LDA-1303\LDA-1307

Worcester Polytechnic Institute: Dormitory Design Project

Major Qualifying Project

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Date Submitted:

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Approved:

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ABSTRACT

This project compared and provided steel and precast concrete structural systems for the new dormitory for Worcester Polytechnic Institute., in compliance with *ASCE 7*, *AISC*, *ACI*, *MSBC*, and *NFPA* provisions. The various components of both gravity and lateral loading were addressed. This project provided cost and performance analyses on both designs and explored the role LEED requirements played in the design. Included are investigations into footing designs, scheduling constraints, and the social and political impacts of the project. A sprinkler system was designed using different materials, and comparisons were made to determine the most cost effective and safe design. To conclude, a code compliance discussion and review of the different life safety aspects of the building was completed.

Executive Summary

Colleges across the nation have been increasing residency capacities on their campuses. Some of the motivating factors have been safety of students and the desirability of on-campus housing for undergraduate and graduate students. Another reason for the increase of capacity is that colleges are expanding to accept more students for both undergraduate and graduate studies. Worcester Polytechnic Institute has been continuously growing and expanding the campus and community, which means that new construction is inevitable. Gateway Park, located along Interstate 290, has become an answer to WPI’s push to expand and help redevelop a deteriorated area of Worcester. Rising numbers of students in recent years have meant the need for additional housing as well. The construction of East Hall (finished in 2008) was one solution to this issue. More recently, the construction of the residence hall at 10 Faraday Street in Gateway Park was another project that would ease the housing strains of an expanding campus. This new residence hall was designed for upperclassmen and graduate students, and was placed in a prime location close to research laboratories.

A residence hall must contain separate spaces for its occupants, including bedrooms, kitchens, bathrooms, and living spaces. Residence halls also usually include a number of supporting spaces, such as corridors, entryways, mechanical rooms, storage, common areas, and laundry facilities. For this project, the new Faraday residence hall was used as the basis for design. Elements in the Faraday residence hall were used to further investigate the building design criteria. The layout of the new dormitory includes suite-style apartments with kitchens and separate bathrooms and common areas used as technology suites. The nature of the layout and room style imposes complications for plumbing, mechanical, and fire protection layouts. One major complication is the system integrations since the various rooms require different plumbing needs, unlike single room dormitories in which a bank of plumbing serves one or two locations in the building. Plumbing, mechanical, electrical, and the fire protection design must be integrated to ensure ease of construction, and this imposes limitations on the building and the architectural layout.

This report focused on the building design and evaluation of the new dormitory, examining the structural aspects of the current construction, as well as environmental issues, fire protection, and alternative construction options. The new residence hall has been constructed by Daniel

O'Connell's Son, Inc. (DOC) using structural steel framing; this report re-creates the steel design process, while posing the use of precast construction as an alternative construction material.

The design of any building takes much thought, analysis, and research, and the new dormitory was no exception. The dormitory design developed in this report first required an understanding of the architectural layouts that had been established by DOC to recognize the geometry imposed on the columns, girders, and beams. As is true for all construction projects taking place in the state of Massachusetts, the building must meet the requirements set forth by the *Massachusetts State Building Code* (*MSBC*). All aspects of construction were addressed, such as means of egress, concrete construction, steel construction, etc., and these standards were essential to the validity of the structural design and the design of the life safety aspects of the structure.

Construction projects also require the use of national standards. This report relied on the use of the American Institute of Steel Construction (*AISC*), the American Concrete Institute (*ACI*), the American Society of Civil Engineering Standards (*ASCE 7*), the *Precast/Prestressed Concrete Institute (PCI),* and the National Fire Protection Association (*NFPA*, particularly: *NFPA 13*, *NFPA 14*, and *NFPA 20*). The fourteenth edition of the *Steel Construction Manual* published by the *AISC* was used to establish properly sized steel members, while *PCI* tables were used to select the appropriate precast concrete members. All design calculations relied on the standards set forth by these organizations, as well as the limitations set forth by Load and Resistance Factor Design (*LRFD*).

Design loads for both gravity and lateral loading were investigated using *RISA-2D* software, Excel spreadsheets, and hand calculations. Due to the unique shape of the building and the parapet around its roof, the loading was complicated to determine. Snow drifting and the varying pressure effects caused by different wind directions were considered. In areas where the loading patterns were quite variable (such as snow drift locations), the larger values were used to standardize member sizes and promote constructability. Constructability refers to reducing the complexity of construction, and a common strategy is to create typical sections of the frame that involve less variation.

Steel girder sizes had to be adjusted after the steel lateral bracing analysis was completed. Using *RISA-2D* and the second-order analysis process, member sizes for the steel bracing were selected; however, varying sizes of the bracing members and girders presented connection issues.

The adjustment of girder sizes was the most reasonable solution. Typical designs for footings and base plates were the final elements of the structural steel design. Equations were used to start the design, but common sense and an understanding of the dimensions of the columns and footing supports were necessary to make modifications to the calculated dimensions.

Since the architectural layout that was followed for this project was the same as that produced by DOC, the steel beam, girder, and column sizes determined in this report could be compared to the member sizes used in DOC's structural plan to compare and contrast design decisions. A 3D *Revit* Model was developed to better visualize the completed steel design.

To expand on the design possibilities, a precast concrete design was established as well; this was done using information from the Precast/Prestressed Concrete Institute. The layout of the concrete members was adjusted and discussed in an effort to select the most efficient design option. The reduction of the number of concrete members used was incorporated, and the limits introduced by precast concrete member connections were addressed.

With a structural outline of the building completed, fire protection aspects were addressed. The first aspect dealt with a code review of the plans. This review was based upon the applicable fire safety codes for the Commonwealth of Massachusetts, and these references included the *Massachusetts State Building Code (MSBC)* and the *Massachusetts State Fire Code*. These codes describe the many active fire protection features, such as sprinkler systems, fire alarm systems, and smoke management systems, that are required installations in particular buildings; they also address the many passive fire protection features, such as the construction of fire walls and smoke barriers, and various fire resistance rating requirements. Review of these elements was necessary to determine whether the building was compliant with the applicable codes and standards. The second aspect of fire protection that was addressed was a design of the sprinkler system required by the *MSBC*. The sprinkler system was designed in accordance with *NFPA 13: Standard for the Installation of Sprinkler Systems*. *NFPA 13* is referenced by the *MSBC* as the manner in which to install a sprinkler system fit for the project. The sprinkler system was designed and laid out based upon the expected hazards and use of the building. Using these hazards and the appropriate fluid mechanics equations, specific sprinklers, fittings, and piping were defined for the system.

By addressing such a variety of aspects of the structural design of the dormitory, a much better understanding was gained on the interconnections amongst the various facets of design. In addition to the structural design options and the fire safety systems, project schedule and cost estimates were investigated to illustrate the construction management aspects of the residence hall project. Cost comparisons between precast concrete and steel design were important to display the variations in these design materials. *RS Means* publications were used to determine the costs of the designed elements; they were also used to project an estimation of the total cost of the project. In order to gain a better understanding of the construction timetable, the schedule created by DOC was analyzed and compacted to make it more comprehensible to the reader.

While creating a project that can be completed within the established timeframe and budget is important, following proper safety and environmental regulations is just as essential. This report looked into these features of the project by detailing the important LEED credits met in the dormitory construction process. Many people do not fully understand the role LEED plays in modern day construction; therefore, this report focused on educating readers on the topic and illustrating its importance. Codes set forth by the *MSBC* direct projects into safe and acceptable environments for their residents. The codes that must be followed can be complicated and detailed. In an effort to simplify and explain the main regulations, a section of the report was dedicated to code compliance issues. These policies cannot be overlooked, because it is these codes that insure that the building will be safe for the large number of people that will be passing in and out of its doors every day.

The topic of people presents yet another issue that accompanies any large project located within a town or city: impacts, both social and political. New buildings have neighbors and affect those around them; this means that buildings cannot be constructed without considering the effect they will have on their surroundings. The beauty of construction is that the addition of one structure can affect thousands of people in very positive ways. The new residence hall will play a significant role in the redevelopment of downtown Worcester by bringing a large population of young people closer to the heart of the city. By interviewing the general manager at the Marriot Courtyard Hotel across from the dormitory, as well as a councilman of Worcester, it was determined that the new residence hall will be a large addition to the push to revitalize Worcester. Not only will the dormitory improve and enhance life on the Worcester Polytechnic Institute campus, it will also enrich and energize the quiet downtown area of Worcester.

In conclusion, this design gave us a better understanding of the scale and resources necessary for a project such as this one. This project has outlined all the intricacies of design projects and shows how all the systems interconnect with one another. Several aspects of the project, such as lateral loading and snow loading, provided challenges during the design process and proved to be more difficult once the topics themselves were thoroughly addressed. If one small aspect of the structural design were to change, many other aspects were adjusted because of this small change. Additionally, we were able to see that the project affects many other entities besides WPI. The social implications of the project extend far beyond the WPI community and have a great impact on the continuing improvements of the City of Worcester.

AUTHORSHIP

This project is a combination of LDA 1303 and LDA 1307. Although the projects are registered separately, the work was done in conjunction to produce one final product. The paragraphs below outline how the work was divided between the members.

Corey Fisher is responsible for the work completed in fire protection engineering. He completed the cost analysis for the project, as well as the code compliance and LEED discussions. He helped with the different steel and precast concrete structural design components, including the completion of the concrete column design and the lateral loading analysis. Additionally, Corey completed an investigation into the social and political impacts of the project.

Kelsey Forward is responsible for the design of the building in both concrete and steel. She developed the Revit 3D model used throughout the report. Kelsey completed the structural footing design for both steel and precast concrete, and is responsible for the discussion on the environmental impacts of the project. She determined the lateral loads, as well as the required steel bracing members. She helped with the LEED certification discussion, the cost analysis, the construction schedule discussion, and the precast concrete beam and girder member selection. Additionally, Kelsey completed an investigation into the social and political impacts of the project.

Shane Ruddy is responsible for the design of the building in both concrete and steel. He selected the beams and girders for the precast concrete structural design, and is responsible for the determination of the snow loading. He determined the lateral loads and created the RISA 2D models used to display these loads. He also determined the required steel bracing members and completed the steel versus precast concrete discussion. He completed the discussion on the construction schedule, including the creation of a Primavera schedule. Additionally, Shane helped with the LEED certification discussion and completed an investigation into the social and political impacts of the project.

Name	Signature	Date
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Shane Ruddy	Shane M. Ruddy	4/24/13

CAPSTONE DESIGN STATEMENT

In order to address engineering designs and the constraints faced in engineering projects, this report considers a number of specific constraints that are relevant and timely in the design of the new residence hall and all real-world projects. The problem this report addressed is how to effectively design a large, residential building in Gateway Park. Architectural layouts and different structural layouts were also designed as needed to accommodate both steel and concrete structures. Both concrete and steel structures were designed for the basis of comparison. Scheduling and cost needed to be analyzed as well to make sure the project would be able to meet required deadlines and to gain an understanding of the project management aspects of the construction process. Fire safety aspects were examined by addressing the needs of a sprinkler system and compliance with state and local fire codes. In order to evaluate the impacts the construction would have on the environment, LEED aspects were investigated. The realistic constraints addressed in this report include: economic, environmental, sustainability, ethical, health and safety, social, and political.

Economic: The financial aspect of construction is a significant factor in the building of the new residence hall and is addressed continuously throughout the report. The costs for steel construction and precast concrete construction were investigated to add to the comparison of both structures. This project analyzed the cost of construction, outlined the cost for structural materials and other elements used in the building (electrical work, plumbing, furnishing, etc.), and determined labor costs. Square foot costs for different aspects of construction were calculated. Additionally, the costs of specific materials for the life safety systems were investigated, such as piping material and sprinkler heads. The investigation determined whether or not different and lower cost materials were able to provide the same level of safety compared to what is typical for a building of this type.

Environmental: Throughout the construction process, environmental aspects created another constraint. The construction site plan addressed this by outlining strategies to mitigate the negative impacts on the environment.

Sustainability: Sustainability has become increasingly important in many large construction projects, and the new residence hall is no exception. Following in the footsteps of the Bartlett Center, East Hall, and the new athletic facility, the new residence hall will be LEED certified.

Therefore, this report addressed several of the LEED certification requirements and what they entailed.

Ethical: This project addressed the construction of the new residence hall using the most up-to-date technology. For scheduling, cost estimating, and modeling, state-of-the-art technology was used including *Revit 2013* and *Primavera*. The design loads are based on the most recent *ASCE 7* standards, and the building is designed to safely withstand all of these loads.

Health and Safety: Ensuring that the building is safe and healthy for its occupants is addressed in this project. Using the *MSBC*, the building layout was analyzed to ensure that safety requirements were met, such as proper hallway and stairwell widths and the number of emergency exits. Other life safety requirements, such as fire resistance ratings, occupant loads, and height and area requirements of the *MSBC*, and their associated criteria for the new residence hall were addressed. Fireproofing of the steel members and analyzing seismic, wind, and gravity loads based on *ASCE 7* design loads was also completed to meet this requirement.

Social: To address the social aspect of this project, the effect of this addition on the WPI community was investigated. The neighbors directly surrounding the building, including the Courtyard Marriott Hotel, were interviewed, and an analysis was done to determine the impact the new dormitory project will have on them. The social constraint focused on how the new dormitory project will affect everyday life for those living in the area and/or attending WPI.

Political: Large construction projects are not always accepted by the neighboring residents and/or communities. The new residence hall will have an impact on the redevelopment of downtown Worcester, and, therefore, required an evaluation of how it would fit into the City. The new residence hall will bring more students even closer to the downtown area yet it is only a small part of the redevelopment taking place at Gateway Park. The report investigated how leaders of the City feel about this project and some of the political steps that are necessary to make construction possible. This includes permits and land acquisition for Gateway Park, which encompasses the specific area of the new residence hall.

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1.0 INTRODUCTION

Worcester Polytechnic Institute has been continuously growing since its beginning in 1865. The student body has been ever increasing, thus causing the campus to expand. The current need on campus is new student housing due to the recent jump in class sizes and the expansion of the graduate programs. WPI currently has nine residence halls accommodating undergraduate students, five of which are specifically for freshmen residence.

Graduate programs, especially in the life sciences and bioengineering, have been growing in the past few years; this is especially true for WPI. WPI has acquired Gateway Park, a twelve acre park in downtown Worcester, with the intention to build-out the park with five buildings. These buildings, expected to be 500,000 total square feet, will feature laboratory, classroom and office space.[1]

Massachusetts has launched the "'Growth Districts Initiative' as a focused means of expediting commercial and residential development within the Commonwealth."[2] Through this initiative, the City of Worcester has created a plan for the redevelopment of downtown Worcester. As seen below in the map of the redevelopment (Figure 1), Gateway Park is the first major section of redevelopment focus.

The area where Gateway Park is located was once a brownfield used for Worcester's large industrial economy, making the area a great location for redevelopment. The City knew that this area needed to be repaired and developed. The location of Gateway Park was critical for the City of Worcester because of its prime location along Interstate 290; this site is the first view of the City for visitors from the North.[3]

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[1] *Gateway Park at WPI*

[2] *Growth districts initiative, 2012*

[3] *The Phoenix Awards*

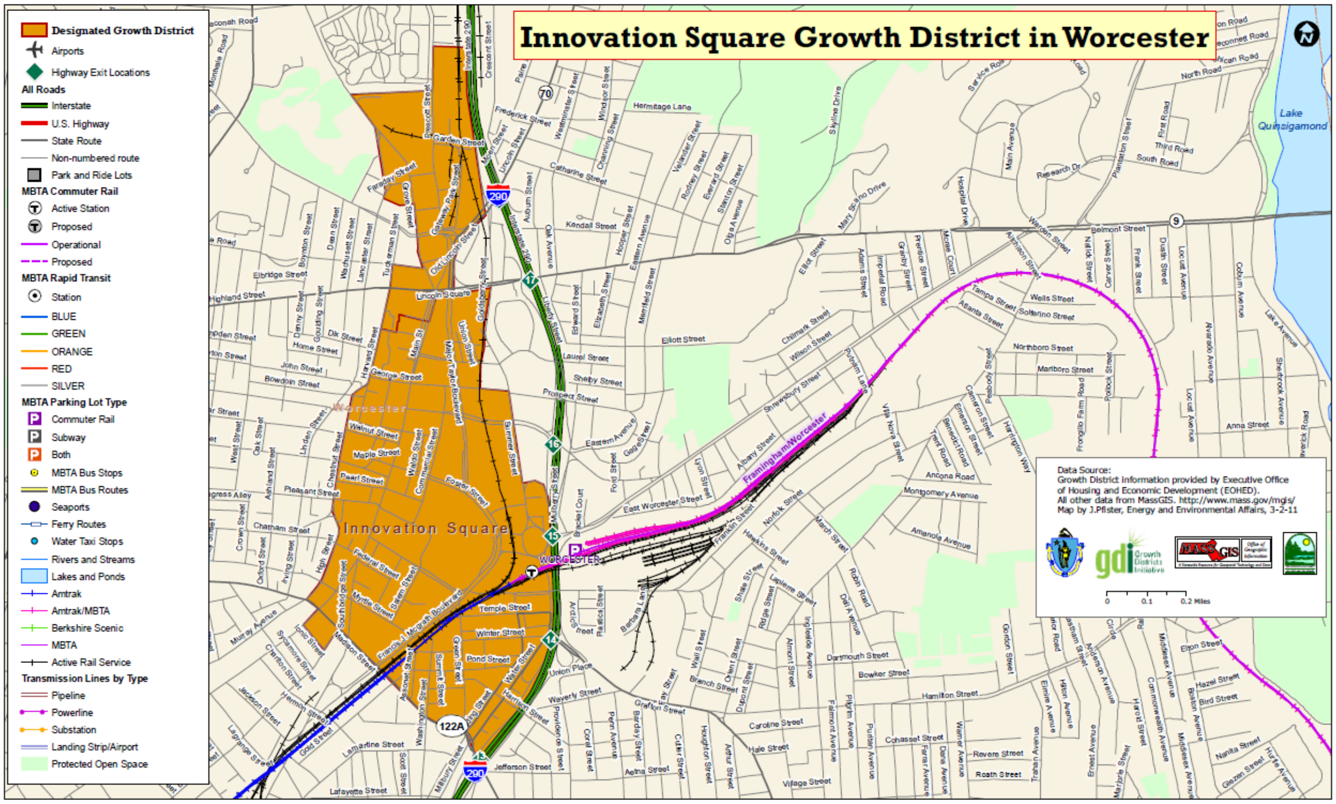

FIGURE 1: MAP OF INNOVATION SQUARE GROWTH DISTRICT IN WORCESTER (TAKEN FROM "GROWTH DISTRICT COMMUNITIES") [4]

The first mixed-use building in Gateway Park was completed in 2007, with the second completed in 2012. The first building, known as Gateway Park 1, "is fully occupied with graduate research laboratories, life science companies, state-of-the-art core facilities, and WPI's Corporate and Professional Education division."[5] Gateway Park 2 houses three of WPI's academic programs, laboratories, office space and classrooms.

The new WPI dormitory, which will house upperclassmen and graduate students, is located in the Gateway Park area. The specific location for the new residence hall in Gateway Park is 10 Faraday Street, at the intersections of Faraday, Grove, and Lancaster Streets as seen in Figure 2 below.

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[4] *Growth district communities, 2012*

[5] *Gateway Park at WPI*

FIGURE 2: GATEWAY PARK DETAILED MAP (TAKEN FROM GATEWAY PARK AT WPI)[6]

The site location is due to the expansion of the WPI campus into Gateway Park. Many WPI faculty members not only have offices that are located in the two current Gateway Park buildings, but also lab space in which seniors and graduate students work on research projects. This makes the location of the new dormitory ideal.

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[6] *Gateway Park Detailed Map*

2.0 Background

Construction projects involve many different factors, which are all crucial to the success of the project. The following paragraphs describe the important aspects that were involved in the design and construction of the new residence hall. The design needed to meet specific regulations set forth by the Commonwealth of Massachusetts and the City of Worcester before any construction could begin. Footings and foundations needed to be designed to support the loads of the building, while the structural design needed to be able to withstand gravity and lateral loads. Investigation into the site conditions for the location of the new building were an important step early on in the project. The resources for cost estimations are described, as well as LEED and its importance to promoting green design. With a better understanding of these elements, a proper design was able to be developed.

2.1 Layouts

In order to begin the process of structurally designing a building one must first understand the architectural layout of the building. The architectural grid and the occupancies of the spaces must first be defined in order to understand the loading of the structure.

For the Worcester Polytechnic Institute’s upperclassmen and graduate dormitory, Daniel O’Connell’s Sons (DOC) was hired to design and build the facility. The architect that was subcontracted by DOC is ADD Inc. Figure 3 and Figure 4 below show the architectural layout of the building. The second, third and fourth floor layouts are typical, thus only the second floor layout is shown.

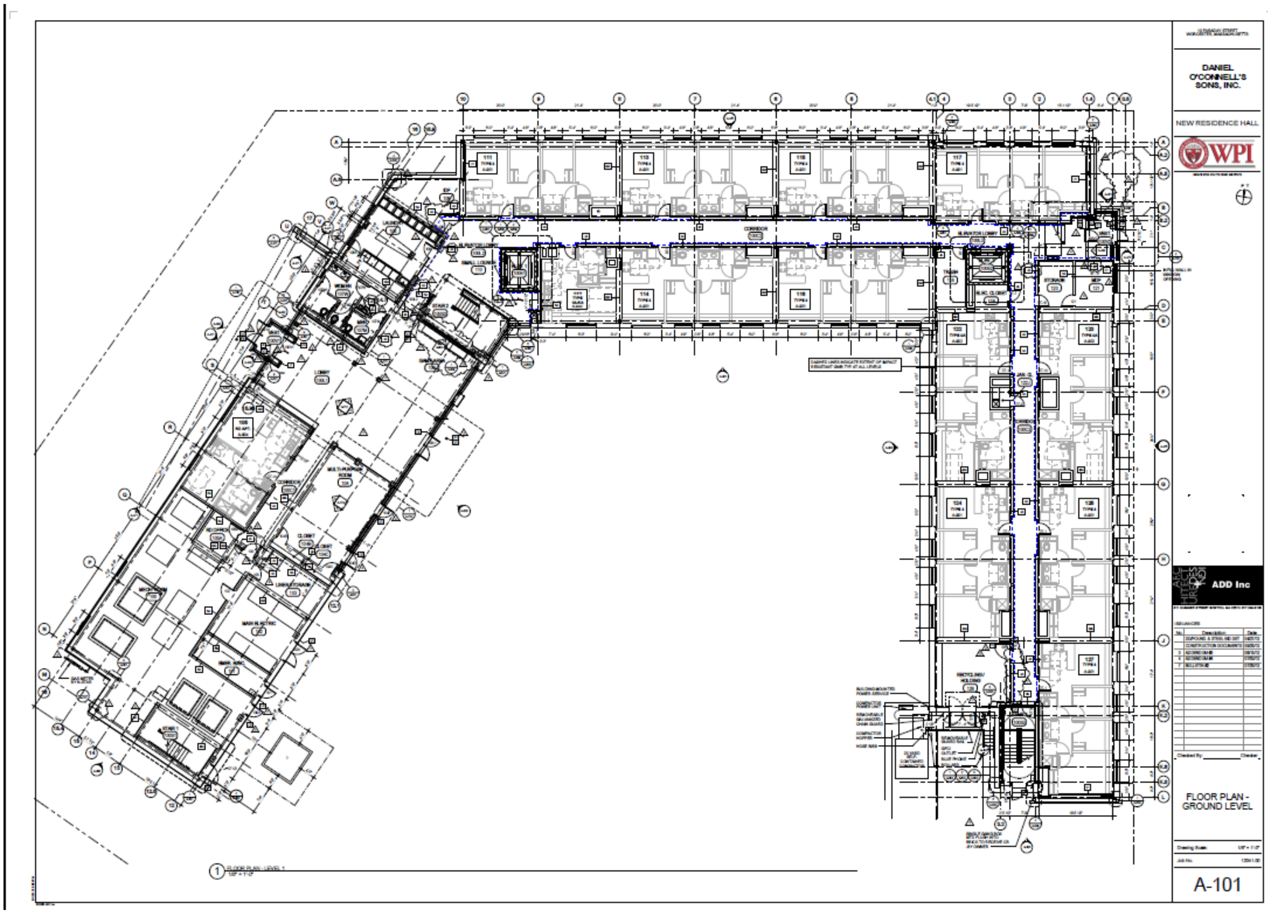

FIGURE 3: ARCHITECTURAL LAYOUT, FIRST FLOOR

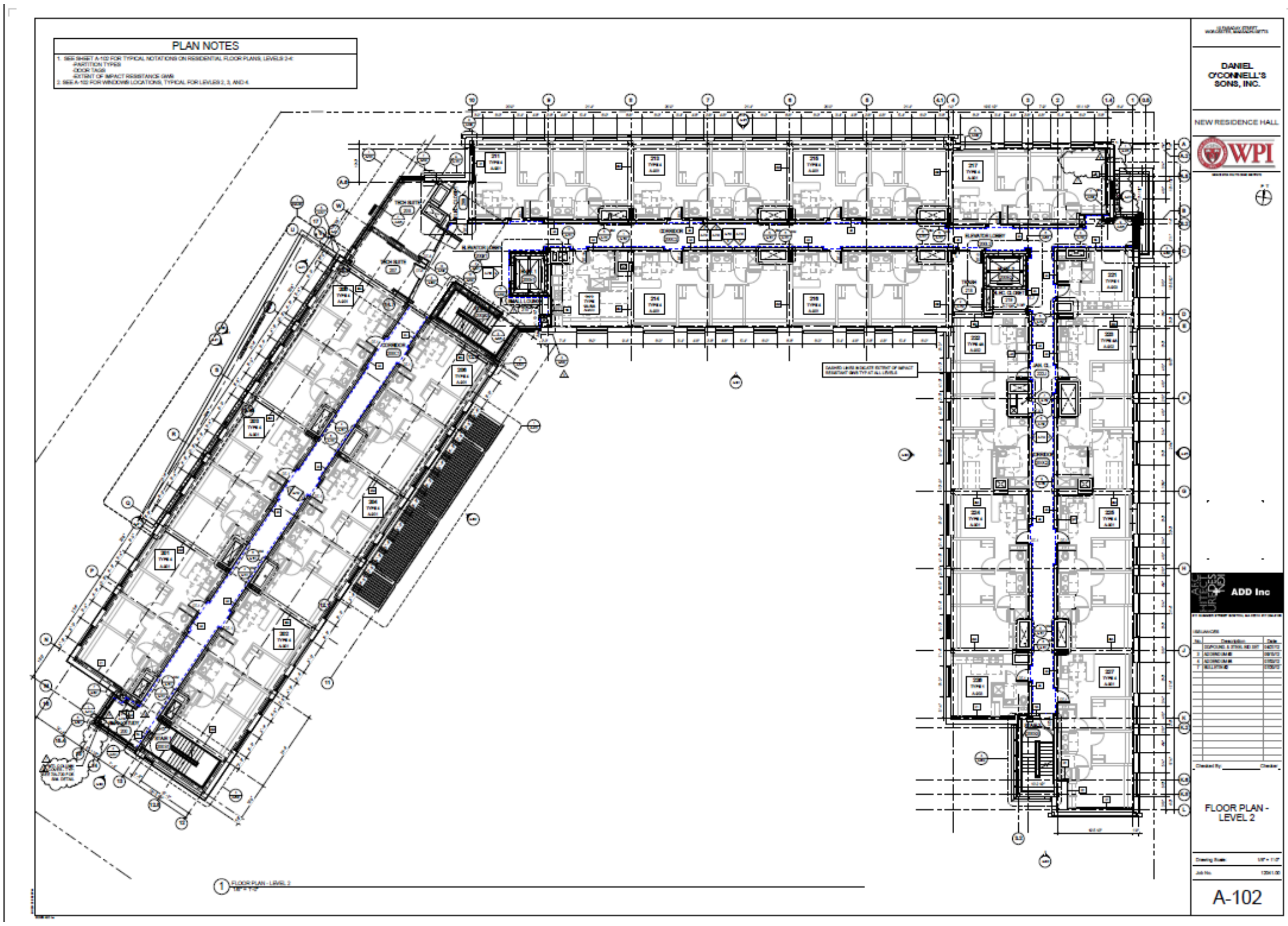

FIGURE 4: ARCHITECTURAL LAYOUT, SECOND FLOOR

Many applications were considered for laying out the architectural grid, such as building code standards, client needs and functional spaces, esthetics and much more.

2.2 MASSACHUSETTS STATE BUILDING CODE

The designs for the new dorm had to follow specific regulations set forth by the Commonwealth of Massachusetts. Every state has its own set of building codes which dictate certain safety requirements and other construction requirements. The *Massachusetts State Building Code* (*MSBC*) has been updated throughout the years and is currently in its eighth edition. The *MSBC* adopts the International Building Code in its entirety and then makes certain additions and deletions for what the lawmakers see fit as best for construction in the State. The *MSBC* is divided into 35 sections, each giving thorough descriptions of the regulations for that topic.[7] For the scope of this project, only a portion of these sections were used, including, but not limited to:

- Chapter 6: Types of Construction
- Chapter 7: Fire and Smoke Protection Features
- Chapter 8: Interior Finishes
- Chapter 9: Fire Protection Systems
- Chapter 10: Means of Egress
- Chapter 16: Structural Design
- Chapter 18: Soils and Foundations
- Chapter 19: Concrete
- Chapter 22: Steel
- Chapter 29: Plumbing Systems (State of Massachusetts, 2010)[8]

Following the building code was essential to the validity of the structural design and the design of the life safety aspects of the structure. Local building codes are important because they give the legal requirements for the design loads for different conditions such as wind and earthquakes. Not only do they establish structural design criteria, such as the minimum design loads that must be followed in Massachusetts, but they also give information on other design factors, such as the exit accessibility for buildings, bracing for frames, and deflection requirements. The code provisions reference documents which outline the required sprinkler types, pressures and flows, and design areas for sprinkler systems. The *MSBC* also outlines aspects such as the width and number of means of egress and the types of interior finishes which are allowed on the walls, ceilings and floors of buildings of this nature. This project, as well as DOC's plans that have already been created, have all been greatly influenced by the provisions of the *MSBC*.

[7] *State of Massachusetts, 2010*

[8] *State of Massachusetts, 2010*

2.3 ZONING

"Zoning is the process of planning for land use by locality to allocate certain kinds of structures in certain areas."[9] Zoning also places restrictions on different building aspects, such as the types of businesses that can be in an area, the height of buildings, the density, etc.[10] The City of Worcester Zoning Ordinance describes the regulations for construction within the different zones of Worcester. There are six major use types for the zoning in Worcester: residential, manufacturing, business, institutional, parks, and conservation areas. Use intensity is also used to subdivide the residential, manufacturing, and business districts.[11] Using Figure 5, it was determined that the location of the new dorm building, as well as all of Gateway Park, is located within the zoning district classified as BG-6.0.[12] BG 6.0 is a business district and has a maximum floor area ratio (FAR) of 6 square feet / 1 square feet.[13] This means that a building in this district may have a floor area that is six times larger than the land area of its lot. This simply means that this district encompasses many multi-story buildings.

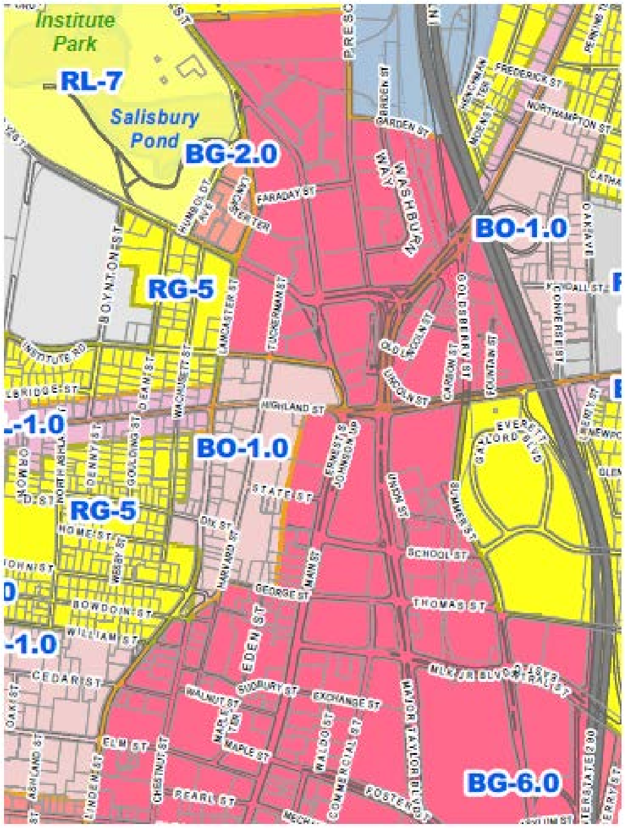

FIGURE 5: WORCESTER ZONING MAP (TAKEN FROM CITY OF WORCESTER (2012B).

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[9] *Murray, 2012*

[10] *Murray, 2012*

[11] *City of Worcester, 2012a*

[12] *City of Worcester, 2012b*

[13] *City of Worcester, 2012a*

The City of Worcester Zoning Ordinance (COWZ) also contains a table depicting the different permitted uses by zoning districts. This table states the different types of structures that can and cannot be built in the different zones. Table 1 shows a small portion of the whole table, but as you can see by the yellow highlights, dormitories are acceptable in the BG-6.0 district.[14]

In addition to limiting the FAR and users, the zoning ordinance gives specific requirements for building in the BG-6.0 district. For example, there is no specific height regulation, but there is a 10 linear foot rear yard setback.[15] Just as the *MSBC* influences building design, so does the City's zoning ordinance.

TABLE 1: PERMITTED USES BY ZONING DISTRICTS (TAKEN FROM CITY OF WORCESTER (2012A))

PERMITTED USES BY ZONING DISTRICTS – TABLE 4.1 RESIDENTIAL USE

	RS 10	RS 7	RL 7	RG 5	BO 1	BO 2	BL 1	BG 2	BG 3	BG 4	BG 6	ML 0.5	ML 1	ML 2	MG 0.5	MG 1	MG 2
1. Bed and Breakfast Establishment	SP	SP	SP	SP	SP	SP	SP	N	N	N	N	N	N	N	N	N	N
2. Continuing care retirement community	SP	SP	SP	SP	SP	SP	SP	SP	SP	SP	SP	N	N	N	N	N	N
3. Dormitory	SP	SP	SP	SP	SP	SP	Y	Y	Y	Y	Y	N	N	N	N	N	N
4. Family day care home	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y
5. Fraternity/sorority/ cooperative residence	SP	SP	SP	SP	SP	SP	Y	Y	Y	Y	Y	N	N	N	N	N	N
6. Group residence (general or limited)	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y
7. Limited Residential Hospice House	SP	SP	SP	SP	N	SP	N	N	N	N	N	N	N	N	N	N	N
8. Lodging house	N	N	N	SP	SP	SP	SP	SP	SP	SP	SP	N	N	N	N	N	N

2.4 Design Standards and Specifications

Building designs are based on different codes and regulations. The residence hall design not only follows the regulations and requirements set forth by the *MSBC* and the COWZ, it also relies heavily on guidance from the American Institute of Steel Construction (*AISC*), the American Concrete Institute (*ACI*), and American Society of Civil Engineering Standards (*ASCE 7*). The fourteenth edition of the *Steel Construction Manual* published by the *AISC* was used to acquire the different dimensions and properties for various structural products that are found in steel design.

[14] *City of Worcester, 2012a*

[15] *City of Worcester, 2012a*

The *Steel Construction Manual* provides the specifications that govern the evaluation of limit states for the design of members and connections. It provides geometric data for standard steel sections and design aids to facilitate proper sizing of members and connections. It was crucial in many different aspects of design, such as determining the minimum and maximum spacing of bolts and checking the various limit states.

ACI 318 provides minimum requirements for the design and construction of concrete structural members. *ACI 318*, similar to the Steel Construction Manual, governs the evaluation of limit states for concrete design. *ASCE 7* provides the current minimum design loads that can be used when designing a building. Live and dead loads are provided, such as the proper snow, wind, and seismic loading requirements.

In addition to the building and structural codes, life safety codes needed to be referenced, primarily codes and standards written by the National Fire Protection Association (*NFPA*). The primary codes that were referenced are *NFPA 13, NFPA 14, and NFPA 20* which are the standards for the installation of sprinkler systems, the installation of standpipes and hose systems, and the installation of stationary pumps for fire protection. These standards give requirements for sprinklers, standpipes and pipes in relation to sizing, materials, and application, amongst others.

2.5 Cost Analysis

Estimating the cost of a construction project is a difficult task. Using *RS Means* is one way to determine accurate cost estimations. *RS Means* is a construction estimation database that provides cost information to the industry. The information is based on U.S. national averages and is adjusted depending on location.[16] Labor costs, material costs, and equipment costs are all included. All of the information is determined based on many different factors, including the size of the project, season of the year, environmental considerations, quality of the work, etc.[17]

RS Means does have limitations, and in the case that certain cost values specific to the project could not be found, interviews with DOC were used. By acquiring lump-sum costs for the

[16] *Reed Construction, 2012*

[17] *Reed Construction, 2012*

dormitory project (i.e. electrical work or plumbing) by interviewing DOC members, an accurate total cost could be calculated and all of the needed cost values could be determined. Putting the cost into dollars/square foot was a good way to make comparisons with the square footage costs of other buildings on the Gateway Park site, as well as the rest of the WPI campus.

An important distinction to know is tangible versus intangible benefits. A tangible benefit is one which is relatively straightforward and can be accounted for, such as saving money. An intangible benefit is one that does not affect the bottom line of the person or organization; these benefits can be harder to define, such as student satisfaction. For this project, only tangible benefits were examined.

2.6 Footing and Foundation Design Factors

There are many different footing and foundation types that can be chosen for large scale construction. Foundations and footings are crucial, as it is their job to transmit loads from the structure to the ground. A footing is often the last structural element of the foundation that loads pass through. They have the important function of spreading out the superimposed load to produce bearing stresses that are within the capacity of the underlying soil.[18] The different factors that were considered in order to determine the best footing and foundation types included:

- Design load
- Soil bearing capacity
- Subsurface formations and the nature of the soil
- Site conditions
- Climatic conditions
- Economic considerations
- Superstructure type
- Special design requirements [19] [20] [21]

[18] *University of Maryland, 2004*

[19] *Chapter 5: Foundation Design*

[20] *Foundation…, 2012*

The development of Gateway Park has been a time-consuming process, due partly to the site conditions of the area. An issue with redeveloping this area was overcoming the stigma of the long industrial past of Worcester, which had led to contamination in the selected site.[22] Contaminants within the twelve-acre area included lead, arsenic, chromium, thallium, nickel zinc, petroleum compounds, polynuclear aromatic hydrocarbons, and several volatile organic compounds.[23] Before construction could begin, the ground needed to be cleared of these contaminants to make it a safe and usable site. Contaminants in the ground were time consuming and costly to remove since the ground must be excavated to depths usually far below the foundation depth. These soils must also be removed from the site and cannot be reused for fill on the site.

2.7 Leadership in Energy and Environmental Design (LEED)

LEED certifications can be met in a variety of ways that must be considered from the beginning of the design. The main LEED credits include sustainability on site, water efficiency, energy and atmosphere, materials and resources, indoor environmental quality, and regional priority. To obtain a LEED certification, the credentials must be addressed in the site selection, the design of the building and throughout construction. Specific applications of "green building" such as alternative roof designs can affect the structural design of the building. This is why it was critical to define the LEED credits in the beginning of the project.

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[21] *University of Maryland, 2004*

[22] *The Phoenix Awards*

[23] *The Phoenix Awards*

3.0 METHODOLOGY

The work accomplished in this report is structured into sections of focus. These sections include architectural layouts, structural steel design, structural precast concrete design, a comparison of precast concrete and steel design, lateral loading and bracing analysis, typical footing design, LEED discussion, sprinkler system design, building code analysis, scheduling, and cost estimation, as seen in Table 2 below.

TABLE 2: METHODOLOGY TOPICS, ACTIVITIES AND RESOURCES

Methodology Topics	Activities	Resources
Layout	Previous architectural layout	DOC
Structural Design (for precast concrete and steel)	Gravity loads	*ASCE 7* , DOC
	Typical bays	*LRFD*
	A-typical bays	*LRFD*
	Structural steel 3D model	Using *Autodesk Revit*
Steel Versus Concrete Design	Construction scheduling	DOC, Research
	Cost	DOC, RS Means
	Material availability	DOC, Research
	Design limitations	DOC, Research
	Safety	Research
Lateral Loading	Seismic loads	*ASCE 7*, Using *RISA-2D*
	Wind loads	*ASCE 7*, Using *RISA-2D*
	Steel bracing design	*ASCE 7*, Research, Using *RISA-2D*
Footing Design	Site conditions	DOC, Research
	Loading	Structural Design, *ACI*
LEED Discussion	Classifications and requirements	Research
Scheduling	Determination of tasks/milestones/duration	DOC, Research, Using *Primavera*
Cost Estimation	Material	DOC, RS Means
	Labor	DOC, RS Means
	Total cost/lazy s-curve	Research
Environmental	Investigation of impacts	Research
Social Implications	Interview	Courtyard-Motel Hotel General Manager
Political Implications	Interview	Worcester District Two Councilor
Fire Safety Investigation	Sprinkler system design	DOC, *NFPA 13*
	Code analysis	*MSBC*

3.1 ARCHITECTURAL LAYOUT AND STRUCTURAL DESIGN BASIS

The original layout for the building, provided by Daniel O'Connell's Sons (DOC), was used as a basis for structural design. The proper design values for gravity loading were calculated and obtained using the *International Building Code* (*IBC*) and *ASCE 7* and can be seen in Chapter 4.1. Special considerations were made throughout the gravity system designs to address areas of design which posed challenges due to geometry of the building and additional loading due to permanent structures on the roof. Effects of the mechanical units located on the roof, the effects of snow drifting, and the effects of the ascetic design on the loading of the building were all addressed. Calculations were performed by hand in order to determine the deflection and loading values for the considered areas. With gravity loading determined, initial structural designs were prepared for both steel and precast concrete systems. A BIM (building information modeling) model for the structural steel design was created using *Revit*.

3.2 Steel Design versus Precast Concrete Design

Two designs were developed, a precast concrete design and a structural steel design. During the design process, key findings and assumptions were made and are outlined in Chapter 4. These findings were used to compare both designs. Some of the key components that were compared were cost, material availability, time, design limitations and safety factors. Costs of the two designs were determined using *RS Means*. Scheduling was another important factor for comparison, and the relevant information was acquired from specific research. Ductility and additional safety procedures were also factored into the comparison, such as spray fireproofing with steel structures.

3.3 Lateral Design

Lateral loading was taken into account once the gravity framing for each material was completed. The effects of wind loading and seismic loading and the implications on the steel design were examined. The major challenge that was faced with seismic loading was caused by the shape of the residence hall. The building had to be split into two separate wings before seismic loads could be determined. Similarly, the profile of the building presented a challenge when investigating the wind loads; wind from all directions had to be considered. In order to perform a proper wind load analysis, the building had to be split into two wings, just as it was for seismic loading. With separate wings, the maximum load values caused by wind and seismic could be determined and accounted for.

3.4 Footing Design

Typical footings were designed for this project in order to further investigate aspects of the structural design process. Reinforced concrete footings were designed in accordance with *ACI* standards and *MSBC* requirements.

3.5 LEED Investigation

An investigation into LEED design criteria was performed to provide an outline of criteria that the building must meet in order to become LEED certified. Detailed descriptions of the credits to be met were provided.

3.6 Construction Schedule

Information was obtained from DOC in order to create a construction schedule. This was a condensed version of DOC's original schedule. The construction schedule was used to demonstrate both the importance of time, due to the fast-track schedule, and how the procurement of materials factors into the comparison of steel versus precast concrete. *Primavera* software was used to create a schedule and bar chart in order to create a visual that highlights the fast-track schedule.

3.7 Cost Estimation

A final cost estimation was performed which included the cost of the structural design and labor required to construct. The costs from the structural design incorporated the costs of material, such as the cost of steel per ton, which were obtained from *RS Means*. Labor cost was based on the quantities of anticipated work for the construction processes and the corresponding labor cost rates.

3.8 Environmental

Environmental factors were also considered in this project. These included various methods to prevent construction runoff into the neighboring areas. The discussion also included the benefits of each method.

3.9 Political and Social Impacts

Interviews were conducted with the general manager of the Courtyard Marriott Hotel, and the District Two Councilor for the City of Worcester in order to determine, the positive and negative, social and political impacts of the new dormitory and Gateway Park on the City. These interviews also helped us to determine if there was any political action taken from WPI in order to acquire the land.

3.10 Sprinkler System Design

A typical sprinkler system per *NFPA 13* requirements was designed for the residence hall based on occupancy hazards and commodity classifications. *NFPA 13, Standard for Installation of Standpipes and Hose Systems, and NFPA 20, Standard for Installation of Stationary Pumps for Fire Protection* were referenced for those aspects of a sprinkler system that are required by either *NFPA 13* or the *MSBC*. Through this design, different options were chosen and processes were compared to determine which materials would provide the designer with the most flexibility with respect to cost and constructability. The options in this comparison included different piping materials, sprinkler sizes, and pump sizes.

3.11 Code Review

The drawings and layouts provided by DOC, labeled as FP-xxx or A-xxx, were evaluated for their compliance with various sections of the *MSBC*. Only select architectural drawings were necessary for this evaluation. The sections evaluated included height and area requirements, use and occupancy characteristics, and means of egress and fire protection ratings. Through this evaluation, a series of charts and tables were developed in order to show the adequacy of the different building features that are regulated by the *MSBC*.

4.0 Structural Design Criteria and Assumptions

The following chapter provides the procedures for the design of specific elements. This chapter includes the processes to determine structural design loads, a-typical snow loading due to drift, lateral loading, as well as steel and precast concrete structural design.

4.1 Structural Design Loads

Loading for this project was based on *LRFD* design. Before any sizing of structural steel or precast concrete elements began, values or calculation strategies for the various design loads were established. *2009 International Building Code* (IBC) Table 1607.1 was the source used to calculate the live loads that would impact building design, shown in Table 3.

TABLE 3: IBC TABLE 1607.1 LIVE LOADING USED

Type of Live Load	Distributed Load (psf)
Roof Live Load	20
Residential Live Load	40
Corridor Live Load	100

Dead loads were more involved than simply using a table. There were several different point loads caused by the mechanical equipment on the roof that needed to be included. The values were gathered from the technical drawings created by DOC. These different concentrated loads affected the moments at various points along the building frame, and, therefore, required a more detailed analysis at their locations. Along with these, dead loads outlined below are exerted on the roof framing. Table 4 portrays the loads used to account for the distribution of MEP (mechanical, electrical, plumbing), ceilings, and insulation throughout the building.

TABLE 4: DEAD LOADING USED

Type of Dead Load	Distributed Load (psf)
MEP	5
Ceiling	3
Insulation	2

The self-weight of the composite decking on the roof and each floor also had to be estimated and added to the dead load. For the steel design, a value of 62.4 pounds per square foot (psf) was calculated to account for the effect of concrete ponding (using 10% of the concrete weight), as shown in Appendix B.1 Calculations for Composite Decking.

4.1.1 Snow Loads

When analyzing the design snow loads for the new residence hall, the first value that needed to be determined was the ground snow load p_g. Using Figure 7-1 from Chapter 7 of *ASCE 7*, the value of p_g=50 psf was determined for Worcester, Massachusetts. Next, the flat roof snow loads were determined using information from *ASCE 7*, seen in Table 5, and in Equation 1.

EQUATION 1: FLAT ROOF SNOW LOAD

$$p_f = 0.7C_eC_tIp_g$$

TABLE 5: SNOW LOAD FACTORS

Factor	Definition	*ASCE 7* Table	Value
C_e	Exposure Factor	7-2	0.9
C_t	Thermal Factor	7-3	1.0
I	Importance Factor	7-4	1.0
p_f = 31.5 psf			

The new residence hall has roofs that are slightly slopped, but because their slope is less than 5°, they could all be placed in the flat roof snow load (p_f) category (*ASCE Chapter 7*).

Varying roof heights can create larger snow loads due to snow drifting. Snow drifts are much more complex than simple flat roof snow loads. Drifts accumulate differently depending on whether the higher-level roof is on the leeward side or the windward side, as shown in Figure 6.

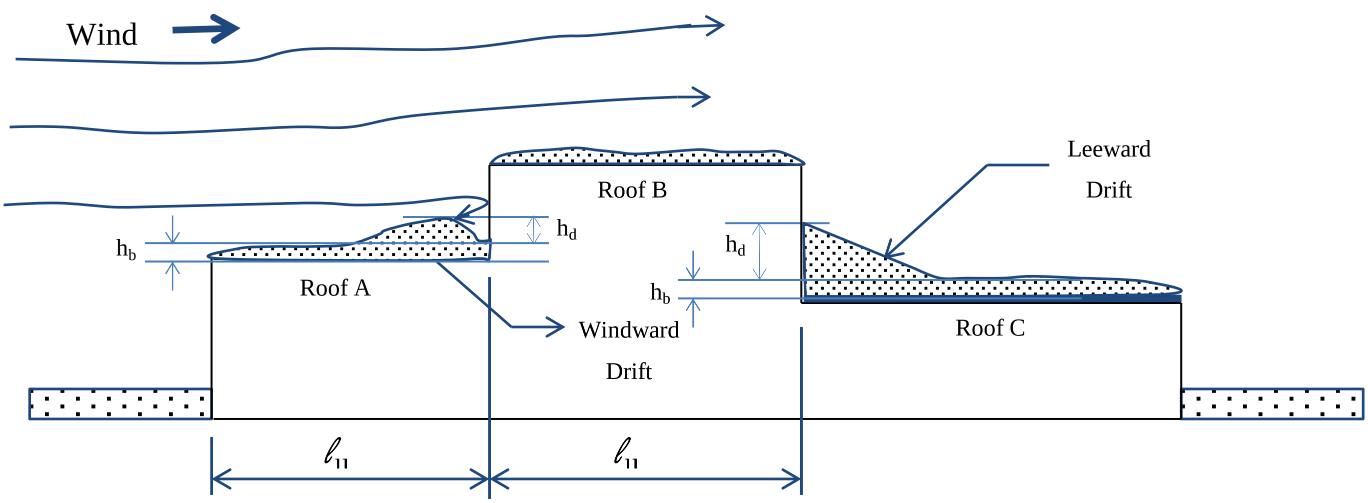

FIGURE 6: LEEWARD AND WINDWARD SNOW DRIFTS, ADAPTED FROM *ASCE 7*

Leeward drifts often create a triangular shaped drift, while windward drifts usually start out as a quadrilateral shape due to the fact that the wind creates a vortex as it collides with the vertical wall directly past the drift. As the height of the drift increases and the wind is redirected over the top of the wall, the drift begins to turn into a triangular shape, as shown in Figure 7. With the varying roof heights, the wind direction dictates whether the length of the lower roof or that of the higher roof will be used as the length of the roof upwind of the drift, l_u. Using l_u and p_g, the drift height (h_d) can be determined using Equation G7-3 from *ASCE 7*:

EQUATION 2: SURCHARGE DRIFT HEIGHT

$$Surcharge\ Drift\ Height, h_d = 0.43\sqrt[3]{l_u}\sqrt[4]{p_g + 10} - 1.5$$

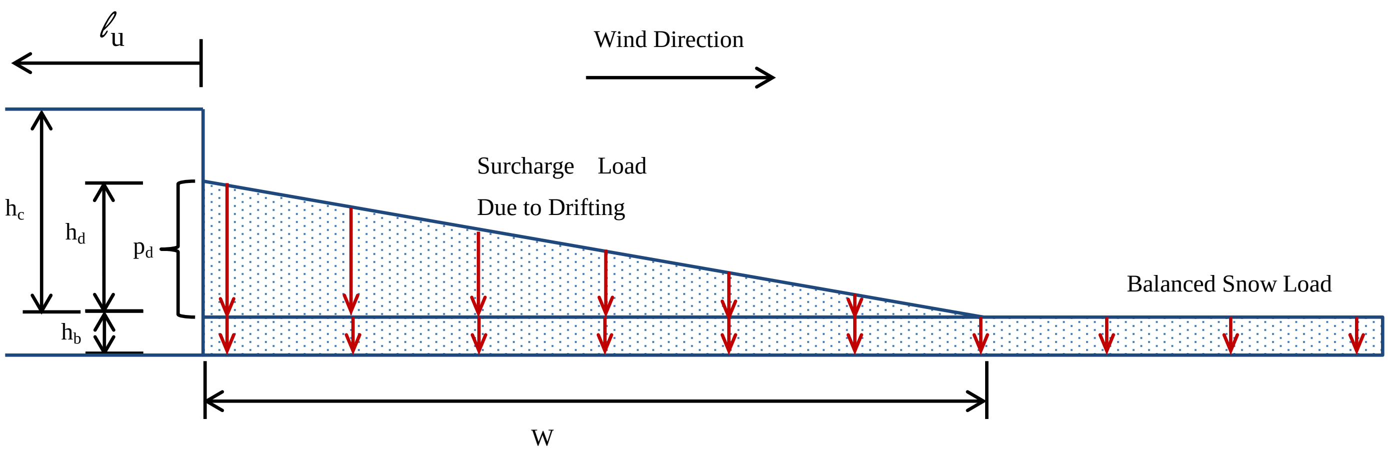

FIGURE 7: SURCHARGE LOADING DUE TO SNOW DRIFT INCLUDING BALANCE SNOW LOAD, ADAPTED FROM *ASCE 7*

Using the calculated drift height, the average snow loads were determined for the areas where drifting was assumed to occur (calculations can be found in Appendix B.2 Calculations for Snow Loading). Following a conservative approach, the location with the largest drift height was used to calculate the maximum load due to drifting for application to the entire roof. For this value, the average load of the maximum value (h_d+h_b) and the minimum value (balanced snow load, which equals the flat roof snow load) was used to approximate the expected snow loading pattern:

$$\frac{120.7\ psf + 31.5\ psf}{2} = 76.1\ psf \approx 80\ psf$$

One exception to the snow loading was the area inside the steel panels surrounding the mechanical equipment on the west wing of the building. Here a conservative value of 120 psf (maximum snow load due to drifting) was used to account for snow build up due to drifting within the confined space and the complex wind effects that may be created by the surrounding steel panel heights.

With the intensity of the snow loading determined, the next step was to calculate the area that the larger snow loads encompass, as shown in Figure 7. The drift width was equal to $4h_d$. The one exception is if the drift height, h_d, is larger than h_c (the height from the top of the balanced snow load to the top of the adjacent roof), then the drift width, w, equals ${4h_d}^2/_{h_c}$, and the drift height simply equals h_c (h_d=h_c) (*ASCE Chapter 7*). Based on these geometric relationships, a width of 8 ft. was used along the parapet, and in order to keep a uniform width, 8 ft. was used as the width around the mechanical equipment as well.

The drift area and the unit weight of the snow were easily found using the equations below.

EQUATION 3: DRIFT AREA

$$Drift\ Area = \frac{1}{2}h_d w = 2(h_d)^2$$

EQUATION 4: UNIT WEIGHT OF SNOW

$$Unit\ Weight\ of\ Snow\ (\gamma) = 0.13p_g + 14 \leq 30\ {}^{lb}/_{ft^3}$$

4.1.2 Lateral Loads

An additional aspect to the design of the residence hall was design of the lateral force resisting system. The lateral force resisting system is crucial in resisting lateral loads such as wind loads and earthquake loads. Wind and earthquake loads force the building to act as a cantilever stemming from the ground. These lateral forces affect a building more as the building increases in height. Buildings with less height may not be as greatly affected but the lateral stability of the building still needs to be checked during the design. Without any lateral stability, the building would give way to an overturning moment and collapse. For this reason, there are frames and cross bracing provided within the structure to offset these loads. The application of wind and seismic loading on the frame can be seen below in Figure 8 and Figure 9.

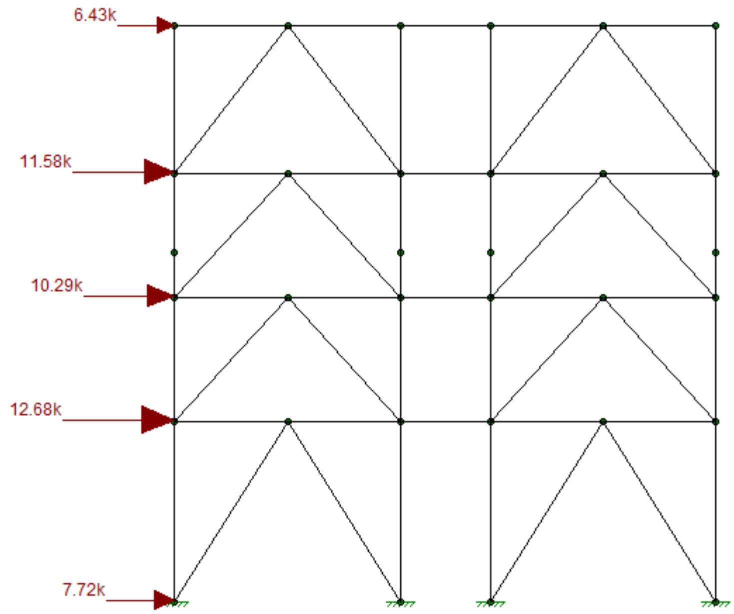

FIGURE 8: VISUAL OF WIND LOADS APPLIED TO FRAME

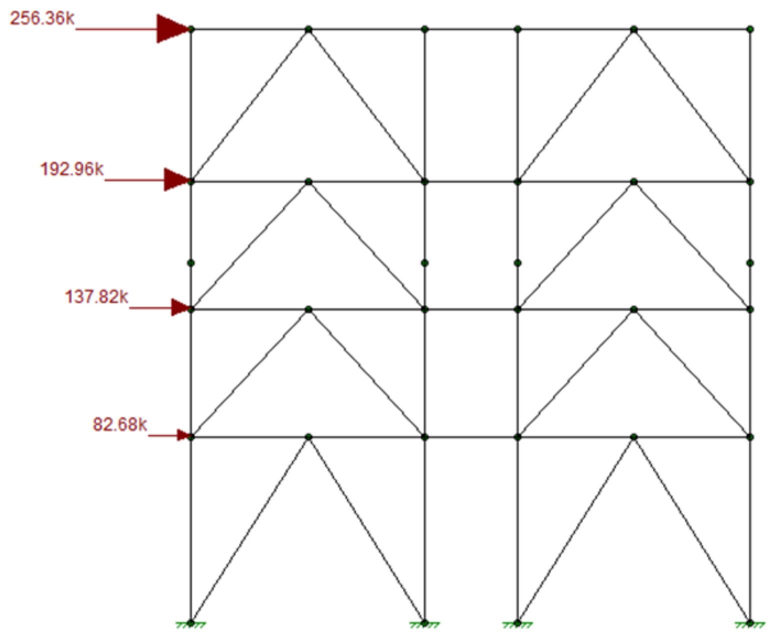

FIGURE 9: VISUAL OF SEISMIC LOADS APPLIED TO FRAME

On the exterior faces of the building, the walls act as slabs and transfer the forces into the girder and beams. The wind pressures and earthquake loading calculated according to the *Massachusetts State Building Code* can be seen in the examples above, Figures 8 and 9. The wind pressures are applied to the building based on whether or not the wall is a windward or leeward wall. Using these pressures, the size of the girders and beams which will carry the load were determined according to the wind loading. Additionally, the earthquake loading was addressed, even though the region is not prone to large earthquakes.

Lateral forces due to wind loading were examined first. *ASCE 7* outlines three different methods that can be used to design buildings and their components to withstand the code-specified wind loads.

1. "Method 1 – Simplified procedure for low-rise simple diaphragm buildings"[24]
2. "Method 2 – Analytical procedure for regular shaped buildings and structures"[25]
3. "Method 3 – Wind tunnel procedure for geometrically complex buildings and structures"[26]

The new residence hall required the second method to analyze wind loading because it did not meet the specifications set forth by Method 1. Due to the various wings of the building, the logical way to examine the wind loads was to look at each wing individually. Many coefficients were determined using various *ASCE* tables and figures in order to, ultimately, determine the design wind pressure, p or the design wind load, F.[27] As used when examining snow drifts, the windward wall refers to the wall located on the upwind side, while the leeward wall refers to the wall on the downwind side of the building.

For the wind loading analysis, the building was split into two sections, the main building and the west wing. The main building is made up of the Faraday St. and Grove St. wings, while the west wing consists of the Lancaster St. wing. For a thorough analysis, the different wind pressures on the building were determined for eight different scenarios, as shown in Table 6. The number of frames for each scenario are also displayed.

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[24] Buyukozturk

[25] Buyukozturk

[26] Buyukozturk

[27] Buyukozturk

TABLE 6: WIND DIRECTION SCENARIOS

	Main Building	Frames	West Wing	Frames
Wind Direction	East to West	10	East to West	7
	West to East	10	West to East	7
	North to South	9	North to South	3
	South to North	9	South to North	3

Before the wind pressure could be calculated, the necessary coefficients had to be determined using *ASCE 7*. Table 7 displays the crucial factors used and where in *ASCE 7* they can be found.

TABLE 7: WIND LOADING COEFFICIENTS, TAKEN FROM *ASCE 7* CHAPTER 6

ASCE 7, Chapter 6	Section	Figures/Tables	Values	
Enclosure Classification				
Classification	6.5.9		Enclosed	
Basic Wind Speed				
V (ft/s)	6.5.4	Figure 6-1	146.67	
Wind Load Importance Factor				
I_w	6.5.5	Table 6-1	1	
Wind Exposure Category				
B	6.5.6.2/6.5.6.3		B	
Wind Topographic Factor				
K_{zt}	6.5.7		1	
Internal Pressure Coefficient				
GC_{pi} ±	6.5.11.1	Figure 6-5	0.18	
Wind Directionality Factor				
K_d	6.5.4	Table 6-4	0.85	
Velocity Pressure Exposure Coefficient				
K_z	6.5.6	Table 6-3	0.85	
Gust Effect Factor				
G	6.5.8.1		0.85	
Velocity Pressure				
q_z (psf)	6.5.10	39.79		
q_h (psf)	6.5.10	39.79		
Design Wind Pressure			$(+GC_{pi})$	$(-GC_{pi})$
p (psf) Windward	6.5.12		19.89	34.22
p (psf) Leeward	6.5.12		-24.07	-9.75
p (psf) Side Walls	6.5.12		-30.83	-16.51
Wall/Roof Pressure Coefficient				
Windward C_p	6.5.11.2.1	Figure 6-6	0.8	
Leeward C_p	6.5.11.2.1	Figure 6-6	-0.5	
Side Wall C_p	6.5.11.2.1	Figure 6-6	-0.7	

Depending on which section of the building was being analyzed, the windward, leeward, and side walls varied. Figure 10 depicts the locations of windward, leeward, and sidewalls when analyzing the main building (Faraday and Grove St. Wings) with wind moving from east to west.

FIGURE 10: DIVISION OF BUILDING FOR WIND ANALYSIS AND DEPECTION OF WINDWARD, LEEWARD AND SIDEWALLS

With the necessary coefficients accounted for, the design wind pressures were calculated using the following equation found in Section 6.5.12.2.1 of *ASCE 7*:

EQUATION 5: DESIGN WIND PRESSURE

$$p = qGC_p - q_i(GC_{pi})(^{lb}/_{ft^2})$$

The different values for C_p (roof/wall pressure coefficient) depending on whether it is a windward, leeward, or side wall account for changes in wind directions that are experienced. GC_{pi} is a positive and negative value, so when calculating p, both options were analyzed, which resulted in a positive and negative value of p, in which the maximum absolute value out of the two was used. The negative values of pressure, often felt on the leeward and side walls of the building, indicate that the walls are experiencing suction as the wind is acting away from the surface, unlike the pushing force felt when the design wind pressure is a positive value.

The pressure was changed into a load (kips) using the tributary area for each level of the building. Figure 11 depicts an example of how the tributary area was determined for a location where the pressure was being applied. For the windward side in the example, the tributary area would be 20’ x 10’ = 200 ft^2.

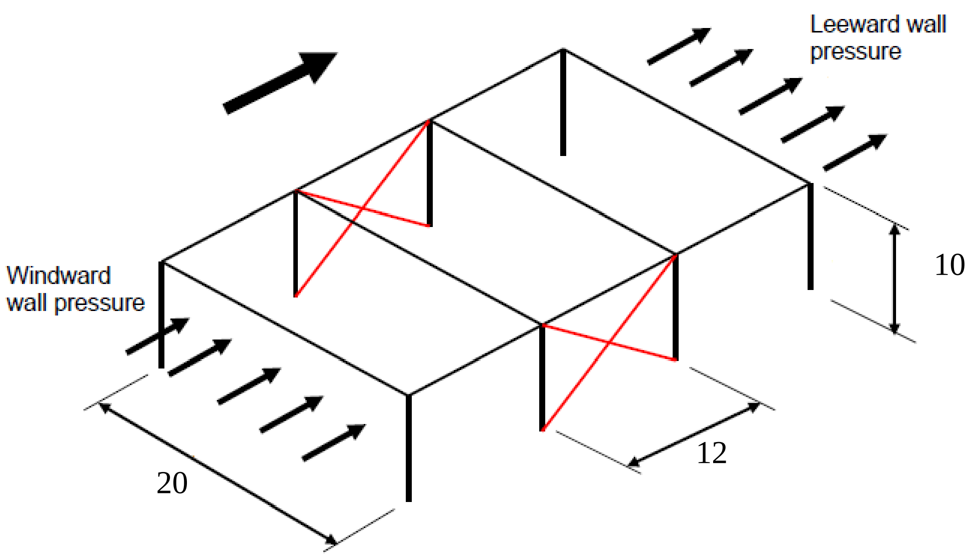


FIGURE 11: EXAMPLE OF TRIBUTARY AREA DURING WIND LOADING, TAKEN FROM *LATERAL FORCE RESISTING SYSTEMS-BRACED FRAMES*

A crucial step in the design of the lateral bracing and the analysis of the frames, was that the wind load calculated for the different wind scenarios needed to be divided by the number of frames for that specific case. The frames used to in the lateral bracing analysis were the same as those used by DOC, as shown in Figure 12. The red lines represent the different frames. For the case described above (Figure 10), with wind hitting the main building: Faraday and Grove St. Wings from east to west, the number of frames is ten. The frames that are parallel to the wind direction were counted because they are the ones being used to help prevent deflection and counteract moment and axial forces. The wind loading experienced by each frame was then determined using the spreadsheet found in Appendix C.1 Sample Spreadsheets for Wind Loading, and the results were used for a *RISA-2D* analysis.

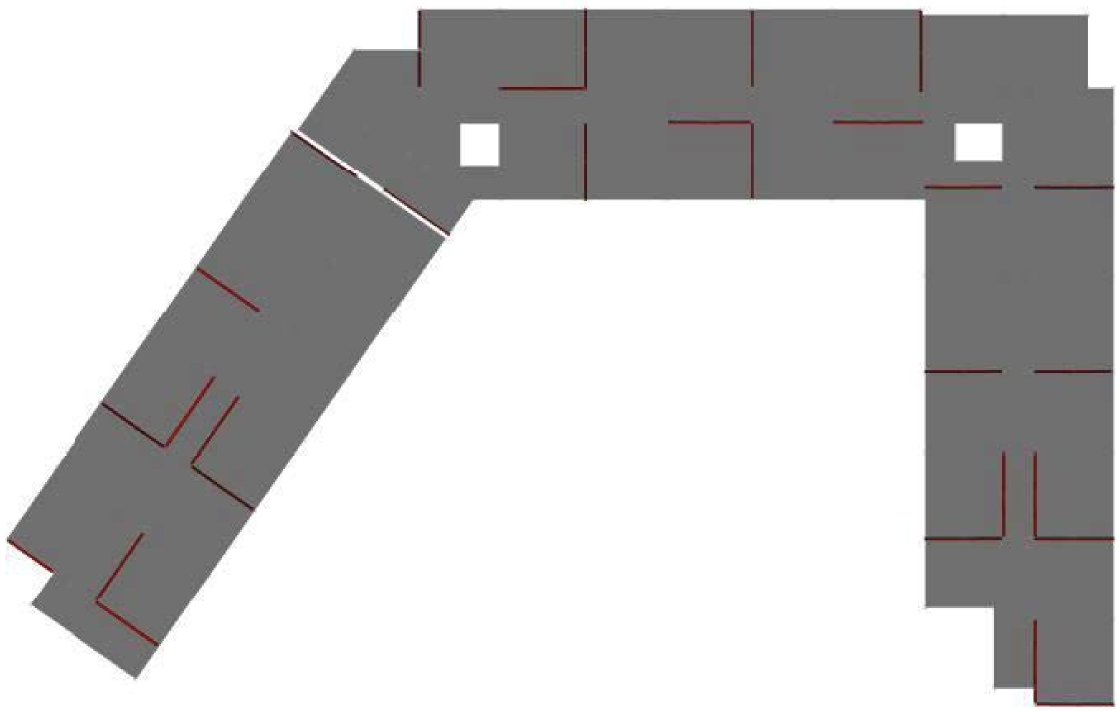


FIGURE 12: LATERAL BRACING FRAME LOCATIONS

The lateral forces due to seismic activity were also important to investigate. The method for determining the loading due to seismic is defined in Chapters 11 and 12 of *ASCE 7*. These chapters outline the essential equations and how to determine the coefficients used within those equations. The different variables and coefficients required to complete this analysis can be seen in Table 8.

TABLE 8: SEISMIC LOADING COEFFICIENTS, TAKEN FROM *ASCE 7*

Coefficient	Value	*ASCE 7*
S_s	0.240	Fig. 22-1
S_1	0.067	Fig. 22-2
F_a	1.6	Table 11.4-1
F_v	2.4	Table 11.4-2
S_{MS}	0.384	Sect. 11.4.3
S_{M1}	0.161	Sect. 11.4.3
S_{DS}	0.256	Sect. 11.4.4
S_{D1}	0.107	Sect. 11.4.4

Coefficient	Value	*ASCE 7*
T_0	0.08	Sect. 11.4.5
T_s	0.42	Sect. 11.45
T_L	6.00	Fig. 22-15
I_e	1.00	Table 11.5-1
Seismic Design Category	B	Table 11.6-1
Response Modification Factor, R[a]	3.25	Table 12.2-1
C_s	0.08	Sect. 12.8.1.1
C_u	1.7	Table 12.8-1
C_t	0.003	Table 12.8-2
x	0.75	Table 12.8-2
h_n	70	
T_a	0.06	Sect. 12.8.2.1

With the coefficients known, the following equation was used to determine the seismic base shear of the building:

EQUATION 6: SEISMIC BASE SHEAR, TAKEN FROM *ASCE 7*

$$V = C_s W, \text{ where } C_s = \frac{S_{DS}}{R/_I}$$

Once the base shear was known, determining the forces due to seismic activity required the use of the following equation:

EQUATION 7: SEISMIC FORCES PER LEVEL, TAKEN FROM BUYUKOZTURK

$$F_x = V_B \frac{W_i h_i}{\sum_{i=1}^{n} W_i h_i}$$

- F_x= Force at given height
- V_B= Base shear
- W_i=Story weight
- h_i=Story height
- n=Number of stories

The story heights of the building were known, while the weights were more challenging to determine. A spreadsheet was developed and used (as seen in Appendix C.2 Sample Spreadsheet for Seismic Loading) to determine the weight of each floor (W_i), including the weight of the beams and supported mechanical equipment (if any). Once the loading for each level of the building was determined, the loads were summed to check if they equaled the base shear and were calculated correctly, as shown in Table 9.

TABLE 9: SEISMIC LOADING VALUES PER LEVEL

V (lbs)	669,834	670		
	lbs	kips		
Fx_2	82,684	83		
Fx_3	137,823	138		
Fx_4	192,963	193		
Fx_{roof}	256,365	256		
	Total	670	Total	Check

With both seismic and wind loading analyzed, the lateral bracing for the steel design of the dormitory was investigated. Steel bracing members were determined by investigating which member sizes could handle the calculated lateral loads.

4.2 Structural Steel Design

In order to complete the design of the steel structure, some key assumptions were made. The snow load on the roof varied as discussed previously due to snow drift. In order to account for the snow load during design, the larger more conservative snow load value of 80 pounds per square foot was used uniformly on the roof adjacent to the mechanical area.

The loading used for steel design can be seen below in Table 10: Typical Roof Loading Conditions and Table 11: Typical Floor Loading Conditions.

TABLE 10: TYPICAL ROOF LOADING CONDITIONS

Loading	Value (psf)
Dead Load	72.37
Snow Load 1	80
Live Load	0
Roof Live Load	20

TABLE 11: TYPICAL FLOOR LOADING CONDITIONS

Loading	Value (psf)
Dead Load	72.37
Snow Load 1	0
Live Load	100

The beams and girders were designed using full composite action and unshored construction. Composite action increases the stiffness and the moment capacity of the member, thus leading to a reduction in member sizes.[28] This reduction lends composite action to become a more favorable selection due to the reduction in cost from decreasing member sizes, even though typically more studs are required in full composite action. The construction of this building was assumed to be unshored, meaning it will not require temporary bracing to support the weight of the

[28] Caprani, Colin

beams, girders, decking, and wet concrete during construction. This means that the beams and girders had to be designed to withstand higher live loads due to construction and the ponding of concrete before it hardens and acts compositely with the steel.

The first step of the design process was to design the beams of the steel structure. Sample beam calculations can be seen in Appendix B.3 Sample Calculation for Steel Design, and the spreadsheet that was developed to streamline the calculations can be found in Appendix C.3 Sample Spreadsheet for Beam Steel Design. The roof beams were designed first, with the loading cases specified for the roof. All beams were designed in the same fashion, with the varying lengths and loading conditions. The exception to this was the beams directly under the mechanical equipment screen wall. As seen in an example provided in Figure 13: North Elevation View of Mechanical Roof Column and Figure 14: East Elevation View of Mechanical Roof Column below, the columns that support the screen wall are connected to the supporting beams underneath.

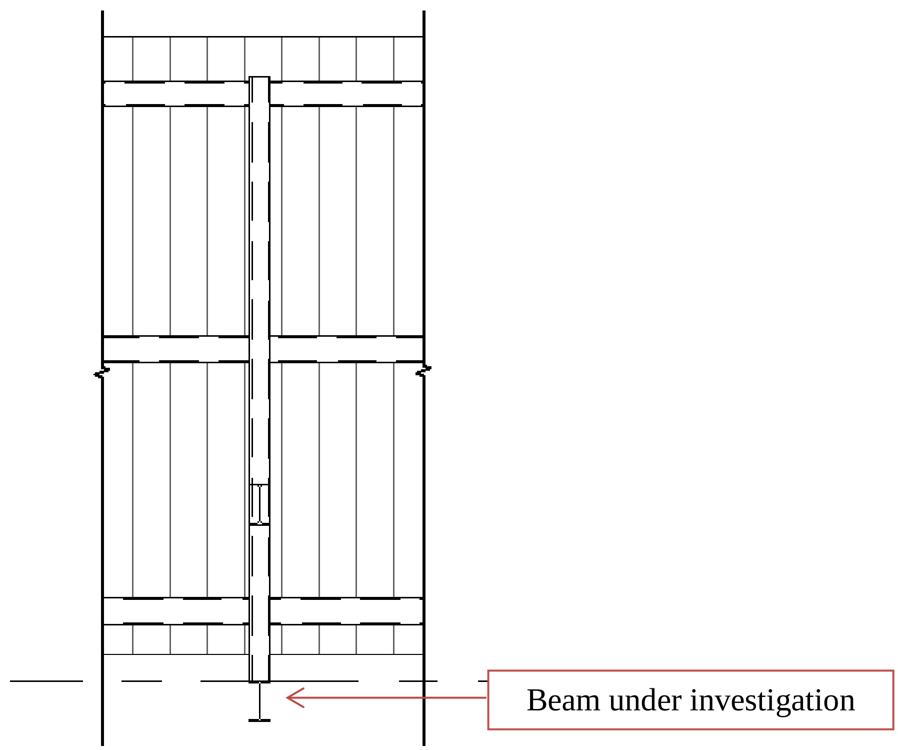


FIGURE 13: NORTH ELEVATION VIEW OF MECHANICAL ROOF COLUMN

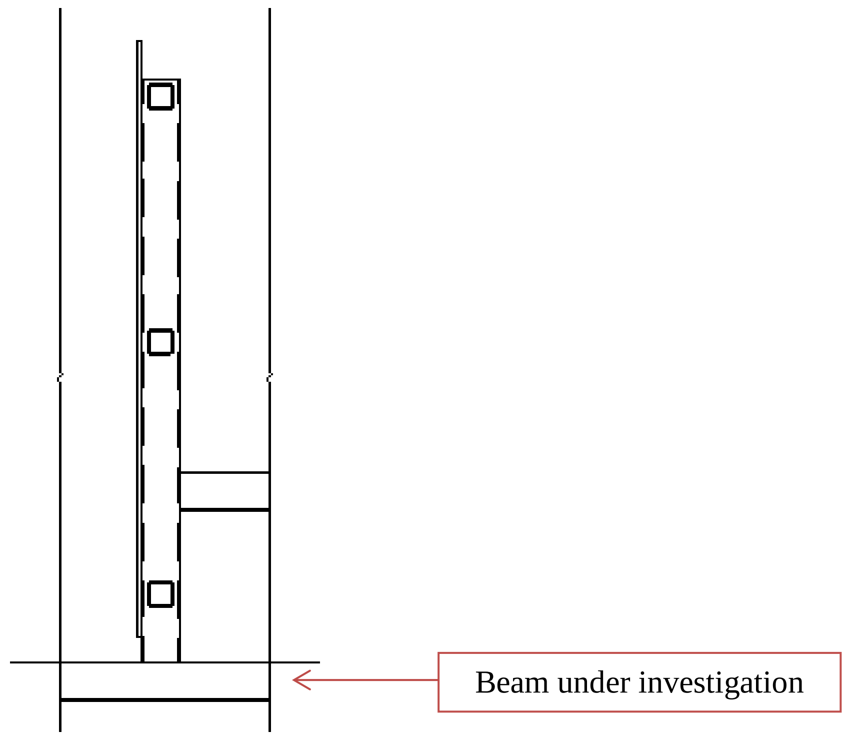


FIGURE 14: EAST ELEVATION VIEW OF MECHANICAL ROOF COLUMN

The column creates a point load on the beam that includes the weight of the column, the weight of the perforated metal screen wall that the column carries, the weight of the girders that support the perforated metal screen wall, and the beam under the mechanical unit that is carried by the column. The supporting beam had to be designed to handle the additional point load. The calculations performed to determine the required beam sizes are shown in Appendix B.4 Sample Calculation for Steel Beam Under Mechanical Screen Wall.

Once the beams were checked for capacity and deflection before and after concrete hardening, the self-weight of the beam was added into the dead load and a second set of calculations were performed to check the capacity and the deflection to ensure the beam is sufficient.

The composite girders were designed in the same fashion as the typical beams except the key difference is that the girders must support the weights of the beams. The beam weight was added into the dead load to determine the girder size needed for sufficient capacity and deflection performance. Again, it was necessary to perform the calculations iteratively by adding the girder's self-weight into the dead load to check that the girder is sufficient. The spreadsheet developed for the design of the girders can be found in Appendix C.4 Sample Spreadsheet for Girder Steel Design.

When designing the beam and girders on the floors below the roof, it was necessary to change the loading to match the floor in which they were located. As discussed in Chapter 4, the loadings on the floor levels were residential live loads and corridor live loads from the *2009 International Building Code*. Floor levels two through four all had the same loading; therefore, it was not necessary to design unique girders and beams for each floor; the beams and girders were designed for one floor and then used for the other floors. The only exception to this was on floor level three. At this level, the exterior girders include a relieving angle which supports the brick work for the top two floor levels. An additional calculation was performed for these girders, in which the total weight of the bricks per linear foot was determined and then added into the dead weight calculation. Once carried out, it was determined that the girders that were originally selected for the other floors were sufficient to carry the additional loading of the bricks.

For both the girder and the beam design, certain members were used throughout the building in order to promote ease of construction or constructability. As an example, certain beam comparisons indicated that the span of the beam changed causing a smaller beam to be sufficient for capacity and deflection. However, for constructability, the same beam size was selected for both cases.

In order to design the columns, it was necessary to complete the design of the beams and the girders. This is because the columns were designed based on carrying the total combined weight of the girders, beams, dead loads, floor and roof live loads, and rood snow loads. The vertical column consists of two W shapes spanning the total height of the building. The bottom most column segment is 30’ 8” tall. The columns were designed as gravity columns, and the spreadsheet that was developed can be found in Appendix C.5 Sample Spreadsheet for Column Steel Design.

4.2.1 Steel Bracing

With the steel structural framing defined, and all of the lateral and gravity loads determined, the next step was to establish the proper lateral bracing members. This step required the use of *RISA-2D* software to evaluate the different frames and determine the axial forces and moment values on the columns and braces. The frame was drawn using the girder sizes which were found using the aforementioned methods. The trial lateral bracing member size was an HSS6X6X5/16. This size was selected because it was reasonable compared to the other member sizes; it was also the same member size used by DOC in their lateral bracing design. The load analysis was done for each *LRFD* combination for each of the three types of frames within the building, shown below in Figure 15.

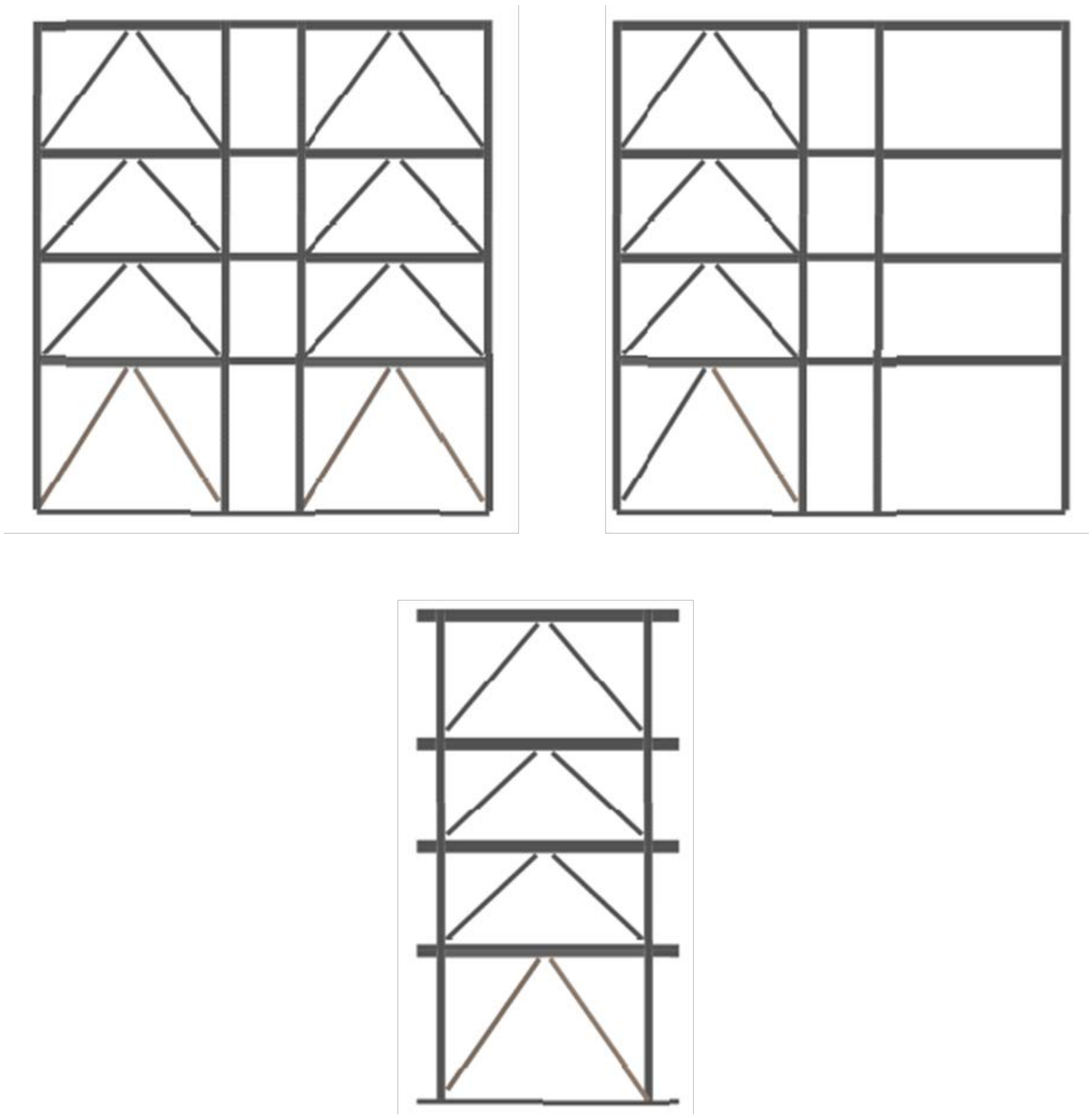


FIGURE 15: THREE TYPICAL LATERAL BRACED FRAMES

RISA-2D was used to check if this bracing was adequate to handle the calculated loading and ensure minimal deflections, moments, and axial loads would occur. The loading used in *RISA-2D* is illustrated on Frame G of the Grove Street Wing. The gravity loads are shown in Figure 16, while the lateral loads are shown in Figure 17.

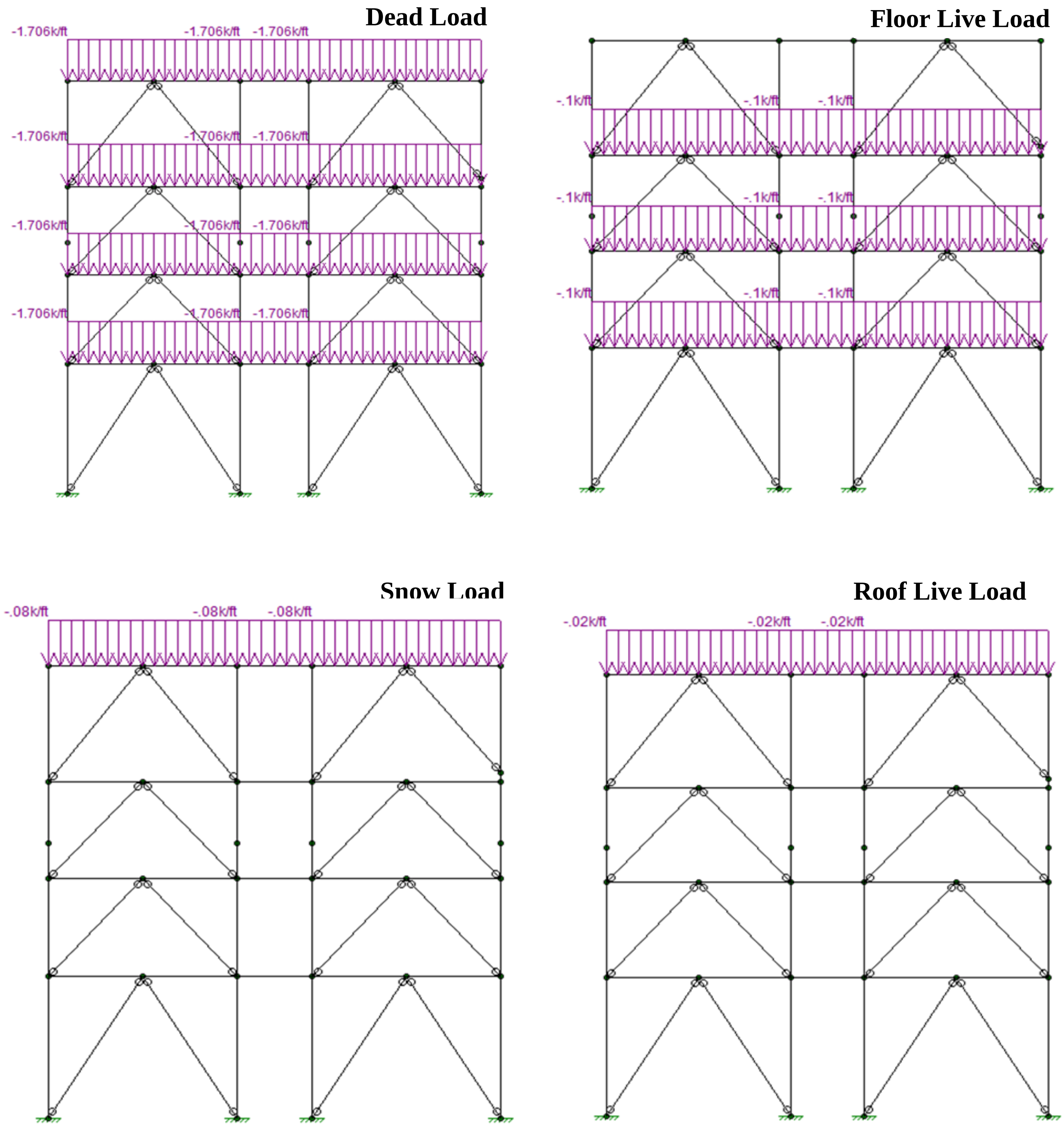


FIGURE 16: GRAVITY LOADING CASES

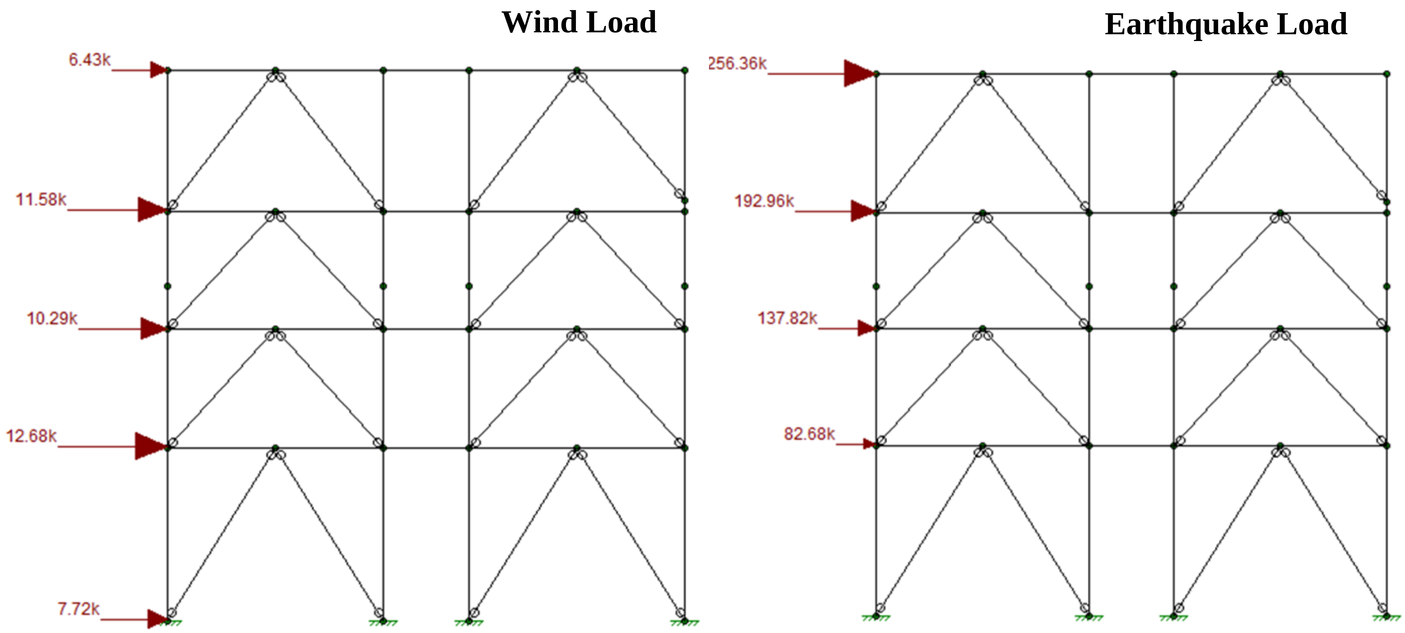
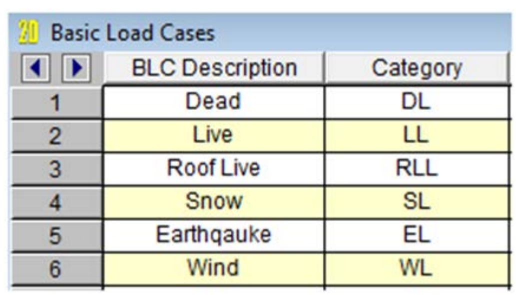


FIGURE 17: LATERAL LOADING

The snow and live loads were easy to input; they simply had to be changed from pounds per foot to kips per foot. The dead load was more complex and required the use of spreadsheets to calculate the loading that would occur on each floor. This was calculated using a conservative approach: the heaviest beams and girders for the frame were used (beam=22 lbs/ft; girder=31 lbs/ft). Using the tributary area, a pound per square foot value could be determined for each floor. The spreadsheet used can be seen in Appendix C.6 Sample Spreadsheet for *RISA-2D* Input - Weights. The wind and earthquake loading used was determined in Chapter 4.1.2 Lateral Loads.

Load combinations were entered into *RISA* by first using the Basic Load Cases tab to give each type of loading a numerical representation, as shown in Figure 18. For example, the dead load is now represented by 1, the live load is now represented by 2, and so on. Using these numbers, the *LRFD* load combinations could be placed into the Load Combinations tab, as shown in Figure 19.


Basic Load Cases

	BLC Description	Category
1	Dead	DL
2	Live	LL
3	Roof Live	RLL
4	Snow	SL
5	Earthqauke	EL
6	Wind	WL

FIGURE 18: BASIC LOAD CASES FROM *RISA-2D*

	Description	Sol...	PD...	SR...	BLC	Factor	BLC	Factor	BLC	Factor
1		☑			1	1.4				
2		☑			1	1.2	2	1.6	4	.5
3		☑			1	1.2	4	1.6	2	.5
4		☑			1	1.2	2	.5	4	.5
5		☑			1	1.2	2	.5	4	.2

FIGURE 19: LOAD COMBINATIONS FROM *RISA-2D*

The load analysis was done for each of the three frame types (Figure 15) and provided values along the columns for both axial (P_{nt}) and moment (M_{nt}) forces due to gravity loading, as well as axial (P_{lt}) and moment (M_{lt}) forces due to lateral loading. Figure 20 illustrates the analysis that was used to determine values of P_{nt} and M_{nt} for Frame G. Appendix D.1 Various *RISA-2D* Load and Frame Analyses displays more analyses done in *RISA-2D* for the different types of frames and load combinations.

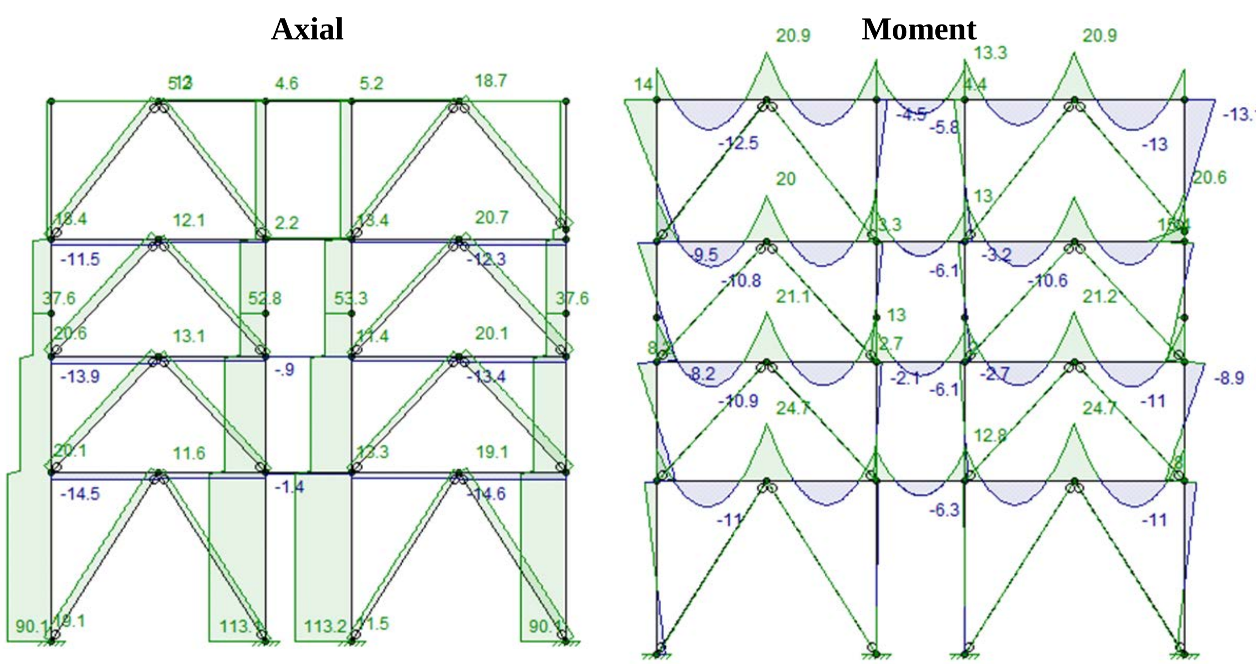


FIGURE 20: AXIAL AND MOMENT DIAGRAMS CREATED USING *RISA-2D*

A second-order analysis could then be completed using the values determined in *RISA-2D*. Using the *AISC* Manual and *Structural Steel Design* by McCormac and Csernak values of B_1 and B_2 were calculated. When “the approximate second-order analysis is used, only B_1 will be computed, as B_2 is not applicable,"[29] and therefore, the moments and axial forces on the columns due to lateral loads become irrelevant when put into the *AISC* H1-1a and *AISC* H1-1b equations. The columns were checked to ensure that they were satisfactory using these equations. A more thorough analysis of the application of these equations can be seen in Appendix C.7 Sample Spreadsheet For Effective Length Method. Web local buckling (WLB) and flange local buckling (FLB) were also checked to confirm that the braced frame could handle the expected loading (Appendix C.7 Sample Spreadsheet For Effective Length Method).[30]

4.3 Precast Concrete Design

In an effort to explore different building materials that could be used to construct the new dormitory, the option of using precast concrete was examined. The superimposed gravity loading for the building was already determined, such as the snow loads, the dead loads, and the various live loads throughout the building. Figure 21 illustrates how the precast concrete columns would be connected to the girders and slabs. The girders rest on corbels located on the side of the columns, while the hollow-core slab rests on top of the girders. This scheme allows everything to fit together smoothly. In the 3rd dimension, girders also sit on corbels and come directly out of and into the page (not shown in this figure). The Precast/Prestressed Concrete Institute (*PCI*) gives information on the necessary sizes for concrete members, and this was the main source that was used for this design process.

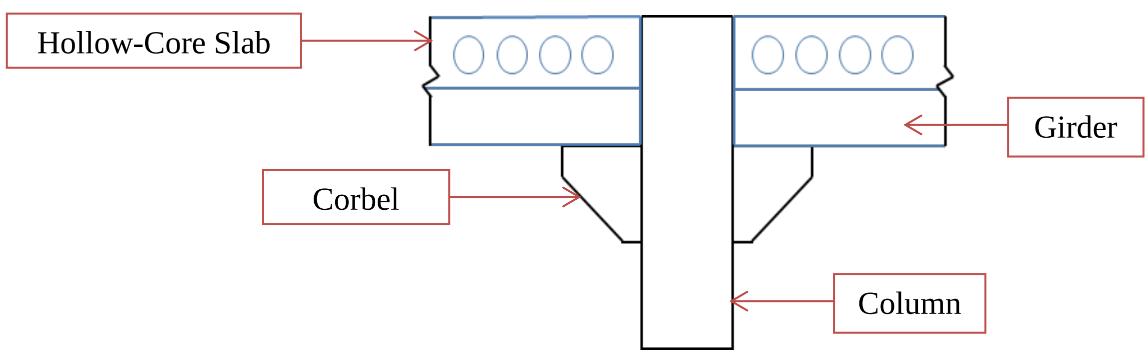


FIGURE 21: PRECAST CONCRETE CONNECTION ILLUSTRATION

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[29] McCormac

[30] McCormac

The first step was to determine the type of slab needed on the roof of the building. A hollow-core slab was chosen over the solid flat-slab because of its lighter weight (while maintaining its structural integrity) and lower building costs. Hollow-core slabs also provide excellent fire resistance. With precise and accurate designs, the voids in the hollow-core slabs can be used for electrical or mechanical runs,[31] however, the design and integration of these services are out of the scope of this MQP. A design based on a superimposed snow load of 80 psf was used to select the hollow-core slab for the roof. In order to have a slab that could be prepared for the installation of a floor covering, a hollow-core slab with a 2-inch normal weight topping was selected.

The next step was selecting the types of girders that would support the slab. For the spandrel girders along the outside of the building (Girder A in Figure 22), an L-shape was the best option, as shown in Figure 23 below.

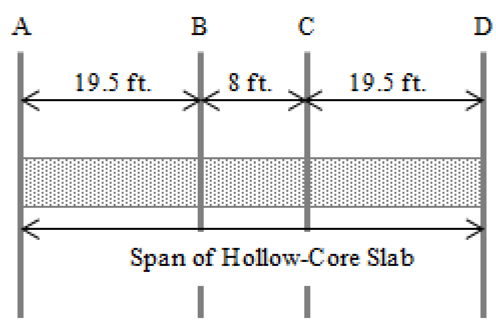

FIGURE 22: PRECAST CONCRETE GIRDER LAYOUT

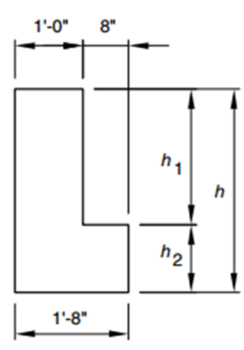

FIGURE 23: PRECAST CONCRETE L-BEAM, TAKEN FROM *PCI* DESIGN HANDBOOK

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[31] Buettner, Donal R., and Roger J. Becker.

The loading for these members would still include the 80 psf snow load, but they would also include the weight of the selected slab. The slab weight is a pounds per foot force, however, it is a line load perpendicular to the length of the girder. Therefore, in order to determine the line load that the girder was required to handle from the hollow-core slab, the following calculations were performed, as seen in Equation 8.

EQUATION 8: PRECAST CONCRETE SLAB LOAD ON PRECAST CONCRETE GIRDERS

$$Slab\ load\ on\ the\ girders = \left(\left(Slab\ weight\ \left(\frac{lb}{ft}\right) \Big/ Slab\ width\ (ft)\right) + 80psf\right) \times (length\ of\ the\ slab\ carried\ by\ the\ girder) = line\ load\ \frac{lb}{ft}$$

The superimposed service load that is calculated was used in conjunction with the “L-Beam Load Tables” published by *PCI* to find the required girder size.

The girders running through the middle of the building, shown as girders B and C in Figure 22: Precast Concrete Girder Layout, had to be different than the L-shaped ones used for the spandrels because they will support a slab on both sides. The best shape was, therefore, an inverted tee beam, shown in Figure 24: Precast Concrete Inverted Tee-Beam.

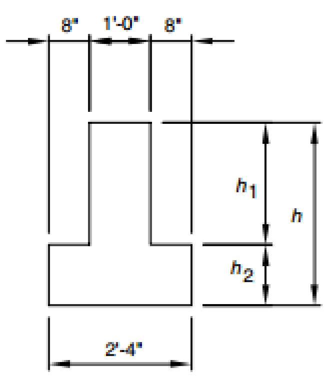

FIGURE 24: PRECAST CONCRETE INVERTED TEE-BEAM, TAKEN FROM *PCI* DESIGN HANDBOOK

The loading on the girder was found using the same equation as above with one minor addition. Now that the slab was located on both sides of the girder, the additional load due to the larger tributary width carried by the girder had to be included.

The hollow-core slab will rest on the girders, sitting atop the 8-inch lip that projects from one side for the L-beams and on both sides for the inverted tee- beam. Beams also needed to be selected. Less beams were required for the concrete design than the steel design. The steel beams were responsible for carrying the weight of the composite decking all around them (found by using tributary areas). The concrete beams are more useful for help with stability of the building when it comes to lateral loading. These beams help to tie the columns together in the direction parallel to the span of the hollow-core slabs. The girders support the slab ends and tie the columns together in the transverse direction.

With the precast concrete slab, beams, and girders selected for the roof, the following floors were examined. All of the elevated floors have the same layout and, therefore, only required the analysis of one floor system to complete the concrete design. Similar steps to the design of the roof floor slab and framing were followed for these floors, with the type and intensity of loading being the only difference. A snow load was no longer present, however, various live loads were. These included the 40 psf loading in the rooms and the 100 psf loading along the corridors.

After the slab, beams, and girders were selected, the precast columns were selected using the previous loads. Determining the loads on the column were much like determining the loading on the girder except that instead of multiplying the loads by the length of slab carried by the girder, the loading on a column depends on the tributary area supported by the column as seen in the equation below.

EQUATION 9: PRECAST CONCRETE SLAB LOAD ON PRECAST CONCRETE COLUMN

$$Slab\ load\ on\ column = \left(\left(Slab\ weight\ (\tfrac{lb}{ft}) \Big/ Slab\ width\ (ft)\right) + 80psf\right) \times (tributary\ area\ supported\ by\ the\ column) = point\ load\ (lb)$$

The tributary area depends on where the column is located in the building. There are two major areas which are to be addressed. Those two areas are when the column is located on the edge of the building, and when the column is located within the center of the building. If the column is located on the edge of the building it will only need to support the slab weight on one side while if it is located in the middle it will need to support the slab weight on both sides using a tributary area method. The two main tributary areas can be seen in Figure 25 below.

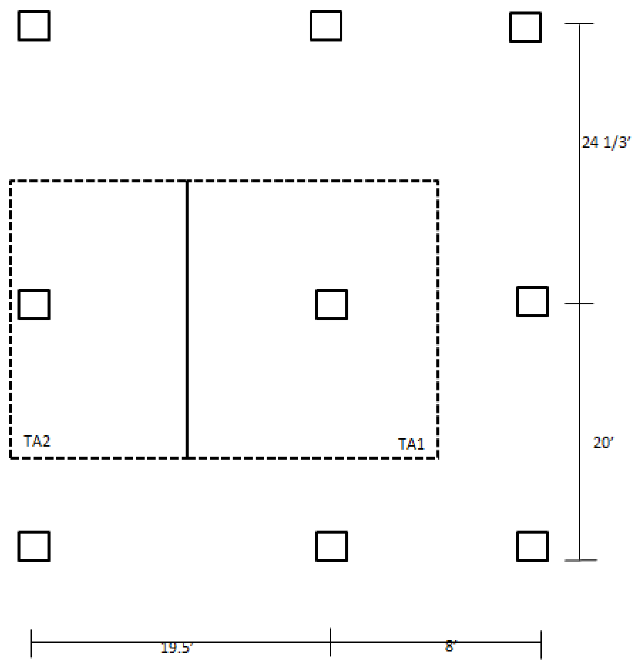

FIGURE 25: DEPICTION OF TWO MAIN TRIBUTARY AREAS

In Figure 25 above, it can be seen that tributary area 1 (TA1) is representative of a column located on the interior of the building while tributary area 2 (TA2) is representative of a column located along the exterior edge of the building. These are the two tributary areas which will be referenced later on in the document while discussing the findings of the column design.

4.4 Footing Design

Footing designs were completed for both the steel design and the precast concrete design. The footing design was completed using a spreadsheet developed using *Structural Steel Design* by Jack C. McCormac and Stephan F. Csernak. This spreadsheet can be found in Appendix C.8 Sample Spreadsheet for Footing Design.

For steel design, it was first necessary to calculate the minimum required area based on the load applied from the column. This load was determined based on the loads that the column carried and the tributary area of the column. The minimum area was also determined based on the compressive strength of the footing. Once the minimum area was determined, the dimensions of the baseplate and the pedestal could be tried for selected. Then the permissible stress was checked against the actual stress exerted.

The actual dimensions were determined based on the calculated deflection. Once the dimensions of the base plate were finalized, the bearing strength of the concrete was checked against the loading applied to the footing to ensure that it was sufficient. Finally, the footing dimensions were calculated.

For precast concrete design, it was first necessary to ensure that the face area of the footing was larger than the face area of the precast concrete column. The load that was applied to the footing from the column was determined in the same manner as steel design. This meant calculating the loads that the column carried and the tributary area of the column. Once the dimensions of the concrete footing were determined the bearing strength was checked against the loading applied to the footing through the column to ensure that it was sufficient.

4.5 Sprinkler System Design

The basis for sprinkler system design is the hazards present in the building and the intended use of that building. The uses of the building are separated into different hazards, Light Hazard, Ordinary Hazard (I and II), and Extra Hazard (I and II) with increasing danger respectively. The hazards present in the building are divided into different commodity classifications. These commodities are labeled Class I-IV or Class A/B/C plastics with their relative dangers increasing respectively. Once it is determined what the general use of the space and what is going to be present in the space it is possible to determine the flow and pressure required at the most hydraulically remote sprinkler in the space.

In order to complete a sprinkler design, hydraulic calculations were performed. Hydraulic calculations use the pressures and flows in the pipes along with the material characteristics of the pipe. The hydraulic calculations start with the most remote sprinkler and work backwards within the design area until all sprinklers are included. These calculations incorporate friction loss due to gravity and elevation as well. The governing equations for hydraulic calculations can be seen below.

EQUATION 10: PRESSURE LOSS PER FOOT DUE TO FRICTION

$$p = \frac{4.52Q^{1.85}}{C^{1.85}d^{4.97}}\ psi/ft$$

- Q=flow (gpm)
- C=Friction Loss Coefficient
- D=internal pipe diameter (in)

EQUATION 11: PRESSURE LOSS PER FOOT DUE TO GRAVITY

$$p = 0.433\ psi/ft$$

EQUATION 12: CONVERSION FACTOR FOR NON-SCHEDULE 40 PIPE

$$\left(\frac{Pipe\ Inside\ Diameter}{Schedule\ 40\ Inside\ Diameter}\right)^{4.87}$$

Equation 10 gives the pressure loss due to friction, Equation 11 gives the pressure loss due to gravity, and Equation 12 is a normalizing equation that creates an equivalency between pipes of differing diameters. The pressure losses are then multiplied by the total number of feet of the applicable pipe size to get a total pressure lost over that many feet. In the pressure loss due to frication equation, the result is heavily dependent on the C Factor. This factor is a characteristic of the pipe material and can be found in *NFPA 13*.

In addition to the pressure loss equations, the flow and pressure present at every sprinkler is necessary. The flows and pressures at each sprinkler can be found using the two equations below.

EQUATION 13: HYDRAULIC FLOW EQUATION

$$Q = k\sqrt{P}$$

EQUATION 14: HYDRAULIC PRESSURE EQUATION

$$P = \left(\frac{Q}{K}\right)^2$$

- K=K Factor for sprinkler

- P=Required Pressure (psi)
- Q=Flow (gpm)

By adding the pressure losses due to friction and gravity and inputting those numbers into these two equations it is possible to determine the pressure and flow required at each sprinkler. Additionally, different fittings will affect the flows and pressures differently. Tables of their equivalent lengths, how much pressure loss there would be in the fitting if it was a straight piece of pipe, are provided in *NFPA 13*. These hydraulic calculations were calculated for a specific design area. This area was established by first determining the number of sprinklers within the design area. This was determined using the equation below.

EQUATION 15: NUMBER OF SPRINKLERS IN DESIGN AREA

$$\#\ of\ sprinklers = \frac{Sprinkler\ operation\ area}{Protection\ Area}$$

The sprinkler operation area is equal to the total square footage of the design area which is found in *NFPA* 13 and the protection area is based on the type of sprinkler used which can also be found in *NFPA* 13. Once the total number of sprinklers was determined, the number of sprinklers required on the most remote branch line was needed. This number of sprinklers was found using the equation below.

EQUATION 16: NUMBER OF SPRINKLERS ALONG MOST REMOTE BRANCH LINE

$$\#\ of\ sprinklers = \frac{1.2\sqrt{Design\ Area}}{Sprinkler\ Spacing}$$

The spacing of the sprinklers was referenced from *NFPA* 13 and was also based on the type of sprinkler. Once both of these values were determined, it was possible to orient the design area over the most remote section of the building. This design area can be seen in Figure 26 below.

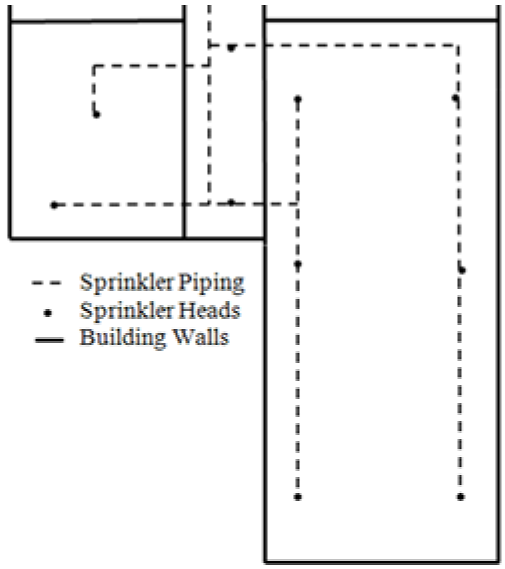

FIGURE 26: LAYOUT OF DESIGN AREA

5.0 Design Results

In this chapter both the steel and the concrete design alternatives will be presented and compared. Design results, discussions and key findings will also be addressed throughout the chapter.

5.1 Structural Steel Design

Below are the steel design layouts for the floor framing. In Figure 27 below, the wings of the building are labeled and will be referred to as such for the report. The two wings, Lancaster Street and Grove Street are highlighted.

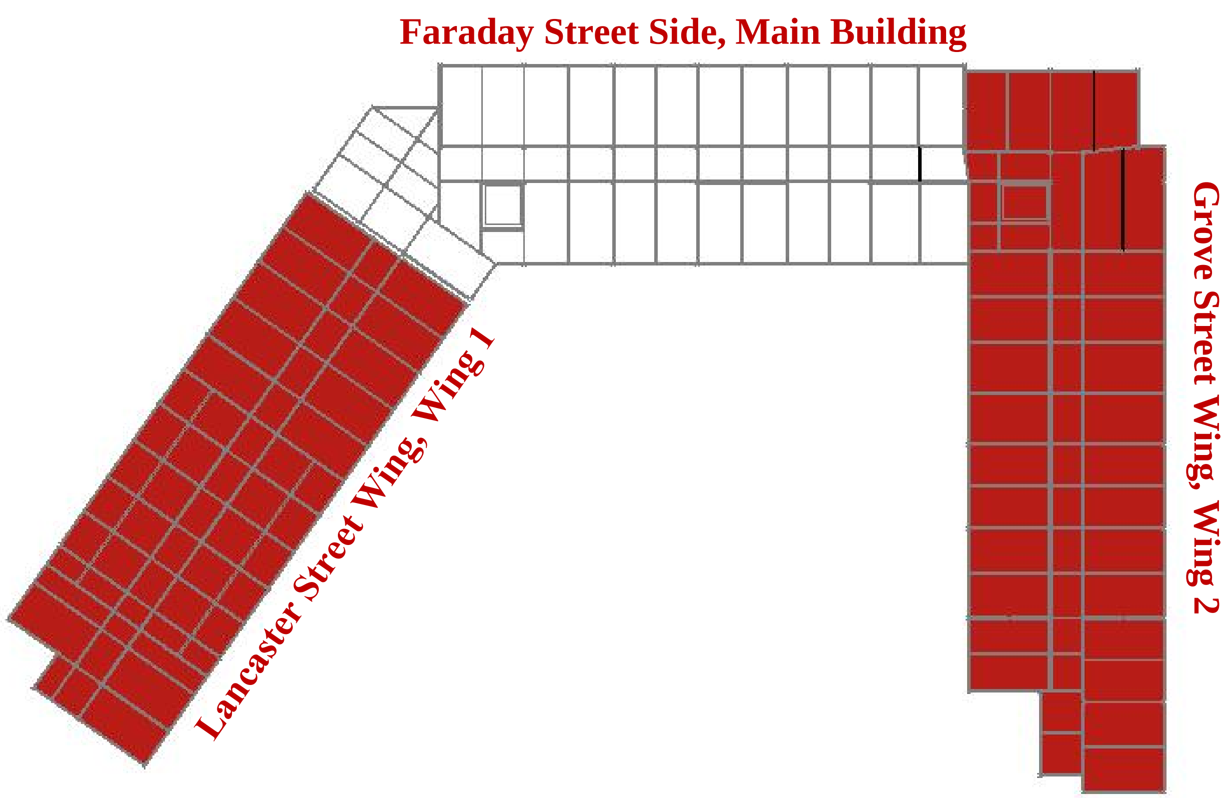

FIGURE 27: BUILDING STEEL LAYOUT AND WING LABELS

In Figure 28, Figure 29, and Figure 30, below, the main building (Faraday Street Side) and each wing (Grove Street Wing and Lancaster Street Wing) were separated in order to display the beam and girder framing steel designs. These figures show the steel designs for the roof of the building. They illustrate the repetitive use of W12X22 beam sizes along the outer spans of the roof and W10X12 beam sizes along the corridor. The frequent use of W14X22 for the girders around the outside of the roof is also portrayed.

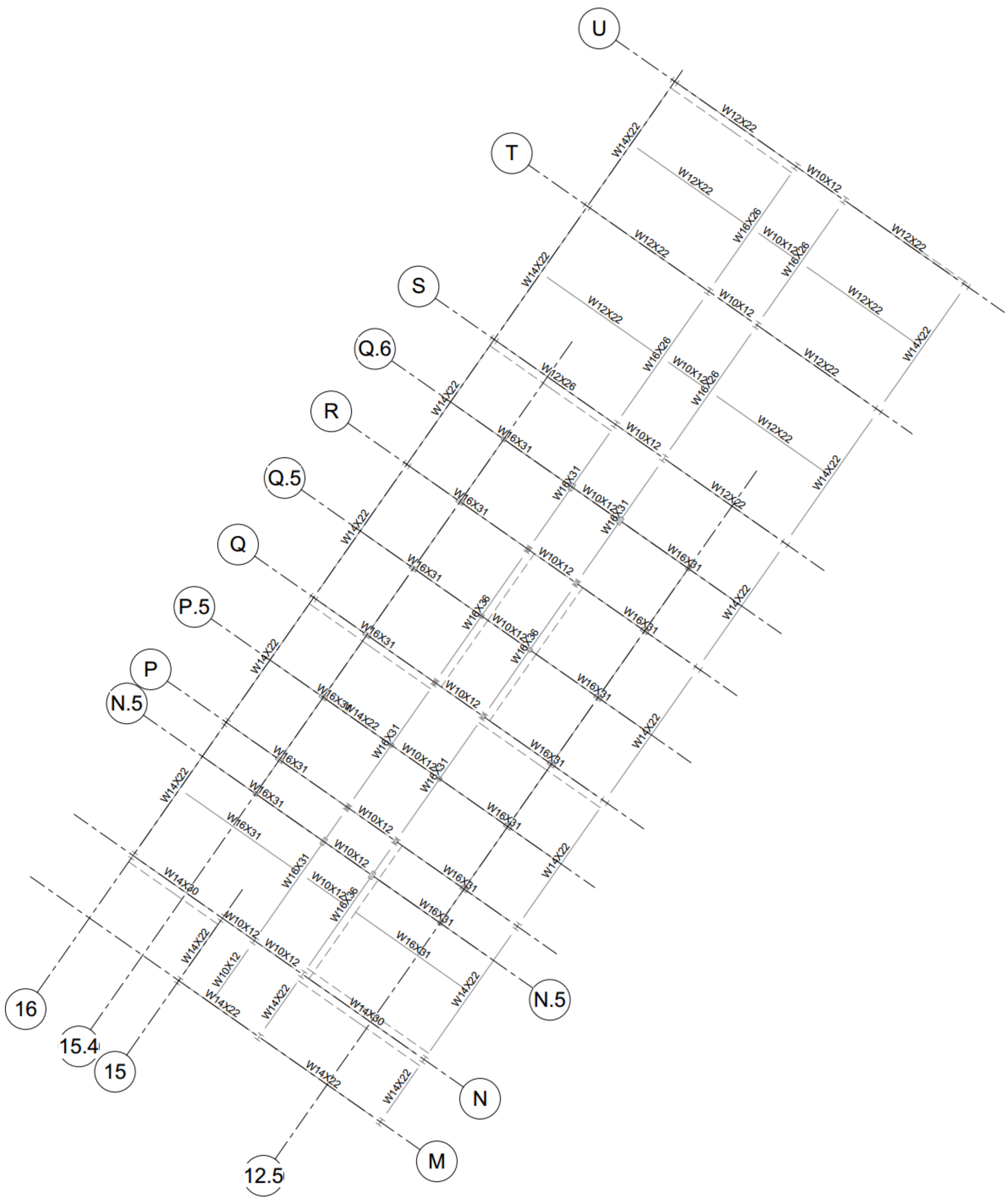

FIGURE 28: LANCASTER STREET WING, WING 1 STEEL LAYOUT ROOF

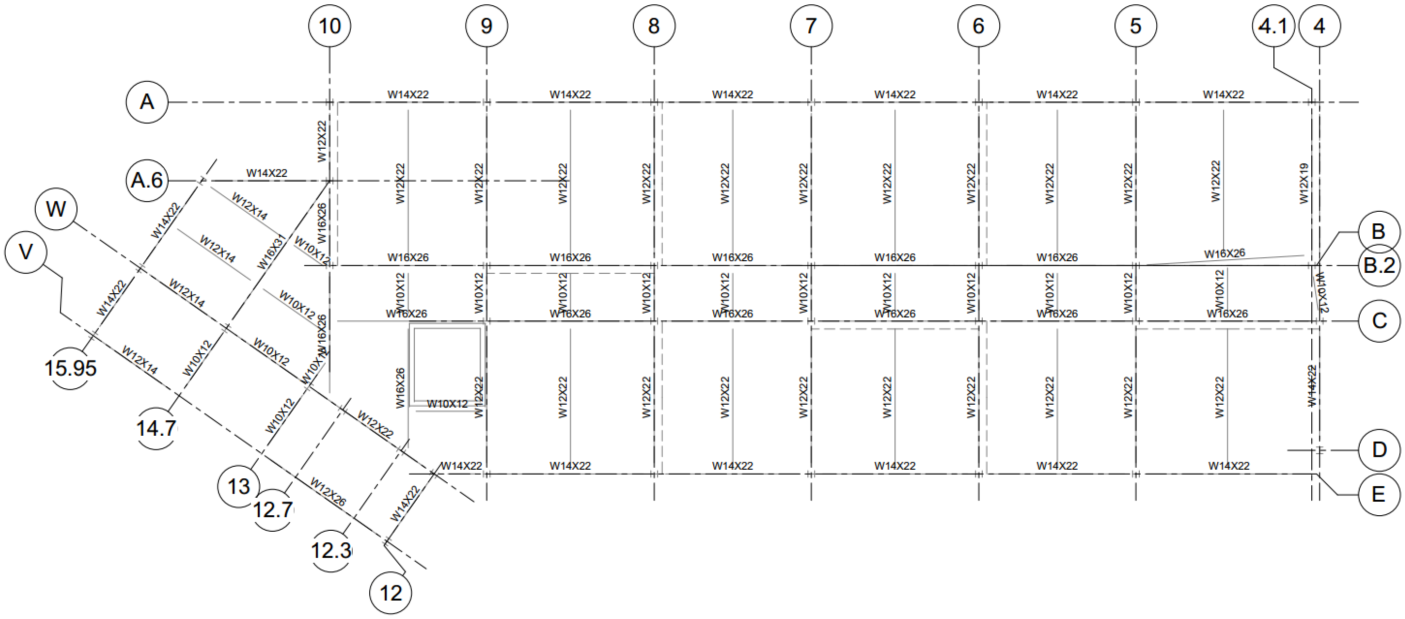

FIGURE 29: FARADAY STREET SIDE, MAIN BUILDING STEEL LAYOUT ROOF

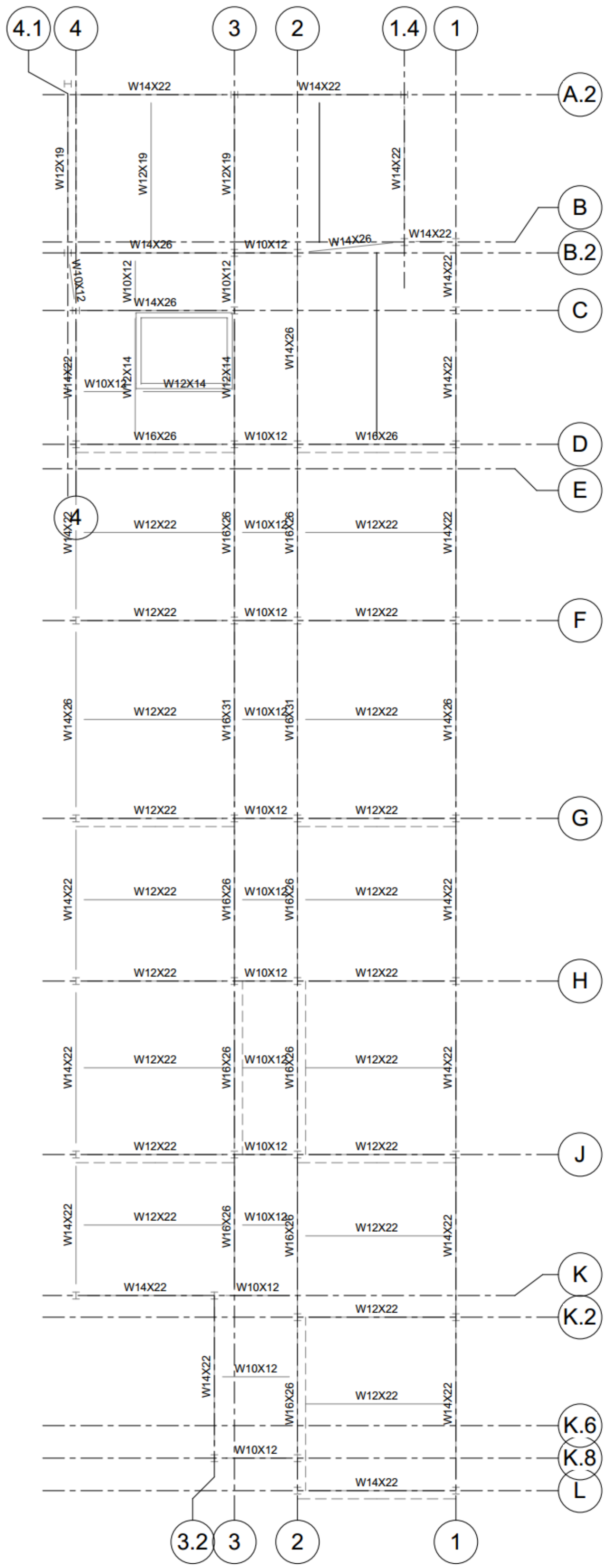

FIGURE 30: GROVE STREET WING, WING 2 STEEL LAYOUT ROOF

The steel designs that are typical for each floor (2-4) are shown below in Figure 31, Figure 32, and Figure 33. They are similar to the steel design for the roof in many ways, including a repetitive use of W12X22 beam sizes along the outer spans of the floor and W10X12 beam sizes along the corridor. The repeated use of W14X22 for the girders around the outside of the roof is also portrayed for floors two through four.

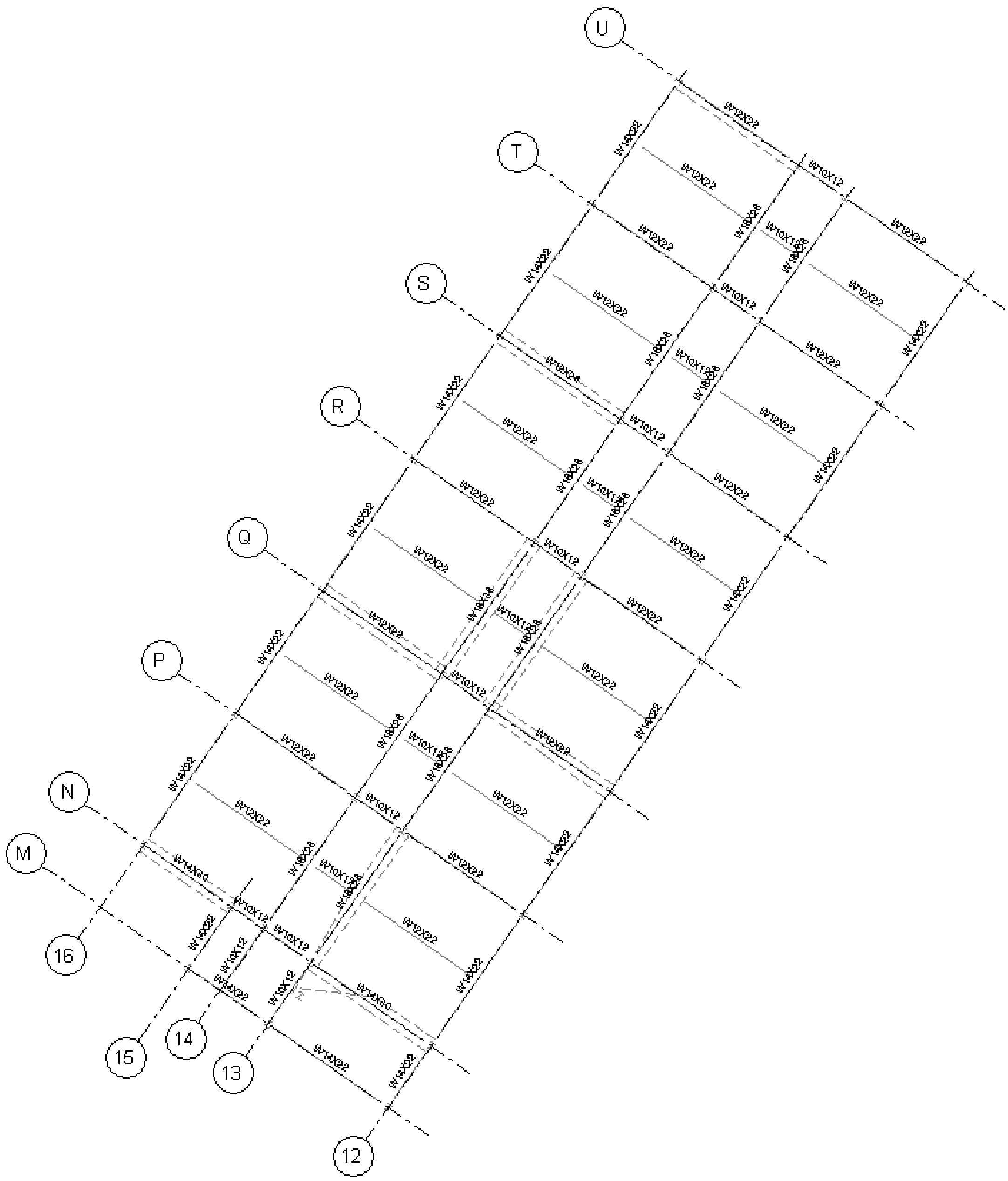

FIGURE 31: LANCASTER STREET WING, WING 1 STEEL LAYOUT FLOORS 2-4

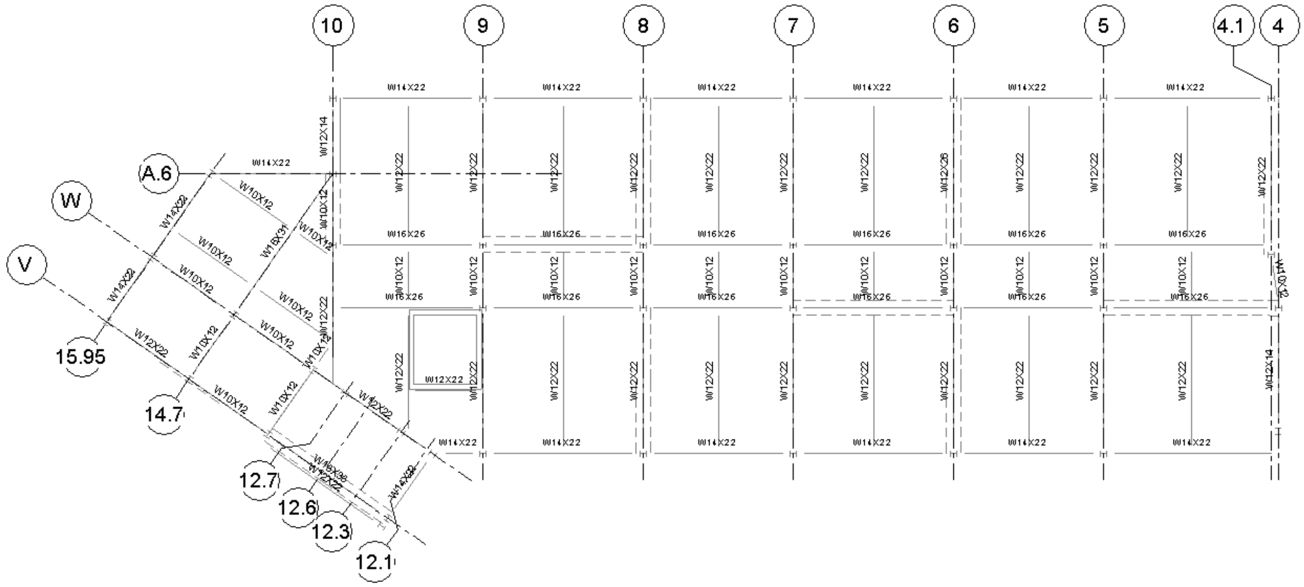

FIGURE 32: FARADAY STREET SIDE, MAIN BUILDING STEEL LAYOUT FLOORS 2-4

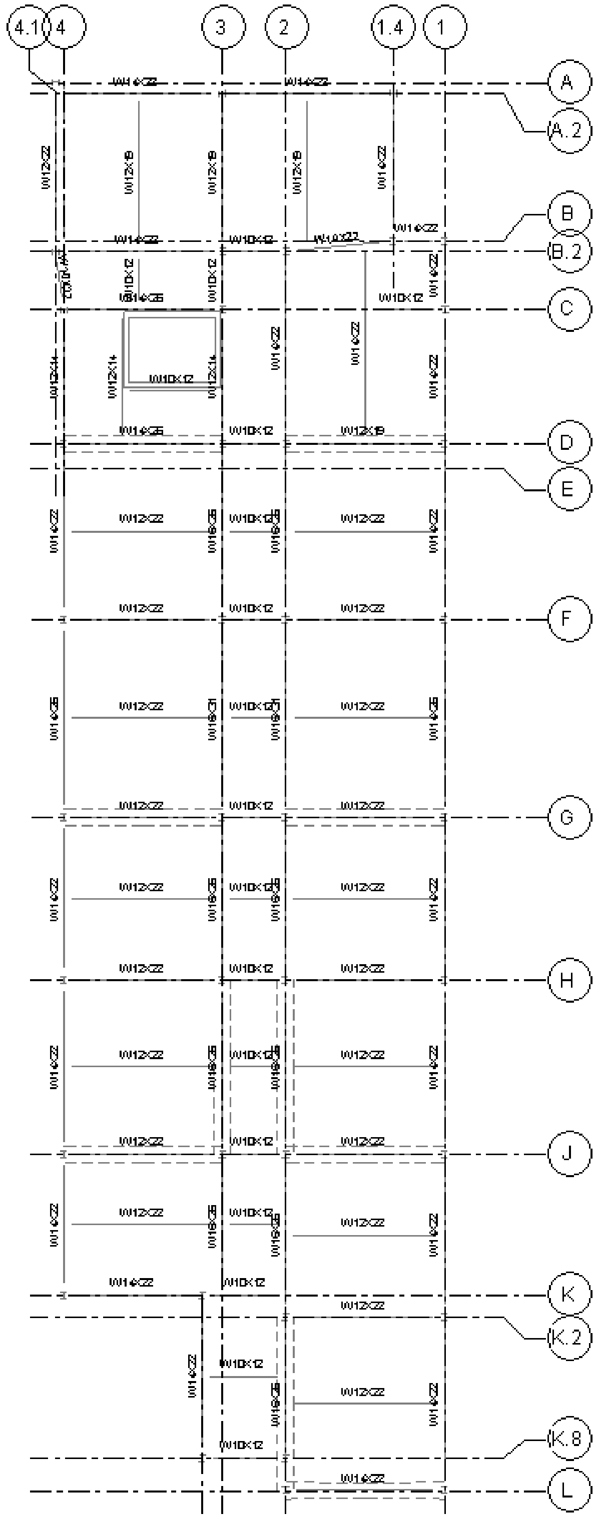

FIGURE 33: GROVE STREET WING, WING 2 STEEL LAYOUT FLOORS 2-4

Some key observations that can be taken from the above figures are the differences in beam and girder sizing between the different floors and the two different wings. The beams and girders designed for the roof were different than those designed for the floors below. This was due to the different loading cases, such as snow, that were applied. The framing plans for the roofs of the wings were also very different in beam and girder sizes because of the mechanical units and screen wall on the Lancaster Street Wing. The structural framing plans for the wings on the floors below the roof were typical of each other. The beams and girders for the entire design were first selected for adequacy for the loading and then for constructability. This means that if a W12X14 was adequate for a girder, but the surrounding girders were required to be W12X22, then the W12X14 was switched to be more uniform which promotes constructability.

5.1.2 Steel Bracing Member Selection

After checking the braced, steel structural frame through *RISA-2D*, it was determined that the HSS6X6X5/16 members were adequate and could handle the gravity and lateral loading. Equation *AISC* H1-1a and *AISC* H1-1b for combined axial and flexural forces ensured that the frames were satisfactory, and the WLB and FLB equations confirmed that local buckling would not be an issue. Through *RISA-2D*, it was also determined that the braced frames should be pin connected to the girders and columns, rather than connected by fixed ends. Pin connections prevented moments from passing into the bracing, unlike the fixed ends, and took some of the stress off the HSS members. Figure 34 and Figure 35 display the difference between these two options by showing a plot of the moment diagrams for each case. They make it very clear that bracing with pin connections is much more advantageous. The red ovals highlight one area where the change is very clear.

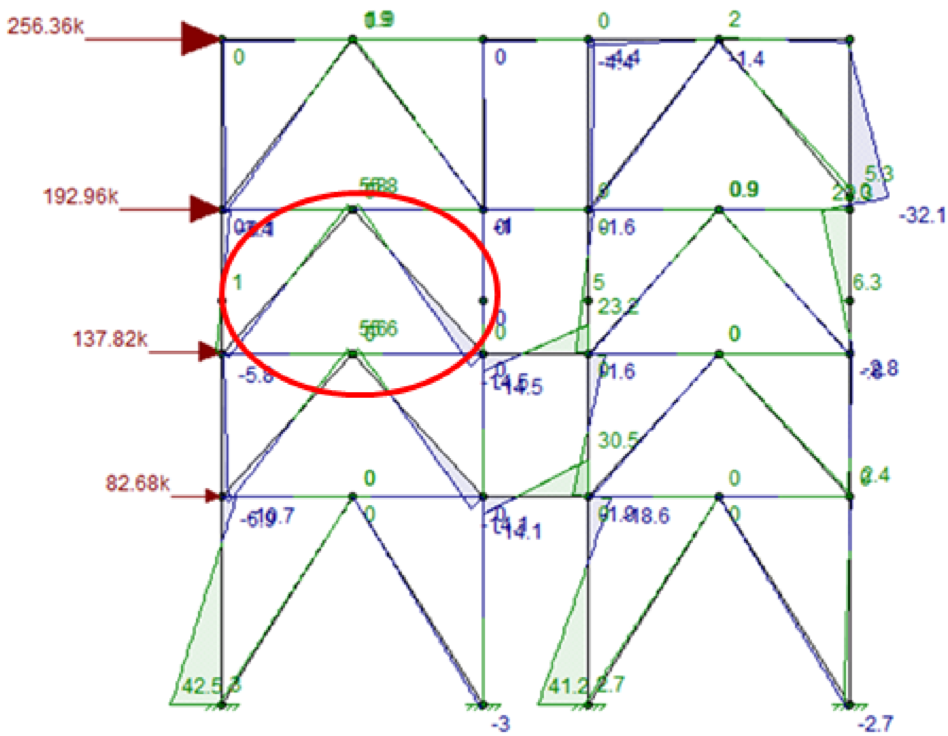

FIGURE 34: FIXED END CONNECTIONS FOR LATERAL BRACED FRAME CREATED USING *RISA-2D*

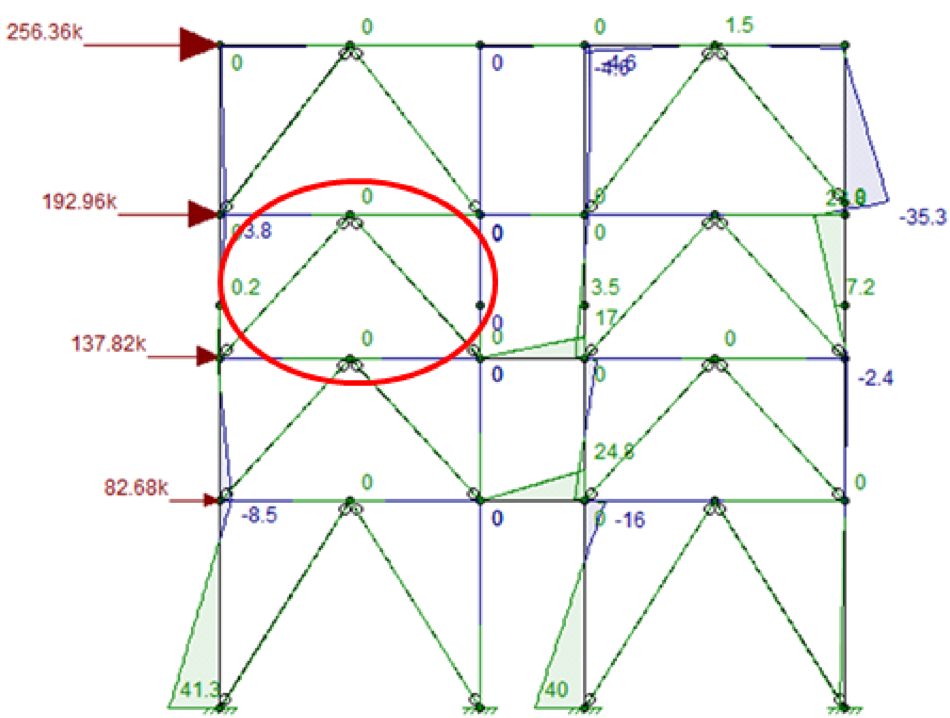

FIGURE 35: PINNED CONNECTIONS FOR LATERAL BRACED FRAME CREATED USING *RISA-2D*

Another outcome of the lateral bracing design was a change in girder sizes around the lateral frames. The HSS members had a width of 6 inches, and this presented a geometry issue with the connection to the girders. The girder width where these connections occurred needed to be larger than 6 inches. Therefore, the widths of all the girders along the frame were checked and changed accordingly. For example, W12X22 girders have a flange width of about 4 inches, and were, therefore, bumped up to W12X26 which have a flange width of about 6.5 inches, as seen in Figure 36 below. W14X26 members were another girder size that was adjusted: they were changed to W14X36 members. The structural diagrams and models presented throughout Chapters 5.1 Structural Steel Design and 5.1.2 Steel Bracing Member Selection account for the change in girders sizes.

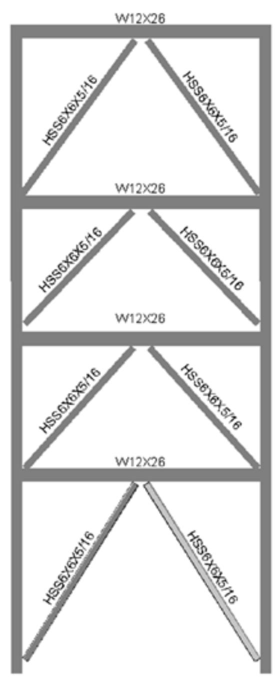

FIGURE 36: TYPICAL STEEL FRAMING PLAN

5.2 Precast Concrete Design

With the necessary steps for selecting the different precast concrete members clearly established, the actual loads were determined and the member sizes were selected from the *PCI*

Tables.[32] The hollow-core slab was the first member chosen, and an example cross section is shown in Figure 37.

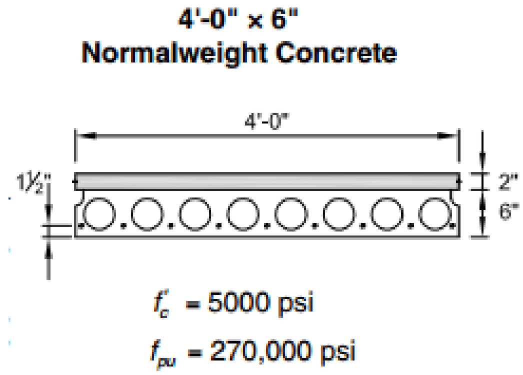

FIGURE 37: HOLLOW CORE PRECAST CONCRETE SLAB, TAKEN FROM *PCI* DESIGN HANDBOOK

Table 12 displays the design data given by the Precast/Prestressed Concrete Institute (*PCI*). The maximum span of the slab is 19' 5 ½", so a value of 20 feet was used for the span length. The yellow highlights in the table depict how the slab type was selected, and the red circle indicates which type was chosen, a 4HC6+2: 66-S

TABLE 12: HOLLOW CORE PRECAST CONCRETE SLAB DESIGN TABLE, TAKEN FROM *PCI* DEISGN HANDBOOK

4HC6 + 2

Table of safe superimposed service load, lb/ft², and cambers, in. **2 in. Normalweight Topping**

Strand designation code	Span, ft																		
	12	13	14	15	16	17	18	19	20	[illegible]	[illegible]	23	24	25	26	27	28	29	30
66-S	333 0.2 0.2	300 0.2 0.2	272 0.2 0.2	248 0.2 0.2	227 0.2 0.2	210 02 0.1	182 0.2 0.1	158 0.2 0.0	136 0.2 –0.1	[illegible] 0.2 –0.2	93 0.1 –0.3	75 0.1 –0.5	59 0.0 –0.7	46 –0.1 –0.9	34 –0.2 –1.2				
76-S		357 0.2 0.2	324 0.3 0.2	296 0.3 0.2	272 0.3 0.2	248 0.3 0.2	216 0.3 0.2	188 0.3 0.1	163 0.3 0.1	137 0.3 0.0	115 0.3 –0.2	95 0.2 –0.3	78 0.1 –0.5	63 0.1 –0.7	50 –0.0 –0.9	38 –0.1 –1.2	27 –0.3 –1.5		
96-S			418 0.4 0.4	384 0.4 0.4	353 0.4 0.4	319 0.5 0.4	279 0.5 0.4	245 0.5 0.4	216 0.5 0.3	186 0.5 0.3	160 0.5 0.2	137 0.5 0.1	116 0.5 –0.1	98 0.4 –0.3	82 0.3 –0.5	68 0.3 –0.7	55 0.1 –1.0	43 0.0 –1.4	33 –0.1 –1.7
87-S			422 0.5 0.5	388 0.5 0.5	359 0.6 0.5	332 0.6 0.6	309 0.7 0.6	288 0.7 0.6	258 0.7 0.5	224 0.7 0.5	195 0.8 0.4	169 0.8 0.4	147 0.7 0.2	127 0.7 0.1	109 0.7 –0.1	94 0.6 –0.3	80 0.5 –0.5	67 0.4 –0.8	55 0.3 –1.2
97-S			473 0.5 0.6	433 0.6 0.6	400 0.7 0.7	371 0.7 0.7	346 0.8 0.7	323 0.8 0.7	288 0.9 0.7	251 0.9 0.7	219 0.9 0.6	192 0.9 0.6	168 1.0 0.5	146 0.9 0.4	127 0.9 0.2	110 0.9 0.0	95 0.8 –0.2	82 0.7 –0.5	70 0.6 –0.8

Strength is based on strain compatibility; bottom tension is limited to $7.5\sqrt{f'_c}$; see pages 3–8 through 3–11 for explanation.
See item 3, note 4, Section 3.3.2 for explanation of vertical line.

[32] *PCI* Design Handbook Seventh Edition

The *PCI* Tables gave the weight load of a 6" hollow-core slab with 2" normal weight topping as 295 pounds per foot of span length. This value was then used in Equation 17 to determine the loading that would be placed on the girders.

EQUATION 17: SLAB LOAD ON THE PRECAST CONCRETE GIRDERS DESIGN VALUES

$$Slab\ load\ on\ the\ girders = \left(\left(\frac{295\frac{lb}{ft}}{4ft}\right) + 80psf\right) \times \left(\frac{19.5ft}{2}\right) = 1499.06\frac{lb}{ft}$$

- 4 ft. is the slab width, used to convert the given weight into psf
- 80 psf is the roof snow load
- (19.5 ft. /2) is half the distance between the girders, used to account for the slab area tributary to this particular girder. This is the only distance used because the slab is only on one side of the girder due to its location on the building.

Looking under the "L-Beam Load Tables" section of the *PCI* Tables and using a superimposed service load of about 1500 lb./ft. and a girder span length of 22 ft., it was determined that a 20LB20 could be used. This size L-beam has an h_1 value of 12 inches and an h_2 value of 8 inches (Figure 38). The 22 foot span length covers the lengths of all the girders except for one section of the building between column lines G and F where the girder span is 24' 4". For this

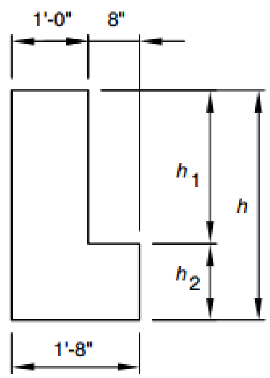

FIGURE 38: PRECAST CONCRETE L-BEAM, TAKEN FROM *PCI* DEISGN HANDBOOK

area, the span length had to be rounded up to 26 ft. to use the *PCI* Tables which increase by values of 2 ft. For this case, the 20LB20 girder was strong enough to handle the load at 22 foot and 26 foot spans, meaning that no change in girder size would be necessary for this larger span.

The interior girders used an inverted tee-beam shape. These members were required to carry the weight of the slab from both sides. This was incorporated by using a tributary width (19.5 ft./2 + 8 ft./2) to encompass the whole area of the hollow-core slab carried by the girder. Using Equation 18, with the addition of (8 ft./2), a superimposed service load of about 2114 lb./ft. was calculated. From the "Inverted Tee-Beam Load Tables" section of the *PCI* Tables, it was concluded that a 28IT20 would be acceptable for all girders in this section, whether they had 22 foot spans or 26 foot spans. The h_1 value was 12 inches, while the h_2 value was 8 inches, which are the same values used for the L-beam and can accommodate a 6 in. slab with 2 in. topping (Figure 39).

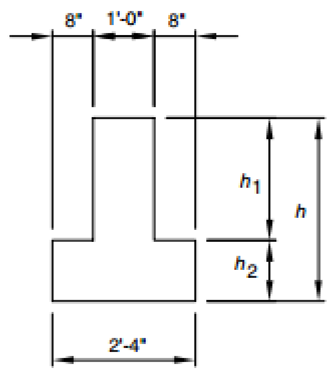

FIGURE 39: PRECAST CONCRETE INVERTED TEE-BEAM, TAKEN FROM *PCI* DEISGN HANDBOOK

In the longitudinal direction, the design only includes beams between the columns, meaning that the concrete design used half the number of beams as the steel design. The concrete beams were rectangular shape, and the loading used to select the proper size was found using the following calculations.

EQUATION 18: LOADING ON THE PRECAST CONCRETE BEAMS DESIGN VALUES

$$Loading\ on\ the\ beams = (73.75\ psf + 80\ psf) \times (1ft) = 153.75\ \frac{lb}{ft}$$

- 73.75 psf is the loading from the slab
- 80 psf is the roof snow load
- 1 ft. is the width of the beam

The beams run parallel to the slab and the maximum span the beams cover is 19.5 feet. The beams were all designed based on this span, so the selected beam size is conservative for beams that cover smaller spans. Using the “Rectangular Beam Load Tables” section of the *PCI* Tables, a 12RB16 rectangular beam was chosen. The dimensions are shown in Figure 40.

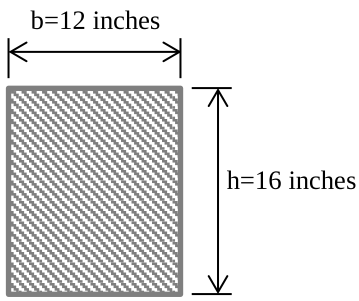

FIGURE 40: PRECAST CONCRETE RECTANGULAR BEAM

Table 13: Precast Concrete Member Sizes shows the results for concrete slab, beam, and girder sizes for all floors, including the roof.

TABLE 13: PRECAST CONCRETE MEMBER SIZES

Floor	Slab Type	Beam Type	Girder Type		
			L-Beam	Inverted Tee-Beam	
				22 Ft. Span	*26 Ft. Span*
Roof	4HC6+2: 66-S	12RB16	20LB20	28IT20	28IT20
4th	4HC6+2: 66-S	12RB16	20LB20	28IT20	28IT24
3rd	4HC6+2: 66-S	12RB16	20LB20	28IT20	28IT24
2nd	4HC6+2: 66-S	12RB16	20LB20	28IT20	28IT24

All precast member sizes were the same for each category, with the one exception being the variation between the roof and the other floors for the 26 foot inverted tee-beam size.

The precast concrete members had large capacities and, therefore, some of the minimum sizes in the *PCI* Tables had capacities that could handle loads much larger than any of the superimposed design loads for the residence hall. In an effort to explore this dilemma, and create a more efficient design, the possibility of removing one row of columns was analyzed. Throughout most of the building, four columns are used to span its width. By removing one row of columns, a larger beam and slab span are created, as shown in Figure 41 below.

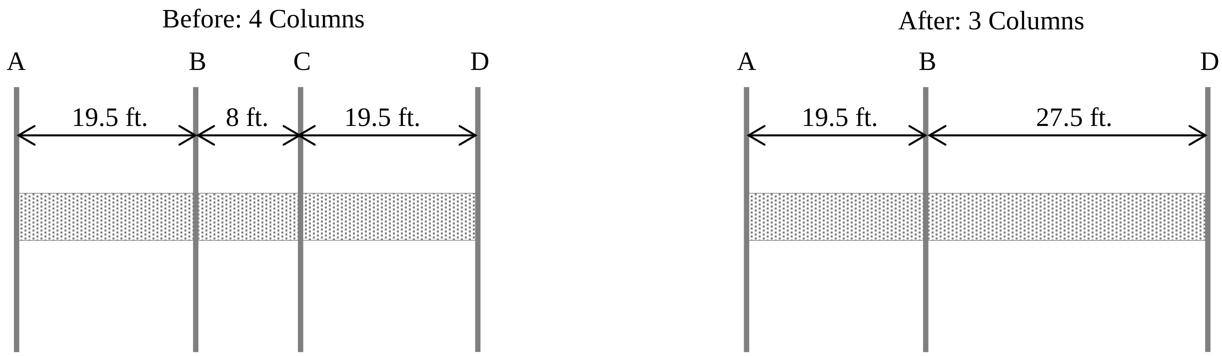

FIGURE 41: PRECAST CONCRETE COLUMN POSSIBILITIES

This option would be feasible, with only minor changes in precast member sizes. The selected 6 inch hollow-core slab can accommodate the increased span; however, a slab with more reinforcing strands would be required. The steel strands running through the member increase the capacity of the concrete members.

In order to compare results, hand calculations were done to determine the required thickness and steel reinforcement needed for a typical concrete slab. Through this investigation, it was determined that a 6-inch slab, with a 1-inch cover, would be satisfactory for the anticipated loads. By analyzing the minimum steel reinforcement area required, it was concluded that #5 bars, spaced 16 inches apart, would be adequate. Detailed calculations can be seen in Appendix B.5 Sample Calculation for Concrete Design. The 7-inch depth used for the hand calculations was only 1 inch smaller than the hollow-core slab previously selected. This analysis displays the many specifications that need to be considered when designing a concrete structure, as well as illustrates the usefulness and effectiveness the *PCI* design tables have in the concrete structural design process.

5.3 Footing Design

For the footing design to support the steel frame, the required area of the baseplate was calculated to be less than the area of the steel column. This meant that the actual area of the baseplate would have to be larger than the calculated area to adhere to design geometry. The dimensions of the column, baseplate, pedestal and footing can be seen below in Table 14.

TABLE 14: STEEL FOOTING DESIGN DIMENSIONS

Column	10"	X	10.1"
Baseplate	13"	X	12"
Pedestal	18"	X	17"
Footing	6'	X	6'

The required pedestal area for the precast concrete was also determined to be less than the area of the concrete column. The concrete column, pedestal and footing dimensions calculated can be seen below in Table 15.

TABLE 15: PRECAST CONCRETE FOOTING DESIGN DIMENSIOINS

Column	12"	X	12"
Pedestal	15"	X	15"
Footing	6'	X	6'

5.4 Steel Versus Precast Concrete

A major choice that must be made in any modern day construction is which construction material to use. For the dormitory, both steel and precast concrete construction was explored. There were advantages and disadvantages of both, and depending on the type of building being constructed, one material is sometimes better suited. In the case of the dormitory, one of the first factors analyzed was the cost of the structural framing for each material. Table 16 displays the cost of precast concrete versus steel construction. It accounts for the beams, girders, columns, slabs, and fireproofing.

TABLE 16: COST COMPARISON STEEL VERSUS PRECAST CONCRETE

Precast Concrete	Steel w/out Fireproofing	Steel w/ Fireproofing
$2,532,980.80	$2,551,225.20	$2,908,897.04

The table depicts a major cost addition that is experienced when building with steel, which is fireproofing. Steel members are noncombustible; however, in temperatures often reached in fires, the strength is greatly reduced. “Steel is an excellent heat conductor, and non-fireproofed steel members may transmit enough heat from one burning compartment of a building to ignite materials with which they are in contact in adjoining sections of the building."[33] Fire-protective covers are necessary to increase the fire resistance ratings for steel members; these include concrete gypsum, mineral fiber sprays, special paints, and other materials. Spray-on fireproofing is the most used fire resistant material and usually consists of either mineral fibers or cementious fireproofing materials. Fireproofing presents an issue because of its high cost, which discourages its use compared to other construction materials.[34] Steel without fireproofing is not an option, but it is used to show the significance that fireproofing has in increasing the overall price of steel construction. Precast concrete is also noncombustible and the high temperatures experienced during fires have little effect on the structural integrity of the concrete.[35] Therefore, concrete is much more advantageous and efficient in terms of fire protection.

One positive aspect of steel construction is that the properties of steel are uniform and do not change throughout, like in concrete. Another positive of steel is its ductility, meaning its ability to withstand extensive deformation without failing when put under large tensile stresses. Steel structures, however, are susceptible to corrosion, and are therefore, required to be painted periodically.[36] Concrete, on the other hand, can be left unpainted without the danger of being damaged by the elements. Precast concrete has a larger thermal mass than steel, meaning that its "capacity to store and regulate internal heat"[37] is much better. Therefore, energy efficiency of precast concrete design is superior to steel construction. When it comes to design limitations, the opportunities with precast concrete design are endless, and more diverse than that of steel.[38] In the case of the new residence hall, the design is fairly straightforward, meaning that both steel and concrete fit the design plan.

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[33] McCormac

[34] McCormac

[35] Building with Precast

[36] Assakkaf

[37] Principles of Eco-Design

[38] Building with Precast

One design limitation that was prevalent for both designs was the various connections between members. Certain areas in the building have members that are larger than required because geometry issues with the connections required thicker members to create a proper fit. For the steel design, this issue was present where the lateral bracing met the girders. Several girders were increased in size in order to make them greater than the width of the bracing members. With the precast design, larger beams were used in the smaller 8-foot span along the corridor of the building. These beams were much larger than necessary, but they allowed the same geometry to be followed across the whole building, and did not require a special order from the fabrication shop. Geometry issues were the only major limitation presented by both designs. More complex buildings would be presented with more design limitations. The comparison between these construction materials is not limited to the design aspect.

Scheduling is important in any construction project, and the time required for steel construction versus precast concrete construction was another factor that was explored. Precast concrete components can be fabricated while the early stages of site work and foundation work are being completed, which allows erection to begin as soon as the foundation work is complete. “As a single-source supplier for a large portion of the structural system, precasters help maintain the critical-path schedule.”[39] If there are issues with the fabrication, the schedule could be severely affected. The precast members are erected very quickly, including the wall panels; this is very beneficial because it allows the building to be enclosed rapidly which reduces the concern for weather.[40]

In many ways, procurement and erection of structural steel is similar to precast construction. Steel fabrication is also completed off-site and delivered to the site where it can be constructed in all seasons. Coordination between the general contractor and the steel contractor is crucial to guarantee the overall construction schedule is followed. Figure 42 illustrates the typical

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[39] Designing with Precast & Prestressed Concrete

[40] Designing with Precast & Prestressed Concrete

Work Breakdown Structure (WBS) for steel construction.[41] Many of the activities are similar to those required in precast concrete construction.

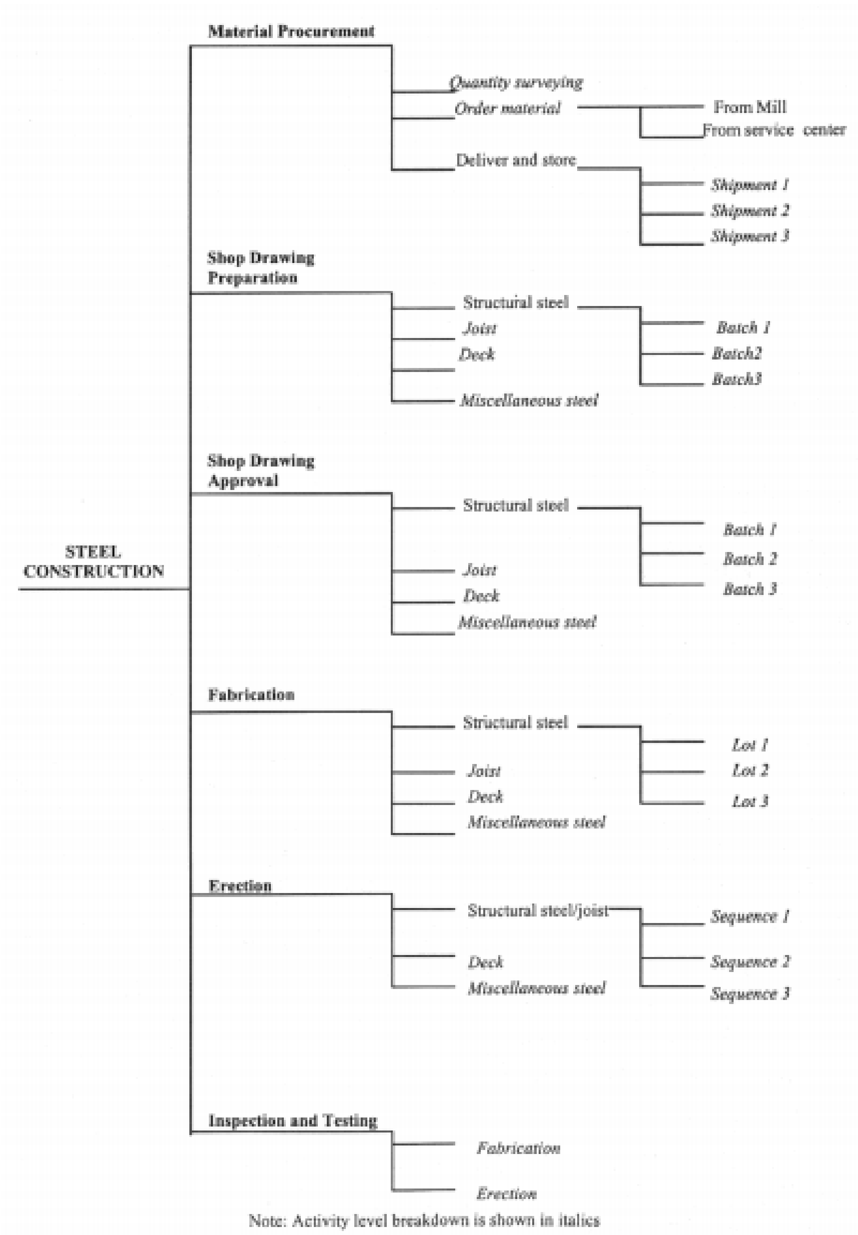

FIGURE 42: TYPICAL WORK BREAKDOWN STRUCTURE (WBS) FOR STEEL CONSTRUCTION, TAKEN FROM CONSTRUCTION MANAGEMENT OF STEEL CONSTRUCTION

[41] Construction Management of Steel Construction

The time required for each type of design varies, and a specific time cannot be put on either method. The duration for each option is very quick, but in the end, the project with the best coordination and fewest delays prevails. Precast concrete lead time is often faster than that of steel members[42], however, depending on when the materials are ordered, this may not have an effect. Because both methods are so similar, they both face related issues that can negatively impact schedule. These include, but are not limited to:

- **Material procurement**: It is important to place orders for the required materials as early as possible. Shortages in materials, delays in production or shipment, unusual member sizes, and overbooked fabrication shops are some of the main problems that can lead to schedule interruptions.
- **Shop drawing approval**: Getting drawings approved can take time, and issues may lead to delays.
- **Coordination among subcontractors**: Delays caused by one subcontractor can affect the outcome of the entire project. Therefore, effective coordination is required between all parties involved.[43]

The outcome for every project is different, but it is evident that for both precast concrete and steel construction, where fabrication of the materials takes place off-site, many issues can arise, which validates the importance of a detailed and organized schedule. Additional discussion on these two options can be seen in 9.0 Cost Estimation.

[42] Bridges

[43] Construction Management of Steel Construction

6.0 CONSIDERATION OF LEED CERTIFICATION

As part of the design/build contract, WPI has laid out the requirement for the residence hall to be LEED certified. In order to achieve LEED certification, the building is required to meet certain criteria and receive a number of different credits and points. To become LEED certified, 40-49 points must be achieved. There are other LEED certification levels that can also be achieved, as shown in Table 17.

TABLE 17: LEED CERTIFICATION LEVELS, TAKEN FROM LEED GREEN BUILDING CERTIFICATION

LEED Certification Levels	Points Required
Certified	40-49
Silver	50-59
Gold	60-79
Platinum	80 and above

The number of points received for each credit is weighted differently and is allocated "based on strategies that will have greater positive impacts on what matters most – energy efficiency and CO_2 reductions." [44] These credits promote sustainable design and lessen the environmental impact. The following seven LEED credits are some of the criteria which the project achieved:

1. Credit Sustainable Sites 7.1
2. Credit Sustainable Sites 7.2
3. Credit Material and Resources 2
4. Credit Material and Resources 4
5. Credit Material and Resources 5
6. Credit Material and Resources 7
7. Credit Indoor Environmental Quality 3.1[45]

Credits 1 and 2 are achieved by having paving and roofing materials compliant with requirements set forth by LEED for solar reflectance index. With materials meeting these criteria, the heating and cooling requirements of the building will be decreased. Credit 3 is aimed at having

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[44] LEED Green Building Certification

[45] USGBC

a waste management plan during construction to limit runoff and the addition of pollutants into the environment. Credit 4 outlines the use of materials with recycled content which is aimed to reuse resources. Credit 5 is required to determine the amount of regionally extracted materials. Materials that come from the region cut down on the amount of fossil fuels burned in shipping those materials from elsewhere in the world. Credit 6 aims to identify the amount of certified wood that is to be used in the building. The last credit, 7, required outlining an air quality management plan for the construction phase of the project.[46]

These LEED credits are the basis for the sustainable design aspect of the project. The seven credits listed above only add up to 8 LEED points, meaning that at least 32 more LEED points are required, as well as a certain number of credits from each individual category, for the dormitory to become certified. There are certain credits for the different categories that do not add points to the count, and are simply required. These credits are known as prerequisites and must be met in conjunction with the other credits (that add up to 40 points) in order for a building to become LEED certified. Additionally, there are some credits which are given a higher priority/more points because they are deemed more important towards the sustainability of a structure.[47] By analyzing DOC's entire LEED checklist, it was determined that the new dormitory will attain at least 43 LEED points.[48] A table portraying the credits that will definitely be achieved by the new residence hall can be seen in Appendix D.2 LEED Credits Checklist.

[46] USGBC

[47] USGBC

[48] Daniel O'Connell's Sons, Inc.

7.0 Scheduling

"A good plan for a construction project is not a plan which can complete the project in the shortest time but it is the plan which can safely execute the project in a given time and budget without compromising the quality and without any litigation."[49] Scheduling is one of the most important aspects of the planning process for construction projects. The scheduling for the residence hall was crucial because the goal was for the dormitory to be opened to students in August 2013 before the start of the school year. The construction process had about one year to be completed; therefore, falling behind schedule could be extremely costly. With students expecting to move into the residence hall in August, if the building was not completed, not only would DOC face cost penalties, students would be left without proper housing.

A condensed version of DOC's schedule was entered into *Primavera*. Figure 43 displays a portion of the timetable for the construction processes. The schedule is split by wing (Lancaster Wing, Faraday Wing, and Grove Wing), creating three similar segments within the one timetable. The labels next to the green bars depict the activity of construction and its associated wing. The complete *Primavera* schedule can be seen in Appendix D.3 Complete *Primavera* Construction Schedule.

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[49] Sharma

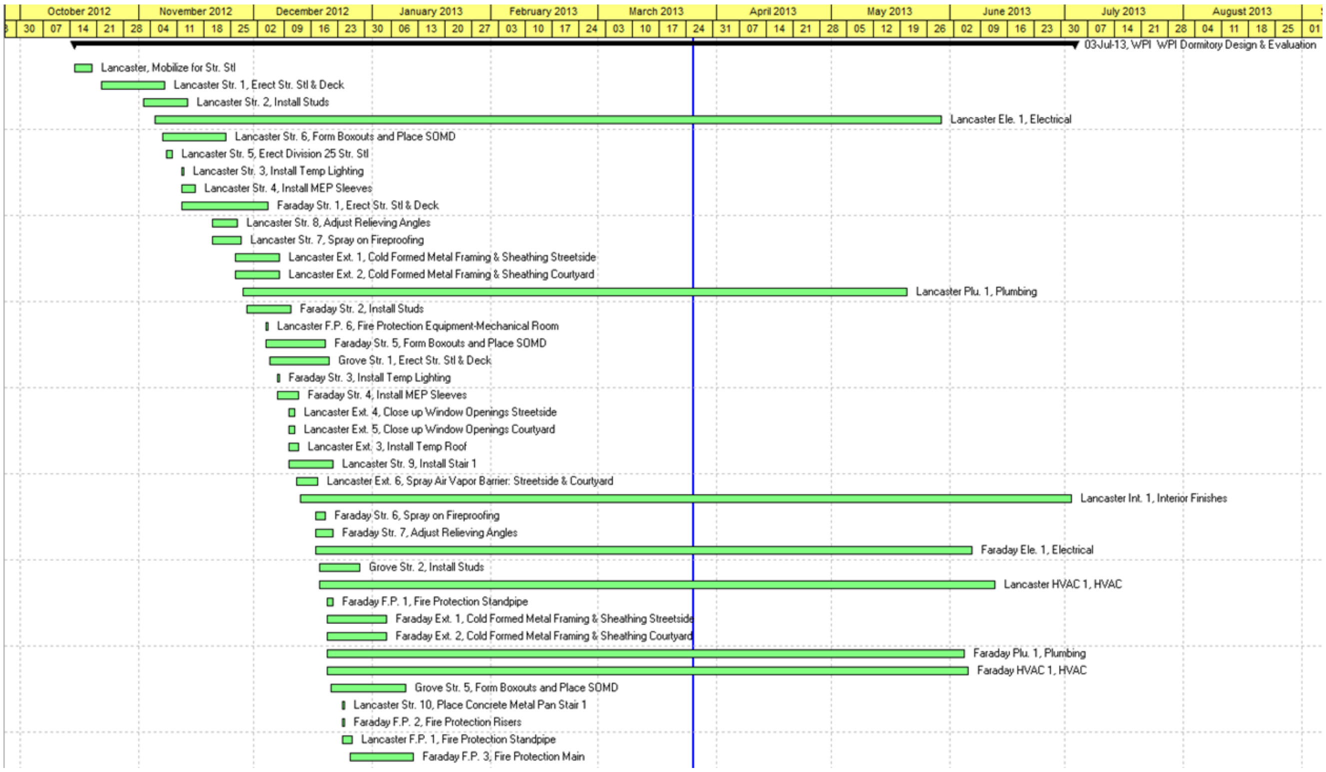

FIGURE 43: *PRIMAVERA* CONSTRUCTION SCHEDULE

Due to the scope of this project, many of the divisions (interior finishes, HVAC, plumbing, electrical, testing) were combined to obtain a total duration. The structural, exterior finishes, and fire protection aspects were left in a more detailed form to display the different features that must be considered. By examining the schedule, the interconnectivity amongst the various activities became apparent. Many activities relied on the completion of other activities before their construction could begin. It became evident that the dependence of early activities to later activities meant that actions that were being completed during overlapping time periods had to confirm that the location of the work or specific building features were already constructed or finished.

In order to understand the time requirements for each division of construction, the total durations for each division were combined from each wing. The division durations were divided by the total duration of the project to determine the percentage of time spent on each division, and this information is displayed in the pie chart shown in Figure 44. Calculations can be seen in Appendix D.4 Schedule Data.

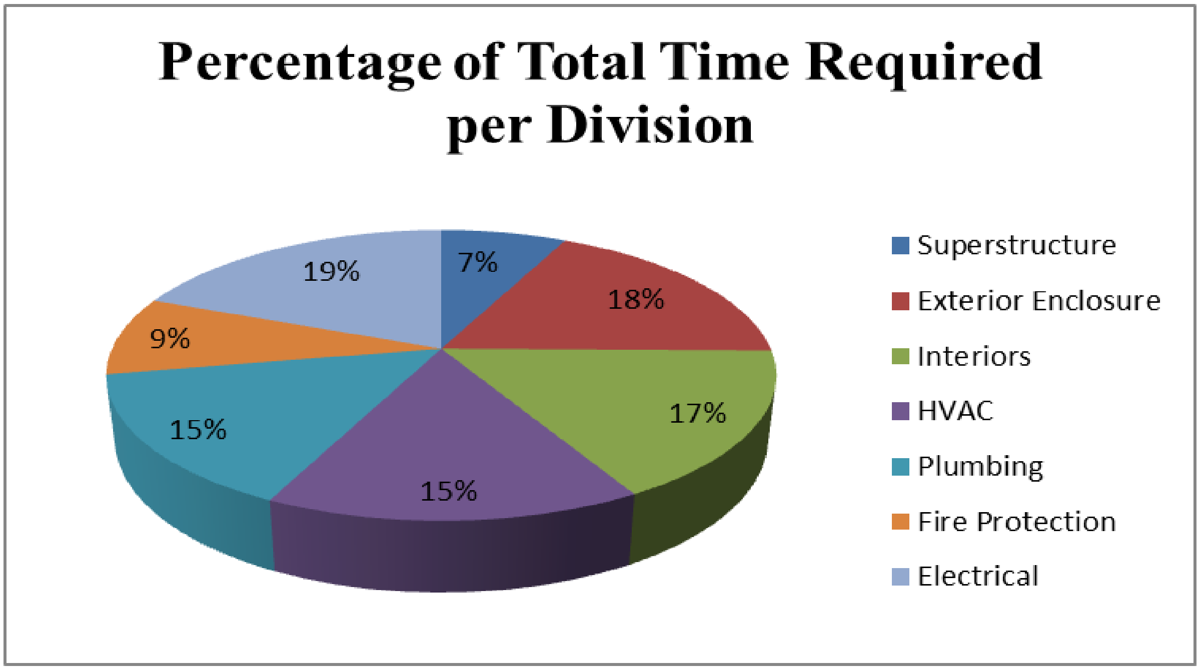

FIGURE 44: PERCENTAGE OF TOTAL TIME REQUIRED PER DIVISION

As illustrated by the pie chart, the MEP aspects of construction consume a majority of the total time in comparison to the other features. The superstructure division was a very small percentage of the total time; however, the percentage used for the pie chart only refers to the steel construction. Footing and foundation work and site work were not included in the superstructure schedule duration. It is interesting to note the large difference between the time required for steel framing construction (superstructure) and the time required for the MEP work. One explanation to this large difference that was discovered was the fact that much of the MEP work required testing after it had been installed, which increased the amount of time before the division was completed. Another explanation for the large percentage difference was that many divisions included much more intricate work than the steel construction. The superstructure is the first aspect of construction, and often times steel erection does not have to be coordinated with the other divisions because it must be completed before installation of the enclosure, MEP, and interiors can even begin. "The structural steel phase has a significant impact on the overall project schedule; completion of the structural frame is generally considered a significant milestone in overall project completion. Completion of the frame allows the work of the architectural, mechanical, electrical, and finally the finishing trades, to proceed."[50] Those divisions that follow the steel erection are often faced with the challenges of coordinating with one another, and many times the start date for one trade relies on the finish dates of other trades.

Scheduling is a very detailed and important aspect of construction projects. When project deadlines are on the line, such as the case for the new residence hall, it is crucial that each activity is completed on time. One late activity can alter the finish date of the entire project. For the new residence hall, the schedule illustrates how the productivity for each day was vital to successful completion of the project; one bad day due to weather or delayed arrival of materials could push back numerous activities and make it very difficult to get back on track.

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[50] Construction Management of Steel Construction

8.0 Sprinkler Design and Layout

According to the *Massachusetts State Building Code*, an automatic sprinkler system is required for the residence hall. This sprinkler must be installed according to the requirements of *NFPA 13: Automatic Sprinkler Systems Handbook*. *NFPA 13* provides the minimum requirements for the design and installation of automatic sprinkler systems. The following section discusses the applicable sections of *NFPA 13* as well as present the layout and design of the system which will be installed in the residence hall.

8.1 System Selection

There are two types of systems that can be installed; wet pipe systems and dry pipe systems. Wet pipe systems have water located within the pipes, while dry pipe systems are filled with pressurized air, and once the system is actuated, water starts to flow through the pipes. There are both positives and negatives to both systems. For dry pipe systems, there is a longer time between the sprinkler being actuated and water flowing onto the fire in comparison to wet pipe systems. Wet pipe systems are prone to freezing in an unheated building which is possible in the northeast. For the residence hall, a wet pipe system was selected because the building will be heated and therefore the pipes are not prone to freezing. Additionally, only one sprinkler system is required due to the size of the building. A sprinkler system protecting a light hazard occupancy, which is determined to be the case in the next section, is capable of protecting up to 52,000 ft^2 of floor space. This limit is a per floor limit because the scope of the code outlines that the design of a sprinkler system according to *NFPA 13* refers to a single fire at one point in the building. In such an event, the sprinklers on only one floor of the building would be operational, and each floor is slightly over 21,000 ft^2, creating a system within the limits of a light hazard occupancy.

8.2 Sprinkler Selection

The selection of the types of sprinklers present was based upon the apparent hazards present in the building, the building occupancy, and the required orifice size for the sprinklers. First, dormitory-type buildings are considered light hazard occupancies. This type of hazard is considered the least prone for fires with the least hazardous materials. Typically, the most hazardous materials found within the building are the mattresses, furniture and personal possessions. Additionally, there is a mechanical room on the first floor which would create an additional hazard classification. This area is considered Ordinary Hazard Group I which allows for the same orifice sized sprinklers to be used as for the remainder of the building.

8.3 Sprinkler Comparison

First, a design area was chosen in order to complete the sprinkler comparison. This design area is required to be in the most hydraulically remote section of the building. In a system such as this one, there is one cross main with branch lines running off of it. The water main enters the building on the Lancaster Wing in the mechanical room which makes the most remote portion of the building the Grove Street wing corner on the third floor. Even though the mechanical room is a higher hazard category, the hydraulic demand is very little because it is where the water main enters the building. In the design area, a comparison was made between using either Schedule 40 Black Steel pipe or CPVC Pipe. The comparison was created in order to determine which pipe material would be more economically feasible when looking at many factors, including the physical cost of materials, maintenance, and the size of the fire pump needed to complete the system.

While designing a sprinkler system, there are many different aspects that are regulated by the code. However, there are also a few aspects which are left up to personal preference. One of those aspects is the type of pipe to be used in the system. A few of the different types of pipe that can be used are copper, steel, and chlorinated polyvinyl chloride (CPVC). Each type pipe is available in different Schedules, 10, 40, 80 etc. The schedule of the pipe refers to the relative thickness of the pipe wall. Higher schedules are thicker and therefore the internal diameter of the pipe decreases. Also, thicker schedules are normally used for harsher environments like exterior or underground applications. Additionally, each type of pipe is usually more prominently used in different scenarios. Although black steel pipe is one of the most common pipe types, the other types of pipe have common uses. CPVC generally comes in an orange color, and since it is plastic, it is lightweight in comparison to the black steel pipe. CPVC is becoming more prominently used in residential occupancies. Also, the Hazen-Williams C facto for CPVC is larger than that of black steel. With a larger C factor the pipe is smoother, and therefore creates less friction which reduces the pressure and flow necessary for the system.

Originally, Schedule 40 Black Steel pipe was chosen for the system. This decision was based solely on its widespread use in sprinkler systems. Once the material was determined, the hydraulic equations for pressure, flow and friction loss were used to determine the pressure and flow first at the most remote sprinkler and then throughout the design area. Once a certain number of sprinklers are included in the system, the pipe size needs to increase which therefore changes the friction factors. This transition can be seen in the sample calculations found in Appendix C.11. Using this material, the required flow is 180.1 gpm at 440.8 psi. The complete calculations can be seen in Appendix C.11 Sprinkler Design Calculation Spreadsheet. Additionally, there is a required inside and outside hose stream of 100 gpm and 150 gpm, respectively. These create a total required flow of 430.1 gpm at 440.8 psi. The static pressure (no flow) at the test hydrant is 140 psi, while the residual pressure (full flow) at the test hydrant is 138 psi. These elements are needed to determine whether or not a fire pump is required to increase flow or if the city supply is adequate. Consequently, the flow and pressure supplied by city water mains are not sufficient, and for this reason, a pump is needed to increase the pressure to the sprinklers. If a pump was not installed, the water would not discharge from the sprinkler at a sufficient pressure and would not achieve the performance requirements of the system. Using the static and residual pressures provided by the city water supply, a water supply curve was created to show the adequacy of the system. The addition of this water supply curve and a pump curve provided by manufacturers, gave the total supply flow and pressure of the combined systems. Provided the combined curve is greater than the necessary flow and pressure, and adequate pump was determined. A pump that would meet the desired requirements for the scenario can be seen in Figure 45 below.

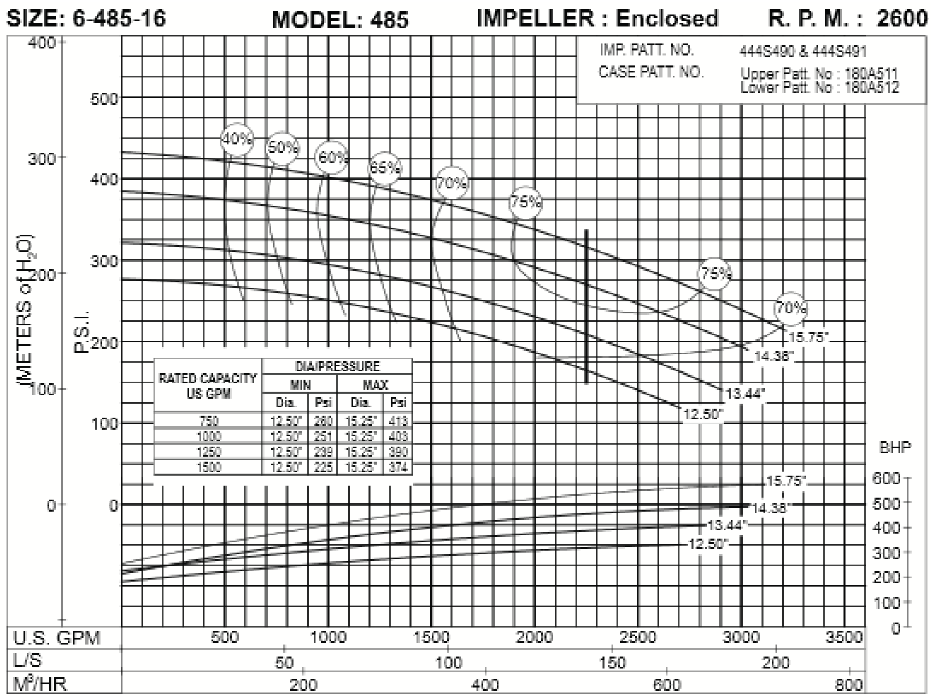

FIGURE 45: ADEQUATE DIESEL PUMP FOR BLACK STEEL DESIGN[51]

As seen in the pump curve above, at 430 gpm, the pump is capable of supplying an additional 450 psi which creates adequate pressure for the system.

For the purpose of the comparison, CPVC pipe was chosen for the system as well. This was because CPVC is becoming increasingly common in residential occupancies due to its flexibility, lightweight properties, and cost. Using this material, the required flow is 169.4 gpm at 274.9 psi. Including the inside and outside hose stream allowances of 100 gpm and 150 gpm, respectively, the total required flow is 419.4 gpm at 274.9 psi. The static pressure (no flow) at the test hydrant is 140 psi, while the residual pressure (full flow) at the test hydrant is 138 psi. Even with a lower required pressure, because the CPVC has less friction loss, a pump is still required to offset the residual pressure supplied by the city water mains. A pump that would meet the desired requirements for the scenario can be seen in Figure 46 below.

[51] Pentair

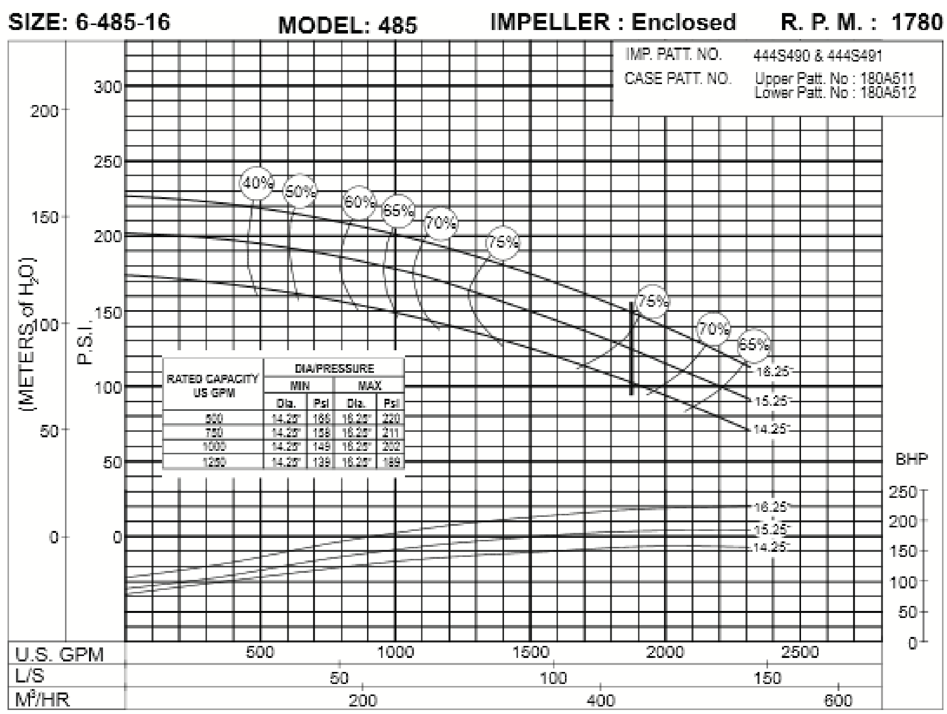

FIGURE 46: ADEQUATE ELECTRIC PUMP FOR CPVC DESIGN[52]

The pump above is a smaller pump than what is required for the black steel design. The pump for the CPVC system is also a pump with an electric driven motor as opposed the diesel driven motor for the black steel system. According to RS Means, a pump of this size has a lower initial cost per unit. Also, diesel pumps have more of a maintenance requirement compared to electric pumps.

8.4 Sprinkler Layout

The hydraulic calculations are completed simultaneously with the layout verification. The hydraulic calculations are dependent on the locations of the sprinklers in comparison to fittings and other construction members. This simultaneous completion makes for a trial and error type design and changing different aspects of the design and layout in order to achieve the best possible design. Changing one aspect of the layout will change the hydraulic calculations of the resulting sprinkler or run of pipe and therefore change the remainder of the elements. While the design was being completed and the characteristics of the sprinklers were chosen, the sprinkler layouts were also

[52] Pentair

verified. The sprinkler layouts were completed on a per room basis as the majority of the rooms are identical with the exception of the ground floor. The layouts of four typical areas can be seen below. These identical floors and rooms make creating the layouts easier as well as promote the constructability of the system.

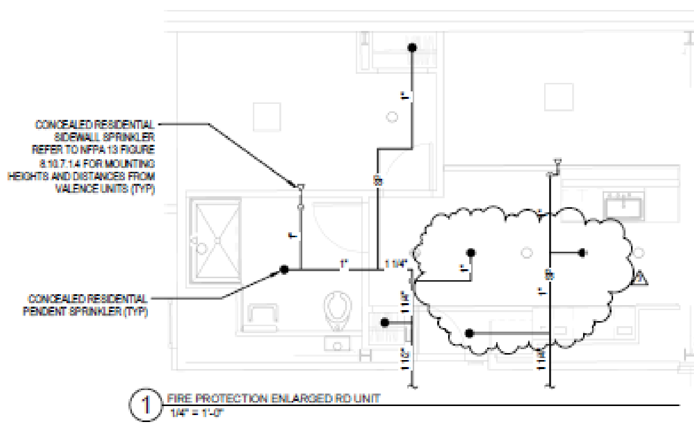

FIGURE 47: TYPICAL SPRINKLER LAYOUT FOR RD UNIT, TAKEN FROM DOC

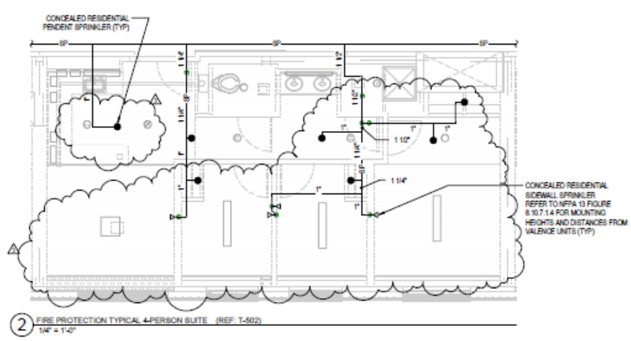

FIGURE 48: TYPICAL SPRINKLER LAYOUT FOR FOUR PERSON SUITE, TAKEN FROM DOC

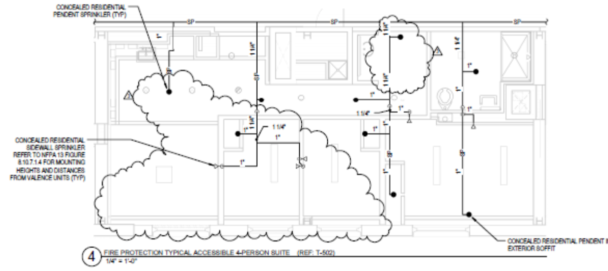

FIGURE 49: TYPICAL SPRINKLER LAYOUT FOR FOUR PERSON ACCESSIBLE SUITE, TAKEN FROM DOC

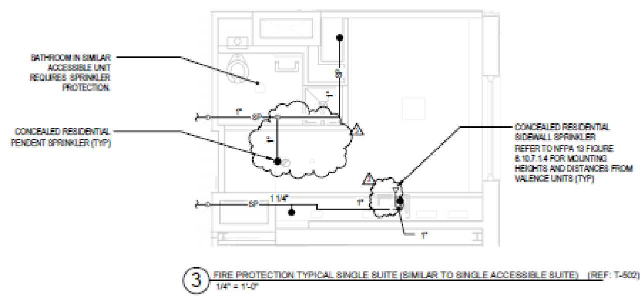

FIGURE 50: TYPICAL SPRINKLER LAYOUT FOR SINGLE SUITE, TAKEN FROM DOC

In addition to the areas shown above, the corridors on each floor will have a typical sprinkler layout. The typical layout for the corridors can be seen in Figure 51 and Figure 52 below.

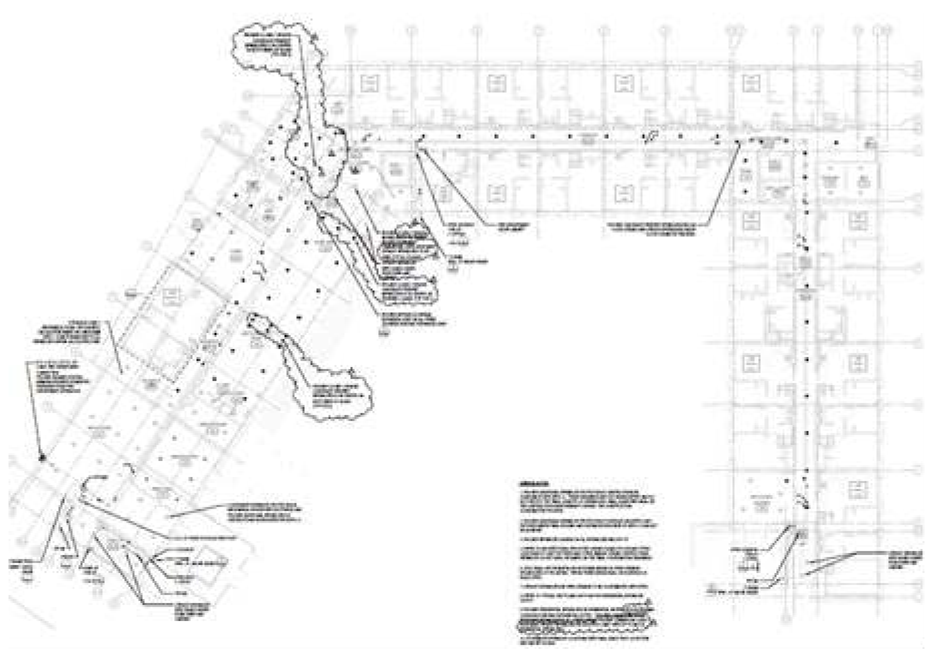

FIGURE 51: GROUND FLOOR COORIDOR SPRINKLER LAYOUT, TAKEN FROM DOC

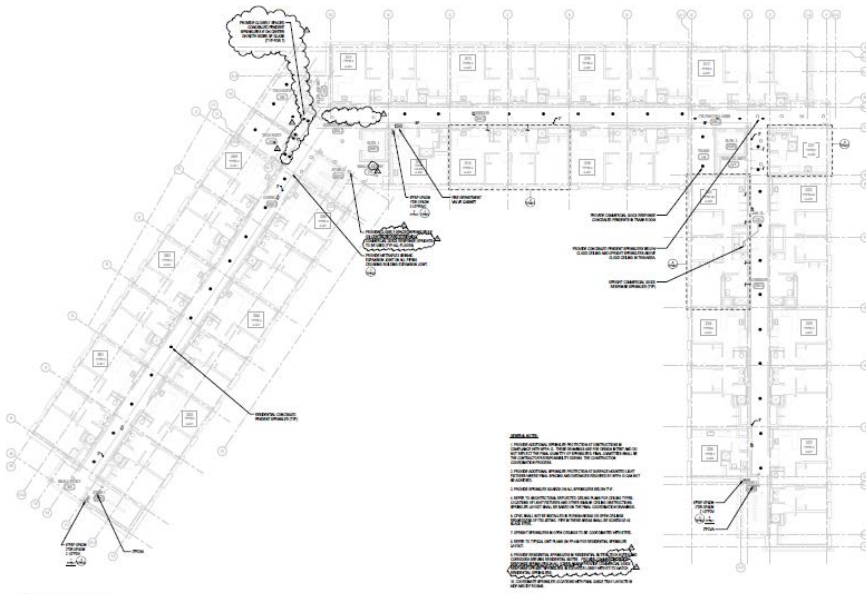

FIGURE 52: FLOORS 2-4 TYPICAL COORIDOR SPRINKLER LAYOUT, TAKEN FROM DOC

Using the designs for black steel and CPVC pipe as outlined earlier in the results section, the layouts above were deemed adequate for use in the residence hall with the stated characteristics. The layouts taken from DOC were verified using the design area created in Section 4.5. As long flow and pressure provided for the design area is adequate, the remaining aspects of the system are only required to meet the spacing and protection area requirements for the specific sprinklers. If the characteristics in the building or the general use of the building were altered, the sprinkler system would have to be checked for adequacy.

9.0 COST ESTIMATION

An important aspect of the project which needs to be addressed is the cost. When it comes to the real world, the final cost would be known until after the job is completed; however, a relatively accurate estimation is determined during the bidding process. This estimation will include all aspects of the project, from the structure itself to the mechanical equipment and testing processes to the final interior finishes of the building.

The cost estimation for this project primarily focused on the steel/concrete structure, fire protection features, spray-on fire proofing (SFRM), HVAC, electrical, and interior/exterior finishes. The need to estimate the cost of SFRM is a necessary aspect of the total cost for the structure because all of the steel members must be coated even though they are not combustible. When steel is heated excessively it loses its capacity to sustain the loads it had safely supported before the heating event. Costs were determined using unit cost values found in the *RS Means* reference book.[53] The table below (Table 18) is going to outline the total costs associated with this building.

TABLE 18: TOTAL COSTS OF PRECAST CONCRETE AND STEEL[54]

Total Costs			
Precast Concrere		**Steel**	
Item	**Cost**	**Item**	**Cost**
Beams	$ 913,000	Beams	$ 763,000
Columns	$ 699,000	Columns	$ 579,000
Slabs	$ 745,000	Slabs	$ 1,033,000
SFRM	N/A	SFRM	$ 358,000
Fire Protection	$ 176,000	Fire Protection	$ 176,000
Total Cost	**$ 2,533,000**	**Total Cost**	**$ 2,909,000**

In essence, the cost estimation offers a comparison of the steel and concrete designs. The cost of the steel beams and columns used in construction were found using tables created in *Revit*

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[53] Waier

[54] Waier

that list the different sized members, the number of elements for each member size, and their total lengths. The detailed tables and cost calculations can be seen in Appendix C.9 Spreadsheet of Beams and Cost for Steel Design and Appendix C.10 Spreadsheet of Columns and Cost for Steel Design. Aside from the steel and concrete design, the remaining features of the building did not differ significantly depending on structural system was used. These main features would be constructed using the same materials, however, the manner in which these features are attached to the structure will change whether steel or concrete is used. Although different, these features will not change the total cost of the design drastically and for the purpose of this comparison, the difference between those costs will be omitted.

As seen in Table 18 above, the steel structure is more costly than the concrete design. The physical beams and columns themselves are not more expensive when it comes to steel. However, all of these beams and columns need to be covered with spray-applied fire resistive material (SFRM) in order to provide the required fire resistance rating. Additionally, the sprinkler system that was designed would have similar differences when it comes to the supporting features as do the other interior features. The major components of the system will not change (sprinkler type, pipe, pump, etc.) but the bracing and members that attach the system to the building structure will change. In comparison to the sprinkler system as a whole, their difference will not account for a large portion of the total cost, and therefore the difference in the cost between the systems whether it was installed in a steel or concrete building is minimal. According to RS Means, a representative residence hall construction project has roughly 8.6% of the total cost towards HVAC ($942,877.645), 11.7% of the total cost towards electrical ($1,282,752.15), 26.4% of the total cost towards interior finishes ($2,894,415.10), and 6.1% of the cost towards interior finishes ($668,785.31). These percentages account for an additional $5,788,830.21 above the total construction costs for both the steel and concrete designs.[55] Figure 53 below displays the percentages of the construction costs per division.

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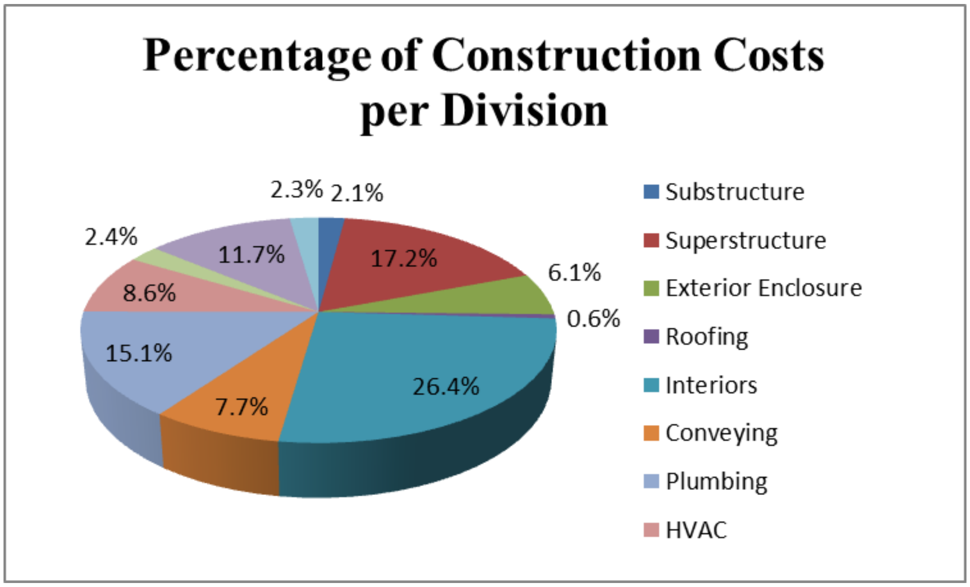

FIGURE 53: PERCENTAGE OF CONSTRUCTION COSTS PER DIVISION

Overall, the steel design ends up being more expensive than the concrete design. This difference is primarily due to the addition of the SFRM on the steel members which is not required for the precast. However, another aspect to look at would be the constructability of each method. The cost for one to be constructed may be vastly different depending on the ease of construction and faster construction times.

10.0 Code Compliance Review

One of the first steps necessary when completing a building design is ensuring that the building is compliant with the applicable codes and standards. The following review will discuss the different aspects of the building and fire codes as they pertain to life safety.

10.1 Applicable Codes and Standards

The Commonwealth of Massachusetts sets forth their building and fire regulations through the Code of Massachusetts Regulations (*CMR*). The applicable codes and standards related to this project are

- *780 CMR*: *Massachusetts State Building Code (8th Edition)*
- *527 CMR*: *Massachusetts Fire Prevention Regulations*

The *Massachusetts State Building Code* is an adoption of the *2009 International Building Code* with state amendments. The State Fire Code, *527 CMR*, is a document that has been written and adopted by the Board of Fire Prevention Regulations (BFPR) throughout the years although they are currently undergoing the adoption process to *NFPA 1: Fire code*. However, as that code has yet to be adopted, *527 CMR* is the document which will be applicable to this building design.

These codes lay out the requirements for how a building is to be constructed, what materials can be used, how those materials will react to fire, amongst many other topics. The specific requirements are identified by determining the type of occupancy and the different hazards that would be present in the space. For example, the hazards that are found in a school are much different than the hazards found in a hospital, and therefore they would need to be addressed in different ways. These codes are written for the protection of property and contents, and for the protection of adjacent properties with a secondary aspect of life safety being addressed to a lesser degree.

10.2 Building Overview

The building in question is a 4-story residence hall for the housing of graduate level students at Worcester Polytechnic Institute (WPI). The residence hall has 72, two bedroom apartment-style suites giving a total of 258 beds. This type of living arrangement classifies the building as a Group R-2 occupancy according to IBC, 2009, Section 310.1. The study areas on each floor and the mechanical areas on the first floor are considered accessory to the dormitory as a whole, and therefore mixed-use occupancy requirements do not need to be considered. The dormitory building is a 4-story/50-ft building with a floor area of 22, 182 ft^2. Type IIB construction is the minimum required construction type for the building; any lesser type of construction than that is constructed out of combustible materials.

10.3 Height and Area

According to Table 503 of the IBC, the limits of Type IIB construction are 4 stories/55 feet and 16,000 ft^2 before allowable increases are introduced for certain considerations. For instance, Type IIB construction is permitted to have building height and area increases if it has an automatic sprinkler system, and the fact that the building fronts on a public way on over 25% of its perimeter. According to IBC Section 504.2, the building is permitted a 1-story and 20-foot height increase; however, this increase cannot exceed 4 stories or 60 feet because the building is group R-2 occupancy.

Additionally, the building is subject to a 200 % area increase because it has an automatic sprinkler system and has more than 1 story above grade plane. The building is subject to increases due to frontage in addition to the increase for sprinklers. This increase for frontage is 70% of the value found in Table 503 of the *MSBC*. The calculations for the building area increases can be seen in Table 19 below.

TABLE 19: PERMISSIBLE HEIGHT AND AREA REQUIREMENTS, BASED ON THE *MSBC*

Group R-2		Code (*Massachusetts State Building Code*)
Height	Floor Area	
4 Stories, 55 ft	16,000 ft^2	Table 503
1 Story, 20 ft		Section 504.2 Height Increases
	32,000 ft^2	Section 506.3 Area Increases (Sprinklers)
	11,200 ft^2	Section 506.2 Area Increases (Frontage)
4 Stories, 60 ft (R-2 Maximum)	**59,200 ft^2**	**Allowed**
4 Stories, 50 ft	**22,182 ft^2**	**Actual**

The maximum total building area is determined by the number of stories above grade plane. Due to the fact that there are three stories above grade plane, the maximum allowable area per is multiplied by three to give the total building area. This factor yields a total building area of 177,600 ft^2 which is much greater than the actual total building area of 88,728 ft^2.

10.4 Fire Separation Characteristics

The physical characteristics of the building are also regulated by *780 CMR*. These characteristics are inclusive of the type of construction and different passive fire protection features including separation of different portions of the building amongst other characteristics. Primarily, these are the fire resistance ratings of various elements of the building construction. First, the fire resistance ratings of the different building elements are laid out in Table 601 of the *MSBC*. The requirements for Type IIB Construction can be seen in Table 20 below.

TABLE 20: FIRE RESISTANCE RATINGS FOR VARIOUS BUILDING ELEMENTS, BASED ON THE *MSBC*

Building Element	Type II B
Primary Structural frame	0
Bearing Walls	
Exterior	0
Interior	0
Non-Bearing Walls	
Exterior	0
Interior	0
Floor Construction and Secondary Members	0
Roof Construction and Secondary Members	0

As seen in Table 20 above, all of the different elements of the building are not required to have a fire resistance rating. This is due to the fact that the construction type is consistent with unprotected steel construction. Steel is considered a non-combustible material; however, unprotected steel has no fire resistance rating. This means that the steel is susceptible to the fire event and will start to be affected by fire at the start of the fire event. In addition to the requirements for the exterior walls in the above table, the fire resistance rating of these walls needs to be confirmed in relation to the fire separation distance (FSD). The FSD is determined by drawing a perpendicular line from the exterior wall to the centerline of the public street or arbitrarily created line between two buildings on the same lot. In this scenario, the FSD is twenty-five feet. The required fire resistance rating can be seen in the table below which is an excerpt of Table 602 of the *MSBC*.

TABLE 21: FIRE RESISTANCE RATINGS DEPENDENT ON FIRE SEPARATION DISTANCES, BASED ON THE *MSBC*

Fire Separation Distance (FSD)	Type of Construction	Fire Resistance Group R-2
FSD < 5 ft	All	1 Hour
5 ft < FSD < 10 ft	IA Others	1 Hour 1 Hour
10 ft < FSD < 30 ft	IA,IB IIB,VB Others	1 Hour 0 Hour 1 Hour
FSD > 30 ft	All	1 Hour

As seen in all of the different elements of the building are not required to have a fire resistance rating. This is due to the fact that the construction type is consistent with unprotected steel construction. Steel is considered a non-combustible material; however, unprotected steel has no fire resistance rating. This means that the steel is susceptible to the fire event and will start to be affected by fire at the start of the fire event. In addition to the requirements for the exterior walls in the above table, the fire resistance rating of these walls needs to be confirmed in relation to the fire separation distance (FSD). The FSD is determined by drawing a perpendicular line from the exterior wall to the centerline of the public street or arbitrarily created line between two buildings on the same lot. In this scenario, the FSD is twenty-five feet. The required fire resistance rating can be seen in the table below which is an excerpt of Table 602 of the *MSBC*.

Table 21 above shows that the exterior walls are required to have a zero hour fire resistance rating based on a FSD equaling twenty-five feet. The required fire resistance of the exterior walls is the greater of the values determined from Table 601 and Table 602 of the *MSBC*. However, both tables require no fire resistance rating for the exterior walls of this building. Additionally, the openings within the exterior wall are also regulated by 780 CMR as well as by the FSD. The table below is an excerpt from Table 705.8 of the *MSBC* which gives percentages of open area in relation to FSD.

TABLE 22: PERCENTAGE OF EXTERIOR WALL AREA OPEN DEPENDENT ON FIRE SEPARATION DISTANCES, BASED ON THE *MSBC*

Fire Separation Distance (FSD)	% of Exterior Wall Area Open
0 ft < FSD < 3 ft	Not Permitted
3 ft < FSD < 5 ft	15%
5 ft < FSD < 10 ft	25%
10 ft < FSD < 15 ft	45%
15 ft < FSD < 20 ft	75%
20 ft< FSD	No Limit

As stated earlier, the FSD around the perimeter of the building is twenty five feet. This twenty five foot distance yields an unlimited area of openings on the exterior walls. The last building characteristic to be addressed is the interior wall separation from specific rooms or areas.

As stated earlier, the FSD around the perimeter of the building is twenty-five feet. This twenty-five foot distance yields an unlimited area of openings on the exterior walls. The last building characteristic to be addressed is the interior wall separation from specific rooms or areas.

Although all of the building elements outlined in Table 601 and 602 of the *MSBC* required no fire resistance rating, there are other special aspects of the building which require a fire resistance rating no matter what the type of construction dictates. These ratings are found throughout Chapter 7, titled Fire and Smoke Protection Features in the *MSBC*. Primarily, according to Section 420 of 780 CMR, there is a required thirty minute separation between dwelling units adjacent, above, or below each other. Additional separation requirements as well as separation type can be seen in Table 23 below.

TABLE 23: BUILDING ELEMENT AND OPENING PROTECTIVE FIRE RESTIANCE RATINGS, BASED ON THE *MSBC*

Building Element	Separation Type	Fire Resistance Rating	Opening Protective Rating
Shaft Enclosures (4 Stories)	Fire Barrier	2 Hours	90 Minutes
Shaft Enclosures (3 Stories)	Fire Barrier	1 Hour	91 Minutes
Stair Enclosures (4 Stories)	Fire Barrier	2 Hours	92 Minutes
Stair Enclosures (3 Stories)	Fire Barrier	1 Hour	93 Minutes
Elevator Machine Room	Fire Barrier	2 Hours	94 Minutes
Emeregency Electrical Room	Fire Barrier	2 Hours	95 Minutes
Corridors Serving Dwellings	Fire Partition	0.5 Hours	20 Minutes
Trash Rooms Over 100 ft^2	Fire Barrier	1 Hour	45 Minutes

As can be seen above, the majority of interior walls are required to have some type of fire resistance rating. The requirements listed above are greater than the requirements of Tables 601 and 602 and therefore must be implemented in the building. These separation requirements are based on the need of property protection, and not life safety requirements.

10.5 Means of Egress

Egress is defined as the action of going out of or leaving a space. Therefore the means of egress would be the physical aspects of a building which allow for the egress of its occupants. The first aspect which must be addressed when it comes to the means of egress is the occupant load. The occupant load will determine the number of required exits and then those exit capacities will have to be sufficient for the occupant load. In addition to the occupant load, the occupancy category of the building will determine other requirements such as door swing, and common path of travel limits, amongst others.

10.6 Occupant Load

Two occupant loads were calculated for this building. The first being the occupant load of the first floor while the second refers to the occupant load of the second through fourth floor as those floors are identical. Each different area within a floor has a specific use, and each of these uses is provided with a different occupant load factor for its expected use. If the usage of a given room were to change, the occupant load would have to be revised to meet the corresponding requirements for the new usage. Each floor is not able to be blanketed with one occupant load factor due to the large differences in use types. Occupant load factors can be found in Table 1004.1.1 of the *MSBC*. The table below (Table 24) outlines the different areas within each floor, the square footage of each area, the differing occupant load factors, and the total number of occupants allowed in each area as well as the floor as a whole.

TABLE 24: OCCUPANT LOAD PER FLOOR BASED ON EXPECTED USE OF AREAS, , BASED ON THE *MSBC*

Floor	Area	Floor Area	Occupanct Load Factor	Occupanct Load
First	Dormitory	15,279 ft^2	50 gross ft^2/occupanct	306
	Mechanical/Storage	2,773 ft^2	300 gross ft^2/occupanct	10
	Multipurpose Room	769 ft^2	7 net ft^2/occupanct	110
	RD Apartment	610 ft^2	200 gross ft^2/occupanct	4
	Trash Room	457 ft^2	50 gross ft^2/occupanct	10
	Laundry Room	381 ft^2	50 gross ft^2/occupanct	8
	Gaming Area	368 ft^2	15 gross ft^2/occupanct	25
	Lobby	1,922 ft^2	15 gross ft^2/occupanct	13
	Lounges	85 ft^2	15 gross ft^2/occupanct	6
			Floor Total	492
Second-Fourth	Dormitory	19,792 ft^2	50 gross ft^2/occupanct	396
	Tech Suite	349 ft^2	15 gross ft^2/occupanct	24
	Study Room/Lounge	237 ft^2	15 gross ft^2/occupanct	16
	Mechanical/Electrical	225 ft^2	300 gross ft^2/occupanct	1
	Trash Room	126 ft^2	300 gross ft^2/occupanct	1
	Small Lounge	91 ft^2	15 gross ft^2/occupanct	7
			Floor Total	445
Building			Total Occupant Load	1,827

As seen above, the first floor has an occupant load of 492 persons while the second, third, and fourth floors each have an occupant load of 445 persons. This gives a total occupant load for the building as 1,827 persons. Based on these occupant loads, the prescribed number of exits required from each floor was determined. The number of required exits is outlined in Section 1021.1 of the *MSBC*. According to Table 1021.1 of the *MSBC*, stories with an occupant load between 1 and 500 people are required to have two exits. Thus, there are two exits on each of the second through fourth stories, and 4 exits on the first story since the story must accommodate the egress of all the building occupants. This is acceptable because the required number of exits from any story must be maintained until the occupants arrive at grade level or a public way. Even if the number of exits is deemed adequate, the exit stairs and doors need to have adequate width to sustain the number of occupants who will be using that exit.

10.7 Doors and Stairs

Doors and stairs must comply with width requirements in order to accommodate the number of occupants of each floor. Even if the occupant load deems otherwise, the minimum width of doors is 32 inches (IBC, 2009, 1008.1.1), and the minimum width of stairs is 44 inches (IBC, 2009, 1009.1). As well as having a certain width, doors are required to open in the direction of egress travel when the occupant load is greater than 50 persons (IBC, 2009, 1008.1.2).

In order to measure the adequacy of the door and stair widths specified for the DOC design, the door widths were divided by 0.15 inches/person and the stair widths were divided by 0.2 inches/person. These quotients determine the number of people who can safely exit through each doorway. Then, adding the persons for each door and stair for each floor provided the total number of occupants who can safely occupy each floor. This number must be greater than the occupant load, otherwise, the doors and stairs are not deemed adequate. Table 25 below outlines the widths of both the stairs and the doors, and the number of people that can safely exit through each door and stair.

TABLE 25: ALLOWABLE OCCUPANTS BASED ON STAIR AND DOOR WIDTHS, BASED ON THE *MSBC*

Floor	Stair	Stair Width	Exit Allowance	Door Width	Exit Allowance	Allowable Occupants
First	Grove Street	61"	0.2	34"	0.15	226
	Main Enterance	N/A	N/A	68"	0.15	453
	Stair 1	N/A	N/A	34"	0.15	226
	Stair 3	50"	0.2	34"	0.15	226
					Floor Occupant Load	1,131
Second-Fourth	Stair 1	50"	0.2	39"	0.15	250
	Stair 3	50"	0.2	39"	0.15	250
					Floor Occupant Load	500

As seen above, the doors and stairs provide an adequate width for the number of occupants in the building. In addition to the stairs being an adequate width, they are also required to be a certain distance apart from each other. This is in case part of the building is compromised then both exit stairs are not affected, and therefore the occupants of the floor can presumably exit safely. This distance is one-third the maximum diagonal of the building, which can be seen in Figure 54 below.

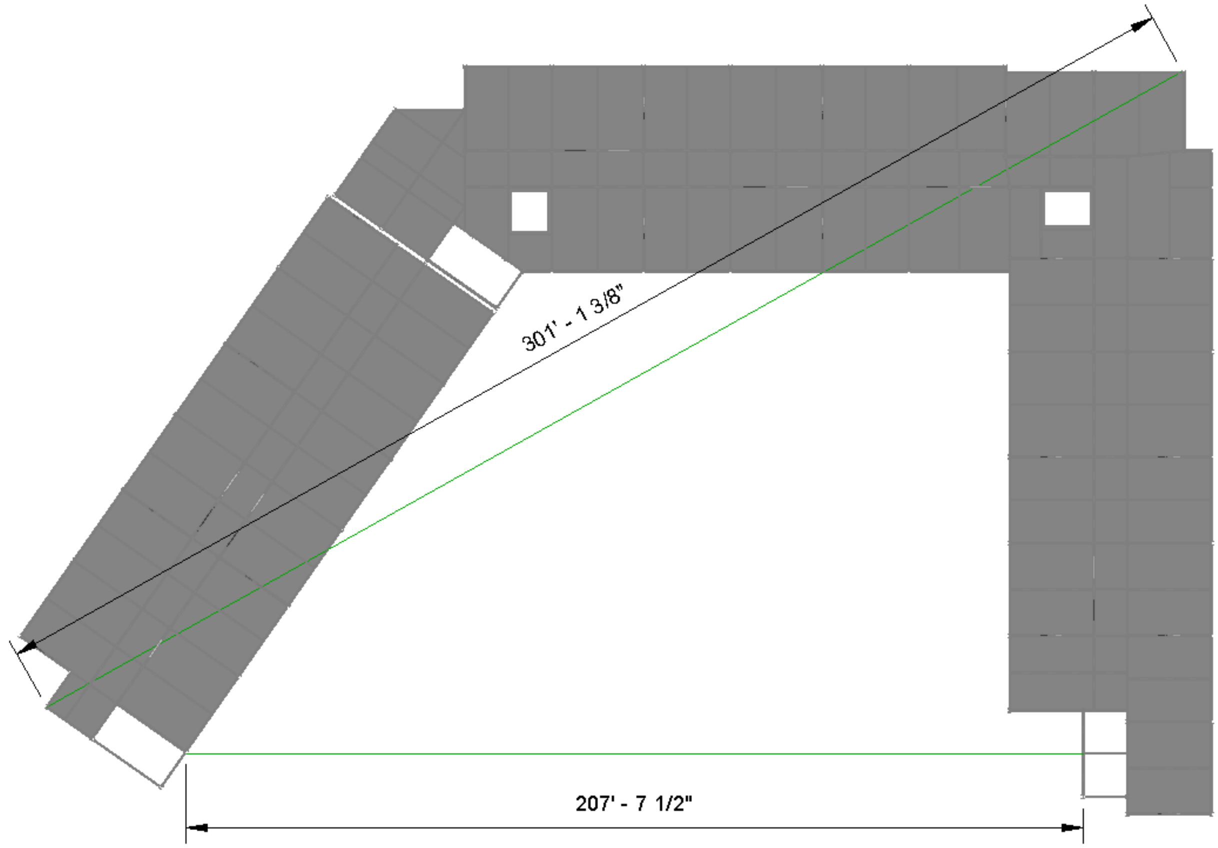

FIGURE 54: MEASURE OF REMOTENESS EXIT STAIRCASES

As seen in the figure above, the distance between the exits is adequate. This is because the distance between the two most remote exits, is 207 ft while the required separation distance is 100 ft. To complete the egress analysis, certain components of the corridors and travel distances were checked. First, the maximum travel distance from any point in the dormitory to an exit access is 250 feet. Additionally, there is a maximum distance of a dead end corridor of fifty feet. However, there is no dead end corridor in the residence hall. Last, the maximum common path of egress travel, the path which one must take before reaching a point where they can choose two distinct paths to reach an exit access, is 125 feet for residential occupancies.

11.0 Consideration of Broader Impacts

Some of the topics presented in this chapter include environmental impacts, social implications, and political implications. Interviews and research was performed in order to provide the following discussions.

11.1 Environmental Impacts

A major environmental concern determined in this project that was focused on was runoff and erosion from the site. Runoff is increased during construction due to the vegetation removed from the site and the increased activity and volume of runoff on site. The removal of vegetation from the site also creates a greater vulnerability to erosion.

In order to decrease or prevent environmental impacts such as erosion, control methods were researched. Control methods aim to disrupt erosion by providing soil stabilization, runoff management and sediment control.[56] Some options that have immediate effects are mulches, erosion control blankets, polymer soils, silt fences and straw bale barriers. The benefits of these control methods include the immediate mitigation effects and the ease of obtaining, placing and removing these systems. Another approach to erosion control is to combine multiple methods. For example, it is popular to find silt fences used in combination with straw bale barriers. Some more labor intensive control methods include constructing a swale, planting vegetation cover and making inlets to retention ponds or other area. These methods tend to provide better runoff control but take more time to create and setup. The removal of these control methods also is more difficult as the vegetation must be removed or the swales need to be filled in. These control methods can also be left permanently if planned ahead of time.[57]

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[56] Broz, Bob, Don Pfost, and Allen Thompson.

[57] Broz, Bob, Don Pfost, and Allen Thompson.

11.2 Social Implications

One of the aspects of any design-build project which needs to be considered is the social impact of the project and the effect it will have on the surrounding community. Although many of the buildings in the direct vicinity of the residence hall are owned by WPI, the owners of the other buildings should be consulted to see their views on the project and how it may affect their business.

One of the businesses that is directly affected by the new residence hall as well as Gateway Park as a whole is the Courtyard Marriot Hotel. The hotel is sandwiched between the Gateway 1 and 2 buildings and the new residence hall. An interview was conducted with Socrates Ramirez (January 2013), the Manager of the hotel, to learn his views on the major construction projects WPI is conducting in general, as well as how he feels the construction may positively or adversely affect his business.[58]

Overall, Mr. Ramirez was very positive about the construction. He sees the construction as a way of bringing more students to campus. The more students that visit the campus and become students at WPI, the more opportunities the hotel has to house family members, especially for special events like graduation and parent's weekend. He also expressed his feelings that any construction is not only good for his business and the hotel industry as a whole, but good for the City as a whole. Mr. Ramirez had seen this type of development in the time he spent living in Providence, RI and believes that the construction and growth that is being seen presently in Worcester is very similar to that which went on in Providence.

The principal negative aspect he saw was the construction process in general. Since a hotel is mainly used for sleeping and business functions, loud noises are definitely not ideal. With construction starting early in the morning, possibly before people wake up for the day, it can become a hindrance for people staying at the hotel. However, Mr. Ramirez knew that this was something that was necessary for growth in the City.

Overall, the construction of the new residence hall, as well as the ever expanding Gateway Park, has a very positive social impact on the immediate area and Worcester as a whole. The

[58] Ramirez

success and positive experience that this construction project gives the City will lead to more construction and a better city as a whole.

11.3 Political Implications

Large construction projects have many obstacles to overcome throughout their construction process. Political issues, such as changes in zoning and the effects a project could have on a neighborhood or city are critical aspects of gaining approvals. How a project fits in with the goals and plans of a city are essential questions to answer. Political figures can be some of the most influential people when it comes to new construction. Numerous people have been involved in the development of the new WPI residence hall in Worcester, and one of the forerunners is Councilor Philip Palmieri. Mr. Palmieri is currently serving his sixth, two-year term as the District Two Councilor. District 2 covers a large area of Worcester, including Lincoln Street, WPI, downtown east of Main Street, UMass, the Canal District north of Kelley Square, and several other areas.[59] Mr. Palmieri grew up in Worcester and is very knowledgeable on the current expansion and redevelopment efforts of WPI and the City of Worcester as a whole. With so many years of experience, Mr. Palmieri has had many interactions with the different colleges of Worcester. He has witnessed WPI grow with the development of Gateway Park, and has seen the effects it has had on the City.

In an interview with Mr. Palmieri (January 2013), significant insight into the political and social impacts that Gateway Park and the new residence hall have and will have on Worcester became clear.[60] Zoning was one of the first issues that was addressed. The zoning for the area of Gateway Park had to be changed from a light industrial zone, because it is not acceptable to build dormitories in this type of zone. Mr. Palmieri explained how, before construction could begin, the zoning for that location was changed so that it could encompass all of the desired Gateway Park buildings, including, but not limited to, the new residence hall. He clarified that in order to get the City to change the zoning of an area, you have to follow a regulatory process. A proposal for the change comes first and is brought in front of the city council and then a subcommittee. Next, the

[59] Councilor Palmieri

[60] Palmieri

planning board looks at the proposal. The most crucial aspect needed when pushing for a zoning change is a critical mass of neighbors to argue that this change is the right thing for everyone in that area. Mr. Palmieri clarified that you cannot simply change the zoning for one street or one block, it needs to be a change for a whole area. This process has many steps, and this was the first political step that needed to take place for construction of Gateway Park and the new dormitory to even begin. This makes it clear that changing the zoning of an area is not a simple task and involves much support and push. Mr. Palmieri expressed the importance of Gateway Park in helping to transition this area of Worcester from its light industrial zone of the past to a flourishing business and multi-purpose district.

The whole reason that Gateway Park was able to be developed in the first place was due to the fact that Worcester Polytechnic Institute partnered with the Worcester Business Development Corporation (WBDC) to purchase the biotechnology park and change this rundown site into an attractive area of Worcester. This depicts the intricacies behind the dormitory project, and how this building did not simply pop up, it took years of planning and many different parties. Mr. Palmieri referred to the redevelopment of this area as turning "brownfields into cornfields."

Focusing more on the political and social aspects of the new dormitory, Mr. Palmieri stressed the benefits this dorm will have on downtown Worcester. Simply stated, a new building used for housing (especially one the size of the new residence hall) close to downtown, means that more people will be within walking distance of the stores, restaurants, bars, etc. that are located downtown. Therefore, downtown Worcester should see an economic improvement. Mr. Palmieri discussed how it is a goal of Worcester to increase the activity of downtown and draw more people, especially younger people, to it. The addition of the dormitory is expected to be a strong contributor to this endeavor. People often see the greater good in adding housing to an area Mr. Palmieri explained, and he believed that the increased foot traffic in Gateway Park and the surrounding areas would only better the City of Worcester.

With any large construction project, in our case the new residence hall (and all of Gateway Park), Mr. Palmieri made it clear that one of the most important features necessary to make it a success is for it to fit in with the goals of the City it is in and be accepted by those around it. There are many political efforts to revitalize and redevelop Worcester, of which, the Gateway Park area of District 2 is the focal point, according to Mr. Palmieri. Therefore, the expansions and additions that WPI makes are often widely accepted because of their beneficial outcomes for Worcester. The new residence hall has a solid political advantage because of the way it will help with the City's plans. Just as Mr. Palmieri described, WPI will continue to be an anchor for the Northern sector of Worcester and the new dormitory will only benefit District 2 and downtown Worcester.

12.0 Conclusions

Worcester Polytechnic Institute (WPI) is growing and expanding its campus and academic offerings, which has created a need for increased residency for graduate students. The construction of a residence hall within the Gateway Park area increases the number of graduate students that are able to pursue graduate studies at WPI. The ability to house graduate students makes WPI a more desirable campus for potential students. The need for the residence hall can be seen in increased acceptance rates and a greater desire to be housed on campus. Before any new buildings are constructed in the Gateway Park region, designs need to be completed to determine their adequacy with state and local building codes and to ensure their stability in adverse conditions.

Throughout this project many different design aspects and conditions were addressed. The design of the primary structural elements was completed for both steel and concrete systems using the requirements of *ASCE 7*, *PCI* and *AISC* as referenced by the *MSBC*. These design aspects included gravity loading, such as dead loads, live loads and snow loads, and lateral loading, such as wind and earthquake loads. Using these loads it was possible to design the building following the load distribution from the concrete slabs, to the beams and girders, to the columns, and down to the footings. While completing the steel design, beams, girders, and columns were selected from *Steel Construction Manual*. The beams, girders, and columns for the concrete design were taken from *PCI* design tables. The design completed by DOC was verified using these techniques, and an alternate design using precast concrete members was completed for comparison. From the comparison, it was evident that the steel design was more costly; however, the steel construction process may be more fluid and have a faster completion time which may offset labor costs to even out the total costs.

In addition to the structural design, some passive measures of fire protection were evaluated to determine their compliance with the provisions of the *MSBC* and *527CMR*. These measures included, but were not limited to, height and area maximums, occupancy loads, and widths of doors and stairs. These aspects are used to ensure the safety of the occupants in making sure buildings with lower safety ratings are less expansive and do not become overcrowded in the event of a fire or other emergency. Many fires with large loss due to fire, like the Station Nightclub Fire in Rhode Island and the Kiss Nightclub Fire in Brazil, have been due to overcrowding and buildings not compliant with the applicable codes. Passive fire protection measures are generally accompanied by active fire protection measures, such as sprinkler systems, to provide additional protection.

The last design aspect which was addressed was the design of the sprinkler system required by the *MSBC*. The sprinkler system was designed in accordance with *NFPA 13*. The system was designed based upon the intended use and occupancy and the typical commodities that are expected to be present within the building. The typical commodities and uses, as well as the orientation of the building, delegate the type and spacing of sprinklers required for the project. For the sake of comparison two different piping materials, black steel and Chlorinated Polyvinyl Chloride (CPVC), were used to determine the required pump size and number of sprinklers which would be required within the building. Black steel pipe was chosen to be used even though it was more costly because CPVC is generally accepted to be used in small residential houses and not large-scale construction project such as this.

In addition to the physical design associated with the project, some construction management aspects, mainly scheduling and cost estimation, were reviewed. The predicted costs for each design aspect, steel, concrete, and fire protection, were determined using the *RS Means Total Construction Costs*. These costs only provide two aspects of the total construction costs. The remainder of the costs (HVAC, electrical, etc.) were estimated in comparison to the structural costs using *RS Means Square Foot Costs* for a representative residential hall. A schedule was created using *Primavera* software. It was a condensed version of the timetable created by DOC, and was used to gain a better understanding of the time required for each division of construction. Using a pie chart to compare the percentages of time for each division, it was determined that the more intricate and detailed aspects of construction, such as electrical and plumbing work, took much more time than the structural steel erection.

Throughout the course of the project, we determined some additional aspects that warrant further research and study. One of the major items for further research is the thermal properties of the steel and concrete structures and their impact on the design and performance of the heating and cooling systems of the building. With many buildings trying to achieve LEED certification, the use of the materials which allow for flexibility of HVAC use is crucial. Additionally, the actual costs of HVAC, Electrical, and other building systems warrant more investigation in an integrated manner instead of approximating those costs as percentages of the structural costs. Last, the hangers and bracing of the sprinkler system can be addressed. There are specific hangers and bracings designed for seismic loading that can be used and should be addressed in further detail in future projects.

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APPENDIX A.1 PROPOSAL – STRUCTURAL

LDA-1303

Worcester Polytechnic Institute: Dormitory Design Proposal

Major Qualifying Project

Submitted by:

Kelsey Forward

Shane Ruddy

Project Advisor:

Professor Leonard Albano

Date Submitted:

October 13, 2012

ABSTRACT

The purpose of this project is to compare and provide alternative steel and concrete structural systems for the new upperclassman and graduate student housing for Worcester Polytechnic Institute. The comparison of the two systems will be based on availability, cost, scheduling, safety and design limitations. Two designs will then be prepared using the material with varying bay sizes and foundations, which will be compliant with *ASCE 7*, *AISC*, *ACI* and *MSBC* provisions. This project will provide cost and performance analyses on both designs and investigate the structural implications of using LEED design specifications. This project will also provide the results of investigating parking conditions, footing designs, architectural layouts and scheduling constraints.

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1.0 Introduction

Worcester Polytechnic Institute has been continuously growing since its beginning in 1865. The student body has ever been increasing, thus causing the campus to expand. The current need on campus is new student housing due to the recent jump in class sizes and the expansion of the graduate programs. WPI currently has nine residential halls accommodating undergraduate students, five of which are specifically for freshmen residence.

Graduate programs, especially in the life sciences and bioengineering, have been growing in the past few years; this is especially true for WPI. WPI has acquired Gateway Park, a twelve acre park in downtown Worcester, with the intention to build-out the park with five buildings. These buildings, expected to be 500,000 total square feet, will feature laboratory, classroom and office space[1].

Massachusetts has launched the "'Growth Districts Initiative' as a focused means of expediting commercial and residential development within the Commonwealth[2]". Through this initiative, the City of Worcester has created a plan for the redevelopment of downtown Worcester. As seen below in the map of the redevelopment (Figure 1), Gateway Park is the first major section of redevelopment focus.

The area where Gateway Park is located was once a brownfield used for Worcester's large industrial economy, making the area a great location for redevelopment. The City knew that this area needed to be repaired and developed. The location of Gateway Park was critical for the City of Worcester because of its prime location along Interstate 290; this site is the first view of the City for visitors from the North[3].

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[1] *Gateway Park at WPI*

[2] *Growth districts initiative, 2012*

[3] *The Phoenix Awards*

FIGURE 2: GATEWAY PARK DETAILED MAP (TAKEN FROM GATEWAY PARK AT WPI)[6]

The site location is due to the expansion of the WPI campus, into Gateway Park. Many WPI faculty members not only have offices that are located in the two current Gateway Park buildings, but also lab space in which seniors and graduate students frequent. This makes the location of the new dormitory ideal.

[6] *Gateway Park Detailed Map*

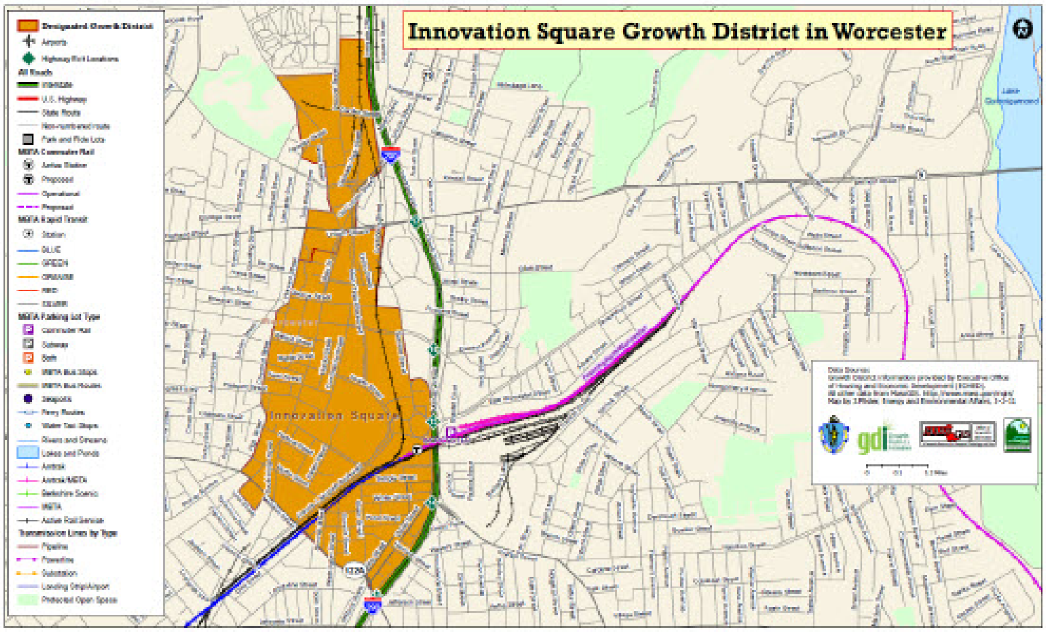

FIGURE 1: MAP OF INNOVATION SQUARE GROWTH DISTRICT IN WORCESTER (TAKEN FROM "GROWTH DISTRICT COMMUNITIES"[4]

The first mixed-use building in Gateway Park was completed in 2007, with the second completed in 2012. The first building, known as Gateway Park 1, "is fully occupied with graduate research laboratories, life science companies, state-of-the-art core facilities, and WPI's Corporate and Professional Education division[5]". Gateway Park 2 houses three of WPI's academic programs, laboratories, office space and classrooms.

The new WPI dormitory, which will house upperclassmen and graduate students, is located in the Gateway Park area. The specific location for the new residence hall in Gateway Park is 10 Faraday Street, at the intersections of Faraday, Grove, and Lancaster Streets as seen in Figure 2 below.

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[4] *Growth district communities, 2012*

[5] *Gateway Park at WPI*

Scope of Work

Constructing a new Gateway building involves numerous organizations, people, and tasks. Due to time constraints, there is no way that every aspect of the construction process could be analyzed, but the project will encompass many of the different design characteristics. The project will focus on the investigation, analysis, and design of a structural steel frame and a precast concrete frame of the new dormitory building located on the corner of Faraday Street and Grove Street. A steel system will be compared to a precast concrete system in order to determine the system best suited for construction of the new dormitory. Different options for the design of the selected structural system will be analyzed, such as different bay sizes. The reason there is a need for a new dormitory on the WPI campus will also be investigated.

The dormitory construction project is a design-build contract between WPI and Daniel O'Connell's Sons (DOC). This report will examine why this type of contract was chosen through interviews with both parties, as well as looking into the different contracts WPI has used in other recent construction endeavors. The Gateway Park project will be examined to understand how this new dormitory fits in with the existing plans.

Design aspects will require the use of different codes, such as the Massachusetts State Building Code and the City of Worcester Zoning Ordinance, to ensure that all the criteria are following proper regulations. The design will also be compliant with ASCE 7, AISC, and ACI requirements. Not only will the steel design be investigated, the foundation will be analyzed and different options will be explored. The Gateway Park area has had issues with soil contamination in the past. We will use geotechnical reports to look into the current situation for the residence hall location. The new dormitory project will obtain LEED certification. Therefore, this concept will be defined and related to the construction process in terms of how it will affect structural design, cost, and layout.

Project management is very important in large scale projects such as this. The project scheduling constraints will be discussed and the cost will be estimated. The cost estimation will be based on information from RS Means and interviews with DOC. Once this work is completed, different options for parking around the new structure will be investigated. All of these activities will create an effective report that displays a proper design for the new residence hall, along with thorough calculations and reasoning behind the major choices in the design concept.

CAPSTONE DESIGN

In order to further address engineering design and the constraints faced in engineering projects, this report will consider specific constraints that are crucial in the design of the new residence hall and all real-world projects. The problem this report faces is how to effectively design a large, residential building in Gateway Park. Architectural layouts and different structural layouts will also be designed. The structural material that will be best for this fast-track construction needs to be determined before the design options can be evaluated for the chosen material. Scheduling and cost need to be analyzed as well to make sure the project will be able to meet required deadlines. In order to make the structure greener and aesthetically appealing, the design of a green roof will be examined. The realistic constraints addressed in this report include: economic, environmental, sustainability, ethical, health and safety, social, and political.

Economic: The financial aspect of construction is a huge factor in the building of the new residence hall and will be addressed continuously throughout the report. The costs for steel construction and precast concrete construction will be investigated during the selection of a construction material and may be an important factor in this process. During the green roof design, evaluating cost will be central to the Return on Investment (ROI) analysis and if a green roof is feasible and worthwhile. This report will analyze the cost of construction, outlining the cost for structural materials, other elements used in the building (electrical work, plumbing, furnishing, etc.), and labor costs. All of these values will allow for a total cost to be determined, which will then be turned into a cost per square foot value. The cost per square foot will be compared to other WPI buildings to get a sense of how expensive or inexpensive it was.

Environmental: Throughout the construction process, environmental aspects create another constraint. The construction site setup will address this by developing plans to mitigate the negative impacts on the environment. An example of this is will be the use of straw bales around the site perimeter to prevent construction runoff into the neighboring areas. The project schedule will display other examples of how the environmental impacts from construction were reduced.

Sustainability: Sustainability plays a huge role in many large construction projects, and the new residence hall is no expectation. Following in the footsteps of the Bartlett Center, East Hall, and the new athletic facility, the new residence hall will be LEED certified. Therefore, this report addresses several of the LEED certification requirements. The sustainability constraint will also be considered during the green roof design of the residence hall.

Ethical: The construction of the new residence hall will use the most up-to-date technology. For scheduling and cost estimating, state of the art technology will be used with Primavera Software. The design loads will be based on the most recent ASCE standards, and the building will be designed to safely withstand all of these loads.

Health and Safety: Ensuring that the building is safe and healthy for its occupants is something that will be addressed. Using the MSBC, the building layout will be designed to meet safety requirements, such as proper hallway and stairwell widths and number of emergency exits. Fireproofing of the steel members and analyzing seismic, wind, and gravity loads based on ASCE 7 design loads will be done to ensure the building will be safe no matter the conditions or elements it faces.

Social: To address the social aspect of this project, the effect of this addition on the WPI community will be investigated. The neighbors directly surrounding the building, including the Courtyard Marriott and homes, will be evaluated to determine what impact the project will have on them. The social constraint will focus more on how this project will affect everyday life for those living in the area and/or attending WPI.

Political: Large construction projects are not always accepted by the neighboring residents and/or communities. The new residence hall will have an impact on the redevelopment of downtown Worcester, and, therefore, will require an evaluation of how it would fit into the City. The new residence hall will bring more students even closer to the downtown area and be only a small part of the redevelopment taking place at Gateway Park. The report will investigate how leaders of the City feel about this project and some of the political steps that are necessary to make construction possible. This includes permits and land acquisition for Gateway Park, which encompasses the specific area of the new residence hall.

2.0 Background

Construction projects involve many different factors, which are all crucial to the project. The following paragraphs describe the important aspects that will be involved in the design and construction of the new residence hall. The design must meet specific regulations set forth by the State of Massachusetts and the City of Worcester before any construction can begin. Footings and foundations must be designed to support the loads of the building, while the structural design must be able to withstand gravity and lateral loads. Investigation into the site conditions for the location of a new building are an important step early on in a project. The resources for cost estimations are described, as well as LEED and its importance to promoting green design. With a better understanding of these elements, a proper design can be developed.

2.1 Layouts

In order to begin the process of structurally designing a building one must first understand the architectural layout of the building. The architectural grid and the occupancies of the spaces must first be defined in order to understand the loading of the structure.

For the Worcester Polytechnic Institute's upperclassmen and graduate dormitory, Daniel O'Connell's Sons (DOC) was hired to design and build the facility. The architect that was subcontracted by DOC is ADD Inc. Figure 3 and Figure 4 below show the architectural layout of the building. The second, third and fourth floor layouts are typical, thus only the second floor layout is shown.

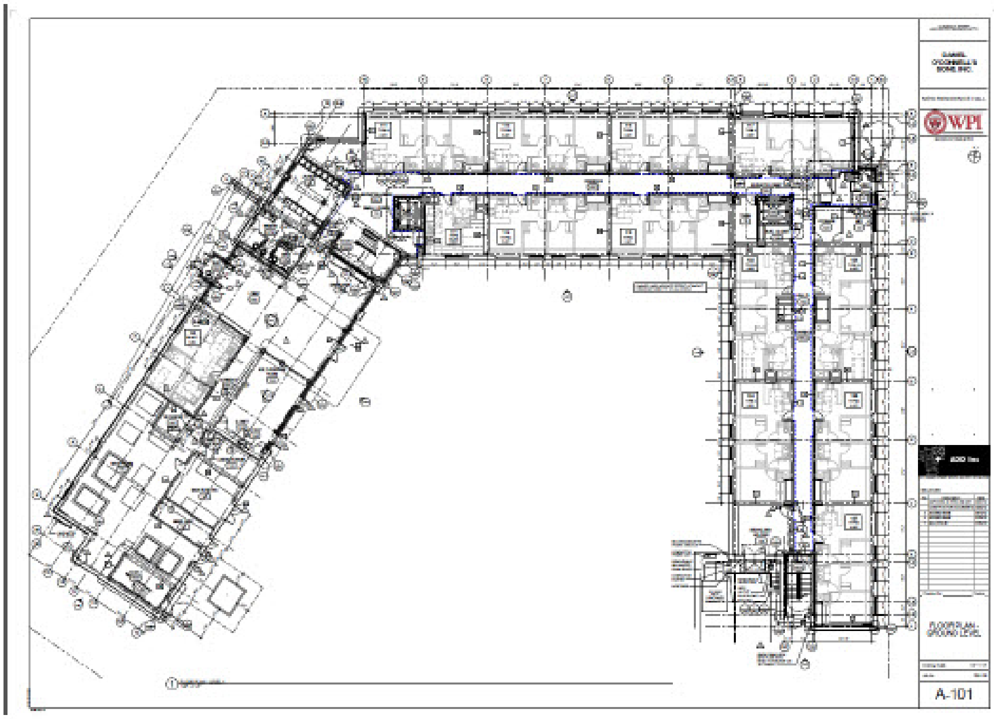

FIGURE 3: ARCHITECTURAL LAYOUT, FIRST FLOOR

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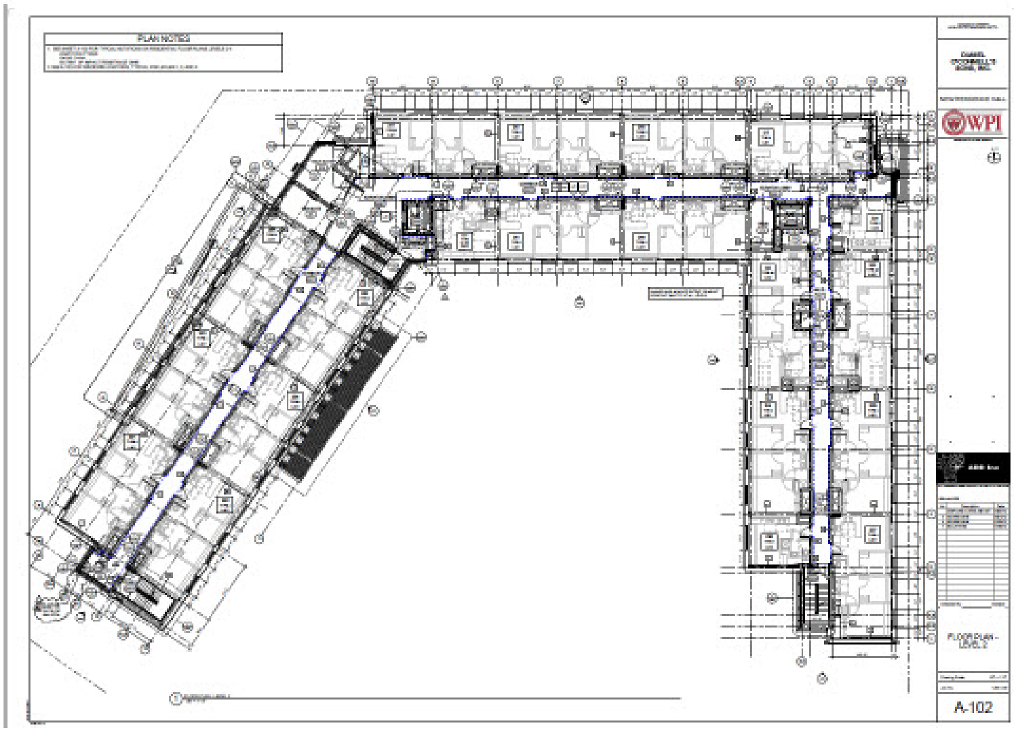

FIGURE 4: ARCHITECTURAL LAYOUT, SECOND FLOOR

Many applications must be considered for laying out the architectural grid. These are building code standards, client needs and functional spaces, esthetics and much more.

2.2 Massachusetts State Building Code

The designs for the new dorm have to follow specific regulations set forth by the state of Massachusetts. Every state has its own set of building codes which dictate certain safety requirements and other construction requirements. The *Massachusetts State Building Code* (MSBC) has been updated throughout the years and is currently in its eighth edition. It references many codes from the *International Building Code* (IBC). The MSBC is divided into 35 sections, each giving thorough descriptions of the regulations for that topic.[7] For the scope of our project, we will only be looking at a portion of these sections, including, but not limited to:

- Chapter 6: Types of Construction
- Chapter 10: Means of Egress
- Chapter 16: Structural Design
- Chapter 18: Soils and Foundations
- Chapter 19: Concrete
- Chapter 22: Steel (State of Massachusetts, 2010)[8]

Following the building codes is crucial to the validity of our structural design because they give the legal requirements for the design loads for different conditions such as wind and earthquakes. Not only do they establish the minimum design loads that must be followed in Massachusetts, but they also give information on other design factors, such as the exit accessibility for buildings, bracing for frames, and deflection requirements. Our work, as well as DOC's plans that have already been created, will be greatly influenced by the provisions of the MSBC.

2.3 ZONING

"Zoning is the process of planning for land use by locality to allocate certain kinds of structures in certain areas.[9]" Zoning also places restrictions on different building aspects, such as the types of businesses that can be in an area, the height of buildings, the density, etc.[10] The City of Worcester Zoning Ordinance describes the regulations for construction within the different zones of Worcester. There are six major use types for the zoning in Worcester: residential, manufacturing, business, institutional, parks, and conservation areas. Use intensity is also used to

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[7] *State of Massachusetts, 2010*

[8] *State of Massachusetts, 2010*

[9] *Murray, 2012*

[10] *Murray, 2012*

subdivide the residential, manufacturing, and business districts.[11] Using Figure 5, it can be determined that the location of the new dorm building, as well as all of Gateway Park, is located within the zoning district classified as BG-6.0.[12] BG 6.0 is a business district and has a maximum floor area ratio (FAR) of 6 square feet / 1 square feet.[13] This means that a building in this district may have a floor area that is six times larger than the land area of its lot. This simply means that this district encompasses many multi-story buildings.

The City of Worcester Zoning Ordinance (COWZ) also contains a table depicting the different permitted uses by zoning districts. This table states the different types of structures that can and cannot be built in the different zones. Table 1 shows a small portion of the whole table, but as you can see by the yellow highlights, dormitories are acceptable in the BG-6.0 district[14].

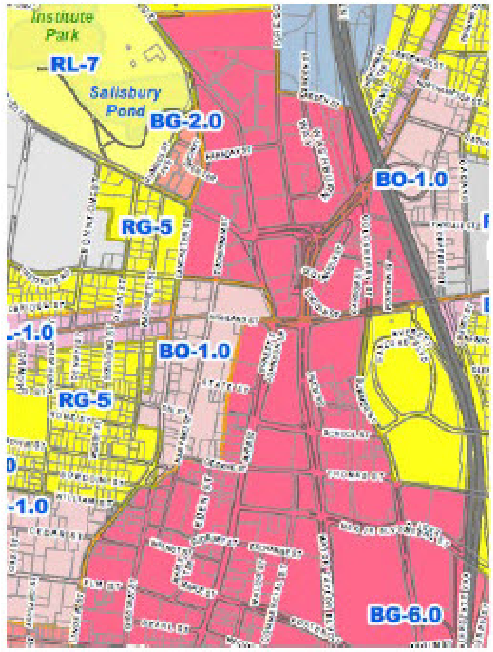

FIGURE 5: WORCESTER ZONING MAP (TAKEN FROM CITY OF WORCESTER (2012B).

In addition to limiting the FAR and users, the zoning ordinance gives specific requirements for building in the BG-6.0 district. For example, there is no specific height regulation, but there is

[11] *City of Worcester, 2012a*

[12] *City of Worcester, 2012b*

[13] *City of Worcester, 2012a*

[14] *City of Worcester, 2012a*

a 10 linear foot rear yard setback[15]. Just as the MSBC influences building design, so does the City's zoning ordinance.

TABLE 1: PERMITTED USES BY ZONING DISTRICTS (TAKEN FROM CITY OF WORCESTER (2012A)

PERMITTED USES BY ZONING DISTRICTS – TABLE 4.1 RESIDENTIAL USE

	RS 10	RS 7	RL 7	RG 5	BO 1	BO 2	BL 1	BG 2	BG 3	BG 4	BG 6	ML 0.5	ML 1	ML 2	MG 0.5	MG 1	MG 2
1. Bed and Breakfast Establishment	SP	SP	SP	SP	SP	SP	SP	N	N	N	N	N	N	N	N	N	N
2. Continuing care retirement community	SP	SP	SP	SP	SP	SP	SP	SP	SP	SP	SP	N	N	N	N	N	N
3. Dormitory	SP	SP	SP	SP	SP	SP	Y	Y	Y	Y	Y	N	N	N	N	N	N
4. Family day care home	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y
5. Fraternity/sorority/ cooperative residence	SP	SP	SP	SP	SP	SP	Y	Y	Y	Y	Y	N	N	N	N	N	N
6. Group residence (general or limited)	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y
7. Limited Residential Hospice House	SP	SP	SP	SP	N	SP	N	N	N	N	N	N	N	N	N	N	N
8. Lodging house	N	N	N	SP	SP	SP	SP	SP	SP	SP	SP	N	N	N	N	N	N

2.4 Design Standards and Specifications

Building designs are based on different codes and regulations. The residence hall design will not only follow the regulations and requirements set forth by the MSBC and the COWZ, it will also rely heavily on guidance from the American Institute of Steel Construction (AISC), the American Concrete Institute (ACI), and American Society of Civil Engineering Standards (ASCE 7). The fourteenth edition of the *Steel Construction Manual* published by the AISC will be used to acquire the different dimensions and properties for various structural products that are found in steel design. The *Steel Construction Manual* provides the specifications that govern the evaluation of limit states for the design of members and connections. It provides geometric data for standard steel sections and design aids to facilitate proper sizing of members and connections. It will be crucial in many different aspects of design, such as determining the minimum and maximum spacing of bolts and checking the various limit states.

ACI 318 provides minimum requirements for the design and construction of concrete structural members. ACI 318, similar to the Steel Construction Manual, will govern the evaluation

[15] *City of Worcester, 2012a*

of limit states for concrete design. ASCE 7 provides the current minimum design loads that can be used when designing a building. Live and dead loads are provided, such as the proper snow, wind, seismic loading requirements.

2.5 Cost Analysis

Estimating the cost of a construction project is a difficult task. Using RS Means is one way to determine accurate cost estimations. RS Means is a construction estimation database that provides cost information to the industry. The information is based on U.S. national averages and is adjusted depending on location[16]. Labor costs, material costs, and equipment costs are all included. All of the information is determined based on many different factors, including the size of the project, season of the year, environmental considerations, quality of the work, etc.[17]

RS Means does have limitations, and in the case that certain cost values specific to the project can't be found, interviews with DOC will be used. By acquiring lump-sum costs for the dormitory project (i.e. electrical work or plumbing) by interviewing DOC members, an accurate total cost can be calculated and all of the needed cost values can be determined. Putting the cost into dollars/square foot is a good way to make comparisons with the square footage costs of other buildings on the Gateway Park site, as well as the rest of the WPI campus.

An important distinction to know is tangible versus intangible benefits. A tangible benefit is one which is relatively straightforward and can be accounted for, such as saving money. An intangible benefit is one that does not affect the bottom line of the person or organization; these benefits can be harder to define such as saving time. For this project, only tangible benefits will be examined.

2.6 Footing and Foundation Design Factors

There are many different footing and foundation types that can be chosen for large scale construction. Foundations and footings are crucial, as it is their job to transmit loads from the structure to the ground. A footing is often the last structural element of the foundation that loads

[16] *Reed Construction, 2012*

[17] *Reed Construction, 2012*

pass through. They have the important function of spreading out the superimposed load.[18] The different factors that must be considered in order to determine the best footing and foundation types include:

- Design load
- Soil bearing capacity
- Subsurface formations and the nature of the soil
- Site conditions
- Climatic conditions
- Economic considerations
- Superstructure type
- Special design requirements [19] [20] [21]

The development of Gateway Park has been a time-consuming process, due partly in fact to the site conditions of the area. An issue with redeveloping this area was overcoming the stigma of the long industrial past of Worcester, which had led to contamination in the selected site.[22] The twelve acre area's contaminants included lead, arsenic, chromium, thallium, nickel zinc, petroleum compounds, polynuclear aromatic hydrocarbons, and several volatile organic compounds.[23] Before construction can begin, the ground must be cleared of these contaminants to make it a safe residential area. Contaminants in the ground is not only time consuming but also costly to remove since the ground must be excavated to depths usually far

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[18] *University of Maryland, 2004*

[19] *Chapter 5: Foundation Design*

[20] *Foundation..., 2012*

[21] *University of Maryland, 2004*

[22] *The Phoenix Awards*

[23] *The Phoenix Awards*

below the foundation depth. These soils must also be removed from the site and cannot be reused for fill on the site.

2.7 Leadership in Energy and Environmental Design (LEED)

LEED certifications can be met in a variety of ways that must be considered from the beginning of the design. LEED credits include sustainability on site, water efficiency, energy and atmosphere, materials and resources, and indoor environmental quality. To obtain a LEED certification, the credentials must be addressed in the site selection, the design of the building and throughout construction. Specific applications of "green building" such as alternative roof designs can affect the structural design of the building. This is why it is critical to define the LEED credits in the beginning of the project.

3.0 METHODOLOGY

The work to be accomplished in this report will be structured into sections of focus. These sections include architectural layouts, structural design, comparison of concrete and steel design, structural systems, lateral loading, typical footing and foundation design, green roof design, scheduling and cost estimation, as seen in Table 2 below.

TABLE 2: METHODOLOGY TOPICS, ACTIVITIES AND RESOURCES

Methodology Topics	Activities	Resources
Layout	Previous architectural layout	DOC
Structural Design (for concrete and steel)	Gravity loads	ASCE 7, DOC
	Lateral loads	ASCE 7
	Typical bays	LRFD, ASCI, MSBC
	A-typical bays	LRFD, ASCI, MSBC
Steel vs. Concrete Design	Construction scheduling	DOC, Research
	Cost	DOC, RS Means
	Material availability	DOC, Research
	Design limitations	Interview DOC, MSBS
	Safety	Research
Chosen Structural System	Long span vs. short span	Interview DOC, Research
	Different material options	Research
	Architectural layouts/building standards	MSBC, Research
Lateral Loads	Effects on structural design	ASCE 7
Footing and Foundation Design	Site conditions	DOC, MSBC, Research
	Loading	Structural Design, ACI
Green Roof Design	Effects on structural design	Research
	Cost	RS Means, Research
Construction Scheduling	Determining tasks/milestones/duration	DOC, Research
Cost Estimation	Material	DOC, RS Means
	Labor	DOC, RS Means
	Total cost/lazy s-curve	Research

3.1 ARCHITECTURAL LAYOUT AND STRUCTURAL DESIGN BASIS

The original layout for the building will be used as a basis for structural design. The proper design values for gravity loading will be obtained from ASCE 7 and from the gravity loads defined on the plan drawings from Daniel O'Connell's Sons. Initial structural designs will be prepared for both steel and precast concrete systems, looking at typical and a-typical bays.

3.2 Steel Design versus Concrete Design

A comparison of steel design versus precast concrete design will be performed. The key components that will be compared will be cost, material availability, time, design limitations and safety factors. The material cost of the two designs will be based on the actual costs of the project and costs determined by RS Means. A comparison of productivity/labor costs will be performed which will also be based on the costs of the bid package from the project (DOC) and from values obtained in RS Means. For steel design, the actual costs of the project will be compared to the RS Means costs to determine an appropriate measure to compare with concrete design. The cost will include material cost and the cost of labor. Procurement times are another important factor for comparison, which will be acquired from RS Means and specific research. The procurement times for each system could become important factors in the decision. Material availability and design limitations will be examined by interviewing key members in DOC. Design limitations can include span sizes, which could also be an important factor in the decision of the structural system. Ductility and additional safety procedures will also be factored in to the decision, such as spray fireproofing with steel structures. These safety factors will be examined for the impacts on cost and lead times for the systems.

3.3 Chosen Structural System

Once the structural system is decided upon, concrete design or steel design, long spans versus short spans will be investigated. The use of different materials will also be investigated, such as, for steel design, rolled W-shapes versus an engineered system. From the investigation of the different materials and the differing span lengths, an alternative design will be created. The architectural layout will most likely need to be to be revised with the new design due to the placement of column lines. An architectural layout will be created within the standards of the *Massachusetts State Building Code* and using the *Graphic Standards Field Guide to Residential Construction* by Dennis J. Hall.

3.4 Lateral Design

Lateral loading will be taken into account once the structural system is selected. The effects of lateral loads and the implications on the design will be examined. Also, rigid frames will be compared to braced frames to determine the better approach for the final design of the building. This comparison will be done by researching the factors of design for both frame systems.

3.5 Footings and Foundation Design

Typical footings will be designed for this project in order to include foundation costs in the decision process. Reinforced concrete footings will be designed in accordance to ACI standards and MSBC requirements. Site conditions will be considered during the design of the footings.

3.6 Green Roof Design

An investigation will be performed on the addition of a green roof on the building. This investigation will include the impacts on the structural design and cost. A Return on Investment (ROI) calculation will be performed that will factor in the tangible benefits of the green roof design, such as money savings from energy savings, and the added material, labor and structural costs of the green roof. Money savings will be determined using average annual energy savings.

3.7 Cost Estimation

Cost estimations will be performed throughout the project, from the beginning when the structural system is being chosen to the concluding ROI calculation for the green roof design. A final cost estimation will be performed which will include the cost from lateral and gravity loading, and labor costs. The costs from lateral and gravity loading will incorporate the costs of material, such as the cost of steel per ton which will be obtained from comparing the bid package that DOC used for the building and RS Means values. Labor cost will be based on the quantities of work for the construction processes, the expected productivity rates for the activities, and the corresponding labor cost rates.

3.8 Scheduling

We will be interviewing and obtaining information from DOC in order to complete a construction schedule. The schedule will be used in order to show the importance of time, due to the fast-track schedule, and how the procurement of materials factor into the structural system selected. We will use Primavera software to create a network diagram to show the relationships of the activities obtained from DOC.

3.9 Environmental

Environmental factors will be considered in this project. This includes the use of straw bales to prevent construction runoff into the neighboring areas. Other applications of prevention will also be examined.

3.10 Political and Social Impacts

We will be conducting interviews with members of DOC, the WPI community, the Marriott Hotel staff, and leaders of the City of Worcester in order to determine the social and political impacts of the new dormitory and Gateway Park on the City. These interviews will also help us to determine if there was any political action taken from WPI in order to acquire the land. The interviews will also be used to determine the social implications of the project, such as the opinion of neighbors and leaders of the City of Worcester.

4.0 DELIVERABLES

Physical deliverables will document and display the results gathered throughout this project. Engineering drawings will be used to illustrate the various design options and load bearing capabilities of the residence hall structural design, while tables will be used to summarize construction characteristics and cost values. The deliverables follow the structure set forth in the methodology as each topic requires an engineering drawing, a table, or a chart to allow the information to be easily understood and depicted. Table 3 below gives an overview of the deliverables that will be produced.

TABLE 3: DELIVERABLES TO BE PRODUCED

Final Deliverables	Notes
Engineering Drawings: Initial Structural Design Including Gravity Loads	For both steel and precast concrete frames
Engineering Drawings: Various Bay Size Designs	For both steel and precast concrete frames
Steel vs. Concrete Comparison Table	Factors for the selection of the building material
Engineering Drawings: Two Steel Structural Building Designs (Different Span Lengths)	One design will be based on DOC's architectural layout
	One will be based on our own architectural layout
Architectural Layouts (including building specifications table)	Outline the various MSBC codes that the layout follows
Engineering Drawings: Lateral Load Design	For the two steel structural designs
Engineering Drawings: Footing & Foundation Design	Based on typical footings
Green Roof Design and ROI	Changes in the schedule, structural design, and cost will be addressed
Revit Model of Completed Design	Dimensions will be displayed
Scheduling: Bar Chart & Network Diagram	Created with Primavera Software
Cost Estimation Table	Structural cost
	Other elements cost
	Labor cost
	Cost per square foot
Final Report	All deliverables will be included

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The first deliverable that will be presented in the report will be engineering drawings of the initial structural design of the building. At this point in the project, the drawings will be based on the architectural layouts created by DOC. No construction material will have been chosen yet, meaning that drawings will be produced for both steel and precast concrete fabrication. Before a material is selected for construction, different bay sizes (typical and atypical) will also be evaluated for both steel and precast. The steel versus concrete comparison table listed in Table 3 will display the pros and cons of the two materials and explain how we came to the decision of using steel for the construction of the building. The table will include the five factors stated in Table 2 in the Methodology: construction scheduling, cost, material availability, design limitations, and safety. The selection of a construction material is a significant aspect of this report, therefore, the comparison table will be very important to use as a final deliverable.

Once a construction material is chosen, the following designs will focus on a structural frame of that material. Two different options for this frame will be created. One drawing will be based on the architectural layout already created by DOC, meaning that all of the rooms, hallways, stairwells, etc. will remain the same size and the structural frame will fit within this layout. For the second engineering drawing, different span lengths will be used and, therefore, the original architectural layout (room sizes, hallway widths and lengths, storage space, etc.) will be altered. This design will be used to see if a stronger, more cost effective structural system can be developed. With the layout changing, a new floor plan will have to be created to fit within the structural frame. This will be necessary to avoid rooms with columns running through them or alterations in stairway locations. Another deliverable, consisting of new architectural layouts, will, therefore, be required. Floor layouts will be created for the first and second floors of the building under the assumption that all floors above the first floor will follow the same layout. The dimensions of the offices, bathrooms, hallways, stairwells, and other rooms will be depicted on the architectural layouts. In order to ensure that the architectural layout created is feasible and acceptable, an understanding of the MSBC will be required. A table will be used in the final deliverable displaying the main MSBC requirements that were met in the architectural layout design. These will include specifications such as: hallway width, number of emergency exits, width of stairwells, etc.

Lateral loads will have to be analyzed for each of the two structural frames, looking into seismic and wind loads. With these deliverables finished, the remaining products for this project will be additional design and management features. Engineering drawings for the footing and foundation designs will be developed based on typical footing designs that will be widely used throughout the layout. These typical footing designs will provide a base for considering foundations and their costs.

The green roof design listed in Table 3 will outline the impacts that a green roof will have on the residence hall design. Due to the changes in loading with the addition of a green roof, a new engineering drawing will have to be created to capture the design changes necessary to encompass the larger gravity loads. Adding a green roof will also lead to a change in the schedule and cost which will be addressed in the final outline of the green roof design. With this deliverable, an ROI will also be developed focusing on tangible aspects, such as additional base cost, additional structural cost, and energy savings, of the green roof design.

In order to display all of the information calculated in an aesthetic way, a Revit model will be created. The model will display dimensions and will be based on one of the floor layouts developed for the selected construction material. A model will allow the readers to visualize all of the information that has been described.

With all of the design factors completed, the final deliverables will focus on scheduling and project cost. A bar chart displaying the activities, their duration, start and finish dates, and total float will be developed using Primavera Software. A network diagram will also be developed using Primavera. The diagram will show the relationships between activities including the critical path for the project. Cost estimation will be the final deliverable for the project. A table will be used to display the cost of the structural elements of the project, such as the steel members and concrete footings. The table will also display the cost of other elements in the building, including electrical work and plumbing. Labor costs will be included and allow us to get a total cost of the project which will be calculated into a cost per square foot. This final value will be compared to the cost per square foot of other buildings built at WPI. The residence hall will have been fully evaluated at this point, allowing the reader to get a thorough understanding of the structural design, as well as the construction process.

All of the aforementioned deliverables will be placed into one final report. This final report will detail the process followed to create the deliverables, as well as relevant background information on the different topics discussed. The final report will provide an overall description and illustration of the building and its various design features and impact on the WPI community.

5.0 CONCLUSIONS

This project will produce a variety of deliverables and rely on many different aspects of civil engineering, from structural engineering to construction project management to environmental engineering. The expectations for this project are that a safe and cost effective design will be created that can be completed in the short amount of time required. It is expected that different floor plans will be feasible, and possibly better, than the plans already set forth by DOC . The results from the examination of the residence hall will allow the reader to gain a better understanding of the various loads that the building must be designed to withstand.

The construction schedule and cost estimation are expected to demonstrate important management aspects of the project and illustrate the relatively quick construction time that is expected, in contrast to other delivery methods. The final result and deliverables will give a full overview of the design of the building, from the footings to the original floor plans to the adjusted floor plans to the steel design to the green roof design. The final report will allow the reader to gain valuable insight and knowledge on the design, schedule, and cost of the new WPI residence hall being constructed at 10 Faraday Street.

6.0 Schedule

Schedule of Anticipated Completion Dates																						
Milestones	A Term		Break	B Term								Break			C Term							
Week Ending	6-Oct	13-Oct	20-Oct	27-Oct	3-Nov	10-Nov	17-Nov	24-Nov	1-Dec	8-Dec	15-Dec	22-Dec	29-Dec	5-Jan	12-Jan	19-Jan	26-Jan	2-Feb	9-Feb	16-Feb	23-Feb	2-Mar
Structual Design Loads																						
Steel Design																						
Concrete Design																						
Comparison of both designs																						
Long Span Design																						
Short Span Design																						
Lateral Loading																						
DRAFT																						
Footing and Foundation Design																						
Green Roof Design																						
Construction Scheduling																						
Cost Estimation																						
DRAFT																						
FINAL DRAFT																						

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Appendix A.2 Proposal – Fire Protection

Worcester Polytechnic Institute:

Dormitory Life Safety Design Proposal

Major Qualifying Project

Submitted by:

Corey Fisher

Project Advisor:

Professor Leonard Albano

Date Submitted:

December 15,2012

ABSTRACT

The purpose of this project is to compare and provide alternative steel and concrete structural systems for the new upperclassman and graduate student housing for Worcester Polytechnic Institute. The comparison of the two systems will be based on availability, cost, scheduling, safety and design limitations. Two designs will then be prepared using the material with varying bay sizes and foundations, which will be compliant with *ASCE 7*, *AISC*, *ACI* and *MSBC* provisions. This project will provide cost and performance analyses on both designs and investigate the structural implications of using LEED design specifications. This project will also provide the results of investigating parking conditions, footing designs, architectural layouts and scheduling constraints. In addition, a sprinkler system in accordance with NFPA 13 requirements will be designed. The sprinkler system will be designed using different materials and a comparison will be made in order to choose the most cost effective and safe design for the property. In the end, a code compliance discussion and review of the different life safety aspects of the building, for example sprinkler systems, standpipe systems, means of egress, and interior finish, amongst others, will be initiated to demonstrate the many aspects of the building which are regulated and how the building meets those regulations.

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1.0 Introduction

This project is being completed in cooperation with Shane Ruddy and Kelsey Forward therefore the introduction to this proposal is adopted from MQP proposal LDA-1303. The revisions and additions to their report will be noted by underlined text.

Worcester Polytechnic Institute has been continuously growing since its beginning in 1865. The student body has ever been increasing, thus causing the campus to expand. The current need on campus is new student housing due to the recent jump in class sizes and the expansion of the graduate programs. WPI currently has nine residential halls accommodating undergraduate students, five of which are specifically for freshmen residence.

Graduate programs, especially in the life sciences and bioengineering, have been growing in the past few years; this is especially true for WPI. WPI has acquired Gateway Park, a twelve acre park in downtown Worcester, with the intention to build-out the park with five buildings. These buildings, expected to be 500,000 total square feet, will feature laboratory, classroom and office space[1].

Massachusetts has launched the "'Growth Districts Initiative' as a focused means of expediting commercial and residential development within the Commonwealth[2]". Through this initiative, the City of Worcester has created a plan for the redevelopment of downtown Worcester. As seen below in the map of the redevelopment (Figure 1), Gateway Park is the first major section of redevelopment focus.

The area where Gateway Park is located was once a brownfield used for Worcester's large industrial economy, making the area a great location for redevelopment. The City knew that this area needed to be repaired and developed. The location of Gateway Park was critical for the City of Worcester because of its prime location along Interstate 290; this site is the first view of the City for visitors from the North[3].

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[1] *Gateway Park at WPI*

[2] *Growth districts initiative, 2012*

[3] *The Phoenix Awards*

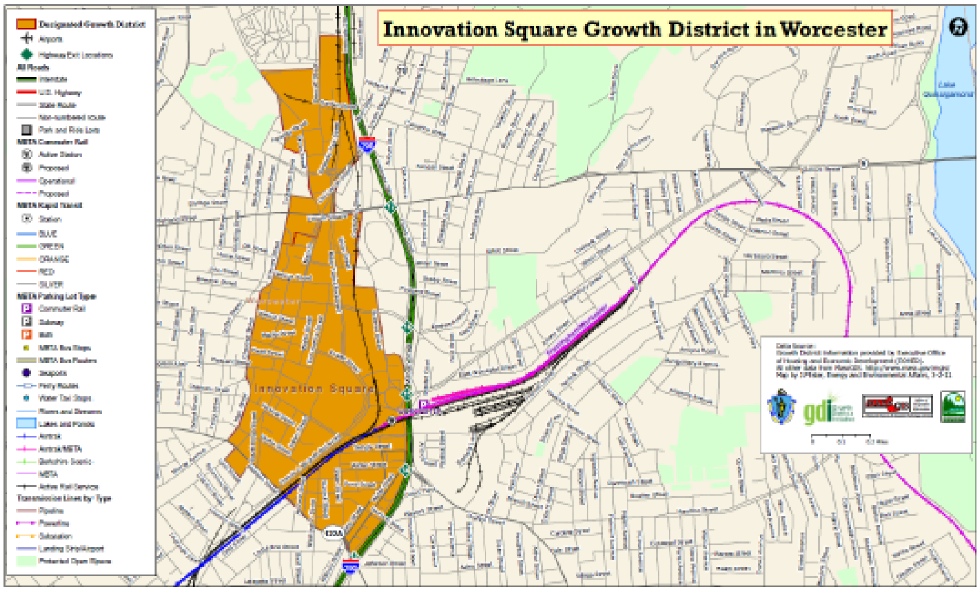

FIGURE 1: MAP OF INNOVATION SQUARE GROWTH DISTRICT IN WORCESTER (TAKEN FROM "GROWTH DISTRICT COMMUNITIES" [4]

The first mixed-use building in Gateway Park was completed in 2007, with the second completed in 2012. The first building, known as Gateway Park 1, "is fully occupied with graduate research laboratories, life science companies, state-of-the-art core facilities, and WPI's Corporate and Professional Education division[5]". Gateway Park 2 houses three of WPI's academic programs, laboratories, office space and classrooms.

The new WPI dormitory, which will house upperclassmen and graduate students, is located in the Gateway Park area. The specific location for the new residence hall in Gateway Park is 10 Faraday Street, at the intersections of Faraday, Grove, and Lancaster Streets as seen in Figure 2 below.

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[4] *Growth district communities, 2012*

[5] *Gateway Park at WPI*

FIGURE 2: GATEWAY PARK DETAILED MAP (TAKEN FROM GATEWAY PARK AT WPI)[6]

The site location is due to the expansion of the WPI campus, into Gateway Park. Many WPI faculty members not only have offices that are located in the two current Gateway Park buildings, but also lab space in which seniors and graduate students frequent. This makes the location of the new dormitory ideal.

[6] *Gateway Park Detailed Map*

Scope of Work

Constructing a new Gateway building involves numerous organizations, people, and tasks. Due to time constraints, there is no way that every aspect of the construction process could be analyzed, but the project will encompass many of the different design characteristics. The project will focus on the investigation, analysis, and design of a structural steel frame and a precast concrete frame of the new dormitory building located on the corner of Faraday Street and Grove Street. A steel system will be compared to a precast concrete system in order to determine the system best suited for construction of the new dormitory. Different options for the design of the selected structural system will be analyzed, such as different bay sizes. The reason there is a need for a new dormitory on the WPI campus will also be investigated. In addition to the structural design, the design of certain life safety systems will be included within the project. Primarily, an automatic sprinkler system will be investigated for the property. Different options for the design will be investigated such as different piping materials and sprinkler heads and those different materials will be used to provide an cost analysis of those materials and determining which materials is best suited for this job.

The dormitory construction project is a design-build contract between WPI and Daniel O'Connell's Sons (DOC). This report will examine why this type of contract was chosen through interviews with both parties, as well as looking into the different contracts WPI has used in other recent construction endeavors. The Gateway Park project will be examined to understand how this new dormitory fits in with the existing plans.

Design aspects will require the use of different codes, such as the Massachusetts State Building Code and the City of Worcester Zoning Ordinance, to ensure that all the criteria are following proper regulations. The design will also be compliant with ASCE 7, AISC, and ACI requirements. Not only will the steel design be investigated, the foundation will be analyzed and different options will be explored. The Gateway Park area has had issues with soil contamination in the past. We will use geotechnical reports to look into the current situation for the residence hall location. The new dormitory project will obtain LEED certification. Therefore, this concept will be defined and related to the construction process in terms of how it will affect structural design, cost, and layout. In addition to the structural codes which need to be

Throughout the design process for the life safety systems, certain codes and standards which are applicable in the state of Massachusetts will need to be referenced such as the state building and fire codes. The different sections of the codes which are applicable, such as means of egress, interior finish, sprinkler systems, and standpipe systems, amongst others, will be discussed for their compliance and for the reasoning as to why a building of this nature requires those aspects of construction and life safety.

Capstone Design

In order to further address engineering design and the constraints faced in engineering projects, this report will consider specific constraints that are crucial in the design of the new residence hall and all real-world projects. The problem this report faces is how to effectively design a large, residential building in Gateway Park. Architectural layouts and different structural layouts will also be designed. The structural material that will be best for this fast-track construction needs to be determined before the design options can be evaluated for the chosen material. Scheduling and cost need to be analyzed as well to make sure the project will be able to meet required deadlines. In order to make the structure greener and aesthetically appealing, the design of a green roof will be examined. The realistic constraints addressed in this report include: economic, environmental, sustainability, ethical, health and safety, social, and political.

Economic: The financial aspect of construction is a huge factor in the building of the new residence hall and will be addressed continuously throughout the report. The costs for steel construction and precast concrete construction will be investigated during the selection of a construction material and may be an important factor in this process. During the green roof design, evaluating cost will be central to the Return on Investment (ROI) analysis and if a green roof is feasible and worthwhile. This report will analyze the cost of construction, outlining the cost for structural materials, other elements used in the building (electrical work, plumbing, furnishing, etc.), and labor costs. All of these values will allow for a total cost to be determined, which will then be turned into a cost per square foot value. The cost per square foot will be compared to other WPI buildings to get a sense of how expensive or inexpensive it was. Additionally, the costs of specific materials for the life safety systems will be investigated such as piping material and sprinkler heads. The investigation will determine whether or not different and lower cost materials will be able to provide the same level of safety compared to what is typical for a building of this type.

Health and Safety: Ensuring that the building is safe and healthy for its occupants is something that will be addressed. Using the MSBC, the building layout will be designed to meet safety requirements, such as proper hallway and stairwell widths and number of emergency exits. Other life safety requirements, such as smoke control, the use of elevators for evacuation, and areas of refuge, amongst a few other topics of the MSBC, will be addressed and will be determined if necessary and to what degree they need to be implemented for the new residence hall. Fireproofing of the steel members and analyzing seismic, wind, and gravity loads based on ASCE 7 design loads will be done to ensure the building will be safe no matter the conditions or elements it faces.

2.0 Background

Construction projects involve many different factors, which are all crucial to the project. The following paragraphs describe the important aspects that will be involved in the design and construction of the new residence hall. The design must meet specific regulations set forth by the State of Massachusetts and the City of Worcester before any construction can begin. Footings and foundations must be designed to support the loads of the building, while the structural design must be able to withstand gravity and lateral loads. Investigation into the site conditions for the location of a new building are an important step early on in a project. The resources for cost estimations are described, as well as LEED and its importance to promoting green design. With a better understanding of these elements, a proper design can be developed.

2.1 Layouts

In order to begin the process of structurally designing a building one must first understand the architectural layout of the building. The architectural grid and the occupancies of the spaces must first be defined in order to understand the loading of the structure.

For the Worcester Polytechnic Institute's upperclassmen and graduate dormitory, Daniel O'Connell's Sons (DOC) was hired to design and build the facility. The architect that was subcontracted by DOC is ADD Inc. Figure 3 and Figure 4 below show the architectural layout of the building. The second, third and fourth floor layouts are typical, thus only the second floor layout is shown.

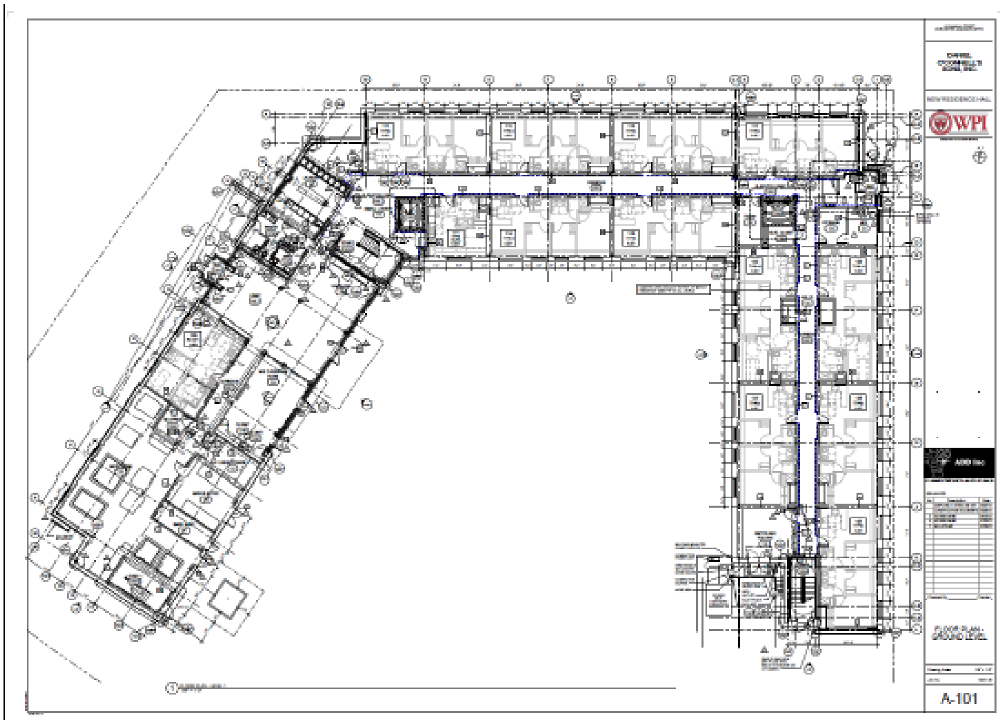

FIGURE 3: ARCHITECTURAL LAYOUT, FIRST FLOOR

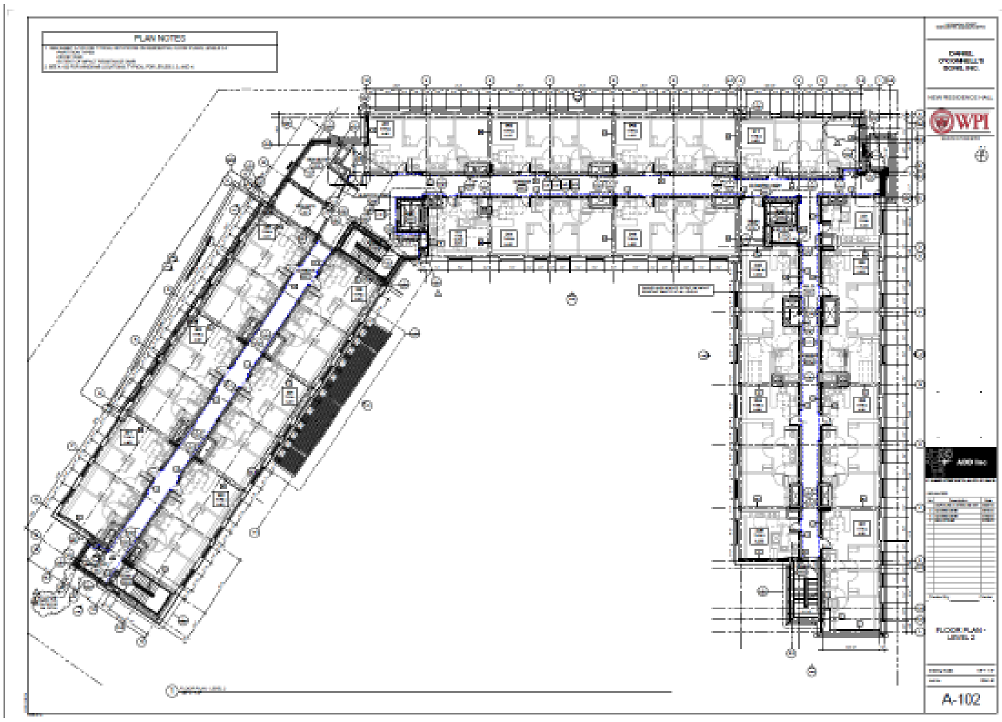

FIGURE 4: ARCHITECTURAL LAYOUT, SECOND FLOOR

Many applications must be considered for laying out the architectural grid. These are building code standards, client needs and functional spaces, esthetics and much more.

2.2 Massachusetts State Building Code

The designs for the new dorm have to follow specific regulations set forth by the state of Massachusetts. Every state has its own set of building codes which dictate certain safety requirements and other construction requirements. The *Massachusetts State Building Code* (MSBC) has been updated throughout the years and is currently in its eighth edition. The MSBC adopts the International Building Code in its entirety and then makes certain additions and deletions for what the lawmakers see fit as best for construction in the State. The MSBC is divided into 35 sections, each giving thorough descriptions of the regulations for that topic.[7] For the scope of our project, we will only be looking at a portion of these sections, including, but not limited to:

- Chapter 6: Types of Construction
- Chapter 7: Fire and Smoke Protection Features

[7] *State of Massachusetts, 2010*

- Chapter 8: Interior Finishes
- Chapter 9: Fire Protection Systems
- Chapter 10: Means of Egress
- Chapter 16: Structural Design
- Chapter 18: Soils and Foundations
- Chapter 19: Concrete
- Chapter 22: Steel (State of Massachusetts, 2010)[8]
- Chapter 29: Plumbing Systems

Following the building codes is essential to the validity of the structural design and the design of the life safety aspects of the structure. These codes are because they give the legal requirements for the design loads for different conditions such as wind and earthquakes. Not only do they establish structural design criteria such as the minimum design loads that must be followed in Massachusetts, but they also give information on other design factors, such as the exit accessibility for buildings, bracing for frames, and deflection requirements. The codes references documents which will outline the required sprinkler types, pressures and flows, and design areas for sprinkler systems. The codes also outline aspects such as the width and number of means of egress and the types of interior finishes which are allowed on the walls, ceilings and floors of buildings of this nature. Our work, as well as DOC's plans that have already been created, will be greatly influenced by the provisions of the MSBC.

2.3 Design Standards and Specifications

Building designs are based on different codes and regulations. The residence hall design will not only follow the regulations and requirements set forth by the MSBC and the COWZ, it will also rely heavily on guidance from the American Institute of Steel Construction (AISC), the American Concrete Institute (ACI), and American Society of Civil Engineering Standards (ASCE 7). The fourteenth edition of the *Steel Construction Manual* published by the AISC will be used to acquire the different dimensions and properties for various structural products that are found in steel design. The *Steel Construction Manual* provides the specifications that govern the evaluation of limit states for the design of members and connections. It provides geometric data for standard steel sections and design aids to facilitate proper sizing of members and connections. It will be crucial in many different aspects of design, such as determining the minimum and maximum spacing of bolts and checking the various limit states.

[8] *State of Massachusetts, 2010*

ACI 318 provides minimum requirements for the design and construction of concrete structural members. ACI 318, similar to the Steel Construction Manual, will govern the evaluation of limit states for concrete design. ASCE 7 provides the current minimum design loads that can be used when designing a building. Live and dead loads are provided, such as the proper snow, wind, seismic loading requirements.

In addition to the building and structural codes, life safety codes will need to be referenced. Primarily, codes and standards written by the National Fire Protection Association (NFPA). The primary codes which will be referenced are *NFPA 13, NFPA 14, and NFPA 20* which are the standards for the installation of sprinkler systems, the installation of standpipes and hose systems, and the installation of stationary pumps for fire protection. These standards give requirements for sprinklers, standpipes and pipes in relation to sizing, materials, and application, amongst others.

2.4 Footing and Foundation Design Factors

There are many different footing and foundation types that can be chosen for large scale construction. Foundations and footings are crucial, as it is their job to transmit loads from the structure to the ground. A footing is often the last structural element of the foundation that loads pass through. They have the important function of spreading out the superimposed load.[9] The different factors that must be considered in order to determine the best footing and foundation types include:

- Design load
- Soil bearing capacity
- Subsurface formations and the nature of the soil
- Site conditions
- Climatic conditions
- Economic considerations
- Superstructure type
- Special design requirements [10] [11] [12]

[9] *University of Maryland, 2004*

[10] *Chapter 5: Foundation Design*

[11] *Foundation…, 2012*

[12] *University of Maryland, 2004*

The development of Gateway Park has been a time-consuming process, due partly in fact to the site conditions of the area. An issue with redeveloping this area was overcoming the stigma of the long industrial past of Worcester, which had led to contamination in the selected site.[13] The twelve acre area's contaminants included lead, arsenic, chromium, thallium, nickel zinc, petroleum compounds, polynuclear aromatic hydrocarbons, and several volatile organic compounds.[14] Before construction can begin, the ground must be cleared of these contaminants to make it a safe residential area. Contaminants in the ground is not only time consuming but also costly to remove since the ground must be excavated to depths usually far below the foundation depth. These soils must also be removed from the site and cannot be reused for fill on the site.

3.0 Methodology

The work to be accomplished in this report will be structured into sections of focus. These sections include architectural layouts, structural design, comparison of concrete and steel design, structural systems, lateral loading, typical footing and foundation design, green roof design, scheduling, cost estimation, sprinkler system design and building code analysis, as seen in Table 2 below.

TABLE 1: METHODOLOGY TOPICS, ACTIVITIES AND RESOURCES

Methodology Topics	Activities	Resources
Layout	Previous architectural layout	DOC
Structural Design (for concrete and steel)	Gravity loads	ASCE 7, DOC
	Lateral loads	ASCE 7
	Typical bays	LRFD
	A-typical bays	LRFD
Steel vs. Concrete Design	Construction scheduling	DOC, Research
	Cost	DOC, RS Means
	Material availability	DOC, Research
	Design limitations	DOC, Research
	Safety	Research
Chosen Structural System	Long span vs. short span	DOC, Research
	Different material options	Research
	Architectural layouts/building standards	MSBC

[13] *The Phoenix Awards*

[14] *The Phoenix Awards*

Footing and Foundation Design	Site conditions	DOC, Research
	Loading	Structural Design, ACI
Green Roof Design	Effects on structural design	Research
	Cost	RS Means, Research
Scheduling	Determining tasks/milestones/duration	DOC, Research
Cost Estimation	Material	DOC, RS Means
	Labor	DOC, RS Means
	Total cost/lazy s-curve	Research
Fire Safety Investigation	Sprinkler System Design	DOC, NFPA 13
	Code Analysis	MSBC

3.1 Architectural Layout and Structural Design Basis

The original layout for the building will be used as a basis for structural design. The proper design values for gravity loading will be obtained from ASCE 7 and from the gravity loads defined on the plan drawings from Daniel O'Connell's Sons. Initial structural designs will be prepared for both steel and precast concrete systems, looking at typical and a-typical bays.

3.2 Steel Design versus Concrete Design

A comparison of steel design versus precast concrete design will be performed. The key components that will be compared will be cost, material availability, time, design limitations and safety factors. The material cost of the two designs will be based on the actual costs of the project and costs determined by RS Means. A comparison of productivity/labor costs will be performed which will also be based on the costs of the bid package from the project (DOC) and from values obtained in RS Means. For steel design, the actual costs of the project will be compared to the RS Means costs to determine an appropriate measure to compare with concrete design. The cost will include material cost and the cost of labor. Procurement times are another important factor for comparison, which will be acquired from RS Means and specific research. The procurement times for each system could become important factors in the decision. Material availability and design limitations will be examined by interviewing key members in DOC. Design limitations can include span sizes, which could also be an important factor in the decision of the structural system. Ductility and additional safety procedures will also be factored in to the decision, such as spray fireproofing with steel structures. These safety factors will be examined for the impacts on cost and lead times for the systems.

3.3 Chosen Structural System

Once the structural system is decided upon, concrete design or steel design, long spans versus short spans will be investigated. The use of different materials will also be investigated, such as, for steel design, rolled W-shapes versus an engineered system. From the investigation of the different materials and the differing span lengths, an alternative design will be created. The architectural layout will most likely need to be to be revised with the new design due to the placement of column lines. An architectural layout will be created within the standards of the *Massachusetts State Building Code* and using the *Graphic Standards Field Guide to Residential Construction* by Dennis J. Hall.

3.4 Lateral Design

Lateral loading will be taken into account once the structural system is selected. The effects of lateral loads and the implications on the design will be examined. Also, rigid frames will be compared to braced frames to determine the better approach for the final design of the building. This comparison will be done by researching the factors of design for both frame systems.

3.5 Footings and Foundation Design

Typical footings will be designed for this project in order to include foundation costs in the decision process. Reinforced concrete footings will be designed in accordance to ACI standards and MSBC requirements. Site conditions will be considered during the design of the footings.

3.6 Sprinkler System Design

A typical sprinkler system per NFPA 13 requirements will be designed for the residence hall based on occupancy hazards and commodity classifications. *NFPA 13, Standard for Installation of Standpipes and Hose Systems, and NFPA 20, Standard for Installation of Stationary Pumps for Fire Protection* will be referenced if those aspects of a sprinkler system are required by either NFPA 13 or the MSBC. Through this design, different materials will be chosen and compared the process to determine which materials will provide the designer with the most flexibility when it comes to cost and constructability. The materials to be included in this comparison may include different piping materials, sprinkler sizes, and pump sizes.

3.7 Code Review

The drawings and layouts provided by DOC, labeled as FP-xxx or A-xxx, will be evaluated for their compliance with the MSBC and other applicable codes. Not all of the architectural drawings will be discussed but many of the drawings give information related to door and stair widths which is important when evaluating the means of egress. Once the information from these drawings is examined, which provisions of the MSBC are applicable to this project must be considered. A discussion of the application of the code to the available information will follow. If there are aspects of the building that appear to not comply with the intent of the code, an identification of what additional information is required, such as the use of a performance based design, will be discussed to give a better understanding of what work was completed.

4.0 DELIVERABLES

Physical deliverables will document and display the results gathered throughout this project. Engineering drawings will be used to illustrate the various design options and load bearing capabilities of the residence hall structural design, while tables will be used to summarize construction characteristics and cost values. The deliverables follow the structure set forth in the methodology as each topic requires an engineering drawing, a table, or a chart to allow the information to be easily understood and depicted. Table 3 below gives an overview of the deliverables that will be produced.

TABLE 2: DELIVERABLES TO BE PRODUCED

Final Deliverables	Notes
Engineering Drawings: Initial Structural Design Including Gravity Loads	For both steel and precast concrete frames
Engineering Drawings: Various Bay Size Designs	For both steel and precast concrete frames
Steel vs. Concrete Comparison Table	Factors for the selection of the building material
Engineering Drawings: Two Steel Structural Building Designs (Different Span Lengths)	One design will be based on DOC's architectural layout
	One will be based on our own architectural layout
Architectural Layouts (including building specifications table)	Outline the various MSBC codes that the layout follows
Engineering Drawings: Lateral Load Design	For the two steel structural designs
Engineering Drawings: Footing & Foundation Design	Based on typical footings

Green Roof Design and ROI	Changes in the schedule, structural design, and cost will be addressed
Revit Model of Completed Design	Dimensions will be displayed
Scheduling: Bar Chart & Network Diagram	Created with Primavera Software
Cost Estimation Table	Structural cost
	Other elements cost
	Labor cost
	Cost per square foot
Sprinkler Design/Layout	Cost and performance comparisons of different materials which could possibly be used in the design
Code Analysis	Application of MSBC provisions to the DOC design documents while integrating a flow chart of the procedure the analysis was completed
Final Report	All deliverables will be included

The first deliverable that will be presented in the report will be engineering drawings of the initial structural design of the building. At this point in the project, the drawings will be based on the architectural layouts created by DOC. No construction material will have been chosen yet, meaning that drawings will be produced for both steel and precast concrete fabrication. Before a material is selected for construction, different bay sizes (typical and atypical) will also be evaluated for both steel and precast. The steel versus concrete comparison table listed in Table 3 will display the pros and cons of the two materials and explain how we came to the decision of using steel for the construction of the building. The table will include the five factors stated in Table 2 in the Methodology: construction scheduling, cost, material availability, design limitations, and safety. The selection of a construction material is a significant aspect of this report, therefore, the comparison table will be very important to use as a final deliverable.

Once a construction material is chosen, the following designs will focus on a structural frame of that material. Two different options for this frame will be created. One drawing will be based on the architectural layout already created by DOC, meaning that all of the rooms, hallways, stairwells, etc. will remain the same size and the structural frame will fit within this layout. For the second engineering drawing, different span lengths will be used and, therefore, the original architectural layout (room sizes, hallway widths and lengths, storage space, etc.) will be altered. This design will be used to see if a stronger, more cost effective structural system can be developed. With the layout changing, a new floor plan will have to be created to fit within the structural frame. This will be necessary to avoid rooms with columns running through them or alterations in stairway locations. Another deliverable, consisting of new architectural layouts, will, therefore, be required. Floor layouts will be created for the first and second floors of the building under the assumption that all floors above the first floor will follow the same layout. The dimensions of the offices, bathrooms, hallways, stairwells, and other rooms will be depicted on the architectural layouts. In order to ensure that the architectural layout created is feasible and acceptable, an understanding of the MSBC will be required. A table will be used in the final deliverable displaying the main MSBC requirements that were met in the architectural layout design. These will include specifications such as: hallway width, number of emergency exits, width of stairwells, etc.

Lateral loads will have to be analyzed for each of the two structural frames, looking into seismic and wind loads. With these deliverables finished, the remaining products for this project will be additional design and management features. Engineering drawings for the footing and foundation designs will be developed based on typical footing designs that will be widely used throughout the layout. These typical footing designs will provide a base for considering foundations and their costs.

With the structural frame completed, the sprinkler system will be able to be designed and laid out. The placement of the sprinklers will be in relation to the structural members. An alternative sprinkler system will be designed in order to facilitate a comparison between materials, cost, and performance of the system. The comparison will contain different piping layouts and sizing and the use of different sprinkler heads. Engineering drawings of the layout will be included in order for the reader to get a visual of how the system will look once installed.

Additionally, the DOC design will be examined in the context of the provisions of the MSBC. This examination will focus primarily on the aspects related to fire protection and life safety, such as means of egress and fire protection systems. The examination will allow the reader to understand the aspects of the life safety design that are governed by the MSBC and how those provisions effect the design.

In order to display all of the information calculated in an aesthetic way, a Revit model will be created. The model will display dimensions and will be based on one of the floor layouts developed for the selected construction material. A model will allow the readers to visualize all of the information that has been described.

All of the aforementioned deliverables will be placed into one final report. This final report will detail the process followed to create the deliverables, as well as relevant background information on the different topics discussed. The final report will provide an overall description and illustration of the building and its various design features and impact on the WPI community.

5.0 Conclusions

This project will produce a variety of deliverables and rely on many different aspects of civil engineering, from structural engineering to construction project management to environmental engineering to fire and life safety. The expectations for this project are that a safe and cost effective design will be created that can be completed in the short amount of time required. It is expected that different floor plans will be feasible, and possibly better, than the plans already set forth by DOC . The results from the examination of the residence hall will allow the reader to gain a better understanding of the various loads that the building must be designed to withstand. Additionally, the deliverables will provide the reader with an understanding of the important life safety systems, such as a sprinkler system, will be incorporated within the building and those systems interaction with each other as well as the interactions with the other features of the building.

The construction schedule and cost estimation are expected to demonstrate important management aspects of the project and illustrate the relatively quick construction time that is expected, in contrast to other delivery methods. The final result and deliverables will give a full overview of the design of the building, from the footings to the original floor plans to the adjusted floor plans to the steel design to the green roof design. Within the adjusted floor plans will include the depiction of a sprinkler system to protect the building against possible fires. The final report will allow the reader to gain valuable insight and knowledge on the design and the reasoning behinds these designs from a code aspect, schedule, and cost of the new WPI residence hall being constructed at 10 Faraday Street.

6.0 SCHEDULE

Schedule of Anticipated Completion Dates																						
Milestones	A Term		Break	B Term								Break			C Term							
Week Ending	6-Oct	13-Oct	20-Oct	27-Oct	3-Nov	10-Nov	17-Nov	24-Nov	1-Dec	8-Dec	15-Dec	22-Dec	29-Dec	5-Jan	12-Jan	19-Jan	26-Jan	2-Feb	9-Feb	16-Feb	23-Feb	2-Mar
Structual Design Loads																						
Steel Design																						
Concrete Design																						
Comparison of both designs																						
~~Long Span Design~~																						
~~Short Span Design~~																						
Lateral Loading																						
DRAFT																						
Corey's Proposal																						
Footing and Foundation Design																						
Green Roof Design																						
Sprinkler System Design																						
Code Compliance Discussion																						
Construction Scheduling																						
Cost Estimation																						
DRAFT																						
FINAL DRAFT																						

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APPENDIX B.1 CALCULATIONS FOR COMPOSITE DECKING

Decking

Type N, 16 gauge composite decking

Composite decking = 57 psf

3" metal decking = 3.3 psf

concrete = 57 psf − 3.3 psf = 53.7 psf

with ponding :

53.7 + 53.7 (10%) = 59.07 psf

+ 3.3 psf

Total composite decking load = 62.4 psf

Appendix B.2 Calculations for Snow Loading

S.D. 1

Snow Drift $\quad W = 4h_d \quad$ flat roof snow load = 31.5 psf

$$h_d = 0.43 \sqrt[3]{\ell_u}\,\sqrt[4]{p_g + 10} - 1.5 \qquad p_g = 50\ \text{lb/ft}^2$$

1.) wind blowing toward the stairwell from the corner

$\ell_u = 117$ ft

$$h_d = 0.43 \sqrt[3]{117}\,\sqrt[4]{50+10} - 1.5$$

$$h_d = 4.35\ \text{ft}$$

$$W = 4h_d = 17.4\ \text{ft}$$

$$\gamma = 0.13\,p_g + 14 \le 30\ \text{lb/ft}^3$$

$$\gamma = 0.13(50) + 14 = 20.5\ \text{lb/ft}^3$$

2.) wind blowing toward the lip of the roof

$\ell_u = 46'8''$

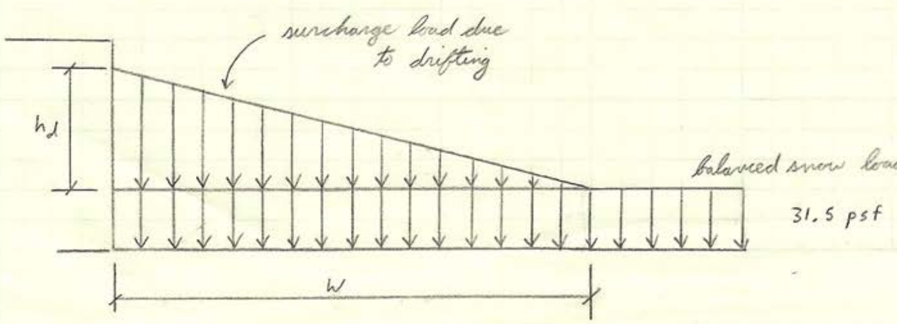
$$h_d = 0.43 \sqrt[3]{46.67}\,\sqrt[4]{50+10} - 1.5$$

$$h_d = 2.81\ \text{ft.}$$

$h_c = 2.00$ ft $\quad$ since $h_c < h_d \longrightarrow h_d = h_c = 2.00$ ft

$$\therefore\ W = \frac{4h_d^2}{h_c} = \frac{4(2.00\ \text{ft})^2}{2\ \text{ft}} = 8\ \text{ft.}$$

S.D. 3



3.) 10' 3.5"

16'-5½" + 3'-0" + 18'-8" + 24'-4" + 20'-0" + 21'-4" + 17-4" − 10'−3.5"

121'−1.5" − 11'−9"

$l_u = 109.375$ ft

$h_d = 0.43\sqrt[3]{110.833}\sqrt[4]{50\,\tfrac{lb}{ft^2}+10} - 1.5$

$h_d = 4.25$ ft.

$W = 4h_d = 17.0$ ft.

APPENDIX B.3 SAMPLE CALCULATION FOR STEEL DESIGN

Beam Design : Steel 1

Average beam spacing : 12 ft.

beam length : 19.5 ft

Loading :

ceiling = $3\ lb/ft^2$

MEP = $5\ lb/ft^2$

insulation = $2\ lb/ft^2$

$10\ lb/ft^2$

roof live load = $20\ lb/ft^2$

(1607.1 → IBC)

total dead load = 10 psf + 62.4 psf = 72.4 psf

snow load = $80.0\ lb/ft^2$ (the maximum snow load will be used for the entire roof to be conservative + promote constructability; using both snow loads, compared to using only the maximum, produces similar results)

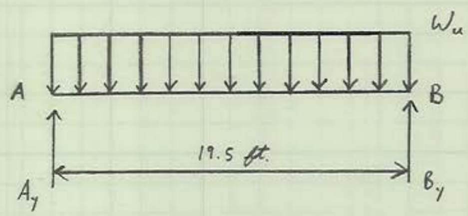
Select W Section :

$Y_{con} = 6.25\ in$

Assume $a = 2\ in$

$Y_1 = 0$

$Y_2 = Y_{con} - a/2 = 6.25\ in - 2\ in/2 = 5.25\ in$



Live Load = 0

beam spacing

Dead Loads = $72.37\ lb/ft^2 \times 12\ ft = 868.44\ lb/ft$

Snow Loads = $80.0\ lb/ft^2 \times 12\ ft = 960\ lb/ft$

Beam Design : Steel 2

Maximum w_u : - using the previously found loads LL, DL, + SL

	w_u (lb/ft)
$U = 1.4 D$	1215.82
$U = 1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$	1522.13
$U = 1.2D + 1.6(L_r \text{ or } S \text{ or } R) + 0.5(L \text{ or } 0.5W)$	2578.13 → governs
$U = 1.2D + 1.0W + 0.5L + 0.5(L_r \text{ or } S \text{ or } R)$	1522.13
$U = 1.2D + 1.0E + 0.5L + 0.2S$	1234.13

$$M_u = M_{max} = \frac{w_u L^2}{8} = \frac{(2578.13 \text{ lb/ft})(19.5 \text{ ft})^2}{8}$$

$$M_u = 122,541.74 \text{ lb}\cdot\text{ft}$$

$$M_u = 122.54 \text{ k}\cdot\text{ft}$$

Beam Selection Using AISC Table 3-19 : W10x12

$\Sigma Q_n = 177$ kips

weight = 12 lb/ft

Interpolation from AISC Table 3-19 :

5 in	5.25 in	5.5 in
132 k·ft	135.5 k·ft	139 k·ft

Trial for selected member : W10x12

$f'_c = 4.00$ ksi

b_e

$$b_E \le 2 \times L/8 = 2 \times \frac{19.5'}{8} \times 12''/' = 58.5 \text{ in} \longrightarrow \text{governs}$$

$$b_E \le 2 \times S/2 = 2 \times \frac{12'}{2} \times 12''/' = 144 \text{ in}$$

$$a = \frac{\Sigma Q_n}{0.85 f'_c b_e} = \frac{177 \text{ K}}{0.85(4 \text{ ksi})(58.5 \text{ in})} = 0.890 \text{ in.}$$

Confirm "a" $\le t_s$

$0.890 \text{ in} \le 6.25 \text{ in}$ √ yes

Beam Design: Steel

3

$$Y_2 = Y_{con} - a/2 = 6.25\ in - \frac{0.890\ in}{2} = 5.81\ in$$

$\phi_b M_n$ from AISC Table 3-19 by interpolation:

5.5 in	5.81 in	6.0 in
139 K ft	142.7 K ft	145 K·ft

Live Load = 0

Dead Load = previous dead load + new beam self weight = 868.44 lb/ft + 12 lb/ft

= 880.44 lb/ft

Snow Load = 960 lb/ft (doesn't change)

Maximum w_u: -using the new load values, incorporating beam self weight

	w_u (lb/ft)
$U = 1.4D$	1232.62
$U = 1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$	1536.53
$U = 1.2D + 1.6(L_r \text{ or } S \text{ or } R) + 0.5(L \text{ or } 0.5W)$	2592.53 → governs
$U = 1.2D + 1.0W + 0.5L + 0.5(L_r \text{ or } S \text{ or } R)$	1536.53
$U = 1.2D + 1.0E + 0.5L + 0.2S$	1248.53

$E = 29,000$ ksi
$I_x = 53.8\ in.^4$ - from AISC Table 1-1

$$M_u = M_{max} = \frac{w_u L^2}{8} = \frac{(2592.53\ lb/ft)(19.5\ ft)^2}{8}$$

$$M_u = 123,226.2\ lb \cdot ft$$

$$M_u = 123.2\ K \cdot ft$$

$$M_u = 123.2\ K \cdot ft < \phi_b M_n = 142.7\ K \cdot ft$$ ✓ Check

Beam Design : Steel

Deflection for beams after hardening of concrete :

$W_{DL} = 0.073 \ ^{K}/_{in}$

$W_{LL} = 0$

$W_{SL} = 0.080 \ ^{K}/_{in}$

changing previous values into $^{K}/_{in}$.

I_{LB} interpolated from Table 3-20 in AISC Manual

5.5 in	5.81 in	6.0 in
247 in.^{4}	258.0 in.^{4}	265 in.^{4}

$$\Delta_{LL} = \frac{5wL^4}{384EI} = \frac{5(0.08)(19.5\times12)^4}{384(29{,}000)(258.0)} = 0.42 \text{ in}$$

$$\Delta_{LL} = 0.42 \text{ in} < {}^{L}/_{360} \text{ or } 1'' \text{ max} = 0.65 \text{ in} \quad \checkmark \text{ Check}$$

$$\Delta_{DL} = \frac{5wL^4}{384EI} = \frac{5(0.073+0.08)(19.5\times12)^4}{384(29{,}000)(258.0)} = 0.80 \text{ in}$$

$$\Delta_{DL} = 0.80 \text{ in} < {}^{L}/_{240} = 0.98 \text{ in} \quad \checkmark \text{ Check}$$

Deflection for beams before concrete hardens :

weight of wet concrete = 59.07 psf
construction live load = 20 psf

$$W_{DL} + W_{LL} = \left(12 \text{ ft} \times \left(3.3 \text{ psf} + 59.07 \text{ psf} + 20 \text{psf}\right) + 12 \ ^{lb}/_{ft}\right) \times \frac{1 \text{ K}}{1000 \text{ lb}} \times \frac{1 \text{ ft}}{12 \text{ in}}$$

$$W_{DL} + W_{LL} = 0.083 \ ^{K}/_{in}$$

$$\Delta_{DL+LL} = \frac{5 W_{DL+LL} L^4}{384 E I_x} = \frac{5\left(0.083 \ ^{K}/_{in}\right)\left(19.5 \text{ ft} \times {}^{12 \text{ in}}/_{1 \text{ ft}}\right)^4}{384 \left(29{,}000 \text{ ksi}\right)\left(53.8 \text{ in}^4\right)} = 2.08 \text{ in}$$

$$\Delta_{DL+LL} = 2.08 \text{ in} \leq {}^{L}/_{300} = \frac{(19.5\times12) \text{ in}}{300} = 0.78 \quad \times \text{ No}$$

Beam Design: Steel

Because deflection did not pass:

Find required I_x:

$$\text{required } I_x = \frac{2.08\,in}{0.78\,in} \times (53.8\,in^4) = 143.5\,in.^4$$

Using AISC Table 3-3, choose a W12x22

$I_x = 156\,in.^4$

$f'_c = 4.00\ ksi$

$\Sigma Q_n = 324^k$ (AISC Table 3-19)

weight $= 22\ lb/ft$

$$a = \frac{\Sigma Q_n}{0.85 f'_c b_e}$$

$$a = \frac{324^k}{0.85\,(4\,ksi)\,(58.5\,in)} = 1.63\,in$$

$t_s = 6.25\,in$

Confirm $a \le t_s$ ✓ Check

$$Y_2 = 6.25 - \frac{1.63}{2} = 5.44\,in$$

Interpolation from AISC Table 3-19:

5 in	5.44 in	5.5 in
271 k·ft	281.5 k·ft	283 k·ft

$W_{DL} = 868.44\ lb/ft + 22\ lb/ft$

$W_{DL} = 890.44\ lb/ft$

$W_{LL} = 0$

$W_{SL} = 960\ lb/ft$

Maximum W_u : - using the new W12x22 beam weight

	W_u (lb/ft)
$U = 1.4D$	1246.62
$U = 1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$	1548.53
$U = 1.2D + 1.6(L_r \text{ or } S \text{ or } R) + 0.5(L \text{ or } 0.5W)$	2604.53 → governs
$U = 1.2D + 1.0W + 0.5L + 0.5(L_r \text{ or } S \text{ or } R)$	1548.53
$U = 1.2D + 1.0E + 0.5L + 0.2S$	1260.53

$E = 29{,}000$ ksi
$I_x = 156$ in.^{4} (AISC Table 1-1)

$$M_u = M_{max} = \frac{w_u L^2}{8} = \frac{(2604.53 \text{ lb/ft})(19.5 \text{ ft})^2}{8} = 123{,}796.57 \text{ lb-ft}$$

$$M_u = 123.80 \text{ k-ft}$$

$M_u = 123.80$ k-ft $< \phi_b M_n = 281.5$ k-ft ✓ Check

Deflection for beams after hardening of concrete:

$W_{DL} = 0.072$ K/in
$W_{LL} = 0$
$W_{KL} = 0.080$ K/in

changing previous values into K/in.

I_{LB} interpolated from Table 3-20 in AISC Manual:

5.0 in	5.44 in	5.5 in
559 in.^{4}	591.6 in.^{4}	596 in.^{4}

$$\Delta_{LL} = \frac{5wL^4}{384EI} = \frac{5(0.08 \text{ K/in})(19.5 \text{ ft} \times \frac{12 \text{ in}}{1 \text{ ft}})^4}{384(29{,}000 \text{ ksi})(591.6 \text{ in}^4)} = 0.18 \text{ in}$$

$\Delta_{LL} = 0.18$ in $\leq L/360$ or 1" max $= 0.65$ in ✓ Check

$$\Delta_{DL} = \frac{5wL^4}{384EI} = \frac{5(0.08 \text{ K/in} + 0.072 \text{ K/in})(19.5 \text{ ft} \times {}^{12 \text{ in}}/_{1 \text{ ft}})^4}{384(29{,}000 \text{ ksi})(591.6 \text{ in}^4)} = 0.35 \text{ in}$$

$\Delta_{DL} = 0.35$ in $\leq L/240 = 0.98$ in ✓ Check

Deflection for girders before concrete hardens :

weight of wet concrete = 59.07 psf
construction live load = 20 psf

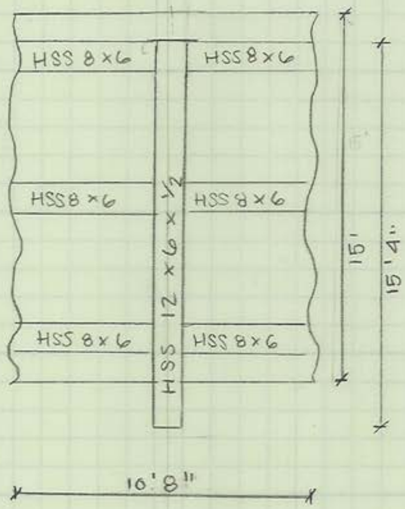
$W_{DL} + W_{LL} = \left(12\ ft \times \left(3.3\ psf + 59.07\ psf + 20\ psf\right) + 22\ ^{lb}/_{ft}\right) \times \frac{1k}{1000\ lb} \times \frac{1ft}{12in}$

$W_{DL} + W_{LL} = 0.084\ ^{k}/_{in.}$

$\Delta_{DL+LL} = \frac{5\ W_{DL+LL}\ L^4}{384\ EI_x} = \frac{5\ (0.084\ ^{k}/_{in})\left(19.5\ ft \times ^{12in}/_{1ft}\right)^4}{384\ (29{,}000ksi)\ (156 in^4)} = 0.72\ in$

$\Delta_{DL+LL} = 0.72 in \leq {}^{L}/_{300} = 0.78 in$ ✓ Check

APPENDIX B.4 SAMPLE CALCULATION FOR STEEL BEAM UNDER MECHANICAL SCREEN WALL



perforated metal wall panel wt. = 0.7 psf

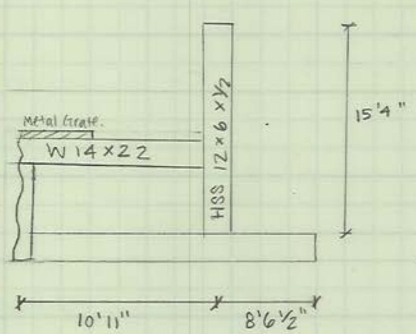
Column wt. (HSS 12×6×½) = 72.9 lb/ft

Girder wt. (HSS 8×6) = 32.58 lb/ft

Beam wt. (W14×22) = 22 lb/ft

P = (0.7 psf)(15')(10'8") + (72.9 lb/ft)(15'4") + 3(32.58 lb/ft)(10'8")

+ (22 lb/ft)(5'5½") = 2938.28 lbs

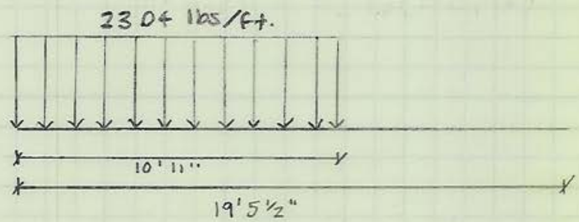


$U = 1.2D + 1.6S + 0.5L$ 2

$S = 120$ psf

$(120\text{ psf})(12')(1.6) = 2304$ lb/ft

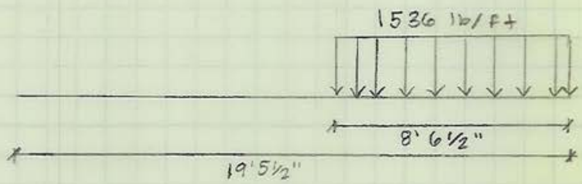
case 1
snow load



$S = 80$ psf

$(80\text{ psf})(12')(1.6) = 1536$ lb/ft

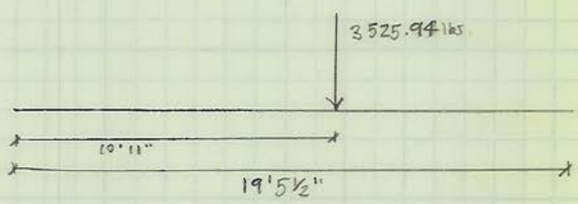
case 2
snow load



$P = 2938.28$ lbs

$(2938.28\text{ lbs})(1.2) = 3525.94$ lbs

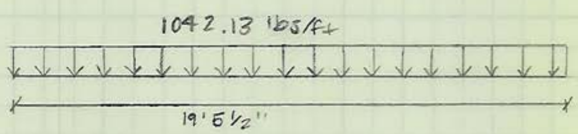
case 3
point load



$D = 72.37$ psf

$(72.37\text{ psf})(12')(1.2) = 1042.13$ lbs/ft

case 4
dead load



case 1

$$\circlearrowleft \Sigma M_A = B_y(19'5\tfrac{1}{2}") - (192\text{ psf})(12\text{ ft})(10'11")(5'5\tfrac{1}{2}") = 0$$

$$B_y = 7055.48\text{ lbs.}$$

$$+\uparrow \Sigma F_y = A_y + 7055.48\text{ lbs} - (192\text{ psf})(12')(10'11") = 0$$

$$A_y = 18096.52\text{ lbs}$$

case 2

$$\circlearrowleft \Sigma M_A = B_y(19'5\tfrac{1}{2}") - (1536\text{ lb/ft})(8'6\tfrac{1}{2}")(15'2\tfrac{1}{4}") = 0$$

$$B_y = 10240.34\text{ lbs}$$

$+\uparrow \sum F_y = A_y + 10240.34 \text{ lbs} - (1536 \text{ lb/ft})(8'6\tfrac{1}{2}") = 0$

$A_y = 2879.66 \text{ lbs}$

Case 3

$+\circlearrowright \sum M_A = B_y(19'5\tfrac{1}{2}") - (3525.94 \text{ lbs})(10'11") = 0$

$B_y = 1978.15 \text{ lbs}$

$+\uparrow \sum F_y = A_y + 1978.15 \text{ lbs} - 3525.94 \text{ lbs} = 0$

$A_y = 1547.79 \text{ lbs}$

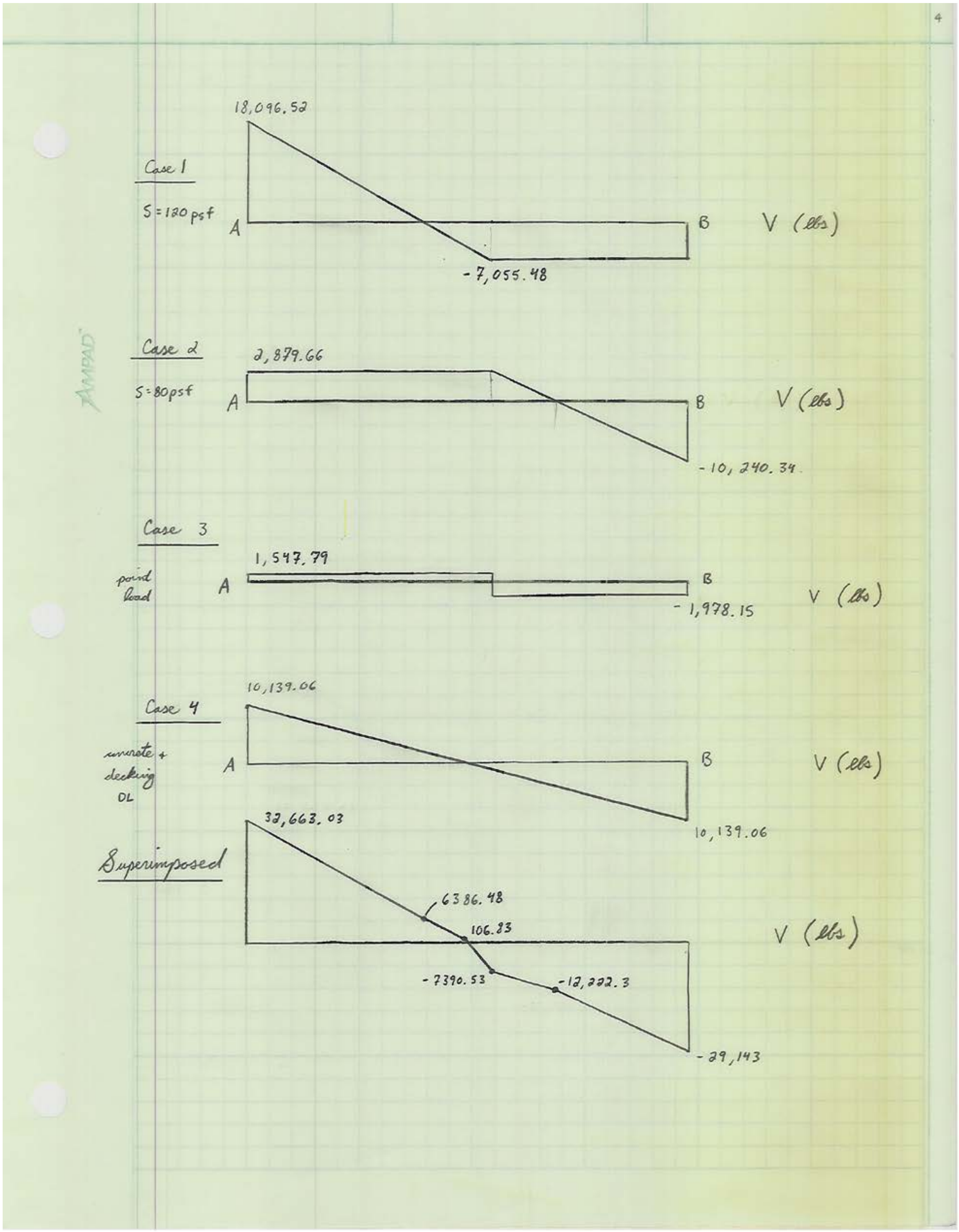
Case 4

$+\circlearrowright \sum M_A = B_y(19'5\tfrac{1}{2}") - (1042.13 \text{ lb/ft})(19'5\tfrac{1}{2}")(9'8\tfrac{3}{4}") = 0$

$B_y = 10{,}139.06$

$+\uparrow \sum F_y = A_y + 10{,}139.06 - (1042.13 \text{ lb/ft})(19'5\tfrac{1}{2}") = 0$

$A_y = 10{,}139.06$


Case 1
S = 120 psf
18,096.52
A
B
V (lbs)
-7,055.48
Case 2
S = 80psf
2,879.66
A
B
V (lbs)
-10,240.34
Case 3
point load
1,547.79
A
B
-1,978.15
V (lbs)
Case 4
concrete + decking DL
10,139.06
A
B
V (lbs)
10,139.06
Superimposed
32,663.03
6386.48
106.83
-7390.53
-12,222.3
V (lbs)
-29,143

Case 1 $\frac{25,152}{10.92} = \frac{18,096.52}{x}$ $x = 7.85$ ft

Case 4 $\frac{10,139.06}{9.73} = \frac{x}{9.73 - 7.85}$ $x = 1959.04$ lb

Case 4 $\frac{10,139.06}{9.73} = \frac{x}{1.187}$ $x = -1236.56$ lb

Case 1 $\frac{-7055.48}{10.92 - 7.85} = \frac{-x}{9.73 - 7.85}$ $x = -4320.62$ lb

Case 2 $\frac{13,120}{8.54} = \frac{2879.66}{x}$ $x = 12.79$ ft

Case 4 $\frac{10,139.06}{9.73} = \frac{x}{1.187}$ $x = -1236.56$ lb

Case 4 $\frac{10,139.06}{9.73} = \frac{x}{3.06}$ $x = -3188.65$ lb

Superimposed:

x - intercept

$$\frac{106.83 + 7390.53}{10.92 - 7.85} = \frac{106.83}{x} \qquad X = 0.0437$$

$$9.73 + 0.0437 = 9.774 \text{ ft}$$

x - intercept = 9.774 ft.

Area under positive side of superimposed shear diagram = 160 K·ft

$$M_u = 160 \text{ K·ft}$$

From Superimposed Shear Diagram:

$M_U = 160$ k·ft.

From Table 3-19 in AISC Manual, and using Interpolation, select:

W10×15 ($\phi_b M_n$)		
5	5.25	5.5
165	169.5	174

assume "a" = 2 in.
YCON = 6.25 in.
Y2 = 5.25 in.

From Table 1-1 ; W10×15 $\Sigma Q_n = 221$ k

$F'_c = 4$ ksi

$b_e \le 2\left(\frac{19.5}{8}\right)12 = 59$ in

$b_e \le 2\left(\frac{12}{2}\right)12 = 144$ in.

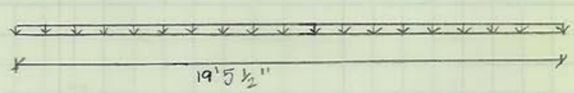
$a = \dfrac{221\text{K}}{0.85(4\text{ksi})(59\text{in})} = 1.11$ in.

$Y2 = 6.25\text{in} - \left(\frac{1.11\text{in}}{2}\right) = 5.69$ in. 1.11 in $\le$ 5.69 in. ✓

From Table 3-19 in AISC Manual, and using Interpolation,

W10×15 ($\phi_b M_n$)		k·ft
5.5	5.69	6.0
174	177.1	182

case 5
selfwt.



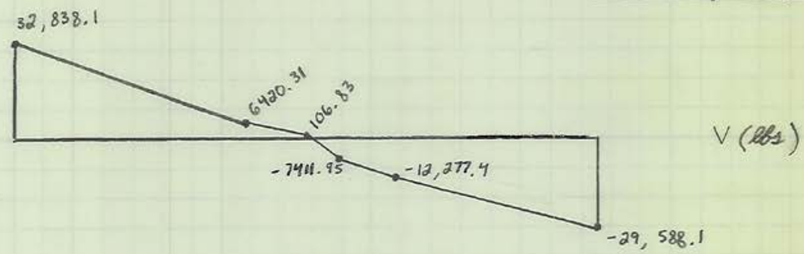
D = 15 lb/ft

(15 lb/ft)(1.2) = 18 psf

7

Case 5 : beam weight (self weight)

New Superimposed



8

Case 5 $\frac{175.1}{9.73} = \frac{x}{7.85}$ $x = 33.83$ lb at 7.85 ft

Case 5 $\frac{175.1}{9.73} = \frac{x}{1.187}$ $x = -21.36$ lb at 10.92 ft

Case 5 $\frac{175.1}{9.73} = \frac{x}{3.06}$ $x = -55.07$ lb at 12.79 ft

From Superimposed Shear Diagram:

$M_u = 160.3$ k·ft $< \Phi_b M_n = 177.1$ k·ft ✓

$E = 29000$ ksi

$I_x = 68.9$ in^4 (From Table 1-1 in AISC Manual)

9

Superimposed Deflections : before concrete hardens

Assume snow load of 120 psf across the entire beam for constructibility

120 psf snow load

$$W = (12'(3.3\text{ psf} + 69.07\text{ psf} + 120\text{ psf}) + 15\text{ lb/ft}) = 2323.44\text{ lb/ft}$$

$$\Delta = \frac{5WL^4}{384EI_x} = \frac{5(2323.44\text{ lb/ft})(19.45833\text{ ft})^4}{384(29000\text{ ksi})(68.9\text{ in}^4)} \times \frac{1728\text{ in.}}{1\text{ ft}} \times \frac{1\text{ k}}{1000\text{ lb.}}$$

$\Delta = 3.75$ in.

Pt. Load = 2.93828 K.

$$\Delta_{Pt.} = \frac{P(a)^2(b)^2}{3EIl} = \frac{(2.93828\text{ k})(131\text{ in})^2(102.5\text{ in})^2}{3(29000)(68.9)(233.4996\text{ in})} = 0.38\text{ in}$$

$\Delta_{total} = 4.13$ in.

$\Delta_{total} = 4.13$ in $\leq L/300 = 0.78$ in ✗

$$\text{Required } I_x = \left(\frac{4.13\text{ in}}{0.78\text{ in}}\right) 68.9\text{ in}^4 = 364.82\text{ in}^4$$

Using Table 3-3 in AISC Manual, select W16×31

$I_x = 375\text{ in}^4$ $\Sigma Q_n = 457$ k

$$a = \frac{457\text{ K}}{0.85(4\text{ ksi})(59\text{ in})} = 2.28\text{ in}$$

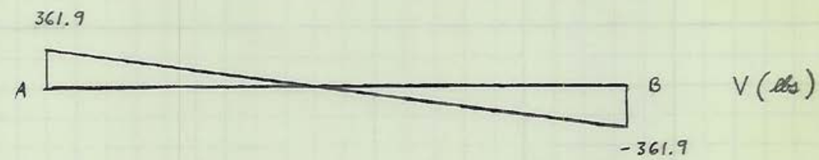
$$Y2 = 6.25\text{ in} - \left(\frac{2.28\text{ in}}{2}\right) = 5.11\text{ in.}$$

From Table 3-19 in AISC Manual, and using interpolation

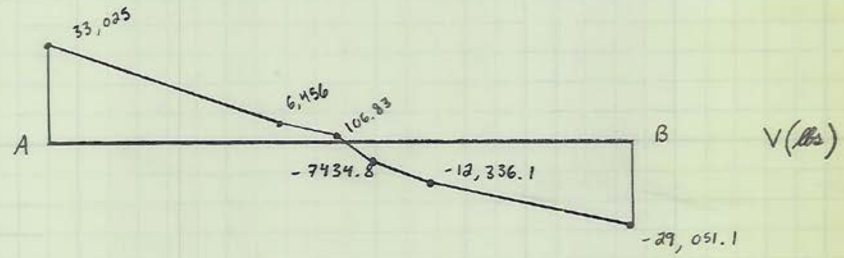
W16×31 $\phi_b M_n = 410.6$ k·ft

10

Case 6 : new beam weight included : W16x31



New Superimposed - W16 x 31



From superimposed shear diagram:

$M_u = 161.2 \text{ k}\cdot\text{ft}$

$M_u = 161.2 \text{ k}\cdot\text{ft} < \phi_b M_n = 410.6 \text{ k}\cdot\text{ft}$

$E = 29{,}000$ ksi

$I_x = 375 \text{ in.}^4$ (From Table 1-1 in AISC Manual)

where it crosses x-axis:

$$\frac{106.83}{x} = \frac{7541.63}{1.19}$$

$x = 0.017$ ft

crosses at = 9.75 ft

Superimposed Deflections - Before Hardening

$W = (12' (3.3 psf + 69.07 psf + 120 psf) + 31 lb/ft) = 2339.44 lb/ft$

$\Delta = \frac{5wL^4}{384EI_x} = \frac{5(2339.44 lb/ft)(19.45833 ft)^4}{384(29000 ksi)(375 in^4)} \times \frac{1728 in}{1 ft} \times \frac{1k}{1000 lb.}$

$\Delta = 0.69 in$

$\Delta_{pt} = \frac{Pa^2b^2}{3EIl} = \frac{(2.93828 k)(131 in)^2(102.5 in)^2}{3(29000 ksi)(375 in^4)(233.4996 in.)} = 0.07 in$

$\Delta_{total} = 0.76 in$

$\Delta_{total} = 0.76 in. \leq 0.78 in$ ✓

Superimposed Deflections - After Hardening

From Table 3-20 in AISC Manual, using Interpoliation

$I_{LB} = 1223.3 in^4$

$W_{LL} = 12' (120 psf) = 1440 lb/ft. = 0.12 k/in.$

$W_{DL} = 12' (57 psf + 10 psf) + 31 lb/ft) = 835 lb/ft. = 0.0696 k/in.$

$P = 2.93828 k$

$\Delta_{LL} = \frac{5(W_{LL})L^4}{384EI_{LB}} = \frac{5(0.12 k/in)(233.4996 in)^4}{384(29000 ksi)(1223.3 in^4)} = 0.13 in.$

$\Delta_{LL} = 0.13 in \leq \frac{L}{360} = 0.65 in$ ✓

$\Delta_{PT} = \frac{Pa^2b^2}{3EIl} = \frac{(2.93828 k)(131 in)^2(102.5 in)^2}{3(29000 ksi)(1223.3 in^4)(233.4996)} = 0.02 in.$

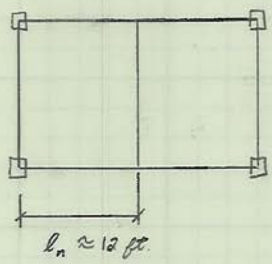
$\Delta_{DL} = \frac{5(W_{DL} + W_{LL})L^4}{384EI_{LB}} = \frac{5(0.0696 + 0.12)(233.4996)^4}{384(29000)(1223.3)} = 0.2 in.$

$\Delta_{DL\,TOTAL} = 0.22 in \leq L/240 = 0.97 in.$ ✓

W16×31

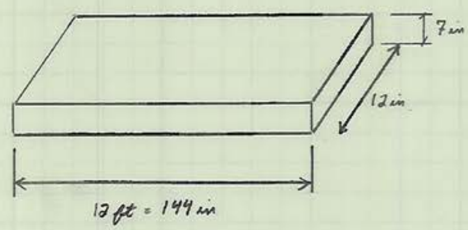
APPENDIX B.5 SAMPLE CALCULATION FOR CONCRETE DESIGN

Concrete slab – new



solid slabs : one end continuous : $\ell/24$

use $\ell_n = 12$ ft $\quad \frac{144 \text{ in}}{24} = 6$ in.



self weight : $150 \text{ lb/ft}^3 \times 7''/12''/' = 87.5$ psf

Loading :

$$\begin{aligned} \text{ceiling} &= 3 \text{ lb/ft}^2 \\ \text{MEP} &= 5 \text{ lb/ft}^2 \\ \text{insulation} &= 2 \text{ lb/ft}^2 \\ \hline & \quad 10 \text{ lb/ft}^2 \end{aligned}$$

Dead load : $10 \text{ lb/ft}^2 + 87.5 \text{ lb/ft}^2 = 97.5 \text{ lb/ft}^2$

Snow load : 80 lb/ft^2

	W_u (psf)
$U = 1.4D$	136.5
$U = 1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$	157
$U = 1.2D + 1.6(L_r \text{ or } S \text{ or } R) + 0.5(L \text{ or } 0.5W)$	245 *
$U = 1.2D + 1.0W + 0.5L + 0.5(L_r \text{ or } S \text{ or } R)$	157
$U = 1.2D + 1.0E + 0.5L + 0.2S$	133

$w_u = 245 \ ^{lb}/_{ft^2} \times 1 \ ft = 245 \ ^{lb}/_{ft}$

$M_u = \frac{1}{10} w \ell_n^2 = \frac{1}{10} \left(245 \ ^{lb}/_{ft}\right)\left(12 \ ft\right)^2$

$M_u = 3528 \ ft \cdot lb = 42,336 \ \frac{lb \cdot in}{1 \ ft}$

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{A_s \ (60,000 \ psi)}{0.85 \ (4000 \ psi)(12 \ in)} = 1.47 \ A_s$

assume $\phi = 0.9$, $\epsilon_s > 0.005$

$\phi M_n = M_u$

$M_n = M_u/_{0.9} = \frac{42,336 \ lb \cdot in}{0.9} = 47,040 \ \frac{lb \cdot in}{ft}$

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

$47,040 \ \frac{lb \cdot in}{ft} = A_s \ (60,000 \ psi)\left(6 \ in - \frac{1.47 A_s}{2}\right)$

$360,000 \ A_s - 44,100 \ A_s^2 = 47,040$

$44,100 \ A_s^2 - 360,000 A_s + 47,040 = 0$

$$\frac{-b \pm \sqrt{b^2 - 4ac}}{2a}$$

$$\frac{360,000 \pm \sqrt{(-360,000)^2 - 4(44,100)(47,040)}}{2 \ (44,100)}$$

$$A_s = \frac{360,000 \pm 348,284.57}{2 \ (44,100)}$$

too large

$A_s = 0.133 \ \frac{in.^2}{1 ft.}$ and $8.03 \ in.^2$

pick $\#4 \longrightarrow A_{\#4} = 0.20 \ in^2$

0.133 in²   1 ft.
0.20 in²   S

$$S = \frac{0.20 \times 12 \text{ in}}{0.11} = 21.82 \text{ in.}$$

Check $A_{s\,min} = \frac{200\, b_w d}{f_y} = \frac{200 (12 \text{ in})(6 \text{ in})}{60{,}000 \text{ psi}} = 0.24 \text{ in}$ ← governs

$$A_{s\,min} = \frac{3\sqrt{f'_c}\, b_w d}{f_y} = \frac{3\sqrt{4000 \text{ psi}}\,(12 \text{ in})(6 \text{ in})}{60{,}000 \text{ psi}} = 0.23 \text{ in.}$$

pick # 5 bar → $A_{\#5} = 0.31 \text{ in}^2$

$$S = \frac{0.31 \text{ in}^2}{0.24 \text{ in}^2} \times 12 \text{ in} = 15.5 \text{ in} \approx 16 \text{ in.}$$

$$\boxed{\#5 \text{ bar @ } 16 \text{ in. spacing}}$$

APPENDIX C.1 SAMPLE SPREADSHEETS FOR WIND LOADING

Grove Street Wing (east to west)		
Top of Elevator/Stairs From ASCE 7		
Enclosure Classification		
Enclosed		
Basic Wind Speed		
V (ft/s)	146.67	
Wind Load Importance Factor		
I_w	1	
Wind Exposure Category		
B		
Wind Topographic Factor		
K_{zt}	1	
Internal Pressure Coefficient		
GC_{pi} ±	0.18	
Corner Zone Dimension		
a (ft)	5	
Mean Roof Height		
h (ft)	60	
Angle of Roof Plane		
θ (degrees)	0	
Wind Directionality Factor		
K_d	0.85	
Velocity Pressure Exposure Coefficient		
K_z	0.85	
Gust Effect Factor		
G	0.85	
Velocity Pressure		
q_z (psf)	39.79	
q_h (psf)	39.79	
Design Wind Pressure	$(+GC_{pi})$	$(-GC_{pi})$
p (psf) Windward	19.89	34.22
p (psf) Leeward	-24.07	-9.75
p (psf) Side Walls	-30.83	-16.51
Wall/Roof Pressure Coefficient		
Windward C_p	0.8	
Leeward C_p	-0.5	
Side Wall C_p	-0.7	

Number of Frames	10

Windward

Level	Height (ft)	Width (ft)	Area (ft^2)	Force (lbs)	Force (k)
Roof	6.67	173	1153	3946.3	3.95
Floor 4	12.00	173	2076	7104.4	7.10
Floor 3	10.67	173	1846	6316.1	6.32
Floor 2	13.34	173	2307	7893.7	7.89
Floor 1	8.00	173	1384	4735.6	4.74

Leeward

Level	Height (ft)	Width (ft)	Area (ft^2)	Force (lbs)	Force (k)
Roof	6.67	154.8	1032	-2484.3	-2.48
Floor 4	12.00	154.8	1858	-4472.3	-4.47
Floor 3	10.67	154.8	1652	-3976.1	-3.98
Floor 2	13.34	154.8	2064	-4969.2	-4.97
Floor 1	8.00	154.8	1238	-2981.1	-2.98

Side Wall 1

Level	Height (ft)	Width (ft)	Area (ft^2)	Force (lbs)	Force (k)
Roof	6.67	191.51	1277	-3936.8	-3.94
Floor 4	12.00	191.51	2298	-7087.3	-7.09
Floor 3	10.67	191.51	2043	-6300.9	-6.30
Floor 2	13.34	191.51	2554	-7874.7	-7.87
Floor 1	8.00	191.51	1532	-4724.2	-4.72

Side Wall 2

Level	Height (ft)	Width (ft)	Area (ft^2)	Force (lbs)	Force (k)
Roof	6.67	114.01	760	-2343.7	-2.34
Floor 4	12.00	114.01	1368	-4219.3	-4.22
Floor 3	10.67	114.01	1217	-3751.2	-3.75
Floor 2	13.34	114.01	1520	-4688.1	-4.69
Floor 1	8.00	114.01	912	-2812.5	-2.81

Side Wall 3

Level	Height (ft)	Width (ft)	Area (ft^2)	Force (lbs)	Force (k)
Roof	6.67	46.67	311	-959.3	-0.96
Floor 4	12.00	46.67	560	-1727.0	-1.73
Floor 3	10.67	46.67	498	-1535.4	-1.54
Floor 2	13.34	46.67	622	-1918.9	-1.92
Floor 1	8.00	46.67	373	-1151.2	-1.15

West Wing (east to west)		
Top of Elevator/Stairs From ASCE 7		
Enclosure Classification		
Enclosed		
Basic Wind Speed		
V (ft/s)	146.67	
Wind Load Importance Factor		
I_w	1	
Wind Exposure Category		
B		
Wind Topographic Factor		
K_{zt}	1	
Internal Pressure Coefficient		
GC_{pi} ±	0.18	
Corner Zone Dimension		
a (ft)	5	
Mean Roof Height		
h (ft)	60	
Angle of Roof Plane		
θ (degrees)	0	
Wind Directionality Factor		
K_d	0.85	
Velocity Pressure Exposure Coefficient		
K_z	0.85	
Gust Effect Factor		
G	0.85	
Velocity Pressure		
q_z (psf)	39.79	
q_h (psf)	39.79	
Design Wind Pressure	$(+GC_{pi})$	$(-GC_{pi})$
p (psf) Windward	19.89	34.22
p (psf) Leeward	-24.07	-9.75
p (psf) Side Walls	-30.83	-16.51
Wall/Roof Pressure Coefficient		
Windward C_p	0.8	
Leeward C_p	-0.5	
Side Wall C_p	-0.7	

Number of Frames	7

Windward

Level	Height (ft)	Width (ft)	Area (ft^2)	Force (lbs)	Force (k)
Roof	6.67	134.0	893	4366.7	4.37
Floor 4	12.00	134.0	1608	7861.2	7.86
Floor 3	10.67	134.0	1430	6988.9	6.99
Floor 2	13.34	134.0	1787	8734.5	8.73
Floor 1	8.00	134.0	1072	5240.1	5.24

Leeward

Level	Height (ft)	Width (ft)	Area (ft^2)	Force (lbs)	Force (k)
Roof	6.67	134.0	893	-3071.9	-3.07
Floor 4	12.00	134.0	1608	-5530.2	-5.53
Floor 3	10.67	134.0	1430	-4916.6	-4.92
Floor 2	13.34	134.0	1787	-6144.6	-6.14
Floor 1	8.00	134.0	1072	-3686.3	-3.69

Side Wall 1

Level	Height (ft)	Width (ft)	Area (ft^2)	Force (lbs)	Force (k)
Roof	6.67	46.67	311	-1370.4	-1.37
Floor 4	12.00	46.67	560	-2467.1	-2.47
Floor 3	10.67	46.67	498	-2193.4	-2.19
Floor 2	13.34	46.67	622	-2741.2	-2.74
Floor 1	8.00	46.67	373	-1644.5	-1.64

Appendix C.2 Sample Spreadsheet for Seismic Loading

	Roof	Floor 4	Floor 3	Floor 2	Floor 1	
Snow Load (psf)	16	0	0	0	0	
Mechanical Equipment (lbs)	43,964	0	0	0		
Live Load (psf)	5	25	25	25	25	
Dead Load (psf)	71.07	71.07	71.07	71.07	67.00	Total (ft.)
Story Height (ft)	13.3333	10.67	10.67	16	0	50.6733
Total Area (ft^2)	22,182	22,182	22,182	22,182	22,182	
Total Weight (lbs)	2,086,155	2,130,919	2,130,919	2,130,919	-	
					TOTAL (lbs)	8,478,914
$W_x h$ (cumulative)	105,712,377	79,568,529	56,831,620	34,094,710	276,207,236	

Cs	0.079

V (lbs)	669,834.17	669.83		
	lbs	kips		
Fx_2	82,683.57	82.68		
Fx_3	137,823.18	137.82		
Fx_4	192,962.79	192.96		
Fx_{roof}	256,364.62	256.36		
	Total	669.83	Total	Check

Appendix C.3 Sample Spreadsheet for Beam Steel Design

Beam Spreadsheet

Loading	Value (psf)
Dead Load	72.37
Snow Load 1	80
Live Load	0
Roof Live Load	20

YCON	6.25
Assume "a"	2
Y1	0
Y2	5.25

Beam Length (ft)	19.5
Beam Spacing (ft)	12

WDL (lb/ft)	868.44
WLL (lb/ft)	-
WSL (lb/ft)	960

Load Combinations	w_u (lb/ft)
U=1.4D	1215.82
U=1.2D + 1.6L + 0.5(L_r or S or R)	1522.13
U=1.2D + 1.6(L_r or S or R) + 0.5(L or 0.5W)	2578.13
U=1.2D + 1.0W + 0.5L + 0.5(L_r or S or R)	1522.13
U=1.2D + 1.0E + 0.5L + 0.2S	1234.13

W10X12		
Interpolation from Table 3-19		
5	5.25	5.5
132	**135.5**	139

Mu (lb*ft)	122,541.65
Mu (k*ft)	122.54

Selection Using AISC Table 3-19	ΣQn (k)	Weight (lbs/ft)
W10X12	177	12

Trial for Selected Member

bE (in)	≤	**59**
bE (in)	≤	**144**

f'c (ksi)	4.00

a (in)	0.89
ts	6.25
Confirm a ≤ ts	Check

Y2	5.81

ϕbMn (k*ft) from Interpolation in AISC Table 3-19		
5.5	5.81	6.0
139	**142.7**	145

WDL (lb/ft)	880.44
WLL (lb/ft)	-
WSL (lb/ft)	960

Load Combinations	w_u (lb/ft)
U=1.4D	1232.62
U=1.2D + 1.6L + 0.5(L_r or S or R)	1536.53
U=1.2D + 1.6(L_r or S or R) + 0.5(L or 0.5W)	2592.53
U=1.2D + 1.0W + 0.5L + 0.5(L_r or S or R)	1536.53
U=1.2D + 1.0E + 0.5L + 0.2S	1248.53

E (ksi)	**29000**	
Ix (in4)	53.8	from Table 1-1

Mu (lb*ft)	123,226.10				
Mu (k*ft)	123.23	<	ϕbMn (k*ft)	142.7	Check

Deflection for beams after hardening of concrete

WDL (k/in)	0.07
WLL (k/in)	-
WSL (k/in)	0.08

ILB (in4) Interpolated from Table 3-20 in AISC Manual		
5.5	5.81	6.0
247	**258.0**	**265**

ΔLL (in)	0.42	≤	L/360 (or 1" max)	0.65	Check
ΔDL (in)	0.80	≤	L/240 (in)	0.98	Check

Deflection for beams before concrete hardens

Wt. wet concrete (psf)	59.07
Construction LL (psf)	20

WDL + WLL (lb/ft)	0.08

ΔDL+LL (in)	2.09	≤	L/300 (in)	0.78	FALSE

If Defelection does not pass (FALSE)

Required Ix (in4)	143.89

New Grider (Using Table 3-3)	Ix (in4)	∑Qn (k)	Weight (lb/ft)
W12X22	156	324	22

f'c (ksi)	4.00

a (in)	1.63
ts	6.25
Confirm a ≤ ts	Check

Y2	5.44

ϕbMn (k*ft) from Interpolation in AISC Table 3-19		
5	5.44	5.5
271	**281.5**	283

WDL (lb/ft)	890.44
WLL (lb/ft)	-
WSL (lb/ft)	960

Load Combinations	**w_u (lb/ft)**
U=1.4D	1246.62
U=1.2D + 1.6L + 0.5(L_r or S or R)	1548.53
U=1.2D + 1.6(L_r or S or R) + 0.5(L or 0.5W)	2604.53
U=1.2D + 1.0W + 0.5L + 0.5(L_r or S or R)	1548.53
U=1.2D + 1.0E + 0.5L + 0.2S	1260.53

E (ksi)	**29000**	
Ix (in4)	156	from Table 1-1

Mu (lb*ft)	123,796.47				
Mu (k*ft)	123.80	<	ϕbMn (k*ft)	281.5	Check

Deflection for girders after hardening of concrete

WDL (k/in)	0.07
WLL (k/in)	-
WSL (k/in)	0.08

ILB (in4) Interpolated from Table 3-20 in AISC Manual		
5.0	5.44	5.5
559	**591.2**	**596**

ΔLL (in)	0.18	≤	L/360 (or 1" max)	0.65	Check
ΔDL (in)	0.35	≤	L/240 (in)	0.98	Check

Deflection for girders before concrete hardens

Wt. wet concrete (psf)	59.07
Construction LL (psf)	20

WDL + WLL (lb/ft)	0.08

ΔDL+LL (in)	0.73	≤	L/300 (in)	0.78	Check

APPENDIX C.4 SAMPLE SPREADSHEET FOR GIRDER STEEL DESIGN

Girder Spreadsheet

Loading	Value (psf)
Dead Load	72.4
Snow Load 1	80
Live Load	0
Roof Live Load	20

YCON	6.25
Assume "a"	2
Y1	0
Y2	5.25

Girder Length (ft)	24.33
Beam Weight (lbs/ft)	22
Beam Spacing (ft)	12
Girder Spacing (ft)	13.75

WDL (lb/ft)	1,020.71
WLL (lb/ft)	-
WSL (lb/ft)	1100

Load Combinations	wu (lb/ft)
U=1.4D	1428.99
U=1.2D + 1.6L + 0.5(Lr or S or R)	1774.85
U=1.2D + 1.6(Lr or S or R) + 0.5(L or 0.5W)	2984.85
U=1.2D + 1.0W + 0.5L + 0.5(Lr or S or R)	1774.85
U=1.2D + 1.0E + 0.5L + 0.2S	1444.85

W10X19		
Interpolation from Table 3-19		
5	5.25	5.5
213	**218**	223

Mu (lb*ft)	220,919.75
Mu (k*ft)	220.92

Selection Using AISC Table 3-19	ΣQn (k)	Weight (lbs/ft)
W10X19	281	19

Trial for Selected Member

bE (in)	≤	**73**
bE (in)	≤	**165**

f'c (ksi)	4.00

a (in)	1.13
ts	6.25
Confirm a ≤ ts	Check

Y2	5.68

ϕbMn (k*ft) from Interpolation in AISC Table 3-19		
5.5	5.68	6.0
223	**227.0**	234

WDL (lb/ft)	1,039.71
WLL (lb/ft)	-
WSL (lb/ft)	1100

Load Combinations	wu (lb/ft)
U=1.4D	1455.59
U=1.2D + 1.6L + 0.5(Lr or S or R)	1797.65
U=1.2D + 1.6(Lr or S or R) + 0.5(L or 0.5W)	3007.65
U=1.2D + 1.0W + 0.5L + 0.5(Lr or S or R)	1797.65
U=1.2D + 1.0E + 0.5L + 0.2S	1467.65

E (ksi)	**29000**	
Ix (in4)	96.3	from Table 1-1

Mu (lb*ft)	222,607.26				
Mu (k*ft)	222.61	<	ϕbMn (k*ft)	227.0	Check

Deflection for girders after hardening of concrete

WDL (k/in)	0.09
WLL (k/in)	-
WSL (k/in)	0.091666667

ILB (in4) Interpolated from Table 3-20 in AISC Manual		
5.5	5.68	6.0
412	**423.4**	**443**

ΔLL (in)	0.71	≤	L/360 (or 1" max)	0.81	Check
ΔDL (in)	1.37	≤	L/240 (in)	1.22	FALSE

Deflection for girders before concrete hardens

Wt. wet concrete (psf)	59.07
Construction LL (psf)	20

WDL + WLL (lb/ft)	0.10

ΔDL+LL (in)	3.32	≤	L/300 (in)	0.973332	FALSE

If Defelection does not pass (FALSE)

Required Ix (in4)	328.87

New Grider	Ix (in4)	∑Qn (k)	Weight (lb/ft)
W16X31	375	457	31

f'c (ksi)	4.00

a (in)	1.84
ts	6.25
Confirm a ≤ ts	Check

Y2	5.33

ϕbMn (k*ft) from Interpolation in AISC Table 3-19		
5.0	5.33	5.5
443	**454.2**	460

WDL (lb/ft)	1,051.71
WLL (lb/ft)	-
WSL (lb/ft)	1100

Load Combinations	**wu (lb/ft)**
U=1.4D	1472.39
U=1.2D + 1.6L + 0.5(Lr or S or R)	1812.05
U=1.2D + 1.6(Lr or S or R) + 0.5(L or 0.5W)	3022.05
U=1.2D + 1.0W + 0.5L + 0.5(Lr or S or R)	1812.05
U=1.2D + 1.0E + 0.5L + 0.2S	1482.05

E (ksi)	**29000**	
Ix (in4)	375	from Table 1-1

Mu (lb*ft)	223,673				
Mu (k*ft)	223.67	<	ϕbMn (k*ft)	454.2	Check

Deflection for girders after hardening of concrete

WDL (k/in)	0.09
WLL (k/in)	-
WSL (k/in)	0.091666667

ILB (in4) Interpolated from Table 3-20 in AISC Manual		
5.0	5.33	5.5
794	**825.0**	**841**

ΔLL (in)	0.36	≤	L/360 (or 1" max)	0.81	Check
ΔDL (in)	0.71	≤	L/240 (in)	1.22	Check

Deflection for girders before concrete hardens

Wt. wet concrete (psf)	59.07
Construction LL (psf)	20

WDL + WLL (lb/ft)	0.10

ΔDL+LL (in)	0.86	≤	L/300 (in)	0.97	Check

APPENDIX C.5 SAMPLE SPREADSHEET FOR COLUMN STEEL DESIGN

Sroof (psf)	80
Droof (psf)	71.06525
Lfloor3 (psf)	100
Dfloor3 (psf)	71.06525
Lfloor2 (psf)	100
Dfloor2 (psf)	71.06525
Lfloor 1 (psf)	100
Dfloor1 (psf)	71.06525
Mechanical Point Load (lbs.)	0

Roof	
Tributary Area (ft2)	167.0625
Beam Wt. (lbs/ft.)	22
Beam Spacing (ft.)	12.15
Girder Wt. (lbs/ft.)	31
Girder Spacing (ft.)	13.75

Floor 3	
Tributary Area (ft2)	167.0625
Beam Wt. (lbs/ft.)	22
Beam Spacing (ft.)	12.15
Girder Wt. (lbs/ft.)	31
Girder Spacing (ft.)	13.75

Floor 2	
Tributary Area (ft2)	167.0625
Beam Wt. (lbs/ft.)	22
Beam Spacing (ft.)	12.15
Girder Wt. (lbs/ft.)	31
Girder Spacing (ft.)	13.75

Floor 1	
Tributary Area (ft2)	167.0625
Beam Wt. (lbs/ft.)	22
Beam Spacing (ft.)	12.15
Girder Wt. (lbs/ft.)	31
Girder Spacing (ft.)	13.75

Concrete Dead Load (psf)	57
MEP (psf)	10

	Pu (lbs)	Pu (k)
1.2D+1.6L+0.5S	143859.7	143.8597

Length of Column (ft.)	32
Assume K1=	1
KL=	32
From Table 1-1	
rx (in)	4.37
ry (in)	2.56

E (psi)	29000
Fy (psi)	50

Using Table 4-1 in AISC Manual: Select
W10X54

KL/rx	87.87185
KL/ry	150

150	Governs	>	4.71√(E/Fy)	113.4318	Check

Fe (ksi)	12.72082
Fcr (ksi)	11.15616
ΦcFcr (ksi)	10.04055

From Table 1-1	
Ag (in2)	15.8

ΦcPn (k)	158.6406	>	Pu	143.8597	Check

APPENDIX C.6 SAMPLE SPREADSHEET FOR *RISA-2D* INPUT - WEIGHTS

		Weight (lbs)
Sroof (psf)	80.0	
Droof (psf)	71.1	72960.3172
Lfloor3 (psf)	100.0	
Dfloor3 (psf)	71.1	72960.3172
Lfloor2 (psf)	100.0	
Dfloor2 (psf)	71.1	72960.3172
Lfloor 1 (psf)	100.0	
Dfloor1 (psf)	71.1	72960.3172
Mechanical Point Load (lbs.)	0.0	

	Weight (lbs)	Weight (k)
	291,841	291.84

Roof	
Tributary Area (ft2)	1026.667
Beam Wt. (lbs/ft.)	22
Beam Spacing (ft.)	12.15
Girder Wt. (lbs/ft.)	31
Girder Spacing (ft.)	13.75

Floor 3	
Tributary Area (ft2)	1026.667
Beam Wt. (lbs/ft.)	22
Beam Spacing (ft.)	12.15
Girder Wt. (lbs/ft.)	31
Girder Spacing (ft.)	13.75

Floor 2	
Tributary Area (ft2)	1026.667
Beam Wt. (lbs/ft.)	22
Beam Spacing (ft.)	12.15
Girder Wt. (lbs/ft.)	31
Girder Spacing (ft.)	13.75

Floor 1	
Tributary Area (ft2)	1026.667
Beam Wt. (lbs/ft.)	22
Beam Spacing (ft.)	12.15
Girder Wt. (lbs/ft.)	31
Girder Spacing (ft.)	13.75

Concrete Dead Load (psf)	57
MEP (psf)	10

APPENDIX C.7 SAMPLE SPREADSHEET FOR EFFECTIVE LENGTH METHOD

P_{nt} (gravity loads) k	89.8			
M_{nt} (gravity loads) ft-	0.2			
Braced Frame, Let K=	1			
L (ft)	31			
KL (ft)	31			
$P_c = \phi_c P_n$ (k)	169.5			
$P_r = P_{nt} + B_2 P_{lt}$	89.8			
P_r/P_c	0.529794	≥	0.2	Use Equation H1-1a
M1	0.1			
M2	0.2			
Cm	0.4			
Pe1 (k)	626.6925			
B1	0.466904			
Mr (ft-k)	0.093381			
φbMp	3330			
P_r/P_c + 8/9 (Mr/Mc)	0.529818	<	1.0	Section is Satisfactory

Member	10X54

Interpolation from Table 4-1 in AISC			
KL (ft)	30	31	32
φcPn (k)	180	169.5	159

Zx (in^3) (Table 1-1)	F_y (ksi)
66.6	50

B_2=0

Table 4-1 in AISC	
L_p (ft.)	L_r (ft.)
9.04	33.6

E (ksi)	29000
I (in4)	303
α	1

L_b (ft.)		L_p (ft.)		L_r (ft.)
31	>	9.04	<	33.6
Zone 2				
Table 1-1 in AISC				
b_f (in)	t_f (in)	h/t_w	S_x (in3)	Z_x (in3)
10	0.615	21.2	60	66.6

FLB	$b_f/2t_f$	≤	$0.38\sqrt{(E/F_y)}$	
	8.13	≤	9.2	Check
WLB	h/t_w	<	$3.76\sqrt{(E/F_y)}$	
	21.2	<	91	Check

APPENDIX C.8 SAMPLE SPREADSHEET FOR FOOTING DESIGN

Base Plate Design

Pu (k)	162.94375		Column	W10X54		ϕc	0.65
			From Table 1-1				
f'c Pedastal (psi)	3000		bf (in)	d (in)			
f'c Footing (psi)	4000		10	10.1			
Base Plate Steel	A36		Fy (ksi)	36			
Assume √A2/A1 =	1		A1 (in2) =	73.73020362	≥	d*bf=	101	False
A1 Minimum (in2) ≥	(bf)(d)						
A1 Minimum (in2) ≥	101						

* The area of the baseplate needs to be larger than the column, therefore use 12"X12" to accommodate 10"x10" with 2" on each side

Try a Base Plate of 12"X12"

Pedestal Area of 17"X17"

A1 (in2) =	144	A2 (in2) =	289

Permissable Stress (psi)			Actual Stress (psi)		
ϕc(0.85(f'c))(√(A2/A1)) =	3130.833333	>	Pu/A1 =	1131.55	Check

Base Plate Dimensions

Δ (in) =	0.7975			Baseplate (in)	13	X	12
				Pedestal (in)	18	X	17
N (in) =	12.7975	≈	13				
B (in) =	11.07692308	≈	12	A1 (in2) =	156		
				A2 (in2) =	306		

Then, Check Bearing Strength of Concrete

√(A2/A1)	1.40				
ΦcPp (k) = ϕc(0.85(f'c))(A1)(√(A2/A1)) =	482.85	>	Pu (k) =	162.94	Check
m (in) =	1.70				
n (in) =	2				
n' (in) =	2.51				
ℓ (in) =	2.51				
t required (in) =	0.64	≈	3/4		

Confirm Pedestal Area is less than four times the area of the baseplate

A2 (in2) =	306	<	4*A1 (in2) =	624	Check

Calculate Footing Size

P (k)=	113.724	Af (ft2)=	28.431
S (k/ft2)=	4		
		Af =	6ft X 6ft

APPENDIX C.9 SPREADSHEET OF BEAMS AND COST FOR STEEL DESIGN

Structural Framing Schedule							
Assembly Code	Assembly Description	Family and Type	Count	Length	Cost/LF	Cost of One @ Length	Total Cost
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	12' - 10 1/8"	$35.75	$ 459.09	$ 459.09
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	12' - 10 3/4"	$35.75	$ 461.16	$ 461.16
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	2	12' - 11 1/8"	$35.75	$ 462.06	$ 924.12
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	13' - 10 1/4"	$35.75	$ 495.39	$ 495.39
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	13' - 10 3/8"	$35.75	$ 495.80	$ 495.80
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	14' - 0 5/8"	$35.75	$ 502.40	$ 502.40
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	14' - 0 7/8"	$35.75	$ 503.09	$ 503.09
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	14' - 1 3/4"	$35.75	$ 505.58	$ 505.58
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	14' - 3"	$35.75	$ 509.35	$ 509.35
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	14' - 3 1/4"	$35.75	$ 510.35	$ 510.35
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	14' - 4 1/8"	$35.75	$ 512.91	$ 512.91
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	14' - 4 1/4"	$35.75	$ 513.30	$ 513.30
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	14' - 4 1/2"	$35.75	$ 513.82	$ 513.82
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	14' - 4 5/8"	$35.75	$ 514.14	$ 514.14
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	14' - 4 3/4"	$35.75	$ 514.83	$ 514.83
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	14' - 4 7/8"	$35.75	$ 515.00	$ 515.00
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	14' - 4 7/8"	$35.75	$ 515.14	$ 515.14
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	2	14' - 5"	$35.75	$ 515.26	$ 1,030.52
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	14' - 5"	$35.75	$ 515.33	$ 515.33
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	3	14' - 5"	$35.75	$ 515.48	$ 1,546.44
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	7	14' - 5 1/8"	$35.75	$ 515.48	$ 3,608.36
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	4	14' - 5 1/8"	$35.75	$ 515.94	$ 2,063.76
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	14' - 5 1/4"	$35.75	$ 516.14	$ 516.14
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	2	14' - 5 1/4"	$35.75	$ 516.24	$ 1,032.48
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	5	14' - 5 1/4"	$35.75	$ 516.32	$ 2,581.60
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	14' - 5 3/8"	$35.75	$ 516.36	$ 516.36
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	5	14' - 5 3/8"	$35.75	$ 516.48	$ 2,582.40
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	14' - 5 3/8"	$35.75	$ 516.55	$ 516.55
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	14' - 5 1/2"	$35.75	$ 516.73	$ 516.73
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	3	14' - 5 1/2"	$35.75	$ 516.87	$ 1,550.61
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	14' - 5 1/2"	$35.75	$ 516.94	$ 516.94
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	14' - 5 3/4"	$35.75	$ 517.49	$ 517.49
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	2	14' - 5 3/4"	$35.75	$ 517.71	$ 1,035.42
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	14' - 5 7/8"	$35.75	$ 518.09	$ 518.09
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	14' - 6"	$35.75	$ 518.32	$ 518.32
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	14' - 6"	$35.75	$ 518.51	$ 518.51
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	14' - 6 1/4"	$35.75	$ 519.11	$ 519.11
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	14' - 6 1/2"	$35.75	$ 519.70	$ 519.70
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	14' - 6 5/8"	$35.75	$ 520.15	$ 520.15
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	9	14' - 6 5/8"	$35.75	$ 520.15	$ 4,681.35
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	14' - 6 5/8"	$35.75	$ 520.24	$ 520.24
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	14' - 6 5/8"	$35.75	$ 520.30	$ 520.30
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	14' - 7 1/8"	$35.75	$ 521.78	$ 521.78
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	14' - 7 1/4"	$35.75	$ 521.93	$ 521.93
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	14' - 7 1/4"	$35.75	$ 522.13	$ 522.13
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	14' - 7 3/8"	$35.75	$ 522.64	$ 522.64
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	3	14' - 7 1/2"	$35.75	$ 522.81	$ 1,568.43
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	2	14' - 7 1/2"	$35.75	$ 523.00	$ 1,046.00
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	4	14' - 7 1/2"	$35.75	$ 523.01	$ 2,092.04
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	14' - 7 5/8"	$35.75	$ 523.22	$ 523.22
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	14' - 7 3/4"	$35.75	$ 523.70	$ 523.70
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	2	14' - 7 7/8"	$35.75	$ 523.93	$ 1,047.86
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	14' - 8"	$35.75	$ 524.23	$ 524.23
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	14' - 9 1/4"	$35.75	$ 527.97	$ 527.97
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	14' - 10 1/2"	$35.75	$ 531.65	$ 531.65
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	14' - 10 1/2"	$35.75	$ 531.69	$ 531.69
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	14' - 11"	$35.75	$ 533.18	$ 533.18
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	14' - 11 1/4"	$35.75	$ 534.05	$ 534.05
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	14' - 11 1/4"	$35.75	$ 534.17	$ 534.17
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	14' - 11 1/2"	$35.75	$ 534.72	$ 534.72
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	2	15' - 0 1/8"	$35.75	$ 536.45	$ 1,072.90
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	15' - 0 1/4"	$35.75	$ 536.89	$ 536.89
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	15' - 0 1/4"	$35.75	$ 536.92	$ 536.92
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	15' - 0 1/2"	$35.75	$ 537.68	$ 537.68
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	15' - 0 1/2"	$35.75	$ 537.69	$ 537.69
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	15' - 1"	$35.75	$ 539.09	$ 539.09
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	4	15' - 1"	$35.75	$ 539.29	$ 2,157.16

B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	2	15' - 1 5/8"	$35.75	$ 541.14	$ 1,082.28
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	15' - 9 5/8"	$35.75	$ 564.76	$ 564.76
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	15' - 9 5/8"	$35.75	$ 564.93	$ 564.93
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	15' - 9 3/4"	$35.75	$ 565.34	$ 565.34
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	15' - 9 3/4"	$35.75	$ 565.43	$ 565.43
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	2	15' - 11 3/4"	$35.75	$ 571.35	$ 1,142.70
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	16' - 3 5/8"	$35.75	$ 582.78	$ 582.78
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	16' - 3 5/8"	$35.75	$ 582.84	$ 582.84
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	16' - 3 3/4"	$35.75	$ 583.06	$ 583.06
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	16' - 3 3/4"	$35.75	$ 583.08	$ 583.08
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	16' - 4 1/2"	$35.75	$ 585.31	$ 585.31
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	16' - 4 5/8"	$35.75	$ 585.69	$ 585.69
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	16' - 4 5/8"	$35.75	$ 585.75	$ 585.75
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	2	16' - 4 7/8"	$35.75	$ 586.45	$ 1,172.90
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	16' - 5 1/8"	$35.75	$ 587.10	$ 587.10
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	2	16' - 5 1/8"	$35.75	$ 587.15	$ 1,174.30
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	16' - 5 1/8"	$35.75	$ 587.19	$ 587.19
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	16' - 5 1/8"	$35.75	$ 587.25	$ 587.25
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	16' - 5 3/8"	$35.75	$ 587.91	$ 587.91
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	2	16' - 5 7/8"	$35.75	$ 589.38	$ 1,178.76
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	2	16' - 6"	$35.75	$ 590.02	$ 1,180.04
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	13	16' - 6 1/8"	$35.75	$ 590.07	$ 7,670.91
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	16' - 6 1/2"	$35.75	$ 591.31	$ 591.31
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	4	16' - 6 3/4"	$35.75	$ 592.12	$ 2,368.48
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	2	16' - 6 7/8"	$35.75	$ 592.34	$ 1,184.68
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	16' - 7 1/2"	$35.75	$ 594.51	$ 594.51
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	16' - 10 5/8"	$35.75	$ 603.47	$ 603.47
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	2	16' - 10 7/8"	$35.75	$ 604.55	$ 1,209.10
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	17' - 0"	$35.75	$ 607.62	$ 607.62
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	17' - 0 1/4"	$35.75	$ 608.35	$ 608.35
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	17' - 0 7/8"	$35.75	$ 610.23	$ 610.23
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	8	17' - 0 7/8"	$35.75	$ 610.23	$ 4,881.84
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	2	17' - 8 5/8"	$35.75	$ 633.38	$ 1,266.76
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	18' - 0 1/4"	$35.75	$ 644.07	$ 644.07
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	2	18' - 2"	$35.75	$ 649.33	$ 1,298.66
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	18' - 2 5/8"	$35.75	$ 651.42	$ 651.42
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	18' - 2 5/8"	$35.75	$ 651.46	$ 651.46
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	18' - 2 3/4"	$35.75	$ 651.75	$ 651.75
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	18' - 2 3/4"	$35.75	$ 651.83	$ 651.83
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	18' - 2 7/8"	$35.75	$ 652.12	$ 652.12
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	18' - 2 7/8"	$35.75	$ 652.22	$ 652.22
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	18' - 3"	$35.75	$ 652.37	$ 652.37
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	2	18' - 3 1/8"	$35.75	$ 652.66	$ 1,305.32
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	18' - 3 1/4"	$35.75	$ 653.34	$ 653.34
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	18' - 3 3/8"	$35.75	$ 653.40	$ 653.40
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	4	18' - 3 3/8"	$35.75	$ 653.44	$ 2,613.76
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	18' - 3 3/8"	$35.75	$ 653.45	$ 653.45
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	2	18' - 3 3/8"	$35.75	$ 653.55	$ 1,307.10
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	18' - 3 3/8"	$35.75	$ 653.57	$ 653.57
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	18' - 3 3/8"	$35.75	$ 653.64	$ 653.64
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	18' - 3 3/8"	$35.75	$ 653.69	$ 653.69
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	4	18' - 3 1/2"	$35.75	$ 653.86	$ 2,615.44
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	18' - 3 1/2"	$35.75	$ 653.92	$ 653.92
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	18' - 3 1/2"	$35.75	$ 653.98	$ 653.98
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	18' - 3 1/2"	$35.75	$ 654.03	$ 654.03
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	3	18' - 3 5/8"	$35.75	$ 654.14	$ 1,962.42
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	18' - 3 5/8"	$35.75	$ 654.17	$ 654.17
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	18' - 3 5/8"	$35.75	$ 654.28	$ 654.28
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	18' - 3 5/8"	$35.75	$ 654.29	$ 654.29
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	18' - 3 5/8"	$35.75	$ 654.33	$ 654.33
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	18' - 3 5/8"	$35.75	$ 654.41	$ 654.41
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	18' - 3 5/8"	$35.75	$ 654.44	$ 654.44
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	3	18' - 3 7/8"	$35.75	$ 654.44	$ 1,963.32
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	18' - 7 3/8"	$35.75	$ 665.59	$ 665.59
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	2	18' - 7 7/8"	$35.75	$ 666.83	$ 1,333.66
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	18' - 7 7/8"	$35.75	$ 666.84	$ 666.84
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	18' - 7 7/8"	$35.75	$ 666.91	$ 666.91
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	18' - 8"	$35.75	$ 667.35	$ 667.35
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	18' - 8 1/8"	$35.75	$ 667.62	$ 667.62
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	2	18' - 8 1/8"	$35.75	$ 667.62	$ 1,335.24

B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	18' - 8 1/4"	$35.75	$ 668.11	$ 668.11
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	2	18' - 8 3/8"	$35.75	$ 668.42	$ 1,336.84
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	18' - 8 3/8"	$35.75	$ 668.45	$ 668.45
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	18' - 8 5/8"	$35.75	$ 669.11	$ 669.11
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	18' - 8 5/8"	$35.75	$ 669.35	$ 669.35
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	18' - 8 5/8"	$35.75	$ 669.38	$ 669.38
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	18' - 8 3/4"	$35.75	$ 669.41	$ 669.41
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	18' - 8 3/4"	$35.75	$ 669.42	$ 669.42
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	19' - 3 1/4"	$35.75	$ 688.85	$ 688.85
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	19' - 3 3/8"	$35.75	$ 689.47	$ 689.47
B10	Superstructure	HSS-Hollow Structural Section: HSS6X6X5/16	1	24' - 8"	$35.75	$ 882.01	$ 882.01
B10	Superstructure	HSS-Hollow Structural Section: HSS8X6X3/8	6	5' - 5 1/2"	$35.75	$ 195.14	$ 1,170.84
B10	Superstructure	HSS-Hollow Structural Section: HSS8X6X3/8	5	7' - 9"	$35.75	$ 276.90	$ 1,384.50
B10	Superstructure	HSS-Hollow Structural Section: HSS8X6X3/8	12	10' - 0"	$35.75	$ 357.50	$ 4,290.00
B10	Superstructure	HSS-Hollow Structural Section: HSS8X6X3/8	6	10' - 0"	$35.75	$ 357.59	$ 2,145.54
B10	Superstructure	HSS-Hollow Structural Section: HSS8X6X3/8	12	10' - 8"	$35.75	$ 381.33	$ 4,576.00
B10	Superstructure	HSS-Hollow Structural Section: HSS8X6X3/8	6	10' - 11"	$35.75	$ 390.17	$ 2,341.02
B10	Superstructure	HSS-Hollow Structural Section: HSS8X6X3/8	7	10' - 11"	$35.75	$ 390.27	$ 2,731.89
B10	Superstructure	HSS-Hollow Structural Section: HSS8X6X3/8	1	11' - 0"	$35.75	$ 393.25	$ 393.25
B10	Superstructure	W-Wide Flange: W10X12	4	4' - 7 1/8"	$24.27	$ 111.50	$ 446.00
B10	Superstructure	W-Wide Flange: W10X12	4	5' - 1 1/2"	$24.27	$ 124.32	$ 497.28
B10	Superstructure	W-Wide Flange: W10X12	3	6' - 5 1/8"	$24.27	$ 155.99	$ 467.97
B10	Superstructure	W-Wide Flange: W10X12	1	6' - 6 5/8"	$24.27	$ 158.92	$ 158.92
B10	Superstructure	W-Wide Flange: W10X12	8	7' - 1"	$24.27	$ 171.92	$ 1,375.36
B10	Superstructure	W-Wide Flange: W10X12	4	7' - 1 7/8"	$24.27	$ 173.62	$ 694.48
B10	Superstructure	W-Wide Flange: W10X12	1	7' - 3 1/4"	$24.27	$ 176.46	$ 176.46
B10	Superstructure	W-Wide Flange: W10X12	53	7' - 9"	$24.27	$ 187.98	$ 9,962.94
B10	Superstructure	W-Wide Flange: W10X12	44	7' - 9"	$24.27	$ 188.09	$ 8,275.96
B10	Superstructure	W-Wide Flange: W10X12	1	7' - 9"	$24.27	$ 188.14	$ 188.14
B10	Superstructure	W-Wide Flange: W10X12	22	8' - 5"	$24.27	$ 204.28	$ 4,494.16
B10	Superstructure	W-Wide Flange: W10X12	21	8' - 5 3/4"	$24.27	$ 205.88	$ 4,323.48
B10	Superstructure	W-Wide Flange: W10X12	1	9' - 4 3/4"	$24.27	$ 227.95	$ 227.95
B10	Superstructure	W-Wide Flange: W10X12	2	9' - 5 5/8"	$24.27	$ 229.71	$ 459.42
B10	Superstructure	W-Wide Flange: W10X12	3	9' - 8 3/4"	$24.27	$ 236.13	$ 708.39
B10	Superstructure	W-Wide Flange: W10X12	1	10' - 0"	$24.27	$ 242.70	$ 242.70
B10	Superstructure	W-Wide Flange: W10X12	12	10' - 0"	$24.27	$ 242.71	$ 2,912.52
B10	Superstructure	W-Wide Flange: W10X12	12	10' - 2 1/2"	$24.27	$ 247.76	$ 2,973.12
B10	Superstructure	W-Wide Flange: W10X12	8	10' - 5 3/4"	$24.27	$ 254.32	$ 2,034.56
B10	Superstructure	W-Wide Flange: W10X12	1	11' - 3 3/4"	$24.27	$ 274.48	$ 274.48
B10	Superstructure	W-Wide Flange: W10X12	3	12' - 2 3/4"	$24.27	$ 296.80	$ 890.40
B10	Superstructure	W-Wide Flange: W10X12	11	12' - 10 1/4"	$24.27	$ 311.98	$ 3,431.78
B10	Superstructure	W-Wide Flange: W10X12	3	13' - 1 1/4"	$24.27	$ 318.02	$ 954.06
B10	Superstructure	W-Wide Flange: W10X12	6	13' - 4 1/8"	$24.27	$ 323.85	$ 1,943.10
B10	Superstructure	W-Wide Flange: W12X14	2	9' - 11 7/8"	$27.85	$ 278.32	$ 556.64
B10	Superstructure	W-Wide Flange: W12X14	1	10' - 0"	$27.85	$ 278.50	$ 278.50
B10	Superstructure	W-Wide Flange: W12X14	1	12' - 2 1/4"	$27.85	$ 339.42	$ 339.42
B10	Superstructure	W-Wide Flange: W12X14	4	13' - 4 1/8"	$27.85	$ 371.62	$ 1,486.48
B10	Superstructure	W-Wide Flange: W12X14	11	16' - 5 1/2"	$27.85	$ 458.36	$ 5,041.96
B10	Superstructure	W-Wide Flange: W12X19	2	19' - 2 1/4"	$32.10	$ 615.85	$ 1,231.70
B10	Superstructure	W-Wide Flange: W12X19	11	19' - 5 1/2"	$32.10	$ 624.61	$ 6,870.71
B10	Superstructure	W-Wide Flange: W12X19	1	20' - 9 1/2"	$32.10	$ 667.41	$ 667.41
B10	Superstructure	W-Wide Flange: W12X22	4	10' - 0"	$36.35	$ 363.50	$ 1,454.00
B10	Superstructure	W-Wide Flange: W12X22	3	13' - 4 1/8"	$36.35	$ 485.04	$ 1,455.12
B10	Superstructure	W-Wide Flange: W12X22	3	17' - 1 5/8"	$36.35	$ 623.03	$ 1,869.09
B10	Superstructure	W-Wide Flange: W12X22	2	18' - 6 5/8"	$36.35	$ 674.46	$ 1,348.92
B10	Superstructure	W-Wide Flange: W12X22	1	18' - 7 1/2"	$36.35	$ 676.87	$ 676.87
B10	Superstructure	W-Wide Flange: W12X22	1	19' - 2"	$36.35	$ 696.90	$ 696.90
B10	Superstructure	W-Wide Flange: W12X22	21	19' - 4 3/4"	$36.35	$ 704.91	$ 14,803.11
B10	Superstructure	W-Wide Flange: W12X22	37	19' - 5 3/8"	$36.35	$ 707.11	$ 26,163.07
B10	Superstructure	W-Wide Flange: W12X22	40	19' - 5 1/2"	$36.35	$ 707.30	$ 28,292.00
B10	Superstructure	W-Wide Flange: W12X22	146	19' - 5 1/2"	$36.35	$ 707.30	$103,265.80
B10	Superstructure	W-Wide Flange: W12X22	2	19' - 5 5/8"	$36.35	$ 707.61	$ 1,415.22
B10	Superstructure	W-Wide Flange: W12X22	1	20' - 1 1/8"	$36.35	$ 730.34	$ 730.34
B10	Superstructure	W-Wide Flange: W12X22	3	20' - 9 1/2"	$36.35	$ 755.78	$ 2,267.34
B10	Superstructure	W-Wide Flange: W12X26	4	19' - 5 3/8"	$41.85	$ 814.10	$ 3,256.40
B10	Superstructure	W-Wide Flange: W12X26	1	19' - 5 1/2"	$41.85	$ 814.32	$ 814.32
B10	Superstructure	W-Wide Flange: W12X26	2	19' - 5 1/2"	$41.85	$ 814.32	$ 1,628.64
B10	Superstructure	W-Wide Flange: W14X22	2	5' - 5 1/2"	$41.31	$ 225.48	$ 450.96
B10	Superstructure	W-Wide Flange: W14X22	4	6' - 4"	$41.31	$ 261.63	$ 1,046.52
B10	Superstructure	W-Wide Flange: W14X22	2	6' - 8 1/4"	$41.31	$ 276.33	$ 552.66

B10	Superstructure	W-Wide Flange: W14X22	1	6' - 8 3/8"	$41.31	$ 276.75	$ 276.75
B10	Superstructure	W-Wide Flange: W14X22	1	6' - 9"	$41.31	$ 278.84	$ 278.84
B10	Superstructure	W-Wide Flange: W14X22	4	7' - 9"	$41.31	$ 319.96	$ 1,279.84
B10	Superstructure	W-Wide Flange: W14X22	2	8' - 5"	$41.31	$ 347.70	$ 695.40
B10	Superstructure	W-Wide Flange: W14X22	2	8' - 5 3/4"	$41.31	$ 350.43	$ 700.86
B10	Superstructure	W-Wide Flange: W14X22	2	10' - 0"	$41.31	$ 413.10	$ 826.20
B10	Superstructure	W-Wide Flange: W14X22	9	10' - 0"	$41.31	$ 413.11	$ 3,717.99
B10	Superstructure	W-Wide Flange: W14X22	4	10' - 0"	$41.31	$ 413.15	$ 1,652.60
B10	Superstructure	W-Wide Flange: W14X22	1	10' - 5 5/8"	$41.31	$ 432.63	$ 432.63
B10	Superstructure	W-Wide Flange: W14X22	7	10' - 5 3/4"	$41.31	$ 432.87	$ 3,030.09
B10	Superstructure	W-Wide Flange: W14X22	4	10' - 8"	$41.31	$ 440.64	$ 1,762.56
B10	Superstructure	W-Wide Flange: W14X22	4	10' - 11"	$41.31	$ 450.86	$ 1,803.44
B10	Superstructure	W-Wide Flange: W14X22	4	10' - 11"	$41.31	$ 450.97	$ 1,803.88
B10	Superstructure	W-Wide Flange: W14X22	1	11' - 0"	$41.31	$ 454.41	$ 454.41
B10	Superstructure	W-Wide Flange: W14X22	4	12' - 10 3/8"	$41.31	$ 531.56	$ 2,126.24
B10	Superstructure	W-Wide Flange: W14X22	2	13' - 2 1/4"	$41.31	$ 544.99	$ 1,089.98
B10	Superstructure	W-Wide Flange: W14X22	1	13' - 2 3/8"	$41.31	$ 545.27	$ 545.27
B10	Superstructure	W-Wide Flange: W14X22	4	13' - 7 1/4"	$41.31	$ 561.85	$ 2,247.40
B10	Superstructure	W-Wide Flange: W14X22	2	16' - 3 1/2"	$41.31	$ 672.92	$ 1,345.84
B10	Superstructure	W-Wide Flange: W14X22	2	16' - 3 1/2"	$41.31	$ 673.11	$ 1,346.22
B10	Superstructure	W-Wide Flange: W14X22	5	16' - 5 1/2"	$41.31	$ 679.88	$ 3,399.40
B10	Superstructure	W-Wide Flange: W14X22	4	17' - 0"	$41.31	$ 702.27	$ 2,809.08
B10	Superstructure	W-Wide Flange: W14X22	4	17' - 4"	$41.31	$ 716.15	$ 2,864.60
B10	Superstructure	W-Wide Flange: W14X22	2	18' - 0 3/4"	$41.31	$ 746.01	$ 1,492.02
B10	Superstructure	W-Wide Flange: W14X22	2	18' - 1 1/2"	$41.31	$ 748.74	$ 1,497.48
B10	Superstructure	W-Wide Flange: W14X22	9	19' - 5 1/2"	$41.31	$ 803.82	$ 7,234.38
B10	Superstructure	W-Wide Flange: W14X22	8	20' - 0"	$41.31	$ 826.08	$ 6,608.64
B10	Superstructure	W-Wide Flange: W14X22	28	20' - 0"	$41.31	$ 826.08	$ 23,130.24
B10	Superstructure	W-Wide Flange: W14X22	8	20' - 0"	$41.31	$ 826.23	$ 6,609.84
B10	Superstructure	W-Wide Flange: W14X22	8	20' - 0"	$41.31	$ 826.31	$ 6,610.48
B10	Superstructure	W-Wide Flange: W14X22	8	20' - 0"	$41.31	$ 826.41	$ 6,611.28
B10	Superstructure	W-Wide Flange: W14X22	7	20' - 5 1/2"	$41.31	$ 845.13	$ 5,915.91
B10	Superstructure	W-Wide Flange: W14X22	4	20' - 10 1/2"	$41.31	$ 862.35	$ 3,449.40
B10	Superstructure	W-Wide Flange: W14X22	7	21' - 4"	$41.31	$ 881.24	$ 6,168.68
B10	Superstructure	W-Wide Flange: W14X22	27	21' - 4"	$41.31	$ 881.28	$ 23,794.56
B10	Superstructure	W-Wide Flange: W14X22	8	21' - 4"	$41.31	$ 881.39	$ 7,051.12
B10	Superstructure	W-Wide Flange: W14X22	8	21' - 4"	$41.31	$ 881.40	$ 7,051.20
B10	Superstructure	W-Wide Flange: W14X22	1	21' - 5 7/8"	$41.31	$ 887.78	$ 887.78
B10	Superstructure	W-Wide Flange: W14X22	8	21' - 8"	$41.31	$ 895.05	$ 7,160.40
B10	Superstructure	W-Wide Flange: W14X22	1	21' - 9"	$41.31	$ 898.66	$ 898.66
B10	Superstructure	W-Wide Flange: W14X22	4	22' - 4"	$41.31	$ 922.59	$ 3,690.36
B10	Superstructure	W-Wide Flange: W14X22	4	23' - 4"	$41.31	$ 963.90	$ 3,855.60
B10	Superstructure	W-Wide Flange: W14X22	3	23' - 6 1/2"	$41.31	$ 972.50	$ 2,917.50
B10	Superstructure	W-Wide Flange: W14X22	2	24' - 6 3/8"	$41.31	$ 1,013.33	$ 2,026.66
B10	Superstructure	W-Wide Flange: W14X22	1	24' - 7"	$41.31	$ 1,015.36	$ 1,015.36
B10	Superstructure	W-Wide Flange: W14X26	1	13' - 2 3/8"	$41.31	$ 545.27	$ 545.27
B10	Superstructure	W-Wide Flange: W14X26	7	19' - 5 1/2"	$41.31	$ 803.82	$ 5,626.74
B10	Superstructure	W-Wide Flange: W14X26	1	20' - 5 1/2"	$41.31	$ 845.13	$ 845.13
B10	Superstructure	W-Wide Flange: W14X26	1	23' - 6 1/2"	$41.31	$ 972.50	$ 972.50
B10	Superstructure	W-Wide Flange: W14X26	8	24' - 4"	$41.31	$ 1,005.21	$ 8,041.68
B10	Superstructure	W-Wide Flange: W14X26	1	24' - 7"	$41.31	$ 1,015.36	$ 1,015.36
B10	Superstructure	W-Wide Flange: W14X30	4	14' - 4"	$47.74	$ 684.13	$ 2,736.52
B10	Superstructure	W-Wide Flange: W14X30	5	19' - 5 1/2"	$47.74	$ 928.93	$ 4,644.65
B10	Superstructure	W-Wide Flange: W16X26	1	9' - 4 3/4"	$41.27	$ 387.62	$ 387.62
B10	Superstructure	W-Wide Flange: W16X26	1	17' - 1 5/8"	$41.27	$ 707.36	$ 707.36
B10	Superstructure	W-Wide Flange: W16X26	4	17' - 4"	$41.27	$ 715.46	$ 2,861.84
B10	Superstructure	W-Wide Flange: W16X26	1	18' - 7 1/2"	$41.27	$ 768.48	$ 768.48
B10	Superstructure	W-Wide Flange: W16X26	2	19' - 5 1/2"	$41.27	$ 803.05	$ 1,606.09
B10	Superstructure	W-Wide Flange: W16X26	8	20' - 0"	$41.27	$ 825.28	$ 6,602.24
B10	Superstructure	W-Wide Flange: W16X26	28	20' - 0"	$41.27	$ 825.28	$ 23,107.84

B10	Superstructure	W-Wide Flange: W16X26	8	20' - 0"	$41.27	$ 825.43	$ 6,603.44
B10	Superstructure	W-Wide Flange: W16X26	6	20' - 0"	$41.27	$ 825.51	$ 4,953.06
B10	Superstructure	W-Wide Flange: W16X26	6	20' - 0"	$41.27	$ 825.61	$ 4,953.66
B10	Superstructure	W-Wide Flange: W16X26	8	21' - 4"	$41.27	$ 880.39	$ 7,043.12
B10	Superstructure	W-Wide Flange: W16X26	23	21' - 4"	$41.27	$ 880.43	$ 20,249.89
B10	Superstructure	W-Wide Flange: W16X26	6	21' - 4"	$41.27	$ 880.54	$ 5,283.24
B10	Superstructure	W-Wide Flange: W16X26	8	21' - 4"	$41.27	$ 880.55	$ 7,044.40
B10	Superstructure	W-Wide Flange: W16X26	8	21' - 8"	$41.27	$ 894.18	$ 7,153.44
B10	Superstructure	W-Wide Flange: W16X26	3	22' - 4"	$41.27	$ 921.70	$ 2,765.10
B10	Superstructure	W-Wide Flange: W16X26	1	22' - 4 1/2"	$41.27	$ 923.50	$ 923.50
B10	Superstructure	W-Wide Flange: W16X26	4	23' - 4"	$41.27	$ 962.97	$ 3,851.88
B10	Superstructure	W-Wide Flange: W16X31	8	19' - 5 3/8"	$49.24	$ 957.85	$ 7,662.80
B10	Superstructure	W-Wide Flange: W16X31	8	19' - 5 1/2"	$49.24	$ 958.12	$ 7,664.96
B10	Superstructure	W-Wide Flange: W16X31	2	20' - 0"	$49.24	$ 984.93	$ 1,969.86
B10	Superstructure	W-Wide Flange: W16X31	2	20' - 0"	$49.24	$ 985.06	$ 1,970.12
B10	Superstructure	W-Wide Flange: W16X31	1	21' - 4"	$49.24	$ 1,050.59	$ 1,050.59
B10	Superstructure	W-Wide Flange: W16X31	2	22' - 11 3/8"	$49.24	$ 1,129.78	$ 2,259.56
B10	Superstructure	W-Wide Flange: W16X31	2	22' - 11 3/8"	$49.24	$ 1,130.17	$ 2,260.34
B10	Superstructure	W-Wide Flange: W16X31	8	24' - 4"	$49.24	$ 1,198.17	$ 9,585.36
B10	Superstructure	W-Wide Flange: W16X36	3	19' - 5 1/2"	$62.34	$ 1,213.02	$ 3,639.06
B10	Superstructure	W-Wide Flange: W16X36	5	21' - 4"	$62.34	$ 1,329.92	$ 6,649.60
B10	Superstructure	W-Wide Flange: W16X36	1	21' - 4"	$62.34	$ 1,330.09	$ 1,330.09
						TOTAL	$763,010.79

APPENDIX C.10 SPREADSHEET OF COLUMNS AND COST FOR STEEL DESIGN

Structural Column Schedule							
Assembly Code	Assembly Description	Family and Type	Count	Length	Cost/LF	Cost of 1 @ length	Total Cost
B10	Superstructure	HSS-Hollow Structural Section-Column: HSS6X6X1/2	10	5' - 4"	$ 35.75	$ 190.67	$ 1,906.70
B10	Superstructure	HSS-Hollow Structural Section-Column: HSS6X6X1/2	4	15' - 4"	$ 35.75	$ 548.17	$ 2,192.68
B10	Superstructure	HSS-Hollow Structural Section-Column: HSS12X6X1/2	14	15' - 4"	$ 93.06	$ 1,426.92	$ 19,976.88
B10	Superstructure	W-Wide Flange-Column: W10X54	101	20' - 0"	$ 101.34	$ 2,026.80	$ 204,706.80
B10	Superstructure	W-Wide Flange-Column: W10X54	108	30' - 8"	$ 101.34	$ 3,107.76	$ 335,638.08
B10	Superstructure	W-Wide Flange-Column: W10X54	4	35' - 4"	$ 101.34	$ 3,580.68	$ 14,322.72

Total Cost	$ 578,743.86

APPENDIX C.11 SPRINKLER DESIGN CALCULATION SPREADSHEET

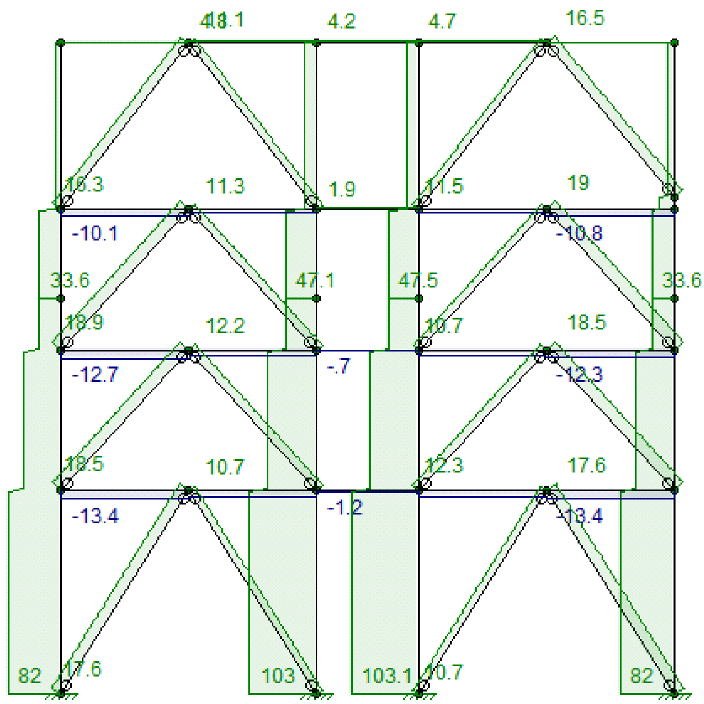
	Categoty	Classification	Notes
Occupancy Classufication	Occupancy Category	Residential (R-2)	IBC 2009 Section 310.1
	Hazard Classification	Light Hazard	NFPA 13 2012 Section A.5.2
Commodity Classification	Alcoholic beverages in glass or metal containers	Class I	NFPA 13 2012 Table A.5.6.3
	Furniture-Wood	Class III	NFPA 13 2012 Table A.5.6.4
	Liquor-Glass Bottles	Class IV	NFPA 13 2012 Table A.5.6.5
	Liquor-Plasting Bottles	Class IV	NFPA 13 2012 Table A.5.6.6
	Mattresses-Standard Box Spring	Class III	NFPA 13 2012 Table A.5.6.7
	Wood Products-Doors, windows, wood cabinets and furniture	Class III	NFPA 13 2012 Table A.5.6.8
Design Area	Area of Sprinkler Operation	1,500 ft^2	NFPA 13 2012 Exhibit 5.4
	Sprinklers in Area	7	Area of Sprinkler Operation/Protection Area
	Sprinklers Along Branch Line	4	[1.2(Design Area)$^{(1/2)}$]/Spacing
System Criteraia	System Type	Wet Pipe	
	Protection Area Limitations	52,000 ft^2	NFPA 13 2012 Section 8.2.1
	Number of systems	1	NFPA 13 2012 Section 8.2.2
Sprinkler Classifications	Protection Area	225 ft^2	NFPA 13 2012 Table 8.6.2.2.1 (a)
	Spacing	15 ft	NFPA 13 2012 Table 8.6.2.2.1 (a)
	Sprinkler Type	Upright	NFPA 13 2012 Section 8.4.1.1
		Pendant	NFPA 13 2012 Section 8.4.1.2
		Sidewall	NFPA 13 2012 Section 8.4.2
	Design Density	0.1 gpm/ft^2	NFPA 13 2012 Exhibit 5.5
	K Factor	5.6	
	Remote Flow	22.5	(Protection Area)(Design Density)
	Discharge Pressure	16.1	(Remote Flow/K Factor)^2
	Temperature Classification	Ordinary (135-170)	NFPA 13 2012 Table 6.2.5.1
Pipe Specifications	Schedule	40	
	C Factor	120	

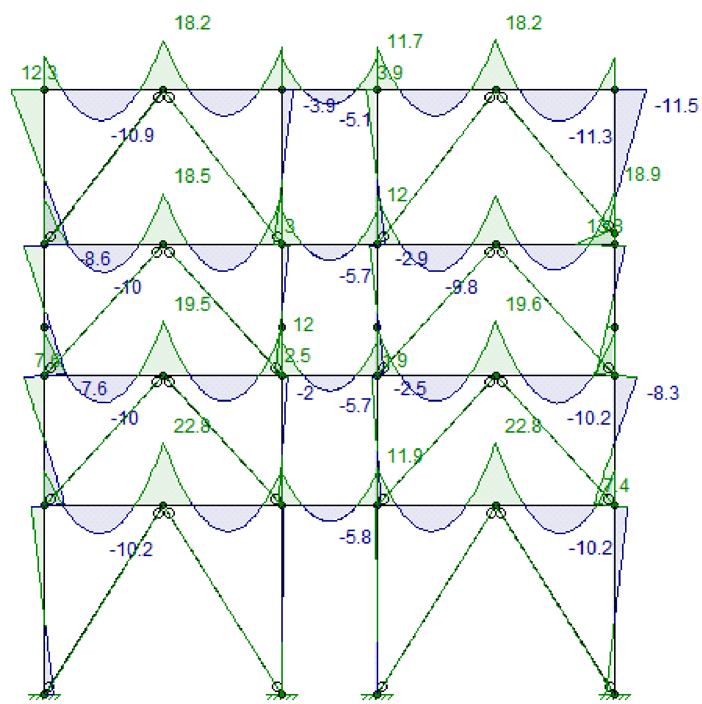
Equipment Number	Flow (gpm)		Nominal Pipe Size	Pipe Diameter (in)	Fittings	Equivalent Pipe Length (ft)		Friction Loss (psi/ft)	Required Pressure (psi)		
Sprinkler 1 to Sprinkler 2	q	22.5				Pipe	13.33333		Pt	16.1	
	Q	22.5	1	1.049	N/A	Fittings	0	0.16181328	Pe	0	
						Total	13.33333		Pf	2.15751	
Sprinkler 2 to Hydraulic Junction Point 1	q	23.92813252				Pipe	6.666667		Pt	18.25751	
	Q	46.42813252	1	1.049	N/A	Fittings	0	0.618047353	Pe	0	
						Total	6.666667		Pf	4.120316	
									Pt	**22.37783**	Pressure from right side of branch line
Sprinkler 3 to Hydraulic Junction Point 1	q	22.5				Pipe	6.666667		Pt	16.1	
	Q	22.5	1	1.049	N/A	Fittings	0	0.16181328	Pe	0	
						Total	6.666667		Pf	1.078755	
									Pt	**17.17876**	Pressure from left side of branch line
Sprinkler 3 Adjustment	k	5.428585379									
	q	25.6800355									
Hydraulic Junction Point 1 to Sprinkler 4						Pipe	6.77085		Pt	22.37783	
	Q	72.10816802	1.5	1.61	1 Tee	Fittings	5	0.17325364	Pe	0	
						Total	11.77085		Pf	2.039343	
Sprinkler 4 to Hydraulic Junction Point 2	q	27.67168969				Pipe	10.052		Pt	24.41717	
	Q	**99.77985771**	1.5	1.61	1 Tee	Fittings	5	0.315966065	Pe	0	
						Total	15.052		Pf	4.755921	
									Pt	**29.17309**	Pressure from left side of HJP2
Sprinkler 5 to Sprinkler 6	q	22.5				Pipe	13.33333		Pt	16.1	
	Q	22.5	1	1.049	N/A	Fittings	0	0.16181328	Pe	0	
						Total	13.33333		Pf	2.15751	
Sprinkler 6 to Sprinkler 7	q	23.92813252				Pipe	13.33333		Pt	18.25751	
	Q	46.42813252	1	1.049	N/A	Fittings	0	0.618047353	Pe	0	
						Total	13.33333		Pf	8.240631	
Sprinkler 7 to Hydraulic Junction Point 2	q	28.82675365				Pipe	26.615		Pt	26.49814	
	Q	75.25488617	1.5	1.61	1 Elbow	Fittings	4	0.187499586	Pe	0	
						Total	30.615		Pf	5.7403	
									Pt	**32.23844**	Pressure from right side of HJP2
Hydraulic Junction Point 2 Adjustment	k	18.4736055									
	q	104.8911109									
Hydraulic Junction Point to top of riser					1 Tee 1	Pipe	408.99		Pt	32.23844	
	Q	180.1459971	1.5	1.61	Elbow 1	Fittings	14	0.942576599	Pe	0	
					Sweep	Total	422.99		Pf	398.7005	
Top of riser to bottom of riser					Swing	Pipe	45		Pt	430.9389	
	Q	**180.1459971**	3	3.068	Check	Fittings	17	0.040791821	Pe	7.361	
					and Gate	Total	62		Pf	2.529093	
									Pt	**440.829**	

APPENDIX D.1 VARIOUS *RISA-2D* LOAD AND FRAME ANALYSES

Frame G

Load Combination 2





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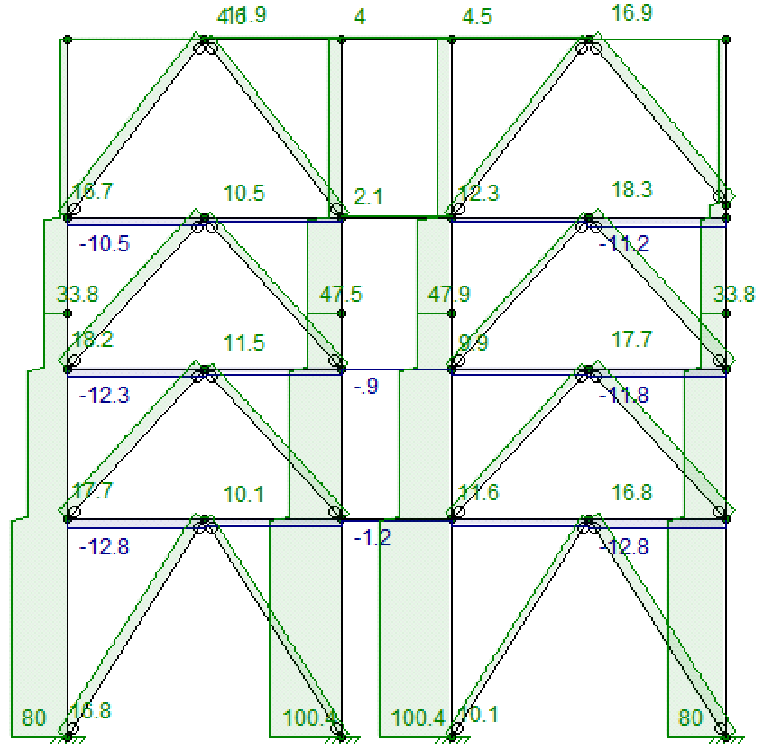

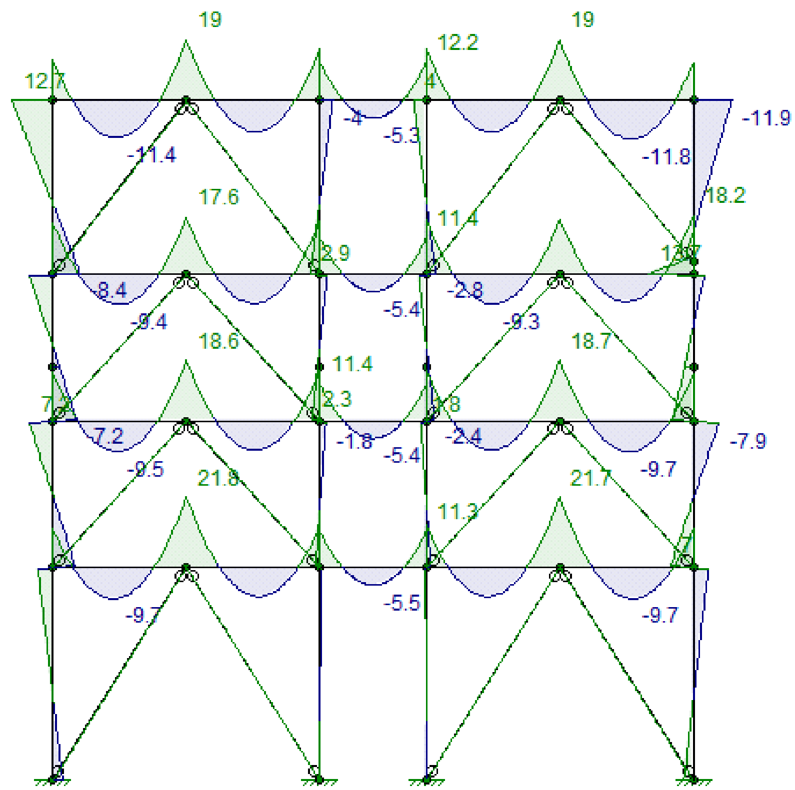

Frame S

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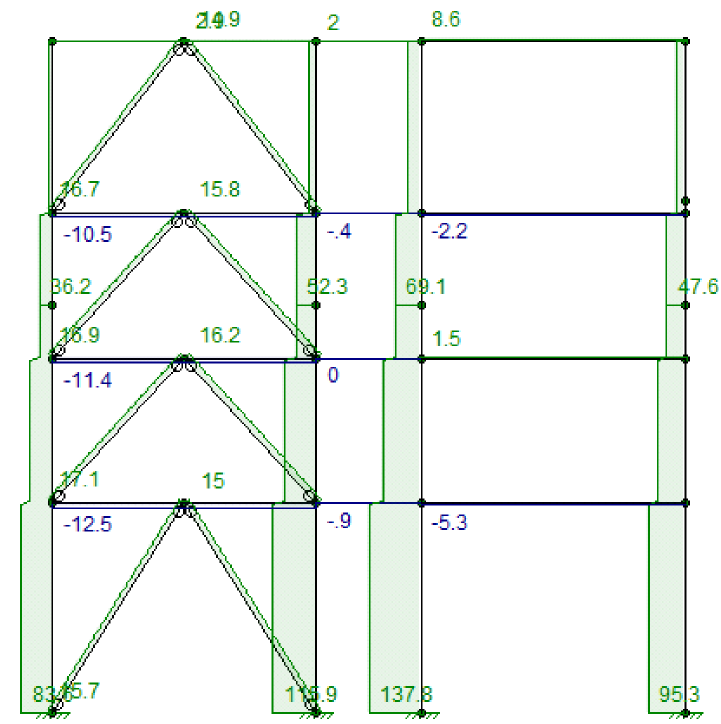

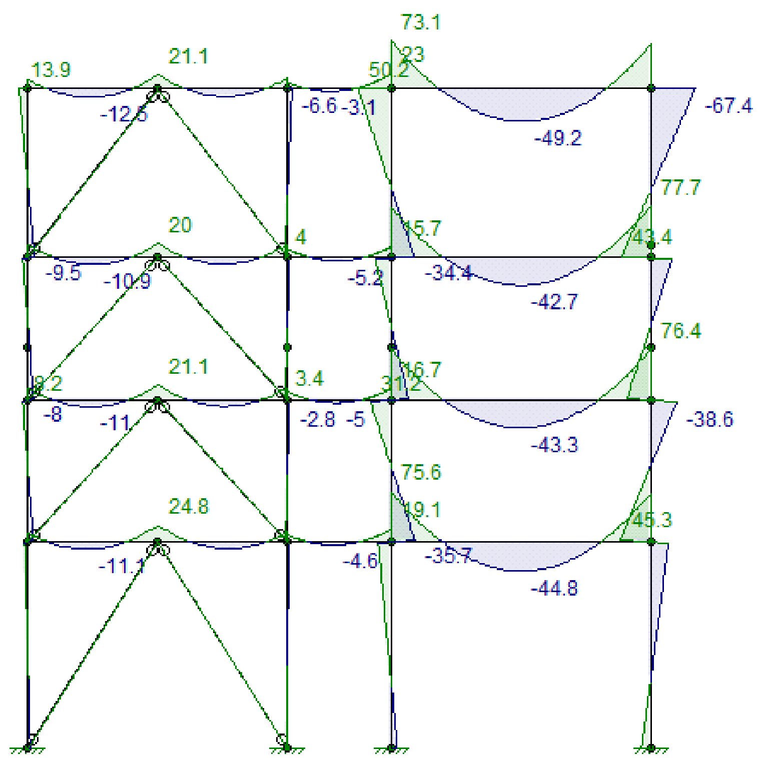

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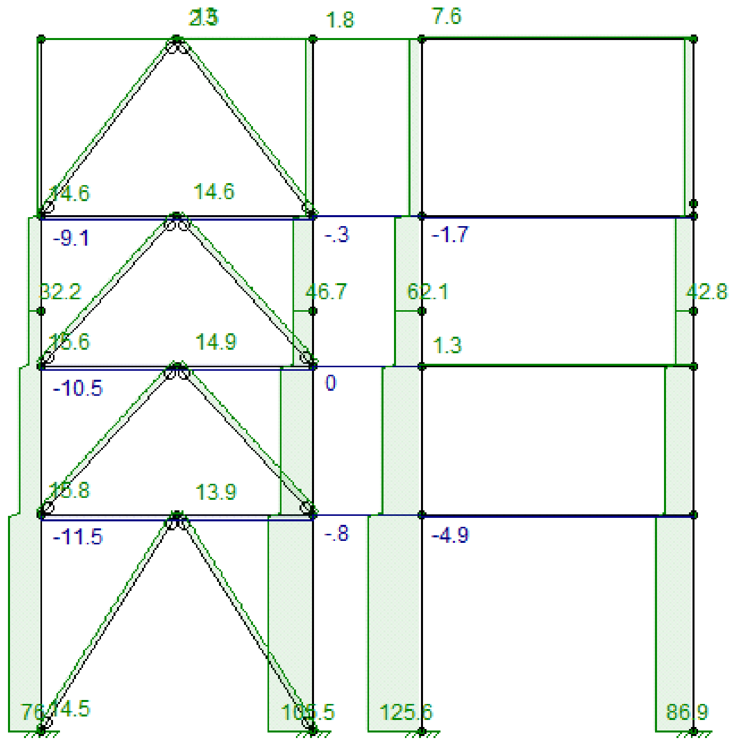

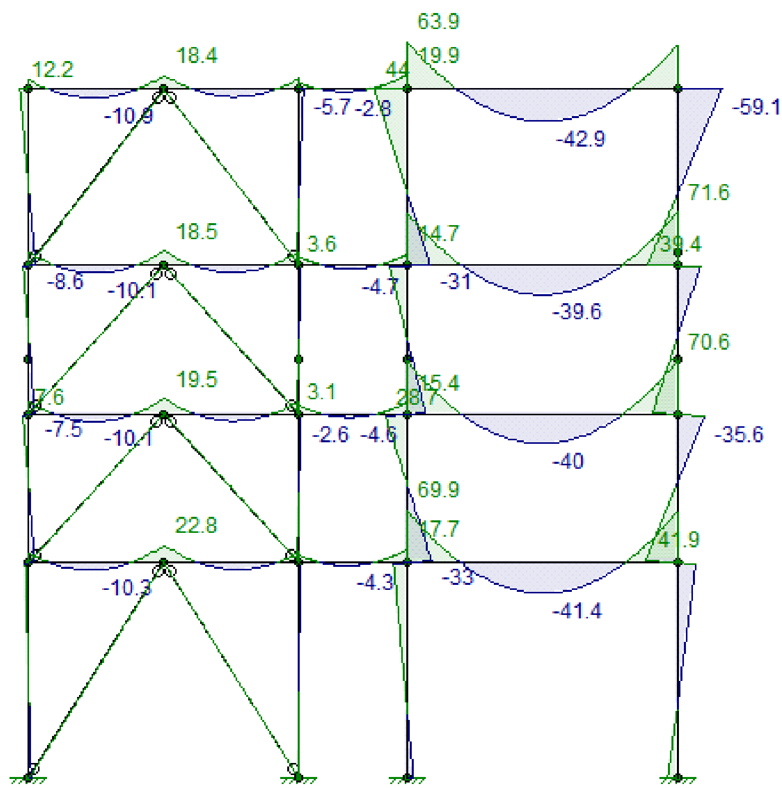

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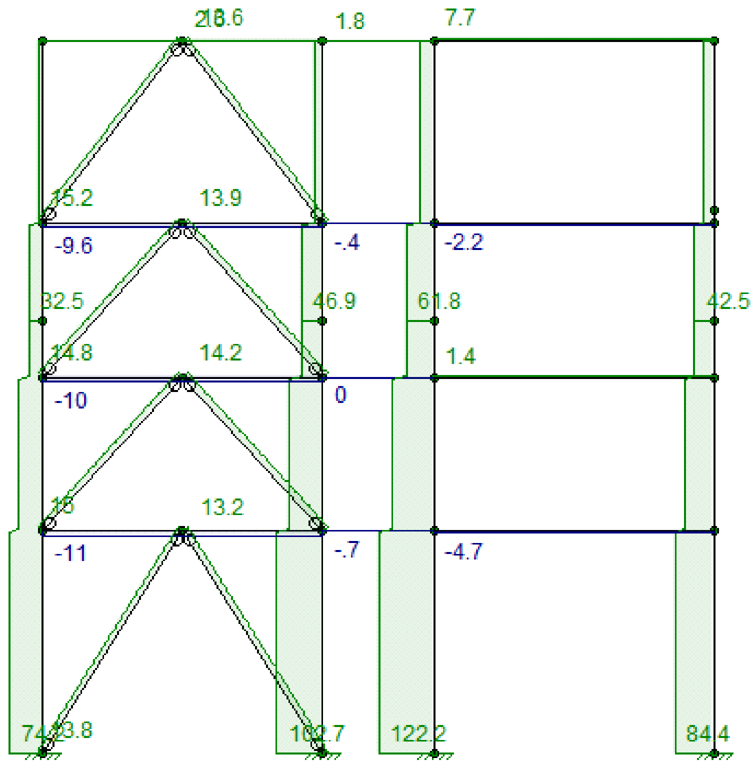

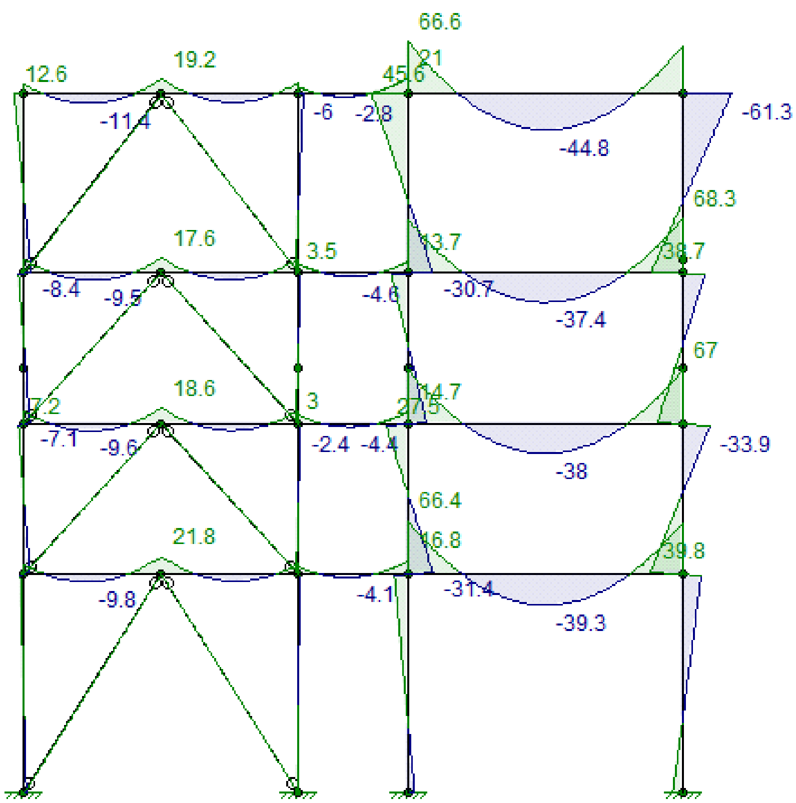

Frame B

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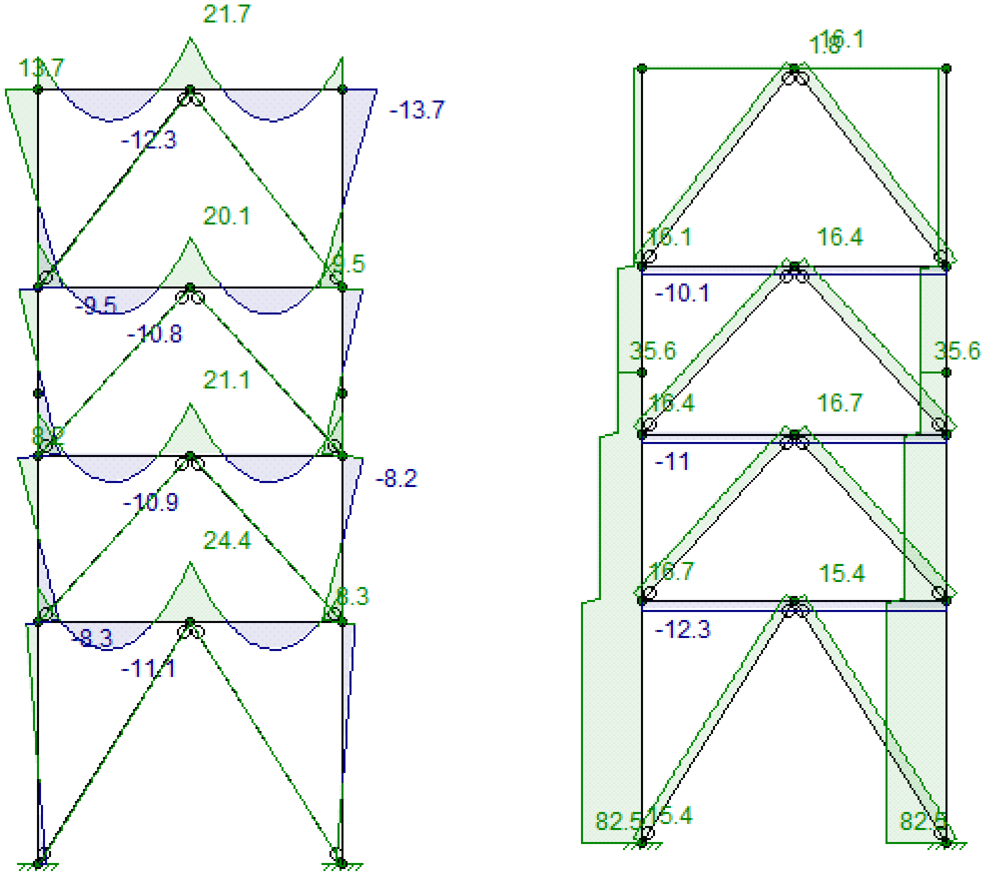

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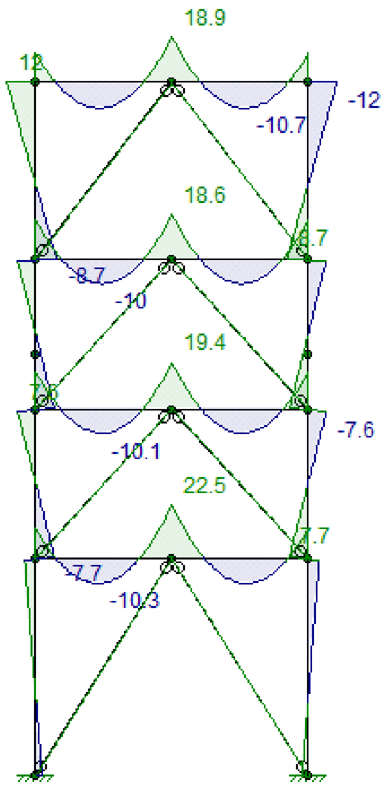

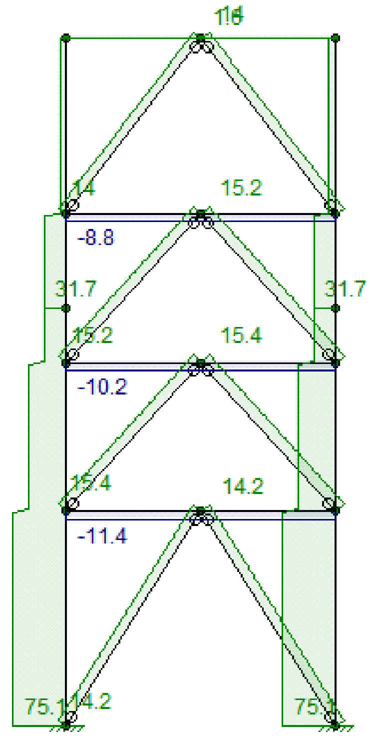

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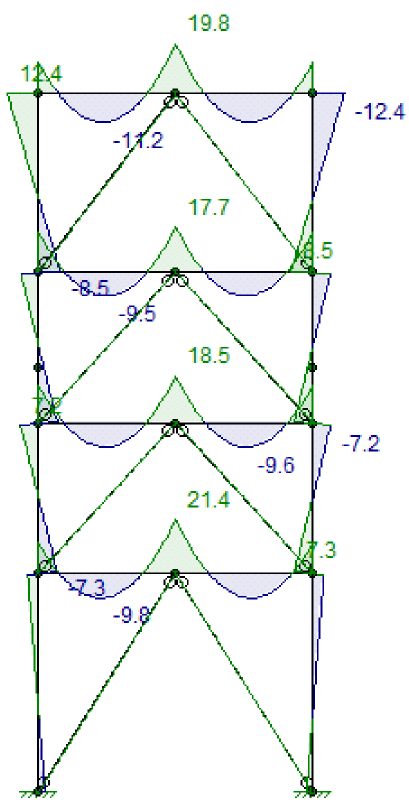
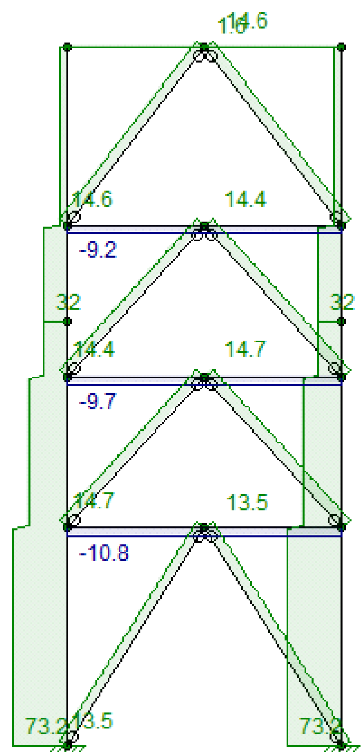


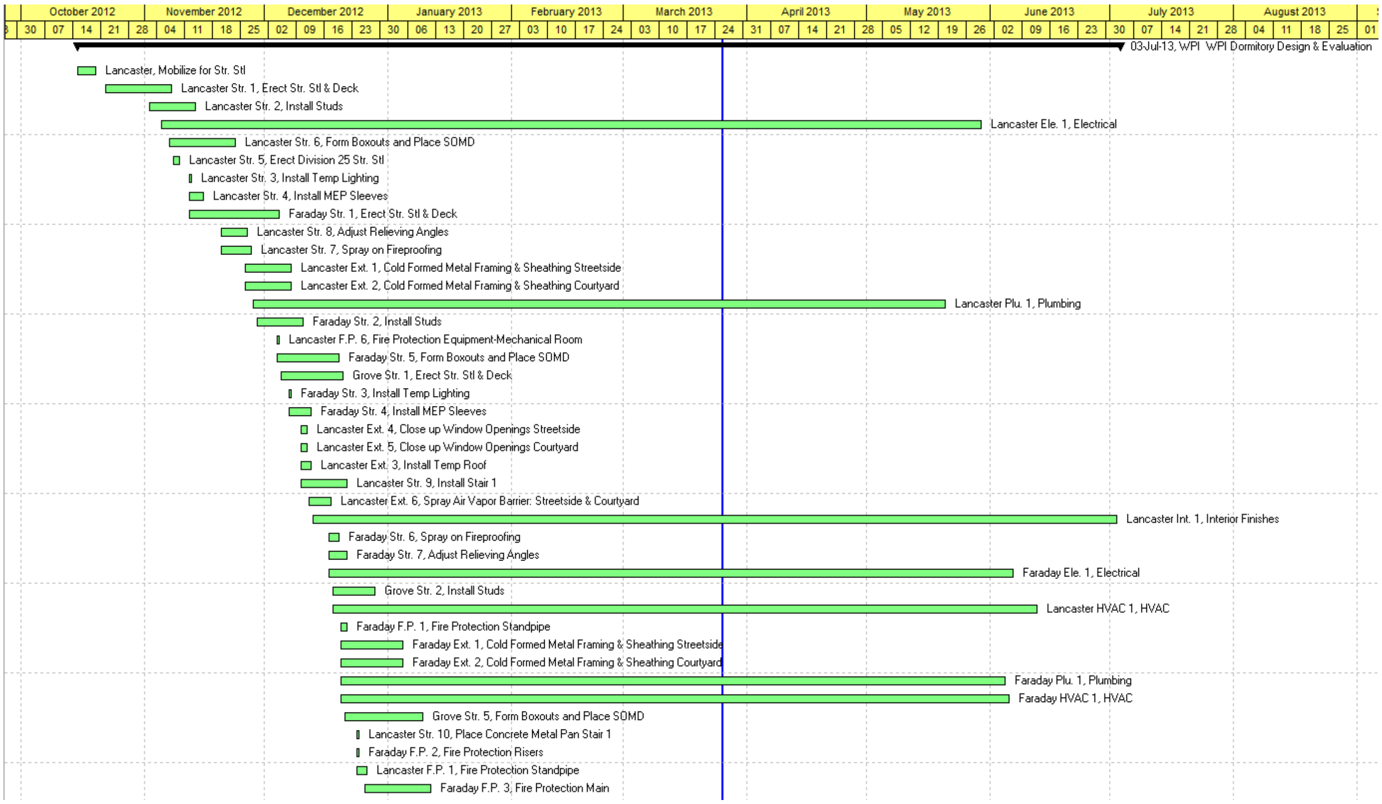


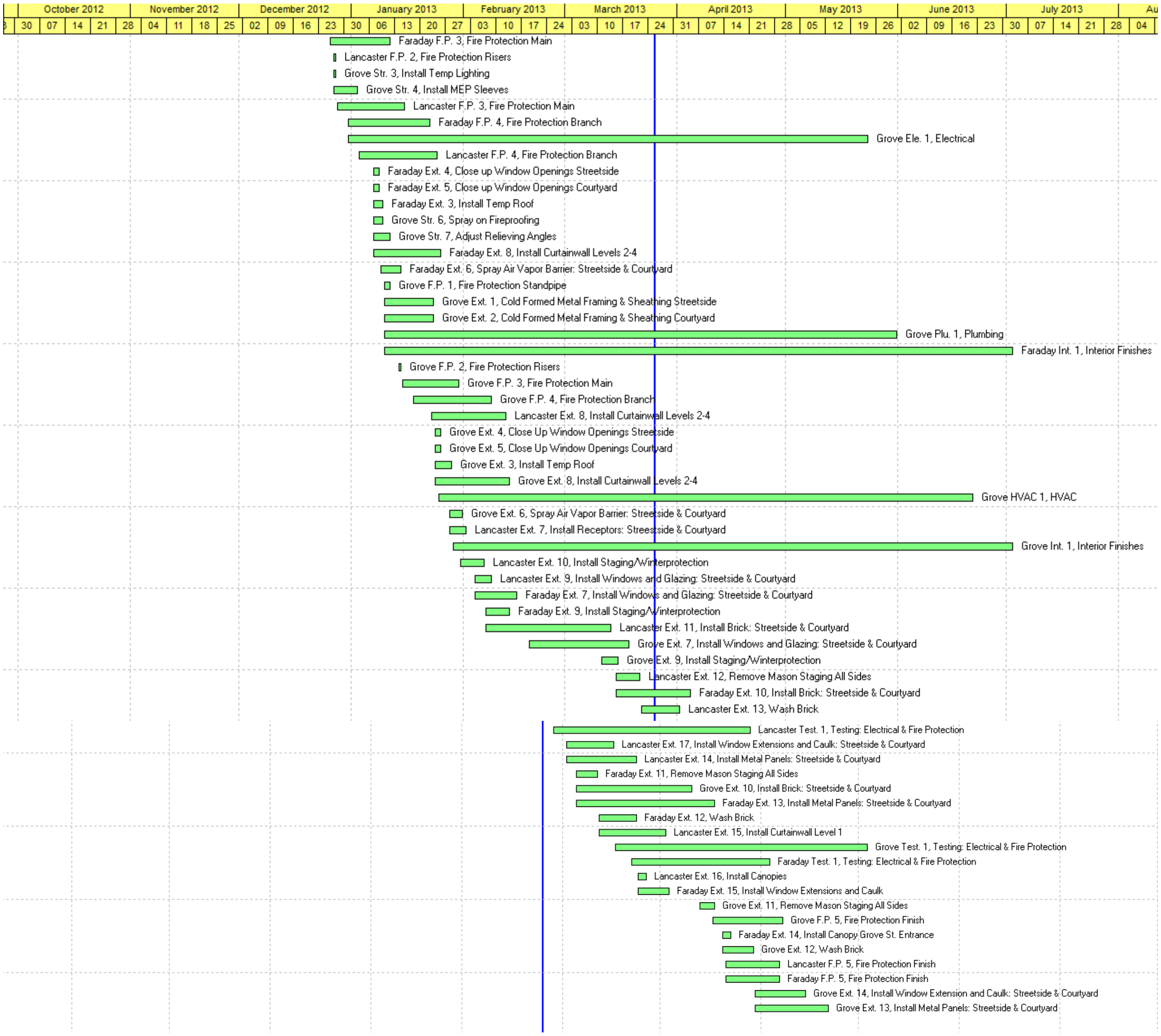
APPENDIX D.2 LEED CREDITS CHECKLIST

LEED Credits	Points Available	Points Attained
Sustainable Sites (SS)		
Site Selection	1	1
Development Density & Community Connectivity	5	5
Alternative Transportation		
Public Transportation Access	6	6
Site Development		
5.1 Protect or Restore Habitat	1	1
5.2 Maximize Open Space	1	1
Heat Island Effect		
7.1 Non-Roof	1	1
7.2 Roof	1	1
Water Efficiency (WE)		
Water Efficient Landscaping		
1.1 Reduce by 50%	2	2
Water Use Reduction		
3.1-30% Reduction (from baseline water use)	2	2
Energy and Atmosphere (EA)		
Optimize Energy Performance		
Exceed ASHRAE 90.1 by 12% New bldg / 8% Exist. Bldg	1	1
Exceed ASHRAE 90.1 by 14% New bldg / 10% Exist. Bldg	1	1
Exceed ASHRAE 90.1 by 16% New bldg / 12% Exist. Bldg	1	1
Exceed ASHRAE 90.1 by 18% New bldg / 14% Exist. Bldg	1	1
Exceed ASHRAE 90.1 by 20% New bldg / 16% Exist. Bldg	1	1
Exceed ASHRAE 90.1 by 22% New bldg / 18% Exist. Bldg	1	1

Materials and Resources (MR)		
Construction Waste Management		
50 % Recycled or Salvaged	1	1
75% Recycled or Salvaged	1	1
Recycled Content		
10% by cost (post-consumer + 1/2 pre-consumer)	1	1
Regional Materials		
10% (by cost) Extracted, Processed & Manufactured Regionally	1	1
Certified Wood	1	1
Indoor Environmental Quality (IEQ)		
Construction IAQ Management Plan		
3.1 During Construction (SMACNA 2008)	1	1
Low-Emitting Materials		
4.1 Adhesives & Sealants (VOC limits)	1	1
4.2 Paints & Coatings (VOC limits)	1	1
4.3 Flooring Systems (CRI Green Label program and	1	1
4.4 Composite Wood & Agrifiber Products (no added ureaformaldehyde resins)	1	1
Controllability of Systems		
6.1 Lighting (individual controls for 90% of occupants)	1	1
6.2 Thermal Comfort (individual controls for 50% of occupants)	1	1
Daylight and Views		
8.2 Views for 90% of Regularly Occupied Areas (direct line of sight via vision glazing)	1	1
Innovation and Design Process (ID)		
Innovation in Design		
1.1 Innovation credit	1	1
LEED Accredited Professional	1	1
Regional Priority Credits (RP)		
1.2 Regional Priority: Specific Credit	1	1
1.3 Regional Priority: Specific Credit	1	1
Grand Total LEED Points		**43**

APPENDIX D.3 COMPLETE *PRIMAVERA* CONSTRUCTION SCHEDULE




October 2012
November 2012
December 2012
January 2013
February 2013
March 2013
April 2013
May 2013
June 2013
July 2013
Au
30 07 14 21 28 04 11 18 25 02 09 16 23 30 06 13 20 27 03 10 17 24 03 10 17 24 31 07 14 21 28 05 12 19 26 02 09 16 23 30 07 14 21 28 04
Faraday F.P. 3, Fire Protection Main
Lancaster F.P. 2, Fire Protection Risers
Grove Str. 3, Install Temp Lighting
Grove Str. 4, Install MEP Sleeves
Lancaster F.P. 3, Fire Protection Main
Faraday F.P. 4, Fire Protection Branch
Grove Ele. 1, Electrical
Lancaster F.P. 4, Fire Protection Branch
Faraday Ext. 4, Close up Window Openings Streetside
Faraday Ext. 5, Close up Window Openings Courtyard
Faraday Ext. 3, Install Temp Roof
Grove Str. 6, Spray on Fireproofing
Grove Str. 7, Adjust Relieving Angles
Faraday Ext. 8, Install Curtainwall Levels 2-4
Faraday Ext. 6, Spray Air Vapor Barrier: Streetside & Courtyard
Grove F.P. 1, Fire Protection Standpipe
Grove Ext. 1, Cold Formed Metal Framing & Sheathing Streetside
Grove Ext. 2, Cold Formed Metal Framing & Sheathing Courtyard
Grove Plu. 1, Plumbing
Faraday Int. 1, Interior Finishes
Grove F.P. 2, Fire Protection Risers
Grove F.P. 3, Fire Protection Main
Grove F.P. 4, Fire Protection Branch
Lancaster Ext. 8, Install Curtainwall Levels 2-4
Grove Ext. 4, Close Up Window Openings Streetside
Grove Ext. 5, Close Up Window Openings Courtyard
Grove Ext. 3, Install Temp Roof
Grove Ext. 8, Install Curtainwall Levels 2-4
Grove HVAC 1, HVAC
Grove Ext. 6, Spray Air Vapor Barrier: Streetside & Courtyard
Lancaster Ext. 7, Install Receptors: Streetside & Courtyard
Grove Int. 1, Interior Finishes
Lancaster Ext. 10, Install Staging/Winterprotection
Lancaster Ext. 9, Install Windows and Glazing: Streetside & Courtyard
Faraday Ext. 7, Install Windows and Glazing: Streetside & Courtyard
Faraday Ext. 9, Install Staging/Winterprotection
Lancaster Ext. 11, Install Brick: Streetside & Courtyard
Grove Ext. 7, Install Windows and Glazing: Streetside & Courtyard
Grove Ext. 9, Install Staging/Winterprotection
Lancaster Ext. 12, Remove Mason Staging All Sides
Faraday Ext. 10, Install Brick: Streetside & Courtyard
Lancaster Ext. 13, Wash Brick
Lancaster Test. 1, Testing: Electrical & Fire Protection
Lancaster Ext. 17, Install Window Extensions and Caulk: Streetside & Courtyard
Lancaster Ext. 14, Install Metal Panels: Streetside & Courtyard
Faraday Ext. 11, Remove Mason Staging All Sides
Grove Ext. 10, Install Brick: Streetside & Courtyard
Faraday Ext. 13, Install Metal Panels: Streetside & Courtyard
Faraday Ext. 12, Wash Brick
Lancaster Ext. 15, Install Curtainwall Level 1
Grove Test. 1, Testing: Electrical & Fire Protection
Faraday Test. 1, Testing: Electrical & Fire Protection
Lancaster Ext. 16, Install Canopies
Faraday Ext. 15, Install Window Extensions and Caulk
Grove Ext. 11, Remove Mason Staging All Sides
Grove F.P. 5, Fire Protection Finish
Faraday Ext. 14, Install Canopy Grove St. Entrance
Grove Ext. 12, Wash Brick
Lancaster F.P. 5, Fire Protection Finish
Faraday F.P. 5, Fire Protection Finish
Grove Ext. 14, Install Window Extension and Caulk: Streetside & Courtyard
Grove Ext. 13, Install Metal Panels: Streetside & Courtyard

APPENDIX D.4 SCHEDULE DATA

	Duration				
Wing	**Lancaster**	**Faraday**	**Grove**	**Total**	**Percentage**
Superstructure	66	49	45	160	7%
Exterior Enclosure	140	129	129	398	18%
Interiors	139	121	108	368	17%
HVAC	122	115	103	340	15%
Plumbing	120	114	99	333	15%
Fire Protection	64	58	72	194	9%
Electrical	163	130	129	422	19%
				2215	