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Structural Steel, Fire Safety and Green Roof Design for New Gateway Park Building

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Structural Steel, Fire Safety and Green Roof Design for New Gateway Park Building

Major Qualifying Project Report

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WORCESTER POLYTECHNIC INSTITUTE

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Degree of Bachelor of Science

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Abstract

This Major Qualifying Project explored the design of a four-story building at Gateway Park in Worcester, Massachusetts. Several designs with different elements were prepared and analyzed to recommend the best option in terms of cost, constructability, performance and usable area. Additionally, different green roof and fire protection designs for the building were investigated and recommended.

Acknowledgements

Our team would like to express our sincere gratitude to project advisor Professor Leonard Albano, for his valuable feedbacks, guidance, support and patience throughout the project. This project would not have been possible was it not for all his help. Our team would also like to thank Professor Guillermo Salazar for providing copies of the drawings with the architectural layout of the actual project.

Authorship

Authorship

Jin Kyung Kim was responsible for writing all materials relating to structural design results, system selection, trapezoidal area, code analysis and automatic fire sprinkler system design.

Long Huynh was responsible for writing all materials relating to lateral forces analysis, frame design results, elevator design results, and green roof design.

Yi Mei Dong was responsible for writing materials relating to foundation design, stair design, *Robot and RISA* comparison, and structure cost analysis.

Jin Kyung Kim

Long Huynh

Dong Yi Mei

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Capstone Design

For this MQP project, the main objective of the group was to serve as structural engineering consultants and provide the structural design for a four-story-office building located at Gateway Park, Worcester, Massachusetts. Along with the structural design, green roof construction and fire protection were implemented to the design. Last, different forms of cost estimation using RS Means Publication were developed to evaluate alternatives and determine the most economical design system. As stated in the ABET General Criterion Curriculum, the designs incorporated engineering standards and realistic constraints which included the following considerations: economic, environmental, sustainability, constructability, ethic, health and safety, and social.

Economic

- Steel cost was calculated for each of the design members, such as beams, girder, columns, studs and frame.
- Once the building design was complete, *RS Means Building Construction Cost Data 2009* was used as a reference to approximate the cost of building per square foot.
- The design that incorporated the best price, constructability and usable space was recommended.

Sustainability

- Beams, girders and columns were placed within the architectural layout to produce efficient and useful spaces.
- Promoted environmental awareness by design green roof.
- Addressed LEED certification of green roof for environmental sustainability.

Constructability

- Alternative design scenarios were developed: composite and non-composite beam-and-slab systems, different bay sizes and beam spacing, and both shored and unshored construction to provide alternatives.
- Maximized repeatability by considering standard size sections, such as the steel member sizes.
- Separation of office and lab spaces will reduce complication during construction.

Ethics

- The design system was established in compliance with the *International Building Code 2009* and *NFPA* publications.
- While cost was an issue, meeting the minimum requirements in terms of performance was the main priority.

Health and Safety

- All Structural system scenarios were designed in compliance with the *International Building Code 2009, AISC Steel Manual, and ASCE 7-05*
- The building was designed with fire protection systems. The fire protection design met the minimum requirements of the codes of *IBC 2009* and *NFPA 13*.

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Chapter 1: Introduction

This project involves numerous design aspects for the new four-story Gateway building located on Grove and Prescott Street, Worcester, Massachusetts. In the early 1900's Worcester was primarily known as a manufacturing industry for producing metal or wire. With the sudden decline in the manufacturing industry, many companies were shut down, and this left numerous empty and unutilized properties that were environmentally damaged from the uses of the early industries. WPI and Worcester Business Development Center (WBDC) took on the daunting task of transforming these brownfields to research center.

Finally in 2007, a four-story Life Science and Bioengineering Center building construction was completed. The 125,000 square feet Life Science and Bioengineering Center is mainly used as laboratories, conference rooms and office spaces. Following the first Gateway Park building construction, WPI announced an agreement with O'Connell Development Group (ODG) of Holyoke, Massachusetts, for the next building at Gateway Park, in 2009. Under the agreement, WPI ground-leased one of the park's four remaining pad-ready sites to ODG, who was responsible for financing, developing, constructing and owning the new building. The ground breaking ceremony took place on April 21st 2011 for the $32million, four-story building with a total area of 92,000 square feet. This building is currently being constructed in front of the parking lot and next to the Life Science and Bioengineering Center.

The architectural design of the building was created by architects at *Perkins+Will*, hired by ODG as architectural designers. The team performed structural design and analysis for the new Gateway Building to meet the demands of the architectural design. The structural design will satisfy all functional and structural aspects for the multi-occupancy building while having a reasonable cost from the building owner's perspective in comparison to the standard cost presented in the *RS Means Square Foot Cost 2009* manual. Structural analysis is essential to any construction and no matter how impressive the architectural plan is, because the structure must have adequate strength, stiffness and stability to withstand all loads. Alternative structural steel frame design scenarios for the new Gateway Building were investigated and the structural design and analysis process was performed in compliance with the *IBC 2009*. The best design scenario was recommended by comparing cost values, constructability, performance and usable area.

To ensure the client that all the codes are satisfied, this project also looked into fire safety design. *IBC 2009* was reviewed to find applicable prescriptive codes required for the minimum fire protection requirements. Two fire protection design systems were investigated where one system did not include sprinklers and the alternative system did include sprinklers which were installed in compliance with *NFPA 13*. These two designs were compared to see the cost and effectiveness of sprinkler systems and what affects it could have on the overall structural design.

Concerned with environmentally friendly buildings, a green roof was designed based on existing structural load capacity, geographical population, and local climate. The design included two alternative types of green roofs with different location of plants and landscaping. Furthermore, the design also looked at effects that the green roof brings to the building in terms of energy performance and improvement of eco system.

Cost estimations were presented for all design systems. There were two alternative methods to estimate the total cost of construction. The first method used the cost per square foot for each design system. The second method used *RS Mean Building Construction Cost Data 2009* to estimate the cost of each member per linear foot.

Chapter 2: Background

2.1 History of Gateway Park

Life Science and Bioengineering center Located on the intersection of Grove Street and Prescott Street is currently used for graduate biology, biochemistry, chemical engineering research and many other research companies. Before building the Life Science Center, Gateway Park was originally a toxic site for steel manufactures, home to various plating, roofing, and paper companies. The following section will present brief information pertaining to Gateway Park located in the city of Worcester, just to the north of downtown.

Back in 1910, Worcester was primarily known as a manufacturing center for producing metal or wire. A sudden decline in the manufacturing industry caused companies to shut down and the unemployment rate to rise. As a result the City was left with several empty and unutilized properties and environmentally damaged land. There were over 200 documented brownfield sites left behind. (Enviormental Proetection Agency, 2007). WPI and Worcester Business Development Center (WBDC) took this ambitious task to transform several brownfields into a research center. WPI's main interest was to create research space for its growing Life Science Department. (Conover, 2007)

In 2007, Life Science and Bioengineering Center was opened to house WPI's growing biotechnology industry and to support the transition of technologies to commercial enterprises. In order to remake the past, Gateway Park was built to blend with surrounding Worcester Buildings of industrial Era, such as the brick mill. This 125,000 square-foot Life Science and Bioengineering Center was mainly used for laboratory, conference and office purpose. The buildings are currently occupied by WPI academic department, research group and companies like Blue Sky Biotech; CellThera; RXi Pharmaceuticals; and the Massachusetts Biomedical

Initiative's life sciences. In addition to the Life Science and Bioengineering Center, a 660-space parking structure and surface lot that cost $12.5 million dollars were opened in the same year for the convenience of the tenants in Life Science and Bioengineering Center and WPI students. (Gateway Park at WPI)

In addition to the expected impact of Gateway Park in the biomedical research in the world, the project has been awarded many times for its commitment to transforming brownfields to a research center. Gateway Park won the Prestigious Phoenix Award and Excellence in Economic Development Award in 2007 for its influences in redeveloping industrial site and urban of economic. (Gateway Park at WPI) In 2008, Gateway Park was nominated by Commonwealth of Massachusetts as the anchor for the state's first Growth District, a new initiative to accelerate job creation in locations that are primed and ready for development. For their commitment, WPI was awarded $5.2 million grant from the Massachusetts Life Sciences Center awarded last year for their new building funds. (Gateway Park at WPI)

2.2 Building Ownership and Future Occupants

As the Life Science and Bioengineering Center is committed to the long-term success of development and revitalizing the neighborhood, the next phase for Gateway development is essential. In 2009, WPI announced an agreement with O'Connell Development Group of Holyoke (ODG) for the next building at Gateway Park. Under the agreement, WPI will ground-lease one of the park's four remaining pad-ready sites to ODG, who is responsible for financing, developing, constructing and owning the new building. For this project, WPI is no longer the building owner, instead WPI will be an important tenant. The new building at Gateway Park will also be a four-story building, with total area of 92,000 square feet. The new building will be located in front of the parking lot and next to the Life Science and Bioengineering Center. (Gateway Park at WPI)

Before the construction begins, approximately half of the building was already leased due to its great reputation. Similar to the Life Science and Bioengineering Center, the new building will be mainly used as laboratory, educational and office spaces, for a range of academic and corporate purposes. (Gateway Park at WPI) WPI will be leasing spaces to serve as a new Biomanufacturing Education and Training Center, an expanded Fire Protection Engineering Department and Research Laboratory, and the Graduate Division of WPI's school of Business. Also, many of the tenants from Life Science and Bioengineering Center, Massachusetts Biomedical Initiatives, Blue Sky Biotech are moving or expanding to this new space. (Gateway Park at WPI). Architectural Footprint

2.3 Structural Design

2.3.1 Steel vs. Concrete

The process of choosing a type of construction is a task structural engineers make from sound judgment. There is no one material superior to the other and the decision depends on which material is the most suitable in accordance with the type of building and the aspects the engineer is looking for in his project.

Today, when competing framing systems are evaluated for projects using comparable, current cost data, structural steel remains the cost leader for the majority of construction projects (Why Do Designers & Owners Choose Structural Steel?, 2012). While economical, steel has very high performance. Although concrete performs well during both natural and manmade disasters as well as requiring no additional fire protection, steel frames with decking and fire protection in total generally cost about 5% to 7% less than concrete framing systems. Along with its relatively low cost, typical structural steel can yield stresses up to 50 ksi in most cases whereas concrete mix and high strength concrete can only yield stresses up to 5 ksi and 15 ksi respectively. (Why Do Designers & Owners Choose Structural Steel?, 2012). Although structural steel is stronger than concrete, it also has higher strength to weight ratio which means that steel structures will be lighter and require less foundation strength. Uniquely, concrete can provide more floors in high rise buildings within a given height restriction because of its lower floor-to-floor heights. However, the higher strength to weight ratio of structural steel allows for longer spans and slender columns, providing greater open floor space for a given footprint. Steel is not only strong but ductile, making it a good candidate for buildings in seismic zones and areas with high wind loads.

Strong and low cost, structural steel provides an architect the freedom to come up with innovative designs. Architects can emphasize grace, slenderness, strength and transparency of frames through structural steel design. Structural steel can be used on a wide variety of buildings from the simplest to the most complex. Overall, structural steel can be seen as a very flexible material that can address a gamut of different design requirements.

Steel ranks as number one as the most recycled material on planet Earth. Structural steel is fully recyclable and does not need further processing. 88% of the structural steel we use today is recycled product. Another merit of using steel is that besides the closed loop recycling process, no water is required or discharged in the environment whether it is during the fabrication process or during the construction stages. (Why Do Designers & Owners Choose Structural Steel?, 2012).

Construction is fastest using structural steel. In terms of starting field work, structural steel may be slow but the framing system can be completed in minimal time through rapid design, fabrication and erection cycle of the structural steel. It is also easy to modify an existing steel structural for any reason. Steel plate can be attached to the flanges or web of sections to strengthen existing columns and beams allowing greater load bearing capacity. Additional floors can be added to existing buildings, old or new. These tasks can be performed even while the building is occupied as it causes little disruption. (Why Do Designers & Owners Choose Structural Steel?, 2012)

2.3.2 Composite vs. Non-Composite

The composite construction's purpose is to enhance the performance of the beam. It is an integral part of the beam. The slabs function is a large plate upon the upper flange of the steel beam, increasing the beam's strength. Composite section has greater stiffness than non-composite sections and smaller deflection. It also has greater ability to take an overload than the composite structure. One more additional advantage of composite construction is possibility of reducing floor depths-an important factor for tall buildings, which leads to reducing building heights. Consequently, there are construction advantages that follow such as reducing material and labor cost for plumbing, wiring, ducting jobs, foundations, labors. The reduced beam depth also reduces the fireproofing costs.

The advantages of composite beam must be weighed against the added construction costs for furnishing and installing the shear connectors; it is not cost effective when used for short spans and lightly loaded applications.

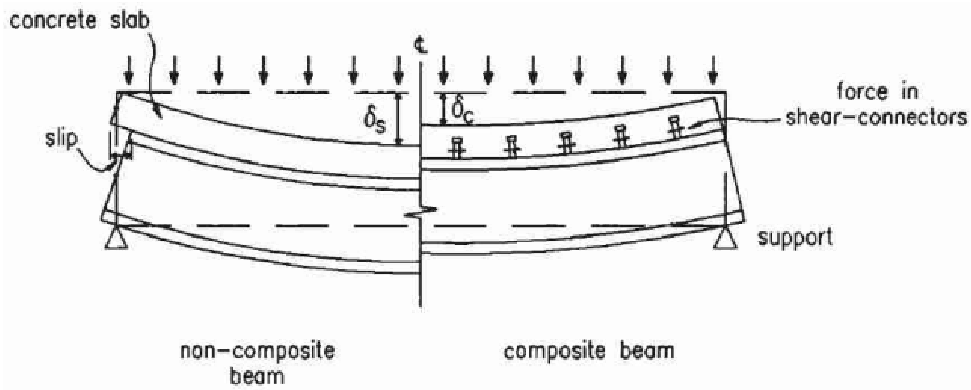


Figure 1: Non-composite beam vs. composite beam behavior

(http://www.colincaprani.com/files/notes/Composite%20Design.pdf)

From figure 1, the non-composite beam deflects further compare with the composite beam. The "I" value has changed. With composite beam, there is a significant increase in the moment capacity. Therefore the metal decking can also be used as permanent formwork for pouring concrete slab, saving construction time. Also for non-composite beams, the concrete slab is not mechanically connected to the steel beam so it behaves independently when there are moderate to large levels of applied force. Because of weak bending nature of concrete, the slab deforms around its own neutral axis. The bottom of the slab slides freely on the flange of the beam, loading to slipping. Figure 2 demonstrates how non-composite beam react to different forces.

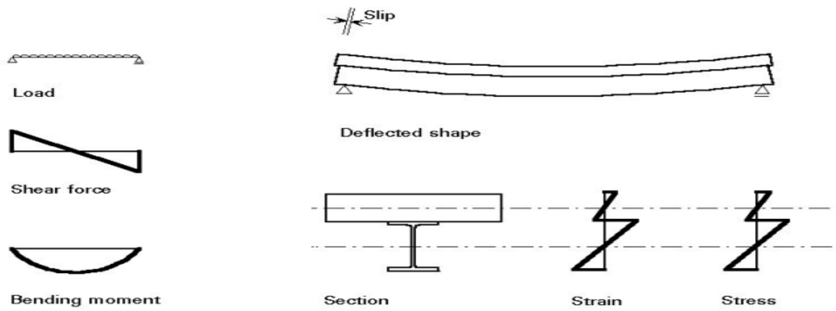


Figure 2: Non-composite beam behavior

(http://www.colincaprani.com/files/notes/Composite%20Design.pdf)

In the case of composite beam, the metal studs are fabricated into the beam flange as a connection between the concrete and steel beam. Therefore two components act together when carry a load. The studs also prevent the slipping behavior. Figure 3 demonstrates how non-composite beam react to different forces.

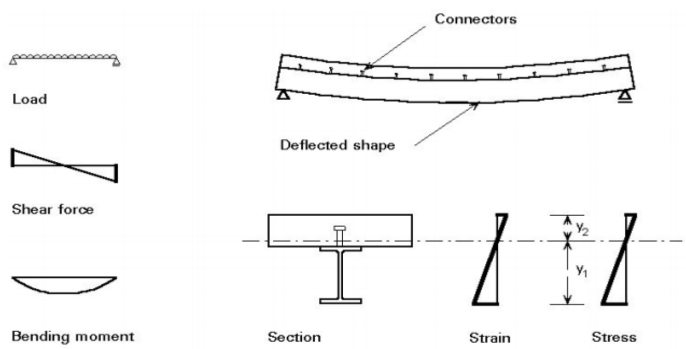


Figure 3: Composite beam behavior

(http://www.colincaprani.com/files/notes/Composite%20Design.pdf)

According to non-composite beam design in RAM SBeam (Bentley.com), a non-composite beam is designed in these following cases 1.There is no positive moment. 2. The maximum negative moment (as for cantilever) is more than twice as large as the maximum positive moment on the beam. 3. Construction time: Both composite beam and non-composite beam have advantages and disadvantages for structural design. Therefore to optimize the design, the engineers must consider all possibilities to suit the project.

2.3.3 Partial Composite

It is assumed that, if enough shear connectors are provided to reach full composite action according to the *AISC Steel Manual*, only a desired value less than that is required. It is more efficient in terms of cost if enough studs are installed to reach the desired strength and deflection behavior. The resulting design section is called a partial composite section; the section that does not have enough stud connectors to reach the full flexural strength of the composite beam. This type of approach provides an effective solution for many practical designs. There is a common practice that total strength of shear studs used in a beam should not be less than 25% of the shearing strength required for full composite scenario. Also, studs must be sufficient to satisfy permissible longitudinal spacing requirements.

2.3.4 Shored VS. Unshored

When unshored construction is chosen, the steel beam must be able to support wet concrete and construction loads until the concrete has sufficiently cured to provide composite action. This obviously makes unshored construction seem more expensive than shored construction as more steel is needed to support greater loads. Shored construction requires the added cost of temporary supports to help the steel beam support the construction loads. The slabs are also susceptible to cracking after the supports are removed. So despite shoring construction sounding like a better option, cost of providing supports and covering of the cracks leads to unshored construction as being the more economical case most of the time.

2.4 Foundation Design

The founndation or substrucure is part of the structure that is usually placed below the surface of the ground. The foundation acts as a load resistor to the undergound soil and rock. When a structure is loaded for settlement, soil is compressed significantly. Thus, it is important to meet the three essential requirements in the process of foundation design: load capacity, total settlement and differential in settlements. (P.Coduto, 1999) To meet these requirements, foundations are designed to transmit the load of the structure to a soil stratum of sufffficient strength and spread the load over a sufficiently large area of that stratum to limit the bearing pressure and soil stress within permissible values.

2.4.1 Shallow and Deep Foundations

Foundations are broadly categorized into two types: shallow foundations and deep foundations. Shallow foundations allow load transmission from structures to near-surface soil or rock. Spread footing foundations and mat foundations are the two types of shallow foundation. A spread footing foundation is a reinforced concrete enlargement located at the bottom of columns or bearing wall. The magnitude of the load and soil properties are the two main factors in the processs of determining the required footing dimensions and reinforcement detail. Mat foundation is another type of shallow foundation that has a physical apperance like a slab that supports an entire building. Mat foundations have the advantage of sturctrual continuity due to the fact that they contain only one large spread footing. Mat foundations have the functionality to reduce or distribute building loads, and results an reduction in differential settlement between adjacent areas. Although spread footing foundations and mat foundations share a lot of similarities, mat foundation is recommended for heavy structures and extreme low soil capacity because mat structure often required thicker individual footing, thus higher contruction cost. (P.Coduto, 1999).

Deep foundations are used for large structures when shallow foundations are insufficent to supporting the poor soil conditions. Deep Foundations can transmit structure loads to deeper soil or rock than shallow foundation. Deep foundations are broadly separated into three categories: piles, drilled shafts and caissons. Piles are rectangular shafts that are typical made of steel, wood or concrete. They are installed vertically and driven into the ground. Drilled shafts are cylindrical, reinforced concrete piers that are installed by first drilling cylinderical holes into the gound, inserting steel reinfocing bars and finished by filling them with concrete. (P.Coduto, 1999)

2.4.2 Geotechnical Report

A geotechnical report was completed by Maguire Group Inc. to determine the soil capacity for the Gateway parking lot in October of 2005. This geotechnical report contains important insights to the soil condition of the area located near where the new Gateway Building will be located. Investigation of the geotechnical report is required in order to determine the proper type of foundation for the new building. Figure 4 is the soil map for the Prescott Street Parking Garage and associated facilities. Since there was not any directed information provided for the new Gateway Building, soil information for the nearest site is used. The plaza is the site located closest to the new Gateway Building and seems to have similar soil conditions. Thus, data for the plaza located on the upper level was studied to determine the proper type of foundation.

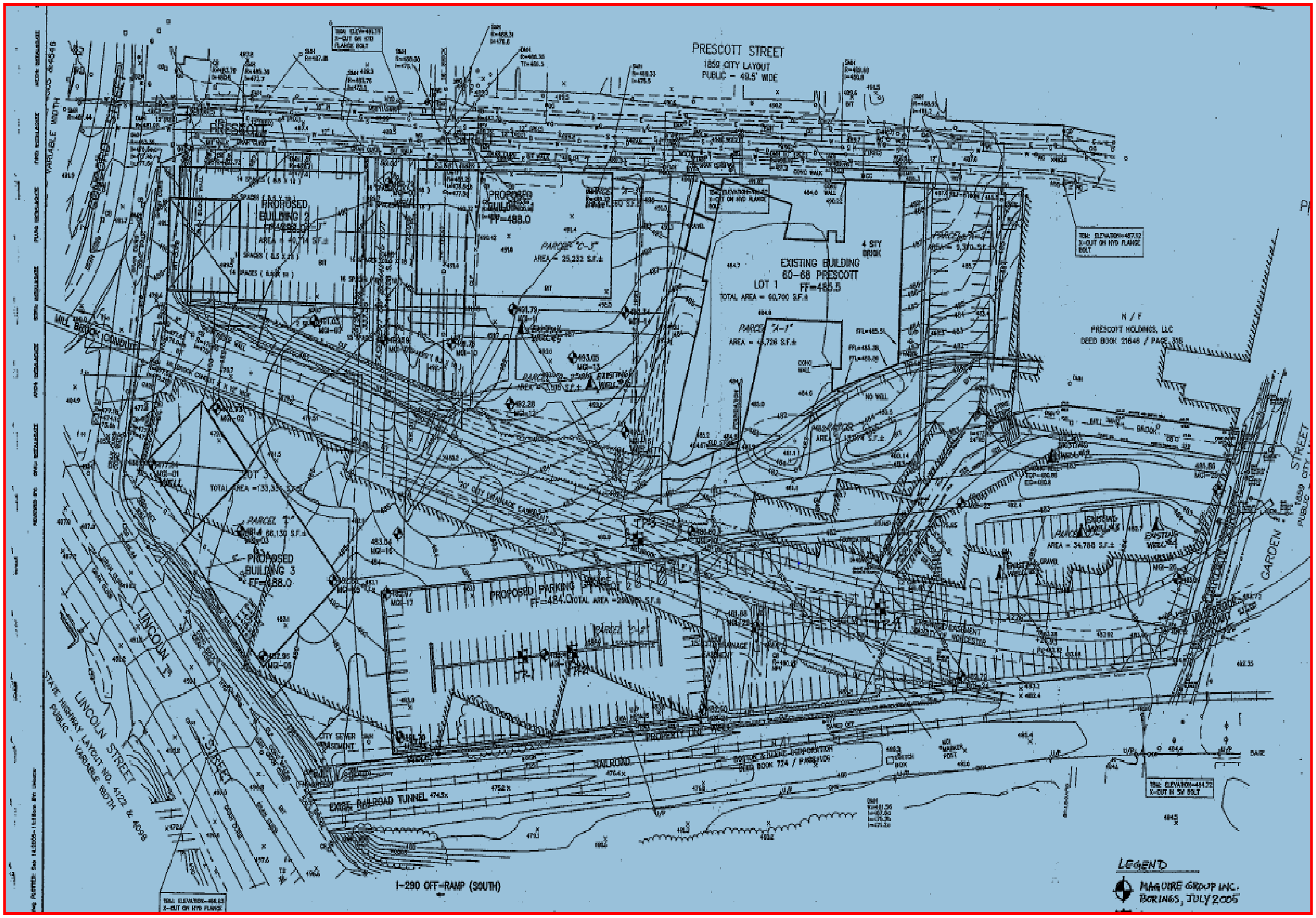


Figure 4: Prescott Street parking garage and associate facilities soil map

The results of the boring conducted for this report show the soil properties beneath the plaza. A copy of a soil profile obtained from the boring data may be found in Appendix D. The plaza area soils excavated between 3 and 10 feet below the grade are described in the report: *Fine to Coarse SAND, trace to little Silt, trace to little gravel, trace cobbles, trace construction material fragments-brick, concrete, wood and asphal, ranging in Unified Soil Classificaion System (USCS) group symbol between SM, SP and SW.* With this detailed decription, the permissible soil capacity for the new building location maybe determined using *Massachusetts Building Code: 780 CMR 120.R Guidance For Selection Of Foundation Material Classes.* (WBDC Gateway Project Proposed Parking Garage and Associated Facilities, 2005)

2.5 Slab-on-ground

A Slab-on-ground is defined in the *ACI 360R Manual* as a concrete slab supported by ground, whose main purpose is to support the applied load by bearing on the ground. To support the various applied loads, different slab thicknesses or stiffening elements such as ribs and beams are determined. Design and construction of slabs-on-ground can be affected by both technical and human factors. Loadings, soil-support systems, joint types and spacing, design method, slab type, concrete mixture, development of maintenance procedures, and construction processes are the technical factors that are involved in the design and construction of Slab-on-ground. Furthermore, human factors like worker's abilities, feedback to evaluate the construction process, and conformance to proper maintenance procedures should also be considered in the process of design and construction of Slab-on-ground. (ACI Manual Of Concrete Practice Part Four., 2010)

There is no single design technique that would cover all of the four different types of slabs. Thus, a number of design methods were identified for each type of slab corresponding to its application. Slab-on-ground are categorized into four types: unreinforced concrete slabs; slabs reinforced to limit crack widths; slabs reinforced to prevent cracking; and structural slabs. For unreinforced concrete slabs, design involves determining the slab thickness without reinforcement. This type of concrete slab is simplest to construct and generally costs less. Slabs reinforced for crack width control are a type of slab that is detailed to minimize the crack width between joints. Bars or welded wire reinforcement are used in this type of slab design to provide moment capacity at a crack section. Slabs reinforced to prevent cracking are the third type of slab, and these are designed to prevent cracking due to load, shrinkage and restrained thermal deformation. Shrinkage compensating concrete is the common material used for this type of slab; it has the highest resistance to shrinking (ACI Manual Of Concrete Practice Part Four., 2010)

Identified appropriate support system is important part in the design and construction of concrete. Information such as geotechnical engineering reports; modulus of subgrade reaction; modulus of subgrade reaction; design of slab support system; site preparation; and inspection and testing of the slab support system are used when addressing issues relating to support systems. Geotechnical reports are performed for most of the building projects today because engineers heavily rely on the soil data that they provide when selecting appropriate foundation design. Geotechnical technical reports include evaluations and recommendations related to the subgrade material and subgrade classification. (ACI Manual Of Concrete Practice Part Four., 2010)

The thickness selection of slabs-on-ground is the most significant design factor affecting slab stiffness. Currently, there are three thickness determination methods developed by different industry organizations: Portland Cement Association (PCA), Wire Reinforcing Institute (WRI) and Corporation of Engineers (COE). The PCA method (ACI Manual Of Concrete Practice Part Four., 2010)is based on Pickett’s analysis. This method is design for interior loadings on the surface of the slab. WRI design chart (ACI Manual Of Concrete Practice Part Four., 2010)for interior loadings are based on the analysis of discrete element of computer model. The slabs are represented by rigid bars, torsion bar for plate twisting, and elastic joints for plate bending. (Wire Reinforcement Institute) The COE (ACI Manual Of Concrete Practice Part Four., 2010) is the thickness design method based on Westergaard's formula for edge stresses in a concrete slabs-on-ground.

2.6 Automatic Sprinkler System

An automatic fire sprinkler is classified as an active fire protection system. The basic system consists of a water supply system, piping system and sprinkler heads. It is important that sufficient water is supplied to the pipes with sufficient pressure and flowrate. If the pipes do not supply water of the required flow and pressure demand, the sprinkler system will not operate properly, which could possibly allow fire to spread. Sprinkler heads, which are connected to pipes, remain closed due to pressure applied by either heat-sensitive glass bulbs or two metal fusible alloys. The glass bulbs or fusible alloys react when ambient temperatures reach the activation temperature allowing the sprinkler head to release water. Figure 3 delineates metal fusible alloys breaking off and setting off a sprinkler head into operation. Although multiple sprinklers are required to protect an entire structure, only one or two sprinkler heads activating will be sufficient to suppress fire in the place of origin. (ArtimNick, 1994).

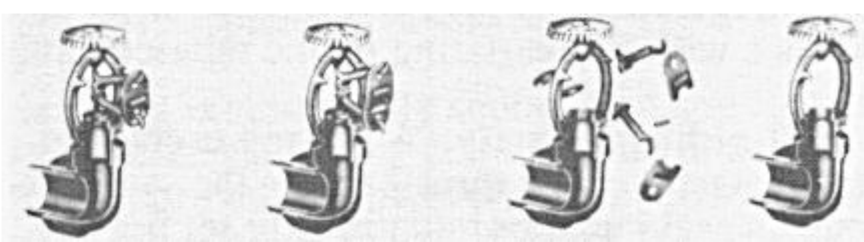


Figure 5: Sprinkler head in operation (ArtimNick, 1994)

Fires in U.S. cities and large industrial fire losses in the late eighteenth and early nineteenth centuries have led to the development of fire sprinklers, building codes, fireproof buildings and the model fires. (Cote, 2008) Although automatic sprinklers are not always required in certain types of buildings according to the building code, “for nearly a century and a half, automatic sprinklers have been the most important single system for automatic control of hostile fires in buildings and many desirable aesthetic and functional features of buildings that might offer concern for fire safety because of the fire growth hazard can be protected by the installation of properly designed sprinkler system.” (Cote, 2008)

An automatic sprinkler system is an efficient means to effectively suppress fires. Studies of fire events with sprinkler systems have shown striking reduction in risks to life safety and property to substantiate for its effectiveness. Some of the facts found are as follows: the risk of dying from fire in your home decreases by about 80% when wet pipe sprinklers are present, the average property loss decreases by 71% per fire, and also 92% of the time sprinklers operated and were effective. (Hall, 2011)

2.7 Green Roofs

In order to obtain LEED (Leadership in Energy and Environmental Design) points for the designed green roof, a brief introduction about LEED and its standards for green credits. Also an overview about green roof will be given along with its benefits. In additional, an introduction to rain water run off problem will be addressed. An introduction to preliminary design will be also discussed.

2.7.1 LEED

LEED, or Leadership in Energy and Environmental Design, (LEED: Leadership in Energy and Environmental Design., 2006), "is an internationally-recognized green building certification system. Developed by the U.S. Green Building Council (USGBC) in March 2000, LEED provides building owners and operators with a framework for identifying and implementing practical and measurable green building design, construction, operations and maintenance solutions." LEED promotes sustainable development practices through a suite of rating systems that recognize and promotes the application of construction practices for better environment and health performance. Buildings comply with LEED standards has many advantages. It improves the surrounding environment by focusing on five important areas of sustainability: site development, water conservation, efficiency energy usage, material selection, and quality of the indoor environment. Building with LEED standard brings a lot of benefits to the owner and occupants. According to USGBC, by applying LEED standards, the building can reduce the energy usage and operating cost as a result of improving self-performance. "Studies show that the energy-efficient electrical and HVAC systems in green building produced a direct 20-year present value energy saving to the facility of approximately $6.00 per square foot to $14.00 per square foot" (RSMeans. Building Construction Cost Data 2012, 2012). LEED also added more asset value to the building and promotes the owners' dedication to sustainability and environmental concern. Besides, with LEED certification, the owner and the builder can benefit from federals and local incentive, and tax credit programs.

A project can archive LEED certification by a process of evaluation of six major sections: sustainable sites, water efficiency, energy and atmosphere, materials and resources, indoor environmental quality, innovation and design process. LEED rating is applied to all type of construction types, from commercial to residential. It works with entire project stages: from design process to construction process, operation, maintenance. LEED extends beyond the project itself and serves its surrounding neighborhood. The certification is categorized into four levels as shown in the Table 1:

Table 1: LEED Construction Certification Levels

Level	Points Required
Platinum	52-69
Gold	39-51
Silver	33-38
Certified	26-32

(LEED: Leadership in Energy and Environmental Design., 2006)

2.7.2 Overview of Green Roof in Building Construction

Recently, WPI has promoted the LEED certified buildings project. It proves that WPI is in progress to achieve better environmental goal, takes advantage of variable local and state incentives.

"*A green roof system is an extension of the existing roof which involves a high quality water proofing and root repellant system, a drainage system, filter cloth, a lightweight growing medium and plants*." (Greenroofs101, 2011)

Just like trees and vegetation everywhere on the planet, a green roof helps mitigate heat in the air and makes the roof top surface cooler than the surrounding air. A green roof can be assembled on most of types of buildings including industrial, residential, and commercial facilities.

2.7.3 How Does the Green Roof Work?

A green roof serves as a shading system that reduces surface temperature below the roof’s associated medium. Therefore, the solar heat transmitted into the building is decreased or re-emitted back to the atmosphere. Furthermore, the vegetation of the green roof acts as a protection underlying media from damage due to UV and wind. The water that transpires through the leaves of the plantings evaporates and cools the surrounding air. The composition of the roof, geographic location, sunlight exposure, determines the amount of temperature reduction can be achieved under sunlight.

There have been numerous research studies comparing the surface temperatures between a green roof and a conventional roof. For example:

- In Chicago, the summer time surface temperature on the green roof ranged from 91 to 119°F on an August day, while the conventional dark group was 169°F.
- In areas of Florida, studies have found that the average temperature on the green roof was 86°F while the light-color conventional roof was 134°F. (GreenRoofsCompendium, 2011)

With the green roof, less heat flows into the building from outside as same as from the inside transfers to the air which also reduces the urban air temperatures. It also is an insulator, preventing the heat from flowing into the building, and therefore reduces the energy needed to cool down the temperature inside buildings. In the winter, the insulation layer prevents the heat lost from inside the building through the roof. Figure 6 presents the temperature differences between a green roof and a conventional roof

Figure 6: Temperature Differences between a Green Roof and Conventional Roof

(GreenRoofsCompendium, 2011)

2.7.4 Benefits of Green Roof

Table 2: Private and Public Benefits of Green Roof

Private Benefits	Public Benefits
Thermal Insulation	Natural Habitats for Animals and Plants
Heat Shield	Storm Water retention
Use of space	Urban Heat Island effect
	Reduction of dirt and smoke level
	Cities and Landscapes

Today, a green roof is an increasingly integral part of building design; besides the “natural look”, the green roof also indicates how the building interacts with surrounding environment. A green roof gives further benefits including waterproofing, water retention, and thermal insulation, improves climatic environment and become a new natural habitat. It widely affects the sustainable development of ecology system.

2.7.5 Types of Green Roofs

There are two main options of green roof; each of them has to have an appropriate design for load bearing capacity, maintenance, plant selection, and budgets.

- Extensive Green Roof: sustains low bearing capacity, not intended to be roof gardens. The cost use to construct is lowest in 3 forms. The mineral substrate layer contains very little nutrients, suitable for low growing plant such as moss, herbs, and grasses. This type of roof usually prefers sun, wind, and drought tolerances plant communities.
- Intensive Roof/Roof Garden: This type is designed for lawn, perennials, bushes and trees along with walkway, benches, playground, even ponds can be addition features. This roof gives more advantages for designers because there is no limitation is required. The harmony between plant communities on this roof has to be considered. And also the maintenance, irrigation, and fertilization frequency are higher than on the extensive roof.

Table 3: Characteristics for different forms of green roof

	Extensive Green Roof	**Intensive Green Roof**
Maintenance	Low	High
Irrigation	No	Regularly
Plant communities	Moss-Sedum-Herbs and Grasses	Lawn or Perennials, Shrubs and Trees
Growing medium thickness	3-4 in	30-40 in
System build-up height	60 - 200 mm	150 - 400 mm on underground garages > 1000 mm
Weight	60 - 150 kg/m^2 13 -30 lb/sqft	180 - 500 kg/m^2 35 - 100 lb/sqft
Costs	Low	High
Use	Ecological protection layer	Park like garden

(Types of green roofs, 2012)

2.7.6 Green Roof Components

A green roof is composed by essential components as shown in figure below accommodating with a table data which interprets the figure. The green roof construction involves several layers and these layers are defined that data table. The structure of intensive green roof will be slightly different compares with the extensive roof.

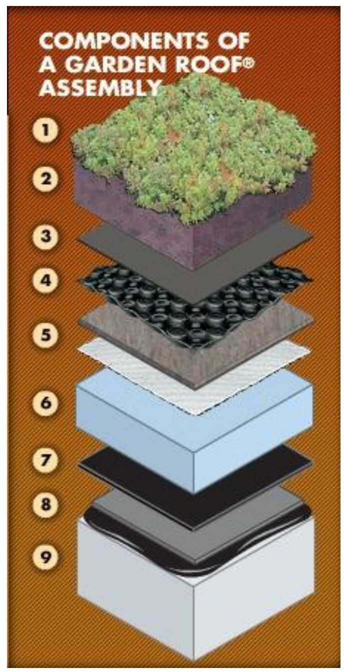


Figure 7: Components of extensive green roof

(HydrotechUSA-Sustainable-design, 2012)

Table 4: Green roof Components

1) Vegetation	Improves aesthetic look, mitigates climate ,heat, and drought tolerance,
2)Engineering growing medium	Supports nutrition for vegetation, water retention, permeability, and density and erosion control necessary to support the green roof.
3)System filter	Prevents particles from being washed out of growing medium; maintains drainage layer's efficiency
4)Drainage/retention/ aeration	Retains water in the troughs; allows excess water to drain away through channels; provides aeration and moisture for upper growing media
5) Moisture mat	Optional, retains moisture and nutrients, and protects root barrier
6)Insulation	Moisture resistant; reduces building heat loss during winter
7)Root barrier	Prevents roots from affecting the roof membrane
8)Roofing membrane	Protects the roof from water
9)Structural Roof deck	Supports the weight of the green roof as well as other live loads and dead loads.

2.7.7 The Rain Water Runoff

According to EPA's analysis, the green roof retained up to 50% of the rain water volume while flat asphalt roofs retained only 14.1% of the precipitation. The rainwater retention by green roof buildings varies from month to month. Retention in cool temperatures (from January to March, October to November) produces more run off, but produces less in the warmer weather months (April to September) (Berghage, 2011). According to the studies in U.S and Europe, about 50% of the annual precipitation in certain climates can be retained by a green roof.

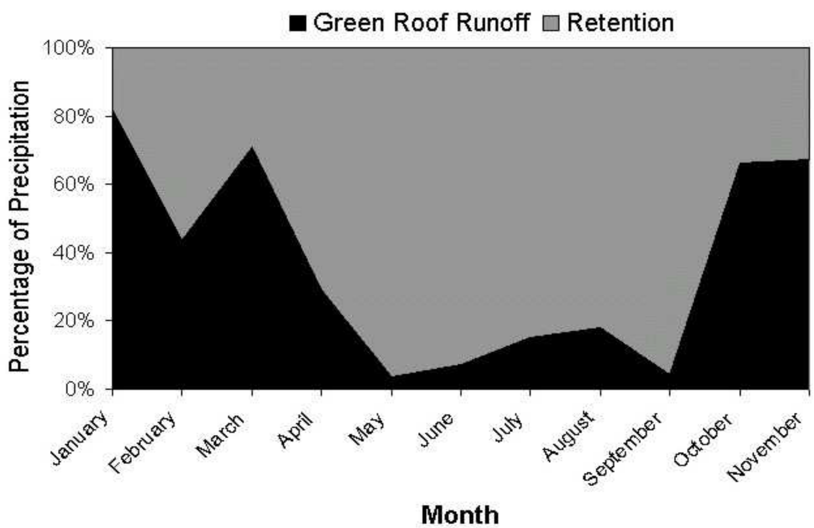


Figure 8: Retention and runoff from green roofs (percentage of average monthly precipitation)

(Berghage, 2011)

The graph provided in Figure 8 shows that in January and March, there is increased runoff as the result of the additional snow or ice melt from the roofs. During that period, the medium of the green roofs may freeze and slowly release the water. Snow also accumulates and melts over longer periods.

The water runoff rate can be calculated using the rational method. In this method, the runoff coefficient of different type of surface is critical. For example, the table below was cited from Green Roof Policies : Tools for encouraging sustainable design by Goya (Ngan, 2004) to shows numerous runoff coefficients

Table 5: Various Runoff Coefficients

Type of Green Roof	Thickness (cm)	Form of Vegetation	Annual Runoff Coefficient
Extensive	2-4	Moss-sedum	0.60
	4-6	Sedum-moss	0.55
	6-10	Sedum-Moss-Herb	0.50
	10-15	Sedum-Herb-Grass	0.45
	15-20	Grass-Herb	0.40
Intensive	15-25	Lawn-small shrub	0.40
	25-50	Lawn-shrub	0.30
	50-50	Lawn-Shrub-Tree	0.10

2.8 Structural Analysis Programs: *Robot* and *RISA*

In the software market today, there are uncountable numbers of computer building structural analysis programs that will speed up the design process significantly. This project also involved the uses of two structural analysis programs: *RISA* and *Robot*.

As stated in the *RISA*-2D user manual, *RISA* is a structural analysis program that creates and analyzes a real-world model of building. *RISA* produces structural analysis results in a few steps. *RISA* begins the calculation by solving the model with the original applied loads, determining the shear force for each member, and deformation is calculated based on the shear force calculated in the previous step. Last, the software reviews the solution results and makes necessary change to the design. For most of the time, the procedure will need to be repeated several times (RISA-2D User Guide, 2010)

As stated in the Autodesk *Robot* Review Manual, Autodesk *Robot* Structural Analysis is a single integrated program used for modeling, analyzing, and designing structures. The program allows user to first create the structure and complete the analysis later. *Robot* also allows informational transfer from other Autodesk modeling software. can analyze the structure in many dynamics: Spectral, seismic, push over, P-delta, buckling deformation and plasticity. *Robot* can also analyze the true structure geometry of frames, plates, and shells, and virtually any defined shapes for any configuration. Moreover, *Robot* can also perform code check calculation of each structural member. (Autodesk Robot Structural Analysis Imperial Getting Started Guide, 2010)

Chapter 3: Methodology

This chapter provides a brief summary for all the methods used to complete the project. A more detailed methodology was included in each of the design chapters.

In the beginning stages the architectural design was reviewed to determine the building occupancies for various floors to designate appropriate design loads using *ASCE Minimum Design Loads for Buildings and Other Structures.*

In order to provide the owner with the most cost effective design, 12 different design alternatives were developed to examine the behavior of different bay size areas, composite and non-composite structural elements, and different beam spacings. After identifying the 12 design alternatives, typical beams, girder and columns were designed using the LRFD method introduced in the *Structural Steel Design 4th Edition* and *Steel Construction Manual.* Once the beam sizes for all the steel members were determined, the foundation spread footing was designed to support the columns using *MA Building Design Code* and the Geotechnical report provided by *Maguire Group*. Cost estimations were presented for all design systems. The *RS Mean Building Construction Cost Data 2009* was referenced to estimate the cost of each member per linear foot.

To evaluate the 12 alternative designs, five criteria were identified by adopting the perspective of the building owner which consisted of total cost, floor depth, number of columns, usable floor area and number of different member sizes. From these criteria, a scoring rubric was established to assess each of the designs and select the best out of them all.

The structural frame design for evaluation only used gravity loads. The design was kept simple in order to finish designs for all 12 design scenarios under limited time. After a design layout was selected, lateral frames, Slab-on-ground, irregular bay areas, elevator and stairs were designed for a more complete structural design.

A lateral frame was designed to resist wind and seismic load effect using the LRFD method. The lateral frame was then modeled in two structural analysis programs *Robot* and *RISA* for second-order moment test. The result produce from the two programs were compared

Slabs-on-ground was designed using the WRI Method introduce in *ACI Manual of Concrete Practice Part Four.* Frames for the stair and elevator were designed for different load requirements. A different bay size was used for the trapezoid area.

After the building design was completed, the square footage cost of the entire building was calculated using *RS Means Building Construction Cost Data 2009.* The cost estimated using the unit cost method was then compared with the cost for a four story office building present in *RS Means Square Foot Costs 2009.* An overall project cost was also approximated.

A green roof was designed by first investigating the performance of structural members under a combination load of green roof assembly's saturated weight and roof's component dead load. A typical roof assembly from HyrotechUSA was used in the computation process. Two types of structural member were designed: composite and non-composite, then one of them was chosen based on cost per square foot. In addition, the amount of rain water runoff was examined with and without the green roof. Last, LEED points were aggregated for the building based on the green roof design.

A code analysis of the building was conducted using the *IBC 2009* edition, to see the fire safety requirements of the Gateway Park Building. A code analysis was performed to consider both a building with and without an automatic sprinkler system. The difference between the two code analyses was identified.

After the code analysis was completed, a sprinkler system was designed using the guidelines and regulations prescribed in *NFPA 13: Standard for the Installation of Sprinkler Systems 2010 Edition.* The step by step procedures were described and the final water demand and cost for sprinkler system was determined using the material costs listed in the *RS Means Building Construction Cost Data 2012*.

Chapter 4: Alternative Designs

4.1 Introduction

For this chapter, the main focus was on examining different bay size areas, composite and non-composite structural elements, and different beam spacings. The first step in the design process was choosing the bay size. Two typical bay sizes were considered in the design with different beam spacings. Repeating the same bay size in the floor plan would increase constructability, lowering the overall cost. However, it is not easy to use a single bay size for the entire building area as it may vary for areas with heavier loads and non-rectangular edges. The two typical bay sizes were determined as: 20’x20’ and 30’ and 30’x30’. Furthermore, three difference filler beam spacing were determined for each of bay size. Three different spacing of 4’, 5’ and 10’ were assigned to bay size 20’x20. Three different spacings of 5’, 6’ and 10’ were assigned for bay size 30’x30’. A typical 5-inch concrete slab and 2-inches metal decking was assumed for this design process. Refer to Figure 9 and 10 for the summaries of design systems and the 12 alternatives that were investigated.

The loading conditions are always identified in the earlier stage of the design process. *Minimum Design Loads for Buildings and Other Structures* commonly known as *ASCE 7* was established by the American Society of Civil Engineer for finding applicable design loads. The building was classified as multiple occupancy that consisting both laboratory and classroom spaces. Thus, different design live loads were considered for the system selection. A comparison of shored and unshored systems was performed in order to provide the owner with the most economical recommendation.

The spread footing foundation design was also established for each bay size. At the end, total cost including beams, girders, columns and footings was completed for each of the alternative. The preliminary cost for each alternative was established using *R.S. Means Building Construction Cost Data 2009.*

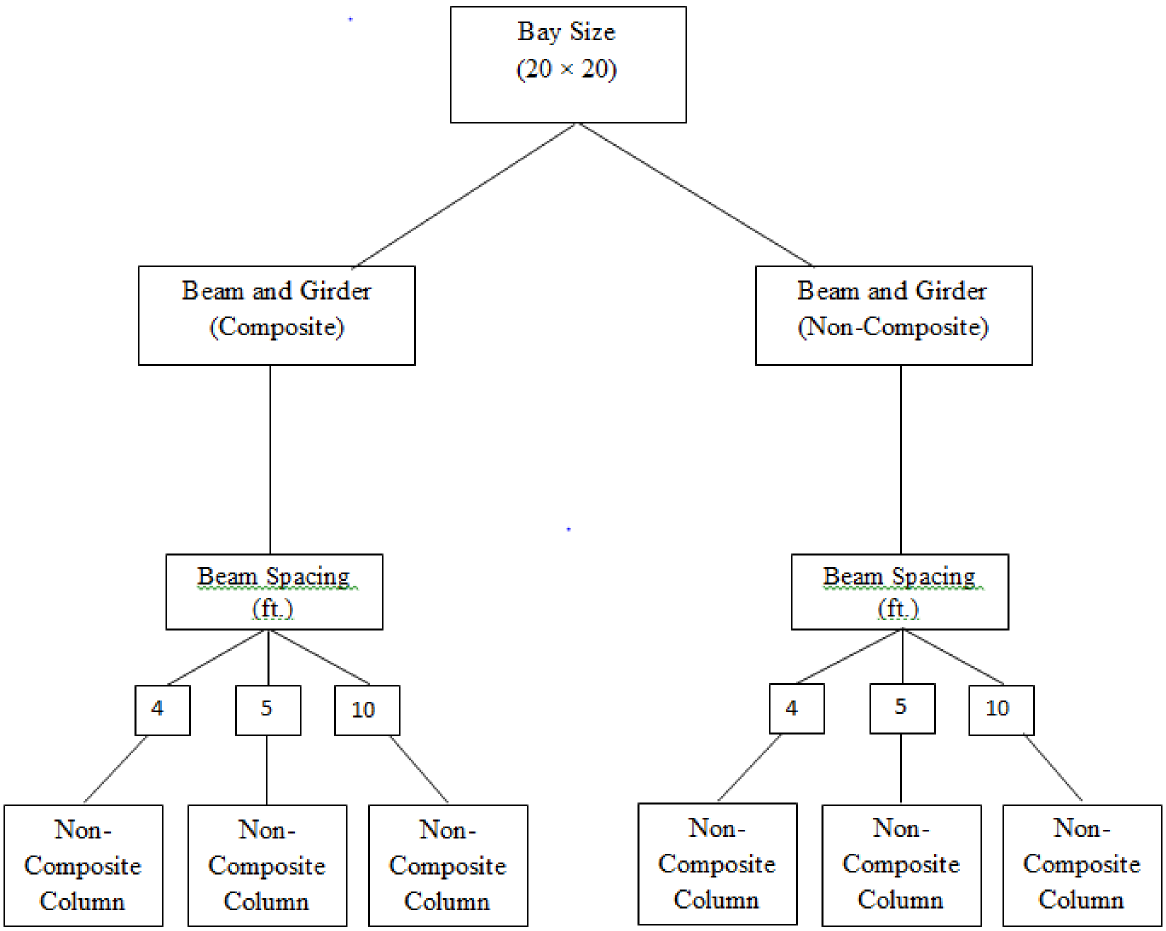


Figure 9: System Group I Alternative Design Scenarios

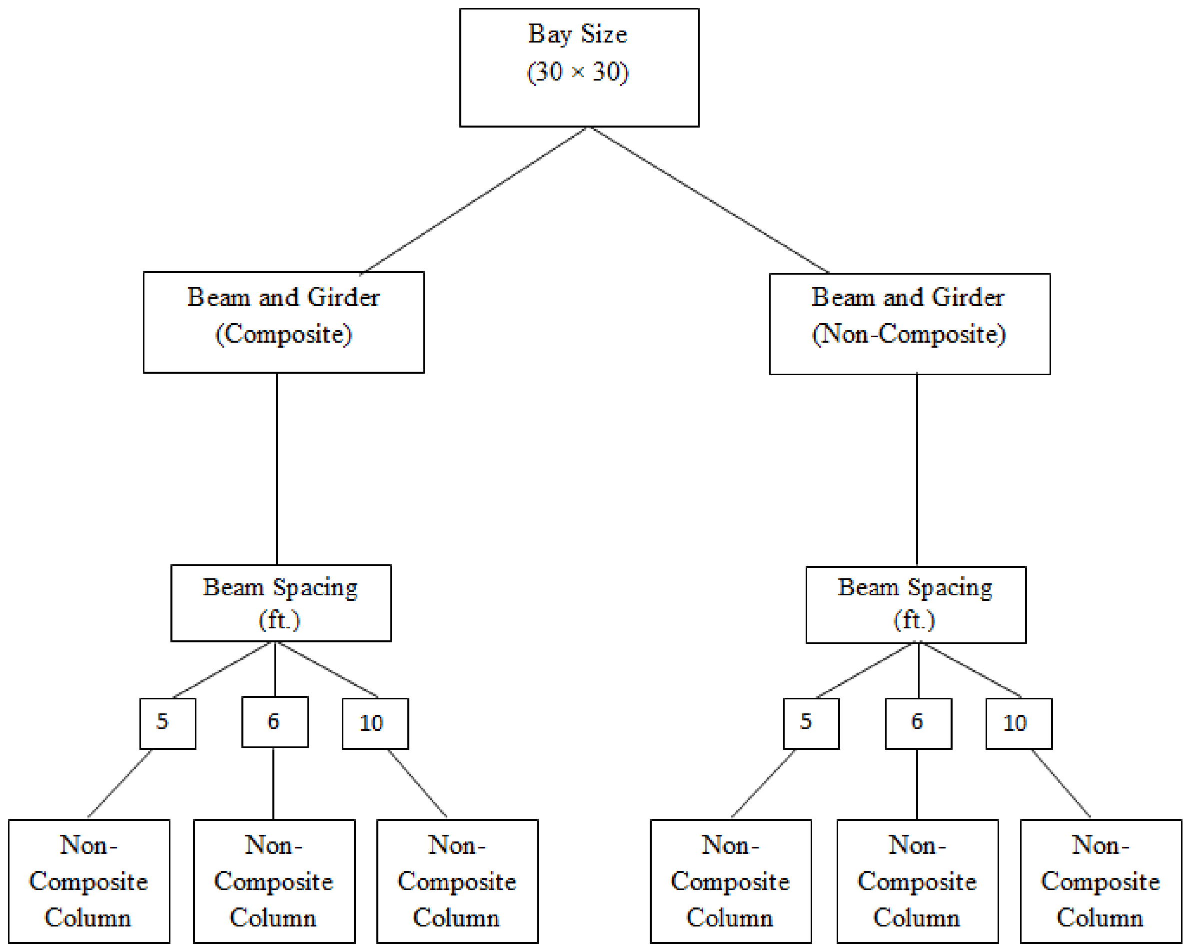


Figure 10: System Group II Alternative Design Scenarios

4.2 Structural Design Methodologies

This section summarizes structural design procedures using flow charts. The flowcharts include: non-composite beam and girder design, composite beam and girder design and non-composite column design. Refer to Appendix B for the step-by-step design process each topic.

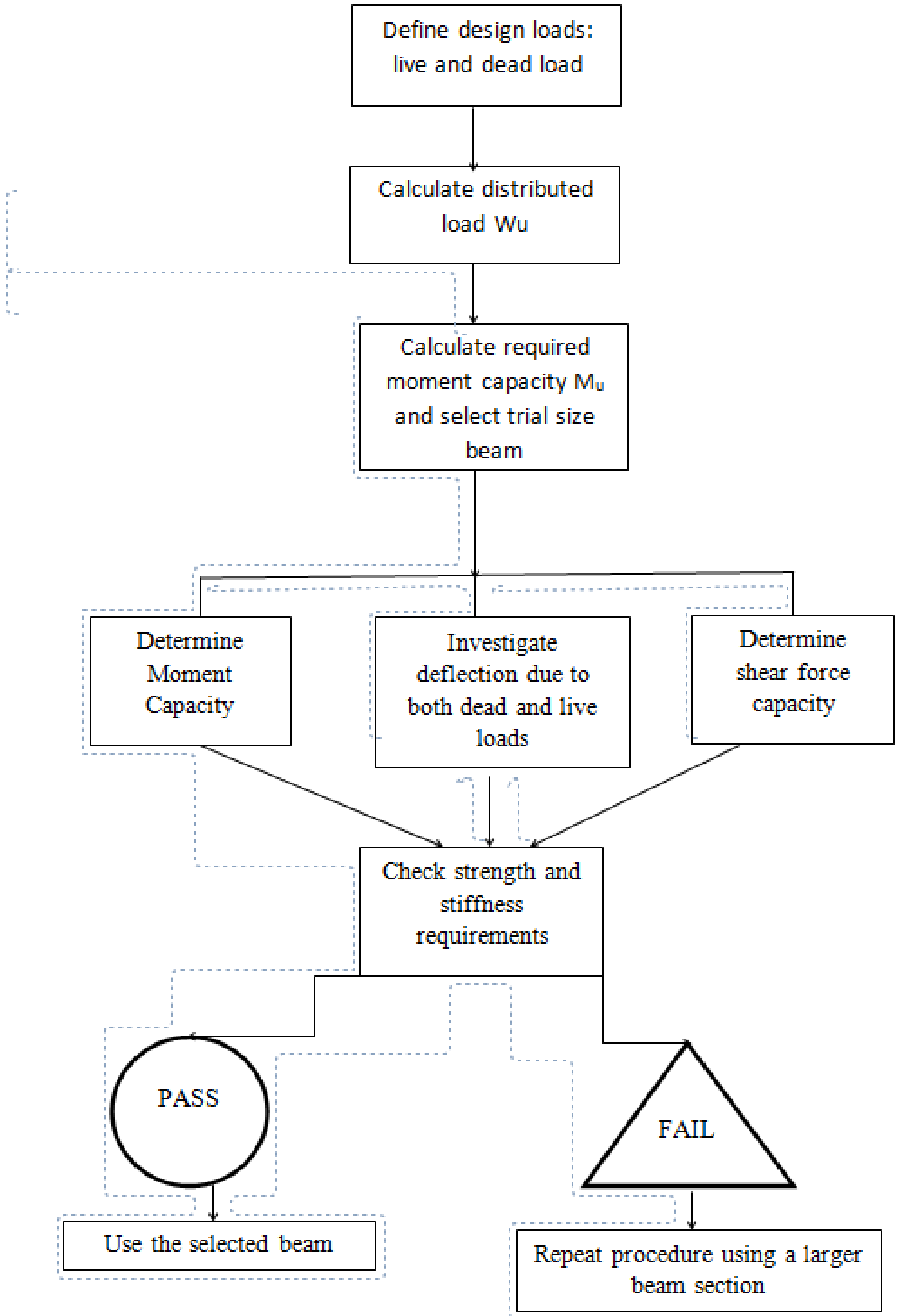


Figure 11: Non-Composite Beams and Girders Design Procedure

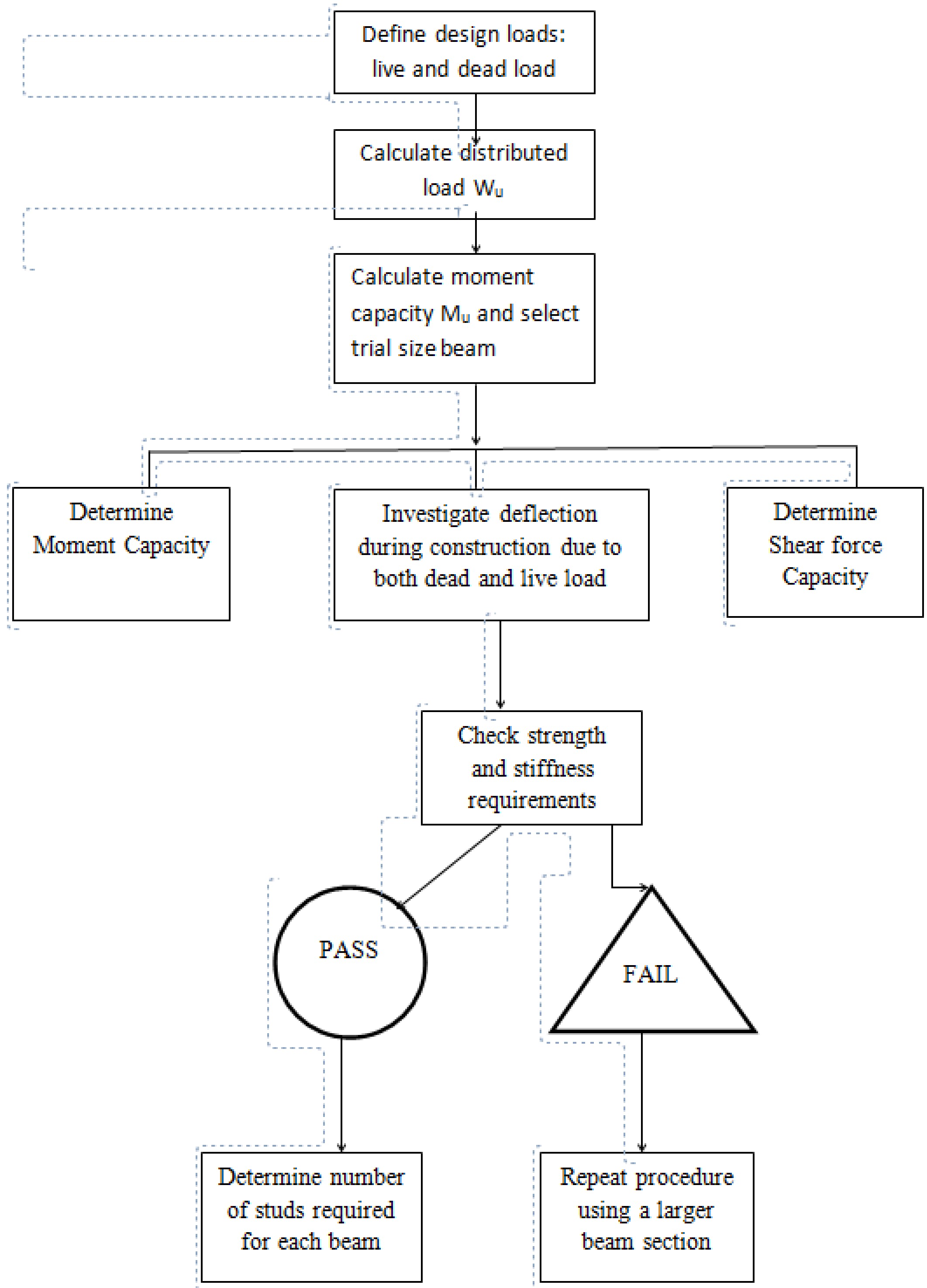


Figure 12: Composite beams and girder design procedure

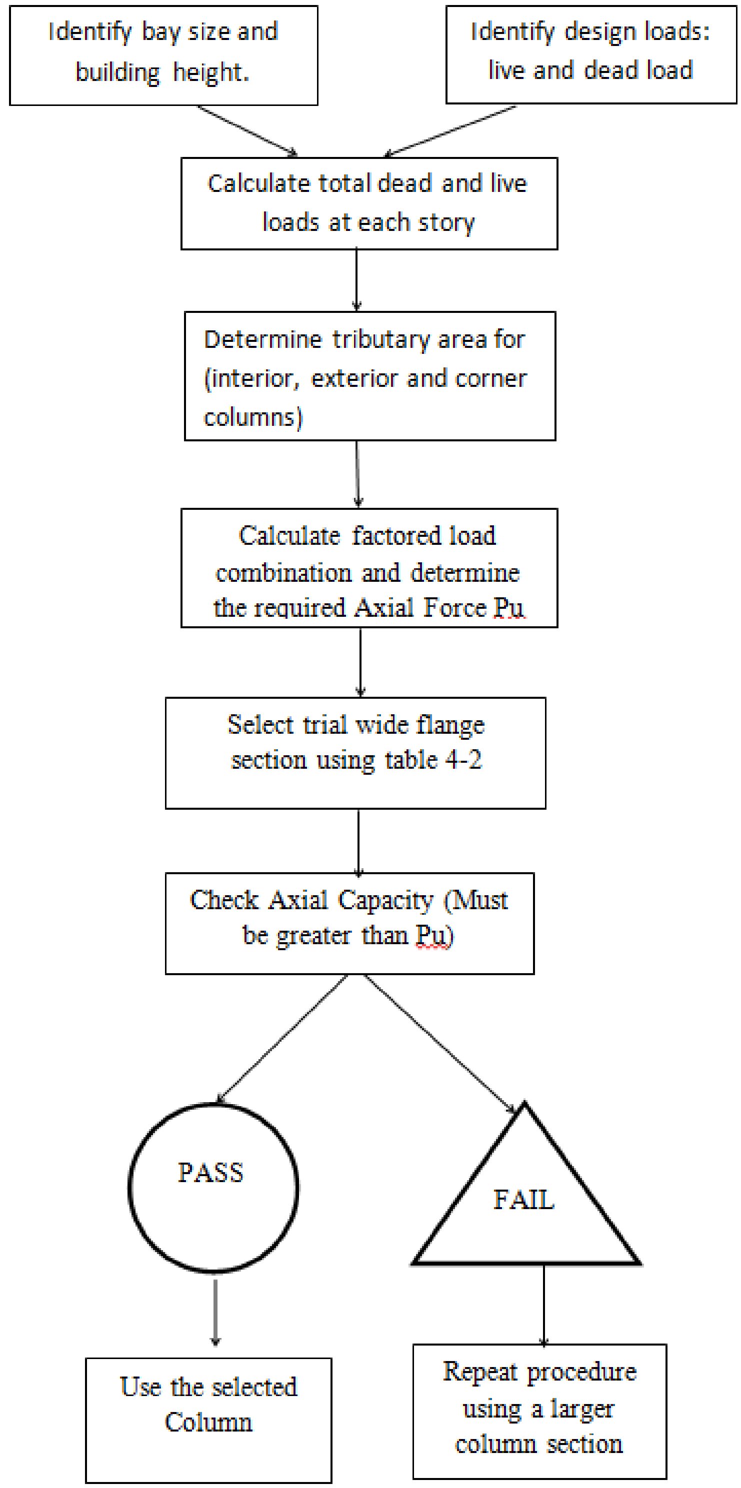


Figure 13: Non-Composite Column Design Procedures

For each structural design scenario, a structural analysis was performed to select the appropriate W-beam sizes for beams, girders and columns. The different levels of load applied to interior and exterior beams and girders and to the interior, exterior and corner columns were all taken into consideration while performing the analyses. Also, after one particular design scenario was selected, a structural analysis was performed on all of the elements mentioned above on that particular design scenario, using an increased live load of 100 psf. The group thought that using a 100 psf live load rather than the 50 psf live load for office space would provide more flexibility for the building owner and would not restrict use to only office space occupancy when leasing the space. While performing the structural analyses a number of trends and relationships were observed. The calculated beam and girder sizes for both composite and non-composite system are presented in Tables 6, 7 and 8.

Table 6: Calculated beam sizes for Non-composite Systems

	Position	Beam size	Girder Size
20’x20’			
4’	Interior	W10×15	W14×34
	Exterior	W10×12	W12×30
5’	Interior	W10×17	W14×34
	Exterior	W10×15	W12×30
10’	Interior	W10×30	W14×34
	Exterior	W10×22	W12×30
30’x30’			
5’	Interior	W16×36	W27×84
	Exterior	W14×30	W24×68
6’	Interior	W14×48	W24×94
	Exterior	W14×34	W24×68
10’	Interior	W21×44	W24×84
	Exterior	W16×45	W21×68

Table 7: Calculated beam sizes for Composite Systems

	Position	Beam Size	# of Studs (3/4")	Girder Size	# of Studs (3/4")
20'x20'					
4'	Interior	W10×12	10	W12×30	12
	Exterior	W10×12	6	W12×19	8
5'	Interior	W10×15	8	W12×30	12
	Exterior	W10×12	6	W12×19	8
10'	Interior	W12×19	10	W12×30	12
	Exterior	W10×15	8	W12×19	10
30'x30'					
5'	Interior	W12×26	12	W21×55	28
	Exterior	W12×16	12	W18×40	20
6'	Interior	W14×26	12	W21×55	20
	Exterior	W12×19	10	W18×40	20
10'	Interior	W14×38	18	W21×55	20
	Exterior	W14×26	12	W18×40	14

Table 8: Non-Composite Column Sizes

Bay Size	Location	Column Size
20'×20'	Interior	W14×61
	Exterior	W14×43
	Corner	W14×34
30'×30'	Interior	W14×99
	Exterior	W12×72
	Corner	W14×61

4.2.1 Calculation Result Analysis

For both the interior and exterior non-composite and composite beams, the moment created by the loads on the beams increased as the spacing increased. This is because when the spacing increases, the tributary area increases, which requires the beam to support an increased amount of total load. However, for both the interior and exterior non-composite and composite girders, the design moments decreased as the spacing of the supported beams decreased. When the beam spacing is increased, it affects the number of beams acting on the girder. The greater the beam spacing, the lesser the number of beams required. Therefore, there are fewer yet heavier beams. For both beams and girders, the interior members resulted in greater member sizes than those for the exterior members because the size of the tributary area of the interior members is greater than the tributary area of the exterior members. This trend was also true for column sizes with corner columns requiring the smallest section sizes, the exterior columns required somewhat larger section sizes, and then the interior columns required the largest section sizes.

For almost all of the beams and girders, it was the construction deflection that governed the member size rather than the required bending moment capacity. The group members tried selecting section sizes that would limit deflections to values closest to the allowed maximum deflection of L/360 in order to reduce the construction costs while providing good performance. For composite beams and girders, most of the section sizes were selected to satisfy the deflection limits for unshored construction, which resulted in allowable moment capacities much larger than required for the design loads. To reduce this allowable moment and to reduce the cost of furnishing and installing the ¾" diameter studs, partial-composite beams and girders were used.

4.2.2 Design with 100 psf Live Load Result Discussion

Through the system selection process (described in chapter 5), a design scenario with composite beams (6' spacing o.c) and girders with a 30'×30' bay size was selected. For this particular design scenario, a new set of structural analysis calculations was performed using a design live load of 100 psf. Because the construction live load (which includes the construction load and the weight of the wet concrete) was used to check the deflection for composite members, the increase in the live load from 50 psf to 100 psf did not affect the construction deflection at all.

Therefore the change in live load only affected the moment acting on the members. To accommodate for greater moments due to the 100 psf live load, it was simply sufficient to use the same beam sizes as for the 50 psf live load scenarios and increase the number of shear studs. Although there were only changes to the number of studs for beams and girders, the member sizes did need to be increased for the columns to meet the demand of a 100 psf live load. With a greater design load, larger column sizes were required.

4.3 Foundation Design

For this section, a proper foundational design was determined for the purpose of transmitting loads from the building structure to the supporting soil sufficiently. Selecting between a shallow foundation and a deep foundation was the first step in the foundation design. The shallow foundation was recommended for this project base on the building height and soil quality near the surface. As introduced in the background, there were two main types of shallow foundations: spread footing foundation and mat foundation. The spread footing foundation was designed specifically to support vertical loads distribued by columns. It was a type of shallow foundation that had a square or sometimes a rectangular shaped concrete footings. Such spread footings are typically recommended for the foundation with reasonable bearing capacity.

4.3.1 Spread Footing Design

A typical footing design was determined to correspond with each of the column sizes. The design of spread footing required a proper determination of the size and depth of the footing in order to meet the permissible soil pressure requirements. The geotechnical report provided by Maguire Group Inc. was used in verifying the allowable bearing capacity. The allowable net bearing capacity was found to be 6 tons per square foot using *Table 120.R1A Allowable Bearing*

Pressure for Foundation Materials from *Massachusetts Building Codes*. The associated soil profile description introduced in the *Massachusetts Building Codes* is *Medium dense gravel, widely graded sand and gravel; and granular ablation till.* In addition, the typical footing sizes of 8’x8’ and 12’x12’ were considered solely in order to maintain uniformity and to reduce labor cost for material cost.

4.3.1.1 Methodology

The figure 14 illustrates the procedures to obtain the adequate footing size for each bay size, and the calculations can be seen in Appendix D.

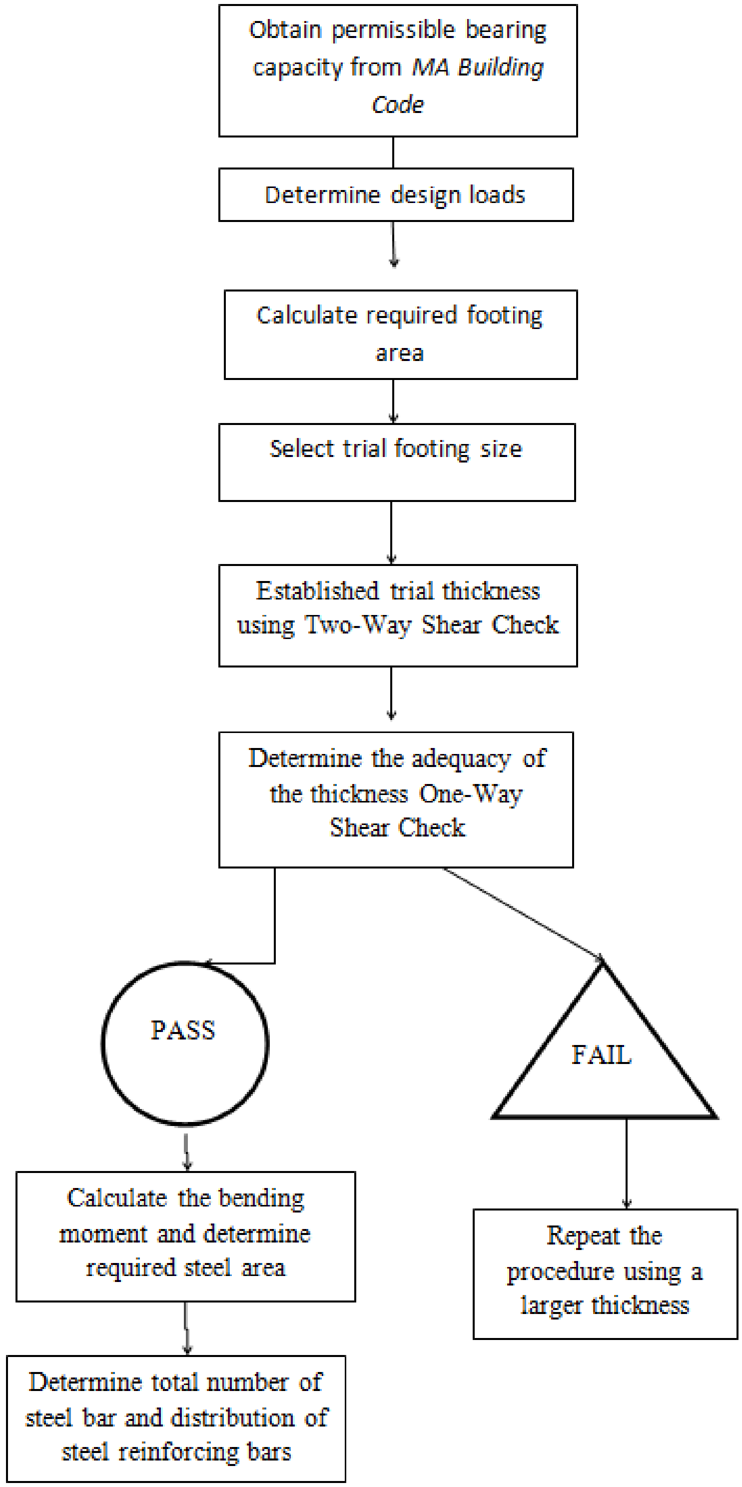


Figure 14: Spread Footing Design Procedures

4.3.1.2 Foundation Results:

Table 9: Footing Sizes for the Two Structural Bays

Bay Size	Interior Footing Size	Bar Size	Exterior Footing Size	Bar Size	Corner Footing Size	Bar Size
20’x20’	8’x8’x20”	9 of No.7	8’x8’x16”	8 of No.7	8’x8’x14”	9 of No.7
30’x30’	12’x12’x24”	12 of no.9	8’x8’x20”	10 of No.8	8’x8’x16”	8 of No.8

Table 9 provides footing sizes for the two structural bays. There were interior, exterior, and corner footings corresponding to the different sizes of columns. Since only the typical sizes of footing were considered, the footing sizes were very much alike with only difference related to their thickness. However in order to maintain consistency, only the largest footing sizes would be recommended for the final design.

4.4 Cost Estimate for all the alternatives

The cost estimate is an essential task in the management of construction projects. The quality of construction management depends on accurate estimation of the construction cost and a reliable schedule for execution of the work. For this section, the cost estimate for each of the alternatives was obtained by combining the costs for beams, girders, columns, and footings. The cost for slab was excluded in the estimation due to the fact that the 5-inch slab was assumed for all the design scenarios. Also, because bay size 20’ x20’ would not completed the building’s structural layout; some of the 20’x30’ and 30’x20’were added. The cost estimate for 20’x20’ included the costs for the two additional bays. All of the cost data were obtained from *Building Construction Cost Data, 2009.* One of the important interpretations when using the manual was that it did not provide all beam sizes; therefore, an interpolation equation was devised to obtain cost data for beam sizes. Moreover, the manual was published two years ago. Thus, proper inflation rate are considered to ensure the accuracy of the cost estimate. The Annual Equivalent Inflation Rate ENR Construction Cost Index was announced to be 3.1%. In addition, a Location Factor for Worcester, MA of 110.1from *RS Means Manual* must be implied in the final cost. Equation 1 illustrates the proper way of addressing the inflation rate and the location factor.

$$Final\ Cost = \left((Original\ Cost) * (1 + 0.031)^{Current\ Year - 2009}\right) * (\frac{110.1}{100})$$

Equation 1: Final Cost Equation by considering both the inflation rate and location factor.

4.4.1 Framing Cost Estimates

Some steps were required in determining the total cost of beams and girders for the non-composite system. The first step in the cost estimate process was to identify all of the member sizes, then to calculate the total number of the identical members. The second step in the estimating process was to calculate the total span length for each member size by multiplying the total number of members and span length for each beam. The span lengths for beams were different due to the fact that there were two different bay sizes. Once the total span length in feet has been defined, the cost for each member size (dollar per linear foot) would be obtained using *data sheet 05 12 23.75* from *Building Construction Cost Data, 2009.* Last but not least, the final step in the cost estimation required totaling the cost of each member size for all four stories. Table 10 through 14 were taken from Appendix E that summarized the total beam and girder cost for both non-composite structural bay sizes.

Table 10: Total beam and girder cost for non-composite bays

Bay Size: 20’x20’	Total Cost (Beam+ Girder)	Bay Size: 30’x30’	Total Cost (Beam+ Girder)
4’ Spacing	$1,332,900	5’ Spacing	$1,588,200
5’ Spacing	$1,256,900	6’ Spacing	$1,716,00
10’ Spacing	$1,096,200	10’ Spacing	$1,167,200

Table 10 showed the bay size 20’ by 20’ with 10’ spacing resulted the lowest cost of the total beam and girder in the composite system comparison. With the intense assessment, another observation had been noticed; the large spacing between the beams ended up a lower overall material cost because a fewer number of beams were used. This cost estimate did not include the cost of either column or foundation; thus, any conclusion derived from the table would be biased.

The procedure for determining the total beam and girder cost for both of the composite and non-composite system were the same with one exception rule. The costs of shear studs must be included in the equation in order to complete the cost for the composite structural system. Table 10 was taken from Appendix E that summarized the total beam and girder cost for the composite structural bays.

Table 11: Total beam and girder cost for composite bay

Bay Size: 20'x20'	Total Cost (Beam+ Girder)	Bay Size: 30'x30'	Total Cost (Beam+ Girder)
4' Spacing	$1,405,200	5' Spacing	$1,134,300
5' Spacing	$1,345,500	6' Spacing	$990,700
10' Spacing	$1,005,600	10' Spacing	$899,600

Table 11 showed the bay size 30' by 30' with 10' spacing resulted the lowest cost of the total beam and girder in the composite system comparison. Another observation apparent to naked eyes was the large spacing between the beams resulted a lower overall material cost because of a fewer number of beams were used. However, this cost estimate did not include either column or foundation cost; thus, any conclusion derived from the table would be biased.

The process used to determine the total cost for the non-composite columns was very similar to the beams and girders. One thing to keep in mind was that the columns sizes were determined based on the total service loaded from all four stories; thus, the column sizes were the same for both of the composite and non-composite system. Thus, the total costs for columns were only compared between the two different sizes of bay. The Total Cost for the columns was obtained after following the three steps. The step one involved the identification of the different column sizes for each one of the structural bay. And, the step two required calculating the total number of column and the total span length of column for each different size. Last step involved calculating the total cost for each different size of column by using the data sheet provided in *RS Mean Building Construction Cost Data.*

Table 12: Total Column Cost for each Bay Size

Bay Size	Total Column Cost
20’x20’	$450,500
30’x30’	$307,300

Table 12 shows that the bay size 30’ by 30’ had the lowest total cost of the column due to the fact that the fewer number of columns were used for larger bay size. However, this cost estimate did not include either beam and girder cost or foundation cost; thus, any conclusion derived from the table would be biased.

4.4.2 Footing Cost

The different spread footing sizes were determined for columns in different sizes. To calculate the total cost for the spread footing, we ought to first obtain the total number of the concrete footing. Total number of footing required for each structural design scheme was the same as the total number of columns. Secondly, we could determine the total required concrete in cubic yard by multiplying the volume for each size of the footing and the total number of footing. Lastly, the concrete cost was found to be $133.1 per cubic yard from *Building Construction Cost Data*, 2009. Thus, the total footing cost was calculated by multiplied the total unit of needed concrete and concrete cost in dollar per cubic yard.

Table 13: Costs of Footing for each Bay Size.

Bay Size	Total Footing Cost$
20’x20’	$46,000
30’x30’	$30,900

Table 13 showed the bay size 30’ by 30’ had the lowest total cost due to the fact that a fewer number of footing was needed for larger bay size. However, this cost estimate did not include any cost for beams; thus, any conclusion derived from the table would be biased.

4.4.3 Complete Cost for Each of the Design Alternative

Once the total cost was determined for all beams, girders, columns, and footings, a completed cost for all the design scenarios were obtained for a comparison. Table 11 shows the overall construction cost and cost per square foot for twelve scenarios. Also, they were ranked from the lowest to the highest. In the economic stand point, a scenario with the lowest price ranked number 1 in the comparison process.

Table 14: Total Cost and Cost per Square foot for the combination of framing and footing

Scenario	Total Cost	Cost per Square foot	Cost Rank
N.20.4.	$2,022,000	$26.7	8
N.20.5	$1,933,000	$25.6	7
N.20.10	$1,744,800	$23.1	6
N.30.5	$2,102,200	$27.8	10
N.30.6	$2,251,907	$29.8	12
N.30.10	$1,609,600	$21.3	4
C.20.4.	$2,106,600	$27.9	11
C.20.5	$2,036,700	$26.9	9
C.20.10	$1,638,900	$21.7	5
C.30.5	$1,571,100	$20.8	3
C.30.6	$1,403,100	$18.6	1
C.30.10	$1,448,500	$19.2	2

Table 14 shows the composite bay size 30’x30’ with 6’ spacing had the lowest overall cost. Thus, design scenario C.30.6 would be recommended to the owner if the system seletion was based on cost alone. However, cost was not the only factor that determined the quality of design project; other factors like usable area and constuctablility should also be included.

Chapter 5: System Selection

5.1 Systematic Approach

In selecting out the best structural system, it is hard to single out one design scenario as the best choice amongst a total of twelve design scenarios. A design scenario cannot be judged and evaluated solely on one criterion. As is the case for many merchandizing products, the least expensive option does not necessarily guarantee high performance or pleasing aesthetics. Just looking at the results table is not ideal as numbers can be deceiving because multi-objective evaluation of structural design scenarios is a complex process. To simplify this process, a systematic approach was taken. The criteria that were deemed important in evaluating the design scenarios were defined to begin the process. Five criteria were established by adopting the perspective of the building owner which consisted of total cost, floor depth, number of columns, usable floor area and number of different member sizes.

5.1.1 System Identification

Twelve design scenarios were all compared in this process. All design scenarios were named based on each individual design's characteristics. For example, the scenario that consisted of a 20'×20' bay size with non-composite beams (spaced 4 feet on center) and girders was named 'NC20.4'. The abbreviation NC was used to represent non-composite, 20 to represent the 20'×20' bay size and 4 to represent the 4 feet spacing of the beams. Therefore, a design scenario named 'C30.10' would consist of a 30'×30' bay size with composite beams (spaced 10' on center) and girders.

5.1.2 Design Objectives

Each criterion and its evaluation are discussed below.

5.1.2.1 Total Cost

This building is a steel structure which means the whole structural frame uses steel. Therefore, looking at the overall steel cost is important. The number or the size of the structural member is the factor that affects the steel cost. Also on the ground floor, all columns are supported by footings. These footing costs were added to the total steel cost to come up with an estimate of the total building cost. The final cost of each system was divided by the total floor area to find out the cost per square foot of floor area. All building owners have a finite amount of budget that he or she is willing to spend and will certainly not pay excessive amounts to construct a building.

5.1.2.2 Floor Depth

Floor depth affects the overall building height. The greater the floor depth, the greater the building height will be and vice versa. Building height directly affects the building cost as an increase or decrease in the building height results in change of the required amount of wall, piping and other materials. The floor depth was calculated by adding the concrete slab and metal decking thickness to the girder depth.

5.1.2.3 Number of Columns

The number of columns affects the cost of footing as well as building usage flexibility. Columns are supported by footings on the ground floor so the number of columns affects the overall footing cost. Columns can be a hindrance when trying to use the building floor space. It might not be the best idea to have a column in the middle of a classroom so the number of columns restricts the flexibility of the space to accommodate changes in architectural layout. The number of columns was obtained by drawing the building layout and incorporating the design bay sizes using the AutoCAD software.

5.1.2.4 Usable Floor Area

The usable floor area is the gross area that the building occupant can actually use. Cost for leasing space in the building is affected by the area, so the greater the usable building area, the more profitable it will be for the building owner. The usable floor area was determined by subtracting the total column area from the total floor area. The total column area was calculated by using the column section area and multiplying it by the number of columns.

5.1.2.5 Number of different Member Size

The structural member uniformity could affect the time of construction. Because when more different structural members are used, it means more information on each of the different sizes are required for the steel production. Also, the mills run on a schedule so one of the structural steel member sizes may take longer than other member sizes to be produced, delaying the overall construction time. Delayed construction time also means an increased cost for the labor required affecting the overall cost. Ideally, one member size for the entire structure would be the best case scenario but such is not actually possible in the real world. As a result, three member sizes (for beams, girders and columns) were used as the best case scenario. For the total number of structural members, a typical building will have different member sizes for external and internal beams, external and internal girders and external, internal and corner columns which results in a possible variety of seven different member sizes, which would mean that there is no member uniformity, hence the worst case scenario.

5.2 Methodology

The architectural layout from Perkins + Will showed the southeast corner of the building having a trapezoidal area. This corner was left out in the comparison process. Only the main building area with the dimension of 210’×90’ was taken into consideration in the design selection process.

With the criteria chosen, a matrix table was created with all of the categories as shown below. A pairwise comparison of the criteria was performed using the matrix table to establish preferences and a rationale weighting scheme.

Table 15: Component matrix table

VS.	Member Uniformity	Floor Depth	Usable Floor Area	# of Columns
Member Uniformity				
Floor Depth				
Usable Floor Area				
# of Columns				

Each group member filled in the matrix table to reduce any bias that may occur. Each of the criterions was compared to one another and each group member filled in the blank spaces in the matrix table with the criterion that was thought to be of a greater significance in terms of evaluating the building design. The following matrix tables were the completed tables by each of the group members.

Table 16: Component matrix table result 1

Member A	Member Uniformity	Floor Depth	Total Column Area	# of Columns
Member Uniformity				
Floor Depth	Member uniformity			
Total Column Area	Column area	Floor depth		
# of Columns	# of columns	# of columns	Column area	

Member Uniformity: 1
Floor Depth: 1
Total Column Area: 2
# of Columns: 2

Table 17: Component matrix table result 2

Member B	Member Uniformity	Floor Depth	Total Column Area	# of Columns
Member Uniformity				
Floor Depth	Member Uniformity			
Total Column Area	Column Area	Column Area		
# of Columns	# of Columns	# of Columns	# of Columns	

Member Uniformity: 1
Floor Depth: 0
Total Column Area: 2
# of Columns: 3

Table 18: Component matrix table result 3

Member C	Member Uniformity	Floor Depth	Total Column Area	# of Columns
Member Uniformity				
Floor Depth	Floor Depth			
Total Column Area	Member Uniformity	Floor Depth		
# of Columns	# of Columns	# of Column	# of Column	

Member Uniformity: 1
Floor Depth: 2
Total Column Area: 0
# of Columns: 3

The results were combined and tallied up to determine a weighting factor for each of the criterion. This was done because the group felt that all the criteria did not have the same value and that some were more important than the others. To apply the significance on the criteria, the number of each selected criterion in the matrix over the total number of tally was converted into a percentage and multiplied to each respective criterion score. The table below shows the combined result of the matrix tables. From a possible of 18 tallies, member uniformity, floor depth, usable floor area and number of columns each received 3, 3, 4 and 8 tallies respectively.

Table 19: Final weighting scales

Criterion	Tally Count	Weighting Factor
Member Uniformity	3	3/18
Floor Depth	3	3/18
Total Column Area	4	4/18
# of Columns	8	8/18
Total	**18**	**18/18**

To score the performance of each criterion, a scoring rubric was established with a 10-point scale. The total number of columns and the floor depth were obtained from the actual architectural design plan by *Perkins + Will*. The numbers obtained were set as standards to establish the grading rubric. Anything above the standard would be considered better and anything below the standard would be considered not as good. For the number of different member sizes, the more variance in member size was thought to be less desirable for constructability. So seven different structural members as explained above, was set as the worst scenario and three different structural member sizes were thought as the best possible option, which would consist of one size for columns, one size for girders and one size for beams. A criterion falling under 'Poor' was given a score of 1-2, 'Fair' a score of 3-4 and so on. The table below shows the grading rubric that was used to grade the attributes of each design.

Table 20: Design system scoring rubric

	Poor (1-2)	Satisfactory (3-4)	Good (5-6)	Very Good (7-8)	Excellent (9-10)
Floor Depth	34" or more	31 – 33	28 – 30	25 – 27	24" below
# of Columns	46 or more	42 – 45	38 – 41	34 –37	33 or less
Usable Floor Area	Less than 90% of floor area	90-93% of floor area	93-95% of floor area	95-98% of floor area	More than 98% of Floor Area
# of different Member Size	7	6	5	4	3

The total cost which was identified as a criterion was not put into the grading rubric. After a score was established for all of the systems, the final score was divided by the cost of its respective cost. This step was carried out so that the final score for each system would reflect how many points the owner would get for each dollar expended per square foot, this ratio provides a measure of the value of a given alternative. The best solution alternative would provide the largest value to the owner. A series of bar charts are provided to display the scoring for each criterion, the total points and the value ratio.

5.3 Results

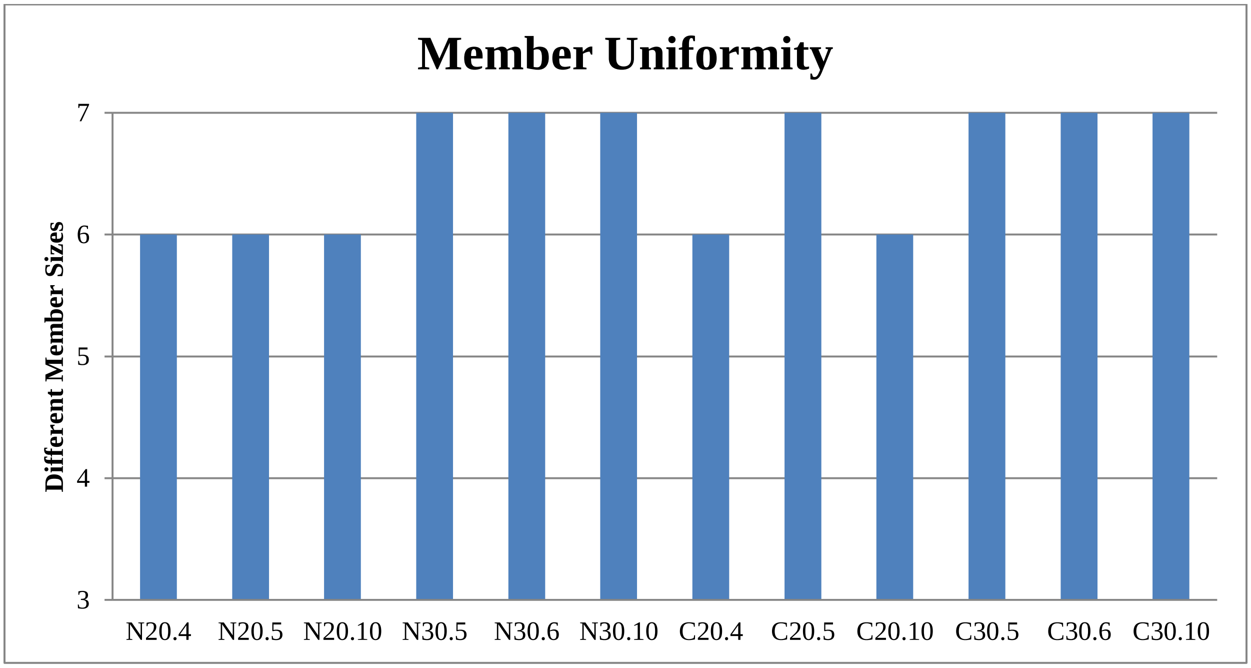


Figure 15: Member uniformity result

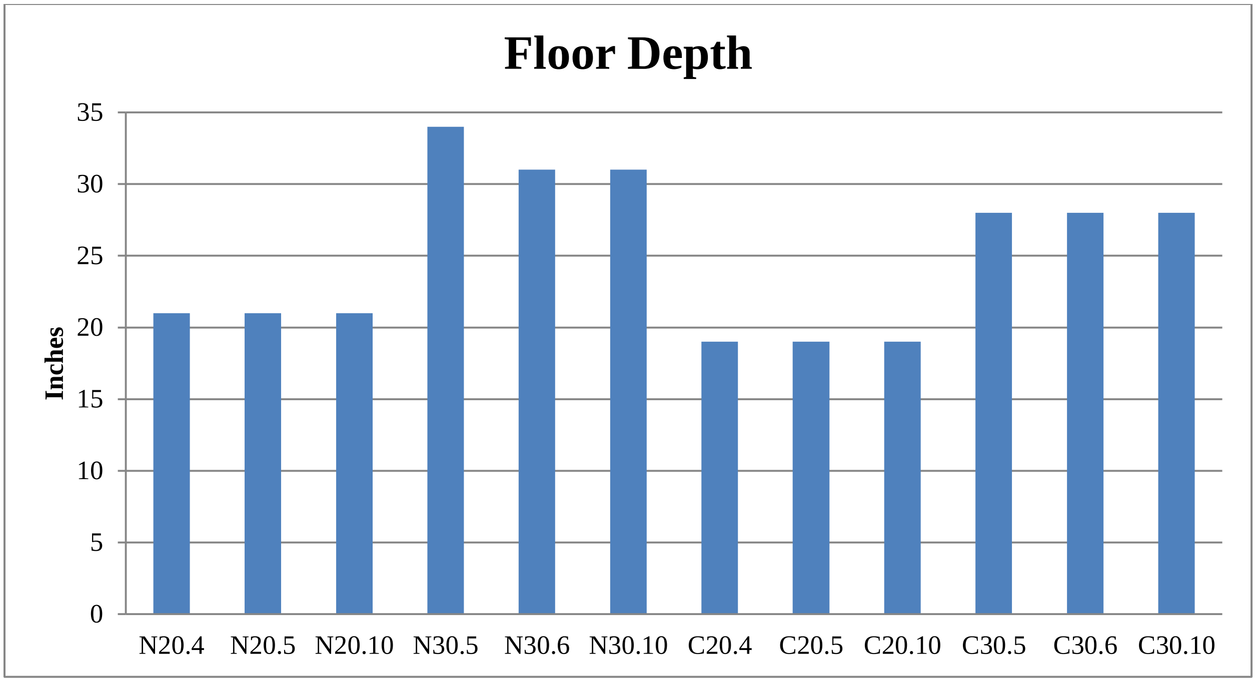


Figure 16: Floor depth result

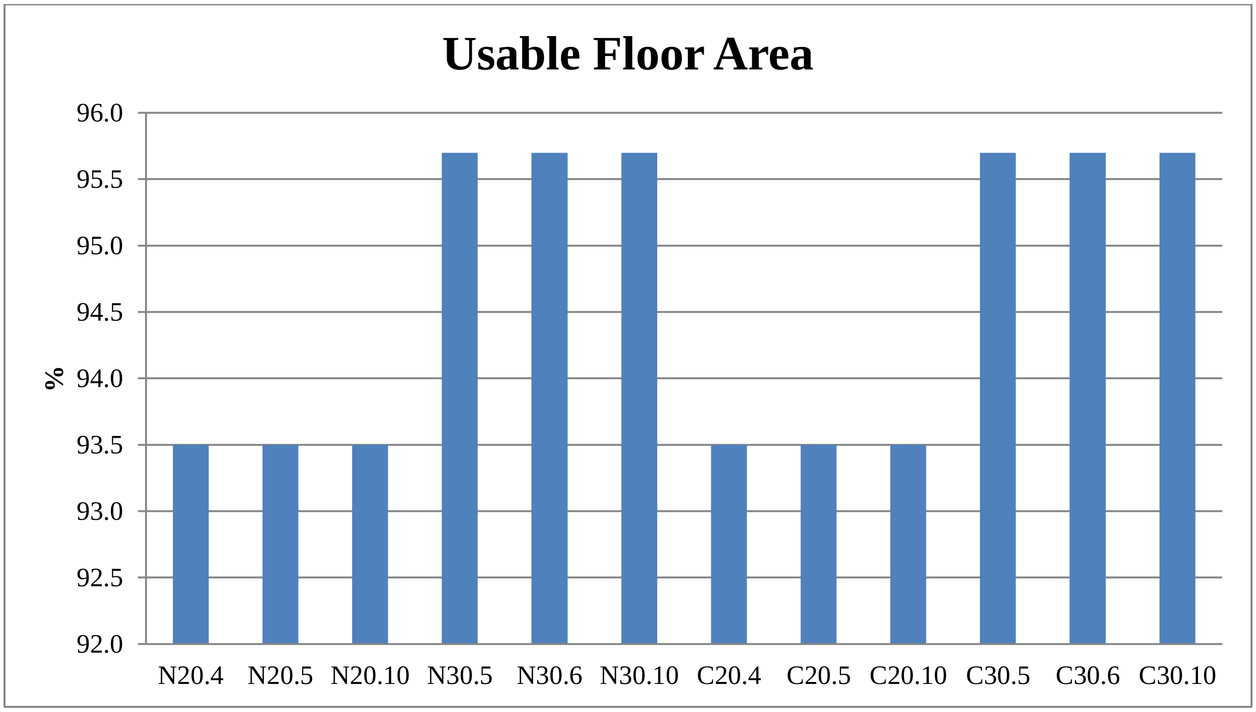


Figure 17: Usable floor area result

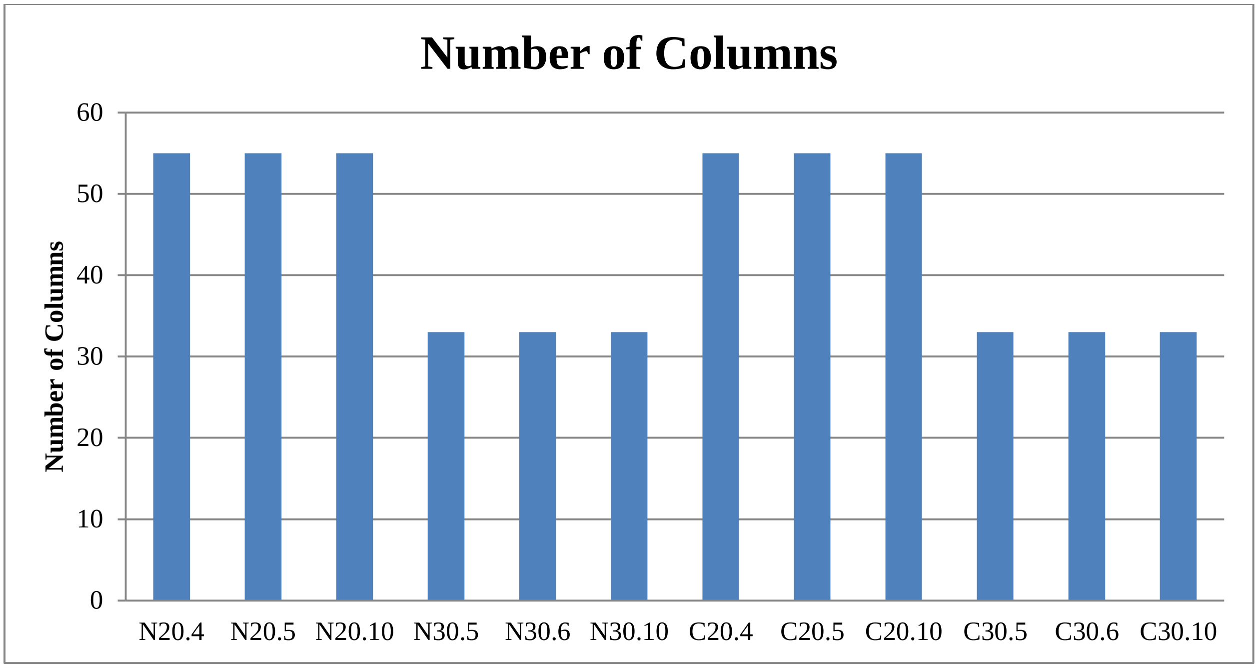


Figure 18: Number of columns result

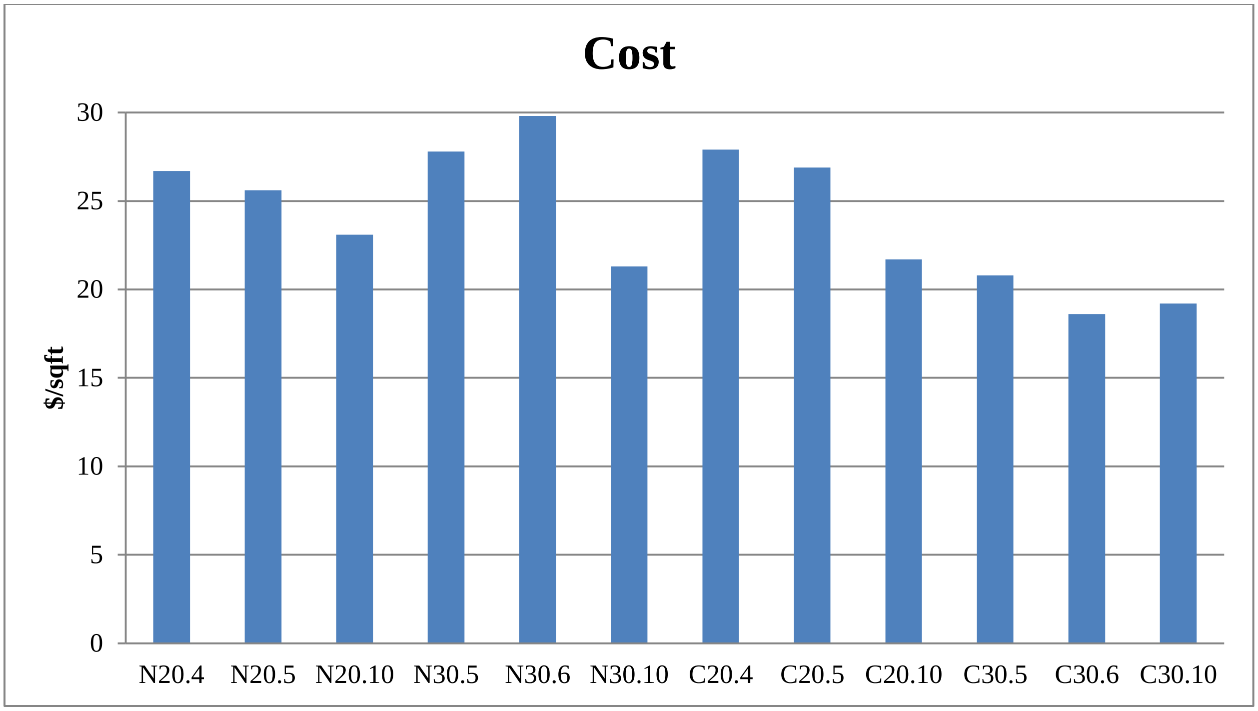


Figure 19: Cost result

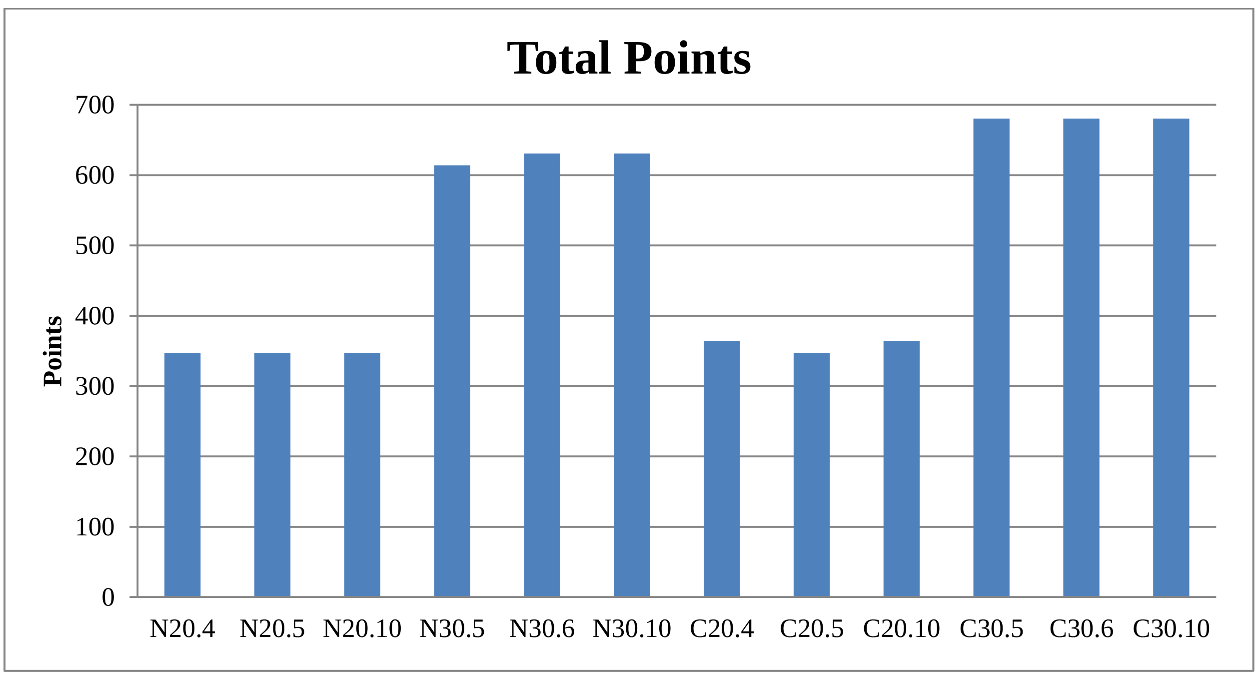


Figure 20: Total points result

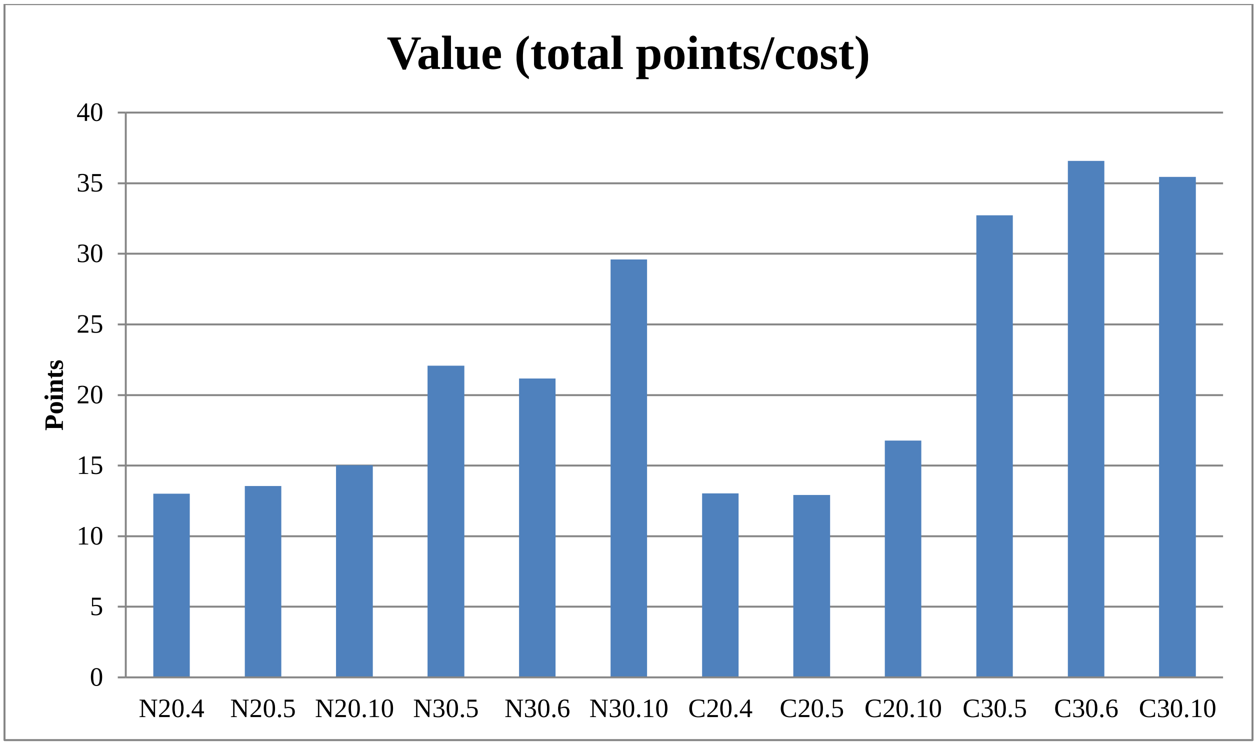


Figure 21: Value of each design system result

The results show that design C30.6 gives the most points in return per $/sqft. It can be seen that the total cost was the most decisive factor in determining the best design scenario. For categories such as number of columns and usable floor area, the numbers were the same for the design scenarios that consisted of the same bay size. The design with a 30’×30’ bay size with composite beams and girders had the most number of points in total and also the lowest costs. As a result, the three design scenarios can be found returning the most value to the owner.

Chapter 6: Lateral Loading

6.1 Introduction

In the structural analysis and design of buildings, in addition to the effect of gravity load created by dead loads, live loads and other loads, the horizontal forces, known as lateral forces also contribute to the deflection of the structure and other impacts. Therefore, the lateral forces must be considered during the design process. The failure of the building under the lateral loads happens due to the sidesway affects generated by wind forces or seismic forces. For that reason, the building must be designed to resist those forces. Some typical solutions are braced frames or moment resistant frames. This chapter will use LRFD design method, guidelines from *ASCE 7-05*, and *Structural Steel Design* (McCormac, 2008) to design the lateral force resisting frame.

6.2 Lateral Loads

Lateral loads may arise from several sources: wind, seismic, soil pressure, eccentricity, unbalanced force, etc. In this project, the magnitude of wind and seismic forces will be the focus for the superstructure.

6.2.1 Wind

Wind force is the main factor of lateral load. Depending on geographical location, the wind magnitude is different. Some regions have fairly high wind forces such as along the coastal area, while others have insignificant to be considered. Wind loads mainly affect the structure of the building by causing positive and negative pressure, and drag forces. The main area under effect of wind loads is the building surface facing directly to the wind, under positive pressure while the leeward side of the building has the negative pressure. The building also experiences the drag effect along the wind direction.

6.2.2 Seismic

Earthquake is the result of the movement of tectonic plates on the Earth. When the earthquake happen, the inertia of the building keeps the building in place, leads to imposition of displacements and forces that damage the structure by creating stress. The area that is under the effect of earthquake suffers live loss and damaged structures. The structures experience random horizontal and vertical movement on the Earth's surface. Seismic design involves proportioning the structural frame so it can withstand the displacement and forces caused by the ground motion. The seismic design in this chapter will emphasize the effect of horizontal base shear.

6.3 Lateral load resisting system

In this project, the lateral load resisting system is a rigid frame. The rigid frame was chosen because it yields opened and flexible use of the interior, such as the location of windows. The connections between members are used to transfer the moment and deformation when the lateral forces are applied. The connections used in this type moment resisting frame are fixed. In this frame, the lateral loads and gravity loads cause the moments to the members. The steel frame is detailed to have sufficient ductility to absorb those lateral forces. The deformation of the frame helps it increase the efficiency when resist the wind forces and seismic loads.

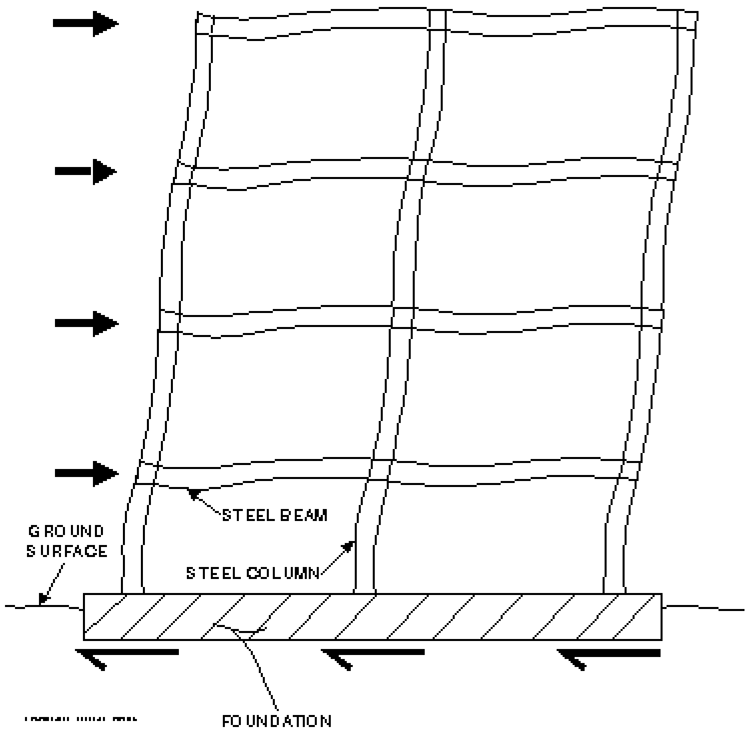


Figure 22: Moment resisting frame

(http://www.propertyrisk.com/refcentr/steel-side.htm)

With moment resisting frame design, there is no requirement for additional members added to the structure, so this is one of advantages of the rigid frame to promote a flexible architectural design and usable area.

In rigid frame design, the deformation is limited to reduce the damage to non-structural components and the noticeable movement of the building. Due to the limitation of deformation, the members are often chosen based on the stiffness requirements. Both gravity and lateral forces contribute to the deformation of the structure. Hence, the governing load combination is used to evaluate the behavior of the frame under combined and gravity lateral loads.

6.4 Determination of Lateral Forces

The design wind load follows the requirement in *International Building Code*, *2009 Edition*. Per *IBC 2009*, the wind load can be determined according to the Chapter 6 of *ASCE 7-05*. The design seismic load can be calculated per chapter 12 in *ASCE 7-05*.

6.4.1Wind Forces

The wind loads were calculated using the guidance of *ASCE 7-05* Main Wind Force Resisting System Method 1 (Simplified Procedure). This chapter provides the factors needed to be considered to adjust the effect of the wind for a specific geographical area. The factors and values necessary to determine the wind loads for the Gateway building are listed below:

Table 21: Factors and values for wind load design

Factor	Notation	Value
Topographic factor: depend on the geographical location of the structure. Because the Worcester area is flat, also shielding providing by surrounding buildings.	K_{zt}	1.0
Importance factor- since the Gateway building considered as the office building, which is category II. (Figure 6-1 ASCE 07-5)	I	1.0
Height: the mean height of the building Mean height is equal to the total height of the building.	h_{mean}	59 ft
Exposure: the building's exposure to the wind. Exposure category B due to Worcester is urban area**.** (Figure 6-2 ASCE 07-5)	λ	1.21
Average Wind Speed: The average wind speed depends on the location of the building and is aggregated from a long period data of monitoring wind speeds. (Figure 6-1)	V	100 mph
Net Wind Pressure- This value is evaluated based on the average wind speed, mean roof height and exposure category. (Figure 6-2 ASCE 07-5)	ps_{30}	15.9 psf
Final wind pressure	$Ps= \lambda K_{zt} I ps_{30}$	19.24sf

6.4.1.1 Beam-column Design

The moment resisting frame was designed to withstand both gravity and lateral loads. For the compressive member, the compressive and flexural forces from gravity loads along with the axial and flexural forces from the lateral loads are analyzed. Following is the steps to perform first order analysis

1
- Determine gravity loads
- Determine wind loads (ASCE 07)
- Determine EQ , investigate all load combinations to identify critical member forces and moments

2
- Using RISA 2.0 to find member forces and joint displacement

3
- Design member sizes

First step was to determine the combination of gravity load and wind loads using LRFD method and ASCE 7-05, and then used the *RISA* 2.0 software to complete the first-order structure analysis. The results obtained were axial forces and moments on critical girders and columns, and also the member displacements. This information was used to design initial adequate girders and columns.

Following the results first-order analysis, it was necessary to perform the second order analysis. The below chart is the process for the 2nd-order analysis.

Magnification factor B1 & B2

- Find C_m by using AISC eq C2-4
- AISC C2-2 : B1=Cm/(1-αPr/Pe1)
- AISC C3-2: B2=1/(1-ΣPnt/ΣPe2)

Euler buckling strength

- No translation AISC eq. C2-5: P_{e1} =2EI/(K1L)^2
- Lateral translation AISC eq. C2-6a: P_{e2}=2EI/(K1L)^2

K factor

- K=1.0 in finding P_{e1}because we assume there was no translation forthe frame
- K for P_{e2} was determing using AISK monograph and values G_A=1.0 , G_B=Σ(Ic/Lc)/Σ(Ig/Lg)

Check WLB &FLB

- WLB: λ<λp=√(E/Fy)
- FLB: bf/2tf<90.5

Check Pr/Pc

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

The magnification factors B1, and B2, along with the Cm parameter were determined for input to the AISC interaction equation, which was used to check the adequacy of the section for combined axial compression and bending moment.

The Euler buckling magnification factors: P_{e1} with no translation and P_{e2} with translation were calculated with different K factors. K=1 was used for P_{e1} due to no translation. For K in $P_{e2,}$ it was determined by using Alignment charts for effective length factor K in sway frame, involving calculation of G_A and G_B based on the moments of inertia and lengths of the member and the adjoining girders and columns.

$$G=\Sigma(Ic/Lc)/\ \Sigma(Ig/Lg)$$

Next, web local buckling and flange local buckling as a part of establishing the flexural capacity were checked before move on to using interaction equation H1-1a and H1-1b in AISC

Pr- axial load , Pc- the strength of the section, along with L_p, L_r, , L_b and flexural strength of the section in Table 3-2 AISC were determined.The last step was to substitute those values to interactive equation below to see of the member was adequate:

Pr/Pc+8/9(Mrx/Mcx+ Mry/Mcy)<1.0 AISC equation H1-1a

Pr/2Pc+(Mrx/Mcx+ Mry/Mcy)<1.0 AISC equation H1-1b

6.4.2 Design Results

Table 22 shows the wind pressure in both vertical and horizontal direction for each zone. The positive values indicate the wind flows toward the surface, while the negative values indicate the suction forces. These values are adjusted based on parameter of height = 59 ft, I = 1.0, λ=1.21

Table 22: Wind pressures

Basic Wind Speed	Zones					
	Horizontal pressure		Vertical pressure			
100 mph	A	C	E	F	G	H
Ps30	15.9	10.5	-19.1	-10.8	-13.3	-8.4
P	18.24	12.7	3-23.11	-13.06	-16.1	-10.16

Table 23: Linear loads and shear forces in transverse and longitudinal direction

	Distributed Wind loads (plf)		Story Forces (k)	
Level	Interior	Exterior	Transverse	Longitudinal
1	186.7	282.82	39.25	20.58
2	186.7	282.82	39.25	20.58
3	186.7	282.82	39.25	20.58
4	93.1	141	19.57	10.26

6.5 Frame and *RISA* 2-D analysis

The rigid frame with moment-resistive connections to withstand lateral forces is shown below

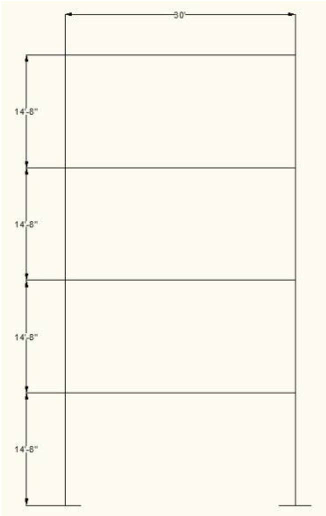


Figure 23: Rigid Frame

With the wind loads determined in section 7.5, the rigid frame is under the effect of those lateral forces.

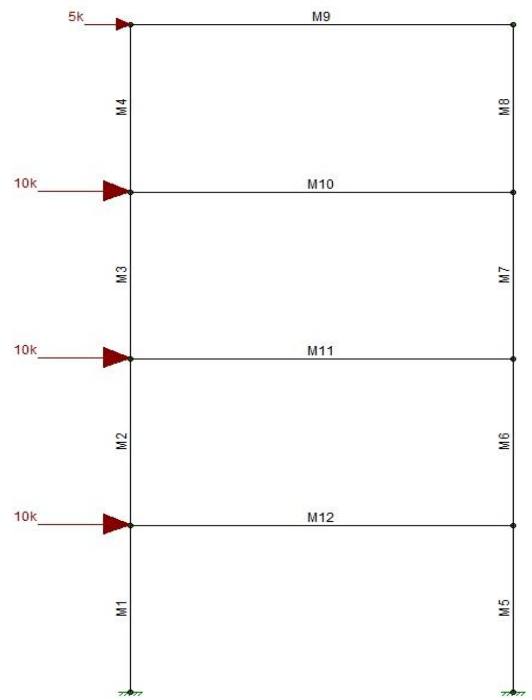


Figure 24: Transverse wind loads

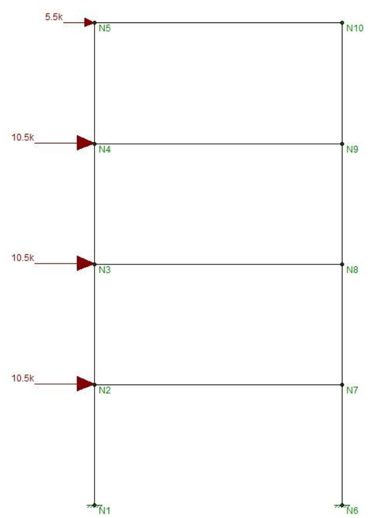


Figure 25: Longitudinal wind loads

The gravity loads also apply on the frame:

Table 24: Design loads for lateral frame

Floor DL (k/ft)	2.1
Roof DL (k/ft)	1.6
LL (k/ft)	0.98
SL (k/ft)	0.68

And the load combination governed is: 1.2DL + 1.6WL + 0.5LL + 0.5SL

The combination of loads were input to *RISA* 2-D. The gravity loads and lateral loads are analyzing separately. The transverse direction was chosen for analyzing because of its larger lateral loads act on the frame compared with longitudinal direction. The values P_{nt} and M_{nt} were calculated when there was only gravity forces act on the frame, while the P_{lt} and M_{lt} based on the displacement of the frame under the wind loads. The program processed and returned the axial loads, also the moment in the table below.

Table 25: Rigid frame values

P_{nt} (k)	169.4
M_{nt} (k-ft)	169.3
P_{lt} (k)	40.26
M_{lt} (k-ft)	336

Also the sway values of each story are also produced; the x defection cannot exceed the story height divided by 360. Id the rigid frame satisfies this limit, the design is acceptable.

Table 26: Rigid frame sway

Story	X-deflection (in)	Height (ft)	Max Sway=H/360
1	0.24	14’8”	0.31
2	0.59	29’4”	0.32
3	0.85	44’	1.47
4	0.99	59’	1.96

With the output from *RISA* 2.0, adequate members were designed to support the combined load. The final rigid frame design is displayed in Table 27.

Table 27: Member sizes for lateral frame

	Level 1	Level 2	Level 3	Level 4
Columns	W14x159	W14x159	W14x159	W14x159
Girders	W16x77	W16x77	W16x77	W16x77

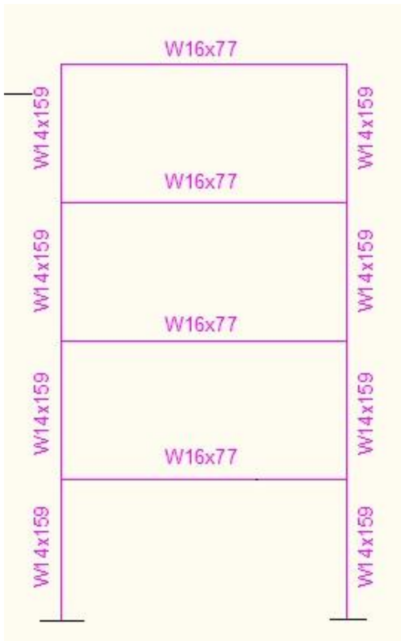


Figure 26: Lateral frame

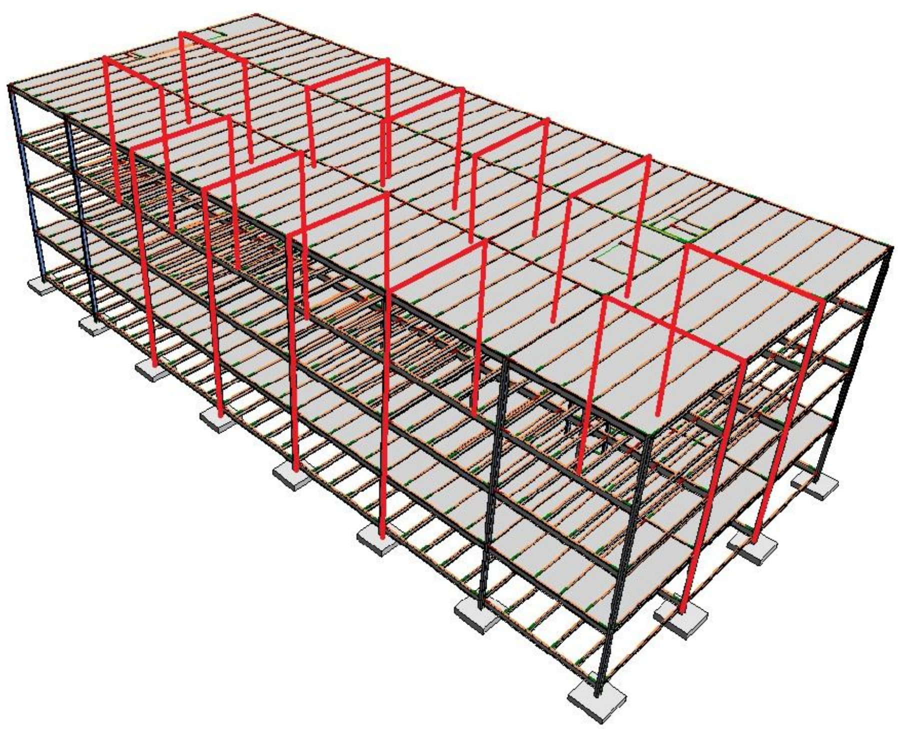


Figure 27: Rigid frame layout of building

A check was performed for longitudinal direction, the results turned out the same design also satisfied lateral loads in that direction and included in appendix G.

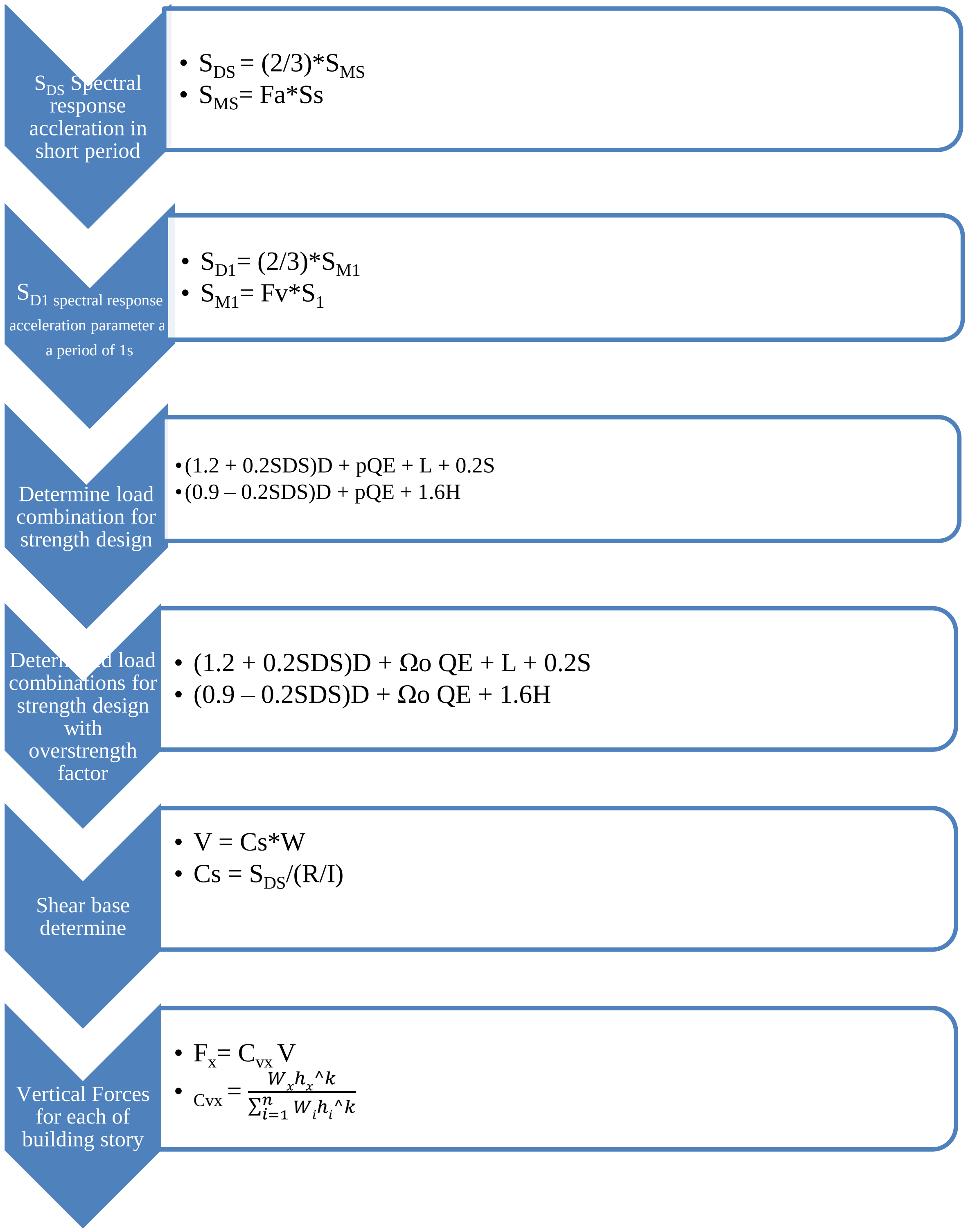
6.6 Seismic Load Design

6.6.1 Seismic Load Criteria

The Gateway building is located in Worcester, MA, where there has been very little or no earthquake effects in history. But it is necessary to check the seismic loading effect for buildings due to its fairly high potential for failure. The design of seismic loads followed the guide of Chapter 12 of *ASCE 7-05: Seismic design requirements for building structures; and Seismic Design chapter in Design of Concrete Structures* (Nilson, 2005). To determine the seismic response of the Gateway building, several parameters were determined. *ASCE 7-05* showed the expected peak acceleration of a single degree-of-freedom system with 0.2 second spectral response acceleration, S_s (short period), along with a 1.0 sec spectral response acceleration S_1. Those two values were based on records from history. There are others factors that contribute to seismic load design process such as soil properties of the site, site classification and occupancy type. The site coefficients F_a and F_v were determined from Table 11.4-1 and 11.4-2 based on the maximum earthquake spectral response acceleration and site class. Table 28 displays all parameters that were considered during the calculation process.

Table 28: Seismic load design parameters

Occupancy Category	III
Site Class	B
S_s	0.25
S_1	0.06
F_a	1.0
F_v	1.0
ρ redundancy factor	1.0
Ω_0 overstrength factor	3.0
R response modification factor	3.5
I occupancy factor	1.25
T_L long period transition	6
C_t building period coefficient	0.028
x building period coefficient	0.8



First step was to calculate the adjusted maximum earth spectral response acceleration S_{MS} and $S_{M1.}$ Then the five percent damped design spectral response acceleration at short period $S_{DS,}$ and at one second period S_{D1} were determined. The governing load combination load was determined for *RISA* 2.0 analysis. Then, the seismic base shear, ‘V’ was calculated based on Cs-seismic response coefficient, and ‘W’, the effective seismic weight of the structure, which combined all the dead loads and partition weights that were applied on the structure. For the Gateway building, three bottom stories had the same weight, while the one on the roof was lighter. The Cs value was checked against its maximum allowable value and the minimum value to single out appropriate value.

Next step was determining the distribution of seismic forces by using the equation below.

$$\text{Cvx} = \frac{Wxhx\hat{}k}{\sum_{i=1}^{n} Wihi\hat{}k}$$

Where F_x is the lateral force at the level x, h_i and h_x are the height from ground to level x, W_i and W_x is the portion of weight assigned to level x.

6.6.2 Design results:

Calculation is shown in Appendix G.2. Below are the lateral loads applied on each story of the building under seismic effects

Table 29: Seismic forces

Level	Later Force (k)
1	0.795
2	0.795
3	0.795
4	0.642

After obtaining the forces, *RISA* 2.0 was used and the same process in wind force design section to analyze the seismic load effect. The below table shows the analyzed results:

Table 30: Seismic member force values

P_{nt} (k)	191.66
M_{nt} (k-ft)	265.23
P_{lt} (k)	2.37
M_{lt} (k-ft)	18.35

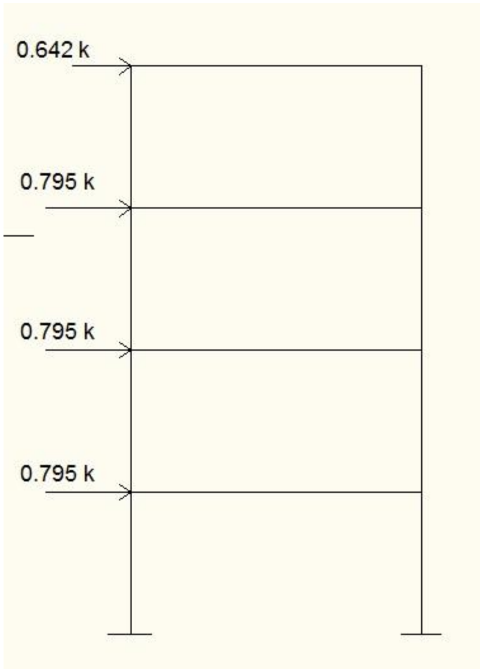


Figure 28: Seismic Loads on Frame

The sway limit of the structure was calculated, Table 31 below shows the results:

Table 31: Sway of the building

Level	X-deflection (in)	Height (ft)	Max Sway=H/360
1	0.009	14’8”	0.35
2	0.033	29’4”	0.7
3	0.064	44’	1.0
4	0.082	59’	1.4

The results satisfied the sway limit of the frame. Hence, the designed frame in section 6.4.1 wind force is still acceptable.

Table 32: Frame design for seismic loads

	Level 1	Level 2	Level 3	Level 4
Columns	W14x159	W14x159	W14x159	W14x159
Girders	W16x77	W16x77	W16x77	W16x77

6.7 *Robot* and *RISA* Comparison

Modern technologies are commonly used to speed up the design process. This project has also incorporated the use of multiple computer programs during the design process. With the help of computer programs, structural analysis can be done in just minutes. In the previous section that discussed the design of the lateral frame design, the computer program *RISA* was used to determine member forces due to gravity and lateral loads. Based on the results obtained from the program, values for Plt, Mlt, Pnt and Mnt were calculated using the AISC load combination equation to produce the governing design values. These values were used to determine the appropriate steel section sizes that would efficiently support the building from lateral and gravity load effects.

The design solutions may vary depending on the different computer programs. For purposes of comparison, two different computer programs, *RISA* and *Robot*, were used to obtain Plt, Mlt, Pnt and Mnt values for the various frames. The solutions were compared using the same design criterion for three simple frames with different heights. Once the comparison for the Plt, Mlt, Pnt, and Mnt values of the three frames was completed, a lateral frame designed in the previous section was modeled in *Robot*. The data obtained was used to determine the required steel section size. Finally, the adequacy of column was checked using the *Robot* Steel Design tab.

6.7.1 Methodology

To better understand the programs**,** three simple frames with different heights were modeled in order to compare the two structural analysis programs *RISA* and *Robot*. LFRD load combinations were not applied in these analyses because this test was only used for the purpose of comparing the two computer programs. Information input to structural analysis programs can be tricky because the different programs may have different ways of interpreting data. Therefore, it was important to first compare the reaction forces for each frame: the reactions should always be the same because they must satisfy the equilibrium conditions. Both programs were set up for these condition states: excluding the p-delta analysis, using same member sizes and applying the same loads. Once the values of $P_{lt,}$ $P_{nt,}$ $M_{l,}$ and M_{nt} were obtained from both programs, comparisons can be made based on the differences in axial forces, and moments. Percentage differences were calculated to distinguish the two programs. The percent of difference is the ratio of differences in value, and then larger value multiplied by 100%. *RISA*'s output was used as the base case for determining the percent of difference since *RISA* was already used in the past.

Furthermore, for the purpose of investigating the difference between the two structural analysis programs, AISC load combination equation was not evaluated.

Frame 1

The first simple frame compared was a one-story plane frame. The geometry is shown below in Figure 29. The columns are pin connected at the end, and the girders are connected with full moment connections. The bay is 30 feet long and 15 feet high. There is a snow load of 1.5 k/ft., and dead load of 0.5 k/ft. distributed along the length of the girder (member 3 in the model). A wind load of 5 kips was applied at the level of the girder (node 2 in the model).

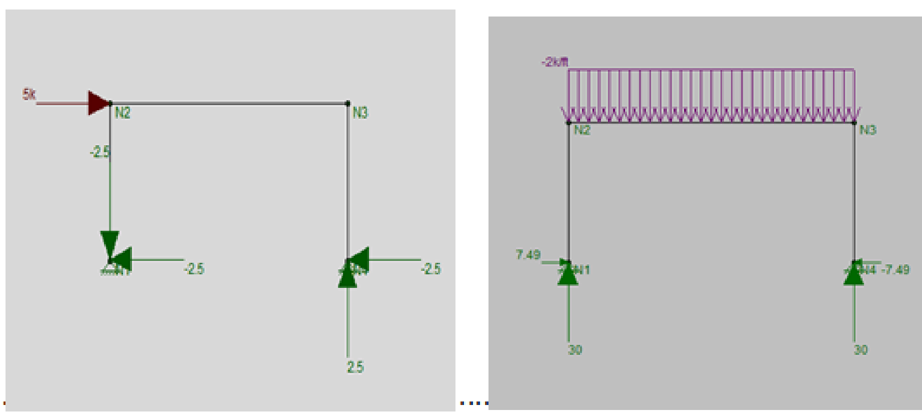


Figure 29: Geometry for one-story frame

Frame 2

A four-story plane frame was the second frame to be analyzed. The geometry is shown in Figure 30.The columns are pin connected at the end. The girders were connected with full moment connections. The bay is 30 feet long, 60 feet high, and each story is 15 feet. A gravity load of 2 k/ft is distributed over the floor beams, and 1.85 k/ft is applied on the roof beam. There was also 3 kips of wind load acting on the roof and 5 kips wind load acting on other floors.

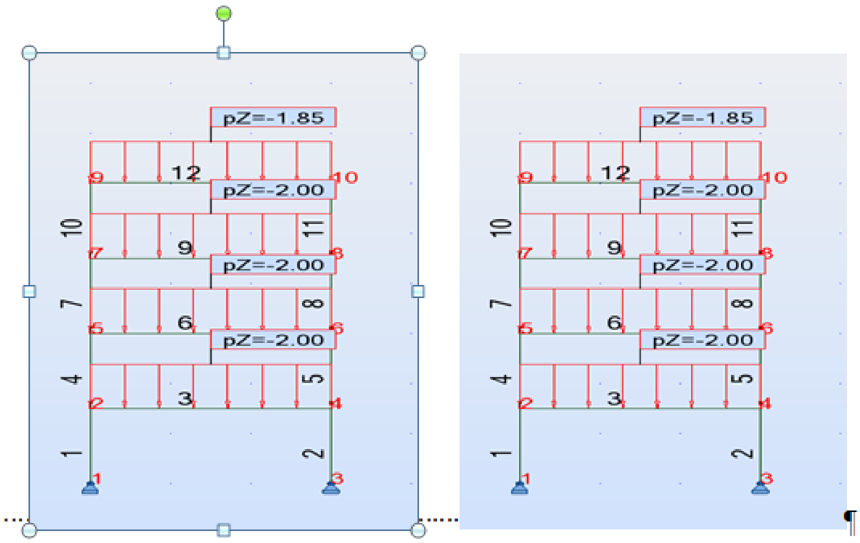


Figure 30: Geometry for a four-story frame

Frame 3

A ten-story plane frame was the third frame to be analyzed. The geometry is shown below in Figure 31. The columns are pin-connected at the end. The girders were connected with full moment connections. The bay is 30 feet long, 150 feet high, and each story is 15 feet. A gravity load of 2 k/ft was distributed over the floor beams, and 1.85 k/ft is applied on the roof beam. There was also 3 kip of wind load acting on the roof and 5 kip of wind load acting on other floors.

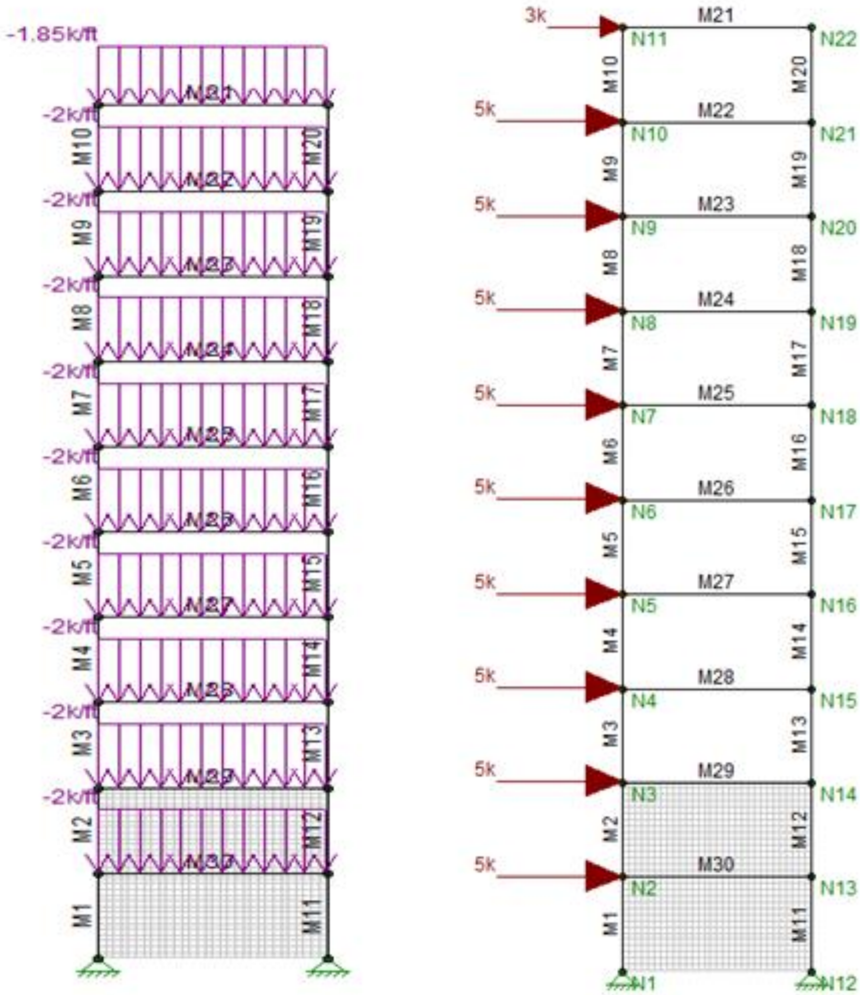


Figure 31: Geometry for a ten-story frame

Frame 4

Sufficient knowledge was developed after modeling and analyzing the three frames in *RISA* and *Robot*. This time, a four-story plane frame with appropriate beam szie was identified and modeled in the program for analysis. After comparing the result from the two programs, this lateral frame design was tested in *Robot* for adequency. After comparing the result from the two programs, this lateral frame was modeled in *Robot* only since frame 4 was already modeled in section 6.5.

This is the lateral frame developed in the previous section. The member size were determined and identify in Figure 32. The columns are fix connected at the end, and the girders are connected to the columns with full moment connections. The bay is 30 feet long, 59 feet high, and each story is 14’-8”. 2.1 k/ft of dead load and 0.98 k/ft of live load were distributed along the spans of the floor girders. 1.6 k/ft of dead loads and 0.68 k/ft of snow load was applied along the length of the roof girder. There were also 3 kips of wind load acting on the roof and 5 kips of wind load acting at the levels of the other floors.

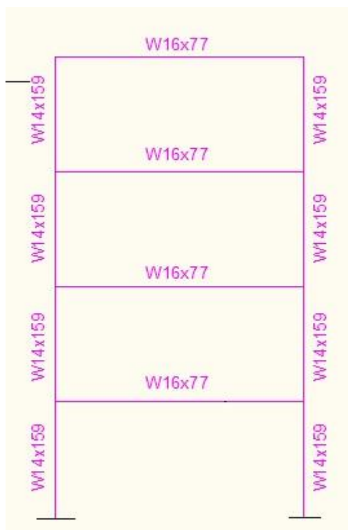


Figure 32: Geometry of a four-story lateral frame

6.7.2 Results

6.7.2.1 Program Analysis Results for the Three Simple Frames

The values for Plt, Mlt, Pnt, and Mnt were established by using both *RISA* and *ROBOT*. Data from the programs were saved as in a spreadsheet and included in the tables below. Based on the results, the percentage differences for the data output from both programs were also determined.

Table 33 presents the data obtained for the one-story frame. Basically the two programs returned the same results, with the exception of a 0.11% difference for factored moment Mnt due to gravity load. Such small differences should not result in any changes in the determination of acceptable columns sizes.

Table 33: Comparison table for a one-story frame

One-Story Frame	*RISA*	*ROBOT*	% of Difference
Plt (Lateral)	2.5	2.5	0.000
Mlt	37.507	37.51	-0.008
Pnt (gravity)	30	30	0.000
Mnt	112.347	112.47	-0.109

Table 34 summarizes the data obtained for the four-story frame. Once again, there was not much difference for values of Plt, Mlt and Pnt, but the variation for Mnt was quite significant for the first three stories. The fourth-story had less than a 1% difference. Such variation required further investigation of the results for the first three stories. The result for a ten-story frame will be used for comparison.

Table 34: Comparison table for a four-story frame

First-Story	*RISA*	*Robot*	% of Difference
Plt (Lateral)	21	21	0.00
Mlt	135.018	135.02	0.00
Pnt (Gravity)	117.75	117.75	0.00
Mnt	0.004	48.06	-1201400.00
Second-Story			
Plt (Lateral)	10.34	10.33	0.10
Mlt	72.601	72.51	0.13
Pnt (Gravity)	87.75	87.75	0.00
Mnt	0.007	76.32	-1090185.71
Third-Story			
Plt (Lateral)	4.438	4.43	0.18
Mlt	44.074	44.01	0.15
Pnt (Gravity)	57.75	57.75	0.00
Mnt	99.664	57.81	42.00
Fourth Story			
Plt (Lateral)	1.339	1.34	-0.07
Mlt	20.081	20.04	0.20
Pnt (Gravity)	27.75	27.75	0.00
Mnt	113.887	114.42	-0.47

Table 35 presents the data obtained for the ten-story frame. The results for stories four through seven are excluded from the table because much of the information is the same as for the first two stories. From the table, the percentage differences for Plt, Mlt, and Pnt of a ten-story frame were relatively minimal, except that the difference in Mnt of the first nine stories was great. One more observation is that the differences in Mlt are relatively high for stories one through story eight.

Table 35: Comparison table for a ten-story frame

First Story	*RISA*	*ROBOT*	% of Difference
Plt (Lateral)	127.5	127.5	0.00
Mlt	719.667	360.02	49.97
Pnt (Gravity)	297.75	297.75	0.00
Mnt	0.004	48.55	-1213650.00
Second-Story			
Plt (Lateral)	98.821	95.89	2.97
Mlt	504.588	208.36	58.71
Pnt (Gravity)	267.75	267.75	0.00
Mnt	0.009	73.97	-821788.89
Third-Story			
Plt (Lateral)	79.258	73.8	6.89
Mlt	487.708	161.96	66.79
Pnt (Gravity)	237.75	237.75	0.00
Mnt	0.009	8.91	-98900.00
Eighth-Story			
Plt (Lateral)	9.576	9.57	0.06
Mlt	61.05	61.02	0.05
Pnt (Gravity)	87.75	87.75	0.00
Mnt	0.007	71.61	-1022900.00
Ninth-Story			
Plt (Lateral)	4.278	4.27	0.19
Mlt	41.695	41.61	0.20
Pnt (Gravity)	57.75	57.75	0.00
Mnt	99.664	58.85	40.95

Tenth-Story			
Plt (Lateral)	1.307	1.3	0.54
Mlt	19.599	19.56	0.20
Pnt (Gravity)	27.75	27.75	0.00
Mnt	113.887	114.32	-0.38

After the analysis of of the results obtained from *RISA* and *ROBOT* for the three frames, some trends were observed. Both programs seem to produce relatively close solution for a one-story frame. However, as the frames exceed one story in height, Mnt became the most concerning factor. Unlike Plt, Mlt, and Pnt, Mnt, which is the moment associated with the no-sway case, was the one result with the most variation in all cases. The variation of Mnt was especially unreasonable in tall frames. Some trends are shown for Mnt that resulted in significant variation. For both the four story and ten story frames, the Mnt values at the top level showed the best agreement, and there were large variation for all of the lower floors. The shear forces for the four-story and ten-story frames were checked to explain such variation in Mnt. In *RISA*, the shear force for the column located on the first story is also zero. However, the shear force for the same column is 3.20 kip in *Robot*. Thus, the cause of the huge variation in Mnt *ROBOT* is because the program included 3.20 kip of force for tall frames automatically. Also, as mentioned above, the Mlt value for the ten-story frame tended to vary. This is because *RISA* Demo is essentially the full *RISA* program with certain limitation such as the ability to store a large number of nodes and members is not available. However, if the model is entered completely, then *RISA*-2d Demo should be able to handle the model. For the 10-story frame, *RISA* would function correctly only if appropriate member sizes are fully entered

6.7.2.2 Program Analysis Results for designed Four-Story lateral frame

After developing sufficient knowledge about *RISA* and *Robot*, the lateral frame designed in the previous section was modeled in *Robot* only, since the *RISA* solution was used in section xx for the lateral frame design process. Second-order moment analysis can be done using *ROBOT* because the appropriate member sizes were already identified for this frame.

From the axial force and moment diagrams developed for this frame, columns located on the first story seemed to resist most of the load. Thus, the comparison only considered columns in the first floors. This time, LRFD load factor equation was applied in all cases. The load equation that resulted in the greatest load combination was 1.2DL+1.6WL+0.5LL+0.5SL. Table 36 presents the solutions for factored force and moment, and the percentage differences were also obtained.

Table 36: Comparison table for designed four story lateral frame

First Story Column	*Robot*	*RISA*	% of Difference
Plt	40.51	40.26	0.600
Mlt	333.73	336.04	-0.693
Pnt	169.35	169.35	0.000
Mnt	91.21	91.05	0.175

For this four-story frame, the percent of variation for Mnt is only 0.175%, as compared to the -1201400.00% indicated in Table 34. The same members were compared from Frame 2. The only difference was that the appropriate member size was identified for this frame, but not for

Frame 2. For Frame 2, the member sizes were set to a standard W8x10. By comparing the results for Frame 2 and Frame 4, a conclusion was made that identified the appropriate~~d~~ member size that would help to lower the variation in output between the two programs.

The designed frame was tested for second-order moment effect using *Robot*. The program was set to use LRFD as the standard design method. Figure 33 shows the Member Verification table for all of the columns after the second-order moment effect was included. All the column sizes were checked for the adequacy in all the load cases. Figure 34 shows an example of the second-order moment calculations. A printed copy of the calculation sheet for column 1 is also included in the Appendix.

ANSI/AISC 360-05 - Member Verification (SLS ; ULS) 1to10 12 13

Results | Messages

Member		Section	Material	Lay	Laz	Ratio	Case	Ratio(vx	Case (vx)	Ratio(vy	Case (vy)
13 Column_13	OK	W 16x77	STEEL	51.37	145.69	0.52	1 DL1	0.00	5 WL	0.00	1 DL1
12 Column_12	OK	W 16x77	STEEL	51.37	145.69	0.53	1 DL1	0.00	5 WL	0.00	1 DL1
10 Column_10	OK	W 16x77	STEEL	51.37	145.69	0.53	1 DL1	0.00	5 WL	0.00	1 DL1
9 Column_9	OK	W 16x77	STEEL	51.37	145.69	0.40	1 DL1	0.00	1 DL1	0.00	1 DL1
8 Column_8	OK	W 14x159	STEEL	28.22	44.98	0.15	1 DL1	0.22	5 WL	0.00	1 DL1
7 Column_7	OK	W 14x159	STEEL	27.60	43.99	0.11	1 DL1	0.40	5 WL	0.00	1 DL1
6 Column_6	OK	W 14x159	STEEL	27.58	43.96	0.14	1 DL1	0.53	5 WL	0.00	1 DL1
5 Column_5	OK	W 14x159	STEEL	27.60	43.99	0.28	5 WL	0.35	5 WL	0.00	1 DL1
4 Column_4	OK	W 14x159	STEEL	28.22	44.98	0.15	1 DL1	0.22	5 WL	0.00	1 DL1
3 Column_3	OK	W 14x159	STEEL	27.60	43.99	0.11	1 DL1	0.40	5 WL	0.00	1 DL1
2 Column_2	OK	W 14x159	STEEL	27.58	43.96	0.14	1 DL1	0.53	5 WL	0.00	1 DL1
1 Column_1	OK	W 14x159	STEEL	27.60	43.99	0.28	5 WL	0.35	5 WL	0.00	1 DL1

Figure 33: Column verification table output from *Robot*

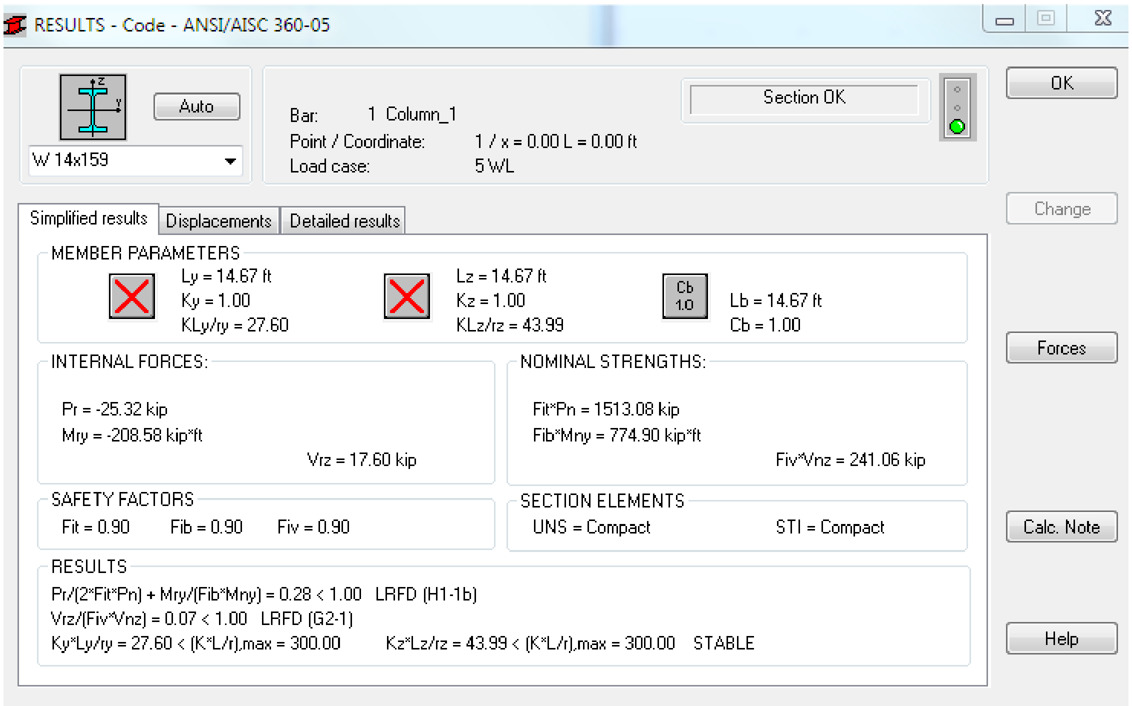


Figure 34: Second-order moment calculated solution sheets for column 1 from *Robot*

Displacement of the frame is an imporant factor in determining the adquacy of the frame. As shown in the figure 35, the maximum lateral dislacement is 1.7708. The total building height is 59 feet, thus the maximum displacement for building is 1.96 inches. The displacement outputed from *Robot* meets the H./360 limit.

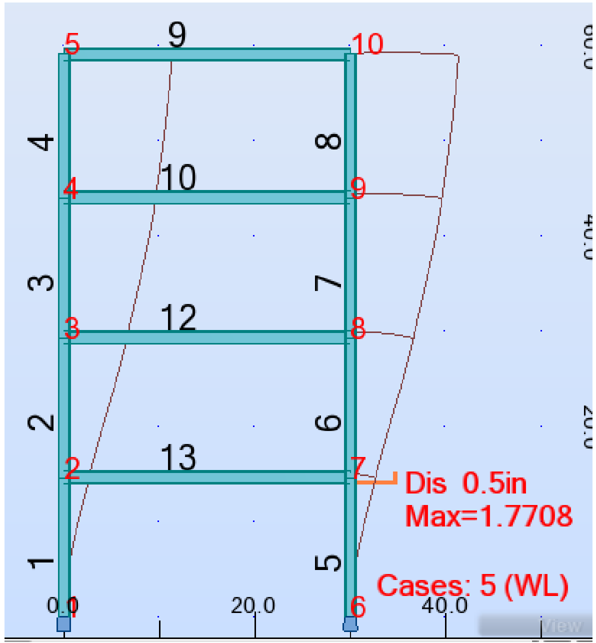


Figure 35: Displacement graph for the lateral frame from *Robot*

6.7.2.3 Conclusion

Both *Robot and RISA* are valuable tools that will speed up the design process. From a given set of input data, both programs can calculate and produce the moment and axial force diagrams for all the members in just minutes. Also, they each have the capability to check the adequacy of all members including second-order moment effects. The solutions that are produced from the two programs would not vary very much if all of the design criterion were set consistently. Setting the design was tricky when using the different programs. As discovered. *Robot* automatically included self-weight when calculating the reaction force for the frames. In addition, *Robot* used the LRFD design method for checking the second-order moment for lateral column. The same method was used in the design of the lateral frame design presented in section xx. Thus, both the hand calculation and the computer analysis proved that W14x159 is an adequate size for the columns in the rigid frame.

Chapter 7: Complete Structural Design

The final structural design recommendation was selected based on the systematic evaluation process presented in chapter 5. However, some of the complicated areas of the structure, such as the Slab-on-ground, the trapezoidal section of the building's floor plan, the elevator shafts, and the stairways were not considered in the earlier evaluation. For this chapter, designs for all of these more complicated areas were considered and included for the final recommendation. The most complicated lateral frame was already completed in chapter 6. The developed design was then included in the determination of final cost in this chapter.

7.1 Slab-on-ground

The purpose of this section is to illustrate a way of selecting appropriate slab thickness for slabs-on-ground design for the floor on the first level. Slabs-on-ground are designed to support applied loads bearing from ground. Since the project was mainly focused on the steel framing design, only the required slab thickness was determined, and not a completed slabs-on-ground design. Slab thickness selection is a major step in the process of completing the design of a slab-on-ground. By knowing the slab thickness, a cost estimate would be obtained relevant to the total amount of concrete needed in the construction of a slab-on-ground.

There are some requirements to follow when selecting the thickness for slabs-on-ground. The requirements include (ACI Manual Of Concrete Practice Part Four., 2010): At least 1/3 the toe footing height to ensure ample resistance against wall sliding; and a smaller amount of contraction control joints is used for thicker slab; and slab must be thick enough to accommodate the reinforcement. For this project, loads are assumed to be uniform for all the sections of the slab. The slope sections and deep end sections are also assumed to have same elevation. (ACI Manual Of Concrete Practice Part Four., 2010)

Construction cost estimates for slabs-on-ground are based on the total amount of concrete needed for construction. The total area of slabs-on-ground is equivalent to the total building area less the total footing area, and then multiplied by the slab thickness. The slab cost will be included in the final cost once the final building layout is completed. Because slab thickness selection method do not included the determination of required numbers of rebar, the cost of rebar would not be considered for the final cost estimate.

7.1.1 Methodology

As introduced in Chapter 2 Background, PCA, WRI and COE are the three different methods for slab thickness selections. Among the three methods, the provisions of the Wire Reinforcing Institute (WRI) presented in the ACI Manual were chosen and followed. The procedure for selecting an appropriate thickness for aisle moment due to uniform loading is outlined in the table below. (ACI Manual Of Concrete Practice Part Four., 2010)

1. Identify soil profile using geotechnical report prepare by Maguire's Group.
2. Obtain Load-bearing values from *IBC 2009 table 1806.2 Presumptive Load-Bearing Values*
3. Classify soil group using *ACI 360R-8 Table 3.1 Unified Soil Classification System (Winterkorn and Fang 1975)*
4. Determine the modulus of subgrade using *ACI 360R-9 figure 3.3- approximate interrelationships of soil classification and bearing values (Portland Cement Associations 1988.)*
5. Select the trial slab thickness value.
6. Check the tensile stress in the top of the concrete slab due to uniform loading using *ACI 360R-65 Figure A2.1 and A2.4.*

7. Use *Figure A2.1Subgrade and slab stiffness relationship with WRI Design* to determine the D/K ratio with known values: trial slab thickness and modulus of subgrade.
8. Use *Figure A2.4-Uniform load design and slab tensile stress chart with WRI design procedure* presents in the figure below to find the required slab thickness. This step required plotting up from aisle width to D/k, then to the right-hand plot edge, and then down through the uniform load value to the left hand edge of the next plot, then horizontally to the allowable stress and down to the design thickness.

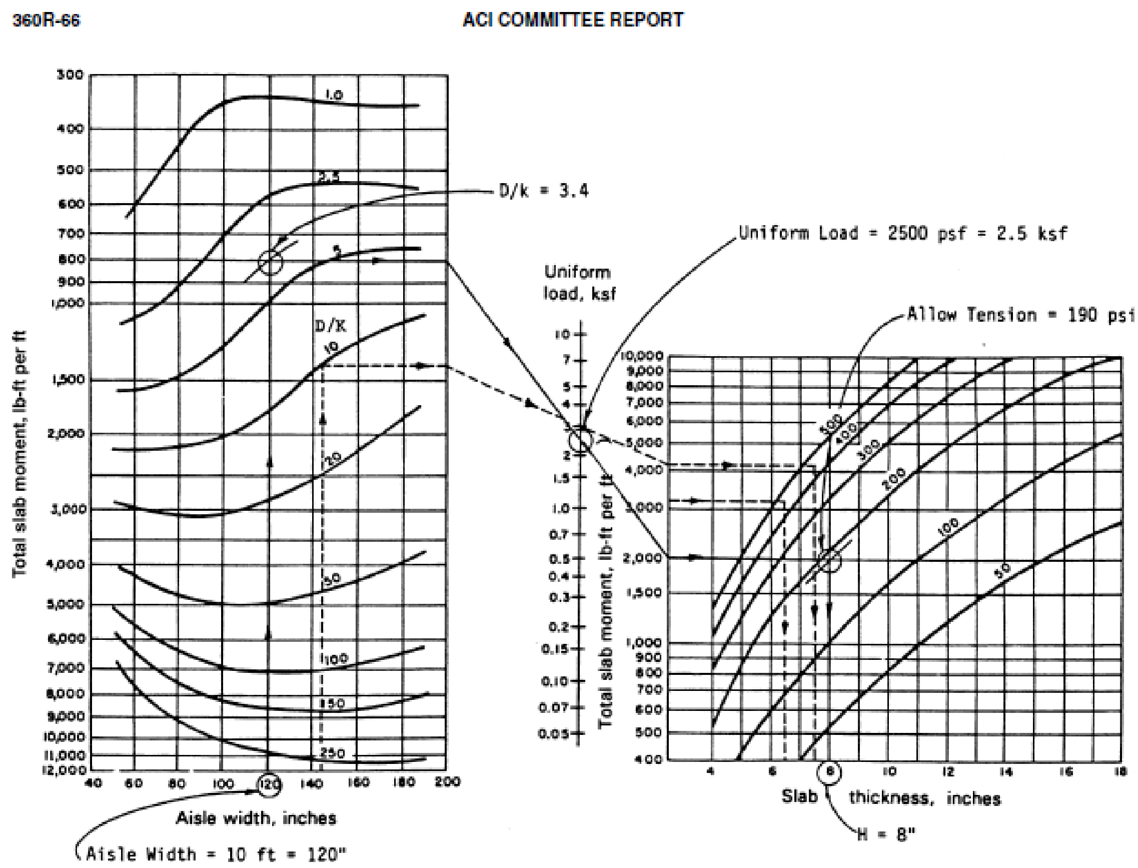


Fig. A2.4—Uniform load design and slab tensile stress charts used with WRI design procedure.

Figure 36: WRI design procedure chart (ACI Manual Of Concrete Practice Part Four., 2010)

9. Lastly, repeat steps 1-7 if the resulting design thickness is different from the assumed value.

7.1.2 Results and Conclusion

As introduced in the methodology, WRI slab thickness selection start with identified soil properties using a geotechnical report. In the geotechnical prepared by *Maguire Group Inc*, soils near the project site are specifically described as:

Fine to Coarse sand, trace to little Silt, trace to little gravel, trace cobbles, trace construction material fragment- brick, concrete, wood, and asphalt, ranging in Unified Soil Classification System (USCS) group symbols between SM, SP, and SW. (WBDC Gateway Project Proposed Parking Garage and Associated Facilities, 2005)

Following the procedures introduced in the methodology, the required variables for determining the slab-on-ground thickness were summarized in table below.

Table 37: Variables for determining the slab-on-ground thickness

Variables	Values
Load Bearing Value (*IBC table 1896.2)*	2000 pcf
Modulus of Subgrade (*ACI 360R-9 figure 3.3)*	400 pcf
D/K Ratio	$3.4x10^5$

A trial thickness of 8 inches was selected based on the D/k value of $3.4*10^5$ and modulus of subgrade. This trial thickness was checked by using step 8 in the procedure. Because the final design thickness was not much difference from the assumed value, 8 inches slab thickness was considered as an appropriate value. Occupancy of the level was not fully clarified in the architectural layout, the floor was assumed to be used as a laboratory for Fire Protection Engineering department. The Slab-on-ground would be pore over the entire floor area since the area bearing the most of the load was undefined.

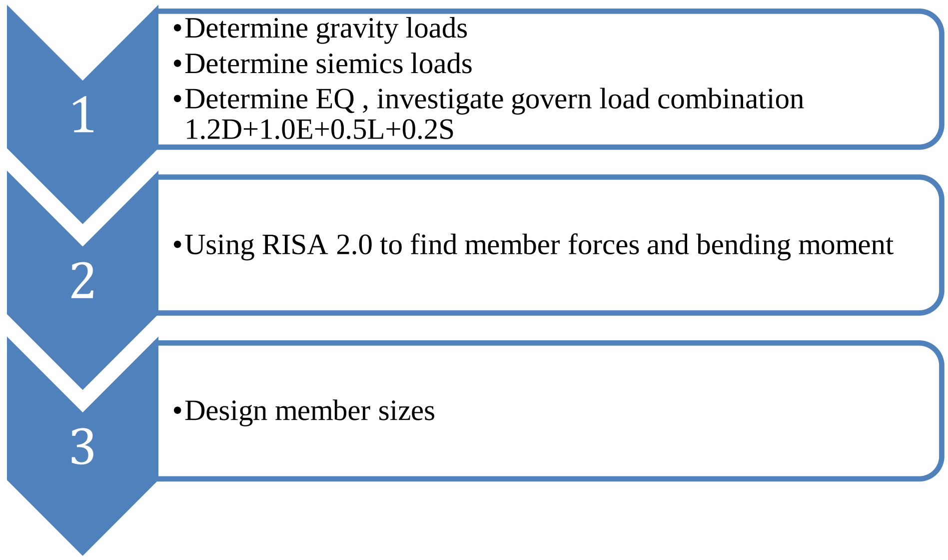
7.2 Elevator Shaft Design

In office buildings, elevators are one of the most essential transport systems for moving people and large equipment between floors. A hydraulic elevator with capacity 2500 lbs was used in this design which is suitable for small office building (*Building Design and Construction Handbook*, 2000). Also, a hydraulic elevator has an advantage that it easily multiplies the pump to lift the elevator cap, therefore reduces the effect on building's structure. The elevator is operated by the hydraulics pump therefore no significant forces impact on the structure of the building. To design the elevator shaft frame, loads are considered such as cap weight, counter weight, and the capacity of the elevator. All the loads were factored for impact which was assumed as dynamics loads resulting from the moving motion.

A moment resisting frame was designed for the elevator shaft. The eccentricities of the structural members cause additional moments in addition to the moment caused by gravity loads. The axial compression forces were also considered because they increased the lateral deflections of the frame through P-Δ effects which also lead to additional moments. A first-order analysis was performed on the members subjected to only bending and axial loads. Secondary moments increased when the frame was subjected to sidesway. It was addressed by calculating Euler buckling factor Pe1 for no translation and Pe2 for lateral translation, and moment capacity Mr. The flexural strength of the member must be equal or greater than the combined results of both first-order and second-order moments, which were considered by using interaction equation H1-1a and H1-1b in AISC for combined axial compression and flexural effects.

Below is the design protocol for elevator shaft:

The moment resisting frame was designed to withstand both gravity and lateral loads. For the compression members, the compressive and flexural forces from gravity loads along with the axial and flexural forces from the lateral loads were analyzed. The steps to perform the first-order analysis are illustrated below.



In this design, the seismic loads were the governing load. The results obtained were axial forces and bending moments on columns with gravity loads and lateral loads treated separately. These member forces were used for the initial design of adequate members.

Following the results from the first-order analysis, it was necessary to perform the second- order analysis to address P-Δ effects. The chart below presents the process for the second-order analysis.

Check LTB

- Column bending capacity: if Lp<Lb
- $M_n = \min [C_b(M_p - (M_p - M_r)*(L_b - L_p)/(L_r - L_p)), M_p]$

K factor

- Effective length factor K=1.0

Euler buckling strength

- No translation AISC eq. C2-5: $P_{e1} = 2EI/(K1L)^2$
- Lateral translation AISC eq. C2-6b: $P_{e2} = R_M(\sum HL/\Delta H)$
- $R_M = 0.85$

Amplification factor

- Find $C_m = 0.6-0.4(M1/M2)$ using AISC eq C2-4
- AISC C2-2 : $B1=Cm/(1-\alpha Pr/Pe1)$
- AISC C3-2 : $B_2 = \frac{1}{1-\frac{\sum P_{nt}}{\sum Pe_2}} \geq 1$

Check WLB &FLB

- WLB: $\lambda < \lambda p = \sqrt{(E/Fy)}$
- FLB: bf/2tf<90.5

Find Mr & Pr

- AISC C2-1a: $M_r = B_1 M_{nt} + B_e M_{lt}$
- AISC C2-1b: $P_r = P_{nt} + B2P_{lt}$

Check Pr/Pc

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

The second-order analysis includes the following factors:

- The factored axial forces P_{nt} from no-sway analysis
- The factored axial forces P_{lt} from sway analysis
- The factored moment M_{nt} from no-sway analysis
- The factored moment M_{lt} from sway analysis

The Euler buckling magnification factors: P_{e1} with no translation and P_{e2} with translation were calculated. K=1 was used for P_{e1} due to no translation gravity loading. P_{e2} was determined by using $\sum H$ (total lateral story shear on that level) and ΔH (first-order interstory drift due to lateral force).

The magnification factors B1, and B2, along with the Cm parameter were determined, which were used to check the adequacy of the section for combined axial compression Pr and bending moment Mr.

Cm = 0.6 – 0.4(M1/M2)

M1 /M2 is the ratio of the smaller moment to the larger moment at the ends of the unbraced length in the plane.

Next, web local buckling and flange local buckling as a part of establishing the flexural capacity were checked before move on to using interaction equation H1-1a and H1-1b in the *AISC Specification*.

Pr- axial load , Pc- the strength of the section, along with $L_{p,}$ $L_{r,}$, L_b and flexural strength of the section in Table 3-2 AISC were determined. The last step was to substitute to the values for Pr, Pc, Mr, and Mc into one of the interactive equations below to investigate whether the member was adequate:

Pr/Pc+8/9(Mrx/Mcx+ Mry/Mcy)<1.0 AISC equation H1-1a

Pr/2Pc+(Mrx/Mcx+ Mry/Mcy)<1.0 AISC equation H1-1b

Table 38 below shows the designed members for elevator shaft including girders and columns size. The columns involved bending about one axis. Figure 36 show the elevator shaft layout within the building's structure.

Table 38: Elevator shaft designed members result

Girders	W10x19
Columns	W12x35

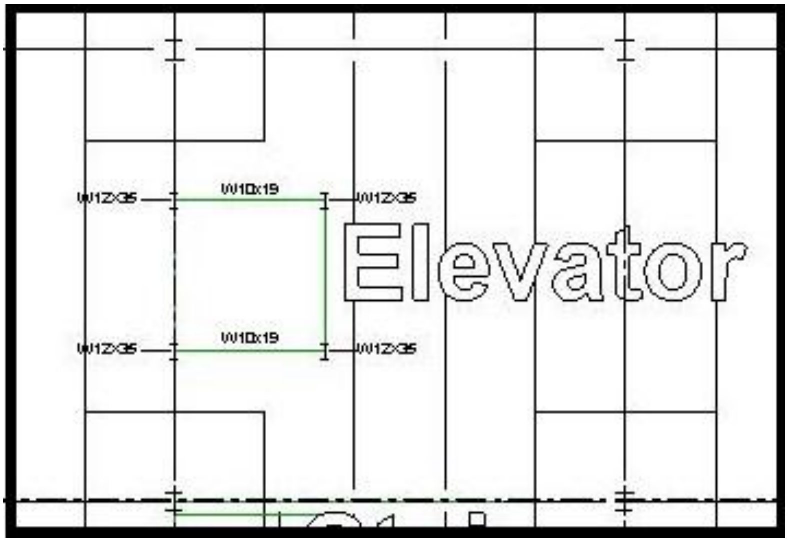


Figure 37: Elevator

7.3 Stair Design

From the architectural layout of the building, there was a typical two-flight staircase located on each side of the new Gateway building. The stairs were spanning from the first floor to the fourth floor with the same spanning length on each of the floor. Figure below shows the architectural plan of the staircases.

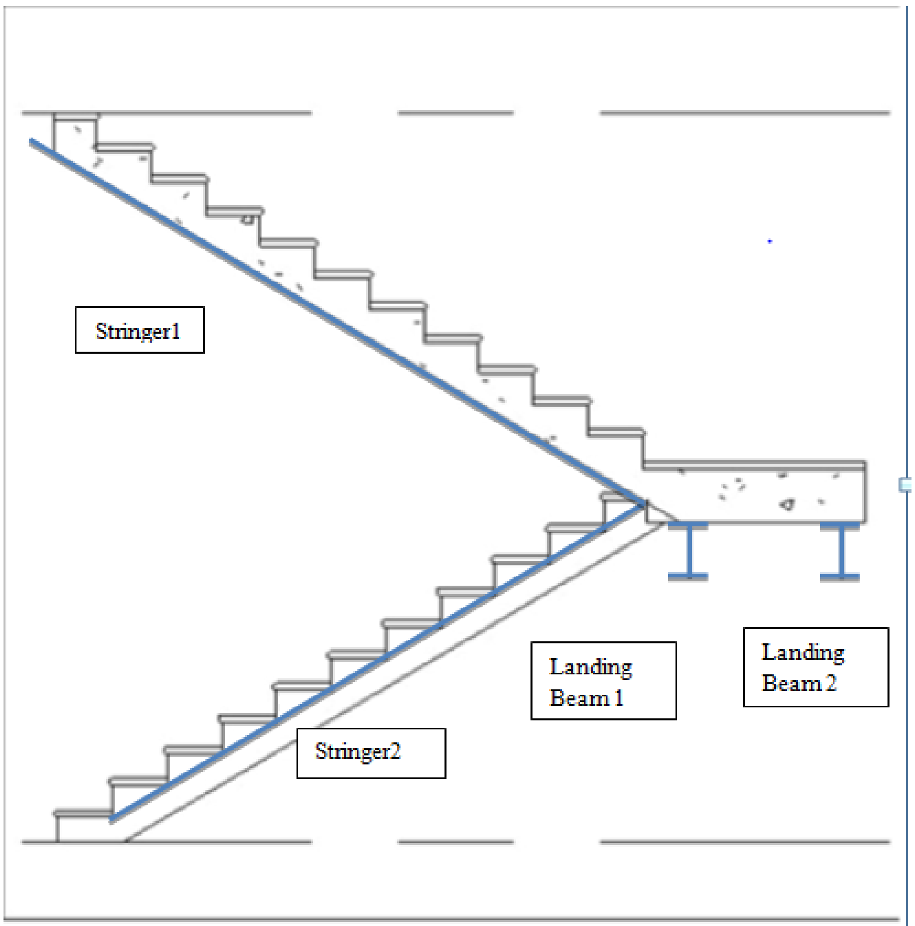


Figure 38: Elevated view of the stair

(http://forums.autodesk.com/autodesk/attachments/autodesk/133/56310/3/Stairs%20Revit.jpg)

In this section, sizes of the beam and column framing that was used to support the stair were calculated. The beams were designed to hold dead loads, including the weight of the concrete slab and metal decking, and live loads. To increase flexibility in the use of building space, the rest of the building was built using the capacity requirement of 100psf. Thus, the design load for this staircase was identified as 63psf for dead load and 100psf for live loads from *ASCE table 4-1*. There was no difference between calculating the member size for staircases and structural floor. However, when calculating the member sizes for the stringers, the stringers were installed an angle to the nearest 34 degrees, the angle was determined by taking the cotangent of stair height over stair width. Figure 39 shows the determined member sizes for the stair. The design also included two W10x12 columns.

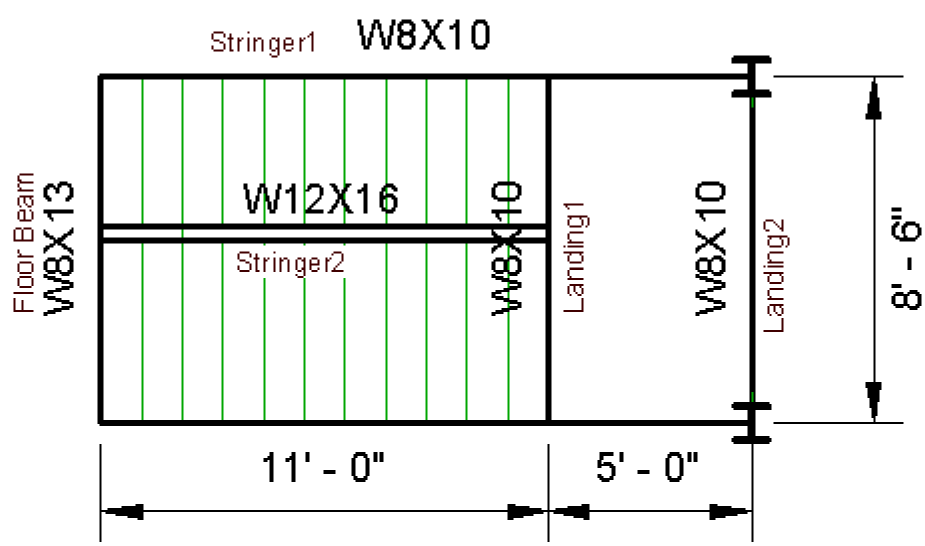


Figure 39: Structural plan view of stair

7.4 Trapezoid Area

The design of the trapezoidal area located the on the east side of the building was considered separately due its irregular shape. The floor frame layout for the trapezoid is presented in Figure 39. For the trapezoidal area, in bay 30x35 and bay 30x40, the beams at locations 1, 2, 3, and 4 were spaced 6 feet O.C. The beam at location 5 has a different tributary width, which extends 3 feet to the left and covers the rest of the bay area on the right side. The size of the beams located at point 5 resulted in larger sizes than for locations 1, 2, 3, and 4 as a result of the expanded tributary width. For all floors, this consistent framing was used in the trapezoidal area.

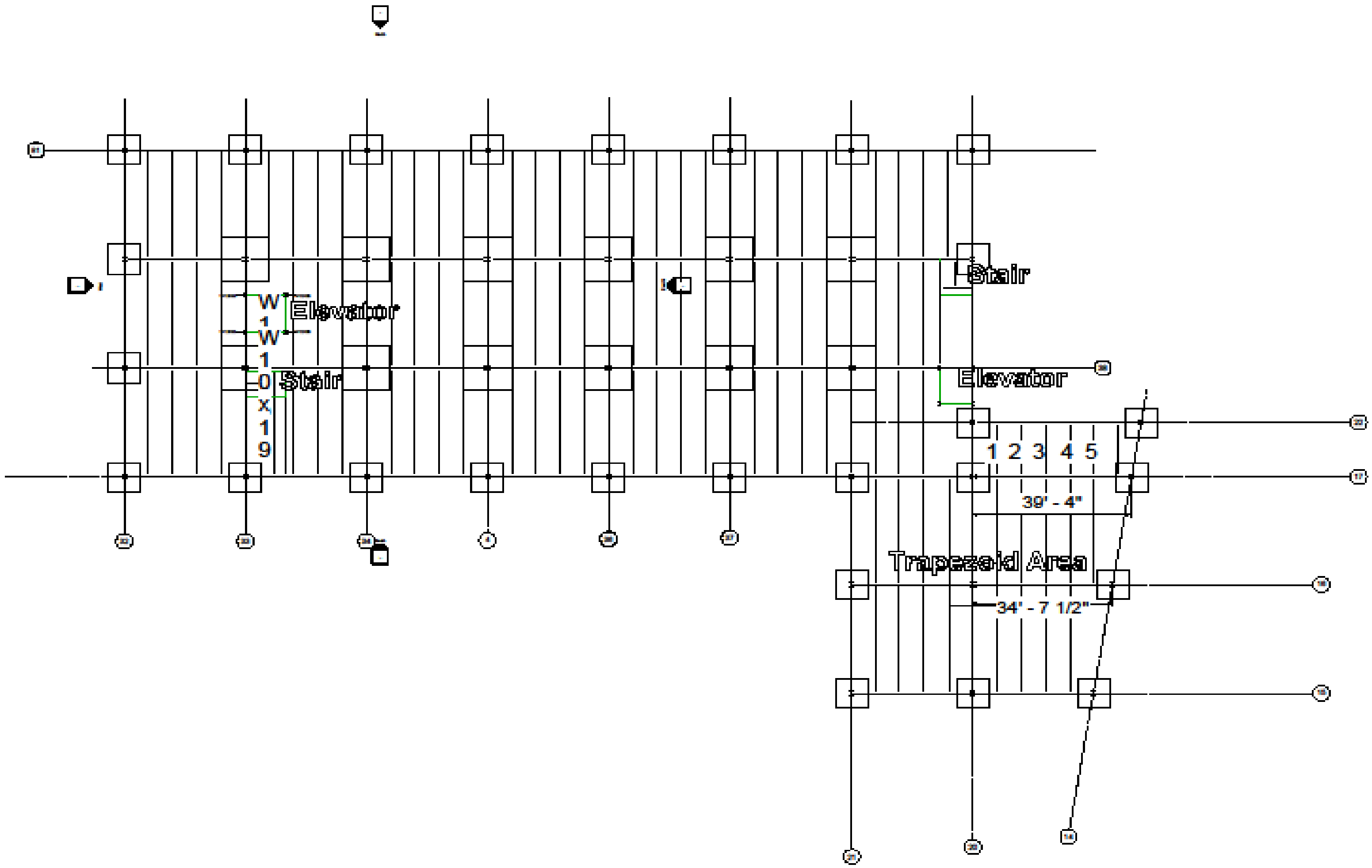


Figure 40: Structural layout for the trapezoid area

7.5 Final Cost

The structural layout of the building was finalized after implementing the designs for the Slab-on-ground, stairs, elevator and trapezoidal area. The total construction cost for all of the designed members was calculated and expressed in dollars per square foot of floor area. This cost estimate involved first obtaining the quantity and dimensions of all the designed members using Revit Scheduler. Second, the unit cost for the designed members was determined using *RS Means Building Construction Cost Data 2009*. The total cost of the designed project was calculated by multiplying the total quantity for each designed member by its corresponding unit cost. The cost estimate obtained from using the unit cost method was then compared with the cost for a four-story office building presented in *RS Means Square Foot Costs 2009*. Since this project only involved the design of the superstructures and substructure, the cost comparisons were limited to these two categories only.

As shown in table below, *RS Means Square Foot Costs 2009* provides a cost breakdown in seven categories. However, costs of Equipment and Furnishing, Special Construction, and Building Site account for zero percent for the overall cost of a four-story office building. Costs were not identified for these categories since typical office buildings do not require any commercial or institutional equipment, integrate construction and ect. In addition, the cost for Services accounts for almost half of the overall cost. The cost of Services included the cost of Conveying, plumbing, HVAC, Fire Protection and Electricals.

Table 39: *RS Means* cost breakdown for a four-story office building

CSI UNIFORMAT	Office
Substructure	**4.40%**
Shell	**27.7%**
Superstructure	10.1%
Exterior Enclosure	15.9%
Roofing	1.60%
Interior	**23.2 %**
Services	**44.8%**
Equipment & Furnishings	**0%**
Commercial Equipment	0%
Institutional Equipment	0%
Vehicular Equipment	0%
Other Equipment	0%
Special Construction	**0%**
Site Construction	**0%**

From Table 39, the substructure account for 4.4% of the overall cost. Substructures were divided into three categories: foundation footing, concrete slab, and foundation wall. Foundation wall was not part of the design in the project, but a typical 8” foundation wall was included for the final cost estimate. This is because cost of foundation wall account for almost 20% of the cost of substructure in *RS Means Square Foot Costs 2009*. From Table 39, superstructure is categorized within the 28% Shell Cost, and it accounted for 10% of the Shell cost. Superstructure included all the floor and roof construction. Floor construction consists of concrete slabs with metal decking and beam. Roof construction consist of a metal deck, open steel joists and interior columns. In addition, the cost of 2” of metal decking for each floor frame was also included.

7.5.1 Cost Estimate Using Unit Cost Method

This section consists of cost estimates for the structure of the building, cost for interiors and services were excluded from the cost estimates since this project did not involve designs in these areas. The costs calculated using the unit cost method was completed for the designed members, and a detailed cost break down is provided in Table 40. The cost of studs for composite construction was also included in the cost calculations because composite beam-and-slab systems were chosen for the final design. Using the unit cost method, the total cost for each of the designed members was calculated separately, and combined at the end for final cost. Table 41 presents the total cost and square footage cost for the substructure and superstructure. A detailed calculation worksheet is included in Appendix E.

Table 40: Breakdown cost for the structure of the building

CSI UNIFORMAT	Structural Elements	Total Cost for each Element
Substructure (4.4%)	Foundation Footing	$48,000
	Slab-on-ground	$80,900
	Foundation Wall	$179,000
Shell (10.1%)	**Superstructure**	
	Steel Members	$2,162,800
	Metal Decking	$181,100
	Studs	$2,500
	Floor Slabs	$212,400
Interior		N/A
Services		N/A
Equipment & Furnishings		N/A
Special Construction		N/A
Site Construction		N/A

Table 41: Total structure cost for the designed project

	Total Cost	$/Square foot
Substructure	$307,821	$3.6
Superstructure	$2,559,000	$30.3
Total Structure (Sub+Super)	$2,866,800	$33.9

7.5.2 *Cost Comparison with RS Means Square Foot Costs*

From *RS Means Square Foot Costs 2009*, the square foot cost for a four-story office building with area of 80,000 square feet is $130.33 for the material cost. The final square footage cost is $174.35 after applying 32% more of labor cost. The final cost with labor cost was used for comparison since the unit cost data included both material and labor costs. The costs of substructure and superstructure account for 4.4% and 10.1% of the total square footage cost. By multiplying these percentages ratio and total square footage cost $174.35. The square footage for each substructure and superstructure was obtained. A comparison of the designed project and RS means square foot cost is tabulated in Table 41.

Table 42: Cost comparison table for designed project and RS Means square foot cost project

	Designed Project	RS Means
Substructure (4.4%)	$3.6	$10.1
Superstructures (10.1%)	$30.3	$23.3
Total Cost	$33.9	$33.4

From Table above, the square footage cost of the substructure cost for the designed project was lower than project presented in the manual by $6.5, while the cost of superstructure for the designed project was higher by $7. Thus, the overall square foot cost for the designed building structure was only 2 % higher than the cost presented in *RS Means Square Foot Costs 2009*. Although the total square footage cost variation was only 2%, the variation for substructure and superstructure was significantly when compared individually. The cost of substructure for designed projected was lower than RS mean because the cost of rebar for the slab-on-ground were not included. The cost of superstructure for designed projected was higher than RS mean because the cost lateral frame of the designed project was very costly, it was responsible for nearly 23% of the cost of superstructure. For such high lateral frame cost, additional cost for special Site Construction included in the CSI Format cost breakdown should be considered.

In conclusion, the square footage cost for the structure is $33.90 which only considers the designs of the structural framing and the foundations. Since the *RS Means* reference was published two years ago, the inflation factor and location factor were also considered. By applying the two cost adjustment factors, the final square footage cost for framing and foundation is $39.7; thus the total structural cost is about $3.35 million. In the *RS Means Square Foot Costs 2009*, the structure cost account for 14.5% of the overall cost. Assuming same percentage values for the cost of the superstructure and substructure were used, the overall building cost projects to $23.1 million using the unit cost method.

Chapter 8: Code Analysis

8.1 Introduction

The *IBC 2009* includes minimum fire protection requirements by which any type of structure must abide. In essence, fire safety requirements or fire codes are integrated within the building code. Before a building can be used, a building permit is required. It is the responsibility of the local AHJ (Authority Having Jurisdiction) to evaluate and ensure that the building is in compliance with the building code provisions. Therefore, in this chapter, one of the group member acted as an AHJ and performed a code analysis to identify the building's appropriate requirements and to compare these requirements with the modified architectural drawings. The detailed requirements and code section references were included in Appendix K. As noted in Chapter 3, the building dimensions and architectural plans were revised to simplify the steel structural frame analysis. The fire code requirements were observed during this revision process in order to avoid violating any dimensional requirements such as the minimum corridor widths. Some code compliance issues could not be compared with the architectural layout because insufficient information was available. For instance the architectural drawings did not reveal any fire resistance ratings of structural elements nor the interior finishing materials.

8.1.1 Automatic Sprinkler System Installation Justification

The building code analysis revealed that this Gateway building was required to be equipped with an automatic sprinkler system. This was due to the A-3 Occupancy rooms located on floors other than a level of exit discharge as well as the building height exceeding 55 feet. However, for this project, a code analysis was performed to consider both a building with and without an automatic sprinkler system. These two analyses were performed to investigate whether installing sprinklers would provide equivalencies for more flexibility in the building design and to assess their effect on the overall cost of the building.

8.2 Modified Floor Plan

8.2.1 First Floor Layout

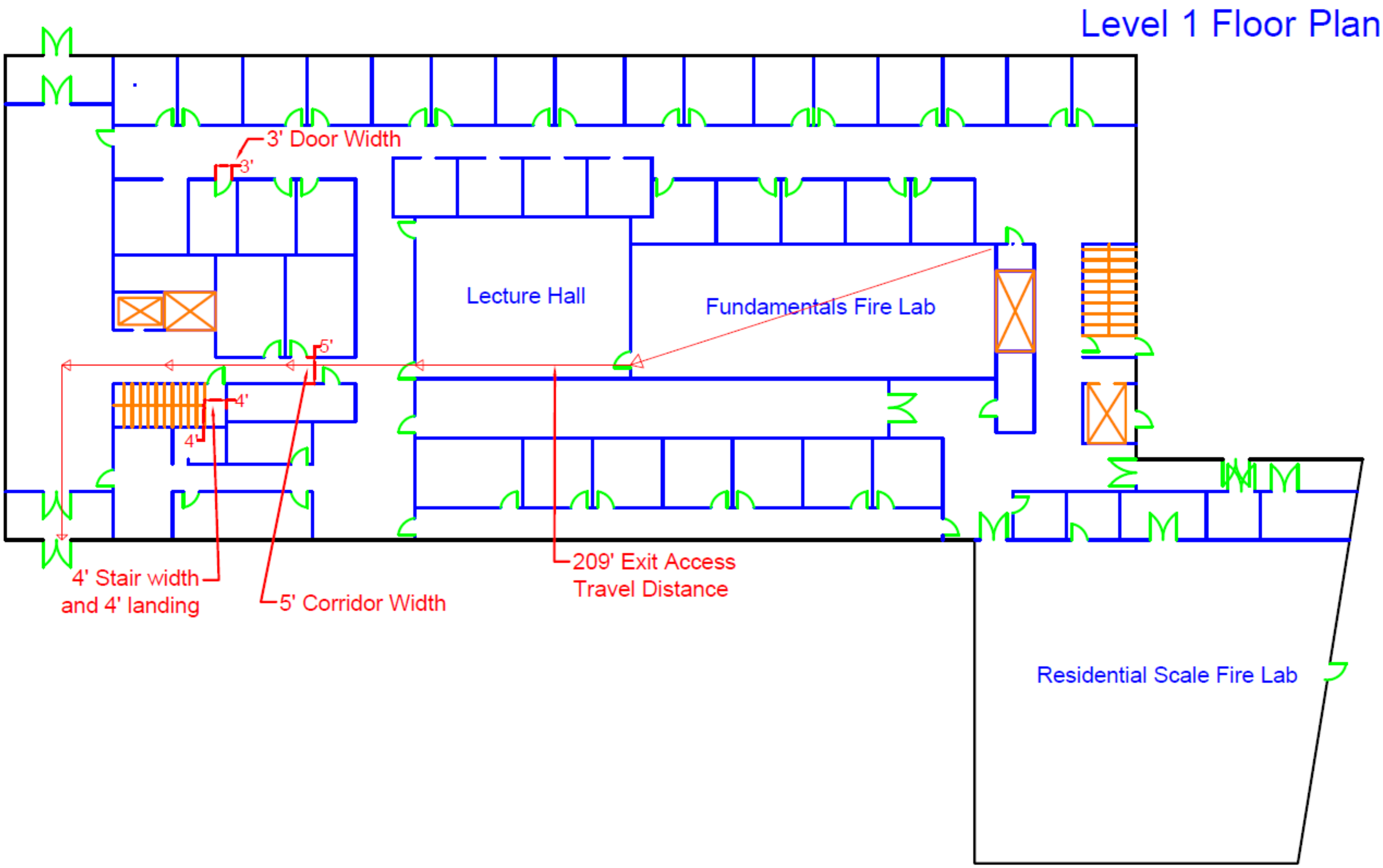


Figure 41: Modified layout of the 1st floor

8.2.2 Second Floor Layout

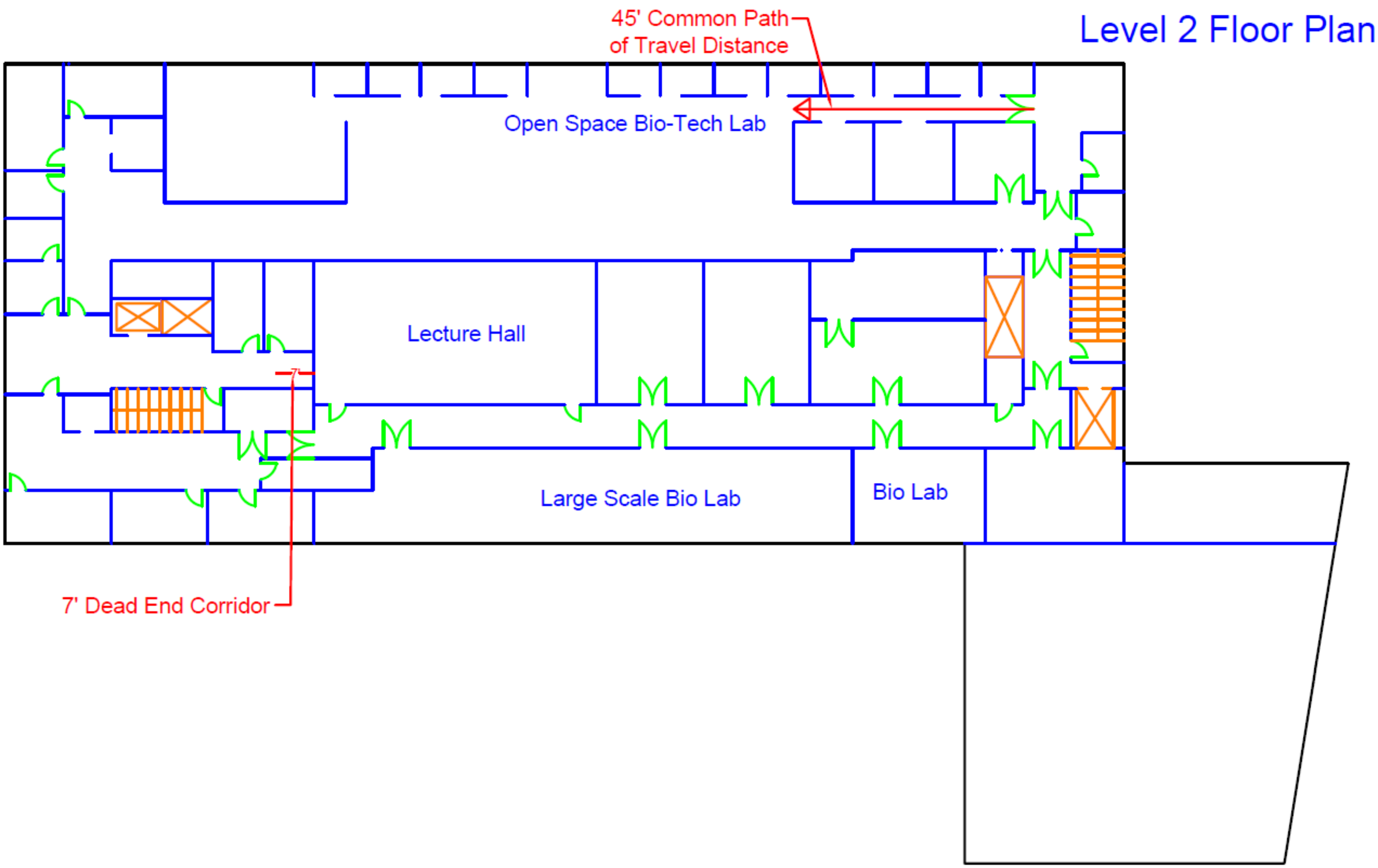


Figure 42: Modified layout of the 2nd floor

8.3 Unsprinklered Building

8.3.1 General Requirements

8.3.1.1 Occupancy Classification

As mentioned previously, the first two floors will be leased by WPI to serve as office space, classrooms, and laboratories. The 3rd and 4th floors are to be leased as office or laboratory spaces. Office space, lecture halls and laboratories are all classified as Group B occupancy. However, due to the size of some lecture halls, which exceed 750 square feet in area, these spaces fall under Group A-3 occupancy. Also, because there are laboratories in which hazardous chemicals will be stored, it was assumed that the amount of such materials would not exceed the limits quantified in Table 307.1(1) to avoid Group H classification. Taking all factors of the building usage in consideration, it was appropriate to classify the building as mixed occupancy consisting of both Group B and Group A-3 occupancies.

8.3.1.2 Construction Type

The June 23rd, 2011 architectural layout version by Perkins + Will identified the building to be of Type IB Construction. However, it should be noted that steel structures can be of either Type I or Type II construction. Since this building consists of an A-3 occupancy group and stands over 55 feet in height, sprinklers are required. If the building was downgraded to Type IIA construction, per *IBC 2009*, the sprinkler system would have acted equivalent to a 1 hour fire resistance rating for building elements as shown on Table 40. The same equivalency does not apply to Type IB buildings.

Table 43: Structural elements fire-resistance rating requirements

Building Element	Fire-Resistance Ratings (hours) Construction Type IB	Fire-Resistance Ratings (hours) Construction Type IIA[d]
Primary structural frame	2	1
Bearing walls		
Exterior	2	1
Interior	2	1
Nonbearing walls and partitions		
Interior	0	0
Floor construction and secondary members	2	1
Roof construction and secondary members	1	1

d. An approved sprinkler system shall be allowed to be substituted for 1-hour fire-resistance-rated construction.

8.3.1.3 Building Height and Area Limitations

The Gateway Park building is a 4-story building standing 57 feet tall with a floor are of 23,400 square feet per floor.

Table 44: Building height and area limitations

Occupancy	Height Limit	Area Limit (per floor)
Group B	11 stories/160ft maximum height	Unlimited
Group A-3	11 stories/160ft maximum height	Unlimited

8.3.1.4 Building Element Fire Resistance Ratings

The chart below shows the typical fire resistance ratings required for building elements.

Building Element	Required Resistance Rating
Structural Frame	2 hours
Exterior Bearing Walls	2 hours
Interior Bearing Walls	2 hours
Nonbearing Walls/Partitions	None
Floor Construction and Secondary Members	2 hours
Roof Construction and Secondary Members	1 hour
Shaft Enclosures Connecting 4 Stories or More	2 Hours

8.3.2 Fire Separations and Resistance Ratings

It is recommended that fire walls be used for the exterior walls of the residential scale fire lab walls and fire barrier walls for the fundamentals fire lab walls. The fire walls will act as a passive fire protection system, containing fire from spreading to other structures. The fire barrier walls will function in the same manner but contain fire from spreading inside the building.

Table 45: Fire separations requirements

Fire Wall Requirements	
Material	Any approved noncombustible materials
Fire Rating	3 hours
Continuity	Continuous from exterior wall to exterior wall
Fire Barrier Requirements	
Material	Materials permitted by the building type of construction
Fire Rating	2 hours
Continuity	Continuous from top of the floor/ceiling assembly below to the underside of the floor or roof sheathing, slab or decking

8.3.3 Means of Egress

8.3.3.1 Occupant Load

Table 46: Occupant load calculation

Floor	**Area**	**Occupant Load**
1st Floor	Office space = 16612 square feet Laboratories = 5588 square feet Lecture Hall = 1200 square feet	16612/100 = 167 persons 5588/50 = 112 persons 1200/15 = 80 persons
		Total Occupant Load = 359 persons
2nd Floor	**Area**	**Occupant Load**
	Office space = 17209 square feet Laboratories = 4760 square feet Lecture Hall = 1431 square feet	17209/100 = 173 persons 4760/50 = 96 persons 1431/15 = 96 persons
		Total Occupant Load = 365 persons

8.3.3.2 Required Number of Exits and Locations

For floors with an occupant load of more than 50 persons and less than 500 persons, a minimum of two exits or exit access doorways shall be provided per floor. Exits or exit access doorways are required from any portion of the exit access.

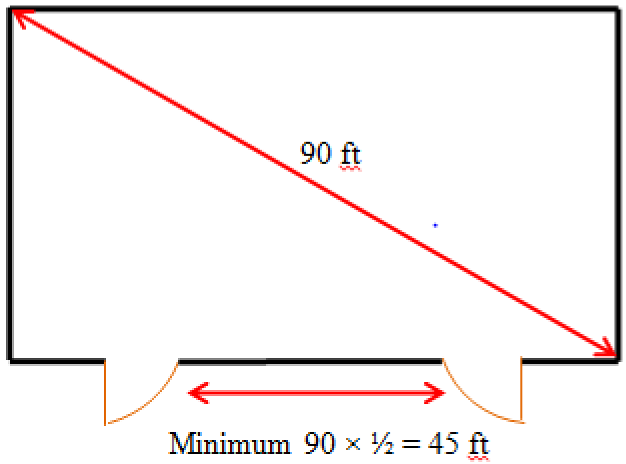


Figure 43: Exit door location requirement

The exit doors or exit access doorways shall be placed a distance apart equal to not less than ½ of the length of the maximum overall diagonal dimension of the building or area to be served measured in a straight line between exit doors or exit access doorways interlocking or scissor stairs shall be counted as one exit stairway. All floors in the building have two or more exits or exit access doorways and are placed not less than ½ of the length of the maximum overall diagonal dimension of the building.

8.3.3.3 Egress Route

Table 47: Maximum allowed travel distances

Reference	Routes	Maximum Travel Distance	Distance Used
1016.1	Exit Access	200 feet	209 feet
1014.3	Common Path of Travel	75 feet	45 feet
1018.4	Dead end Corridors	20 feet	7 feet

The exit access travel distance of 209 feet exceeds the allowed code requirement of 200 feet. However, the distance allowed increases with the installation of sprinkler systems, which was highlighted in Section 8.4.3.

8.3.3.4 Egress Component Widths

Table 48: egress component allowed widths

Reference	Egress Components	Minimum Required Widths	Width Used
1008.1.1	Doors	32 in	36 in
1009.1	Stairs	44 in	48 in
1009.5	Stair Landings	48 in	48 in
1018.2	Corridors	44 in	60 n

8.3.3.5 Egress **Component Fire Resistance Ratings**

Table 49: Fire resistance rating requirements of egress components

Reference	Egress Components	Minimum Fire Resistance Rating
715	Corridor Doors	20 minutes
1022.1	Stair Enclosure	2 hours
Table 1018.1	Corridor Walls	1 hour

8.3.3.6 Elevators

An enclosed elevator lobby shall be provided at each floor where an elevator shaft enclosure connects more than three stories. Elevator lobbies shall have at least one means of egress complying with Chapter 10 and other provisions within this code.

8.3.3.7 Exit Signs and Illumination

Exits shall be marked by an approved exit sign, readily visible from any direction of egress travel. Exit signs shall be placed so that no point in an exit access corridor is more than 100 feet from the nearest visible exit sign. It shall be illuminated at all times and extra power source shall be provided so that it is illuminated for a minimum duration of 90 minutes

8.3.3.8 Openings

Openings in a fire barrier shall be limited to a maximum aggregate width of 25% of the length of the wall and the maximum area of any single opening shall not exceed 156 square feet. A minimum of 90 minutes fire resistance rating shall be provided in openings in exit enclosures.

8.3.4 Interior Finishing

Table 50: Interior finishing materials

Interior Elements	Finishing Materials
Exit Enclosures and Exit Passageways	Class A Materials
Corridors	Class B Materials
Rooms and Enclosed Spaces	Class C Materials

8.3.5 Fire Protection Systems

An approved automatic sprinkler system is required for buildings 55 feet or more in height with approved audible devices connected to every automatic sprinkler system. A manual fire alarm system and manual fire alarm boxes shall be installed for unsprinklered Group B occupancy buildings. Class III standpipe systems shall be installed throughout buildings where the floor level of the highest story is located more than 30 feet above the lowest level of fire department vehicle access. 1 ½ -inch hose stations and 2 ½ -inch hose connections are provided by Class III standpipe system to supply water for use by building occupants and fire departments. Portable fire extinguishers shall be installed in new and existing Group A and B occupant buildings.

8.4 Equivalencies Provided by Installing Sprinkler Systems

This section only highlights the changes made in the building requirements due to the addition of sprinkler systems. The change in value or wording was highlighted in yellow. Except for all the equivalencies mentioned in this section, all of the requirements listed in section 8.2 will be valid for sprinklered buildings as well.

8.4.1 Building Height and Area Limitations

Table 51: Building height and area limitations

Occupancy	Height Limit	Area Limit (per floor)
Group B	12 stories/180ft maximum height	Unlimited
Group A-3	12 stories/180ft maximum height	Unlimited

8.4.2 Exit Locations

Table 52: Exit door location requirement

Exit Components	Remoteness
Exit doors or exit access doorways	1/3 of the length of the maximum overall diagonal dimension of the building

8.4.3 Egress Routes

Table 53: Maximum allowed travel distances

Reference	Routes	Maximum Travel Distance	Distance Used
1016.1	Exit Access	300 feet	209 feet
1014.3	Common Path of Travel	100 feet	45 feet
1018.4	Dead end Corridors	50 feet	7 feet

8.4.4 Egress Component Fire Resistance Ratings

Table 54: Fire resistance rating requirements for egress components

Reference	Egress Components	Minimum Fire Resistance Rating
715	Corridor Doors	20 minutes
1022.1	Stair Enclosure	2 hours
Table 1018.1	Corridor Walls	0 hour

8.4.4 Interior Finishing

Table 55: Interior finishing materials

Interior Elements	Finishing Materials
Exit Enclosures and Exit Passageways	Class B Materials
Corridors	Class C Materials
Rooms and Enclosed Spaces	Class C Materials

8.4.5 Fire Protection Systems

Table 56: Fire protection system requirements

System	Requirement
Manual Fire Alarm System	Not required
Manual Fire Alarm Box	Not required
Standpipe System	Class I (2 ½ -inch hose connections)
Portable Fire Extinguisher	Not required

Chapter 9: Automatic Sprinkler System Design

9.1 Introduction

The code analysis of the Gateway Park building revealed that a sprinkler system was required. However, since the third and fourth floors of the building do not yet have a set of definite floor plans, the design process was conducted only for the first and second floors. This section describes the key procedures in a step-by-step manner for successfully designing sprinkler systems. The guidelines and regulations prescribed in *NFPA 13: Standard for the Installation of Sprinkler Systems 2010 Edition* were used to perform the design process. While performing the architectural plan revision as discussed in Chapter 8, the sprinkler system layout and its design guidelines from *NFPA 13* was taken into consideration. For example, some room dimensions were reduced so that only one sprinkler was sufficient to cover the entire room area, or the locations of some of the rooms were altered somewhat to smooth out complications in terms of sprinkler spacing requirements. The specific references for the sprinkler system design layout, such as the maximum and minimum spacing between sprinklers, were included in Appendix L.

9.2 Classification of Occupancy Hazards

It is important to correctly classify the occupancy hazards for sprinkler installation. According to these occupancy hazard classifications, the fire hazards could be quantified and the heat release rates could be well identified. For different occupancy hazards, the sprinkler system design criteria changes.

The Gateway Park building consists of office spaces, lecture halls and labs. This building has a mixed occupancy use, and as a result, three different occupancy hazard groups for sprinkler systems were identified. Office space was considered a light hazard occupancy, the lecture hall was considered an ordinary hazard group 1 occupancy, and the fire lab was considered as extra hazard group 2 occupancy because it was assumed that there would be flammable liquids and materials.

9.3 Identifying Construction Type

The ceiling construction type of the building has a great influence on the design and performance of sprinkler systems. Depending on the construction type, the position of the deflectors, allowed coverage area and sprinkler head spacing can all be altered. Certain types of ceiling construction could obstruct sprinkler spray patterns from effectively suppressing a fire, and they could also prevent the development of hot gas layers near the sprinkler heads. The formation of these hot gas layers is necessary for timely sprinkler operation. The ceiling construction types are classified as obstructed or unobstructed according to *NFPA 13*.

- *Obstructed construction*: Panel construction and other construction where beams, trusses, or other members impede heat flow or water distribution in a manner that materially affects the ability of sprinklers to control or suppress a fire.
- *Unobstructed construction*: Construction where beams, trusses, or other members do not impede heat flow or water distribution in a manner that materially affects the ability of sprinklers to control or suppress a fire. Unobstructed construction has horizontal structural members that are not solid, where the openings are at least 70 percent of the cross-section area and the depth of the member does not exceed the least dimension of the openings or all construction types where the spacing of structural members exceeds 7½ ft on center.

Figure 44: Example of an unobstructed roof construction type ceiling

(http://www.helpinaflash.com/House-Projects/Suspended-Ceiling-Installation.cfm)

It was assumed that the Gateway building would have suspended ceilings below the floor slabs where all piping and ducts would be installed and extended. Examples of suspended ceilings include mineral fiber, mineral wool and metal suspended ceilings. With the proposed configuration, the Gateway Building was classified as unobstructed construction type.

9.4 Sprinkler System Type

There are a variety of sprinkler system types available. Some are more suitable than the others depending on the building conditions It was assumed the building will be heated during the freezing winter seasons and the pipes will be protected from freezing conditions. For the Gateway Park building, a wet-pipe sprinkler system was selected. Water under pressure is always present in wet pipe systems. As a result, water is immediately discharged once a sprinkler activates under heat. Simple and reliable, wet pipe system is the most commonly used sprinkler system. Wet-pipe sprinkler systems contain the fewest components, resulting in less room for error. Therefore wet-pipe systems are the least likely system to malfunction, providing great reliability. Because of their simplistic nature, wet-pipe systems are economical, as installation is quick, easy, and requires little capital. Maintenance and modification of wet-pipe sprinkler systems only involves shutting down the water supply and draining the pipes, which is a relatively simple process compared with other types of systems. In order to restore wet-pipe systems, the fused sprinkler heads are replaced and the water supply is turned back on to reinstate sprinkler protection. There is no need to reset control equipment.

9.5 Pipe Material

Pipes are essential in supplying sprinkler heads with water at the required pressure and flow rate. Piping comes in different materials and sizes. It is crucial to choose the right piping material as they are subject to damage which could result in leakages or further damage due to corrosions. Although CPVC are good for light hazard occupancies, Schedule 40 steel pipe was chosen as it is the most commonly used and the building consists of both ordinary and extra hazard occupancies as well.

9.6 Number of Risers

For light and ordinary hazard occupancies, NFPA 13 allows for a maximum protection area of 52,000 square feet per floor for a single sprinkler system riser or combined system riser. For extra hazard occupancy, the allowed maximum protection area is 40,000 square feet per floor. One sprinkler system riser per floor, located along the west staircase, was used for the design process. The final sprinkler design layout was included in Section 9.8.

9.7 Sprinkler Head Types

Numerous sprinkler head types exist with some preferred over others under differing conditions. Some examples of different sprinkler head types are pendant, upright or sidewall sprinkler heads. Even within these sprinkler head types, characteristics differ with various options for the orifice size, temperature rating, installation orientation and water distribution characteristics. The sprinkler's K-factor, or discharge coefficient, is the rate at which water is delivered through the sprinkler as a function of inlet pressure. The K-factor is directly proportional to the orifice size. The specified K-factor depends largely on the occupancy classification and "a nominal sprinkler head with a K-factor of 5.6 is commonly referred to as the standard orifice sprinkler." (Bell, 2007). Sprinklers have different temperature ratings which can be distinguished by different colors in the glass bulb or the color code in the sprinkler head.

Table 6.2.5.1 Temperature Ratings, Classifications, and Color Codings

Maximum Ceiling Temperature		Temperature Rating		Temperature Classification	Color Code	Glass Bulb Colors
°F	°C	°F	°C			
100	38	135–170	57–77	Ordinary	Uncolored or black	Orange or red
150	66	175–225	79–107	Intermediate	White	Yellow or green
225	107	250–300	121–149	High	Blue	Blue
300	149	325–375	163–191	Extra high	Red	Purple
375	191	400–475	204–246	Very extra high	Green	Black
475	246	500–575	260–302	Ultra high	Orange	Black
625	329	650	343	Ultra high	Orange	Black

Figure 45: Sprinkler head temperature ratings from (*NFPA 13)*

Three different sprinkler head types were used in this project as there were three different occupancy hazards presented. The various design criteria and the different types of sprinkler heads selected area listed in Table 56.

Table 57: Sprinkler system design values

Design Criteria	Characteristic Values for Design		
Occupancy classification	Light hazard occupancy	Ordinary Hazard Group 1	Extra hazard occupancy group 2
System protection area (for 1st and 2nd floors)	38581 sqft	2631 sqft	5588 sqft
Ceiling Construction Type	Unobstructed	Unobstructed	Unobstructed
Maximum protection area per sprinkler	130 sqft	120 sqft	105 sqft
Maximum spacing between sprinklers	15 feet	15 feet	15 feet
Minimum spacing between sprinklers	6 feet	6 feet	6 feet
Maximum sprinkler distance from walls	7.5 feet	7.5 feet	7.5 feet
Minimum sprinkler distance from walls	1 feet	6 feet	7 feet
Deflector Position	1 inch from ceiling	1 inch from ceiling	1 inch from ceiling
Sprinkler Head	Upright-standard	Upright-standard	Upright-standard
Orifice Size	K-5.6	K-8.2	K-11.2
Temperature Rating	Ordinary	Ordinary	Extra Hazard
Design Area and Density	1500 sqft and 0.10 gpm/sqft	1500 sqft and 0.15 gpm/sqft	2500 sqft and 0.40 gpm/sqft
Hose Stream requirements	100 gpm (30 minutes)	250 gpm (60 – 90 minutes)	500 gpm (90 – 120 minutes)

Using the design criteria established the sprinkler layout was established. The room and corridor dimensions were determined to see how many sprinklers were required for certain spaces in the floor. After the number of sprinkler heads was determined, the sprinkler spacing and distances from walls were set based on the design criteria. Some areas required more sprinkler heads than the calculated number due to the dimensional design criteria. The pipes were laid out in attempts to minimize the total pipe length while all areas were protected.

9.8 Pipe Size Configuration Approach and Water Demand

After the sprinkler system layout was established, the necessary pipe sizes were determined. It is important to adjust the piping sizes accordingly so that the sprinkler heads are supplied with sufficient water to suppress fires while accounting for pressure loss in the pipes due to friction and elevation changes, and also trying to keep the piping costs at minimum. For certain cases, there might not be adequate water supply from the main to meet the demands of the sprinkler system and fire pumps may be required to meet the pressure and flow demands. It might even be less expensive to install fire pumps than using large pipes. However water demand can be largely affected by the overall design layout.

9.8.1 Pipe Schedule Method

The Pipe Schedule Method (NFPA 13) is a standardized method to determine pipe sizing. This method was developed by *NFPA* to make the pipe sizing process easier than using the hydraulic calculation method. The procedure requires counting the number of sprinkler heads on the branch line and selecting the appropriate pipe sizes from the pipe schedule tables incorporated in *NFPA 13*. There are separate tables for each occupancy hazard classifications as shown in Figure 46 and 47.

Table 22.5.2.2.1 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	4 in.	See Section 8.2

For SI units, 1 in. = 25.4 mm.

Figure 46: Light hazard pipe schedule (NFPA 13)

Table 22.5.3.4 Ordinary Hazard Pipe Schedule

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..	20 sprinklers	2½ in.	25 sprinklers
3 in.	40 sprinklers	3 in.	45 sprinklers
3½ in.	65 sprinklers	3½ in.	75 sprinklers
4 in.	100 sprinklers	4 in.	115 sprinklers
5 in.	160 sprinklers	5 in.	180 sprinklers
6 in.	275 sprinklers	6 in.	300 sprinklers
8 in.	See Section 8.2	8 in.	See Section 8.2

For SI units, 1 in. = 25.4 mm.

Figure 47: Ordinary hazard pipe schedule (*NFPA 13*)

The pipe schedule method was used to determine initial estimates for the pipe sizes. Through contacting the *City of Worcester*, it was found that the water supply on Prescott Street has a pressure of 136 psi (City of Worcester, 2012)

9.8.2 Hydraulic Calculation

After the pipe sizes were established using the pipe schedule method, a hydraulic calculation was performed within each of the occupancy hazard group areas to make sure the pipe sizes were supplying sufficient water while meeting the water supply limits.

The density/area approach design method was used to identify the minimum flow required at the most remote sprinkler in a designated area.

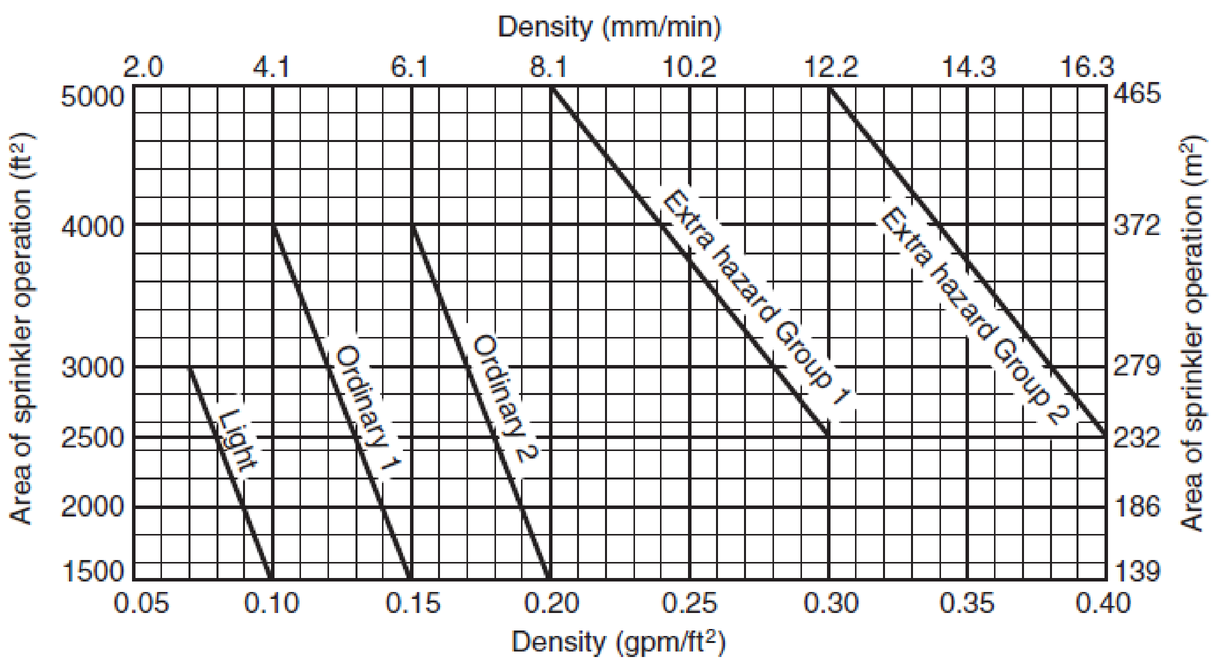


FIGURE 11.2.3.1.1 Density/Area Curves.

Figure 48: Density/area curves (*NFPA 13*)

After the area and density were chosen from the appropriate curve in Figure 47, a section from the design layout was selected. The number of sprinkler heads was counted and the coverage area of a single sprinkler system was calculated. Then, the following equations were used to determine the minimum flow and pressure required at the most remote sprinkler:

$$\text{Equation 1: } Q = A \times d$$

$$\text{Equation 2: } P = (K/Q)^2$$

Q is the flowrate in gpm, A is the single sprinkler head area coverage, d is the density in gpm/ft^2, P is the pressure in psi and K is the sprinkler k-factor.

The pressure loss in the pipes was determined using the Hazen-William equation:

$$\text{Equation 3: } P_L = \left(\frac{4.52Q^{1.85}}{C^{1.85}D^{4.87}}\right)L$$

P_L is the total pressure loss due to friction in psi, L is the pipe length in feet (which should include the fittings and devices within the pipe), Q is the flowrate from the sprinkler in gpm, C is the Hazen-Williams coefficient and D is the internal pipe diameter in inches. The result is the pressure loss, and the pressure loss for each of the pipes was summed up to calculate the total pressure loss in the pipes due to friction. The Hydraulic calculation spreadsheets were incorporated in Appendix M.2.

9.8.3 Water Demand Results

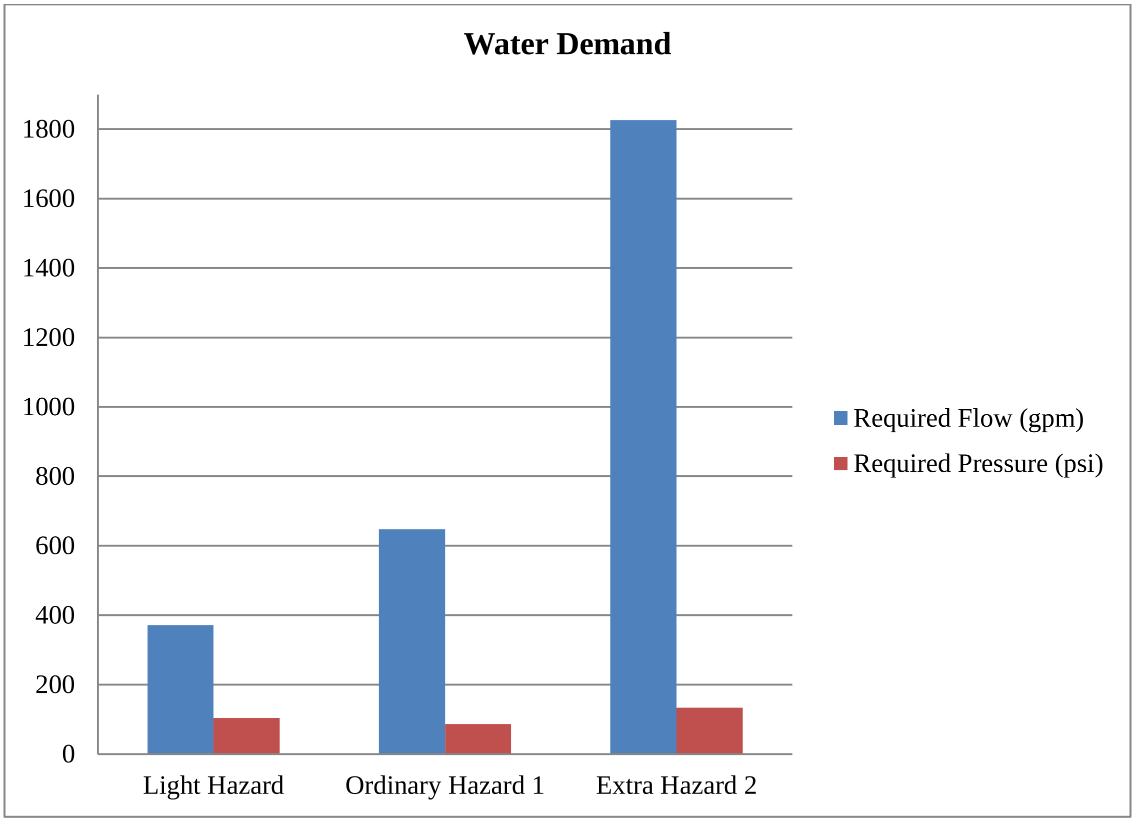


Figure 49: Water demand for all occupancy hazard groups

Figure 49 shows water demand with the required pressure and flow for each occupancy hazards. The water demand revealed that the extra hazard area required a supply of 133 psi at 1826 gpm, which was lower than the water supply pressure from the water main. Because only the water pressure from the water main supply could be obtained, it was assumed that the flow of 1826 gpm could also be met by the water main supply. For extra hazard occupancies, each sprinkler coverage area was smaller than the coverage area of light hazard sprinklers and all the sprinkler heads had a K-factor of 11.2. With a greater orifice size, the water demand was greater for each sprinkler head than the demands of sprinklers in other occupancy hazard areas. Also, the hose stream water flow requirement was greatest for extra occupancy hazard group with 500 gpm required, increasing the total demand as well.

To reduce the water demand and pipe sizes, a separate riser just for the extra hazard area may be used or even fire pumps may be installed for additional water supply. Although such action could possibly decrease the demands on the water supply and the required branch pipe sizes, installing new risers or fire pumps introduces additional cost.

9.9 Sprinkler Design Layout

9.9.1 First Floor Sprinkler Layout

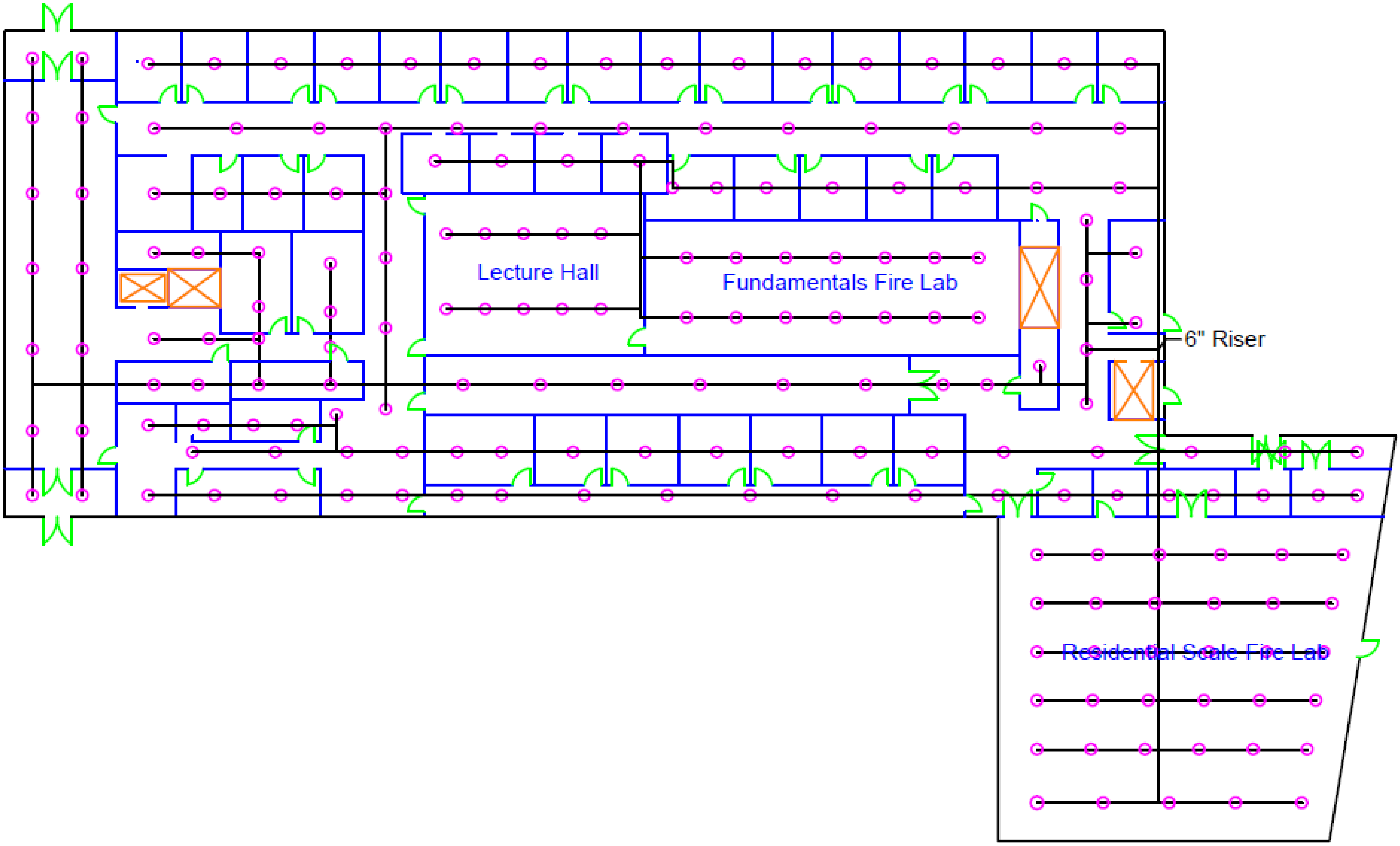


Figure 50: Sprinkler system layout of 1st floor

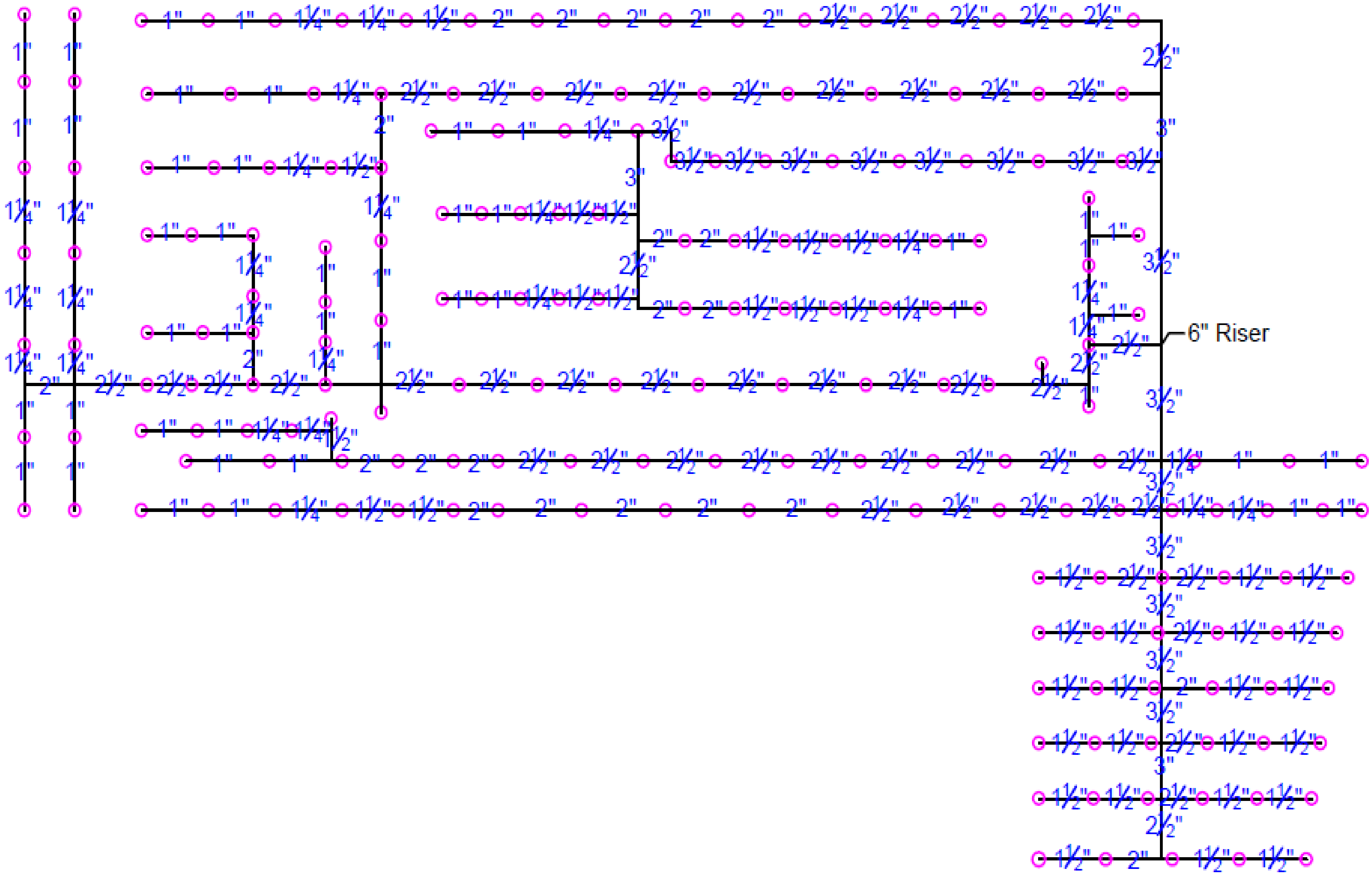


Figure 51: 1st floor pipe sizes

9.9.2 Second Floor Sprinkler Layout

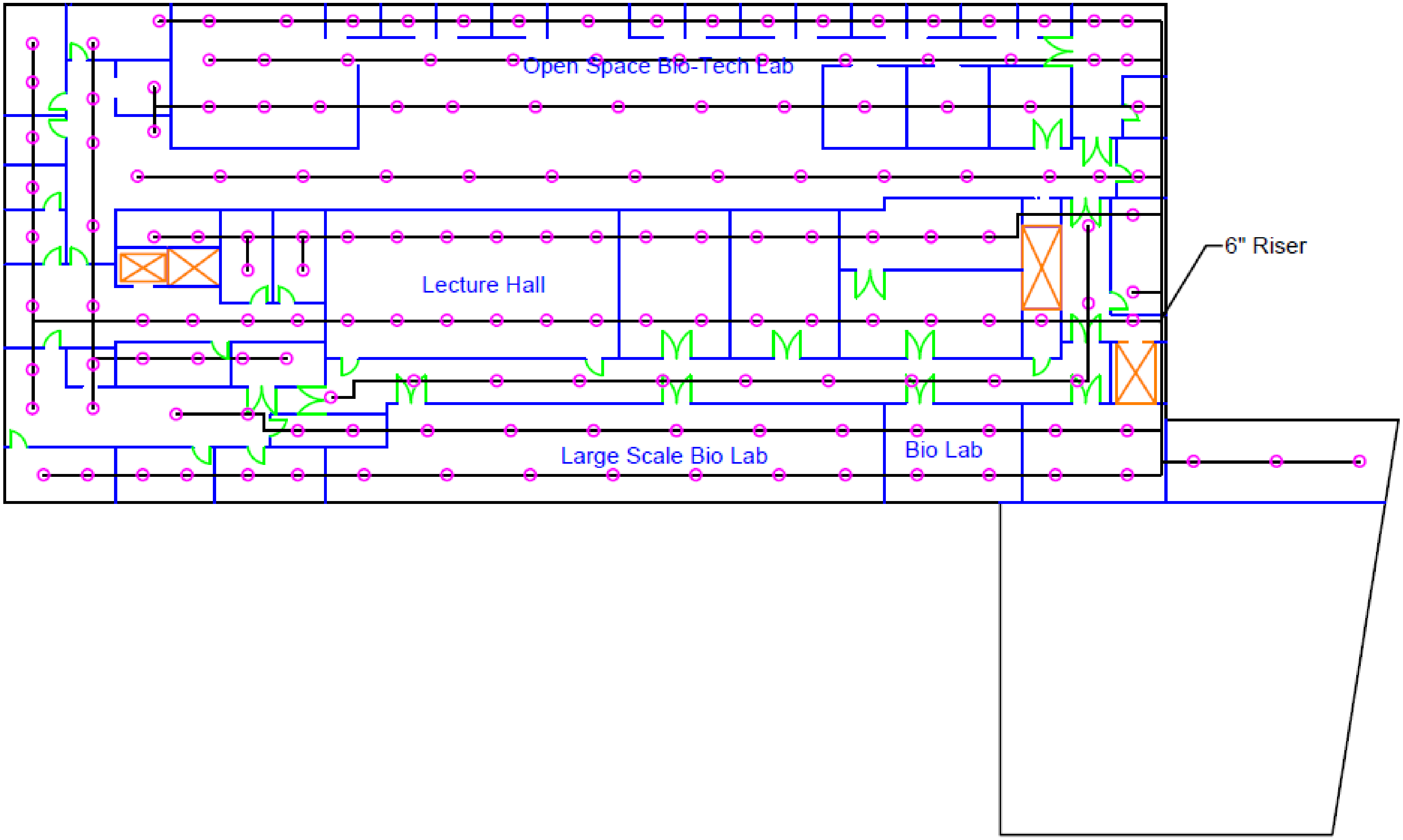


Figure 52: Sprinkler system layout of 2nd floor

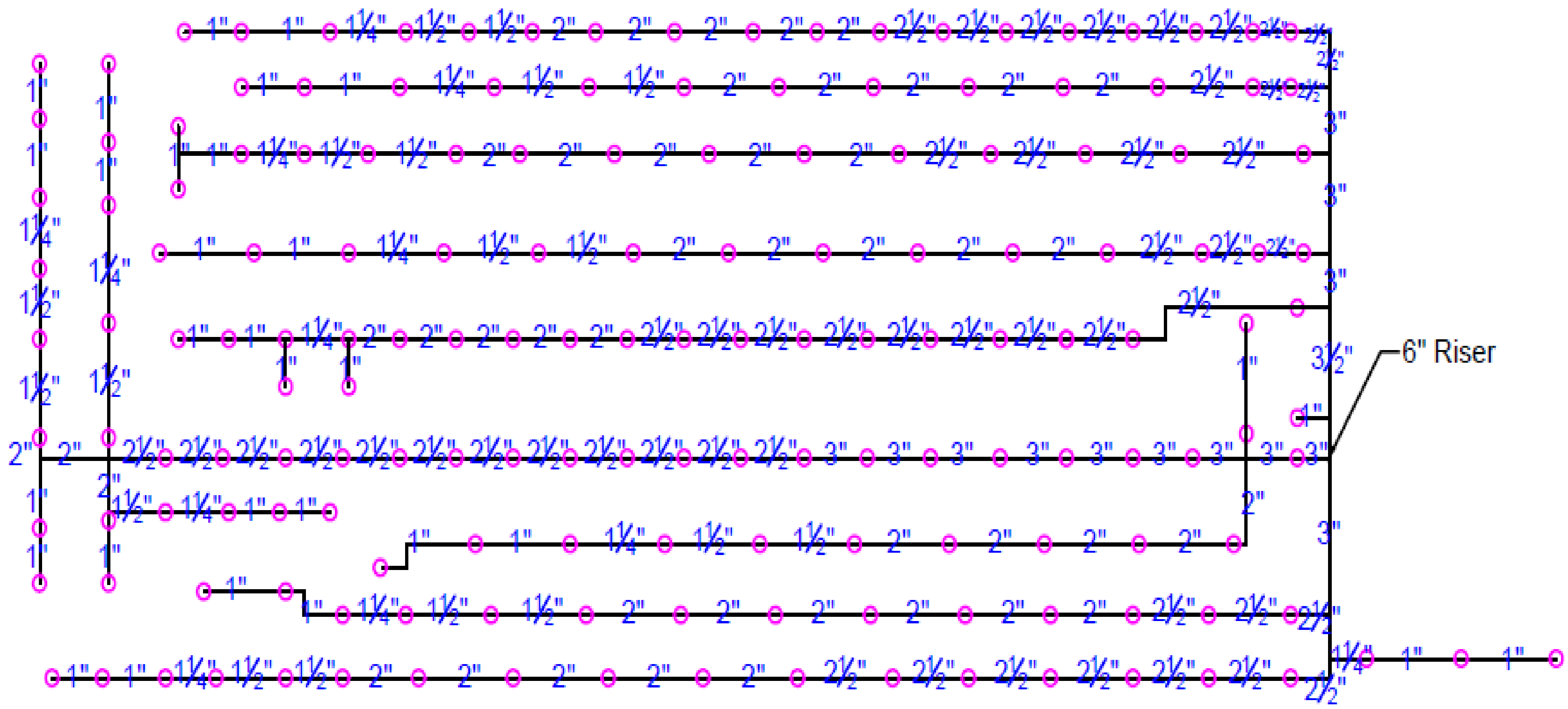


Figure 53: 2nd floor pipe sizes

9.10 Sprinkler System Cost

9.10.1 Cost Estimate

Material costs were referenced from the *RSMeans Building Construction Cost Data 2012*. Costs were obtained for the sprinkler heads and pipes. The cost value obtained incorporated material, labor and equipment cost. The total length of piping was measured for each of the pipe sizes used, and the number of sprinkler heads was counted for each of the different K-factor sprinkler heads. This was done separately for different occupancy hazard areas to see if there were large differences in sprinkler system costs depending on occupancy hazard classification. Figure 54 shows the final cost of the sprinkler system.

9.10.2 Sprinkler System Final Cost

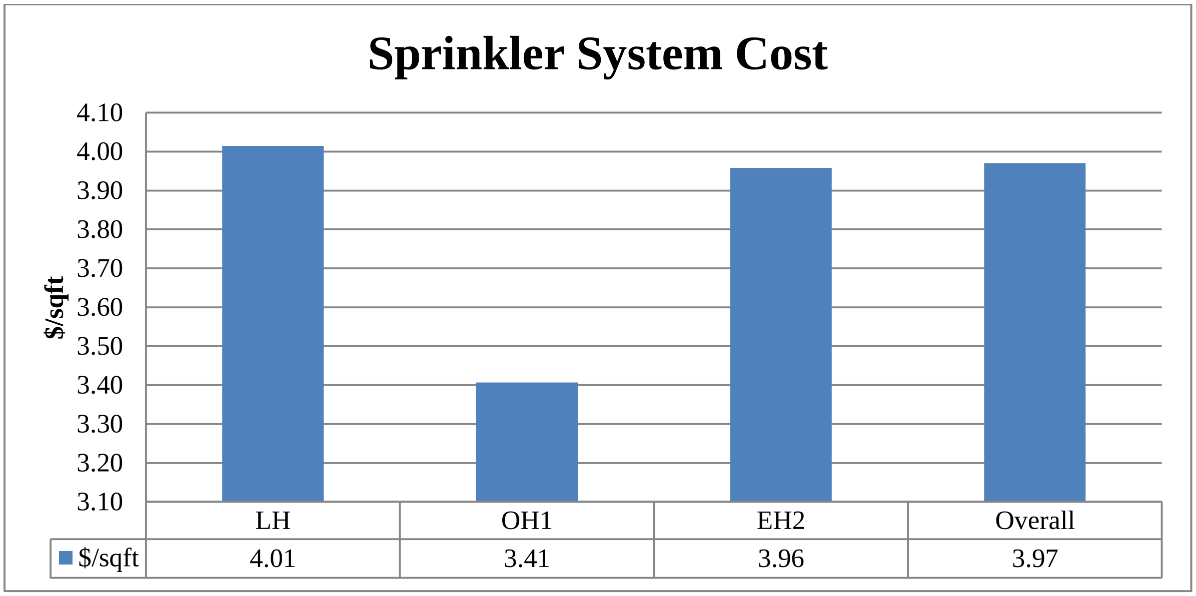


Figure 54: Sprinkler system cost ($/sqft)

The results showed that the overall cost of installing sprinkler system was about $3.97 per square foot. The light hazard area turned out to be the most costly. It was initially thought that the light hazard area would be the least costly with the extra hazard area being the most expensive because the extra hazard area required sprinkler heads with the largest orifice size with reduced coverage area per sprinkler and greater water demand leading to larger pipe sizes. The ordinary hazard area was the least expensive.

The sprinkler head cost was greater for smaller orifice size sprinklers and the pipe costs increased in direct proportion with the pipe diameter. So in terms of just sprinkler head costs, the light hazard area, which was equipped with 5.6 K-factor sprinklers, had the highest sprinkler head cost while the extra hazard area, equipped with 11.2 K-factor sprinklers, had the lowest sprinkler head costs. Also the cost could have been affected the by the interior design. The ordinary hazard and extra hazard areas were smaller in total area and in small portions of the total floor area. The entire building area excluding the labs and lecture halls were classified as light hazard; however, due to the interior design, certain areas required extra sprinkler heads for proper coverage. For example, the floor layout includes 5-foot corridors between offices and separate branch lines were required to serve these corridors.

The cost for each occupancy hazards showed that the cost of sprinkler system is affected by multiple factors. Sprinkler head orifice size, pipe sizes and floor layout all seem to affect the overall sprinkler system cost but it is hard to single out any one of the factors as the most influential in final cost and probably varies for different projects.

The overall cost of about $3.97/square foot for sprinkler system may not be so expensive. For instance, Chapter 8 showed that sprinkler system substitutes for other passive fire protection systems, such as portable fire extinguishers, and provides more flexibility to the design, such as an increase in maximum exit access travel distance.

Chapter 10: Green Roof Design

10.1 Extensive Green Roof Specifications and Design

Figure 55: Extensive green roof on 29 Garden St, Harvard University, Cambridge, MA

(http://www.hydrotechusa.com/garden-projects.htm)

As mentioned in the background section, an additional green roof to a building is an effective strategy to achieve more LEED points. A green roof helps reduce the urban heat island effect, contributes to storm water mitigation, increases the aesthetic value, and demonstrates proof for WPI's commitment to environmental sustainability. To continue the path of LEED-certified buildings such as East Hall and the new WPI recreation center, a typical green roof, specifically the Garden Roof Assembly from Hydrotech Inc., was applied to the new Gateway building.

10.2 Design Procedure

The existing steel roof structure of the Gateway building was used to analyze the effect of new live load and dead loads associated with the green roof. The members were chosen using the

LRFD method. The roof snow load was determined based on the provisions of the *Massachusetts State Building Code*. The dead load of the roof was the combination of the weight of the green roof assembly and the concrete slab. Using the load combination, the moment capacity of the members and the deflections were determined. Two types of members were calculated: one was non-composite, the other was for a composite beam-and-slab system. The composite members carry most of the weight of the green roof and cost less than the other system. Therefore, it was chosen for the green roof design.

10.3 Loadings

From the information given by Hydrotech INC. the details of each component of their green roof system are shown in the figure below:

Figure 56: Green roof components

(Hydrotech-GardenRoof, 2011)

Table 58: Components details for garden roof assembly

Component	Depth Inches	Wet Weight lb/sf
Lite top extensive	4.0	24.0
System Filter	0.01	0.03
Gardendrain GR15 or GR 30	1.18	1.6
Moisture Mat	0.19	1.2
StyroFoam Insulation	4.0	0.68
Root Stop/Hydroflex 30	0.1	0.8
Roof membrane MM6125	0.25	1.4
Total Assembly	9.73	29.71

(HydrotechUSA-Sustainable-design, 2012)

Assembly notes:

- Water Retention Capacity: 1.5 in of rain (0.93gal/sf)
- For roof deck slopes from 0 to 2:12, in this case was flat roof

Determine the snow roof load:

Beside the roof dead loads of structural members and components, the snow load is also a critical design load that was considered. The ground snow load of Worcester was determined to be 55 lb/sf by using Table 1604.10 Ground snow load of the *Massachusetts State Building Code*. To obtain the actual roof snow load, several design factors need to be considered as shown in the table below

Table 59: Design snow load parameters

Parameters	Values
Ground Snow Load (psf)	55
Exposure factor (C_e)	0.9
Thermal factor (C_t)	1.1
Snow load shape coefficient (μi)	1.3
Formula 5.1: SL(psf) = SL=μi*Ce*Ct*sk	40

The superimposed design loads for structural analysis of the green roof include the total green roof assembly saturated weight, the concrete slab weight and roof snow loads. The values of each type of load are shown in the table below:

Table 60: Extensive green roof design loads

Type of load	Value (psf)
Green roof assembly wet weight	29.71
Concrete slab weight	62.5
Snow load (Live Load)	40

10.4 Roof Structure Layout

The green roof system was placed on the top of the existing 210ft x 90 ft flat roof, which has18900 square feet area. Columns that support structure were placed at a 30 ft spacing. The beams were located at a 5 ft spacing o.c. The figure below shows the total area of the roof.

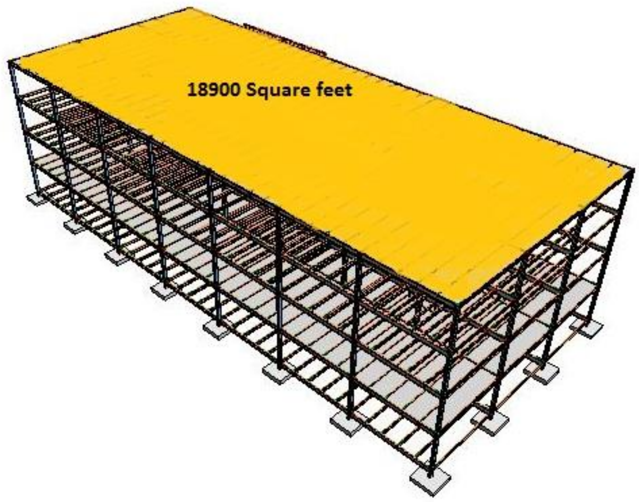


Figure 57: Area of the existing roof

The green roof was placed in an area that has 5 ft offsets from the edges of the building for accessibility. Therefore the actual roof area for the green roof was reduced to 16000 sq ft. The resulting area is illustrated as a green shaded region in figure below

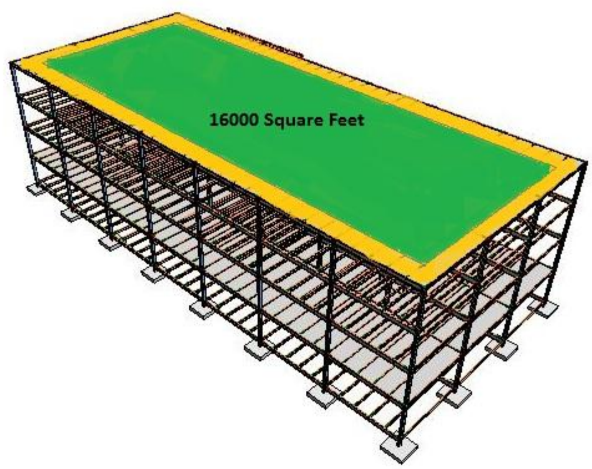


Figure 58: Green roof area

10.5 Results

Using the Hydrotech Garden assembly (Hydrotech-GardenRoof, 2011), the superimposed design loads were used to investigate the adequacy of the existing structural design by using the LRFD method (see Appendix N). The method used in this chapter was the same as in section 4.2: structural design. The results for both the non-composite and composite beam-and-slab framing systems are shown in the tables below:

Load combination: 1.2 DL + 1.6 LL

Table 61: Non-composite members

Type	Member size	Deflection (in)	Moment (ft-k)
Non-composite interior beam	W16x36	0.93	103.1
Non-composite exterior beam	W14x30	0.71	53.98
Non-composite interior girder	W27x84	0.92	646.15
Non-composite exterior girder	W24x68	0.76	352.44

Table 62: Composite Members

Type	Member size	Deflection (in)	Moment (ft-k)
Composite interior beam	W12x26	1.0	98.24
Composite exterior beam	W12x16	1.0	49.12
Composite interior girder	W21x55	0.2	610
Composite exterior girder	W18x40	0.2	339

As the result, the existing structural layout is adequate for the new green roof design. The new green roof was placed on a 5" concrete slab with dimension 200ft x 80 ft. The beams are placed horizontally at a 5 ft spacing on center for every 30ftx 30ft bay. The layout of these structural members is shown in Figure 59:

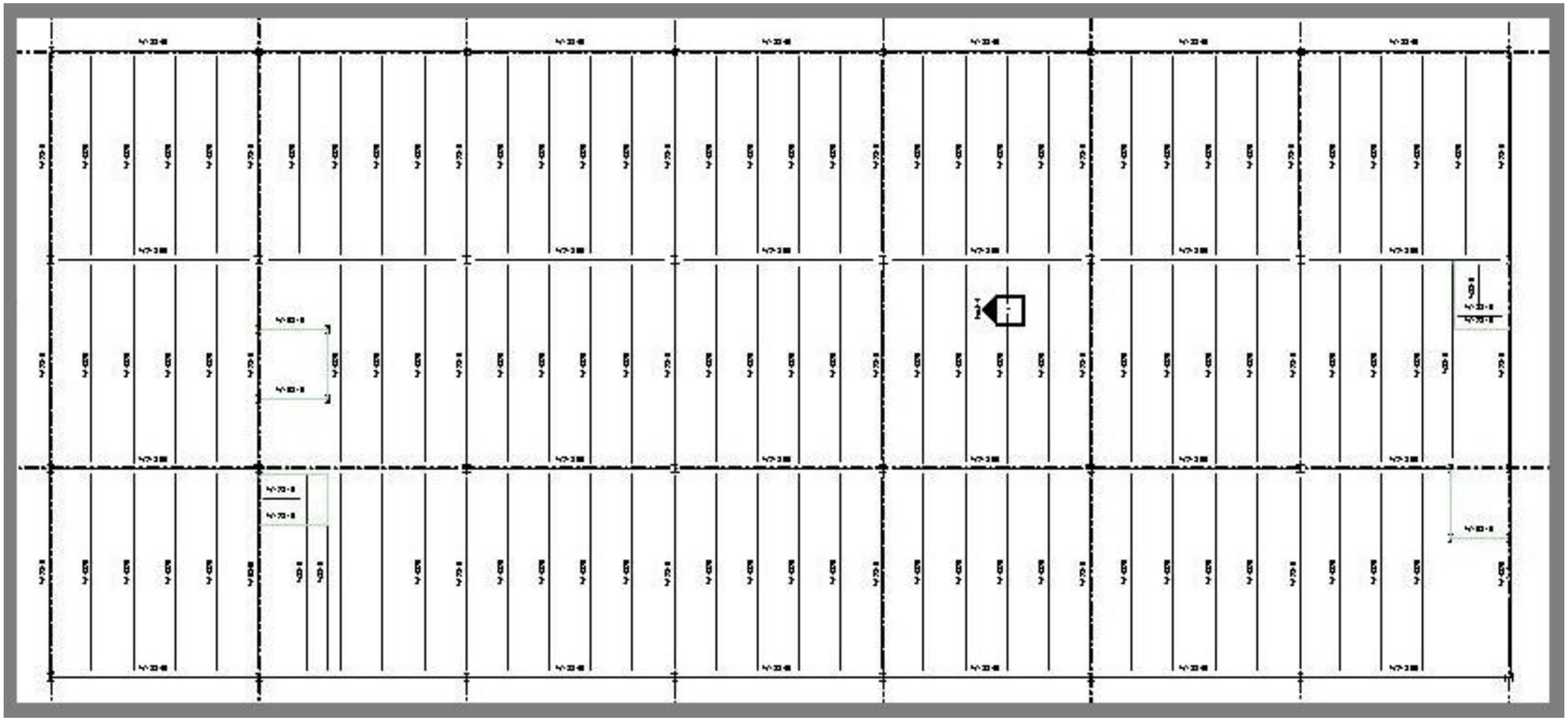


Figure 59: Roof Framing System

10.6 Costs

The costs for the components that comprise the designed extensive green roof were obtained primarily from the *RS Means Building Construction Costs 2012* (RSMeans. Building Construction Cost Data 2012, 2012), under section 33 63.10

Table 63: Cost break down for green roof components

Component	Description	Unit Cost ($/ft^2)	Total Cost
1) Vegetation	Grass/herbs	0.68	$10880
2)Engineering growing medium	Hoist and soil mix 4 in depth	2.76	$44160
3)System filter	Non-woven landscape filter fabric*	0.12	$2000
4)Drainage/retention/ aeration	WEARWELL drainage mat*	1.8	$29400
5) Moisture mat	Moisture retention barrier and reservoir	4.08	$65280
6)Insulation	Polystyrene 4" thick R20	2.37	$37920
7)Root barrier	Root barrier	2.06	$32960
8)Roofing membrane	Fluid applied rubber membrane	7.35	$117600
		Total	**$340200**

*cost obtained from www.autorain.com and WEARWELL Inc.

Based on the *RS Means Building Construction Costs 2012*, the tables below show estimated costs for non-composite and composite structural members that support the green roof system.

Table 64: Non- composite roof framing steel cost

Non-Composite Member Type		Member Size	Total Number of Beams	Total Span Length (ft.)	Unit Cost (plf)	Total Beam Cost$
Beam	Interior	W16×36	124	3720	73	270332
Beam	Exterior	W14×30	5	150	62	9225
Girder	Interior	W27×84	14	420	159	66780
Girder	Exterior	W24×68	13	390	130	50700
				Total Cost (USD)		**397037**

Table 65: Composite roof framing steel cost

Composite Member Type		Member Size	Total Number of Beams	Total Span Length (ft)	Unit Cost ($plf)	Total Beam Cost	Total Num. of Studs	Total Stud Cost	Total Cost
Beam	Interior	W12×26	124	3720	54	200880	12	66.72	200946.7
Beam	Exterior	W12×16	5	150	36	5400	12	66.72	5466.72
Girder	Interior	W21×55	14	420	107.88	45309.6	28	155.68	45465.28
Girder	Exterior	W18×40	13	390	81	31590	20	111.2	31701.2
								Total Cost (USD)	**283580**

The above tables reflect that the composite beam-and-slab system costs less than the non-composite system while their performance still meets the strength and deflection requirements. Therefore, for cost control, the composite system was chosen for preparing the final cost estimate for the green roof construction. The table below shows the break down costs for a complete green roof installation package

Table 66: Estimated total cost of extensive green roof

Item	Item Description	Quantity	Unit of Measure	Unit Cost	Total Cost
Concrete Slab	5" thickness. 4 ksi	247	Cubic Yard	$117	$28900
Design and Specification	5-10% of total cost				$17000-$34000
Project Administration & Site review	2.5%-5% of the total cost				$8500-$17000
Beams & Girders	W shape. Yeild strength 50 ksi	See table 41			$283580
Green Roof Construction	Extensive	See table 39			$340200
		Total Cost			**$678180-703680**

For the upcoming years after the green roof is built, there will be operating costs associated with maintaining and irrigating the roof. The table below provides estimates for the cost of each activity.

Table 67: Life cycle cost of green roof in future

Item	Cost/unit	Total
Maintenance	$1.25-$2.00 /ft^2 for the first 2 years	$20000-$32000
Irrigation	$2.00-$4.00 /ft^2	$32000-$64000
	Total	$42000-$96000

Guideline for Green Roofs by Beck and Kuhn (pp.15-16) (Kuhn, 2003)

Based on the estimated installation cost, the initial investment for green roof system seems to be high, about 1/32 of the total cost of the building. However, there are federal and state tax incentives for green roof usage that encourage owners to improve their building's environmental impact. Table 67 below shows detailed incentive from federal and state programs.

Table 68: Federal and state tax incentives

Type of Incentive	Description	Credit amount
Clean Energy Stimulus & Investment Assurance Act	Recoup 30% of the green roof cost in federal tax credit. No limit on commercial roof. Green roof must cover at least 50% of the total roof surface.	$109710-$117360
Energy Policy Act of 2005 (Potentially)	Federal tax credits of up to $1.80 per sq ft if the building meet ASHRAE standards	$28800
	Total	$138510-$146160

(Green Roof Legislation, Policies and Tax Incentives, 2012)

The Gateway building has a green roof area of about 16000 ft^2, which is 90% of the total roof area. It is qualified for Clean Energy Stimulus & Investment Assurance Act. The owner will get from $138510 to $146160 in federal tax credits when the green roof is finished. That incentive amount covers about 1/3 of the total cost of installing the green roof. This amount of incentive will help the owner to cover at least 2 years maintenance during the life time cycle of the green roof. Moreover, the green roof is potentially qualified for the Energy Policy Act 2005 which awards additional federal tax credit if the building meets ASHRAE standards. For each square foot of the green roof area, there will be $1.80 credit awarded. For a long term usage of about 20 years, green roof brings more benefits and reduces the maintenance cost for the owners.

10.7 Storm Water Runoff

The effectiveness of the green roof for storm water detention depends on its major components. A roof with greater vegetation depth and a better drainage system will increase the water detention rate. For this project, a 4 inch deep grass and Hydrotech drainage could serve to retain over 50% of the rain water. (Hydrotech-GardenRoof, 2011)

A calculation of the peak runoff rate (in cubic feet per second) was computed using the following rational method (Weiler, 2009):

$$Q=C*I*A$$

Q = peak runoff rate (cubic ft per sec)

C = Runoff coefficient (from 0 to 1)

I = rainfall intensity in inches per hour for the design storm frequency and for the time of concentration of the drainage area

A = area of drainage area (in acres)

Rain fall intensity is the depth of rainfall per unit of time, usually expressed in inches per hour. Figure below is the synthetic 24-hr rainfall time distribution curves for Massachusetts. In this project, a1-hour duration was investigated because it has the largest value which is suitable for conservative designed. The corresponding rainfall intensity was 2.5in/hr.

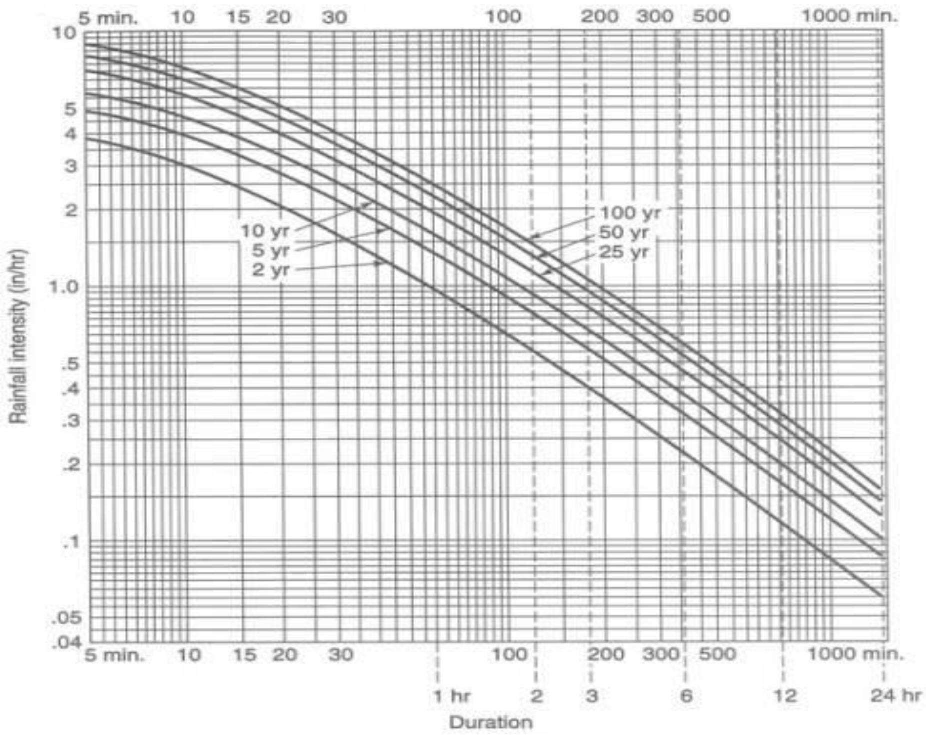


Figure 60: Rainfall intensity in Massachusetts

(Center for Energy Efficiency and Renewable Energy, UMass, Copyright, 2002)

The runoff coefficient for a conventional flat roof with gravel and tar was assumed 0.85. An analysis for the runoff coefficient of the green roof system was conducted to estimate the correct coefficient for the system. This value is defined as runoff divided by the corresponding rainfall

$$C = \frac{runoff\ (in)}{rainfall\ (in)}$$

According to Garden Roof Planning Guide of *Hydrotech Inc*., the extensive green roof has water retention capacity 1.5 in. Therefore the runoff amount is 1 in, rainfall amount is 2.5 in. Hence, the runoff coefficient is 0.4.

The main component of Hydrotech assembly is the drainage/retention Garderdrain with arrays of channels on both top and underside that support maximum drainage of water even when the roots grow into the layer. The table below shows the key parameters and their values for computing the amount of water runoff.

Table 69: Runoff computational parameters

Parameters	Conventional Roof	Green Roof
A(ft^2)	16000	16000
C	0.85	0.4
I (in per hr)	2.5	2.5
Q (cb ft per sec)	0.78	0.37

From the results, the extensive green roof is able to detain about 47% of the rainfall run off, which reduced the flow rate from 0.78 cubic feet per sec to 0.37 cubic feet per sec. While giving advantage to water runoff control, the detained water impacts the building's roof structure. When it rains, the roof loading will be increased by a weight of 0.41 lbs/ft^2. A worse scenario will occur if rain and snow coincide, and the structure of the roof will have to sustain a larger load. These added loads have been factored into the LRFD design equations for evaluating the performance of structural members to ensure the roof will not collapse under these scenarios.

A green roof is an effective tool to control storm water discharge. Green roof fits into the sub category of on-site storm water control technology (Berghage, 2011). This tool is very affective for urban area where runoff causing problems such as peak flow rate increased.

10.8 LEED points for green roof

The table below outlines the LEED credits that can be earned by installing and operating a green roof:

Table 70: LEED credits for green roof

	Intent	Requirement	Technologies/Strategies
Site Credit 6: Storm Water Management (1 Point)	Limit the disruption of natural water flows by minimizing storm water runoff, increasing on-site infiltration and reducing contaminants	Implement a storm water management plan that results in a 25% decrease in the rate and quantity of storm water runoff	Reducing impervious surface, maximize on-site storm water infiltration, and retain pervious and vegetated area. Capture rainwater from impervious area for ground water recharge or reuse within building. Use green roof
Site Credit 7: Landscape and exterior design to reduce heat island (2 points)	Reducing heat island effect to minimize the impact on climate, human, and habitat	Use light colored/high-albedo materials, Energy Star roof compliant, high-reflectance and low emissivity roofing for minimum 75% of the roof surface. Or install green roof at least 50% of roof area	Vegetation is the ultimate high-albedo materials. It cools down the surrounding air and filter dust
Materials Credit 4: Recycled Content (1 Points)	Increasing demand for building products that have not corporate recycled material, reducing the environmental impact from making new material	25% of building materials that contain in aggregate a minimum weighted average of 20% post-consumer recycled content material, or 40% post-industrial recycled content material	Use monolithic waterproofing Environmental Grade (EV), MM6125EV, qualified for 25% post-consumer recycled material. Using Gardendrain water retention/drainage components contains post-industrial recycled materials

((HydrotechUSA-Sustainable-design, 2012)

Chapter 11: Conclusion and Recommendations

11.1 Structural Design Recommendations

Twelve different structural scenarios were investigated for a four-story office building located at Gateway Park, Worcester, Massachusetts. Each of the schemes had different variables in terms of the structural bay dimensions, beam spacing, and the use of composite or non-composite beam-and-slab construction. The scoring system established in section 5.2 revealed that Design C30.6 would provide the building owner with the best value solution. Value was determined from the number of points rewarded for every dollar spent per square foot of construction. Design C30.6 provided 36.6 points per dollar, and the average number of points from all twelve designs was 21.8 points per dollar.

Design C30.6 consisted of 30’×30’ bay areas with 6 feet beam spacing on center and composite beams and girders. This design had the highest points for dollar per square foot ratio because it had structural and architectural benefits in net usable floor area, number of columns and an economical structural steel frame.

For Design C30.6, typical spreading footings, a floor slab-on-ground, lateral frame, automatic fire sprinkler system, and green roof were designed using appropriate design methods introduced for each of the respective components. The completed structural layout with identified dimensions is present in figure below.

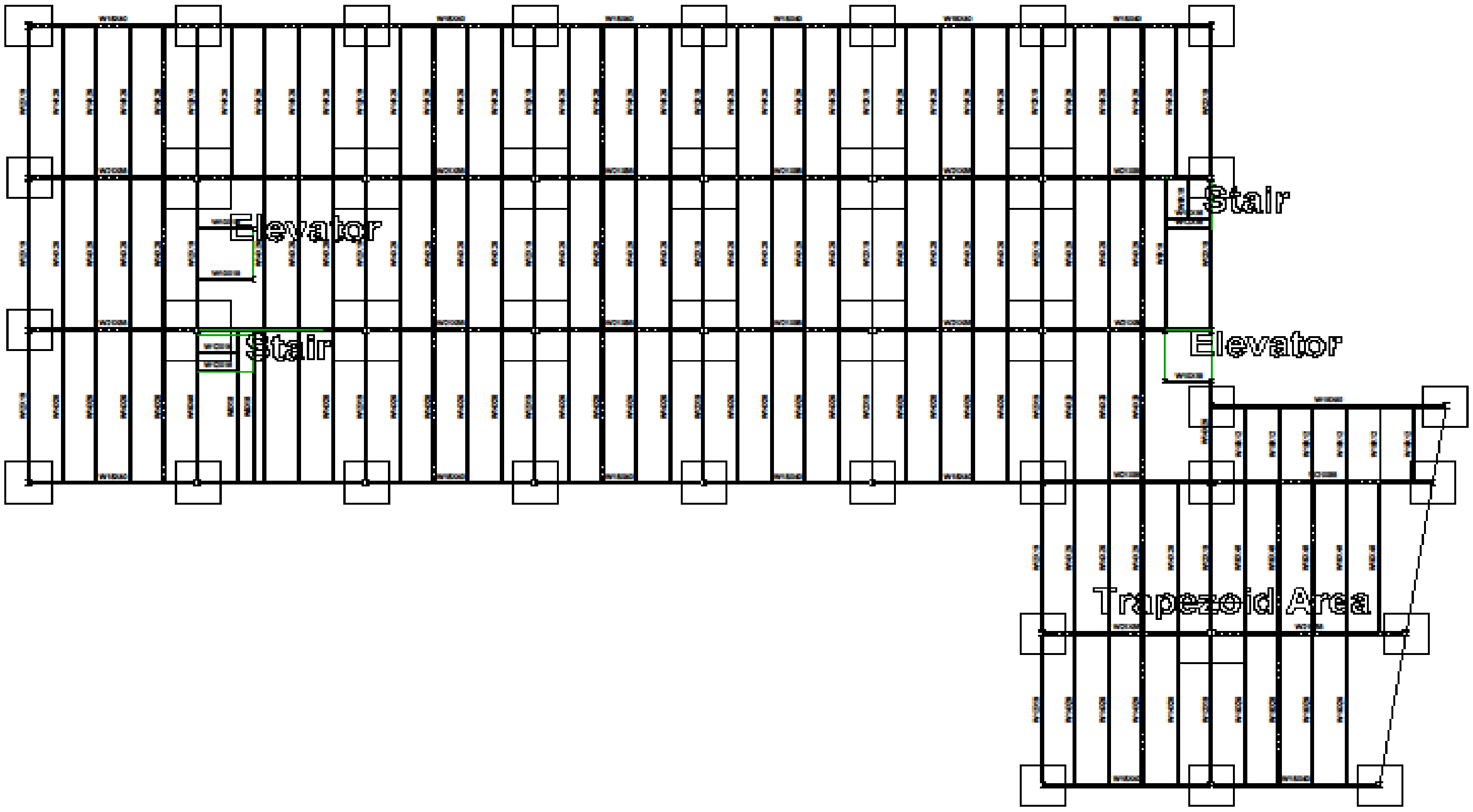


Figure 61: Structural floor plan (double click to view the details)

Figure below presents the foundation layout for the building. By studying the geotechnical report, spread footings were determined as the most appropriate type of the foundation for this building. The spread footings were designed to support the columns and bearing the ground, thus there is a footing designed to correspond with each of the column sizes. From the foundation layout, three different sizes of footing were determined for the interior, exterior and corner columns. A concrete slab-on-ground was also designed for floor on level to support the applied loads from the ground. Using WRI slab thickness selection method introduced in the *ACI Manual*, an 8-inch thick slab-on-ground was determined for the floor area.

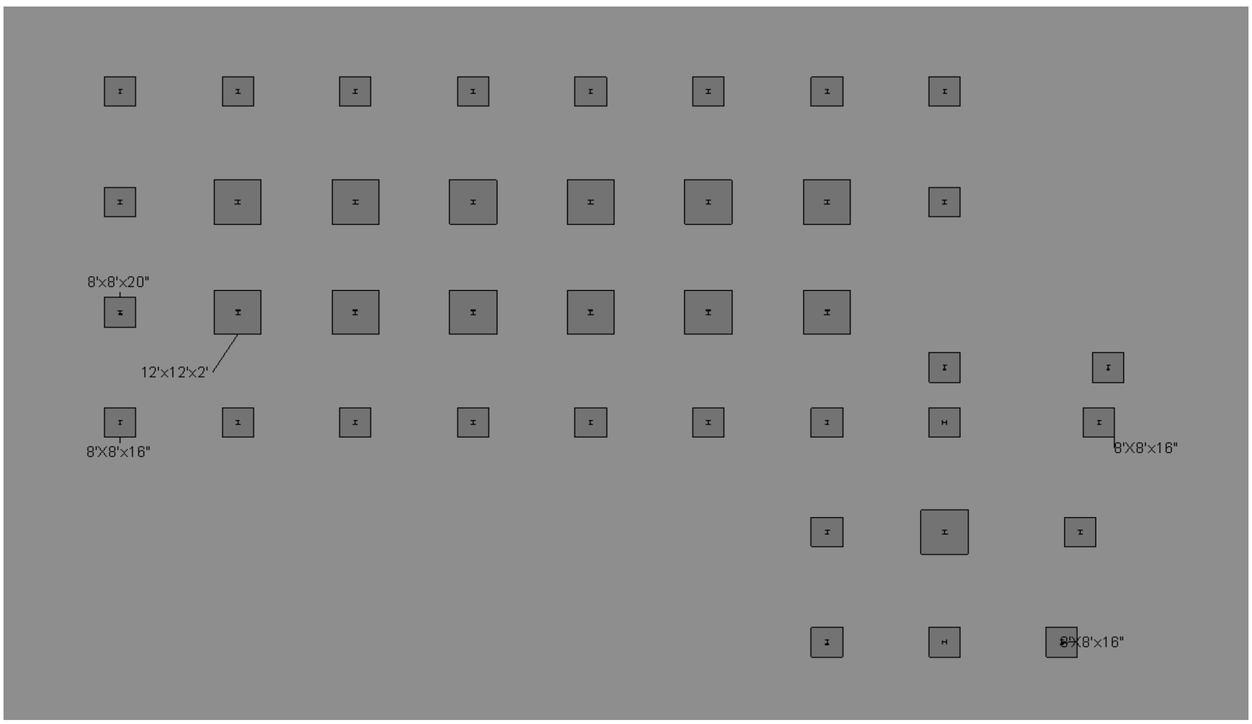


Figure 62: Foundation plan

11.2 Lateral Frame Recommendations

The later frame is an important component for the stability of the building's structure. A rigid frame was chosen for this design to avoid architectural conflicts. The rigid frame uses moment-resistive connections to perform against lateral forces including wind, and seismic. The lateral frame was designed using LRFD and *ASCE 7* method along with *Robot* modeling for second-order moment analysis. The result outputs from *Robot* verified the adequacy of the designed structure in all load cases. The proposed frame consists of beams W14x159 sections as columns and W16x77 sections for the girders. There are twelve frames total for the building's structure as shown in the figure 63 below:

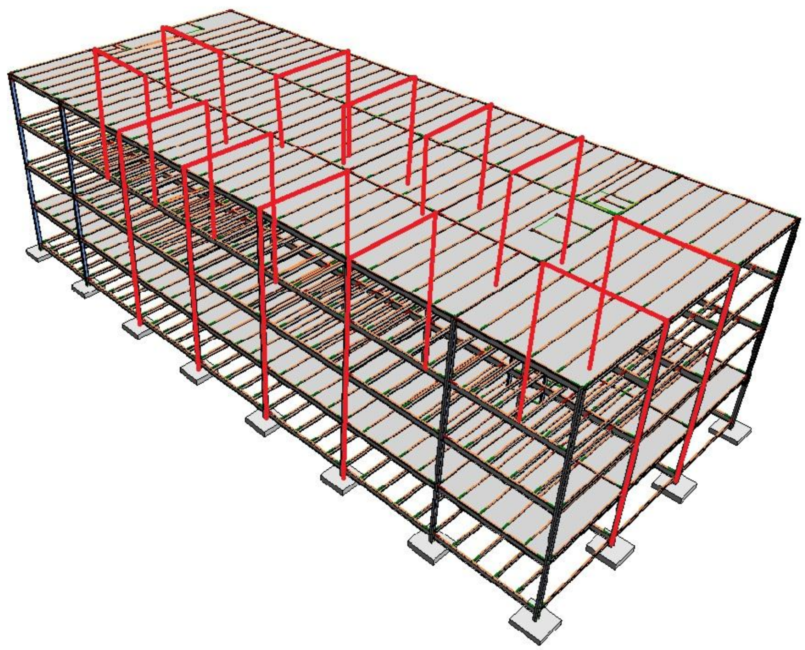


Figure 63: Lateral frames layout

11.3 Cost Recommendations

In conclusion, the square footage cost for the structures including: spreading footing, slab-on-ground, structural steel framing, metal decking and concrete floor slab is \$39.7.The square footage cost for the structural frame was calculated using the unit cost method. It is stated in *RS Means Square Foot Costs 2009*, the structure cost account for 14.5% of the overall cost for a typical four-story office building. By applying the same ratio, the overall cost of the designed building was approximated as $23.1 million. In a recent news report the developer ODG announced that $32 million was invested for this actual project. (Gateway Park at WPI) The overall cost for the building designed in this project seems much lower, but with 85% of the building cost undefined, an accurate scale-up of the total costs from the cost estimates for the superstructure and substructure is not expected.

11.4 Green roof Recommendations

The extensive green roof designed for this building brings a lot of advantages to the owner and occupants in term of reducing building operation cost and promoting environmental concerns. It also helps improve the environmental sustainability, while effectively contributing to storm water control. Although the investment for the new green roof is fairly high $678180 to $703680, there are federal and state tax incentive programs that encourage builders and owners to incorporate the sustainability design into their buildings, making it more affordable. Particularly for this project, a potential amount of tax credits from $138510 to $146160 may be awarded to the owners. Although the initial construction cost for green roof is more expensive compare to conventional roof, but in a long term usage, the green roof has a better life cycle cost benefits such as reducing energy usage, conserving natural resources to create a better sustainable living environment.

11.5 Automatic Fire Sprinkler System Recommendations

Section 9.11.1 revealed that installing sprinkler systems would cost about $3.97 per square foot. This overall cost included areas with light, ordinary and extra hazard groups. The cost, when broken down by each hazard group, revealed that the light hazard group had the highest cost at $4.01 per square foot. This demonstrates the fact that sprinkler system costs do not depend solely on material costs. The occupancy hazard, sprinkler orifice size, interior layout, piping layout, all have direct effects on sprinkler system cost.

The hydraulic calculations revealed water demand was highest in the extra hazard area. The overall demand including the hose stream requirement was 133 psi at 1826 gpm. The need for fire pumps could not be determined due to insufficient information. If this water demand seems too high and the building owner would like to reduce the demand, there could be several options to explore. The most common methods to reduce water demand are to use fire pumps, install a separate riser for supplying just the extra hazard area, or increasing the pipe sizes. However, all methods would add extra costs.

For a sprinkler system supplied with water of 133 psi at 1826 gpm, the overall cost of $3.97 per square foot seems economical. The building owner should be aware that installing sprinkler systems reduces cost in other fire protection systems as identified in Chapter 8.

11.6 Ideas for Future Work

This project entailed details on several different areas such as different types of structural components, automatic fire sprinkler systems and green roof design. However, there were areas our team would have liked to delve into for further work if time had permitted.

For structural analysis, this project only looked at steel members. Initially, there were thoughts about comparing steel and concrete and construction to highlight the advantages and disadvantages of each. Such work could be taken on by future MQP groups.

For green roof design, other types of design such as intensive roof or lawn with variety of green roof components other than Hydrotech could be investigated. Combination designs of solar electricity system and solar thermal system could also be considered. Those types of design ultimately promote sustainable building with LEED certification along with the owners' reputation in the industry.

For automatic fire sprinkler systems, exploring several different sprinkler layouts for the same floor design to see how pipe layouts could have an impact on the cost and water demand of sprinkler systems would be interesting. Also, different types of sprinkler heads such as ESFR (early suppression fast response) or extended coverage could be used to see how different characteristics of sprinkler heads could impact the layout and overall cost.

Students could investigate further into the structural analysis computer software *Robot*. One suggestion would be modeling the whole building frame in *Robot* and checking the building adequacy by applying LRFD code requirements. However, *Robot* Professional is required for such building analysis.

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Appendix A: Proposal

Project # LDA - 1207

Steel Structural, Fire Safety and Green Roof Design for Gateway Park Building

Major Qualifying Project Report

Submitted to the Faculty of

WORCESTER POLYTECHNIC INSTITUTE

In partial fulfillment of the requirements for the

Degree of Bachelor of Science

By

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Jin Kyung Kim

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Long Huynh

____

Dong Yi Mei

DATE: October 14, 2011

Approved by:

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Professor Leonard D. Albano, Faculty Advisor

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Abstract

This Major Qualifying Project explores the design of a four-story building at Gateway Park in Worcester, Massachusetts. Several designs with different elements will be presented and analyzed to recommend the best design in terms of cost, constructability, performance and usable area. Different green roof and fire protection designs for the building will be investigated and recommended.

Capstone Design

For this MQP project, the main objective of the group is to serve as structural engineering consultants and provide steel design for a four-story-office building located at Gateway Park, Worcester, Massachusetts. Along with the structural design, green roof and fire protection will be implemented to the design. Lastly, different forms of cost estimation using RS means manual will be developed to produce and determine the most economical design system. As stated in the ABET General Criterion Curriculum, the designs will incorporate engineering standards and realistic constraints that include the following considerations: economic, environmental, sustainability, constructability, ethic, health and safety, and social.

Economic

- Steel cost will be calculated for each of the design members, such as beams, girder, columns, studs and frame.
- Once the building design is complete, *RS Means Building Construction Cost Data 2009* will be used as the reference to approximate the cost of building per square foot.
- Design with relatively low price will be recommended to the owner.

Sustainability

- Beam, girder and columns will be placed in the legitimate section to produce efficient and useful spaces.
- Promoting environmental awareness by investigating the environmental and structural implication of green roof design.
- Acquiring LEED certification for environmental sustainability.

Constructability

- Alternative design scenarios were developed: composite and non-composite design members, different bay sizes and beam spacing and shored and unshored construction to provide alternatives.
- Maximizing repeatability by considering the standard size materials, like the steel member sizes.
- Separation of office and lab spaces will reduce complication during construction.

Ethics

- The design systems will be in compliance with the *International Building Code 2009* and *NFPA* publications.
- While cost will be an issue, meeting the minimum requirement in terms of performance will be the main priority.

Health and Safety

- All Structural system scenarios will be designed in compliance with the *International Building Code, AISC Steel Manual, and ASCE 7-05*
- The building will be designed with fire protection systems. The fire protection design will meet the minimum requirements of the codes in NFPA publications.

Introduction

This project involves numerous design aspects for the new four-story Gateway building located on Grove and Prescott Street, Worcester, Massachusetts. In the early 1900's Worcester was primarily known as a manufacturing industry for producing metal or wire. With the sudden decline in the manufacturing industry, many companies were shut down and this left numerous empty and unutilized properties which led to a widespread of environmental damage. WPI and Worcester Business Development Center (WBDC) took on the daunting task of transforming brownfield to research center. Finally in 2007, a four-story Life Science and Bioengineering Center building construction was completed. The 125,000 square feet Life Science and Bioengineering Center is mainly used as laboratories, conference rooms and office spaces. Following the first Gateway Park building construction, WPI announced an agreement with O'Connell Development Group of Holyoke (ODG) for the next building at Gateway Park, in 2009. Under the agreement, WPI ground-leased one of the park's four remaining pad-ready sites to ODG, who was responsible for financing, developing, constructing and owning the new building. The ground breaking ceremony took place on April 21st 2011 for the 32million dollar, four-story building with a total area of 92,000 square feet. This building is currently being constructed in front of the parking lot and next to the Life Science and Bioengineering Center.

The architectural design of the building was obtained through Professor Salazar, an associate professor for the Civil and Environmental Engineering department at WPI, which was designed by architects at *Perkins+Will*, hired by ODG as architectural designers.

The team will perform structural design and analysis for the new Gateway Building to meet the demands of the architectural design. The structural design will satisfy all functional and structural aspects for the multi-occupancy building while having reasonable cost and being aesthetically pleasing. Structural analysis is essential to any construction and no matter how impressive the architectural plan is, the structure must have adequate strength, stiffness and stability to withstand all loads. Alternative structural steel frame design scenarios for the new Gateway Building will be investigated and the structural design and analysis process will be performed in compliance with the *IBC 2009*. The best design scenario will be recommended by comparing cost values, constructability, performance and usable area.

To ensure the client that all the codes are satisfied, this project will also look into fire safety design. *IBC 2009* will be reviewed to find applicable prescriptive codes required for the minimum fire protection requirements. Two fire protection design systems will be investigated where one system will not include sprinklers and the alternative system will include sprinklers which will be installed in compliance with *NFPA 13*. These two designs will be compared to see the cost and effectiveness of sprinkler systems and what affects it could have on the overall structural design.

Concerned with environmentally friendly buildings, a green roof will be designed based on existing structural load capacity, geographic, and local climate. The design will include two alternative types of green roofs with different location of plants and landscaping. The design will be in compliance with *IBC 2009*, EPA standard requirements, and building's LEED certification.

Furthermore, the design will also look at effects that the green roof brings to the building in terms of energy performance and improvement of eco system.

Cost estimations will be presented for all design systems. There will be two alternative methods to estimate the total cost of construction. The first method will be using the cost per square foot for each design system. The second method will use RS Mean Building Construction Cost Data 2009 to estimate the cost of each member per linear foot.

Scope of Work

This project will address a number of questions regarding the structural, fire protection, green roof and cost analysis aspects of a four-story steel structure Gateway Park building. The focus questions will be presented in this section along with the methods that will be used in order to analyze and answer the problems.

Structural Systems Design

As there are numerous methods for designing steel structural frames, alternative design scenarios with each containing different elements will be investigated. All the scenarios are shown on Figure 1 and Figure 2. All the scenarios will be analyzed and evaluated.

- **Focus Questions**

What bay size is the most economical? Are larger bay sizes more economical?

How effective are composite and non-composite beam and girders in terms of performance and cost?

How does the spacing of beams affect the cost? Are bigger or smaller spacing more economical?

What affect do composite or non-composite columns have on the usability?

How do lateral forces impact the design?

How do shored and unshored construction method compare in terms of cost and constructability?

What will happen to the structural design and its cost if a greater loading than the required loading is used to allow greater flexibility for the building owner?

Design Systems:

Figure 1: System Group I Alternative Design Scenarios

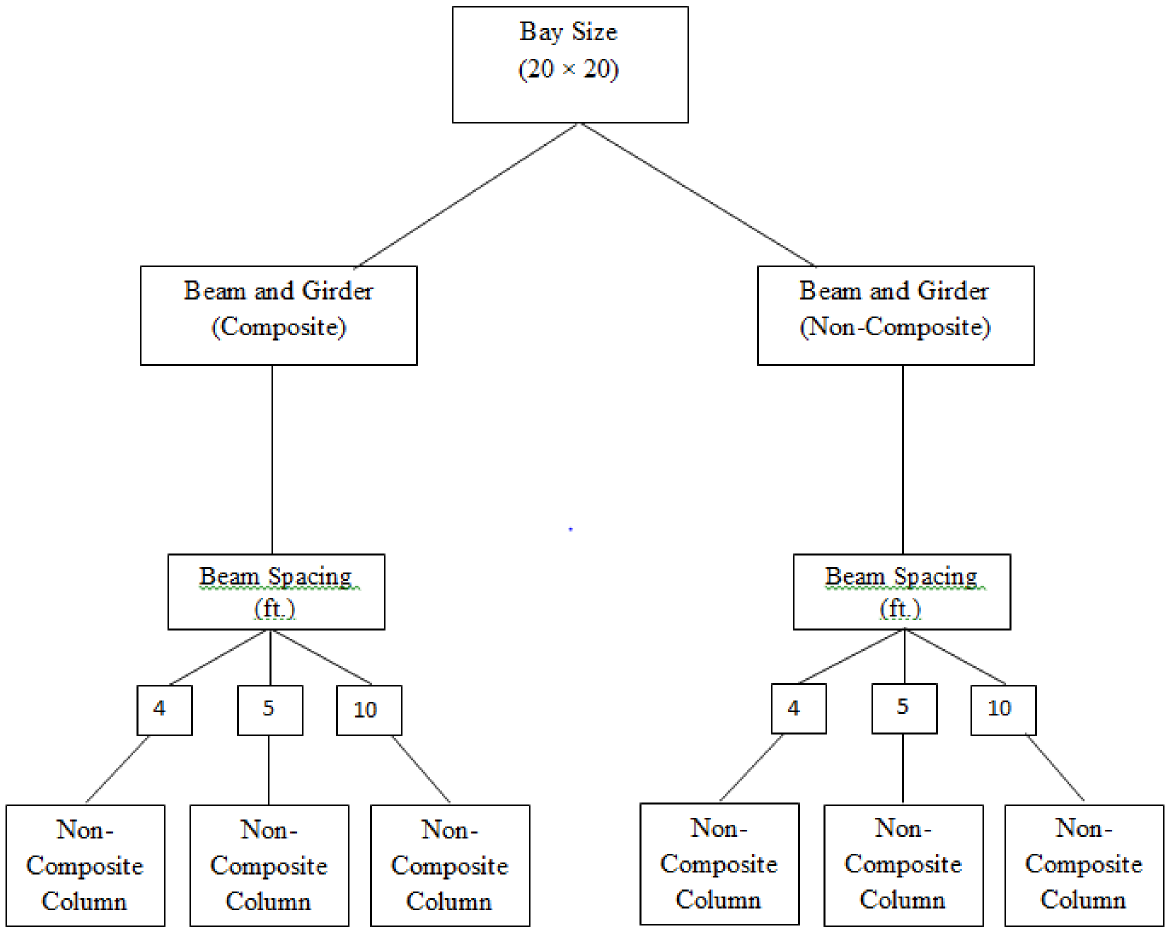
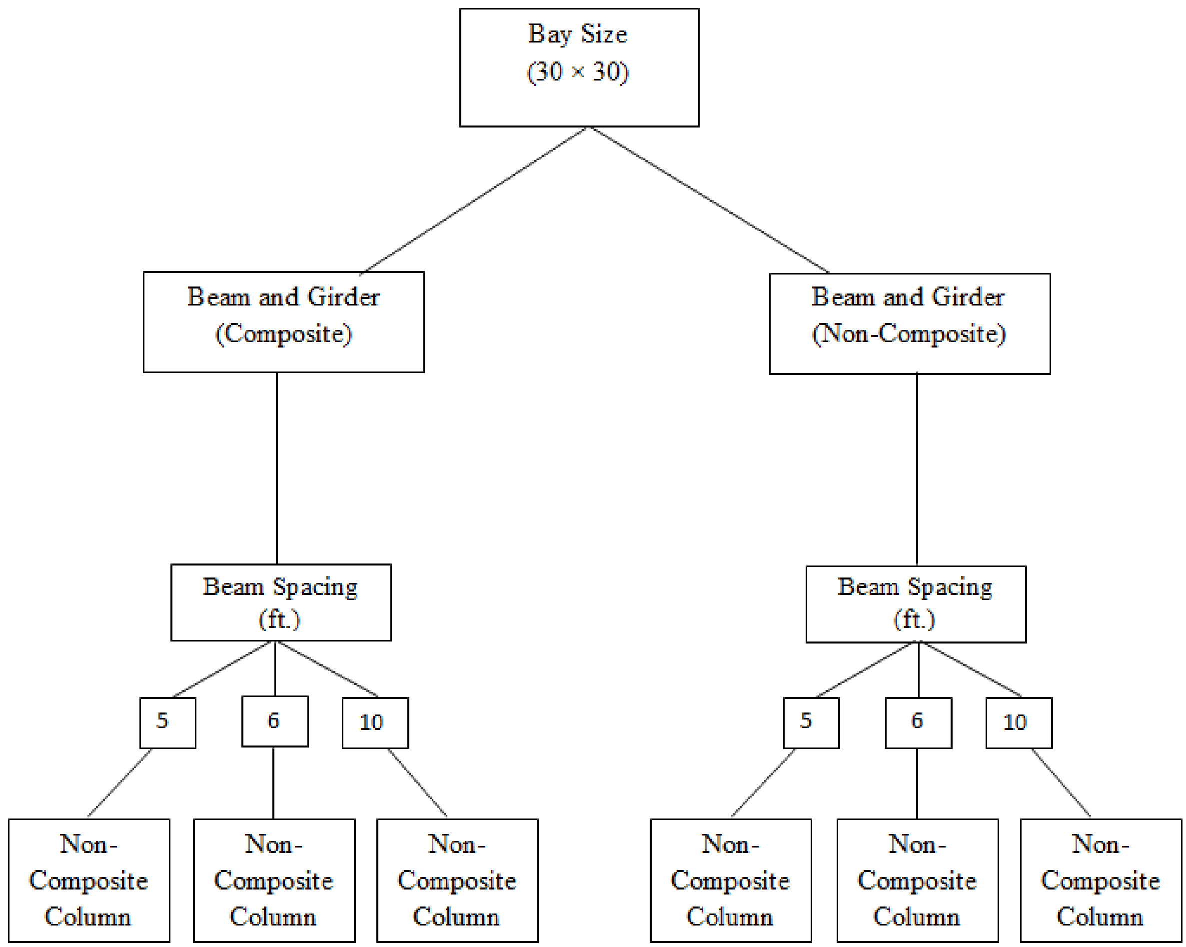


Figure 2: System Group II Alternative Design Scenarios



- **Methods**

The structural analysis will consist of multiple floor and roof framing schemes with each having different characteristics. The main focus will be on examining different bay size areas, composite and non-composite structural elements and different beam spacing. The first step in the design process is choosing the bay size. Two typical bay sizes are considered in the design with different design members. Repeating the same bay size in the floor plan would increase constructability, lowering the cost. However, it is not easy to use a single bay size for an entire building area as it may vary for areas with heavier loads and non-rectangular edges.

Beams and girders are two main horizontally spanning steel members that are very similar in shape and structure. Beams are designed to resist vertical loads, and then transfer loads to girders. Column is the vertical steel member that is designed to transfer the girder loads to the foundation of the building. For this project, a typical concrete footing design will be used instead composing a new footing design. Thus, the number of footing required is simplified to number of columns

on sublevel. The spanning length of the beam is an important factor that determines the required number of columns. Fewer and larger columns are required to support the beams with larger span, which lower the number of footing requirement. Inversely, additional and smaller columns are required to support the beams with smaller span. Thus more footing is required for beams for smaller span.
A typical 6 inch concrete slab design is assume for this design process. Slab thickness is tested by *span length/24.* The calculated ratio must not exceed 6 inches to become the adequate design scenario. Due to the limited available strength in flexure table mention in AISC Design Manual, beams are required not to exceed 40 feet span and 20 feet spacing. Three beam spacing of 4 feet, 5 feet and 10 feet will be investigated.

The loading conditions will be considered in the earlier stage of the design process for determining the legitimate beam member sizes. *Minimum Design Loads for Buildings and Other Structures* commonly known as ASCE7 is entitled by the American Society of Civil Engineer for finding applicable design loads. Table 1 lists the loading requirements for an office building.

Table 71: References for Loading Requirements

Loading Requirement	Reference
Dead Loads	Steel Manual
Live Loads	IBC 2009 Table 1607.1
Live Load Element Factor K_{LL}	IBC 2009 Table 1076.9.1
Snow Loads	ASCE 7- Chapter 7
Wind Loads	ASCE 7- Chapter 6
Earthquake Loads	ASCE 7

The building will be multiple occupancy containing some laboratory spaces and conference rooms. Thus different building design loads will be considered for the selected system design. *AISC Steel Manual* will be used in determining design member sizes. Lastly, *RISA*-2D software will be used for analyzing the lateral and gravitational load effects. Braced and rigid frames will be compared and members will be designed for both set of frames by using the interaction equation listed on the *AISC Steel Construction Manual*.

Unshored construction will be assumed for each design. Costs will be calculated for all of the design systems. After reviewing and analyzing all systems, one design system will be recommended based on the cost, performance and constructability. Another cost analysis will be done on the final recommendation design system using shored construction, in order to compare unshored and shored construction costs. Additionally, the recommended design scenario will be redesigned with heavier loadings using both shored and unshored construction. Using a heavier loading than the required amount can provide flexibility in terms of building use for the owner.

Design Scenario Selection

Structural scheme selection is one of the most important tasks for structural design. To determine a design scenario with the best performance, each system will be evaluated base on three factors: cost, constructability and usability. By analyzing and weighting in these selection factors, the best design scenario will be recommended to the owner.

Generally, cost is the most important factor for clients in the process of selecting the design. Thus, cost will account for higher percentage in the design selection. For each alternative, the cost for fabrication and erection of the structural steel and the equation shown on Figure 3 will be used to calculate the steel cost of design per square foot.

Figure 3: Steel Cost per Square Foot

$$Cost\left(\frac{\$}{sf}\right) = \left(\frac{Beam\ Weight, \left(\frac{ton}{ft}\right)}{Beam\ Spacing, (ft)} * Cost\ per\ ton\right) + \left(\frac{No.of\ Studs * Cost\ per\ stud}{Beam\ Spacing\ (ft), * Beam\ Span\ (ft)}\right)$$

Ensuring the ease of construction is important in order to lower the overall material and labor cost. Constructability is evaluated by comparing the member uniformity and how well the structural plan collaborates with the floor plan. Usability is another important property to evaluate in the process of selecting the structural scheme. Total usable area refers to the total free living space. Buildings with larger living spaces will provide greater benefits for the owners. By comparing the three properties for each design scenario, two design scenarios with the best average performance will be selected for further estimation and investigation.

- **Deliverables**

All Deliverables for the structural design component can be found in Table 6 under the topic areas of Architectural, Structural Elements Comparisons and Structural Calculations.

Fire Protection Design

Large portions of the building code include fire safety codes. These codes and standards are included as a means to minimize the possibility and effects of fire. Fire codes address the minimum fire protection requirements that a structure must adhere to for construction phases, design process and a fully completed and occupied building. It is essential to meet the fire code requirements in order to prevent and suppress fires which help protect life and property while minimizing the damage. Structural fire protection is generally achieved through both a combination of active and passive fire protection systems. Active fire protection refers to manual or automatic fire detection and suppression. Passive fire protection refers to fire resistant compartments, such as special walls with fire resistance ratings. An engineer has the freedom to play around with different fire protection systems and based on the choices, it could have significant cost reductions as fire codes generally lessen the fire criteria requirements if certain types of fire protection systems are used. It is widely thought that a sprinkler system is one of the most effective fire protection systems and this section will take a close look at a design with sprinkler system and one without sprinkler systems.

- **Focus Questions I**

What benefits do automatic fire sprinkler systems offer?

Which is more economical when considering sprinklered vs. non-sprniklered building?

What modifications could be made for structural elements and fire protection systems if sprinkler systems are installed?

Will installing sprinkler systems give architects more freedom in terms of floor design?

- **Methods**

IBC 2009 Edition and NFPA 13 2010 Edition will be reviewed to find the applicable codes to the Gateway Building. The following charts, which list some (not all) the general codes in terms of fire protection, will be used as a reference to help guide the fire protection design process. Two solutions of sprinklered and non-sprinklered buildings for fire protection design will be presented and analyzed to compare and explore the benefits of installing sprinkler systems.

Table 2: Code Analysis for Non-Sprinklered Gateway Building

Classification of occupancies for the building	**IBC Chapter 3** – Use and Occupancy Classification
Construction Types classification	**IBC Table 601** – Fire resistance rating requirements for building elements
Building limitations of heights and areas based on Type of Construction	**IBC Table 503** – Allowable Building Heights and Areas (based on construction type)
Fire resistance ratings for fire barriers	**IBC Table 707.3.9** – Fire-Resistance Rating Requirements for Fire Barrier Assemblies or Horizontal Assemblies between Fire Areas
Requirements and fire ratings for fire partitions	**IBC Section 709** – fire rating, requirements, materials, continuity
Fire doors and shutters requirements	**IBC Table 715.4** – Fire Door and Fire Shutter Fire Protection Ratings
Structural element protection requirements	**IBC Table 720.1** – Minimum Protection of Structural Parts Based on Time Periods for Various Noncombustible Insulating Materials
Wall fire rating requirements	**IBC Table 720.1(2)** – Rated Fire-Resistance Periods for Various Walls and Partitions
Floors and Roofs	**IBC Table 720.1(3)** – Minimum Protection for Floor and Roof Systems
Concrete Slab Thickness	**IBC Table 721.2.1.1** – Minimum Equivalent

	Thickness of Cast-in-Place or Precast Concrete Walls, Load-Bearing or Nonload-Bearing
Interior Finishing	**IBC Table 803.9** – Interior Wall and Ceiling Finish Requirements by Occupancy
Egress System Terms	**IBC Section 1002.1** – Definitions (for different egress systems)
Egress System Requirements	**From IBC Chapter 10** **1003.2** – Means of egress shall have a ceiling height of not less than 7 feet 6 inches **1004.1.1** – Maximum Floor Area Allowances per Occupant **1005.1** – Minimum required egress width **1006.1** – Illumination of egress is required **1007.2.1** – at least one required accessible means of egress shall be an elevator is required for buildings with four or more stories **1007.3** – exit stairway needs to have a minimum clear width of 48 inches **1009.1** – 44inches minimum for stairways **1016.1** – Exit access travel distance **1018.1** – Corridor Fire-resistance rating **1018.6** – Fire-resistance-rated corridors shall be continuous from the point of entry to an exit and shall not be interrupted by intervening rooms **1021.1** – Minimum Number of Exits for Occupant Load **1022.1** – enclosure requirements and fire resistance ratings

Table 3: Code Analysis for Installing Sprinkler Systems

Occupancy hazard and commodity classification	**NFPA 13 Chapter 5** – Classification of Occupancies and Commodities
Ceiling Construction Type	**NFPA13 Chapter 3.7** – Construction Definitions
Sprinkler System Allowable Area	**NFPA 13 Chapter 8.2** – System Protection Area Limitations
Coverage area of single sprinkler head and spacing of sprinklers	**NFPA 13 Chapter 8.6.2.2** – Maximum Protection Area of Coverage
Sprinkler spacing requirements for small room	**NFPA 13 Chapter 8.6.3.2.4** – allowed to be 9ft from any wall
Distance from walls	**NFPA 13 Chapter 8.8.3.2** – Maximum Distance from Walls
Sprinkler placement to avoid discharge obstruction	**NFPA 13 Chapter 8.8.5.1.2** – sprinkler arrangements
Vertical obstructions	**NFPA 13 8.8.5.2.2** – Distance from Suspended or Floor-Mounted Vertical Obstructions
Piping systems	**NFPA 13 Chapter 6.3 –** Aboveground pipe and tube **NFPA 13 Table 22.5.2.2.1** – Light Hazard Pipe Schedules
Sprinkler temperature rating	**NPFA 13 Chapter 8.3.2 –** Temperature ratings
Sprinkler responsiveness	**NFPA 13 Chapter 8.3.3 –** Thermal Sensitivity

- **Focus Questions II**

What type of sprinkler system will be the most suitable to the Gateway Park Building?

What type of sprinkler heads should be used?

What material piping systems should be used?

- **Methods**

A literature review of books and reliable internet articles will be performed to gain sufficient knowledge of the types of sprinkler systems, sprinkler heads and piping materials that are available. The set of components that seems to be the most suitable with this building will be used in the design process.

- **Deliverables**

All deliverables related to fire protection design can be found in Table 6 under the Fire Safety Code Analysis and Sprinkler System Design Analysis Summary categories.

Sustainable Design

Green roof, also called as living roof or eco-roof, is technology that incorporates planting vegetation with landscaping on top of the roof. This technology has been applied to a majority of the European countries in the past 40 years. This type of technology provides many benefits in environmentally, economically, and in energy consumption where land resources are limited and energy source is expensive. Green roof helps mitigate heat in the air, save energy cost by being a natural insulation and retain storm water. In the larger scale, green roof improves climatic environment by reducing urban heat island effect. It widely affects sustainable development of the ecology system. For the past decade, green roof has been implemented increasingly across America. The U.S Environmental Protection Agency (EPA) plays a key role for green roof development in the country. It has comprised a compendium of strategies for reducing Urban Heat Island Effect, setting up webcast and conference calls on green roof topics, and composing community actions on green roof database of locals and states' initiatives to reduce heat island effect. To promote such efforts, a green roof design will be created for this project.

- **Questions**

Does the existing structure have enough load capacity for additional green roof?

How could the green roof be incorporated into the LEED certification aspect?

What type of green roof should be implemented for the building?

What growing media and plants will be used?

How does the additional green roof comply with the roof's slope and drainage system?

How will the accessibility and maintenance be integrated with the design?

What effect will the green roof bring to energy usage and storm water management?

How much will it cost to install and perform maintenance for the green roof?

- **Methods**

The existing building's structural capacity will be analyzed to ensure additional installation of a new green roof is feasible when including weights of green roof components such as insulation, waterproofing membrane, growing media, fully saturated soil, mature plant, and other landscaping items. In order to be certified for the Leadership in Energy and Environmental Design (LEED) rating system, the design has to meet several requirements under various categories. Points will be accumulated as much as possible to reach LEED rating scale. The types of green roof, growing plants will be chosen depending on the local climate, geographical location, and intended use of the building. Also, there will be an evaluation of the effects the green roof will bring to the building, based on information found doing research. The effect on green roof over the energy usage will be measured by comparing thermodynamic properties of the roof components, as well as the amount of heat transfer between the building interior and exterior of the building. The drainage system will be designed after reviewing the Urban

Drainage and Flood Control District's guidance in Volume 3 of the Urban Storm Drainage Criteria Manual, and data from the Water Capture Quality of the EPA. Occupational Safety and Health Administration (OSHA) 29 CFR 1910 Subpart D and 29 CFR 1926 Subpart M will be used as a guide for accessibility and safety requirements. Also, based on the slope of the roof, an appropriate growing media and strategies for weight distributing will be developed. Furthermore, an estimated cost will be calculated for the design.

Table 4: Compilation of Various Manuals and Regulations

Subject	Preference source	Section
LEED Certification	LEED Green Building Design and Certification 2009	• Certification Application • Certification Strategy
		Sustainable site: • Prerequisite 1 • site selection, • Development density and Community Connectivity • Protect or restore habitat, • Storm water design • Heat Island effect-Roof
		Energy and Atmosphere
		Regional Priority
Drainage System	Urban Storm Drainage Criteria Manual-Volume 3	Urban Drainage and Flood Control District's guidance
	EPA	Water Capture Quality
Accessibility and safety	Occupational Safety and Health Administration (OSHA)	29 CFR 1910 Subpart D
		29 CFR 1910 Subpart D
Green Roof Design Strategies	EPA	Heat Island Mitigation-Green Roofs Chapter
	Climate Protection Partnership Division in the U.S. EPA	Green Roof Compendium
Thermal Performance of Green Roof	National Research Council of Canada	Report No. NRCC-46412
Estimate the impacts of green roof on energy	Heatislandmitigationtool.com	Mitigation Impact Screening Tool (MIST)
Budget/Cost estimate	Green Roofs for Healthy Cities Green roof Design 101 Introductory Course Participant Manual	Cost estimate and budget

- **Deliverables**

All sustainable design deliverables can be found on Table 6 under the Green Roof Design category.

Cost Analysis and 3D Modeling

Cost estimation of a construction project is an important task in the management of construction projects. The quality of construction management depends on accurate estimation of the construction cost. While steel cost per square foot estimation used as base in making decisions on structural scheme selection, an overall cost estimation will be introduced to the owner for a better understanding of financial needs. For this section, an overall cost is analyzed for the two structural design scenarios selected during the scenario design analysis. After comparing the overall cost for the two design scenarios, a design scenario with lower overall cost will be selected for the breakdown cost using CSI Uniformat II. The following questions address ways to determine the overall cost. Answers will be obtained for these questions and provides owner with better understanding of the total investment needs.

- **Questions**

Which of the two selected system designs will have the lowest approximated overall building cost per square foot by using RS Means Square Foot Cost Data Manual, 2009?

Would the system with lower steel cost per square foot have a lower overall cost per square foot?

How can the relationship between the cost and performance for the design systems be investigated?

Does high cost guarantee high performance?

- **Methods**

The Square Foot and Cubic Foot Estimation will first be constructed using *RS Means Square Foot Cost Data Manual, 2011*. This method of calculation will allow the group to develop a schematic design and generate a cost estimate based on the building's size and use. The *RS Means Square Foot Cost Data Manual, 2011* provides cost per square foot values for certain buildings based on past construction projects. Since the building will contain lab and office spaces, individual data for the two different building occupancies is provided in the manual. References are provided in the manual. In the *Foot Cost Data Manual, 2011*, Page 177 provides model costs for a three-story office building, and page 111 provides model costs for a two-story laboratory building. By figuring out the total office and lab space, the overall building cost will be obtained using the shown on Figure 4.

Figure 4: Overall Cost Using Square Foot and Cubic Foot Estimation

$$Total\ Cost\ (\$) = \left(Office\ Cost\ \left(\frac{\$}{sf}\right) * Office\ Area + Lab\ Cost\ \left(\frac{\$}{sf}\right) * lab\ area\right) * 1.7$$

** 1.7 is the Worcester location factor*

To have a better understanding of which elements account for the most money in the construction cost, another method of cost estimation using CSI Uniformat II will be constructed following the preliminary building cost estimate generated using the Square Foot and Cubic Estimation. CSI Uniformat II classifies construction cost into seven categories: Substructure, Shell, Interiors, Services, Equipment & Furnishings, Special Construction and Demolition, and Site work. Each of the categories will account for certain percentage of the overall cost. Table 5 shows the breakdown of Uniform II for each of the two respective building examined in the RS Means. With the limited information on the scope of the project, Site work and Special Construction will not have an impact on the Uniformat II cost.

Table 5: Uniformat II Cost Distribution

CSI UNIFORMAT	College Lab	Office
Substructure	**11.30%**	**4.40%**
Shell	**18.70%**	**29.60%**
Superstructure	6.70%	12.20%
Exterior Enclosure	7.70%	15.80%
Roofing	4.30%	1.60%
Interior	**23.30%**	**22.70%**
Services	**45.60%**	**43.30%**
Conveying	0.00%	8.90%
Plumbing	17.10%	2.80%
HVAC	14.50%	11.80%
Fire Protection	1.90%	2.80%
Electrical	12.10%	17.00%
Equipment & Furnishings	**1.10%**	**0%**
Special Construction	**0%**	**0%**
Site Construction	**0%**	**0%**

Associated category cost is obtained by multiplying by each percentage and developed overall cost using S Square Foot and Cubic Foot Estimation.

A 3-D model for the structural scheme with the lowest overall cost will be created using *AutoCAD Revit Structure* software. This *Revit* building model will provide both physical and analytical representation of the building. All structural components: footing, columns, slabs, beams, girder and brace will be implemented in the drawing in the order presented. Architectural components will be added to the drawing to ensure structural components are placed on the right location that will provide support to building elements while not blocking the living spaces. The completed Revit drawing will be imported to *Autodesk Robot* for further structural analysis. The results will be presented on bars in diagram form. However, few experimental trials of smaller projects are recommended to get familiar with this new software.

- **Deliverables**

All deliverables for Cost Analysis and 3-D Modeling can be found in Table 6 under Cost Analysis and 3-D Modeling categories.

Deliverables

Table 6: Project Deliverables

Topic Areas	Deliverables
Architectural	Autocad Drawings of the floor plan
Structural Elements Comparisons	Floor plan with column locations
	Composite vs. Non-composite structural elements
	Bay sizes
	Beam Spacing
	Shored vs. Unshored construction
	Design with heavier loading exceeding the required loading
	Cost
Structural Calculations	Compare different design loads
	Partial Composite beams and girders
	Non-composite beams and girders
	Non-Composite columns
	Shored construction
	Cost
Fire Safety Code Analysis	Requirements for non-sprinklered building
	Requirements for sprinklered building
Sprinkler System Design Analysis Summary	Sprinkler System layout (containing underground piping, cross mains, branch lines, risers and sprinkler location)
	Sizes and materials
	Sprinkler locations
	Sprinklers and components manufacture data sheets
	Sprinkler system hydraulic calculations
	Cost
Green Roof Design	Alternative green roof designs
	Analyzed energy usage impacts
	LEED certified points
	Cost estimation of the green roof system
Cost Analysis	Overall Cost using Square Foot Cost
3D Modeling	Breakdown Cost Using CSI Uniformat II
	Autodesk Revit 3D structural modeling
	AutoDesk *Robot* structural analysis

Project Schedule

Tasks	B-term (in Weeks)							B-term (in Weeks)						
Steel Design (all)	8	9	10	11	12	13	14	15	16	17	18	19	20	21
Composite beam /girder system calculation														
Non Composite beam/girder system calculation														
Non-composite column design														
Steel cost estimate and system selection														
Design member analysis using different load scenarios														
Steel cost estimate and system selection														
Shored vs. unshored construction for selected system														
Final structural layout														
Braced Frame Analysis														
Rigid Frame Analysis														
Writing														
Environmental (LH)														
Identify type of green roof-define the using purpose														
Analyzing structural load capacity														
Design Green Roof														
Explore the effect of green roof														
LEED certification														
Cost estimate for the system														
Fire Protection (JK)														
Define occupancy, construction type, commodities classification, write up of code analysis														
Look up and select piping material, sprinkler systems														
Make layout of floor with sprinklers														
Look at cost of installing sprinkler system and compare														
Cost Analysis and 3D Modeling (YD)														
Square Foot Cost Estimation														
CSI Uniformat Cost estimation														
3D Modeling using Revit														
Structural Analysis using Autodesk Robot														
Writing														
Poster														
Finalize Report														

Conclusion

By performing this Major Qualifying Project, each of the group members will apply knowledge into real world applications. Working on areas of structural design, fire protection design, green roof design and LEED accreditation along with cost estimation will provide an invaluable experience. Not only will these processes require applying previously learned knowledge, it will also require additional researching, learning and implementing new materials to execute the project. Each team member will effectively practice collaboration and communication in a teamwork environment.

Appendix B: Structural Calculation Worksheets

B.1 Design Loads

The design loads for both composite and non-composite systems same same. All of the design load values were obtained from *International Building Code 2009 and Minimum Design Load for Building and Other Structure.* Table below identity all the required design load values that are appropriate for this project.

Loads	**Loading specification**	**Dead load lb/ft^2**
Live loads	Office buildings	50
	Laboratory	100
Dead loads	5” Concrete slab and 2” metal decking (includes ponding)	63
	Mechanical, Electrical & Plumbing (MEP)	5
	Ceiling Construction	2
	Structural Steel framing at each level floor	8
	Exterior walls (exterior beam)	48
Roof or Snow loads	Snow load	50

The service load combinations were obtained using following equation:

1. Wu=1.2 DL + 1.6 LL;
2. Wuroof=1.2 DL + 1.6 SL

B.2 Non-Composite Beam Design

Non-Composite Beam Design for 5 Feet Spacing (30x30 bay size)

$LL := \frac{50 \cdot 5}{1000} = 0.25$ k/ft $DL := \frac{[63 + (5 + 2) + 8] \cdot 5}{1000} = 0.39$ k/ft $SN := \frac{45 \cdot 5}{1000} = 0.225$ k/ft

$W_u := 1.2DL + 1.6LL = 0.868$ k/ft $W := DL + LL = 0.64$ k/ft

$M_u := \frac{W_u \cdot 30^2}{8} = 97.65$ ft-k $M := \frac{W \cdot 30^2}{8} = 72$ k/ft

Values from Table 1-1 and Table 3-2 for Beam W16x36

$A := 10.6$ in^2 $d := 15.7$ in $I_x := 448$ in^4

$L_p := 3.00$ ft $t_w := 0.295$ in $E := 29000$ ksi

$L_r := 9.17$ ft $k_{des} := 0.832$ in $F_y := 50$ ksi

$L := 30$ ft $Z_x := 64.0$ in^3

Moment Check for Zone 1 (Lr < Lp) $\phi_b := 0.9$

$W_{ubeam} := 1.2 \cdot (DL + 0.036) + 1.6 \cdot LL = 0.911$

$M_{ubeam} := \frac{W_{ubeam} \cdot L^2}{8} = 102.51$ ft-k $M_{px} := \frac{F_y \cdot Z_x}{12} = 266.667$ ft-k

$\phi_b \cdot M_{px} = 240$ ft-k

φMubeam < Mp, OK

Shear Stress Check

$$h := d - 2 \cdot k_{des} = 14.036$$

$$\frac{h}{t_w} = 47.58 \quad < \quad 2.24\left(\frac{E}{F_y}\right)^{\frac{1}{2}} = 53.946$$

$$C_v := 1.0$$

$$V_n := 0.6 \cdot F_y \cdot d \cdot t_w \cdot C_v = 138.94 \text{ k}$$

$$V_u := \frac{L}{2} \cdot W_u = 13.02 \quad \text{k}$$

Vu < Vn, OK

Deflection Check

$$\Delta := \frac{M \cdot L^2}{161 \cdot I_x} = 0.898 \quad \text{in}$$

The deflection of the dead and live load is smaller than 1 inch so OK

Non-Composite Girder Design for 5 Feet of Spacing (30' x 30' bay size)

$LL := \frac{50 \cdot 5}{1000} = 0.25$ k/ft $DL := \frac{[63 + (5 + 2) + 8] \cdot 5 + 36}{1000} = 0.426$ k/ft $SN := \frac{45 \cdot 5}{1000} = 0.225$ k/ft

$W_u := 1.2DL + 1.6LL = 0.911$ k/ft $P := 30 \cdot (DL + LL) = 20.28$ k

$P_u := W_u \cdot 30 = 27.336$ k $M := P \cdot 5 + P \cdot 10 + P \cdot 15 - P \cdot \frac{15}{2} = 456.3$ ft-k

$M_u := P_u \cdot 5 + P_u \cdot 10 + P_u \cdot 15 - P_u \cdot \frac{15}{2} = 615.06$ ft-k

Values from Table 1-1 and Table 3-2 for Beam W27x84

$A := 24.8$ in^2	$d := 26.7$ in	$I_x := 2850$ in^4
$L_p := 7.31$ ft	$t_w := 0.285$ in	$E := 29000$ ksi
$L_r := 20.8$ ft	$k_{des} := 1.24$ in	$F_y := 50$ ksi
$L := 30$ ft	$Z_x := 244$ in^3	

Moment Check for Zone 1 (Lr < Lp) $\phi_b := 0.9$

$W_{ubeam} := 1.2 \cdot (DL + 0.034) + 1.6 \cdot LL = 0.952$

$P_{ubeam} := W_{ubeam} \cdot 30 = 28.56$ k

$M_{ubeam} := P_{ubeam} \cdot 5 + P_{ubeam} \cdot 10 + P_{ubeam} \cdot 15 - P_{ubeam} \cdot \frac{15}{2} = 642.6$ ft-k

$M_{px} := \frac{F_y \cdot Z_x}{12} = 1.017 \times 10^3$ ft-k

$\phi_b \cdot M_{px} = 915$ ft-k

ϕMubeam < Mp, OK

Shear Stress Check

$h := d - 2 \cdot k_{des} = 24.22$

$\frac{h}{t_w} = 84.982 \quad < \quad 2.24\left(\frac{E}{F_y}\right)^{\frac{1}{2}} = 53.946$

$C_v := 1.0$

$V_n := 0.6 \cdot F_y \cdot d \cdot t_w \cdot C_v = 228.28$ k

$V_u := \frac{L}{2} \cdot W_u = 13.668$ k

Vu < Vn, OK

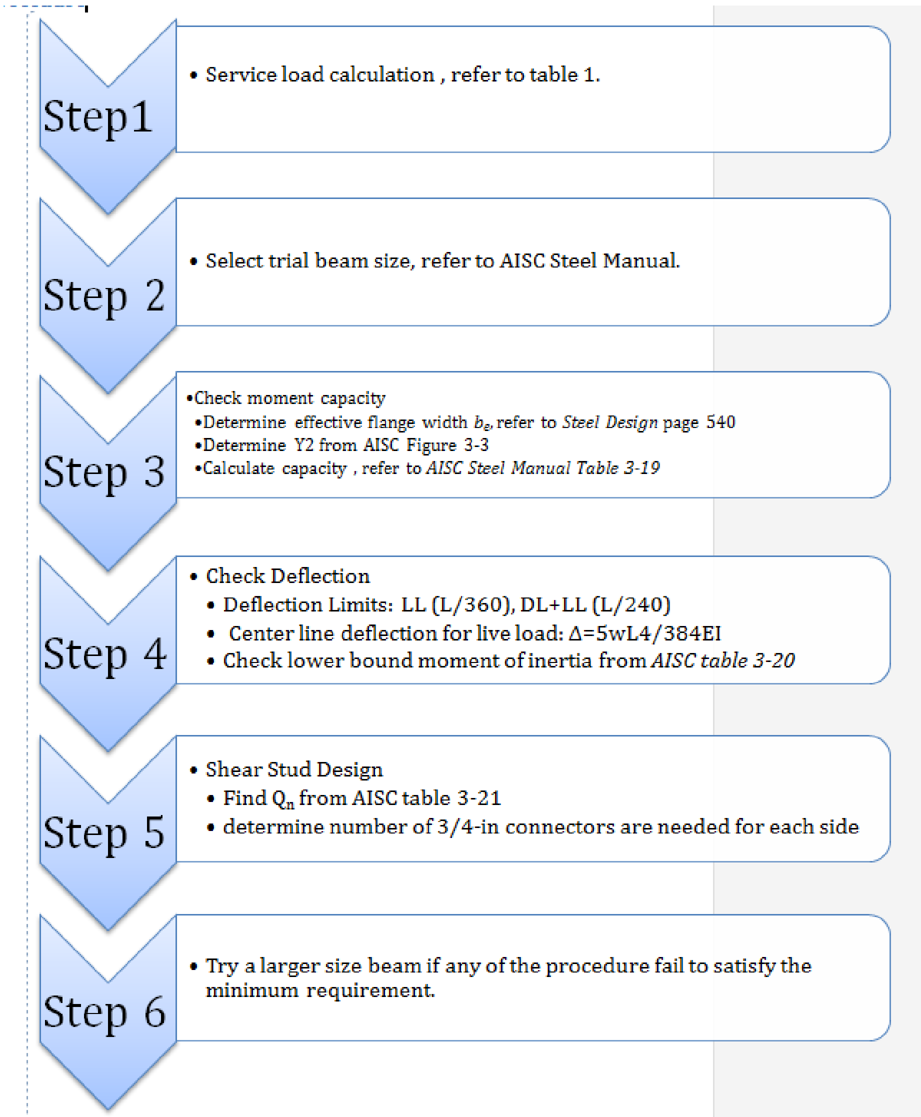
Deflection Check

$\Delta := \frac{M \cdot L^2}{161 \cdot I_x} = 0.895$ in

The deflection of the dead and live load is smaller than 1 inch so OK

B.3 Composite Beam and Girder Design Calculation

B.3.1 Step By step Procedure



B.3.2 Mathcad Calculation Sheets

Composite Interior Beam Design for 5' spacing (30x30 Bay Size)'

$L := 30$ ft $F_y := 50$ ksi $E := 29000$ ksi

S=5 ft $f_c := 4$ ksi

$LL := \frac{50 \cdot 5}{1000} = 0.25$ k/ft $DL := \frac{[63 + (5 + 2) + 8] \cdot 5}{1000} = 0.39$ k/ft

$W_u := 1.2 DL + 1.6 LL = 0.868$ k/ft

$M_u := \frac{W_u \cdot L^2}{8} = 97.65$ ft-k

$b_{e1} := 2 \cdot \frac{L}{8} \cdot 12 = 90$ in $b_{e2} := 2 \cdot \frac{S}{2} \cdot 12 = 60$ in

The following variables were assumed:

$y_{con1} := 7$ in $a := 2$ in

$y_1 := 0$ in $y_2 := y_{con1} - \frac{a}{2} = 6$ in

The following values were obtained from Table 1-1 and 3-2 for beam size W12x26

$I_x := 204$ in^4

$A := 7.65$ in^2

Deflection Check during construction:

$W_{beam} := 26$ plf

$DL_{wetconcrete} := \left(\frac{5}{12} \cdot 145 S\right) \cdot 1.1 = 332.292$ $\frac{lbs}{ft}$

$LL := 20 \cdot 8 = 100$ $\frac{lbs}{ft}$

$Wu_{Unfactored} := \frac{(DL_{wetconcrete} + LL) + W_{beam}}{1000} = 0.458$ $\frac{k}{ft}$

$M_{deflec} := \frac{Wu_{Unfactored} \cdot L^2}{8} = 51.558$

$\Delta_{DL} := \frac{5 \cdot Wu_{Unfactored} \cdot L^4}{384 E \cdot I_x} \cdot 1728 = 1.412$ 37 in

Composite Interior Girder Design for 5'spacing (30'x30 Bay Size)

$L := 30$ ft $\quad F_y := 50$ ksi $\quad E := 29000$ ksi

$S := 5$ ft $\quad f_c := 4$ ksi $\quad W_{beam} := 26$ plf

$LL := \dfrac{50 \cdot 5}{1000} = 0.25$ k/ft $\quad DL := \dfrac{[63 + (5 + 2) + 8] \cdot 5}{1000} + \dfrac{W_{beam}}{1000} = 0.416$ k/ft

$W_u := 1.2 DL + 1.6 LL = 0.899$ k/ft

$$M_u := \frac{W_u \cdot L^2}{8} = 101.16 \quad \text{ft-k}$$

$b_{e1} := 2 \cdot \dfrac{L}{8} \cdot 12 = 90$ in $\quad b_{e2} := 2 \cdot \dfrac{L}{2} \cdot 12 = 360$ in

The following variables were assumed:

$y_{con1} := 7$ in $\quad a := 2$ in

$y_1 := 0$ in $\quad y_2 := y_{con1} - \dfrac{a}{2} = 6$ in

in^4 in^2

The following values were obtained from Table 1-1 for beam size W21x55

$I_x := 1140 \quad A := 16.2$

Deflection Check during construction:

$W_{beam} := 55$ plf

$$DL_{wetconcrete} := \left(\frac{5}{12} \cdot 145 L\right) \cdot 1.1 = 1.994 \times 10^3 \quad \frac{lbs}{ft}$$

$$LL := 20 \cdot L = 600 \quad \frac{lbs}{ft}$$

$$Wu_{Unfactored} := \frac{(DL_{wetconcrete} + LL) + W_{beam}}{1000} = 2.649 \quad \frac{k}{ft}$$

$$M_{deflec} := \frac{Wu_{Unfactored} \cdot L^2}{8} = 297.984 \quad \text{ft} - \text{k}$$

$$\Delta_{DL} := \frac{5 \cdot Wu_{Unfactored} \cdot L^4}{384 E \cdot I_x} \cdot 1728 = 1.46 \quad \text{in}$$

30

$$\Delta_{limit} := \frac{(L \cdot 12)}{240} = 1.5 \quad \text{in} \qquad \Delta_{limit} > \Delta_{DI} \qquad \text{Pass}$$

Check Strength

Assume Partial Composite Y1=6 to reduce number of studs.

Sum Q.n=292 From Table4-19

Required 'a':

$$a_{req} := \frac{292}{0.85 f_c \cdot b_{e1}} = 0.954 \quad \text{in}$$

$$y_{2req} := y_{con1} - \frac{a_{req}}{2} = 6.523 \quad \text{in}$$

ϕbMn Interpolation from Table 3-19

$$y := 6.5$$

$$M_{p1} := 756$$

$$M_{p2} := 767$$

$$M_p := M_{p1} + \frac{(y_{2req} - y)}{0.5}(M_{p2} - M_{p1}) = 756.503 \quad \text{ft-k}$$

Mu < Mp, moment check is satisfied

Shear Stress Check (Table 3-6)

$$V_u := \frac{L \cdot W_u}{2} = 13.488 \quad \text{k} \qquad \phi V_n := 234 \quad \text{k}$$

Vu < ϕ Vn, stress check is ok.

Required Number of 3/4" Studs

$$Q_{nstud} := 21.2$$

$$N_{studs} := \frac{292.2}{Q_{nstud}} = 27.547$$

28 studs required.

$$\text{Spacing} := \frac{12 \cdot L}{28} = 12.857 \quad \text{in}$$

Interior Composite Girder Design for 6' spacing (30x30 Bay Size)'

$L := 30$ ft $F_y := 50$ ksi $E := 29000$ ksi

$S := 30$ ft $f_c := 4$ ksi

$LL := \frac{100 \cdot 6}{1000} = 0.6$ k/ft $DL := \frac{78 \cdot 6 + 26}{1000} = 0.494$ k/ft

$W_u := 1.2 \cdot DL + 1.6 \cdot LL = 1.553$ k/ft

$P_u := W_u \cdot S = 46.584$ k/ft

$M_u := P_u \cdot 36 - P_u \cdot 12 - P_u \cdot 6 = 838.512$ ft-k

$b_{e1} := 2 \cdot \frac{L}{8} \cdot 12 = 90$ in

$b_{e2} := 2 \cdot \frac{S}{2} \cdot 12 = 360$ in

The following variables were assumed:

$y_{con1} := 7$ in

$a := 1$ in

$y_1 := 0.392$

$y_2 := 7 - \frac{a}{2} = 6.5$ in **beam size W12x19**

in^4 $A := 5.57$ in^2

The following values were obtaineed from Table 1-1 and 3-2 for beam size W21x55

$I_x := 1140$ in^4 k $\Sigma Q_n := 69.1$ $A := 16.2$ in^2

$\Sigma Q_n := 488$ k

Required 'a':

$$a_{req} := \frac{\Sigma Q_n}{0.85 f_c \cdot b_{e1}} = 1.595 \qquad \text{in}$$

$$y_{2req} := y_{con1} - \frac{a_{req}}{2} = 6.203 \qquad \text{in}$$

ϕ bMn Interpolation from Table 3-19

$y := 6.0$

$M_{p1} := 865$

$M_{p2} := 884$

$$M_p := M_{p1} + \frac{\left(y_{2req} - y\right)}{0.5}\left(M_{p2} - M_{p1}\right) = 872.699 \qquad \text{ft-k} \qquad Mp > Mu=838.5 \text{ ft-k Ok}$$

Deflection Check

$$C_{LL} := \frac{20 \cdot S}{1000} = 0.6 \qquad WC := \frac{145 \cdot \frac{5}{12} \cdot S \cdot 1.1}{1000} = 1.994$$

$$W := C_{LL} + WC + .055 = 2.649$$

$$\Delta_C := 1728 \cdot \frac{5 \cdot W \cdot L^4}{384 \cdot E \cdot I_x} = 1.46 \qquad \text{in}$$

Flange Local Buckling Check

$$\frac{b_f}{2t_f} := 7.87 \quad < \quad 0.38 \cdot \left(\frac{E}{F_y}\right)^{\frac{1}{2}} = 9.152 \qquad \text{OK}$$

Web Local Buckling Check

$$\frac{h}{t_w} := 50.0 \quad < \quad 3.768 \left(\frac{E}{F_y}\right)^{\frac{1}{2}} = 90.745 \qquad \text{OK}$$

Shear Stress Check

$V_u := \frac{L \cdot W_u}{2} = 23.292$ k $Vu < \phi V_n = 234$ OK

Stud Designs using table 3-21

$Q_n := 21.5$ K

$N_{stud} := \frac{\Sigma Q_n \cdot 2}{Q_n} = 45.395$ 46 3/4" Studs are required

$Spacing := \frac{12 \cdot L}{46} = 7.826$ in

Excel Spread Sheets

Composite Design: Interior Beams for Bay Size 30'x30'

Composite Interior Beam Design for Bay Size: 30'x30'			
Spacing	5	6	10
DL	78.00	78.00	78.00
LL	50.00	50.00	50.00
Span	30.00	30.00	30.00
Wu	0.17	0.17	0.17
Pu	0.87	1.04	1.74
Mu	97.65	117.18	195.30
be1	90.00	90.00	90.00
be2	60.00	72.00	120.00
Selected W Section			
Ycon	7.00	7.00	7.00
Assume a	2.00	2.00	2.00
y1	0.00	0.00	0.00
Y2	6.00	6.00	6.00
Trial Beam Size	W12x26	W14x26	W14x38
Area	7.65	7.69	11.20
Ix	204.00	245.00	385.00
Deflection During Construction			

DLwetconcrete	332.29	398.75	664.58
LL	100.00	120.00	200.00
Wbeam	26.00	26.00	38.00
Unfactored Wu	0.46	0.54	0.90
Mu	51.56	61.28	101.54
E	29000.00	29000.00	29000.00
Defle	1.41	1.40	1.47
Defle Limit	1.50	1.50	1.50
Check Strength			
Sum Qn	95.60	96.10	140.00
fc	4.00	4.00	4.00
be(lower)	60.00	72.00	60.00
Areq	0.47	0.39	0.69
Y2`	6.77	6.80	6.66
Interpolation			
Patial-Com Y1=	7.00	7.00	7.00
Y2	6.50	6.50	6.00
StrengthLower	215.00	234.00	349.00
Strengthupper	259.00	238.00	354.00
Mu	238.38	236.43	355.57
φ	0.85	0.85	0.85
φMu	202.62	200.97	302.23
Shear Check			
Check Shear	13.02	15.62	26.04
Shear Limit (Table 3.2)	84.30	106.00	131.00
Total Number of Studs Required			
Qnu	17.20	17.20	17.20
# of stud	11.12	11.17	16.28
even # of stud	12.00	12.00	18.00
spacing	30.00	30.00	20.00

Composite Design: Interior Girders for Bay Size 30’x30’

Composite Interior Girder Design for Bay Size: 30'x30'			
Beam Weight	26	26	38
Spacing	5	6	10
DL	0.416	0.494	0.818
LL	0.25	0.3	0.5
Span	30	30	30
Wu	0.8992	1.0728	1.7816
Pu	26.976	32.184	53.448
Mu	606.96	579.312	534.48
be1	90	90	90
be2	360	360	360
Select W Section			
Ycon	7	7	7
Assume a	2	2	2
y1	0	0	0
Y2	6	6	6
Trial Girder Size	W21x55	W21x55	W21x55
Area	16.2	16.2	16.2
Ix	1140	1140	1140
Deflection During Construction			
DLwetconcrete	1993.75	1993.75	1993.75
LL	600	600	600
Wbeam	55	55	55
Unfactored Wu	2.64875	2.64875	2.64875
Mu	59.596875	47.6775	26.4875
E	29000	29000	29000
Defle	1.460177518	1.460177518	1.460177518
Defle Limit	1.5	1.5	1.5
Check Strength			
Sum Qn	292	203	203
fc	4	4	4
be(lower)	90	90	90
Areq	0.954248366	0.663398693	0.663398693
Y2`	6.522875817	6.668300654	6.668300654
Interpolation			
Patial-Com Y1=	6	7	7
Y2	6.5	6.5	6.5
StrengthLower	756	687	687
Strengthupper	767	695	695

Mu	756.503268	689.6928105	689.6928105
φ	0.85	0.85	0.85
φMu	643.0277778	586.2388889	586.2388889
Shear Check			
Check Shear	13.488	16.092	26.724
Shear Limit (3-2)	234	234	234
Total Number of Studs Required			
Qnu	21.2	21.2	21.2
# of stud	27.54716981	19.1509434	19.1509434
even # of stud	28	20	20
spacing	12.85714286	18	18

B.4 Non-Composite Column Calculation

Corner Column calculation 30'x30' bay size

$L := 30$ ft

Load combination 1.2D+1.6L+0.5S

$DL := 78$ psf $\quad LL := 100$ psf

$A_{tributary} := 15 \cdot 15 = 225$ ft^2

$W_{girder} := 94 \cdot L \cdot 2 = 5.64 \times 10^3$ lbs

$D_{TL} := 4 \cdot \left[\left(DL \cdot A_{tributary} \right) + W_{girder} \right] = 9.276 \times 10^4$ lbs

$L_{TL} := 3 \cdot \left(LL \cdot A_{tributary} \right) = 6.75 \times 10^4$ lbs

$S_{TL} := 1 \cdot \left(S \cdot A_{tributary} \right) = 1.012 \times 10^4$ lbs

$$P_u := \frac{\left(1.2 D_{TL} + 1.6 L_{TL} + 0.5 S_{TL}\right)}{1000} = 224.375 \quad k$$

Assume: $\frac{KL}{r} := 50$ $\quad \phi F_{cr} := 37.5$ ksi from table 4.22

$$A_{required} := \frac{P_u}{\phi F_{cr}} = 5.983 \quad in^2$$

Try W14x74

$A_{beam} := 21.8$ in^2 $\quad r_x := 6.04$ in $\quad r_y := 2.48$ in

Use $K := 1$

$$\frac{KL}{r_x} := \frac{K \cdot L \cdot 12}{r_x} = 59.603 \qquad \frac{KL}{r_y} := \frac{K \cdot L \cdot 12}{r_y} = 145.161$$

Use $\frac{KL}{r_y}$ and table 4.22

$x := 145.161 - 145 = 0.161$

$\phi F_{cr1} := 10.7$

$\phi F_{cr2} := 10.6$

$$\phi F_{crf} := \phi F_{cr1} - \frac{x}{1.0} \cdot \left(\phi F_{cr1} - \phi F_{cr2} \right) = 10.684$$

$\phi P_n := \phi F_{crf} \cdot A_{beam} = 232.909$ $\quad \phi Pn > Pu$ OK

Exterior Column calculation 30'x30' bay size

$L := 30$ ft

Load combination 1.2D+1.6L+0.5S

$DL := 78$ psf $\quad LL := 100$ psf

$A_{bay} := 15 \cdot 30 = 450$ ft^2

$W_{girder} := 94 \cdot L \cdot 2 = 5.64 \times 10^3$ lbs

$D_{TL} := 4 \cdot \left[\left(DL \cdot A_{bay}\right) + W_{girder}\right] = 1.63 \times 10^5$ lbs

$L_{TL} := 3 \cdot \left(LL \cdot A_{bay}\right) = 1.35 \times 10^5$ lbs

$S_{TL} := 1 \cdot \left(S \cdot A_{bay}\right) = 2.025 \times 10^4$ lbs

$$P_u := \frac{\left(1.2 D_{TL} + 1.6 L_{TL} + 0.5 S_{TL}\right)}{1000} = 421.677$$ k

Assume: $\frac{KL}{r} := 50$ $\quad \phi F_{cr} := 37.5$ ksi from table 4.22

$$A_{required} := \frac{P_u}{\phi F_{cr}} = 11.245$$ in^2

Try W14x90

$A_{beam} := 26.5$ in^2 $\quad r_x := 6.14$ in $\quad r_y := 3.70$ in

Use $K := 1$

$$\frac{KL}{r_x} := \frac{K \cdot L \cdot 12}{r_x} = 58.632 \qquad \frac{KL}{r_y} := \frac{K \cdot L \cdot 12}{r_y} = 97.297$$

Use $\frac{KL}{r_y}$ and table 4.22

$x := 97.297 - 97 = 0.297$

$\phi F_{cr1} := 22.6$

$\phi F_{cr2} := 22.3$

$$\phi F_{crf} := \phi F_{cr1} - \frac{x}{1.0} \cdot \left(\phi F_{cr1} - \phi F_{cr2}\right) = 22.511$$

$\phi P_n := \phi F_{crf} \cdot A_{beam} = 596.539$ $\quad \phi$ Pn > Pu OK

B.5 Structural Design with 100psf design Live load

100psf Exterior Composite Beam Design for 6' spacing (30'x30' Bay Size)'

$L := 30$ ft $F_y := 50$ ksi $E := 29000$ ksi

$S := 3$ ft $f_c := 4$ ksi

$$LL := \frac{100 \cdot S}{1000} = 0.3$$

$$DL := \frac{(78 + 48) \cdot S}{1000} = 0.378$$

$W_u := 1.2 DL + 1.6 LL = 0.934$ k/ft^2

$M_u := \frac{W_u \cdot 30^2}{8} = 105.03$ ft-k

$b_{e1} := 2 \cdot \frac{L}{8} \cdot 12 = 90$ in

$b_{e2} := 2 \cdot \frac{S}{2} \cdot 12 = 36$ in

The following variables were assumed:

$y_{con1} := 7$ in

$a := 1$ in

$y_1 := 3.12$

$y_2 := 7 - \frac{a}{2} = 6.5$ in

The following values were obtaineed from Table 1-1 and 3-2 for beam size W12x19

$I_x := 120$ in^4 $A_s := 5.57$ in^2

$\Sigma Q_n := 69.1$ k

Required 'a':

$$a_{req} := \frac{\Sigma Q_n}{0.85 f_c \cdot b_{e2}} = 0.565 \quad in$$

$$y_{2req} := y_{con1} - \frac{a_{req}}{2} = 6.718 \quad in$$

ϕ bMn Interpolation from Table 3-19

$y := 6.5$

$M_{p1} := 151$

$M_{p2} := 153$

$$M_p := M_{p1} + \frac{(y_{2req} - y)}{0.5}(M_{p2} - M_{p1}) = 151.871 \quad \text{ft-k} \qquad Mp > Mu=105.03 \text{ ft-k Ok}$$

Deflection Check

Deflection during construction:

$$C_{LL} := \frac{20 \cdot S}{1000} = 0.06 \quad \text{k/ft^2} \qquad WC := \frac{145 \cdot \frac{5}{12} \cdot S \cdot 1.1}{1000} = 0.199$$

$$W := C_{LL} + WC + .026 = 0.285$$

$$\Delta_C := 1728 \frac{5 \cdot W \cdot L^4}{384 E \cdot I_x} = 1.495 \quad in$$

Flange Local Buckling Check

$$\frac{b_f}{2t_f} := 5.98 \quad < \quad 0.38 \left(\frac{E}{F_y}\right)^{\frac{1}{2}} = 9.152 \qquad \text{OK}$$

Web Local Buckling Check

$$\frac{h}{t_w} := 48.1 \quad < \quad 3.76 \left(\frac{E}{F_y}\right)^{\frac{1}{2}} = 90.745 \qquad \text{Ok}$$

Shear Stress Check Using Table 3-6

$V_u := \frac{L \cdot W_u}{2} = 14.004$ k   Vu< ϕV_n=72.8 OK

Stud Designs using table 3-21

$Q_n := 17.2$ K

$N_{stud} := \frac{\Sigma Q_n \cdot 2}{17.2} = 8.035$ 10 3/4" Studs are required

$Spacing := \frac{12 \cdot L}{10} = 36$

Appendix C: Structural Steel Design Summary

Non-Composite Beams					
		Beam size	**Allowable $\phi_b M_{px}$ (K-ft)**	**Actual M_u (K-ft)**	**Deflection (Inches)**
20'x20'					
4'	Interior	W10×15	60.00	35.62	0.923
	Exterior	W10×12	47.25	23.12	0.813
5'	Interior	W10×17	70.12	44.42	0.971
	Exterior	W10×15	60.00	28.90	0.793
10'	Interior	W10×30	137.25	88.6	0.935
	Exterior	W10×22	97.5	57.8	0.926
30'x30'					
5'	Interior	W16×36	240.00	102.51	0.898
	Exterior	W14×30	177.38	65.03	0.951
6'	Interior	W14×48	294	123.66	0.990
	Exterior	W14×34	204.75	78.03	0.977
10'	Interior	W21×44	357.75	201.24	0.955
	Exterior	W16×45	308.63	130.05	0.944

Non-Composite Girders					
		Girder Size	**Allowable $\phi_b M_{px}$ M_p (K-ft)**	**Actual M_u (K-ft)**	**Deflection (Inches)**
20'x20'					
4'	Interior	W14×34	204.75	180.77	0.924
	Exterior	W12×30	161.63	114.43	0.912
5'	Interior	W14×34	204.75	185.84	0.960
	Exterior	W12×30	161.63	119.2	0.950
10'	Interior	W14×34	204.75	181.26	0.957
	Exterior	W12×30	161.63	118.24	0.942
30'x30'					
5'	Interior	W27×84	915.00	642.60	0.895
	Exterior	W24×68	663.75	414.45	0.969
6'	Interior	W24×94	952.5	615.6	0.912
	Exterior	W24×68	663.75	396.58	0.927
10'	Interior	W24×84	840.0	552.48	0.937
	Exterior	W21×68	600.0	363.0	0.970

Composite Beams							
		Beam Size	**# of Studs (3/4")**	**Spacing between studs (in)**	**Allowable φM_p (K-ft)**	**Actual M_u (K-ft)**	**Construction Deflection (in)**
20'x20'							
4'	Interior	W10×12	10	24	83.81	34.7	0.83
	Exterior	W10×12	6	40	70.3	23.1	0.43
5'	Interior	W10×15	8	30	88.8	43.4	0.81
	Exterior	W10×12	6	40	70.3	28.9	0.53
10'	Interior	W12×19	10	24	129.5	86.8	0.84

	Exterior	W10×15	8	30	89	57.8	0.81
30'x30'							
5'	Interior	W12×26	12	30	202.6	97.7	1.41
	Exterior	W12×16	12	30	128.0	65	1.42
6'	Interior	W14×26	12	30	201.0	117.2	1.40
	Exterior	W12×19	10	36	129.6	78.03	1.35
10'	Interior	W14×38	18	20	302.2	195.3	1.47
	Exterior	W14×26	12	30	201.2	130.05	1.41

Composite Girders

		Girder Size	# of Studs (3/4")	Spacing between studs (in)	Allowable ϕM_p (K-ft)	Actual M_u (K-ft)	Construction Deflection (in)
20'x20'							
4'	Interior	W12×30	12	20	213.2	170.1	0.92
	Exterior	W12×19	8	30	152.3	112.7	0.84
5'	Interior	W12×30	12	20	213.2	177.2	0.92
	Exterior	W12×19	8	30	152.3	117.04	0.84
10'	Interior	W12×30	12	20	213.2	175.9	0.92
	Exterior	W12×19	10	24	149.6	116.5	0.84
30'x30'							
5'	Interior	W21×55	28	12.9	643.0	606.96	1.25
	Exterior	W18×40	20	18	414.4	390.7	1.39
6'	Interior	W21×55	20	18	586.2	579.3	1.25
	Exterior	W18×40	20	18	414.4	380.7	1.39
10'	Interior	W21×55	20	18	586.2	534.5	1.25
	Exterior	W18×40	14	25.8	441.6	351.5	1.39

Columns

Bay Size	Location	Column Size	Allowable ϕPn (k)	Actual Pu (k)
20'×20'	Interior	W14×61	265.07	261.29
	Exterior	W14×43	178.88	133.91
	Corner	W14×34	86.18	70.22
30'×30'	Interior	W14×99	657.35	600.28
	Exterior	W12×72	340.04	313.68
	Corner	W14×61	188.06	170.38

# of Structural Members

20'×20'	Beam Location	# of Beams	Girder Location	# of Girders	Column Location	# of Columns
4'	Interior	130	Interior	9	Interior	18
	Exterior	5	Exterior	16	Exterior	6
					Corner	3
5'	Interior	103	Interior	9	Interior	18
	Exterior	5	Exterior	16	Exterior	6
					Corner	3
10'	Interior	49	Interior	9	Interior	18
	Exterior	5	Exterior	16	Exterior	6
					Corner	3

Non-Composite Irregular Bay Structural Members

Bay Size	Spacing	Beam Location	Size	# of Beams	Girder Location	Size	# of Girders	Column Location	Size	# of Columns

20×30	4’	Interior	W12×45	43	Interior	W14×53	18	Interior	W14×99	20
		Exterior	W12×30	2	Exterior			Exterior	W12×72	8
								Corner	W14×61	0
	5’	Interior	W12×53	34	Interior	W14×53	18	Interior	W14×99	20
		Exterior	W12×35	2	Exterior			Exterior	W12×72	8
								Corner	W14×61	0
	10’	Interior	W12×87	16	Interior	W14×53	18	Interior	W14×99	20
		Exterior	W12×72	2	Exterior			Exterior	W12×72	8
								Corner	W14×61	0
30×20	5’	Interior	W10×17	28	Interior	W27×84	3	Interior	W14×99	20
		Exterior			Exterior	W24×68	2	Exterior	W12×72	8
								Corner	W14×61	0
	6’	Interior	W10×19	24	Interior	W24×94	3	Interior	W14×99	20
		Exterior			Exterior	W24×68	2	Exterior	W12×72	8
								Corner	W14×61	0
	10’	Interior	W10×30	16	Interior	W24×84	3	Interior	W14×99	20
		Exterior			Exterior	W21×68	2	Exterior	W12×72	8
								Corner	W14×61	0

Composite Irregular Bay Structural Members

Bay Size	Spacing	Beam Location	Beam Size	# of Studs	# of Beams	Girder Location	Girder Size	# of Studs	# of Girders	Column Location	Column Size	# of Columns
20×30	4’	Interior	W12×22	10	43	Interior	W14×34	12	18	Interior	W14×99	20
		Exterior	W10×17	12	2	Exterior				Exterior	W12×72	8
										Corner	W14×61	0
	5’	Interior	W12×26	12	34	Interior	W14×38	14	18	Interior	W14×99	20
		Exterior	W10×22	10	2	Exterior				Exterior	W12×72	8
										Corner	W14×61	0
	10’	Interior	W12×38	18	16	Interior	W16×31	22	18	Interior	W14×99	20
		Exterior	W12×26	12	2	Exterior				Exterior	W12×72	8
										Corner	W14×61	0
30×20	5’	Interior	W10×12	6	28	Interior	W21×55	28	3	Interior	W14×99	20
		Exterior				Exterior	W18×40	20	2	Exterior	W12×72	8
										Corner	W14×61	0
	6’	Interior	W10×15	8	24	Interior	W21×55	20	3	Interior	W14×99	20
		Exterior				Exterior	W18×40	20	2	Exterior	W12×72	8
										Corner	W14×61	0
	10’	Interior	W12×19	10	16	Interior	W21×55	20	3	Interior	W14×99	20
		Exterior				Exterior	W18×40	14	2	Exterior	W12×72	8
										Corner	W14×61	0

30’×30’	Beam Location	# of Beams	Girder Location	# of Girders	Column Location	# of Columns
5’	Interior	124	Interior	14	Interior	15
	Exterior	5	Exterior	13	Exterior	15
					Corner	3
6’	Interior	103	Interior	14	Interior	15
	Exterior	5	Exterior	13	Exterior	15
					Corner	3
10’	Interior	61	Interior	14	Interior	15
	Exterior	5	Exterior	13	Exterior	15
					Corner	3

Appendix D: Footing Design

D.1 Geotechnical Report Provided by Maguire Group Inc. for Gateway Parking Lot Development

VI. Recommendations and Discussion

The following are our geotechnical recommendations with brief discussion, based upon a review of project borings, laboratory testing of soil, our project research and experience in the local Worcester area. Reference in this report to the State Building Code will refer to the current, 6th Edition of the Massachusetts State Building Code, 780 CMR. Recommendations for and discussion concerning each of the five project design component are presented as stand-alone sections, some repetition of information is to be expected.

Plaza Fill

Reference: Figure 2, Exploration Location Plan, Schematic Subsurface Profile Notes, Figure 3, Plaza Fill Schematic Subsurface Profile, Figure 8, Inferred Ground Water Isopachs, July 2005, Table 3, Summary Soil Testing Program, Table 4, Summary Ground Water Well Monitoring Data, Appendix I Boring Logs and Appendix II Grain Size Data.

- The proposed Plaza area is approximately 1 acre in size. In excess of approximately 60% of the proposed Plaza is located within the "upper," higher elevation site area, which will require mass soil excavation to attain finished grade. The existing Plaza area grade ranges between approximately elevation +491 and +493. Proposed Plaza finished grade ranges between elevation +483 and +488, averaging elevation +485, necessitating between 3 and 10 feet of soil removal.
- Ground water level, July 2005, within the Plaza area ranges between elevation +474 and +475, 17 to 18 feet below existing grade and 9 to 13 feet below Plaza finished grade. Ground water level will not be a factor in the soil removal operation.
- The State Building Code defines a wide range of granular soils that can be utilized as "prepared fill" for structural use. Prepared fill materials are identified, in part, as soil materials contained in Table 1804.3, class 6 through 8, described as glacial till, widely graded sands and gravels to non-plastic silty sands.
- Plaza area soils to be excavated extend between 3 and 10 feet below existing grade and are site specifically described as:
    - Fine to Coarse SAND, trace to little Silt, trace to little gravel, trace cobbles, trace construction material fragments - brick, concrete, wood and asphalt, ranging in Unified Soil Classification System (USCS) group symbols between SM, SP and SW.

D.2 Spread Footing Design Procedure

Step1

- Determine Soil Bearing Capacity From *Massachusetts Building Code*
- F'c=3Ksi, Fy is 50Ksi, ∅=0.85

Step 2

- Solve for required area by (Live load + Dead Load)/Bearing Capacity
- Determine trial size for footing
- Calculate factored net soil pressure q_{nu}=p_u/area of footing

Step 3

- **Determine the footing depth d base Two-Way Shear Check**
  - Calculate critical perimeter $b_{0,}$ b_0=4*(18+d)
  - Calculate shear force $V_{u,}$ V_u= q_{nu} (footing area-effective area)
  - Calculate the Nominal Shear Strength $V_{c,}$ V_c=4*Sqrt(F'c)*Footing Area*d
  - ∅Vc must be greater than V_u to pass the two way shear check

Step 4

- **One- Way Shear Check**
  - Calculate $V_{u2,}$ $V_{u2=}$ q_{nu*}Effective Area
  - Calculate V_{c2}, V_{c2}=2*Sqrt(F'c)*footing width*d
  - V_{c2} must be greater than V_{u2} to pass the one way shear check

Step 5

- **Bending Moment Calculation**
  - **Assume a=2 in**
  - **M_u=** q_{nu}*footing width*(distance from column to the footing^2/2)

Step 6

- **Design Flexure Reinforcement**
  - Calculate require steel area A_s= M_u/.90*Fy*(d-1)
  - Calculate $A_{s,min}$= (3*Sqrt(F'c)/Fy)*footing area*d
  - $A_{s,min2}$=(200/F'c)*footing area*d
  - determine number of Bars

D.3 Spread Footing Excel Sheet

Footing Designs for Bay Size 20'x20'				
	Interior	**Exterior**	**Corner**	
Column Size :	W14x61	W14x43	W14x34	
bf	10.00	8.00	6.75	in
d	14.00	13.70	14.00	in
DL	130.20	67.80	36.64	Kips
LL	60.00	30.00	15.00	Kips
Pu	252.24	129.36	67.97	Kips
Permissible Bearing Capacity	6.00	6.00	6.00	KSF
F'c	3.00	3.00	3.00	Ksi
Fy	50.00	50.00	50.00	ksi
Concrete Unit Weight	145.00	145.00	145.00	pcf
Distance below Grade	5.00	5.00	5.00	ft.
Area Require	31.70	16.30	8.61	ft^2
Typical Footing Size	8'x8'	8'x8'	8'x8'	
Double Shear Check				
Qnu	3.94	2.02	1.06	ksf
d	14.00	10.50	8.00	in
Effective Area	58.56	59.93	60.64	
Vu	230.78	121.14	64.40	Kips
Bo	128.00	114.00	104.00	in
Vc	392.61	262.25	182.28	Kips
φVc	333.72	222.91	154.94	Kips
Single Shear Check				
Vu2	61.80	31.69	16.65	
Vc2	147.23	110.42	84.13	kips
φVc2	125.14	93.86	71.51	kips
Bending Moment				
Mu	2589.87	1328.20	697.86	in-kip

As	4.43	3.11	2.22	in^2
Checking Minimum reinforcement Ratio				
Asmin	4.17	3.12	2.38	in^2
But not less than				
As,min	5.38	5.38	5.38	in^2
Bar No.	#7	#7	#7	
Cross section Area	0.60	0.60	0.60	in^2
Total No. of Bars	9.00	9.00	9.00	
Final d	18.50	15.00	12.50	in
Final Footing Dimension	8'x8'x19"	8'x8'x15"	8'x8'x13"	

Footing Designs for Bay Size 30'x30'				
	Interior	**Exterior**	**Corner**	
Column Size :	W14x99	W12x72	W14x61	
bf	14.20	12.00	10.00	in
d	14.00	12.30	13.90	in
DL	303.40	163.00	92.70	Kips
LL	135.00	67.50	33.75	Kips
Pu	580.08	303.60	165.24	Kips
Permissible Bearing Capacity	6.00	6.00	6.00	KSF
F'c	3.00	3.00	3.00	Ksi
Fy	50.00	50.00	50.00	ksi
Concrete Unit Weight	145.00	145.00	145.00	pcf
Distance below Grade	5.00	5.00	5.00	ft.
Area Require	73.07	38.42	21.08	ft^2
Typical Footing Size	12'X12'	8'x8'	8'x8'	
Double Shear Check				
Qnu	4.03	2.11	1.15	ksf
d	20.00	14.00	10.00	in
Effective Area	135.97	59.20	140.03	
Vu	547.74	124.81	160.69	Kips

Bo	152.00	128.00	112.00	in
Vc	666.03	392.61	245.38	Kips
φVc	566.13	333.72	208.57	Kips
Single Shear Check				
Vu2	151.06	79.06	43.03	
Vc2	315.49	220.84	157.74	kips
φVc2	268.16	187.72	134.08	kips
Bending Moment				
Mu	6963.86	3644.72	1983.71	in-kip
As	8.14	6.23	4.90	in^2
Checking Minimum reinforcement Ratio				
Asmin	8.93	6.25	4.46	in^2
But not less than				
As,min	11.52	8.06	5.76	in^2
Bar No.	#9	#8	#8	
Cross section Area	1.00	0.79	0.79	in^2
Total No. of Bars	12.00	10.00	8.00	
Final d	24.50	18.50	14.50	in
Final Footing Dimension	12'X12'X25"	8'x8'x19"	8'x8'x15"	

Appendix E: Cost Estimate Spread Sheets

E.1 Cost: Non-Composite Beam and Girder

Non-Composite		Beam Size	Total Number of Beams	Total Span Length (ft.)	Cost (plf)	Total Beam Cost$	Girder Size	Total Number of Beams	Total Span Length (ft.)	Cost (plf)	Total Girder Cost $
20’x20’											
4’	Interior	W10×15	130	2600	37	96200	W14×34	9	180	69	12420
	Exterior	W10×12	5	100	32	3200	W12×30	16	320	62	19699
5’	Interior	W10×17	103	2060	41	83863	W14×34	9	180	69	12420
	Exterior	W10×15	5	100	37	3700	W12×30	16	320	66	20979
10’	Interior	W10×30	49	980	65	63700	W14×34	9	180	69	12420
	Exterior	W10×22	5	100	50	5000	W12×30	16	320	66	20979
20’x30’											
4’	Interior	W12x45	43	1290	90	115670	W14x53	18	360	104	37440
	Exterior	W12x30	2	60	62	3693					
5’	Interior	W12x53	34	1020	104	106335	W14×53	18	360	104	37440
	Exterior	W12x35	2	60	71	4260					
10’	Interior	W12x87	16	480	167	80160	W14×53	18	360	104	37440
	Exterior	W12x72	2	60	140	8400					
30'x20'											
5’	Interior	W10x17	28	560	41	22798	W27x84	3	90	159	14310
	Exterior						W24x68	2	60	130	7800
6’	Interior	W10x19	24	480	44	21326	W24x94	3	90	179	16110
	Exterior						W24x68	2	60	130	7800
10’	Interior	W10x30	16	320	65	20800	W24x84	3	90	160	14400
	Exterior						W21x68	2	60	179	10740
30’x30’											
5’	Interior	W16×36	124	3720	73	270332	W27×84	14	420	159	66780
	Exterior	W14×30	5	150	62	9225	W24×68	13	390	130	50700
6’	Interior	W14×48	103	3090	95	292778	W24×94	14	420	179	75180
	Exterior	W14×34	5	150	69	10350	W24×68	13	390	130	50700
10’	Interior	W21×44	61	1830	88	160125	W24×84	14	420	160	67200
	Exterior	W16×45	5	150	89	13388	W21×68	13	390	131	51090

E.2 Cost: Composite Beams

Composite		Beam Size	Total Number of Beams	Total Span Length	Cost (plf)	Total Beam Cost	Total Num. of Studs	Total Stud Cost	Total Cost
20’x20									
4’	Interior	W10×12	246	4920	32	157440	10	55.6	157496
	Exterior	W10×12	9	180	32	5760	6	33.36	5793.36
5’	Interior	W10×15	196	3920	37	145040	8	44.48	145084
	Exterior	W10×12	9	180	32	5760	6	33.36	5793.36
10’	Interior	W12×19	96	1920	44.43	85305.6	10	55.6	85361.2
	Exterior	W10×15	9	180	37	6660	8	44.48	6704.48
20’x30									
4’	Interior	W12x22	43	1290	47	60630	10	55.6	60685.6
	Exterior	W10×17	2	60	40.7143	2442.86	12	66.72	2509.58
5’	Interior	W12x26	34	1020	54	55080	12	66.72	55146.7
	Exterior	W10x22	2	60	50	3000	10	55.6	3055.6
10’	Interior	W12x38	16	480	76.6	36768	18	100.08	36868.1
	Exterior	W12x26	2	60	54	3240	12	66.72	3306.72
30’x20									
5’	Interior	W10x12	28	560	32	17920	6	33.36	17953.4
	Exterior								
6’	Interior	W10x15	24	480	37	17760	8	44.48	17804.5
	Exterior								
10’	Interior	W12×19	16	320	44.43	14217.6	10	55.6	14273.2
	Exterior								
30’x30									
5’	Interior	W12×26	124	3720	54	200880	12	66.72	200947
	Exterior	W12×16	5	150	36	5400	12	66.72	5466.72
6’	Interior	W14×26	103	3090	53	163770	12	66.72	163837
	Exterior	W12×19	5	150	44.43	6664.5	10	55.6	6720.1
10’	Interior	W14×38	61	1830	76.33	139684	18	100.08	139784
	Exterior	W14×26	5	150	53	7950	12	66.72	8016.72

E.3 Cost: Composite Girders

Composite		Girder Size	Total # of Girders	Total Span Length	Cost (plf)	Total Girder Cost	Total Num. of studs	Total Stud Cost	Total Cost
20’x20’									
4’	Interior	W12×30	40	800	61.56	49248	12	66.72	49314.7
	Exterior	W12×19	20	400	44.43	17772	8	44.48	17816.5
5’	Interior	W12×30	40	800	61.56	49248	12	66.72	49314.7
	Exterior	W12×19	20	400	44.43	17772	8	44.48	17816.5
10’	Interior	W12×30	40	800	61.56	49248	12	66.72	49314.7
	Exterior	W12×19	20	400	44.43	17772	10	55.6	17827.6
20’x30’									
4’	Interior	W14x34	18	360	69	24840	12	66.72	24906.7
	Exterior								
5’	Interior	W14x38	18	360	76.3333	27480	14	77.84	27557.8
	Exterior								
10’	Interior	W16x31	18	360	63.5	22860	22	122.32	22982.3
	Exterior								
30’x20’									
5’	Interior	W21x55	3	90	107.875	9708.75	28	155.68	9864.43
	Exterior	W18x40	2	60	81	4860	20	111.2	4971.2
6’	Interior	W21x55	3	90	107.875	9708.75	20	111.2	9819.95
	Exterior	W18x40	2	60	81	4860	20	111.2	4971.2
10’	Interior	W21x55	3	90	107.875	9708.75	20	111.2	9819.95
	Exterior	W18x40	2	60	81	4860	14	77.84	4937.84
30’x30’									
5’	Interior	W21×55	14	420	107.88	45309.6	28	155.68	45465.3
	Exterior	W18×40	13	390	81	31590	20	111.2	31701.2
6’	Interior	W21×55	14	420	107.88	45309.6	20	111.2	45420.8
	Exterior	W18×40	13	390	81	31590	20	111.2	31701.2
10’	Interior	W21×55	14	420	107.88	45309.6	20	111.2	45420.8
	Exterior	W18×40	13	390	81	31590	14	77.84	31667.8

E.5 Cost: Non-Composite Columns and Concrete Footings

Interior Column	Column Size	Total Number of columns	Total Span Length (ft.)	Cost (plf)	Total Column Cost $	Footing Size	Footing volume (CF)	Typical Cost (per CY)	Cost (per CF)	Cost per Footing	Total Footing Cost $	Cost for Column +Footing $
20’x20’	W14x61	18	1049.9	118.5	124386.4	8'x8'x19"	101.3	133.1	4.9	499.5	8991.6	133378.0
20'x30'	W14x99	20	1166.6	187.5	218737.5	12'x12'x25'	300.0	133.1	4.9	1478.9	29577.8	248315.3
30'x30'	W14x99	15	875.0	187.5	164053.1	12'x12'x25'	300.0	133.1	4.9	1478.9	22183.3	186236.5
Exterior Column												
20’x20’	W14x43	6	350.0	85.5	29923.3	8'x8'x15"	80.0	133.1	4.9	394.4	2366.2	32289.5
20'x30'	W12x72	8	466.6	140.0	65329.6	8'x8'x19"	101.3	133.1	4.9	499.5	3996.3	69325.9
30'x30'	W12x72	15	875.0	140.0	122493.0	8'x8'x19"	101.3	133.1	4.9	499.5	7493.0	129986.0
Corner Column												
20’x20’	W14x34	3	175.0	69.0	12074.3	8'x8'x13"	69.3	133.1	4.9	341.8	1025.4	13099.7
30'x30'	W14x61	3	175.0	118.5	20731.1	8'x8'x15"	80.0	133.1	4.9	394.4	1183.1	21914.2

E.6 Cost: Cost per Square foot for all the Alternatives

System Names	Total Beam Cost	Total Girder Cost	Total Beam and Girder Cost for 4 Stories	Interior Column Cost (Include Footing.)	Exterior Column Cost (Include Footing.)	Corner Column Cost (Include Footing.)	Total Cost	Total Cost after applying Location Factor	Total Cost after applying inflation rate	Cost Per Square Foot
N.20.4.	241561	91669	1332921	381693	101615	13100	1727714	1902213	2021978	26.7
N.20.5	219483	94749	1256930	381693	101615	13100	1651723	1818547	1933045	25.6
N.20.10	178060	95979	1096157	381693	101615	13100	1490950	1641536	1744888	23.1
N.30.5	279557	117480	1588150	186236	129986	21914	1796300	1977727	2102246	27.8
N.30.6	303128	125880	1716030	186236	129986	21914	1924181	2118523	2251907	29.8
N.30.10	173513	118290	1167210	186236	129986	21914	1375361	1514272	1609612	21.3
C.20.4.	244437	106874	1405244	381693	101615	13100	1800037	1981841	2106620	27.9
C.20.5	226885	109480	1345459	381693	101615	13100	1740252	1916018	2036652	26.9
C.20.10	146514	104882	1005584	381693	101615	13100	1400377	1541816	1638890	21.7
C.30.5	206413	77166	1134320	186236	129986	21914	1342470	1478060	1571120	20.8
C.30.6	170557	77122	990715	186236	129986	21914	1198866	1319951	1403057	18.6
C.30.10	147801	77089	899557	186236	129986	21914	1237694	1362701	1448498	19.2

E.7 Final Cost Calculation

	Dimensions	Count	Length	total length	Cost (LF)	Total beam cost
Beam	W8x10	15	30	450	28.5	12825
	W8x10	5	8	40	28.5	1140
	W10x12	18	15	270	32	8640
	W10x19	5	8.333333333	41.66666667	44.43	1851.25
	W10x19	10	10	100	44.43	4443
	W12x16	20	7	140	36	5040
	W12x19	122	30	3660	44.43	162613.8
	W14x26	1	20	20	53	1060
	W14x26	428	30	12840	53	680520
	W16x26	12	30	360	53	19080
	W16x40	20	30	600	80	48000
	W18x40	71	30	2130	81	172530
	W18x40	3	42	126	81	10206
	W21x55	78	30	2340	107.88	252439.2
	W21x55	3	35	105	107.88	11327.4
	W21x55	3	39.33333333	118	107.88	12729.84
Column	W12x35	27	3	81	71	5751
	W12x35	59	8	472	71	33512
	W12x72	33	3	99	140	13860
	W12x72	62	14	868	140	121520
	W14x61	4	33	132	40.71	5373.72
	W14x61	3	47	141	40.71	5740.11
	W14x61	5	62	310	40.71	12620.1
	W14x99	1	33	33	65	2145
	W14x99	13	62	806	65	52390
Frame	W14x159	96	14.67	1408.32	226	318280.32
	W16x77	48	30	1440	130	187200
Total						2162837.74

	Dimension	Count	rebar	no of rebar	Total No. of Rebar	Unit Cost	total cost
Footing Rebar	8x8x16	10	no7	8	80	23.5	1880
	8x8x20	17	no8	10	170	31.5	5355
	12x12x2	13	No9	12	156	63	9828
							17063

	Dimension	Count	$/each	Total
Footing	8x8x16	10	370	3700
	8x8x20	17	370	6290
	12x12x2	13	370	4810
		Area		14800
Slab-on-ground	8"	18900	4.28	80892
Floor Slabs	5"	63399.75	3.35	212389.1625
				293281.1625

Superstructure	floor Slab	212389
	Steel Frames	2162837
	Metal Decking	181147.08
	Studs	2590.96
		2558964.04
Substructure	Footing	47963
	Slab-on-ground	80892
	Foundation	178965.2778
		307820.2778

	Total Cost	$/sf
Structure-Cost Data (2009)	2866784.32	33.91
Structure(with inflation)	3355055.20	39.69
Total building cost	23138311.71	273.72

Appendix F: System Selection Matrix

Long	Member Uniformity	Floor Depth	Total Column Area	# of Columns
Member Uniformity				
Floor Depth	Member uniformity			
Total Column Area	Column area	Floor depth		
# of Columns	# of columns	# of columns	Column area	

Member Uniformity: 1
Floor Depth: 1
Total Column Area: 2
# of Columns: 2

Vs.	Member Uniformity	Floor Depth	Total Column Area	# of Columns
Member Uniformity				
Floor Depth	Member Uniformity			
Total Column Area	Column Area	Column Area		
# of Columns	# of Columns	# of Columns	# of Columns	

Member Uniformity: 1
Floor Depth: 0
Total Column Area: 2
# of Columns: 3

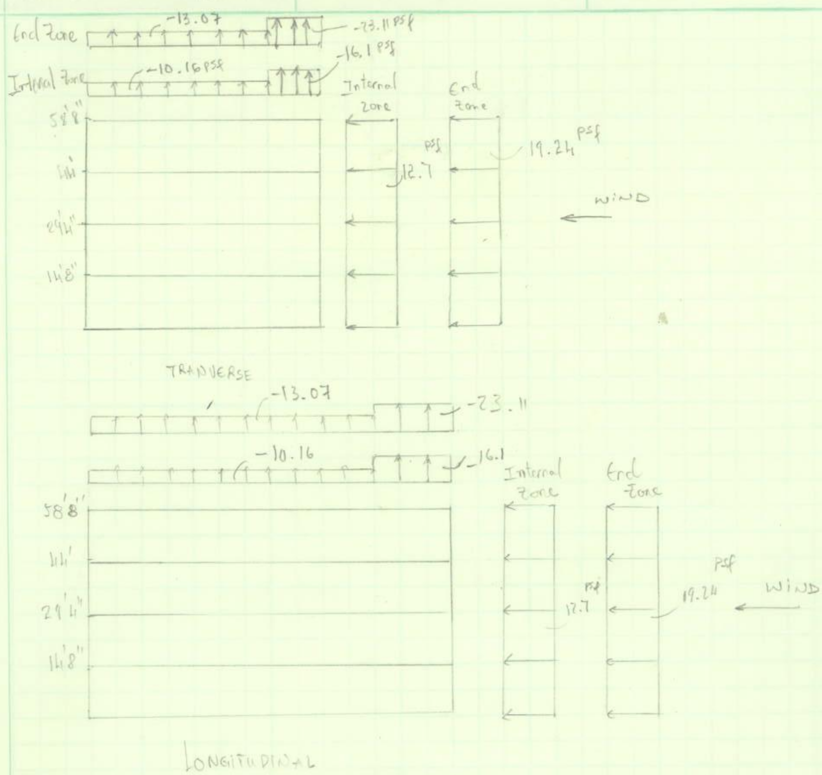
Mei	Member Uniformity	Floor Depth	Total Column Area	# of Columns
Member Uniformity				
Floor Depth	Floor Depth			
Total Column Area	Member Uniformity	Floor Depth		
# of Columns	# of Columns	# of Column	# of Column	

Member Uniformity: 1
Floor Depth: 2
Total Column Area: 0
# of Columns: 3**Total Tally**: 18
Member Uniformity: 3
Floor Depth: 3
Total Column Area: 4
# of Columns: 8

Appendix G: Lateral Design

G.1 Wind Force Calculation

WIND LOAD CALCULATION



LENGTH OF HIGHER END ZONE PRESSURE.

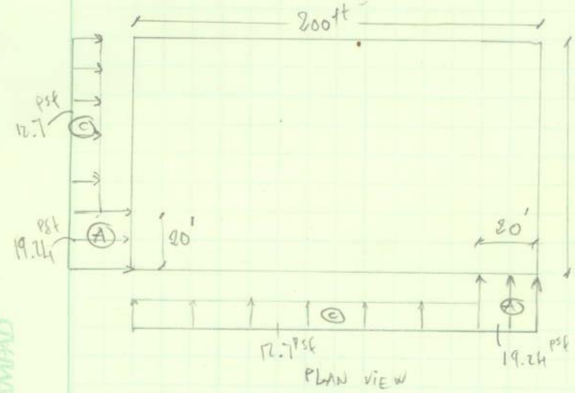
$0.4(h_{mean}) = 0.4 \times 59' = 23.6^{ft}$

$0.1(\text{least width}) = 0.1 \times 100' = 10^{ft}$

$a = 10^{ft}$ (lesser of the two)

$2a = 2 \times 10 = 20^{ft}$

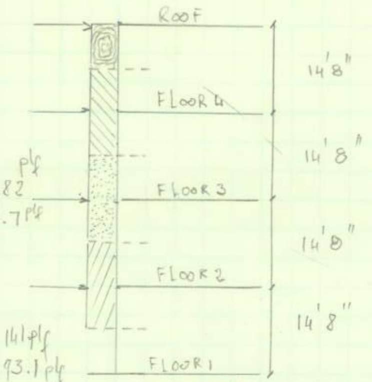
DETERMINE SHEARFORCE @ EACH LEVEL.



PLAN VIEW

TRIBUTARY HEIGHT:

Floor 2: 14'8"
Floor 3: 14'8"
Floor 4: 14'8"
ROOF : 7'4"



TRIBUTARY WIND LOADS

LINEAR LOAD:

FLOOR 2 : $(19.24^{psf})(14.7') = 282.82^{plf}$
$(12.7^{psf})(14.7') = 186.7^{plf}$

FLOOR 3 = FLOOR 4 = FLOOR 2
END: 282.82^{plf}
INT: 186.7^{plf}

ROOF : END: $(19.24)(7.33) = 141\,plf$
INT: $(12.7)(7.33) = 93.1\,plf$

FORCES @ EACH STOREY

LONGITUDINAL DIRECTION – END ZONE WIDTH = 20', INTERIOR ZONE = 100-20 = 80^{ft}

WIND LOAD: $(20)(19.24) + (80)(12.7) = 1400.8 = 1.4^{K}$

AT EACH STORY: WIND LOAD × TRIBUTARY HEIGHT.

FLOOR 2 = $(1.4^{K})\cdot(14.7') = 20.58^{K}$

FLOOR 3 = FLOOR 4 = FLOOR 2 = 20.58^{K}

ROOF = $(1.4)(7.33) = 10.26^{K}$

TRANSVERSE DIRECTION: END ZONE WIDTH: 20^{ft}, INTERIOR ZONE: $200-20 = 180^{ft}$

WIND LOAD: $(20)(19.24) + (180)(12.7) = 2.67^{K}$

FLOOR 2 = $(2.67^{K})(14.7') = 39.25^{K}$

FLOOR 3 = FLOOR 4 = FLOOR 2 = 39.25^{K}

ROOF = $(2.67)(7.33) = 19.57^{K}$

G.1.1 Rigid Frame- Transverse Direction

$$P_{e2} := \frac{3.14^2 \cdot E \cdot I_x}{(K_x \cdot 14.7 \cdot 12)^2} = 7.143 \times 10^3$$

$$B_2 := \frac{1}{1 - \frac{6 \cdot P_{nt}}{2 \cdot P_{e2}}} = 1.077$$

$$P_r := P_{nt} + B_2 \cdot P_{lt} = 212.744$$

LTB

$$M_n := 1.0 \cdot \left[\phi M_{px} - \left(\phi M_{px} - 0.7 \cdot \frac{f_Y}{1000} \cdot S_x \right) \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] = 1.169 \times 10^3 \qquad k - ft$$

FLB

$$\lambda_p := 0.38 \cdot \left(\frac{E}{f_Y} \right)^{0.5} = 0.289$$

WLB

$$\frac{h}{t_w} = \blacksquare \cdot 15.3$$

$$M_{cx} := 1169 \qquad k - ft$$

$$M_{rx} := B_1 \cdot M_{nt} + B_2 \cdot M_{lt} = 532.195$$

$$\frac{P_r}{P_c} = 0.117$$ less than 0.2

Use $H1 - 1b$

$$\frac{P_r}{2 \cdot P_c} + \left(\frac{M_{rx}}{M_{cx}} \right) = 0.514$$ less than 1.0 ok

G.2 Seismic forces design

$C_{smin} := 0.01$ sec (govern)

Combination load for strength design (LRFD)

$(1.2 + 0.2S_{DS})D + Q_E + 0.5L + 0.2S$

$(0.9 - 0.2S_{DS})D + Q_E + 1.6H$

Combination load for strength design with overstrength factor (LRFD)

$(1.2 + 0.2S_{DS})D + 2.5 \cdot Q_E + 0.5L + 0.2S$

$(0.9 - 0.2S_{DS})D + 2.5Q_E + 1.6H$

$DL_{floor} := \frac{140 \cdot 30^2}{1000} = 126$ k

$DL_{roof} := \frac{106.67 \cdot 30^2}{1000} = 96.003$ k

$LL := \frac{65 \cdot 30^2}{1000} = 58.5$ k

$SL := \frac{45 \cdot 30^2}{1000} = 40.5$ k

$W_1 := (1.2 + 0.2S_{DS})DL_{floor} + 0.5LL + 0.2SL = 192.75$ k

$W_2 := 192.75$ k

$W_3 := 192.75$ k

$W_4 := (1.2 + 0.2S_{DS})DL_{roof} + 0.5LL + 0.2SL = 155.754$ k

Base Shear

$V := C_{smin} \cdot (W_1 + W_2 + W_3 + W_4) = 7.34$ k

Vertical Distribution Force $k := 1$

$\Sigma := W_1 + 2W_2 + 3 \cdot W_3 + 4 \cdot W_4 = 1.78 \times 10^3$ k

because h is constant

$F_{x1} := \frac{W_1}{\Sigma} \cdot V = 0.795$ k

$F_{x2} := \frac{W_2}{\Sigma} \cdot V = 0.795$ k

$F_{x3} := \frac{W_3}{\Sigma} \cdot V = 0.795$ k

$F_{x4} := \frac{W_4}{\Sigma} \cdot V = 0.642$ k

G.2.1Rigid Frame- Seismic Load

Column W14x159

$L_b := 14.$ ft | $A := 46.$ in^2 | $I_x := 199$ in^4 | $E := 2900$ | $f_Y := 5000$

$h_0 := 13.$ in | $r_{ts} := 4.51$ in | $J := 19.$ in^4 | $S_x := 25$ in^3 | $r_y := 4$ in

$\frac{h}{t_w} := 15.$ | $\phi M_{px} := 108$ | $L_r := 66.$ | $L_p := 14.1$ | $\phi P_n := 182$

Girder W16x77

$I_{girder} := 111$ in^4

$L_{girder} := 3$ ft

Using RISA 2.0

$P_{nt} := 191.6$ k | $M_{nt} := 265.2$ k − ft | $K := 1.$

$P_{lt} := 2.3$ k | $M_{lt} := 18.3$ k − ft | $\alpha := 1.$

$$P_c := \phi P_n$$

Find B1 and B2

$$P_{e1} := \frac{3.14^2 \cdot E \cdot I_x}{(K \cdot L_b \cdot 12)^2} = 1.829 \times 10^4$$

$$C_m := 1 - 0.4 \frac{(P_{nt} + P_{lt})}{P_{e1}} = 0.996$$

$$B_1 := \frac{C_m}{1 - \alpha \cdot \frac{P_{nt} + P_{lt}}{P_{e1}}} = 1.006$$

$G_A := 1.$ (default)

$$G_B := \frac{\left(\frac{I_x}{L_b \cdot 12}\right)}{\left(\frac{I_{girder}}{L_{girder} \cdot 12}\right)} = 3.659$$

So $K_x := 1.$ from K chart

$$P_{e2} := \frac{3.14^2 \cdot E \cdot I_x}{(K_x \cdot 14.7 \cdot 12)^2} = 7.143 \times 10^3$$

$$B_2 := \frac{1}{1 - \frac{6 \cdot P_{nt}}{2 \cdot P_{e2}}} = 1.088$$

$$P_r := P_{nt} + B_2 \cdot P_{lt} = 194.237$$

LTE

$$M_n := 1.0\left[\phi M_{px} - \left(\phi M_{px} - 0.7 \cdot \frac{f_Y}{1000} \cdot S_x\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] = 1.169 \times 10^3 \qquad k - ft$$

FLB

$$\lambda_p := 0.38 \cdot \left(\frac{E}{f_Y}\right)^{0.5} = 0.289$$

WLB

$$\frac{h}{t_w} = \blacksquare \cdot 15.3$$

$$M_{cx} := 1169 \qquad k - ft$$

$$M_{rx} := B_1 \cdot M_{nt} + B_2 \cdot M_{lt} = 286.893$$

$$\frac{P_r}{P_c} = 0.107$$ less than 0.2

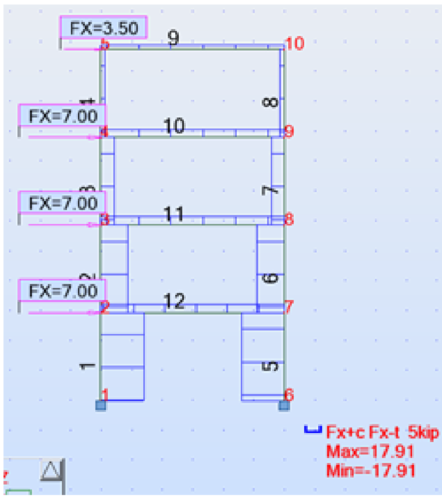
Use H1 − 1b

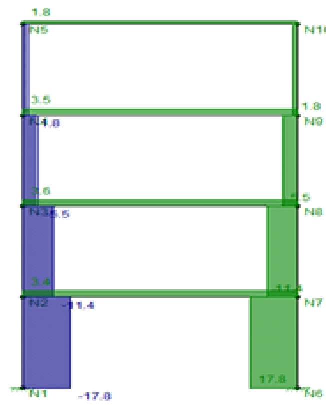
$$\frac{P_r}{2 \cdot P_c} + \left(\frac{M_{rx}}{M_{cx}}\right) = 0.299$$ less than 1.0 ok

G.3. *RISA* and *ROBOT* COMPARISON

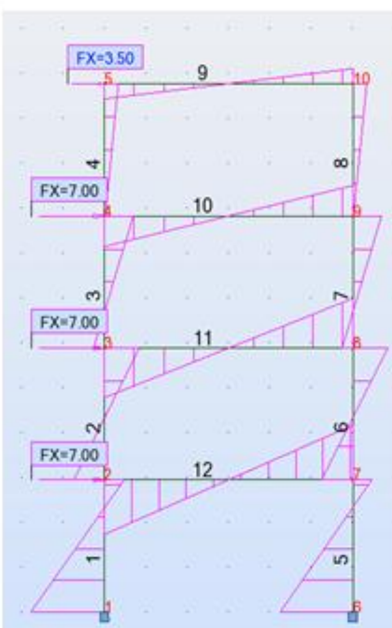
G.3.1 Simple Four-Story Frame Models

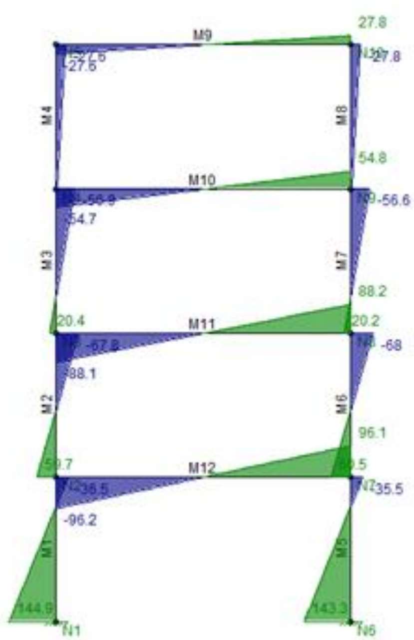
Axial Force





Moment

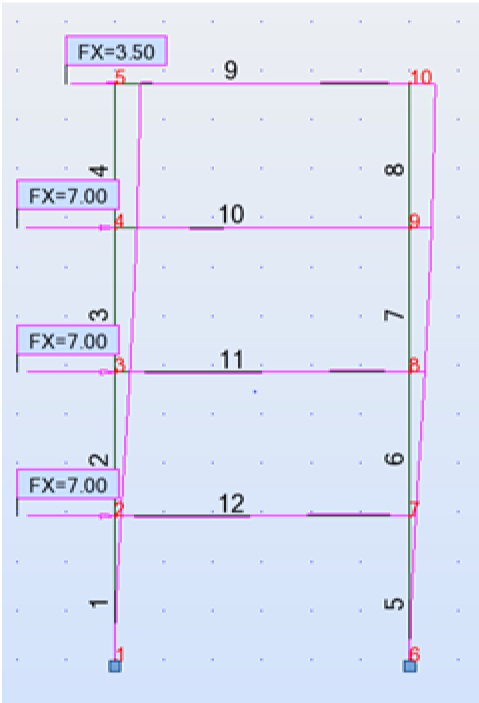


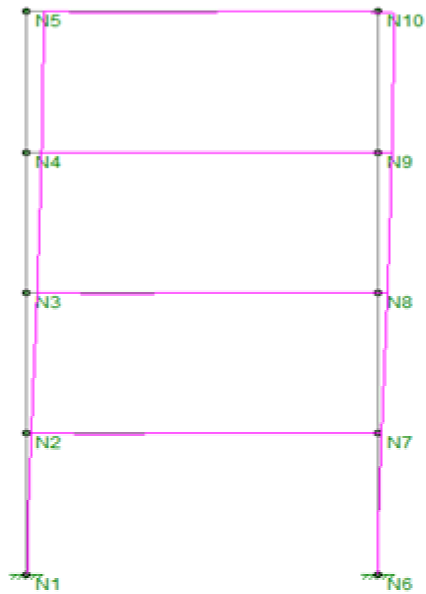


Bar/Node/Case			MY (kip-ft)
1/	1/	1	-143.35
1/	2/	1	38.65
2/	2/	1	-59.39
2/	3/	1	67.82
3/	3/	1	-21.10
3/	4/	1	56.30
4/	4/	1	1.66
4/	5/	1	27.03
5/	6/	1	-141.03
5/	7/	1	37.13
6/	7/	1	-60.65
6/	8/	1	68.17
7/	8/	1	-20.79
7/	9/	1	55.85
8/	9/	1	1.13
8/	10/	1	27.21
9/	5/	1	27.03
9/	10/	1	-27.21
10/	4/	1	54.64
10/	9/	1	-54.72
11/	3/	1	88.93
11/	8/	1	-88.96
12/	2/	1	98.03
12/	7/	1	-97.78

	L...	Member Label	S...			Mome...
31	8	M7	1	5	5	20.194
32			2	5	5	1
33			3	5	5	-18.194
34			4	5	5	-37.389
35			5	5	5	-56.583
36	8	M8	1	1	1	-1.801
37			2	1	1	-8.297
38			3	1	1	-14.792
39			4	1	1	-21.288
40			5	1	1	-27.784
41	8	M9	1	1	-	-27.645
42			2	1	-	-13.788
43			3	1	-	.069
44			4	1	-	13.926
45			5	1	-	27.784
46	8	M10	1	3	-	-54.719
47			2	3	-	-27.343
48			3	3	-	.032
49			4	3	-	27.407
50			5	3	-	54.782
51	8	M11	1	3	-	-88.15
52			2	3	-	-44.07
53			3	3	-	.009
54			4	3	-	44.088
55			5	3	-	88.167
56	8	M12	1	3	-	-96.24
57			2	3	-	-48.166
58			3	3	-	-.092
59			4	3	-	47.981
60			5	3	-	96.055

Deflection





Deflection output table

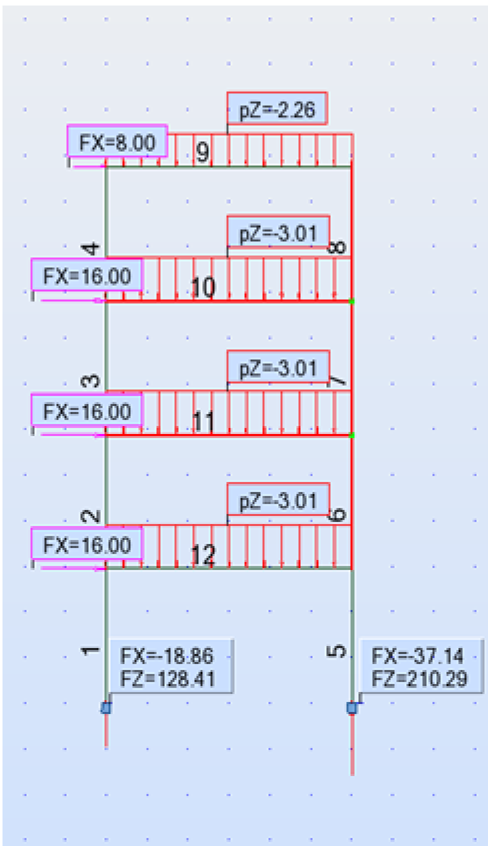
Node/Case	UX (in)	UZ (in)	RY (Rad)
1/ 1	0.0	0.0	0.0
2/ 1	0.0758	0.0021	0.001
3/ 1	0.1868	0.0034	0.001
4/ 1	0.2707	0.0040	0.000
5/ 1	0.3175	0.0042	0.000
6/ 1	0.0	0.0	0.0
7/ 1	0.0749	-0.0021	0.001
8/ 1	0.1858	-0.0034	0.001
9/ 1	0.2697	-0.0040	0.000
10/ 1	0.3170	-0.0042	0.000

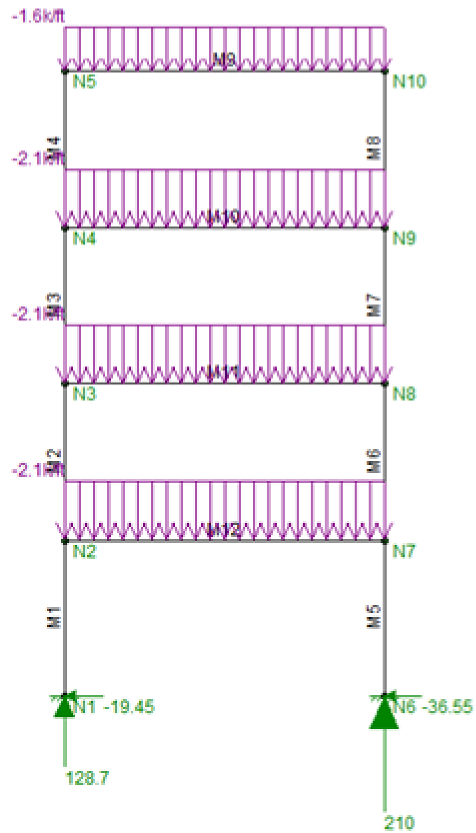
Joint Deflections (By Combination)

	L...	Joint Label	X [in]	Y [in]	Rotatio...
1	8	N1	0	0	0
2	8	N2	.107	.002	-7.04e-4
3	8	N3	.258	.003	-6.519e-4
4	8	N4	.371	.004	-4.149e-4
5	8	N5	.433	.004	-2.211e-4
6	8	N6	0	0	0
7	8	N7	.106	-.002	-7.003e-4
8	8	N8	.257	-.003	-6.522e-4
9	8	N9	.37	-.004	-4.162e-4
10	8	N10	.433	-.004	-2.24e-4

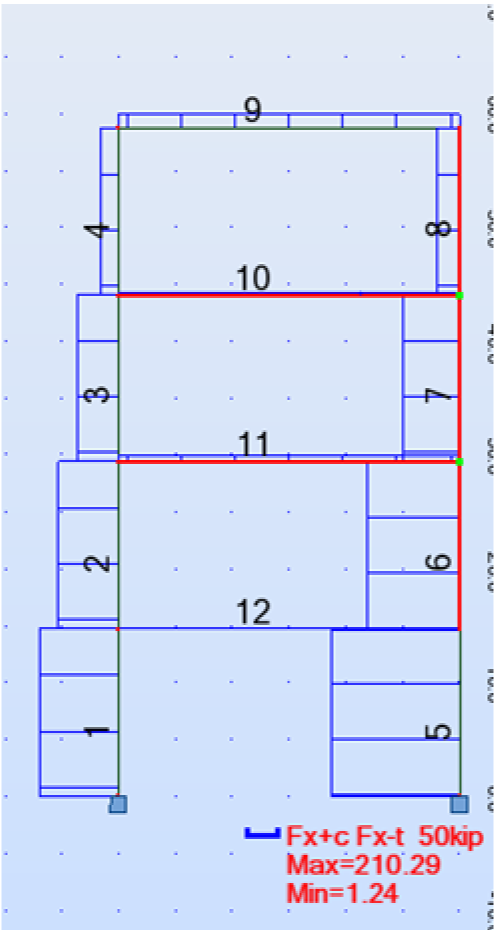
G.3.2 Lateral load + Gravity load (1.2DL+ 1.6WL+ 0.5LL+0.5LL)

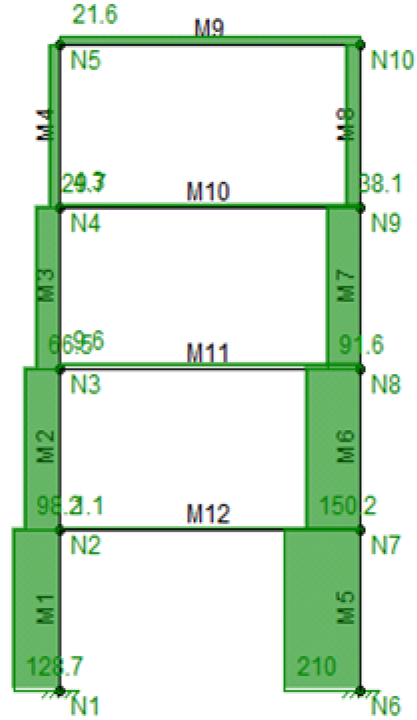
Joint Reaction



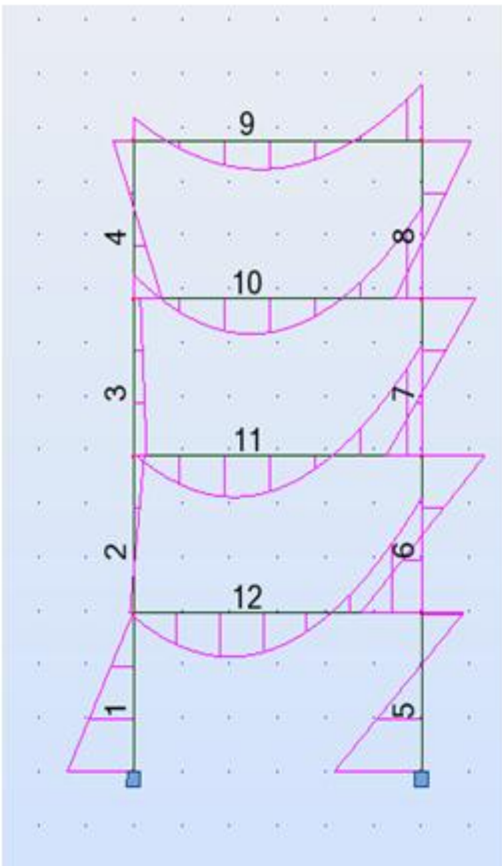


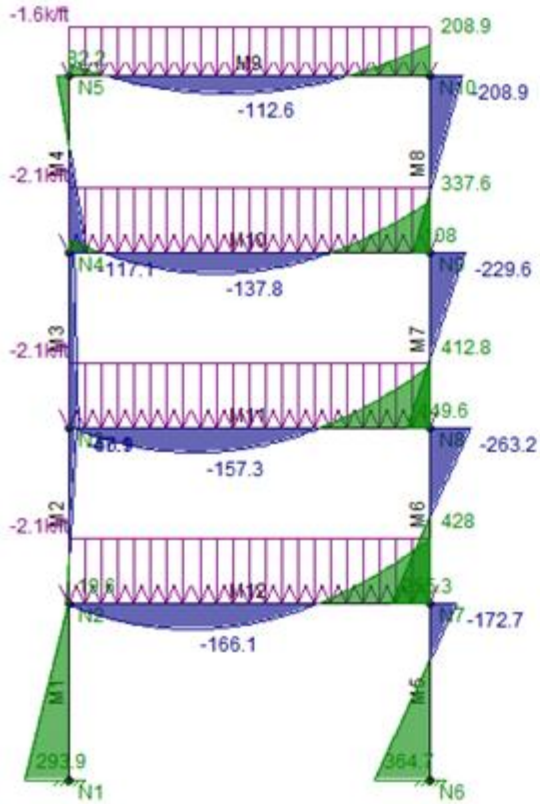
Axial Force


9
4
8
10
3
7
11
2
6
12
1
5
Fx+c Fx-t 50kip
Max=210.29
Min=1.24


21.6
M9
N5
N10
M4
M8
38.1
M10
N4
N9
M3
M7
91.6
M11
N3
N8
M2
M6
M12
N2
N7
M1
M5
210
N1
N6

Moment


9
10
11
12
1
2
3
4
5
6
7
8


-1.6k/ft
208.9
N5
M9
-112.6
208.9
M4
M8
337.6
-2.1k/ft
M10
N4
117.1
-137.8
-229.6
M3
M7
412.8
-2.1k/ft
M11
49.6
-157.3
-263.2
M2
M6
428
-2.1k/ft
M12
N2
-166.1
-172.7
M1
M5
N1
N6

G.3.3 *Robot* Second-Order moment calculation sheet

STEEL DESIGN

CODE: *ANSI/AISC 360-05 An American National Standard, March 9,2005*
ANALYSIS TYPE: Member Verification

CODE GROUP:
MEMBER: 1 Column_1 **POINT:** 1 **COORDINATE:** x = 0.00 L = 0.00 ft

---

LOADS:
Governing Load Case: 5 WL

MATERIAL:
STEEL Fy = 36.00 ksi Fu = 58.00 ksi E = 29000.00 ksi

SECTION PARAMETERS: W 14x159

d=14.98 in	Ay=37.045 in2	Az=11.160 in2	Ax=46.700 in2
b=15.56 in	Iy=1900.000 in4	Iz=748.000 in4	J=19.700 in4
tw=0.75 in	Sy=253.672 in3	Sz=96.113 in3	
tf=1.19 in	Zy=287.000 in3	Zz=146.000 in3	

MEMBER PARAMETERS:

Ly = 14.67 ft	Lz = 14.67 ft	
Ky = 1.00	Kz = 1.00	Lb = 14.67 ft
KLy/ry = 27.60	KLz/rz = 43.99	Cb = 1.00

INTERNAL FORCES:
Pr = -25.32 kip
Mry = -208.58 kip*ft
Vrz = 17.60 kip

NOMINAL STRENGTHS:
Fit*Pn = 1513.08 kip
Fib*Mny = 774.90 kip*ft
Fiv*Vnz = 241.06 kip

SAFETY FACTORS
Fib = 0.90 Fit = 0.90 Fiv = 0.90

SECTION ELEMENTS:
UNS = Compact STI = Compact

VERIFICATION FORMULAS:
Pr/(2*Fit*Pn) + Mry/(Fib*Mny) = 0.28 < 1.00 LRFD (H1-1b) Verified
Vrz/(Fiv*Vnz) = 0.07 < 1.00 LRFD (G2-1) Verified
Ky*Ly/ry = 27.60 < (K*L/r),max = 300.00 Kz*Lz/rz = 43.99 < (K*L/r),max = 300.00 STABLE

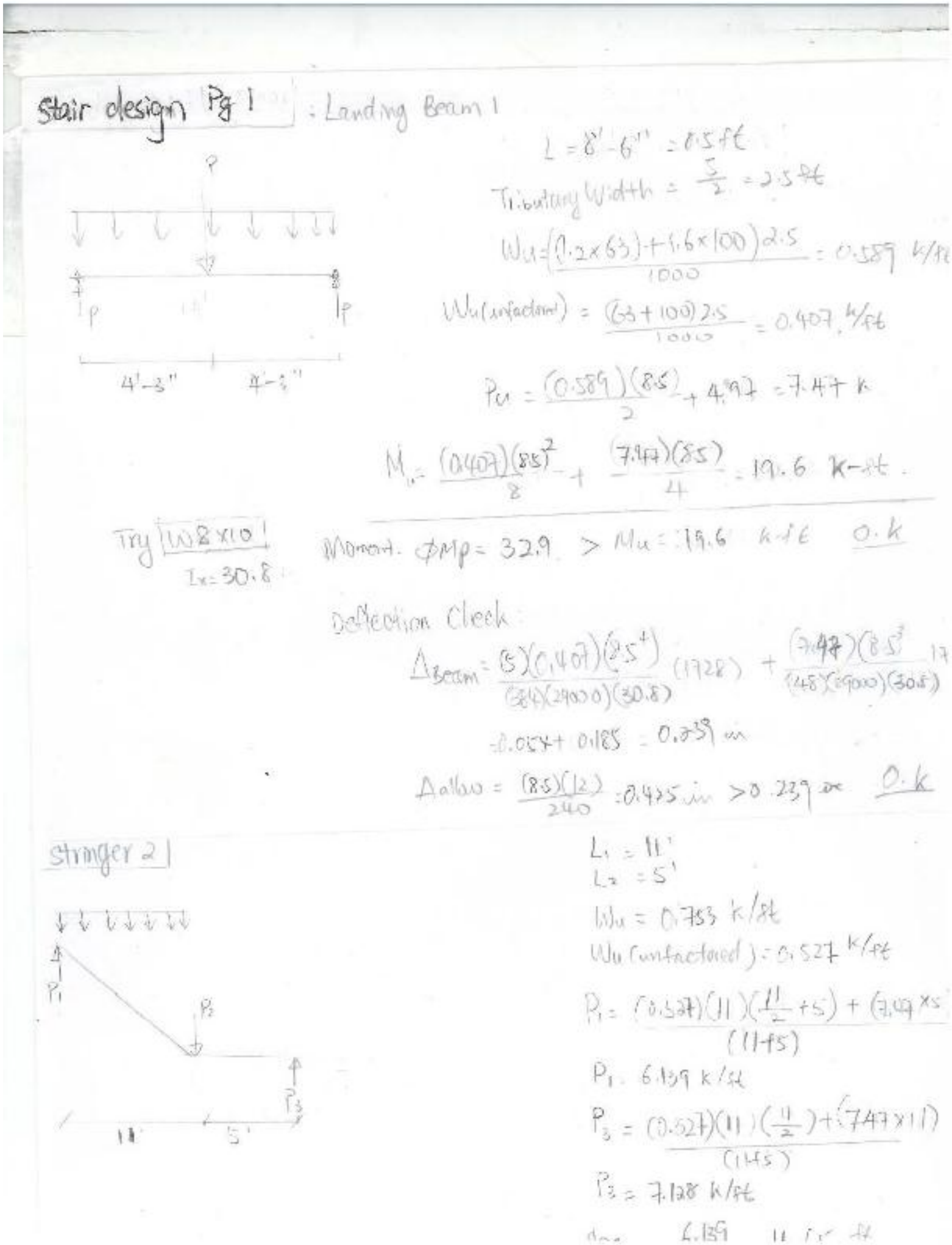
LIMIT DISPLACEMENTS

Deflections *Not analyzed*

Displacements
vx = 0.4134 in < vx max = L/150.00 = 1.1736 in Verified
Governing Load Case: 5 WL
vy = 0.0000 in < vy max = L/150.00 = 1.1736 in Verified
Governing Load Case: 1 DL1

Appendix H. Elevator and Stair Design

H.1 Stair Design



Stair design Pg 1 : Landing Beam 1

$L = 8'\text{-}6'' = 8.5\,ft$

Tributary Width $= \frac{5}{2} = 2.5\,ft$

$W_u = \frac{(1.2 \times 63) + 1.6 \times 100)2.5}{1000} = 0.589\ k/ft$

$W_{u(unfactored)} = \frac{(63+100)2.5}{1000} = 0.407\ k/ft$

$P_u = \frac{(0.589)(8.5)}{2} + 4.97 = 7.47\ k$

$M_u = \frac{(0.407)(8.5)^2}{8} + \frac{(7.47)(8.5)}{4} = 19.6\ k\text{-}ft.$

Try $\boxed{W8 \times 10}$ $I_x = 30.8$

Moment. $\phi M_p = 32.9 > M_u = 19.6\ k\text{-}ft$ O.k

Deflection Check:

$\Delta_{Beam} = \frac{(5)(0.407)(8.5^4)}{(384)(29000)(30.8)}(1728) + \frac{(7.47)(8.5)^3}{(48)(29000)(30.8)}(1728)$

$= 0.054 + 0.185 = 0.239\ in$

$\Delta_{allow} = \frac{(8.5)(12)}{240} = 0.425\ in > 0.239\ in$ O.k

Stringer 2

$L_1 = 11'$

$L_2 = 5'$

$W_u = 0.753\ k/ft$

$W_{u\,(unfactored)} = 0.527\ k/ft$

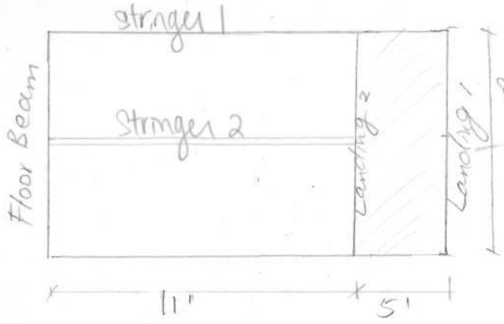
$P_1 = \frac{(0.527)(11)(\frac{11}{2}+5) + (7.47 \times 5)}{(11+5)}$

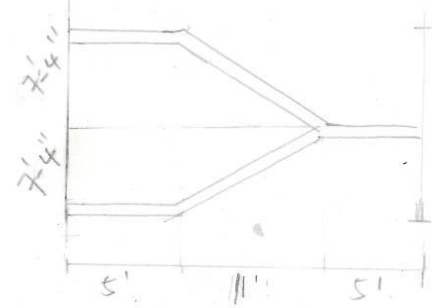
$P_1 = 6.139\ k/ft$

$P_3 = \frac{(0.527)(11)(\frac{11}{2}) + (7.47 \times 11)}{(11+5)}$

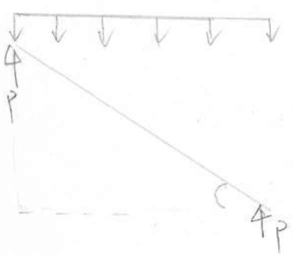
$P_3 = 7.128\ k/ft$

Stair design Pg 2





Stringer 1



DL = 63 PSF

LL = 100 PSF

$\text{Span} = \sqrt{7.33^2 + 12^2} = 13.21$

$\text{Tributary Width} = \frac{4.25}{2} = 2.125 \text{ ft}$

$\measuredangle = \tan^{-1}\left(\frac{7.33}{11}\right) = 33.678^\circ$

$W_u = \frac{\left(1.2\left(\frac{63}{\cos 33.678}\right) + 1.6(100)\right)3'}{1000} = 0.753 \text{ k/ft}$

$W_u \text{ (unfactored)} = \frac{\left(\frac{63}{\cos 33.68} + 100\right)3'}{1000} = 0.527 \text{ k/ft}$

$P_u = \frac{WL}{2} = \frac{(0.753)(13.21)}{2} = 4.97 \text{ k}$

$M_u = \frac{WL^2}{8} = \frac{(0.527)(13.21)^2}{8} = 11.50 \text{ k-ft}$

$\boxed{\text{Try } W8\times10}$

$I_x = 30.8 \text{ in}^4$

$E = 29000$

Moment check: $\phi M_p = 32.9 \text{ k-ft} > M_u$ O.

Deflection check: $\text{Beam } \Delta = \frac{5wL^4}{384EI} = \frac{(5)(0.527)(13.21)^4}{(384)(29000)(30.8)}(1728)$

$= 0.408$

$\text{Allowable } \Delta = \frac{L}{240} = \frac{(14.07)(12)}{240} = 0.661 \text{ in}$

Stair Design B3

$M_u = (6.139)(11.65) - \frac{(0.527)(11.65^2)}{2}$

$M_u = 35.756$ ft

Try W12x16

$I_x = 103 \text{ in}^4$

Moment check: $\phi M_p = 75.4$ k-ft $> M_u = 43.09$ k-ft O.k

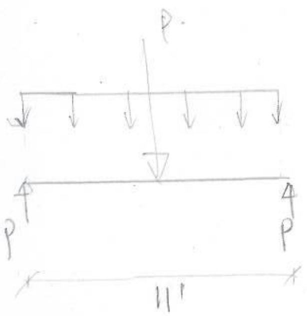
$\Delta_{Beam} = \frac{(5)(0.527)(5+11)^4}{(384)(29000)(103)} 1728 + \frac{(6.139)(5+11)^3}{(48)(29000)(103)}(1728)$

$= 0.26 + 0.303$

$= 0.563$ in.

$\Delta_{allowed} = \frac{(16)(12)}{240} = 0.80$ in $> \Delta_{Beam}$ O.k

Landing B2



$L = 11$

$W_u = 0.589$ k/ft

$W_{u(unfactored)} = 0.407$ k/ft

$P_u = \frac{(0.589)(11)}{2} = 3.24$ k

$M = \frac{(0.407)(12)^2}{8} = 6.16$ k-ft

Try 8x10 $I_x = 30.8 \text{ in}^4$

Moment check:

$\phi M_p = 32.9$ k-ft $> M_u$ k-ft O.k.

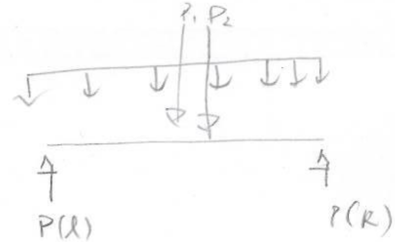
Deflection check:

$\Delta_{Beam}: \frac{(5)(0.407)(11)^4}{(384)(29000)(30.8)} 1728 = 0.15$ in

$\Delta_{allowed} = \frac{(11)(12)}{240} = 0.55$ in O.k.

Stair design Pg $\boxed{4}$

Floor Beam



$L = 8.5\ ft$ $\quad$ W_u (unfactored) : 0.55 k/ft

$W_u = (1.6)(100)\left(\frac{11}{2}\right)/1000$

$= 0.88$ kip/ft

$P_u = (P_{string\ 1})(2) = (4.97)(2) = 9.94\ k$

$P(l) = (0.88)\left(\frac{8.5}{2}\right) + 9.94$

$= 13.66\ k$

$$M_u = \frac{(0.55)(8.5)^2}{8} + \frac{(9.94)(8.5)}{4} = 26.09\ k\text{-}ft$$

$\boxed{\text{Try } W10\times12}$

$I_x = 53.8$

moment check:

$\phi M_p = 46.9\ k\text{-}ft > 26.09\ k\text{-}ft$

Deflection check

$$\Delta_{beam} = \frac{(5)(0.55)(8.5^4)}{(384)(29000)(53.8)} 1728 + \frac{(9.94)(8.5)^3}{(48)(29000)(53.8)} 1728$$

$= 0.207$ in

$$\Delta_{Allow} = \frac{(8.5)(12)}{240} = 0.425\ in > 0.207\ in \quad \underline{\text{check}}$$

Column Design $\quad L = 14.8''$

$P_u = (P_{stringer\ 2} + \text{Landing } 2)(4)$

$= (19.128 + 3.24)(4)$

$= 92.4\ k$

Assume $\frac{KL}{r} = 50 \quad \phi F_{cr} = 37.5$

$$\text{AREA Required} = \frac{P_u}{\phi F_{cr}} = \frac{92.4}{37.5} = 2.46\ in^2$$

Try: W10X12 $\quad$ Area $= 2.96\ in^2$ $\quad r_x = 3.22$ in $\quad r_y = 0.841$ in

$$\frac{KL}{r_x} = \frac{(1)(14.67)(12)}{3.22} = 45.138$$

Stair design Pg 5

Use $\frac{KL}{r_y}$ From table 4-22

Interpolation $\frac{KL}{r_y} = 101$ $\phi F_{cr} = 21.3$ ksi

$\frac{KL}{r_y} = 102$ $\phi F_{cr} = 21$ ksi

$$\phi F_{cr} = (21) + \frac{(101.172 - 102)}{(102 - 101)}(21.3 - 21) = 21.052$$

$$\phi P_n = (\phi F_{cr})(AREA)$$

$$= (21.052)(4.41)$$

$$= 92.8 \text{ k} > 92.4$$ O.K

H.2 Elevator Design

Elevator Seismic design

From table 1-1 (ASCE 7-05) , the Gateway building is category III
total building height $h := 59$ ft

Figure 22.1 $S_s := 0.25$ Table 11.4-1 $F_a := 1.0$

Figure 22.2 $S_1 := 0.06$ Table 11.4-2 $F_v := 1.0$

Site B

Design spectral response acceletration parameter at short periods

$$S_{DS} := \frac{2}{3} \cdot F_a \cdot S_s = 0.167$$

Qualify for Equivalent lateral force analysis

$$S_{D1} := \frac{2}{3} \cdot F_v \cdot S_1 = 0.04$$

From table 11.6-1, when S_{DS} greater or equal 0.167 and design cat. B

The redundancy factor $\rho := 1.0$

From the table 12.2-1, the building will be an ordinary steell moment frame

The overstrength factor $\Omega_0 := 3.0$

The response modification factor $R := 3.5$

From table 11.5-1,

The occupancy important factor $I := 1.25$

From figure 22-15

Long period transition $T_L := 6$

Building period coefficient $12.8 - 2$

$C_t := 0.028$ $x := 0.8$

Fundamental period of structure

$$T_a := C_t \cdot (h \cdot 12)^x = 5.336$$

Ta less than TL

$$C_s := \frac{S_{DS}}{\frac{R}{I}} = 0.06 \quad \text{sec}$$

C_s cannot exeed the value

$$C_{smax} := \frac{S_{D1}}{T_a \cdot \frac{R}{I}} = 2.677 \times 10^{-3} \quad \text{sec}$$

$C_{smin} := 0.01$ sec (govern)

Combination load for strength design (LRFD)

$(1.2 + 0.2S_{DS})D + Q_E + 0.5L + 0.2S$

$(0.9 - 0.2S_{DS})D + Q_E + 1.6H$

Combination load for strength design with overstrength factor (LRFD)

$(1.2 + 0.2S_{DS})D + 2.5 \cdot Q_E + 0.5L + 0.2S$

$(0.9 - 0.2S_{DS})D + 2.5Q_E + 1.6H$

$DL_{floor} := \frac{282 \cdot 8^2}{1000} = 18.048$ k $\qquad DL_{roof} := \frac{136 \cdot 8^2}{1000} = 8.704$ k

$LL := \frac{134.5 \cdot 8^2}{1000} = 8.608$ k $\qquad SL := \frac{45 \cdot 8^2}{1000} = 2.88$ k

$W_1 := (1.2 + 0.2S_{DS})DL_{floor} + 0.5LL + 0.2SL = 27.139$ k

$W_2 := W_1 = 27.139$ k

$W_3 := W_1 = 27.139$ k

$W_4 := (1.2 + 0.2S_{DS})DL_{roof} + 0.5LL + 0.2SL = 15.615$ k

Base Shear

$V := C_{smin} \cdot (W_1 + W_2 + W_3 + W_4) = 0.97$ k

Vertical Distribution Force $\qquad k := 1$

$\Sigma := W_1 + 2W_2 + 3 \cdot W_3 + 4 \cdot W_4 = 225.295$ k

because h is constant

$F_{x1} := \frac{W_1}{\Sigma} \cdot V = 0.117$ k

$F_{x2} := \frac{W_2}{\Sigma} \cdot V = 0.117$ k

$F_{x3} := \frac{W_3}{\Sigma} \cdot V = 0.117$ k

$F_{x4} := \frac{W_4}{\Sigma} \cdot V = 0.067$ k

Elevator design

Trib. Width	$W := 4 \ \mathbf{ft}$
Floor DL	$DL_{floor} := 140 \cdot 4 = 560 \ \frac{lb}{ft}$
Standard Elevator cab Wt	$W_{elevator} := 222 \ \frac{lb}{ft}$
wt of counter Wt	$W_{counterweight} := 250 \ \frac{lb}{ft}$
Live load (capacity)	$LL_{capacity} := 277.8 \ \frac{lb}{ft}$
Floor LL	$LL_{floor} := 260 \ \frac{lb}{ft}$
Assume beam wt	$W_{beam} := 96 \ \frac{lb}{ft}$
Total DL	$DL := DL_{floor} + W_{elevator} + W_{counterweight} + W_{beam} = 1.128 \times 10^{3} \ \frac{lb}{ft}$
Total LL	$LL := LL_{capacity} + LL_{floor} = 537.8 \ \frac{lb}{ft}$

Values from Table 1-1 and Table 3-2 for Beam W10x19

$A := 5.62$ in^2	$d := 10.2$ in	$I_x := 96.3$ in^4
$L_p := 3.09$ ft	$t_w := 0.25$ in	$E := 29000$ ksi
$L_r := 9.72$ ft	$k_{des} := 0.695$ in	$F_y := 50$ ksi
$L_b := 9$ ft	$Z_x := 21.6$	$\phi_b := 0.9$

Moment check

$$Wu := \frac{1.2 \cdot DL + 1.6 \cdot LL}{1000} = 2.214 \ \ \mathbf{k - ft}$$

$$Mu := \frac{Wu \cdot L_b^{\ 2}}{8} = 22.418 \ \ \mathbf{ft - k}$$

$$M_{px} := \frac{50 \cdot Z_x}{12} = 90$$

$$\phi_b \cdot M_{px} = 81$$

Shear Stress Check for Beam

$\mathbf{h} := \mathbf{d} - 2\cdot\mathbf{k_{des}} = 8.81$

$\frac{\mathbf{h}}{\mathbf{t_w}} = 35.24 \quad \blacksquare < \blacksquare \quad 2.24\left(\frac{\mathbf{E}}{\mathbf{F_y}}\right)^{\frac{1}{2}} = 53.946$

$\mathbf{C_v} := 1.0$

$\mathbf{V_n} := 0.6\cdot\mathbf{F_y}\cdot\mathbf{d}\cdot\mathbf{t_w}\cdot\mathbf{C_v} = 76.5$ k

$\mathbf{V_u} := \frac{\mathbf{L_b}}{2}\cdot\mathbf{Wu} = 9.963$ k

Deflection

$\mathbf{W} := \frac{\mathbf{DL} + \mathbf{LL}}{1000} = 1.666$

$\mathbf{M} := \frac{\mathbf{W}\cdot 30^2}{8} = 187.403$

$\mathbf{\Delta} := \frac{\mathbf{M}\cdot\mathbf{L_b}^2}{161\cdot\mathbf{I_x}} = 0.979$ in less than 1 in OK

Column design for gravity loads

Elevator cab size (tributary area) 8' x 10'

Length of rail and bracket supports $\mathbf{L_{rail}} := 12$ **ft**

Standard elevator cab wt $\mathbf{W_{elev}} := 4000$

wt of counter wt $\mathbf{W_{counter}} := 4500$

Floor DL $140\cdot 8\cdot 10 = 1.12\times 10^4$ **lbs**

$\mathbf{DL_{total}} := 4000 + 4500 + 11200 = 1.97\times 10^4$ **lbs**

Elevator capacity 2500 lbs

Floor LL $65 \cdot 8 \cdot 10 = 5.2 \times 10^3$ **lbs**

$$\mathbf{LL_{total}} := 2500 + 5200 = 7.7 \times 10^3 \quad \mathbf{lbs}$$

$$\mathbf{P_u} := \frac{(1.2 \cdot \mathbf{DL_{total}} + 1.6 \cdot \mathbf{LL_{total}})}{1000} = 35.96 \quad \mathbf{k}$$

Assume: $\frac{\mathbf{KL}}{\mathbf{r}} = \blacksquare \cdot 50$

Try column size **W12x35**

$\mathbf{A_1} := 10.63 \quad \mathbf{in}^2$

$\mathbf{r_y} := 1.54 \quad \mathbf{in}$

$\mathbf{K} := 1$

$\mathbf{L} := 14.7 \quad \mathbf{ft}$

$$\frac{\mathbf{KL}}{\mathbf{r_y}} := \frac{\mathbf{K} \cdot \mathbf{L} \cdot 12}{\mathbf{r_y}} = 114.545$$

$\mathbf{\phi F_c} := 17.2$ from 4.22 AISC

$\mathbf{\phi P_n} := \mathbf{\phi F_c} \cdot \mathbf{A_1} = 182.836$ k $\blacksquare > \mathbf{P_u}$ **ok**

dead load	Pnt	20.75	
	Mnt	5.2	
Live Load	Pnt	8.25	
	Mnt	2.5	
Snow Load	Pnt	1	
	Mnt	1.16	
Seismic Load	Pnt	1	
	Mnt	2.3	
1.2D+1.0E+0.5L+0.2S			
pnt	29.225		
plt	1		
Mnt	7.722		
Mlt	2.3		
∑H	0.12		
h	0.14		
∑Pe2=Rm (∑HL/h)	128.52		
∑Pnt	58.45		
B2=1/(1-∑Pnt/∑Pe2)	1.834166	>	1 (OK)
M1	0.232		
M2	6.24		
Cm	0.585128		
Pr=Pnt+B2Plt	31.05917		
Pel=πEI/(KL)^2	834.0178		
B1=Cm/(1-α(Pr/pel)	0.607761	use	1
required second order strength values			
Pr=Pnt+B2Plt	31.05917		
Mr=B1Mnt+B2Mlt	11.94058		
Evaluate Pr/Pc			
K	1		
(KL/r)y	114.5455	govern	
(KL/r)x	27.63916		
ØcFc	17.2	aisc table 4.22	
A	10.3		
ØPn	177.16		
Pr/Pc	0.175317	< 0.2	
use H1-1a : Pr/Pc+(Mrx/Mcx)			
		Lr	16.7
S	45.6	Lp	5.44
Mp	192	Lb	14.7
Mn	143.4796		
ØMn	129.1316		
Pr/Pc+(Mrx/Mcx)	0.257511	< 1.0 (OK)	

Appendix I: Trapezoid Area Steel Framing Calculation

I.1 Interior Beam Sample Calculation

50psf Interior Composite Beam Design for 6' spacing (35'x30' Bay Size)'

$L := 30$ ft $\quad F_y := 50$ ksi $\quad E := 29000$ ksi

$S := 8$ ft $\quad f_c := 4$ ksi

$$LL := \frac{50 \cdot S}{1000} = 0.4$$

$$DL := \frac{(78 + 48) \cdot S}{1000} = 1.008$$

$$W_u := 1.2 \cdot DL + 1.6 \cdot LL = 1.85 \quad \text{k/ft^2}$$

$$M_u := \frac{W_u \cdot L^2}{8} = 208.08 \quad \text{ft-k}$$

$$b_{e1} := 2 \cdot \frac{L}{8} \cdot 12 = 90 \quad \text{in}$$

$$b_{e2} := 2 \cdot \frac{S}{2} \cdot 12 = 96 \quad \text{in}$$

The following variables were assumed:

$y_{con1} := 7$ in

$a := 1$ in

$y_1 := 3.79$

$y_2 := 7 - \frac{a}{2} = 6.5$ in

The following values were obtaineed from Table 1-1 and 3-2 for beam size W16x26

$I_x := 301$ in^4 $\quad A := 9.13$ in^2

$\Sigma Q_n := 96$ k

Required 'a':

$$a_{req} := \frac{\Sigma Q_n}{0.85 f_c \cdot b_{e1}} = 0.314 \quad \text{in}$$

$$y_{2req} := y_{con1} - \frac{a_{req}}{2} = 6.843 \quad \text{in}$$

ϕbMn Interpolation from Table 3-19

$$y := 6.5$$

$$M_{p1} := 255$$

$$M_{p2} := 259$$

$$M_p := M_{p1} + \frac{(y_{2req} - y)}{0.5}(M_{p2} - M_{p1}) = 257.745 \text{ ft-k} \qquad \text{Mp > Mu=208.1 ft-k} \quad \text{Ok}$$

Deflection Check

Deflection during construction:

$$C_{LL} := \frac{20 \cdot S}{1000} = 0.16 \text{ k/ft^2} \qquad WC := \frac{145 \cdot \frac{5}{12} \cdot S \cdot 1.1}{1000} = 0.532$$

$$W := C_{LL} + WC + .026 = 0.718$$

$$\Delta_C := 1728 \cdot \frac{5 \cdot W \cdot L^4}{384 \cdot E \cdot I_x} = 1.498 \quad \text{in}$$

Flange Local Buckling Check

$$\frac{b_f}{2t_f} := 7.97 \quad < \quad 0.38 \cdot \left(\frac{E}{F_y}\right)^{\frac{1}{2}} = 9.152 \qquad \text{OK}$$

Web Local Buckling Check

$$\frac{h}{t_w} := 56.8 \quad < \quad 3.768 \left(\frac{E}{F_y}\right)^{\frac{1}{2}} = 90.745 \qquad \text{Ok}$$

Shear Stress Check Using Table 3-6

$V_u := \frac{L \cdot W_u}{2} = 27.744 \ k$ Vu< ϕV_n=131 OK

Stud Designs using table 3-21

$Q_n := 17.2 \quad K$

$N_{stud} := \frac{\Sigma Q_n \cdot 2}{17.2} = 11.163$ 12 3/4" Studs are required

$Spacing := \frac{12 \cdot L}{12} = 30$

I.2 Interior Girder Calculation

Interior Composite Girder Design for 6' spacing (42x15 Bay Size)'

$L := 42$ ft $\quad F_y := 50$ ksi $\quad E := 29000$ ksi

$S := 7.5$ ft $\quad f_c := 4$ ksi

$LL := \frac{100 \cdot S}{1000} = 0.75$ k/ft $\qquad DL := \frac{(78+48) \cdot S + 12}{1000} = 0.957$ k/ft

$W_u := 1.2 \cdot DL + 1.6 \cdot LL = 2.348$ k/ft

$P_u := W_u \cdot S = 17.613$ k/ft

$M_u := 1056$ ft-k

$b_{e1} := 2 \cdot \frac{L}{8} \cdot 12 = 126$ in

$b_{e2} := 2 \cdot \frac{S}{2} \cdot 12 = 90$ in

The following variables were assumed:

$y_{con1} := 7$ in

$a := 1$ in

$y_1 := 1.67$

$y_2 := 7 - \frac{a}{2} = 6.5$ in

The following values were obtaineed from Table 1-1 and 3-2 for beam size W21x62

$I_x := 1330$ in^4 $\qquad A := 18.3$ in^2

$\Sigma Q_n := A \cdot F_y = 915$ k

Required 'a':

$$a_{req} := \frac{\Sigma Q_n}{0.85 f_c \cdot b_{e2}} = 2.99 \quad \text{in}$$

$$y_{2req} := y_{con1} - \frac{a_{req}}{2} = 5.505 \quad \text{in}$$

ϕbMn Interpolation from Table 3-19

$$y := 5.5$$

$$M_{p1} := 1210$$

$$M_{p2} := 1240$$

$$M_p := M_{p1} + \frac{(y_{2req} - y)}{0.5}(M_{p2} - M_{p1}) = 1.21 \times 10^3 \text{ ft-k} \qquad \text{M p > Mu=1056 ft-k} \quad \text{Ok}$$

Deflection Check

$$C_{LL} := \frac{20 \cdot 8}{1000} = 0.15 \qquad WC := \frac{145 \cdot \frac{5}{12} \cdot 8 \cdot 1.1}{1000} = 0.498$$

$$W := C_{LL} + WC + .070 = 0.718$$

$$\Delta_C := 1728 \frac{5 \cdot W \cdot L^4}{384 E \cdot I_x} = 1.304 \quad \text{in}$$

Flange Local Buckling Check

$$\frac{b_f}{2t_f} := 6.70 \quad < \quad 0.38 \cdot \left(\frac{E}{F_y}\right)^{\frac{1}{2}} = 9.152 \qquad \text{OK}$$

Web Local Buckling Check

$$\frac{h}{t_w} := 46.8 \quad < \quad 3.768 \left(\frac{E}{F_y}\right)^{\frac{1}{2}} = 90.745 \quad \text{OK}$$

Shear Stress Check

$$V_u := \frac{L \cdot W_u}{2} = 49.316 \;\; k$$ Vu< ϕV_n=256 OK

Stud Designs using table 3-21

$Q_n := 21.5$ K

$$N_{stud} := \frac{\Sigma Q_n \cdot 2}{Q_n} = 85.116$$ 46 3/4" Studs are required

$$Spacing := \frac{12 \cdot L}{46} = 10.957 \, in$$

Appendix J: Code Analysis

J.1 Unsprinklered Building Code Analysis

IBC 2009 Code Analysis (unsprinklered building)	
Occupancy Classification	
303.1	Assembly (Group A-3) – classroom areas are larger than 750sqft
304.1	Business Occupancy (Group B) – offices and laboratories
Construction Type	
Table 601	Construction Type IB
Height and Area Limitations	
Table 503	Group A-3: 11 stories (160ft maximum height) and unlimited area
	Group B: 11 stories (160ft maximum height) and unlimited area
Fire Resistance Ratings for Building Elements	
Table 601	2-hrs required for Structural Frame
	2-hrs required for exterior bearing walls
	2-hrs required for interior bearing walls
	No rating required for interior nonbearing walls and partitions
	2-hrs required for floor construction and secondary members
	1-hr required for roof construction and secondary members
708.4	2-hrs required for shaft enclosures connecting four stories or more
Fire Separations and Resistance Ratings	
Table 706.4	3-hrs required for fire walls
706.3	Fire walls shall be of any approved noncombustible materials
706.5	Fire walls shall be continuous from exterior wall to exterior wall and extend at least 18 inches beyond the exterior surface of exterior walls
Table 707.3.9	2-hrs required for fire barrier assemblies or horizontal assemblies between fire areas
707.2	Fire barriers shall be of materials permitted by the building type of construction
707.6	Openings in a fire barrier shall be limited to a maximum aggregate width of 25% of the length of the wall and the maximum area of any single opening shall not exceed 156 sqft
710.3	Smoke barriers shall be of materials permitted by the building type of construction with a minimum of 1 hour fire rating. Each smoke barriers form an effective membrane continuous from outside wall to outside wall and from the top of the foundation or floor/ceiling assembly below the underside of the floor or roof sheathing, deck or slab above including continuity through concealed spaces, such as those found above suspended ceilings, and interstitial structural and mechanical spaces. (not required for
Means of Egress	
Table 1004.1.1	100 gross occupant load factor for business area 50 gross occupant load factor for laboratories 15 gross occupant load factor for lecture halls
715	20 min fire rating shall be provided for corridor doors and 90 min fire rating

	shall be provided in openings in exit enclosures
715.4.8	All fire doors shall be self or automatic closing
Table 715.4	All fire doors and fire shutter assembly shall meet the fire ratings in accordance to the table
715.4.4	Doors in exit enclosures and exit passageways to 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.
715.4.8.3	Automatic closing fire doors in corridors (door connecting to the open-air covered ramp or to the atrium) to have not more than a 10-second delay before the door starts to close after the smoke detector is actuated and smoke detector should be installed in accordance with Section 907.3
708.14.1	Enclosed elevator lobby shall be provided at each floor where an elevator shaft enclosure connects more than three stories. Elevator lobbies shall have at least one means of egress complying with Chapter 10 and other provisions within this code
Table 1015.1	At least two or more exits or exit access doorway shall be present as the building occupant load exceeds 50 persons
1015.2.1	Exits or exit access doorways are required from any portion of the exit access, the exit doors or exit access doorways shall be placed a distance apart equal to not less than 1/2 of the length of the maximum overall diagonal dimension of the building or area to be served measured in a straight line between exit doors or exit access doorways interlocking or scissor stairs shall be counted as one exit stairway
1008.1.1, 1008.1.3	Doors shall have a minimum clear width of 32 inches, and shall not exceed 5 pounds for the opening force except for fire doors
1009.1	Stairs shall have a minimum width of 44 inches
1009.2	80 inches of headroom clearance shall be provided
1009.4.2	Riser shall be 4 to 7 inches and tread depth shall be 11 inches minimum
1009.5	Stair landings shall be 48 inches maximum
1016.1	Maximum exit access travel distance shall be within 200ft
1014.3	Maximum common path of travel does shall not exceed 75 feet
1018.4	Maximum length of dead end corridors shall not exceed 20 feet
1018.2	The minimum corridor width shall be not less than 44 inches
1018.3	No obstructions shall be present in corridors
1011	Exits shall be marked by an approved exit sign, readily visible from any direction of egress travel. Exit signs shall be placed so that no point in an exit access corridor is more than 100 feet from the nearest visible exit sign. It shall be illuminated at all times and extra power source shall be provided so that it is illuminated for a minimum duration of 90 minutes
1022.1	Stair enclosure shall have a minimum of 2 hours fire rating
Table 1018.1	Corridor walls shall have a minimum of 1-hr fire rating
Openings	
713	All penetrations shall comply with this section
716	All ducts and air transfer openings shall comply with this section
Interior Finishing	

Table 803.9	Exit Enclosures and exit passageways shall be of Class A materials
	Corridors shall be of Class B materials
	Rooms and enclosed spaces shall be of Class C materials
804.4.1	Interior floor finish and floor covering materials in exit passageways and corridors shall have at least a Class II material complying with the DOCFF-1 test
804.2	Interior floor finish and floor covering materials required by Section 804.4.1 shall be of Class I or II materials and shall be classified in accordance with NFPA 253
Required Fire Protection Systems	
903.2.1.3	An automatic sprinkler system shall be provided for Group A-3 occupancies where the fire area is located on a floor other than a level of exit discharge serving such occupancies.
903.2.11.3	Approved automatic sprinkler system is required for buildings 55 feet or more in height
903.4.2	Approved audible devices shall be connected to every automatic sprinkler system
907.5	A fire alarm system shall annunciate at the panel and shall initiate occupant notification upon activation
907.2.2	Manual fire alarm boxes shall be installed
905.3.1	Class III standpipe systems shall be installed throughout buildings where the floor level of the highest story is located more than 30 feet above the lowest level of fire department vehicle access
906.1	Fire extinguishers shall be installed in new and existing Group A and B with sprinkler system

J.2 Sprinklered Building Code Analysis

IBC 2009 Code Analysis (sprinklered building)	
Occupancy Classification	
303.1	Assembly (Group A-3) – classroom areas are larger than 750sqft
304.1	Business Occupancy (Group B) – offices and laboratories
Construction Type	
Table 601	Construction Type IB
Height and Area Limitations	
Table 503	Group A-3: 11 stories (160ft maximum height) and unlimited area
	Group B: 11 stories (160ft maximum height) and unlimited area
504.2	Where a building is equipped throughout with an approved automatic sprinkler system, the value specified in Table 503 for maximum building height is increased by 20 feet and the maximum number of stories is increased by one.
Fire Resistance Ratings for Building Elements	
Table 601	2-hrs required for Structural Frame
	2-hrs required for exterior bearing walls
	2-hrs required for interior bearing walls
	No rating required for interior nonbearing walls and partitions

	2-hrs required for floor construction and secondary members
	1-hr required for roof construction and secondary members
708.4	2-hrs required for shaft enclosures connecting four stories or more
Fire Separations and Resistance Ratings	
Table 706.4	3-hrs required for fire walls
706.3	Fire walls shall be of any approved noncombustible materials
706.5	Fire walls shall be continuous from exterior wall to exterior wall and extend at least 18 inches beyond the exterior surface of exterior walls
Table 707.3.9	2-hrs required for fire barrier assemblies or horizontal assemblies between fire areas
707.2	Fire barriers shall be of materials permitted by the building type of construction
707.6	Openings in a fire barrier shall be limited to a maximum aggregate width of 25% of the length of the wall and the maximum area of any single opening shall not exceed 156 sqft
710.3	Smoke barriers shall be of materials permitted by the building type of construction with a minimum of 1 hour fire rating. Each smoke barriers form an effective membrane continuous from outside wall to outside wall and from the top of the foundation or floor/ceiling assembly below the underside of the floor or roof sheathing, deck or slab above including continuity through concealed spaces, such as those found above suspended ceilings, and interstitial structural and mechanical spaces. (not required for
Means of Egress	
Table 1004.1.1	100 gross occupant load factor for business area 50 gross occupant load factor for laboratories 15 gross occupant load factor for lecture halls
715	20 min fire rating shall be provided for corridor doors and 90 min fire rating shall be provided in openings in exit enclosures
715.4.8	All fire doors shall be self or automatic closing
Table 715.4	All fire doors and fire shutter assembly shall meet the fire ratings in accordance to the table
715.4.4	Doors in exit enclosures and exit passageways to 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.
715.4.8.3	Automatic closing fire doors in corridors (door connecting to the open-air covered ramp or to the atrium) to have not more than a 10-second delay before the door starts to close after the smoke detector is actuated and smoke detector should be installed in accordance with Section 907.3
708.14.1	Enclosed elevator lobby shall be provided at each floor where an elevator shaft enclosure connects more than three stories. Elevator lobbies shall have at least one means of egress complying with Chapter 10 and other provisions within this code
Table 1015.1	At least two or more exits or exit access doorway shall be present as the building occupant load exceeds 50 persons
1015.2.1	Exits or exit access doorways are required from any portion of the exit access, the exit doors or exit access doorways shall be placed a distance apart equal to

	not less than 1/3 of the length of the maximum overall diagonal dimension of the building or area to be served measured in a straight line between exit doors or exit access doorways interlocking or scissor stairs shall be counted as one exit stairway
1008.1.1, 1008.1.3	Doors shall have a minimum clear width of 32 inches, and shall not exceed 5 pounds for the opening force except for fire doors
1009.1	Stairs shall have a minimum width of 44 inches
1009.2	80 inches of headroom clearance shall be provided
1009.4.2	Riser shall be 4 to 7 inches and tread depth shall be 11 inches minimum
1009.5	Stair landings shall be 48 inches maximum
1016.1	Maximum exit access travel distance shall be within 300ft
1014.3	Maximum common path of travel does shall not exceed 100 feet
1018.4	Maximum length of dead end corridors shall not exceed 50 feet
1018.2	The minimum corridor width shall be not less than 44 inches
1018.3	No obstructions shall be present in corridors
1011	Exits shall be marked by an approved exit sign, readily visible from any direction of egress travel. Exit signs shall be placed so that no point in an exit access corridor is more than 100 feet from the nearest visible exit sign. It shall be illuminated at all times and extra power source shall be provided so that it is illuminated for a minimum duration of 90 minutes
1022.1	Stair enclosure shall have a minimum of 2 hours fire rating
Table 1018.1	Corridor walls shall have a minimum of 0-hr fire rating
Openings	
713	All penetrations shall comply with this section
716	All ducts and air transfer openings shall comply with this section
Interior Finishing	
Table 803.9	Exit Enclosures and exit passageways shall be of Class B materials
	Corridors shall be of Class C materials
	Rooms and enclosed spaces shall be of Class C materials
804.4.1	Interior floor finish and floor covering materials in exit passageways and corridors shall have at least a Class II material complying with the DOCFF-1 test
804.2	Interior floor finish and floor covering materials required by Section 804.4.1 shall be of Class I or II materials and shall be classified in accordance with NFPA 253
Required Fire Protection Systems	
903.2.1.3	An automatic sprinkler system shall be provided for Group A-3 occupancies where the fire area is located on a floor other than a level of exit discharge serving such occupancies.
903.2.11.3	Approved automatic sprinkler system is required for buildings 55 feet or more in height
903.4.2	Approved audible devices shall be connected to every automatic sprinkler system
907.5	A fire alarm system shall annunciate at the panel and shall initiate occupant notification upon activation

907.2.2	Manual fire alarm boxes shall not be required for sprinklered buildings
905.3.1	Class I standpipe systems shall be installed throughout buildings where the floor level of the highest story is located more than 30 feet above the lowest level of fire department vehicle access
906.1	Fire extinguishers not required in new and existing Group A and B with sprinkler system

Appendix K: Automatic Sprinkler System Design

K.1 NFPA 13 2010 Edition Guidelines

Reference Section	Design Criteria	Light Hazard Occupancy	Ordinary Hazard Group 1	Extra Hazard Group 2
5.2, 5.3.1 5.4.2	Occupancy classification	Light hazard occupancy	Ordinary Hazard Group 1	Extra hazard occupancy group 2
8.2.1	System protection area limitation	52,000 sqft	52,000 sqft	40,000 sqft
3.7	Ceiling Construction Type	Unobstructed	Unobstructed	Unobstructed
Table 8.6.2.2.1(a), Table 8.6.2.2.1(c)	Maximum protection area per sprinkler (pendant standard spray)	225 sqft	130 sqft	130 sqft
Table 8.6.2.2.1(a), Table 8.6.2.2.1(c)	Maximum spacing between sprinklers	15 feet	15 feet	15 feet
8.6.3.4.1	Minimum spacing between sprinklers	6 feet	6 feet	6 feet
8.6.3.2.1	Maximum sprinkler distance from walls	7.5 feet	7.5 feet	7.5 feet
8.6.3.3	Minimum sprinkler distance from walls	4 inches	4 inches	4 inches
8.6.4.1.1.1	Deflector Position	Min = 1 inch Max = 12 inches	Min = 1 inch Max = 12 inches	Min = 1 inch Max = 12 inches
Figure 11.2.3.1.1	Design Area and Density	1500 sqft and 0.10 gpm/sqft	1500 sqft and 0.15 gpm/sqft	2500 sqft and 0.40

				gpm/sqft
Table 11.2.3.1.2	Hose Stream requirements	100 gpm (30 minutes)	250 gpm (60 – 90 minutes)	500 gpm (90 – 120 minutes)

K.2 Hydraulic Calculations

Light Hazard									
Nozzle Type & Location	**Flow (GPM)**	**Pipe D (in)**	**Fittings**	**Pipe Eq Length (ft)**	**Friction Loss**	**Req Pressure**	**K-factor**	**Notes**	
1 (BL-1)	13.000	1.049		11.000	0.645	6.034	5.6	Q = 13	P = 5.389031
2	26.756	1.049		14.000	3.121	9.156	5.6		
3	43.701	1.380		14.000	2.035	11.190	5.6		
4	62.434	1.380		15.000	4.218	15.408	5.6		
5	84.415	1.380	Tee (8 ft)	14.500	7.123	**22.531**	5.6		
7	33.750	1.049		12.000	4.111	40.433	5.6	Right side of cross main	
6	69.359	1.049	Tee (5 ft)	13.500	17.533	57.966	5.6	Q = 43.242	
CM to BL-2	127.657	2.067	Tee (12 ft)	21.000	3.100	25.631	25.215	BL K = 28.263	
BL-2 to CM	270.744	2.067		81.000	48.047	73.678	25.215		
CM to Riser	270.744	2.459		118.5	30.172	103.850	25.215		

Ordinary Hazard 1

Nozzle Type & Location	Flow (GPM)	Pipe D (in)	Fittings	Pipe Eq Length (ft)	Friction Loss	Req Pressure	K-factor	Notes	
1 (BL-1)	48.000	2.067	ELS (5 ft)	13.500	0.326	18.694	11.2	Q = 48	P = 18.36735
BL-1 to CM	**96.424**	2.067		1.500	0.132	**18.825**	11.2	BL K = 11.10183	
6 (BL-2)	15.750	1.049		7.000	0.586	4.275	8.2	Q = 15.75	P = 3.68921
5	32.704	1.049		7.000	2.262	6.537	8.2		
4	53.670	1.380		7.000	1.488	8.025	8.2		
3	76.899	1.610		7.000	1.366	9.391	8.2		
2	**102.028**	1.610	Tee (8 ft)	15.000	4.939	**14.330**	8.2		
BL-2 to CM to BL-3	198.452	2.469		9.500	1.335	34.490	33.792		
BL-3 to CM to BL-4	294.876	2.469		4.500	1.316	35.806	33.792		
BL-4 to RIS	396.904	3.068	Tee (15 ft), 3 ELS (7 ft)	205.5	36.145	86.281	33.792		

Extra Hazard 2									
Nozzle Type & Location	**Flow (GPM)**	**Pipe D (in)**	**Fittings**	**Pipe Eq Length (ft)**	**Friction Loss**	**Req Pressure**	**K-factor**	**Notes**	
1 (BL-1)	42.000	1.610		12.000	0.765	14.827	11.2	Q = 42	P = 14.0625
2	**85.127**	2.067	Tee (10 ft)	20.000	1.395	**16.222**	11.2		
5	42.000	1.610		12.000	0.765	14.827	11.2	Right side of cross main	
4	85.127	1.610		12.000	2.826	17.654	11.2		
3	132.185	2.067	Tee (10 ft)	12.000	1.889	**19.543**	11.2	Q= 93.434	
CM to BL-2	**225.620**	2.469	Tee (12 ft)	22.000	3.920	**23.463**	11.2		
6 (BL-2)	42.000	1.610		9.802	0.625	14.687	11.2	Q = 42	P = 14.0625
7	84.923	1.610		9.802	2.298	16.986	11.2		
8	**131.082**	2.469	Tee (10 ft)	10.401	0.679	**17.664**	11.2		
11	42.000	1.610		9.802	0.625	14.687	11.2	Right side of cross main	
10	84.923	1.610		9.802	2.298	16.985	11.2		
9	131.082	2.469	Tee (12 ft)	20.010	1.306	18.291	11.2	Q = 128.817	
CM to BL-3	354.436	3.068	Tee (12 ft)	21	2.996	44.123	11.2		
12 (BL-3)	42.000	1.610		10.099	0.644	14.706	11.2	Q = 42	P = 14.0625
13	84.951	1.610		10.099	2.369	17.076	11.2		
14	**131.232**	2.469	Tee (12 ft)	13.802	0.902	**17.978**	11.2		
17	42.000	1.610		10.099	0.644	14.706	11.2	Right side of cross main	
16	84.951	1.610		10.099	2.369	17.076	11.2		
15	131.232	2.469	Tee (12 ft)	16.052	1.049	18.125	11.2	Q = 130.698	
CM to BL-3	485.135	3.548	Tee (17 ft)	21	2.638	64.886	11.2	BL K = 60.226	
CM to RIS	1326.53114	3.548	5 Tee (17 ft)	85	68.656	133.542	11.2		

K.3 Sprinkler System Cost

Light Hazard Occupancy		
Component	**Quantity**	**Total Cost (Material, Labor and Equipment)**
K-5.6 Sprinkler Head	283 sprinkler heads	$18112
1” Piping	593.1 feet	$12455.10
1 ¼” Piping	412.5 feet	$9900
1 ½ “ Piping	273.7 feet	$7389.90
2” Piping	723 feet	$25305
2 ½” Piping	1076.1 feet	$52190.85
3” Piping	24 feet	$1440
3 ½” Piping	170 feet	$12495
Total Area = 34693		**Cost = 139287.85**

Ordinary Hazard Group 1 Occupancy		
Component	**Quantity**	**Total Cost (Material, Labor and Equipment)**
K-8.2 Sprinkler Head	22 sprinkler heads	$1303.5
1” Piping	28 feet	$588
1 ¼” Piping	14 feet	$336
1 ½ “ Piping	28 feet	$756
2” Piping	51.5 feet	$1802.50
2 ½” Piping	69.4 feet	$3365.90
3” Piping	13.5 feet	$810
Total Area = 2631 sqft		**Total Cost = 8963.40**

Extra Hazard Group 2 Occupancy		
Component	**Quantity**	**Total Cost (Material, Labor and Equipment)**
K-11.2 Sprinkler Head	49 sprinkler heads	$2695
1” Piping	164 feet	$3444
1 ¼” Piping	18 feet	$432
1 ½ “ Piping	287.3 feet	$7757.10
2” Piping	40 feet	$1400
2 ½” Piping	63 feet	$3055.50
3” Piping	9 feet	$540
3 ½” Piping	38 feet	$2793
Total Area = 5588 sqft		**Cost = 22116.60**

Appendix L: Green roof structural analysis

L.1 Non-composite Beams

Non-Composite Beam Design for 5 Feet Spacing (30x30 bay size)

$LL := \frac{40 \cdot 5}{1000} = 0.2$ k/ft $\qquad DL := \frac{(62.5 + 29.71) \cdot 5}{1000} = 0.461$ k/ft

$W_u := 1.2DL + 1.6LL = 0.873$ k/ft $\qquad W := DL + LL = 0.661$ k/ft

$M_u := \frac{W_u \cdot 30^2}{8} = 98.242$ ft-k $\qquad M := \frac{W \cdot 30^2}{8} = 74.36\{$ k/ft

Values from Table 1-1 and Table 3-2 for Beam W16x36

$A := 10.6$ in^2	$d := 15.7$ in	$I_x := 448$ in^4
$L_p := 3.00$ ft	$t_w := 0.295$ in	$E := 29000$ ksi
$L_r := 9.17$ ft	$k_{des} := 0.832$ in	$F_y := 50$ ksi
$L := 30$ ft	$Z_x := 64.0$ in^3	

Moment Check for Zone 1 (Lr < Lp) $\qquad \phi_b := 0.9$

$W_{ubeam} := 1.2 \cdot (DL + 0.036) + 1.6 \cdot LL = 0.916$

$M_{ubeam} := \frac{W_{ubeam} \cdot L^2}{8} = 103.102$ ft-k $\qquad M_{px} := \frac{F_y \cdot Z_x}{12} = 266.667$ ft-k

Shear Stress Check

$$h := d - 2 \cdot k_{des} = 14.036$$

$$\frac{h}{t_w} = 47.58 \quad < \quad 2.24\left(\frac{E}{F_y}\right)^{\frac{1}{2}} = 53.946$$

$$C_v := 1.0$$

$$V_n := 0.6 \cdot F_y \cdot d \cdot t_w \cdot C_v = 138.94 \text{ k}$$

$$V_u := \frac{L}{2} \cdot W_u = 13.099 \quad \text{k}$$

Vu < Vn, OK

Deflection Check

$$\Delta := \frac{M \cdot L^2}{161 \cdot I_x} = 0.928 \quad \text{in}$$

The deflection of the dead and live load is smaller than 1 inch so OK

L.2 Non-composite Girder

Non-Composite Girder Design for 5 Feet of Spacing (30' x 30' bay size)

$LL := \frac{40 \cdot 5}{1000} = 0.2$ k/ft $DL := \frac{(62.5 + 29.71) \cdot 5 + 36}{1000} = 0.497$ k/ft $SN := \frac{45 \cdot 5}{1000} = 0.225$ k/ft

$W_u := 1.2DL + 1.6LL = 0.916$ k/ft

$P := 30 \cdot (DL + LL) = 20.912$ k

$M := P \cdot 5 + P \cdot 10 + P \cdot 15 - P \cdot \frac{15}{2} = 470.509$ ft-k

$P_u := W_u \cdot 30 = 27.494$ k

$M_u := P_u \cdot 5 + P_u \cdot 10 + P_u \cdot 15 - P_u \cdot \frac{15}{2} = 618.611$ ft-k

Values from Table 1-1 and Table 3-2 for Beam W27x84

$A := 24.8$ in^2	$d := 26.7$ in	$I_x := 2850$ in^4
$L_p := 7.31$ ft	$t_w := 0.285$ in	$E := 29000$ ksi
$L_r := 20.8$ ft	$k_{des} := 1.24$ in	$F_y := 50$ ksi
$L := 30$ ft	$Z_x := 244$ in^3	

Moment Check for Zone 1 (Lr < Lp)

$\phi_b := 0.9$

$W_{ubeam} := 1.2 \cdot (DL + 0.034) + 1.6 \cdot LL = 0.957$

$P_{ubeam} := W_{ubeam} \cdot 30 = 28.718$ k

$M_{ubeam} := P_{ubeam} \cdot 5 + P_{ubeam} \cdot 10 + P_{ubeam} \cdot 15 - P_{ubeam} \cdot \frac{15}{2} = 646.151$ ft-k

$M_{px} := \frac{F_y \cdot Z_x}{12} = 1.017 \times 10^3$ ft-k

$\phi_b \cdot M_{px} = 915$ ft-k

ϕMubeam < Mp, OK

Shear Stress Check

$h := d - 2 \cdot k_{des} = 24.22$

$$\frac{h}{t_w} = 84.982 \quad < \quad 2.24\left(\frac{E}{F_y}\right)^{\frac{1}{2}} = 53.946$$

$C_v := 1.0$

$V_n := 0.6 \cdot F_y \cdot d \cdot t_w \cdot C_v = 228.28$ k

$V_u := \frac{L}{2} \cdot W_u = 13.747$ k

Vu < Vn, OK

Deflection Check

$$\Delta := \frac{M \cdot L^2}{161 \cdot I_x} = 0.923 \quad \text{in}$$

The deflection of the dead and live load is smaller than 1 inch so OK

L.3 Composite Beam

Composite Beam Design for 5' spacing (30x30 Bay Size)'

$L := 30$ ft $\qquad F_y := 50$ ksi $\qquad E := 29000$ ksi

$S := 5$ ft $\qquad f_c := 4$ ksi

$LL := \frac{40 \cdot 5}{1000} = 0.2$ k/ft $\qquad DL := \frac{(62.5 + 29.71) \cdot 5}{1000} = 0.461$ k/ft

$W_u := 1.2 \cdot DL + 1.6 \cdot LL = 0.873$ k/ft

$M_u := \frac{W_u \cdot L^2}{8} = 98.242$ ft-k

$b_{e1} := 2 \cdot \frac{L}{8} \cdot 12 = 90$ in $\qquad b_{e2} := 2 \cdot \frac{S}{2} \cdot 12 = 60$ in

The following variables were assumed:

$y_{con1} := 7$ in

$a := 2$ in

$y_1 := 0$ in $\qquad y_2 := y_{con1} - \frac{a}{2} = 6$ in

The following values were obtaineed from Table 1-1 and 3-2 for beam size W12x26

$I_x := 204$ in^4

$A := 7.65$ in^2

$Q_n := 95.6$ k

Required 'a':

$a_{req} := \frac{Q_n}{0.85 f_c \cdot b_{e2}} = 0.469$ in

$y_{2req} := y_{con1} - \frac{a_{req}}{2} = 6.766$ in

ϕbMn Interpolation from Table 3-19

$y := 6.0$

$M_{p1} := 215$

$M_{p2} := 259$

$$M_p := M_{p1} + \frac{(y_{2req} - y)}{0.5}(M_{p2} - M_{p1}) = 282.38 \quad \text{ft-k}$$

Mu < Mp, moment check is satisfied

Deflection Check

wet concrete and beam weight deflection check:

$$w := \frac{5}{12} \cdot 145 \cdot S + 26 = 328.083 \qquad \frac{\text{lbs}}{\text{ft}}$$

$$M_{deflec} := \frac{w \cdot L^2}{8 \cdot 1000} = 36.909 \qquad \text{ft} - \text{k}$$

$$\Delta_{DL} := \frac{M_{deflec} \cdot L^2}{161 \cdot I_x} = 1.01 \, \text{in}$$

Liveload deflection check (Table 3-20):

$I_{lb1} := 891$ in^4 $\qquad I_{lb2} := 942$ in^4

$$I_{lb} := I_{lb1} + \frac{(y_{2req} - y)}{0.5}(I_{lb2} - I_{lb1}) = 969.1 \qquad \text{in^4}$$

$$M_{LL} := \frac{LL \cdot L^2}{8} = 22.5 \quad \text{ft-k}$$

$$\Delta_{LL} := \frac{M_{LL} \cdot L^2}{161 \cdot I_{lb}} = 0.13 \qquad \text{in}$$

Flange Local Buckling Check

$$\frac{b_f}{2 \cdot t_f} := 5.98 \quad < \quad 0.38 \cdot \left(\frac{E}{F_y}\right)^{\frac{1}{2}} = 9.152$$

Web Local Buckling Check

$$\frac{h}{t_w} := 48.1 \quad < \quad 3.76 \cdot \left(\frac{E}{F_y}\right)^{\frac{1}{2}} = 90.553$$

Shear Stress Check (Table 3-6)

$V_u := \frac{L \cdot W_u}{2} = 13.099$ k $\qquad \phi V_n := 106$ k

Vu < ϕVn, stress check is ok.

Required Number of 3/4" Studs

$Q_{nstud} := 17.2$

$N_{studs} := \frac{Q_n \cdot 2}{Q_{nstud}} = 11.116 \qquad$ 12 studs required.

$Spacing := \frac{12 \cdot L}{12} = 30$ in

L.4 Composite Girder

Composite Girder 5 feet spacing (30'x30')

$L := 30$ ft $\quad F_y := 50$ ksi $\quad E := 29000$ ksi

$S := 5$ ft $\quad f_c := 4$ ksi

$LL := \frac{40 \cdot 5}{1000} = 0.2$ k/ft $\quad DL := \frac{(62.5 + 29.71) \cdot 5 + 26}{1000} = 0.487$ k/ft $\quad SN := \frac{45 \cdot 5}{1000} = 0.225$ k/ft

$W_u := 1.2DL + 1.6LL = 0.904$ k/ft

$P_u := W_u \cdot 30$ k

$M_u := P_u \cdot 5 + P_u \cdot 10 + P_u \cdot 15 - P_u \cdot \frac{15}{2} = 610.511$ ft-k

$b_{e1} := 2 \cdot \frac{L}{8} \cdot 12 = 90$ in $\quad b_{e2} := 2 \cdot \frac{S}{2} \cdot 12 = 60$ in

The following variables were assumed:

$y_{con1} := 7$ in

$a := 2$ in

$y_1 := 0$ in

$y_2 := y_{con1} - \frac{a}{2} = 6$ in

The following values were obtaineed from Table 1-1 and 3-2 for bezm size W21x55

$I_x := 1140$ in^4

$A := 7.65$ in^2

$Q_n := 292$ k

Required 'a':

$$a_{req} := \frac{Q_n}{0.85 f_c \cdot b_{e1}} = 0.954 \text{ in}$$

$$y_{2req} := y_{con1} - \frac{a_{req}}{2} = 6.523 \text{ in}$$

ϕbMn Interpolation (Table 3-19)

$$y := 6.0$$

$$M_{p1} := 756$$

$$M_{p2} := 767$$

$$M_p := M_{p1} + \frac{(y_{2req} - y)}{0.5}(M_{p2} - M_{p1}) = 767.503 \text{ ft-k}$$

Mu < Mp, moment check is satisfied

Deflection Check

wet concrete and girder weight deflection check:

$$w := \frac{5}{12}\cdot 150\cdot S + 40 = 352.5 \qquad \frac{\text{lbs}}{\text{ft}}$$

$$M_{deflec} := \frac{w\cdot L^2}{8\cdot 1000} = 39.656 \qquad \text{ft} - \text{k}$$

$$\Delta_{DL} := \frac{M_{deflec}\cdot L^2}{161\cdot I_x} = 0.19 \text{ in}$$

Liveload deflection check (Table 3-20):

$$I_{lb1} := 1670 \qquad I_{lb2} := 1760$$

$$I_{lb} := I_{lb1} + \frac{(y_{2req} - y)}{0.5}(I_{lb2} - I_{lb1}) = 1.764 \times 10^3$$

$$M_{LL} := \frac{LL\cdot L^2}{8} = 22.5$$

$$\Delta_{LL} := \frac{M_{LL}\cdot L^2}{161\cdot I_{lb}} = 0.071$$

Flange Local Buckling Check

$$\frac{b_f}{2\cdot t_f} := 6.93 \quad < \quad 0.38\cdot\left(\frac{E}{F_y}\right)^{\frac{1}{2}} = 9.152$$

Web Local Buck

$$\frac{h}{t_w} := 46.5 \quad < \quad 3.76\cdot\left(\frac{E}{F_y}\right)^{\frac{1}{2}} = 90.553$$

Shear Stress Check (Table 3-6)

$V_u := \frac{L \cdot W_u}{2} = 13.567$ k $\phi V_n := 146$ k

Vu < ϕVn, stress check is ok.

Required Number of 3/4" Studs

$Q_{nstud} := 21.5$

$N_{studs} := \frac{Q_n \cdot 2}{Q_{nstud}} = 27.163$ 28 studs required.

$Spacing := \frac{12 \cdot L}{28} = 12.857$ in