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MQPSL1-VOKE

DEVELOPMENT OF 32 PRESCOTT STREET AT GATEWAY PARK

A Major Qualifying Project

Submitted to the Faculty

of the

WORCESTER POLYTECHNIC INSTITUTE

in partial fulfillment of the requirements for the

Degree of Bachelor of Science

In Civil Engineering

By

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Abstract

Gateway Park at WPI is a mixed-use complex for life sciences and biotechnology companies. The goal of this MQP was to investigate, design, and analyze a proposed mixed-use development that will be located at Gateway Park WPI. The proposed facility will serve as: office and industrial space for new life science companies, retail space, and graduate or upper-class housing. This MQP presents: a functional layout and floor plans, a structural analysis, an evaluation of the impact on existing traffic and parking conditions, an overview of obtaining Gold LEED certification, and a cost estimate.

Acknowledgements

Although there were only three people who worked on writing this project directly, there were many more who worked behind the scenes and without their selfless support we could not have completed the project. First, we would like to express a sincere thank you to John Weaver and Bill Carkin of the Worcester Business Development Corporation. The folks at the WBDC were always willing to give us a hand, providing many drawings and plans as well as the *Gateway Master Plan* document that would prove pivotal for project. It was with their help, that we found this MQP. Next, we would like to thank WPI's President Berkey for answering questions regarding Gateway Park by means of email in a timely manner. Jeff Solomon and Alfredo DiMauro, of WPI, were also a great help and provided our MQP team with valuable insight of WPI's goals and direction with the Gateway Park development. For technical assistance, we would like to thank WPI's Professor Mathisen, Professor Salazar, and Professor Tao for their expertise. We would also like to thank Brad Davis of the *AISC Solutions Center* for answering various steel design questions. Most importantly, we would like to express our gratitude towards our MQP advisors, Professor Albano and Professor LePage. Together they have provided numerous hours of guidance and direction, along with the countless hours they have spent reading all the revisions of our report. The advisors were always willing and able to answer our technical questions, and they kept an open door at their office so that even the quickest of questions could be addressed in a timely manner.

Authorship

Michael O'Brien

Michael O. was responsible for writing the Capstone Design Statement. For the background of the report, he wrote the sections on the Transformation of Prescott-Grove Street District to Gateway Park, Gateway Park Today, *MASTAN2*, and *AMLink Material Weight Calculator* section. For the methodology, Michael O. co-authored the Design of a Structural System, Design of a Gravity System, Design of Typical Connections, and the sections on Wind Loads and Seismic Loads. Michael O. helped to write the introduction for the LEED section of the Report. He performed all structural calculations by hand to check Jodi-Lee S.'s spreadsheet calculations. Michael O. performed all calculations for the connections of the structural members. He worked on all 2-D CAD drawings for the building. He correspondingly built the model of the building on *MASTAN2* to be used for the structural analysis. Michael O. also created the model room renderings in *Autodesk Revit*. For the Findings, Michael O. wrote the sections on Long Span and Short Span as well as the Economic Evaluation. Michael O. took lead role on looking at suggestions for further study. He also created this Authorship page and the Acknowledgements page.

Jodi-Lee Smith

Jodi-Lee S. was responsible for writing the Introduction, and working on sections of the Background. Jodi-Lee S. was also responsible for the section on Cornell University's Seismic and Wind Force Calculator, the Coduto Spreadsheets and the background section on the Vierendeel Frame. For the Methodology, she co-authored Design of a Structural System, Design and Analysis of a Gravity System, and Design of Typical Connections sections. Jodi-Lee S. also wrote the Column Design for Lateral Loads and Design of

Foundations section of the report. Respectively, she used *MASTAN2* to analyze the effects of lateral loads and subsequently found the required column sizes with combined axial and lateral conditions. Jodi-Lee S. performed all calculations of the design of foundations and correspondingly wrote the section of the Findings on the Selection and Design of Foundations. Jodi-Lee S. also wrote the section on Comparison and Selection of Conceptual Design for the Findings. She also created a structural Revit model for both the short-span and long-span design. Jodi-Lee S. took lead role on organizing the Appendices of this report.

Ryan Worsman

Ryan W. was responsible for writing the section on Lot Six of Gateway Park and the sections in the methodology on Site Planning, Development of Drainage Calculations, Traffic and Analysis, and Development of a Preliminary Cost Estimate sections. He authored the Findings sections on Site-Planning and Drainage as well as Traffic and Parking. Ryan W. conducted field studies and data analysis on traffic flow rates and current traffic loads. Based on these volumes he projected the expected growth and re-analyzed the intersections based on future loads. The traffic studies he conducted included modal split calculations to verify the use of green options of transportation. Ryan W. conducted two separate cost estimates, one for the long-span design and one for the short span design, accommodating for the mixed uses of the building. He finished the cost estimate by comparing the price of both structural configurations to other buildings on WPI campus to ensure financial feasibility. He also performed drainage calculations Ryan W. also investigated and wrote the section on Obtaining LEED certification including an estimate of expected cost increase based on additional expenses of green technologies. He also created the architectural Revit model of the building.

Capstone Design Statement

The MQP team has been prepared for engineering practice from a variety of related coursework. The capstone design experience is an important part of becoming a professional engineer. The capstone design experience serves to bring together previously learned skills and new concepts learned independently, while incorporating current engineering standards and building codes. To this aim this MQP incorporated several realistic constraints by considering various aspects of the project: economic, environmental and sustainability, constructability, health and safety, and social and political. The following paragraphs will describe each aspect in more detail.

Economics

The first constraint this project considered was economics. The project took into consideration the cost per square foot of the building design per *RS Means 2011* using a unit cost approach. The most economical design of either a long span or short span structure was selected as the final structure.

Environmental and Sustainability

This MQP examined the project from both an environmental, as well as a sustainable perspective. As with most projects taking place today in the United States, sustainability was a major concern and therefore required an adequate amount of attention. This project considered general LEED certification criteria. Traffic impact on the surrounding area was assessed. Furthermore, impacts to storm water runoff were evaluated.

Constructability

Constructability was another important constraint of this project. The MQP team first examined the advantages and disadvantages of building one versus two buildings on 32 Prescott Street at Gateway Park. Constructability is defined as "the optimum use of construction knowledge and experience in planning, design, procurement, and field operations to achieve the overall project objectives" (Construction Industry Institute, 1986). The usability and functionality of the interior layout of the building was a major design factor. The MQP team took into consideration time constraints of shored construction for the composite beam-and-slab systems. Furthermore, the MQP team took caution to ensure that the design was simple, standardized sections and geometries were specified, and the frame layout was repetitive so that the construction process can be as efficient as possible.

Health and Safety

All construction projects need to account for the health and safety of the building's occupants. One of the primary ways this design accounted for the safety of building occupants was the application of the *Massachusetts Building Code* which references the *2009 International Building Code* (*IBC*) as a standard for all construction. The *IBC* was consulted to determine the minimum requirements for the frequency, width, and travel distance of each means of egress. Additionally, this project used the *IBC* to determine the requirements for fire walls to separate certain occupancy types since this is a multi-use building. The beams, girders, columns, and footings were all designed to comply with the *IBC* provisions for safely transferring the dead loads and live loads of the building.

Social and Political

Lastly, this project considered social and political constraints. Gateway Park as a whole has gone through the required permitting and zoning procedures to become an approved project within the City of Worcester. Grants and subsidies were a major part of the development of Gateway Park so the MQP team investigated why the grants were given and when and how they were utilized. This project examined some of Worcester's zoning laws to confirm that site and building design and usages meet the current regulations of the city. Socially, the impact on the surrounding community and ties to Worcester Polytechnic Institute were also considered. The MQP group spoke to consultants and planners involved in similar projects throughout the city.

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Goal

The goal of this MQP is to investigate, design, and analyze a proposed mixed-use development that will serve as: office and industrial space for new life science companies, retail space, and Graduate housing. This MQP will also analyze the impact of the proposed building to the existing traffic, parking conditions.

1 Introduction

Gateway Park LLC. is a joint effort between Worcester Polytechnic Institute (WPI) and other private profit and non-profit organizations to revitalize the Prescott-Grove Street District, commonly known as Gateway Park. In order to achieve the development goals that align with the City of Worcester and the Gateway Park LLC., the *Gateway Park Master Plan* was written and submitted to Worcester in 2001 (Wallace Floyd Design Group, 2001). More specifically, the *Gateway Park Master Plan* "was commissioned to assess the development potential of the area, based on market and physical characteristics, and to create an achievable vision for the area to guide future development and both public and private investment decisions" (Wallace Floyd Design Group, 2001). The *Gateway Park Master Plan* is a comprehensive long term plan that guides the development of 63 acres including 11 acres now known as Gateway Park at WPI.

Gateway Park at WPI initially began as a collaborative effort between Worcester Polytechnic Institute and the Worcester Business Development Corporation (WBDC). However, in 2010 WPI and the WBDC reached a new agreement that stated that WPI would be the exclusive owner of Gateway Park at

WPI, with WBDC shifting their role from co-owner to more of "a development role on a consulting basis," (Worcester Polytechnic Institute, 2010). In order to ensure that WPI growth only serves to "raise the university to new levels of quality and prestige" its development is guided by its *Strategic Plan- New Vision, New Ideas, and New Resources II* ("Strategic Plan"). This document was first written in 1996, and has since been revised twice to account for WPI's growth and development. Goal seven of WPI's *Strategic Plan* expresses WPI's desire to "develop non-traditional sources of revenue as a means of strengthening WPI financially and keeping it affordable" (Worcester Polytechnic Institute, 2008). This desire is the predominant driving force behind the development and expansion Gateway Park at WPI.

WPI aims to develop Gateway Park as "a mixed-use, science-based neighborhood providing opportunities for corporate partnerships and income from rents and ground leases," (Worcester Polytechnic Institute, 2008). In 2007 WPI completed the construction of its first building-a 125,000 square-foot Life Sciences and Bioengineering Center. On April 21, 2011 O'Connell Development Group broke ground for a new four-story facility that will house laboratory, educational, and office spaces for a range of academic and corporate uses. In keeping with goal seven of the *Strategic Plan,* WPI seeks to develop a new mixed-used development at 32 Prescott Street.

One of the constraints to this development is the location of the Millbrook Culvert as it bisects 32 Prescott Street. The culvert must remain easily accessible for maintenance and repairs, and as a result, it cannot be permanently obstructed, thus complicating the design solution for a potential new building or buildings located at

32 Prescott Street. This constraint necessitates a design solution that is cost effective and constructible, yet avoids obstructing the culvert. Although WPI owns the land, it plans to lease it to private life science developers interested in expanding their businesses. The goal of this MQP is to investigate, design, and analyze a proposed mixed-use development that will serve as: office and industrial space for new life science companies, retail space, and Graduate or Upper-class housing. This MQP will also analyze the impact of the proposed building to the existing traffic, and parking facilities.

2 Background

The focus of this MQP is to investigate, design, and analyze a proposed mixed-use development at Gateway Park at WPI. This section shall present information on the history of Gateway Park and 32 Prescott Street, and present information regarding software that was utilized in the design and analysis

2.1 Transformation of Prescott-Grove Street District to Gateway Park

During the industrial age, vibrant steel mills occupied the area currently known as Gateway Park. This area in Worcester flourished until the late 1950s; eventually production moved to other parts of the world and Worcester was left with an abundance of abandoned buildings. Figure 1 shows Gateway Park prior to its revitalization.

Figure 1: Gateway Park Prior to Revitalization

Contamination was a problem associated with many of these abandoned sites. Today, within the City of Worcester, there are more than 200 brownfield sites that are documented (Brownfields Success Story, 2009). However, despite the number of brownfield sites, there are less than 100 acres open for development in all of Worcester. In a city where non-developed land is scarce, Gateway Park is a prime location due to its close proximity to WPI, Main Street, Interstate 190 (I-190), and Interstate 290 (I-290). The cleanup process was partially funded by two $350,000 loans issued by the Massachusetts Development Finance Agency and $200,000 from a 2005 EPA Brownfields Revolving Loan Fund awarded to the city of Worcester. By 2006, cleanup of the site was completed; the entire site is now ready to be built on, and any contamination levels are below the accepted maximum designated by the EPA (Brownfields Success Story, 2009).

2.2 Gateway Park Today

Gateway Park in total is 63 acres. Of the 63 acres, 11 acres are considered Gateway Park at WPI; this land is highlighted in Figure 2: 2007 Gateway Park Plan. The old Millbrook culvert which runs beneath many of the properties in Gateway Park poses many problems when current construction is considered. The 11- acre site was originally owned by seven different individuals; however, Gateway Park, LLC. was able to negotiate and purchase all of this land (The Phoenix Awards, 2007). By March, 2010 WPI took over as the sole owner of Gateway Park at WPI, however the WBDC still assists in consulting efforts (Cohen, 2010).

Figure 2: 2007 Gateway Park Plan, Gateway Park Highlighted in Yellow

The primary focus of Gateway Park is bringing life sciences and bioengineering to the area, revitalizing it beyond its former splendor. As stated in a report concerning Gateway Park, “the cleanup of an environmentally blighted and economically stagnant area has opened up a new ‘gateway’ to unite and capitalize on Worcester’s burgeoning life science industry and WPI’s leadership and vision in bioengineering and life sciences” (Carey & Conover, 2007). Cost alone is one factor that will make Gateway Park an asset to bioengineering companies. Rent is less than half that in the Boston/Cambridge area with Worcester offices renting for $20-$35 per sq. ft. near WPI versus $45-$95 near MIT in a recent cost analysis (Facts and Figures, 2011). Worcester boasts thirteen prominent colleges, and five medical facilities, three of which are also schools, such as the UMASS Medical School. These institutions help to fuel the need for more biotechnology and life sciences research and facilities. Prominent companies have already been leasing space at Gateway Park and, with more office space to be built such as that proposed in this report, many top companies will look at Worcester as a destination that is more economical and practicable than Cambridge.

2.3 Lot Six of Gateway Park

Lot six is proposed to be one of the last lots in Gateway Park at WPI to be developed. In Figure 3, lots two and three are under development, and the current Gateway Life Sciences building is partially situated on lot two and on the “Newgate Properties” Lot. Lot six abuts Lincoln Street, Concord Street, and Prescott Street. The lot also borders the Boston & Maine Corporation’s rail lines which are typically just used for freight trains. The location of the culvert can also be seen in this figure. The lot's proximity to I-290 also increases its potential value as a location for new businesses, whether offices or retail space.

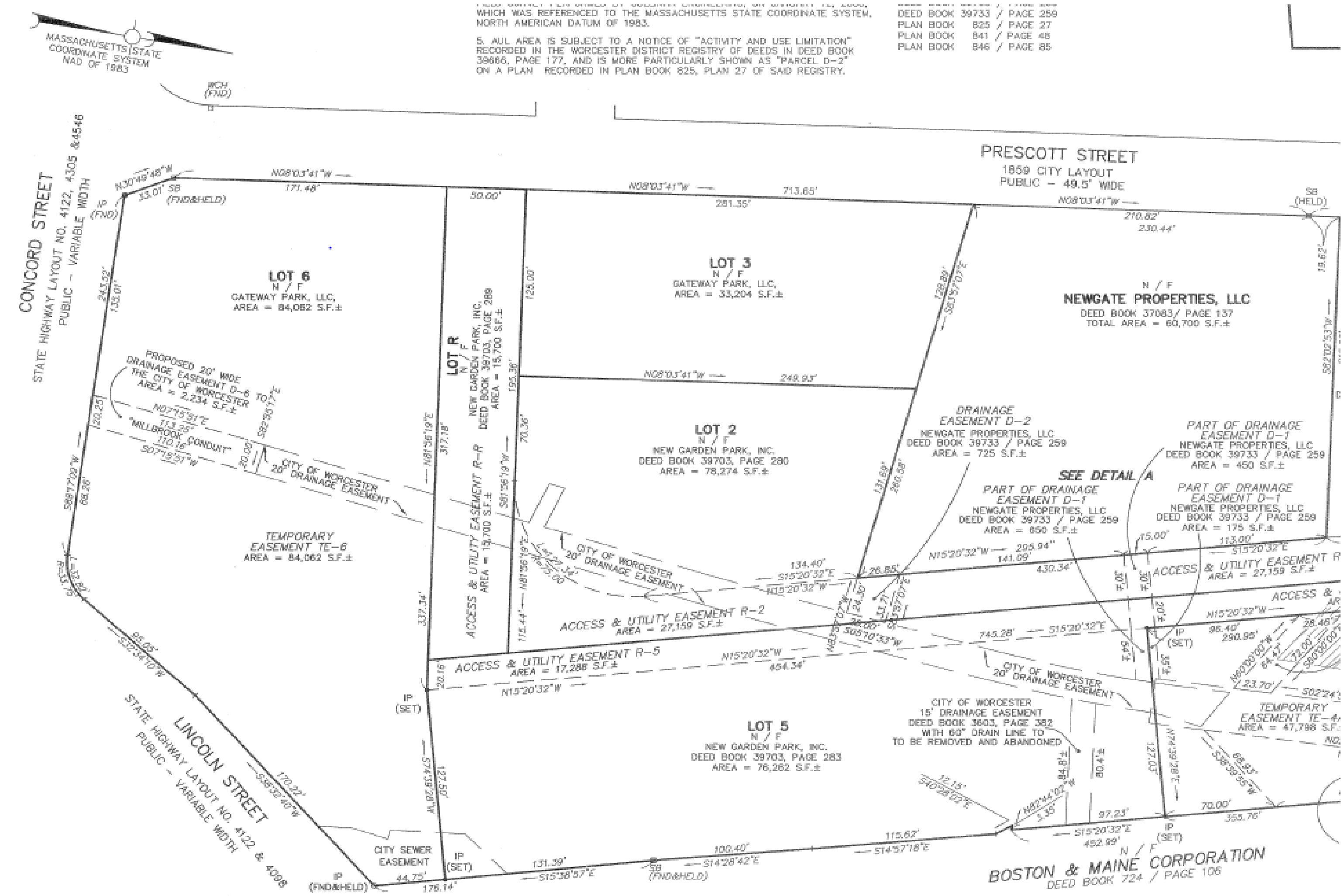


Figure 3: 2006 Gateway Park Parcel Survey
(Engineering, 2006)

The *Gateway Master Plan* makes several recommendations pertaining to two proposed buildings; Table 1 outlines proposed building requirements.

Table 1: Building criteria for Lot 6

Building 1	Building 2
Corner of Prescott and Concord Street	Corner of Lincoln and Concord Street
Development may take place before Lincoln Square is reconfigured	Development may take place before Lincoln Square is reconfigured
Will be visible from I-290	Will be visible from I-290
“Prominent new building” Office space	“Prominent new building” Office space
Research and development	Research and development
20,000 square feet per floor/ 100,000 square feet total	20,000 square feet per floor/ 160,000 square feet total
4-7 floors	8-10 floors
300 parking spaces required	480 parking spaces required
	Parking facility “b” for Gateway Park: 270 spaces below grade

(Wallace Floyd Design Group, 2001)

The 84,062 square foot lot is vacant, and recently grass has been planted to improve the aesthetics of Gateway Park. Currently, the MQP Group is led to believe that the reason there are two separate buildings envisioned for this one lot is to avoid the permanent obstruction of the Millbrook Culvert. The culvert needs to be fully accessible for maintenance purposes. From a site planning perspective this means that there can be neither vertical obstruction, for a set height of at least 21 feet, nor also for a certain distance laterally, allowing excavation. This vertical distance of 21 feet allows truck and heavy equipment access based on a Caterpillar

450E Backhoe Loader, the largest of the Caterpillar family's backhoe loaders; larger excavators could be used once ground is broken (Caterpillar, 2007).

This location was selected as an MQP topic for a variety of reasons. First, this project presents unique challenges due to its proximity to major problematic traffic areas in Worcester. Next, the culvert poses a separate problem which will be investigated, namely by considering one versus two building on lot six. Most importantly, since this project is related to WPI, the group of students felt a connection with working on this project especially knowing that its results could be examined and used by WPI in the future.

2.4 Computer-Aided Structural Analysis

In order to analyze the effects of loads on the structure two computer-aided structural analysis programs were utilized. The MQP team decided to utilize *MASTAN2* and Cornell University's Seismic and Wind Force Calculator (Ochshorn, 2009). These programs enabled the MQP team to quickly and efficiently analyze the statically indeterminate structure, and determine the design values for the structural loads.

2.4.1 MASTAN2

Since the frame of this building is a statically indeterminate structure, computer software was used to aid in the calculations of member moments and axial loads. The program of choice for this MQP was *MASTAN2* developed by Professor Ronald Ziemian of Bucknell University and Professor William McGuire of Cornell University. *MASTAN2* was chosen due to its simple interface and quick learning curve. Member sizes, properties, and fixities were first defined. Each loading condition was input in *MASTAN2* individually; for example, all the live-loads were analyzed first, then all the dead-loads, etc. For this MQP, first-order elastic analysis was utilized to determine the moments, axial loads, and node deflection for lateral loads. Additionally, second-order effects were handled in an approximate manner with multipliers B1 and B2 as outlined in the *AISC Steel Manual* section C2.1 (American Institute of Steel Construction, 2008). The program also provided the MQP team a visual analysis of how certain loading configurations affect the structure's deflection through animation.

2.4.2 Cornell University's Seismic and Wind Force Calculator

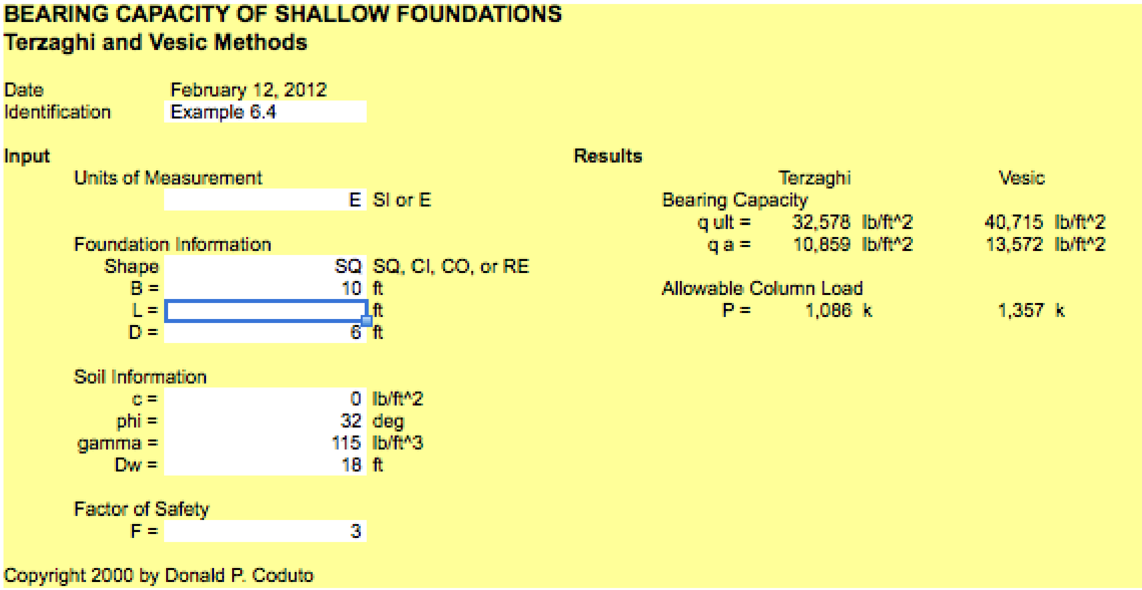
In order to determine the design values for the seismic and wind forces that this building could potentially experience the MQP group decided to utilize Cornell University's *Seismic and Wind Force Calculator* (Ochshorn, 2009). The design values obtained from the program are based on ASCE/SEI 7-05, *Minimum Design Loads for Buildings and Other Structures.* This calculator enables users to input "general data (city, importance factor), seismic data (site class, seismic force resisting system), and wind data (exposure category, plan and parapet dimensions, and coefficients for directionality and topography)," and site specific information such as the height of each story and the dead load for each story. Once the data has been entered, the program will determine the windward pressure, the leeward pressure, and the seismic story forces. The results obtained from the use of the *Seismic and Wind Force Calculator* were input into *MASTAN2* to determine the structural response to lateral loads.

2.4.3 AMLink Material Weight Calculator

The *AMLink Material Weight Calculator* is an online resource that enables users to quickly determine the weight of a particular object (AMLink Materal Weight Calculator). The calculator allows users to specify: the material, the density of the given material, quantity, shape and dimensions of the material. Once all the information is entered into the program the weight of the object can be determined. This program enabled the MQP team to make an estimate as to the average weight of each story in order to calculate the seismic loads.

2.4.4 Coduto Spreadsheets

In order to determine an appropriate foundation width and depth the MQP team utilized two spreadsheets developed by Coduto (Coduto, 2001). The first spreadsheet, *Bearing Capacity of Shallow Foundations*, enables the user to determine the maximum bearing pressure for a given foundation shape, embedment depth, D, footing width, B, soil type, and factor of safety. For design applications the user can vary the footing width B until the value obtained for the permissible load P is greater than the service load. The corresponding allowable bearing capacity can then be noted and used to design the footing. Figure 4 and Figure 5 display a sample view of the Coduto spreadsheets that were utilized.



BEARING CAPACITY OF SHALLOW FOUNDATIONS
Terzaghi and Vesic Methods

Date February 12, 2012
Identification Example 6.4

Input

Units of Measurement
E SI or E

Foundation Information
Shape = SQ SQ, CI, CO, or RE
B = 10 ft
L = ft
D = 6 ft

Soil Information
c = 0 lb/ft^2
phi = 32 deg
gamma = 115 lb/ft^3
Dw = 18 ft

Factor of Safety
F = 3

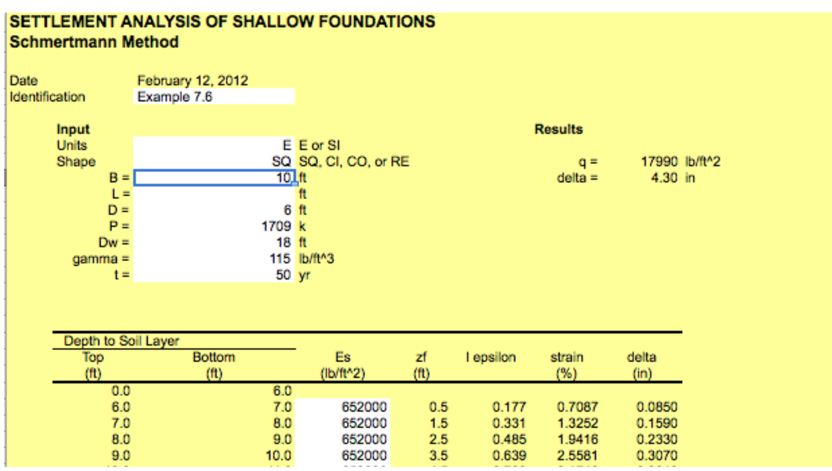
Results

	Terzaghi		Vesic	
Bearing Capacity				
q ult =	32,578	lb/ft^2	40,715	lb/ft^2
q a =	10,859	lb/ft^2	13,572	lb/ft^2
Allowable Column Load				
P =	1,086	k	1,357	k

Copyright 2000 by Donald P. Coduto

Figure 4: Bearing Capacity of Shallow Foundation Spreadsheet

After the footing width is determined based on bearing capacity, the second spreadsheet *Settlement Analysis of Shallow Foundations-Schmertmann* can be used to determine the minimum footing width that satisfies settlement criteria and its corresponding allowable bearing capacity. The resulting footing size was determined by the limiting of the two design approaches.



SETTLEMENT ANALYSIS OF SHALLOW FOUNDATIONS
Schmertmann Method

Date February 12, 2012
Identification Example 7.6

Input
Units E E or SI
Shape SQ SQ, CI, CO, or RE
B = 10 ft
L = ft
D = 6 ft
P = 1709 k
Dw = 18 ft
gamma = 115 lb/ft^3
t = 50 yr

Results
q = 17990 lb/ft^2
delta = 4.30 in

Depth to Soil Layer Top (ft)	Bottom (ft)	Es (lb/ft^2)	zf (ft)	I epsilon	strain (%)	delta (in)
0.0	6.0					
6.0	7.0	652000	0.5	0.177	0.7087	0.0850
7.0	8.0	652000	1.5	0.331	1.3252	0.1590
8.0	9.0	652000	2.5	0.485	1.9416	0.2330
9.0	10.0	652000	3.5	0.639	2.5581	0.3070

Figure 5: Settlement Analysis of Shallow Foundations-Schmertmann Method

2.5 Vierendeel Frame

A Vierendeel frame was designed for the bridged section of the development since it allows for clearer rectangular spaces by eliminating the need for diagonal bracing (MacLeod, 2005). "The analogy between a Vierendeel frame and a beam is similar to that for a parallel chord truss except that the shear mode component is due to the bending of the chords and the posts rather than to axial deformation of the diagonals and posts" (MacLeod, 2005). The challenge of designing a Vierendeel frame is that it is a highly indeterminate structural system, and it is difficult to determine the load path intuitively and identify which elements are predominantly bearing the loads. Since the calculations for this statically indeterminate structure would be extensive, the MQP group utilized *MASTAN2* to analyze the frame. Figure 6 displays an elevation view of the overall building structure and depicts the location of the Vierendeel frame.

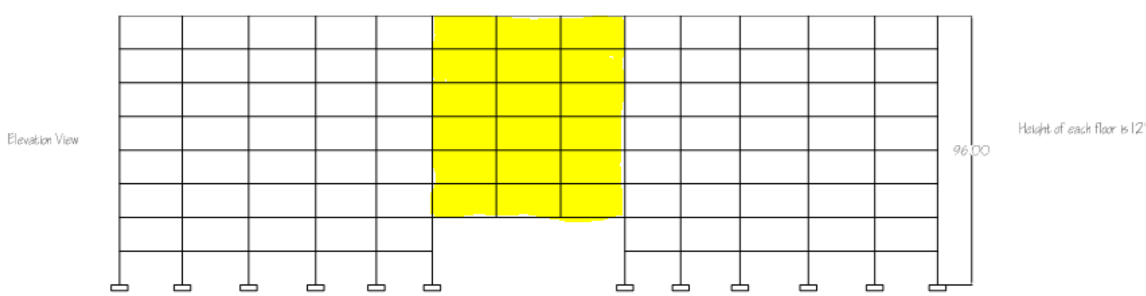


Figure 6: Vierendeel Frame is Shown Highlighted in Yellow

3 Methodology

Goal seven of the *WPI Strategic Plan* expresses WPI's desire to generate revenue from non-traditional sources. To this aim, WPI seeks to develop Gateway Park as a mixed-used life sciences and biotechnology center. This MQP investigated, designed, and analyzed a proposed mixed-use development that will serve as: office and industrial space for new life science companies, retail space, and graduate housing. Furthermore, this MQP analyzed the impact of the proposed development on the existing traffic and parking conditions. In order to accomplish these goals, the following objectives were executed:

- Conducted a programming phase
- Constructed a site plan
- Conducted a preliminary analysis and comparison of design options
- Developed a building layout design
- Developed an engineering design
    - Structural
    - Site
- Developed a construction schedule and cost estimate
- Conducted a traffic and parking analysis
    - Examined practicality of site layout to traffic patterns

The proceeding sections will provide a detailed look into how these objectives were executed.

3.1 Programming Phase

The programming phase was designed to break up the total square footage for conceptual design A and conceptual design B into their major parts. In order to complete the space allocations the needs of every intended occupant of the building was taken into account. For WPI the primary needs to be satisfied are more research and development space and graduate student housing. According to interviews conducted with WPI officials, outside companies will be targeted to occupy the building. The external companies will require both office space and research labs. In order to accommodate all of these building functions careful planning was used to effectively respond to all of the needs of each potential client that will be occupying the building. For example keeping noise generating uses, such as laboratories, away from residential dwellings or ensuring adequate sound proofing will aid in keeping all occupants satisfied.

The proposed development located at 32 Prescott Street lies within the Mixed Use Development Zone Overlay which "is intended to provide for the coordinated and mixed development of residential, business, institutional and open/recreational space uses the City of Worcester" (City of Worcester, 2011). The allocation of usages and space within the Mixed Use Development Zone Overlay is governed by the following guidelines displayed in Figure 7 and Figure 8.

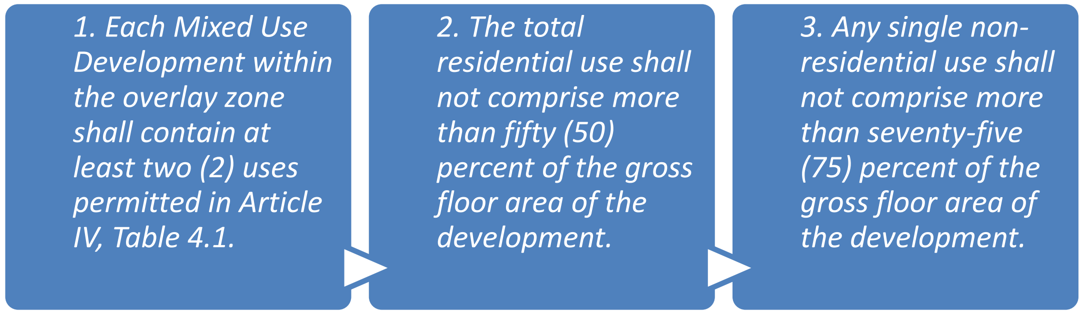


Figure 7: Guidelines 1, 2, and 3 for a Mixed Used Development Zone Overlay

(City of Worcester, 2011)

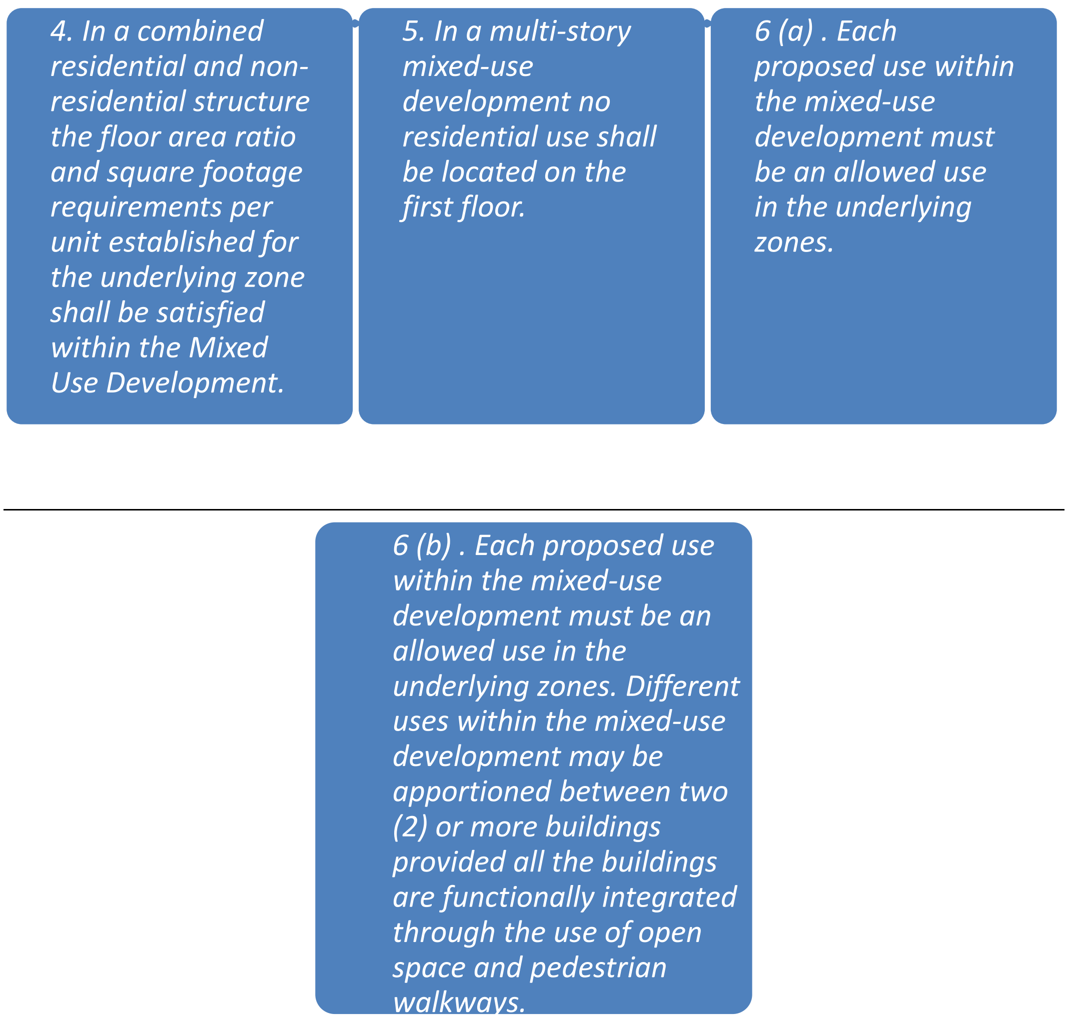


Figure 8: Guidelines 4, 5, and 6 for a Mixed Use Development Zone Overlay
(City of Worcester, 2011)

3.2 Site Planning

A site plan is a critical part to any building project. The Worcester Zoning Ordinance was examined to determine the required setbacks from streets and other nearby buildings. The most recent amendments to The Zoning Ordinance of the City of Worcester went into effect on June 14, 2011. The purpose of the Zoning Ordinance is “to promote the health, safety and general welfare of the public and to contribute to the implementation of the City’s ongoing comprehensive planning process” (City of Worcester Zoning Ordinance, 2011). This MQP will follow provisions set forth by the Zoning Ordinance to meet the document’s purpose.

Parking requirements were examined as well as the flow of vehicular traffic and pedestrian traffic from the proposed development to other buildings at Gateway Park and towards WPI campus. Once a suitable square footage for the development was determined, the proposed development was situated on the lot minding the city’s ordinances. Furthermore, the anticipated uses of the buildings and adequate space for parking were considered, as retail space needs as much visibility as possible without compromising the necessary parking areas.

As part of the site plan, utility design and connections needed to be considered. Using available plans from the city water, gas, electricity, and sewer connections were examined to see where the necessary connections from the street to the proposed development could be made. The site drainage was also examined for all areas of the site including changes by adding roofs, parking lots and walkways as well as all the pervious surfaces that may be affected by development.

3.3 Development of Conceptual Designs

In order to select a design option that best suits the needs of Gateway Park and the WPI community, two conceptual design alternatives were analyzed and compared. The criteria used in the preliminary development of each alternative are displayed in Figure 9.

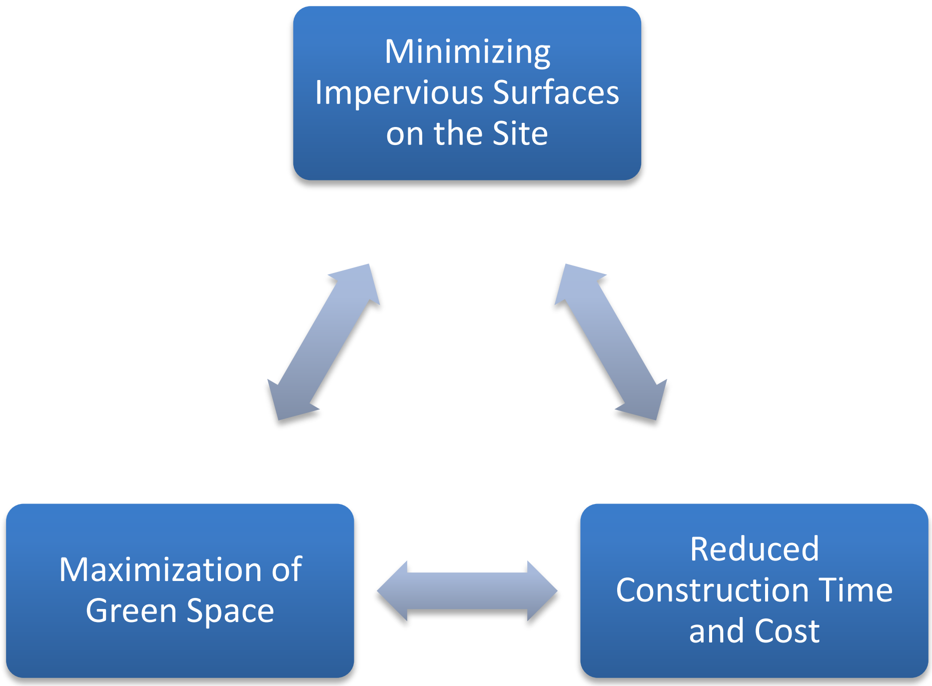


Figure 9: Criteria Used in the Preliminary Development of Each Design Alternative

Conceptual design A proposes the construction of two separate buildings, the first on the corner of Prescott Street and Concord Street, and the second on the corner of Concord Street and Lincoln Street. Conceptual design B proposes the construction of one building on the lot that will incorporate both of the first two buildings into one design. Each conceptual design was developed based on site planning and zoning restrictions.

Each conceptual design was developed for the following usages: office, industrial, research and development and residential units. The total development will be approximately 240,000 square feet and will require a certain amount of parking spaces depending on zoning requirements. The construction will mark the completion of a prominent building seen from I-290 as part of the entrance to Worcester. According to the *Gateway Park Master Plan* the development must be constructed with red brick and glass façade to enhance street visibility and match the exterior of the existing buildings (Gateway Park Master Plan, 2001).

3.4 Comparison and Selection of Conceptual Design

After conceptual design A and conceptual design B were developed, they had to be analyzed and compared so a design could be selected for further development. In order to select a design option the pros and cons of each design alternative were evaluated based on the following criteria:

- Time for construction
- Accessibility of culvert
- Aesthetical impact on the Gateway Park at WPI
- Cost of construction
- Zoning restrictions
- Maximization of green space

An aspect that was heavily considered through the entire comparison and selection process was sustainable design, specifically through LEED Certification. LEED Certification "or Leadership in Energy and Environmental Design, is an internationally-recognized green building certification system" (U.S. Green Building Council, 2011). The MQP group used the LEED point system for new construction and major renovations to assist in determining if conceptual design A or conceptual design B is more successful in meeting the LEED certification.

In order to have a quantitative method of identifying the preferred conceptual design the MQP team calculated an average score for each design. A maximum of four points was allocated between conceptual design A and B for each evaluation criteria. The total score for each design option was determined by the summation of each team member's score. The design with the larger numerical score was selected as the conceptual design from which the rest of the project will be developed.

3.5 Building Layout Design

Based on the results from Section 3.1 Programming Phase the layout of the design was established. To ensure that the building layout maximizes each of the three planned usages the MQP team utilized *Time Saver Standards* and *Architect's Studio Companion* (Watson, Crosbie, & Callender, 1999). Additionally, the design was developed so that the layout promoted efficient travel through the development for all its users as well as provided adequate means of egress in the event of an emergency. A great example of this is having the retail space on the first floor exposed to street passersby. The building layout design was also developed to ensure that the sunlight entering the building was maximized to reduce the cost of lighting and heating. An important aspect of developing the building layout was determining the location of the hard constraints, such as means of egress and restrooms, as well as the grid of girders and columns for the design and analysis of the structural system.

3.5.1 Determination of Hard Constraints

In order to design a structural system the MQP team had to identify the locations and dimensions of the hard constraints such as the stairs, elevators, and main restrooms. The

MQP team decided to incorporate a central core, which would be repeated on floors one through five. The central core would be comprised of: male and female restrooms, two elevators, HVAC MEP, and a janitor's closet. The dimensions of the elevators were obtained from the book *Architectural Graphic Standards* (Hoke, 2000). The dimensions and minimum requirements for stairs, corridors, means of egress, and restrooms were determined utilizing the 2009 *International Building Code (IBC)*. The proceeding steps provide explicit details on the sections of the code that were consulted for determining the corridor width, the width of stairwells, and the number of egresses required per floor .

STEP 1: Determine the Occupant Loads

a) Building usages

   Chapter 3 of the *IBC* was used to determine the usages of the building. Based on the proposed usages the following subsections were consulted: 304.1 Business Group B, 306.1 Factory Industrial Group, 306.3, Factory Industrial F-2 Low-Hazard Occupancy, 309.1 Mercantile group M, 310.1 Residential Group R.

b) Square footage of floor

   The gross square footages of each floor was determined

c) Floor area in square feet per occupant was determined based on the building usages identified in STEP 1, part a.

d) The occupant load for each floor was determined using the equation below:

$$Occupant\ Load: \frac{Gross\ Square\ Footage}{Floor\ Area\ in\ Square\ Footage\ Per\ Occupant}$$

STEP 2: Determine the Minimum Number of Means of Egress

a) The minimum number of means of egress per story was then determined by utilizing Section 1014.2.3.3 of the *IBC* (International Code Council, 2009). The minimum number of egresses was determined using the occupant load found in STEP 1, part d and the table below from the *IBC:*

Table 2: Occupant Load and Minimum Number of Exists Required

OCCUPANT LOAD (persons per story)	MINIMUM NUMBER OF EXITS (per story)
1-500	2
501-1,000	3
More than 1,000	4

(International Code Council, 2009)

STEP 3: Determine the Minimum Width of Stairs in inches

a) The width of the stairs was determined using Section 1009.1 of the *IBC* (International Code Council, 2009).

b) The number of means of egress was found using the occupant load determined in STEP 1, part d, and the minimum number of means of egress determined in STEP 2

$$Width\ of\ Stairs = MIN(\frac{.2*Occupant\ Load}{Number\ of\ MeanMeans\ of\ Egress}, \frac{.2*Occupant\ Load}{Number\ of\ Means\ of\ Egress}, 44")$$

STEP 4: Determine the Minimum Corridor Width

a) The width of the corridors was determined using Section 1018.2 of the *IBC* (International Code Council, 2009).

b) The width of the corridor cannot be less than .3*occupant load or 44 inches minimum. The width of the corridor was found using the occupant load determined using the occupant load

$$Width\ of\ Corridor = MIN\left(\frac{.3*Occupant\ Load}{Number\ of\ MeansMeans\ of\ Egress}, \frac{.3*Occupant\ Load}{Number\ of\ Means\ of\ Egress}, 44"\right)$$

STEP 5: Determine the Minimum Required Number of Bathrooms

a) The minimum required amount of bathrooms was determined using Table 2902.1 of the *IBC* (International Code Council, 2009). Table 2902.1 displays the minimum number of plumbing fixtures required for a particular type of occupancy.

Once the hard constraints were determined using STEPS 1 through STEP 5, framing plans and building layouts for a long and short span bay size were developed based on the locations of the hard constraints.

3.6 Design of a Structural System

The structural system transfers loads from building construction, occupancy, and natural effects such as wind, and earthquakes to the supporting foundation. The effects of gravity loads on a steel frame were first investigated. The objective was to select a cost-effective system for the dead and live loads based on the long span and short span; this is accounting for the cost of steel. Two alternative typical bays for the entire building were designed. In order to design the structure the following tasks were executed:

- The development of an interior framing plan to fit the functional layout
- The determination of structural loads
- The analysis of a long-span and short-span structural bays

- The evaluation of a full composite beam-and-slab design with a concrete slab on metal decking
- The determination of the shape and size of structural members

The *Massachusetts State Building Code* used to determine the gravity loads and to assist in the design (Massachusetts Board of Building Regulations and Standard, 2011). The *AISC Specifications for Load Resistance Factor Design (LRFD)* was used to determine the member and component design (American Institute of Steel Construction, 2008). Figure 10 displays an elevation view of the structure for analysis.

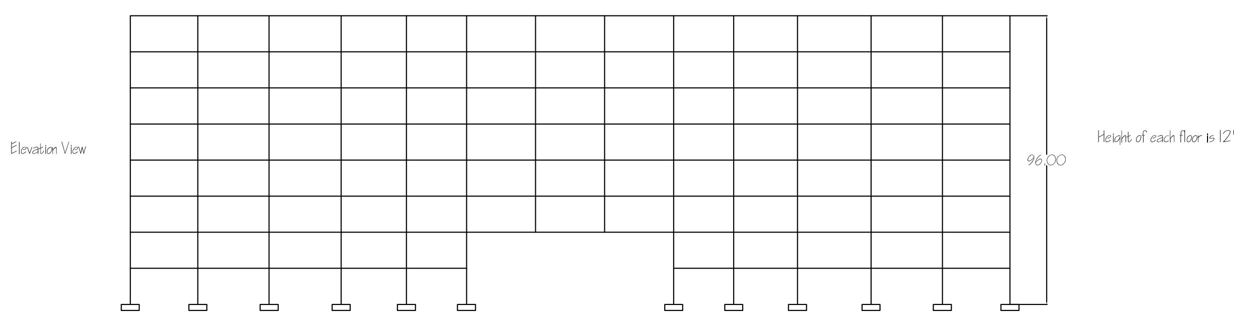


Figure 10: Elevation View of Structure

3.7 Design and Analysis of a Gravity System

Once a framing plan had been developed for the long and short span it was necessary to select the framing plan option that offered a more cost-effective system. This was accomplished by analyzing the dead and live loads for each span option; thus accounting for the cost of steel. The scope of the design and analysis of a gravity system included filler beams, girders, and supporting columns for a typical structural bay in both the long span and short span options. Separate designs for the roof and floor framing requirements were investigated. The flexural strength and serviceability (deflection) criteria were used for the basis of the design. In order to design and analyze the gravity system, the building geometry and the gravity loads were considered. The design and analysis process involved five main steps displayed in Figure 11.

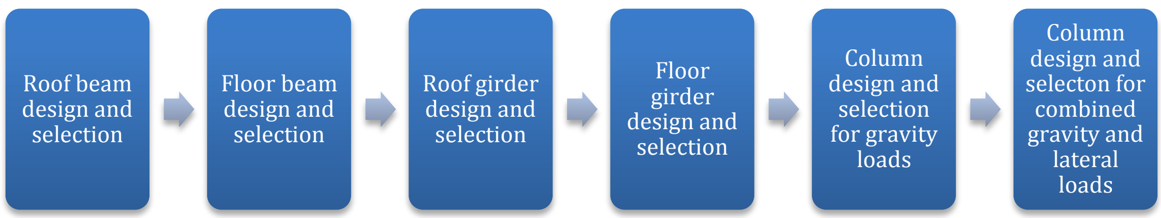


Figure 11: Main Steps in the Design and Analysis of a Gravity System

Chapter 16 of the *Massachusetts State Building Code* was used to obtain the minimum design values for live load and snow load. Additionally, Table 1607.1 of the *IBC*, which the *Massachusetts State Building Code* references, was used to identify the minimum uniformly distributed live loads and concentrated loads. The following sections will present the steps that were taken to successfully design and select suitable member sizes.

3.7.1 Roof and Floor Member Design and Analysis

Determining the appropriate beam or girder size required an iterative process. If the member failed either of the following tests the MQP team had to restart the process until a member size was identified that passed both the strength and serviceability design requirements:

- Strength: $\Phi_b M_n \geq M_u$
- Serviceability:
  - $\Delta_{LL\ during\ service} \leq L/360$ or 1" max

- $\Delta_{\text{during construction}} \leq L/360$ or 1" max

During design and member selection it was very important that careful attention was paid to the weights of the members that the MQP team selected. This was of utmost importance since it is essential that the final design was structurally sound yet economically feasible. The main steps in the design process were: determining W_u and then M_u, selecting a W section based on M_u (assumed simply supported conditions), checking the strength of the W section before the concrete hardens, and determining the service load deflection after composite action has taken place. Figure 12 and Figure 13 shows the more detailed steps in the design process.

Step 1: Determined W_u and M_u

- Determined the unfactored loads
- Used LRFD load Combinations to determined $M_u = W_u L^2/8$

Step 2: Based on this M_u a W section was Selected

- Determined Y_{con}
- Assumed value of stress block depth, a
- Selected a trial section from Table 3-19 of *Steel Construction Manual* (Based on Mu)
- Assumed $\Sigma Q_n = A_{sy} * Fy$ or use Table 3-20 pg 3-200 of *Steel Construction Manual*
- Solved for a ; $a = \Sigma Q_n / (.5 f'_c b_e)$
- $Y2 = Y_{con} - (a/2)$ *Y2 is the distance from the C.G concrete Flange to Top Flange of the beam*
- Determined φM_n from Table 3-19 of *Steel Construction Manual* . Interpolation must be larger than M_u else beam failed and started over. If beam passed continued to Step 3

Step 3: Designed Shear Studs

- Determined Q_n from AISC Table 3-21 of *Steel Construction Manual*
- $N = 2 * \Sigma Q_n / Q_n$ (from Table 3-21 of *Steel Construction Manual*) Round up to nearest whole number. Used that many studs on each side of beam center line.
- Checked fit of shear studs in decking

Figure 12: Step 1, Step 2, Step 3 to Design Beams and Girders

Step 4: Checked Strength of W section before Concrete Hardens

- Assumed that the wet concrete is a live load during construction, and added a 20 psf construction live load
- Accounted for other dead loads(weight of girder, beam, decking and slab ponding, ceiling, MEP, insulation)
- Determined w_u and M_u using LRFD

Step 5: Determined Service Load Deflection After Composite Action- (lower bound I_x)

- Determined Ix (lower bound) using Table 3-20 of *Steel Construction Manual* based Y1 and Y2 = some value and Linear interpolation
  - Determined deflection Using Δ_L= $ML*L^2/(C1*Ix > L/360$ C1= Found pg 3-8 Fig 3-2
  - If deflection was excessive then determined an appropriate lower bound Ix and used table 3-20 to select a beam

Step 6: Determine Service Load Deflection Before Concrete Hardens

- Determined M_D using $w_u L^2/8$
- Determined Ix(not lower bound) using Table 1-1
  - Determined Deflection Using Δ_{DL}= $M_D*L^2/(C1*Ix > L/360)$ (pg 553 max Δ) C1= Found pg 3-8 Fig 3-2 of *Steel Construction Manual*
- If deflection was excessive then determined an appropriate lower bound Ix and used table 3-20 to select a beam
- Once both strength and serviceability are met, beam is adequate

Figure 13: Step 4, Step 5, and Step 6 to Design and Analyze Beams and Girders

3.7.2 Gravity Column Design and Analysis

In order to determine the gravity column design an iterative process was utilized. This process is outlined through a flowchart in Figure 14, Figure 15 and Figure 16. However, it should be noted that this process was simplified through the use of Table 4-1 in the *AISC Steel Manual*; once equivalence was established between the flowchart and the use of Table 4-1, the flowchart was rendered inefficient and Table 4-1 was used extensively. Column sections were specified in two-story lengths of 24 feet. The lower level columns must support the floor loads from all of the overlying stories plus the loads from the roof. For example, the first column tier must support the loads from floors 2 through 8, plus the roof, and the third and fourth floor column must support the loads from floors 4 through 8, plus the roof. Therefore the design process follows the load path from the roof level down to the footing level (base column). In addition, the load combinations for each column tier are:

$$1.2D + 1.6L + 0.5S,$$

Equation 1: Load Combination for Maximum Live Load

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

Equation 2: Load Combination for Maximum Snow Load

where, D- dead load from all overlying floors plus the roof

L -live load from all overlying floors

S - Snow load for roof

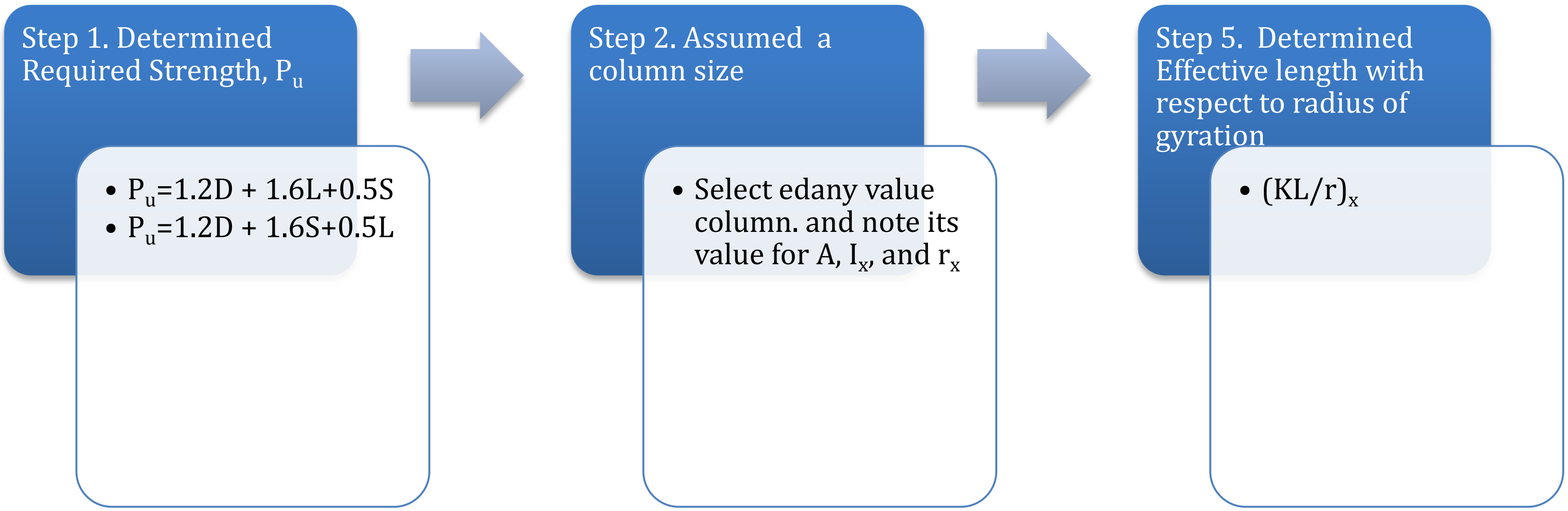


Figure 14: Step 1, Step 2, Step 3, Step 4, Step 5 to Determine Gravity Column Design and Analysis

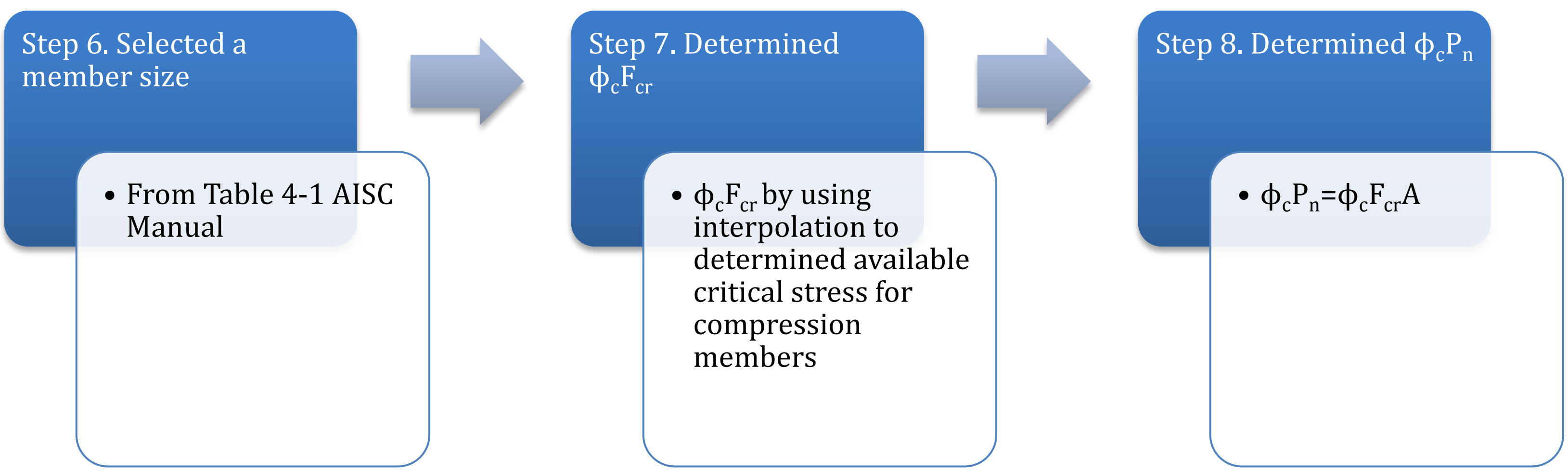


Figure 15: Step 6, Step 7, Step 8 to Determine Column Design and Analysis

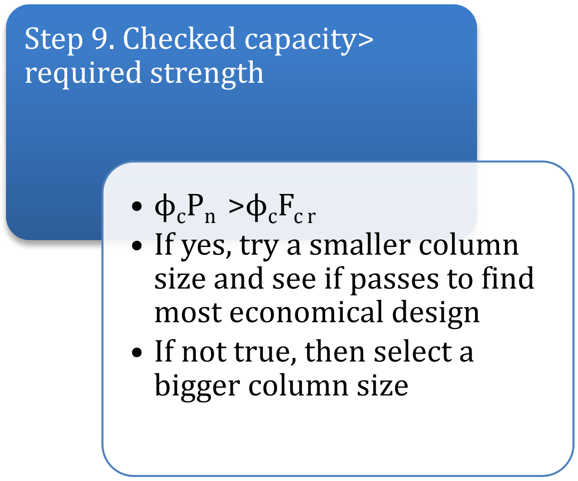


Figure 16: Step 8 to Determine Column Design and Analysis

For the simplified way to determine the column size, first the factored axial load was calculated. Next, with an effective length KL=12ft, the lightest W-section was chosen from Table 4-1 in the *AISC Steel Manual* with $\phi_c P_n \geq Pu$. Next, the column self-weight was determined and this load was added to the axial dead load to determine Pu. The available strength $\phi_c P_n$, was then checked against the load Pu, if it was greater, then the column was considered adequate for gravity loads.

The tributary area used in the design calculations was determined by comparing all the bay sizes of each span. The largest numerical value was selected and used to design a typical bay. The following images display the tributary area utilized in the design of the columns for both the short span and the long span alternatives. Figure 17 and Figure 20 displays the tributary areas used for design of columns in the short span, the long span and the bridged area.

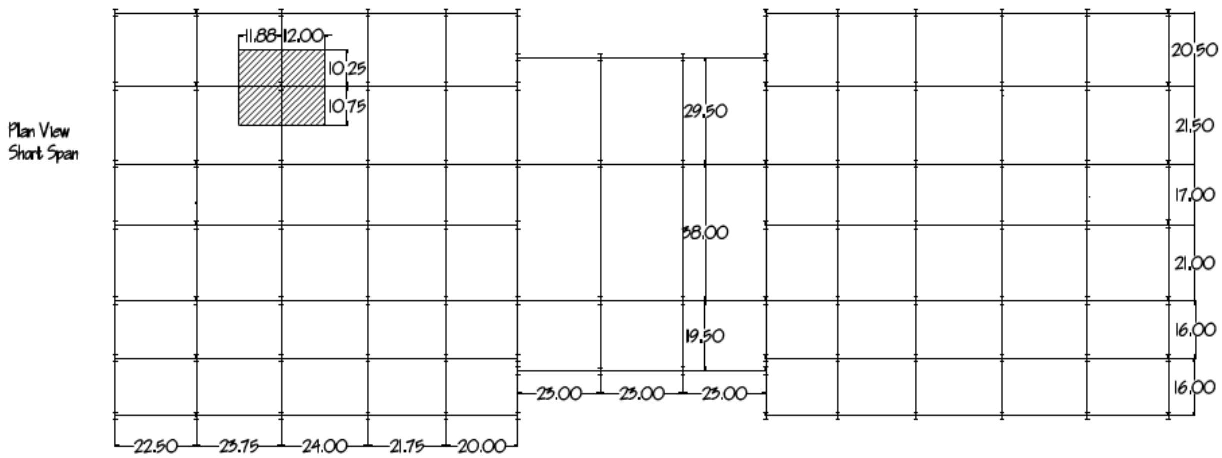


Figure 17: Tributary Areas Used for Design of Columns in Short Span and Bridged Section

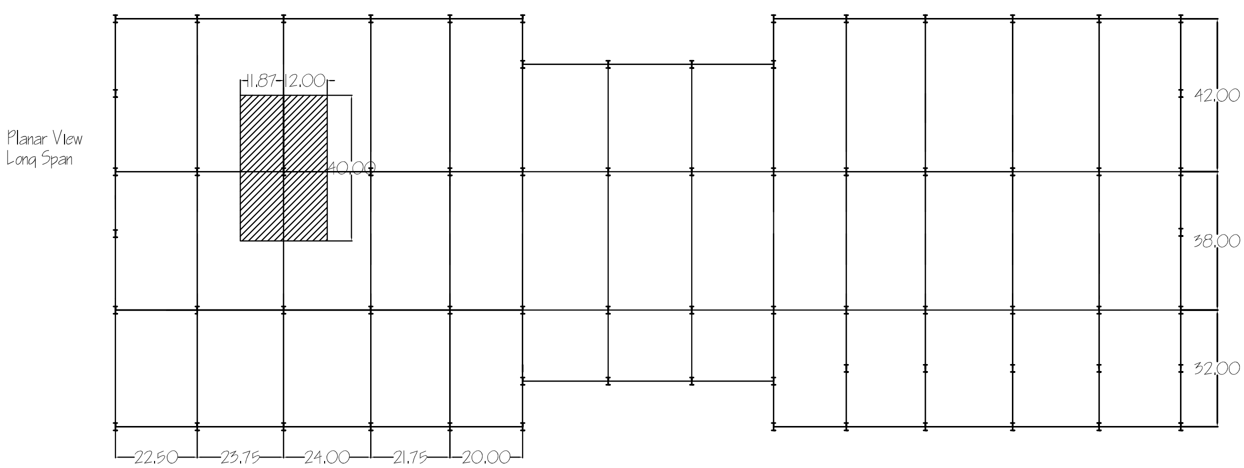


Figure 18: Tributary Area Used for Design of Columns in Long Span

After the columns were designed care had to be taken to ensure that there was not an abrupt change in column size from the ground to the roof; the size of columns needs to either remain constant or decrease progressively from ground to roof. This was an important step since an objective of the design is to ensure that it is both cost effective and constructible. To ensure this the base column was anchored and the upper columns were made larger so that there was a gradual taping of the member size. This is a concern for constructability since consistent column splices, or a gradual progression of column sizes will eliminate the need for column splices. *AISC* recommends that "Some of the specific topics that should be considered for constructability are repetition of member sizes to reduce construction cost, spacing of column splices to strike a balance between economical size and cost of splice details, and the use of *AISC* standard connection details, which are familiar and easier to review and install" (Arber, 2010).

3.7.3 Design of Typical Connections

For many years riveting was the accepted method used for connecting the members of steel structures. Today, however, welding and bolting are the methods that are used to make structural steel connections. Each method offers its own advantages and disadvantages for a given connection. For this project, the MQP team investigated a typical bolted connection from the beam-to-girder and girder-to-column for the long-span framing plan.

The bolting of steel structures is a very rapid field erection process compared to field welding. Since it is a rapid process this means that it could reduce the schedule duration and the amount of money that is allocated for job overhead, and hopefully increase savings. Additionally, bolted connections require less skilled labor than welding.

There are different types of bolts that can be used for connecting steel members. There are unfinished bolts, and high strength bolts. Unfinished bolts are classified by the ASTM as A307 bolts and are made from carbon steel with stress-strain characteristics very similar to A36 steel. They are typically used in light structures subjected to static loads and for secondary members (such as purlins, girts, bracing, platforms, small trusses, and so forth). High strength bolts rely on a certain amount of pre-tension as a part of their installation. Common bolt sizes for buildings range from ¾" to 7/8", and typically use either of the following two grades of steel:

$$\boldsymbol{A325} \qquad \boldsymbol{F_u = 120ksi}$$

$$\boldsymbol{A490} \qquad \boldsymbol{F_u = 150ksi}$$

Bolts have associated available shears strength. Table 10-4 in the *AISC* manual provides the engineer or fabricator with two bearing type connections that can either have threads included in their shear plane or the threads are excluded from the shear plane. For example the designation A325-X refers to threads excluded from the shear plane, and A325-N indicates that the threads are included in the shear plane. The available shear capacity is increased if the threads are excluded from the shear plane, since the full diameter of the bolt can be used to resist the shear. Consequently it follows that an A325-X bolt has larger available shear strength than an A-325N bolt. However, it should be noted that it may actually be beneficial to assume type A325-N bolts for connection design since the structural engineer or fabricator will not have to consider if the threads will fall in the shear plane of the connection.

The design process for designing bolts involved the following steps and reference sections from the *AISC Steel Construction Manual:*

1. Determine the member size- AISC D2(a)
2. Determine the number of bolts required- AISC J3.6
3. Establish the geometry of the bolt layout- AISC J3.10
4. Check the rupture on the net area and block shear- AISC D2(b) and J4.3

5. Use the geometry of the bolt layout to establish the geometry of the plate (the thickness of the plate must satisfy the load requirements and size is governed by the depth of the beam or girder web)

For the design of bolted connections the MQP team checked each of the limit states provided within Section J3 of the *AISC* manual based on a single angle connection. The limit states include:

1. Shear on the bolts
2. Bolt bearing on the angle of the web
3. Shear fracture through the angle leg
4. Shear yield through the angle leg
5. Bolt bearing/ tear out on the beam web

Prior to examining the limit states it was necessary to determine the shear capacity of the beam or girder under study to ensure it can transfer calculated reaction V_u using the following equation:

$$\boldsymbol{\Phi V_n = \Phi .6 F_y A_w \geq \ V_u}$$

if h/t_w is greater less than 2.24 $\sqrt{\frac{E}{F_y}}$ *then* Φ*=1.0 may be used*

Equation 3: Shear Capacity of the Structural Member

Each of the abovementioned limit states must be considered in the design of a bolted connection. The shear capacity of the bolt is given by <u>Equation 4</u>:

$$\boldsymbol{\Phi R_n = .75(F_{nv} A_b)}$$

Equation 4: Shear or Tension Capacity of the Bolt

Equation 4 can also be used to determine the number of bolts required, by dividing the calculated reaction V_u by the shear capacity of the bolt.

One design concern was the angle thickness. The angle thickness must be sufficient to develop the desired strength of the connection. It was determined by looking at the following limit states: tearing/ bearing capacity of the bolt, the shear rupture on the net area of the angle, and the shear yield on the gross area of the angle. The MQP team utilized Equation 5 through Equation 7 and the shear strength of the bolt to determine the thickness. It should be noted that bolt capacity may be larger than the calculated reaction V_u since a whole number of bolts needs to be used. Once the MQP team calculated a thickness for each of the limit states the team selected the largest value to govern.

The bearing or tear out in the vicinity of a bolt is given by $V_u \leq \Sigma\Phi R_n$ and this value is summed for each individual bolt. To determine if bearing or tear out governs the MQP team utilized Equation 5. The left hand term of the equation is the tear out term and the right hand side of the equation is the bearing capacity. The bearing capacity is the upper bound capacity in the vicinity of each bolt.

$$\boldsymbol{\Phi R_n = \Phi(1.2L_c t F_u \leq \Phi(2.4 d_b t F_u)}$$

Equation 5: Bearing or Tear out

The shear rupture on the net area of the angle and the shear yield on the gross area of the angle are given by Equation 6 and Equation 7respectively.

$$\boldsymbol{\Phi R_n = .6F_u\left(L_{angle} - nd_e\right)t \geq V_u}$$

Equation 6: Shear Rupture on the Net Area of the Angle

$$\boldsymbol{\Phi R_n = .6F_y L t \geq V_u}$$

Equation 7: Shear Yield on the Gross Area of the Angle

Next, one may select the largest of the three t values (thickness) within Equation 5 through Equation 7. With the required value for t known, *AISC* Table 1-7 was used to identify a suitable angle.

3.8 Lateral Force Analysis

For this proposed building, the MQP team took into account lateral loads in terms of both wind loads and seismic loads. The classification of the forces is in the transverse (North-South) direction and the longitudinal (East-West) direction. Referring to the design of a typical frame, the transverse loading acts on a typical story of 12 feet in height by a 40 foot width for the long span, and a 19.25 foot width for the short span. For the lateral loads on the longitudinal side of the building, the bay size was 12 feet in height by 24 feet in width (the long and short spans have the same framing pattern in this direction). The area of the typical bay was used to determine a point load on each floor in units of kips instead of using a distributed force model involving pounds per square foot. The MQP team examined the effects due to lateral loads produced in both the transverse and longitudinal directions using *MASTAN2* for analysis based on planar (2-D) sections.

3.8.1 Wind Loads

For the wind loads, both windward (positive) and leeward (negative) pressures were addressed. Design values for these pressures were all determined using Cornell University's *Seismic and Wind Force Calculator* (Ochshorn, 2009). Figure 19 displays a visual representation of the wind force calculator that was utilized. Section 6.13 of APPENDIX A : STRUCTURAL CALCULATIONS summarizes the loading values that were used in the *MASTAN2* models. From there, values for moments and axial forces on any member of the frame could be determined in *MASTAN2*; an example of the moments produced from the wind acting on the transverse direction of the building is presented in Figure 20.

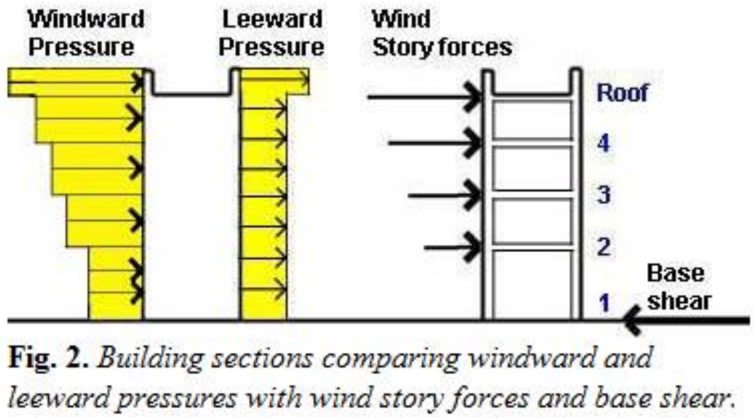


Figure 19: Diagram depicting windward pressure and leeward pressure (Ochshorn, 2009)

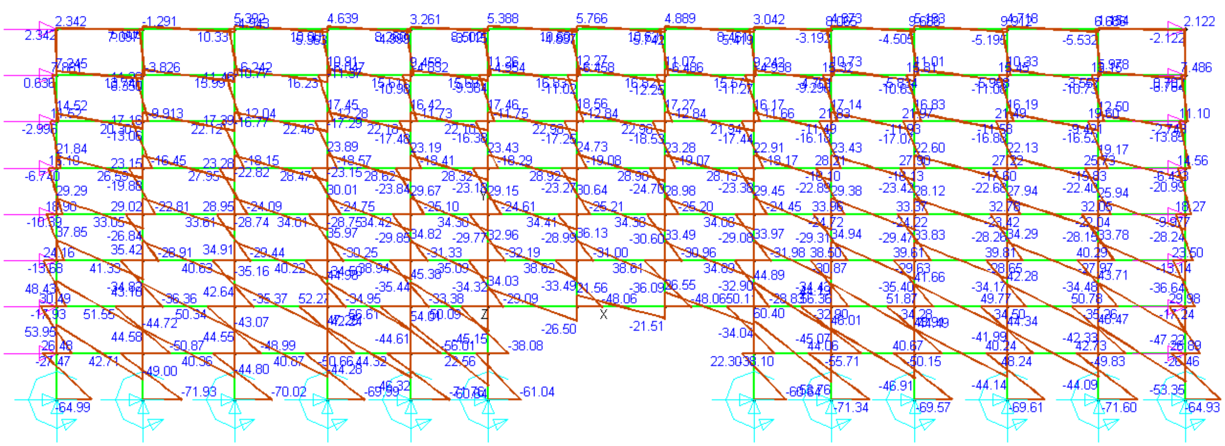


Figure 20: Moments on Entire Frame due to Wind-Load

3.8.2 Seismic Loads

To calculate the seismic loads on the building using the online calculator, first the approximate weight of each story had to be determined. To determine the approximate weight of each story, another online calculator was used (AMLink Material Weight Calculator). In order to use the *AMLink Material Weight Calculator*, the approximate slab thicknesses and weight of concrete, as well as dimensions of the building had to be known. The first and second floors have an approximate weight of 1,516,000 pounds per story, and the third through eighth floors have an approximate weight of 1,983,000 pounds per story. The output values for the seismic loads were given as total forces acting at each story level; thus, the total story forces had to be translated into point loads acting on a structural frame based on the tributary areas area for each frame respectively. Section 6.14 in APPENDIX A : STRUCTURAL CALCULATIONS summarizes the seismic forces that were input into *MASTAN2*.

3.8.3 Column Design for Lateral Loads

The story stiffness method was utilized to determine approximate multipliers for second-order effects. Based on the initial column sizes designed in Section 4.8.2 Gravity Column Design and analysis the members were redesigned utilizing the story stiffness method. The story stiffness method was selected to ensure that the columns can resist combined axial and bending effects. Figure 21 below displays the method used to design columns for combined effects.

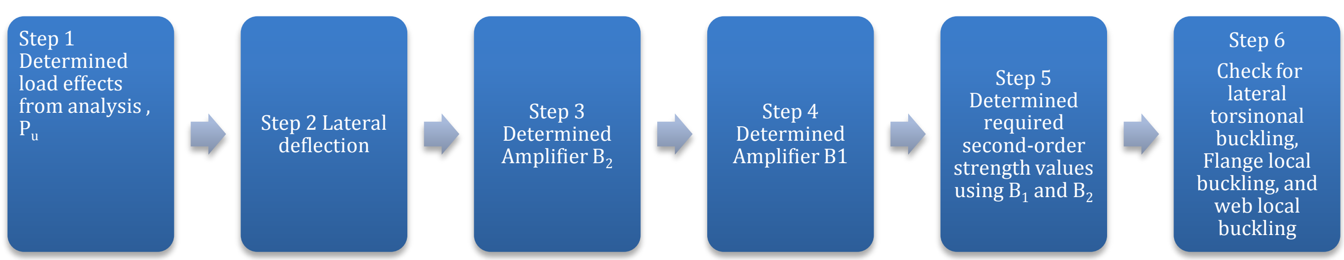


Figure 21: Steps 1-5 of the Story Stiffness Method used in Designing Columns for Combined Axial and Bending Effects

3.9 Design of the Foundation System

The *Gateway Park Geotechnical Report* (Maguire Group Inc., 2005) suggests that shallow foundations should be utilized. To this aim the MQP team decided to design and analyze shallow type foundations. Footings were designed to support both the long-span and short-span structural frame options. The MQP team would then compare the cost of each span's foundation. A typical footing foundation was designed to support a maximum allowable live load and a maximum allowable dead load with the foundation design focusing on just those columns that just resist gravity loads. The footing foundations were designed based on the soil conditions outlined in the *Gateway Park Geotechnical Report* (Maguire Group Inc., 2005). An analysis of the Gateway Park geotechnical report was used to establish the allowable bearing capacities by developing: a soil profile for the site, suitable design soil parameters, and a design chart that was used to size the footings to support various column loads.

3.9.1 Development of a Soil Profile

The purpose of analyzing the boring logs was to develop a soil profile for the site. According to Appendix 1 of *Gateway Park Geotechnical Report* borings MGI 01, MGI 02, MGI 03, MGI 05, and MGI 06 were specfically taken to provide data on the soil conditions at 32 Prescott Street (Maguire Group Inc., 2005). Figure 22 shows the locations of borings MGI 01, MGI 02, MGI 03, MGI 05, and MGI 06 that were used to develop the soil profile (Maguire Group Inc., 2005).

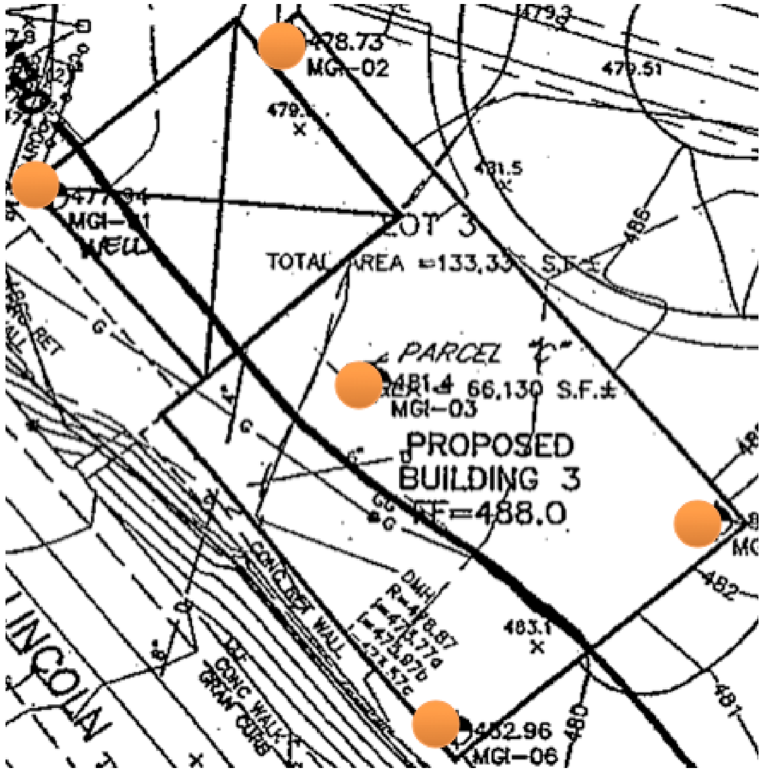


Figure 22: Boring Logs Selected for Development of Soil Profiles

According to *Gateway Park Geotechnical Report* (Maguire Group Inc., 2005) the proposed site will be developed on the existing subsoil and an 8-foot fill. The soil profile was developed by first identifying the soil type and its unified soil classification according to the sample descriptions at various depths that are presented in the boring logs. The results of the subsurface soil exploration based on the Maguire Report are displayed in Table 3.

Table 3: Subsurface Soil Exploration From Maguire Report

	Density	Sand Type	Average Strata Thickness (Feet)	USCS Group Symbol Range
Proposed Fill	Medium To dense	Fine to Medium	8	SP, SM
Surficial Fill "Upper Level"	Medium Dense to Very Dense	Fine to Medium	10	SP, SM
Glacial Outwash	Medium Dense to Very Dense	Fine to Coarse	32	SM, SW, SP, GP, GW

Once descriptions and average strata thicknesses for each soil type were determined, the corresponding unit weights of the soil above the groundwater table and below the groundwater table were obtained utilizing Table 3.2 in *Foundation Design Principles and Practices* (Coduto, 2001). A visual representation was then developed to display the relation between the depth and thickness of each layer, the soil type, the unit weight, and the location of the water table. Figure 23 displays the characteristic soil profile that was developed and used in the design and analysis of the foundations.

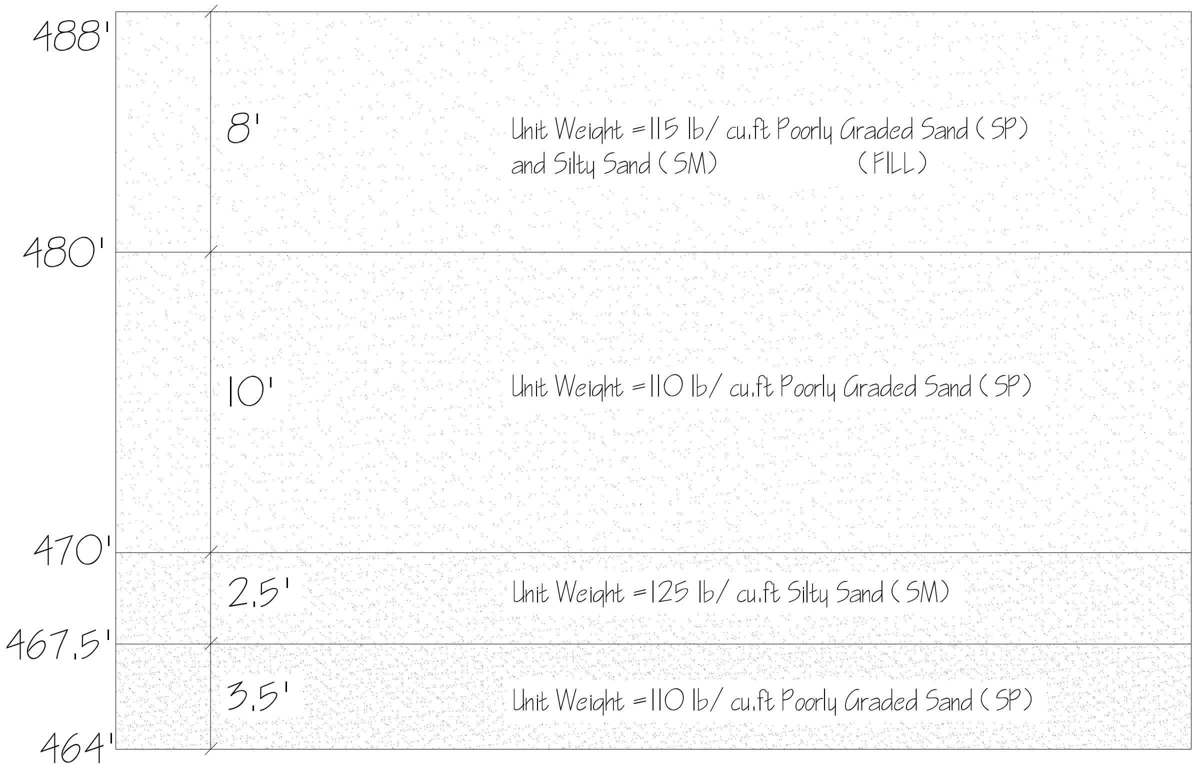


Figure 23: Soil Profile Based on Boring Log Data in Maguire Geotechnical Reports

3.9.2 Selection of a Foundation System

The type of foundation system selected for the design is dependent on the local soil conditions and the individual needs of the building. The *Gateway Park Geotechnical Report* (Maguire Group Inc., 2005) explicitly states that the proposed development at 32 Prescott Street should utilize shallow type foundations (Maguire Group Inc., 2005). The type of footing selected was based on the size of the design loads, soil type at depth, and site constraints such as property lines and the location of the culvert.

3.9.3 Bearing Capacity Considerations

Foundations transmit structural loads, inducing compressive and shear stresses in the supporting soil. If the footing of the foundation is too small, or the soil's bearing pressure is not sufficient, a bearing capacity failure may occur. In this case, the shear stresses exceed the shear strength of the soils. To avoid failure a sufficient design for bearing capacity is required. Consequently the first step required the MQP team to select an appropriate factor of safety. A suitable design factor of safety was selected using the guidelines outlined in Chapter 6.4 of *Foundation Design Principles and Practices* (Coduto, 2001). This value was then checked against the minimum value of two specified by Section 1809.5 of the *International Building Code* (International Code Council, 2009).

The MQP team conducted a bearing capacity analysis. To accomplish this analysis, the bearing capacity spreadsheet developed by Donald Coduto was utilized to determine an appropriate width and length for a spread footing so that it can support the maximum column axial load (Coduto, 2001). The bearing capacity analysis was conducting using the bearing spreadsheet, and the permissible column load, P, was computed that corresponded to the defined factor of safety. The next step involved selecting a series of footing widths, B and determining their corresponding P values. This process was continued until the MQP team computed the value for P so that it was slightly larger than the maximum design column load. Table 4 displays the assumptions and shear strength parameters utilized in the calculations. Although Section 1809.4 of the *IBC* specifies that the minimum embedment depth below undisturbed ground surface is 12 inches (International Building Code, 2009) for areas that experience cold temperatures, the 8th Edition of the *Massachusetts Building Code* states that, "*foundations and other permanent supports of buildings must be protected by "extending below the frost line of the locality" or other methods. The 8th Edition does not specify a particular frost line depth. Four feet has been traditionally accepted in MA as a reasonable default frost line depth for foundation* *design*," (Mass. Building Code, 2011). Section 1809.4 of the *IBC* states that shallow foundations must have a minimum footing width of 12 inches (International Building Code, 2009).

Table 4: Soil Parameters and Assumptions for Determining the Maximum Allowable Axial Load

Soil Parameters and Assumptions	Value	Reason Selected
c(lb/ft^2)	0	Geotechnical Repot by the Maguire Group (Maguire Group Inc., 2005)
***Φ* (degrees)**	32	Geotechnical Repot by the Maguire Group (Maguire Group Inc., 2005)
ϒ(lbs/ft^3)	115	The foot is embedded in the soil to a depth of 4 feet. Based on this fact it lies in the clay soil with the corresponding unit weight.
Depth to Water Table (feet)	18	Based on the design soil profile developed this was the shallowest level observed
Factor of Safety	3.5	Was selected based on guidelines outlined in Chapter 6.4 of *Foundation Design Principles and Practices* (Coduto, 2001). This is a reasonable value for a factor of safety for sandy soil with: minimal site characterization data, moderate soil variability, high importance of structure, and consequence of failure.
Minimum Embedment Depth, D (feet)	4	Was selected based on guidelines outlined in *Massachusetts Building Code 8th Edition* (Mass. Building Code, 2011)

3.9.4 Settlement Analysis

Once an initial spread footing size was determined the MQP team checked the general soil shear case and conducted a settlement analysis to ensure that the foundation will not settle excessively. It is important to do a settlement analysis because if a soil failure doesn't occur due to insufficient bearing capacity, then excessive settlement can cause damage to the foundation or other structural or non-structural aspects of the building. By conducting a settlement analysis it is possible to reduce the differential settlements. A trial-and-error approach was utilized and the value of the footing width was adjusted until the computed settlement matched the permitted value. Coduto's spreadsheet *Settlement Analysis of Shallow Foundations* was utilized.

3.9.5 Structural Design of a Typical Footings

Coduto's spreadsheet *Settlement Analysis of Shallow Foundations* was utilized to determine the minimum footing size that can both sustain the maximum design column load determined in Section 4.10.3 and produce a predicted settlement that is less than the maximum allowable settlement. Equation 8 displays the design requirements for settlement. There is no factor of safety in Equation 8 because the factor is already included in δ_a.

$$\delta \leq \delta_a$$

Equation 8: Design Requirements Based on Settlement

Based on the information presented in Table 2.1 of *Foundation Design Principles and Practices* the maximum allowable settlement ranges from .5 inches – 2.0 inches (Coduto, 2001). The smallest footing width that can satisfy both strength and settlement requirements was selected. The design was limited to a concentrically loaded footing, and consequently column bases subjected to overturning moments and base shear were not considered.

After the development of the plan dimensions and minimum embedment depth of the spread footings, the next steps involved structural design of the reinforced concrete footing. The structural design is important because it ensures that the foundation has sufficient structural integrity to safely transmit the design loads from the structure to the ground. A concrete strength of 4000 psi and reinforcing steel of 60,000 psi were utilized to determine the thickness of the foundation and the size, number and spacing of reinforcing bars.

The structural design of the footing was completed in compliance with *ACI-318* standards. In addition, the embedment depth to the base of the footing was checked against the *Massachusetts Building Code* criteria. Design results were presented as typical details and drawings. Figure 24 provides a visual representation of the concepts of footing width, B, embedment depth, D, footing thickness, T, column width, c, in relation to the axial load, Pu, and the column base

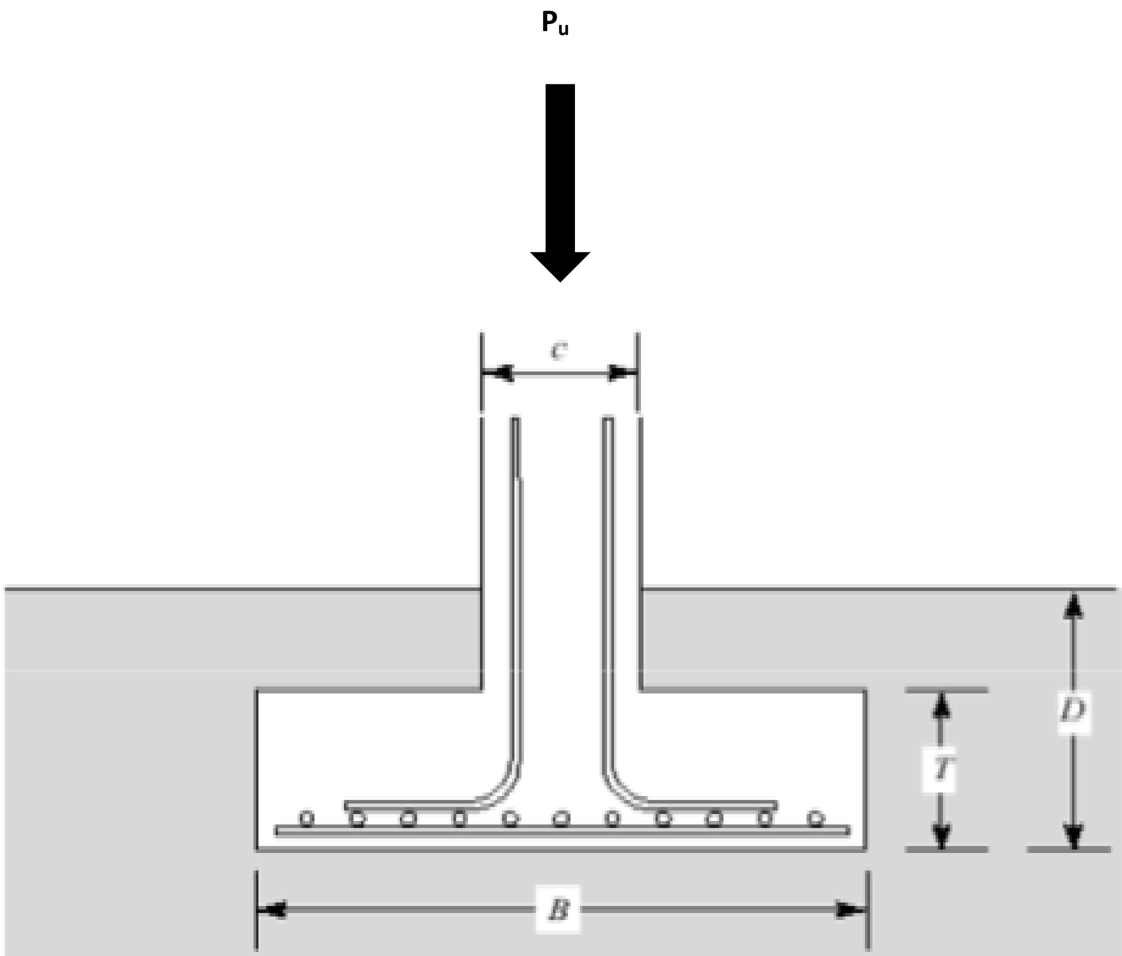


Figure 24: Square Spread Footing (Prieto-Portar L. , 2009)

3.10 Development of Drainage Calculations

To assess the expected increase in storm water runoff to the surrounding areas certain assumptions had to be made in order to accommodate the inherently unpredictable nature of rain storms. To ensure the design storm events exceed the City of Worcester's 25 year storm design requirements, several assumptions were used to calculate the added impact of the 2, 10, 25, and 100 year storms to the area. The rational method was used to complete all calculations (Portection, 2002) The 25 year storm was applied as a baseline for design with a factor of safety, and the 100 year storm was used to present a worst case scenario.

The following assumptions were used for these calculations:

- Original runoff coefficient: 0.15 (lawn with heavy soil and mostly flat)
- Final runoff coefficient: 0.5 (light industrial) (Portection, 2002)
- Rainfall intensity for 2, 10, 25, and 100 year storms are 3.0, 4.5, 5.3, 6.5 inches per hour respectively for the Worcester, MA area. (Portection, 2002)

STEP 1: Peak Flow rate from each storm

a) This was done for current site conditions and then a second time for future site conditions.

   1) $Q = ciA$

      i) Q is the peak flow rate in cubic feet per second

      ii) *i* is the rainfall intensity

      iii) A is the area of the site in question

STEP 2: Find the pre-development volume of runoff for each storm

a) The volumes were found by multiplying the flow rates seen in STEP 1 with the corresponding time of concentration for the 2, 10, 25, 100 year design storms.

   1) $V = \Delta Q t_c$

      i) V is the volume

      ii) ΔQ is the flow rate increase

      iii) t_c is the time of concentration for the design storm. (t^{-}_{c} is 45 min) (Portection, 2002)

STEP 3: Find the additional volume for each storm

a) When calculating the volume the assumption was made that the pre-existing conditions of the site were acceptable and therefore only the impacts of the proposed development were assessed.

1) $\Delta V = V_{fin} - V_{init}$

i) V_{fin} is the final Volume

ii) V_{init} is the Volume

Once the added volumes had been found, the team investigated options for where the additional water should be diverted. The first solution considered was a ground water infiltration system to retain much of the runoff on the site. This was ruled out due to the high water table (18 feet). With such a high water table it would be potentially dangerous to the building's foundations to reintroduce the water by these means. Since infiltration systems were ruled out it was decided that the excess water will be diverted to the Millbrook Conduit, which is where storm water is currently diverted.

3.11 Traffic and Parking Analysis

The traffic and parking analysis was conducted in four major steps that coincided with the usage of the building. This enabled the team to ensure that the flow to and from the building will not impede the traffic flow in the surrounding area. Figure 25 displays the intersection of Salisbury St. and Grove St. The intersection was selected for its close proximity to the site. This intersection is expected to be the most heavily impacted by the construction of this building as seen in the EIR traffic analysis. (Group, 2008)

Figure 25: Intersection of Salisbury Street and Grove Street3

STEP 1: Conduct an intersection traffic analysis

The first step was conducting an intersection traffic analysis to provide baseline data for the Grove Street/Salisbury Street intersection. This included completing traffic and turning counts for the intersection. Using the computer program *MCTrans: HCS2000* this intersection's level of service was evaluated and compared to the projected LOS in the *Gateway Park Master Plan* (McTrans Moving Technology, 2011).

STEP 2: Approximate trip generations

The second step was approximating the number of trips per day that this new construction will bring to the area. This was estimated by using the *ITE Trip Generation Handbook* (Engineers, 2008). The MQP team followed the procedure outlined in Chapter 7.5 Procedure for Estimating Multi-Use Trip Generation of the *ITE Trip Generation*

Handbook with a few modifications to accommodate the mixed use construction (Engineers, 2008).

The process of developing an accurate estimate of trips generated was challenging because the *ITE Handbook* does not include trip generations for mixed use developments. To ensure an accurate result the MQP team compiled an estimate based on the occupancy expectations of each part of the building. This procedure enabled the MQP team to estimate how many trips will be introduced since different occupancy use-groups will generate different volumes of traffic.

The expected trips generated by residents include an expected number of cars per household. This expected number of trips and vehicles was then reduced by the expected value of residents that live on site and will not have vehicles. The reduction was taken for a compilation of college campus' statistics from across the country for data see APPENDIX F: MODAL SPLIT DATA. An example of the modal split data collected in a study of Ohio State University is as follows (Flynn, 2011):

1. Example
    a. 70% walk
    b. 19% cabs
    c. 5% car
    d. 6% bike
2. Summary 76% of campus residents do not have cars on campus

The estimated traffic volumes were then compared to the ones used in the *Gateway Master Plan* to confirm or refute expected increased loadings. If variations greater than 20% existed between the *Gateway Master Plan* and the estimates generated by this MQP team, then further studies of the intersection would have been conducted to confirm the team's results for level of service (LOS).

STEP 3: Pedestrian traffic

Due to on-site housing units there may be a significant increase in pedestrian traffic. This increase in pedestrian traffic could necessitate more crosswalks and, in turn, affect traffic flow. To accommodate this increase a curb cut and cross walk is recommended directly across from the main entrance to the building to ensure accessibility for people with mobility issues access to surrounding buildings.

STEP 4: Parking

Due to the usages of the building the tenants will require both day and night parking accommodations. Once the expected traffic in and out of the building was assessed, an expected need for number of spaces was compiled and then used in the design and expansion of surface lots and parking garages.

3.12 Development of a Preliminary Cost Estimate

WPI is interested in achieving the lowest possible overall project cost that will accomplish its objectives outlined in goal 7 of the *WPI Strategic Plan*. The cost estimate is vital to the development of the project since it gives the owner an idea of the expected cost of the project prior to construction.

A cost estimate was developed for the building at Gateway Park primarily using *RS-Means* square footage estimate values. (RSMeans, 2012) Square footage costs were not used for the steel cost and concrete cost because sufficient design information was available to base costs on a quantity takeoff. These two aspects were tied into the structural aspects of the building and included a long span and short span comparison. The steel cost was compiled on a price per pound basis and the concrete cost was estimated based on cubic yards required.

Since this is a multi-use building four different estimates needed to be combined to produce an accurate figure. The four combined estimates were for, university lab space, residential housing, restaurant or mercantile use, and industrial space. If there were ever any question as to which aspect should be used the most expensive option was chosen to ensure the final estimate would be conservative.

3.13 LEED Certification

For this project, designing a building to obtain a level of the U.S. Green Building Council's LEED Certification was a primary goal. In the past, WPI has built two buildings that earned some level of LEED certification, with East Hall being the most efficient on campus achieving a Gold Certification. Obtaining LEED certification is based upon obtaining a benchmark number of points that help to make a building more sustainable within the environment. The criteria for different levels of LEED certification is outlined in Table 5 (U.S. Green Building Council, 2011).

Table 5: LEED Point Classification Criteria

Level of Certification	Number of LEED points required
Certified	40-49
Silver	50-59
Gold	60-79
Platinum	80 +

The USGBC's document, *LEED 2009 for New Construction and Major Renovations Rating System,* is broken down into seven sections where possible points can be obtained. Five of the seven sections have prerequisites that are required before any points can be obtained in the category. Table 6 presents the possible number of points available in each category.

Table 6: Possible LEED Points per Category

Category	Total Possible Number of Points
Sustainable Sites	26
Water Efficiency	10
Energy and Atmosphere	35
Materials and Resources	14
Indoor Environmental Quality	15
Innovation and Design	6
Regional Priority	4
Total	**110**

During the construction scheduling and build out the owner needs to inform the contractor of the final LEED target. While the project is being developed precautionary steps must be taken to ensure environmental protection. Due to the type of design and analysis of the building a primary LEED estimate was conducted using the criteria specified in *LEED 2009 New Construction and Major Renovations*. A secondary estimate was also conducted to include expected points earned after construction completion. These points are based on the assumption that throughout construction and during the purchasing of equipment, excluding those in this proposal for the building, the LEED criteria will be consulted to ensure a sustainable new addition to the community.

3.14 Development of a 3D Model with Revit

The MQP team utilized *Autodesk Revit Architecture* and *Autodesk Revit Structure* to develop 3D models of the proposed structure. Prior to developing the 3D models *Google Earth* and *Civil AutoCAD* were used to import a *Google Earth* image of the site and the surrounding environs. After the *Google Earth* image was imported, an architectural model was developed to display the floor layout, room configurations and exterior finishes. Additionally, *Autodesk Revit Structures* was used to develop two separate structural models to depict the beam and girder layouts and sizing for the long span and short span structural frame options.

4 Findings

This section presents the MQP team's results from the objectives outlined within Section 4 of this document. The findings section has been broken down into seven major parts which represent the main results from the MQP team's design and analysis. This MQP will present: a functional layout and floor plans, a structural analysis, an evaluation of the impact on existing traffic and parking conditions, and a preliminary construction schedule and cost estimate. The findings will be presented in the following order:

- Programming phase
- Comparison and selection of a design alternative
- Building layout and framing plan
- Structural design
- Evaluation of the impact on existing traffic and parking conditions
- Construction cost estimate
- Obtaining LEED certification
- Revit architectural model

4.1 Programming Phase

The programming phase is designed to translate the objectives for a facility into functional spaces and their associated floor areas. The proposed development located at 32 Prescott Street lies within the Mixed Use Development Zone Overlay which "is intended to provide for the coordinated and mixed development of residential, business, institutional and open/recreational space uses the City of Worcester" (City of Worcester, 2011).

The building usages for 32 Prescott Street were determined using the Worcester Zoning Ordinance and feedback from WPI President Berkey and Jeff Solomon, WPI Vice President/ Chief Financial Officer of Finance and Operations. Based on the information provided by President Berkey the proposed development will be divided into four main categories: retail, industrial, research and development, and residential. In order to complete the space allocations the needs of every intended occupant of the building must be taken into account.

A programming phase was developed for both conceptual design A and conceptual design B. For both conceptual design A and B the entire first floors will be utilized as retail space. This area is ideal for retail space due to its curb appeal, serving to easily attract passersby with ample store front for road display. The office space will provide a location for research and development companies to compile data, plan sales, and take care of paperwork. The residential portion will satisfy WPI's needs for its currently nonexistent graduate housing, or as potential housing for researchers at Gateway Park. A major advantage of locating the graduate housing in the same complex as Gateway Park is that many graduate students can walk to their labs or classes, reducing vehicular traffic during the day from the added lab and office space.

The increase in laboratory space at Gateway Park will also allow for the addition of much needed research facilities for the life sciences. The life sciences are one of the fastest growing areas of development in Worcester. Along with being a developing industry in Worcester, it is furthermore a swiftly expanding major here at WPI. One of the added benefits of sharing this building with outside companies is the possibility for WPI graduates to procure jobs in a growing industry. All of the research lab space being created also creates a need for supporting office space within close proximity.

Based on the group's understanding of WPI's goals as outlined in the WPI *Strategic Plan,* the MQP team was able to allocate square footages per floor to each of the major building uses. Table 7 and Table 8 display the allocation of square footages to building usages for conceptual design A and conceptual design B. In conceptual design A the uses of the two buildings have been broken up both by floor and by occupant. All the upper floors of the smaller building are allocated to residential dwellings. The larger building is broken up to contain three usages. As stated previously the first floor is retail. The second through the third floors are for industrial usage, and the fifth through the eighth floors are used for research and development. In conceptual design B there are two legs of the building, the East and West legs. For the first floor, two restaurants are proposed on the East leg, while retail space is proposed on the West leg. The second and third floors are both designated for industrial usage. The fourth and fifth floors are for research and development. The sixth through eighth floors are allocated to residential dwellings.

Table 7: Allocation of Square Footages to Building Usages for Conceptual Design A

Building	Floors	Building Space Per Floor(sf)	Retail(sf)	Industrial (sf)	Research & Development(sf)	Residential Units(sf)	Total Proposed (sf)
Building 1	1	19,000	19,000				19,000
	2,3,4	19,000				57,000	57,000
Building 2	1	18,000	18,000				18,000
	2,3,4	18,000		54,000			54,000
	5,6,7,8	18,000			72,000		72,000
							220,000

Table 8: Allocation of Square Footages to Building Usages for Conceptual Design B

Floors	Building Space Per Story(sf)	Retail(sf)	Industrial (sf)	Research & Development(sf)	Residential Units(sf)	Total Proposed (sf)
1	25,088	25,088				25,088
2;3	25,088;31,091		56,179			56,179
4;5	31,091			62,182		62,182
6;7;8	31,091				93,273	93,273
						236,722

In both conceptual designs the floors are used as barriers between individual uses. Industrial use is kept on the lower floors to ensure heavy materials and machinery will remain near the ground floor. Office space is located in the middle of the building to maintain a close proximity to the industrial space that it is intended to support. The location of the office space will also serve as a noise buffer between the industrial space and the residential dwellings.

The maximum allowable square footage for the development is constrained by the maximum allowable square footage for Gateway Park as approved during the Environmental Permitting process. This is because lot 3 in the *Gateway Master Plan* was allocated an additional 20,000 square feet, and the size of the entire development is constrained by a maximum allowable square footage. Both conceptual designs are below the maximum allowed area of 240,000 square feet since conceptual design A has a total area of 220,000 square feet while conceptual design B has a total area of 230,000 square feet.

It was important to examine the restrictions on floor areas since the Mixed Use Development Zone Overlay within the Worcester Zoning Ordinance specifies that the total residential use shall not exceed more than fifty percent of the gross floor area, and a single non-residential use shall not comprise more than 75% of the gross floor area of the development.

4.2 Comparison and Selection of Conceptual Design

A first-order analysis of two preliminary designs was considered: conceptual design A, involving two separate buildings, and conceptual design B involved one bridged building from the third to top floor. Accessibility of the Millbrook Culvert, construction costs, time for construction, site planning and zoning restrictions, maximization of green spaces, and the aesthetic impact on Gateway Park were the six major attributes defined for comparisons to help determine a more suitable building solution for 32 Prescott Street.

The first comparison between the two conceptual designs was based on the accessibility of the culvert. Initially two buildings were proposed for development on lot 6 because this would enable the culvert to be easily accessed for maintenance. The MQP team proposed conceptual design B, a single bridged building, which leaves sufficient clear space for heavy equipment to access the culvert below. Thus, when these two design options are compared, there is no advantage regarding culvert access since both options leave the culvert fully accessible.

Next, the potential cost of construction was compared for the two proposals. President Berkey and the *Gateway Master Plan* both planned on having the two buildings in conceptual design A built in two separate phases, approximately 5-7 years apart (Gateway Park Master Plan, 2001). When considering the current costs to construct two buildings, the second building, even if the same size, would have increased construction costs due to inflation. Furthermore, mobilization costs would be double for having two separate projects spread out over a few years. However, the cost of constructing conceptual design B would face challenges too with a more complicated design consisting of a single bridged building.

The time for construction was another major consideration in this project. Conceptual design B would be completed earlier and thus would start generating revenue sooner than the two-phased construction of conceptual design A. Furthermore, with pedestrian traffic and construction noise issues, one phase of construction, as is considered in conceptual design B, would be safer and cause less issues.

Next, the group considered the zoning restrictions and permitting costs. Both conceptual design options are considered a mixed-use development overlay according to the Worcester Zoning Ordinance. The permitting costs and fees for conceptual design A would be higher than conceptual design B since there are two separate projects occurring at two different periods of time; this would lead to an increase in the number of site inspections that would have to take place. Accordingly, conceptual design B would have an advantage when considering the legal and permitting aspects of this project.

Maximizing green space is another important aspect now more than ever before. With two separate buildings, conforming to the maximum number of stories by the *Gateway Master Plan*, the amount of pervious space for conceptual design A is 55%. For conceptual design B, the MQP Team was able to reduce the base level foot print of each building (below the level of the bridge) and thus increase the open space on the lot to 70%. Although the total square footages of both buildings are nearly identical, conceptual design B can gain approximately 6,000 square feet per floor where the building is bridged.

The MQP Team took LEED certification into account with regard to the development footprint. For 32 Prescott Street, the open space needs to exceed 17,512.9 square feet to obtain this LEED point; both development footprints meet this requirement and would obtain this applicable LEED point. Other criteria for LEED certification is discussed in greater detail in Section 5.10 Obtaining Gold LEED Certification.

Lastly, the aesthetic appeal of both conceptual designs was examined. For conceptual design A, there is the potential of constructing two “sister” buildings with each building having complementary features. Conceptual design B also has much potential to be aesthetically pleasing, since a bridge between the two buildings could be considered the “gateway” to Gateway Park with a footpath going between the two bridged buildings.

The MQP team took all of the abovementioned factors into consideration for the selection of a conceptual design. Group members were assigned to allocate 4 points for each abovementioned factor. The choice of 4 points was determined for the case that neither design option has a clear advantage so two points each could be allocated, yet it still leaves room for one design to receive a slight advantage over the other (3 points versus 1 point). The scores are summarized in Table 9. Each design has a total score presented; this is the total score based on the sum of each group member’s allocation of points. The maximum total number of possible points for a design option is 72.

Based on the allocation of points, conceptual design B received the higher score by 22 points. Thus, the MQP team decided that conceptual design B would have more advantages for WPI. Furthermore, conceptual design B would create a unique project for the MQP team and pose its own set of new challenges that the team would like to address. The rest of this MQP was developed to investigate conceptual design B (a single, bridged building).

Table 9: Comparisons between Conceptual Design A and Conceptual Design B

	Accessibility of Culvert	**Cost of Construction**	**Time for Construction**	**Site Planning and Zoning Restrictions**	**Maximization of Green Spaces**	**Aesthetical Impact on Gateway Park**	**Total Score**
Conceptual Design A	-Fully Accessible	-Since constructed in two phases inflation increases construction costs -Mobilization costs will double	-Increased time for construction since buildings built in two main phases	-Mixed use overlay development permits two buildings -Additional permitting costs for two separate buildings	-56% of lot space unoccupied by structures - One additional LEED point	-Potential to develop two "sister" buildings	
Score	6	4	1	5	5	4	25
Conceptual Design B	-Fully Accessible	-Additional need for bridging the two buildings increases cost -All built at once therefore, decreases mobilization costs	-One main phase of construction which results in relatively shorter time for construction	-Less permitting costs since only one building	-70% of lot space unoccupied by structures - One additional LEED point	-Since located right by I-290 and Route 122 a single building with an bridge connecting both sides could have a more profound impact	
Score	6	8	11	7	7	8	47

4.3 Building Layout and Framing Plan

Based on the group's understanding of WPI's goals as outlined in the *WPI Strategic Plan*, the MQP team was able to allocate square footages per floor to each of the major building uses. Figure 26, Figure 27 and Figure 28 display the floor layout.

Figure 26: Building Layout Floors 1 and 2

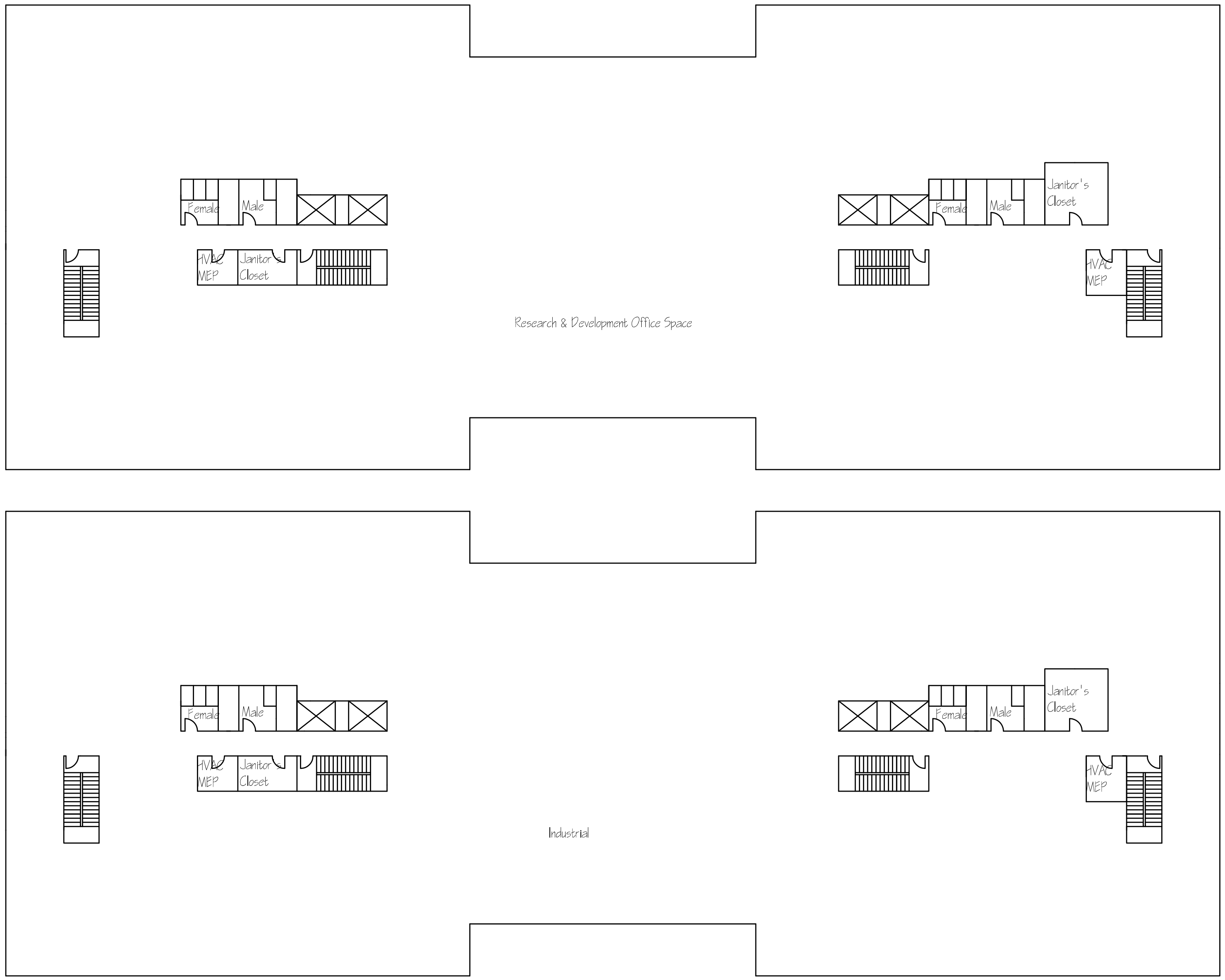


Figure 27: Building Layout for Floors 3 Through 5

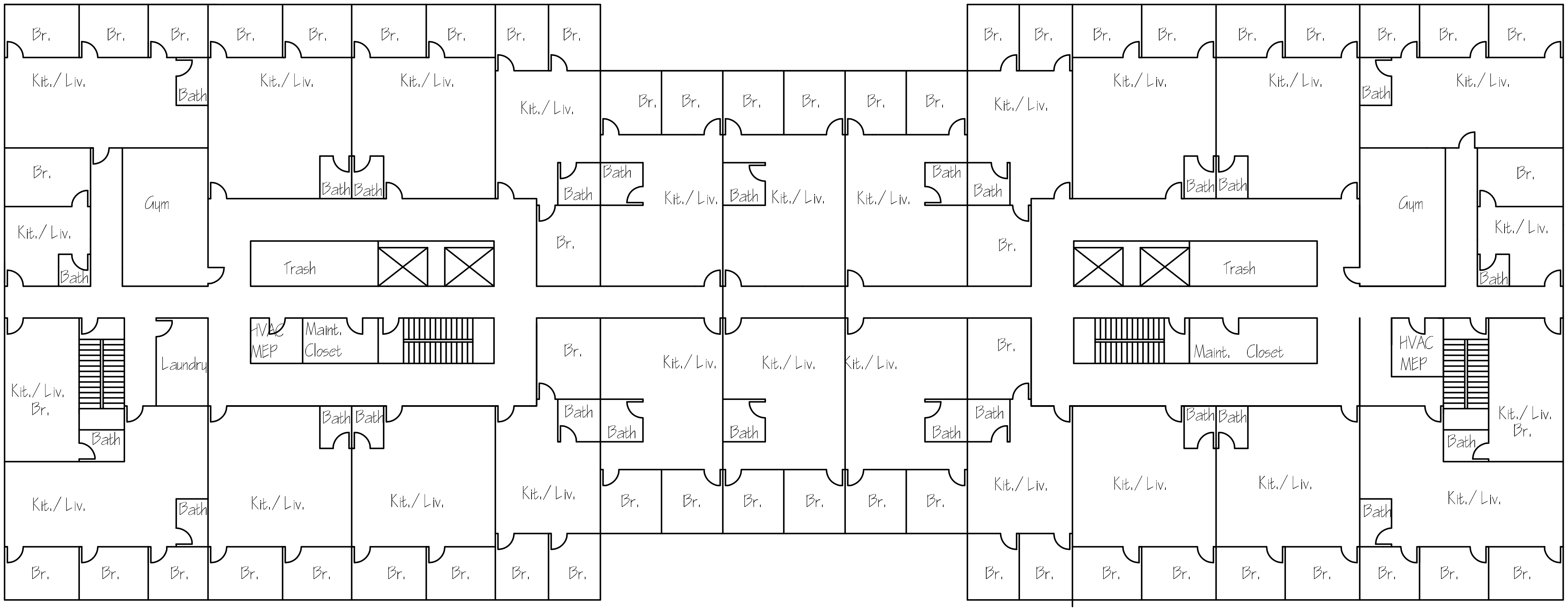


Figure 28: Building Layout for Floors 6 Through 8

4.4 Structural Design

For the structural design, there were two framing plans considered: short span and long span. One of the objectives of this MQP was to determine whether the short span or long span would have an economic advantage over the other. The cost of steel for the framing including labor and the cost of cement utilized for the foundations were calculated to determine if one of the spans would have a cost advantage. The framing plan for the long span results in a reduced quantity of beams, girders, and columns, so the MQP team decided to investigate if this reduction in total number of members was offset by the increase in the size of structural steel members.

Although each individual member would be sized differently due to different bay sizes and loading patterns the MQP team decided to design for a typical beam and girder. A typical beam and girder were selected based on the largest tributary area and then used as typical designs throughout the building. It should be noted that this is a conservative approach; however, it could save time in the field or save fabrication costs by standardizing the connections between similar sized members. For the gravity load analysis, all calculations were initially done by hand and checked with an Excel spreadsheet. Systems of rigid frames were chosen as the lateral load resisting system for this MQP. For the consideration of lateral load effects, *MASTAN2* was utilized to determine moments and axial loads on the structure since the frame is statically indeterminate. In particular, the frame has a unique design due to the utilization of a Vierendeel frame to bridge together the two sides of the building and create elevated usable space.

The results for structural design and foundations will be presented as follows:

- Long span
- Short span
- Vierendeel frame
- Design of Typical Connections
- Foundations
- Revit Model
- Evaluation and selection of a design alternative

4.4.1 Long Span

Figure 29 shows the 3-D model of the long span design. The steel design details of this span option are presented in the following sections.

4.4.1.1 Beam and Girder Design

Figure 30 displays the beam and girder spacing in addition to typical member sizes. All sections utilize composite construction with 5/8-inch diameter shear studs, and the stud spacing is indicated as note on the diagram. For the long span, all beam sections were designated to be W24X55 sections. For the long span, all girder sections were designated to be W24X55 sections, the same size as the beams. The calculated deflections and relevant calculations are summarized in APPENDIX A : STRUCTURAL CALCULATIONS. Table 10 summarizes the number of sections for the long span.

Table 10: Beams and Girders Summary for Long Span

	Beams	**Girders**
Size	W24X55	W24X55
Number of Sections	882	280

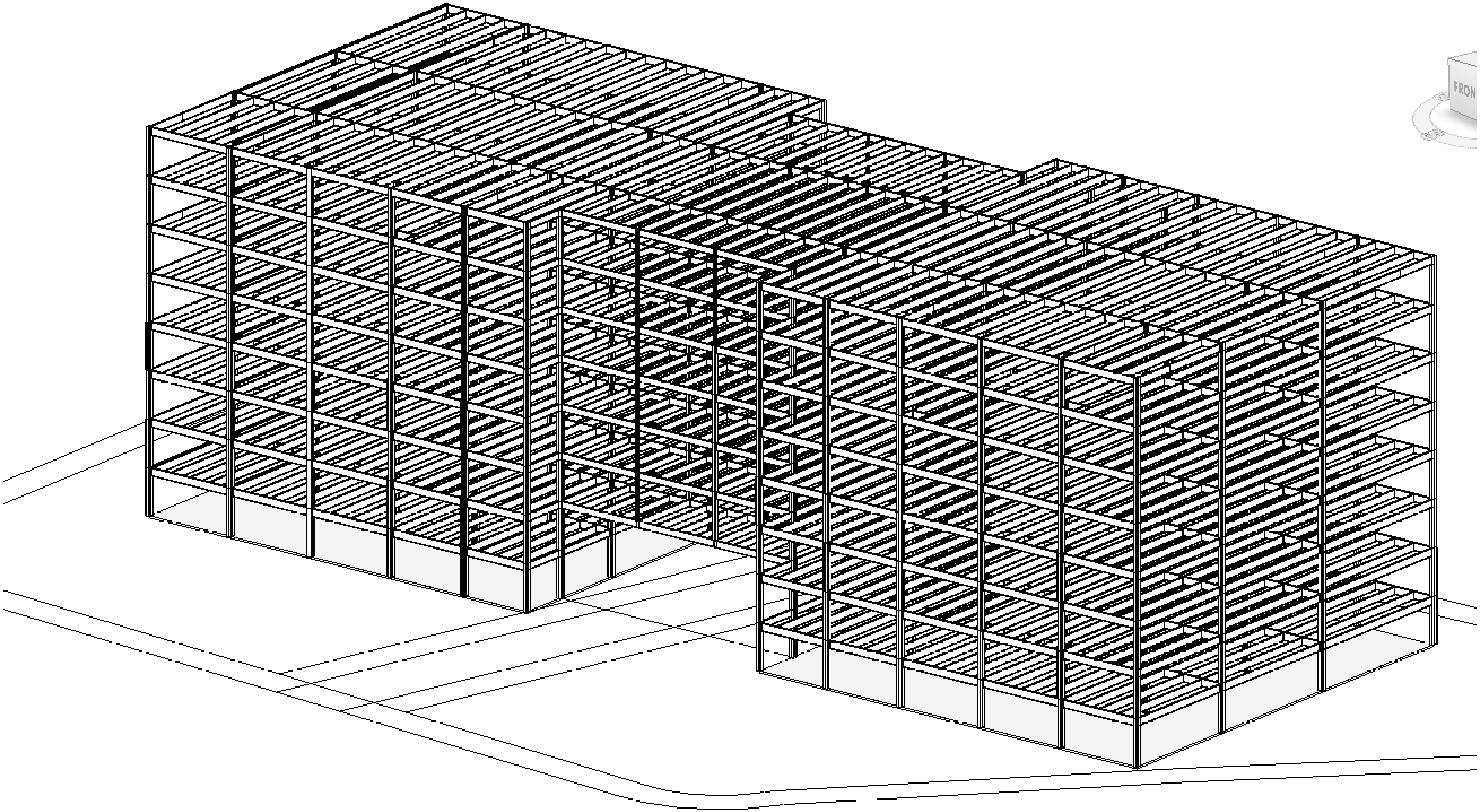


Figure 29: Long Span Structural Framing System Generated Using Revit

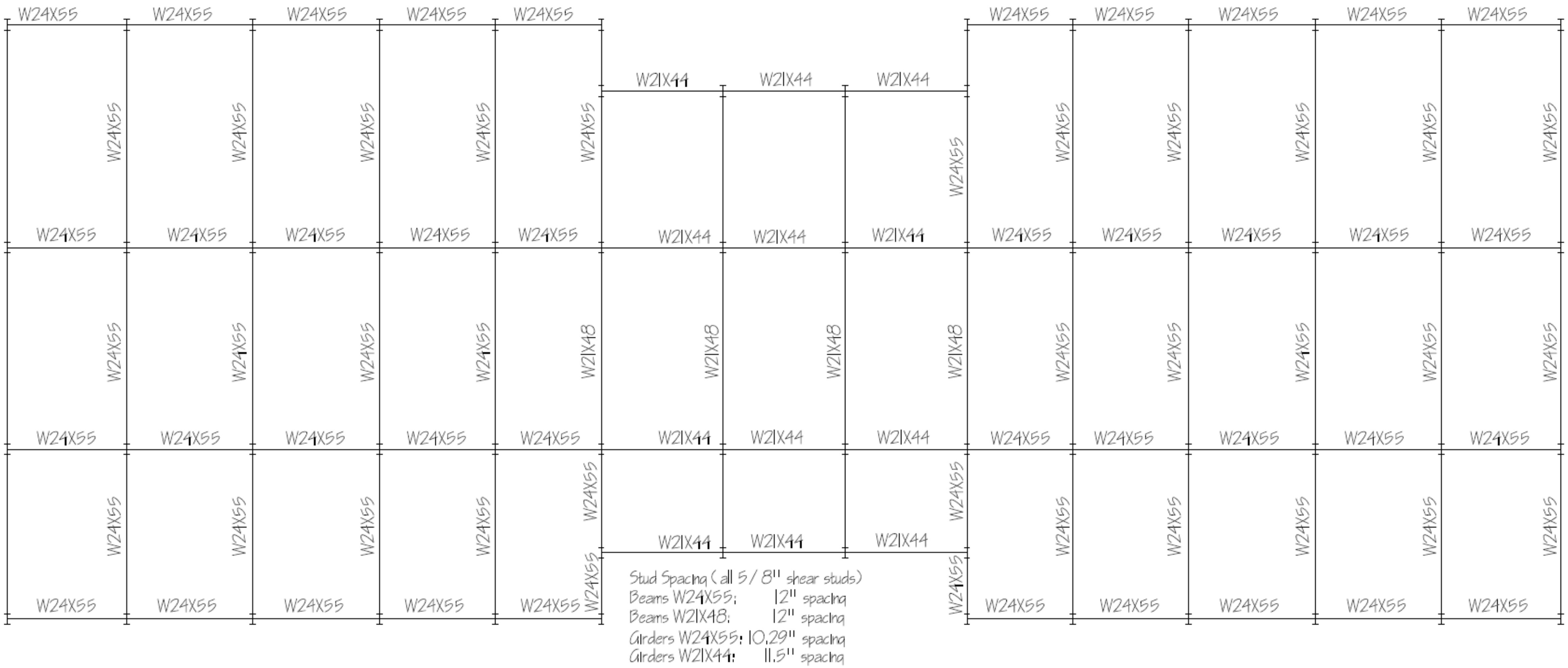


Figure 30: Long Span Typical Beam and Girder Sizes with Stud Spacing

4.4.1.2 Column Design

Gravity loads were used to establish preliminary column sizes. However, when lateral load considerations were present the size of the members changed significantly. Even in a moment resisting frame it is very important to consider both wind and seismic effects, especially when considering the size of such a building and its relative mass. Table 11 displays a summary of the column sizes for the long span.

Table 11: Column Size Summary for Long Span

Story	Gravity Loads	Combined Gravity and Lateral Loads	Number of Columns
1-2	W12X79	W12X106	44
3-4	W10X54	W12X65	44
5-6	W10X39	W12X65	44
7-8	W8X31	W12X53	44

4.4.2 Short Span

Figure 31 shows the 3-D model of the short span design. The steel design details of this span option are presented in the following sections.

4.4.2.1 Beam and Girder Design

Figure 32 displays the framing plan with the required beam and girder sizes. All sections utilize composite construction with 5/8-inch diameter shear studs, and the stud spacing is indicated as note on the diagram. For the short span, all beam sections were designated to be W12X19 sections. For the short span, all girder sections were designated to be W18X40 sections. The given deflections and relevant calculations are outlined in APPENDIX A : STRUCTURAL CALCULATIONS. Table 12 summarizes the number of sections required for the short span.

Table 12: Beam and Girder Summary for Short Span

	Beams	Girders
Size	W12X19	W18X40
Number	1792	490

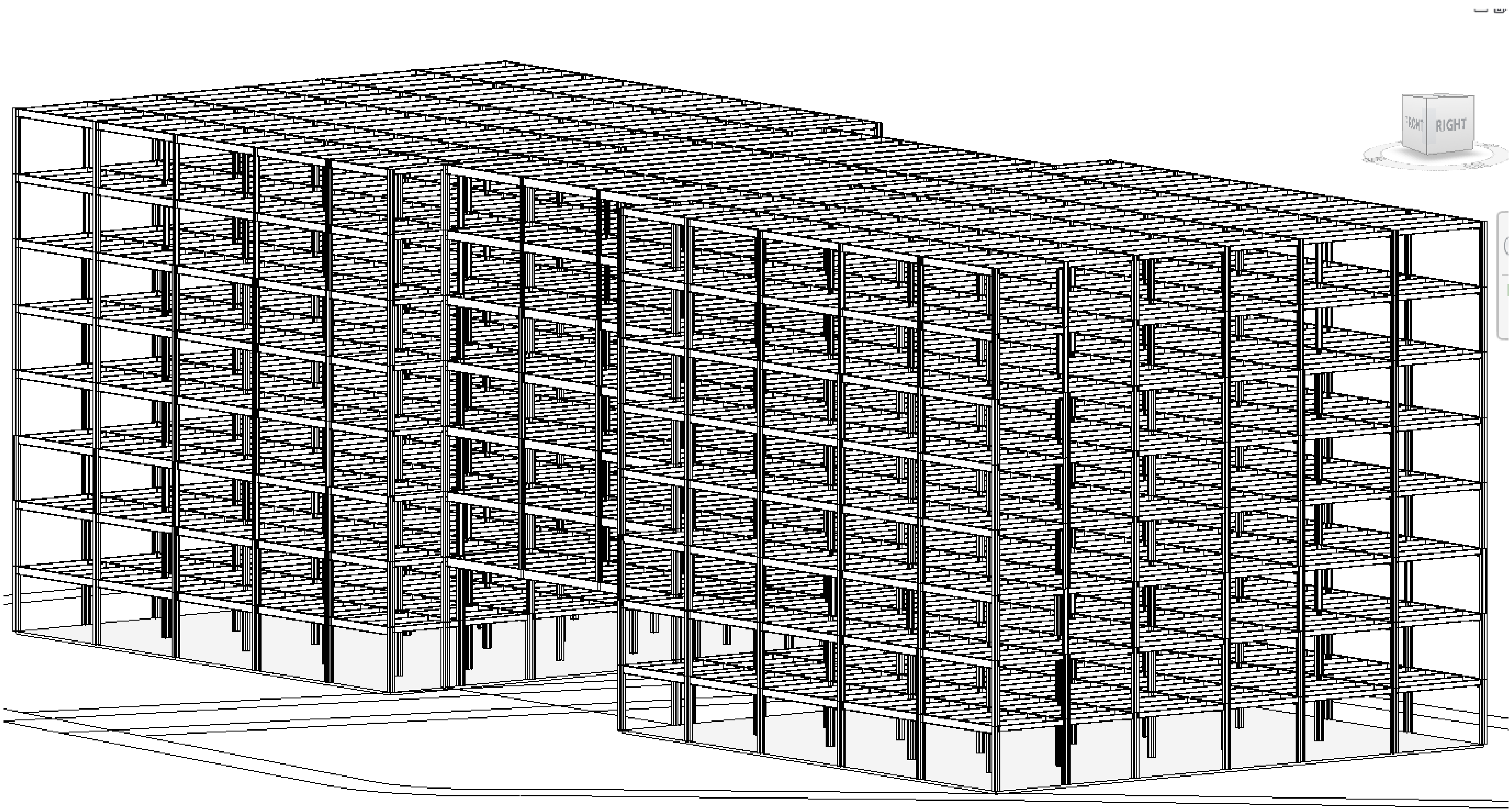


Figure 31: Short Span Structural System Generated by Revit

Figure 32: Short Span Typical Beam and Girder Sizes with Stud Spacing

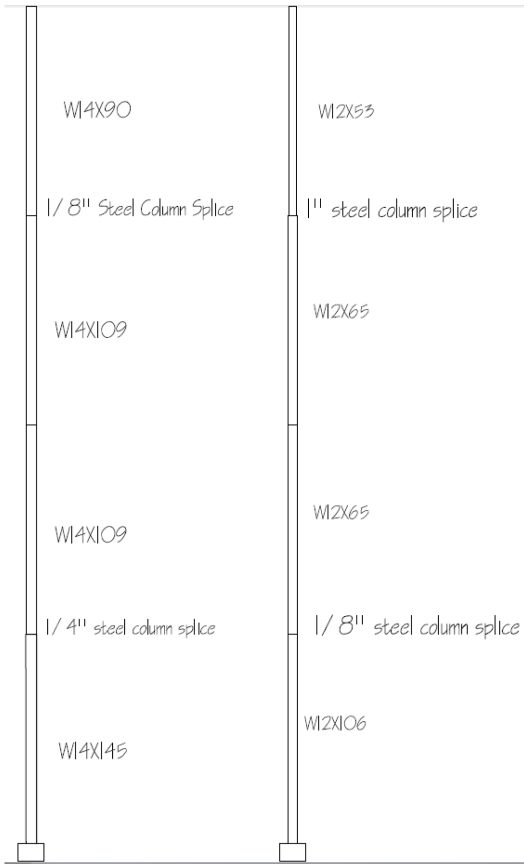


Figure 33: Columns sizes with combined axial and lateral loads, Long Span is on left and Short Span is on right

4.4.2.2 Column Design

Similar to the long span design, the column sizes were first defined according to gravity loads only. Similar to the column designs for the long-span option, when lateral load considerations were present, the size of the members changed significantly. APPENDIX A : STRUCTURAL CALCULATIONS

Table 13 and Figure 33 displays the column sizes for the short span for combined axial and lateral loads. APPENDIX A : STRUCTURAL CALCULATIONS discusses the required increase in columns sizes.

Table 13: Column Sizes for Short Span

Story	Gravity Loads Only	Combined Gravity and Lateral Loads	Number of Columns
1-2	W14X145	W14X145	80
3-4	W14X99	W14X109	80
5-6	W12X65	W14X109	80
7-8	W8X31	14X90	80

4.4.3 Vierendeel Frame

For the Vierendeel frame, beam and girder sizes were consistent; however, the size of the "shared" columns changed based on the differences in tributary area and resultant reaction forces for the two different spans.

4.4.3.1 Beam and Girder Design

For the Vierendeel frame, all beam sections were designated to be W21X48 sections. These beams will use 5/8" shear studs. All girder sections were designated to be W21X44 sections. These girders will use 5/8" shear studs. When calculating the size of these members, the deflections governed design. The calculated deflections are summarized in APPENDIX A : STRUCTURAL CALCULATIONS

4.4.3.2 Column Design

The Vierendeel frame presented a unique set of challenges for the design of the columns. The interior columns do not carry weight to the ground, instead their main purpose is to transfer vertical shear through to the girder sections. The two lines of columns that are "shared" between the Vierendeel frame and either the short or long-span main frames have larger moments and axial forces than the other columns of the main frame since these columns are transferring the loads from the Vierendeel frame to the ground as well as loads from their respective tributary areas. The shared columns are highlighted in red, and the interior columns are highlighted in green in Figure 34. Table 14 displays the short span and long span Vierendeel frame columns respectively.

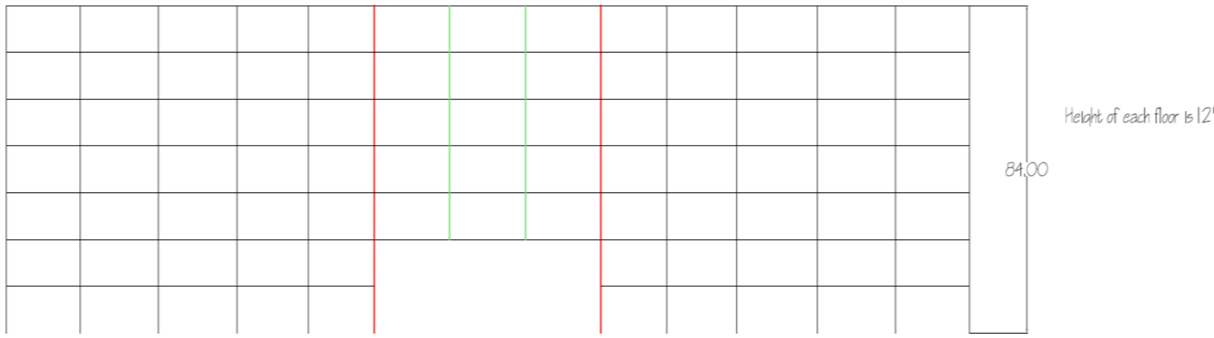


Figure 34: Identification of Interior and Exterior ("Shared") Columns within the Vierendeel Frame

Table 14: Long Span Vierendeel Frame Columns

Column Type and Story	Long Span Column Size	Short Span Column Size
Shared 1-2	W14X145	14X132
Shared 3-4	W14X132	14X90
Shared 5-6	W14X109	14X90
Shared 7-8	W14X90	14X90
Vierendeel 3-4	W10X88	W10X88
Vierendeel 5-6	W10X54	W10X54
Vierendeel 7-8	W8X31	W8X31

4.4.4 Design of Typical Connections

For the economic evaluation of short span versus long span, the MQP team did not calculate the cost of connections separately for each span type. Instead, the MQP team designed a "typical" beam-to-girder and a girder-to-column connection using the W24X55 beam and girder sizes that comprise the framing for the long-span design. Both single angle connections use ¾" diameter Type A325-N bolts.

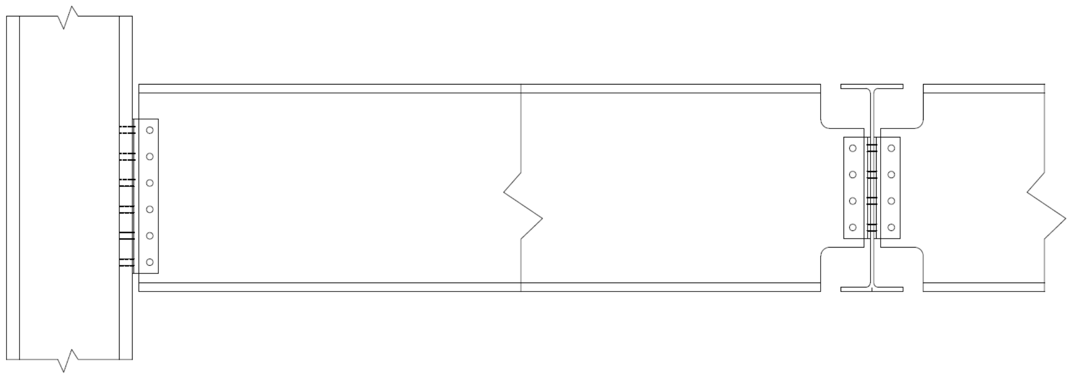


Figure 35: Full System with Connections

4.4.5 Beam-to-Girder Connection

For the beam-to-girder connection, a L3X3X3/16X11.5 single angle would be used. Figure 36 shows the detailing dimensions for one of the angles, and Figure 37 shows a drawing of the connection itself.

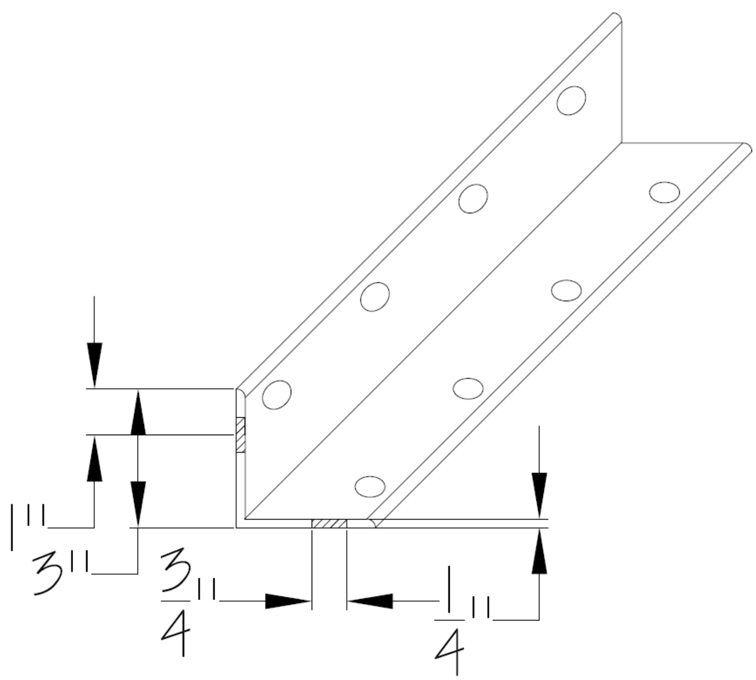


Figure 36: L3X3X3/16X11.5 Angle, 3/4 Bolts Spaced at 3" Center-to-Center

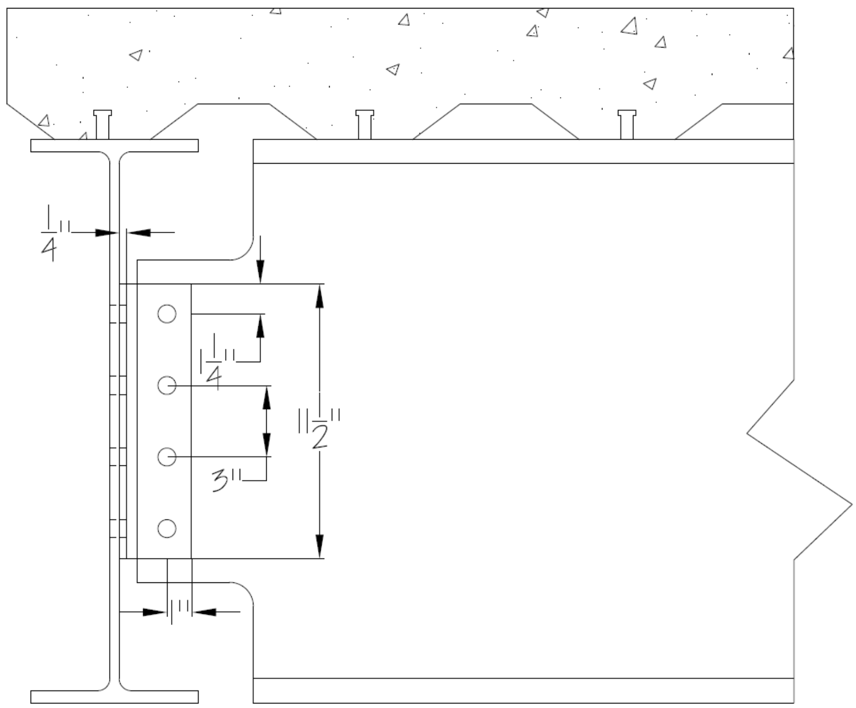


Figure 37: Full Detail of Beam-to-Girder Connection with Metal Decking and Concrete Slab

4.4.6 Girder-to-Column Connection

For the girder-to-column connection, a L3X3X5/16X17.5 angle would be used. Figure 38 shows the dimension of the angle, and Figure 39 shows a drawing of the connection itself.

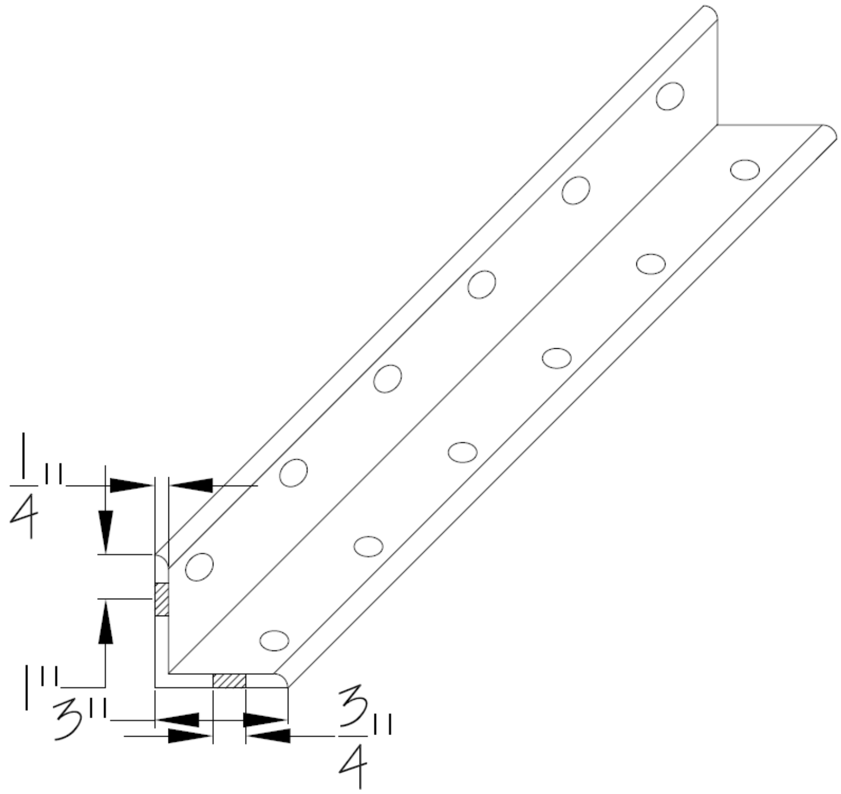


Figure 38: L3X3X5/16X17.5 Angle, 3/4 Bolts Spaced at 3" Center-to-Center

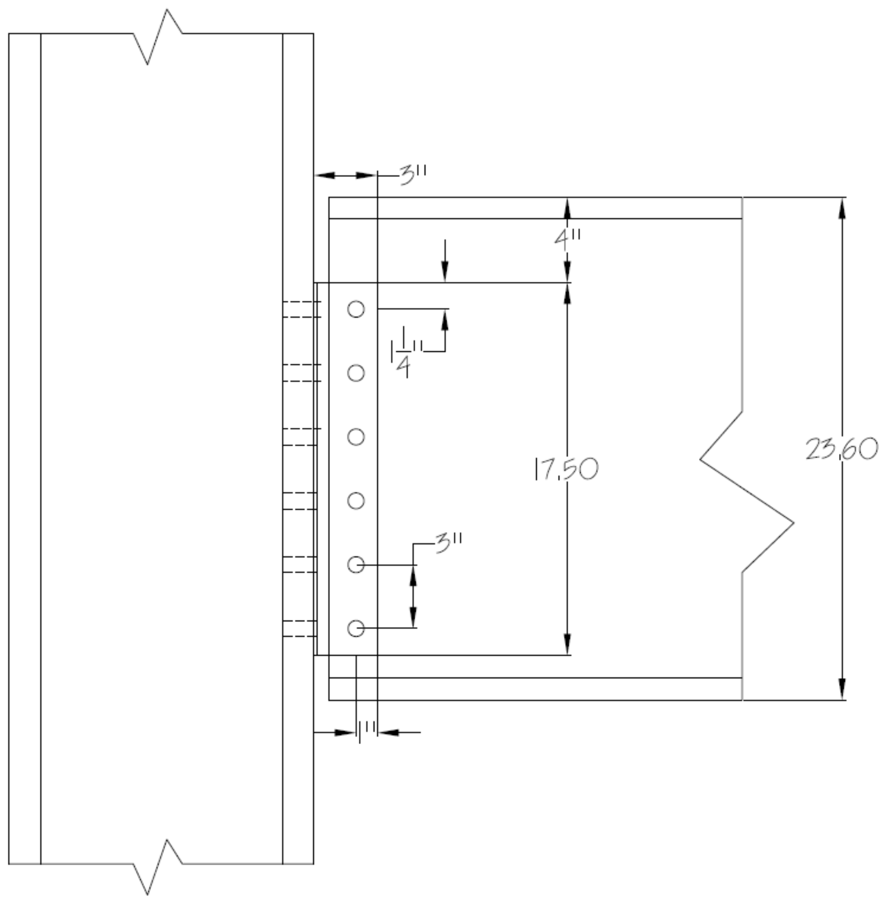


Figure 39: Typical Girder to Column Connection with Dimensions

4.4.7 Foundation Design

The foundations were designed based on bearing capacity and settlement requirements. The MQP team encountered several challenges during the design and analysis of the spread footings. The framing plan for the long span required large member sizes, which had a great impact on the final foundation design. In comparison with the short span the long span foundations experience a service load of approximately 1031 kips which is about almost twice the service load that the short span experiences. For both spans settlement was the governing design criteria, and consequently foundations had to be designed to ensure that the width and embedment depth were sufficient.

For the long span the footing width had to be sufficient to control differential settlements within tolerable limits so as not exceed the maximum allowable settlement limit of 1.03 inches. According to calculations a foundation width of 21 feet would be required to satisfy bearing capacity and settlement design criteria; however, this footing size will not be feasible based on the column spacing so the MQP team is recommending that a mat foundation is best suited for the long span. Table 15 shows a preliminary foundation design summary for a proposed mat foundation, and Figure 40 displays a visual representation of a mat foundation.

It should be noted that the design summary for the mat presented in Table 15 are based on calculations for a typical square spread footing and are no the actual results for a mat foundation. These results are presented to give a sense of proportion and prepare a construction estimate for the amount of concrete required. Therefore it is recommended that the structural design of the mat foundation be designed for strength and serviceability requirements by first evaluating the strength requirements using the factored loads, and then evaluating the mat deformations.

Table 15: Foundation Design Summary Long Span

Mat Thickness, T(ft)	**3**
Embedment Depth, D(Ft)	6
Number of Bars (#)	5
Bar Size Designation	18
Area of Steel (in^2)	20

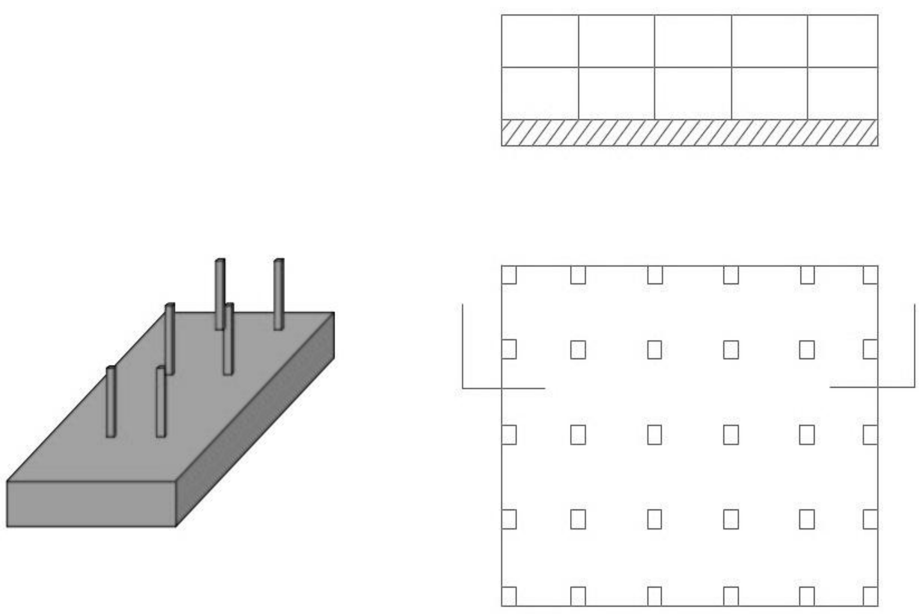


Figure 40: Representation of Mat Foundation

Similar to the long span the footing width was designed so that it would not exceed the maximum allowable settlement limit of 1.03 inches. A typical foundation size was determined for the interior columns and the exterior columns. Specifying a typical foundation size facilitates ease of construction since all foundations have the same width, thickness, and embedment depth. The foundations were designed to be able to carry a maximum allowable load of 549 kips which is greater than the service load of 483 kips. Table 16 shows the foundation design summary for a typical interior and exterior footing, and Figure 41 displays a typical square footing for the short span.

Table 16: Design Summary for a Typical Interior and Exterior Square Spread Footing for the Short Span Alternative

Number of Spread Footings (#)	**74**
Width, B(ft)	10
Thickness, T(ft)	2
Embedment Depth, D(Ft)	7
Number of Bars (#)	8
Bar Size Designation	8
Area of Steel (in^2)	6.32

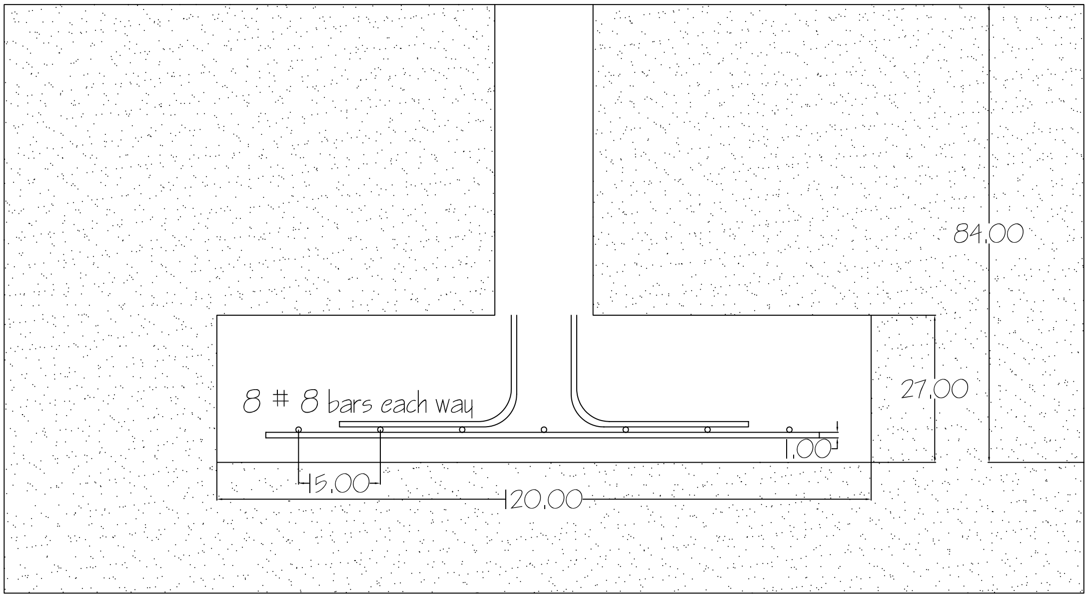


Figure 41: Typical Square Spread Footing for Short Span

Due to the large width of each spread footing and the small length of some the structural bays in the transverse direction, the MQP group recommends the redesign of these spread footings as combined footings since it is possible that "the columns will be so close to each other that the two isolated stress zones in the soil areas will overlap" (Prieto-Portar L. , 2008).

For the footings that support the frame and the Vierendeel frame the MQP team recommends combined footings because of the large axial service loads that they experience and their close proximity to the culvert limits the permissible size of the footings. "A useful application of a combined footing is if one (or several) columns are placed right at the property line. The footings for those columns cannot be centered around the columns. The consequent eccentric load would generate a large moment in the footing. By tying the exterior footing to an interior footing through a continuous footing, the moment can be substantially reduced, and a more efficient design is attained" (Prieto-Portar L. , 2008). Figure 42 identifies the square spread footings that the MQP team recommends to be redesigned as combined footings. These footings are enclosed in a red rectangle.

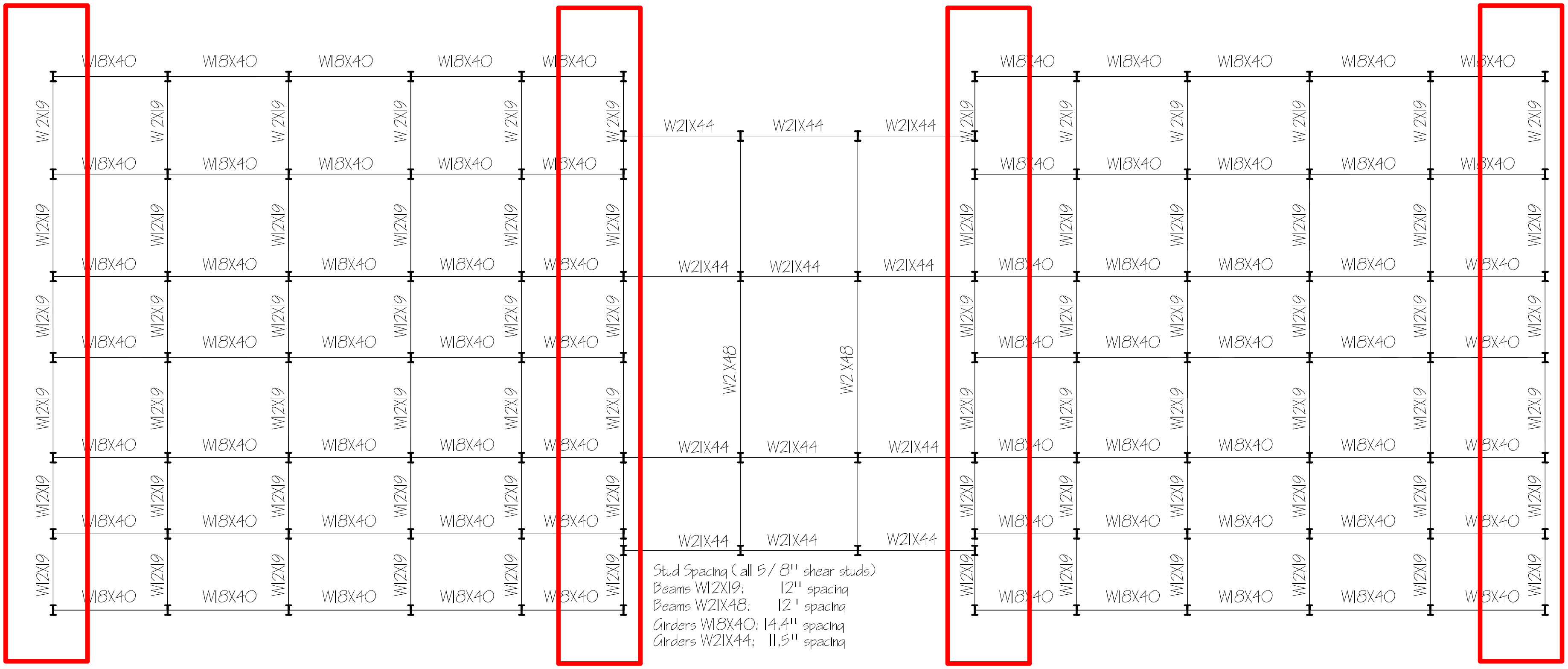


Figure 42: Square Spread Footings to be Redesigned as Combined Footings

4.4.8 Evaluation of Long Span versus Short Span

One of the overall goals of this MQP was to determine if a structural framing plan with long spanning beams or short spanning beams would be more economical. In order to evaluate the costs the MQP team decided to examine difference in cost due to steel for the structural frame and the concrete cost for the foundations.

According to the *AISC's* online article *Understanding the Supply Chain,* the structural framing system typically accounts for 10%-12% of the total building cost (AISC, 2012). For this project the estimated cost of structural framing system will be approximately 20% of the total building cost instead of the typical 10%-12% because of the Vierendeel frame. At an estimated $870 per ton of steel, not including the cost of labor, there was a difference between the long span and short span option (MetalBulletin, 2012). Although the short span is comprised of more members it requires less tons of steel than the long span, thus resulting in structural steel weight savings. However, this is offset by the labor costs that are more pronounced; this is similar to placing concrete for the foundations.

There is a drastic difference in the number of members when comparing the short span and long span. This is because the short span essentially splits the long span in half in the transverse direction accounting for an additional 1,236 members. The increase in the number of members increases the number of connections, the time to construction the frame, and the labor costs. Because of these increases, the MQP team is led to believe that although there would be a savings of approximately $450,000 in materials costs for the short span, once connections and labor are considered, the advantage may be given to the long-span design.

The late John Ruddy, formerly of Structural Affiliates International, is referenced in *Modern Steel Construction* as suggesting that the most economical framing has a bay area of about 1000 square feet (Carter, Murray, & Thornton, 2001). Many of the long span bays range from 800-1000 square feet, where as a typical bay in the short span is approximately half the area of the long span. Additionally, Ruddy suggested that all beams span the long-direction and be about 1.25 to 1.5 times the width of the girder span. The short span alternative does not meet this requirement since most of the beams are spanning the transverse direction since the span is too short. Figure 43 shows one bay on the short span where this is an issue, however there are multiple for this framing design.

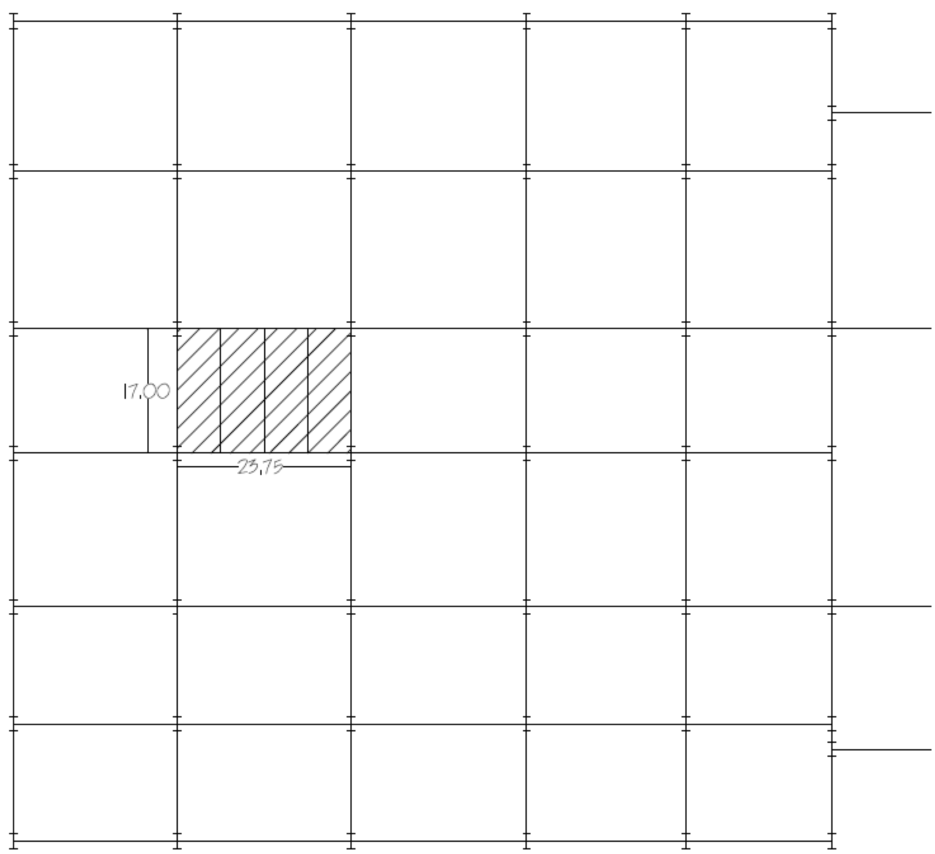


Figure 43: Beam Length Short than Girder Length for Several Short-Span Bays

The foundations were evaluated based on the cost for concrete and the formwork required. Based on the design and analysis of the foundations the short span will require 88 square footings with a thickness of approximately two feet, whereas the long span will require a mat foundation with a thickness of three feet. More formwork would be required for the short span and therefore result in an increase in cost. Calculations suggest that there is not a significant difference in the cost for concrete with the long span costing a total of $819,541 and the short span having a cost of $766,501.

From a constructability perspective the long span may be easier to construct since it has fewer structural members than the short span, approximately 1236, and will require less formwork for foundations. Additionally, since the long span has fewer columns it allows for more flexibility in the use of the floor space in the offices and restaurants.

Table 17 summarizes the weight of steel and estimated cost of each design option without consideration of the erection costs; it also identifies the cost of concrete construction required for the foundation systems.

Table 17: Cost Comparison between Long Span and Short Span for Steel and Concrete

Span	No. Members	Total Weight of Structural Steel (Tons)	Estimated Cost of Steel ($)	Type of Shallow Foundation	Cost per Square Foot of Concrete ($/sq.ft)	Cost of Concrete ($)
Long Span	1692	1551.32	1,349,654	Mat	$3.46	819,541
Short Span	2928	1039.22	904,118	Spread Footing	$3.24	766,501
Difference	1236	510	445,577		0.22	530,40

There were many other considerations made over the course of this MQP which would affect the economic outcome of this project. Figure 44 displays the methods that the

MQP team utilized to potentially make each structure more cost effective and the anticipated construction activities more efficient.

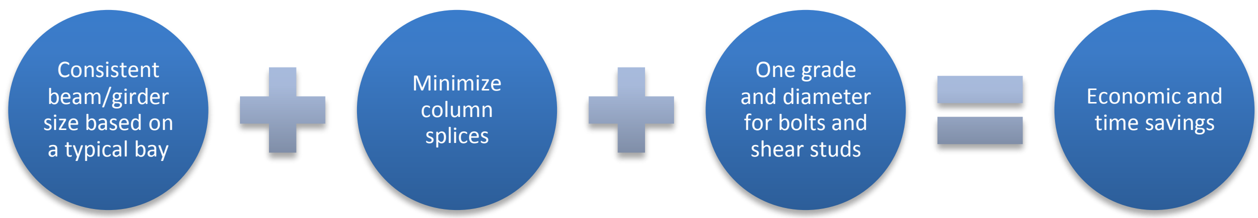


Figure 44: Methods Utilized to Increase Economic and Time Savings

Although certain bays could use different member sizes than other bays, and in some cases, even varying member sizes for particular beams within a bay, the adherence to repetition and uniformity would generally be more economical. This is because everything is simplified, from inventory control, to erection costs. It is also said that in order to be cost effective, one needs to order at least 20 tons from a steel mill of a given size; for a typical 55 pound/foot girder, this would equate to over 725 feet of steel, or about 32 members for the short-span bay girders (Carter, Murray, & Thornton, 2001). According to a *Modern Steel Construction* article, when quantities less than a “mill quantity (approximately 20 tons)” are used, the fabricator typically purchases sections from a service center with an added premium cost (Carter, Murray, & Thornton, 2001). Although there were several members in both designs of long span and short span that did not meet this quantity, most well exceed it.

Another general consideration was column splices. Ordering steel for the columns in 24-foot sections that span over two stories, as opposed to single story, 12-foot sections, would provide a significant cost savings. Furthermore, using similarly sized sections would also cut down on the amount of steel used to connect each section. *Modern Steel Construction's* April, 2001 article cites that "the labor involved in making a column splice equates to about 500 [pounds] of steel" (Carter, Murray, & Thornton, 2001). The added amount of column splices for the short span, 280 column-to-column connections, versus 172 for the long span, would give a difference of about $23, 500 based on the value given by *Carter et.* al. and the given price of steel.

One more consideration to be noted was the use of a single grade and diameter of bolts. Although sometimes, it may be more cost effective to use different size or grade bolts, it is standard to use the same size throughout a project. This is to reduce the risk of errors in construction and having an under-built structure if an inadequate size or strength bolt were to be used on a job with multiple possible sizes. For this project, 5/8" shear studs were used for all beams and girders for composite beam-slab construction, and ¾" ASTM A325 bolts were used for all beam-girder and girder-column connections. By using ¾" diameter bolts for connections, typical spacing and edge distances without the use of special equipment can be utilized. Both spans were built within this means.

Although the initial weight savings of the short span may prove more cost effective, the long span design is more typical of what is seen in modern building construction today. With the universal use of 50 ksi steel, beams can easily span over 40 feet as seen in the long span design, while still using a relatively light W-Section. For the design, the MQP team recommends the long-span design.

The MQP team decided to investigate how the cost of the long span and short span compares to the other WPI buildings that were constructed within the last 7 years. Table 18 displays a building cost comparison between the two spans and East Hall and the new

FPE building. The cost of comparison illustrates that the final cost of the long span design is very cost efficient for its usage. This low cost will allow provide the owners with the opportunity to make a return on their investment in a shorter period of time.

Table 18: Building Cost Comparison

	Long Span Design Alternative	Short Span Design Alternative	East Hall	New FPE Building
Number of Stories	8	8	5	5
Total Square Footage	236,722	236,722	103,610	92,000
Total Cost ($)	$42,773,382	$44,301,786	33,000,000	35,000,000
Total Cost/ Square Foot	$180.69	$187.15	$319	$380.437
Weight of steel (Tons)	1551.327	1039.216	500	Information Not Available
Cost of Steel ($)	$5,572,550	$6,825,838	$2,200,000	Information Not Available
Cost of steel/ square foot ($)	$23.54	$28.83	$21.23	Information Not Available

4.5 Development of Drainage Calculations

The increase in storm water runoff to the surrounding areas was calculated under four conditions to ensure the design storm events exceed the City of Worcester's 25 year storm design requirements. The four design storms that were used to assess the added impact of the proposed building where the 2, 10, 25, and 100 year storms to the area. The rational method was used to complete all calculations. (Portection, 2002) The 25 year storm was applied as a baseline for design with a factor of safety, and the 100 year storm was used to present a worst case scenario.

As stated earlier the following assumptions were used for these calculations:

- Original runoff coefficient: 0.15 (lawn with heavy soil and mostly flat)

- Final runoff coefficient: 0.5 (light industrial) (Portection, 2002)
- Rainfall intensity for 2, 10, 25, and 100 year storms are 3.0, 4.5, 5.3, 6.5 inches per hour respectively for the Worcester, MA area. (Portection, 2002)

STEP 1: Peak Flow rate from each storm

This was done for current site conditions and then a second time for future site conditions.

$Q_p = ciA$

$$Q_{init-2} = (.15) * \frac{3.0in}{hr} * (1.93acres) = .87cfs$$

$$Q_{init-10} = (.15) * \frac{4.5in}{hr} * (1.93acres) = 1.30cfs$$

$$Q_{init-25} = (.15) * \frac{5.3in}{hr} * (1.93acres) = 1.53cfs$$

$$Q_{init-100} = (.15) * \frac{6.5in}{hr} * (1.93acres) = 1.88cfs$$

$$Q_{fin-2} = (.5) * \frac{3.0in}{hr} * (1.93acres) = 2.9cfs$$

$$Q_{fin-10} = (.5) * \frac{4.5in}{hr} * (1.93acres) = 4.34cfs$$

$$Q_{fin-25} = (.5) * \frac{5.3in}{hr} * (1.93acres) = 5.11cfs$$

$$Q_{fin-100} = (.5) * \frac{6.5in}{hr} * (1.93acres) = 6.27cfs$$

STEP 2: Find the pre-development volume of runoff for each storm

The volumes were found by multiplying the flow rates seen in STEP 1 with the corresponding time of concentration for the 2, 10, 25, 100 year design storms.

2) $V = \Delta Q t_c$

$V_{Init} = Q_{init} t_c$

$$V_{Init2} = (.87cfs) * 45min * \frac{60s}{min} = 2{,}349ft^3$$

$$V_{Init10} = (1.30cfs) * 45min * \frac{60s}{min} = 3{,}510ft^3$$

$$V_{Init25} = (1.53cfs) * 45min * \frac{60s}{min} = 4{,}131ft^3$$

$$V_{Init100} = (1.88cfs) * 45min * \frac{60s}{min} = 5{,}076ft^3$$

STEP 3: Find the post-development volume of runoff for each storm

The volumes were found by multiplying the flow rates seen in STEP 1 with the corresponding time of concentration for the 2, 10, 25, 100 year design storms.

1) $V_{Fin} = Q_{Fin}t_c$

$V_{Fin} = Qt_c$

V_{fin} is the final Volume

$$V_{Fin2} = (2.9cfs) * 45min * \frac{60s}{min} = 7{,}830ft^3$$

$$V_{Fin10} = (4.34cfs) * 45min * \frac{60s}{min} = 11{,}718ft^3$$

$$V_{Fin25} = (5.11cfs) * 45min * \frac{60s}{min} = 13{,}797ft^3$$

$$V_{Fin100} = (6.27cfs) * 45min * \frac{60s}{min} = 16{,}929ft^3$$

STEP 4: Find the additional volume for each storm

When calculating the volume the assumption was made that the pre-existing conditions of the site were acceptable and therefore only the impacts of the proposed development were assessed.

$\Delta V = V_{fin} - V_{init}$

$\Delta V_2 = 7{,}830ft^3 - 2{,}349ft^3 = 5{,}481ft^3$

$\Delta V_{10} = 11{,}718ft^3 - 3{,}510ft^3 = 8{,}208ft^3$

$\Delta V_{25} = 13{,}797ft^3 - 4{,}131ft^3 = 9{,}666ft^3$

$\Delta V_{100} = 16{,}929ft^3 - 5{,}076ft^3 = 11{,}853ft^3$

Once the added volumes were found, it was decided that due to the exceedingly high ground water table (18ft) the additional water would pose a threat to the buildings foundations. Since this threat eliminated any option of ground water infiltration tt is recommended that all excess runoff, not naturally infiltrated, be diverted to the Millbrook Conduit, where storm water is currently routed.

4.6 Traffic and Parking

The traffic and parking results are broken into four major groups that coincided with the usage of the building. By doing this the team ensured the traffic flow to and from the building will have minimal impact on the surrounding area.

4.6.1 Current Intersection Traffic

Through an intersection field study the current loads on the intersection were collected. The traffic analysis provided baseline data for the Grove Street/Salisbury Street intersection. This included traffic and turning movement counts for the intersection. The existing conditions can be seen in Table 19. Using the computer program *MCTrans: HCS2000*, this intersection's level of service was evaluated. When evaluated under current conditions the intersections LOS analysis results in an "F" rating based on the intersection delay times.

Table 19: Vehicles currently through the intersection in peak hour

Vehicles through the intersection	Current Traffic Loads
EBLTR	746
WBLT	818
WBR	359
NBL	42
NBTR	186
SBL	438
SBTR	117
Total growth	0
Maximum delay (s)	-

4.6.2 Intersection Growth

In *MCTrans: HCS2000*, a growth rate of 4% per year for 10 years was then applied to the intersection to accommodate the expected increase in traffic over the time interval between now and anticipated construction. These traffic figures for existing and projected growth were then combined with the expected traffic increases from the new mixed use building. The results can be seen in Table 20. This resulted in a notable increase in vehicle delay during peak hour traffic.

Table 20: Total Traffic Loads Due to Proposed Building and Growth

Vehicles through the intersection	Total Traffic Loads Due to Proposed Building
EBLTR	1406
WBLT	820
WBR	716
NBL	186
NBTR	277
SBL	636
SBTR	916
Total growth	4957
Maximum delay (s)	358.7

4.6.3 EIR Comparison and Delay Mitigation

In the MQP team's analysis of the intersection, it was found that the current Peak-hour LOS and the expected LOS for the intersection after the new building is constructed were classified as F, which was the same finding as presented in the *Gateway Master Plan*. The final comparison of the total traffic increase due to the proposed building in the future and the EIR's full build-out can be seen in Table 21. This shows that the increase in the traffic volumes due to the proposed building is proportionally lower than the expected peak after *Gateway Park's* completion. The 13% variation is because the EIR accounts for the full build-out of *Gateway Park* and the analyses only accommodate the current development plus the addition of this proposed building.

In order to mitigate the effects of adding extra traffic to the intersection, the *Gateway Master Plan* calls for the intersection signals to be retimed. Per a meeting with Jon Weaver of the WBDC it was confirmed that the intersection timing had not been implemented prior to the field study. (Weaver, 2011) This retiming will result in the delay reduction from, what the analysis found to be, 359 seconds to 270 for the growth caused by the proposed building. The 89 second decrease results in the intersection having substantially less delay than if no upgrades were initiated. Though significantly improved through the implementation of the new signal timing the intersections LOS remained "F".

Table 21: Proposed building and EIR comparison

Vehicles through the intersection	Total Traffic Loads Due to Proposed Building	EIR Full Build-out
EBLTR	1406	1661
WBLT	820	971
WBR	716	966
NBL	186	71
NBTR	277	384
SBL	636	765
SBTR	916	849
Total growth	4957	5667
Maximum delay (s)	358.7	309.1

4.6.4 Trip Generation Impacts

It was found that the approximate number of trips added to and from the building would be 3254 in total, and the breakdown can be seen in Table 22. 60% of the trips to and from the building will be funneled through the Salisbury St. Grove St. intersection with the remaining 30% leaving through other directions. (Group, 2008) The increase was incorporated into the LOS analysis above and resulted in a final number of vehicles through the intersection during peak hour to be 4957 as seen in Table 21.

Table 22: Trips Generated per day

Usage	Trips
Restaurant	800
Retail	803
Dwelling units	756
Research and development	680
Industrial	215
Total	**3,254**

It is expected that the estimated 3,254 trip addition is higher than what will be observed in real world conditions. There is a reduction expected in the number of trips per day due to individuals who work and live on site. The expected decrease is 65% based on a compilation of statistics from college campuses from across the country (see APPENDIX F:). This decrease was not incorporated into the traffic analysis in an effort to maintain a conservative estimate of impact. It is anticipated that the residents will have an effect on traffic through shuttle usage and pedestrian traffic.

4.6.5 Pedestrian traffic

Due to on-site housing units there will be an increase in pedestrian traffic. This increase in pedestrian traffic necessitates an added crosswalk. To accommodate this increase a curb cut and cross walk are recommended directly across from the main entrance to the building to ensure that people with mobility issues have access to the surrounding buildings.

4.6.6 Parking

Due to the usage of the building the tenants will require both day and night parking accommodations. For an expected need the number of spaces was compiled and then used in the design and expansion of surface lots and parking garage; see Table 23. (Engineers, 2008) It is recommended that an extra bay be added to the South side of the parking garage. This additional bay, when combined with the growth of the surface parking on the Eastern side of the building, will accommodate the 531 required additional spaces.

Table 23: Parking Requirements

Usage	Sq.Ft.	Spaces per X	Spaces
Industrial	56,179	1/1000 Sq. Ft.	56
Research and Development	62,182	1/300 Sq.Ft.	207
Retail	12,544	1/300 Sq.Ft.	42
Restaurant	12,544	.5/Occupant	200
Residential	93,273	.33/Dwelling	26
Total			531 Spaces

4.7 Cost Estimate

After the design was completed two final cost estimates were produced. The first containing the cost for short span construction and the second containing long span construction. The breakdown of the two structures can be seen in Appendix F. The combination of the estimates resulted in a final cost for construction being: $44,301,786 for the short span with a final square footage cost of $187.15 dollars per square foot and $42,773,382 for the long span with a final square footage cost of $180.69 dollars per square foot. The cost for the long span option was very cost efficient and when compared to similar buildings on campus (East Hall $319/sq.ft.) currently and is therefore the recommended design for this project. The significant cost reduction when compared to East Hall may be due to the developments in green technology and the added expense East Hall incurred from its green roof. A summary of the final estimate can be seen in Table 24. This value of $180.69 dollars per square foot also compared to the final cost of East Hall, WPI's most recent green building, with a final cost of $33 million dollars which is approximately $319 per square foot. (U.S. Green Building Council, 2011) Once compared against these figures the project is not only financially feasible but a prime next step in the expansion of the WPI community.

Table 24: Cost Estimate for Long Span and Short Span

			% of Total	Cost Per S.F.	Cost	Specialty Areas
A Substructure Short Span			2.70%	$4.61	$1,091,771.63	
A1010	Standard Foundations			$3.46	$819,541	
A1030	Slab on Grade			$1.15	$272,230	
B Shell Short Span			28.90%	$90.78	$12,956,937.80	
B1010	Steel Construction			$2.2/lbs	$6,825,838	
	Steel Erection			$36.04	$1,706,460	
A Substructure Long Span			2.70%	$4.39	$1,038,731.42	
A1010	Standard Foundations			$3.24	$766,501	
A1030	Slab on Grade			$1.15	$272,230	
B Shell Long Span			28.90%	$49.44	$11,703,649.80	
B1010	Steel Construction			$2.2/lbs	$5,572,550	
B1020	Roof Construction			$7.73	$1,829,861	

			% of Total	Cost Per S.F.	Cost	Specialty Areas
B2010	**Exterior Walls**			**$8.15**	**$1,929,284**	
B2020	**Exterior Windows**			**$2.67**	**$632,048**	
B2030	**Exterior Doors**			**$4.38**	**$1,036,842**	
B3010	**Roof Coverings**			**$2.47**	**$584,703**	
B3020	**Roof Openings**			**$0.50**	**$118,361**	
C Interiors			**27.60%**	**$30.07**	**$7,118,230.54**	
C1010	**Partitions**			**$5.77**	**$1,365,886**	
C1020	**Interior Doors**			**$7.15**	**$1,692,562**	Residential
C1030	**Fittings**			**$2.73**	**$646,251**	
C2010	**Stair Construction**			**$3.31**	**$783,550**	
C3010	**Wall Finishes**			**$2.70**	**$639,149**	Residential
C3020	**Floor Finishes**			**$4.92**	**$1,164,672**	
C3030	**Ceiling Finishes**			**$3.49**	**$826,160**	
D Services			**40.80%**	**$57.02**	**$16,697,835**	
		+Retail		**$6.70**		
		+Industrial		**$2.25**		
		+R&D		**$23.19**		
		+Residential		**$15.69**		
D1010	**Elevators and Lifts**			**$14.23**	**$3,368,554**	
D2010	**Plumbing Fixtures**			**$15.69**	**$1,463,453**	Residential
D2010	**Plumbing Fixtures**			**$6.70**	**$168,090**	Restaurant
D2010	**Plumbing Fixtures**			**$2.25**	**$126,403**	Factory
D2010	**Plumbing Fixtures**			**$23.19**	**$1,442,001**	Lab

			% of Total	Cost Per S.F.	Cost	Specialty Areas
D2020	Domestic Water Distribution			$1.90	$449,772	
D2040	Rain Water Drainage			$0.12	$28,407	
D3010	Energy Supply			$5.90	$1,396,660	
D3050	Terminal & Package Units			$18.80	$4,450,374	
D4010	Sprinklers			$2.98	$705,432	
D4020	Standpipes			$1.61	$381,122	
D5010	Electrical Service/Distribution			$2.23	$527,890	
D5020	Lighting and Branch Wiring			$8.69	$2,057,114	
D5030	Communications and Security			$0.38	$89,954	
D5090	Other Electrical Systems			$0.18	$42,610	
G Building Sitework			0.00%	$0.00	$0	
SubTotal Short Span			100%	$182.48	$37,864,775	
Contractor Fees (General Conditions,Overhead,Profit)			10.00%	$16.00	$3,786,477	
Architectural Fees			7.00%	$11.20	$2,650,534	
Total Building Cost Short Span				$187.15	$44,301,786	
SubTotal Long Span			100%	$140.92	$36,558,447	
Contractor Fees (General Conditions,Overhead,Profit)			10.00%	$15.44	$3,655,845	
Architectural Fees			7.00%	$10.81	$2,559,091	

			% of Total	Cost Per S.F.	Cost	Specialty Areas
User Fees			0.00%	$0.00	$0	
Total Building Cost Long Span				$180.69	$42,773,382	

4.8 Obtaining LEED Certification

Throughout the entire design of the building environmentally friendly alternatives for construction were always given a high priority to ensure LEED certification could be obtained. After fully completing the design the estimates were compiled based on the recommendations provided in this report and an expected result during construction. The points expected to be awarded for the primary and secondary estimates can be seen below in Table 25. Until the building is officially commissioned no point will be officially awarded. The exact break down of where these points are expected to be awarded is in 16 APPENDIX J.

Table 25: LEED Points Summary

		Proposed Design Assessment		
Category	Total Possible Number of Points	Primary Estimate	Secondary Estimate	Prerequisites achieved
Sustainable Sites	26	23	1	Yes
Water Efficiency	10	8		Yes
Energy and Atmosphere	35	7	15	Yes
Materials and Resources	14	3	4	Yes
Indoor Environmental Quality	15	11	2	Yes
Innovation and Design	6	1	3	n/a
Regional Priority	4	2	0	n/a
Total	**110**	55	25	n/a

A project targeting some level of LEED certification will incur added costs throughout construction (Consultants, 2003). The additional costs are added on to four categories. The first constraint is that the construction process is more time intensive. Areas of the site must be kept undisturbed and care must be taken to prevent the addition of any contamination to the building. The materials used to construct the building will also be more expensive. Reusing materials that have been used in construction previously can first, be expensive to procure because their price includes the cost of salvaging the materials. The reused materials are also expensive to install because they have been specialized for another scenario and therefore may require extra work to install. These two hindrances aside, it is important to reuse materials to preserve our fragile environment. For this same reason energy saving appliances need to be installed wherever possible. Examples of these are dual flush toilets and low flow faucets. The installation of appliances such as these can greatly reduce the environmental footprint of a building. The last aspect is during construction; special care must also be taken to prevent erosion and sediment contamination to the surrounding area.

These added costs are aspects of construction that the owner must account for during the planning phase. In the long run these extra steps and costs can pay for themselves both directly and indirectly. The direct return comes in the form of long-term energy conservation from efficient building operation. The indirect return on investment comes in the form of future savings when considering the global cost to the environment. As more sustainable construction is incorporated throughout the world the non-renewable energy demand will diminish due to reliance on alternative sources. The environmental return is the most substantial benefit and is also the most necessary. For a complete breakdown of the expect LEED points earned see appendix K.

4.8.1 Cost Increase of LEED construction

It should also be noted that green construction does come with an added cost to the whole project. A representation of this can be seen in Table 26 Low-Flow fixtures were used as an example because the numbers for these elements can be finalized at this phase of the design. The 125 low-flow toilets will add approximately $20,000 to the cost of the project versus the purchase of conventional toilets. After all the fixtures are considered, there is an added expense of almost $50,000 above the cost of conventional fixtures. This seems like a very large cost but, when compared to the full scope of the project, it is really rather small. Most buildings will experience a cost increase of between 0.5% and 5% of the total cost by constructing with sustainable intentions (Consultants, 2003). To keep the cost within this range the owner must have green construction in mind from the very first steps toward construction.

Table 26: Fixture Cost Increase

Plumbing Fixtures	Standard Fixture: Price	Low-flow Fixture: Price	% cost Increase
Toilet	$229Kohler	$389 American Standard	41 %
Urinal	$139 American Standard	$399 Sloan	65 %
Sink	$89 Delta	$120 American Standard	26 %
Shower	$24 Delta	$30 Delta	20 %

4.8.2 Sustainable Sites

Being a brownfields site in the city of Worcester the site is in a prime location to receive several points for sustainability. The site will have many amenities to promote efficient modes of transit, including its close proximity to an interconnected network for public transportation. The onsite parking has been reduced by almost 10% from residents working on site and is reduced even further by its close proximity public transportation. The site is designed to maximize green space while reducing light pollution and heat island effect to minimize its negative contributions to the environment.

4.8.3 Water Efficiency

Throughout purchasing and construction close attention will be paid to water conservation both in usage and in waste. A primitive percentage estimate on the water conservation that low flow appliances can produce is displayed in Table 27. Since the fixtures outlined in Table 27 are the primary water uses for the building there will be a substantial decrease in volume of water used. Through these reductions this new building will have an approximately 50% reduction in water consumption when compared to the amount of water that would be used if low flow fixtures were not installed.

Table 27: Potential Water Usage Reduction

Fixtures	Flow rates for Conventional Fixtures (gpm/gpf)	Flow Rates for Energy efficient Fixtures (gpm/gpf)	% Reduction
Toilet	1.6	0.8	50 %
Urinal	1.0	0.125	87.5 %
Sink	2.2	1.2	46 %
Shower	2.5	1.2	52 %

4.8.4 Energy and Atmosphere

Most of the points this construction would achieve in this category would need to be proven after construction completion. This is because predicted values must be compared against the actual measured values to prove the reduction. All of the systems within the building will need to be equipped with energy saving features like day vs. night usage. The night vs. day usage will incorporate features that will vent at specific times of the day based on the season for maximum heating in the winter and cooling in the summer, and use natural heating as much as possible. The HVAC system will need to run using no CFC-based refrigerants to eliminate its contribution to ozone depletion. Through effective planning and purchasing a substantial amount of energy consumption can be saved over the life cycle of the building.

4.8.5 Materials and Resources

This LEED section is comprised of two major portions. The first piece is building reuse of onsite structure. In this section no points will be collected for building reuse because this is a new construction project. The second major portion is the use of locally manufactured materials. Since the reuse points were impossible to achieve it will be important to pay extra attention to the second aspect of the category. During purchasing it will be necessary to use materials that were manufactured within 500 miles of the project site. If the materials were already used and can be reused in this construction that is also very beneficial to the project and to sustaining the environment for years to come. While these materials are being put in place careful planning is necessary to ensure that very little waste is produced. This waste reduction will help to mitigate the added construction cost and to minimize the volume of materials that will be sent off to landfills.

4.8.6 Indoor Environmental Quality

Most of the points that are within this category are easy to achieve through meticulous construction planning and efficient operation of systems in place. This is achieved by using low-emitting materials for construction to minimize contaminants to which the occupants could be exposed after construction has been completed. During construction, care must be taken to ensure that absorptive materials don't collect contaminants that may create an exposure hazard to the occupants later on. To ensure the exposure will be minimized the building must be flushed out prior to occupancy. Once the building is occupied the air quality within the building will need to be monitored to encourage and sustain a healthy environment. The air quality is not the final step in maintaining a high indoor environmental quality. To ensure the well-being of the occupants and to increase productivity the occupants must also have a great deal of control of lighting and temperature within their space.

4.8.7 Innovation in Design

This building may receive extra points in this area for its roof design. The two largest portions of the roof are glass. This glass roof allows the natural light to penetrate into the stair wells and the hall ways of the floor below. The glass roof will serve as more than just a means to add natural lighting to the building; it will also allow for natural heating of the building interior to reduce energy cost.

4.8.8 Regional Priority

Much like the name indicates the Regional priority points vary by location throughout the United States and some other countries. The Regional priority points will add at least 2 points toward LEED certification. The first extra point will be awarded for a reduction in the heat island effect from non-roof reflected heat. The second regional priority point that will be awarded is for the reduction in heat island effect from the roof. This roof reflected heat is being reduced by the installation of a white roof. The Regional priority points serve as extra incentive to implement sustainable design in the areas where they are most needed. In the area this building is to be constructed emphasis is put on using green energy sources and reducing the heat island effect caused by development.

4.9 Revit Architectural Model

Models of the building were produced using *Autodesk Revit Architecture*. Both a structural model and an architectural model were created. The structural model was created to show the framing plan as a 3-dimensional system, since *MASTAN2* was only used for 2-D analyses. The 3D structural model helps one to easily see the difference in the number of members between the short span and the long span. It also helps to put the member sizes into perspective. The architectural model was used primarily for ensuring proper layout of the rooms, showing the windows in the building, and the exterior curtain walls. The architectural renderings could also be used to showcase a demo room for marketing the space towards either student residents or office workers. Figure 46, Figure 47, Figure 48 and Figure 44 display renderings of sample residential units created using *Autodesk Revit Architecture.*

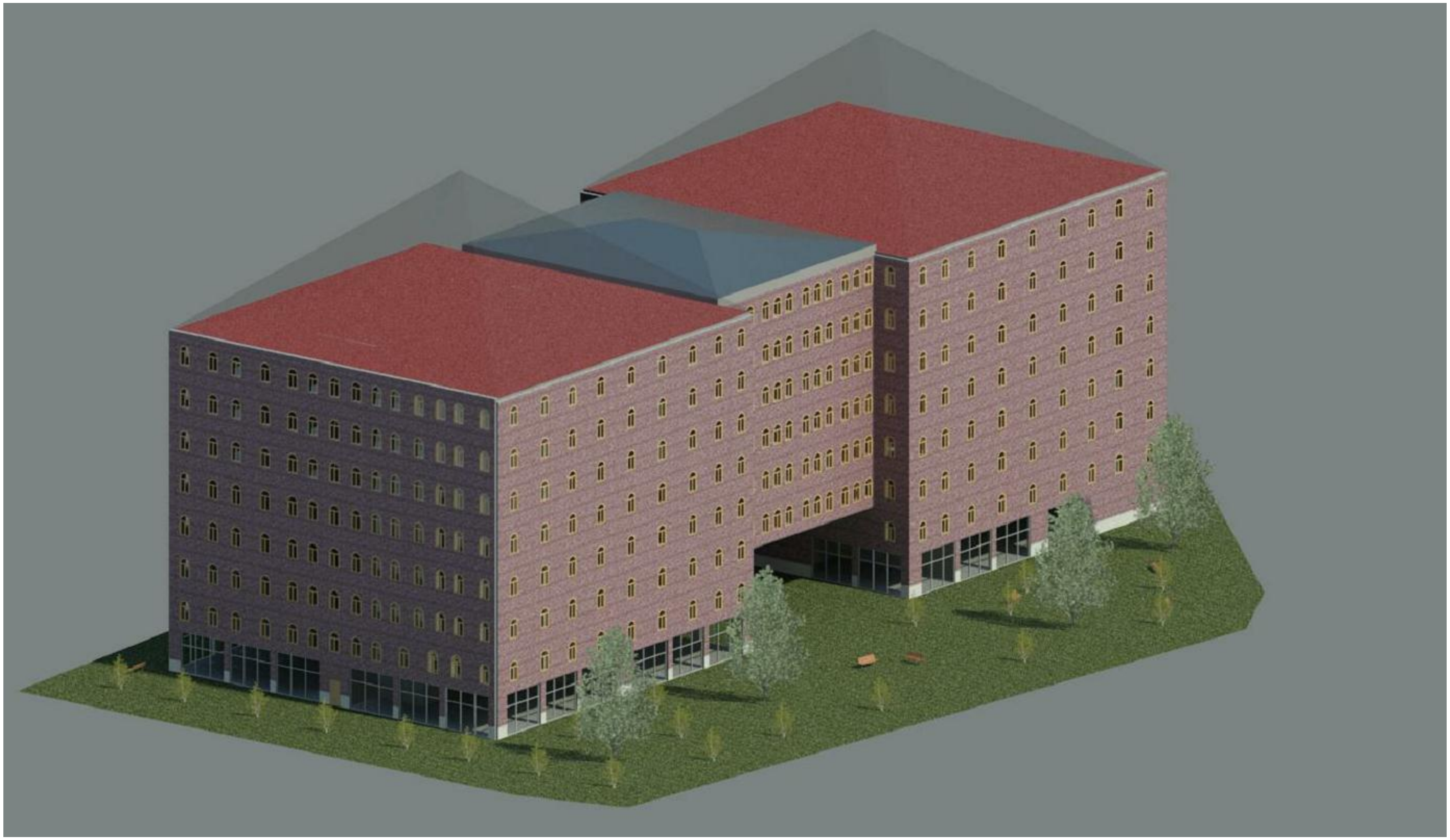


Figure 45: Rendering of Conceptual Design Generated with Revit

Figure 46: Rendering Generated with *Autodesk Revit* of Typical Apartment Unit

Figure 47: Typical Dorm-Style Double Bedroom

Figure 48: Bedroom with Desk and Shelf

5 Conclusion and Final Design Recommendations

Based on the MQP team's design and analysis of the short-span and long–span alternatives, it has been decided that the long-span solution offers an economic advantage. This will result in cost savings of approximately $280,982 over the short-span approach. Additionally, the owner could potentially reduce the total construction time since there are less members and connections.

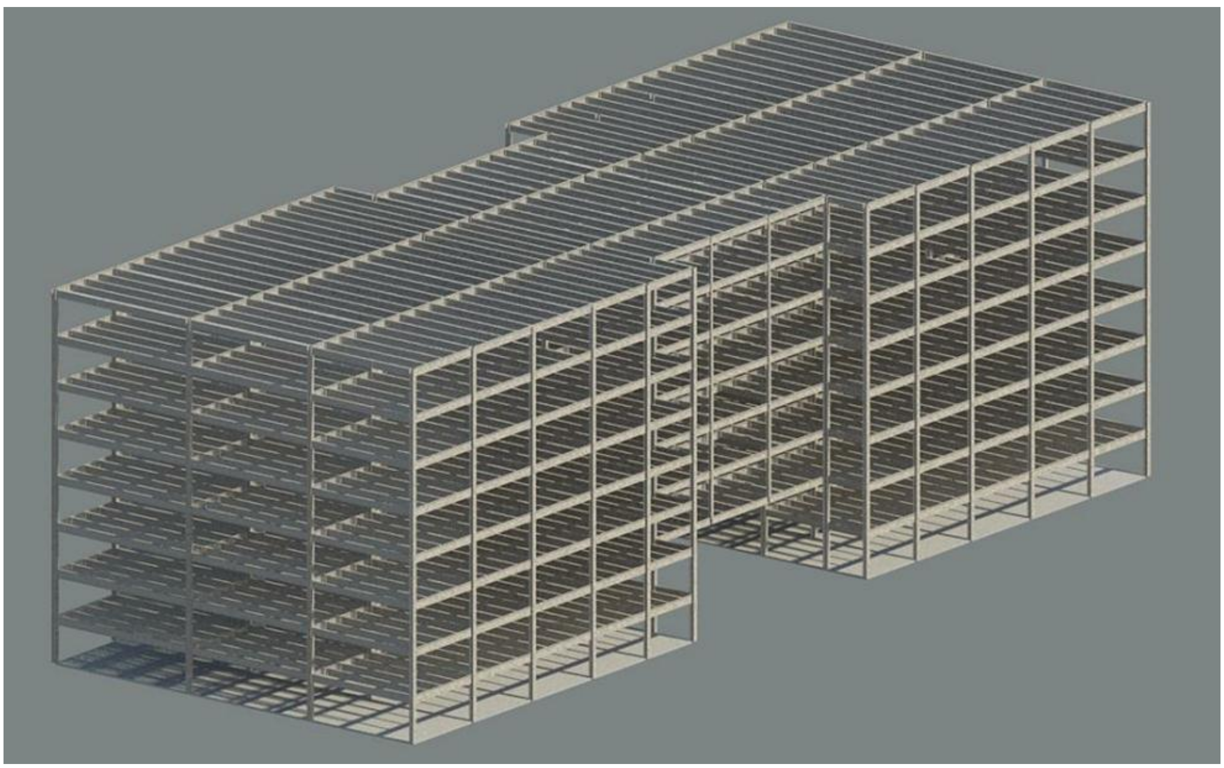


Figure 49: Long Span Structural

6 Recommendations and Areas for Further Study

This section presents some topics that the MQP group did not get to investigate due to time constraints but believes that they should be investigated by future project groups.

6.1 Bracing

For ease of analysis, the MQP team decided to design a rigid frame with moment-resisting girder-to-column connections. Past MQP's have investigated if this is the most cost effective method. It was found that this is typically only done if there is a reason not to provide one of the other forms of bracing; for example, diagonal bracing elements may block elevator doors, stairwells, or block off windows in the building (Frascotti, Richard, & Toomey, 2008).

Figure 50: Example of Bracing (Source: http://www.sigi.ca/engineering/)

There are three main types of bracing that could be considered for further study: cross bracing, chevron bracing, and eccentric bracing. Cross bracing is the most common type of bracing; however it is also the most restrictive. It should be noted that all bracing would typically have to be applied to multiple column lines to be effective.

Cross bracing can often block off other areas or potentially become large enough to take up once useable room space and affect the soffit details (AISC, 2002). *AISC*'s *Designing with Structural Steel: A Guide For Architects* has provided Figure 51 and Figure 52 showing the typical set-up of cross bracing.

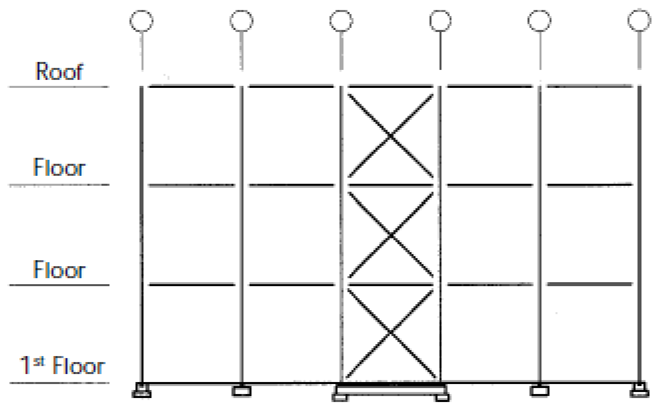


Figure 51: Cross Braced Frame Example (AISC *Designing with Structural Steel*, 2002)

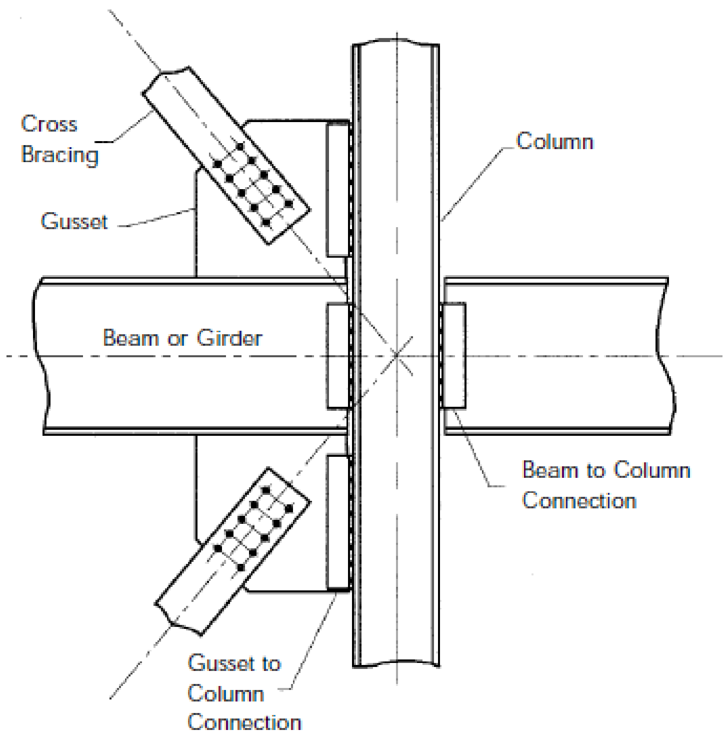


Figure 52: Cross Bracing Connections (AISC *Designing with Structural Steel*, 2002)

Chevron Bracing is another common type of bracing. The key advantage to Chevron bracing over standard cross bracing is that Chevron bracing allows "the architect to consider placing doorways and corridors through the bracing lines on a building" (AISC, 2002). This design configuration contributes vertical support to the girders as well and becomes a major part of the structural frame. Thus the bracing members need to be designed to carry adequate gravity loads. An advantage to this system is that lighter beams or girders can typically be used since they benefit from the intermediate support. Figure 53 shows a typical layout of Chevron bracing in a small building.

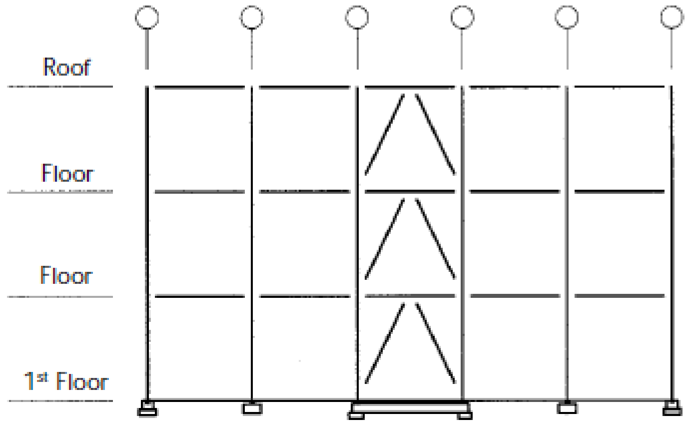


Figure 53: Example of Chevron Bracing (AISC *Designing with Structural Steel*, 2002)

Eccentrically braced frames are the last major type of bracing. This type of bracing is typically used in areas subject to higher seismic loads. The major difference in this type of bracing with Chevron bracing is that the bracing gussets are connected within the span of the beam or girder as opposed to the direct center with Chevron bracing. This eccentricity introduces additional bending into the system response which increases ductility. Figure 54 shows a standard connection to a beam or girder for a eccentrically braced frame.

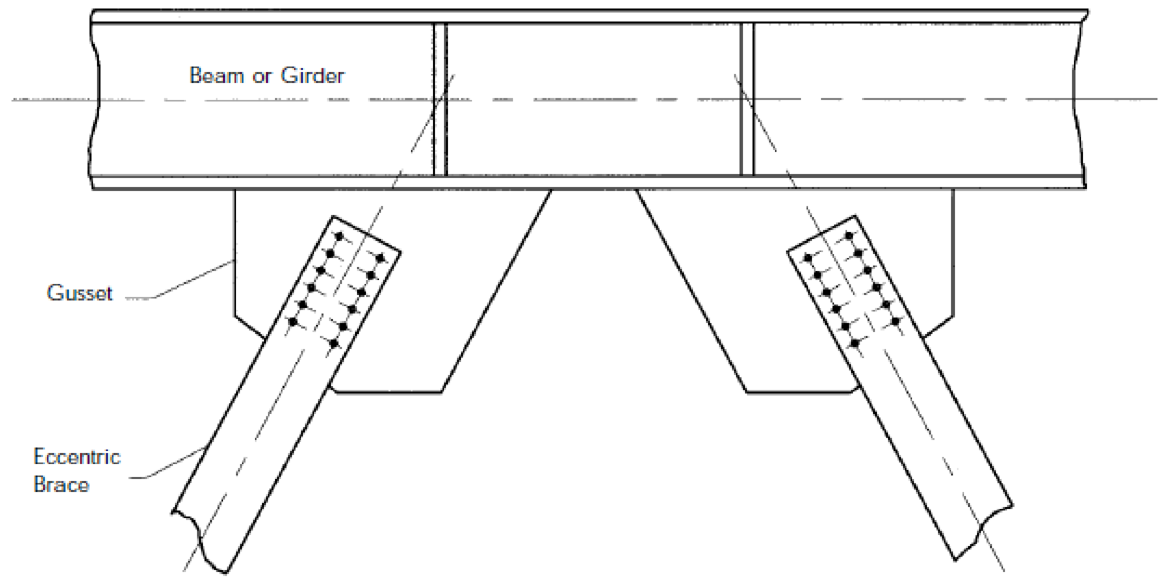


Figure 54: Example of Eccentrically Braced Connections (AISC *Designing with Structural Steel*, 2002)

6.2 Use of Reinforced Concrete versus Steel

Another interesting topic for consideration would be to design the building using reinforced concrete instead of structural steel framing. Since most buildings in New England are designed using steel, the MQP team decided that this would be the best design type for the project. Both the advantages and disadvantages of steel and concrete should be compared before making finalized decisions on a construction type.

Steel has a higher strength-to-weight ratio than concrete and therefore smaller foundations could be used. Furthermore, steel can span longer distances and thus makes for fewer columns splitting up useable space in the proposed building. It should also be noted that "the typical steel column occupies 75% less floor space than an equivalent concrete column" (AISC Importance of Framing Selection). Steel is also more predictable and reliable than concrete. A weak steel beam is far less common than a bad batch of concrete being produced. Several studies cite that steel construction is typically 5-7% less expensive than concrete construction (AISC Importance of Framing Selection). Steel is also modifiable, meaning that connections and members can be changed or fit to accommodate renovations or additions to a building in the future.

Concrete is readily available and much quicker to erect a building than steel. Alfred G. Gerosa, president of Concrete Alliance Inc., says that "it is not uncommon for cast-in-place reinforced concrete buildings to rise one floor every other day. Developers can finish jobs faster, earn a profit, recoup capital, and move on to the next project" (Madsen, 2005). Although in the current economy saving time and moving onto the next project may not be the primary concern, it would certainly be a concern in a strong economy, perhaps when WPI plans on developing Gateway Park further. Concrete typically has better sound resistance than steel construction, and this aspect of performance could be an important consideration for the residential areas of the proposed building. It should also be noted that from a fire-protection standpoint, concrete can generally be considered a safer building material than unprotected steel. To make a steel structure safer for fire conditions, typically a spray-applied fireproofing material or concrete encasement is used. However, for fireproofing steel, many materials also have disadvantages such as installation time or susceptibility to damage after installation (Goode, 2004).

6.3 Structural Design for Fire-Safety

Designing a building to be safe during fire conditions is another area of which the MQP team would suggest further study. There are many considerations for the design for fire conditions. The MQP team touched upon code-based requirements, such as the required egress widths which correspond to the life-safety of persons in the event of a fire. However, the MQP team did not go into the required fireproofing materials or the possibility of introducing a performance-based design approach for certain areas of the building. With the lobby areas on the first floor, there is a great potential to open each lobby up to the second floor or through each industrial and office space area, creating atria in each leg of the building. This would present a unique set of design challenges in which the engineer would need to prove to the authority-having-jurisdiction (AHJ) that the design of the building meets certain performance criteria based on the scenario design fires. Much of the design comes down to how much the owner/developer of the building may be willing to spend. For example, does the owner want to invest the time and money to examine the egress times and ensure that the building will not collapse before all occupants exit? Does the owner want to preserve the building in the event of a structural fire? Performance-based design for fire conditions involves asking "what could happen" and "how will the building respond?" Different types of occupancies account for different types of possible design fire scenarios. Both passive and active fire-protection systems could be examined; however, for a Civil Engineering MQP, passive fire-protection and structural performance would take a primary consideration.

6.4 Baseplates

In order for the columns to connect to the foundations, baseplates would have to be designed. Since the columns in this case are small in comparison to the foundations, there would be no sizing issues where the plates are far too large. The purpose of the plates is for the load from the column "to be spread over a sufficient area to keep the footing from being overstressed" which is much similar to how a footing spreads the load to the soil so as to prevent a particular area of soil from being overstressed (McCormac, 2008). Chapter 14 of the *AISC Steel Manual* presents information on the design of base plates.

6.5 Curtain Walls

The design and connections of curtain walls is another potential topic for future studies. The structural system for the building was designed with nonbearing walls. The current enclosure was only used to consider the effects of lateral loads on the building frame. Large glass curtain walls could be utilized to give the building a modern look which is important for a technology driven area such as Gateway Park at WPI.

6.6 Added Weight/Cost of Steel for Gravity to Lateral Load System

Gravity loads were used to establish preliminary member sizes, then lateral loads were considered to determine the final member sizes. A potential topic for further study would be how much of a premium does one pay for a lateral load bearing system. A thorough cost analysis could be performed based on either the increase of girder/column sizes or the cost to install bracing for the structural frame. The MQP team looked into seeing if there was any correlation between the increase of Z_x and I_x for the frame designed to resist lateral loads. It should be noted that the gravity analysis did not involve plastic capacity and this only became a point for concern in some cases when lateral load effects were considered. Table 28 and Table 29 summarize the findings.

Table 28: Short Span Column Summary

Story	Gravity Loads	Combined Loads	Increase in Z_x	Increase in I_x
1-2	W12X79	W12X106	119 to 164 = 37.82% increase	662 to 933 = 40.9% increase
3-4	W10X54	W12X65	66.6 to 96.8= 45.35 % increase	303 to 533= 75.9% increase
5-6	W10X39	W12X65	46.8 to 96.8 = 106.84% increase	209 to 533 = 155.0% increase
7-8	W8X31	W12X53	30.4 to 77.9 = 156.25% increase	110 to 425 = 286.4% increase

Table 29: Long Span Column Summary

Story	Gravity Loads	Combined Loads	Increase in Z_x	Increase in I_x
1-2	W14X145	W14X145	260 to 260= 0% increase	1710 to 1710 = 0% increase
3-4	W14X99	W14X109	173 to 192 = 10.98% increase	1110 to 1240 = 11.7% increase
5-6	W12X65	W14X109	96.8 to 192 = 175% increase	533 to 1240 = 132.6% increase
7-8	W8X31	14X90	30.4 to 157 = 416.45% increase	110 to 999= 808.2% increase

The MQP team noticed that columns on the upper floors needed to become much larger sections to support the added lateral loads (which were greatest at the top of the building, and decreased towards elevation level). The columns at the bottom of the structure for the long span did not require an increase in size since heavy W-sections were already in use. The correlations between column size and story height has been plotted in Figure 55 and Figure 56.

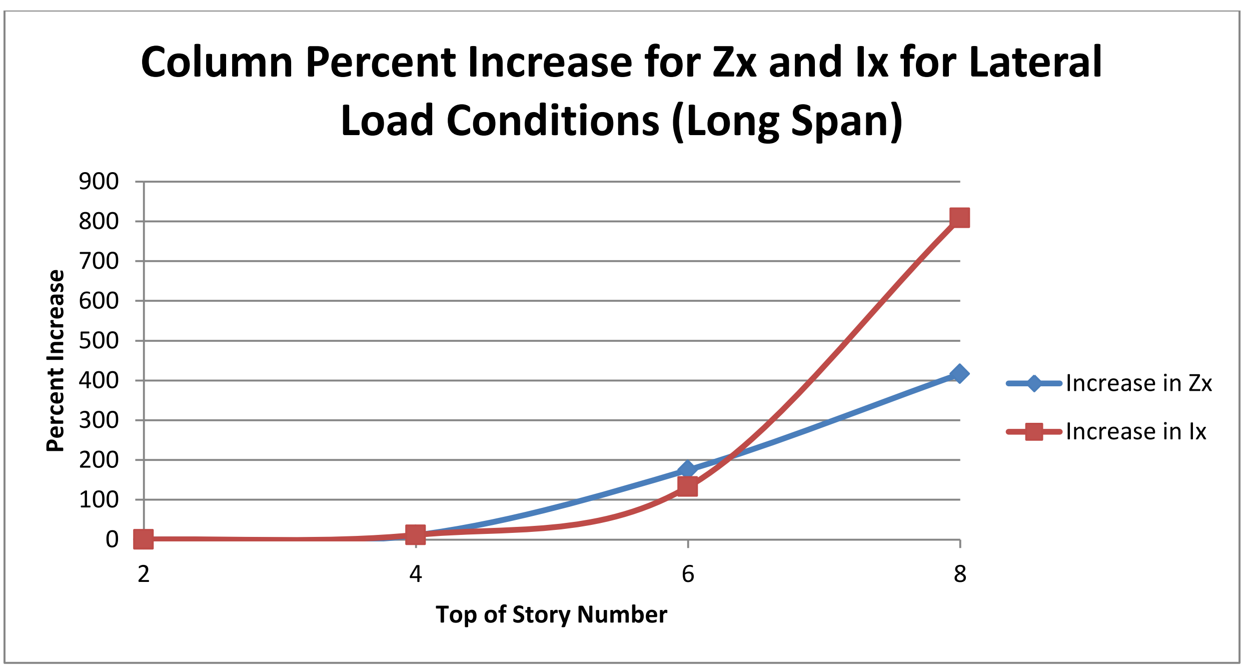


Figure 55: Column Increases of Zx and Ix for Long Span

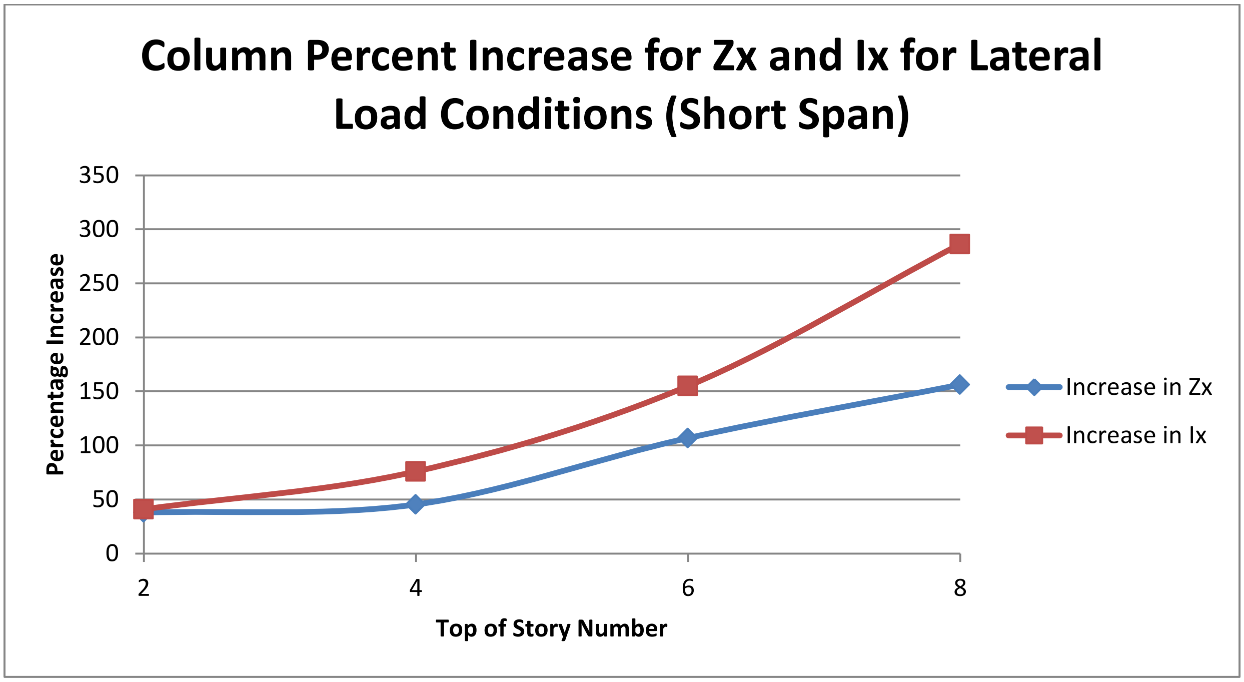


Figure 56: Column Increases of Zx and Ix for Short Span

Figure 57 and Figure 58 show that the increase in size of the members for the long span was much more dramatic than the short span. This may be because in one way, having so many columns for the short span is acting in it owns as a form of bracing, or perhaps because the shorter girder lengths in the span are more effective in restraining the columns and so the lateral drift is not as large. The MQP team believes that there are possibilities in the future for MQP's to do a full study on the way that a structural frame would increase when comparing a frame with just gravity loads to lateral and gravity loads.

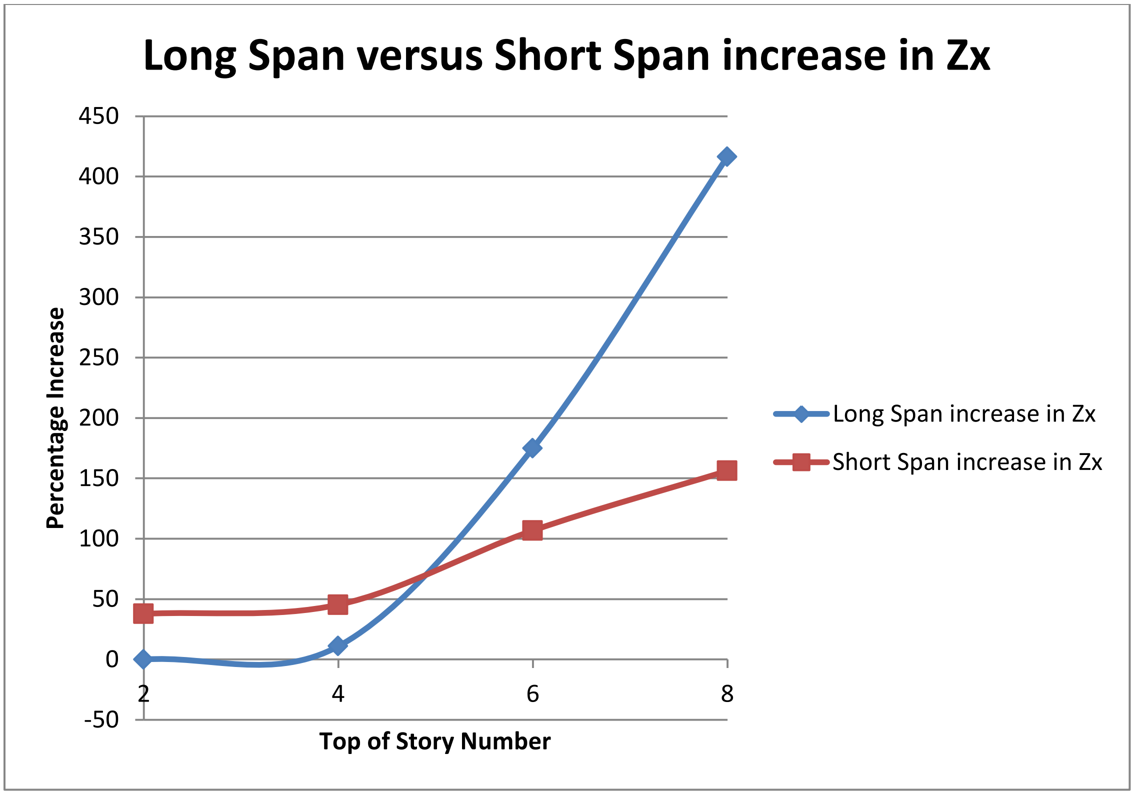


Figure 57: Increase in Zx for Combined Axial and Lateral Load Conditions

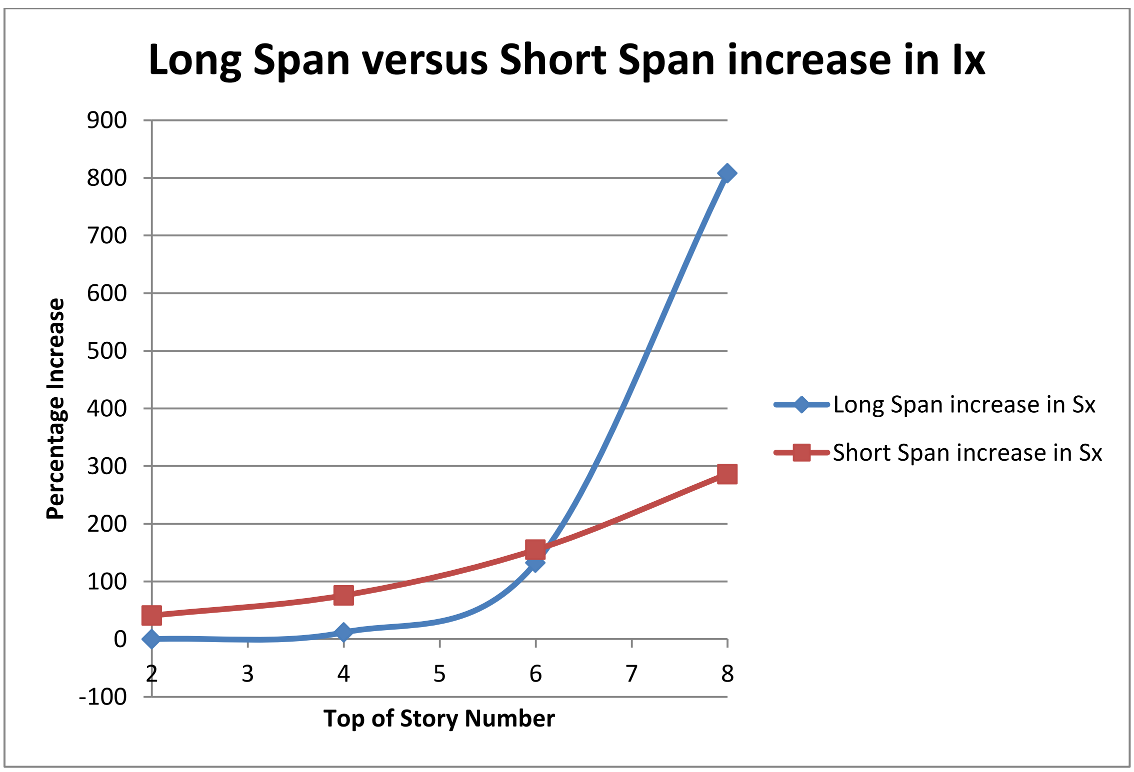


Figure 58: Increase in Ix for Combined Axial and Lateral Load Conditions

6.7 Use of Efficient Structural Software

For this MQP, the team utilized the *MASTAN2* as the structural analysis software. During the initial usage, the MQP team was led to believe that it would be an adequate program to handle all of the team's work. However, due to some of the programs constraints and lack of modern features, the MQP team believes that the program in part slowed down much of the work.

MASTAN2 had several limitations. For building the frame, each node has to be input manually by using coordinates, unlike many other programs where one can simply "draw" in the nodes/members. To change member sizes, one needs to individually select each member; you cannot "highlight" multiple members, or specify "all beam" or "all columns," for example. *MASTAN2* does not give member deflections at the mid-point, you can only find deflections of members based on the node displacements. If there was a particular member for which one wanted to know the midpoint displacement, one had to select the member, delete it, create a node at the midpoint, create two members connecting to that midpoint, attach section criteria, and then recreate the load acting on that member. A subsequent analysis would then provide displacement information for the newly created node.

Many other operations in *MASTAN2* were just as tedious as finding the displacement of members. Unlike other programs where all loading conditions can be analyzed on the same frame, a separate file had to be saved for each loading condition. *MASTAN2* also does not give member data if you were to click on a particular member. This is a problem when changing the member size of, for example, all columns. If one were to potentially forget a column when switching from W14X99 to W14X109, there is no convenient means to be certain that all members were changed without going through and selecting all of the columns again and changing them to the newer section again.

There are several programs of which the MQP looked into using for this project, but ultimately did not use. *RISA 2-D* was one such program. *RISA 2-D* is installed on the school computers as a demo program. This is a great program for smaller structures; however, since the school only has the rights to use the demo version of this program, there is a limit on the number of members one can build a model for and still save. With such a large structure, it would be near impossible to rebuild the structure every time the MQP group worked on the project and still finish this project on time. The MQP team also tried installed the educational version of *RISA 2-D* on a personal laptop, however, that program would crash whenever trying to save the model, so that was not an viable option either. *Autodesk Robot* was another program that was considered for structural analysis; after talking to other groups working with the program, it was decided that it is currently too "buggy," subject to many glitches, and has a very steep learning curve. *SAP2000* was another program of which the MQP team acquired information about. The group installed *SAP2000* onto one of the personal laptops. *SAP2000* seems to be one of the best programs on the market with the power to do many operations which would have sped up this MQP. Nonetheless, the team did not use this program due to the total amount of time invested into *MASTAN2* by the end of B-Term when this program was installed. For other MQP groups performing a structural analysis, the MQP team would recommend installing this program at the start of A-Term and working through all of the given tutorials provided on the *SAP2000* website.

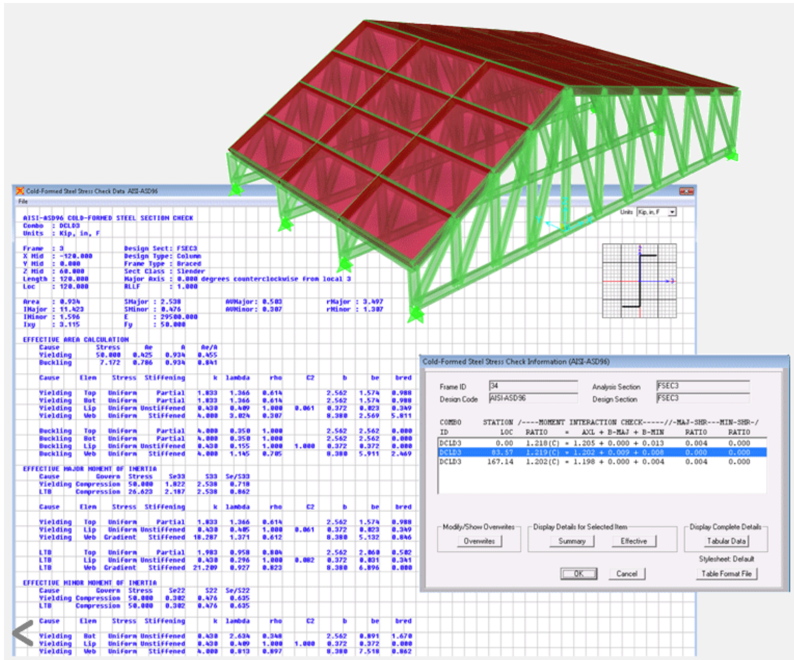


Figure 59: SAP 2000 Software

Talking to other MQP groups, many had the similar issues in using an adequate structural analysis software. Many students would like to see the computers in the MQP Lab, located on the 2nd story of the south side of Kaven Hall, turn into an area actually used for MQP activities. There could be 5-10 computers with dedicated structural software on computers designed to run these programs, such as what is at many other schools' Civil Engineering departments. The computers currently in this room are out-dated compared to the other computers found in Kaven Hall. Although free student programs, such *Autodesk Robot*, can be installed on personal laptops for free, this only works if there is a student in the group with a laptop powerful enough to run this software. There is also no assistance to be found if there are errors running this software on a personal computer. Even computers such as those in Kaven Hall room 202 have trouble running the larger Revit models created without freezing and lagging severely. The MQP team hopes that in future years, structural analysis software is made readily available to the students in a dedicated room for working on MQP on high-powered computers.

6.8 In-depth Parking Analysis

This development is requiring the addition of 531 parking spaces to account for additional vehicles parking on site. This figure is an over estimate and was compiled prior to the reduction from residents that work on site. It is expected that the number of new spaces actually required for this building is far less than 531. The MQP team feels that further study should be conducted to estimate the exact magnitude of this reduction.

The team noted three major factors could have a role in reducing the parking requirements. The first is a study based on the number of spaces that could be eliminated due to unutilized spaces that are within the existing Gateway parking garage. A large number of vacant spaces was noted throughout all times of the day at which the team visited the garage. The second factor is WPI's close proximity to the network of public transportation. This will allow site users to utilize alternate, environmentally friendly modes of transit other than individual vehicles. The third potential reduction the team noted was WPI campus as a whole will be adding a new parking facility below the baseball field adjacent to Park Avenue. This garage will allow for additional parking on campus and may further reduce the need for additional parking at Gateway Park.

These areas of reduction would require a study that would encompass the entire WPI campus parking system as a whole. In completing this study WPI would be able to contribute to a greener community by reducing impervious surfaces and encouraging alternative modes of transportation. This would in turn reduce WPI's carbon foot print as well as their contributions to the Worcester heat island effect.

7 APPENDIX A : STRUCTURAL CALCULATIONS

Sample Short Span Hand Calculations-Beams

by: Mike

Checked by: Jodi-Lee

Michael O'Brien | MQP | Live + Dead Loads | Pg 1

24' x 21.5' bay

Short span

Concrete = 145 lb/cuft

LL Residential = 40 psf

LL offices = 50 psf

L_R Roof Maint. = 20 psf

D = 57 psf (3 psf for decking)

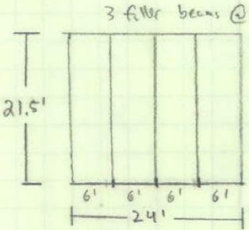
Ceiling + Insulation + MEP =

3 psf + 2 psf + 5 psf = 10 psf

LL offices = 20 psf partitions

Snow load = 55 psf

3 filler beams @ 6'



Trib area = 21.5' x 6'

D = (57 − 3)1.1 = 59.4 psf + 3 = 62.4 psf (Accounts for ponding)

Design for offices since greater LL.

LL + Partitions = (50 + 20)(6 ft) = 420 lb/ft

Ceiling + Insulation + MEP = 10 psf (6 ft) = 60 lb/ft

Concrete + Decking = 62.4 psf (6') = 374.4 lb/ft

Calculate loads for office bay since it will be the greatest

$W_u = 1.2D + 1.6L = 1.2(374.4 + 60) + 1.6(420) = 1193.28\ lb/ft = 1.193\ k/f$

Assume simply supported

$$M_u = \frac{W_u L^2}{8} = \frac{1.193(21.5)^2}{8} = 68.9^{\prime k}$$

Calculate loads for roof from snow load + maintenance

$S = 55\,psf\,(6') = 330\ lb/f$ ← (use $1.2D + 1.6S$)

$L_R = 20\,psf\,(6') = 120\ lb/f$

$W_u = 1.2(374.4 + 60) + 1.6(330) = 1049.28\ lb/ft = 1.049\ k/f$

Assume simply supported

$$M_u = \frac{W_u L^2}{8} = \frac{1.049(21.5)^2}{8} = 60.6^{\prime k}$$

5 1/2" slab

assume a = 1"

$$Y_2\ \text{assumed} = Y_{con} - \frac{a}{2} = 5\tfrac{1}{2}'' - \frac{1''}{2} = 5''$$

Identify alternative beam size. Must satisfy $\phi M_n \geq M_u$ from AISC table 3-19.

Roof beams + floor beams try the following:

Shape	PNA	ΣQ_n	ϕM_n	Page
W10X12	7	44.2	76.5	3-189

Michael O'Brien | MQP | Beam Sizing - Short span | pg2

W10X12 $\Sigma Q_n = 44.2$ $b_e = 6' = 72"$ or $\frac{2(L)}{8} = \frac{2(21.5)}{8} = 5.375 * 12 = 64.5"$

Calculate the actual depth of the concrete stress block + actual value of Y_2
$f'_c = 4000\,psi = 4ksi$

$$a = \frac{\Sigma Q_n}{.85 f'_c b_e} = \frac{44.2}{.85(4)(64.5)} = .20"$$

Calculate actual value for Y_2

$$Y_2 = Y_{con} - \frac{a}{2} = 5\tfrac{1}{2} - \frac{.20}{2} = 5.40"$$

Calculate actual capacity of composite section using interpolation

$$\phi M_n = 76.5 + \frac{.40}{.5}(78.1 - 76.5) = 77.8'^k > 68.9'^k \text{ (floors) or } 60.6'^k \text{ (roof)}$$

Use ΣQ_n from table 3-20 to calculate the number of 3/4" ϕ studs required along the beam span
$\Sigma Q_n = 44.2$ AISC requires studs to be 2X thickness of beam
$T_f = 3/16"$ use Q_n 3/8"
Assume 1 weak stud per rib.
$Q_n = 4.31K$

$$N = \frac{2\Sigma Q_n}{Q_n} = \frac{2(44.2)}{4.31} = 20.5 = 22 \text{ studs}$$ (11 studs on each side of max moment on the beam).

Check serviceability deflection
From table 3-20 in AISC manual:

$$I_{LB} = 124 + \frac{.40}{.5}(131 - 124) = 129.6\,in^4$$

Limit service live load to $\frac{L}{360} = \frac{21.5(12)}{360} = .72"$ (controls) or 1"

Use unfactored LL to check deflection
Roof: 330 lb/f = .33k/f
Offices: 420 lb/f = .42k/f

Try office deflection first, since this load is greater

$$\Delta_c = \frac{5wL^4}{384EI} = \frac{5(.42)(21.5 \times 12)^4}{384(29\times10^3)(129.6)} \cdot \frac{1'}{12"} = .53"$$

New

Michael O'Brien | MQP | Beam Sizing - Short Span | Pg 3

W10X12

Next check the strength and deflection performance of the beam for use in unshored construction. Assume weight of wet concrete is a LL and an additional LL of 20 psf,

Limit unshored deflection to .72" maximum.

Steel beam weight = 12 lb/f   Concrete w/ ponding = 59.4 (6') = 356.4 lb/f

20 psf (6') = 120 lb/f   Decking = 3 psf (6') = 18 lb/f

$W_u = 1.2(12+18) + 1.6(356.4+120) = 798.24 = .798 k/f$

$M_u = \frac{.798(21.5)^2}{8} = 46.1^{1k} < 47.25^{1k}$ ✓ o.k. $M_n = F_y Z_x = 50(12.6)\frac{1}{12} = 52.5$
$M_p = \phi M_n = .9(52.5) = 47.25^{1k}$

Use I_x of beam to check deflection

$\Delta c = \frac{5(.012+.018+.3564+.12)(21.5\times12)^4}{384(29\times10^3)(53.8)} \frac{1'}{12''} = 1.56'' > .72''$

Not okay! Beam needs to be stiffer for construction.

Next, solve for I_x to ensure beam doesn't deflect too much when concrete is wet.

$(.72)(384)(29\times10^3)(I_x)(12) = 5(.012+.018+.3564+.12)(21.5\times12)^4$

$I_x \geq 116.7 in^4$

Lightest beam that satisfies inequality is W12X19

Calculate new Mu due to added beam weight:

Roof $W_u = 1.2(374.4+60+19) + 1.6(330) = 1071.6 = 1.072$ klf

Floor $W_u = 1.2(374.4+60+19) + 1.6(420) = 1216.1 = 1.216$ klf

Roof $M_u = \frac{1.072(21.5)^2}{8} = 61.9^{1k}$ W12X19 w/ PNA = 7 and $Y_2 = 5''$ (assumed) gives $\phi M_n = 143^{1k}$, $\Sigma Q_n = 69.7k$

Floor $M_u = \frac{1.216(21.5)^2}{8} = 70.3^{1k}$

Calculate depth of stress block

$a = \frac{\Sigma Q_n}{.85 f'_c b_e} = \frac{69.7}{.85(4)(64.5)} = .32''$

Calculate actual value for Y_2

$Y_2 = Y_{con} - \frac{a}{2} = 5\frac{1}{2} - \frac{.32}{2} = 5.34''$

Calculate actual capacity of composite section using interpolation

$\phi M_n = 143 + \frac{.34}{.5}(145-143) = 144.36^{1k} > 61.9^{1k}$ (roof) or 70.3^{1k} (floors)

New

Michael O'Brien | MQP | Beam Sizing - short span | pg 4

W12 x 19

Check serviceability deflection (max .72")

Find I_{LB} using interpolation (from table 3-20)

$$I_{LB} = 267 + \frac{.34}{.5}(280-267) = 275.84$$

Check roof deflection first

$$\Delta_c = \frac{5(.33)(21.5\times12)^4}{384(29\times10^3)(275.84)}\frac{1}{12} = .20'' \checkmark \text{ o.k.}$$

Check office deflection

$$\Delta_c = \frac{5(.42)(21.5\times12)^4}{384(29\times10^3)(275.84)}\frac{1'}{12''} = .25'' \checkmark \text{ o.k.}$$

Next check strength and deflection performance of the beam for use in unshored construction. Assume weight of wet concrete is a LL and additional construction LL of 20 psf. Limit deflection to .72"

Steel beam = 19 lb/f Concrete = 356.4 lb/f LL = 120 lb/f Decking = 18 lb/f

$Z_x = 24.7$ $M_n = F_y Z_x = 50(24.7)(\frac{1}{12}) = 102.9$ 'k

$\Phi M_n = .9(102.9) = 92.625$ 'k

$W_u = 1.2(18+19) + 1.6(356.4+120) = 806.6$ lb/f $= .807$ k/f

$$M_u = \frac{.807(21.5)^2}{8} = 46.6 < 92.625\text{'k} \checkmark \text{ o.k.}$$

Use I_x of beam only ($I_x = 130 \text{ in}^4$)

$$\Delta_c = \frac{5(.019+.3564+.12+.018)(21.5\times12)^4}{384(29\times10^3)(130)}\frac{1'}{12''} = .65'' < .72'' \checkmark \text{ o.k.}$$

New C

Michael O'Brien | MQP | Beam Sizing - Short Span | Pg 5

W12X19 w/ 1.5" L.O.K. - Floor metal decking, (12" rib spacing)

11 studs on each side of max moment
22 studs total

$N = \frac{2\Sigma Q_n}{Q_n}$ $\quad 22 = \frac{2(69.7)}{Q_n}$ $\quad Q_n \geq 6.34k$

All studs are assumed to be in weak position
Choose ½" shear studs, $Q_n = 7.66k$

Thus $\Sigma Q_n = 84.26k$

Check that strength of concrete cannot be greater than strength of stud.

$Q_n = .5 A_{sc}\sqrt{f'_c E_c} \leq R_g R_p A_{sc} F_u$

$A_{sc} = \frac{\pi(.5)^2}{4} = .1963 \text{ in}^2$

For beams, decking orientated perpendicular, 1 stud/rib.
$R_g = 1.0$
$R_p = .6$

$E_c = (w^{1.5})\sqrt{f'_c} = (145^{1.5})\sqrt{4} = 3492$

$Q_n = .5(.1963)\sqrt{4(3492)} \leq (1)(.6)(.1963)(65)$

$Q_n = 11.6 \leq 7.66$ ✓ $Q_n = 7.66$ o.k.

Check that 12" stud spacing falls within acceptable limits

Minimum spacing $= 6d = 6(\tfrac{1}{2}") = 3"$

Maximum spacing $= 8t_s = 8(5.5") = 44"$

Our spacing = 12" $\quad 3" \leq 12" \leq 44"$ ✓ o.k.

Summary
Use W12X19 for beams
Use ½" Shear studs w/ 12" spacing
Composite deflection = .28" (roof), .36" (floors) < .72" max
Construction deflection = .65"

7.1 Sample Short Span Hand Calculations- Girders

Michael O'Brien | MQP | Short Span - Girders | Pg 1

Load Calculations:

Roof:

$S = 55(21.5) = 1183$ lb/lf

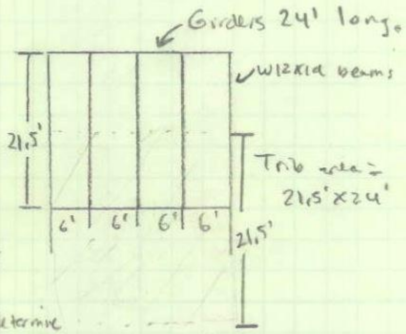
Ceiling + Insulation + MEP = 10 psf(21.5) = 215 lb/lf

Concrete + Decking = 62.4 psf(21.5) = 1342 lb/lf

Beam weight = (5 beams)(19 lb/lf)(21.5') = 2042.5 lb (total)

Beam weight = $\frac{2042.5}{24'} = 85.1$ lb/lf

Office live load w/ partitions = (50+20)(21.5) = 1505 lb/lf



Look @ unshored deflection approximate to determine girder size based on I_x required, Concrete is LL, Construction LL of 20 psf

Assume girder weight 40 lb/lf

20 psf(21.5) = 430 lb/lf

Concrete w/ ponding = 59.4(21.5') = 1277 lb/lf

Decking = 3 psf (21.5') = 64.5 lb/lf

Limit deflection to $\frac{L}{360} = \frac{24(12)}{360} = .8''$ or $1''$ (governs)

$$.8(384)(29\times10^3)(I_x)(12) = 5(.04 + .0645 + 1.277 + .43 + .085)(24\times12)^4$$

$$I_x \geq 610$$

Lightest beam to satisfy inequality **W18x40** (from table 3-3)

$I_x = 612$ in^4

$Z_x = 78.4$ in^3

W18x40 w/ ϕ PNA = 7'' (assumed) $Y_2 = 5''$ (also assumed)

gives $\Phi M_n = 422^{k}$

$\Sigma Q_n = 147$ K

Calculate M_u knowing beam size:

Roof $W_u = 1.2(1342 + 40 + 215 + 85.1) + 1.6(1183) = 3911$ lb/lf $= 3.91$ k/lf

Floor $W_u = 1.2(1342 + 40 + 215 + 85.1) + 1.6(1505) = 4426.5$ lb/lf $= 4.43$ k/lf

$$\text{Roof } M_u = \frac{3.91(24)^2}{8} = 281.5^{\text{ k}}$$

$$\text{Floor } M_u = \frac{4.43(24)^2}{8} = 319.0^{\text{ k}}$$

Calculate depth of stress block $b_e = 21.5' = 258''$ or $b_e = \frac{2(L)}{8} = \frac{2(24)}{8} = 6'(12'') = 72''$

$$a = \frac{\Sigma Q_n}{.85 f'_c b_e} \qquad a = \frac{147}{.85(4)(72)} = .6''$$

Calculate actual value for Y_2

$$Y_2 = Y_{con} - \frac{a}{2} = 5\tfrac{1}{2} - \frac{.6}{2} = 5.20''$$

Calculate actual capacity of composite section using interpolation

$$\Phi M_n = 422 + \frac{.2}{.5}(428 - 422) = 424.4^{\text{ k}} > 281.5^{\text{ k}} \text{ (roof) or } 319.0^{\text{ k}} \text{ (floor)} \quad \checkmark \text{ ok.}$$

Michael O'Brien | MQP | Short-Span Girders | A2

W18X40

Check serviceability deflection (max .8")

Find I_{LB} using interpolation

$I_{LB} = 1070 + \frac{.2}{.5}(1100-1070) = 1082\ in^4$

Check deflection of roof first

$$\Delta_c = \frac{5(1.183)(24\times12)^4}{384(29\times10^3)(1082)}\frac{1'}{12''} = .28'' < .8''\ \checkmark\ o.k.$$

Check deflection of offices

$$\Delta_c = \frac{5(1.505)(24\times12)^4}{384(29\times10^3)(1082)}\frac{1'}{12''} = .36'' < .8''\ \checkmark\ o.k.$$

Next, check strength + deflection performance of the beam for use in unshored construction. Assume weight of wet concrete is a LL and additional construction LL of 20 psf. Limit deflection to .8"

Steel girder = 40 lb/f Concrete w/ ponding: 59.4 (21.5') = 1277 lb/f

LL = 20(21.5') = 430 lb/f Decking = 3 psf (21.5'): 64.5 lb/f

Use I_x of beam only (I_x = 612 in⁴)

$$\Delta_c = \frac{5(.04+1.277+.43+.0645+.065)(24\times12)^4}{384(29\times10^3)(612)}\frac{1'}{12''} = .798'' < .8''\ \checkmark\ o.k.$$

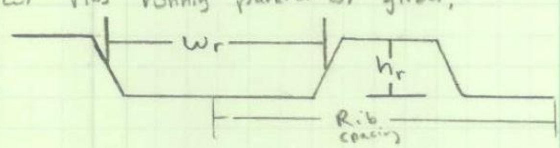
$M_n = F_y Z_x = 50(78.4)\frac{1'}{12''} = 326.7$

$W_u = 1.2(40+64.5) + 1.6(1277+430) = 2856.61\ lb/f = 2.86\ k/f$

$$M_u = \frac{2.86(24)^2}{8} = 205.9'^k < 326.7'^k\ \checkmark\ o.k.$$

Using 1.5" L.O.K. - Floor Metal decking w/ ribs running parallel w/ girder,

$W_r = 6''$ $h_r = 1.5''$



$\frac{W_r}{h_r} = \frac{6}{1.5} = 4 \geq 1.5$

With decking parallel, $R_g = 1.0$, $R_p = .75$ $A_{sc} = \frac{\pi(.5)^2}{2} = .1963$

Try ½" shear studs (since same size as beams)

$Q_n = .5(.1963)\sqrt{4(3492)} \leq 1(.75)(.1963)(65)$ $E_c = 145^{1.5}\sqrt{4} = 3492$

$11.6 \leq 9.57k$

Check Q_n from table 3-21, w/ f'_c = 4 ksi, Q_n = 9.57k

Calculate # of ½" shear studs to be used along girder:

$N = \frac{2\Sigma Q_n}{Q_n} = \frac{2(147)}{9.57} = 30.72 = 32 = 16$ on each side of max moment (center) of girder.

Determine spacing: Minimum spacing = $6d = 6(\frac{1}{2}) = 3''$

Maximum spacing = $8t_s = 8(5.5) = 44''$

Our spacing = $\frac{288}{2}\left(\frac{1}{16}\right) = 9''$ spacing $3'' < 9'' < 44''\ \checkmark\ o.k.$

Summary Use W18X40 for girders

7.2 Sample Short Span Excel Calculations-Beams

By: Jodi-Lee

Checked by: Mike

SHORT SPAN BEAM-FLOOR

Short Span

Load Combination	psf	Trib area(ft)	lb/ft
Dead Loads			
Beam			0
Decking and Slab+ Ponding	62.4	6	**374.4**
Ceiling	3	6	**18**
MEP	5	6	**30**
Insulation	2	6	**12**
Total Dead Load			**434.4**
Roof Live Load			
Maintenance	20	6	**120**
Snow	55	6	**330**
Total Roof Live Load			450
Floor Live Load			
Office	50	6	**300**
Residential	40	6	**240**
Partitions	20	6	**120**
Total Floor Live Load(Office)			420
Total Floor Live Load (Residential)			360

Maximum floor live load occurs for office

STEP 1: LRFD Factored Loading			
	1.2D	1.6L	Wu(k)
Beams on Roof	521.28	528	1.04928
Beams on Floor Office	521.28	672	1.19328

length of bay(ft)	21.5	beam spacing(ft)	6

STEP 2: Determine Mu (1/8WuL^2)	
Beams on Roof(kip-ft)	61
Beams on Floor(kip-ft)	69

Above Does not Include Weight of Beam

Assume a=1	Therefore Y=Ycon=5.5-1/2	5

FOR FLOOR TRY A W 10X12

STEP 3: Use of Tables to Determine Values

Composite W shapes Table 3.19 (pg 3-189)	Mu(ft-kip)	69
From Table 1-1	Area (in^2)	3.54
	I(in^4)	53.8
	Fy(ksi)	50

Step 4: Assume ΣQn= Asy*Fy

Full Composite	ΣQn(k)	177
Partial Composite=(PNA=7)	ΣQn(k)	44.2
STEP 5: Determine Be	Be= 2l/8	Be2=2s/2
Select a Be Value (minimum value of the two)	5.375	6
	2L/8 Controls	

Step 6: Determine a

a=ΣQn/(.85*f'c*bE)	ΣQn(kips)		44.2
	F'c(ksi)	4	
	be'	64.5	
	a	0.202	

Step 7: Interpolate to Find Mu Using Table 3-19

a) Calculate Y2	Ycon	5.5
	a/2	0.100775194
	Y2	5.399
b) Interpolate	upper value	78.1
from table 3-19	lower value	76.5
from table 3-19	fraction	0.798
	ΦbMn	77.78

Step 8: Determine Deflection Due to Live Load

	LL(kip-ft)	0.42
	L(ft)	21.5
Step (8a) calculate Moment Due to live Load	Ml	24.268125
Step (8b) Interpolate- Using Table 3-20	Lower Y2	124
From Table 3-21	Upper Y2	131
From Table 3-20	Fraction	0.798
	Interpolated	129.589

Step (8c) Determine Δl

=Ml*(L^2)/161*Ilower Bound

Ml	24.2681 25
L	21.5
Loading Constant	161
Ilb	129.5891473
ΔL	0.538
L/360	0.7167
1 " Max	1

Next: check to see if will have okay deflection during unshored construction

Step (9d) Determine if ΔL is Sufficient

Deflection okay

STEP 9: Determine Loads during Unshored Construction

Check Unshored construction Deflection

		psf	Trib area(ft)
Dead Loads			
Decking	3	6	
Beam		6	lb/ft
Live Loads			18
LLWork	20	6	12
Llconcrete	59.4	6	
Total Unfactored Loads			120
			356.4
Factored Loads			506.4

LRFD Factored Loading			
	1.2D	1.6L	
Beams on Roof	[redacted]	[redacted]	
Beams on Floor Office	36	762.24	Wu(k)
			[redacted]
			0.79824

Step 10: Determine if Mu <φMn

φ	0.9
L(ft)	21.5
Zx(Table 1-1)	12.6
Mu	46.123
φMn	47.25

Step 11: Determine Deflection Due to Unshored Construction

	Unfactored Loads	0.5064	
		L(ft)	21.5
	Ix	53.8	
	Max Δ allow	0.72	
a) calculate Moment Due to live Load	Ml	29.260425	
b) Deflection	DL	1.56	
d)Determine if ΔL is Sufficient	No good try new beam size		

Step 12: Solve For new Ix	117.2
Table 3-3 pg 3-21	Select a W 12 X 19 Since 117.2 in^4 > 84.0in^4

Step 13: Select new Size Beam

Using Table 3-20	W 12 X 19

Determine New Mu(Including Weight of Beam)

Load Combination	psf	Trib area(ft)	
Dead Loads			
Beam			lb/ft
Decking and Slab+ Ponding	62.4	6	
Ceiling	3	6	19
MEP	5	6	**374.4**
Insulation	2	6	**18**
Total Dead Load			**30**
Roof Live Load			**12**
Maintenance	20	6	**434.4**
Snow	55	6	
Total Roof Live Load			**120**
Floor Live Load			**330**
Office	50	6	450
Residential	40	6	
Partitions	20	6	**300**
Total Floor Live Load(Office)			**240**
Total Floor Live Load (Residential)			**120**

			420
STEP 1: LRFD Factored Loading			360
	1.2D	1.6L	
Beams on Roof	521.28	528	
Beams on Floor Office	521.28	672	Wu(k)
			1.04928
length of bay(ft)	21.5	beam spacing(ft)	1.19328
STEP 2: Determine Mu (1/8WuL^2)			
Beams on Roof(kip-ft)	61		6
Beams on Floor(kip-ft)	69		

Above Does not Include Weight of Beam

	Therefore	
Assume a=1	Y=Ycon=5.5-1/2	5

FOR FLOOR TRY A W 10X12

STEP 3: Use of Tables to Determine Values

Composite W shapes Table 3.19 (pg 3-189)	Mu(ft-kip)	69
From Table 1-1	Area (in^2)	4.16
	I(in^4)	130
	Fy(ksi)	50
	Zx	24.7

Step 4: Assume ΣQn= Asy*Fy

Full Composite	ΣQn(k)	279
Partial Composite=(PNA=7) ASIC Table 3-20 pg 3-205	ΣQn(k)	69.7
STEP 5: Determine Be	Be= 2l/8	Be2=2s/2
	5.375	6
Select a Be Value (minimum value of the two)	2L/8 Controls	

c)Determine a

a=ΣQn/(.85*f'c*bE)	ΣQn(kips)	69.7
	F'c(ksi)	4
	be'	64.5
	a	0.32

d) Interpolate to Find Mu Using Table 3-19

a) Calculate Y2	Ycon	5.5
	a/2	0.158914729
	Y2	5.341085271
b) Interpolate	upper value	145
from table 3-19	lower value	143
from table 3-19	fraction	0.682170543
	ΦbMn	144.3643411

f) Determine Deflection Due to Live Load		
	LL(kip-ft)	0.42
	L(ft)	21.5
i) calc ulate Moment Due to live Load	Ml	24.268125

ii) Interpolate- Using Table 3-20	Lower Y2	267
	Upper Y2	280
	Fraction	0.682170543
	Interpolated	275.8682171

g) Determine Δl		
=Ml*(L^2)/161*Ilower Bound	Ml	24.268125
	L	21.5
	Loading Constant	161
	Ilb	275.8682171
	ΔL(")	0.25257223
Controls	L/360("(	0.716666667
	Max"	0.716
	Deflection okay	

STEP 11: Determine Loads during Unshored Construction

Check Unshored construction Deflection

Assume wet concrete is a live load and has a value of 20psf

Dead Loads	psf	Trib area(ft)	
Decking		3	6
Beam		6	lb/ft
Live Loads			18
LLWork	20	6	19
Llconcrete	59.4	6	
Total Unfactored Loads			120

			356.4
Step 12: Determine Deflection Due to Unshored Construction			513.4
	Unfactored Loads	0.5134	
		L(ft)	21.5
	Ix	130	
Controls	Max Δ allow	0.72	
a) calculate Moment Due to live Load	Ml	29.7	
b) Deflection	DL	0.6552	
d)Determine if ΔL is Sufficient	Deflection okay		

Step: Check Strength of beam for unshored Construction

Determine Factored Loads

Dead Loads		psf	Trib area(ft)
Decking	3	6	
Beam		6	lb/ft
Total Dead Loads (Unfactored)			18
Live Loads			19
LLWork	20	6	37

Llconcrete	59.4	6	
Total Live Loads (Unfactored)			120
			356.4
	Live	**Dead**	476.4
Factor	1.2	1.60	
Factored Loads	0.0444	0.76224	**Total**
Mu (ft-kip)	46.6086675		0.807
Mn= FyZx (ft-kip)	102.9166667		
Mp=φMn= FyZx (ft-kip), *Where ϕ=.9*	92.625		
Is Strength okay? IFF Mu< φMn	Strength Okay		

SELECT a W 12 X 19 FOR FLOOR BEAMS IN SHORT SPAN

7.3 Sample Short Span Excel Calculations-Girders

SHORT SPAN GIRDER-FLOOR

LONG SPAN BEAM-FLOOR

STEP 1: Determine Loads during Unshored Construction

Check Unshored construction Deflection

Dead Loads	psf	Trib area(ft)	lb/ft
Decking	3	21.5	64.5
Beam		21.5	40
Live Loads			
LLWork	20	21.5	430
Llconcrete	59.4	21.5	1277.1
Total Unfactored Loads			1811.6

Factored Loads

LRFD Factored Loading			
	1.2D	1.6L	Wu(k)
Beams on Roof			
Beams on Floor Office	125.4	2731.36	2.85676

Step 2: Determine Deflection Due to Unshored Construction

	Unfactored Loads	1.8116
	L(ft)	24
Assume Ix=300	Ix	300
	Max Δ allow	1.00
a) calculate Moment Due to live Load	Ml	130.4352

	L/360(L in ")	0.8
b) Deflection	DL	1.56
d)Determine if ΔL is Sufficient	No good try new beam size	

Step 3: Solve For new Ix (in^4)

Table 3-3 pg 3-21	583.3126957
Select a W 18 X 40 Since Ix=612 > 583	

SHORT SPAN GIRDER FLOOR

Loading Conditions	
No Beams(EA)	5
Wt of each Beam(lb/ft)	19
Tributary Area(ft)	21.5
Beam weight Over Tributary area(lbs)	2042.5
Length of Girder(ft)	24
Beam Weight on Each Girder At 30'(lbs/ft)	85.1

Load Combination	**psf**	**Trib area(ft)**	**lb/ft**
Dead Loads			
Weight of Girder			40
Weight of Beam			85.1
Decking and Slab+ Ponding	62.4	21.5	1341.6
Ceiling	3	21.5	64.5
MEP	5	21.5	107.5

Insulation	2	21.5	43
Total Dead Load			1681.7
Roof Live Load			
Snow	55	21.5	1182.5
Floor Live Load	50	21.5	1075
Office	50	21.5	**1075**
Residential	40	21.5	**860**
Partitions	20	21.5	**430**
Total Floor Live Load(Office)			1505
Total Floor Live Load (Residential)			1290

STEP 4: LRFD Factored Loading			
	1.2D	1.6L	Wu
Girders on Roof	2018.045	1892	3.910045
Girders On Floor	2018.045	2408	4.426045

STEP 5: Determine Mu (1/8WuL^2)	
Girders On Roof	282
Girders on Floor	319

Above Does not Include Weight of Girder

Calculate Y2

Ycon	5.5
assume a=1	1.00
Y2	5

W 18X40 PNA=7

STEP 6: Use of Tables to Determine Values

Composite W shapes Table 3.19	Mu(ft-kip)	758
From Table 1-1	Area (in^2)	11.8
	I(in^4)	612
	Fy	50

Step 4: Assume ΣQn= Asy*Fy	ΣQn	590
	ΣQn (from book)	422
STEP 5: Determine Be	Be= 2l/8	Be2=2s/2
	6	21.5
Select a Be Value	2L/8 Controls	

Determine a

a=ΣQn/(.85*f'c*bE)	ΣQn(kips)	147
	F'c(ksi)	4
	be'	72
	a	0.60

Step 6: Interpolate to Find Mu Using Table 3-19

a) Calculate Y2	Ycon	5.5
	a/2	0.300245098
	Y2	5.199754902
b) Interpolate	upper value	428
from table 3-19	lower value	422

from table 3-19	fraction	0.399509804
	ΦbMn	424.3970588

ΦbMn>Mu OKAY!

Step 8: Determine Deflection Due to Live Load

	LL(kip-ft)	1.505
	L(ft)	24
a) calc ulate Moment Due to live Load	Ml	108.36
b) Interpolate- Using Table 3-20	Lower Y2	1070
	Upper Y2	1100
	Fraction	0.399509804
	Interpolated	1081.985294
c) Determine Δl		
=Ml*(L^2)/161*Ilower Bound	Ml	108.36
	L	24
	Loading Constant	161
	Ilb	1081.985294
	ΔL	0.358297886
	L/360	0.8
	1 " Max	1

Deflection okay

STEP 11: Determine Loads during Unshored Construction

Check Unshored construction Deflection

Dead Loads	psf	Trib area(ft)	lb/ft

Decking	3	21.5	64.5
Beam		21.5	85.1
Live Loads			
LLWork	20	21.5	430
Llconcrete	59.4	21.5	1277.1
Total Unfactored Loads			1856.704167

Step 12: Determine Deflection Due to Unshored Construction

	Unfactored Loads	1.856704167
	L(ft)	24
	Ix (in^4)	612
	Max Δ allow	0.80
a) calculate Moment Due to live Load	Ml	133.6827
b) Deflection	DL	0.78
d)Determine if ΔL is Sufficient	Deflection okay	

Check if Mu< φMn

Mu (ft-kip)	758
Mn= FyZx (ft-kip)	3700
Mp=φMn= FyZx (ft-kip), *Where ϕ=.9*	3330
Is Strength okay? IFF Mu< φMn	Strength Okay

7.4 Sample Long Span Hand Calculations-Beams

By: Mike

Checked by: Jodi-Lee

Michael O'Brien | MQP | Beam Sizing – Long Span

24' x 42' bay

Long Span

Concrete = 145 lb/cu ft

LL Residential = 45 psf

LL Offices = 50 psf

LR Roof Maintenance = 20 psf

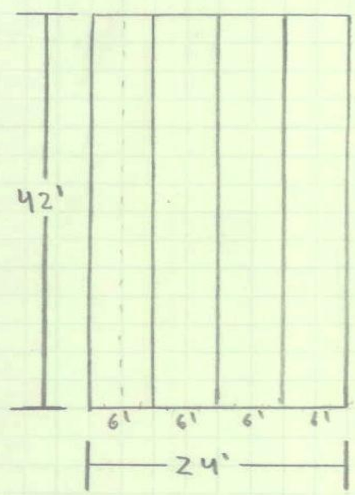
D = 57 psf (3 psf for decking)

Ceiling + Insulation + MEP = 10 psf

LL offices = 20 psf for partitions

Snow Load = 55 psf

D = (57−3)1.1 = 59.4 psf + 3 = 62.4 psf (accounts for ponding)



Design for offices since greater LL.

LL + Partitions = (50 + 20)(6') = 420 lb/f

Ceiling + Insulation + MEP = 10(6') = 60 lb/f

Concrete + Decking = 62.4(6') = 374.4 lb/f

S = 55 (6) = 330 lb/f

Look @ unshored deflection approximate to determine beam size based on I_x required.

Assume steel beam weight of 24 lb/f   Concrete w/ ponding: 59.4(6') = 356.4 lb/f

Assume construction LL of 20 psf = 20 psf(6') = 120 lb/f   Decking: 3(6') = 18 lb/f

Limit deflection to $\frac{L}{360} = \frac{42(12)}{360} = 1.4''$ or $1''$ (controls)

$384(29\times10^3)(I_x)(12) = 5(.024 + .018 + .3564 + .12)(42 \times 12)^4$

$I_x \geq 1251\ in^4$

Lightest beam that satisfies inequality <u>W24X55</u> (from table 3-3)

Calculate M_u knowing beam size:

Roof $w_u = 1.2(374.4 + 60 + 55) + 1.6(330) = 1115\ lb/f = 1.12\ k/f$

Floor $w_u = 1.2(374.4 + 60 + 55) + 1.6(420) = 1259\ lb/f = 1.26\ k/f$

Roof $M_u = \frac{1.12(42)^2}{8} = 246.96^{1k}$

W24X55 w/ PNA = 7 $Y_2 = 5''$ (assumed)

gives $\phi M_n = 720^{1k}$

$\Sigma Q_n = 203$

Floor $M_u = \frac{1.26(42)^2}{8} = 277.8^{1k}$

Calculate depth of stress block

$a = \frac{\Sigma Q_n}{.85 f'_c b_e}$ $a = \frac{203}{.85(4)(72)} = .83''$

$b_e = 6' = 72''$ or $\frac{2(42)(12)}{8} = 126''$, use b_e as 72''

Calculate actual value for Y_2

$Y_2 = Y_{con} - \frac{a}{2} = 5\frac{1}{2} - \frac{.83}{2} = 5.09''$

Michael O'Brien	MQP	Beam Sizing - Long Span	A2

W24X55

Calculate actual capacity of composite section using interpolation

$\phi M_n = 720 + \frac{.09}{.5}(727-720) = 721.3^{1k} > 246.96^{1k}$ (roof) or 277.8^{1k} (floors)

Check serviceability deflection (max 1")

Find I_{LB} using interpolation

$I_{LB} = 2260 + \frac{.09}{.5}(2320-2260) = 2270.8\ in^4$

Check deflection of roof first

$$\Delta_c = \frac{5(.33)(42\times 12)^4}{384(29\times 10^3)(2270.8)} \frac{1'}{12"} = .35" \checkmark \text{o.k.}$$

Check deflection of offices

$$\Delta_c = \frac{5(.42)(21.5\times 12)^4}{384(29\times 10^3)(2270.8)} \frac{1'}{12"} = .45" \checkmark \text{o.k.}$$

Next check strength + deflection performance of the beam for use in unshored construction. Assume weight of wet concrete is a LL and additional construction LL of 20 psf. Limit deflection to 1"

Steel beam = 55 lb/f   Concrete = 356.4 lb/f   LL = 120 lb/f   Decking = 18 lb/f

$Z_x = 134$ $M_n = F_y Z_x = 50(134)\frac{1}{12} = 558.3^{1k}$

$\phi M_n = .9(558.3) = 502.5^{1k}$

$W_u = 1.2(18+55) + 1.6(356.4+120) = 849.8\ lb/f = .850\ k/f$

$$M_u = \frac{.850(42)^2}{8} = 187.4^{1k} < 502.5^{1k} \checkmark \text{o.k.}$$

Use I_x of beam only   ($I_x = 1350$)

$$\Delta_c = \frac{5(.055+.3564+.12+.018)(42\times 12)^4}{384(29\times 10^3)(1350)} \frac{1'}{12"} = .98" < 1" \checkmark \text{o.k.}$$

Michael O'Brien | MQP | Beam Sizing - Long Span | 3

W24X55 w/ 1.5" L.O.K. - Floor Metal Decking (12" rib spacing)
21 studs on each side of max moment
42 studs total

$$N = \frac{2\Sigma Q_n}{Q_n} \qquad 42 = \frac{2(203)}{Q_n} \qquad Q_n = 9.67K$$

All studs are assumed to be in weak position.
Choose 5/8" shear studs, $Q_n = 12.0K$

Check that 12" spacing falls within acceptable limits
Minimum spacing $= 6d = 6(5/8) = 3.75"$
Maximum spacing $= 8t_s = 8(5.5) = 44"$

$3.75" \leq 12" \leq 44"$ ✓ o.k.

Summary: Use W24X55 Beams for long span
Use 5/8" shear studs w/ 12" spacing
Composite deflection: .35" (roof), .45" (floors)
Construction deflection: .98"

7.5 Sample Short Long Hand Calculations-Girders

Michael O'Brien | MQP | Girders – Long Span

Load Calculations:

Roof: $S = 55psf(42') = 2310\ lb/f$

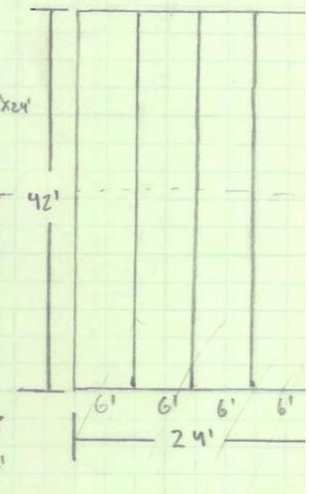
Ceiling + Insulation + MEP = 10 psf (42') = 420 lb/f

Concrete + Decking = 62.4 psf (42') = 2620.8 lb/f

Beam Weight = (5 beams)(55 lb/f)(42') = 11550 lb (total)

Beam Weight = $\frac{11550}{24'} = 481\ lb/f$

Trib Area = 42'x24'



Office LL w/ partitions = (50+20)(42) = 2940 lb/f

Look @ unshored deflection approximate to determine girder size based on I_x required, concrete is a LL, Construction LL of 20 psf.

Assume girder weight 80 lb/f

Concrete w/ ponding = 59.4(42') = 2498 lb/f

LL 20psf (42') = 840 lb/f

Decking 3psf (42') = 126 lb/f

Limit deflection to $\frac{L}{360} = \frac{24(12)}{360} = .8''$ or 1" max

$(.8)384(29\times10^3)(I_x)(12) = 5(.08+2.498+.840+.126+.481)(24\times12)^4$

$I_x \geq 1295$

Lightest beam to satisfy inequality is <u>W24X55</u> (from table 3-3)

$I_x = 1350\ in^4$

$Z_x = 134\ in^3$

From table 3-19 W24X55 w/ PNA = 7" (assumed) $Y_2 = 5''$ (also assumed)

(from table 3-19)

gives $\Phi M_n = 720'^k$

$\Sigma Q_n = 203$

Calculate depth of stress block $b_e = 42' = 504''$ or $\frac{2(24)}{8} = 6'(12) = 72''$

$a = \frac{\Sigma Q_n}{.85(f'c)b_e}$ $a = \frac{203}{.85(4)(72)} = .83''$

Calculate actual value for Y_2

$Y_2 = Y_{con} - \frac{a}{2} = 5\frac{1}{2} - \frac{.83}{2} = 5.09''$

Calculate actual capacity of composite section

$\Phi M_n = 720 + \frac{.09}{.5}(727-720) = 721.3'^k$ $62.4(42) = 2620.8$

Check compared to roof/floor M_u values

Roof $w_u = 1.2(2621+55+420+481)+1.6(2310) = 7980\ lb/f = 7.98\ k/f$

Floor $w_u = 1.2(2621+55+420+481)+1.6(2940) = 8996\ lb/f = 8.996\ k/f$

Roof $M_u = \frac{7.98(24)^2}{8} = 575.5'^k < 721.3'^k$ ✓ o.k.

Floor $M_u = \frac{8.996(24)^2}{8} = 648.0'^k < 721.3'^k$ ✓ o.k.

Michael O'Brien | MQP | Long-Span Girders | pg 2

W24x55

Check serviceability deflection (.8" max)

Find ILB using interpolation

$I_{LB} = 2260 + \frac{.01}{.5}(2320-2260) = 2270.8 \text{ in}^4$

Check deflection of roof first

$$\Delta_L = \frac{5(2.310)(24 \times 12)^4}{384(24\times10^3)(2270.8)} \frac{1'}{12"} = .26" \checkmark \text{ o.k.}$$

Check floor deflection

$$\Delta_e = \frac{5(2.940)(24 \times 12)^4}{384(24\times10^3)(2270.8)} \frac{1'}{12"} = .33" \checkmark \text{ o.k.}$$

Next, check strength + deflection performance of the beam for use in unshored construction. Assume weight of wet concrete is a LL and additional construction LL of 20 psf. Limit deflection to .8"

Steel girder = 55 lb/lf — Concrete w/ ponding = 59.4(42') = 2498 lb/lf

LL = 20(42) = 840 lb/lf — Decking = 3(42) = 126 lb/lf

Beam weight = 481 lb/lf (connected to girders)

Use I_x of beam only ($I_x = 1350$)

$$\Delta_L = \frac{5(.055 + 2.498 + .840 + .126 + .481)(24 \times 12)^4}{384(24\times10^3)(1350)} \frac{1'}{12"} = .76" < .8" \checkmark \text{ o.k.}$$

$M_n = F_y Z_x = 50(134)\frac{1}{12} = 558'^k$

$\phi M_n = .9(558) = 502.5'^k$

$w_u = 1.2(55+126+481) + 1.6(840+2498) = 6135 \text{ lb/lf} = 6.12 \text{ k/lf}$

$$M_u = \frac{6.12(24)^2}{8} = 440.6'^k < 558'^k \checkmark \text{ o.k.}$$

Using 1.5" L.O.K. Floor metal decking w/ ribs running parallel w/ girder

$w_r = 6"$, $h_r = 1.5"$ $\quad \frac{w_r}{h_r} = 4$

Try 5/8" shear studs (same as beam stud size)

$Q_n = 15k$ since $\frac{w_r}{h_r} \geq 1.5$ (from AISC table 3-21)

Calculate # of 5/8" shear studs to be used along girder

$N = \frac{2\Sigma Q_n}{15} = \frac{2(203)}{15} = 27 = 28 = 14$ on each side of max moment of girder

Determine spacing: Minimum Spacing = 6d = 3.75"

Maximum Spacing = 8ts = 8(5.5") = 44"

Our spacing = $\frac{288}{2}\left(\frac{1}{14}\right) = 10.29"$

$3.75" \leq 10.29" \leq 44"$ ✓ o.k.

Summary: Use **W24x55** for girders

Use 5/8" shear studs w/ 10.29" spacing

Composite deflection: .26" (roof), .33" (floors)

Construction deflection: .76"

7.6 Sample Long Span Excel Calculations-Beams

By: Jodi-Lee

Checked by: Mike

LONG SPAN BEAM-FLOOR

LONG SPAN BEAM-FLOOR

STEP 1: Determine Loads during Unshored Construction

Check Unshored construction Deflection

Dead Loads	psf	Trib area(ft)	lb/ft
Decking	3	6	18
Beam		6	24
Live Loads			
LLWork	20	6	120
Llconcrete	59.4	6	356.4
Total Unfactored Loads			518.4

Factored Loads

LRFD Factored Loading			
	1.2D	1.6L	Wu(k)
Beams on Roof			
Beams on Floor Office	50.4	762.24	0.81264

Step 2: Determine Deflection Due to Unshored Construction

Unfactored Loads	0.5184
L(ft)	42

Assume Ix=300	Ix	300
	Max Δ allow	1.00
a) calculate Moment Due to live Load	Ml	114.3072
b) Deflection	DL	4.17
d)Determine if ΔL is Sufficient	No good try new beam size	

Step 12: Solve For new Ix (in^4)	
Table 3-3 pg 3-21	1252.409322

Select a W 24 X 55 Since Ix=1350 > 1252

Load Combination	**psf**	**Trib area(ft)**	**lb/ft**
Dead Loads			
Beam			55
Decking and Slab+ Ponding	62.4	6	**374.4**
Ceiling	3	6	**18**
MEP	5	6	**30**
Insulation	2	6	**12**
Total Dead Load			**489.4**
Roof Live Load			
Maintenance	20	6	**120**

Snow	55	6	**330**
Total Roof Live Load			450
Floor Live Load			
Office	50	6	**300**
Residential	40	6	**240**
Partitions	20	6	**120**
Total Floor Live Load(Office)			420
Total Floor Live Load (Residential)			360

Maximum floor live load occurs for office

STEP 1: LRFD Factored Loading			
	1.2D	1.6L	Wu(k)
Beams on Roof	587.28	528	1.11528
Beams on Floor Office	587.28	672	1.25928

length of bay(ft)	42	beam spacing(ft)	6

STEP 2: Determine Mu (1/8WuL^2)	
Beams on Roof(kip-ft)	246
Beams on Floor(kip-ft)	278

Above Does not Include Weight of Beam

Assume a=1	Therefore Y=Ycon=5.5-1/2	5

STEP 3: Use of Tables to Determine Values

Composite W shapes Table 3.19 (pg 3-189)	Mu(ft-kip)	278
From Table 1-1	Area (in^2)	16.2
	I(in^4)	1350
	Fy(ksi)	50

	Zx(in^3)	134

W 24 X 55

Step 4: Assume ΣQn= Asy*Fy

Full Composite	ΣQn(k)	810
Partial Composite=(PNA=7)	ΣQn(k)	203
STEP 5: Determine Be	Be= 2l/8	Be2=2s/2
	10.5	6
Select a Be Value (minimum value of the two)	2s/2 Controls	

Determine a

a=ΣQn/(.85*f'c*bE)	ΣQn(kips)	203
	F'c(ksi)	4
	be'	72
	a	0.829

Step 6: Interpolate to Find Mu Using Table 3-19

a) Calculate Y2	Ycon	5.5
	a/2	0.414624183
	Y2	5.085
b) Interpolate	upper value	727
from table 3-19	lower value	720
from table 3-19	fraction	0.171
	ΦbMn	721.20

Step 8: Determine Deflection Due to Live

Load

a) calc ulate Moment Due to live Load

b) Interpolate- Using Table 3-20

LL(kip-ft)	0.42
L(ft)	42
Ml	92.61
Lower Y2	2320
Upper Y2	2320
Fraction	0.171
Interpolated	2320.0

c) Determine Δl

=Ml*(L^2)/161*Ilower Bound

Ml	92.61
L	42
Loading Constant	161
Ilb	2320.0
ΔL	0.437
L/360	1.4000
1 " Max	1

Next: check to see if will have okay deflection during unshored construction

d)Determine if ΔL is Sufficient Deflection okay

STEP 9: Determine Loads during Unshored Construction

Check Unshored construction Deflection

Dead Loads

	psf	Trib area(ft)	lb/ft
Decking	3	6	18
Beam		6	55

Live Loads

LL_{Work}	20	6	120
$Ll_{concrete}$	59.4	6	356.4
Total Unfactored Loads			549.4

Factored Loads

LRFD Factored Loading			
	1.2D	1.6L	Wu(k)
Beams on Roof	[redacted]	[redacted]	[redacted]
Beams on Floor Office	87.6	762.24	0.84984

Step 10: Determine if Mu <φMn

φ	0.9
L(ft)	42
Zx(Table 1-1)	134
Mu	187.390
φMn	502.5

Step 11: Determine Deflection Due to Unshored Construction

Unfactored Loads	0.5494
L(ft)	42
Ix	1350

	Max Δ allow	1.00
a) calculate Moment Due to live Load	Ml	121.1427
b) Deflection	DL	0.98
d)Determine if ΔL is Sufficient	Deflection okay	
Is Strength okay? IFF Mu< φMn	Strength Okay	

SELECT a W 24 X 55 FOR FLOOR BEAMS IN LONG SPAN

7.7 Sample Long Span Excel Calculations-Girder

LONG SPAN GIRDER-FLOOR

LONG SPAN BEAM-FLOOR

STEP 1: Determine Loads during Unshored Construction

Check Unshored construction Deflection

Dead Loads	psf	Trib area(ft)	lb/ft
Girder Weight			80
Decking	3	42	126
Beam		42	481
Live Loads			
LLWork	20	42	840
Llconcrete	59.4	42	2494.8
Total Unfactored Loads			4021.8

Factored Loads

LRFD Factored Loading			
	1.2D	1.6L	Wu(k)

Beams on Roof			
Beams on Floor Office	728.4	5335.68	6.06408

Step 2: Determine Deflection Due to Unshored Construction

	Unfactored Loads	4.0218
	L(ft)	24
Assume Ix=300	Ix	300
	Max Δ allow	1.00
a) calculate Moment Due to live Load	Ml	289.5696
	L/360(L in ")	0.8
b) Deflection	DL	3.45
d)Determine if ΔL is Sufficient	No good try new beam size	

Step 3: Solve For new Ix (in^4)

Table 3-3 pg 3-21	1294.96964
Select a W 24 X 55 Since Ix=1350>1294	

LONG SPAN GIRDER FLOOR CONT'D

Loading Conditions	
No Beams(EA)	5
Wt of each Beam(lb/ft)	55
Tributary Area(ft)	42
Beam weight Over Tributary area(lbs)	11550

Length of Girder(ft)	24		
Beam Weight on Each Girder At 30'(lbs/ft)	481.3		
Load Combination	**psf**	**Trib area(ft)**	**lb/ft**
Dead Loads			
Weight of Girder			55
Weight of Beam			481.3
Decking and Slab+ Ponding	62.4	42	2620.8
Ceiling	3	42	126
MEP	5	42	210
Insulation	2	42	84
Total Dead Load			3577.1
Roof Live Load			
Snow	55	42	2310
Floor Live Load	50	42	2100
Office	50	42	**2100**
Residential	40	42	**1680**
Partitions	20	42	**840**
Total Floor Live Load(Office)			2940
Total Floor Live Load (Residential)			2520

STEP 4: LRFD Factored Loading			
	1.2D	1.6L	Wu
Girders on Roof	4292.46	3696	7.98846
Girders On Floor	4292.46	4704	8.99646
STEP 5: Determine Mu (1/8WuL^2)			

Girders On Roof	575
Girders on Floor	648

Above Does not Include Weight of Girder

Calculate Y2

Ycon	5.5
assume a=1	1.00
Y2	5

W 24X55 PNA=7

STEP 5: Use of Tables to Determine Values

Composite W shapes Table 3.19	Mu(ft-kip)	648
From Table 1-1	Area (in^2)	16.2
	I(in^4)	1350
	Fy	50

Step 6: Assume ΣQn= Asy*Fy	ΣQn	810
	ΣQn (from book)	422
STEP 7: Determine Be	Be= 2l/8	Be2=2s/2
	6	42
Select a Be Value	2L/8 Controls	

Determine a

a=ΣQn/(.85*f'c*bE)	ΣQn(kips)	203
	F'c(ksi)	4
	be'	72
	a	0.83

Step 6: Interpolate to Find Mu Using Table 3-19

a) Calculate Y2	Ycon	5.5
	a/2	0.414624183
	Y2	5.085375817
b) Interpolate	upper value	727
from table 3-19	lower value	720
from table 3-19	fraction	0.170751634
	ΦbMn	721.1952614

ΦbMn>Mu OKAY!

Step 8: Determine Deflection Due to Live Load

	LL(kip-ft)	2.94
	L(ft)	24
a) calc ulate Moment Due to live Load	Ml	211.68
b) Interpolate- Using Table 3-20	Lower Y2	2260
	Upper Y2	2320
	Fraction	0.170751634
	Interpolated	2270.245098
c) Determine Δl		
=Ml*(L^2)/161*Ilower Bound	Ml	211.68
	L	24
	Loading Constant	161

	Ilb	2270.245098
	ΔL	0.333582829
	L/360	0.8
	1 " Max	1
Deflection okay		

STEP 9: Determine Loads during Unshored Construction

Check Unshored construction Deflection

Dead Loads	psf	Trib area(ft)	lb/ft
Decking	3	42	126
Beam		42	40
Live Loads			
LLWork	20	42	840
Llconcrete	59.4	42	2494.8
Total Unfactored Loads			3500.8

Step 10: Determine Deflection Due to Unshored Construction

	Unfactored Loads	3.5008
	L(ft)	24
	Ix (in^4)	1350
	Max Δ allow	0.80
a) calculate Moment Due to live Load	Ml	252.0576
b) Deflection	DL	0.67
d)Determine if ΔL is Sufficient	Deflection okay	

Step 11: Check if Mu< φMn

Mu (ft-kip)	647.74512
Mn= FyZx (ft-kip)	6700
Mp=φMn= FyZx (ft-kip), *Where* ϕ*=.9*	6030

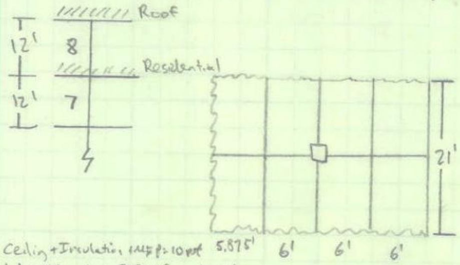
7.8 Sample Short Span Hand Calculations- Columns (Gravity Loads)

By: Mike

Checked by: Jodi-Lee

Michael O'Brien | MQP | Interior Columns (Short Span)

Floors 7-8 Columns are 24' long



Ceiling + Insulation + MEP = 10 psf

LL = 40 psf + 20 psf (partitions) = 60 psf

Concrete + decking = 62.4 lb/f

Beam weight = (4 beams)(21')(19 lb/f) = 1596 lb (per floor)

Girder weight = 23.875(40 lb/f) = 955 lb (per floor)

S = 55 psf = 55(21 × 23.875) = 27575.625 lb (roof only)

Lr: 20 psf = 20(21 × 23.875) = 10027.5 lb (roof LL)

LL = 60(21 × 23.875) = 30082.5 lb (Residential floors only)

Concrete + MEP = 72.4(21 × 23.875) = 36300 lb (per floor)

Try both loading conditions $(S > L_r)$

$P_u = 1.2D + 1.6(S \text{ or } L_r) + .5L$ or $P_u = 1.2D + 1.6L + .5(S \text{ or } L_r)$

LC1 $P_u = 1.2(2(1596+955) + 2(36300)) + 1.6(27575) + .5(30082.5) = 152404 \text{ lb} = 152.4\text{K}$

LC2 $P_u = 1.2(2(1596+955) + 2(36300)) + 1.6(30082.5) + .5(27575.6) = 155162 \text{ lb} = 155.2\text{K}$

$1.2D + 1.6L + .5S$ governs in this case

Since the frame is braced, use $K = 1.0$ (conservative approach)

Because the unbraced length is the same in both the x-x and the y-y direction, and r_x exceeds r_y for all W-shapes, y-y axis buckling will govern.

Assume $\frac{KL}{r} = 50$ $\phi_c F_{cr} = 37.5$ (Available critical stress for compression members; table 4-22 in AISC manual)

$A \text{ required} = \frac{P_u}{\phi_c F_{cr}} = \frac{155.2}{37.5} = 4.14 \text{ in}^2$

Try smallest W section listed in table 4-1, W8X31 $(r_y = 2.02 \text{ in}, A = 9.12 \text{ in}^2)$

$\left(\frac{KL}{r}\right)_y = \frac{1(12)(12'')}{2.02} = 71.28$

Interpolate (from table 4-22)

$\phi_c F_{cr} = 31.1 + .28(30.8 - 31.1) = 31.016 \text{ ksi}$

Michael O'Brien | MQP | Interior Columns (short span)

<u>W8X31</u> (floors 7-8)

Check $\phi_c P_n = \phi_c F_{cr} A = 31.016(9.12) = 283K > 155.2K$ ✓ o.k.

From table 4-1, $\phi_c P_n = 283K = \phi_c F_{cr} A$ (calculated)
Thus, from here on, we will select lightest W-section from table 4-1, instead of using interpolation w/ table 4-22.

Check column selfweight:
$24'(31 lb/f) = 744 lb$

$P_u = 1.2(744 + 2(1596+955) + 2(36300)) + 1.6(30082.5) + .5(27575) = 156.1K$

$156.4K < 283K$ ✓ o.k.

<u>Floors 5-6</u>

$P_u = 1.2(744 + 4(1596+955) + 4(36300)) + 1.6(3(30082.5)) + .5(27575) = 345561 lb = 345.6K$

Try <u>W10X39</u>, from AISC table 4-1, $\phi_c P_n = 351K$ as lightest W section w/ $\phi_c P_n > P_u$.

Add column selfweight
$24'(39) = 936 lb + 744 lb = 1680 lbs$

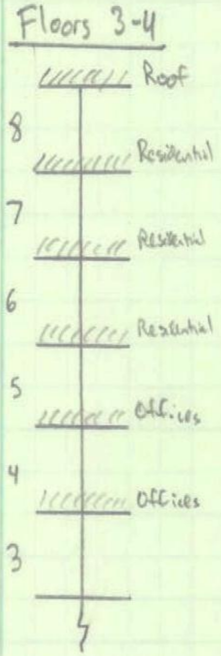
$P_u = 1.2(1680 + 4(1596+955) + 4(36300)) + 1.6(3(30082.5)) + .5(27575) = 346.7K$

$P_u < \phi_c P_n$

$346.7 < 351$ ✓ o.k.

Michael O'Brien | MQP | Interior Columns (Short Span) Pg 3

Floors 3-4



Office LL: 50 psf + Partitions = 20 psf => 70 psf
Industrial (light manufacturing) = 125 psf
Beams = 1596 lb
Girders = 955 lb
Concrete + MEP = 36300 lb
Residential LL: 30082.5 lb (per floor)
Office LL: 70(21×23.875) = 35096 lb (per floor)
Snow: 27575 lb

$P_u = 1.2(1680 + 6(1596+955) + 6(36300)) + 1.6(3(30082.5) + 2(35096)) + .5(27575) = 552232\text{ lb}$

$P_u = 552.2\text{ K}$

With KL = 12, from table 4-1 choose a W section with $\phi_c P_n \geq P_u$

Lightest W section that satisfies inequality is W10X54

$\phi_c P_n = 565\text{K} > 552.2\text{K}$ ✓ o.k.

Add column self weight

$24'(54\text{ lb/ft}) = 1296\text{ lb}$ $\quad 1680 + 1296 = 2976\text{ lbs}$

New $P_u = 1.2(2976 + 6(1596+955) + 6(36300)) + 1.6(3(30082.5) + 2(35096)) + .5(27575) = 553787\text{ lb} = 553.8\text{K}$

$P_u < \phi_c P_n$

$553.8\text{K} < 565\text{K}$ ✓ o.k.

7.9 Sample Short Span Excel Calculations- Columns (Gravity Loads)

By: Jodi-Lee

Checked by: Mike

COLUMN DESIGN SHORT SPAN

WT FROM FLOORS 8 & ROOF

	l	w	
Dimensions	**23.875**	21	
	Beams	**Giders**	**Total**
Wt (lbs/ft(	19	40	
Trib Area	**21**	23.875	
Wt lbs	**399**	955	
Number Beams *wt 1	**1596**	955	**2551**

Step 1: Determine Loading Conditions

Select larger of	*1.2D+1.6 L +.5S*
	1.2D+1.6S+.5L

Dead Loads-Floor	**Psf**	**Tributary Area(Ft^2)**	**lb**
Weight of Concrete (lb)	62.40	501.38	31285.80
Ceiling (lb)	3.00	501.38	1504.13
MEP(lb)	5.00	501.38	2506.88
Insulation(lb)	2.00	501.38	1002.75
Floor beams+ girders(lb)			2551.00
Column Weight			744.00

Dead loads per floor			
8			38.85055
6,7,8			0.00
4,5,6,7,8			0.00
2,3,4,5,6,7,8			0.00
Total Dead Load (kips)			38.85

Live Loads Floor			
Office	50	501.375	**25068.75**
Residential	40	501.375	**20055**
Industrial(Light manufacturing)	125	501.38	**62671.875**
Partitions	20	501.375	**10027.5**
Live Loads per Floor			
8			30.08
6,7,8			0.00
4,5,6,7,8			0.00
2,3,4,5,6,7,8			0.00
Total Live Load Floor			30.08
Roof Dead Load			
Since same dead loads Dead load on roof=Dead load on Floor			38.85
Roof Live Load			

Roof Snow Load			
Snow	55	501.38	27.575625

Factored Load	Larger of:	*1.2D+1.6 L +.5S*	155.16
		1.2D+1.6S+.5L	152.40
	Larger		155.1611325

Step 2: Assign A value for

Fy (ksi)	50.00
f'c(ksi)	4.00

Step 3: Select A KL value

Assume Kl/r=	50
Step 4: Determine Loads	
Loads	
Pu= larger value of factored loads	155

Step 5: Identify Value for ΦcFcr assuming KL/r=50

ΦcFcr	37.5

Step 6: Identify Required Area

Pu/ΦcFcr	4.14

Step 7: Section Properties

Select A member size so that Area > Area required

obtained from Table 1-1 of the AISC Steel Manual 13th Edition

Try a W 8 X 31

A(in2)	9.12		
rx (in)	3.470		
ry (in)	2.020		
Step 8: Determing Capacity			
Using Table 4-22 of AISC Steel determine the capacity of the column through Interpolation			
height per floor(Ft)	12		
KL/rx	41.499		
KL/ry	71.287	Governs	

Use Table 4-22 To determine ΦcFcr

KL/r lower	71.000		Table 4-22
ΦcFcr lower ksi	31.100		
KL/r upper	72.000		
ΦcFcr upper (ksi)	30.800		
ΦcFcr actual (ksi)	31.014	30.71386139	
ΦcPn (kips)	282.847		
Check that ΦcPn>Pu	282.847	> YES	155

Therefore Select Beam size W 8 X 31

Step 9: Add Column weight to check still adequate

Check with new column Weight

Factored Load	Larger of:	*1.2D+1.6 L +.5S*	156.05
		1.2D+1.6S+.5L	153.30
	Larger		156.0539325
Check that ΦcPn>Pu	282.847	>	156
		YES	
Keep Beam size W 8 X 31			

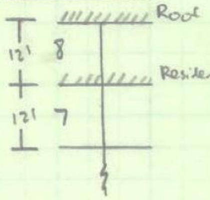
7.10 Sample Long Span Hand Calculations- Columns (Gravity Loads)

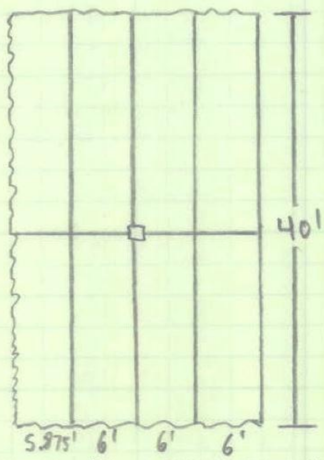
By: Mike

Checked by : Jodi-Lee

Michael O'Brien | MQP | Long Span (Interior Columns) Pg 1

Floors 7-8   Columns are 24' long





Beam weight = (4 beams)(40')(55 lb/ft) = 8.8k

Girder weight = 23.875(55) = 1.313 K

Residential LL = 40 psf + 20 psf (partitions) = 60 psf (23.875' x 40') = 57300 lbs

DL = (62.4 + 10)(23.875 x 40) + (8800 + 1313) = 79255 lb   (per floor)

S = 55 psf (23.875' x 40') = 52525   (roof only)

Calculate required strength

$P_u = 1.2D + 1.6L + .5S$

$P_u = 1.2(2(79255)) + 1.6(57300) + .5(52525) = 308155 \text{ lb} = 308.2\text{K}$ (governs)

or $P_u = 1.2D + 1.6S + .5L$

$P_u = 1.2(2(79255)) + 1.6(52525) + .5(57300) = 302902 \text{ lb} = 302.9\text{K}$

Since the frame is braced, use $K = 1.0$

Because the unbraced length is the same in both the x-x and the y-y direction, and r_x exceeds r_y for all W-shapes, y-y axis buckling will govern.

From table 4-1 in AISC manual, select lightest W section that satisfies $\Phi_c P_n \geq P_u$

Try a W8X35 , $\Phi_c P_n = 320\text{K}$

Check beam self-weight

$(24')(35 \text{ lb/ft}) = 840 \text{ lb}$

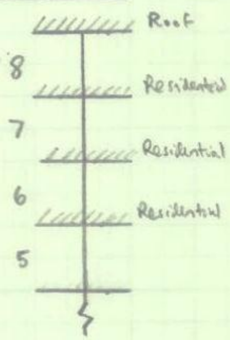
New $P_u = 1.2(840 + 2(79255)) + 1.6(57300) + .5(52525) = 309163 \text{ lb} = 309.2\text{K}$

$\Phi_c P_n > P_u$

$320\text{K} > 309.2\text{K}$ ✓ o.k.

Michael O'Brien | MQP | Long Span (Interior Columns) 1/2

Floors 5-6



Residential LL: 57300 lb (per floor)
DL = 79255 lb (per floor)
S = 52525 lb (roof only)

Calculate required strength

$P_u = 1.2(840 + 4(79255)) + 1.6(3(57300)) + .5(52525) = 682734 lb = 682.7K$

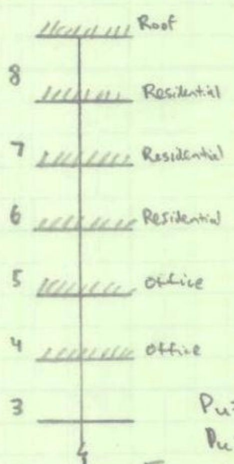
From table 4-1 in AISC manual, select lightest W section that satisfies $\phi_c P_n \geq P_u$

Try a W12x65 $\phi_c P_n = 727K$

Check column self weight
$(24')(65 lb/ft) = 1560 lbs$ $1560 lbs + 840 lbs = 2400 lbs$
$P_u = 1.2(2400 + 4(79255)) + 1.6(3(57300)) + .5(52525) = 684607 lb = 684.6K$

$\phi_c P_n > P_u$
$727K > 684.6K$ ✓ o.k.

Floors 3-4



Office LL = 50 psf + 20 psf (partitions) = 70 psf

Offices LL = $70(23.875' \times 40') = 66850 lbs$ (per floor)

$P_u = 1.2(2400 + 6(79255)) + 1.6(3(57300) + 2(66850)) + .5(52525) = 1088738 lb$
$P_u = 1088.7K$

From table 4-1, select lightest W section that satisfies $\phi_c P_n \geq P_u$

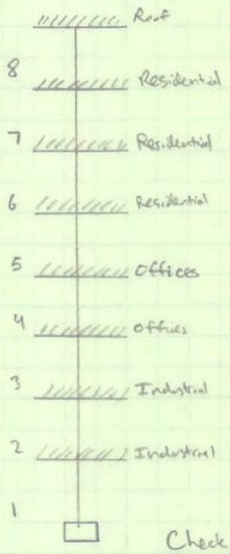
Try a W14x99, $\phi_c P_n = 1170K$

Check column self-weight
$(24')(99 lb/ft) = 2376 lb$ $2376 + 2400 = 4776 lbs$
New $P_u = 1.2(4776 + 6(79255)) + 1.6(3(57300) + 2(66850)) + .5(52525) = 1091590 lbs$
$P_u = 1091.6K$

$\phi_c P_n > P_u$
$1170K > 1091.6K$ ✓ o.k.

Michael O'Brien | MQP | Long Span (Interior Columns)

Floors 1-2



Industrial (light manufacturing)

$LL = 125psf + 20psf = 145psf(23875 \times 40) = 138475lb$ (per floor)

Res $LL = 57300lb$

Offices $LL = 66850lb$

$DL = 79255lb$ (per floor)

$S = 52525$

$P_u = 1.2(4776 + 8(79255)) + 1.6(3(57300) + 2(66850) + 2(138475)) + .5(52525) = 1724921lb$

$P_u = 1724.9K$

$KL = 12'$

From table 4-1 select lightest W section that satisfies $\phi_c P_n \geq P_u$

Try <u>W14X145</u>, $\phi_c P_n = 1740K$

Check column selfweight

$(24')(145lb/ft) = 3480lb$ $3480 + 4776 = 8256lb$

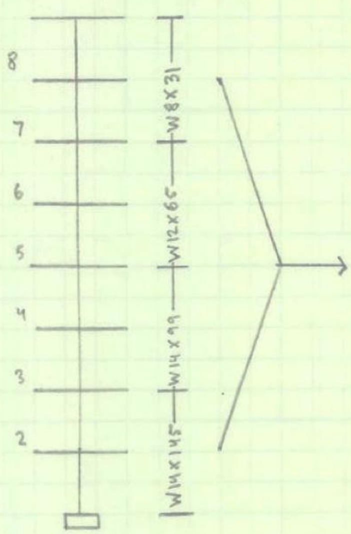
New $P_u = 1.2(8256 + 8(79255)) + 1.6(3(57300) + 2(66850) + 2(138475)) + .5(52525) = 1729098lb$

$P_u = 1729.1K$

$\phi_c P_n \geq P_u$

$1740K \geq 1729.1K$ ✓ o.k.

Summary



Columns increase in size from roof to ground ✓ o.k.

7.11 Sample Long Span Hand Calculations- Columns (Gravity Loads)

By: Jodi-Lee

Checked by: Mike

COLUMN DESIGN LONG SPAN

WT FROM FLOORS 8 & ROOF

	l	w	
Dimensions	**23.875**	40	
	Beams	Giders	**Total**
Wt (lbs/ft(	55	55	
Trib Area	**40**	23.875	
Wt lbs	**2200**	1313.125	
Number Beams *wt 1	**8800**	1313.125	**10113.125**

Step 1: Determine Loading Conditions

Select larger of	*1.2D+1.6 L +.5S*
	1.2D+1.6S+.5L

Dead Loads-Floor	**Psf**	**Tributary Area(Ft^2)**	**lb**
Weight of Concrete (lb)	62.40	955.00	59592.00
Ceiling (lb)	3.00	955.00	2865.00
MEP(lb)	5.00	955.00	4775.00
Insulation(lb)	2.00	955.00	1910.00
Floor beams+ girders(lb)			10113.13
Column Weight			840.00

Dead loads per floor			
8			79.255125
6,7,8			0.00
4,5,6,7,8			0.00
2,3,4,5,6,7,8			0.00
Total Dead Load Floor (kips)			79.26

Live Loads Floor			
Office	50	955	**47750**
Residential	40	955	**38200**
Industrial(Light manufacturing)	125	955.00	**119375**
Partitions	20	955	**19100**
Live Loads per Floor			
8			57.30
6,7,8			0.00
4,5,6,7,8			0.00
2,3,4,5,6,7,8			0.00
Total Live Load Floor			57.30
Roof Dead Load			
Since same dead loads Dead load on roof=Dead load on Floor			79.26
Roof Live Load			

Roof Snow Load			
Snow	55	955.00	52.525

Factored Load	Larger of:	*1.2D+1.6 L +.5S*	308.15
		1.2D+1.6S+.5L	302.90
	Larger		308.1548

Step 2: Assign A value for

Fy (ksi)	50.00
f'c(ksi)	4.00

Step 3: Select A KL value

Assume Kl/r=	50
Step 4: Determine Loads	
Loads	
Pu= larger value of factored loads	308

Step 5: Identify Value for ΦcFcr assuming KL/r=50

ΦcFcr	37.5

Step 6: Identify Required Area

Pu/ΦcFcr	8.22

Step 7: Section Properties

Select A member size so that Area > Area required

obtained from Table 1-1 of the AISC Steel Manual 13th Edition

Try a W 8 X 35

A(in2)	10.30		
rx (in)	3.510		
ry (in)	2.030		
Step 8: Determine Capacity			
Using Table 4-22 of AISC Steel determine the capacity of the column through Interpolation			
height per floor(Ft)	12		
KL/rx	41.026		
KL/ry	70.936	Governs	

Use Table 4-22 To determine ΦcFcr

KL/r lower	70.000		Table 4-22
ΦcFcr lower ksi	31.400		
KL/r upper	71.000		
ΦcFcr upper (ksi)	31.100		
ΦcFcr actual (ksi)	31.119		
ΦcPn (kips)	320.528		
Check that ΦcPn>Pu	320.528	>	308
		YES	

Therefore Select Beam size W 8 X 35

Step 9: Add Column weight to check still adequate

Check with new column Weight

Factored Load	Larger of:	*1.2D+1.6 L +.5S*	309.16
		1.2D+1.6S+.5L	303.91
	Larger		309.1628
Check that ΦcPn>Pu	320.528	>	309
		YES	
Keep Beam size W 8 X 35			

7.12 Analysis for Lateral Loads

This section presents the results from *MASTAN2* and sample calculations that were used to determine the column sizes while accounting for combined bending and axial forces.

7.12.1 Wind Load Results from *MASTAN2*

The MQP team took into account the lateral loads in terms of wind loads. For this, there were two lateral load conditions that went into the *MASTAN2* program. The first input was the loads for the long span, with a tributary area of 12'x40 feet. The second input was for the short span, with a tributary area of 12'x19.25'. Both windward (positive) and leeward (negative) pressures were addressed. The pressures were all determined using Cornell University's *Seismic and Wind Force Calculator* (Ochshorn, 2009).

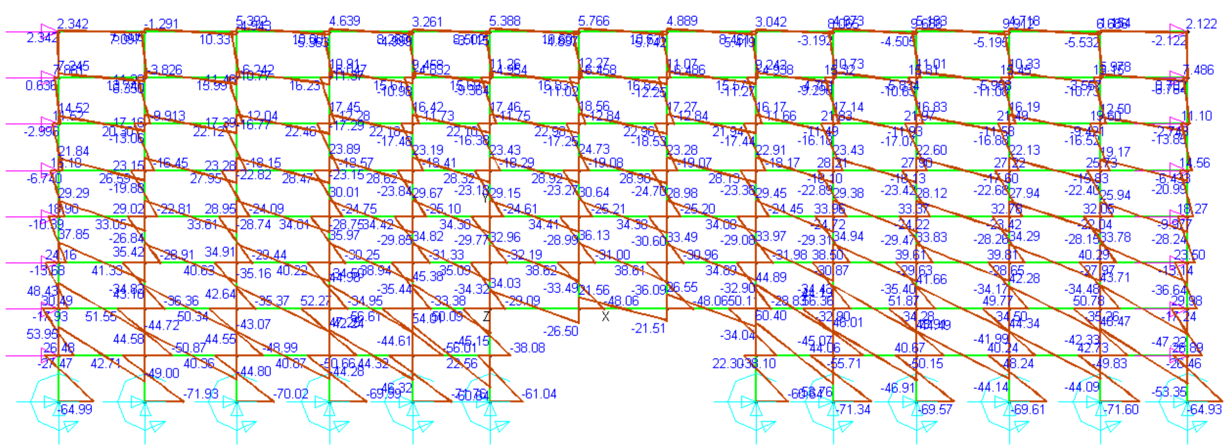


Figure 60: Moments on Entire Frame due to Wind-Load

Table 30: Wind-Load Pressure and Forces

Story	Top of story Height (ft)	windward pressure (psf)	windward long span force(k)	windward short span force(k)	leeward pressure (psf)	long span leeward force (k)	short span leeward force (k)
8	96	18.75	9.0	4.3	-11.72	-5.6	-2.7
7	84	18.04	8.7	4.2	-11.72	-5.6	-2.7

6	72	17.27	8.3	4.0	-11.72	-5.6	-2.7
5	60	16.39	7.9	3.8	-11.72	-5.6	-2.7
4	48	15.38	7.4	3.6	-11.72	-5.6	-2.7
3	36	14.16	6.8	3.3	-11.72	-5.6	-2.7
2	24	12.62	6.1	2.9	-11.72	-5.6	-2.7
1	12	11.03	5.3	2.5	-11.72	-5.6	-2.7

Table 31 and Table 32 summarize the loading values that were used in the *MASTAN2* models. From there, values for moments and axial forces on any member of the frame could be determined in *MASTAN2*.

Table 31: Wind-Load Pressure and Forces acting on Transverse Side of Building

Story	Top of story Height (ft)	Windward pressure (psf)	Windward long span force(k)	Windward short span force(k)	Leeward pressure (psf)	Long span leeward force (k)	Short span leeward force (k)
8	96	18.75	9.0	4.3	-11.72	-5.6	-2.7
7	84	18.04	8.7	4.2	-11.72	-5.6	-2.7
6	72	17.27	8.3	4.0	-11.72	-5.6	-2.7
5	60	16.39	7.9	3.8	-11.72	-5.6	-2.7
4	48	15.38	7.4	3.6	-11.72	-5.6	-2.7
3	36	14.16	6.8	3.3	-11.72	-5.6	-2.7
2	24	12.62	6.1	2.9	-11.72	-5.6	-2.7
1	12	11.03	5.3	2.5	-11.72	-5.6	-2.7

Table 32: Wind-Load Pressures and Forces acting on Longitudinal Side of Building

Story	Top of story Height (ft)	Windward pressure (psf)	Windward span force(k)	Leeward pressure (psf)	Leeward force (k)
8	96	18.75	5.4	-11.72	-3.4
7	84	18.04	5.2	-11.72	-3.4
6	72	17.27	5.0	-11.72	-3.4
5	60	16.39	4.7	-11.72	-3.4
4	48	15.38	4.4	-11.72	-3.4
3	36	14.16	4.1	-11.72	-3.4
2	24	12.62	3.6	-11.72	-3.4
1	12	11.03	3.2	-11.72	-3.4

7.13 Seismic Loads

Table 33 and Table 34 summarize the values that were used in *MASTAN2* for analysis.

Table 33: Seismic Forces acting on the transverse side of building

Story	Top of story Height (ft)	Seismic Force For Entire Story (k)	Seismic Force for Long Span Bay (k)	Seismic Force for Short Span Bay (k)
8	96	56.1	20.0	9.6
7	84	47.2	16.9	8.1
6	72	38.7	13.8	6.7
5	60	30.6	10.9	5.3
4	48	22.9	8.2	3.9
3	36	15.8	5.6	2.7
2	24	7.2	2.6	1.2
1	12	2.9	1.0	0.5

Table 34: Seismic Forces acting on the longitudinal side of building

Story	Top of story Height (ft)	Seismic Force For Entire Story (k)	Seismic Force for Bay (k)
8	96	56.1	4.6
7	84	47.2	3.9
6	72	38.7	3.2
5	60	30.6	2.5
4	48	22.9	1.9
3	36	15.8	1.3
2	24	7.2	0.6
1	12	2.9	0.2

7.14 Sample Short Span Hand Calculations- Columns (Lateral Loads)

MQP ✓ 1/20/12

WPI
Department of Civil & Environmental Engineering
CE3006 Design of Steel Structures
Worksheet for Amplified First-Order Elastic Analysis (Story Stiffness Method)

Identify load combination equation for investigation:

Calculation of P_r and M_r for input to interaction equation
Refer to preliminary column and girder sizes, member forces, and lateral deflection values obtained from structural analyses.

Column load effects from analysis	
Factored axial force P_{nt} from no-sway analysis (gravity loads)	1.2D+1.6L+.5S= 650.1 K
Factored axial force P_{lt} from sway analysis (lateral loads)	1.2D+1.6L+.5S= use Y_i nominal load Lateral Force = .002Y_i Y_i=1.2D+1.6L .002Y_i = .002(4078.12) = 8.16K
Factored moment M_{nt} from no-sway analysis (gravity loads)	1.2D+1.6W+.5L+.5S = 146.85 ft·K
Factored moment M_{lt} from sway analysis (lateral loads)	1.2D+1.6W+.5L+.5S = 125.028 ft·K
Lateral deflection (story drift) from analysis	
Total story shear ΣH (lateral loads input to deflection analysis for the story)	7.0 K (12ft)
Lateral deflection (drift) for story Δ_H (obtained from deflection analysis and loading ΣH)	.00035 in (12 ft)
Amplifier B_2	
Total elastic critical buckling load for the story $$\Sigma P_{e2} = R_M \frac{\Sigma HL}{\Delta_H}$$ where R_M = 0.85 for moment frames and L = story height.	$\Sigma P_{e2} = .85 \frac{(7.0)(12')(12''/1ft)}{(.00035 in)} = 2,448,000$

1/20/12

Total factored gravity load for the story $\sum P_{nt}$ Note: Sum the factored gravity loads for all columns in the story.	$1.2(1498.3)+1.6(1432.6)+.5(126.4) = 4141K$
$$B_2 = \frac{1}{1-\frac{\alpha \Sigma P_{nt}}{\Sigma P_{e2}}} \geq 1$$ where $\alpha = 1.0$ for LRFD.	$B_2 = \frac{1}{1-\frac{1.0(4141)}{2448000}} = 1.0017$
Amplifier B_1	
M_1 = smaller factored column end moment due to gravity load (no sway) analysis	136.816 ftk
M_2 = larger factored column end moment due to gravity load (no sway) analysis	146.85 ftk
Indicate: single or reverse curvature	Reverse
$C_m = 0.6 \pm 0.4(M_1/M_2)$ Use + for single curvature (*hurt*). Use – for reverse curvature (*help*).	$C_m = .6 - .4\left(\frac{136.82}{146.85}\right) = .227$
$P_r = P_{nt} + B_2 P_{lt}$ where P_{nt}, P_{lt}, and B_2 are defined above.	$P_r = 650.1 + 1.0017(8.16) = 658.27K$
Elastic critical buckling load for column $P_{e1} = \pi^2 EI/(K_1L)^2$ where $K_1 = 1.0$ Note: This load capacity refers to the no sway case (gravity loading).	$P_{e_1} = \frac{\pi^2(29000)(662)}{(1.0(12))^2(144)} = 9137.57$
$$B_1 = \frac{C_m}{1-\alpha P_r/P_{e1}} \geq 1$$ where $\alpha = 1.0$ for LRFD.	$B_1 = \frac{.227}{1-1\left(\frac{658.27}{9137.57}\right)} = .245$ thus use 1.0

Required second-order strength values	
$P_r = P_{nt} + B_2 P_{lt}$ where P_{nt}, P_{lt}, and B_2 are defined above.	658.27 K
$M_r = B_1 M_{nt} + B_2 M_{lt}$ where M_{nt}, M_{lt}, B_1, and B_2 are defined above.	1.0(146.85) + 1.0017(125.028) = 272.1 ftk

Interaction Equations
Calculated values for Pr and Mr are to substituted into appropriate AISC interaction equation H1-1a or H1-1b for combined flexural and axial forces.

Michael O'Brien	MQP	W12X79 Columns

Use
H1-1a: For $\frac{P_r}{P_c} \geq .2$ or H1-1b: For $\frac{P_r}{P_c} < .2$

For W12X79 $\frac{P_r}{P_c} = \frac{658.27}{887k} = .74$
From table 4-1 (KL=12.0ft)

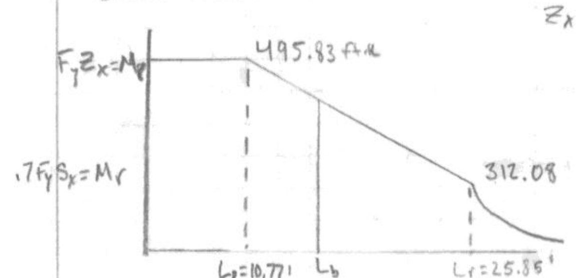
Thus use H1-1a
Next, establish M_{cx}, think of column like a beam

Check WLB = $\frac{h}{t_w} = 20.7 < 3.76\sqrt{\frac{E}{F_y}} = 90.5$ o.k. (avoids WLB)

Check FLB: $\frac{b_f}{2t_f} = 8.22 < .38\sqrt{\frac{E}{F_y}} = 9.2$ o.k. (avoids FLB)

Check LTB:

$Z_x = 119$ $L_p = 1.76 r_y \sqrt{\frac{E}{F_y}} = 1.76(3.05)\sqrt{\frac{29000}{50}} =$



$.7F_y S_x = .7(50)(107)$

$L_r = \pi r_{ts}\sqrt{\frac{E}{0.7F_y}} = \pi(3.43)\sqrt{\frac{29000}{.7(50)}} = 310 \text{ in} = 25.85 \text{ ft}$

$L_p < L_b < L_r$ so interpolation is required

$M_{nx} = M_p - \frac{(L_b - L_p)}{(L_r - L_p)}(M_p - M_r) = 495.83 - \left(\frac{12 - 10.77}{25.85 - 10.77}\right)(495.83 - 312.08) = 480.8 \text{ ft-k}$

$M_{cx} = .9M_{nx} = .9(480.8) = 432.8 \text{ ft-k}$

H1-1a: $\frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_{rx}}{M_{cx}}\right) = .75 + \frac{8}{9}\left(\frac{312.08}{432.8}\right) = 1.39 \geq 1.0$ Not o.k.

Try larger column size based on S_x or Z_x
(Try W12X87 $Z_x = 132$, $S_x = 118$)

Try a larger section
Govern by bending
Ch 3-2 look @ Z_x or S_x to try next size.
Ch 5 "columns"

7.15 Sample Excel Calculations- Columns (Lateral Loads)

Geometric Properties From Table 1-1 AISC Manual

Fy(ksi)	50	Ix	96
A (in^2)	15.6	Zy(in^3)	29.1
Sx (in^3)	70.6	Iy(in^4)	95.8
Zx(in^3)	77.9	Lp(ft)	
Sy(in^3)	19.2	Lr(ft)	
rts (in)	2.79	ry(in)	2.48
rx(in)	5.23		
h/t_w	28.1		
$b_f/2t_f$	8.69		
.7*Fy	35		

Loads	Axial	Moment
Dead	60.35	83.86
Live	21.35	50.05
Snow	22.3	21.58
Wind	1.112	22.63
Earthquake	0.557	11.11

STEP 3: Determine Factored Loads and Select Governing Load Combination

Factored Loads	**Axial**	**Moment**
Case 1- 1.4 D	84.49	117.404
Case 2- 1.2D+1.6L+.5(Lr or S)	106.58	180.712
	117.73	191.502
Case 3- 1.2D+1.6(Lr or S)+(.5L or .8W)	118.775	125.657
Goverening for Case 3	108.9896	153.264
Case 4- 1.2D+1.6W+.5L+.5(Lr or S)	96.0242	172.655
Case 5- 1.2D+1.0E+.5L+.2S	106.707	170.067
Governing Loads	118.775	191.502

Factored Loads	**Pu(k)**	**Mnt(k-ft)**	**Mlt(ft-k)**	**Plt(k)**
Case 1- 1.4 D	84.532	117.404	0	0
Case 2- 1.2D+1.6L+.5(Lr or S)	106.984	180.712	0	0
	118.139	191.502	0	0
Case 3- 1.2D+1.6(Lr or S)+(.5L or .8W)	83.246	125.657	0	0
	72.456	100.632	19.296	0.996

Goverening for Case 3	118.942	160.185	0	0
Case 4- 1.2D+1.6W+.5L+.5(Lr or S)	106.056	136.447	39.092	2.492
Case 5- 1.2D+1.0+.5L+.2S	87.708	129.973	10.64	0.539
Governing Loads	118.942	191.502	0	1.034034
				use min lateral load =.002Yi

Yi= total factored gravity loads at ith level Sum of L and Dead

Yi (k)	517.017
Min lateral Load =.002Yi (kip)	1.034034

STEP 4: Column Load Effects from Analysis

No Sway Analysis

Axial Load (kip)	118.94
Moment Force (kip-ft)	
Factored 1.2D +1.6L+.5S	191.50

Sway Analysis

Factored axial Force Pnt from no sway analysis (gravity Laods) (kips)	118.94
Factored axial Force Plt from sway analysis (Lateral Loads) (kips)	1.034
Factored Moment Mnt from no-sway anaysis(gravity Loads) (kip-ft)	191.50
Factored Moment Mlt from sway anaysis(lateral Loads) (kip-ft)	0.00

STEP 5: Lateral Deflection (Story Drift) from Analysis

LONG SPAN Lateral Deflection Drift For Story Delta H (obtained)

Floor	**Total (in)**
ΣHTotal Story shear (lateral loads input to deflection analysis for the story) (kip)	8.8
ΔH Lateral deflection (drift) for story (obtained from deflection analysis and loading ΣH)	0.00031
Story Height (Ft)	12

STEP 6: Amplifier B2

Total elastic critical buckling load for the story

ΣPe2 = RmHL/ΔH; where Rm=.85 for moment frames and L= story height	3474580.645			
Total factored gravity load for the story ΣPnt Note: Sum the factored gravity loads for all columns in the story				
	DL	LL	S	Total Factored
Unfactored	372.06	141.09	129.37	
Total Gravity Load Factored 1.2D +1.6S+.5L(kip)	446.472	70.545	206.992	724.009
ΣPnt	724.009			
B2= 1/(1-αΣPnt/ΣPe2) ≥ 1	1.000208416			
Check if B2 ≥ 1	1.000208416			

STEP 7: Amplifier B1

Step 4(a) **M1**= smaller factored column end moment due to gravity load (no sway) analysis	DL	LL	S	
Unfactored	77.79	46.53	17.07	
Factored	93.348	74.448	8.535	176.331
Step 4(b)**M2**= larger factored column end moment due to gravity load (no sway) analysis	DL	LL	S	
Unfactored	84.39	50.45	17.57	
Factored	101.268	80.72	8.785	190.773
Indicate: Single/ reverse curvature	Double-reverse			
Cm= 0.6 ± .4(M1/M2) use + for single curvature(hurt) and - for reverse curvature (help)				
Cm (Single Curvature)	[redacted]			
Cm (Reverse Curvature)	0.230281015			
Pr=Pnt + B2Plt; where Pnt, Plt, and B2 are defined above	119.9762495			
Pe1= Elastic critical buckling load from column Pe1=πEI/(KL)^2; where K1=1.0 *Note refers to no sway case	1320.984796			
B1=Cm/(1-αPr/Pe1) ≥1; where α=1.0 for LRFD	0.253285224			
B1 Value to Use	1			

Required Second Order-Strength Values

Pr=Pnt+B2Plt	119.9762495
Mr=B1+Mnt + B2Mlt	191.502

Step 8: Calculate the nomial Gross Tensile Strength

Pn= FyAg

Step 9: Yielding

Mnx=Mp=FyZx (kip-ft)	324.583

Step 10: Lateral Torsional Buckling

Lb (ft)	12
Lp(ft)	8.76
Lr(ft)	21.01

Lateral Buckling applies Since Lp ≤Lb≤Lr

Step 11: Determine if Yielding or Lateral Buckling Applies

Cb *According to AISC Manual =1.0 for LRFD*	1
Mp= FyZx(kip-ft)	324.583
Mr= .7*Sx*Fy (kip-ft)	205.9166667
Lb-Lp (ft)	3.24
Lr-Lp (Ft)	12.25
Mnx (kip-ft)	293.2075463
Step 12: Check if Mn is less than Mp	yes Mn< Mp
Use Lower value	293.208

Step 13: Determine Mcx

L	12
Kr*L/ry	58.06451613
KxL/rx	27.5334608
Larger Value Governs so se Table 4-1 in AISC Manual	
KL=	12
φcPn *From Table 4-1 in ASIC manual*	547
Mcx=φb*Mnx	263.89

Step 14:Select an Interaction Equation

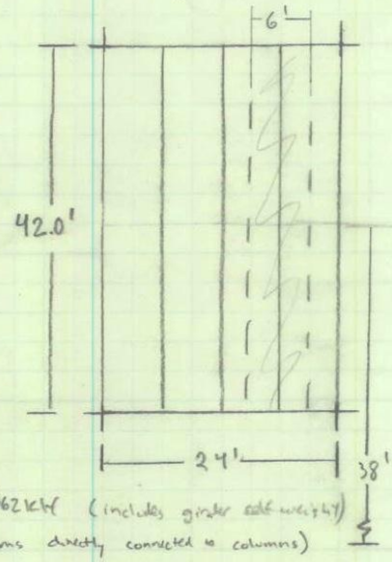
Pr *Required axial Capacity from interaction equation worksheet*	119.9762495
Pc *Required Axial Capacity*	547
Pr/Pc	0.219335008
If Pr/Pc ≥.2 use H1-1a from AISC Manual	0.86440031
If Pr/Pc <.2 use H1-1b from AISC Manual	0.835365061
Use interaction Equation H1-1b OKAY PASSES USE W 12 X 53	

7.16 Sample Hand Calculations- Connections

Michael O'Brien | MQP | Beam – Girder Connections | 1

Typical connection, conservative estimate, based on long-span design.
Will look @ office based LL

W24x55 Girders
W24x55 Beams
Bolts ¾ in ϕ Type A325-N



Beams: (trib area = 6ft)
Unfactored loads
LL= (80psf + 20psf)(6ft) = 420 lb/lf = .420 K/f
DL= (62.4+10)psf(6ft) = 724 lb/lf = .724 K/f
Beam self weight = 55 lb/lf = .055 K/f
Total DL= .779 K/f
1.2D+1.6L= 1.2(.779)+1.6(.420) = 1.6068 K/f

Girders (trib area= 38ft)
Unfactored loads
LL= 70(38) = 2660 lb/lf = 2.66 K/f
DL= 72.4(38) = 2751.2 = 2751.2+55 = 2806.2 = 2.8062 K/f (includes girder self weight)

$\frac{(2\text{ beams})(55\text{ lb/f})(38\text{ft})}{24\text{ft}}$ = 261 lb/f (other 2 beams directly connected to columns)
= .261 K/f

Total DL= 2.8062 + .261 = 3.067 K/f
1.2D+1.6L= 1.2(3.067)+1.6(2.66) = 7.9364 K/f

1.) Verify that beam + girder sizes have adequate shear resistance for the given design loads

Beams W24x55

$\frac{h}{t_w} = 54.6 > 2.24\sqrt{\frac{E}{F_y}} = 53.9$ $\qquad V_u = \frac{1.6068(42)}{2} = 33.7K$

(A W24x55 is one of 8 W-sections that does not meet the requirements of section G2.1(a) on page 16.1-65 of the AISC steel manual.

$\frac{h}{t_w} \leq 1.10\sqrt{\frac{K_v E}{F_y}} = 1.10\sqrt{\frac{5(29000)}{50}} = 59.24$ Thus use $\phi = .9$

$\phi V_n = \phi(.6F_y)A_w$ where $A_w = d \cdot t_w$, $A_w = 23.6(.395) = 9.322\ in^2$

$\phi V_n = .9(.6(50))(9.322) = 251.7K > 33.7K$ ✓ o.k.

Girders

$V_u = \frac{7.9364(24)}{2} = 95.2368K$ $\qquad \phi V_n > V_u$

251.7K > 95.24K ✓ o.k.

Michael O'Brien | MQP | Beam- Girder Connections 2

2.) Bolts $3/4''$ A325-N from AISC table J3.2, $F_{nv}=60$ KSI
(threads included from shear planes) ⟶ $F_{nv}=48$ KSI for threads not exc.

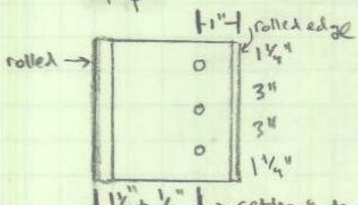
$A_b=\frac{\pi d^2}{4}=\frac{\pi}{4}\left(\frac{3}{4}\right)^2=.4418\text{ in}^2$

$\phi R_n=1(.75)(48)(.4418)=15.9\text{ k/bolt}$

1 shear plane

$n=\frac{V_u}{\phi R_n}$ $\quad \frac{33.7}{15.9}=2.12\text{ bolts}\ :\ 3\text{ bolts}$

Using table J3.4,
$3/4''\phi$ $\quad 1\frac{1}{4}''$ C to edge for sheared edges $\quad 1''$ C to edge for rolled edges



3d spacing preferred for bolts, however, 3" spacing common b/c 1" max bolt (3d) so fabricators typically use this.

setback to accomodate / to get to fit properly. Need 3" angle for $1\frac{3}{4}''$ workable gauge. (p1-46)

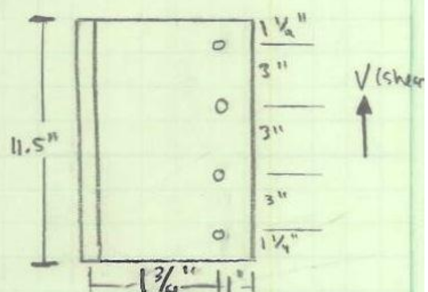
$\frac{T}{2}\leq L\leq T$ $\quad$ T= flat part of web's length, T= 20.75"

$\frac{20.75}{2}\leq 8.5''\leq 20.75''$ $\quad 8.75\not\geq 10.375$, ✗ Not o.k.

To keep current spacing, add 3" so total is 11.5" w/ 4 bolts
$10.375''\leq 11.5''\leq 20.75''$ ✓ o.k.

Now, we will look @ strength of angle; angle thickness t depends on this. A system of 3 equations to solve for t, w/ the largest value governing.

1.) Tearing / Bearing of bolts (AISC J3.10)
(tearing) $1.2L_c t F_y\leq 2.4 d_b t F_u$ (bearing upper bound)
L_c= clear distance in direction of force
$L_c=1.25-\frac{1}{2}\left(\frac{3}{4}''+\frac{1}{8}''\right)=.81''$ (for bottom bolt)
$L_c=3-2\left(\frac{1}{2}\right)\left(\frac{3}{4}''+\frac{1}{8}''\right)=2.13''$ (top 3 bolts)
2 holes $\quad$ 1/2 hole size



Bearing: $\phi R_n=.75(2.4)(3/4)tF_u=1.35tF_u$

Tearing, Bottom Bolt
$\phi R_n=.75(1.2)(.81)(tF_u)=.792tF_u<1.35tF_u$

Top 3 bolts:
$\phi R_n=.75(1.2)(2.13)(tF_u)=1.917tF_u>1.35tF_u$

bearing governs here

Thus, for the bottom bolt, tearing governs, for the other three bolts bearing governs.

Thus, $\phi R_n=.792tF_u+3(1.35tF_u)\geq 33.7\text{K}$

$t\geq\frac{33.7}{(.792+3(1.35))(58)}$ $\quad F_u=58$ KSI

$t\geq .120\text{ in}$ ($1/8''$ closest)

2.) Angle Shear Rupture

$\phi R_n = \phi .6 F_u (L - n d_e) t \geq 33.7k$

$\phi R_n = .75(.6) F_u (11.5 - 4(3/4'' + 1/8'')) t \geq 33.7\,k$

$3.6 F_u t \geq 33.7k$

$t \geq .16$ in 3/16" closest (governs) ⟹

3.) Shear yield on gross area

$\phi R_n = \phi(.6 F_u) L t \geq 33.7k$

$\phi R_n = 1.0(.6 F_u) 11.5 t \geq 33.7k$

$t \geq .08$ 1/8" angle closest

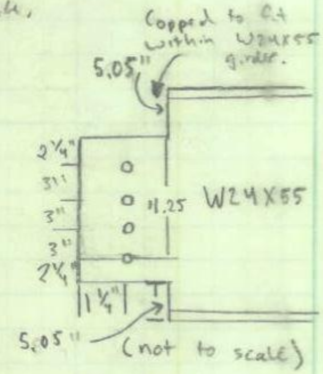
Due to a 2" workable gauge, we will choose a $\angle 3 \times 3 \times 3/16 \times 11.5$ angle

Next, check bearing on beam web

$t_w = .395''$ $F_u = 65$ KSI

$\phi R_n = n(\phi) 2.4 d_b t F_u \geq 37.7k$

$\phi R_n = 4(.75)(2.4)(3/4)(.395)(65) = 138.6k > 37.7k$ ✓ o.k.



4.) Check block shear rupture on web of beam

$F_u A_{nt} + .6 F_y A_{nv} \leq F_u A_{nt} + .6 F_y A_{gv}$

$A_{nt} = ((1.25 - \frac{1}{2}(3/4'' + 1/8''))(.395) = .32 \text{ in}^2$ (net shear area)

$A_{gv} = (11.25)(.395) = 4.44 \text{ in}^2$

$A_{nv} = (11.25 - 3.5(3/4'' + 1/8''))(.395) = 3.23 \text{ in}^2$

$F_u A_{nt} + .6 F_u A_{nv} = 65(.32) + .6(65)(3.23) = 146.77K$ (governs)

$F_u A_{nt} + .6 F_y A_{gv} = 65(.32) + .6(50)(4.44) = 154k$

$\phi R_n = .75(146.77) = 110K > 33.7K$ ✓ o.k.

Use same design for connection of angle to girder web
All limit states will be met since girder is same size.
→ We would also need to investigate shear yield across gross area of the tab and shear rupture across net area of the tab since both the top + bottom of the beam are coped.

Shear yield on gross area of tab Tab is 13.5" long.

$\phi R_n = \phi(.6 F_u) L t \geq 33.7K$

$\phi R_n = 1.0(.6(65))(13.5)(.395) \geq 33.7$

$207.97k \geq 33.7k$ ✓ o.k.

Shear rupture across net area of the tab

$\phi R_n = \phi(.6 F_u)(L - n d_e) t \geq 33.7K$

$\phi R_n = .75(.6)(65)(13.5 - 4(3/4'' + 1/8''))(.395) \geq 33.7k$

$115.5\,k \geq 33.7\,k$ ✓ o.k.

As a check, table 10-10 in AISC manual gives the following eq.

$R_n = C \times r_n$ $\phi R_n = .75(1.99)(120)$

Michael O'Brien | MQP | Girder - Column Connection 4

As previously determined, the design $V_u = 95.24\ k$ for Girder to Column connections. We already know that the girder has adequate shear resistance for its own loads, and for columns, this would transfer to an axial load.

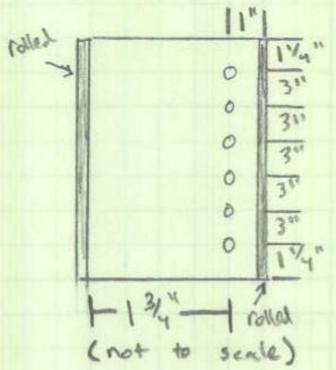
Bolts, 3/4" A-325-N $F_{nv} = 48\ ksi$, (threads not excluded)

$A_b = .4418\ in^2$

$\phi R_n = 15.9\ k/\text{bolt}$

$$n = \frac{V_u}{\phi R_n} = \frac{95.24}{15.9} = 5.98\ \text{bolts} = 6\ \text{bolts}$$

Using table J3.4, 3/4"ϕ => 1 1/4" c to edge for sheared edges, 1" c to edge for rolled edges



Check clearance for a W24X55

$T = 20.75"$ $L = 17.5"$

$\frac{T}{2} \leq L \leq T$

$10.375" \leq 17.5" \leq 20.75"$ ✓ o.k.

Now, look at strength of angle, angle thickness t depends on this. A system of two equations to solve for t, w/ the largest value governing.

1.) Tearing / Bearing of bolts (AISC J3.10)

(tearing) $1.2 L_c t F_y \leq 2.4 d_b t F_u$

L_c = clear distance in direction of force

$L_c = 1.25 - \frac{1}{2}(\frac{3}{4}" + \frac{1}{8}") = .81"$ (for bottom bolt)

$L_c = 3 - 2(\frac{1}{2})(\frac{3}{4} + \frac{1}{8}) = 2.13"$ (top 5 bolts)

Bearing:

$\phi R_n = .75(2.4)(\frac{3}{4}) t F_u = 1.35 t F_u$

Tearing, Bottom Bolt:

$\phi R_n = .75(1.2)(.81)(t F_u) = .792 t F_u < 1.35 t F_u$

Tearing, Top 5 Bolts:

$\phi R_n = .75(1.2)(2.13)(t F_u) = 1.917 t F_u > 1.35 t F_u$ (thus bearing governs here)

Bottom bolt, tearing governs, other bolts, bearing governs.

Thus $\phi R_n = .792 t F_u + 5(1.35) t F_u \geq 95.24k$

$F_u = 58\ ksi$

$$t \geq \frac{95.24}{(.792 + 5(1.35))(58)}$$

$t \geq .218$ in (1/4" closest)

2.) Angle shear rupture

$\phi R_n = \phi .6 F_u (L - n d_e) t \geq 95.24k$

$\phi R_n = .75(.6)(F_u)(17.5 - 6(\frac{3}{4} + \frac{1}{8})) t \geq 95.24$

$t \geq .298"$ (5/16" closest) governs

3.) Shear yield on gross area W24X55 Girders, $V_u = 95.24k$

$\phi R_n = \phi(.6F_y)Lt \geq 95.24k$

$\phi R_n = 1.0(.6)(59)(17.5)t \geq 95.24k$

$t \geq .16''$ (3/16'' closest)

Angle shear rupture governs, use **L3X3X 5/16 X17.5** angle

Next, check bearing on girder web

$t_w = .395''$, $F_u = 65$ ksi

$\phi R_n = n\phi 2.4 d_b t F_u \geq 95.24k$

$\phi R_n = 6(.75)(2.4)(3/4)(.395)(58) \geq 95.24$

$185.6k \geq 95.24k$ ✓ o.k.

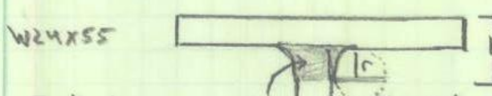
4.) Check block shear rupture on web of beam (although not coped, so typically don't need to do this).

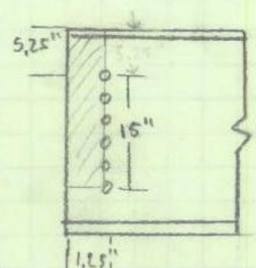
$F_u A_{nt} + .6F_y A_{nv} \leq F_u A_{nt} + .6F_y A_{gv}$

$t_w = .395''$, $F_u = 65$ ksi

A_{nt} = Net Area Subject to Tension $= (1.25 - \frac{1}{2}(3/4 + 1/8))(.395) = .32''$

A_{gv} : Gross Area Subject to Shear $= (20.25 - k)(.395) + m + t_f(b_f)$





$r = k - t_f$

$r = 1.01 - .505 = .505$ in

$m = rt_w + 2(r^2 - \frac{\pi r^2}{4})$

$m = .505(.395) + 2(.505^2 - \frac{\pi(.505)^2}{4})$

$m = .199475 + .109458$

$m = .3089$ (shaded area of web)

$A_{gv} = (20.25 - 1.1)(.395) + .309 + .505(7.01) = 11.41 in^2$

A_{nv}: Net Area Subject to Shear

$A_{nv} = ((20.25 - 1.1 - 5.5(3/4 + 1/8))(.395) + .309 + .505(7.01) = 9.51 in^2$

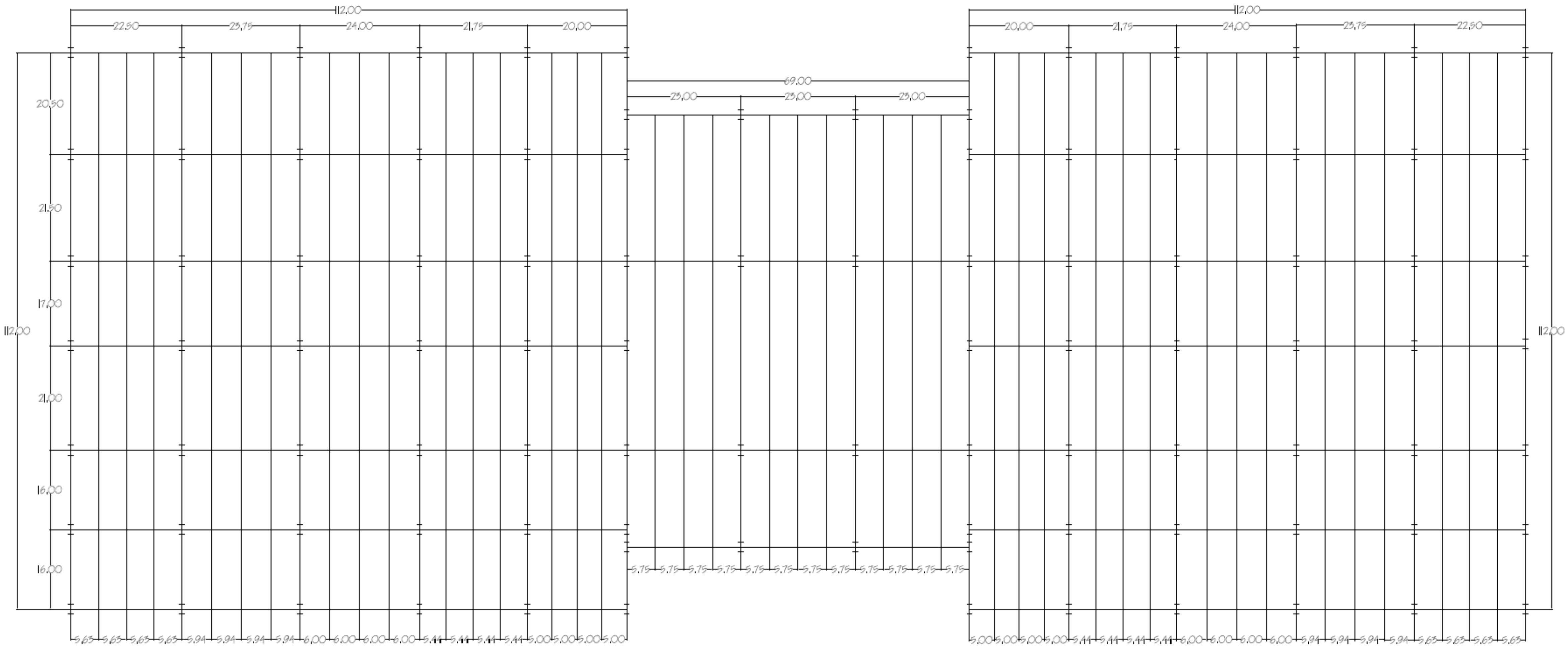
$F_u A_{nt} + .6F_u A_{nv} = 65(.32) + .6(65)(9.51) = 391.7K$

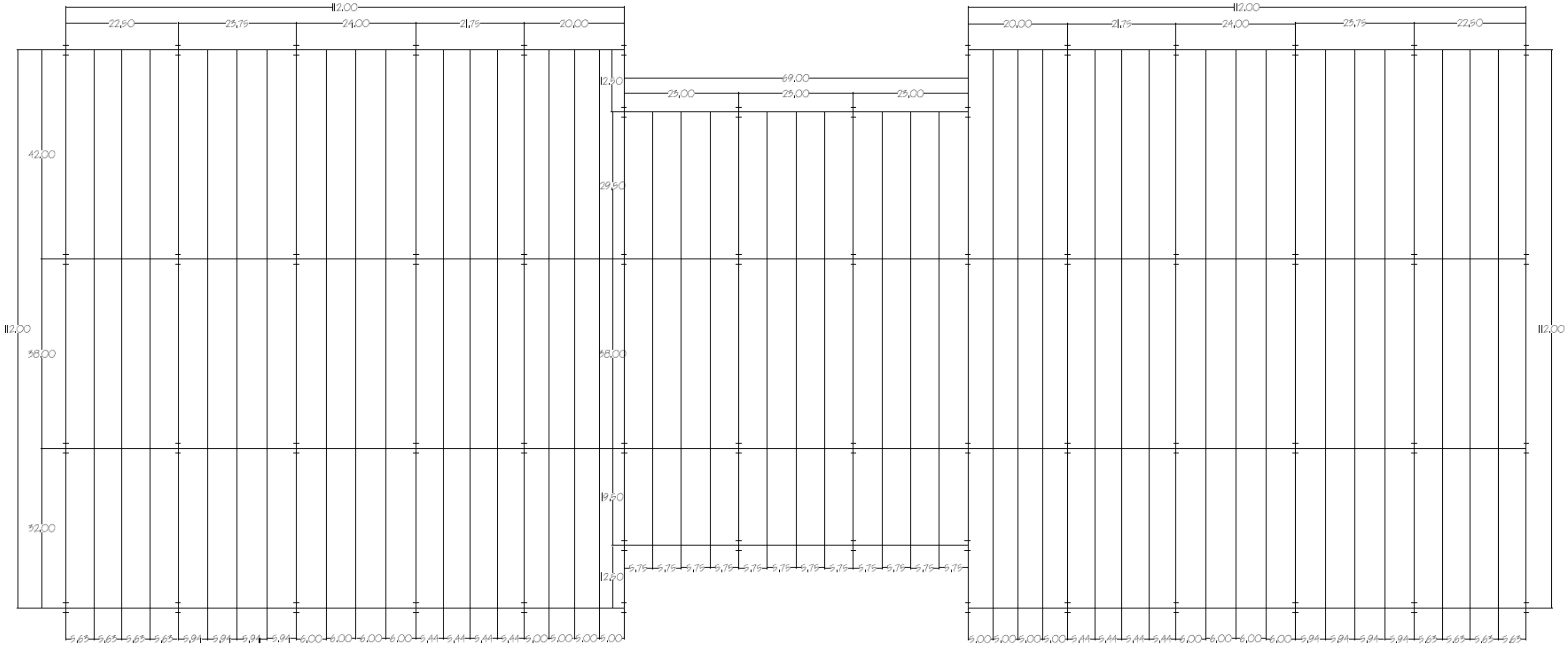
$F_u A_{nt} + .6F_y A_{gv} = 65(.32) + .6(50)(11.41) = 363.1k$ (governs)

$\phi R_n = .75(363.1) = 272.3k \leq 95.24k$ ✓ o.k.

We will use same design for connection of angle to column since column is thicker than girder designed for, and all limit states will be met.

8 APPENDIX B: STRUCTURAL CAD DRAWINGS




112.00
22.50
23.75
24.00
21.75
20.00
69.00
23.00
42.00
38.00
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12.50
29.50
19.50
5.75
5.63
5.94
6.00
5.44
5.00

9 APPENDIX C: FOUNDATION DESIGN

9.1 Soil Properties

Table 35: Properties of Each Soil Layer in Design Soil Profile

	Mix of Poorly Graded Sand (SP) and Silty Sand (SM)	Poorly Graded Sand (SP)	Silty Sand (SM)	Well –Graded Sand (SW)
Elevation of Layer(ft)	488	480	470	468.5
Thickness of Layer (ft)	8	10	2.5	3.5
Total Unit Weight (pcf)	115	110	125	132.5
Dry Unit Weight (pcf)	115	110	92.5	115
Angle of Internal Friction (ϕ') (Cohesionless Soils)	32	32	32	32

Table 36: Soil Parameters, and Assumptions used to Develop the Bearing Capacity Analysis

Soil Parameters and Assumptions	Value	Reason Selected
c(lb/ft^2)	0	According to Geotechnical Repot by the Maguire Group
ϕ(degrees)	32	According to Geotechnical Repot by the Maguire Group.
ϒ(lbs/ft^3)	100	The foot is embedded in the soil to a depth of 6-7 feet. Based on this fact it lies in the clay soil with the corresponding unit weight.
Depth to Water Table (feet)	18	Based on the design soil profile developed this was the shallowest level observed
Factor of Safety	3.5	Was selected based on guidelines outlined in Chapter 6.4 of *Foundation Design Principles and Practices* (Coduto, 2001). This is a reasonable value for a factor of safety for sandy soil with: minimal site characterization data, moderate soil variability, high importance of structure, and consequence of failure.
Embedment Depth, D (feet)	4	Was selected based on guidelines outlined in Chapter 8.1 of *Foundation Design Principles and Practices* (Coduto, 2001), which displays minimum depth of embedment for square footings. The IBC specifies that for areas with freeze cycles the minimum embedment depth should be 4 feet (International Code Council, 2009).

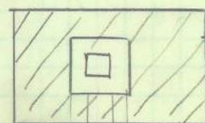
9.2 Sample Hand Calculations Foundations-Square Spread Footing

FOUNDATIONS | Short Span | Typ Interior SQ

P_u Factored load = $1.2D + 1.6L + .5S$
$= 1.2(230.01^k) + 1.6(225.44) + (19.51).5$
$= 646.515^k$

D = 7 ft (Based on settlement analysis > 4 ft Min MA Building code)
B = 10 ft = 120" c = 0 $\phi = 32°$ $\gamma = 115\ lb/ft^2$ t = 3.5
f'_c = 4,000 psi
f_y = 60,000 psi
$M_u = 0$; since no applied moment

STEP 1: Determine effective depth based on 2 way shear analysis
Sketch
Critical area at a distance ($d/2$) from a column face.



Try T = 12"
d = T - 1 bar ϕ - 3in = 8"

$$V_{uc} = \left(\frac{P_u}{4} + \frac{M_u}{c+d}\right)\left(\frac{B^2 - (c+d^2)}{B^2}\right)$$

23 c = 18

$$= \left(\frac{646.515^k}{4} + \frac{M_u}{c+d}\right)\left(\frac{(120)^2 - (18-8^2)}{(120)^2}\right)$$

0 (assumed above)

= 207401.114

$V_{nc} = 4\sqrt{f'_c}\, b_o d$ b_o = perimeter of critical area
d = effective depth

$b_o = 8" + 18" = 26"$
$= 4\sqrt{4{,}000}\,(26")(8")$
= 52,620.30 lb

$\phi V_{nc} = (.85)(52)(52620.30) = 44727.20$

Check is $V_{uc} > \phi V_{nc}$?
no ∴ not acceptable try bigger size.

FOR T = 24"
V_{uc} = 16516.4014 ϕV_{nc} = 163426.50 NOT Okay

TRY T = 27'

$$V_{uc} = \left(\frac{P_u}{4} + \frac{M_u}{c+d}\right)\left(\frac{B^2 - (c+d^2)}{B^2}\right) = \left(\frac{646.515^k}{4} + 0\right)\left(\frac{120^2 - (18-23^2)}{120^2}\right)$$

= 167364.3 lb

$V_{nc} = 4\sqrt{f'_c} \cdot b_o d = 4\sqrt{4000}\,(41")(23") = 238562.23$ lb

$\phi V_{nc} = .85(V_{nc} = 238562.23) = 202777.8927$

$V_{uc} = 167364.3 < 238562$ okay! d = 23"

FOUNDATION DESIGN | SHORT SPAN | SQ FOOTING Typ interior

$\frac{\delta_D}{\delta}$ = .5" (since a flexible structure) coduto = pg 248

According to table 7.5 in Foundations Design Principle and Practices δ_D/δ ratio is $\approx$ .5 $\therefore$ diff settlements may be large as (.5)(2") = 1.

Necessary to design footing so total settlement no greater than .516/.5 = 1.032"
Thus allowable settlement must be reduced to δ_a = 1.032"

STEP 6/7: Use classical method spreadsheet by Coduto
max BEARING PRESSURE = q = 10921 lb/ft^2
δ = 2.14"
is $\delta < \delta_a$? is 2.14" < 1.032" no choose lower q value

STEP 8: Use Schertmann Spreadsheet by Coduto.
→ using the δ_a value obtained from step 6/7 and the techniques described in Chp 7 of Foundations Design Principles and Practices, perform a settlement analysis and the footing with the largest applied normal load.
Determine max bearing pressure that keeps total settlement within tolerable limits ($\delta \le \delta_a$)

q_a = 6037 lb/ft^2
δ = 1.01" } From Schertmann spreadsheet
B = 10'
D = 7'
δ = 1.02" $\le$ 1.032"

STEP 9: let q_a = lower value from q_a from step 4 (bearing capacity) versus step 7 (due to settlement select) q_a = 6037 lb/ft^2

STEP 10: DETERMINE THE ACTUAL REQUIRED FOOTING WIDTH.

$B = \sqrt{A} = \sqrt{\dfrac{P + W_f}{q_a + u_D}}$

P = 483.674^k
u_D = 0 since significant height above H_2O table
W_f = (7')(150 lb/ft^3)B^2 → wt conc
= 1050B^2

$B^2 = \dfrac{P = 483{,}}{6037} \dfrac{\times10^3 + 1050B^2}{+0}$

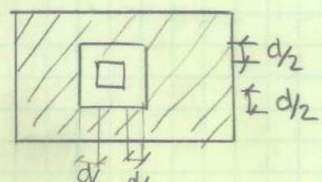
B = 10.0' $\approx$ 10' To satisfy Bearing capacity and settlement analysis

FOUNDATIONS | Short Span | Typ Interior SQ

P_u Factored load = 1.2D + 1.6L + .5S
$= 1.2(230.1^k) + 1.6(225.44) + (19.51).5$
$= 646.515^k$

D = 7 ft (Based on settlement analysis > 4 ft Min MA. Building code
B = 10 ft = 120" c = 0 $\phi = 32°$ γ = 115 lb/ft^2 t = 3.5
f'_c = 4,000 psi
f_y = 60,000 psi
$M_u = 0$; since no applied moment

STEP 1: Determine effective depth based on 2 way shear analysis
Sketch
critical area at a distance ($d/2$) from a column face.



Try T = 12"
d = T - 1 bar ϕ - 3in = 8"

$$V_{uc} = \left(\frac{P_u}{4} + \frac{M_u}{c+d}\right)\left(\frac{B^2 - (c+d^2)}{B^2}\right)$$ 23 c = 18

$$= \left(\frac{646.515^k}{4} + \frac{M_u}{c+d}\right)\left(\frac{(120)^2 - (18-8^2)}{(120)^2}\right)$$
0 (assumed above)

= 207401.114

$V_{nc} = 4\sqrt{f'_c}\, b_o d$ b_o = perimeter of critical area
d = effective depth
b_o = 8" + 18" = 26"
$= 4\sqrt{4{,}000}\,(26")(8")$
= 52620.30 lb

$\phi V_{nc} = (.85)(52)(52620.30) = 44727.20$

Check is $V_{uc} > \phi V_{nc}$?
no $\therefore$ not acceptable try bigger size.

FOR T = 24"
V_{uc} = 16516.4014 ϕV_{nc} = 163426.50 NOT Okay

TRY t = 27'
$$V_{uc} = \left(\frac{P_u}{4} + \frac{M_u}{c+d}\right)\left(\frac{B^2 - (c+d^2)}{B^2}\right) = \left(\frac{646.515^k}{4} + 0\right)\left(\frac{120^2 - (18 - 23^2)}{120^2}\right)$$
= 167364.3 lb
$V_{nc} = 4\sqrt{f'_c}\cdot b_o d = 4\sqrt{4000}\,(41")(23") = 238562.23$ lb
$\phi V_{nc} = .85(V_{nc} = 238562.23) = 202777.8927$
$V_{uc} = 167364.3 < 238562$ okay! d = 23"

Foundations	Short Span	Typ interior SQ

Step 12: determine EFFECTIVE DEPTH FOR 1-way Shear

Try $T = 24''$

$d = T - 1\text{bar}\,\phi - 3'' = 12' - 1 - 3 = 20''$

$$V_{uc} = \left(\frac{B - c - 2d}{B}\right)\sqrt{P_u + \frac{6M_u}{B}}^{\,2} + V_u$$

V_u = 0 since no applied load

M_u = 0 since no applied moment

$$= \left(\frac{120 - 18'' - 2(20)}{120}\right)\sqrt{646.515}$$

$V_{uc} = 415.43^{k}$

$V_{nc} = 2b_w \cdot d \cdot \sqrt{f'_c} = 2(120)(8'')(4000)^{1/2}$

$= 121143.462$

$\Phi = .85$

$\Phi V_{nc} = 103216\ \text{lb}$

$\Phi V_n = 103216\ \text{lb} > 415,413\ \text{lb}$

STEP 13: DETERMINE Avg value FOR EFFECTIVE DEPTH FOR one-way and TWO-way Shear steel runs in 2 directions USE AVERAGE value for d

$$d_{avg} = \frac{d_{one\text{-}way} + d_{two\text{-}way}}{2} = \frac{20'' + 23''}{2}$$

$= 21.5''$ Round up to $22''$

STEP 14: DETERMINE STEEL AREA REQ

$$A_s = \left(\frac{f'_c\, b}{1.176 F_y}\right)\left(d - \sqrt{d^2 - \frac{2.353 M_{uc}}{\Phi f'_c b}}\right)$$

$$L = \frac{B - c}{2} = \frac{120'' - 18''}{2} = 51''$$

$$M_{uc} = \frac{P_u L^2}{2B} + 0 = \frac{(646.5)(51)^2}{2(120)} = 7001025\ \text{in-lb}$$

FOUNDATIONS | SHORT SPAN | Typ interior SQ.

$$A_S = \frac{4000 \cdot 120''}{1.176 \cdot 60{,}000}\left(22'' - \sqrt{(22)^2 - \frac{2.353 \cdot 700102.5}{(.9)(4000)(120')}}\right)$$

$A_S = 6.0165 \; in^2$

Check minimum steel.

$A_{s\,min} \geq (.0018)(27)(120)$

$\geq 5.83 \; in^2$

$6.02 \; in^2 > 5.83 \; in^2$ okay!

USE Coduto table 9.1 in Foundation Design Principles and design.

USE 8 #8 bars to Give Total area $= 6.0165 \; in^2$

clear space between bars $= \frac{B}{(n+1)} - 1''$

$n = \#$ of bars

$= \frac{120}{8+1} - 1''$

$= 12.3''$ okay

STEP 15: Check development length. for column 12x106

l_d supplied $= L - 3 = 51 - 3 = 48''$

$\frac{c + K_{tr}}{d_b}$ — $K_{tr} = 0$ (for spread footings to be conservative)

$d_b = 1''$ (coduto table 9-1, pg 357)

column dimensions 12.x 12.9

$= 12.2'' / 1'' = 12.2 > 2.5$ so USE 2.5

$\frac{c + K_{tr}}{d_b} = \frac{12.9}{1} > 2.5$ USE 2.5

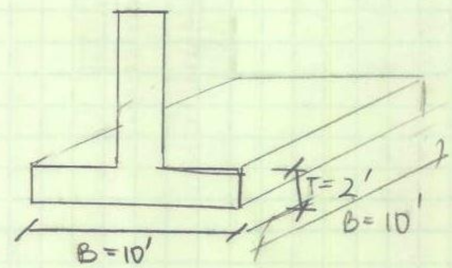
$$l_d = \frac{3}{40} \cdot \frac{f_y}{\sqrt{f'_c}} \cdot d_b \cdot \frac{\alpha \beta \gamma \lambda}{\frac{c + K_{tr}}{d_b}} = \frac{3}{40} \cdot \frac{60000}{\sqrt{4{,}000}} \cdot \left(\frac{(1)(1)(1)(1)}{2.5}\right) = 28$$

$= 28''$

$L_d = 28'' < L_{supplied} = 48$ so ∴ OK

FOUNDATIONS | COST ESTIMATE | SHORT SPAN.

SKETCH.



Volume of concrete (cf) = L(ft) · width (ft) · height (ft)
= 10' · 10' · 24 = 2400 cf
VOLUME CONCRETE cy = 2400 cf / 27 = 88.88 cy
ADD 5% FOR WASTE and Round off
VOLUME of CONCRETE cy = 88.88 · 1.05 = 93.33 cy
USE 94 cy / footing

70 FOOTINGS in short span that do not support
Total = 6580 cy for non vierndel
concrete

10 APPENDIX D: DRAINAGE CALCULATIONS

$$Q_p = ciA$$

$$Q_{init-2} = (.15) * \frac{3.0in}{hr} * (1.93acres) = .87cfs$$

$$Q_{init-10} = (.15) * \frac{4.5in}{hr} * (1.93acres) = 1.30cfs$$

$$Q_{init-25} = (.15) * \frac{5.3in}{hr} * (1.93acres) = 1.53cfs$$

$$Q_{init-100} = (.15) * \frac{6.5in}{hr} * (1.93acres) = 1.88cfs$$

$$Q_{fin-2} = (.5) * \frac{3.0in}{hr} * (1.93acres) = 2.9cfs$$

$$Q_{fin-10} = (.5) * \frac{4.5in}{hr} * (1.93acres) = 4.34cfs$$

$$Q_{fin-25} = (.5) * \frac{5.3in}{hr} * (1.93acres) = 5.11cfs$$

$$Q_{fin-100} = (.5) * \frac{6.5in}{hr} * (1.93acres) = 6.27cfs$$

$$V_{Init} = Qt_c$$

$$V_{Init2} = (.87cfs) * 45min * \frac{60s}{min} = 2{,}349ft^3$$

$$V_{Init10} = (1.30cfs) * 45min * \frac{60s}{min} = 3{,}510ft^3$$

$$V_{Init25} = (1.53cfs) * 45min * \frac{60s}{min} = 4{,}131ft^3$$

$$V_{Init100} = (1.88cfs) * 45min * \frac{60s}{min} = 5{,}076ft^3$$

$$V_{Fin} = Qt_c$$

$$V_{Fin2} = (2.9cfs) * 45min * \frac{60s}{min} = 7{,}830ft^3$$

$$V_{Fin10} = (4.34cfs) * 45min * \frac{60s}{min} = 11{,}718ft^3$$

$$V_{Fin25} = (5.11cfs) * 45min * \frac{60s}{min} = 13{,}797ft^3$$

$$V_{Fin100} = (6.27cfs) * 45min * \frac{60s}{min} = 16{,}929ft^3$$

$$\Delta V = V_{fin} - V_{init}$$

$$\Delta V_2 = 7{,}830ft^3 - 2{,}349ft^3 = 5{,}481ft^3$$

$$\Delta V_{10} = 11{,}718ft^3 - 3{,}510ft^3 = 8{,}208ft^3$$

$$\Delta V_{25} = 13{,}797ft^3 - 4{,}131ft^3 = 9{,}666ft^3$$

$$\Delta V_{100} = 16{,}929ft^3 - 5{,}076ft^3 = 11{,}853ft^3$$

11 APPENDIX E: TRIP GENERATION RATE CALCULATIONS

Based on trip generation data taken from the trip generational information report:

The 7700 sq.ft. of restaurant will generate an approximate maximum of 800 trips per day

Eq. not given

The 12,544 sq.ft. of retail space will generate an approximate maximum of 803 trips per day

Eq. not given

The 93,000 sq.ft. of dwellings (72 apartments) will generate be an approximate maximum of 756 trips per day

Using low rise apartment building since the building has 3 stories of residential floors.

Eq $T= 5.12X+387.53$

The 62,000 sq.ft. of research and development will generate an approximate maximum of 680 trips per day

Using research and development

Eq $\ln T =0.82\ln X+3.14$

The 50,000 sq.ft. of industrial space will generate an approximate maximum of 215 trips per day

using industrial manufacturing

Eq $T=3.88X*20.70$

Table 37: Trip Counts

Usage	Trips
Restaurant	800
Retail	803
Dwelling units	756
Research and development	680
Industrial	215
Total	**3,254 trips per day**

12 APPENDIX F: MODAL SPLIT DATA

Miami University Oxford campus

- Walk 77.5%,
- Personal vehicle 13.6%,
- Miami Metro 4.8%,
- Bike 3.5%,
- Apartment shuttle 0%,
- Other 0.6%
- Sum 81%

Ohio State University

- Walk 70%
- Taxi 19%
- Car 5%
- Bike 6%
- Sum 76%

Cornell University

- Car 19%
- Carpool 5.5%
- Transit 37.7%
- Walk 30.9%
- Bike 4%
- Other2.9%
- Sum 50%

University of California Davis

- Drive 43.5%
- Transit 4.3%
- Walk 4.3%
- Bicycle 47.8%
- Sum 52%

Colombia Univeristy

- Auto 6.0%
- Taxi 2.5%
- Subway 40.0%
- Bus 3.5%
- Shuttle 1.0%

- Walk Only 47%
- Sum 68%

Average: 65% don't use vehicles

13 APPENDIX G: COST ESTIMATES

		Long Span	% of Total	Cost Per S.F.	Cost	
A Substructure			**2.70%**	**$4.61**	**$1,091,771.63**	
A1010	**Standard Foundations**			**$3.46**	**$819,541**	
A1030	**Slab on Grade**			**$1.15**	**$272,230**	
	Slab on grade, 4" thick, non industrial, reinforced				**$0**	
B Shell			**28.90%**	**$79.00**	**$18,701,884.50**	
B1010	**Steel Construction**			**$870/ton**	**$1,349,654**	
	Steel Erection			**$53.10**	**$11,221,131**	
B1020	**Roof Construction**			**$7.73**	**$1,829,861**	
	Floor, composite slab on steel beam, 25'x25' bay, 4"slab, 21.5" total depth, 40 PSF superimposed load, 82 PSF total load					
B2010	**Exterior Walls**			**$8.15**	**$1,929,284**	
	Brick veneer wall, standard face, 20 ga x 3-5/8" NLB @ 24" metal stud back-up, running bond					
B2020	**Exterior Windows**			**$2.67**	**$632,048**	
	Aluminum flush tube frame, for 1/4"glass,1-3/4"x4", 5'x6' opening, no intermediate horizontals					
	Glazing panel, plate glass, 1/4" thick, clear					
B2030	**Exterior Doors**			**$4.38**	**$1,036,842**	
	Door, aluminum & glass, without transom, full vision, double door, hardware, 6'-0" x 7'-0" opening					
	Door, aluminum & glass, with transom, non-standard, double door, hardware, 6'-0" x 10'-0" opening					
	Door, steel 18 gauge, hollow metal, 1 door with frame, no label, 3'-0" x 7'-0" opening					
B3010	**Roof Coverings**			**$2.47**	**$584,703**	
	Roofing, asphalt flood coat, gravel, base sheet, 3 plies 15# asphalt felt, mopped					

	Insulation, rigid, roof deck, composite with 2" EPS, 1" perlite					
	Roof edges, aluminum, duranodic, .050" thick, 6" face					
	Gravel stop, aluminum, extruded, 4", mill finish, .050" thick					
B3020	**Roof Openings**			**$0.50**	**$118,361**	
	Skylight, plastic domes, insulated curbs, 30 SF to 65 SF, single glazing					
	Roof hatch, with curb, 1" fiberglass insulation, 2'-6" x 3'-0", galvanized steel, 165 lbs					
	Smoke hatch, unlabeled, galvanized, 2'-6" x 3', not incl hand winch operator					
C Interiors			**27.60%**	**$30.07**	**$7,118,230.54**	
C1010	**Partitions**			**$5.77**	**$1,365,886**	
	Metal partition, 5/8"fire rated gypsum board face, no base,3 -5/8" @ 24" OC framing, same opposite face, no insulation					
	Gypsum board, 1 face only, exterior sheathing, fire resistant, 5/8"					
	Add for the following: taping and finishing					
	1/2" fire ratedgypsum board, taped & finished, painted on metal furring					
C1020	**Interior Doors**			**$7.15**	**$1,692,562**	
	Door, single leaf, wood frame, 3'-0" x 7'-0" x 1-3/8", birch, solid core					
	Door, single leaf, wood frame, 3'-0" x 7'-0" x 1-3/8", birch, hollow core					
	Locksets, heavy duty cylindrical, non-keyed, passage					
	Locksets, heavy duty cylindrical, keyed, single cylinder function					
C1030	**Fittings**			**$2.73**	**$646,251**	Residential only
	Cabinets, residential, wall, two doors x 48" wide					
C2010	**Stair Construction**			**$3.31**	**$783,550**	
	Stairs, steel, cement filled metal pan & picket rail, 12 risers, with landing					
C3010	**Wall Finishes**			**$2.70**	**$639,149**	
	Painting, interior on plaster and drywall, walls & ceilings, roller work, primer & 2 coats					
	Painting, interior on plaster and drywall, walls & ceilings, roller work, primer & 2 coats					

	Vinyl wall covering, fabric back, medium weight					
	Ceramic tile, thin set, 4-1/4" x 4-1/4"					
C3020	**Floor Finishes**			**$4.92**	**$1,164,672**	residential
	Carpet tile, nylon, fusion bonded, 18" x 18" or 24" x 24", 24 oz					
	Carpet tile, nylon, fusion bonded, 18" x 18" or 24" x 24", 35 oz					
	Carpet, padding, add to above, minimum					
	Carpet, padding, add to above, maximum					
	Vinyl, composition tile, minimum					
	Vinyl, composition tile, maximum					
	Tile, ceramic natural clay					
C3030	**Ceiling Finishes**			**$3.49**	**$826,160**	
	Gypsum board ceilings, 1/2" fire rated gypsum board, painted and textured finish, 7/8"resilient channel furring, 24" OC support					
D Services			**40.80%**	**$57.02**	**$16,697,835**	
		+Retail		**$6.70**		
		+Industrial		**$2.25**		
		+R&D		**$23.19**		
		+Residential		**$15.69**		
D1010	**Elevators and Lifts**			**$14.23**	**$3,368,554**	
	Traction, geared passenger, 3500 lb,15 floors, 10' story height, 2 car group, 350 FPM					
D2010	**Plumbing Fixtures**			**$15.69**	**$1,463,453**	Residential
	Kitchen sink w/trim, countertop, PE on CI, 24" x 21", single bowl					
	Laundry sink w/trim, PE on CI, black iron frame, 24" x 20", single compt					
	Service sink w/trim, PE on CI, corner floor, 28" x 28", w/rim guard					
	Bathroom, lavatory & water closet, 2 wall plumbing, stand alone					
	Bathroom, three fixture, 2 wall plumbing, lavatory, water closet & bathtub, stand alone					
D2010	**Plumbing Fixtures**			**$6.70**	**$168,090**	Restaurant

	Water closet, vitreous china, tank type, 2 piece close coupled			
	Urinal, vitreous china, wall hung			
	Lavatory w/trim, vanity top, PE on CI, 20" x 18"			
	Kitchen sink w/trim, countertop, stainless steel, 44" x 22" triple bowl			
	Service sink w/trim, PE on CI, corner floor, wall hung w/rim guard, 24" x 20"			
	Shower, stall, baked enamel, terrazzo receptor, 36" square			
	Water cooler, elec, floor mounted, refrigerated compartment type, 1.5 GPH			
D2010	**Plumbing Fixtures**	**$2.25**	**$126,403**	Factory
	Water closet, vitreous china, bowl only with flush valve, wall hung			
	Urinal, vitreous china, stall type			
	Lavatory w/trim, vanity top, PE on CI, 19" x 16" oval			
	Kitchen sink w/trim, countertop, stainless steel, 33" x 22" double bowl			
	Service sink w/trim, PE on CI, corner floor, wall hung w/rim guard, 22" x 18"			
	Group wash fountain, precast terrazzo, circular, 54" diameter			
	Shower, stall, baked enamel, terrazzo receptor, 36" square			
	Water cooler, electric, floor mounted, dual height, 14.3 GPH			
D2010	**Plumbing Fixtures**	**$23.19**	**$1,442,001**	Lab
	Water closet, vitreous china, bowl only with flush valve, wall hung			
	Urinal, vitreous china, wall hung			
	Lavatory w/trim, wall hung, PE on CI, 18" x 15"			
	Lab sink w/trim, polyethylene, single bowl, double drainboard, 54" x 24" OD			
	Service sink w/trim, vitreous china, wall hung 22" x 20"			
	Shower, stall, fiberglass 1 piece, three walls, 36" square			
	Water cooler, electric, wall hung, wheelchair type, 7.5 GPH			
D2020	**Domestic Water Distribution**	**$1.90**	**$449,772**	
	Gas fired water heater, commercial, 100< F rise, 600 MBH input, 576 GPH			

D2040	**Rain Water Drainage**			**$0.12**	**$28,407**	
	Roof drain, DWV PVC, 4" diam, diam, 10' high					
	Roof drain, DWV PVC, 4" diam, for each additional foot add					
D3010	**Energy Supply**			**$5.90**	**$1,396,660**	
	Apartment building heating system, fin tube radiation, forced hot water, 30,000 SF area,300,000 CF vol					
D3050	**Terminal & Package Units**			**$18.80**	**$4,450,374**	
	Rooftop, multizone, air conditioner, schools and colleges, 25,000 SF, 95.83 ton					
D4010	**Sprinklers**			**$2.98**	**$705,432**	
	Wet pipe sprinkler systems, steel, light hazard, 1 floor, 10,000 SF					
	Wet pipe sprinkler systems, steel, light hazard, each additional floor, 10,000 SF					
	Standard High Rise Accessory Package 16 story					
D4020	**Standpipes**			**$1.61**	**$381,122**	
	Wet standpipe risers, class III, steel, black, sch 40, 6" diam pipe, 1 floor					
	Fire pump, electric, with controller, 5" pump, 100 HP, 1000 GPM					
	Fire pump, electric, for jockey pump system, add					
D5010	**Electrical Service/Distribution**			**$2.23**	**$527,890**	
	Service installation, includes breakers, metering, 20' conduit & wire, 3 phase, 4 wire, 120/208 V, 2000 A					
	Feeder installation 600 V, including RGS conduit and XHHW wire, 2000 A					
	Switchgear installation, incl switchboard, panels & circuit breaker, 2000 A					
D5020	**Lighting and Branch Wiring**			**$8.69**	**$2,057,114**	
	Receptacles incl plate, box, conduit, wire, 4 per 1000 SF, .5 W per SF, with transformer					
	Miscellaneous power, 1 watt					

	Central air conditioning power, 4 watts					
	Motor installation, three phase, 460 V, 15 HP motor size					
	Motor feeder systems, three phase, feed to 200 V 5 HP, 230 V 7.5 HP, 460 V 15 HP, 575 V 20 HP					
	HID fixture, 8'-10' above work plane, 100 FC, type C, 8 fixtures per 1800 SF					
D5030	**Communications and Security**			**$0.38**	**$89,954**	
	Communication and alarm systems, includes outlets, boxes, conduit and wire, fire detection systems, 50 detectors					
D5090	**Other Electrical Systems**			**$0.18**	**$42,610**	
	Generator sets, w/battery, charger, muffler and transfer switch, gas/gasoline operated, 3 phase, 4 wire, 277/480 V, 30 kW					
G Building Sitework			**0.00%**	**$0.00**	**$0**	
SubTotal			**100%**	**$170.71**	**$43,609,721**	
Contractor Fees (General Conditions,Overhead,Profit)			**10.00%**	**$18.42**	**$4,360,972**	
Architectural Fees			**7.00%**	**$12.90**	**$3,052,680**	
User Fees			**0.00%**	**$0.00**	**$0**	
Total Building Cost				**$202.02**	**$51,023,374**	

		Short Span	% of Total	Cost Per S.F.	Cost
A Substructure			**2.70%**	**$4.39**	**$1,038,731.42**
A1010	**Standard Foundations**			**$3.24**	**$766,501**
A1030	**Slab on Grade**			**$1.15**	**$272,230**
	Slab on grade, 4" thick, non industrial, reinforced				**$0**
B Shell			**28.90%**	**$80.24**	**$18,995,088.06**

B1010	**Steel Construction**			**$870/ton**	**$904,118**
	Steel Erection			**$54.34**	**$11,959,870**
B1020	**Roof Construction**			**$7.73**	**$1,829,861**
	Floor, composite slab on steel beam, 25'x25' bay, 4"slab, 21.5" total depth, 40 PSF superimposed load, 82 PSF total load				
B2010	**Exterior Walls**			**$8.15**	**$1,929,284**
	Brick veneer wall, standard face, 20 ga x 3-5/8" NLB @ 24" metal stud back-up, running bond				
B2020	**Exterior Windows**			**$2.67**	**$632,048**
	Aluminum flush tube frame, for 1/4"glass,1-3/4"x4", 5'x6' opening, no intermediate horizontals				
	Glazing panel, plate glass, 1/4" thick, clear				
B2030	**Exterior Doors**			**$4.38**	**$1,036,842**
	Door, aluminum & glass, without transom, full vision, double door, hardware, 6'-0" x 7'-0" opening				
	Door, aluminum & glass, with transom, non-standard, double door, hardware, 6'-0" x 10'-0" opening				
	Door, steel 18 gauge, hollow metal, 1 door with frame, no label, 3'-0" x 7'-0" opening				
B3010	**Roof Coverings**			**$2.47**	**$584,703**
	Roofing, asphalt flood coat, gravel, base sheet, 3 plies 15# asphalt felt, mopped				
	Insulation, rigid, roof deck, composite with 2" EPS, 1" perlite				
	Roof edges, aluminum, duranodic, .050" thick, 6" face				
	Gravel stop, aluminum, extruded, 4", mill finish, .050" thick				
B3020	**Roof Openings**			**$0.50**	**$118,361**
	Skylight, plastic domes, insulated curbs, 30 SF to 65 SF, single glazing				
	Roof hatch, with curb, 1" fiberglass insulation, 2'-6" x 3'-0", galvanized steel, 165 lbs				
	Smoke hatch, unlabeled, galvanized, 2'-6" x 3', not incl hand winch operator				
C Interiors			**27.60%**	**$30.07**	**$7,118,230.54**

C1010	**Partitions**	**$5.77**	**$1,365,886**	
	Metal partition, 5/8"fire rated gypsum board face, no base,3 -5/8" @ 24" OC framing, same opposite face, no insulation			
	Gypsum board, 1 face only, exterior sheathing, fire resistant, 5/8"			
	Add for the following: taping and finishing			
	1/2" fire ratedgypsum board, taped & finished, painted on metal furring			
C1020	**Interior Doors**	**$7.15**	**$1,692,562**	
	Door, single leaf, wood frame, 3'-0" x 7'-0" x 1-3/8", birch, solid core			
	Door, single leaf, wood frame, 3'-0" x 7'-0" x 1-3/8", birch, hollow core			
	Locksets, heavy duty cylindrical, non-keyed, passage			
	Locksets, heavy duty cylindrical, keyed, single cylinder function			
C1030	**Fittings**	**$2.73**	**$646,251**	Residential only
	Cabinets, residential, wall, two doors x 48" wide			
C2010	**Stair Construction**	**$3.31**	**$783,550**	
	Stairs, steel, cement filled metal pan & picket rail, 12 risers, with landing			
C3010	**Wall Finishes**	**$2.70**	**$639,149**	
	Painting, interior on plaster and drywall, walls & ceilings, roller work, primer & 2 coats			
	Painting, interior on plaster and drywall, walls & ceilings, roller work, primer & 2 coats			
	Vinyl wall covering, fabric back, medium weight			
	Ceramic tile, thin set, 4-1/4" x 4-1/4"			
C3020	**Floor Finishes**	**$4.92**	**$1,164,672**	residential
	Carpet tile, nylon, fusion bonded, 18" x 18" or 24" x 24", 24 oz			
	Carpet tile, nylon, fusion bonded, 18" x 18" or 24" x 24", 35 oz			
	Carpet, padding, add to above, minimum			
	Carpet, padding, add to above, maximum			
	Vinyl, composition tile, minimum			
	Vinyl, composition tile, maximum			
	Tile, ceramic natural clay			

C3030	**Ceiling Finishes**			**$3.49**	**$826,160**	
	Gypsum board ceilings, 1/2" fire rated gypsum board, painted and textured finish, 7/8"resilient channel furring, 24" OC support					
D Services			**40.80%**	**$57.02**	**$16,697,835**	
		+Retail		**$6.70**		
		+Industrial		**$2.25**		
		+R&D		**$23.19**		
		+Residential		**$15.69**		
D1010	**Elevators and Lifts**			**$14.23**	**$3,368,554**	
	Traction, geared passenger, 3500 lb,15 floors, 10' story height, 2 car group, 350 FPM					
D2010	**Plumbing Fixtures**			**$15.69**	**$1,463,453**	Residential
	Kitchen sink w/trim, countertop, PE on CI, 24" x 21", single bowl					
	Laundry sink w/trim, PE on CI, black iron frame, 24" x 20", single compt					
	Service sink w/trim, PE on CI, corner floor, 28" x 28", w/rim guard					
	Bathroom, lavatory & water closet, 2 wall plumbing, stand alone					
	Bathroom, three fixture, 2 wall plumbing, lavatory, water closet & bathtub, stand alone					
D2010	**Plumbing Fixtures**			**$6.70**	**$168,090**	Restaurant
	Water closet, vitreous china, tank type, 2 piece close coupled					
	Urinal, vitreous china, wall hung					
	Lavatory w/trim, vanity top, PE on CI, 20" x 18"					
	Kitchen sink w/trim, countertop, stainless steel, 44" x 22" triple bowl					
	Service sink w/trim, PE on CI, corner floor, wall hung w/rim guard, 24" x 20"					
	Shower, stall, baked enamel, terrazzo receptor, 36" square					
	Water cooler, elec, floor mounted, refrigerated compartment type, 1.5 GPH					

D2010	**Plumbing Fixtures**			**$2.25**	**$126,403**	Factory
	Water closet, vitreous china, bowl only with flush valve, wall hung					
	Urinal, vitreous china, stall type					
	Lavatory w/trim, vanity top, PE on CI, 19" x 16" oval					
	Kitchen sink w/trim, countertop, stainless steel, 33" x 22" double bowl					
	Service sink w/trim, PE on CI, corner floor, wall hung w/rim guard, 22" x 18"					
	Group wash fountain, precast terrazzo, circular, 54" diameter					
	Shower, stall, baked enamel, terrazzo receptor, 36" square					
	Water cooler, electric, floor mounted, dual height, 14.3 GPH					
D2010	**Plumbing Fixtures**			**$23.19**	**$1,442,001**	Lab
	Water closet, vitreous china, bowl only with flush valve, wall hung					
	Urinal, vitreous china, wall hung					
	Lavatory w/trim, wall hung, PE on CI, 18" x 15"					
	Lab sink w/trim, polyethylene, single bowl, double drainboard, 54" x 24" OD					
	Service sink w/trim, vitreous china, wall hung 22" x 20"					
	Shower, stall, fiberglass 1 piece, three walls, 36" square					
	Water cooler, electric, wall hung, wheelchair type, 7.5 GPH					
D2020	**Domestic Water Distribution**			**$1.90**	**$449,772**	
	Gas fired water heater, commercial, 100< F rise, 600 MBH input, 576 GPH					
D2040	**Rain Water Drainage**			**$0.12**	**$28,407**	
	Roof drain, DWV PVC, 4" diam, diam, 10' high					
	Roof drain, DWV PVC, 4" diam, for each additional foot add					
D3010	**Energy Supply**			**$5.90**	**$1,396,660**	
	Apartment building heating system, fin tube radiation, forced hot water, 30,000 SF area,300,000 CF vol					
D3050	**Terminal & Package Units**			**$18.80**	**$4,450,374**	

	Rooftop, multizone, air conditioner, schools and colleges, 25,000 SF, 95.83 ton		
D4010	**Sprinklers**	**$2.98**	**$705,432**
	Wet pipe sprinkler systems, steel, light hazard, 1 floor, 10,000 SF		
	Wet pipe sprinkler systems, steel, light hazard, each additional floor, 10,000 SF		
	Standard High Rise Accessory Package 16 story		
D4020	**Standpipes**	**$1.61**	**$381,122**
	Wet standpipe risers, class III, steel, black, sch 40, 6" diam pipe, 1 floor		
	Fire pump, electric, with controller, 5" pump, 100 HP, 1000 GPM		
	Fire pump, electric, for jockey pump system, add		
D5010	**Electrical Service/Distribution**	**$2.23**	**$527,890**
	Service installation, includes breakers, metering, 20' conduit & wire, 3 phase, 4 wire, 120/208 V, 2000 A		
	Feeder installation 600 V, including RGS conduit and XHHW wire, 2000 A		
	Switchgear installation, incl switchboard, panels & circuit breaker, 2000 A		
D5020	**Lighting and Branch Wiring**	**$8.69**	**$2,057,114**
	Receptacles incl plate, box, conduit, wire, 4 per 1000 SF, .5 W per SF, with transformer		
	Miscellaneous power, 1 watt		
	Central air conditioning power, 4 watts		
	Motor installation, three phase, 460 V, 15 HP motor size		
	Motor feeder systems, three phase, feed to 200 V 5 HP, 230 V 7.5 HP, 460 V 15 HP, 575 V 20 HP		
	HID fixture, 8'-10' above work plane, 100 FC, type C, 8 fixtures per 1800 SF		
D5030	**Communications and Security**	**$0.38**	**$89,954**
	Communication and alarm systems, includes outlets, boxes, conduit and wire, fire detection systems, 50 detectors		

D5090	**Other Electrical Systems**			**$0.18**	**$42,610**
	Generator sets, w/battery, charger, muffler and transfer switch, gas/gasoline operated, 3 phase, 4 wire, 277/480 V, 30 kW				
G Building Sitework			**0.00%**	**$0.00**	**$0**
SubTotal			**100%**	**$171.72**	**$43,849,885**
Contractor Fees (General Conditions,Overhead,Profit)			**10.00%**	**$18.52**	**$4,384,988**
Architectural Fees			**7.00%**	**$12.97**	**$3,069,492**
User Fees			**0.00%**	**$0.00**	**$0**
Total Building Cost				**$203.21**	**$51,304,365**

14 APPENIDX H: INTERVIEW TRANSCRIPT

For interview conducted with Mr. Jeffrey S. Solomon, Chief Financial Officer and Vice President for Finance and Operations

Date conducted: October 31, 2011

Introduction to be read to interviewee

We are a group of students working with Professor Albano and Professor LePage on our MQP. For our MQP, we are investigating, designing, and analyzing a proposed mixed-use development that will serve as: office and industrial space for new life science companies, retail space, and Graduate or Upper-class housing. This MQP will also analyze the impact of the proposed building to the existing traffic and parking. The proposed development is taking place on Lot 6 of the Gateway Plan.

1. We recently reviewed the WPI strategic plan in particular goals 7 and 5 and were wondering how the vision may have changed since its publication in 2008?
   Response: WPI would like to see Gateway be comprised of approximately 40-60% Life Sciences, and the plan generally remains the same

2. Would a building containing upper-class/graduate-housing be something that WPI might be interested in constructing? If so, would it be run by WPI or through an independent firm?
   Response: *Yes on the vacant lot at 75 Grove Street. That's where we would consider undergraduate housing for upper-class students. It would be similar to East Hall; suite style apartments, tech suites. Parking could be across the site at the national grid site. We are currently in negotiations with them. (Thinking of developing that site internally or with a development (privatized approach). However, it will not be graduate housing; we can use Salisbury estates for graduate housing.*

3. Would WPI be interested in further developing campus facilities at Gateway supporting academic and co-curricular needs, similar to what is being constructed right now with the Fire Protection Combustion Lab?
   Response: *There could potentially be another academic building at gateway, however there are no plans for that right now. Depending on what happens with the market; nothing in the short term for sure.*
   WPI's Goals are:
   - Move the strategic plan for gateway forward
   - Free up space in Higgins lab to expand on campus
   - To do more work force training at Gateway Park
   - Provide more opportunities for graduate research and state of the art facilities. This is a major part of strategic plan overall

WPI is definitely interested in some housing there as a well

4. What is the time-frame of constructing these buildings?

Response: Resident hall in the next two years at 75 Grove Street. WPI cannot push anymore for the other sites as we are already over extended with current projects. It is a little premature to develop other site. The intention is to have a public private partnership with a lot of federal funding. For the building under construction WPI received a lot of partnerships and subsidies. Market will have to show that there is demand prior to development. Worcester cannot support the rents in even decent times but construction costs are the same as Cambridge. Cannot charge enough rent due to the "Worcester Delta" so we must mitigate costs through subsidy programs. E.g. new market tax credits, MA life center helping with fit out of building, or a number of incentives. Getting $20/ sq. foot in rent is a struggle especially if there is something around the corner for 14$/ sq. $25/SF in new building is low and only possible because of these subsidies.

5. Have two connected buildings spanning the culvert in Lot 6 been considered as an alternative design to two separate buildings?

Response: Single building that bridges the culvert yes. *A number of studies have been done by architect and independent firms. Maximizing that site will be very important. Do not want to end up orphaning a part of the site*. *The parking garage supports 75-80% of build out of park. Whatever someone does on lot they will have to solve that parking problem.* The challenge is the culvert and addressing the parking issue. An underground parking garage is not likely, rather we can extend the existing garage to expand the parking. Above ground parking is $16000 a space vs. $50,000 underground. Hazardous material gets very expensive to deal with and may be run into below grade on that site. Ask Jon Weaver for conceptual designs of Lot 6 A and 6B or lot 6 and 6A. Jon may have some files on that bridged building.

6. Are there plans to use any aspect of this building to help produce revenue as discussed in Goal 7 of *The Strategic Plan?*

Response: WPI doesn't see themselves renting anymore space at Gateway or developing lot 6 themselves. WPI has other needs such as: a garage, a residence hall, and converting alumni gym for academic space so it can be used for classes. Space being freed up as fire protection moving out. We don't see WPI pushing out with another building; only possibility is if we are very successful with Fundraising. WPI wants to put the business school building on library parking lot.

7. Are there specific companies that have already expressed interest in leasing space if more was to be built?

Response: No specific companies. O'Connell is marketing the development of lot 6 with a couple brokers (Keller and Sckaowski.) Few thousand square feet for each of the floors. Smaller local firms will invest but the bigger companies may want to expand. O'Connell's development interest depends on how this current development at Gateway pans out and honestly, interest in the building has been slow.

8. Are there other long-term facility needs that might be able to be accommodated at Gateway?
   Response: Maybe a new academic building or a residence hall

9. If WPI is interested in developing a mixed-used facility, then what is WPI's approach to determining feasibility?

Response: WPI cannot be too prescriptive. We will try to ensure they are mixed-use but predominantly life sciences. Have design requirement. We would like the buildings to look Cannot look too different from buildings down there(brick and glass façade)

10. Would WPI prefer development by a third party? If yes, why?

Response: WPI does prefer development by a 3rd party for those things. WPI has already put a ton of money into the infrastructure and cleaning up and " is not prepared to put any capital down there now a 3rd party developer has to do this"

15 APPENDIX I: BRIEF HISTORY OF WORCESTER

In 1722 Worcester was incorporated and officially became a town of the Commonwealth of Massachusetts (WorcesterMA.gov). Worcester is an area with a rich history of a working class. Today, Worcester is once again growing as a community.

Figure 61: Circus comes to Worcester (E. B. Luce Photography, 2009)

The Blackstone River is one of the main reasons for Worcester's past success. At the peak of the Industrial Revolution here in America, the valley of the Blackstone River housed over 1,100 mills (Rittman). As industry was growing along the river, the need to move goods between Worcester and Providence was increasing. Accordingly, more important to industry than the Blackstone River was the Blackstone Canal.

In the 1820s, two separate companies started constructing a canal to connect Worcester to Providence. One company started in 1824 in Providence, while the other company started in 1826 in Worcester. In 1825, the two companies combined together as the Blackstone Canal Company. By November 1828, the canal was in full use; usage of the canal peaked in 1832. The canal utilized 49 locks to transport boats from Providence to Worcester compensating for the 438 foot change in elevation (Eckilson, 2007).

By 1833, legal action was taken against the Blackstone Canal Company by many mill owners over water rights violations. It was said that the canal would have to restore water to the river within one hour of lockage. Despite efforts to make this possible, it was unmanageable to do such. By 1840, the Massachusetts Supreme Court ruled that the company pay $8,450 in fines. However, the company was unable to pay this debt, as it was already behind on repairs to the canal. By 1845, the company was sold to the Providence and Worcester Railroad. Finally, the canal saw its final use in 1848 (Salotto, 2000). By this time, the Providence and Worcester Railroad had begun to use the canal's banks to lay down tracks.

Figure 62: Busy Main Street with Trolley Cars (E. B. Luce Photography, 2009)

The year 1848 was also when Worcester officially became a city. In 1849, Main Street was paved for the first time ending the treachery of dust clouds from sunny days and mud following a period of rainy days. Many people would consider Worcester along with the Blackstone Valley to be the birthplace of the Industrial Revolution here in America.

Figure 63: Old Main Street Worcester (E. B. Luce Photography, 2009)

In 1893, the first known triple-decker was built in Worcester (Krim, 1977). Triple-deckers were comfortable apartments where the owner would typically live on the second floor. The owner would then lease out the other two units to tenants who also worked in the Worcester area. The neighborhoods comprised of these triple-deckers were typically viewed as safe as they consisted of local factory workers and their families alike.

Figure 64: Main Street Worcester, 1962 (Worcester Telegram)

By the turn of the century the population of Worcester was estimated to be at 118,000 persons. Population peaked in Worcester in the 1950s when the population was just over 203,000 persons. It has continued to decline until the 1990s (U.S. Census Bureau, June).

16 APPENDIX J: LEED POINTS

			LEED 2009 for New Construction and Major Renovations			
			Sustainable Sites		Possible Points:	26
Y	?	N				
Y			Prereq 1	Construction Activity Pollution Prevention		
Y			Credit 1	Site Selection		1
Y			Credit 2	Development Density and Community Connectivity		5
Y			Credit 3	Brownfield Redevelopment		1
Y			Credit 4.1	Alternative Transportation—Public Transportation Access		6
Y			Credit 4.2	Alternative Transportation—Bicycle Storage and Changing Rooms		1
Y			Credit 4.3	Alternative Transportation—Low-Emitting and Fuel-Efficient Vehicles		3
Y			Credit 4.4	Alternative Transportation—Parking Capacity		2
		N	Credit 5.1	Site Development—Protect or Restore Habitat		1
Y			Credit 5.2	Site Development—Maximize Open Space		1
	?		Credit 6.1	Stormwater Design—Quantity Control		1
	?		Credit 6.2	Stormwater Design—Quality Control		1
Y			Credit 7.1	Heat Island Effect—Non-roof		1
Y			Credit 7.2	Heat Island Effect—Roof		1
Y			Credit 8	Light Pollution Reduction		1
			Water Efficiency		Possible Points:	10
Y			Prereq 1	Water Use Reduction—20% Reduction		
Y			Credit 1	Water Efficient Landscaping		2 to 4
		N	Credit 2	Innovative Wastewater Technologies		2
Y			Credit 3	Water Use Reduction		2 to 4
			Energy and Atmosphere		Possible Points:	35
Y			Prereq 1	Fundamental Commissioning of Building Energy Systems		
Y			Prereq 2	Minimum Energy Performance		0
Y			Prereq 3	Fundamental Refrigerant Management		
	?		Credit 1	Optimize Energy Performance		1 to 19
	?		Credit 2	On-Site Renewable Energy		1 to 7
Y			Credit 3	Enhanced Commissioning		2
Y			Credit 4	Enhanced Refrigerant Management		2
Y			Credit 5	Measurement and Verification		3
	?		Credit 6	Green Power		2
			Materials and Resources		Possible Points:	14
Y			Prereq 1	Storage and Collection of Recyclables		0
		N	Credit 1.1	Building Reuse—Maintain Existing Walls, Floors, and Roof		1 to 3
		N	Credit 1.2	Building Reuse—Maintain 50% of Interior		1

				Non-Structural Elements		
Y			Credit 2	Construction Waste Management		1 to 2
	?		Credit 3	Materials Reuse		1 to 2
Y			Credit 4	Recycled Content		1 to 2
Y			Credit 5	Regional Materials		1 to 2
	?		Credit 6	Rapidly Renewable Materials		1
	?		Credit 7	Certified Wood		1
			Indoor Environmental Quality		Possible Points:	15
Y			Prereq 1	Minimum Indoor Air Quality Performance		0
Y			Prereq 2	Environmental Tobacco Smoke (ETS) Control		0
	?		Credit 1	Outdoor Air Delivery Monitoring		1
	?		Credit 2	Increased Ventilation		1
Y			Credit 3.1	Construction IAQ Management Plan—During Construction		1
Y			Credit 3.2	Construction IAQ Management Plan—Before Occupancy		1
Y			Credit 4.1	Low-Emitting Materials—Adhesives and Sealants		1
Y			Credit 4.2	Low-Emitting Materials—Paints and Coatings		1
Y			Credit 4.3	Low-Emitting Materials—Flooring Systems		1
Y			Credit 4.4	Low-Emitting Materials—Composite Wood and Agrifiber Products		1
Y			Credit 5	Indoor Chemical and Pollutant Source Control		1
Y			Credit 6.1	Controllability of Systems—Lighting		1
Y			Credit 6.2	Controllability of Systems—Thermal Comfort		1
Y			Credit 7.1	Thermal Comfort—Design		1
Y			Credit 7.2	Thermal Comfort—Verification		1
		N	Credit 8.1	Daylight and Views—Daylight		1
		N	Credit 8.2	Daylight and Views—Views		1
			Innovation and Design Process		Possible Points:	6
	?		Credit 1.1	Innovation in Design:Glass Roof		1
	?		Credit 1.2	Innovation in Design: Specific Title		1
	?		Credit 1.3	Innovation in Design: Specific Title		1
	?		Credit 1.4	Innovation in Design: Specific Title		1
	?		Credit 1.5	Innovation in Design: Specific Title		1
Y			Credit 2	LEED Accredited Professional		1
			Regional Priority Credits		Possible Points:	4
Y			Credit 1.1	Regional Priority:7.1		1
Y			Credit 1.2	Regional Priority: 7.2		1
		N	Credit 1.3	Regional Priority: Specific Credit		1
		N	Credit 1.4	Regional Priority: Specific Credit		1
			Total		Possible Points:	110

17 APPENDIX K: MQP PROPOSAL

Dept. of Civil & Env. Engineering

MQP Proposal

Development of 32 Prescott Street at Gateway Park

Michael O'Brien
Jodi-Lee Smith
Ryan Worsman

Submitted to:
Professor Albano
Professor LePage

Abstract

Gateway Park at WPI is a mixed-use complex for life sciences and biotechnology companies. The goal of this MQP is to investigate, design, and analyze a proposed mixed-use development that will be located at Gateway Park WPI. The proposed facility will serve as: office and industrial space for new life science companies, retail space, and graduate or upper-class housing. This MQP will present: a complete building design, a structural analysis, an evaluation of the impact on existing traffic and parking conditions, and a preliminary construction schedule and cost estimate.

Goal

The goal of this MQP is to investigate, design, and analyze a proposed mixed-use development that will serve as: office and industrial space for new life science companies, retail space, and Graduate or Upper-class housing. This MQP will also analyze the impact of the proposed building to the existing traffic, parking conditions.

Introduction

Gateway Park LLC. is a joint effort between Worcester Polytechnic Institute (WPI) and other private profit and non-profit organizations to revitalize the Prescott-Grove Street District, commonly known as Gateway Park. In order to achieve the development goals that align with the City of Worcester and the Gateway Park LLC., the *Gateway Park Master Plan* was written and submitted to Worcester in 2001. More specifically, the *Gateway Park Master Plan* "was commissioned to assess the development potential of the area, based on market and physical characteristics, and to create an achievable vision for the area to guide future development and both public and private investment decisions" (Wallace Floyd Design Group, 2001). *The Gateway Park Master Plan* is a comprehensive long term plan that guides the development of 63 acres including 11 acres now known as Gateway Park at WPI.

Gateway Park at WPI initially began as a collaborative effort between Worcester Polytechnic Institute and the Worcester Business Development Corporation (WBDC). However, in 2010 WPI and WBDC reached a new agreement that stated that WPI will be the exclusive owner of Gateway Park at WPI, with WBDC shifting their role from co-owner to more of “a development role on a consulting basis,” (Worcester Polytechnic Institute, 2010). In order to ensure that WPI growth only serves to “raise the university to new levels of quality and prestige” its development is guided by its *Strategic Plan- New Vision, New Ideas, and New Resources II (“Strategic Plan”).* This document was first written in 1996, and has since been revised twice to account for WPI’s growth and development. Goal seven of the WPI *Strategic Plan* expresses WPI’s desire to "Develop non-traditional sources of revenue as a means of strengthening WPI financially and keeping it affordable” (Worcester Polytechnic Institute, 2008). This desire is the predominant driving force behind the development and expansion Gateway Park at WPI.

WPI aims to develop Gateway Park as “a mixed-use, science-based neighborhood providing opportunities for corporate partnerships and income from rents and ground leases,” (Worcester Polytechnic Institute, 2008). In 2007 WPI completed the construction of its first building-a 125,000 square-foot Life Sciences and Bioengineering Center. On April 21, 2011 O’Connell Development Group broke ground for a new four-story facility that will house a new laboratory, educational, and office spaces for a range of academic and corporate uses. In keeping with goal seven of WPI’s *Strategic Plan* WPI seeks to develop a new mixed-used development at 32 Prescott Street.

One of the constraints to this development is the location of the Millbrook Culvert as it bisects 32 Prescott Street. The culvert must remain easily accessible for maintenance and repairs, and as a result, it cannot be permanently obstructed, thus complicating the design solution for a potential new building or buildings located at 32 Prescott Street. This constraint necessitates a design solution that is cost effective and constructible, yet avoids obstructing the culvert. Although WPI owns the land, it plans to lease it to private life science developers interested in expanding their businesses. The goal of this MQP is to investigate, design, and analyze a proposed mixed-use development that will serve as: office and industrial space for new life science companies, retail space, and Graduate or Upper-class housing. This MQP will also analyze the impact of the proposed building to the existing traffic, parking.

Background

The focus of this MQP is to investigate, design, and analyze a proposed mixed-use development at Gateway Park at WPI. This section shall present information on the history of Gateway Park and 32 Prescott Street.

Transformation of Prescott-Grove Street District to Gateway Park

During the industrial age, vibrant steel mills occupied the area currently known as Gateway Park. This area in Worcester flourished until the late 1950s; eventually production moved to other parts of the world and Worcester was left with many abandoned buildings. Contamination was a problem associated with many of these abandoned sites. Today, within the city of Worcester, there are more than 200 brownfield sites that are documented (Brownfields Success Story, 2009). However, despite this there are less than 100 acres open for development in all of Worcester. In a city where non-developed land is scarce, Gateway Park is a prime location due to its close proximity to WPI, Main Street, Interstate 190 (I-190), and Interstate 290 (I-290). The cleanup process took advantage of two $350,000 loans issued by the Massachusetts Development Finance Agency and $200,000 from a 2005 EPA Brownfields Revolving Loan Fund awarded to the city of Worcester. By 2006, cleanup of the site was completed; the entire site is now ready to be built on, and any contamination levels are below the accepted maximum designated by the EPA (Brownfields Success Story, 2009).

Gateway Park Today

Gateway Park in total is 63 acres. Of the 63 acres, 11 acres are considered Gateway Park at WP; this land is highlighted in Figure 65.

The old Millbrook culvert which runs beneath many of the properties in Gateway Park poses many problems when current construction is considered. The 11- acre site was originally owned by seven different individuals; however Gateway Park, LLC. was able to negotiate and purchase all of this land (The Pheonix Awards, 2007). By March, 2010 WPI took over as the sole owner of Gateway Park at WPI, however the WBDC will still assist in consulting efforts (Cohen, 2010).

Figure 65: 2007 Gateway Park Plan

The primary focus of Gateway Park is bringing life sciences and bioengineering to the area, revitalizing it beyond its former splendor. As stated in a report concerning Gateway Park, “the cleanup of an environmentally blighted and economically stagnant area has opened up a new ‘gateway’ to unite and capitalize on Worcester’s burgeoning life science industry and WPI’s leadership and vision in bioengineering and life sciences” (Carey & Conover, 2007). Cost alone is one factor that will make Gateway Park an asset to bioengineering companies. Rent is less than half that in the Boston/Cambridge area with Worcester offices renting for $20-$35 per sq. ft. near WPI versus $45-$95 near MIT in a recent cost analysis (Facts and Figures, 2011). Worcester boasts thirteen prominent colleges, and five medical facilities, three of which are also schools, such as the UMASS Medical School. These institutions help to fuel the need for more biotechnology and life sciences research and facilities. Prominent companies have already been leasing space at Gateway and with more office space to be built such as that proposed in this report; many top companies will look at Worcester as a destination that is more economical and practicable than Cambridge.

Lot Six of Gateway Park

Lot six is proposed to be one of the last lots in Gateway Park at WPI to be developed. In Figure 3, lots two and three are under development, and the current Gateway Life Sciences building is partially situated on lot two and on the “Newgate Properties” Lot. Lot six abuts Lincoln Street, Concord Street, and Prescott Street in Worcester. The lot also borders the Boston & Maine Corporation’s rail lines which are typically just used for freight trains. The lot's proximity to I-290 also increases its potential value as a location for new businesses, whether offices or retail space. The *Gateway Master Plan* makes several recommendations pertaining to two proposed buildings; Table 38 outlines proposed building requirements.

Table 38: *Gateway Park Master Plan* Proposed Building Designs

Building 1	Building 2
Corner of Prescott and Concord Street	Corner of Lincoln and Concord Street
Development may take place before Lincoln Square is reconfigured	Development may take place before Lincoln Square is reconfigured
Will be visible from I-290	Will be visible from I-290
“Prominent new building” Office space	“Prominent new building” Office space
Research and development	Research and development
20,000 square feet per floor/ 100,000 square feet total	20,000 square feet per floor/ 160,000 square feet total
4-7 floors	8-10 floors
300 parking spaces required	480 parking spaces required
	Parking facility “b” for Gateway Park: 270 spaces below grade

(Wallace Floyd Design Group, 2001)

The 84,062 square foot lot is vacant, and recently grass has been planted to improve the aesthetics of Gateway Park. Currently, the MQP Group is led to believe that the reason there are two separate buildings envisioned for this one lot is to avoid the permanent obstruction of the Millbrook Culvert. The culvert needs to be fully accessible for maintenance purposes. From a site planning perspective this means that there can be neither vertical obstructions for a set height (allowing truck and heavy equipment access) nor also for a certain distance laterally, allowing excavation.

This location was selected as an MQP topic for a variety of reasons. First, this project presents unique challenges due to its proximity to major problematic traffic areas in Worcester. Next, the culvert poses a separate problem which will be investigated, namely by considering one versus two building on lot six. Most importantly, since this project is related to WPI, the group of students felt a connection with working on this project especially knowing that its results could be examined and used by WPI in the future.

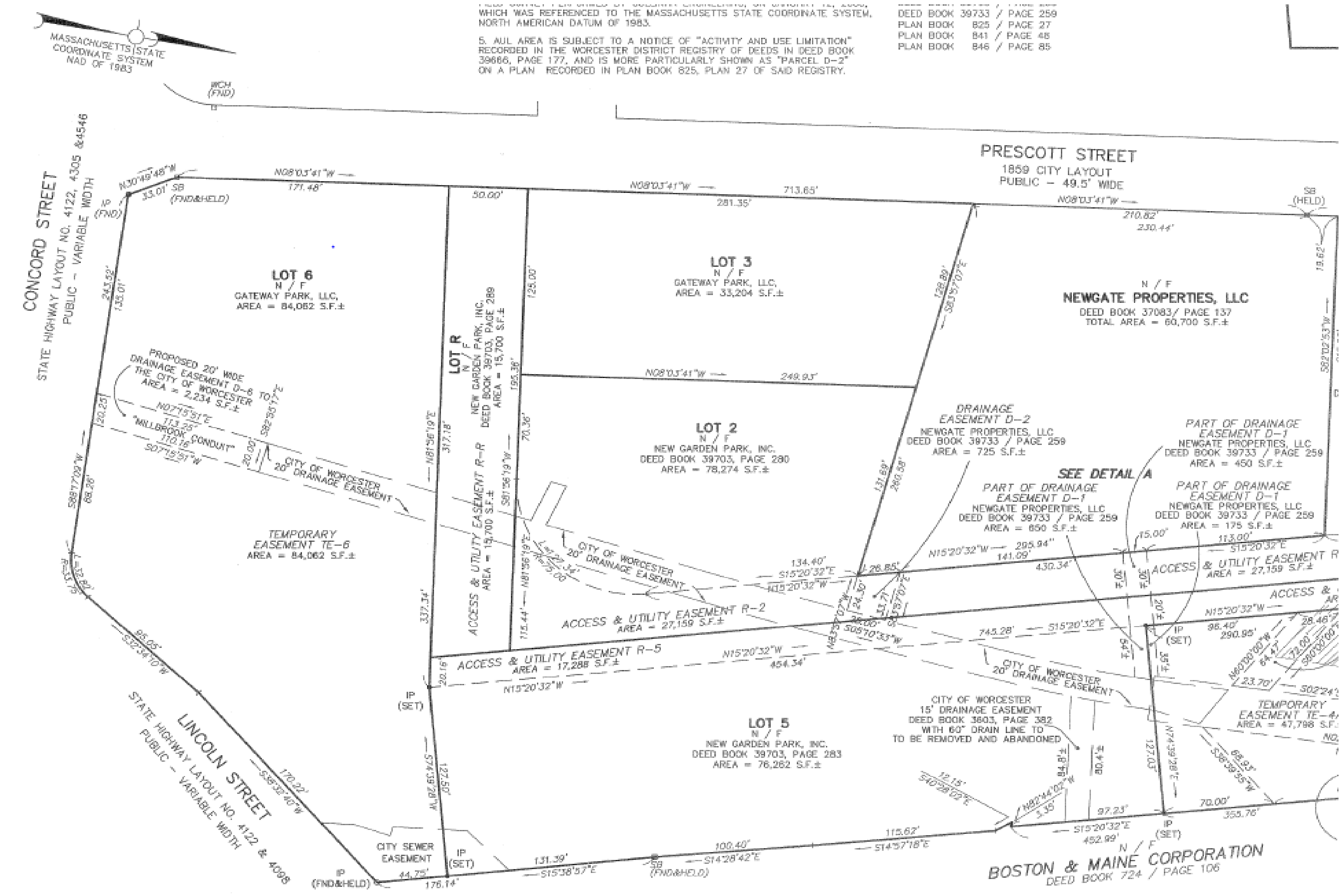


Figure 66: 2006 Gateway Park Parcel Survey
(Engineering, 2006)

Methodology

Goal seven of the WPI *Strategic Plan* expresses WPI's desire to generate revenue from non-traditional sources. To this aim, WPI seeks to develop Gateway Park as a mixed-used life sciences and biotechnology center. This MQP will investigate, design, and analyze a proposed mixed-use development that will serve as: office and industrial space for new life science companies, retail space, and graduate or upper-class housing. Furthermore, this MQP will analyze the impact of the proposed building to the existing traffic and parking conditions. In order to accomplish these goals, the following objectives have to be accomplished:

- Conduct a programming phase
- Construct site plan
- Conduct a preliminary analysis and comparison of design options
- Develop a building layout design
- Develop an engineering design
- Develop a construction schedule and cost estimate
- Conduct a traffic and parking analysis

The proceeding sections will provide a detailed look into how these objectives will be executed.

Programming Phase

The programming phase is designed to break up the structures total square footage into its major parts. In order to complete the space allocations the needs of every intended occupant of the building must be taken into account. For WPI the major needs to be satisfied are more research and development space and graduate student housing. Other companies outside of WPI will also be targeted to occupy the building. The external companies will require both office space and research labs. In order to accommodate all of these building functions careful planning must be used to comfortably cater to all of the parties that will be occupying the building. A great example of this is keeping noise generating uses, such as laboratories, away from residential dwellings or ensuring adequate sound proofing.

Site Planning

A site plan is a critical part to any building project. The Worcester Zoning Ordinance will have to be examined first to determine the required setbacks from streets and other nearby buildings. Parking will need to be examined as well as flow of vehicular traffic and pedestrian traffic from the proposed development to other buildings at Gateway Park and towards WPI campus. Next, once a suitable square footage for a building is determined, the proposed building can be situated on the lot minding the City's ordinances. Furthermore, the use of the buildings will have to be considered, as retail space would need to be visible to people passing by on Concord Street and Lincoln Street.

As part of the site plan, utility design and connection will need to be considered. Using available plans from the City, water, gas, electricity, and sewerage connections will be examined to see where they connect from the street to the proposed development. Furthermore, drainage will be examined from all areas of the site including the roofs, and the parking lots and walkways.

Zoning and Implications

The most recent amendments to The Zoning Ordinance of the City of Worcester went into effect on June 14, 2011. The purpose of the Zoning Ordinance is "to promote the health, safety and general welfare of the public and to contribute to the implementation of the City's ongoing comprehensive planning process" (City of Worcester Zoning Ordinance, 2011). This MQP will follow provision set forth by the Zoning Ordinance to meet the document's purpose.

Development of Conceptual Designs

In order to select a design option that best suits the needs of Gateway Park and the WPI community, two conceptual design alternatives will be analyzed. The criteria used in the preliminary evaluation each alternative are: minimizing impervious surfaces on the site; reduced construction time and cost; keeping the city beautiful by maximizing green space and construction. Conceptual Design A is the construction of two separate buildings, the first on the corner of Prescott Street and Concord Street, and the second on the corner of Concord Street and Lincoln Street. Conceptual Design B is the construction of one building on this lot that will incorporate both of the first two buildings into one design. Each conceptual design will be developed based on site planning and zoning restrictions.

The constraints that the total construction must satisfy are: usages as office, industrial, research and development, as well as residential units. The total construction will be approximately 240,000 square feet and will require a certain amount of parking spaces depending on zoning requirements. This construction will mark the completion of a prominent building seen from I-290 as part of the entrance to Worcester. They will be constructed with red brick and glass façade to enhance street visibility and keep with traditional construction.

Preliminary Evaluation for LEED Certification

LEED Certification "or Leadership in Energy and Environmental Design, is an internationally-recognized green building certification system" (U.S. Green Building Council, 2011). The MQP group will use the LEED point system for new construction and major renovations to assist in determining if conceptual design A or conceptual Design B is more successful in meeting the LEED certification.

Comparison and Selection of Conceptual Design

After two conceptual design alternatives have been developed and analyzed, either Conceptual Design A or Conceptual Design B will be selected. In order to select a design option the pros and cons of each design option shall be evaluated based on the following criteria:

- Time for construction
- Location of culvert
- Aesthetical impact on the Gateway Park at WPI

Building Layout Design

The building layout design of this project is heavily contingent on two aspects. Based on the results from Section 3.1 Comparison and Selection of Conceptual Design and Section 3.4 Programming Phase the layout design can be established. To ensure that the building layout maximizes each of three usages the

MQP team shall utilize *Time Saver Standards* and *Architect's Studio Companion.* In order to develop an efficient structure, multiple building layouts and configurations will be considered. Beyond this, the design will also incorporate a layout that will promote efficient travel through the building or buildings for all its users as well as provide adequate means of egress in the event of an emergency. A great example of this is having the retail space on the first floor exposed to street passersby. The design will also maximize usage of sunlight to reduce the cost of lighting and heating the building.

Engineering Design

The engineering design phase is composed of several tasks such as the design of:

- Structural System
- Exterior Curtain Walls
- Foundations

The following sections will provide more details on how the MQP team will design and evaluate the abovementioned items. Based on the analysis of each item a final engineering design will be selected.

Structural System

The structural system serves to transfer loads between the interconnected structural members of the frame. The effect of gravity loads on a steel frame will be investigated. Two alternative typical bays for the entire building will be designed. In order to design the structure the following tasks shall be executed:

- The determination of structural loads
- The determination of a structural bay size
- The development of an interior framing plan
- The determination of the shape and size of structural members
- An evaluation of a full composite and partial composite beam-and-slab design will be used, a concrete slab on metal decking, a solid concrete slab

The load resistance factor design (LRFD) code will be used to determine the gravity loads and then the ASIC Steel Construction Manual shall be used to assist in the design.

Exterior Curtain Walls

Several curtain walls will be considered, however, only two options will be designed and evaluated, and an analysis on the impact of using different options will be presented. *Fundamentals of Building Construction* by Allen and Iano will be used as a reference text for curtain wall design. This reference will help us to understand how these enclosures are connected to the frame of the building and what load they would put on the frame. The next step involves defining the gravity loads and designing the exterior columns and girders. The load resistance factor design (LRFD) code will be used to determine the gravity loads and then the *ASIC Steel Construction Manual* shall be used to assist in the member design of the exterior columns and girders.

Design of Foundations

The footings shall be designed based on the two frame designs options: a long span and a short span. This will enable the MQP team to determine if a particular frame option incurs larger foundation costs over another. The footing foundations will be designed based on the Gateway Park Geotechnical Report done by the Maguire Group in 2005. This submission will present:

- An analysis of the Gateway Park geotechnical report to establish bearing capacities. More specifically, this involves developing: a soil profile for the site, suitable design soil parameters, and a design chart that will be used to size the footings to support various column loads.
- A design of piers for a column that can support a maximum allowable live load and a maximum allowable dead load

The foundation system analysis will be conducted for the selected design and shall include: column footings, wall footings, foundation walls, and the concrete slab on grade.

Selection of Structural System

The section of the structural system will be based on a combination of three factors:

- Cost of the design based on steel costs ($/lb) and concrete costs ($/cu yd.)
- Usability of floor space based on the location of columns
- Ability to meet LEED criteria

Following the selection of a structural system the structural frame will be designed for lateral loading. Finally, standard connections for the frame will be designed. Final deliverables will include a list of beam sizes, structural drawings for the structural bays, frame and framing connections. Once the abovementioned tasks have been completed the engineering design will be complete and a construction schedule and cost estimate can be developed.

Construction Schedule

The final cost estimate will be organized into a spreadsheet based on the CSI Uniformat divisions list. Furthermore since the project won't commence before 2016 engineering economics shall be used to account for inflation and the time value of money.

Since the construction schedule will be based on a conceptual design, many intricacies of the actual construction will not be accounted for; therefore the schedule will only display major milestones. "Card Tricks" will be used to develop a schedule. The use of card tricks involves using color-coded cards for each trade or discipline. The cards are placed on a large, printed timeline to represent the different stages of the project. Once predecessors and successors have been established, the tasks of the project can be imported into Primavera, a Gantt chart will be created and the critical path of the project will be identified.

Cost Estimate

Constructability and economic feasibility are two important factors that affect a project's development, and execution. To this aim the group will prepare a construction schedule and a preliminary cost estimate. The cost estimate will be developed using both *2011 RS Means Square Foot Cost* and calculated values based on the current cost of steel (per pound) and concrete (per cubic yard) as shown in

Table 39. Using this information, the total estimated cost for the building and site can then be determined.

Table 39: Components of Construction Cost Estimate

2011 RS Means Square Foot Cost	Calculated Design Quantities
Assemblies	**Steel**
Building Construction (Labor)	**Concrete**
Masonry	
Interior	
Mechanical	
Plumbing	
Fire Protection Systems	
HVAC	
Pavement	
Site Work & Landscaping	

Traffic and Parking Analysis

The traffic and parking analysis will be done through three major steps that coincide with the usage of the building. The first step will be figuring out the approximate number of vehicles that this new construction will bring to the area by using the *ITE Trip Generation Handbook.* The MQP team will follow the procedure outlined in Chapter 7.5 Procedure for Estimating Multi-Use Trip Generation of the

ITE Trip Generation Handbook. This procedure will enable the MQP team to find out how many vehicles will be introduced since different occupancy use-groups generate different amounts of traffic. Approximate figures for more thorough analysis of the intersections and roads throughout Gateway Park can then be conducted.

The second step is linked to the previous variable, since there are students in the housing units there may be a significant increase in pedestrian traffic. This increase in pedestrian traffic may necessitate more crosswalks. The design of crosswalks will be established from the use of Chapter 5 of The *MassDOT Project Development Guide* and *The Massachusetts Safety Traffic Toolbox Series* (Mass Highway, 2008).

Finally, once numbers have been compiled, field tests can be run on certain intersections in the area to ensure that they maintain an acceptable Level of Service (LOS) using the computer program *MCTrans: HCS2000*. A few of the intersections surrounding the lot will be chosen to give a brief overview of the expected traffic changes to the area. If recalibration or redesign is necessary it will also be included.

Project Schedule

A project schedule has been developed using Microsoft Project. The project schedule is shown in a Gantt chart and the critical path is highlighted in red. By identifying the critical path the group is recognizing the vital tasks that need to be completed to finish this MQP by the March deadline.

Conclusion

In summary, the motivating force behind this MQP is WPI's desire to continue to further develop Gateway Park as "a mixed-use, science-based neighborhood providing opportunities for corporate partnerships and income from rents and ground leases" (Worcester Polytechnic Institute, 2008). To ensure that the deliverable responds to the needs of Gateway LLC., the development and execution of this project will be guided by the *Gateway Master Plan* and WPI's *Strategic Plan.* The MQP group's overall aim is to develop a structural design, conduct a preliminary cost analysis, provide a construction schedule and conduct traffic and parking analysis.

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