklers

Step No.	Nozzle ID. and location	Flow in gpm	Pipe Size	Pipe Fiitings and Devices	Equiv. Pipe Length	Friction Loss psi/Foot	Summary Pressure	Notes
	1	12.00	1.00	12.00	12.00		4.59	Pmin=4.59psi
1	to 1	-	1.05	0.00	0.00	0.05	0.00	Qmin=K√P
	2	12.00	40.00	0.00	12.00		0.61	Qmin=12gpm
	2	12.77	1.00	12.00	12.00		5.20	
2	to 2	-	1.05	0.00	0.00	0.19	0.00	
	3	24.77	40.00	0.00	12.00		2.32	
	3	15.35	1.25	12.00	12.00		7.52	
3	to 3	-	1.38	0.00	0.00	0.12	0.00	
	4	40.12	40.00	0.00	12.00		1.49	
	4	16.80	1.50	10.00	10.00		9.00	K=Q/√P
4	to 8	-	1.61	0.00	0.00	0.11	0.00	K=56.92/√10.12
	8	56.92	40.00	0.00	10.00		1.12	K=17.89
	8	56.91	2.00	46.00	46.00		10.12	Q=K√p
5	to 5-6-7-8	-	2.07	ELL	4.00	0.12	0.00	Q=17.89*√10.12
	12	113.83	40.00	0.00	50.00		5.97	Q=56.91
	12	56.91	2.00	46.00	46.00		16.09	
6	to 9-10-11-12	-	2.07	ELL	4.00	0.25	0.00	
	13	170.74	40.00	0.00	50.00		12.64	

	13	30.02	2.00	30.00	30.00		28.73	
7	to 13	-	2.07	0.00	0.00	0.34	0.00	
	14	200.76	40.00	0.00	30.00		10.23	
	14	-	3.00	70.00	70.00		38.97	
8	to 14	-	3.07	0.00	0.00	0.05	0.00	
	15	200.76	40.00	0.00	70.00		3.49	
	15	-	3.50	63.00	63.00		42.46	
9	to 15	-	3.55	0.00	0.00	0.02	0.00	
	16	200.76	40.00	0.00	63.00		1.55	
	16	-	3.50	45.00	45.00		44.00	
10	to 16	-	3.55	0.00	0.00	0.02	0.00	
	17	200.76	40.00	0.00	45.00		1.11	
	17	-	4.00	9.00	9.00		45.11	
11	to 17	-	4.03	0.00	0.00	0.01	0.00	
	18	200.76	40.00	0.00	9.00		0.12	
Totals		200.76					45.23	

System 1-Pipe Schedule Method-K8.0 Sprinklers

Step No.	Nozzle ID. and location	Flow in gpm	Pipe Size	Pipe Fiitings and Devices	Equiv. Pipe Length	Friction Loss psi/Foot	Summary Pressure	Notes
	1	12.00	1.00	12.00	12.00		2.25	Pmin=2.25psi
1	to 1	-	1.05	0.00	0.00	0.05	0.00	Qmin=K√P
	2	12.00	40.00	0.00	12.00		0.61	Qmin=12gpm

	2	13.52	1.00	12.00	12.00		2.86	
2	to 2	-	1.05	0.00	0.00	0.20	0.00	
	3	25.52	40.00	0.00	12.00		2.45	
	3	18.43	1.25	12.00	12.00		5.31	
3	to 3	-	1.38	0.00	0.00	0.15	0.00	
	4	43.95	40.00	0.00	12.00		1.76	
	4	21.27	1.50	10.00	10.00		7.07	K=Q/√P
4	to 8	-	1.61	0.00	0.00	0.14	0.00	K=65.23/√8.51
	8	65.23	40.00	0.00	10.00		1.44	K=22.36
	8	65.22	2.00	46.00	46.00		8.51	Q=K√p
5	to 5-6-7-8	-	2.07	ELL	4.00	0.15	0.00	Q=22.36*√8.51
	12	130.45	40.00	0.00	50.00		7.68	Q=65.22
	12	65.22	2.00	46.00	46.00		16.19	
6	to 9-10-11-12	-	2.07	ELL	4.00	0.33	0.00	
	13	195.67	40.00	0.00	50.00		16.26	
	13	31.90	2.00	30.00	30.00		32.46	
7	to 13	-	2.07	0.00	0.00	0.43	0.00	
	14	227.57	40.00	0.00	30.00		12.90	
	14	-	3.00	70.00	70.00		45.36	
8	to 14	-	3.07	0.00	0.00	0.06	0.00	
	15	227.57	40.00	0.00	70.00		4.40	
	15	-	3.50	63.00	63.00		49.76	
9	to 15	-	3.55	0.00	0.00	0.03	0.00	
	16	227.57	40.00	0.00	63.00		1.95	
	16	-	3.50	45.00	45.00		51.71	

10	to 16	-	3.55	0.00	0.00	0.03	0.00	
	17	227.57	40.00	0.00	45.00		1.39	
	17	-	4.00	9.00	9.00		53.10	
11	to 17	-	4.03	0.00	0.00	0.02	0.00	
	18	227.57	40.00	0.00	9.00		0.15	
Totals		227.57					53.26	

System 1-Resized Mains-K5.6 Sprinklers

Step No.	Nozzle ID. and location	Flow in gpm	Pipe Size	Pipe Fiitings and Devices	Equiv. Pipe Length	Friction Loss psi/Foot	Summar y Pressure	Notes
	1	12.00	1.00	12.00	12.00		4.59	Pmin=4.59psi
1	to 1	-	1.05	0.00	0.00	0.05	0.00	Qmin=K√P
	2	12.00	40.00	0.00	12.00		0.61	Qmin=12gpm
	2	12.77	1.00	12.00	12.00		5.20	
2	to 2	-	1.05	0.00	0.00	0.19	0.00	
	3	24.77	40.00	0.00	12.00		2.32	
	3	15.35	1.25	12.00	12.00		7.52	
3	to 3	-	1.38	0.00	0.00	0.12	0.00	
	4	40.12	40.00	0.00	12.00		1.49	
	4	16.80	1.50	10.00	10.00		9.00	K=Q/√P
4	to 8	-	1.61	0.00	0.00	0.11	0.00	K=56.92/√10.12
	8	56.92	40.00	0.00	10.00		1.12	K=17.89
	8	56.91	2.50	46.00	46.00		10.12	Q=K√p
5	to 5-6-7-8	-	2.47	ELL	4.00	0.05	0.00	Q=17.89*√10.1

								2
	12	113.83	40.00	0.00	50.00		2.51	Q=56.91
	12	56.91	2.50	46.00	46.00		12.64	
6	to 9-10-11-12	-	2.47	ELL	4.00	0.11	0.00	
	13	170.74	40.00	0.00	50.00		5.32	
	13	23.73	3.00	30.00	30.00		17.96	
7	to 13	-	3.07	0.00	0.00	0.05	0.00	
	14	194.47	40.00	0.00	30.00		1.41	
	14	-	3.50	70.00	70.00		19.37	
8	to 14	-	3.55	0.00	0.00	0.02	0.00	
	15	194.47	40.00	0.00	70.00		1.62	
	15	-	3.50	63.00	63.00		20.99	
9	to 15	-	3.55	0.00	0.00	0.02	0.00	
	16	194.47	40.00	0.00	63.00		1.46	
	16	-	3.50	45.00	45.00		22.44	
10	to 16	-	3.55	0.00	0.00	0.02	0.00	
	17	194.47	40.00	0.00	45.00		1.04	
	17	-	4.00	9.00	9.00		23.49	
11	to 17	-	4.03	0.00	0.00	0.01	0.00	
	18	194.47	40.00	0.00	9.00		0.11	
Totals		194.47					23.60	

System 1-Resized Mains-K8.0 Sprinklers

Step No.	Nozzle ID. and location	Flow in gpm	Pipe Size	Pipe Fiitings and Devices	Equiv. Pipe Length	Friction Loss psi/Foot	Summary Pressure	Notes
	1	12.00	1.00	12.00	12.00		2.25	Pmin=2.25psi
1	to 1	-	1.05	0.00	0.00	0.05	0.00	Qmin=K√P
	2	12.00	40.00	0.00	12.00		0.61	Qmin=12gpm
	2	13.52	1.00	12.00	12.00		2.86	
2	to 2	-	1.05	0.00	0.00	0.20	0.00	
	3	25.52	40.00	0.00	12.00		2.45	
	3	18.43	1.25	12.00	12.00		5.31	
3	to 3	-	1.38	0.00	0.00	0.15	0.00	
	4	43.95	40.00	0.00	12.00		1.76	
	4	21.27	1.50	10.00	10.00		7.07	K=Q/√P
4	to 8	-	1.61	0.00	0.00	0.14	0.00	K=65.23/√8.51
	8	65.23	40.00	0.00	10.00		1.44	K=22.36
	8	65.22	2.50	46.00	46.00		8.51	Q=K√p
5	to 5-6-7-8	-	2.47	ELL	4.00	0.06	0.00	Q=22.36*√8.51
	12	130.45	40.00	0.00	50.00		3.23	Q=65.22
	12	65.22	2.50	46.00	46.00		11.74	
6	to 9-10-11-12	-	2.47	ELL	4.00	0.14	0.00	
	13	195.67	40.00	0.00	50.00		6.84	
	13	24.14	3.00	30.00	30.00		18.59	
7	to 13	-	3.07	0.00	0.00	0.06	0.00	
	14	219.81	40.00	0.00	30.00		1.77	
	14	-	3.50	70.00	70.00		20.36	

8	to 14	-	3.55	0.00	0.00	0.03	0.00	
	15	219.81	40.00	0.00	70.00		2.03	
	15	-	3.50	63.00	63.00		22.39	
9	to 15	-	3.55	0.00	0.00	0.03	0.00	
	16	219.81	40.00	0.00	63.00		1.83	
	16	-	3.50	45.00	45.00		24.22	
10	to 16	-	3.55	0.00	0.00	0.03	0.00	
	17	219.81	40.00	0.00	45.00		1.31	
	17	-	4.00	9.00	9.00		25.53	
11	to 17	-	4.03	0.00	0.00	0.02	0.00	
	18	219.81	40.00	0.00	9.00		0.14	
Totals		219.81					25.67	

System 1-Resized Branchlines-K5.6 Sprinklers

Step No.	Nozzle ID. and location	Flow in gpm	Pipe Size	Pipe Fiitings and Devices	Equiv. Pipe Length	Friction Loss psi/Foot	Summary Pressure	Notes
	1	12.00	1.25	12.00	12.00		4.59	Pmin=4.59psi
1	to 1	-	1.38	0.00	0.00	0.01	0.00	Qmin=K√P
	2	12.00	40.00	0.00	12.00		0.16	Qmin=12gpm
	2	12.20	1.25	12.00	12.00		4.75	
2	to 2	-	1.38	0.00	0.00	0.05	0.00	
	3	24.20	40.00	0.00	12.00		0.58	
	3	12.93	1.50	12.00	12.00		5.33	
3	to 3	-	1.61	0.00	0.00	0.05	0.00	

	4	37.14	40.00	0.00	12.00		0.61	
	4	13.65	1.50	10.00	10.00		5.94	K=Q/√P
4	to 8	-	1.61	0.00	0.00	0.09	0.00	K=50.79/√6.84
	8	50.79	40.00	0.00	10.00		0.91	K=19.42
	8	50.78	2.00	46.00	46.00		6.85	Q=K√p
5	to 5-6-7-8	-	2.07	ELL	4.00	0.10	0.00	Q=19.42*√6.84
	12	101.57	40.00	0.00	50.00		4.84	Q=50.78
	12	50.78	2.00	46.00	46.00		11.68	
6	to 9-10-11-12	-	2.07	ELL	4.00	0.20	0.00	
	13	152.35	40.00	0.00	50.00		10.24	
	13	26.22	2.00	30.00	30.00		21.92	
7	to 13	-	2.07	0.00	0.00	0.27	0.00	
	14	178.57	40.00	0.00	30.00		8.24	
	14	-	3.00	70.00	70.00		30.16	
8	to 14	-	3.07	0.00	0.00	0.04	0.00	
	15	178.57	40.00	0.00	70.00		2.81	
	15	-	3.50	63.00	63.00		32.97	
9	to 15	-	3.55	0.00	0.00	0.02	0.00	
	16	178.57	40.00	0.00	63.00		1.25	
	16	-	3.50	45.00	45.00		34.22	
10	to 16	-	3.55	0.00	0.00	0.02	0.00	
	17	178.57	40.00	0.00	45.00		0.89	
	17	-	4.00	9.00	9.00		35.11	
11	to 17	-	4.03	0.00	0.00	0.01	0.00	
	18	178.57	40.00	0.00	9.00		0.10	

Totals		178.57					35.20	

System 1-Resized Branchlines-K8.0 Sprinklers

Step No.	Nozzle ID. and location	Flow in gpm	Pipe Size	Pipe Fiitings and Devices	Equiv. Pipe Length	Friction Loss psi/Foot	Summary Pressure	Notes
	1	12.00	1.25	12.00	12.00		2.25	Pmin=2.25psi
1	to 1	-	1.38	0.00	0.00	0.01	0.00	Qmin=K√P
	2	12.00	40.00	0.00	12.00		0.16	Qmin=12gpm
	2	12.42	1.25	12.00	12.00		2.41	
2	to 2	-	1.38	0.00	0.00	0.05	0.00	
	3	24.42	40.00	0.00	12.00		0.59	
	3	13.87	1.50	12.00	12.00		3.00	
3	to 3	-	1.61	0.00	0.00	0.05	0.00	
	4	38.28	40.00	0.00	12.00		0.64	
	4	15.28	1.50	10.00	10.00		3.65	K=Q/√P
4	to 8	-	1.61	0.00	0.00	0.10	0.00	K=53.56/√4.65
	8	53.56	40.00	0.00	10.00		1.00	K=24.84
	8	53.56	2.00	46.00	46.00		4.65	Q=K√p
5	to 5-6-7-8	-	2.07	ELL	4.00	0.11	0.00	Q=24.84*√4.65
	12	107.12	40.00	0.00	50.00		5.34	Q=53.56
	12	53.56	2.00	46.00	46.00		9.98	
6	to 9-10-11-12	-	2.07	ELL	4.00	0.23	0.00	
	13	160.68	40.00	0.00	50.00		11.30	
	13	25.83	2.00	30.00	30.00		21.28	

7	to 13	-	2.07	0.00	0.00	0.30	0.00	
	14	186.52	40.00	0.00	30.00		8.93	
	14	-	3.00	70.00	70.00		30.21	
8	to 14	-	3.07	0.00	0.00	0.04	0.00	
	15	186.52	40.00	0.00	70.00		3.05	
	15	-	3.50	63.00	63.00		33.26	
9	to 15	-	3.55	0.00	0.00	0.02	0.00	
	16	186.52	40.00	0.00	63.00		1.35	
	16	-	3.50	45.00	45.00		34.61	
10	to 16	-	3.55	0.00	0.00	0.02	0.00	
	17	186.52	40.00	0.00	45.00		0.96	
	17	-	4.00	9.00	9.00		35.57	
11	to 17	-	4.03	0.00	0.00	0.01	0.00	
	18	186.52	40.00	0.00	9.00		0.10	
Totals		186.52					35.68	

System 1-Resized Mains and Branchlines-K5.6 Sprinklers

Step No.	Nozzle ID. and location	Flow in gpm	Pipe Size	Pipe Fiitings and Devices	Equiv. Pipe Length	Friction Loss psi/Foot	Summary Pressure	Notes
	1	12.00	1.25	12.00	12.00		4.59	Pmin=4.59psi
1	to 1	-	1.38	0.00	0.00	0.01	0.00	Qmin=K√P
	2	12.00	40.00	0.00	12.00		0.16	Qmin=12gpm
	2	12.20	1.25	12.00	12.00		4.75	
2	to 2	-	1.38	0.00	0.00	0.05	0.00	

	3	24.20	40.00	0.00	12.00		0.58	
	3	12.93	1.50	12.00	12.00		5.33	
3	to 3	-	1.61	0.00	0.00	0.05	0.00	
	4	37.14	40.00	0.00	12.00		0.61	
	4	13.65	1.50	10.00	10.00		5.94	K=Q/√P
4	to 8	-	1.61	0.00	0.00	0.09	0.00	K=50.79/√6.84
	8	50.79	40.00	0.00	10.00		0.91	K=19.42
	8	50.78	2.50	46.00	46.00		6.85	Q=K√p
5	to 5-6-7-8	-	2.47	ELL	4.00	0.04	0.00	Q=19.42*√6.84
	12	101.57	40.00	0.00	50.00		2.04	Q=50.78
	12	50.78	2.50	46.00	46.00		8.88	
6	to 9-10-11-12	-	2.47	ELL	4.00	0.09	0.00	
	13	152.35	40.00	0.00	50.00		4.31	
	13	20.34	3.00	30.00	30.00		13.19	
7	to 13	-	3.07	0.00	0.00	0.04	0.00	
	14	172.69	40.00	0.00	30.00		1.13	
	14	-	3.50	70.00	70.00		14.32	
8	to 14	-	3.55	0.00	0.00	0.02	0.00	
	15	172.69	40.00	0.00	70.00		1.30	
	15	-	3.50	63.00	63.00		15.63	
9	to 15	-	3.55	0.00	0.00	0.02	0.00	
	16	172.69	40.00	0.00	63.00		1.17	
	16	-	3.50	45.00	45.00		16.80	
10	to 16	-	3.55	0.00	0.00	0.02	0.00	
	17	172.69	40.00	0.00	45.00		0.84	

	17	-	4.00	9.00	9.00		17.63	
11	to 17	-	4.03	0.00	0.00	0.01	0.00	
	18	172.69	40.00	0.00	9.00		0.09	
Totals		172.69					17.72	

System 1-Resized Mains and Branchlines-K8.0 Sprinklers

Step No.	Nozzle ID. and location	Flow in gpm	Pipe Size	Pipe Fiitings and Devices	Equiv. Pipe Length	Friction Loss psi/Foot	Summary Pressure	Notes
	1	12.00	1.25	12.00	12.00		2.25	Pmin=2.25psi
1	to 1	-	1.38	0.00	0.00	0.01	0.00	Qmin=K√P
	2	12.00	40.00	0.00	12.00		0.16	Qmin=12gpm
	2	12.42	1.25	12.00	12.00		2.41	
2	to 2	-	1.38	0.00	0.00	0.05	0.00	
	3	24.42	40.00	0.00	12.00		0.59	
	3	13.87	1.50	12.00	12.00		3.00	
3	to 3	-	1.61	0.00	0.00	0.05	0.00	
	4	38.28	40.00	0.00	12.00		0.64	
	4	15.28	1.50	10.00	10.00		3.65	K=Q/√P
4	to 8	-	1.61	0.00	0.00	0.10	0.00	K=53.56/√4.65
	8	53.56	40.00	0.00	10.00		1.00	K=24.84
	8	53.56	2.50	46.00	46.00		4.65	Q=K√p
5	to 5-6-7-8	-	2.47	ELL	4.00	0.04	0.00	Q=24.84*√4.65
	12	107.12	40.00	0.00	50.00		2.25	Q=53.56
	12	53.56	2.50	46.00	46.00		6.89	

6	to 9-10-11-12	-	2.47	ELL	4.00	0.10	0.00	
	13	160.68	40.00	0.00	50.00		4.75	
	13	19.11	3.00	30.00	30.00		11.65	
7	to 13	-	3.07	0.00	0.00	0.04	0.00	
	14	179.80	40.00	0.00	30.00		1.22	
	14	-	3.50	70.00	70.00		12.87	
8	to 14	-	3.55	0.00	0.00	0.02	0.00	
	15	179.80	40.00	0.00	70.00		1.40	
	15	-	3.50	63.00	63.00		14.27	
9	to 15	-	3.55	0.00	0.00	0.02	0.00	
	16	179.80	40.00	0.00	63.00		1.26	
	16	-	3.50	45.00	45.00		15.53	
10	to 16	-	3.55	0.00	0.00	0.02	0.00	
	17	179.80	40.00	0.00	45.00		0.90	
	17	-	4.00	9.00	9.00		16.43	
11	to 17	-	4.03	0.00	0.00	0.01	0.00	
	18	179.80	40.00	0.00	9.00		0.10	
Totals		179.80					16.53	

J2: Sprinkler System 2 Hydraulic Calculations

System 2-Pipe Schedule Method-K5.6 Sprinklers

Step No.	Nozzle ID. and location	Flow in gpm	Pipe Size	Pipe Fiitings and Devices	Equiv. Pipe Length	Friction Loss psi/Foot	Summary Pressure	Notes

	1	19.50	1.00	13.00	13.00		12.12	Pmin=12.12psi
1	to 1	-	1.05	0.00	0.00	0.12	0.00	Qmin=K√P
	2	19.50	40.00	0.00	13.00		1.61	Qmin=19.5gpm
	2	20.75	1.00	13.00	13.00		13.73	
2	to 2	-	1.05	0.00	0.00	0.47	0.00	
	3	40.25	40.00	0.00	13.00		6.17	
	3	24.98	1.25	13.00	13.00		19.90	
3	to 3	-	1.38	0.00	0.00	0.30	0.00	
	4	65.24	40.00	0.00	13.00		3.96	
	4	27.36	1.50	13.00	13.00		23.87	K=Q/√P
4	to 4	-	1.61	0.00	0.00	0.28	0.00	K=92.59/√27.44
	5	92.60	40.00	0.00	13.00		3.58	K=17.67
	5	-	2.00	15.00	15.00		27.45	
5	to 5	-	2.07	ELL	4.00	0.08	0.00	
	6	92.60	40.00	0.00	19.00		1.55	
	6	95.13	2.50	82.00	82.00		29.00	Q=K√p
6	to 7-8-9-10-6	-	2.47	ELL	4.00	0.13	0.00	Q=17.67*√28.99
	11	187.73	40.00	0.00	86.00		10.90	Q=95.13
	11	-	3.00	45.00	45.00		39.90	
7	to	-	3.07	0.00	0.00	0.04	0.00	
	12	187.73	40.00	0.00	45.00		1.98	
	12	-	3.50	40.00	40.00		41.88	
8	to	-	3.55	0.00	0.00	0.02	0.00	
	13	187.73	40.00	0.00	40.00		0.87	
	13	-	4.00	16.50	16.50		42.75	

9	to	-	4.03	ELL	4.00	0.01	0.00	
	14	187.73	40.00	0.00	20.50		0.24	
Totals		187.73					42.99	

System 2-Pipe Schedule Method-K8.0 Sprinklers

Step No.	Nozzle ID. and location	Flow in gpm	Pipe Size	Pipe Fiitings and Devices	Equiv. Pipe Length	Friction Loss psi/Foot	Summary Pressure	Notes
	1	19.50	1.00	13.00	13.00		5.94	Pmin=12.12psi
1	to 1	-	1.05	0.00	0.00	0.12	0.00	Qmin=K√P
	2	19.50	40.00	0.00	13.00		1.61	Qmin=19.5gpm
	2	21.99	1.00	13.00	13.00		7.55	
2	to 2	-	1.05	0.00	0.00	0.50	0.00	
	3	41.49	40.00	0.00	13.00		6.52	
	3	30.02	1.25	13.00	13.00		14.08	
3	to 3	-	1.38	0.00	0.00	0.36	0.00	
	4	71.51	40.00	0.00	13.00		4.70	
	4	34.67	1.50	13.00	13.00		18.78	K=Q/√P
4	to 4	-	1.61	0.00	0.00	0.35	0.00	K=106.17/√23.38
	5	106.17	40.00	0.00	13.00		4.61	K=21.95
	5	-	2.00	15.00	15.00		23.38	
5	to 5	-	2.07	ELL	4.00	0.10	0.00	
	6	106.17	40.00	0.00	19.00		1.99	

	6	110.58	2.50	82.00	82.00		25.38	Q=K√p
6	to 7-8-9-10-6	-	2.47	ELL	4.00	0.17	0.00	Q=21.95*√25.38
	11	216.75	40.00	0.00	86.00		14.23	Q=110.58
	11	-	3.00	45.00	45.00		39.61	
7	to	-	3.07	0.00	0.00	0.06	0.00	
	12	216.75	40.00	0.00	45.00		2.58	
	12	-	3.50	40.00	40.00		42.19	
8	to	-	3.55	0.00	0.00	0.03	0.00	
	13	216.75	40.00	0.00	40.00		1.13	
	13	-	4.00	16.50	16.50		43.32	
9	to	-	4.03	ELL	4.00	0.02	0.00	
	14	216.75	40.00	0.00	20.50		0.31	
Totals		216.75					43.64	

System 2-Resized Mains-K5.6 Sprinklers

Step No.	Nozzle ID. and location	Flow in gpm	Pipe Size	Pipe Fiitings and Devices	Equiv. Pipe Length	Friction Loss psi/Foot	Summar y Pressure	Notes
	1	19.50	1.00	13.00	13.00		12.12	Pmin=12.12psi
1	to 1	-	1.05	0.00	0.00	0.12	0.00	Qmin=K√P
	2	19.50	40.00	0.00	13.00		1.61	Qmin=19.5gpm
	2	20.75	1.00	13.00	13.00		13.73	
2	to 2	-	1.05	0.00	0.00	0.47	0.00	
	3	40.25	40.00	0.00	13.00		6.17	

	3	24.98	1.25	13.00	13.00		19.90	
3	to 3	-	1.38	0.00	0.00	0.30	0.00	
	4	65.24	40.00	0.00	13.00		3.96	
	4	27.36	1.50	13.00	13.00		23.87	K=Q/√P
4	to 4	-	1.61	0.00	0.00	0.28	0.00	K=92.59/√27.44
	5	92.60	40.00	0.00	13.00		3.58	K=17.67
	5	-	2.50	15.00	15.00		27.45	
5	to 5	-	2.47	ELL	4.00	0.03	0.00	
	6	92.60	40.00	0.00	19.00		0.65	
	6	95.13	3.00	82.00	82.00		28.10	Q=K√p
6	to 7-8-9-10-6	-	3.07	ELL	4.00	0.04	0.00	Q=17.67*√28.99
	11	187.73	40.00	0.00	86.00		3.79	Q=95.13
	11	-	3.50	45.00	45.00		31.88	
7	to	-	3.55	0.00	0.00	0.02	0.00	
	12	187.73	40.00	0.00	45.00		0.98	
	12	-	4.00	40.00	40.00		32.86	
8	to	-	4.03	0.00	0.00	0.01	0.00	
	13	187.73	40.00	0.00	40.00		0.47	
	13	-	4.00	16.50	16.50		33.33	
9	to	-	4.03	ELL	4.00	0.01	0.00	
	14	187.73	40.00	0.00	20.50		0.24	
Totals		187.73					33.57	

System 2-Resized Mains-K8.0 Sprinklers

Step No.	Nozzle ID. and location	Flow in gpm	Pipe Size	Pipe Fiitings and Devices	Equiv. Pipe Length	Friction Loss psi/Foot	Summar y Pressure	Notes
	1	19.50	1.00	13.00	13.00		5.94	Pmin=12.12psi
1	to 1	-	1.05	0.00	0.00	0.12	0.00	Qmin=K√P
	2	19.50	40.00	0.00	13.00		1.61	Qmin=19.5gpm
	2	21.99	1.00	13.00	13.00		7.55	
2	to 2	-	1.05	0.00	0.00	0.50	0.00	
	3	41.49	40.00	0.00	13.00		6.52	
	3	30.02	1.25	13.00	13.00		14.08	
3	to 3	-	1.38	0.00	0.00	0.36	0.00	
	4	71.51	40.00	0.00	13.00		4.70	
	4	34.67	1.50	13.00	13.00		18.78	K=Q/√P
4	to 4	-	1.61	0.00	0.00	0.35	0.00	K=106.17/√23.3 8
	5	106.17	40.00	0.00	13.00		4.61	K=21.95
	5	-	2.50	15.00	15.00		23.38	
5	to 5	-	2.47	ELL	4.00	0.04	0.00	
	6	106.17	40.00	0.00	19.00		0.84	
	6	110.58	3.00	82.00	82.00		24.22	Q=K√p
6	to 7-8-9-10-6	-	3.07	ELL	4.00	0.06	0.00	Q=21.95*√25.38
	11	216.75	40.00	0.00	86.00		4.94	Q=110.58
	11	-	3.50	45.00	45.00		29.16	
7	to	-	3.55	0.00	0.00	0.03	0.00	
	12	216.75	40.00	0.00	45.00		1.27	

	12	-	4.00	40.00	40.00		30.44	
8	to	-	4.03	0.00	0.00	0.02	0.00	
	13	216.75	40.00	0.00	40.00		0.61	
	13	-	4.00	16.50	16.50		31.05	
9	to	-	4.03	ELL	4.00	0.02	0.00	
	14	216.75	40.00	0.00	20.50		0.31	
Totals		216.75					31.36	

System 2-Resized Branchlines-K5.6 Sprinklers

Step No.	Nozzle ID. and location	Flow in gpm	Pipe Size	Pipe Fiitings and Devices	Equiv. Pipe Length	Friction Loss psi/Foot	Summary Pressure	Notes
	1	19.50	1.25	13.00	13.00		12.12	Pmin=12.12psi
1	to 1	-	1.38	0.00	0.00	0.03	0.00	Qmin=K√P
	2	19.50	40.00	0.00	13.00		0.42	Qmin=19.5gpm
	2	19.83	1.25	13.00	13.00		12.54	
2	to 2	-	1.38	0.00	0.00	0.12	0.00	
	3	39.33	40.00	0.00	13.00		1.55	
	3	21.03	1.50	13.00	13.00		14.10	
3	to 3	-	1.61	0.00	0.00	0.12	0.00	
	4	60.36	40.00	0.00	13.00		1.62	
	4	22.20	2.00	13.00	13.00		15.72	K=Q/√P
4	to 4	-	2.07	0.00	0.00	0.07	0.00	K=82.56/√16.58
	5	82.57	40.00	0.00	13.00		0.86	K=20.28
	5	-	2.00	15.00	15.00		16.58	

5	to 5	-	2.07	ELL	4.00	0.07	0.00	
	6	82.57	40.00	0.00	19.00		1.25	
	6	85.63	2.50	82.00	82.00		17.83	Q=K√p
6	to 7-8-9-10-6	-	2.47	ELL	4.00	0.10	0.00	Q=20.28*√17.83
	11	168.20	40.00	0.00	86.00		8.90	Q=85.63
	11	-	3.00	45.00	45.00		26.73	
7	to	-	3.07	0.00	0.00	0.04	0.00	
	12	168.20	40.00	0.00	45.00		1.62	
	12	-	3.50	40.00	40.00		28.35	
8	to	-	3.55	0.00	0.00	0.02	0.00	
	13	168.20	40.00	0.00	40.00		0.71	
	13	-	4.00	16.50	16.50		29.05	
9	to	-	4.03	ELL	4.00	0.01	0.00	
	14	168.20	40.00	0.00	20.50		0.20	
Totals		168.20					29.25	

System 2-Resized Branchlines-K8.0 Sprinklers

Step No.	Nozzle ID. and location	Flow in gpm	Pipe Size	Pipe Fiitings and Devices	Equiv. Pipe Length	Friction Loss psi/Foot	Summary Pressure	Notes
	1	19.50	1.25	13.00	13.00		5.94	Pmin=12.12psi
1	to 1	-	1.38	0.00	0.00	0.03	0.00	Qmin=K√P
	2	19.50	40.00	0.00	13.00		0.42	Qmin=19.5gpm
	2	20.18	1.25	13.00	13.00		6.36	

2	to 2	-	1.38	0.00	0.00	0.12	0.00	
	3	39.68	40.00	0.00	13.00		1.58	
	3	22.55	1.50	13.00	13.00		7.95	
3	to 3	-	1.61	0.00	0.00	0.13	0.00	
	4	62.23	40.00	0.00	13.00		1.72	
	4	24.86	2.00	13.00	13.00		9.66	K=Q/√P
4	to 4	-	2.07	0.00	0.00	0.07	0.00	K=87.09/√10.61
	5	87.10	40.00	0.00	13.00		0.95	K=26.74
	5	-	2.00	15.00	15.00		10.61	
5	to 5	-	2.07	ELL	4.00	0.07	0.00	
	6	87.10	40.00	0.00	19.00		1.38	
	6	92.59	2.50	82.00	82.00		11.99	Q=K√p
6	to 7-8-9-10-6	-	2.47	ELL	4.00	0.12	0.00	Q=26.74*√11.99
	11	179.69	40.00	0.00	86.00		10.06	Q=92.59
	11	-	3.00	45.00	45.00		22.04	
7	to	-	3.07	0.00	0.00	0.04	0.00	
	12	179.69	40.00	0.00	45.00		1.83	
	12	-	3.50	40.00	40.00		23.87	
8	to	-	3.55	0.00	0.00	0.02	0.00	
	13	179.69	40.00	0.00	40.00		0.80	
	13	-	4.00	16.50	16.50		24.67	
9	to	-	4.03	ELL	4.00	0.01	0.00	
	14	179.69	40.00	0.00	20.50		0.22	
Totals		179.69					24.89	

System 2-Resized Mains and Branchlines-K5.6 Sprinklers

Step No.	Nozzle ID. and location	Flow in gpm	Pipe Size	Pipe Fiitings and Devices	Equiv. Pipe Length	Friction Loss psi/Foot	Summar y Pressure	Notes
	1	19.50	1.25	13.00	13.00		12.12	Pmin=12.12psi
1	to 1	-	1.38	0.00	0.00	0.03	0.00	Qmin=K√P
	2	19.50	40.00	0.00	13.00		0.42	Qmin=19.5gpm
	2	19.83	1.25	13.00	13.00		12.54	
2	to 2	-	1.38	0.00	0.00	0.12	0.00	
	3	39.33	40.00	0.00	13.00		1.55	
	3	21.03	1.50	13.00	13.00		14.10	
3	to 3	-	1.61	0.00	0.00	0.12	0.00	
	4	60.36	40.00	0.00	13.00		1.62	
	4	22.20	2.00	13.00	13.00		15.72	K=Q/√P
4	to 4	-	2.07	0.00	0.00	0.07	0.00	K=82.56/√16.58
	5	82.57	40.00	0.00	13.00		0.86	K=20.28
	5	-	2.50	15.00	15.00		16.58	
5	to 5	-	2.47	ELL	4.00	0.03	0.00	
	6	82.57	40.00	0.00	19.00		0.53	
	6	85.63	3.00	82.00	82.00		17.10	Q=K√p
6	to 7-8-9-10-6	-	3.07	ELL	4.00	0.04	0.00	Q=20.28*√17.83
	11	168.20	40.00	0.00	86.00		3.09	Q=85.63
	11	-	3.50	45.00	45.00		20.19	
7	to	-	3.55	0.00	0.00	0.02	0.00	
	12	168.20	40.00	0.00	45.00		0.80	

	12	-	4.00	40.00	40.00		20.99	
8	to	-	4.03	0.00	0.00	0.01	0.00	
	13	168.20	40.00	0.00	40.00		0.38	
	13	-	4.00	16.50	16.50		21.37	
9	to	-	4.03	ELL	4.00	0.01	0.00	
	14	168.20	40.00	0.00	20.50		0.20	
Totals		168.20					21.57	

System 2-Resized Mains and Branchlines-K8.0 Sprinklers

Step No.	Nozzle ID. and location	Flow in gpm	Pipe Size	Pipe Fiitings and Devices	Equiv. Pipe Length	Friction Loss psi/Foot	Summar y Pressure	Notes
	1	19.50	1.25	13.00	13.00		5.94	Pmin=12.12psi
1	to 1	-	1.38	0.00	0.00	0.03	0.00	Qmin=K√P
	2	19.50	40.00	0.00	13.00		0.42	Qmin=19.5gpm
	2	20.18	1.25	13.00	13.00		6.36	
2	to 2	-	1.38	0.00	0.00	0.12	0.00	
	3	39.68	40.00	0.00	13.00		1.58	
	3	22.55	1.50	13.00	13.00		7.95	
3	to 3	-	1.61	0.00	0.00	0.13	0.00	
	4	62.23	40.00	0.00	13.00		1.72	
	4	24.86	2.00	13.00	13.00		9.66	K=Q/√P
4	to 4	-	2.07	0.00	0.00	0.07	0.00	K=87.09/√10.61
	5	87.10	40.00	0.00	13.00		0.95	K=26.74
	5	-	2.50	15.00	15.00		10.61	

5	to 5	-	2.47	ELL	4.00	0.03	0.00	
	6	87.10	40.00	0.00	19.00		0.58	
	6	92.59	3.00	82.00	82.00		11.19	Q=K√p
6	to 7-8-9-10-6	-	3.07	ELL	4.00	0.04	0.00	Q=26.74*√11.99
	11	179.69	40.00	0.00	86.00		3.49	Q=92.59
	11	-	3.50	45.00	45.00		14.68	
7	to	-	3.55	0.00	0.00	0.02	0.00	
	12	179.69	40.00	0.00	45.00		0.90	
	12	-	4.00	40.00	40.00		15.58	
8	to	-	4.03	0.00	0.00	0.01	0.00	
	13	179.69	40.00	0.00	40.00		0.43	
	13	-	4.00	16.50	16.50		16.01	
9	to	-	4.03	ELL	4.00	0.01	0.00	
	14	179.69	40.00	0.00	20.50		0.22	
Totals		179.69					16.23	

J3: Sprinkler System Cost

System 1- Pipe Schedule

Pipe size	Qty(L.F.)	Unit hours	Unit Price	Net Price	Labor hours
1"	666	0.151	$3.87	$2,577.42	100.566
1 1/4"	296	0.18	$4.89	$1,447.44	53.28
1 1/2"	115.5	0.2	$5.75	$664.13	23.1
2"	317	0.25	$7.70	$2,440.90	79.25
2 1/2"	238.5	0.32	$12.05	$2,873.93	76.32

3"	79.5	0.372	$15.50	$1,232.25	29.574
3 1/2"	118	0.4	$25.50	$3,009.00	47.2
4"	9	0.444	$23.00	$207.00	3.996
Totals	1839.5			$14,452.06	413.286
Sprinkler Type	**Qty.**	**Unit Hours**	**Unit Price**	**Net Price**	**Labor hours**
Pendent-k5.6	143	0.1	$22.30	$3,188.90	14.3
			Total=	$17,640.96	427.586
			30%adj=	$22,933.25	556

System 1- Resized Mains and Branchlines

Pipe size	Qty(L.F.)	Unit hours	Unit Price	Net Price	Labor hours
1 1/4"	666	0.18	$4.89	$3,256.74	119.88
1 1/2"	296	0.2	$5.75	$1,702.00	59.2
2"	115.5	0.25	$7.70	$889.35	28.875
2 1/2"	317	0.32	$12.05	$3,819.85	101.44
3"	238.5	0.372	$15.50	$3,696.75	88.722
3 1/2"	79.5	0.4	$25.50	$2,027.25	31.8
4"	127	0.444	$23.00	$2,921.00	56.388
Totals	1839.5			$18,312.94	486.305
Sprinkler Type	**Qty.**	**Unit Hours**	**Unit Price**	**Net Price**	**Labor hours**
Pendent-k5.6	143	0.1	$22.30	$3,188.90	14.3

Total=	$21,501.84	500.605
30%adj=	$27,952.39	651

System 2- Pipe Schedule

Pipe size	**Qty(L.F.)**	**Unit hours**	**Unit Price**	**Net Price**	**Labor hours**
1"	778	0.151	$3.87	$3,010.86	117.478
1 1/4"	376	0.18	$4.89	$1,838.64	67.68
1 1/2"	118	0.2	$5.75	$678.50	23.6
2"	56	0.25	$7.70	$431.20	14
2 1/2"	68	0.32	$12.05	$819.40	21.76
3"	65	0.372	$15.50	$1,007.50	24.18
3 1/2"	40	0.4	$25.50	$1,020.00	16
4"	17	0.444	$23.00	$391.00	7.548
Totals	1518			$9,197.10	292.246
Sprinkler Type	**Qty.**	**Unit Hours**	**Unit Price**	**Net Price**	**Labor hours**
Pendent-k5.6	117	0.1	$22.30	$2,609.10	11.7
			Total=	$11,806.20	303.946
			30%adj=	$15,348.06	395

System 2- Resized Mains and Branchlines

Pipe size	Qty(L.F.)	Unit hours	Unit Price	Net Price	Labor hours
1 1/4"	778	0.18	$4.89	$3,804.42	140.04
1 1/2"	376	0.2	$5.75	$2,162.00	75.2
2"	118	0.25	$7.70	$908.60	29.5
2 1/2"	56	0.32	$12.05	$674.80	17.92
3"	68	0.372	$15.50	$1,054.00	25.296
3 1/2"	65	0.4	$25.50	$1,657.50	26
4"	57	0.444	$23.00	$1,311.00	25.308
Totals	1518			$11,572.32	339.264
Sprinkler Type	**Qty.**	**Unit Hours**	**Unit Price**	**Net Price**	**Labor hours**
Pendent-k5.6	117	0.1	$22.30	$2,609.10	11.7
			Total=	$14,181.42	350.964
			30%adj=	$18,435.85	456

J4: Alpert Method Activation Times

System1

Design Parameters

H	r	RTI	Ta
2.44	3.1	50	20

Time step	**time(t)**	**Q(kW)**	**ΔTg**	**Tg**	**u**	**ΔTd**	**Td**
0	0	0	0	0	0	0	20

1	10	4	2.613312	22.61331	0.193172	0.229717	20.22972
2	20	18	7.123055	27.12306	0.318919	0.778574	21.00829
3	30	40	12.12992	32.12992	0.416176	1.43495	22.44324
4	40	72	17.94897	37.94897	0.506253	2.206511	24.64975
5	50	113	24.24024	44.24024	0.588324	3.00527	27.65502
6	60	162	30.81967	50.81967	0.663379	3.773433	31.42846
7	70	221	37.90934	57.90934	0.735734	4.542792	35.97125
8	80	289	45.33322	65.33322	0.804556	5.267363	41.23861
9	90	366	53.06466	73.06466	0.870463	5.938653	47.17726
10	100	452	61.08158	81.08158	0.933906	6.552945	53.73021
11	110	546	69.28083	89.28083	0.994614	7.090949	60.82116
12	120	650	77.82042	97.82042	1.054131	7.597494	68.41865
13	130	763	86.59663	106.5966	1.111984	8.051785	76.47044
14	140	885	95.59757	115.5976	1.168345	8.458505	84.92894

System 2

Design Parameters

H	r	RTI	Ta
2.44	3.28	50	20

Time step	**time(t)**	**Q(kW)**	**ΔTg**	**Tg**	**u**	**ΔTd**	**Td**
0	0	0	0	0	0	0	20
1	10	4	2.516807	22.51681	0.184296	0.216092	20.21609
2	20	18	6.860012	26.86001	0.304267	0.732962	20.94905
3	30	40	11.68198	31.68198	0.397054	1.352612	22.30167

4	40	72	17.28615	37.28615	0.482993	2.082774	24.38444
5	50	113	23.34509	43.34509	0.561293	2.841044	27.22548
6	60	162	29.68155	49.68155	0.6329	3.572985	30.79847
7	70	221	36.5094	56.5094	0.701931	4.308191	35.10666
8	80	289	43.65913	63.65913	0.76759	5.003091	40.10975
9	90	366	51.10506	71.10506	0.83047	5.649212	45.75896
10	100	452	58.82594	78.82594	0.890997	6.242557	52.00152
11	110	546	66.7224	86.7224	0.948916	6.764481	58.766
12	120	650	74.94663	94.94663	1.005699	7.256715	66.02272
13	130	763	83.39876	103.3988	1.060893	7.699439	73.72215
14	140	885	92.0673	112.0673	1.114665	8.096785	81.81894

J5: Hekestad and Delichatsios Activation Times Method

System 1

Design Parameters

H	r	RTI	Ta	α	Cp	p	g	Xconv
2.44	3.1	50	293	0.0452	1.005	1.205	9.81	0.7

α_c=	0.03164	A=	0.0276	D=	0.3928

Time step	time(t)	t*2f	t*2	u/u*2	ΔT/ΔT*2	ΔT*2	u*2/ΔT*2	Y	ΔTd	Td
0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	20.00
1	10	1.85	1.20	0.30	1.08	0.00	0.51	0.00	0.00	20.00
2	20	1.85	2.40	0.29	1.05	1.57	0.51	0.03	0.15	20.15
3	30	1.85	3.59	0.29	1.05	7.32	0.51	0.14	0.66	20.81
4	40	1.85	4.79	0.29	1.05	14.68	0.51	0.28	1.27	22.08
5	50	1.85	5.99	0.29	1.05	23.14	0.51	0.44	1.90	23.98
6	60	1.85	7.19	0.29	1.05	32.46	0.51	0.61	2.53	26.51
7	70	1.85	8.39	0.29	1.05	42.52	0.51	0.81	3.13	29.65
8	80	1.85	9.58	0.29	1.05	53.21	0.51	1.01	3.70	33.34
9	90	1.85	10.78	0.29	1.05	64.46	0.51	1.22	4.23	37.57
10	100	1.85	11.98	0.29	1.05	76.24	0.51	1.44	4.71	42.28
11	110	1.85	13.18	0.29	1.05	88.48	0.51	1.68	5.15	47.43
12	120	1.85	14.38	0.29	1.05	101.17	0.51	1.92	5.55	52.98
13	130	1.85	15.57	0.29	1.05	114.27	0.51	2.16	5.91	58.89
14	140	1.85	16.77	0.29	1.05	127.75	0.51	2.42	6.23	65.12
15	150	1.85	17.97	0.29	1.05	141.61	0.51	2.68	6.53	71.65

16	160	1.85	19.17	0.29	1.05	155.80	0.51	2.95	6.79	78.44
17	170	1.85	20.37	0.29	1.05	170.33	0.51	3.23	7.02	85.46

System 2

Design Parameters

H	r	RTI	Ta	α	Cp	p	g	Xconv
2.44	3.28	50	293	0.0452	1.005	1.205	9.81	0.7

α_c=	0.03164	A=	0.02765	D=	0.4083

Time step	time(t)	t*2f	t*2	u/u*2	ΔT/ΔT*2	ΔT*2	u*2/ΔT*2	Y	ΔTd	Td
0	0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	20.00
1	10	1.91	1.20	0.30	1.08	0.00	0.51	0.00	0.00	20.00
2	20	1.9	2.40	0.29	1.05	1.28	0.49	0.02	0.12	20.1

		1								2
3	30	1.91	3.59	0.29	1.05	6.64	0.49	0.13	0.62	20.74
4	40	1.91	4.79	0.29	1.05	13.57	0.49	0.26	1.20	21.94
5	50	1.91	5.99	0.29	1.05	21.55	0.49	0.42	1.82	23.76
6	60	1.91	7.19	0.29	1.05	30.37	0.49	0.59	2.44	26.20
7	70	1.91	8.39	0.29	1.05	39.89	0.49	0.77	3.03	29.23
8	80	1.91	9.58	0.29	1.05	50.01	0.49	0.97	3.59	32.82
9	90	1.91	10.78	0.29	1.05	60.68	0.49	1.17	4.11	36.94
10	100	1.91	11.98	0.29	1.05	71.83	0.49	1.39	4.60	41.53
11	110	1.91	13.18	0.29	1.05	83.44	0.49	1.61	5.04	46.57
12	120	1.91	14.38	0.29	1.05	95.47	0.49	1.85	5.44	52.01
13	130	1.91	15.57	0.29	1.05	107.89	0.49	2.09	5.80	57.81
14	140	1.91	16.77	0.29	1.05	120.68	0.49	2.33	6.13	63.94
15	150	1.91	17.97	0.29	1.05	133.82	0.49	2.59	6.43	70.37
16	160	1.91	19.17	0.29	1.05	147.29	0.49	2.85	6.69	77.06
17	170	1.91	20.37	0.29	1.05	161.07	0.49	3.12	6.93	84.00

Appendix K: Structural Selection

K.1 Floor Depth

Slab Thickness	First Floor Largest Girder	Depth of Largest Girder	**Floor Depth**
3	W14x30	13.8	**16.8**
3	W14x30	13.8	**16.8**
5	W14x30	13.8	**18.8**
3	W18x55	18.1	**21.1**
3	W14x61	13.9	**16.9**
3	W14x61	13.9	**16.9**
5	W14x61	13.9	**18.9**
3	W21x73	21.2	**24.2**
3	W14x30	13.8	**16.8**
3	W14x30	13.8	**16.8**
5	W14x30	13.8	**18.8**
3	W18x55	18.1	**21.1**
3	W14x61	13.9	**16.9**
3	W14x61	13.9	**16.9**
5	W14x61	13.9	**18.9**
3	W21x73	21.2	**24.2**
3	W14X30	13.8	**16.8**
3	W14X30	13.8	**16.8**
3	W18x15	18.1	**21.1**
3	W18x15	18.1	**21.1**

2nd Floor

Largest Girder	Depth of Largest Girder	**Floor Depth**
W14x30	13.8	**16.8**
W14x30	13.8	**16.8**
W14x30	13.8	**18.8**
W18x55	18.1	**21.1**
W14x61	13.9	**16.9**
W14x61	13.9	**16.9**
W14x61	13.9	**18.9**
W21x73	21.2	**24.2**
W14x30	13.8	**16.8**
W14x30	13.8	**16.8**
W14x30	13.8	**18.8**
W18x55	18.1	**21.1**
W14x61	13.9	**16.9**
W14x61	13.9	**16.9**
W14x61	13.9	**18.9**
W21x73	21.2	**24.2**
W14X30	13.8	**16.8**
W14X30	13.8	**16.8**
W18x15	18.1	**21.1**
W18x15	18.1	**21.1**

3rd Floor		
Largest Girder	**Depth of Largest Girder**	**Floor Depth**

W14x30	13.8	**16.8**
W14x30	13.8	**16.8**
W14x30	13.8	**18.8**
W18x55	18.1	**21.1**
W14x61	13.9	**16.9**
W14x61	13.9	**16.9**
W14x61	13.9	**18.9**
W21x73	21.2	**24.2**
W14x30	13.8	**16.8**
W14x30	13.8	**16.8**
W14x30	13.8	**18.8**
W18x55	18.1	**21.1**
W14x61	13.9	**16.9**
W14x61	13.9	**16.9**
W14x61	13.9	**18.9**
W21x73	21.2	**24.2**
W14X30	13.8	**16.8**
W14X30	13.8	**16.8**
W18x15	18.1	**21.1**
W18x15	18.1	**21.1**

4th Floor		
Largest Girder	Depth of Largest Girder	**Floor Depth**
W14x30	13.8	**16.8**
W14x30	13.8	**16.8**

W14x30	13.8	**18.8**
W18x55	18.1	**21.1**
W16x57	16.4	**19.4**
W16x57	16.4	**19.4**
W16x57	16.4	**21.4**
W21x83	21.4	**24.4**
W14x30	13.8	**16.8**
W14x30	13.8	**16.8**
W14x30	13.8	**18.8**
W18x55	18.1	**21.1**
W16x57	16.4	**19.4**
W16x57	16.4	**19.4**
W16x57	16.4	**21.4**
W21x83	21.4	**24.4**
W14X30	13.8	**16.8**
W14X30	13.8	**16.8**
W18x15	18.1	**21.1**
W18x15	18.1	**21.1**

Totoal Floors Depth	SCORE	
67.2	73.3	73.3
67.2	73.3	73.3
75.2	62.2	62.2
84.4	49.4	49.4
70.1	69.3	69.3

70.1	69.3	69.3
78.1	58.2	58.2
97	31.9	31.9
67.2	73.3	73.3
67.2	73.3	73.3
75.2	62.2	62.2
84.4	49.4	49.4
70.1	69.3	69.3
70.1	69.3	69.3
78.1	58.2	58.2
97	31.9	31.9
67.2	73.3	73.3
67.2	73.3	73.3
84.4	49.4	49.4
84.4	49.4	49.4

K.2 Net Usable Area

Composite Columns for *Non* Composite Beams and Girders

Bay Size	**# Interior Columns**	**Type of Column**	**Area**	**NUA**	**Score**
Bay Size 25x25				50.36111	49.63889
				50.36	
Fist Floor					

	15	W12X26	196
Second Floor			
	14	W10X15	144
Third Floor			
	14	W6X12	100
Fourth Floor			
	14	W6X8.5	64

Scenario	Score	Area Used
1A	30.8	69.17
2A	30.8	69.17
3A	30.8	69.17
4A	49.6	50.36
1B	30.8	69.17
2B	30.8	69.17

3B	30.8	69.17
4B	49.6	50.36
1C	40.7	59.31
2C	40.7	59.31
3C	40.7	59.31
4C	64.1	35.93
1D	47.4	52.61
2D	47.4	52.61
3D	47.4	52.61
4D	56.2	43.82
1E	30.8	69.17
2E	40.7	59.31
3E	49.6	50.36
4E	64.1	35.93

K.3 Member Uniformity

1A		2A		3A		4A	
W10x17	720	W10x17	570	W12x22	270	W12x26	456
W10x12	30	W10x12	30	W10x17	30	W10x19	24
W14x30	120	W14x30	120	W14x30	120	W18x55	72
W12X22	60	W12X22	60	W12X22	60	W16x40	48
W6x8.5	192	W6x8.5	192	W6x8.5	192	W6x8.5	97
W6x12	36	W6x12	36	W6x12	36	W6x12	21
W10x15	36	W10x15	36	W10x15	36	W10x15	21
						W12x35	21
						W8x18	20

1B		2B		3B		4B	
W12X16	30	W12x30	380	W14x38	290	W14x34	304
W10x33	720	W14x26	190	W14x34	20	W16x31	152
W14x61	80	W12X16	20	W12x26	10	W12x26	24
W16x57	40	W12x19	10	W14x61	80	W21x73	48
W12X34	40	W14x61	80	W16x57	40	W21x83	24
W14x38	20	W16x57	40	W12X34	40	W18x50	32
W6x8.5	192	W12X34	40	W6x8.5	192	W18x60	16
W6x12	36	W14x38	20	W6x12	36	W6x8.5	97

W10x15	36	W6x8.5	192	W10x15	36	W6x12	21
		W6x12	36			W10x15	21
		W10x15	36			W12x35	21
						W8x18	20

1C		2C		3C		4C	
W10x17	720	W10x17	570	W12x22	270	W12x26	456
W10x12	30	W10x12	30	W10x17	30	W10x19	24
W14x30	120	W14x30	120	W14x30	120	W18x55	72
W12X22	60	W12X22	60	W12X22	60	W16x40	48
W6x20	36	W6x20	36	W6x20	36	W6x15	24
W6x15	60	W6x15	60	W6x15	60	W6x12	4
W6x12	4	W6x12	4	W6x12	4	W6x20	4
W8x28	36	W8x28	36	W8x28	36	W6x25	20
W8x35	36	W8x35	36	W8x35	36	W8x24	21
W8x24	30	W8x24	30	W8x24	30	W8x31	20
W8x31	26	W8x31	26	W8x31	26	W8x40	20
W10x49	36	W10x49	36	W10x49	36	W8x28	4
						W10x49	21
						W10x39	21
						W12x65	21

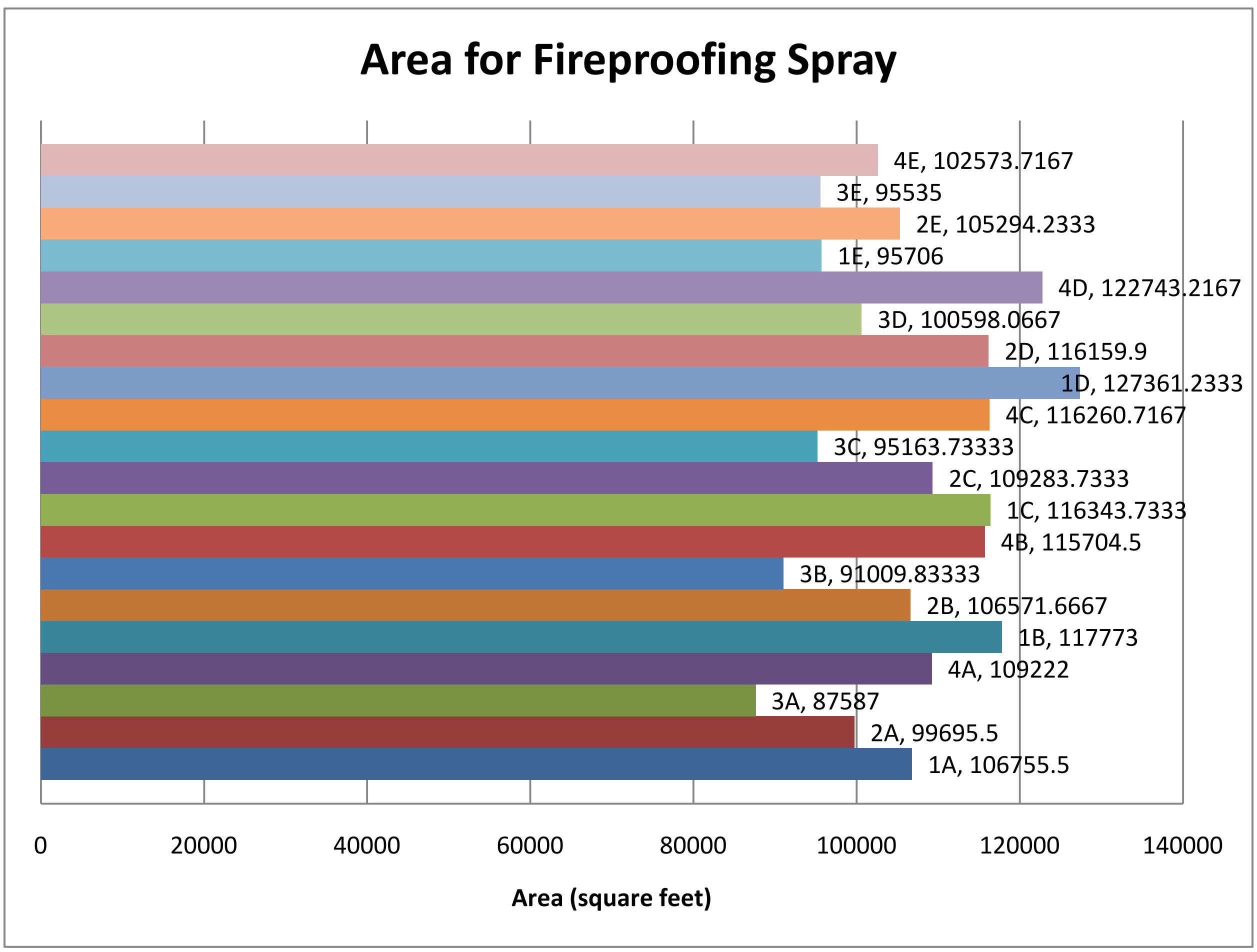
1D		2D		3D		4D	
W12X16	30	W12x30	380	W14x38	290	W14x34	304
W10x33	720	W14x26	190	W14x34	20	W16x31	152

W14x61	80	W12X16	20	W12x26	10	W12x26	24
W16x57	40	W12x19	10	W14x61	80	W21x73	48
W12X34	40	W14x61	80	W16x57	40	W21x83	24
W14x38	20	W16x57	40	W12X34	40	W18x50	32
W6x20	36	W12X34	40	W6x20	36	W18x60	16
W6x15	60	W14x38	20	W6x15	60	W6x15	24
W6x12	4	W6x20	36	W6x12	4	W6x12	4
W8x28	36	W6x15	60	W8x28	36	W6x20	4
W8x35	36	W6x12	4	W8x35	36	W6x25	20
W8x24	30	W8x28	36	W8x24	30	W8x24	21
W8x31	26	W8x35	36	W8x31	26	W8x31	20
W10x49	36	W8x24	30	W10x49	36	W8x40	20
		W8x31	26			W8x28	4
		W10x49	36			W10x49	21
						W10x39	21
						W12x65	21

1E		2E		3E		4E	
12K5	1470	12K5	1470	16K7	936	16K7	936
12K1	30	12K1	30	14K4	24	14K4	24
W14x61	120	W14x61	120	W21x73	72	W21x73	72
W12X34	60	W12X34	60	W21x83	48	W21x83	48
W6x8.5	192	W6x20	36	W6x8.5	97	W6x15	24
W6x12	36	W6x15	60	W6x12	21	W6x12	4
W10x15	36	W6x12	4	W10x15	21	W6x20	4

		W8x28	36	W12x35	21	W6x25	20
		W8x35	36	W8x18	20	W8x24	21
		W8x24	30			W8x31	20
		W8x31	26			W8x40	20
		W10x49	36			W8x28	4
						W10x49	21
						W10x39	21
						W12x65	21

K.4 Fire Proofing Area



Appendix L: Structural Steel Member Under Elevated Temperature (sample excel sheet)

A 125Mb excel was constructed to perform the numerical calculation for temperature change of steel and steel properties changes under elevated temperature. Due to the length of excel sheet, only two samples are shown here to display how the excel application was constructed.

								Source: Swedish Steel Design Manual			© CE 514, 2008						in2	m2				
										light concret properties:				Steel	Concrete		144	0.09290304				
	hc=	0.025	kW/m^2 C		As=	0.0016258	ρs=	7850	kg/m3	ρi=	1750	kg/m3		Fy=50ksi			in	m				
	σ=	5.67E-11	kW/m^2 K		b=	0.100076	Cps=	0.54	kJ/kgK	Cpi=	0.96	kJ/kgK		E=29000ksi			12	0.3048				
	ε=	0.5			h=	0.148082	Δt=	1	sec	ki=	0.0008	kW/mK										
					Vs/Ai=	0.003276				ti(1" thickness)=	0.0254	m										
		E 119	Steel member/I beam					Concrete					Steel Rod								non-restrained	
t (min)	t (sec)	Temp. (C	Temp C	Fy (ksi)	E (ksi)	ε_{th}	σ_{th} (ksi)	Temp. (C)	fc (ksi)	E (ksi)	ε_{th}	σ_{th} (ksi)	Temp C	Fy (ksi)	E (ksi)	ε_{th}	σ_{th} (ksi)	Load Capacity	Load (wt.+th.)	Passed/Failed	Load	Test
0	0	21	20	50	29000	1.5458E-20	4.4829E-16	20	4	4350	1.84E-07	0.0008004	20	60	29000	1.5458E-20	4.48E-16	555.2091886	86.9370107	Passed	86.937	Passed
0.016667	1	38.70932	20	49.752	28928	1.5458E-20	4.4717E-16	21.28634	4	4350	1.84E-07	0.0008004	21.286	59.678	28922	1.5649E-05	0.452583	553.5321741	88.35884082	Passed	86.937	Passed
0.033333	2	55.33963	20	49.752	28928	1.5458E-20	4.4717E-16	23.5392	4	4350	1.84E-07	0.0008004	23.539	59.636	28911	4.3087E-05	1.24569	553.3990588	90.85046259	Passed	86.937	Passed
0.05	3	70.30711	20	49.752	28928	1.5458E-20	4.4717E-16	26.51134	4	4350	1.84E-07	0.0008004	26.511	59.578	28897	7.9348E-05	2.292891	553.218015	94.14034037	Passed	86.937	Passed
0.066667	4	83.91427	20	49.752	28928	1.5458E-20	4.4717E-16	30.01034	4	4350	1.84E-07	0.0008004	30.01	59.508	28879	0.00012213	3.526916	552.9973665	98.0171427	Passed	86.937	Passed
0.083333	5	96.38791	20	49.752	28928	1.5458E-20	4.4717E-16	33.88581	4	4350	1.84E-07	0.0008004	33.886	59.427	28859	0.00016962	4.895111	552.7439848	102.3154565	Passed	86.937	Passed
0.1	6	107.9024	20	49.752	28928	1.5458E-20	4.4717E-16	38.02005	4	4350	1.84E-07	0.0008004	38.02	59.338	28836	0.00022042	6.356144	552.4637739	106.9054273	Passed	86.937	Passed
0.116667	7	118.5949	20	49.752	28928	1.5458E-20	4.4717E-16	42.32094	4	4350	1.84E-07	0.0008004	42.321	59.242	28812	0.00027342	7.877547	552.1618919	111.6850553	Passed	86.937	Passed
0.133333	8	128.5748	20	49.752	28928	1.5458E-20	4.4717E-16	46.71658	4	4350	1.84E-07	0.0008004	46.717	59.14	28786	0.00032773	9.433858	551.8428591	116.5743483	Passed	86.937	Passed
0.15	9	137.9314	20	49.752	28928	1.5458E-20	4.4717E-16	51.15109	4	4350	1.84E-07	0.0008004	51.151	59.035	28758	0.00038268	11.00518	551.5106249	121.5108086	Passed	86.937	Passed
0.166667	10	146.7378	20	49.752	28928	1.5458E-20	4.4717E-16	55.5813	4	4350	1.84E-07	0.0008004	55.581	58.926	28730	0.00043773	12.57607	551.1686184	126.4459039	Passed	86.937	Passed
0.183333	11	155.0552	20	49.752	28928	1.5458E-20	4.4717E-16	59.97417	4	4350	1.84E-07	0.0008004	59.974	58.815	28701	0.00049248	14.13464	550.819795	131.3422787	Passed	86.937	Passed
0.2	12	162.9351	20	49.752	28928	1.5458E-20	4.4717E-16	64.30476	4	4350	1.84E-07	0.0008004	64.305	58.702	28672	0.0005466	15.67184	550.466681	136.1715332	Passed	86.937	Passed
0.216667	13	170.4212	20	49.752	28928	1.5458E-20	4.4717E-16	68.55456	4	4350	1.84E-07	0.0008004	68.555	58.589	28642	0.00059985	17.18091	550.1114173	140.9124425	Passed	86.937	Passed
0.233333	14	177.551	20	49.752	28928	1.5458E-20	4.4717E-16	72.71018	4	4350	1.84E-07	0.0008004	72.71	58.476	28612	0.00065207	18.65694	549.7558024	145.5495236	Passed	86.937	Passed
0.25	15	184.3569	20	49.752	28928	1.5458E-20	4.4717E-16	76.7623	4	4350	1.84E-07	0.0008004	76.762	58.363	28582	0.00070312	20.09645	549.4013314	150.0718775	Passed	86.937	Passed

							steel properties			air properties			ceiling/gypsum board properties			
	hc=	0.025	kW/m^2 C		Top/bottom chord		ρs=	7850	kg/m3	ρ=	1	kg/m3	ρi=	800	kg/m3	input
	σ=	5.67E-11	kW/m^2 K		As(BC)=	0.301625	Cps=	0.54	kJ/kgK	Cp=	1	kJ/kgK	Cpi=	1.7	kJ/kgK	forumla/equations
	ε=	0.5			Vs=	0.000674	Δt=	1	sec				ki=	0.00017	kW/mK	selective input
Fy=50ksi	Control	V=	0.10236	m	As/Vs (BC)=	447.39							ti=	0.0127	m	
E=29000k	Volume	H=	0.4318	m	As(TC)=	0.195263	Diagnoal web									
	parameters	L=	1	m	As/Vs (TC)=	289.62	As/Vs (4/d)=	229.06								

		E 119		Steel member/bottom chord			Steel member/Top chord			Steel member/web rod			Load	Bottom Chord			Top Chord			Web Rod			
(min)	t (sec)	Temp. (C	T space	Temp C	Fy (ksi)	E (ksi)	Temp C	Fy (ksi)	E (ksi)	Temp C	Fy (ksi)	E (ksi)	V_{TL} (k/in)	M (k-in)	σ_b (ksi)	Pass/Fail	M (k-in)	σ_b (ksi)	Pass/Fail	V (kips)	P (kips)	σ (ksi)	Passed/Failed
0	0	22	21	20	50	29000	20	50	29000	20	50	29000	0.031167	350.625	20.00428	Passed	350.625	20.00428	Passed	4.675	6.611448	17.80989	Passed
0.016667	1	38.70932	21	20.003	49.75148	28928	20.0019	49.751	28928	20.002	49.751	28928	0.031167	350.625	20.00428	Passed	350.625	20.00428	Passed	4.675	6.611448	17.80989	Passed
0.033333	2	55.33963	21	20.006	49.75143	28928	20.0038	49.751	28928	20.003	49.751	28928	0.031167	350.625	20.00428	Passed	350.625	20.00428	Passed	4.675	6.611448	17.80989	Passed
0.05	3	70.30711	21	20.009	49.75139	28928	20.0057	49.751	28928	20.005	49.751	28928	0.031167	350.625	20.00428	Passed	350.625	20.00428	Passed	4.675	6.611448	17.80989	Passed
0.066667	4	83.91427	21	20.012	49.75134	28928	20.00759	49.751	28928	20.006	49.751	28928	0.031167	350.625	20.00428	Passed	350.625	20.00428	Passed	4.675	6.611448	17.80989	Passed
0.083333	5	96.38791	21	20.015	49.7513	28928	20.00948	49.751	28928	20.008	49.751	28928	0.031167	350.625	20.00428	Passed	350.625	20.00428	Passed	4.675	6.611448	17.80989	Passed
0.1	6	107.9024	21	20.018	49.75125	28928	20.01137	49.751	28928	20.009	49.751	28928	0.031167	350.625	20.00428	Passed	350.625	20.00428	Passed	4.675	6.611448	17.80989	Passed
0.116667	7	118.5949	21	20.02	49.75121	28928	20.01325	49.751	28928	20.01	49.751	28928	0.031167	350.625	20.00428	Passed	350.625	20.00428	Passed	4.675	6.611448	17.80989	Passed
0.133333	8	128.5748	21	20.023	49.75117	28928	20.01513	49.751	28928	20.012	49.751	28928	0.031167	350.625	20.00428	Passed	350.625	20.00428	Passed	4.675	6.611448	17.80989	Passed
0.15	9	137.9314	21	20.026	49.75112	28928	20.01701	49.751	28928	20.013	49.751	28928	0.031167	350.625	20.00428	Passed	350.625	20.00428	Passed	4.675	6.611448	17.80989	Passed
0.166667	10	146.7378	21	20.029	49.75108	28928	20.01888	49.751	28928	20.015	49.751	28928	0.031167	350.625	20.00428	Passed	350.625	20.00428	Passed	4.675	6.611448	17.80989	Passed
0.183333	11	155.0552	21	20.032	49.75104	28927	20.02075	49.751	28928	20.016	49.751	28928	0.031167	350.625	20.00428	Passed	350.625	20.00428	Passed	4.675	6.611448	17.80989	Passed
0.2	12	162.9351	21	20.035	49.75099	28927	20.02261	49.751	28928	20.018	49.751	28928	0.031167	350.625	20.00428	Passed	350.625	20.00428	Passed	4.675	6.611448	17.80989	Passed
0.216667	13	170.4212	21	20.038	49.75095	28927	20.02447	49.751	28928	20.019	49.751	28928	0.031167	350.625	20.00428	Passed	350.625	20.00428	Passed	4.675	6.611448	17.80989	Passed
0.233333	14	177.551	21	20.04	49.75091	28927	20.02633	49.751	28928	20.021	49.751	28928	0.031167	350.625	20.00428	Passed	350.625	20.00428	Passed	4.675	6.611448	17.80989	Passed
0.25	15	184.3569	21	20.043	49.75086	28927	20.02818	49.751	28928	20.022	49.751	28928	0.031167	350.625	20.00428	Passed	350.625	20.00428	Passed	4.675	6.611448	17.80989	Passed
0.266667	16	190.867	21	20.046	49.75082	28927	20.03003	49.751	28927	20.024	49.751	28928	0.031167	350.625	20.00428	Passed	350.625	20.00428	Passed	4.675	6.611448	17.80989	Passed
0.283333	17	197.106	21	20.049	49.75078	28927	20.03188	49.751	28927	20.025	49.751	28928	0.031167	350.625	20.00428	Passed	350.625	20.00428	Passed	4.675	6.611448	17.80989	Passed
0.3	18	203.0956	21	20.052	49.75074	28927	20.03372	49.751	28927	20.027	49.751	28928	0.031167	350.625	20.00428	Passed	350.625	20.00428	Passed	4.675	6.611448	17.80989	Passed