Mixed-Use Academic/Parking Structure Design

A Major Qualifying Project Report

submitted to the Faculty of

WPI

in partial fulfillment of the requirements for the

Degree of Bachelor of Science

by

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Abstract

This project developed architectural plans and a structural design for a new, three-story, steel framed academic building above a three-story concrete parking garage. The building will house the new School of Business and additional on-campus parking. The design included steel schedules for the academic building as well as design drawings and specifications for the concrete members of the parking garage.

Authorship Page

Mark Arnold, Tariq Aziz Azizi, Andrew Mackenzie, and Kyle Warren developed this Major Qualifying Project (MQP) for Worcester Polytechnic Institute for the Bachelor of Science Degree requirement. The four team members worked on all portions of the project and contributed to the conception, design, and final report for the project as shown here. The team used a dynamic system of task delegation in order to complete all necessary parts.

Acknowledgements

First and foremost, we would like to thank our project advisors **Edward Swierz** and **Tahar El-Korchi** for their assistance on this project. They devoted their time and effort providing us with essential information to complete all respective parts of the project. Their guidance allowed us to take away valuable lessons about the design process and was by far the most valuable part of the MQP process.

A special thanks is due to **Professor Albano** who helped answer our questions about the computer software and steel design. We appreciate his sincere concern and time he allotted for us in his busy schedule.

Sincere gratitude is also due to **Yvette Ruttledge** and **Alfredo DiMauro** from the WPI Facilities Department who provided us with pivotal information about WPI's development plan. They also gave us valuable insight into other considerations we should account for when initiating our design. Without their help, we would never have been able to produce a purposeful design for WPI.

Our team appreciates the Associate Dean of WPI's School of Business **Karen A. Hebert-Maccaro** for assisting us develop the layout of the academic building. She articulated all of the needs and desires of the School of Business for their desired building. We integrated all of this information into the design of our mixed-use building.

We would also like to acknowledge **Alford Green** for working with us on this project during the fall semester of 2011. It was an honor working with him and he was a valuable teammate. We wish Alford well in all of his future endeavors.

Finally, we wish to thank **Worcester Polytechnic Institute** for the assets and tools it provides to the student body. We appreciate the opportunity to initiate and complete such a valuable process with engaged faculty.

Executive Summary

Worcester Polytechnic Institute (WPI) has been experiencing rapid growth in recent years. This growth has put a strain on the amount of available academic space and has caused severe on-campus parking shortages. In addition, WPI's School of Business plans to double its undergraduate and graduate student body within the next 5 years. The School of Business currently has its administrative offices in Washburn Labs, while its sub-branches and academic spaces are located all across campus. With the anticipated student population growth and the current lack of one unified space, the School of Business is in need of a building dedicated to their department.

In WPI's 2003 revised Master Plan, the George C. Gordon Library's parking lot on Boynton Street had been identified as part of the Institute's Core Academic and Administrative Campus (CAAC) zone. Therefore, this space would be ideal for a future building to house WPI's School of Business. We believe that a structure designed to integrate both parking and academic space would be aligned with WPI's Master Plan, alleviate the parking problem and satisfy the growth needs of the School of Business. This option would also allow for future development of the George C. Gordon library lot.

We developed five different architectural designs that integrate both a building to house the School of Business and a parking structure. Based on a set of criteria developed to ensure that the needs of the Institute and the School of Business are aligned with the President's strategic plan and the WPI Mater Plan, we selected a final design. The selected final design includes a three-story academic building on top of a three-story parking garage. By combining the academic building and parking garage in one structure, we minimized the required footprint, and freed additional land that could be developed as green space or for other structures in the future.

The academic building was designed as a steel structure. The column layout was selected to maximize office and academic space. We utilized ETABS, the structural engineering software by CIS Computers, to complete a frame analysis on the steel structure. We confirmed the results of the analysis with hand calculation checks according to AISC codes.

Interfacing the steel structure to the concrete structure was one of the main engineering design challenges. Our primary goal was to maximize the number of parking spaces within a reasonable cost. After studying different ways to transfer the steel column axial loads, we decided to place each steel column on top of concrete columns and transfer the loads directly to the foundation. The academic building's first floor is a system of ten foot double-tee beams with an additional four inch slab to support

the loads. Standard double-tee beams were also used for the flooring system throughout the remainder of the parking garage.

We also analyzed the use of different foundation systems in the building. Spread footings were found to be the most cost efficient way to resist the gravity and lateral loads from the columns. A retaining wall was also designed to resist the soil pressure on the exterior walls of the parking garage. Once the structural design was completed, we used Costworks (cost estimation computer software by Reed Construction Data Inc) to estimate the total cost of the building. We calculated the square foot cost to be $307.42 for the academic building and $125.90 for the parking garage.

This MQP report presents a comprehensive design and construction plan for a new School of Business and parking structure. Our team strongly recommends that WPI considers this design as a significant benefit to the entire student body.

Capstone Design Requirement

This project satisfies all relevant Accreditation Board for Engineering and Technology (ABET) requirements. Our team incorporated almost all of our WPI-attained knowledge in the completion of this project. In addition, we conducted intensive research to gain understanding in numerous areas where our prior knowledge was not adequate.

First and foremost, we had to analyze the need for our project. In order to understand the need, we conducted several interviews with relevant WPI staff and faculty to get accurate information. Through this we were able to comprehend what societal impacts our project would have on WPI and the Worcester community. Our team gained a detailed understanding of WPI's needs and desires which allowed us to continue on with the other phases of our project.

We also considered numerous themes concerning environmental and sustainability impacts. As the project would be creating a new structure on WPI campus, we needed to comply with the WPI president's desire to have all new buildings comply with Leadership in Energy and Environmental Design (LEED) requirements. Therefore, we researched LEED in order to successfully incorporate its goals in the design of our mixed-use structure. In addition, we used Geographic Information System (GIS) software to complete a land-use study on our site. We analyzed numerous environmental layers in this software in order to determine the feasibility of developing on the site.

While designing both the steel and concrete structures, we adhered to applicable codes. In order to complete this, we researched numerous building codes and design processes during the conceptual phase. We incorporated this knowledge into our design options. Our team also included numerous safety specifications within the cost estimate to reflect the necessary fire protection technologies which would be included during construction.

In addition, our team also considered manufacturability throughout the design process. As a team we decided to make the design as feasible as possible in terms of constructability. We considered this perspective every time we had to make a decision between different possibilities to allow the steel and concrete fabricators to complete their jobs in an efficient manner. Manufacturability also reflected on the overall cost of the design. We aimed to create an economical option in order for it to be achievable for WPI. Finally, using RSMean's Costworks, we prepared a detailed cost estimate for the entire design. This estimate included estimates for all relevant areas of the design and gave our team an overall idea of the effects different systems have on the cost of structural projects.

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1.0 Introduction

Worcester Polytechnic Institute (WPI) is a private engineering institution located in the city of Worcester, Massachusetts. The Institute offers over 50 undergraduate and graduate degree programs in a variety of departments. WPI's project-based curriculum requires all students to complete at least 3 rigorous projects that involve solving real life problems. Among these, the Innovative Global Perspective Program, which gives students a chance to travel abroad for a project makes the school one of the premiere engineering institutions in the nation. As a result, WPI has seen a tremendous student growth in the past several years. The school's undergraduate population has grown 16.5% in the past decade [1].

The most recent class of 2015 shows just how attractive WPI is to pre-freshmen and college bound students. There was a 7:1 ratio for the number of applications received to the number of spaces available. WPI is currently ranked number one in the nation in part-time MBA by US News, and PayScaler ranks WPI graduates among the highest paid in the nation [1]. While this publicity and growth proves WPI's distinguished academic system, it has also caused severe on-campus parking shortages and has put a strain on the amount of available academic space.

WPI has made efforts to alleviate the parking crisis. The Institute made the decision to construct parking structures with the new East Hall and Gateway Park buildings [2]. However, at least two hundred more parking spaces are needed to bring the number to acceptable levels[1]. Although it is not explicitly outlined where parking structures are to be constructed, the Institute's Master Plan of development reveals that there is a need for a new parking structure [3].

In addition to growth in other departments, WPI's School of Business plans to double its undergraduate and graduate student body within the next 3-5 years [4]. Currently the School of Business has 41 full time employees and staff who teach approximately 1000 graduate students every year. The School of Business's administrative office is currently in Washburn Labs while its

[1] A Typical university requires parking for 10-50% of their student body. Assumed WPI needed 20% parking based on large population near campus (.20*4954 undergrad and grad students=990 spaces). Based on WPI 2009-2010 Annual Report, assumed 80% of faculty need spaces. (.80*1030 faculty and staff=825). Total number of spaces predicted to be (990+825=**1815 spaces**).

sub-branches and academic spaces are located all across campus. The School of Business is already facing challenges in providing academic spaces for its current student body. Although the majority of graduate business classes occur in the evenings and the weekends when classroom demand is low, the increase of about 135 students each year has already placed a strain on the department's resources [4].

The 2003 revised Master Plan details the expected population size and proposed lot-usage for WPI's Campus [3]. The George C. Gordon Library's parking lot on Boynton Street has been identified as part of the Institute's Core Academic and Administrative Campus (CAAC) zone and has been set aside to host a future building for WPI's Business School. From preliminary analysis we concluded that a structure designed for parking and academic space for the School of Business would be in line with the Master Plan, and would also reduce the parking issue and keep the library's lot within the CAAC for further development.

The following sections of this report fully outline the issues and other considerations in order for the proposed design to be considered successful. First, a detailed background presents all pertinent information and research used in the creation of the design. Next, a comprehensive methodology outlines the design process and traces the steps taken to complete all necessary sections. These sections are followed by a proposed design and a preliminary structural cost estimation of the project, which WPI can use to further its development. Finally, this report recommends certain steps and several other considerations WPI should recognize to successfully implement the project.

2.0 Literature Review

In order to clearly understand all driving forces for this project, certain research was conducted. The first step was to define the need and purpose of the project. Once our team established the parameters, we conducted technical research by consolidating the project's criteria and brainstorming necessary additional information. We researched all applicable topics within our determined scope in order to optimize the design process. This literature review is divided into four sections: Background, Project Specifications, Architectural Design, and Cost Estimation.

2.1 Background

In order to successfully design any building, the designers must understand the needs, impacts, and constraints of any given design proposal. As Albert Einstein stated, "If we knew what it was we were doing, it would not be called research, would it?" [5]. In order to achieve an adequate level of knowledge and understanding of the client's needs, the team conducted extensive research and professional interviews to gather vital information. We systematically outlined relevant information and began using several different avenues and media to find what we needed.

2.1.1 WPI's Vision

All projects come from a need envisioned by the Owner. This project's overall goal is to satisfy as many of WPI's goals as possible in the most efficient way. According to the president's office and our interview with President Berkey, WPI has already met its goal of reaching an undergraduate enrollment of 3600, but WPI still aims to increase the graduate enrolment from 1000 to 1600 students by the year 2015 [2]. President Berkey is seeking to establish WPI as a global leader in technological education and to develop its position as a national university [2]. WPI's President's vision statement also includes developing campus facilities according to an explicit plan for supporting academic and co-curricular needs, property maintenance, improving alumni relations, developing non-traditional sources of revenue to strengthen the Institute financially and keeping WPI affordable for prospective students [2] [6].

Although the goals stated above may not directly affect this project, it creates the need for the project to occur. Increasing the number of students will cause an increase in the demand for additional academic facilities and on-campus parking. In addition, if WPI is going to play a role in advanceing technological education and increase its prestige as a nationally renowned Institute, it must be able to provide the adequate academic space for its students.

On an ambitious plan to raise the Institute to new levels of quality and prestige, WPI has constructed and renovated several new academic and residential buildings in the past eight years. Examples of these projects are the Bartlett Center, East Hall residential building and parking garage, Gateway Park academic space and parking garage, and the new Recreational Area on the west side of the campus which will open in Fall 2012 [2]. Because of the 2007 vision, all the new buildings listed above are to be environmentally sensitive and are LEED-certified structures[2] [7]. In addition to these improvements, President Berkey also desires to renovate Kaven Hall, Stratton Hall, Alumni Gym and the Gateway project center as well as renovate and increase the number of residence halls so that they can accommodate 70% of the undergraduate student body [2]. There is also an ongoing plan for an underground parking facility under the softball field on the west side of campus that will hold about 550 vehicles [8].

The decision to build the new parking garage under the surface of the softball field may help the current parking issues. However, after talking with the facilities office, we believe that there will be a continued need for parking spaces on campus. Seeing as how WPI already has over 3,600 undergraduate students, which is over the expected maximum number of students, our team estimates that there is a demand for approximately 1,815 on-campus parking spaces[3]. Adding the new parking garage under the softball field brings the total available parking spaces to 1,200. Our team believes that having the additional 220 parking spaces under the proposed School of Business will ease the parking needs on the east side of the campus.

2.1.2 WPI Master Plan

To gain a better understanding of what WPI's needs were, our group met with Mr. Alfredo DiMauro, WPI's Assistant Vice President of Facilities. According to Mr. DiMauro, the WPI facilities

[2] *In February 2007, WPI's Board of Trustees voted to adopt a policy calling for all future buildings on campus to be environmentally friendly and LEED-certified structures.

[3] Typical university requires parking for 10-50% of their student body. Assumed WPI needed 20% parking based on large population near campus (.20*4954 undergrad and grad students=990 spaces). Based on WPI 2009-2010 Annual Report, assumed 80% of faculty need spaces. (.80*1030 faculty and staff=825). Total number of spaces predicted to be (990+825=**1815 spaces**).

department and President Berkey developed a master plan from 2003 to 2005 [8]. This plan shows how WPI would like to expand and explains the intended use for the land that WPI owns. A key feature of the plan outlines the intended layout of parking around campus. The Master Plan seeks to eliminate all faculty and student parking from around academic buildings in the center of campus [3]. However, several exceptions exist to this parking plan. WPI plans to keep several spaces available for visitors, deliveries and handicapped needs. This new plan would create a much more pedestrian friendly campus, adding to the overall WPI experience.

Because this project will have a major impact on WPI's campus, our team sought to understand WPI's future vision of its campus. Mr. DiMauro stated that WPI is seeking to mold certain areas for specific purposes. WPI is trying to keep all academic and administrative buildings within the area surrounded by Boynton Street, Institute Road, Park Avenue and Salisbury Street. Furthermore, WPI aims to use Olin Hall, Higgins Laboratories and Alden Memorial as secondary boundaries within this area [8]. This important requirement directly impacts the intended use of the Library parking lot area, which will be the focus of our project.

Mr. DiMauro also reminded us that this project should not affect any future plans for WPI or create an obstacle to any plans that haven't been thought of yet. He pointed out that the Civil Engineering Department has out-grown Kaven Hall and that a building built on the northern end of the library parking lot could interfere with WPI's possible expansion of Kaven [8]. The Master Plan demonstrated that the project must have a specific goal and task, but should also consider the future for how our design will affect WPI's future ambitions.

Facilities also mentioned WPI's goal for making the campus "greener" which will be discussed in a later section [7]. Mr. DiMauro also pointed out that the Art Walk which runs east to west between East Hall and Founders Hall should be continued across Boynton Street [8]. The Art Walk is a premeditated plan created by Dr. Berkey and the Facilities Office to create an aesthetic walking route from the WPI Gordon Library to the Worcester Art Museum on Lancaster Street. He also stated that it would be nice to have a covered walkway to the top of the hill to provide a safe passageway for pedestrians in the winter, especially with the snow and ice on the walkways. Our team inquired about the access road that comes down from Boynton Hall. In response, he said it is part of the fire access to campus, and it must be either kept in its current location or rerouted before being altered. This is an important issue as it will impact our project's design location [8].

In conclusion Mr. DiMauro gave our team access to blueprints of East Hall and its parking garage. The meeting with Mr. DiMauro was a great success for our team as we did learn several of WPI's goals which include:

1. Keep library parking lot an academic area
2. School of Business building in library parking lot
3. Increase parking capacity of the east side of campus
4. Continue Art Walk
5. Keep access road from Boynton Hall to Boynton St.
6. All new buildings must be LEED certified on campus
7. Plan for future projects

In addition to WPI's vision, our team also investigated the specific needs for WPI's School of Business. Our group met with Ms. Karen Hebert-Maccaro who is the Associate Dean of the School of Business at WPI. From this detailed interview we uncovered the following general goals of the School of Business [4]:

1. Acquire a single building dedicated to the School of Business
2. Double graduate student population in 3-5 years
3. Total of 33 full-time faculty in 3-5 years with personal office space
4. Increase department staff to 7 members in 3-5 years
5. One classroom for 70 students
6. Four classrooms for 30-50 students
7. One computer lab or classroom designed for laptop use.
8. Six small (12-person capacity) conference rooms
9. Location: Dean's choice is near Gateway Park

All of these points will be taken into consideration in our design to accommodate the School of Business' needs.

2.1.3 Project Purpose

The project seeks to provide a response to two of WPI's needs: 1) increase campus's parking capacity and 2) create an academic building for the School of Business at WPI. The first part of our project seeks to create approximately 220 spaces in a parking garage footprint that is half the size of the current parking lot. The second part of our project will create an academic building for the School of Business above the parking garage, thus optimizing the use of the space.

The relatively small physical footprint of the building allows for a green space on the east side of campus by making the current parking lot behind the library into a field for outdoor activities. This would fulfill WPI's goal of creating more green space and give students on the east side of campus a place for outdoor recreation. Keeping the School of Business on campus will also give it a more campus-like feel as compared to Gateway, which is a business park. Freeing up the current library parking lot area also allows for the expansion of Kaven Hall in the future. The same is true for new construction that WPI may need in the next century. The green space will offer numerous uses and options to WPI in both the short and long term. With every aspect of this project, we aimed to create the most beneficial proposal for the re-development of the Gordon Library parking lot.

2.2 Project Specifications for Architectural Design

The proposed building will have a number of features, which meet the needs of the expanding Business School and WPI's Master Plan. The minimum required features of the academic section include:

1. Six small (12-person capacity) conference rooms
2. Five (50-person capacity) classrooms
3. Two (70-person capacity) lecture halls
4. One computer lab (30 stations)
5. Thirty three faculty offices
6. Seven staff offices and cubicles

The Master Plan suggests that 400 parking spaces be created and requires that any new building be LEED certified [3] [6]. This building aims to meet both of these requirements. Our group aims to achieve LEED-NC (New Construction) Platinum certification by having the following features:

- Solar-powered lighting
- Green roof
- Sub-grade rainwater storage
- Ten percent preferred parking spaces for hybrid vehicles
- Hybrid plug-in stations

- Five percent parking spaces for carpool vehicles
- Bicycle spaces
- Compact car spaces
- Handicapped-accessible parking spaces
- Replacement of impervious surfaces with green spaces

2.2.1 Project Site Evaluation

WPI has a rather uniquely configured campus because of Boynton Hill. The proposed area of our project is located on the southeast corner of WPI's main campus area (area surrounded by Park Avenue, Salisbury Street, Boynton Street, and Institute Road). Figure 1 shows the proposed area with respect to the rest of WPI campus.

Figure 1: Site Location [9]

The site runs from the corner of the Boynton Street-Institute Road intersection to the midpoint of Boynton Hill on the south side of the WPI library. The site area is approximately 300' x 400'. The 400' dimension would run adjacent to Boynton Street. It is important to note that these are not the proposed building dimensions. In addition, this structure will require substantial excavation which will pose a logistical problem during the construction phase. The construction will have to designate an area for all of the excavated soil. In Figure 1, Boynton Hill increases in elevation from East to West. Therefore, the excavation contractors will have to remove large amounts of soil from the western side of the construction site to make room for the parking garage.

Another obstacle of construction will be the preservation of the WPI emergency access road, Skull Tomb, and Coombs walkway. The access road is essential for WPI and it must be preserved for fire safety and emergency reasons. With our current projected site area, the construction will obstruct the access road in possibly two areas. Therefore, we may have to consider redirecting the access road to the northwest before beginning construction. Another consideration we must make is the location of the Skull Tomb. The Skull Society has existed since 1911 and is heavily ingratiated in WPI folklore [10]. The tomb is the location where the Skulls meet. Therefore it is essential that we properly consider the Skull Tomb. Our site includes the Skull Tomb in the south most region of our site so it may be necessary to adjust the building's perimeter. Finally, the Coombs Walkway up the southern side of Boynton Hill provides students from Institute and Founders Halls with a route to campus. We took this into consideration with our site plan as it lies on the southern edge of our perimeter.

Another major issue with our site is the effect it will have on traffic. Institute Road and Boynton Street are pivotal access roads to WPI. Many faculty and students travel and park on these streets and any construction will affect the traffic and parking on those approaches. Our design will have to account for these interruptions, and we will have to make a concerted effort to minimize its negative impacts on the community.

2.2.2 Geotechnical studies

In order to design the foundation, it is important to analyze the soil conditions of the site. Since the geotechnical report for the current site is not available, the team had to rely on the geotechnical reports of the nearby buildings. The existing soil conditions of the site are extrapolated from 1998 soil tests, 2007 East Hall and Recreational Center Geotechnical Investigations. During the 1998 test 10 borings were taken by Haley & Aldrich INC. [11]. The locations of the boring are shown in Figure 2. The parking lot is at an Elevation of 502 $\pm$ 2 ft above sea level [11].

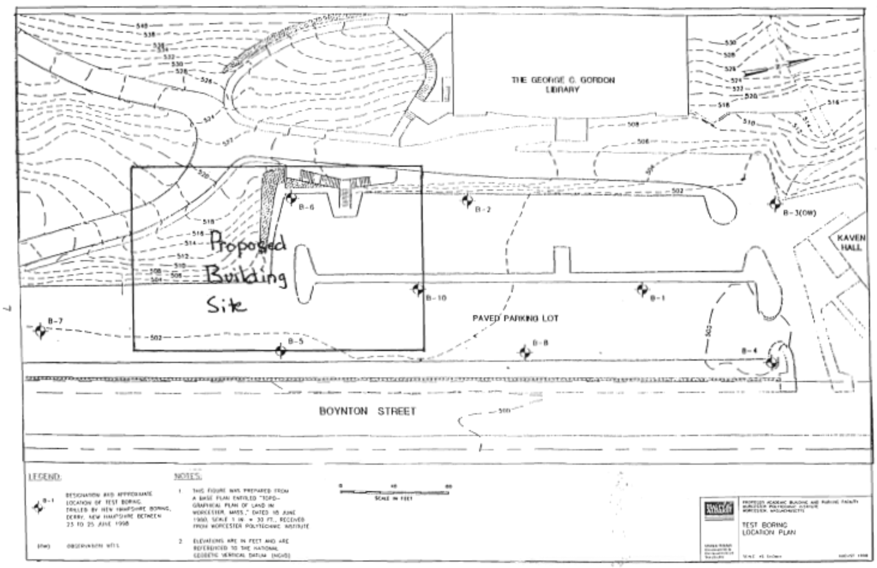


Figure 2: Soil Boring Site Locations [11]

The results of the soil boring investigation showed that the site consists of glaciofluvial and glaciolacustrine deposits which come from glaciers washing the material into lakes. Haley & Aldrich identified that the glaciofluvial deposits consisted of “medium dense to very dense light brown to olive grey coarse to fine sand with varying amounts of silt, coarse to fine gravel, cobbles, and boulders." The report stated the glaciolacustrine consisted of “loose to medium dense yellowish orange to tan salty fine sand to fine sandy silt with varying amounts of medium to coarse sand and fine gravel” [11].

In 2007 Haley & Aldrich Inc. conducted a Geotechnical Design Investigation diagonally across from the proposed site for the East Hall Dormitories as well as at the proposed Recreational Center site that is now in construction. Without the ability to conduct an individual geotechnical report at the precise site location, our team must rely on cross-referencing the reports to establish the most accurate portrait of the site. The East Hall report was weighted more heavily as it resides closer to the site than the Recreational Center. However, both reports found glaciofluvial and glaciolacustrine deposits along with glacial till and fill. Below is a summary of the important aspects that are relevant to this project. The East Hall Geotechnical report is included in Appendix A [12].

1. Ground Water 13.8 ft below EL. 488.7 however this may vary slightly.
2. Maximum Net allowable bearing pressure of 4.0 ksf.

3. If footing width is less than 3 ft, maximum allowable bearing pressure should be reduced by 1/3.
4. Footings must be at least 18 inches wide.
5. Bottom of footing should be positioned at least 4ft below adjacent ground or slab surface exposed to freezing.
6. Minimum 4 in thickness of compacted Granular Fill provided between footing and the bottom of slabs.
7. Seismic design: the recommend soil profile type is S3 with a site coefficient equal to 1.5.
8. Perimeter foundation drain system should consist of:
    a. A perforated drain pipe positioned at the exterior base of the foundation wall.
    b. Well-drained backfill against the below grade walls and below upper slab levels.
    c. Water stops at construction joints.
    d. Damp proofing of the exterior of walls.
9. Under slab drainage system recommended were slab level is 8 ft or deeper than the adjacent exterior ground surface. (See full report for complete details)
10. No confirmed bedrock was encountered in test borings, although refusal was encountered at several locations.
11. No liquefaction expected.

2.2.3 Sustainable Development

Sustainable development has many definitions and it is not exclusively an environmental concept. What is sustainable for one group of people may not be sustainable for another. For some people the idea of sustainable life can be living longer, maintaining well-being or, it can just be the capacity to endure. Sustainable development focuses on the triple bottom line: economic, environmental, and societal. A general definition of sustainable development can be "the development that meets the need of the present without compromising the ability of future generations to meet their personal needs" [13]. While the definition above is internationally accepted, the reaction to it is different in every region and good practices for sustainable construction may not be the same for all societies around the world. Therefore, the developed rating systems and techniques used will differ.

In an effort to encourage sustainable ideas in the United States, the topics of sustainable construction and green buildings have grown in popularity. Sustainable construction can be any type of construction (office buildings, railroads, highways, etc.) whereas green buildings focus on improving vertical constructions. Energy efficiency is a high priority in many countries, especially cold countries like the United States. Consuming less energy greatly affects the economy as well as the environment. In order to improve energy efficiency in vertical buildings, the US Green Building Council (USGBC) developed the Leadership in Energy and Environmental Design (LEED) in 1998 [14]. It is intended to provide building owners and operators a strategic outline on how to design, construct, operate and maintain practical and measurable green buildings. Since its start LEED Projects have grown substantially and as of October 6th 2011 there are about 35,000 participating projects and over 1.5 billion square feet of LEED Certified Commercial Buildings in 90 countries. As of 2011 many familiar companies and institutions require their new buildings to be LEED-Certified [14].

In February 2007, WPI's Board of Trustees voted to adopt a policy calling for all future buildings on campus to be environmentally friendly and LEED-certified structure projects [6]. As a result of this policy there are currently two Gold-level LEED-Certified Residential and Office Buildings, as well as two other to-be LEED-Certified Buildings currently under construction. Since WPI requires all its new buildings to be LEED-Certified, we decided the new parking garage and academic building must be designed to be LEED-Certified Platinum. The reasons for our decision are mentioned below.

2.2.3.1 LEED-NC 2009

LEED is an internationally recognized green building certification system. LEED standards provide strategies and techniques intended to improve performances such as energy savings, water efficiency, CO_2 emission reduction, improved indoor environmental quality, and stewardship of resources and sensitivity to their impacts. While LEED-Certification system is a part of USGBC's "Green Building Certification Institution" (GBCI) it still operates independently in order to deliver ethical and unbiased conclusions [14].

2.2.3.1.1 LEED-NC 2009 Rating System

The LEED-NC 2009 rating system is the latest system delivered by USGBC. It covers most commercial, institutional and industrial projects and includes residential construction of facilities of four or more stories. In LEED 2009 there are 100 possible base points plus an additional 10 points for two other categories. The base points are distributed between the five main categories such as, Sustainable Sites, Energy and Atmosphere, Materials and Resources and Indoor Environmental Quality, while the additional points are for Regional Priorities and Innovative design. None of the credits are mandatory besides the 8 prerequisites in LEED 2009. The credits are worth a certain number of points and a combination of credit points add up to a certain level of Certification. Under the LEED-NC 2009 system commercial buildings can receive 4 different levels of certification (i.e. Platinum, Gold, Silver and Certified) [14]. Table 1 shows a summary of certification level, total prerequisites, and points.

Table 1: A Summary of LEED Certification Level [14]

	LEED-NC 2009
Prerequisites	8
Base Points from Main Credit Subcategories	100
Innovation in Design Points	6
Regional Priority Points	4
Total	110
Points for Certified	40–49
Points for Certified Silver	50–59
Points for Certified Gold	60–79
Points for Certified Platinum	80+

2.2.3.2 LEED-NC 2009 Registration

The LEED-NC 2009 Registration Documents and forms are all online at www.GBCI.org. The process to register a new construction project is straight forward and it is designed in a visually interactive way to ease the process for users. The registration of a new project costs USGBC members a flat fee of $900.00 and $1200.00 for non-members [15]. After registering a project an application and examination process follow, and certification is awarded after that process is completed. To achieve any sort of certification through the LEED-NC program, the building must be looked over by a LEED accredited professional and a LEED green associate [15].

2.2.3.3 Green Roofs

Green roofs provide an environmentally friendly and aesthetic option to increase the sustainability of any design. The two main types of green roofs are intensive (deep) and extensive (shallow).

Intensive roofs range from 6 inches to 15 inches thick, and they can host a wider range of flora. Thickness of green roofs usually comes from the structural capacity of a building as well as the owner's budget. These intensive roofs are typically designed to be more interactive and beneficial to the user. A downside to intensive designs also includes the increased maintenance cost, due to the larger flora which requires special care and treatment on a regular basis. Another constraint is the restriction of roof slopes as intensive green roof systems can only be installed on slopes up to 3% [16].

On the other hand, extensive roofs are lighter and more economical, but are limited in the types of plants that can be supported. Extensive roofs are typically 2 or 3 inches thick and are host to various types of grasses and flowers. These roofs typically have fewer layers than intensive roofs which contribute to their lower weights. This results in an average imposed weight of 17 pounds per square foot. Another interesting aspect of these roofs is their ability to be constructed on slopes of up to 33%. Due to the restricted selection of flora, extensive roofs require less maintenance than intensive roofs after installation [16].

2.2.4 Selected Materials

There are a number of different materials that could be used to design the specified building. We decided to design the academic building using a steel frame and the parking garage using prestressed and reinforced concrete members. Other materials used throughout the building include glass, brick, and aluminum.

The parking structure roof will be made of prestressed double-tees with a concrete slab to withstand the loads of the first floor of the academic building. This method makes use of the concrete's high compressive strength and the steel's high tensile strength in reducing the slab's thickness and deflection. These characteristics allow for increased spans between supports as compared to standard reinforced concrete. The floors of the parking garage will be composed of double-tee beams which will be prefabricated. A benefit of concrete is that it requires about half as much energy to produce per unit weight as hot rolled steel beams which is a benefit to the environment [17].

We selected hot rolled steel beams as the primary framing material for the office building structure above the parking garage. The decision was made according to LEED requirements, architectural design of the building, ease of construction, weight of the material and material cost. The steel will be faster to construct than concrete, allowing for the overall construction process to increase productivity once the concrete garage is completed. In addition, using steel will allow the academic building to be more flexible in its layouts. WPI's Boynton Hall was constructed in 1868 and is still being used as an administrative building [1]. Therefore, this new academic building will fill various uses throughout its existence. A steel frame allows the building to be more easily modified in order to change the layout in the distant future.

2.3 Architectural Design

The final design will be developed to accommodate WPI's request for educational needs, parking space and green space. Since we are proposing the building per WPI's vision and there was no prior architectural program or design, the team drafted preliminary architectural designs based on an iterative process with feedback from our client. Our current design (3 story office building on top of a 3 story parking garage) was selected per the School of Business's needs; the community's parking demand and the overall view of the campus. The next section of the paper will discuss our selection procedure and our other alternative options.

2.3.1 Preliminary Design Selection

Although our design team was presented with certain criteria and constraints, we aimed to maintain a conceptual "tabula rasa" perspective. Through brainstorming sessions we developed conceptual designs that were refined after consultations with our client. These preliminary designs not only aimed to satisfy the needs of the School of Business or WPI's Master Plan. Below are the common objectives our team wanted to include in the preliminary designs.

1. Successfully blend into the brick-dominated surrounding area (especially with height)
2. Present a slightly modern edge to distinguish itself from the rest of WPI

3. Minimum environmental impact and footprint
4. Feasible access to pedestrian and vehicular entrances
5. Minimum impact on traffic on Institute Road and Boynton Street
6. Increase pedestrian safety around the Institute/Boynton intersection with increased lighting

These architectural preferences, coupled with the School of Business's needs, provided the impetus and guidance to create five preliminary options.

The first design outlined a three story parking garage, with one level being below the current grade of the site. The parking garage would have a green space/recreational area on the roof. A benefit of this design was an increase in the number of parking spaces to approximately 400. This design would also not have required extensive excavation involved with construction. However, disadvantages included the loss of current parking spaces during construction from the Gordon Library parking lot. The basic floor plan of the parking garage is shown in Figure 3. The left portion of the figure shows what the building would look like from the street (east) and the right portion shows what the building would look like from Kaven Hall (north).

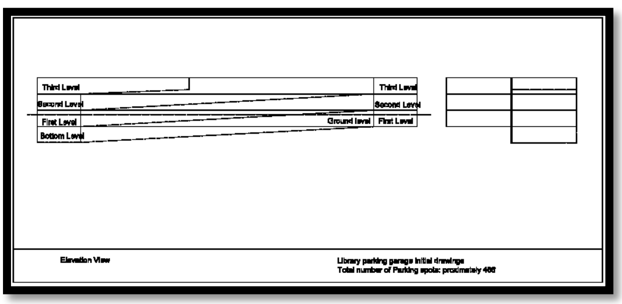


Figure 3: Large Parking Garage Design Option

Another design option included a residential building with an underground parking garage. The residential hall would have the same architectural design as East Hall and because East Hall is already LEED certified this would have simplified the design process. We considered this design option to provide more housing for its students. However, after studying the WPI's Master Plan and interviewing Mr. DiMauro and Ms. Hebert-Maccaro, our team realized that the space is designated for an academic building [8]. A downside to this design option was that it did not increase the number of parking spaces provided behind the library. The garage would only hold approximately the same number of spaces as the current library parking lot (approximately 220).

There was also a lot of excavation included with this design as the entire parking garage would be subgrade. Figure 4 shows the design from the street view. Another drawback to the design was designing ramps to get down to the parking because of the width of the current parking lot and the space available to us. We considered changing the use of the building from residential to academic, however finally decided to scrap the sketch due to the excessive land use.

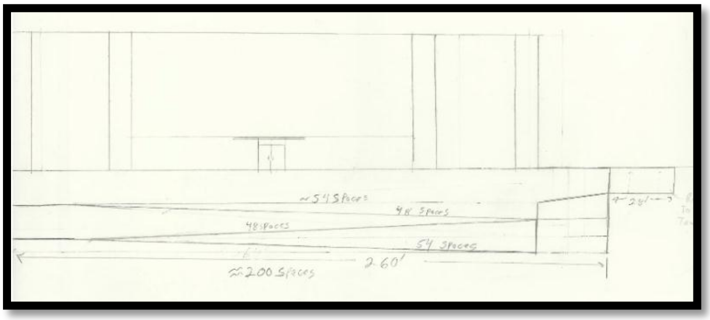


Figure 4: East Hall with Below Grade Parking Garage Option

Our third design option was to have an office building located next to a parking garage. The advantage of this option was that having two separate building would make the design process easier. We would not have to account for any vertical transfer loads between buildings as we would have to if the academic building was placed on top of the garage. However, this design would increase the footprint of the buildings, taking away precious land from WPI's future developments. Approximately 24,600 square feet of land would be used therefore providing less green space for the library parking lot area. Figure 5 shows that the parking structure is on the right side of the building when looking from the street because the access road runs adjacent to the west. Some pros for this option would be that the number of parking spaces would be increased and that there would be a short, protected walk from the parking garage to the academic building. However, the office building would be smaller than if we built it on top of the parking garage.

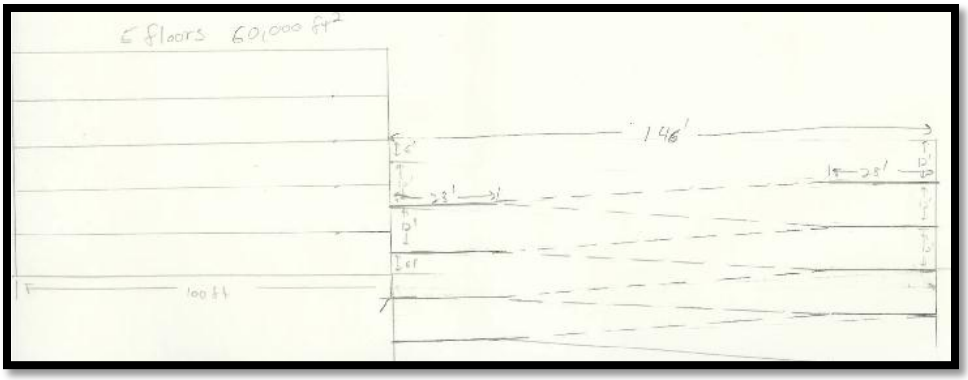


Figure 5: Two Separate Buildings Option

Our fourth design option was to have underground parking with recreational area on top next to an office/academic building. All parking would be below grade however the roof of the parking garage is fifteen feet above ground level so that cars can get into the parking garage as shown in Figure 6. The benefit for this option would be its recreational area. The Boynton Street side of campus would greatly benefit from this space. However, as with the third option, there is no increase in the number of parking spaces.

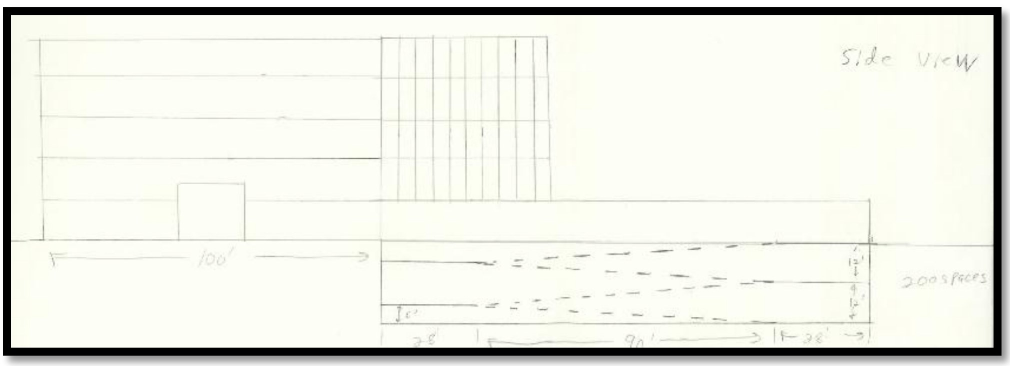


Figure 6: Underground Parking Garage and Green Roof with Adjacent Academic Building Option

After meeting with President Berkey, the Associate Dean of the School of Business and the facilities manager, the fifth and final design option was chosen. This design includes a three story parking garage consisting of one level below grade and two levels above grade. An academic building would be located directly on top of this concrete parking garage. This option was selected according to the WPI School of Business's current needs. Figure 7 shows the front of the building as seen on Boynton Street. Stairs going up the front of the parking garage up to the office building will provide access to the front doors. The initial architectural layouts of the academic building and parking structure are shown in Figure 8, 9, 10, 11, 12, and 13.

Figure 7: Three Story Academic Building on Top of Three Story Parking Garage Final Design Option

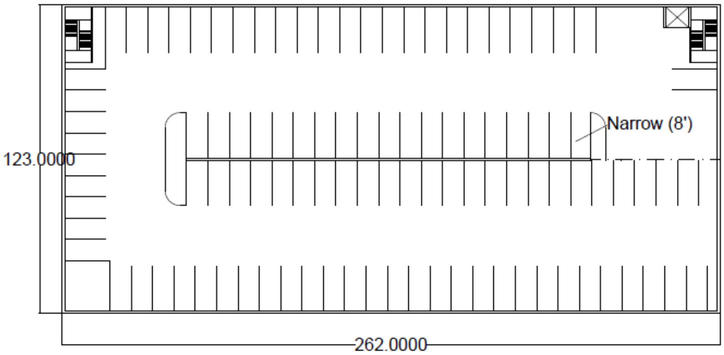


Figure 8: Basement Level of Parking.

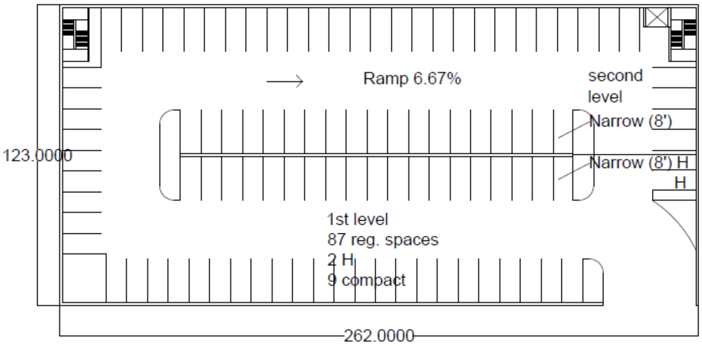


Figure 9: First Level of Parking

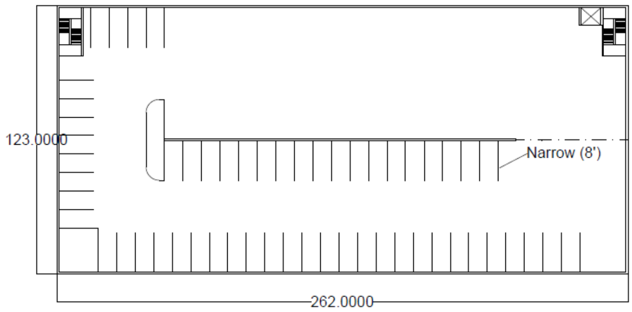


Figure 10: Second Level of Parking

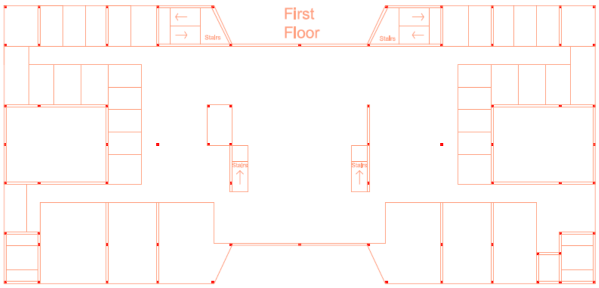


Figure 11: First Floor Tentative Layout Option

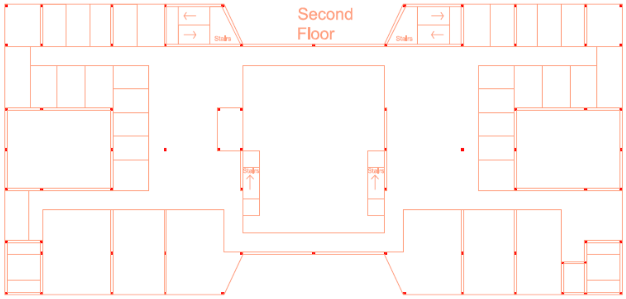


Figure 12: Second Floor Tentative Layout Option

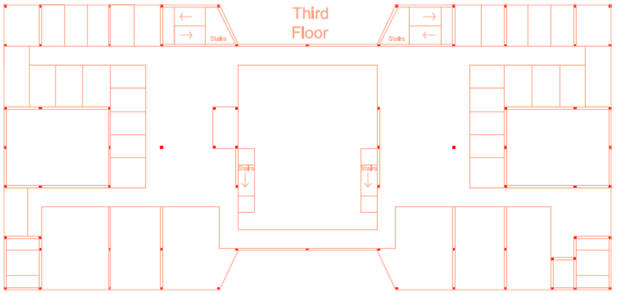


Figure 13: Third Floor Tentative Layout Option

2.3.2 Parking Garage Specifications

The parking garage will consist of three levels with a footprint of 123' X 262'. It will include parking for approximately 220 cars, which includes 7 handicap spaces and 12 compact car spaces. The length of the garage allows the design to have ramps only on one side of the garage so a large portion of the parking will not be sloped. All of the parking spaces in the garage will be perpendicular spaces (90°) to provide parking to people traveling in both directions around the parking garage.

The parking space layout for the garage is shown in Figures 8-10. Figures 8-10 also show the space left for a staircase in two corners and elevator in the upper right corner of the drawing. The handicap parking will be located right next to the elevator.

2.3.3 Academic Building Specifications

The building above the parking garage is intended to be used by the WPI School of Business. The building will provide faculty and staff offices as well as classrooms for graduate and undergraduate level classes. As stated by the Associate Dean of the Department, Ms. Karen Hebert-Maccaro, a new building will centralize the department by making it more efficient and productive [4].

The building's footprint will be 123' X 262' excluding the 60' X 40' outdoor sitting area as shown in Figure 11-13. There is also an entrance to the building on the west side so that people will be able to enter through this door from Boynton Hill.

2.4 Cost Estimation

Project cost is always a driving factor on a construction project; the pre-design cost of a project depends on the needs of the owner, the specification and complexity of the site, height of the building and much more. The field of project management typically deals with the scheduling and economic analysis of proposed structural designs. In this field, there are several different methods of completing a project.

Design-build projects consist of a bidding process which creates a unified build team. A design-build contractor is the person who is in charge of hiring contractors instead of completing separate bids. This process requires the general design-build contractor and the owner to be on the same page throughout the process [19].

Fast tracked projects have the benefit of compressing the typical duration of projects into shorter periods of time. Making decisive decisions and well-established lines of communication are essential in order for fast tracked projects to be successful. Different sides of the design phase often occur simultaneously whereas in a typical design-build process the process would prove to be more systematical [20].

3.0 Methodology

Structural design practice is controlled by various codes, regulations, architectural program and budget. This chapter presents the methods used throughout the structural design phase of this project.

3.1 Land Use and Development

Before starting the structural design, the proposed land was analyzed to determine the feasibility of development on the site. The team used the Geographic Information System (GIS) and ArcMap 10 computer software to generate numerous physical and climate-related constraints. We looked at these several layers both overlapped and set individually to ensure the architectural plan did not violate city or state codes. Massachusetts town boundaries were used as the base map layer and then other pertinent layers were added to evaluate their impacts on the proposed architectural plan. The examined layers are listed below in order of precedence:

1. EOT major roads
2. Roads
3. Wetlands
4. Flood plains
5. Early postglacial layer
6. Postglacial layer
7. Stratified deposits
8. Till bedrock
9. Environmental justice areas
10. Town boundaries

Our team's primary focus was on hydrological, topographical, and ecological layers. As the team already had a rather specific lot area, we did not need to concern ourselves with major roads. We added environmental justice areas as a layer to ensure there were no special species of flora or fauna living within the proposed lot. This layer also considers the previous treatment of the land as a factor to adjust its future protection. Our group also investigated the flood plain of the area and looked for possible problems with flood insurance rates. Finally, we concerned ourselves with the soil composition of the location. GIS offers various layers which analyze both visible and nonvisible soil compositions. This allowed us to determine the feasibility of excavating the proposed site.

3.2 Steel Design Methodology

As mentioned above, the building is a combined steel and concrete structure. The three story academic building on top of the parking garage will be constructed as a braced frame system. The bracings used throughout the structure to withstand lateral loads are shown in Figure 14. The steel frame was designed to support dead loads (self-weight of the members plus any additional dead loads applied on the roof and floor beams), snow, rain seismic and wind. The following sections discuss the methods used to design steel frame members and connections in addition to specifications and codes required throughout the design process.

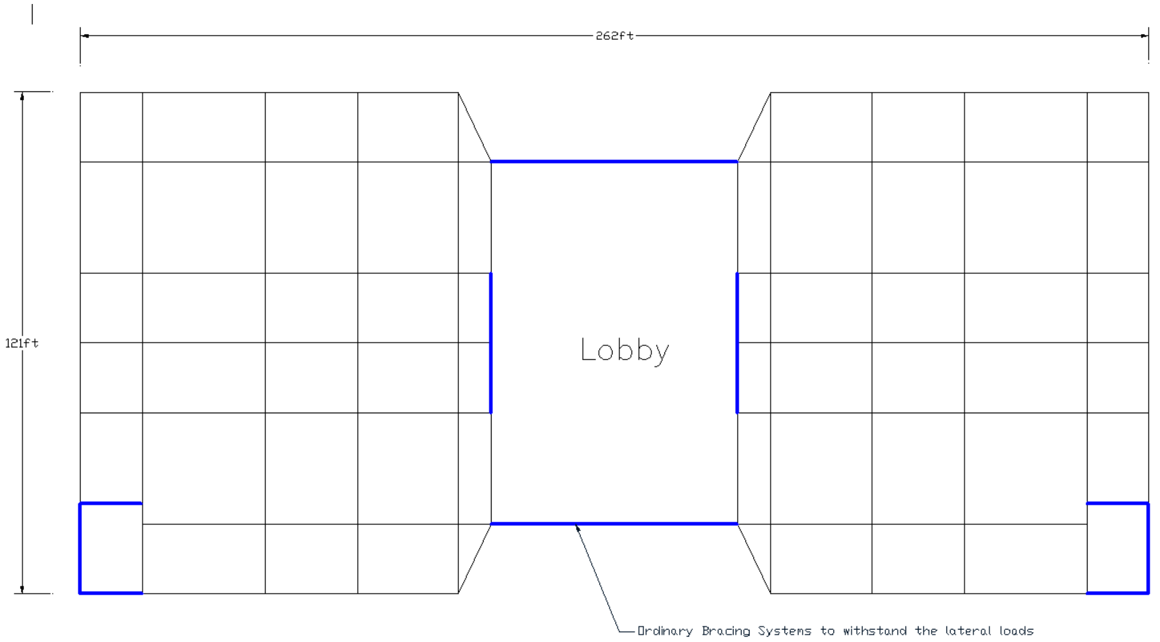


Figure 14: Plan View showing the proposed steel bracing for lateral support

3.2.1 Roof Load Analysis

Based on consultations with WPI and following the trend of recent construction on campus, our team decided to incorporate a green roof on top of the three story academic portion of the building [16]. The team decided that staff and students should be able to access the roof to provide recreational space. We also determined what accessories needed to be placed on the roof in addition to the green components. Based on the needs of the academic portion of the structure, our team discovered that the building would require HVAC units for air circulation. The roof provides the most practical location for these units. Finally, our team decided that the roof would have to compliment the interior of the academic building in at least one way. We determined that a transparent structure above the lobby would increase the ambient lighting inside the building. Therefore we compared the aesthetic value, feasibility, and drainage of these structures in order to choose the final design.

Although intensive roof systems would provide numerous possibilities for designing a garden, they will increase the dead load on the roof slab and increase the size of members throughout the academic building and parking structure. This could reduce the economy of the entire design. On the other hand, an extensive roof system would limit the type of flora however would not lead to additional structural demands. Our team decided that an extensive green roof would be the best option. Once we determined the layout and surface areas of all the mechanical items on the roof, the next step was determining the weights these systems would exert on the roof slab according to IBC 2009 and LRFD. Calculations of the dead load due to the green space were made using the equation:

$$D_{GR} = \text{Average weight of green spaces (psf)} * \text{Area of space (sq.ft.)}$$[4]

The total dead loads were put as the sum of the individual mechanical equipment (i.e. HVAC), green roof (D_{GR}) and self-weight of members (beams, girders, and composite slab). In addition to these dead loads, the roof live load due to persons walking on the roof for maintenance and other purposes were calculated based on Table 1607.1 in IBC [21].

Other loads acting on the roof of the academic building include snow, wind and rain loads. The Massachusetts State Board of Building Regulations and Standards' 780 CMR document, Table 1604.10, states that the minimum snow loads (p_g) that should be considered for a building in Worcester is 55 psf. The team used this value, along with other variables, to determine the final flat roof snow load (p_f) for the chosen load combination.

$$p_f = 0.7C_eC_tIp_g$$

Where: C_e = Exposure factor (ASCE 7-05, Table 7-2) [22]

[4] Psf: Pounds per square-foot sq.ft.: Square-foot

C_t = Thermal factor (ASCE 7-05, Table 7-3) [22]

I = Importance factor (ASCE 7-05, Table 7-4) [22]

The p_f value for the garage design included the loads associated with snow drift. Snow may accumulate on the sides of the building after being blown over the academic building's roof. The following excerpt from ASCE 7-05 outlines the procedure of determining the drift snow load:

"Snow that forms drifts comes from a higher roof or, with the wind from the opposite direction, from the roof on which the drift is located. These two kinds of drifts ("leeward" and "windward respectively) are shown in Fig. 7-7 [ASCE7-05]. The geometry of the surcharge load due to snow drifting shall be approximated by a triangle as shown in Fig. 7-8. Drift loads shall be superimposed on the balanced snow load. If h_c/h_b, is less than 0.2, drift loads are not required to be applied. For leeward drifts, the drift height h_d shall be determined directly from Fig. 7-9 [in ASCE7-05] using the length of the upper roof. For windward drifts, the drift height shall be determined by substituting the length of the lower roof l_u, in Fig. 7-9 [in ASCE7-05]. And using three-quarters of h_d as determined from Fig. 7-9[in ASCE7-05] as the drift height. The larger of these two heights shall be used in design. If this height is equal to or less than h_c, the drift width, w, shall equal $4h_d$ and the drift height shall equal h_d. If this height exceeds h_c, the drift width, w, shall equal $4h_d^2/h_c$ and the drift height shall equal h_c. However, the drift width, w, shall not be greater than $8h_c$. If the drift width, w, exceeds the width of the lower roof, the drift shall be truncated at the far edge of the roof, not reduced to zero there. The maximum intensity of the drift surcharge load, p_d, equals $h_d\gamma$ where snow density, γ, is defined in Eq. 7-3:

$$\gamma = 0.13p_g + 14 \text{ but not more than 30 pcf (7-3)"}$$

One of the major lateral loads that needed to be designed for was the wind load. This is the uniform force exerted on the windward face of a building due to the wind. The wind load varies proportionally with the wind speed. The basic wind speed, V, for buildings in Worcester, was found to be 100 mph (Table 1604.10; 780 CMR). This value was used in a series of steps for "Method 1- Simplified Procedure" outlined below and referenced in ASCE 7-05 [22]:

I. “The *basic wind speed V* shall be determined in accordance with Section 6.5.4. The wind shall be assumed to come from any horizontal direction.

I. An *importance factor I* shall be determined in accordance with Section 6.5.5.

II. An *exposure category* shall be determined in accordance with Section 6.5.6.

III. A height and exposure adjustment coefficient, *λ,* shall be determined from Fig. 6-2 (See Appendix B).”

The load due to the pooling of rain on the roof of the building as a result of the blockage of the primary drains was calculated using the equation:

$$R = 5.2(d_s + d_h)$$

Where: R = Rain load on the un-deflected roof, in psf.

d_s = Depth of water on the un-deflected roof up to the inlet of the secondary drainage system when the primary drainage system is blocked (i.e., the static head), in in.

d_h = Additional depth of water on the un-deflected roof above the inlet of the secondary drainage system at its design flow (i.e., the hydraulic head), in in (ASCE 7-05) [22].

At this stage, the governing load combination on the roof of the academic building was determined. This is the load case from the following list of LRFD load combinations which has the greatest sum and will set the minimum design load of the element. The following load combinations are equations 16-1 through 16-6 in IBC 2009 [21]:

I. $U = 1.4 (D+F)$

II. $U = 1.2(D+F) + 1.6(L+H) + 0.5(Lr \text{ or } S \text{ or } R)$

III. $U = 1.2(D+F) + 1.6(Lr \text{ or } S \text{ or } R) + (f_1L \text{ or } 0.8W) + 1.6H$

IV. $U = 1.2D + 1.6W + f_1 L + 0.5(Lr \text{ or } S \text{ or } R) + 1.6H$

V. $U = 1.2D + 1.0E + f_1L + 0.5S + 1.6H$

VI. $U = 0.9D + (1.0E \text{ or } 1.6W) + 1.6H$

Where: f_1 = 1.0 for floors in places of public assembly, for live loads in excess of 100 pounds per square foot (4.79 kN/m2), and for parking garage live load.

f_1 = 0.5 for other live loads.

U = Design/Ultimate load

D = Dead load

L = Live load due to occupancy

L_r = Roof live load

S = Snow load

W = Wind load

R = Rain load excluding ponding effects

H = Loads due to lateral Earth or pore pressure

E = Earthquake load

F = Flood load

3.2.2 Floor Load Analysis

The next step in the design process was to plan the layout of the floors in the academic building. This was guided by the number, size and uses of the different rooms. Figure 11 shows a potential layout of the first floor of the academic section of the building with two main lecture halls, 10 faculty offices, 12 student conference rooms and lounges. The advantage of such a layout is the location of the academic space. The majority of the student academic space is located on the very accessible first floor.

Once the layout of the rooms and the potential locations of columns were identified, the loads acting on the floors of the academic building were calculated according to IBC 2009 Equations 16-1 to 16-6 as previously discussed in the *Roof Load Analysis* section [21]. The team determined the governing load case on each floor in order to design the steel beam system.

There were a number of possible design systems for the roof and floor structure. The most typical of these systems are: corrugated steel decking with a thin concrete slab, open-web joists, composite (steel beam and concrete), concrete-pan, and flat slab. Appendix C shows the advantages of each system.

In order to choose the most suitable system, we considered three main characteristics. We considered constructability, economy, and weight imposed on the structure. Our team qualitatively compared and contrasted the separate systems against one another. Based on the information presented in Appendix C, our team decided that steel decking would be the best-use system for our design.

3.2.3 Design Specifications

Through the steel design phase we followed the building codes and specifications provided by AISC-2010 (American Institute of Steel Construction) and IBC 2009 standards. The loads affecting the structure were calculated according to ASCE 7-05 (American Society of Civil Engineers, Minimum Design Loads for Buildings and other Structures) and followed the LRFD method [22]. For more information on LRFD refer to ASCE 7-05. The main intent of these specifications is to provide the loadings which cause the largest stresses on the structure, therefore designing conservatively.

3.2.4 Computer Design Software

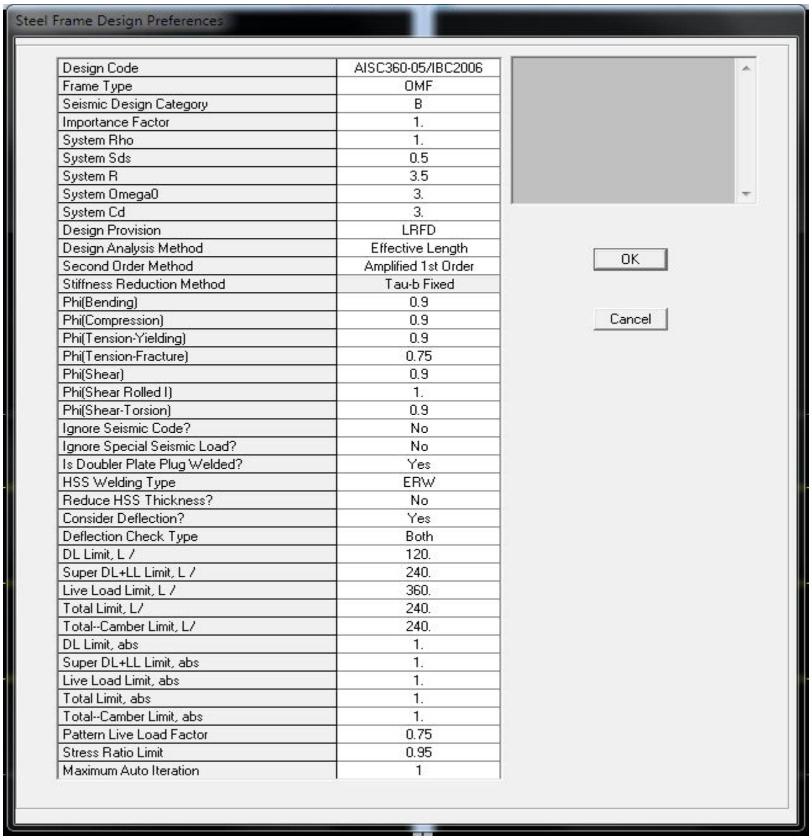
Using computer software to analyze and design can greatly reduce the time required for the task and is likely to increase the accuracy of the analysis. For this project we reviewed three (RISA, STAAD Pro, ASDIP Modules and ETABS) structural steel design software packages which are commonly used. We analyzed these by considering their user interface, functionality, analysis process and design output for each software. Based on these criteria, the team decided to use ETABS as the main frame analysis software.

3.2.4.1 ETABS

ETABS is a building analysis and design software provided by Computers & Structures Inc. Its system is built around a physical object based graphical user interface. The structural engineering computer software allows users to create and analyze integrated building models that have moment resisting frames, braced frames, rigid and flexible floors, sloped roofs, ramps, and parking structures [23]. Its object oriented interface makes the data entry easy and expedites the design and analysis process. However, In order to achieve accurate results, the user must have knowledge of finite element analysis (the method used by the software) and must be familiar with the ETABS's interface. Due to ETABS's broad usage and capability, our team selected ETABS as our main source for the frame analysis of the steel structure. The team used manuals and online tutorials to learn about the ETABS's interface and analysis process.

The ETABS design method is conducted in three steps: Modeling, which includes defining the member properties and assigning the applicable loads. Analysis; calculates the reactions, deflections and moments on each member and joint. Design; the process is broken into steel design, and concrete design. The design part of the method chooses the member sizes according to the previously determined design parameters.

Our team first defined the materials for the metal decking, composite floor slab, frame members (beams, columns, braces and girders) and then modeled the building using a pre-defined list of structural members. The appropriate load cases (described in sections 3.2.1 and 3.2.2) were applied to members. The ETABS analysis was conducted using the design parameters of the building which included load combinations for dead, live, super dead, cladding, wind, seismic, and equipment loads. After the design was complete, the software selected the most economical members from the pre-defined list. The steel frame design was according to the values shown in Figure 15.



Design Code	AISC360-05/IBC2006
Frame Type	OMF
Seismic Design Category	B
Importance Factor	1.
System Rho	1.
System Sds	0.5
System R	3.5
System Omega0	3.
System Cd	3.
Design Provision	LRFD
Design Analysis Method	Effective Length
Second Order Method	Amplified 1st Order
Stiffness Reduction Method	Tau-b Fixed
Phi(Bending)	0.9
Phi(Compression)	0.9
Phi(Tension-Yielding)	0.9
Phi(Tension-Fracture)	0.75
Phi(Shear)	0.9
Phi(Shear Rolled I)	1.
Phi(Shear-Torsion)	0.9
Ignore Seismic Code?	No
Ignore Special Seismic Load?	No
Is Doubler Plate Plug Welded?	Yes
HSS Welding Type	ERW
Reduce HSS Thickness?	No
Consider Deflection?	Yes
Deflection Check Type	Both
DL Limit, L /	120.
Super DL+LL Limit, L /	240.
Live Load Limit, L /	360.
Total Limit, L/	240.
Total--Camber Limit, L/	240.
DL Limit, abs	1.
Super DL+LL Limit, abs	1.
Live Load Limit, abs	1.
Total Limit, abs	1.
Total--Camber Limit, abs	1.
Pattern Live Load Factor	0.75
Stress Ratio Limit	0.95
Maximum Auto Iteration	1

Figure 15: ETABS Steel Frame Design Preferences

The software creates extensive outputs for the values that it calculates which were checked with hand calculations. While computers reduce the amount of calculation necessary for analysis, it is still required for the design engineer to have solid understanding of the methods used during the analysis process. In order to demonstrate our team's comprehension of structural design, we produced various hand calculations to ensure the accuracy of the ETABS output. Appendix D contains a sample of a beam, girder and column design output. In the next section we discuss different methods we will use throughout the project to design for stability, tension members, compression members, beams and connections. The outputs from the ETABS and the member sizes selected for each section of the steel frame is provided in Appendix E

3.2.5 Design of Roof Truss

For the design of roof of the lobby for the academic building will be covered in glazed glass (*DuPoint high strength laminated Butacit PCB*) in order to allow for the maximum amount of natural lighting. Due to the 60ft width of the lobby, the roof structure was determined to consist of 5 steel trusses that are eight feet tall. The steel truss was chosen due to its lighter construction than large steel beams and its ability to span greater distances in a more economical fashion. The truss also provides a slope for better drainage of the water and snow and will allow more natural light to enter the lobby. Sap2000 structural engineering software was used to create the full lobby frame with the trusses in order to check the truss against lateral and gravity loads. Figure 16 shows the frame layout, and the analysis for the frame layout is shown in Appendix F.

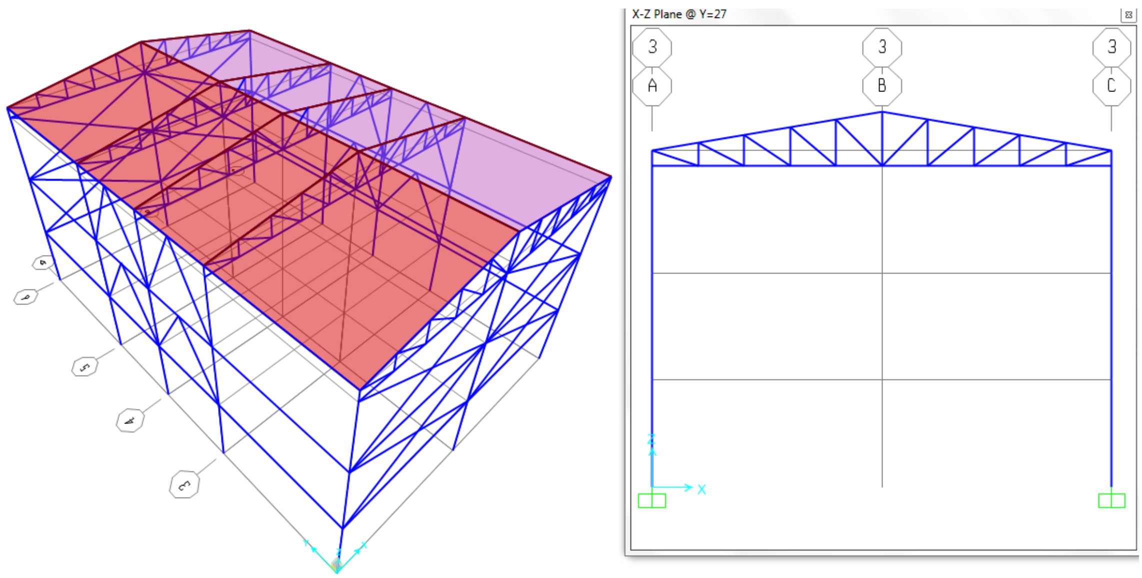
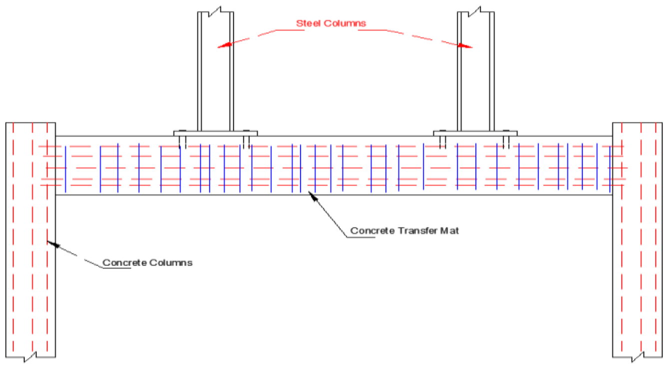


Figure 16: Lobby truss and frame layout by SAP2000

3.3 Transfer Loads

An engineering challenge arose interfacing the steel structure to the concrete parking garage. Although there were several options to consider, our team distinguished the best design by considering cost, and constructability. We had to successfully transfer all of the loads from the steel columns, which support the entire academic building, to the concrete structure. Since the floor plan of the office building was unique from the floor plan of the parking garage, it proved to be a challenge for us to determine the correct layout. We considered using a thick concrete mat, transfer beams, and concrete columns supporting the steel columns loads.

The first option, using a concrete mat, was primarily considered to simplify the design process. However, quick calculations proved this option to be uneconomical. The uniform thickness of the transfer mat would require large volumes of concrete and high percentage of



reinforcing steel.

Figure 17 shows an example of a concrete mat used to carry loads from two steel columns.

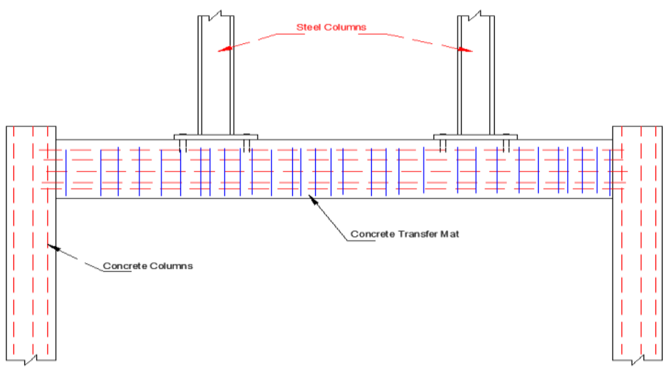


Figure 17: Mat Transfer Diagram [24]

The second option, using transfer beams, initially proved promising. Our plan included having post-tensioned beams span from the perimeter to the central axis of the garage floor. These beams would support the steel columns as well as the concrete slab for the first floor of the academic building. Figure 18 shows an example of a post-tensioned transfer beam. After several quick calculations, our team discovered that the required sizes of these beam made the option impractical. The large depths of the beams would increase the overall height of the structure and prove difficult to place in the field.

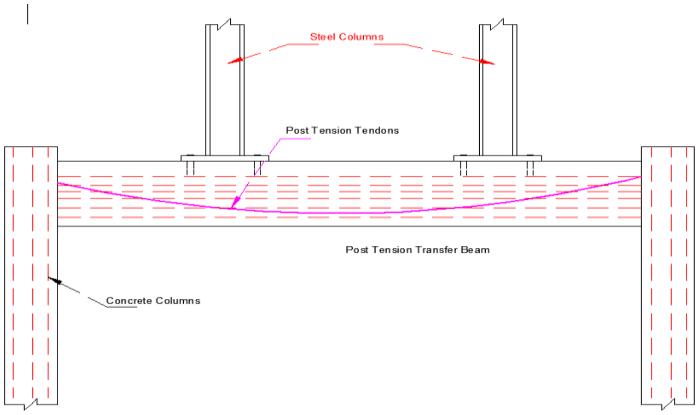


Figure 18: Post Tension Transfer Beam Example [25]

The final option we considered was using reinforced concrete columns as direct supports for the steel columns. The concrete columns were designed for moments, buckling, and axial compressive forces, and minimum cross section area of the concrete columns were selected to have enough capacity for the applied loads; and adequate dimensions to support the steel baseplate. The column design also considered the parking layout and whether the dimensions for the columns could be changes to increase the number of parking spaces. The exterior columns will be braced by the exterior girders; while the interior beams are not un-braced due to their location.

3.4 Parking Garage Layout

Before we began the structural design of the parking garage, our team created a conceptual layout of the garage. Our aim was to optimize the layout to accommodate the largest number of parking spaces. In order to accomplish this, our team analyzed preexisting parking structures around WPI to gain a sense of parking structure layouts. We examined 37 Dean Street as well as the Gateway parking garage. We took measurements of parking spaces and annotated the ramp designs into sketches for later conceptual use. These sketches are shown in Section 2.3.1.

In addition to looking at previously existing structures, we used available text resources which gave insight to parking structure layouts and requirements. The design required adherence to the proper regulations and considered both regional and national codes. The design progressed to an efficient layout of the parking spaces as they will bring in annual revenue to WPI. Therefore the design must exhaust all plausible options to optimize the amount of spaces. One resource we used to determine the layout was "Design, Construction and Maintenance of Cast-in-Place Post Tensioned Concrete Parking Structures" [18]. This source covered the majority of information we needed for developing the layout.

3.4.1 Pre-stressed Double-tee Design

In order to design the floor system for the parking garage we studied two different systems; post tensioned (PT) concrete slab and pre-stressed precast double- tee beams shown in Figure 19 and Figure 20. Our team found the use of pre-stressed precast double-tees to be more feasible. Figure 58 in Appendix C shows the benefits of using a PT concrete system.

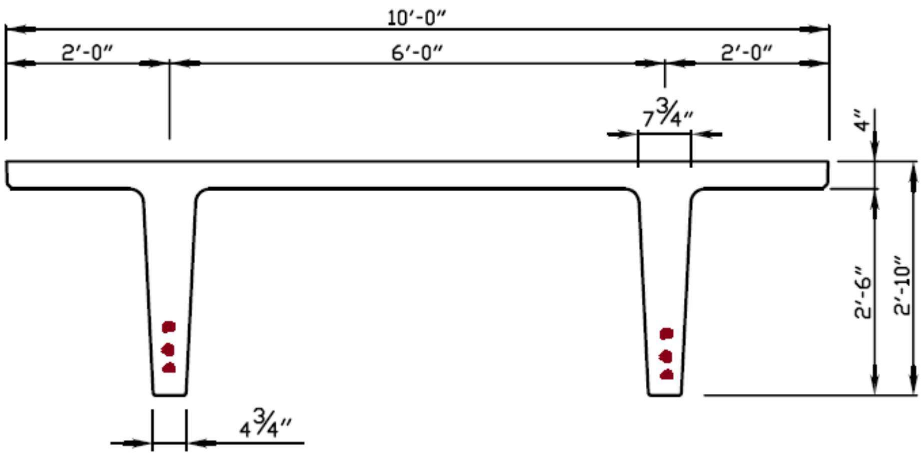
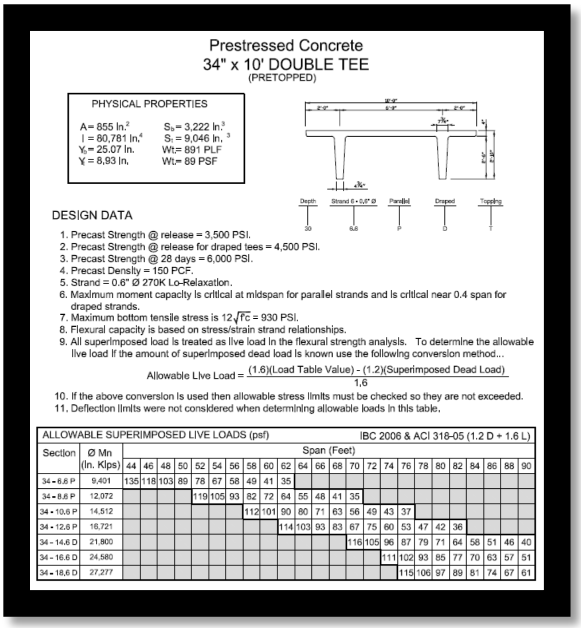


Figure 19: Pre-stressed Double-tee Member [27]

Figure 20: Double-tee Floor System [28]

The use of precast prestressed double-tees simplifies the overall design process. In addition, our observations of the local WPI parking garages indicated that double-tee design was common for these applications. Concrete casting companies publish sheets with predetermined structural properties. An example of these specifications can be seen in Figure 21.



Prestressed Concrete
34" x 10' DOUBLE TEE
(PRETOPPED)

PHYSICAL PROPERTIES

$A = 855$ In.^{2} $S_b = 3{,}222$ In.^{3}
$I = 80{,}781$ In.^{4} $S_t = 9{,}046$ In.^{3}
$Y_b = 25.07$ In. Wt= 891 PLF
$Y_t = 8.93$ In. Wt= 89 PSF

DESIGN DATA

1. Precast Strength @ release = 3,500 PSI.
2. Precast Strength @ release for draped tees = 4,500 PSI.
3. Precast Strength @ 28 days = 6,000 PSI.
4. Precast Density = 150 PCF.
5. Strand = 0.6" Ø 270K Lo-Relaxation.
6. Maximum moment capacity is critical at midspan for parallel strands and is critical near 0.4 span for draped strands.
7. Maximum bottom tensile stress is $12\sqrt{f'c} = 930$ PSI.
8. Flexural capacity is based on stress/strain strand relationships.
9. All superimposed load is treated as live load in the flexural strength analysis. To determine the allowable live load if the amount of superimposed dead load is known use the following conversion method...

$$\text{Allowable Live Load} = \frac{(1.6)(\text{Load Table Value}) - (1.2)(\text{Superimposed Dead Load})}{1.6}$$

10. If the above conversion is used then allowable stress limits must be checked so they are not exceeded.
11. Deflection limits were not considered when determining allowable loads in this table.

ALLOWABLE SUPERIMPOSED LIVE LOADS (psf) — IBC 2006 & ACI 318-05 (1.2 D + 1.6 L)

Section	Ø Mn (In. Kips)	Span (Feet) 44	46	48	50	52	54	56	58	60	62	64	66	68	70	72	74	76	78	80	82	84	86	88	90
34 - 6.6 P	9,401	135	118	103	89	78	67	58	49	41	35														
34 - 8.6 P	12,072					119	105	93	82	72	64	55	48	41	35										
34 - 10.6 P	14,512								112	101	90	80	71	63	56	49	43	37							
34 - 12.6 P	16,721										114	103	93	83	67	75	60	53	47	42	36				
34 - 14.6 D	21,800														116	105	96	87	79	71	64	58	51	46	40
34 - 16.6 D	24,580																111	102	93	85	77	70	63	57	51
34 - 18.6 D	27,277																	115	106	97	89	81	74	67	61

Figure 21: Fabricated Double-tee Specifications [29]

The choice of a precast double-tee for the parking garage was mainly driven by the loads and the sizes available. The first step was to determine the dead and live loads on the concrete. The IBC minimum live load is 40psf for a garage, but this value was increased to 45psf to be conservative. The dead load for the parking was 15psf because only the lighting and fire suppression systems needed to be accounted for; this did not include the self-weight of the double-tee beams.

The strand patterns were found by using a *PCI Design Handbook* Pre-topped Double-Tee Load Tables [29]where the allowable safe superimposed live loads per span and strand pattern are also displayed. From these tables we selected the lightest member for the needed loads. The table also gave us the estimated camber of the beam at erecting and estimated long term camber. Cambering theses beams allows for increased load capacity that is still is within code requirements.

3.4.2 Girder Design

The weight from double-tees is transferred into the concrete girders that span between the columns on both the perimeter and interior areas of the garage. The size of the girders was estimated according to ACI code, and the girders were checked against moment distribution, torsional effects and shear failure to determine adequacy. We expected stirrup steel and longitudinal steel to both be required for flexural, shear and torsional effects.

3.4.3 Corbel Design

When designing a corbel the main constraint is the possible shear failure from the reaction load. The first step in the design is calculating the load from the double-tees. In addition, the location of the point loads from the double-tees need to be determined so that the corbels could be correctly placed. After completing the initial layout of the corbels, we checked the structural requirements, which included the nominal shear capacity $V_n < 0.2f'_c b_w d$ and $< 800 b_w d$ by ACI Code. A horizontal tensile force needed to be included to insure that the minimum capacity was available which is equal to $0.2V_u$. The area of the primary steel (horizontal closed stirrups) could then be determined by adhering to the greater of the following cases. The amount of primary steel had to meet the minimal ACI requirement.

$$A_{sc} = 2/3(A_{vf}) + A_n \qquad \text{Or} \qquad A_{sc} = A_f + A_n \qquad A_{sc}\,\text{Min} = 0.04(f'_c/fy)bd$$

The dimension of the corbel was also restrained by $e/d \leq 1.0$ by ACI Code, where *e* was the moment arm and d was the depth of the steel. Using a 45 degree corbel, ACI dictates that the bearing area must be no less than 0.5d. In addition, the dimensions from the top to the bottom of the closed stirrups must not be greater than 2/3 d. The steel angle should be selected to insure that the concrete is not impacted by tension A_{sc} steel. Figure 22 shows these design requirements in detail.

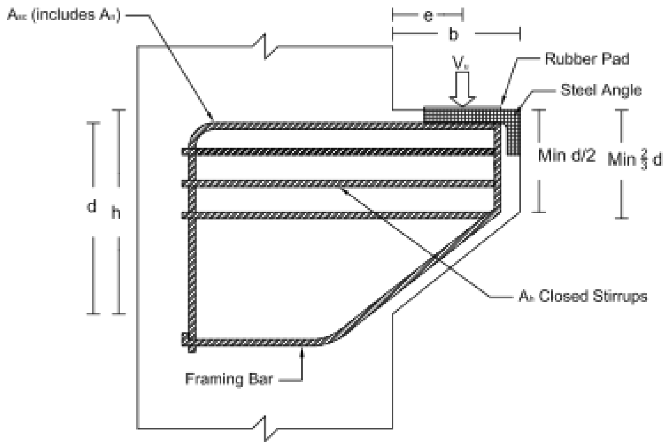


Figure 22: Single Corbel Design [30]

Double corbels used in the centerline of the concrete parking garage were designed in a similar manner except for the closed stirrups being connected to each other as shown in Figure 23.

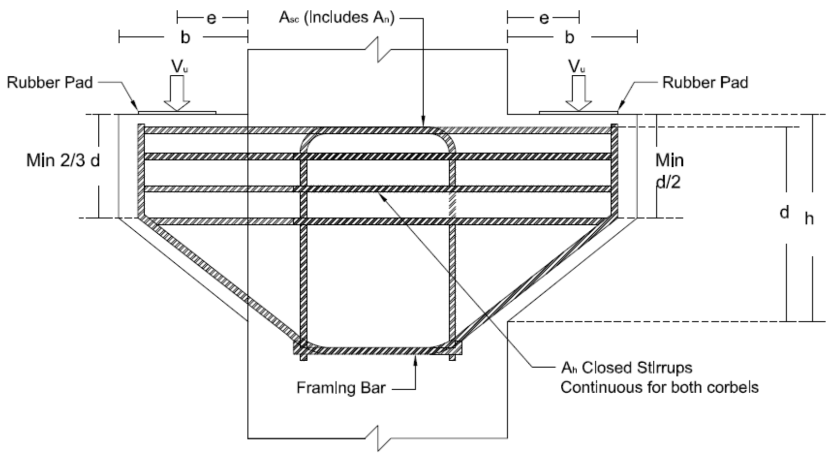


Figure 23: Double Corbel Design [30]

3.5 Foundation Design

The foundation design for this structure will be broken into three parts. The calculations and results for each section are provided in the appropriate appendixes.

1. Footing Design - Appendix M
2. Retaining Wall Design – Appendix N
3. Parking Structure Shear Wall - Appendix L

However, before designing the footings and the retaining wall, it is necessary to analyze the site soil conditions. The general procedure to analyze the soil condition is as follows.

3.5.1 Soil conditions

Before designing the footing or the retaining wall, it is necessary to check the soil condition of the site. Soil conditions can be analyzed in several ways. The Schmertmann Method[5] looks at the soil profile and the blow data and therefore results in a more reliable soil capacity. Figure 24, Figure 25, and Figure 26 shows the soil diagrams and boring locations for the site, the data is obtained from the soil tests in 1998. These give the depths of each layer which Schmertmann's method uses to accurately evaluate the soil for supporting a structure.

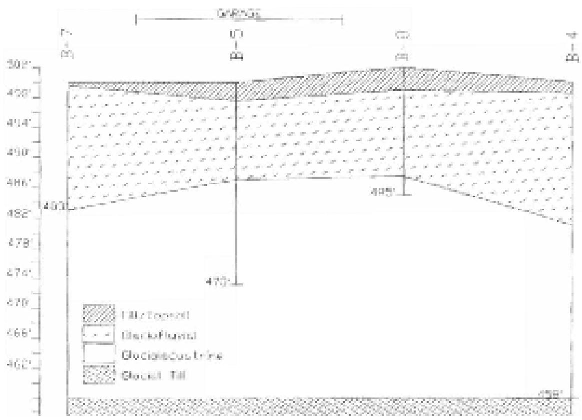


Figure 24: Sediment locations for Borings 4-7 [11]

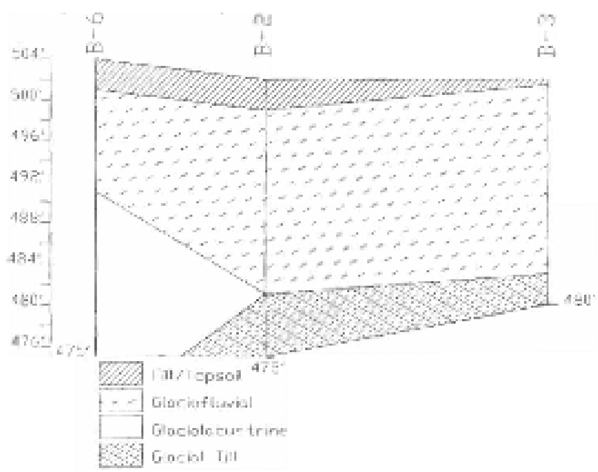


Figure 25: Sediment locations for Borings 2,3 & 6 [11]

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[5] Book Reference (Schmertmann, 1970 & 1978)

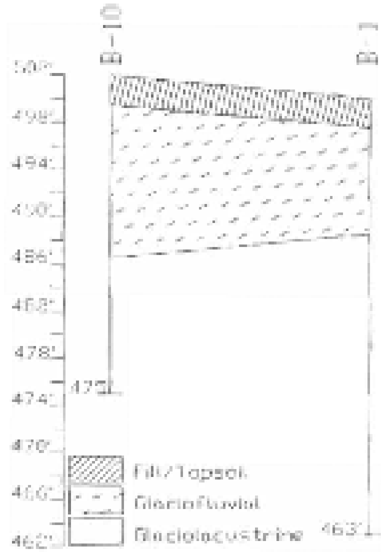

Figure 26: Sediment locations for Borings 1 & 10 [11]

After locating the layers in the soil the Schmertmann method can be applied.

The settlement of the layer can be calculated by

$$\sum_{i=0}^{n} \frac{I_Z \Delta z}{E_s}$$

E_s is the modulus of elasticity strength factor (kilo-Newtons per meter), I_z is the influence factor in layer n, and Δz is the change in depth of the layer (meters).

This summation can then be placed into the equation below to find the actual settlement.

$$\delta = C_1 x\, C_2\, x\, \Delta\sigma \sum_{i=0}^{n} \frac{I_Z \Delta z}{E_s}$$

Where $\Delta\sigma$ is the change in net stress due to the applied load, and C_1 & C_2 are the correction factors.

C_1 is the factor of the strain relief due to embedment.

$C_1 = 1 - 0.5\frac{\sigma_v'}{q-\sigma_v'} \geq 0.5$

$C_2 = 1 + 0.2 \log_{10}(\frac{t(years)}{0.1})$

The test conducted in 1998, although useful, did not conduct a static cone test which Schmertmann uses for the calculation of the modulus of elasticity. Stathis Payiatakis (Payiatakis, 1977) developed a method of calculating static cone test. His research found that Melzer's formula has the most suitable correlation for the blow tests.

$N_m = 7.57 + 1.05N_{field}$

From Melzer's formula the modulus of elasticity can be calculated with Payiatakis's equation below.

$$E_s = 2K*N_m$$

Where, K is the variable that payiatakis's developed to provide a better representation of the soil in respect to the strength values. The K-value by Payiatakis's = 3-4, therefore K = 3.5 is a reasonable average for clean, fine-to-medium sands and slightly silty sands that Haley & Aldrich INC found.

In conclusion Schmertmann's method does take into account the most factors for a foundation and will be used for designing the foundation of the structure.

After this preliminary research was completed our team was provided with the geotechnical reports for East Hall and Recreational Center on campus done by Haley and Aldrich INC. These reports parallel with soil and site conditions found on the site and gave allowable bearing pressure of 4ksf for the foundation. The geotechnical data by Haley & Aldrich INC was considered to be sufficient for the scope of this project and the methods described were used as reference only.

3.5.2 Footing design due to loadings

The footing design process for this project is shown in Appendix M and is according codes and site specifications. The footing design [31] generally took into account the following loads. The footing design had to consider.

- Axial Load
- Lateral Load
- Moments
- Loads from base slab

To begin with the Bearing Pressure on the soil (q) must be calculated using

$$q= [(\frac{P + W_i}{A})] - u_D$$

Where,

P = vertical Column Loading

W_i = Weight of foundation (including weight of soil above foundation)

A = Base area of footing (B^2)

u_D = pore water pressure at bottom of foundation.

$u_D = \gamma_w z_{wD}$

Where, γ_w = specific weight of water

z_{wD} = Depth of the foundation – Depth of the water table

The foundation will have square footings, but will also need to incorporate continuous footing for the walls of the parking garage.

The design of the structure above will bring out whether the loads act through the centroid of the footing. However, if this is not the case, the eccentricity must be calculated and used to design the footing to account for the load not acting through the centroid. The eccentricity (e) of the bearing pressure can be calculated as:

$$e = \frac{Pe_1}{P+Wf}$$

Or, for continuous footings

$$e = \frac{P}{b}e_1 \frac{\frac{P}{b}e_1}{\frac{P}{b}+\frac{w_f}{b}}$$

Where e_1 = eccentricity of applied vertical load

P = vertical Column Loading

P/b = applied vertical load per unit length of foundation

W_f/b = weight of unit length of foundation

W_i = Weight of foundation (including weight of soil above foundation)

If the moment loads are non-uniform then the following equation must be used to calculate the eccentricity of the bearing pressure.

$$e = \frac{M}{M+w_f}$$

Or, for continuous footings

$$e = \frac{\frac{M}{b}}{\frac{M}{b}+\frac{w_f}{b}}$$

Where, M = applied moment load

M/b = applied vertical load per unit length of foundation

W_f/b = weight of unit length of foundation

W_i = Weight of foundation (including weight of soil above foundation)

If the eccentric or moment loads occur only in the width direction, then

$$q_{min} = \left(\frac{P+w_f}{A} - u_D\right)\left(1-\frac{6e}{b}\right)$$

Where, q_{min}= Minimum bearing pressure

q_{max}= Maximum bearing pressure

e = eccentricity of bearing pressure distribution

B = Width of foundation

P = Column Load

W_f = Weight of foundation

A = Base area of foundation

If the resultant loading on the base is eccentric in both directions of the footing, then the bearing pressure is:

$$q = \left(\frac{P + w_f}{A} - u_D\right) \left(1 \pm \left(\frac{6e}{b} \pm \frac{6el}{L}\right)\right.$$

Where, q= Bearing pressure

e = Eccentricity of bearing pressure distribution

B = Width of foundation

P = Column Load

W_f = Weight of foundation

A = Base area of foundation

L = Length of footing

From our soil conditions, we know that the soil is "medium dense to very dense light brown to olive grey coarse to fine sand with varying amounts of silt, coarse to fine gravel, cobbles, and boulders." It is also known that sections of the soil are "loose to medium dense yellowish orange to tan salty fine sand to fine sandy silt with varying amounts of medium to coarse sand and fine gravel".

From this description and looking at Coduto's Table 8.3 [31], the coefficient of friction μ can be conservatively chosen as equal to 45. From the μ-value, the effective friction angle (φ') for the soil beneath the foundation can be found.

$$\phi' = \tan^{-1}\mu$$

After calculating the bearing pressure on the soil, its bearing capacity can be found.

For square foundations

$$q_{ult} = 1.3c'N_c + \sigma'_{zD}N_q + 0.4\gamma' B N_\gamma$$

For continuous foundation

$$q_{ult} = c'N_c + \sigma'_{zD}N_q + 0.5\gamma' B N_\gamma$$

Where, q_{ult} = Ultimate bearing capacity

c' = Effective cohesion for soil beneath foundation

σ'_{zD} = Vertical effecting stress ate depth D below the ground surface

$\sigma'_{zD} = \gamma D - u$

Where, N_q = Terzaghi's bearing capacity factors (Coduto Table 6.1)

N_γ = Terzaghi's bearing capacity factors (Coduto Table 6.1)

B = Width of foundation

Due to the water table being at approximately 9 feet below the ground surface, its effects on the soils must be considered. The effective weight will change according to the following formula:

$$\gamma' = \gamma_b = \gamma - \gamma_w$$

From the ultimate bearing capacity the allowable bearing capacity can be calculated

$$q_a = q_{ult}/F$$

Where, q_a = allowable bearing capacity

q_{ult} = Ultimate bearing capacity

F = factor of safety

When designing a foundation the bearing pressure does not exceed the allowable bearing pressure.

$$q \leq q_a$$

The safety factor (F) will be chosen by taking into account any building codes, the prevailing soil type, soil variability, and the likelihood of the design load ever occurring.

After calculating the bearing pressure the induced stresses beneath the foundation will be looked at shown in Coduto's textbook, pg. 210 [31].

3.5.3 Retaining Wall Design

The retaining wall design for the building was completed by hand due to the columns being incorporated with the retaining wall. The wall was therefore designed as a beam with the soiling being the applied load. This method was conducted due to columns providing fixed support. Therefore the retaining wall could not be design as a free standing retaining wall.

3.5.4 Shear Wall

Reinforced concrete shear walls are used in buildings to resist the horizontal loads, such as seismic and wind. Shear walls are usually placed along the sides of the buildings, but in a parking garage the sides are usually open. Therefore the shear walls in this design were placed further inside the building shown in Figure 46. In order to resist loads in both directions, the shear walls were designed to be T shaped in the center of the garage. The elevator shaft and staircase walls were also used as shear walls. The design of the shear wall in the parking structure is shown in Appendix L. The shear wall was designed using Advanced Structural Design International Program (ASDIP). The wall is designed as a multipurpose support; it will support the concrete structure against lateral loads and the weight of the steel structure above.

3.6 Cost Estimation

After conducting all of the structural processes for the design, our team created a cost estimate using the computer software CostWork by RSMeans. We initially created a rough estimate of the building design to get a preliminary idea of the project's scale. To complete this initial draft, we completed the parameters for seven different sections. The sections are listed below.

1. Substructure
2. Shell
3. Interiors
4. Services
5. Equipment and Furnishings
6. Special Construction
7. Building Sitework

For the first five sections, our team inputted the most accurate data as possible into the software based on our design. However, for certain architectural sections we had to make assumptions as to what the architect would design. This included fittings, doors, and ceiling finishes. Another important note is that for this preliminary cost estimation our team excluded any information from the Special Construction and Building Sitework sections.

In addition to the outputs of the preliminary CostWorks data, we also calculated the square foot cost of the most recently constructed WPI Recreational Center. This was simply a division of the total square footage of the building from the total cost. After computing this, our team compared the cost per square foot of our design to that of the Recreational Center.

After completing this preliminary cost estimation, our team decided that a more precise and accurate cost estimation was necessary. We decided to continue to use the CostWork software with more detailed input data. In order to make the cost more exact we had to do more research on Site and Specialty Construction because the software did not account for these two conditions. Since our building has both steel and concrete we had to run the cost estimation program twice, once for each case. Then we had to make slight changes to the costs in both to more accurately reflect our building. For example, we had to delete the cost for foundation on the steel building because it is placed on top of the parking garage instead of foundations and we had to increase the size of the foundations in the parking garage to account for the extra loads coming from the steel structure.

4.0 Results and Discussion

The results are obtained from the background research and design process outlined in the methods section and following various building codes. These results reflect on the feasibility of the desired objectives of this project. We present these results in a sequential order reflecting the top-down procedure mentioned in the methodology.

4.1 Geographic Information System

As stated in our methodology, the land for the project must be appropriate for development. Figure 27 shows the finalized GIS map with the prioritized layers as noted in the methodology. Based on the selected layers, we made several important conclusions about the proposed site. The first important finding was the lack of any serious threat from surface flooding. This is extremely useful knowing that the proposed site is not hazardously located on a flood plain, especially with Institute Park's pond in close proximity. Our team also confirmed that the site is not within any setbacks for bodies of water or wetlands. Once again, the pond poses the closest potential design issue.

Figure 27: ArcMap 10 Generated Model [1]

We also considered the soil composition. The GIS results showed that there was no exposed bedrock on the site. However, the soil is primarily composed of glaciolacustrine and glaciofluvial deposits. This is corroborated by the findings found in the Haley and Aldrich geotechnical reports for the East Hall and Recreational Center projects. The deposits pose no significant issue for design or construction.

The GIS output also showed that the site is considered an environmental justice area. This means that due to previous ordinances, the land has not been preserved the way it should have been. Essentially, Worcester zoning has bookmarked the land to ensure it is properly treated in the future. This only reinforces the need for a low impact sustainable design. The design must aim to impose the smallest footprint as possible on the outlying area. This will ensure that WPI has an efficient and sustainable building which will create more opportunities in the future. Our team considered this output in addition to the LEED design criteria in order to make an effective structure which would fulfill the needs of WPI and Worcester.

In summary, the GIS results successfully show that there is no direct obstacle to the development of the site. We considered these results for the placement and as supplemental information for the soil composition from the geotechnical reports. After receiving these results, we continued on in the design process adhering to the top-down approach.

4.2 Academic Building Design

This section provides the recommended design information for the steel framed portion of the building. The frame layout and elevation view for each floor is placed under separate drawings (Drawings 1-5 attached in Appendix F). The interior layout of rooms and partitions for each story is viewed in Figure 28, Figure 29, and Figure 30. Complete analysis and calculation for the steel components are provided in Appendices. The design of the academic building consisted of the ETABS output for the steel frame, hand calculations, lobby roof truss system, and column baseplate design.

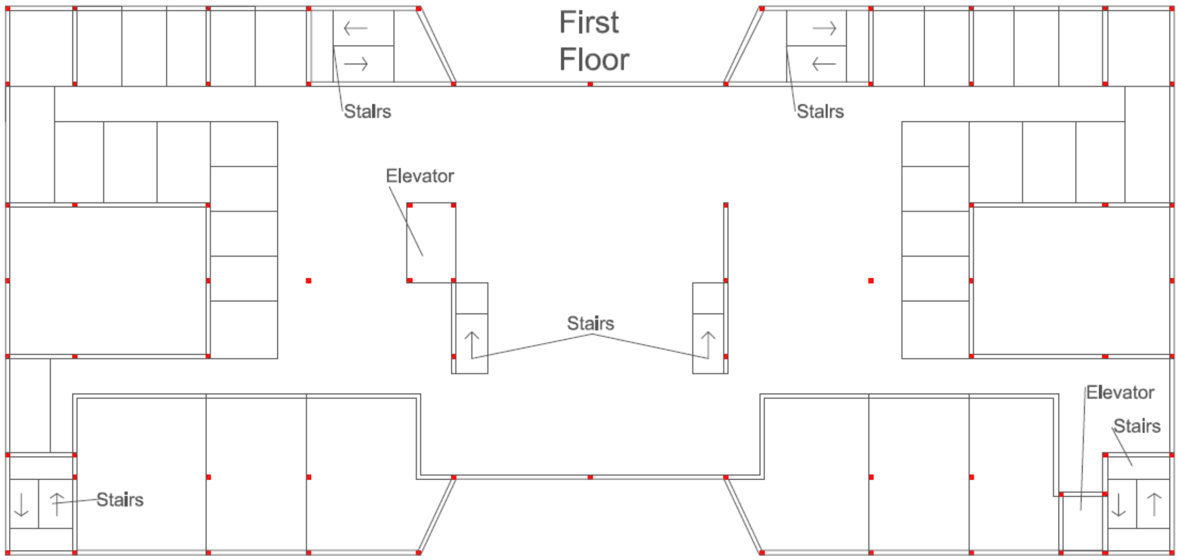

Figure 28: First Floor Interior Layout

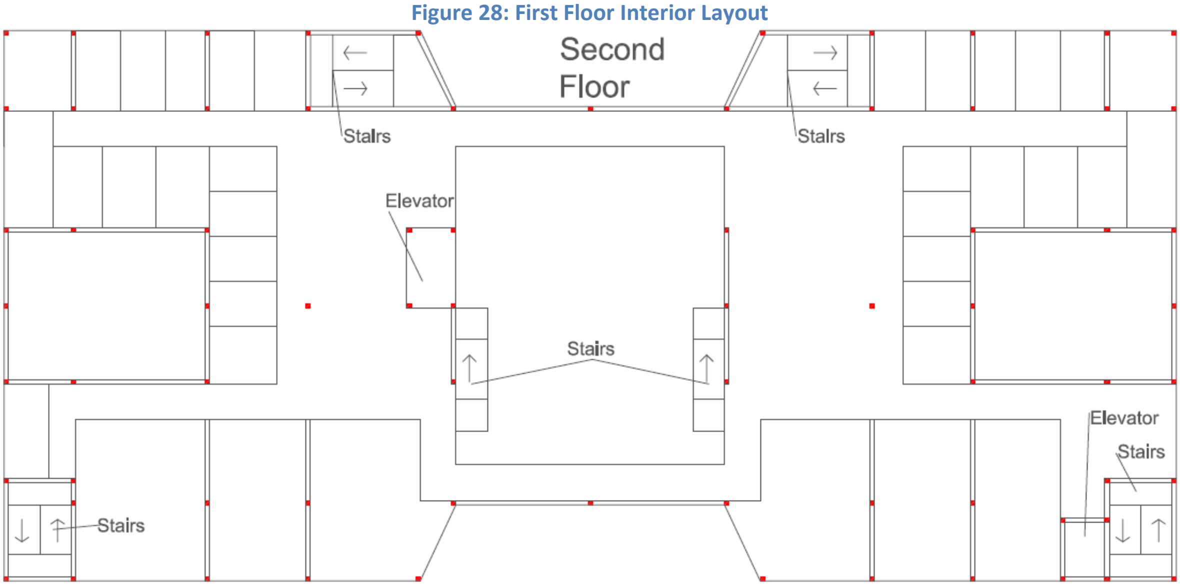

Figure 29: Second Floor Interior Layout

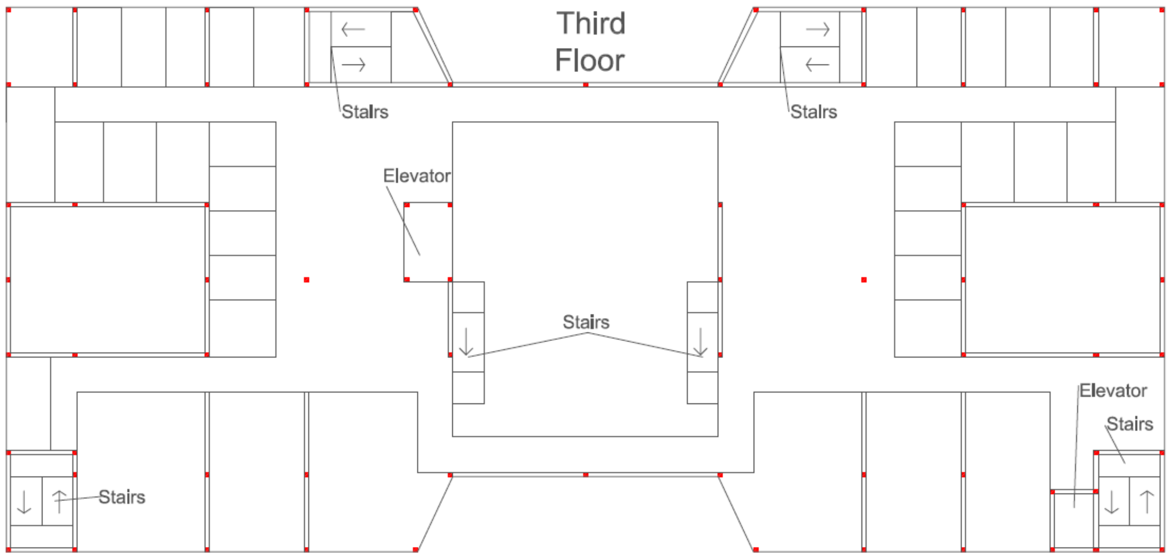

Figure 30: Third Floor Interior Layout

4.2.1 Steel Frame Computer Analysis

ETABS software was used to conduct the preliminary lateral and gravity load analysis. We used the results from the ETABS analysis to select appropriate composite beams, girders, columns, number of studs and bracing as required. The complete summary of our design is presented in Appendix E of this paper. The summary report presents the recommended member sizes and their reaction against axial and lateral loads. A 3D view of the steel frame is shown in Figure 31.

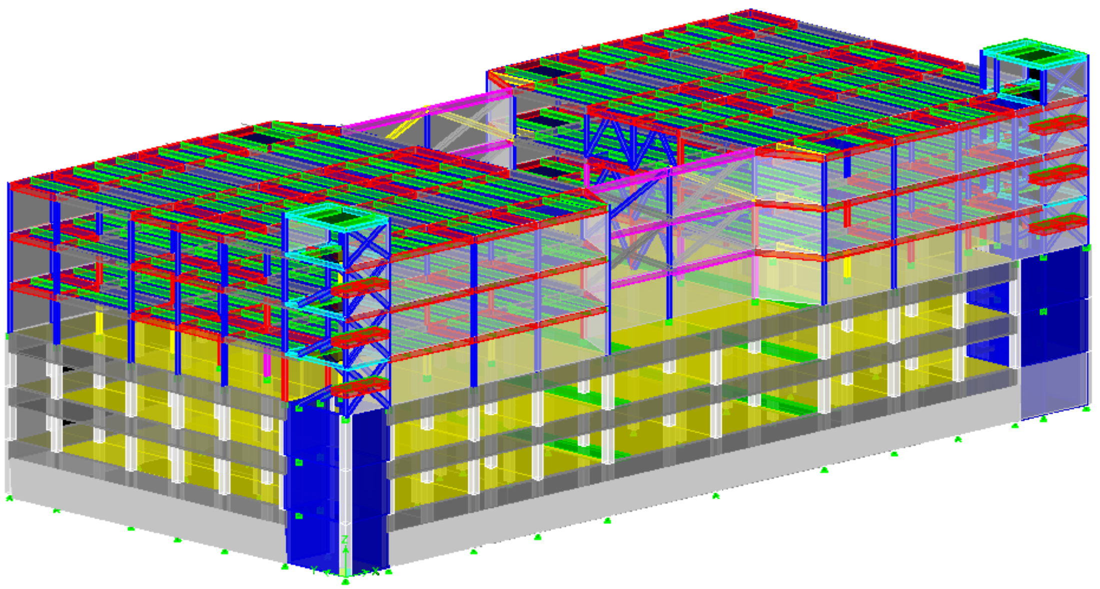

Figure 31: ETABS Finalized Design Showing Steel Frame and Concrete Parking Garage

4.2.1.1 Roof Truss Design

The Lobby roof truss's are analyzed according AISC manual and designed using SAP 2000 Computer Engineering Software. A total of 5, 6 foot trusses support the load combination and glazing on top of the lobby. Figure 32 shows the truss. The total factored load on the truss from the glass roof, snow, wind, earthquake, and self-weight was calculated to be 121pcf on the windward direction and 111pcf on the leeward direction of the roof. Drawing 01 attached on Appendix F shows the front view for the truss as well as provides the suggested member sizes for the frame. More detailed analysis and hand calculations are provided in Appendix F.

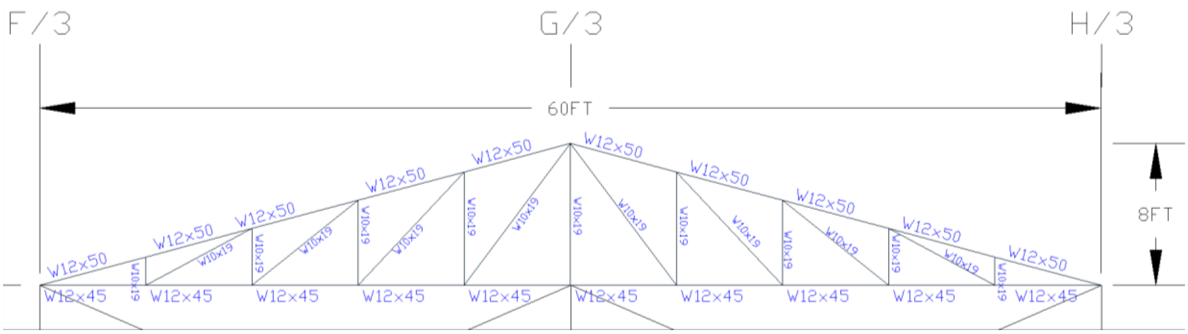

Figure 32 Lobby Roof Truss

4.2.1.2 Slab Design

Using 110pcf lightweight concrete, the slab design for the academic building was determined to be 5.5 inches thick with 1.5 inch 20 gauge composite metal deck. This configuration meets IBC Code, which requires a slab thickness of at least 3.6 inches for 2-hour fire rating for lightweight concrete [2]. The team chose 110pcf lightweight concrete for the slab in accordance with CSI Guide specification for structural lightweight concrete, section 033013, which is based on ASTM standards. This concrete will contain expanded shale, clay or slate as aggregate to achieve the specified light weight density. The decking gage is determined to be 20 Gauge and was chosen due to the span between the composite beams (7.5 feet to 8 feet apart). The beams and girders are designed as partial composite members and the required number of shear studs to achieve this property was selected according to ETABS's AISC specification and then later on calculated by hand (shown in the next section) in order to confirm the results.

4.2.2 Hand Calculations

In order to check the ETABS frame analysis results, we designed the steel beams and columns for one bay by hand. As with the remainder of the design process, we designed the steel top-down. The bay that we chose to design for was C7-D6 along with all of the calculations, which are in Appendix D. All of the calculations were done using the specifications set in the AISC manual.

4.2.2.1 Beam and Girder Design

The beams and girders were designed for partial composite action using shear studs, where the neutral axis is located below the flange. The beam sizes are summarized in Table 2 below. Several reasons explain why these beams and girders are smaller than that of the software program. The first reason is that these beams were only designed for dead, live, and snow loads, but ETABS included wind and earthquake loads. Also, for convenience, all of the beams were designed to be the same size in the software. However, in order to make fabrication easier, we designed all beams to be one size and all girders to be one size. This will reduce the overall project cost and simplify the construction phase.

Table 2: Selected Beams

Beams/Girder	Roof/Floor Level	Load w	Moment M_u (ft-k)	Connecting Beam Size	Number of Studs
Beams	Roof	1.2 k/ft	44.2	W12X16	22
	Floor	1.5 k/ft	54.2	W12X16	22
C7-D7	Roof	58 k	217.5	W16X31	44
	Floor	70 k	262.5	W16X31	44
C6-D6	Roof	44 k	165	W16X31	44
	Floor	54 k	202.5	W16X31	44

4.2.2.2 Column Design

In order to calculate the loads on the columns, preliminary calculations were done on beams and girders outside of the bay to determine their end reactions. In order to minimize the amount of different size columns only two sizes were chosen for this bay, shown in Table 3.

Table 3: Selected Columns

Column	Floor Level	Load (k)	Column Size	$\Phi_c P_n$
C7	3	100.7	W12X40	281
	2	222	W12X40	281
	1	343.3	W12X53	477
D7	3	90.1	W12X40	281
	2	198.6	W12X40	281
	1	307.1	W12X53	477
C6	3	77.5	W12X40	281
	2	171	W12X40	281
	1	264.5	W12X53	477
D6	3	73.2	W12X40	281
	2	157	W12X40	281
	1	240.8	W12X53	477

4.2.2.3 Connection Design

The beam-column connections were designed to be simple double angle connections. Using Table 10-1 in the AISC manual we decided that 3 rows and 1 column of ¾" A325 bolts would be sufficient to carry the loads being transferred to the column. We then calculated the minimum length for the angle legs and chose to use L4X4X¼ angles for the connections. We designed for the largest load being transferred so that the same connection could be used at all beam-column connections.

4.2.3 Base plate Design

The design of the base plate for the column was considered for a W16x57 column when considering the bearing strength and size of the concrete column (30in by 30in), anchor size, and the applied shear, moment and gravity loads. The plate size, as shown in Figure 11, was determined to be 28 inch by 20 inch with a thickness of 1.5 inch to allow for adequate distribution of the load and the AISC limit states. We used six 1-inch diameter anchor bolts to transfer the load from the steel column through the baseplate by bearing on the grout to the concrete column below. A fillet weld was specified around the column as shown in Figure 33: Base plate Design

Calculations are placed in appendix G.

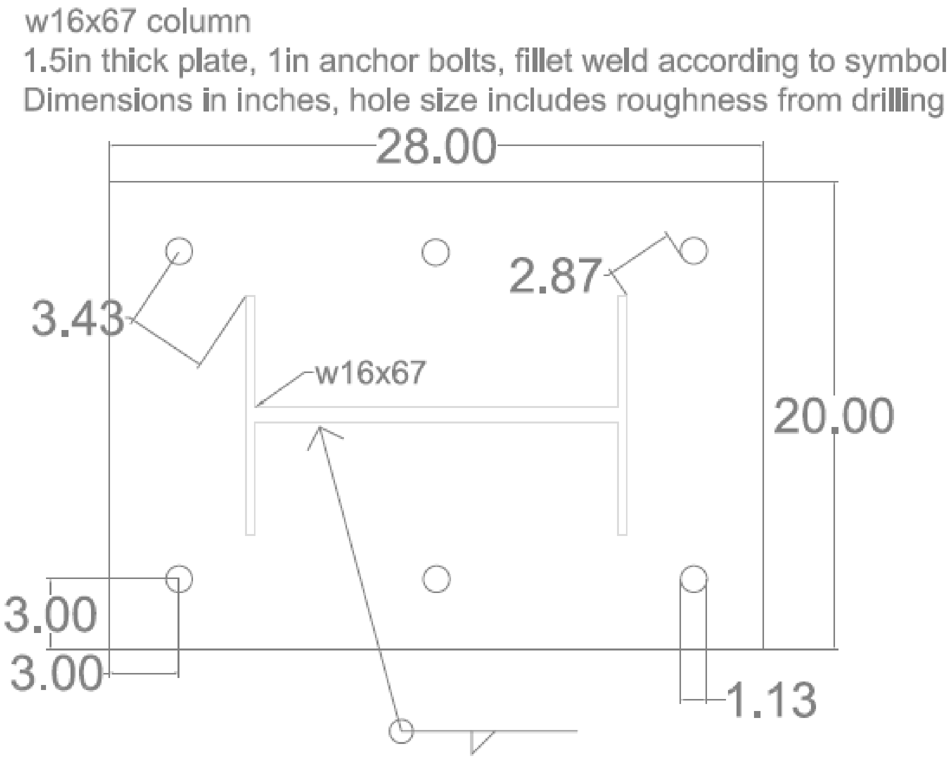

Figure 33: Base plate Design

4.3 Concrete Parking Structure Design

The parking garage results are presented in the following sections beginning with the top of the parking garage, which is also the floor of the academic building. The parking space layout was governed by the design of the structure. The parking layouts are shown in Figure 34, Figure 35, and Figure 36 from 11.5ft below grade to 11.5ft above grade.

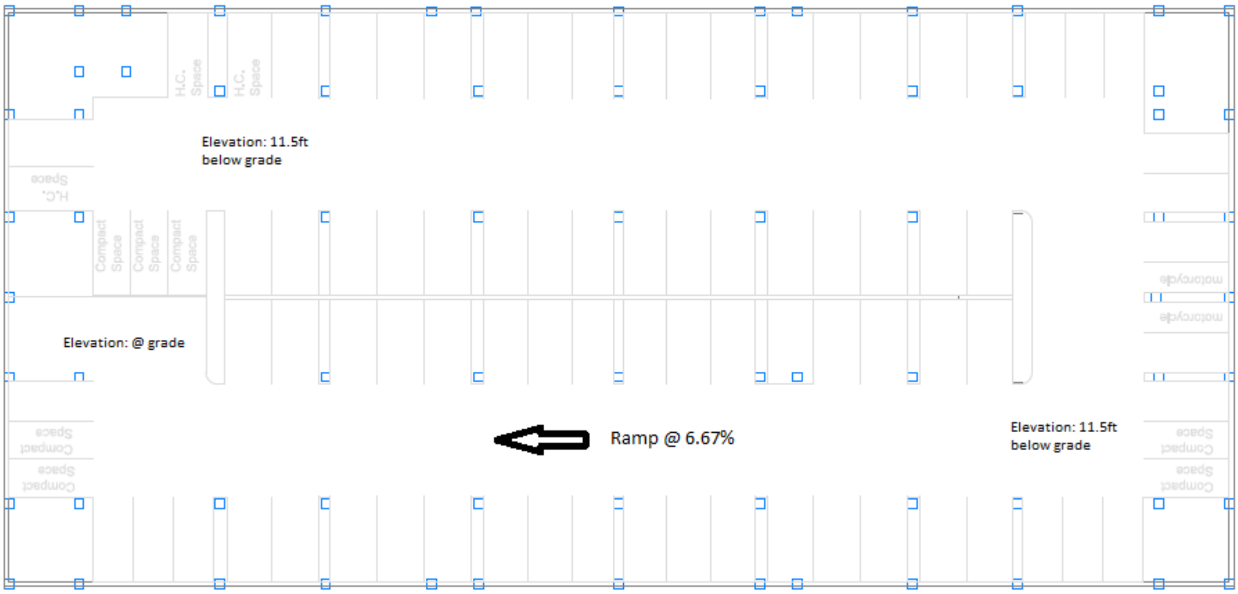

Figure 34: Garage Plan View Basement Level

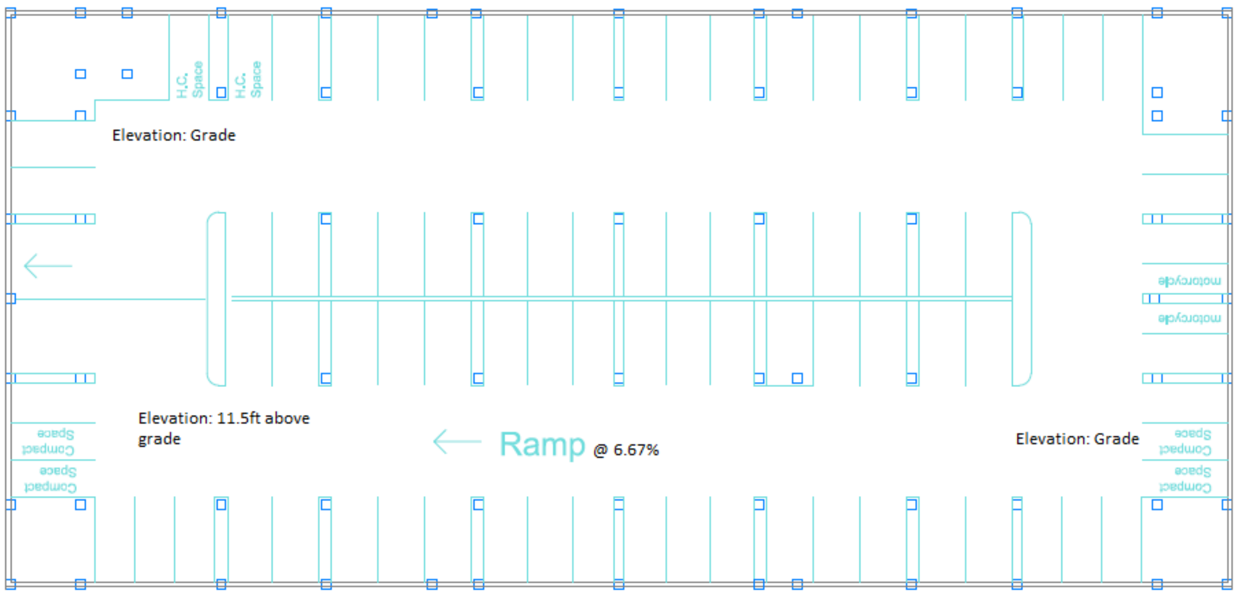

Figure 35: Garage Plan View First Level

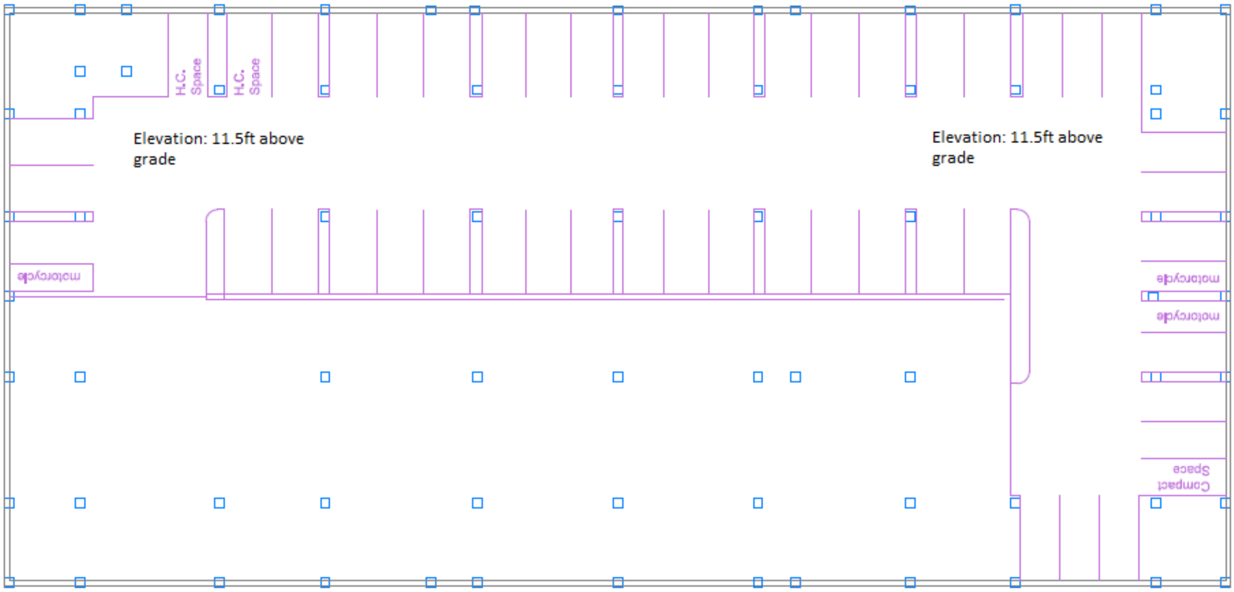

Figure 36: Garage Plan View Second Level

4.3.1 Academic First Floor Design

The structure that forms the first level floor of the academic building was determined to be formed of a 2 in. lightweight concrete slab and pre-topped light weight double-tee beams. According to ACI 216.1-97 code Table 3.1 a 3.6 in slab depth for light weight concrete is required for 2-hr fire resistance rating [3]. The selected double-tee beam, shown in Table 4, is adequate for the live and dead loads applied in the first floor of the academic building.

The load values for double-tees presented in the PCI Design Handbook take into account all pre-stress losses during fabrication and the length of the member's lifetime. Do the large live and dead loads on the floor these beams will not have dapped webs. Figure 48.

Table 4: Acedimic First Floor Double-tee Selection

Double-tee Selected	10LDT34
Allowable Safe Superimposed Service Load	187 psf
Strand Pattern	208-D1
Estimated camber at erection, in. **Estimated long-time camber, in.**	3.5 in 4.8 in
10-10 foot wide beam	LDT-lightweight concrete double-tee
PCI Table	Appendix H

Due to the cambering of the double-tee beams a raised floor system should be used to provide a level first floor for the academic building.

4.3.2 Column Design

The Excel spreadsheet showing the column design is in Appendix K. First, we determined that the height of the interior columns would require slenderness to be considered. We conducted the design following the non-sway criteria as dictated in Chapter 10 of ACI 318-08 [4]. We designed the columns to all be uniform in size to reduce the overall cost of the project by increasing the constructability of the design. The main difference in the design of the interior versus the exterior columns lies with the effective length of the columns. Although both column locations are 34.5 feet long, the exterior columns are braced by the 60"x18" exterior girders. Therefore the effective length of the exterior columns is only 11.5 feet.

We designed a 30"x 30" column with a 3 inch concrete cover on all sides in order to create a conservative design for weather and fire. We specified 4,000psi concrete for the columns. The steel layout consists of four #10 and eight #9 axial bars tied with #4 bars 10" on center and #3 intermediate ties also 10" on center. This amounts to about 1.4% steel in the columns. This column design supports the maximum axial loads and turning moments for both the interior and exterior locations.

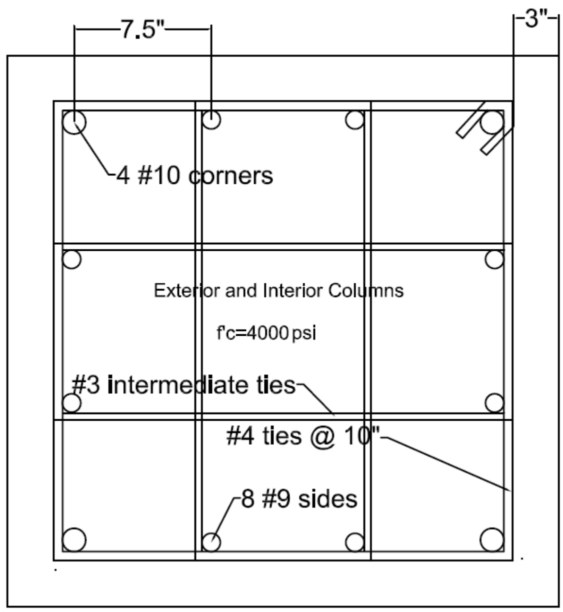

Figure 37: Interior and Exterior Column Cross Section

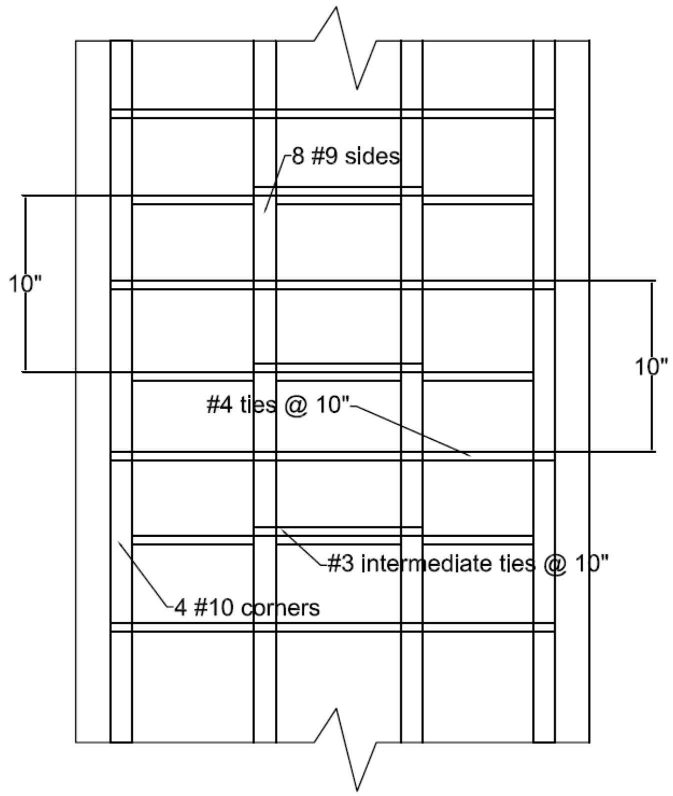

Figure 38: Interior and Exterior Column Elevation

4.3.3 Garage Floor Design

The double-tees for the garage floors were selected from the PCI Design Handbook double-tee tables. The live load is 45 psf and the dead is 15 psf for Mechanical, Electrical, and Plumbing (MEP). Our selections from the PCI tables (shown in Appendix H) are listed in Table 5 [4].

Table 5: Double-tee selection

Double-tee Selected	10LDT34
Allowable Safe Superimposed Service Load	85 psf
Strand Pattern	168-S
Estimated camber at erection, in. **Estimated long-time camber, in.**	1.7 in 2.4 in
10-10 foot wide beam LDT-lightweight concrete double-tee 34-Depth of web	

PCI Table	Appendix H

The selected double-tee is adequate for the service load requirements of the garage. The load values for double-tees presented in the PCI Design Handbook take into account all pre-stress loses during fabrication and length of the member's lifetime. In Addition, we will specify to the fabricator and the double-tees have dapped ends. (Figure 17) These dapped ends act in a similar way as coped steel beams. The double-tee elevation section is shown in Figure 38.

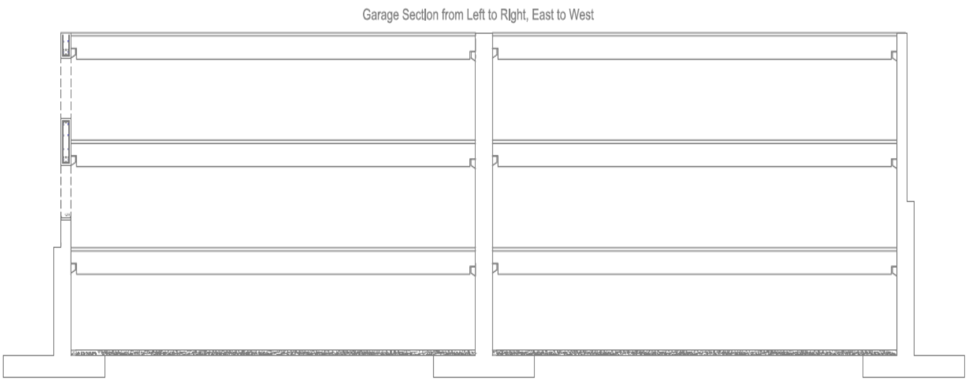

Figure 39 Garage Section

Figure 40 Double-Tee dapped web

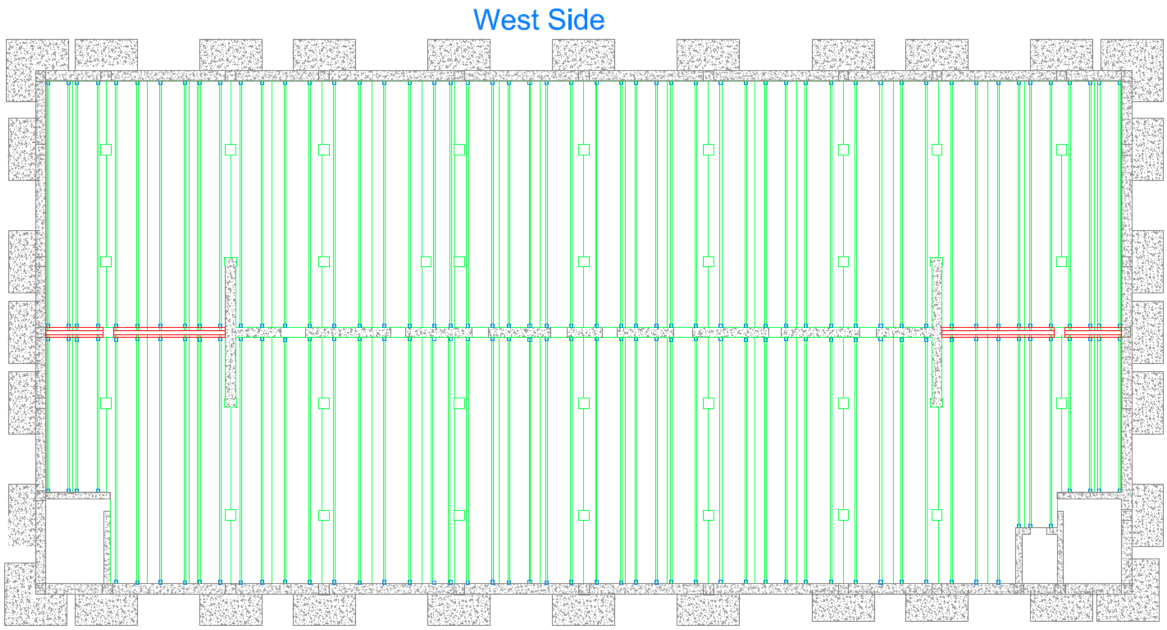

Figure 41: Garage level double-tee layout

Table 6: Garage Floor Labels

Green	10' & 12' Double-Tee Beams
Red	28' & 16' Inverted-Tee Beams
Blue	Corbels
Gray and Shaded Gray	Garage shear walls, supporting walls, retaining walls, exterior column footings

4.3.4 Girder Design

The results for concrete girder design are divided into exterior girders and beams. The exterior girders serve two purposes. First, the girders support the corbels which support the double-tees. Secondly it provides a barrier for the parking spaces around the perimeter. The team selected these girders to be a 60" deep and 18" wide. These dimensions would allow for the beam to serve all purposes in one element. The area of the steel was found to be 6 square inches and was determined according to the span between the columns, self-weight and the number of corbels that it would support. The largest exterior span is 30 feet which supports 6 corbels with a combined load of 155 kips. Due to the shear from the loads the stirrup steel was determined to be #4U bars spaced at 10 inches. The torsional steel was required due to the moment created by the corbels. 8 #6 bars were placed in the longitudinal direction and the stirrups were #4 U bars @ 10o.c. The calculations are placed in appendix G and the steel layouts are shown in Figure 42 and Figure 43.

Table 7 East Girder Steel Summery

Girder Design	
Width	18 in
Torsional Steel	
#4 U stirrups @ 10in o.c. (Overlap 14in)	
8 #6 Bars	
Flexural Steel	
9 #9 Bars (6 bottom, & 3 top)	
Shear Steel	
#4 U stirrups @ 10in o.c. for 10ft from end of beam Overlap 14in)	

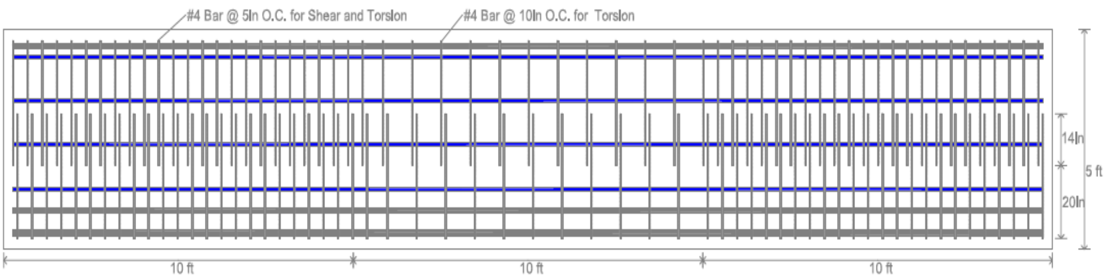

Figure 42: East Side Exterior girder steel layout

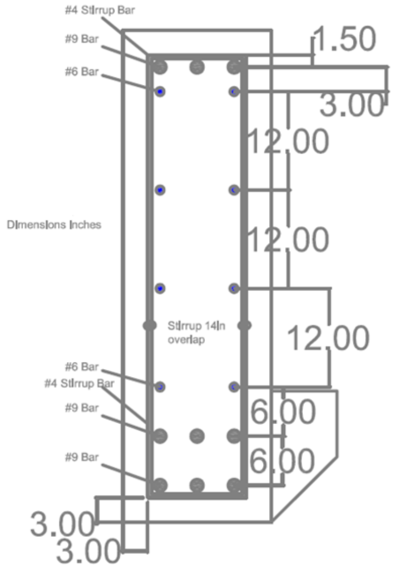

Figure 43: East Side Girder End View

The design of the North and South Beams is exactly similar, except for the absence of torsional affects, and therefore only flexural steel is required. Table 8 summarizes the steel reinforcement and Figure 44, Figure 45 show the steel layout.

Table 8 North and South Beam Steel Summary

North and South Beam Design	
Width	18 in
Flexural Steel	
9 #9 Bars (6 bottom, & 3 top)	
Shear Steel	
#4 U stirrups @ 10in o.c. for 10ft from end of beam Overlap 14in)	
#4 U stirrups @ 24in o.c. for Center of beam 10ft Overlap 14in)	

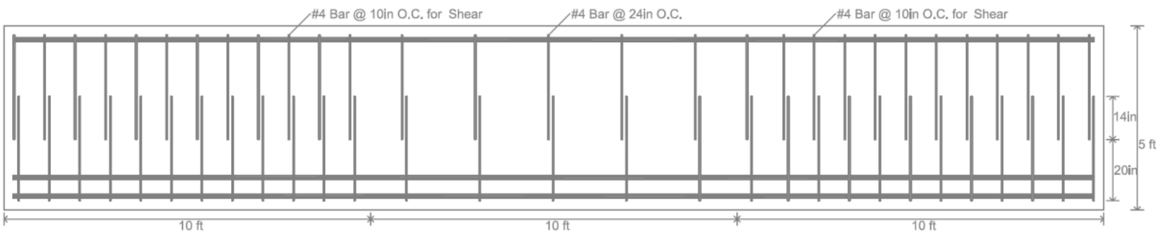

Figure 44 North & South Beam Steel Layout

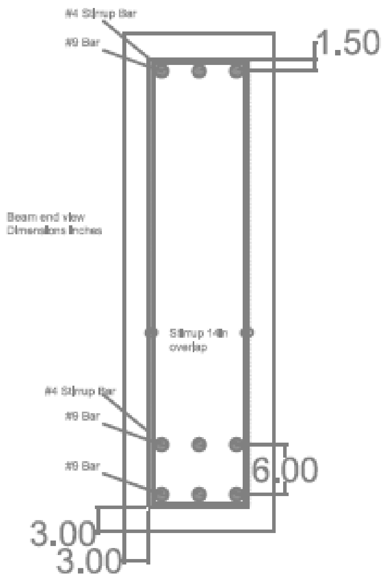

Figure 45: North and South Side Beam End View

4.3.5 Corbel Design

The corbels were designed to support the loads from the double-tee flanges. Our team calculated the largest load to be 43.44 kips for a 10 foot wide double-tee. The calculations of the corbel and the area of steel are in Appendix J.

Table 9: Corbel Design

Corbel Load	25.8 Kips (for 10 foot wide DT)
Width (From front)	8 in
d (depth of steel from bottom of corbel)	9 in
Height of Corbel	12 in
Bearing depth (from face of beam) > 0.5d	8 in
Front vertical depth of corbel	8 in
Spacing distance of steel < 2/3d	8 in
Area of steel for flexure and direct tension A_{sc}	0.93 in^2 (3 #5 Bars)
Area of Hoop Steel A_h	0.66 in^2 (3 #3 Bars)

In addition to the reinforcing steel a steel angle will be attached on the corner of the corbel as shown in Figure 22 with studs. The chosen angle properties results are shown in Table 10.

Table 10: Steel angle for corbel

Steel Angle Design	
Width of angle (From front of corbel)	6 in
Bearing depth	6 in
Vertical depth	4 in
Vertical width	4 in
Angle Thickness	3/8

4.3.6 Inverted-Tee Beam Design

The inverted-tee Beams were selected from the PCI Design Handbook Inverted-Tee Beam tables. The live and dead loads for the appropriate floors were applied to the double-tees, which sit on inverted-tee beam.

Our selections from the PCI tables (shown in Appendix I) are listed in Table 5 [4].

Table 11: Inverted-Tee Beam Selection

Inverted-Tee Beam Selected	Garage 1st Level	Garage 1st Level	Garage 2nd Level	Garage 2nd Level
Live and Dead Load	Live: 45 psf Dead: 83 psf (Includes Double-Tee weight)	Live: 45 psf Dead: 83 psf (Includes Double-Tee weight)	Live: 100 psf Dead: 108 psf (Includes Double-Tee weight)	Live: 100 psf Dead: 108 psf (Includes Double-Tee weight)
Beam Notation	28IT40	28IT28	28IT52	28IT52
Allowable Safe Superimposed Service Load	8630 lb/ft	8305 lb/ft	9980 lb/ft	9980 lb/ft
Number of Strands	19	13	24	24
End Y_s	4.21 in	3.08 in	5.17	5.17
Span	28 ft	14 ft	28	14 ft
Flange Depth increased from	No change	No Change	16 in to 22 in	16 in to 22 in
28-28 in. wide beam IT-Inverted-Tee Beam 40-Depth of beam				
PCI Table Appendix I				

Both selected inverted tees are adequate for the service load requirements of the garage and academic first floor. The load values for inverted-tee beams presented in the PCI Design Handbook take into account all prestress losses during fabrication and the length of the member's lifetime. The inverted beams locations are shown in Figure 41. Due to the load difference on the floors and different spans large depths had to be selected and therefore precast beam flange will need to be increase as specified in Table 11 to allow for the double-tee beams to be level with the top of the inverted tee beam.

4.3.7 Shear Wall Design

Reinforced concrete shear walls were used to resist the horizontal forces acting on the building. Appendix L shows the results of the shear wall design. The boundary elements consist of having an l_w = 34', h_w = 35', and an h = 18". There are two of these walls pictured in Figure 46 in the east to west plane (top to bottom in the figure). The design states that 4 ksi concrete is necessary in addition to the vertical and horizontal stirrup steel reinforcement.

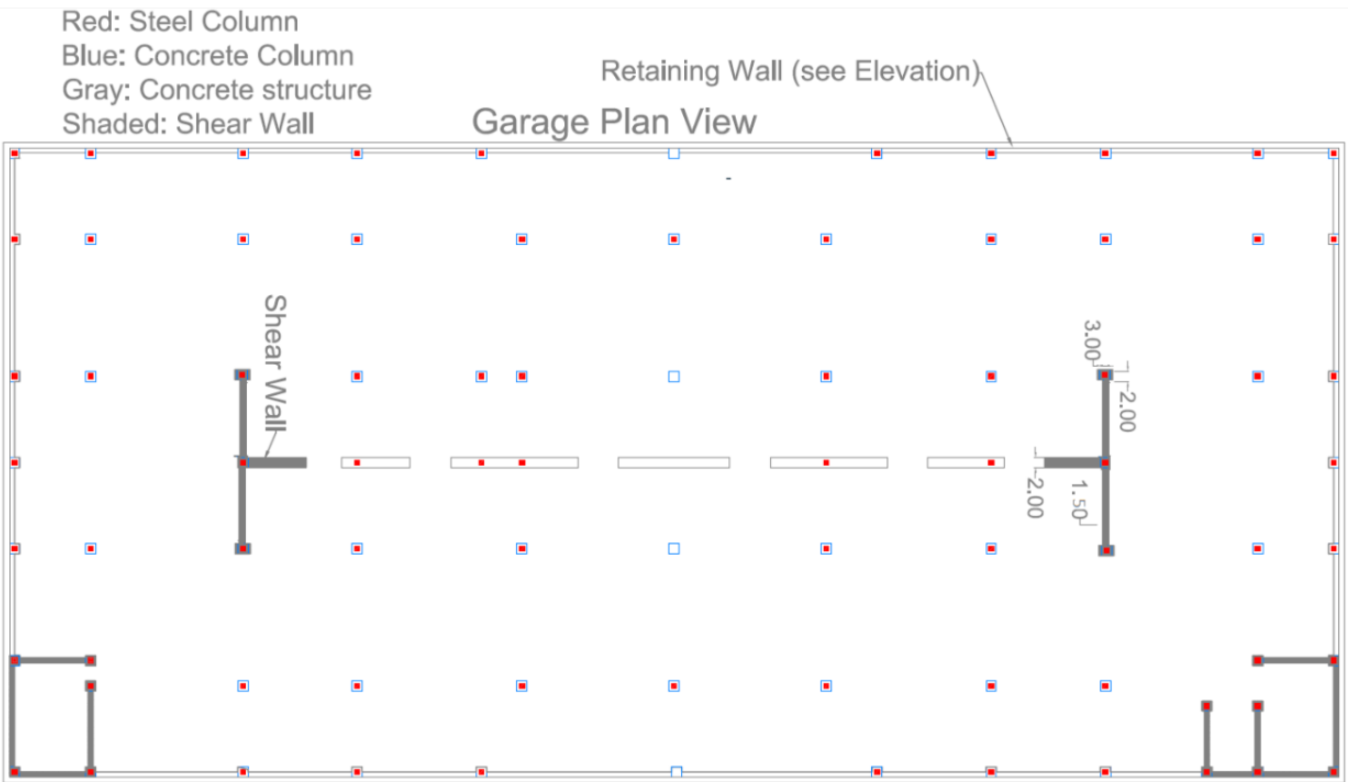

Figure 46: Shear Wall Diagram

4.3.8 Retaining Wall Design

The retaining structures in the garage consist of three types. The west side is 34.5 ft tall (Figure 48). The north and south walls are 14.5 ft tall (Figure 47) and the east wall is also 14.5 ft tall and only differs from the north and south walls by the corbel layout (Figure 49).The west side of the wall extends all the way to the base of the steel structure, while east, north and south sides only extend to the base of the first floor of the parking garage.

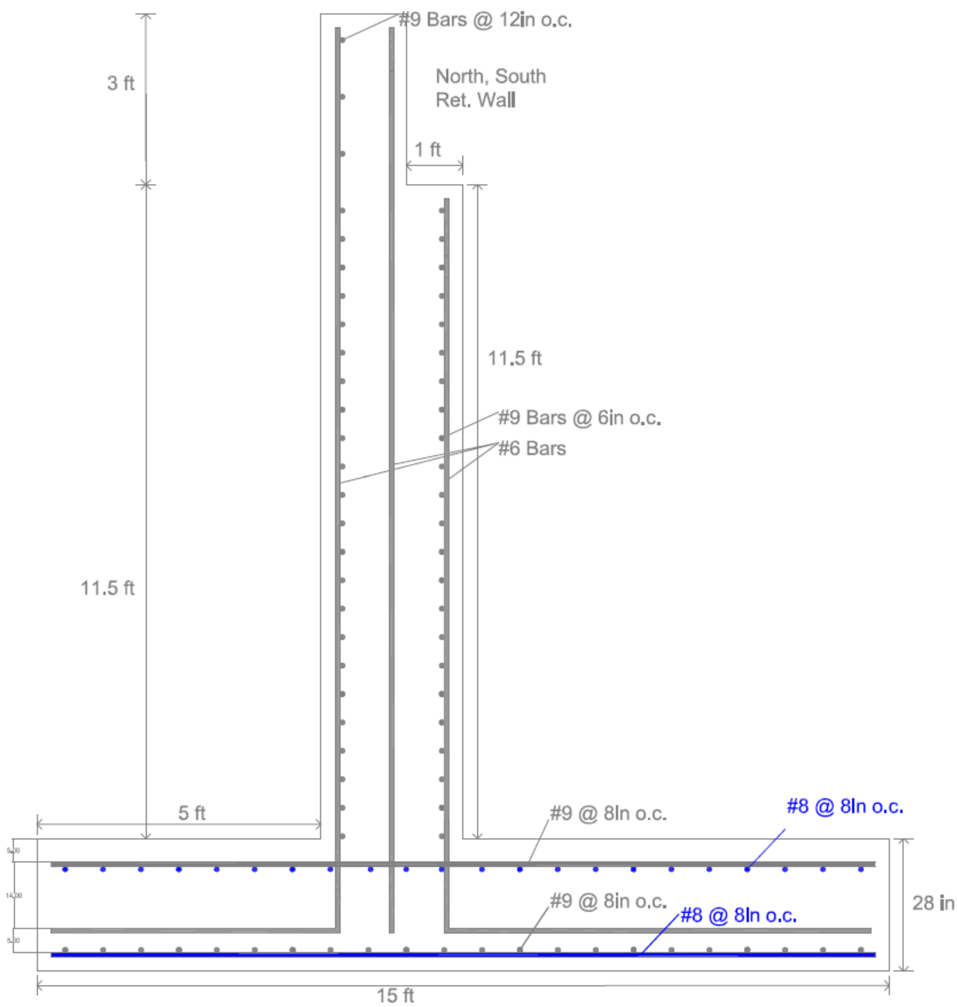

Figure 47: East & North Side Retaining Wall

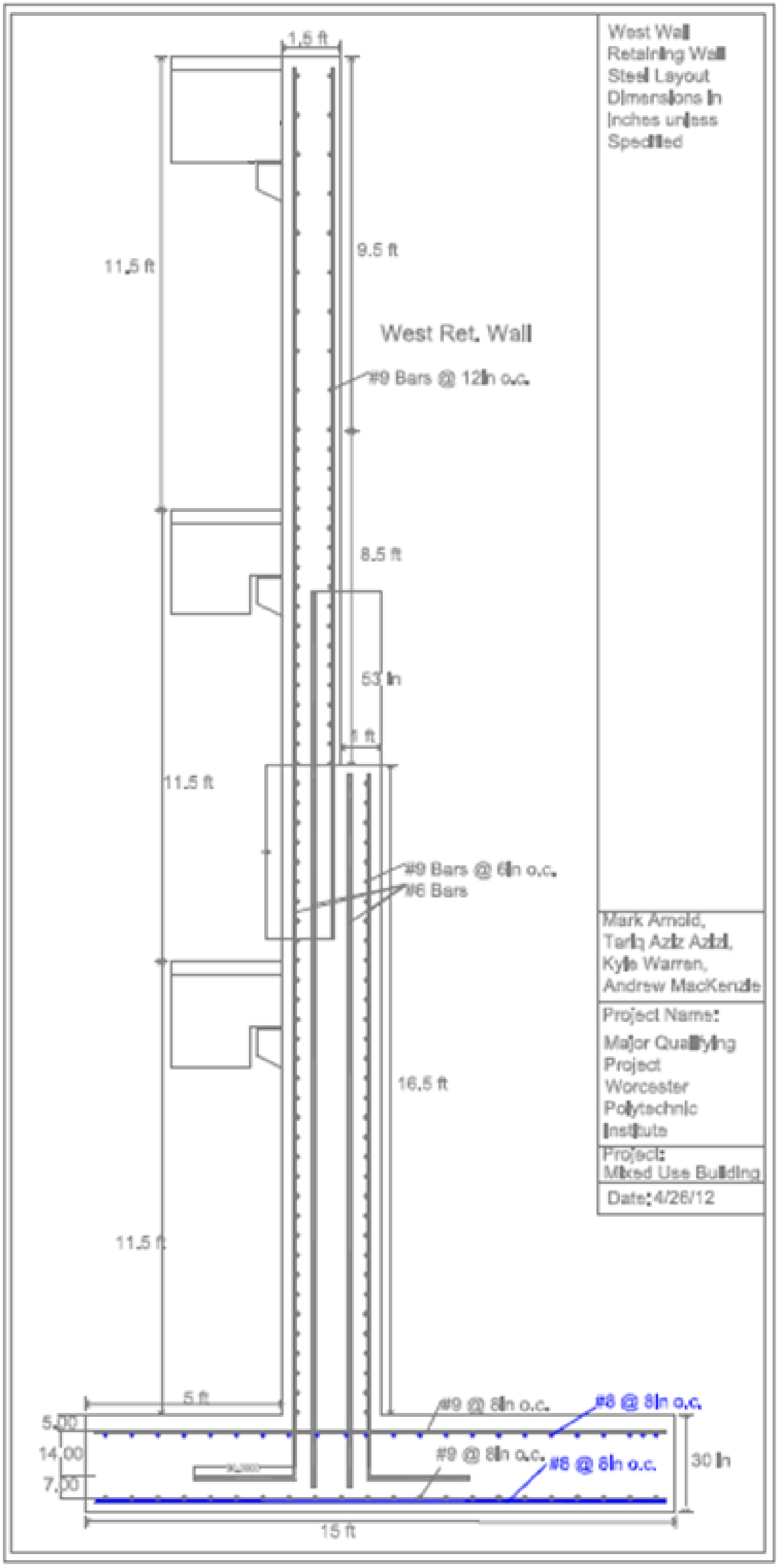

Figure 48: West Side Retaining Wall

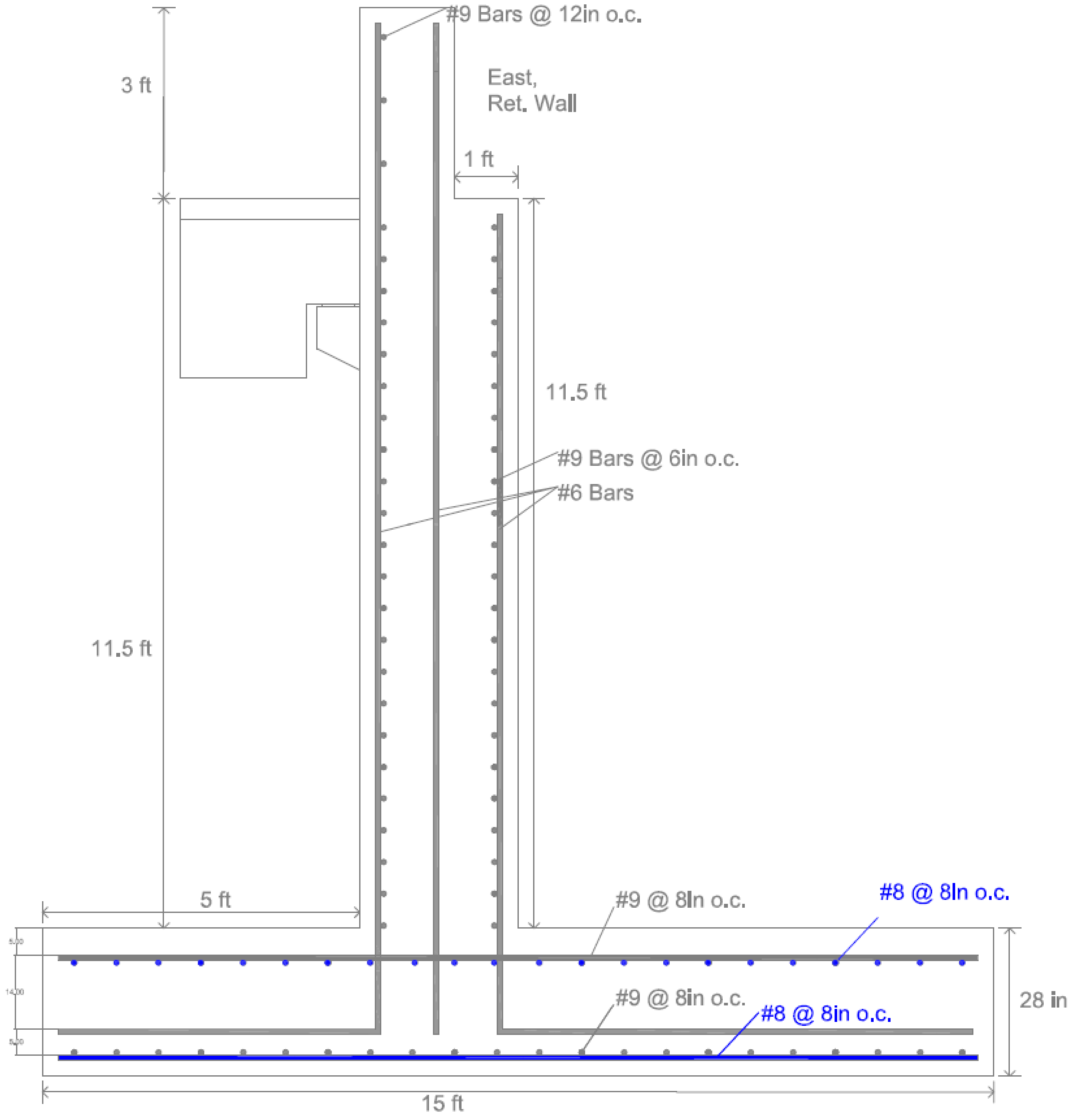

Figure 49: East Side Retaining Wall

4.3.9 Foundation Design

As mentioned earlier, our team utilized information from adjacent build sites for our geotechnical data. For the soil conditions shown in Appendix A, the net allowable soil bearing pressure was 4 ksf according to Haley & Aldrich's East Hall report [5]. From this consideration we completed the design of the foundation. Due to the design gravity load of 800 kips, shear load of 20 kip, and a moment of 100 kip-feet the initial footing size was 15 square feet. The design of a footing took into account the gravity, moment and shear loads on the connecting column. The design also checked for soil pressure, soil cover, sliding, overturning, punching shear, and development length.

The results are summarized in Table 12, and the complete calculations are in Appendix M.

Table 12: Foundation Results Summery

Column footing	
Depth	30 in
Width	15 ft
Length	15 ft
Longitudinal reinforcement	
Number of Bars at top section	#9 bars spaced @ 8" o.c.
Number of Bars at bottom section	#8 bars spaced @ 8 " o.c.
Transverse flexural reinforcement	
Number of Bars at bottom section	#9 Bars spaced @8" o.c.
Number of Bars at top section	#8 bars spaced @ 8 " o.c.
Dowel Steel (Figure 51: Column Dowel Configuration)	8 #8 bars, d_l (vertical) = 53 in, Lab splice for column steel = 30 in
F'c = 4000 psi	

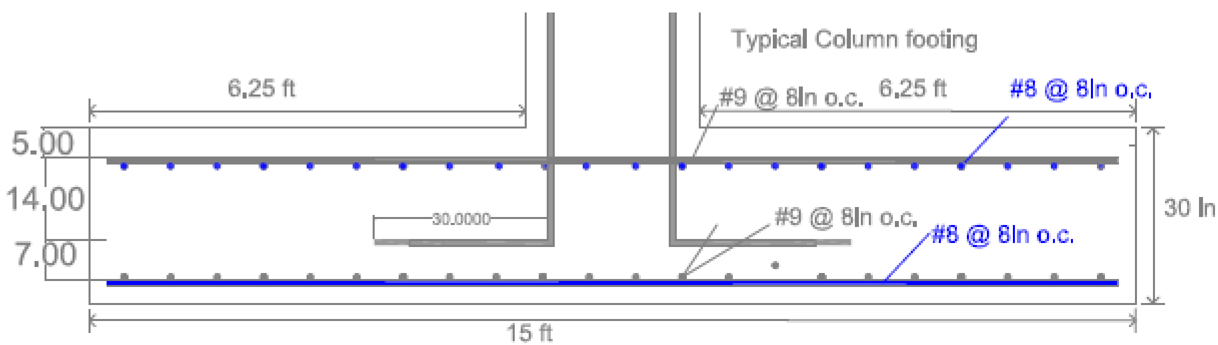

Figure 50: Combined Footing Design Section

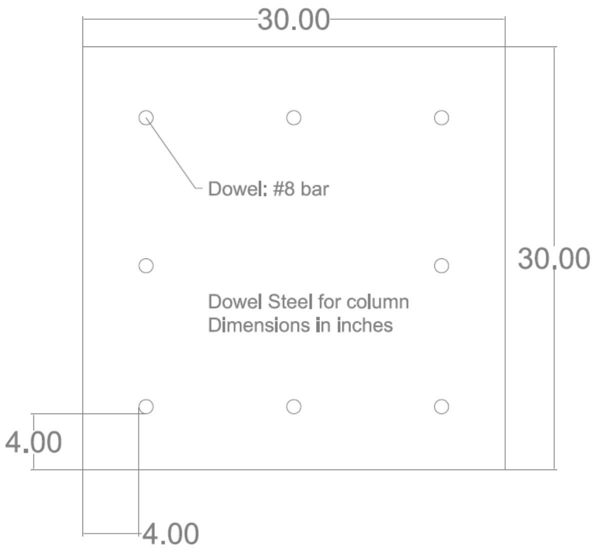

Figure 51: Column Dowel Configuration

From the Hadley and Aldrich geotechnical report it was suggested that a perimeter foundation drain and under slab systems should be implemented to cope with the storm water and the water table which was found to be only 13.8 ft below the sites surface [5]. We recommend that the design includes a system identical to Figure 52.

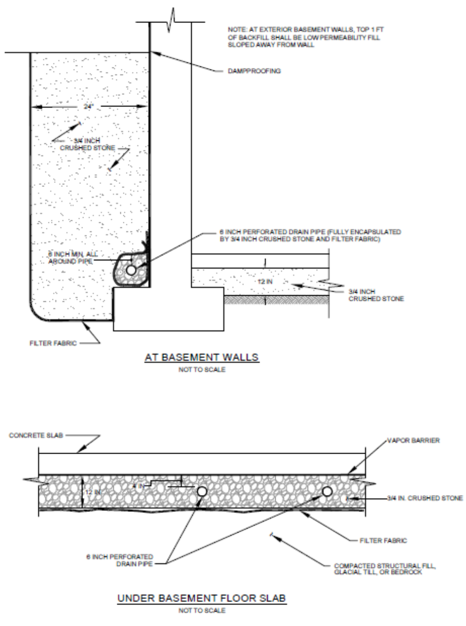

Figure 52: Foundation Drainage System [5]

4.4 Cost Estimation

The preliminary cost estimate put the building at a cost of $42,000,000 with a $307.42 per square feet for the steel building and $125.9 per square feet for parking garage. Appendix O shows the cost breakdown for each section, and RSMeans CostWork software was used to calculate the total cost of the structure. The cost estimate also factored in the larger foundation, retaining walls, concrete columns, landscaping, deep excavation, and site work. This resulted in a higher cost than what would be typically expected in a similar parking garage. However the cost estimation is preliminary and does not include detained furnishing.

5.0 Conclusions/Recommendations

As stated earlier in the background section, WPI must plan for its expansion and development in order to continue its successful operation. However, with the continuous demand for faculty and facilities from the ever increasing student body, it has become hard to achieve all of WPI's objectives with one project. However, with this project, our team aims to offer WPI a realistic and beneficial opportunity.

5.1 Cost Considerations

The cost of the building and construction are not the only cost effects when considering a project. The initial cost of $42,000,000 does not take into account the cost of disrupting the area and current parking lot. The loss of over 200 parking spaces for at least two years would increase the current parking problems and also reduce revenue from commuter students who use the current lot. The severe disruption to pedestrian traffic could also have implication as the walk way adjacent to the library would be inaccessible due to the construction site.

The longer term cost of a building is another major cost consideration. The addition of a building's utility bill, maintenance and staff should be considered before construction can begin. Despite all of the costs associated with the project, WPI must remember the important role it will fulfill.

5.2 Proposed Timeline

Depending on WPI's current financial resources, this project would be best suited to be completed as soon as possible. WPI's undergraduate and graduate student populace continues to grow annually with no foreseeable change in sight. This will place an even greater pressure on the School of Business who already has trouble scheduling classrooms for its students. The department, which boosts WPI's overall prestige when compared to similar universities, will be inhibited from progressing.

With respect to parking, WPI currently has several small parking lots. The lack of sufficient parking space is evident when walking down Boynton Street or Institute Road during business hours. Many people who purchase WPI parking permits often have to search the rows of the WPI library parking lot for that one spot that is not occupied.

Our team believes that WPI should consider this design as a legitimate long-term solution. In addition, it is in WPI's best interest to review this design, brainstorm alterations, and finally set the plan in motion as soon as possible. We recommend that WPI aims to have the facility completed by August of 2015 in order for it to be used for the 2015-2016 academic year. Problems never get better with time; therefore, WPI should consider this design as a realistic option which should materialize in the near future.

5.3 Design Considerations

Although this project addressed the major structural design of this academic building and parking garage, there are some points that should be addressed for future consideration. Our goal was to obtain the most economical design while keeping a maximum number of parking spaces. Our initial goal to transfer the steel columns through the use of transfer beams was found to be expensive. This system could be further studied if WPI is willing to pay the extra cost for the additional parking spaces that this system provides.

Another possible option that should be further researched is whether adding another level to the parking garage as suggested in our report could be economically reasonable. In addition, several obstacles associated with making this happen would be the ground water table, increased excavation costs and possible site complications. However, this option would increase the parking capacity and overall benefit to WPI.

This design increases the parking capacity while reducing the overall footprint. There are a total of 219 car spaces, which include 7 handicapped and 12 compact spaces. It is also important to note that many of the spaces are wider than required by code and therefore can easily suit wider vehicles. There are also an additional 8 motorcycle spaces which are not included in the number of car spaces. This integrated structure would not only increase WPI's academic and parking capacity but would also be LEED certified. It would also create a green space where the existing Gordon Library parking lot is located.

5.4 Final Recommendations

After conducting all of our research and creating our final design, our team has several recommendations for WPI. These recommendations outline the necessary steps WPI must complete in order to successfully construct and implement the structure into WPI's community. After completing all of the following steps, WPI will have a highly functional and purposeful building on its campus which will serve WPI in years to come.

Review and analyze the overall purpose and specifications of the included design.

WPI should consider this design and distribute it to all parties who would either be involved in its development or who are interested in its purpose. Several parties who should be notified of the design include WPI's Facilities Department and School of Business.

Once the design is properly disseminated, respective parties such as the board of trustees, should communicate amongst one another to converse about their opinions. These reviewers should contemplate the benefits and drawbacks of the proposal in order to consider the overall feasibility of the design. After considering all relevant aspects, WPI officials should note the merits of our design and conclude that this design is the best-use option for WPI to develop in the near future.

Coordinate with construction firms and contractors to develop an attainable construction plan.

Assuming WPI approves the design, WPI must find a suitable construction company to complete the plan. For this, WPI needs to decide which method of construction it would like to follow. As an initial design is provided with this design, our team recommends that WPI follows a typical Design-Build process so that all desired features comes to fruition. During this time, WPI should finalize a project schedule with the contractor in order to establish an estimated completion date early on.

In addition to the schedule, WPI should also agree to a final cost of the project with the general contractor. Before the total cost is finalized, WPI and the contractor must anticipate possible obstacles which may occur during the design process. This will set the construction up for success and set the entire project off in the right direction.

Follow through with the construction of the mixed-use building.

Once the logistical setup for the project is completed, WPI must follow through with the construction according to its established schedule. All contractors and WPI officials must aim to stay on time throughout the process. At the same time, all parties involved must be flexible and expeditiously resolve any unforeseen problems or hurdles. WPI must follow through with the construction phase until the entire project is completed according to the design specifications.

Create a plan for the transfer of the School of Business to the new academic building.

WPI needs to ensure that the School of Business can smoothly move into the new building as soon as the construction is completed. If WPI plans to have the building operational by the beginning of the 2015-2016 academic year, an efficient facility transfer will be necessary in order to avoid delays. In addition, the vacated Washburn wing which previously housed the School of Business will be able to be used for other purposes. This will provide room for the Mechanical Engineering Department to expand its facilities in the building.

Construct a specific building maintenance plan to guarantee the building's longevity.

Finally, WPI must appropriate and budget sufficient resources to preserve this asset. WPI Facilities should routinely inspect and evaluate the best maintenance methods. Any problems should be addressed at the time of discovery in order to prevent premature deterioration over time.

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Appendix A

Haley & Aldrich East Hall Geotechnical Report

1. INTRODUCTION

1.1 Site Location and Surface Conditions

The project site is located between Boynton Street (to the west) and Dean Street (to the east), on the WPI campus in Worcester, Massachusetts (see Figure 1, Project Locus). The site is bordered to the north by The Church of Our Saviour of Worcester. To the south of the property is the multi-story masonry Founders Hall and a student parking lot.

The site is currently occupied by a two-story masonry building utilized by the campus police, a two-story masonry building utilized by the Human Resources Department, a three-story masonry institutional building, a one-story garage, and an asphalt-paved parking lot.

The site is relatively level with surface grades ranging from approximately El. 500 to El. 503.

1.2 Proposed Development

The proposed development consists of a student residence building and a parking structure. The residence building is planned as a 5-story structure, with the lowest level finished slab near existing site grades, at El. 502.6. Maximum building interior column loads, provided to us by Cannon Design on 29 December 2006, are on the order of 300 to 350 kips (interior columns) and 250 kips (exterior columns).

The parking structure is planned with a portion of the lowest level below grade, at El. 494.8, as provided by Cannon Design on 19 January 2007.

1.3 Elevation Datum

Elevations in this report are in feet measured relative to the National Geodetic Vertical Datum of 1929 (NGVD 1929).

1.4 Design Team

The design team for the project consists of the following participants:

Owner:	Worcester Polytechnic Institute
Architect/Structural Engineer:	Cannon Design
Civil Engineer:	Cullinan Engineering Co. Inc.
Geotechnical Engineer:	Haley & Aldrich, Inc.

2. SUBSURFACE EXPLORATIONS AND LABORATORY TESTING

2.1 Test Borings

A total of eight test borings (B1 through BS) were conducted at the project site between 2 and 4 January 2007. The test borings were drilled by Seaboard Drilling of Springfield, Massachusetts, and were observed by a Haley & Aldrich geologist: The test borings, located as shown on Figure 2, ranged in depth from 23.5 to 38.3 ft. The test boring logs are included in Appendix A.

A groundwater observation well was installed in test boring B7 to permit observation of stabilized water levels. The observation well installation report is included in Appendix B.

Ground surface elevations at exploration locations were approximated from available site topographic information provided by Cannon Design.

2.2 Laboratory Soil Tests

Limited laboratory grain size analyses (Percent Finer than No. 200 Sieve) were performed on two samples of the naturally-deposited soils obtained from the borings. The purpose of the tests was to aid in soil classification and to evaluate the likelihood of liquefaction or seismically-induced settlements to occur for any loose, "clean" (low fine-grained soil content) sandy soils observed at the site. The results of the laboratory tests are summarized below:

Exploration Designation	Sample	Depth	Stratum	Percent Passing No. 200 Sieve
B1	S4	15-17 ft	Glaciolacustrine Deposit	44.6
B3	S4	15-17ft	Glaciolacustrine Deposit	47.0

3. SITE AND SUBSURFACE CONDITIONS

3.1 General

In general, the subsurface explorations conducted for this study revealed soil and groundwater conditions that are consistent with our local experience and understanding of site area geology. The subsurface conditions typically consist of sandy glaciofluvial soils overlying less dense silty glaciolacustrine soils.

3.2 Subsurface Soil Conditions

The explorations encountered the following sequence of subsurface units, listed below in order of increasing depth below ground surface. A summary of the conditions encountered in each of the explorations is included in the attached Table I.

Fill: A layer of fill was encountered in all of the test borings, ranging in thickness from 5 to 6 ft. The fill consisted of medium dense to dense poorly graded SAND, with varying amounts of gravel and silt. At several locations the fill consisted of SILT.

Glaciofluvial Deposits: Glaciofluvial deposits were encountered in most explorations beneath the fill. The glaciofluvial deposits ranged in thickness from 3.5 to 9.5 ft and consisted of medium dense to dense, well graded or poorly graded SAND, with varying amounts of gravel and silt.

Glaciolacustrine Deposits: Glaciolacustrine deposits were encountered in most explorations beneath the glaciofluvial deposits. The glaciolacustrine deposits ranged in thickness from 5.5 to 21 ft and consisted of loose to medium dense, silty SAND.

Glacial Till: Each boring was terminated within the glacial till deposits. The glacial till consisted of medium dense to very dense, silty SAND with gravel, silty GRAVEL with sand, or sandy SILT, occasionally bonded. The depth to the top of the glacial till ranged from 13.5 to 29.5 ft (El. 473 to El. 488.5).

No confirmed bedrock was encountered in test borings, although refusal was encountered at several locations.

3.3 Groundwater

Boring B7 was completed as a groundwater monitoring well at the proposed parking structure. Groundwater was observed at a depth below grade of 13.8 ft (El. 488.7). Water levels vary with precipitation, season, and other environmental factors. As a result, groundwater levels encountered during and after construction may differ from those observed during the recent explorations.

4. DESIGN CONSIDERATIONS AND RECOMMENDATIONS

4.1 General

Earthwork for construction of foundations and slabs for the proposed residence building (lowest floor El. 502.6) will require excavations of approximately 4 ft in existing fill soils.

Construction of foundations and slabs for the parking structure (lowest floor level El. 494.8), will require excavations up to 12 ft deep in various soil deposits.

Bedrock is not anticipated to be encountered within the depth of excavation required.

Depths of cuts are referenced herein to the footing levels. Some additional, localized excavation will be required for construction of elevator pits and utilities.

4.2 Foundation and Slab Design Recommendations

The existing pavement, topsoil, fill, and existing structures are unsuitable for support of the proposed building structural loads. The proposed building and parking structure walls, columns, elevator pits, and other structural elements should be supported on conventional reinforced concrete footing foundations bearing in the naturally-deposited silty and sandy bearing soils (glacial soils). Where required locally, compacted engineered fill (Compacted Granular Fill described below) should be placed beneath structures after removal of any unsuitable materials.

Fill was observed in the test borings to depths of 5 to 6 ft below ground surface (bottom of fill approximately El. 495 to 497). With the building floor slab at El. 502.6, "typical" footing depths would result in interior footings bearing at El. 501.1 and exterior footings bearing at El. 498.6. Therefore, some over-excavation to remove the existing fill beneath proposed footings will be required. The over-excavated depth should be backfilled with Compacted Granular Fill.

As a general guideline, foundation design and construction must conform to the applicable provisions of the latest edition (Sixth) of the Massachusetts State Building Code (Code). Specific recommendations for foundation design follow:

- Footings should be designed using a maximum net allowable bearing pressure of 4.0 ksf.

- For footings with a least lateral dimension (width) Jess than 3 ft, the maximum allowable bearing pressure should be reduced to a value equal to one-third of the recommended allowable bearing pressure multiplied by the least lateral footing dimension in feet. For example, a 1.5-ft wide footing should be designed using an allowable bearing pressure equal to 1/3 x 4.0 ksf x 1.5 = 2.0 ksf.

- Footings should have a least lateral dimension of 18 in. or greater.

- Bottoms of footings should be positioned at least 4 ft below adjacent ground or slab surface exposed to freezing. Footings in heated interior locations should bear at least 18 in. below the adjacent slab surface.

- Footings should bear below a reference line drawn upward and outward on a 1.5H: IV slope from the bottom of any adjacent utilities or underground structures.

- The lowest building floors may be designed and constructed using typical concrete slab-on-grade construction, bearing on a minimum 8-in. thick layer of Compacted Granular Fill, after removal of pavements, existing structures, organic matter, or other visible unsuitable materials.

- A minimum 4-in. thickness of Compacted Granular Fill should be provided between tops of footings and the bottom of slabs. Stones larger than 2/3 of the lift thickness should be removed from the fill between the footings and the slab.

- Compacted Granular Fill below footings and slabs should be placed within the zone beneath imaginary lines extending 2 ft laterally beyond footings and slabs and down on a 1H:1V slope to the top of the natural inorganic bearing soils.

- The soils below the proposed buildings are judged not susceptible to liquefaction.

- For seismic design, the recommended soil profile type is S3, with a site coefficient equal to 1.5.

- A coefficient of friction between concrete and bearing soil of 0.4 should be used to calculate ultimate sliding resistance. A factor of safety of at least 1.5 should be applied to calculate the allowable sliding resistance.

4.3 Settlements

At the recommended allowable bearing pressures, it is anticipated that the settlement of individual new footings bearing on the natural foundation bearing soils and constructed as recommended herein, under static loading conditions, will not exceed about 3/4 in. (total and differential settlement). It is expected that most of the settlement will occur during construction as building loads are added.

4.4 Foundation and Slab Drainage and Waterproofing

The lowest floor level in the proposed residence will be slightly above planned adjacent exterior grades. Therefore, no foundation or slab drainage is required for the residence building.

The lowest floor level in the proposed parking structure will be up to 7 ft below planned adjacent exterior grades. The available water level data indicates groundwater was approximately 6 ft below the lowest floor level at the time of the investigation.

It is recommended that a permanent, perimeter foundation drain system be provided along all below grade walls where the adjacent floor slab is 2 ft or deeper below the adjacent exterior finished grade. The drain system is intended to collect and drain any infiltrating surface or seepage water which might otherwise become trapped against below-grade walls and seep into the building or exert hydrostatic pressures on the walls.

The perimeter foundation drain system should consist of:

a) a perforated drain pipe positioned at the exterior base of the foundation wall;
b) well-draining backfill against the below-grade walls and below upper slab levels;
c) waterstops at construction joints; and
d) dampproofing of the exterior of walls.

The drain pipe should consist of 4 or 6 in. diameter continuous, perforated PVC or corrugated HDPE drain pipe completely surrounded by a 6 in. minimum thick zone of drainage fill (3/4 in. size crushed stone) which is in turn completely surrounded by a non woven filter fabric such as Mirafi 140N or equivalent. Perforations in the drain pipe should be positioned downward. Backfill within 2 ft laterally of the exterior walls above the drain pipe and drainage fill should consist of 3/4-in. crushed stone separated from adjacent soil by non woven filter fabric. The perimeter foundation drain system is shown schematically in Figure 3.

An underslab drainage system is also recommended, where the lowest slab levels are 8 ft or deeper than the adjacent exterior ground surface. Underslab drainage systems should consist of a grid of perforated drain pipes bedded midway in a layer of crushed stone with discharge outlets through the perimeter foundation wall. The grid configuration should be established when interior and exterior grading is finalized. The underslab drainage system should consist of a 6-in. thick layer of 3/4-in. crushed stone drainage fill beneath the slab placed on a non woven filter fabric such as Mirafi 140N or equivalent. Drainage pipes, consisting of 4 in. diameter perforated PVC or corrugated HDPE, should be bedded midway in the locally thickened zone of 3/4 in. crushed stone (4 inches of stone all-around pipe). The underslab drainage system is also shown schematically in Figure 3.

Pipe inverts should be positioned at an elevation between the footing bearing grade and 6 in. below the level of the underside of the adjacent interior floor slab, pitching to discharge points at a slope of at least 0.5 percent. Ideally, water collected by the foundation drainage systems should be directed in pipes by gravity away from the building to the site stormwater system. Depending on the configuration (elevations and proximity) of the site stormwater system, it may be necessary for water to be directed by gravity to sump pit(s) within the garage where it would then be pumped to the stormwater system. Cleanouts should be installed in the systems to facilitate maintenance.

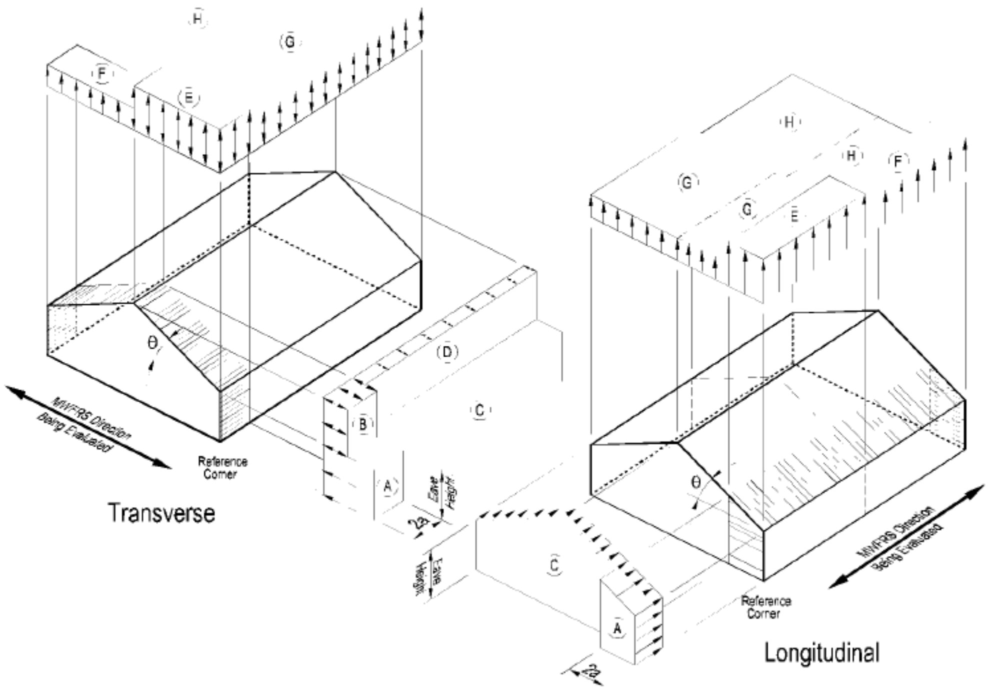
The drain system should be designed and constructed to provide redundant flow paths from each point in the system to a minimum of two discharge points to the site stormwater system. The drain system should be designed to prevent backflow from the site storrnwater system into the foundation drains.

Dampproofing of below grade basement walls is recommended. Waterstops or "keys" should be provided at construction joints in below-grade retaining walls, and between such walls and footing.

To limit water infiltration into the perimeter foundation drain system, it is recommended that the upper twelve inches of backfill within approximately 10ft of the buildings, in unpaved areas, should consist of topsoil or other silty soil having relatively low permeability (such as glacial till). In general, the ground surface immediately around buildings should be sloped downward away from the structure to direct the surface runoff.

Appendix B

Main Wind Force Resisting System – Method 1		h ≤ 60 ft.
Figure 6-2	Design Wind Pressures	Walls & Roofs
Enclosed Buildings		



Notes:

1. Pressures shown are applied to the horizontal and vertical projections, for exposure B, at h=30 ft (9.1m), I=1.0, and K_{zt} = 1.0. Adjust to other conditions using Equation 6-1.
2. The load patterns shown shall be applied to each corner of the building in turn as the reference corner. (See Figure 6-10)
3. For the design of the longitudinal MWFRS use $\theta = 0°$, and locate the zone E/F, G/H boundary at the mid-length of the building.
4. Load cases 1 and 2 must be checked for $25° < \theta \leq 45°$. Load case 2 at 25° is provided only for interpolation between 25° to 30°.
5. Plus and minus signs signify pressures acting toward and away from the projected surfaces, respectively.
6. For roof slopes other than those shown, linear interpolation is permitted.
7. The total horizontal load shall not be less than that determined by assuming $p_S = 0$ in zones B & D.
8. The zone pressures represent the following:

   Horizontal pressure zones – Sum of the windward and leeward net (sum of internal and external) pressures on vertical projection of:

A - End zone of wall	C - Interior zone of wall
B - End zone of roof	D - Interior zone of roof

   Vertical pressure zones – Net (sum of internal and external) pressures on horizontal projection of:

E - End zone of windward roof	G - Interior zone of windward roof
F - End zone of leeward roof	H - Interior zone of leeward roof

9. Where zone E or G falls on a roof overhang on the windward side of the building, use E_{OH} and G_{OH} for the pressure on the horizontal projection of the overhang. Overhangs on the leeward and side edges shall have the basic zone pressure applied.
10. Notation:

    a: 10 percent of least horizontal dimension or 0.4h, whichever is smaller, but not less than either 4% of least horizontal dimension or 3 ft (0.9 m).

    h: Mean roof height, in feet (meters), except that eave height shall be used for roof angles <10°.

    θ: Angle of plane of roof from horizontal, in degrees.

Main Wind Force Resisting System – Method 1		h ≤ 60 ft.
Figure 6-2 (cont'd)	Design Wind Pressures	Walls & Roofs
Enclosed Buildings		

Simplified Design Wind Pressure, p_{s30} (psf) *(Exposure B at h = 30 ft., K_{zt} = 1.0, with I = 1.0)*

Basic Wind Speed (mph)	Roof Angle (degrees)	Load Case	Zones									
			Horizontal Pressures				Vertical Pressures				Overhangs	
			A	B	C	D	E	F	G	H	EOH	GOH
85	0 to 5°	1	11.5	-5.9	7.6	-3.5	-13.8	-7.8	-9.6	-6.1	-19.3	-15.1
	10°	1	12.9	-5.4	8.6	-3.1	-13.8	-8.4	-9.6	-6.5	-19.3	-15.1
	15°	1	14.4	-4.8	9.6	-2.7	-13.8	-9.0	-9.6	-6.9	-19.3	-15.1
	20°	1	15.9	-4.2	10.6	-2.3	-13.8	-9.6	-9.6	-7.3	-19.3	-15.1
	25°	1	14.4	2.3	10.4	2.4	-6.4	-8.7	-4.6	-7.0	-11.9	-10.1
		2	-------	-------	-------	-------	-2.4	-4.7	-0.7	-3.0	-------	-------
	30 to 45	1	12.9	8.8	10.2	7.0	1.0	-7.8	0.3	-6.7	-4.5	-5.2
		2	12.9	8.8	10.2	7.0	5.0	-3.9	4.3	-2.8	-4.5	-5.2
90	0 to 5°	1	12.8	-6.7	8.5	-4.0	-15.4	-8.8	-10.7	-6.8	-21.6	-16.9
	10°	1	14.5	-6.0	9.6	-3.5	-15.4	-9.4	-10.7	-7.2	-21.6	-16.9
	15°	1	16.1	-5.4	10.7	-3.0	-15.4	-10.1	-10.7	-7.7	-21.6	-16.9
	20°	1	17.8	-4.7	11.9	-2.6	-15.4	-10.7	-10.7	-8.1	-21.6	-16.9
	25°	1	16.1	2.6	11.7	2.7	-7.2	-9.8	-5.2	-7.8	-13.3	-11.4
		2	-------	-------	-------	-------	-2.7	-5.3	-0.7	-3.4	-------	-------
	30 to 45	1	14.4	9.9	11.5	7.9	1.1	-8.8	0.4	-7.5	-5.1	-5.8
		2	14.4	9.9	11.5	7.9	5.6	-4.3	4.8	-3.1	-5.1	-5.8
100	0 to 5°	1	15.9	-8.2	10.5	-4.9	-19.1	-10.8	-13.3	-8.4	-26.7	-20.9
	10°	1	17.9	-7.4	11.9	-4.3	-19.1	-11.6	-13.3	-8.9	-26.7	-20.9
	15°	1	19.9	-6.6	13.3	-3.8	-19.1	-12.4	-13.3	-9.5	-26.7	-20.9
	20°	1	22.0	-5.8	14.6	-3.2	-19.1	-13.3	-13.3	-10.1	-26.7	-20.9
	25°	1	19.9	3.2	14.4	3.3	-8.8	-12.0	-6.4	-9.7	-16.5	-14.0
		2	-------	-------	-------	-------	-3.4	-6.6	-0.9	-4.2	-------	-------
	30 to 45	1	17.8	12.2	14.2	9.8	1.4	-10.8	0.5	-9.3	-6.3	-7.2
		2	17.8	12.2	14.2	9.8	6.9	-5.3	5.9	-3.8	-6.3	-7.2
105	0 to 5°	1	17.5	-9.0	11.6	-5.4	-21.1	-11.9	-14.7	-9.3	-29.4	-23.0
	10°	1	19.7	-8.2	13.1	-4.7	-21.1	-12.8	-14.7	-9.8	-29.4	-23.0
	15°	1	21.9	-7.3	14.7	-4.2	-21.1	-13.7	-14.7	-10.5	-29.4	-23.0
	20°	1	24.3	-8.4	16.1	-3.5	-21.1	-14.7	-14.7	-11.1	-29.4	-23.0
	25°	1	21.9	3.5	15.9	3.5	-9.7	-13.2	-7.1	-10.7	-18.2	-15.4
		2	-----	-----	-----	-----	-3.7	-7.3	-1.0	-4.6	-----	-----
	30 to 45	1	19.6	13.5	15.7	10.8	1.5	-11.9	0.6	-10.3	-6.9	-7.9
		2	19.6	13.5	15.7	10.8	7.6	-5.8	6.5	-4.2	-6.9	-7.9
110	0 to 5°	1	19.2	-10.0	12.7	-5.9	-23.1	-13.1	-16.0	-10.1	-32.3	-25.3
	10°	1	21.6	-9.0	14.4	-5.2	-23.1	-14.1	-16.0	-10.8	-32.3	-25.3
	15°	1	24.1	-8.0	16.0	-4.6	-23.1	-15.1	-16.0	-11.5	-32.3	-25.3
	20°	1	26.6	-7.0	17.7	-3.9	-23.1	-16.0	-16.0	-12.2	-32.3	-25.3
	25°	1	24.1	3.9	17.4	4.0	-10.7	-14.6	-7.7	-11.7	-19.9	-17.0
		2	-------	-------	-------	-------	-4.1	-7.9	-1.1	-5.1	-------	-------
	30 to 45	1	21.6	14.8	17.2	11.8	1.7	-13.1	0.6	-11.3	-7.6	-8.7
		2	21.6	14.8	17.2	11.8	8.3	-6.5	7.2	-4.6	-7.6	-8.7
120	0 to 5°	1	22.8	-11.9	15.1	-7.0	-27.4	-15.6	-19.1	-12.1	-38.4	-30.1
	10°	1	25.8	-10.7	17.1	-6.2	-27.4	-16.8	-19.1	-12.9	-38.4	-30.1
	15°	1	28.7	-9.5	19.1	-5.4	-27.4	-17.9	-19.1	-13.7	-38.4	-30.1
	20°	1	31.6	-8.3	21.1	-4.6	-27.4	-19.1	-19.1	-14.5	-38.4	-30.1
	25°	1	28.6	4.6	20.7	4.7	-12.7	-17.3	-9.2	-13.9	-23.7	-20.2
		2	-------	-------	-------	-------	-4.8	-9.4	-1.3	-6.0	-------	-------
	30 to 45	1	25.7	17.6	20.4	14.0	2.0	-15.6	0.7	-13.4	-9.0	-10.3
		2	25.7	17.6	20.4	14.0	9.9	-7.7	8.6	-5.5	-9.0	-10.3

Unit Conversions—1.0 ft = 0.3048 m; 1.0 psf = 0.0479 kN/m^2

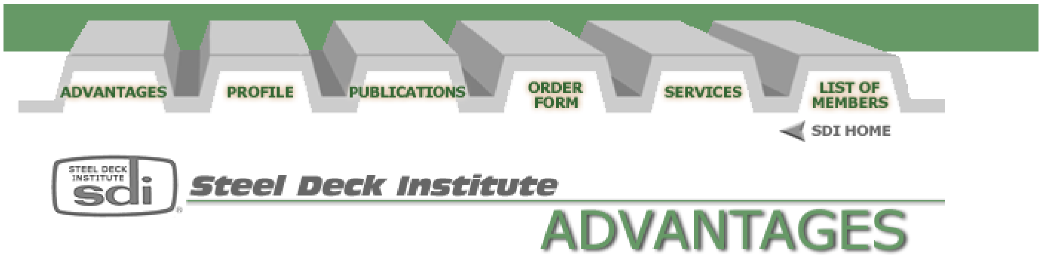
Main Wind Force Resisting System – Method 1		h ≤ 60 ft.
Figure 6-2 (cont'd)	Design Wind Pressures	Walls & Roofs
Enclosed Buildings		

Adjustment Factor for Building Height and Exposure, λ

Mean roof height (ft)	Exposure		
	B	C	D
15	1.00	1.21	1.47
20	1.00	1.29	1.55
25	1.00	1.35	1.61
30	1.00	1.40	1.66
35	1.05	1.45	1.70
40	1.09	1.49	1.74
45	1.12	1.53	1.78
50	1.16	1.56	1.81
55	1.19	1.59	1.84
60	1.22	1.62	1.87

Figure 53: IBC Wind Pressures [21]

Appendix C



Versatility
Steel decks complying with SDI Specifications are available from the member companies in various depths and rib spacings, with and without stiffening elements, with and without acoustical material, cellular and non-cellular, and in varying material thicknesses. This extensive choice makes steel deck applicable to a wide range of projects and structural designs.

Structural Strength with Less Weight
The properties of steel are used with maximum efficiency in the design and fabrication of steel decks, resulting in products with a high strength-to weight ratio. As a result, delivery, erection, and structural framing costs can be lower than with other systems.

Attractive Appearance
Although steel deck is primarily a structural component, it is visually attractive when left exposed in other applications. With the properly specified factory and field coatings, steel deck is easy to maintain, durable, and esthetically pleasing.

All Weather Construction
Steel deck can be erected in most weather conditions, eliminating the costly delays that can occur with other types of floor and roof systems.

Required Fire Ratings
U.L. fire resistance ratings on standard roof and floor assemblies have been obtained by the Steel Deck Institute. Individual SDI manufacturers have ratings on their own products. Most fire resistance requirements can be met with products manufactured by SDI members.

Uniform Quality
Through engineering and continuously refined production techniques, SDI manufacturers produce decks that conform to specified standards.

Proven Durability
Steel deck in place and performing satisfactorily for more than a half century is indicative of the product's durability.

Economy and Value
Value is determined by combining initial costs, life-cycle costs, and overall performance. Steel deck assemblies are the best value in floor and roof designs. They combine low cost with top performance.

Figure 54: Steel Decking Benefits [35]

WHY USE WEB JOIST?

ECONOMY: Web Joist can be used successfully in many projects because it is a very economical floor and roof system when compared to other construction materials – especially when ceiling, decking, floor fill materials and labor costs are considered. Utilizing these lighter weight systems will generally result in reduced size of bearing walls, foundations and footings.

VERSATILITY: Longer spanability, with open webs for ductwork, and top or bottom chord bearing capability. Web Joist trusses may be supported on frame, steel, masonry, or concrete construction. Web Joist trusses weigh less than most other structural members – approximately 4 plf for Version 1 and 7 plf for Version 2. This feature facilitates quick and easy installation without the need for expensive mechanical equipment in most projects.

MECHANICAL ACCESS: The space between webs facilitates installation of ductwork, wiring, and plumbing to significantly reduce the system depth and cost.

FIRE SPRINKLERS: Web Joist trusses are unobstructed construction as defined by the 1999 edition of NFPA 13 for Fire Sprinkled Construction which eliminates sprinkling in each truss space.

ONE HOUR FIRE RATING: For one hour, fire resistive assemblies, see ICBO or LA City Report.

NAILABLE CHORDS: The wood chords are 3 1/2" wide. The decking or ceiling can be directly attached to the chords to minimize construction time and labor.

ENGINEERED DESIGNS: Each Web Joist truss is custom designed by a computer program for its span, depth and loading requirements. The Web Joist engineering department will provide designs for all trusses on a project.

Figure 55: Roof Web Joists [36]

What is Composite Design?

Composite design marries some of steel and concrete's best attributes together for an efficient structural system. Let's start by thinking about a structural system that isn't composite design. Structural steel beams placed at 4' on center with a steel deck spanning perpendicular which will have 4" of concrete placed on top of the steel deck is not a composite system. That means the steel beams will carry their own weight, the weight of the steel deck and concrete above and whatever live load gets applied. The steel deck and the concrete must carry their own weight and the live load spanning from steel beam to steel beam. Another way to state the proposition: the steel beam acts on its own structurally and the steel deck and concrete act on their own structurally.

A composite system ties together that steel beam and concrete floor and forces them to act as a single structural unit. Some connector on top of the steel beam makes the steel and concrete act as one unit. The steel beam can't slide independently of the concrete slab, the two are bonded together. Since the concrete is strong in compression, the composite system can be quite efficient structurally. The figure below illustrates the concept.

Figure 56: Composite Design benefits

Flat slabs

Flat slabs are highly versatile elements widely used in construction, providing minimum depth, fast construction and allowing flexible column grids.

Benefits

Construction

Construction of flat slabs is one of the quickest methods available. Lead times are very short as this is one of the most common forms of construction.

Procurement

Because this is one of the most common forms of construction, all CONSTRUCT members and many other concrete frame contractors can undertake this work.

Cost, whole life cost, value

Flat slabs are particularly appropriate for areas where tops of partitions need to be sealed to the slab soffit for acoustic or fire reasons. Flat slabs are considered to be faster and more economic than other forms of construction, as partition heads do not need to be cut around downstand beams or ribs.

Flat slabs can be designed with a good surface finish to the soffit, allowing exposed soffits to be used. This allows exploitation of the building's thermal mass in the design of heating, ventilation and cooling requirements, increasing energy efficiency.

Mechanical and Engineering

Flat slabs provide the most flexible arrangements for services distribution as services do not have to divert around structural elements.

Figure 57: Benefits of Flat Slabs [37]

- **Initial and Life-Cycle Costs Savings**: Post-tensioned structural systems often provide initial cost savings and better life-cycle costs when compared to other framing systems.
- **Low Maintenance:** Loss of parking spaces during maintenance work results in loss of revenue and inconvenience. Post -tensioned floors are relatively crack-free, resulting in minimal maintenance issues.
- **Crack Control-Watertightness:** Post-tensioned structural systems eliminate closely spaced joints and help ensure water-tightness by placing the floor in bi-axial compression, thereby controlling and counteracting shrinkage and flexural cracks.
- **Smooth Riding Surfaces:** The elimination of closely spaced joints results in a superior riding surface. Differential deflections across joints are not an issue in cast-in-place post-tensioned floors.
- **Lighting and Security:** The long spans, wide beam spacing, and flat surfaces provided by post-tensioned structures enhance lighting and improve patron security. The open structural systems also enhance lighting and create a more welcoming environment.
- **Fire Resistance:** The slab thickness and beam widths commonly used in post-tensioned structures provide fire ratings that meet code requirements.
- **Functional Flexibility:** Post-tensioned construction allows long column-free spans and is adaptable to other functional requirements of parking structures such as the need to accommodate slopes to drains, straight or curved ramps, warped surfaces that provide smooth transitions between ramps and level floors, and irregular plan layouts.
- **Versatility:** Cast-in-place, post-tensioned construction allows for versatility in structural layout. Structural floor configurations are virtually limitless. Cast-in-place forms can be used to create helical ramps, spiral ramps, compound slopes and more.
- **Aesthetics:** Curvilinear shapes and forms are easily achieved in a cost-effective manner with cast-in-place structures. Architectural treatments of exposed concrete are easily achieved.
- **Reduced Structural Depth:** Post-tensioning can reduce structural depths by one-third or more in comparison to other systems. In the case of underground parking garages, structural depth reduction is desirable to reduce excavation, soil retention system costs, and dewatering costs in sites with high water tables.
- **Deflection and Vibration Control:** The pre-compression, draped tendons and the monolithic nature of cast-in-place concrete garages significantly reduce deflection problems. The monolithic construction, continuity and rigid connections between beams and columns reduce vibrations.
- **Lateral Loads:** Monolithic connections between slabs, beams and columns provide rigid frame action to resist wind and moderate seismic loads.
- **Seismic Loads:** Post-tensioned parking structures performed well in the 1971 San Fernando, 1989 Loma Prieta, and the 1994 Northridge earthquakes. Research has shown that, under some conditions, unbonded post-tensioning improves the behavior of moment frames under seismic loads. Code provisions developed by the Building Seismic Safety Council allow the use of unbonded post-tensioning in ductile moment resisting frames.
- **Structural Integrity:** Research and experience have shown that post-tensioned structures inherently provide structural integrity under abnormal and catastrophic loading. Well detailed cast-in-place post-tensioned structures have significantly higher structural integrity, redundancy and resistance to catastrophic loading than precast systems.
- **Construction Advantages:** Both the time and cost of hauling prefabricated pieces from the plant to the job site and the need for heavy lifting are eliminated. Large panel flying forms, modular forming systems and ever improving concrete technology continue to enhance the construction speed and economy of cast-in-place post-tensioned parking structures.

Figure 58: Post Tensioned garage structure benefits

Appendix D

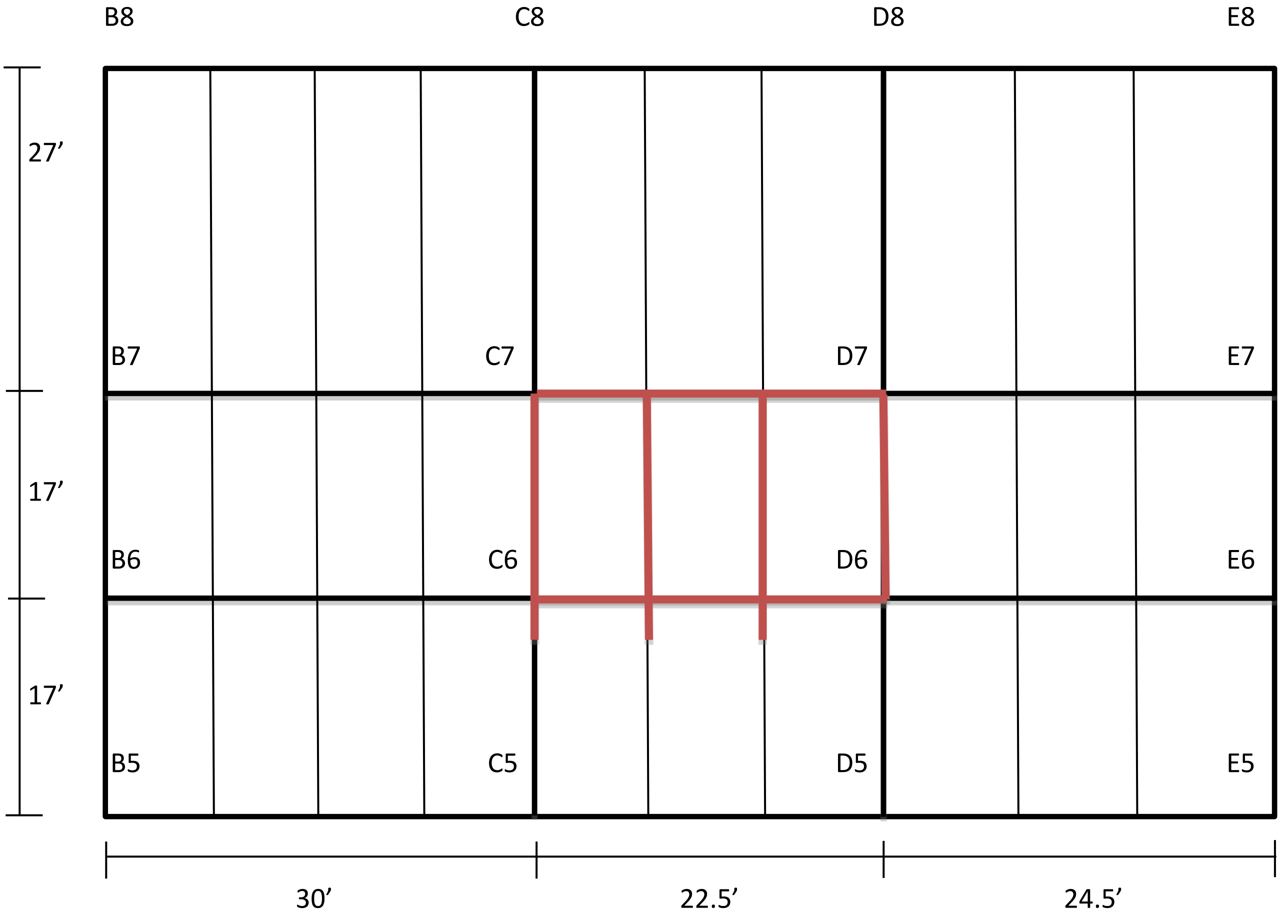


Figure 59: Steel Design Bays

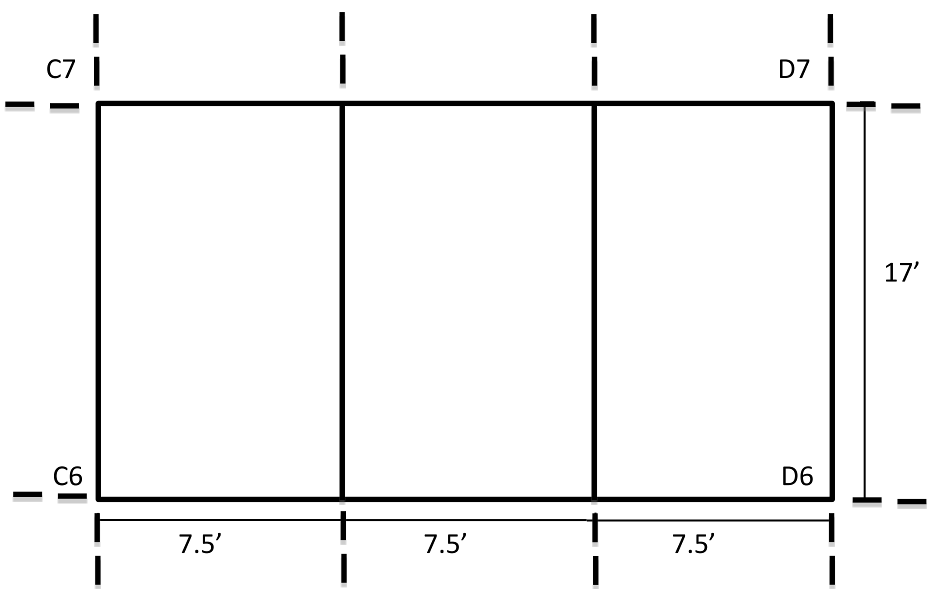


Figure 60: Zoom-In of Bay

7.5 ft on center spacing $f'_c = 4$ ksi 2 in steel deck

$L = 17$ ft $w_c = 110$ pcf $h_r = 2$ in

$t_c = 3$ in $F_y = 50$ ksi $w_r = 6$ in

Roof Beams

DL = 56.5 psf LL = 20 psf SL = 55 psf

DL = .42 k/ft LL = .15 k/ft SL = .42 k/ft

$w_u = 1.2D + 1.6S + .5L = 1.2(.42) + 1.6(.41) + .5(.15) = 1.2$ k/ft

$M_u = \frac{w_u(L^2)}{8} = \frac{1.2(17^2)}{8} = 44.2$ ft-k

$b_e = 2\,(1/8 * L * 12) = 2\,(.125*17*12) = 51$ in ⟵ Use

$b_e = 2(T_w/2 * 12) = 2(3.75*12) = 90$ in

$Y_{con} = 2 + 3 = 5$

Assume a = 2

Y1 = 0

$Y2 = Y_{con} - \frac{a}{2} = 5 - 1 = 4$

Table 3-19 TRY W12X16 (A = 4.17 in^2, I_x = 103 in^4)

Construction loads

w = (3/12)*110*7.5 + 16 = 222 lbs/ft

$M = \frac{.222(17^2)}{8} = 8$ ft-k

$\Delta = \frac{M*L^2}{C_{1*}I_x} = \frac{8*17^2}{161*103} = .14\ in < 2.5\ in$ **OK** ✓

$\Sigma Q_n = A_s*F_y = 4.71(50) = 236$

A required $= \frac{\Sigma Q_n}{.85*f'_c*b_e} = \frac{236}{.85*4*51} = 1.4 < 3$ ✓

Y1 = 0

$Y2 = 5 - \frac{1.4}{2} = 4.3$

$\Phi_b M_n$ from manual by interpolation

$177 + \frac{.3}{.5} * (185 - 177) = 181.8$ ft-k > 44.2 ft-k **OK** ✓

USE W12X16

Table 3-21 ⟶ $Q_n = 21.2$ ¾" studs

No. of studs $= \frac{2*\Sigma Qn}{Q_n} = \frac{2*236}{21.2} = 22.3$

Use 23 studs on each side of the maximum moment

Floor Beams

DL = 34.5 psf LL = 100 psf

DL = .26 k/ft LL = .75 k/ft

w_u = 1.2D + 1.6L = 1.2(.26) + 1.6(.75) = 1.5 k/ft

$M_u = \frac{w_u(L^2)}{8} = \frac{1.5(17^2)}{8}$ = 54.2 ft-k

USE W12X16

Use 23 studs on each side of the maximum moment

Girders

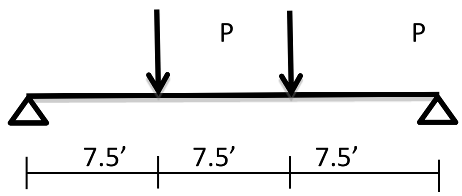


Figure 61: Girders for bay

Roof Girder C7-D7

M_{max} = P*7.5 = 29*7.5 = 217.5 ft-k

V_{max} = P = 29 kips

Y_{con} = 5" assume a = 2 Y1 = 0 Y2 = 4

Table 3-19 TRY W16X31 (A = 9.13 in^2 , I_x = 375 in^4)

Construction loads

w = (3/12)*110 + 31 + 2600 = 2.66 kips/ft

M = $\frac{2.66(22.5^2)}{8}$ = 168.3 ft-k

$\Delta = \frac{M*L^2}{C_{1*}I_x} = \frac{168.3*22.5^2}{158*375} = 1.4\ in < 2.5\ in$ **OK** ✓

$\Sigma Q_n = A_s*F_y$ = 9.13(50) = 456.5

A required = $\frac{\Sigma Q_n}{.85*f'_c*b_e} = \frac{456.5}{.85*4*67.5} = 1.99 < 3$ ✓

Y1 = 0

Y2 = 5 - $\frac{1.99}{2}$ = 4

$\Phi_b M_n$ from manual

409 ft-k > 217.5 ft-k **OK** ✓

USE W16X31

Table 3-21 ⟶ Q_n = 21.2 ¾" studs

No. of studs = $\frac{2*\Sigma Qn}{Q_n} = \frac{2*456.5}{21.2} = 43.1$

Use 44 studs on the outer two thirds of the girder

Roof Girder C6-D6

M_{max} = P*7.5 = 22*7.5 = 165 ft-k

V_{max} = P = 22 kips

USE W16X31
Use 44 studs on the outer two thirds of the girder

Floor Girder C7-D7
M_{max} = P*7.5 = 35*7.5 = 262.5 ft-k
V_{max} = P = 35 kips

USE W16X31
Use 44 studs on the outer two thirds of the girder
Floor Girder C6-D6
M_{max} = P*7.5 = 27*7.5 = 202.5 ft-k
V_{max} = P = 27 kips

USE W16X31
Use 44 studs on the outer two thirds of the girder

Columns

Table 13: Loads on Columns

Column	Roof Load	Floor Load
C7	100.7	121.3
D7	90.1	108.5
C6	77.5	93.5
D6	73.2	83.8

Equations used
$L_e = L*k*12$
$r = \sqrt{\frac{I}{A}}$
$S = \frac{L_e}{r}$
$F_e = \frac{\pi^2 * E}{S^2}$
If $F_e < .44 * F_y$, then $F_{cr} = .877*F_e$, otherwise $F_{cr} = .658^{\frac{F_y}{F_e}} * F_y$
$P_n = F_{cr}*A$
$\Phi_c P_n = .9*P_n$

Table 14: 3rd Floor Columns

Column	Load (k)	L (ft)	L_e (in)	I (in^4)	A (in^2)	r (in)	S	E (ksi)	F_e (ksi)	F_y (ksi)	F_{cr} (ksi)	P_n (k)	$\Phi_c P_n$ (k)
C7	100.7	15	180	44.1	11.7	1.94	92.7	29000	33.3	50	26.7	312	281
D7	90.1	15	180	44.1	11.7	1.94	92.7	29000	33.3	50	26.7	312	281
C6	77.5	15	180	44.1	11.7	1.94	92.7	29000	33.3	50	26.7	312	281
D6	73.2	15	180	44.1	11.7	1.94	92.7	29000	33.3	50	26.7	312	281

Use W12X40 columns

Table 15: 2nd Floor Columns

Column	Load (k)	L (ft)	L_e (in)	I (in^4)	A (in^2)	r (in)	S	E (ksi)	F_e (ksi)	F_y (ksi)	F_{cr} (ksi)	P_n (k)	$\Phi_c P_n$ (k)
C7	222	15	180	44.1	11.7	1.94	92.7	29000	33.3	50	26.7	312	281
D7	198.6	15	180	44.1	11.7	1.94	92.7	29000	33.3	50	26.7	312	281
C6	171	15	180	44.1	11.7	1.94	92.7	29000	33.3	50	26.7	312	281
D6	157	15	180	44.1	11.7	1.94	92.7	29000	33.3	50	26.7	312	281

Use W12X40 columns

Table 16: 1st Floor Columns

Column	Load (k)	L (ft)	L_e (in)	I (in^4)	A (in^2)	r (in)	S	E (ksi)	F_e (ksi)	F_y (ksi)	F_{cr} (ksi)	P_n (k)	$\Phi_c P_n$ (k)
C7	343.3	15	180	95.8	15.6	2.48	72.6	29000	54.3	50	34	530	477
D7	307.1	15	180	95.8	15.6	2.48	72.6	29000	54.3	50	34	530	477
C6	264.5	15	180	95.8	15.6	2.48	72.6	29000	54.3	50	34	530	477
D6	240.8	15	180	95.8	15.6	2.48	72.6	29000	54.3	50	34	530	477

Use W12X53 columns

View of columns C7, D7, C6, and D6 from the side

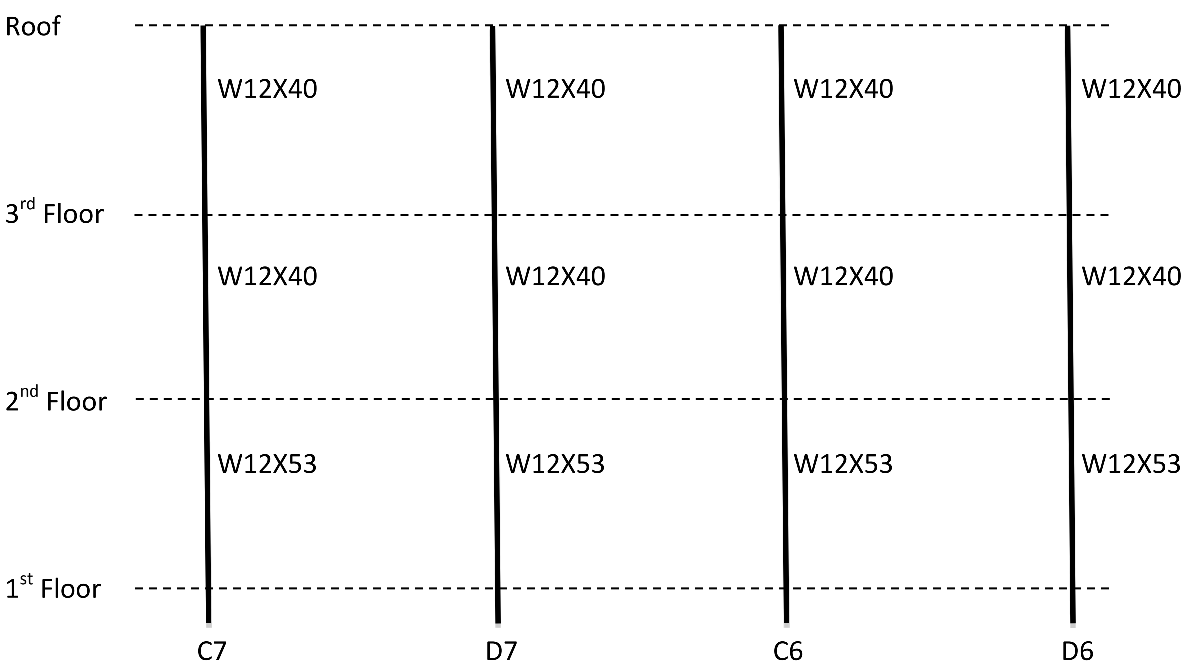


Figure 62: Column Layout

Connections

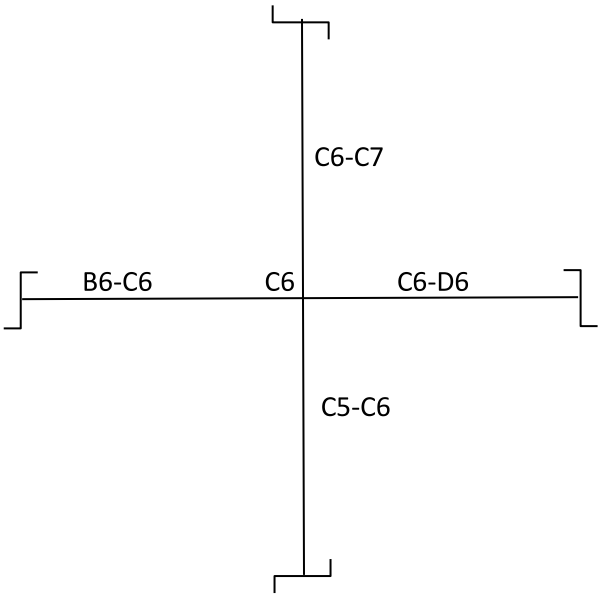


Figure 63: Connections

Table 17: Connection Transfer Loads

Beam	R_u Roof (kips)	R_u Floor (kips)	t_w (in)
B6-C6	33	40.5	.275
C6-D6	22	27	.275
C6-C7	11	13.5	.220
C5-C6	11	13.5	.220

W12X40 Column t_f = .515 in t_w = .295 in
W12X53 Column t_f = .575 in t_w = .345 in

Double angle connections
F_y = 36 ksi F_u = 58 ksi
Use ¾ in A325-N bolts   Table 7-16 $H_2 = 1\frac{3}{8}$" min $C_1 = 1\frac{1}{4}$" min

Table 10-1, 3 rows of bolts
Try angle thickness of ¼", ΦR_n = 76.4 kips > 40.5 kips ✓

Leg length = Angle thickness + H_2 + C_1 + 1 = $\frac{1}{4} + 1\frac{3}{8} + 1\frac{1}{4} + 1 = 3\frac{7}{8}$ use 4"
Use 2Ls 4 X 4 X ¼ in A36 for all connections

Appendix E

ETABS®

Parking Garage and Academic Building

Project Number: Preliminary Analysis of the Steel Structure

Prepared for

President Berkey

AISC360-05/IBC2006
Steel Frame Design Report

Prepared by

Tariq Aziz Azizi
Mark Arnold
Andrew Mackenzie
Kyle Warren

Model Name: School of Business Steel Building Frame

24 April 2012

ETABS v9.7.2 File :WPI School of Business Steel Frame Units:Kip-in April 25, 2012

ETABS Frame Analysis output summary is provided in pages 1-8, Detailed analysis output is summarized in pages 9-41 of this report.

PROJECT INFORMATION

Company Name = Worcester Polytechnic Institute

Client Name = President Berkey

Project Name = Parking Garage and Academic Building

Project Number = Preliminary Analysis of the Steel Structure

Frame Type = Ordinary Braced Frame

Engineer = Tariq Aziz Azizi

Checker = Mark Arnold

Supervisor = Edward J. Swierz

Design Code = AISC 14, IBC2009,

STORY DATA

STORY	SIMILAR TO	HEIGHT	ELEVATION
STORY4	None	120.000	588.000
STORY3	None	156.000	468.000
STORY2	STORY1	156.000	312.000
STORY1	None	156.000	156.000
BASE	None		0.000

STATIC LOAD CASES

STATIC CASE	CASE TYPE	AUTO LAT LOAD	SELF WT MULTIPLIER	NOTIONAL FACTOR	NOTIONAL DIRECTION
DEAD	DEAD	N/A	1.0000		
LIVE	REDUCE LIVE	N/A	0.0000		

QUAKE QUAKE IBC2006 0.0000

WIND WIND ASCE7-05 0.0000

SDEAD SUPER DEAD N/A 0.0000

CLAUDING SUPER DEAD N/A 0.0000

SNOW SNOW N/A 0.0000

A U T O W I N D A S C E 7 - 0 5

Case: WIND

AUTO WIND INPUT DATA

Exposure From: Area Objects

Top Story: STORY3

Bottom Story: BASE

No parapet is included

Basic Wind Speed, V = 100 mph

Exposure Category = B Importance Factor, Iw = 1

Kzt = 1 Kd = 0.85

Gust Factor, G = 0.85

AUTO WIND CALCULATION FORMULAS

P = wind pressure = WindwardCp qz G + qh G LeewardCp

qz, the velocity pressure, = 0.00256 Kz Kzt Kd Iw V^2 psf

Kz is the velocity pressure exposure coefficient

Kz = 2.01 (z / zg)^(2/alpha) for 15 feet <= z <= zg

Kz = 2.01 (15 / zg)^(2/alpha) for z < 15 feet

z is the distance from the specified bottom story to the point considered

zg and alpha are specified in ASCE7-05 Table 6-2

qh is the velocity pressure at the specified top story level

AUTO WIND CALCULATION RESULTS

zg = 1200   alpha = 7.0   qh = 16.4316

AUTO WIND STORY FORCES

STORY	FX	FY	FZ	MX	MY	MZ
STORY4	0.00	0.00	0.00	0.000	0.000	0.000
STORY3	0.00	0.00	0.00	0.000	0.000	0.000
STORY2	0.00	-37.88	0.00	0.000	0.000	27.206
STORY1	0.00	-70.87	0.00	0.000	0.000	15.179

A U T O   S E I S M I C   I B C 2 0 0 6

Case: QUAKE

AUTO SEISMIC INPUT DATA

Direction: X + EccY

Typical Eccentricity = 5%

Eccentricity Overrides: No

Period Calculation: Program Calculated

Ct = 0.028 (in feet units)

x = 0.8

Top Story: STORY4

Bottom Story: BASE

R = 3.5   I = 1.25

Ss = 0.21g   S1 = 0.07g   TL = 6

Site Class = B   Fa = 1   Fv = 1

hn = 588.000 (Building Height)

AUTO SEISMIC CALCULATION RESULTS

Sds = 0.1400g   Sd1 = 0.0467g

T Used = 0.3118 sec   W Used = 4669.17

V Used = 0.0500W = 233.46   K Used = 1.0000

AUTO SEISMIC STORY FORCES

STORY	FX	FY	FZ	MX	MY	MZ
STORY4	9.96	0.00	0.00	0.000	0.000	-131.461
STORY3	109.64	0.00	0.00	0.000	-12.678	-7411.772
STORY2	75.72	0.00	0.00	0.000	-8.455	-4834.611
STORY1	38.14	0.00	0.00	0.000	-4.228	-2385.788

D I A P H R A G M M A S S D A T A

STORY	DIAPHRAGM	MASS-X	MASS-Y	MMI	X-M	Y-M
STORY4	D1	2.774E-01	2.774E-01	6.062E+05	1560.000	130.626
STORY3	D1	3.785E+00	3.785E+00	4.658E+06	1560.282	729.711
STORY2	D1	3.883E+00	3.883E+00	4.787E+06	1560.718	726.971
STORY1	D1	3.906E+00	3.906E+00	4.821E+06	1560.214	725.439

A S S E M B L E D P O I N T M A S S E S

STORY	UX	UY	UZ	RX	RY	RZ
STORY4	2.774E-01	2.774E-01	0.000E+00	0.000E+00	0.000E+00	6.062E+05
STORY3	3.837E+00	3.837E+00	0.000E+00	0.000E+00	0.000E+00	4.658E+06
STORY2	3.975E+00	3.975E+00	0.000E+00	0.000E+00	0.000E+00	4.787E+06
STORY1	4.004E+00	4.004E+00	0.000E+00	0.000E+00	0.000E+00	4.821E+06
BASE	1.194E-01	1.194E-01	0.000E+00	0.000E+00	0.000E+00	0.000E+00
Totals	1.221E+01	1.221E+01	0.000E+00	0.000E+00	0.000E+00	1.487E+07

C E N T E R S O F C U M U L A T I V E M A S S & C E N T E R S O F R I G I D I T Y

STORY	DIAPHRAGM	/----------CENTER OF MASS----------		//--CENTER OF RIGIDITY--/	
LEVEL	NAME	MASS	ORDINATE-X	ORDINATE-Y	ORDINATE-X	ORDINATE-Y
STORY4	D1	2.774E-01	1560.000	130.626	1543.781	82.907
STORY3	D1	4.062E+00	1560.262	688.801	1561.498	479.095
STORY2	D1	7.946E+00	1560.485	707.456	1556.584	502.828
STORY1	D1	1.185E+01	1560.396	713.383	1552.814	483.191

M O D A L P E R I O D S A N D F R E Q U E N C I E S

MODE NUMBER	PERIOD (TIME)	FREQUENCY (CYCLES/TIME)	CIRCULAR FREQ (RADIANS/TIME)
Mode 1	0.45050	2.21974	13.94703
Mode 2	0.44822	2.23106	14.01819
Mode 3	0.35612	2.80805	17.64352
Mode 4	0.34440	2.90364	18.24412
Mode 5	0.34424	2.90494	18.25230
Mode 6	0.34405	2.90654	18.26234
Mode 7	0.33814	2.95736	18.58165
Mode 8	0.31185	3.20668	20.14814
Mode 9	0.28105	3.55805	22.35590
Mode 10	0.27923	3.58125	22.50165
Mode 11	0.24399	4.09846	25.75140
Mode 12	0.24236	4.12615	25.92536

M O D A L L O A D P A R T I C I P A T I O N R A T I O S

(STATIC AND DYNAMIC RATIOS ARE IN PERCENT)

TYPE	NAME	STATIC	DYNAMIC
Load	DEAD	0.0299	0.0482
Load	LIVE	0.0086	0.0000
Load	QUAKE	38.0359	27.1585
Load	WIND	63.9146	2.8960
Load	SDEAD	0.0072	0.0000
Load	CLAUDING	0.0009	0.0000
Load	SNOW	0.0062	0.0000
Accel	UX	33.7759	21.0083

Accel	UY	96.9339	83.7044
Accel	UZ	0.0000	0.0000
Accel	RX	99.9069	99.5712
Accel	RY	32.8996	23.4576
Accel	RZ	79.2008	69.6669

TOTAL REACTIVE FORCES (RECOVERED LOADS) AT ORIGIN

LOAD	FX	FY	FZ	MX	MY	MZ
DEAD	1.620E-12	-4.895E-11	6.226E+03	4.443E+06	-9.707E+06	-2.099E-07
LIVE	7.596E-13	-6.674E-11	5.224E+03	3.787E+06	-8.107E+06	-2.125E-07
QUAKE	-2.335E+02	-1.568E-10	-6.484E-14	-6.753E+01	-7.268E+04	1.788E+05
WIND	3.956E-12	1.442E+02	4.108E-15	-1.613E+04	-4.618E+00	2.249E+05
SDEAD	1.174E-13	-8.309E-12	1.050E+03	7.671E+05	-1.638E+06	-1.045E-08
CLAUDING	-2.544E-14	-7.894E-12	2.623E+02	1.547E+05	-4.090E+05	-1.044E-07
SNOW	2.243E-13	7.350E-12	1.356E+03	9.786E+05	-2.144E+06	6.030E-08

S T O R Y   F O R C E S

STORY	LOAD	P	VX	VY	T	MX	MY
STORY4	QUAKE	3.331E-15	-9.959E+00	-1.697E-14	1.432E+03	5.471E-13	-1.195E+03
STORY3	QUAKE	1.732E-13	-1.196E+02	8.602E-10	8.885E+04	-2.835E+01	-1.596E+04
STORY2	QUAKE	1.443E-13	-1.953E+02	-2.806E-11	1.487E+05	-5.605E+01	-4.198E+04
STORY1	QUAKE	-6.484E-14	-2.335E+02	-1.568E-10	1.788E+05	-6.753E+01	-7.268E+04
STORY4	WIND	-1.429E-14	3.911E-14	3.172E-13	5.838E-10	-3.998E-11	2.669E-11
STORY3	WIND	-2.572E-14	1.521E-12	2.356E-13	1.377E-06	3.004E+02	3.796E+00
STORY2	WIND	-3.553E-14	2.963E-12	3.788E+01	5.909E+04	-3.945E+03	5.074E+00
STORY1	WIND	4.108E-15	3.955E-12	1.087E+02	1.696E+05	-1.613E+04	-4.618E+00

STORY DRIFTS

STORY	DIRECTION	LOAD	MAX DRIFT
STORY4	X	QUAKE	1/9505
STORY4	Y	QUAKE	1/18442

STORY3	X	QUAKE	1/9536
STORY2	X	QUAKE	1/8309
STORY1	X	QUAKE	1/8939
STORY4	Y	WIND	1/84226
STORY3	Y	WIND	1/66381
STORY2	Y	WIND	1/19279
STORY1	Y	WIND	1/7705

DISPLACEMENTS AT DIAPHRAGM CENTER OF MASS

STORY	DIAPHRAGM	LOAD	UX	UY	RZ
STORY4	D1	QUAKE	0.0459	0.0000	-0.00002
STORY3	D1	QUAKE	0.0432	0.0001	-0.00001
STORY2	D1	QUAKE	0.0296	0.0000	-0.00001
STORY1	D1	QUAKE	0.0140	0.0000	0.00000
STORY4	D1	WIND	0.0000	-0.0216	0.00000
STORY3	D1	WIND	0.0000	-0.0204	0.00000
STORY2	D1	WIND	0.0000	-0.0200	0.00000
STORY1	D1	WIND	0.0000	-0.0137	0.00000

STORY MAXIMUM AND AVERAGE LATERAL DISPLACEMENTS

STORY	LOAD	DIR	MAXIMUM	AVERAGE	RATIO
STORY4	QUAKE	X	0.0481	0.0459	1.049
STORY3	QUAKE	X	0.0526	0.0432	1.217
STORY2	QUAKE	X	0.0362	0.0297	1.221
STORY1	QUAKE	X	0.0175	0.0140	1.246
STORY4	WIND	Y	0.0217	0.0216	1.006
STORY3	WIND	Y	0.0205	0.0204	1.002
STORY2	WIND	Y	0.0201	0.0200	1.003
STORY1	WIND	Y	0.0138	0.0137	1.005

Steel Column Design - Capacity Check Output

Steel Column Design - Capacity Check Output

Story Level	Column Line	Section Name	Moment Interaction Check Ratio = AXL + B33 + B22	Shear22 Ratio	Shear33 Ratio
STORY3	C42	W16X36	0.839 = 0.086 + 0.697 + 0.056	0.190	0.002
STORY2	C42	W16X57	0.866 = 0.248 + 0.554 + 0.063	0.182	0.003
STORY1	C42	W16X50	0.825 = 0.440 + 0.342 + 0.042	0.099	0.002
STORY3	C43	W16X36	0.626 = 0.056 + 0.569 + 0.001	0.153	0.000
STORY2	C43	W16X50	0.723 = 0.234 + 0.487 + 0.002	0.157	0.000
STORY1	C43	W16X36	0.915 = 0.522 + 0.390 + 0.004	0.104	0.001
STORY3	C44	W16X50	0.719 = 0.061 + 0.605 + 0.052	0.173	0.002
STORY2	C44	W16X50	0.921 = 0.291 + 0.569 + 0.061	0.180	0.003
STORY1	C44	W16X50	0.873 = 0.462 + 0.373 + 0.038	0.108	0.002
STORY3	C45	W16X36	0.548 = 0.074 + 0.415 + 0.059	0.105	0.002
STORY2	C45	W16X50	0.721 = 0.275 + 0.386 + 0.060	0.125	0.003
STORY1	C45	W16X50	0.728 = 0.445 + 0.243 + 0.039	0.071	0.002
STORY3	C47	W16X36	0.144 = 0.038 + 0.045 + 0.060	0.012	0.002
STORY2	C47	W16X36	0.306 = 0.221 + 0.042 + 0.044	0.013	0.002
STORY1	C47	W16X36	0.365 = 0.322 + 0.024 + 0.018	0.009	0.001
STORY3	C48	W16X36	0.234 = 0.058 + 0.117 + 0.060	0.029	0.002
STORY2	C48	W16X36	0.473 = 0.321 + 0.106 + 0.046	0.032	0.002
STORY1	C48	W16X36	0.555 = 0.478 + 0.058 + 0.020	0.016	0.001
STORY3	C50	W16X36	0.109 = 0.014 + 0.058 + 0.037	0.016	0.001
STORY2	C50	W16X36	0.122 = 0.045 + 0.045 + 0.031	0.013	0.001
STORY1	C50	W16X36	0.099 = 0.065 + 0.021 + 0.013	0.007	0.001
STORY4	C51	W16X36	0.058 = 0.035 + 0.005 + 0.018	0.001	0.001
STORY3	C51	W16X36	0.138 = 0.043 + 0.094 + 0.001	0.034	0.001
STORY2	C51	W16X36	0.167 = 0.068 + 0.089 + 0.010	0.034	0.001
STORY1	C51	W16X36	0.238 = 0.081 + 0.153 + 0.004	0.070	0.003
STORY4	C52	W16X36	0.079 = 0.021 + 0.015 + 0.044	0.004	0.001
STORY3	C52	W16X36	0.237 = 0.031 + 0.015 + 0.191	0.014	0.012
STORY2	C52	W16X36	0.260 = 0.055 + 0.004 + 0.201	0.017	0.013
STORY1	C52	W16X36	0.196 = 0.064 + 0.003 + 0.129	0.016	0.012
STORY3	C53	W16X36	0.244 = 0.036 + 0.088 + 0.120	0.022	0.004
STORY2	C53	W16X36	0.396 = 0.203 + 0.078 + 0.115	0.024	0.005
STORY1	C53	W16X36	0.405 = 0.304 + 0.040 + 0.061	0.011	0.003
STORY3	C54	W16X36	0.134 = 0.047 + 0.018 + 0.069	0.006	0.003
STORY2	C54	W16X36	0.324 = 0.258 + 0.014 + 0.052	0.006	0.002
STORY1	C54	W16X36	0.415 = 0.383 + 0.005 + 0.026	0.003	0.001
STORY3	C55	W16X36	0.145 = 0.053 + 0.033 + 0.060	0.009	0.002
STORY2	C55	W16X36	0.367 = 0.289 + 0.032 + 0.046	0.010	0.002
STORY1	C55	W16X36	0.471 = 0.431 + 0.017 + 0.024	0.006	0.001
STORY3	C57	W16X36	0.622 = 0.311 + 0.247 + 0.064	0.070	0.003
STORY2	C57	W16X50	0.834 = 0.526 + 0.245 + 0.064	0.080	0.003
STORY1	C57	W16X67	0.628 = 0.451 + 0.129 + 0.048	0.050	0.003
STORY3	C58	W16X36	0.676 = 0.311 + 0.311 + 0.054	0.091	0.002
STORY2	C58	W16X50	0.879 = 0.522 + 0.300 + 0.057	0.098	0.003

Steel Column Design - Capacity Check Output

Story Level	Column Line	Section Name	Moment Interaction Check Ratio = AXL + B33 + B22	Shear22 Ratio	Shear33 Ratio
STORY1	C58	W16X67	0.648 = 0.448 + 0.160 + 0.040	0.062	0.002
STORY3	C59	W16X36	0.503 = 0.227 + 0.272 + 0.004	0.078	0.000
STORY2	C59	W16X36	0.846 = 0.557 + 0.285 + 0.003	0.084	0.000
STORY1	C59	W16X50	0.755 = 0.594 + 0.159 + 0.002	0.046	0.000
STORY3	C60	W16X36	0.725 = 0.315 + 0.375 + 0.035	0.106	0.002
STORY2	C60	W16X50	0.862 = 0.497 + 0.316 + 0.049	0.110	0.002
STORY1	C60	W16X67	0.631 = 0.434 + 0.151 + 0.046	0.058	0.003
STORY3	C61	W16X36	0.555 = 0.272 + 0.237 + 0.045	0.063	0.002
STORY2	C61	W16X50	0.683 = 0.434 + 0.196 + 0.053	0.070	0.002
STORY1	C61	W16X50	0.913 = 0.696 + 0.123 + 0.094	0.035	0.004
STORY3	C62	W16X36	0.611 = 0.295 + 0.284 + 0.031	0.077	0.001
STORY2	C62	W16X50	0.741 = 0.471 + 0.231 + 0.040	0.082	0.002
STORY1	C62	W16X57	0.847 = 0.653 + 0.130 + 0.064	0.037	0.003
STORY3	C65	W16X36	0.616 = 0.289 + 0.294 + 0.034	0.081	0.001
STORY2	C65	W16X50	0.747 = 0.462 + 0.239 + 0.047	0.085	0.002
STORY1	C65	W16X57	0.859 = 0.640 + 0.136 + 0.082	0.039	0.004
STORY3	C66	W16X36	0.421 = 0.208 + 0.209 + 0.004	0.059	0.000
STORY2	C66	W16X36	0.743 = 0.521 + 0.219 + 0.003	0.065	0.000
STORY1	C66	W16X50	0.680 = 0.555 + 0.123 + 0.002	0.036	0.000
STORY3	C69	W16X36	0.581 = 0.290 + 0.238 + 0.053	0.070	0.002
STORY2	C69	W16X50	0.770 = 0.493 + 0.222 + 0.056	0.073	0.003
STORY1	C69	W16X57	0.887 = 0.667 + 0.146 + 0.074	0.042	0.003
STORY3	C70	W16X36	0.302 = 0.233 + 0.009 + 0.060	0.003	0.002
STORY2	C70	W16X36	0.658 = 0.584 + 0.012 + 0.062	0.005	0.003
STORY1	C70	W16X50	0.665 = 0.626 + 0.005 + 0.033	0.004	0.001
STORY3	C71	W16X36	0.352 = 0.085 + 0.194 + 0.073	0.050	0.002
STORY2	C71	W16X36	0.672 = 0.426 + 0.160 + 0.087	0.049	0.004
STORY1	C71	W16X36	0.855 = 0.664 + 0.120 + 0.071	0.032	0.003
STORY4	C72	W16X36	0.060 = 0.015 + 0.003 + 0.042	0.003	0.002
STORY3	C72	W16X36	0.109 = 0.051 + 0.014 + 0.044	0.005	0.002
STORY2	C72	W16X36	0.189 = 0.189 + 0.000 + 0.000	0.002	0.001
STORY1	C72	W16X36	0.278 = 0.261 + 0.008 + 0.008	0.005	0.000
STORY4	C73	W16X36	0.195 = 0.019 + 0.042 + 0.134	0.015	0.005
STORY3	C73	W16X36	0.419 = 0.068 + 0.094 + 0.258	0.024	0.009
STORY2	C73	W16X36	0.625 = 0.280 + 0.078 + 0.268	0.024	0.012
STORY1	C73	W16X36	0.717 = 0.415 + 0.069 + 0.233	0.021	0.009
STORY4	C75	W16X36	0.212 = 0.033 + 0.045 + 0.135	0.018	0.005
STORY3	C75	W16X36	0.366 = 0.082 + 0.024 + 0.260	0.005	0.009
STORY2	C75	W16X36	0.589 = 0.305 + 0.013 + 0.270	0.005	0.012
STORY1	C75	W16X36	0.665 = 0.421 + 0.011 + 0.233	0.010	0.009
STORY3	C76	W16X36	0.136 = 0.033 + 0.087 + 0.017	0.023	0.001
STORY2	C76	W16X36	0.195 = 0.195 + 0.000 + 0.000	0.021	0.001
STORY1	C76	W16X36	0.325 = 0.286 + 0.036 + 0.003	0.010	0.000
STORY3	C77	W16X36	0.120 = 0.029 + 0.090 + 0.002	0.024	0.000
STORY2	C77	W16X36	0.162 = 0.162 + 0.000 + 0.000	0.020	0.000
STORY1	C77	W16X36	0.268 = 0.236 + 0.031 + 0.000	0.009	0.000
STORY3	C78	W16X36	0.173 = 0.038 + 0.091 + 0.044	0.024	0.002
STORY2	C78	W16X36	0.343 = 0.223 + 0.080 + 0.040	0.024	0.002

Steel Column Design - Capacity Check Output

Story Level	Column Line	Section Name	Moment Interaction Check Ratio = AXL + B33 + B22	Shear22 Ratio	Shear33 Ratio
STORY1	C78	W16X36	0.394 = 0.331 + 0.042 + 0.021	0.012	0.001
STORY3	C79	W16X36	0.168 = 0.039 + 0.091 + 0.039	0.024	0.001
STORY2	C79	W16X36	0.343 = 0.231 + 0.079 + 0.033	0.024	0.001
STORY1	C79	W16X36	0.398 = 0.342 + 0.042 + 0.014	0.012	0.001
STORY3	C80	W16X36	0.644 = 0.272 + 0.321 + 0.051	0.092	0.002
STORY2	C80	W16X50	0.801 = 0.458 + 0.293 + 0.050	0.098	0.002
STORY1	C80	W16X57	0.869 = 0.619 + 0.185 + 0.065	0.053	0.003
STORY3	C81	W16X36	0.646 = 0.270 + 0.323 + 0.054	0.093	0.002
STORY2	C81	W16X50	0.805 = 0.453 + 0.294 + 0.058	0.098	0.003
STORY1	C81	W16X57	0.882 = 0.611 + 0.185 + 0.085	0.053	0.004
STORY3	C82	W16X36	0.436 = 0.099 + 0.326 + 0.010	0.082	0.000
STORY2	C82	W16X36	0.786 = 0.488 + 0.290 + 0.008	0.087	0.000
STORY1	C82	W16X50	0.681 = 0.522 + 0.156 + 0.003	0.045	0.000
STORY3	C83	W16X36	0.583 = 0.247 + 0.330 + 0.005	0.096	0.000
STORY2	C83	W16X50	0.747 = 0.418 + 0.316 + 0.013	0.104	0.001
STORY1	C83	W16X50	0.852 = 0.651 + 0.193 + 0.008	0.056	0.001
STORY3	C84	W16X36	0.291 = 0.050 + 0.185 + 0.056	0.046	0.002
STORY2	C84	W16X36	0.490 = 0.278 + 0.167 + 0.045	0.051	0.002
STORY1	C84	W16X36	0.525 = 0.416 + 0.088 + 0.021	0.023	0.001
STORY3	C85	W16X36	0.276 = 0.022 + 0.106 + 0.149	0.026	0.005
STORY2	C85	W16X36	0.337 = 0.069 + 0.107 + 0.161	0.029	0.006
STORY1	C85	W16X36	0.240 = 0.097 + 0.059 + 0.084	0.015	0.003
STORY3	C86	W16X36	0.407 = 0.034 + 0.208 + 0.165	0.052	0.006
STORY2	C86	W16X36	0.491 = 0.098 + 0.221 + 0.171	0.058	0.007
STORY1	C86	W16X36	0.478 = 0.288 + 0.110 + 0.080	0.029	0.003
STORY3	C94	W16X36	0.492 = 0.061 + 0.376 + 0.055	0.093	0.002
STORY2	C94	W16X36	0.762 = 0.339 + 0.368 + 0.055	0.108	0.002
STORY1	C94	W16X36	0.915 = 0.644 + 0.235 + 0.036	0.080	0.002
STORY4	C294	W16X36	0.140 = 0.020 + 0.088 + 0.032	0.018	0.001
STORY3	C294	W16X36	0.425 = 0.035 + 0.237 + 0.153	0.077	0.010
STORY2	C294	W16X36	0.486 = 0.077 + 0.250 + 0.159	0.079	0.010
STORY1	C294	W16X36	0.430 = 0.204 + 0.153 + 0.073	0.091	0.010
STORY4	C295	W16X36	0.070 = 0.033 + 0.019 + 0.018	0.004	0.001
STORY3	C295	W16X36	0.165 = 0.047 + 0.104 + 0.015	0.038	0.002
STORY2	C295	W16X36	0.167 = 0.078 + 0.082 + 0.007	0.032	0.001
STORY1	C295	W16X36	0.286 = 0.211 + 0.069 + 0.007	0.066	0.002
STORY4	C296	W16X36	0.206 = 0.033 + 0.043 + 0.130	0.017	0.005
STORY3	C296	W16X36	0.373 = 0.084 + 0.029 + 0.260	0.006	0.009
STORY2	C296	W16X36	0.592 = 0.306 + 0.015 + 0.270	0.006	0.012
STORY1	C296	W16X36	0.666 = 0.422 + 0.010 + 0.234	0.009	0.009
STORY4	C297	W16X36	0.197 = 0.020 + 0.039 + 0.138	0.014	0.005
STORY3	C297	W16X36	0.415 = 0.072 + 0.087 + 0.256	0.022	0.009
STORY2	C297	W16X36	0.628 = 0.298 + 0.063 + 0.267	0.020	0.012
STORY1	C297	W16X36	0.736 = 0.453 + 0.053 + 0.231	0.018	0.009
STORY4	C298	W16X36	0.348 = 0.014 + 0.013 + 0.321	0.002	0.012
STORY3	C298	W16X67	0.749 = 0.037 + 0.011 + 0.702	0.006	0.037
STORY2	C298	W16X89	0.793 = 0.060 + 0.017 + 0.716	0.004	0.041
STORY1	C298	W16X67	0.564 = 0.232 + 0.011 + 0.321	0.008	0.019

Steel Column Design - Capacity Check Output

Story Level	Column Line	Section Name	Moment Interaction Check Ratio = AXL + B33 + B22	Shear22 Ratio	Shear33 Ratio
STORY3	C300	W16X36	0.130 = 0.032 + 0.085 + 0.013	0.022	0.001
STORY2	C300	W16X36	0.194 = 0.194 + 0.000 + 0.000	0.021	0.001
STORY1	C300	W16X36	0.325 = 0.285 + 0.037 + 0.003	0.011	0.000
STORY3	C301	W16X36	0.119 = 0.029 + 0.089 + 0.001	0.024	0.000
STORY2	C301	W16X36	0.164 = 0.161 + 0.003 + 0.000	0.019	0.000
STORY1	C301	W16X36	0.269 = 0.236 + 0.032 + 0.000	0.010	0.000
STORY3	C302	W16X36	0.177 = 0.038 + 0.092 + 0.047	0.024	0.002
STORY2	C302	W16X36	0.342 = 0.224 + 0.078 + 0.041	0.023	0.002
STORY1	C302	W16X36	0.394 = 0.331 + 0.043 + 0.021	0.012	0.001
STORY3	C303	W16X36	0.167 = 0.039 + 0.092 + 0.036	0.024	0.001
STORY2	C303	W16X36	0.341 = 0.231 + 0.078 + 0.032	0.023	0.001
STORY1	C303	W16X36	0.399 = 0.342 + 0.043 + 0.014	0.012	0.001
STORY3	C304	W16X36	0.108 = 0.014 + 0.059 + 0.034	0.016	0.001
STORY2	C304	W16X36	0.119 = 0.045 + 0.043 + 0.031	0.013	0.001
STORY1	C304	W16X36	0.100 = 0.065 + 0.022 + 0.013	0.007	0.001
STORY3	C305	W16X36	0.547 = 0.067 + 0.420 + 0.059	0.107	0.002
STORY2	C305	W16X50	0.721 = 0.262 + 0.399 + 0.060	0.128	0.003
STORY1	C305	W16X50	0.675 = 0.422 + 0.219 + 0.034	0.064	0.001
STORY3	C306	W16X36	0.403 = 0.034 + 0.209 + 0.160	0.052	0.006
STORY2	C306	W16X36	0.488 = 0.098 + 0.222 + 0.167	0.059	0.007
STORY1	C306	W16X36	0.473 = 0.288 + 0.108 + 0.078	0.029	0.003
STORY3	C307	W16X36	0.148 = 0.051 + 0.038 + 0.059	0.010	0.002
STORY2	C307	W16X36	0.368 = 0.282 + 0.041 + 0.045	0.012	0.002
STORY1	C307	W16X36	0.467 = 0.419 + 0.025 + 0.023	0.008	0.001
STORY3	C308	W16X36	0.238 = 0.057 + 0.116 + 0.065	0.028	0.002
STORY2	C308	W16X36	0.483 = 0.315 + 0.116 + 0.052	0.034	0.002
STORY1	C308	W16X36	0.560 = 0.471 + 0.063 + 0.027	0.017	0.001
STORY3	C309	W16X36	0.469 = 0.060 + 0.367 + 0.043	0.091	0.002
STORY2	C309	W16X36	0.736 = 0.332 + 0.357 + 0.048	0.105	0.002
STORY1	C309	W16X36	0.889 = 0.634 + 0.224 + 0.032	0.076	0.002
STORY3	C310	W16X36	0.267 = 0.022 + 0.101 + 0.144	0.025	0.005
STORY2	C310	W16X36	0.314 = 0.069 + 0.101 + 0.144	0.027	0.006
STORY1	C310	W16X36	0.221 = 0.098 + 0.052 + 0.072	0.013	0.003
STORY3	C311	W16X36	0.136 = 0.038 + 0.043 + 0.054	0.012	0.002
STORY2	C311	W16X36	0.303 = 0.217 + 0.044 + 0.041	0.013	0.002
STORY1	C311	W16X36	0.362 = 0.319 + 0.025 + 0.019	0.009	0.001
STORY3	C312	W16X36	0.227 = 0.058 + 0.115 + 0.054	0.028	0.002
STORY2	C312	W16X36	0.470 = 0.319 + 0.108 + 0.044	0.032	0.002
STORY1	C312	W16X36	0.553 = 0.475 + 0.057 + 0.021	0.016	0.001
STORY3	C313	W16X36	0.290 = 0.050 + 0.187 + 0.053	0.046	0.002
STORY2	C313	W16X36	0.488 = 0.278 + 0.166 + 0.044	0.050	0.002
STORY1	C313	W16X36	0.526 = 0.416 + 0.089 + 0.021	0.024	0.001
STORY3	C314	W16X36	0.177 = 0.034 + 0.093 + 0.050	0.028	0.002
STORY2	C314	W16X36	0.164 = 0.069 + 0.035 + 0.059	0.012	0.002
STORY1	C314	W16X36	0.136 = 0.093 + 0.009 + 0.033	0.005	0.001
STORY3	C315	W16X36	0.078 = 0.013 + 0.062 + 0.003	0.018	0.000
STORY2	C315	W16X36	0.146 = 0.146 + 0.001 + 0.000	0.008	0.000
STORY1	C315	W16X36	0.211 = 0.207 + 0.001 + 0.004	0.003	0.000

Steel Column Design - Capacity Check Output

Story Level	Column Line	Section Name	Moment Interaction Check Ratio = AXL + B33 + B22	Shear22 Ratio	Shear33 Ratio
STORY3	C316	W16X50	0.724 = 0.062 + 0.607 + 0.055	0.174	0.002
STORY2	C316	W16X50	0.885 = 0.256 + 0.570 + 0.058	0.180	0.003
STORY1	C316	W16X50	0.813 = 0.406 + 0.371 + 0.036	0.108	0.001
STORY3	C318	W16X36	0.415 = 0.257 + 0.094 + 0.063	0.027	0.003
STORY2	C318	W16X36	0.795 = 0.640 + 0.088 + 0.067	0.027	0.003
STORY1	C318	W16X50	0.783 = 0.686 + 0.055 + 0.041	0.017	0.002
STORY3	C319	W16X36	0.193 = 0.193 + 0.000 + 0.000	0.019	0.000
STORY2	C319	W16X36	0.548 = 0.475 + 0.070 + 0.003	0.021	0.000
STORY1	C319	W16X36	0.794 = 0.734 + 0.058 + 0.003	0.016	0.001
STORY3	C320	W16X36	0.672 = 0.307 + 0.307 + 0.058	0.090	0.002
STORY2	C320	W16X50	0.873 = 0.518 + 0.296 + 0.060	0.097	0.003
STORY1	C320	W16X67	0.647 = 0.445 + 0.160 + 0.043	0.062	0.002
STORY3	C321	W16X36	0.650 = 0.300 + 0.283 + 0.067	0.082	0.003
STORY2	C321	W16X50	0.849 = 0.510 + 0.273 + 0.065	0.090	0.003
STORY1	C321	W16X67	0.634 = 0.439 + 0.146 + 0.049	0.057	0.003
STORY3	C322	W16X36	0.673 = 0.307 + 0.310 + 0.056	0.090	0.002
STORY2	C322	W16X50	0.871 = 0.518 + 0.298 + 0.054	0.098	0.002
STORY1	C322	W16X67	0.644 = 0.446 + 0.157 + 0.041	0.061	0.003
STORY3	C323	W16X36	0.578 = 0.290 + 0.240 + 0.048	0.070	0.002
STORY2	C323	W16X50	0.769 = 0.492 + 0.224 + 0.052	0.074	0.002
STORY1	C323	W16X57	0.882 = 0.666 + 0.145 + 0.071	0.041	0.003
STORY3	C324	W16X36	0.411 = 0.216 + 0.191 + 0.004	0.054	0.000
STORY2	C324	W16X36	0.735 = 0.532 + 0.200 + 0.003	0.060	0.000
STORY1	C324	W16X50	0.680 = 0.568 + 0.109 + 0.003	0.032	0.000
STORY3	C325	W16X36	0.562 = 0.299 + 0.209 + 0.054	0.060	0.002
STORY2	C325	W16X50	0.760 = 0.503 + 0.201 + 0.057	0.066	0.003
STORY1	C325	W16X57	0.890 = 0.678 + 0.130 + 0.082	0.037	0.004
STORY3	C328	W16X36	0.640 = 0.269 + 0.315 + 0.056	0.091	0.002
STORY2	C328	W16X50	0.799 = 0.451 + 0.289 + 0.059	0.096	0.003
STORY1	C328	W16X57	0.881 = 0.609 + 0.187 + 0.085	0.053	0.004
STORY3	C329	W16X36	0.429 = 0.099 + 0.323 + 0.008	0.081	0.000
STORY2	C329	W16X36	0.780 = 0.487 + 0.286 + 0.008	0.086	0.000
STORY1	C329	W16X50	0.680 = 0.521 + 0.156 + 0.003	0.045	0.000
STORY3	C330	W16X36	0.578 = 0.247 + 0.328 + 0.003	0.096	0.000
STORY2	C330	W16X50	0.743 = 0.418 + 0.314 + 0.012	0.103	0.000
STORY1	C330	W16X50	0.852 = 0.651 + 0.194 + 0.008	0.056	0.001
STORY3	C717	W16X36	0.638 = 0.272 + 0.317 + 0.049	0.092	0.002
STORY2	C717	W16X50	0.796 = 0.457 + 0.290 + 0.049	0.097	0.002
STORY1	C717	W16X57	0.871 = 0.619 + 0.187 + 0.065	0.054	0.003
STORY3	C718	W16X36	0.572 = 0.295 + 0.227 + 0.049	0.066	0.002
STORY2	C718	W16X50	0.761 = 0.500 + 0.212 + 0.049	0.070	0.002
STORY1	C718	W16X57	0.878 = 0.676 + 0.136 + 0.065	0.039	0.003
STORY3	C719	W16X36	0.519 = 0.272 + 0.181 + 0.066	0.052	0.003
STORY2	C719	W16X50	0.702 = 0.464 + 0.174 + 0.064	0.057	0.003
STORY1	C719	W16X57	0.829 = 0.630 + 0.115 + 0.084	0.033	0.003
STORY3	C797	W16X36	0.499 = 0.088 + 0.343 + 0.067	0.087	0.002
STORY2	C797	W16X36	0.803 = 0.447 + 0.282 + 0.075	0.086	0.003
STORY1	C797	W16X50	0.675 = 0.488 + 0.148 + 0.040	0.043	0.002

Steel Column Design - Capacity Check Output

Story Level	Column Line	Section Name	Moment Interaction Check Ratio = AXL + B33 + B22	Shear22 Ratio	Shear33 Ratio
STORY3	C798	W16X36	0.421 = 0.071 + 0.255 + 0.095	0.065	0.003
STORY2	C798	W16X36	0.667 = 0.362 + 0.212 + 0.093	0.065	0.004
STORY1	C798	W16X36	0.811 = 0.574 + 0.161 + 0.076	0.043	0.003
STORY3	C799	W16X36	0.062 = 0.062 + 0.000 + 0.000	0.001	0.000
STORY2	C799	W16X36	0.215 = 0.212 + 0.002 + 0.001	0.002	0.000
STORY1	C799	W16X36	0.278 = 0.266 + 0.011 + 0.001	0.003	0.000
STORY3	C800	W16X36	0.036 = 0.036 + 0.000 + 0.000	0.001	0.000
STORY2	C800	W16X36	0.205 = 0.205 + 0.000 + 0.000	0.002	0.000
STORY1	C800	W16X36	0.268 = 0.266 + 0.000 + 0.002	0.002	0.000

Steel Column Design - Special Seismic Requirements

Steel Column Design - Special Seismic Requirements

Story Level	Column Line	Section Name	Section Class	Cont. Plate Area	Dbl. Plate Thickness	B/C Ratio Major	B/C Ratio Minor
STORY3	C42	W16X36	Compact				
STORY2	C42	W16X57	Seismic				
STORY1	C42	W16X50	Seismic				
STORY3	C43	W16X36	Compact				
STORY2	C43	W16X50	Seismic				
STORY1	C43	W16X36	Compact				
STORY3	C44	W16X50	Seismic				
STORY2	C44	W16X50	Seismic				
STORY1	C44	W16X50	Seismic				
STORY3	C45	W16X36	Compact				
STORY2	C45	W16X50	Seismic				
STORY1	C45	W16X50	Seismic				
STORY3	C47	W16X36	Compact				
STORY2	C47	W16X36	Compact				
STORY1	C47	W16X36	Compact				
STORY3	C48	W16X36	Compact				
STORY2	C48	W16X36	Compact				
STORY1	C48	W16X36	Compact				
STORY3	C50	W16X36	Compact				
STORY2	C50	W16X36	Compact				
STORY1	C50	W16X36	Compact				
STORY4	C51	W16X36	Compact				
STORY3	C51	W16X36	Compact				
STORY2	C51	W16X36	Compact				
STORY1	C51	W16X36	Compact				
STORY4	C52	W16X36	Compact				
STORY3	C52	W16X36	Compact				
STORY2	C52	W16X36	Compact				
STORY1	C52	W16X36	Compact				
STORY3	C53	W16X36	Compact				
STORY2	C53	W16X36	Compact				
STORY1	C53	W16X36	Compact				
STORY3	C54	W16X36	Compact				

Steel Column Design - Special Seismic Requirements

Story Level	Column Line	Section Name	Section Class	Cont. Plate Area	Dbl. Plate Thickness	B/C Ratio Major	B/C Ratio Minor
STORY2	C54	W16X36	Compact				
STORY1	C54	W16X36	Compact				
STORY3	C55	W16X36	Compact				
STORY2	C55	W16X36	Compact				
STORY1	C55	W16X36	Compact				
STORY3	C57	W16X36	Compact				
STORY2	C57	W16X50	Seismic				
STORY1	C57	W16X67	Compact				
STORY3	C58	W16X36	Compact				
STORY2	C58	W16X50	Seismic				
STORY1	C58	W16X67	Compact				
STORY3	C59	W16X36	Compact				
STORY2	C59	W16X36	Compact				
STORY1	C59	W16X50	Seismic				
STORY3	C60	W16X36	Compact				
STORY2	C60	W16X50	Seismic				
STORY1	C60	W16X67	Compact				
STORY3	C61	W16X36	Compact				
STORY2	C61	W16X50	Seismic				
STORY1	C61	W16X50	Seismic				
STORY3	C62	W16X36	Compact				
STORY2	C62	W16X50	Seismic				
STORY1	C62	W16X57	Seismic				
STORY3	C65	W16X36	Compact				
STORY2	C65	W16X50	Seismic				
STORY1	C65	W16X57	Seismic				
STORY3	C66	W16X36	Compact				
STORY2	C66	W16X36	Compact				
STORY1	C66	W16X50	Seismic				
STORY3	C69	W16X36	Compact				
STORY2	C69	W16X50	Seismic				
STORY1	C69	W16X57	Seismic				
STORY3	C70	W16X36	Compact				
STORY2	C70	W16X36	Compact				
STORY1	C70	W16X50	Seismic				
STORY3	C71	W16X36	Compact				
STORY2	C71	W16X36	Compact				
STORY1	C71	W16X36	Compact				
STORY4	C72	W16X36	Compact				
STORY3	C72	W16X36	Compact				
STORY2	C72	W16X36	Compact				
STORY1	C72	W16X36	Compact				
STORY4	C73	W16X36	Compact				
STORY3	C73	W16X36	Compact				
STORY2	C73	W16X36	Compact				
STORY1	C73	W16X36	Compact				
STORY4	C75	W16X36	Compact				
STORY3	C75	W16X36	Compact				

Steel Column Design - Special Seismic Requirements

Story Level	Column Line	Section Name	Section Class	Cont. Plate Area	Dbl. Plate Thickness	B/C Ratio Major	B/C Ratio Minor
STORY2	C75	W16X36	Compact				
STORY1	C75	W16X36	Compact				
STORY3	C76	W16X36	Compact				
STORY2	C76	W16X36	Compact				
STORY1	C76	W16X36	Compact				
STORY3	C77	W16X36	Compact				
STORY2	C77	W16X36	Compact				
STORY1	C77	W16X36	Compact				
STORY3	C78	W16X36	Compact				
STORY2	C78	W16X36	Compact				
STORY1	C78	W16X36	Compact				
STORY3	C79	W16X36	Compact				
STORY2	C79	W16X36	Compact				
STORY1	C79	W16X36	Compact				
STORY3	C80	W16X36	Compact				
STORY2	C80	W16X50	Seismic				
STORY1	C80	W16X57	Seismic				
STORY3	C81	W16X36	Compact				
STORY2	C81	W16X50	Seismic				
STORY1	C81	W16X57	Seismic				
STORY3	C82	W16X36	Compact				
STORY2	C82	W16X36	Compact				
STORY1	C82	W16X50	Seismic				
STORY3	C83	W16X36	Compact				
STORY2	C83	W16X50	Seismic				
STORY1	C83	W16X50	Seismic				
STORY3	C84	W16X36	Compact				
STORY2	C84	W16X36	Compact				
STORY1	C84	W16X36	Compact				
STORY3	C85	W16X36	Compact				
STORY2	C85	W16X36	Compact				
STORY1	C85	W16X36	Compact				
STORY3	C86	W16X36	Compact				
STORY2	C86	W16X36	Compact				
STORY1	C86	W16X36	Compact				
STORY3	C94	W16X36	Compact				
STORY2	C94	W16X36	Compact				
STORY1	C94	W16X36	Compact				
STORY4	C294	W16X36	Compact				
STORY3	C294	W16X36	Compact				
STORY2	C294	W16X36	Compact				
STORY1	C294	W16X36	Compact				
STORY4	C295	W16X36	Compact				
STORY3	C295	W16X36	Compact				
STORY2	C295	W16X36	Compact				
STORY1	C295	W16X36	Compact				
STORY4	C296	W16X36	Compact				
STORY3	C296	W16X36	Compact				

Steel Column Design - Special Seismic Requirements

Story Level	Column Line	Section Name	Section Class	Cont. Plate Area	Dbl. Plate Thickness	B/C Ratio Major	B/C Ratio Minor
STORY2	C296	W16X36	Compact				
STORY1	C296	W16X36	Compact				
STORY4	C297	W16X36	Compact				
STORY3	C297	W16X36	Compact				
STORY2	C297	W16X36	Compact				
STORY1	C297	W16X36	Compact				
STORY4	C298	W16X36	Compact				
STORY3	C298	W16X67	Compact				
STORY2	C298	W16X89	Seismic				
STORY1	C298	W16X67	Compact				
STORY3	C300	W16X36	Compact				
STORY2	C300	W16X36	Compact				
STORY1	C300	W16X36	Compact				
STORY3	C301	W16X36	Compact				
STORY2	C301	W16X36	Compact				
STORY1	C301	W16X36	Compact				
STORY3	C302	W16X36	Compact				
STORY2	C302	W16X36	Compact				
STORY1	C302	W16X36	Compact				
STORY3	C303	W16X36	Compact				
STORY2	C303	W16X36	Compact				
STORY1	C303	W16X36	Compact				
STORY3	C304	W16X36	Compact				
STORY2	C304	W16X36	Compact				
STORY1	C304	W16X36	Compact				
STORY3	C305	W16X36	Compact				
STORY2	C305	W16X50	Seismic				
STORY1	C305	W16X50	Seismic				
STORY3	C306	W16X36	Compact				
STORY2	C306	W16X36	Compact				
STORY1	C306	W16X36	Compact				
STORY3	C307	W16X36	Compact				
STORY2	C307	W16X36	Compact				
STORY1	C307	W16X36	Compact				
STORY3	C308	W16X36	Compact				
STORY2	C308	W16X36	Compact				
STORY1	C308	W16X36	Compact				
STORY3	C309	W16X36	Compact				
STORY2	C309	W16X36	Compact				
STORY1	C309	W16X36	Compact				
STORY3	C310	W16X36	Compact				
STORY2	C310	W16X36	Compact				
STORY1	C310	W16X36	Compact				
STORY3	C311	W16X36	Compact				
STORY2	C311	W16X36	Compact				
STORY1	C311	W16X36	Compact				
STORY3	C312	W16X36	Compact				
STORY2	C312	W16X36	Compact				

Steel Column Design - Special Seismic Requirements

Story Level	Column Line	Section Name	Section Class	Cont. Plate Area	Dbl. Plate Thickness	B/C Ratio Major	B/C Ratio Minor
STORY1	C312	W16X36	Compact				
STORY3	C313	W16X36	Compact				
STORY2	C313	W16X36	Compact				
STORY1	C313	W16X36	Compact				
STORY3	C314	W16X36	Compact				
STORY2	C314	W16X36	Compact				
STORY1	C314	W16X36	Compact				
STORY3	C315	W16X36	Compact				
STORY2	C315	W16X36	Compact				
STORY1	C315	W16X36	Compact				
STORY3	C316	W16X50	Seismic				
STORY2	C316	W16X50	Seismic				
STORY1	C316	W16X50	Seismic				
STORY3	C318	W16X36	Compact				
STORY2	C318	W16X36	Compact				
STORY1	C318	W16X50	Seismic				
STORY3	C319	W16X36	Compact				
STORY2	C319	W16X36	Compact				
STORY1	C319	W16X36	Compact				
STORY3	C320	W16X36	Compact				
STORY2	C320	W16X50	Seismic				
STORY1	C320	W16X67	Compact				
STORY3	C321	W16X36	Compact				
STORY2	C321	W16X50	Seismic				
STORY1	C321	W16X67	Compact				
STORY3	C322	W16X36	Compact				
STORY2	C322	W16X50	Seismic				
STORY1	C322	W16X67	Compact				
STORY3	C323	W16X36	Compact				
STORY2	C323	W16X50	Seismic				
STORY1	C323	W16X57	Seismic				
STORY3	C324	W16X36	Compact				
STORY2	C324	W16X36	Compact				
STORY1	C324	W16X50	Seismic				
STORY3	C325	W16X36	Compact				
STORY2	C325	W16X50	Seismic				
STORY1	C325	W16X57	Seismic				
STORY3	C328	W16X36	Compact				
STORY2	C328	W16X50	Seismic				
STORY1	C328	W16X57	Seismic				
STORY3	C329	W16X36	Compact				
STORY2	C329	W16X36	Compact				
STORY1	C329	W16X50	Seismic				
STORY3	C330	W16X36	Compact				
STORY2	C330	W16X50	Seismic				
STORY1	C330	W16X50	Seismic				
STORY3	C717	W16X36	Compact				
STORY2	C717	W16X50	Seismic				

Steel Column Design - Special Seismic Requirements

Story Level	Column Line	Section Name	Section Class	Cont. Plate Area	Dbl. Plate Thickness	B/C Ratio Major	B/C Ratio Minor
STORY1	C717	W16X57	Seismic				
STORY3	C718	W16X36	Compact				
STORY2	C718	W16X50	Seismic				
STORY1	C718	W16X57	Seismic				
STORY3	C719	W16X36	Compact				
STORY2	C719	W16X50	Seismic				
STORY1	C719	W16X57	Seismic				
STORY3	C797	W16X36	Compact				
STORY2	C797	W16X36	Compact				
STORY1	C797	W16X50	Seismic				
STORY3	C798	W16X36	Compact				
STORY2	C798	W16X36	Compact				
STORY1	C798	W16X36	Compact				
STORY3	C799	W16X36	Compact				
STORY2	C799	W16X36	Compact				
STORY1	C799	W16X36	Compact				
STORY3	C800	W16X36	Compact				
STORY2	C800	W16X36	Compact				
STORY1	C800	W16X36	Compact				

Steel Beam Design - Capacity Check Output

Steel Beam Design - Capacity Check Output

Story Level	Beam Bay	Section Name	Moment Interaction Check Ratio = AXL + B33 + B22	Shear22 Ratio	Shear33 Ratio
STORY3	B162	W21X50	0.132 = 0.000 + 0.132 + 0.000	0.045	0.000
STORY2	B162	W21X50	0.272 = 0.000 + 0.272 + 0.000	0.098	0.000
STORY1	B162	W21X50	0.248 = 0.000 + 0.248 + 0.000	0.085	0.000
STORY3	B164	W21X50	0.056 = 0.000 + 0.056 + 0.000	0.026	0.000
STORY2	B164	W21X50	0.148 = 0.000 + 0.148 + 0.000	0.063	0.000
STORY1	B164	W21X50	0.118 = 0.000 + 0.118 + 0.000	0.050	0.000
STORY3	B165	W21X62	0.055 = 0.000 + 0.055 + 0.000	0.031	0.000
STORY2	B165	W21X62	0.135 = 0.000 + 0.135 + 0.000	0.073	0.000
STORY1	B165	W21X62	0.112 = 0.000 + 0.112 + 0.000	0.058	0.000
STORY3	B166	W21X50	0.058 = 0.000 + 0.058 + 0.000	0.023	0.000
STORY2	B166	W21X50	0.141 = 0.000 + 0.141 + 0.000	0.056	0.000
STORY1	B166	W21X50	0.111 = 0.000 + 0.111 + 0.000	0.044	0.000
STORY3	B168	W21X50	0.044 = 0.000 + 0.044 + 0.000	0.021	0.000
STORY2	B168	W21X50	0.102 = 0.000 + 0.102 + 0.000	0.052	0.000
STORY1	B168	W21X50	0.075 = 0.000 + 0.075 + 0.000	0.040	0.000
STORY4	B178	W14X22	0.042 = 0.000 + 0.042 + 0.000	0.028	0.000
STORY3	B178	W21X50	0.023 = 0.010 + 0.010 + 0.003	0.003	0.000
STORY2	B178	W21X50	0.025 = 0.014 + 0.009 + 0.003	0.003	0.000
STORY1	B178	W14X22	0.101 = 0.028 + 0.070 + 0.003	0.032	0.000
STORY3	B179	W14X22	0.243 = 0.052 + 0.180 + 0.011	0.036	0.001
STORY2	B179	W14X22	0.206 = 0.003 + 0.189 + 0.014	0.036	0.001
STORY1	B179	W14X22	0.246 = 0.206 + 0.039 + 0.000	0.037	0.001
STORY4	B180	W14X22	0.049 = 0.000 + 0.049 + 0.000	0.029	0.000

Steel Beam Design - Capacity Check Output

Story Level	Beam Bay	Section Name	Moment Interaction Check Ratio = AXL + B33 + B22	Shear22 Ratio	Shear33 Ratio
STORY3	B180	W14X22	0.340 = 0.000 + 0.340 + 0.000	0.080	0.000
STORY2	B180	W14X22	0.664 = 0.000 + 0.664 + 0.000	0.150	0.000
STORY1	B180	W14X22	0.652 = 0.000 + 0.652 + 0.000	0.150	0.000
STORY3	B181	W14X22	0.029 = 0.000 + 0.029 + 0.000	0.014	0.000
STORY2	B181	W14X22	0.045 = 0.000 + 0.045 + 0.000	0.021	0.000
STORY1	B181	W14X22	0.055 = 0.000 + 0.055 + 0.000	0.024	0.000
STORY3	B182	W21X50	0.075 = 0.000 + 0.075 + 0.000	0.061	0.000
STORY2	B182	W21X50	0.129 = 0.000 + 0.129 + 0.000	0.118	0.000
STORY1	B182	W21X50	0.150 = 0.000 + 0.150 + 0.000	0.123	0.000
STORY3	B183	W21X50	0.191 = 0.000 + 0.191 + 0.000	0.099	0.000
STORY2	B183	W21X50	0.361 = 0.000 + 0.361 + 0.000	0.179	0.000
STORY1	B183	W21X50	0.366 = 0.000 + 0.366 + 0.000	0.178	0.000
STORY3	B184	W21X50	0.358 = 0.000 + 0.358 + 0.000	0.112	0.000
STORY2	B184	W21X50	0.573 = 0.000 + 0.573 + 0.000	0.193	0.000
STORY1	B184	W21X50	0.560 = 0.000 + 0.560 + 0.000	0.192	0.000
STORY3	B185	W21X50	0.466 = 0.000 + 0.466 + 0.000	0.156	0.000
STORY2	B185	W21X62	0.650 = 0.000 + 0.650 + 0.000	0.255	0.000
STORY1	B185	W21X62	0.651 = 0.000 + 0.651 + 0.000	0.257	0.000
STORY3	B186	W21X62	0.107 = 0.000 + 0.107 + 0.000	0.062	0.000
STORY2	B186	W21X62	0.287 = 0.000 + 0.287 + 0.000	0.115	0.000
STORY1	B186	W21X62	0.261 = 0.000 + 0.261 + 0.000	0.113	0.000
STORY3	B187	W21X50	0.128 = 0.000 + 0.128 + 0.000	0.050	0.000
STORY2	B187	W21X50	0.237 = 0.000 + 0.237 + 0.000	0.093	0.000
STORY1	B187	W21X50	0.220 = 0.000 + 0.220 + 0.000	0.091	0.000
STORY3	B188	W21X50	0.125 = 0.000 + 0.125 + 0.000	0.049	0.000
STORY2	B188	W21X50	0.192 = 0.000 + 0.192 + 0.000	0.088	0.000
STORY1	B188	W21X50	0.218 = 0.000 + 0.218 + 0.000	0.091	0.000
STORY3	B189	W21X62	0.107 = 0.000 + 0.107 + 0.000	0.061	0.000
STORY2	B189	W21X62	0.165 = 0.000 + 0.165 + 0.000	0.089	0.000
STORY1	B189	W21X62	0.262 = 0.000 + 0.262 + 0.000	0.113	0.000
STORY3	B190	W21X50	0.109 = 0.000 + 0.109 + 0.000	0.046	0.000
STORY2	B190	W21X50	0.180 = 0.000 + 0.180 + 0.000	0.085	0.000
STORY1	B190	W21X50	0.217 = 0.000 + 0.217 + 0.000	0.089	0.000
STORY3	B202	W21X50	0.082 = 0.000 + 0.082 + 0.000	0.028	0.000
STORY2	B202	W21X50	0.167 = 0.000 + 0.167 + 0.000	0.062	0.000
STORY1	B202	W21X50	0.162 = 0.000 + 0.162 + 0.000	0.055	0.000
STORY3	B203	W21X50	0.184 = 0.000 + 0.184 + 0.000	0.063	0.000
STORY2	B203	W21X50	0.411 = 0.000 + 0.411 + 0.000	0.135	0.000
STORY1	B203	W21X50	0.380 = 0.000 + 0.380 + 0.000	0.117	0.000
STORY3	B205	W21X50	0.425 = 0.000 + 0.425 + 0.000	0.146	0.000
STORY2	B205	W21X62	0.606 = 0.000 + 0.606 + 0.000	0.233	0.000
STORY1	B205	W21X62	0.602 = 0.000 + 0.602 + 0.000	0.236	0.000
STORY3	B206	W21X50	0.310 = 0.000 + 0.310 + 0.000	0.100	0.000
STORY2	B206	W21X50	0.410 = 0.000 + 0.410 + 0.000	0.130	0.000
STORY1	B206	W21X50	0.494 = 0.000 + 0.494 + 0.000	0.178	0.000
STORY3	B207	W21X50	0.387 = 0.000 + 0.387 + 0.000	0.124	0.000
STORY2	B207	W21X62	0.554 = 0.000 + 0.554 + 0.000	0.208	0.000
STORY1	B207	W21X50	0.723 = 0.000 + 0.723 + 0.000	0.216	0.000

Steel Beam Design - Capacity Check Output

Story Level	Beam Bay	Section Name	Moment Interaction Check Ratio = AXL + B33 + B22	Shear22 Ratio	Shear33 Ratio
STORY3	B208	W21X50	0.242 = 0.000 + 0.242 + 0.000	0.077	0.000
STORY2	B208	W21X50	0.379 = 0.000 + 0.379 + 0.000	0.136	0.000
STORY1	B208	W21X50	0.369 = 0.000 + 0.369 + 0.000	0.137	0.000
STORY3	B210	W21X50	0.355 = 0.000 + 0.355 + 0.000	0.103	0.000
STORY2	B210	W21X50	0.439 = 0.000 + 0.439 + 0.000	0.132	0.000
STORY1	B210	W21X50	0.528 = 0.000 + 0.528 + 0.000	0.181	0.000
STORY3	B211	W21X50	0.521 = 0.000 + 0.521 + 0.000	0.163	0.000
STORY2	B211	W21X68	0.639 = 0.000 + 0.639 + 0.000	0.234	0.000
STORY1	B211	W21X68	0.641 = 0.000 + 0.641 + 0.000	0.238	0.000
STORY3	B212	W21X50	0.173 = 0.000 + 0.173 + 0.000	0.056	0.000
STORY2	B212	W21X50	0.298 = 0.000 + 0.298 + 0.000	0.104	0.000
STORY1	B212	W21X50	0.320 = 0.000 + 0.320 + 0.000	0.110	0.000
STORY3	B213	W21X50	0.331 = 0.000 + 0.331 + 0.000	0.115	0.000
STORY2	B213	W21X50	0.644 = 0.000 + 0.644 + 0.000	0.202	0.000
STORY1	B213	W21X50	0.644 = 0.000 + 0.644 + 0.000	0.202	0.000
STORY3	B214	W21X50	0.279 = 0.000 + 0.279 + 0.000	0.082	0.000
STORY2	B214	W21X50	0.448 = 0.000 + 0.448 + 0.000	0.147	0.000
STORY1	B214	W21X50	0.453 = 0.000 + 0.453 + 0.000	0.148	0.000
STORY3	B215	W21X50	0.407 = 0.000 + 0.407 + 0.000	0.129	0.000
STORY2	B215	W21X62	0.579 = 0.000 + 0.579 + 0.000	0.205	0.000
STORY1	B215	W21X62	0.584 = 0.000 + 0.584 + 0.000	0.207	0.000
STORY3	B216	W21X50	0.234 = 0.000 + 0.234 + 0.000	0.070	0.000
STORY2	B216	W21X50	0.348 = 0.000 + 0.348 + 0.000	0.121	0.000
STORY1	B216	W21X50	0.357 = 0.000 + 0.357 + 0.000	0.125	0.000
STORY3	B217	W21X50	0.414 = 0.000 + 0.414 + 0.000	0.145	0.000
STORY2	B217	W21X62	0.605 = 0.000 + 0.605 + 0.000	0.236	0.000
STORY1	B217	W21X62	0.604 = 0.000 + 0.604 + 0.000	0.236	0.000
STORY3	B218	W21X50	0.346 = 0.000 + 0.346 + 0.000	0.102	0.000
STORY2	B218	W21X50	0.535 = 0.000 + 0.535 + 0.000	0.180	0.000
STORY1	B218	W21X50	0.533 = 0.000 + 0.533 + 0.000	0.181	0.000
STORY3	B219	W21X50	0.496 = 0.000 + 0.496 + 0.000	0.158	0.000
STORY2	B219	W21X68	0.636 = 0.000 + 0.636 + 0.000	0.235	0.000
STORY1	B219	W21X68	0.644 = 0.000 + 0.644 + 0.000	0.239	0.000
STORY3	B220	W21X50	0.098 = 0.000 + 0.098 + 0.000	0.043	0.000
STORY2	B220	W21X50	0.180 = 0.000 + 0.180 + 0.000	0.081	0.000
STORY1	B220	W21X50	0.180 = 0.000 + 0.180 + 0.000	0.080	0.000
STORY4	B221	W14X22	0.016 = 0.000 + 0.016 + 0.000	0.003	0.000
STORY3	B221	W14X22	0.150 = 0.000 + 0.150 + 0.000	0.070	0.000
STORY2	B221	W14X22	0.365 = 0.000 + 0.365 + 0.000	0.131	0.000
STORY1	B221	W14X22	0.376 = 0.000 + 0.376 + 0.000	0.130	0.000
STORY4	B222	W18X35	0.006 = 0.000 + 0.006 + 0.000	0.002	0.000
STORY3	B279	W21X50	0.110 = 0.000 + 0.110 + 0.000	0.042	0.000
STORY2	B279	W21X50	0.230 = 0.000 + 0.230 + 0.000	0.092	0.000
STORY1	B279	W21X50	0.207 = 0.000 + 0.207 + 0.000	0.079	0.000
STORY3	B280	W21X50	0.122 = 0.000 + 0.122 + 0.000	0.055	0.000
STORY2	B280	W21X50	0.260 = 0.000 + 0.260 + 0.000	0.116	0.000
STORY1	B280	W21X50	0.233 = 0.000 + 0.233 + 0.000	0.102	0.000
STORY3	B281	W21X50	0.160 = 0.000 + 0.160 + 0.000	0.027	0.000

Steel Beam Design - Capacity Check Output

Story Level	Beam Bay	Section Name	Moment Interaction Check Ratio = AXL + B33 + B22	Shear22 Ratio	Shear33 Ratio
STORY2	B281	W21X50	0.131 = 0.000 + 0.131 + 0.000	0.061	0.000
STORY1	B281	W21X50	0.177 = 0.000 + 0.177 + 0.000	0.051	0.000
STORY3	B282	W21X50	0.101 = 0.000 + 0.101 + 0.000	0.045	0.000
STORY2	B282	W21X50	0.216 = 0.000 + 0.216 + 0.000	0.088	0.000
STORY1	B282	W21X50	0.212 = 0.000 + 0.212 + 0.000	0.088	0.000
STORY3	B283	W21X50	0.096 = 0.000 + 0.096 + 0.000	0.046	0.000
STORY2	B283	W21X50	0.207 = 0.000 + 0.207 + 0.000	0.090	0.000
STORY1	B283	W21X50	0.220 = 0.000 + 0.220 + 0.000	0.092	0.000
STORY3	B290	W21X50	0.150 = 0.000 + 0.150 + 0.000	0.027	0.000
STORY2	B290	W21X50	0.243 = 0.000 + 0.243 + 0.000	0.060	0.000
STORY1	B290	W21X50	0.238 = 0.000 + 0.238 + 0.000	0.053	0.000
STORY3	B291	W21X50	0.067 = 0.000 + 0.067 + 0.000	0.024	0.000
STORY2	B291	W21X50	0.140 = 0.000 + 0.140 + 0.000	0.055	0.000
STORY1	B291	W21X50	0.114 = 0.000 + 0.114 + 0.000	0.041	0.000
STORY3	B293	W21X101	0.190 = 0.002 + 0.188 + 0.000	0.021	0.000
STORY2	B293	W21X101	0.289 = 0.000 + 0.288 + 0.000	0.042	0.000
STORY1	B293	W21X101	0.249 = 0.002 + 0.247 + 0.000	0.030	0.000
STORY3	B295	W21X101	0.237 = 0.001 + 0.236 + 0.000	0.025	0.000
STORY2	B295	W21X101	0.314 = 0.000 + 0.313 + 0.000	0.044	0.000
STORY1	B295	W21X101	0.256 = 0.000 + 0.256 + 0.000	0.030	0.000
STORY4	B300	W14X22	0.031 = 0.000 + 0.031 + 0.000	0.025	0.000
STORY4	B301	W14X22	0.011 = 0.000 + 0.011 + 0.000	0.009	0.000
STORY3	B1379	W21X50	0.133 = 0.000 + 0.133 + 0.000	0.045	0.000
STORY2	B1379	W21X50	0.271 = 0.000 + 0.271 + 0.000	0.098	0.000
STORY1	B1379	W21X50	0.248 = 0.000 + 0.248 + 0.000	0.085	0.000
STORY3	B1380	W21X50	0.056 = 0.000 + 0.056 + 0.000	0.026	0.000
STORY2	B1380	W21X50	0.148 = 0.000 + 0.148 + 0.000	0.063	0.000
STORY1	B1380	W21X50	0.118 = 0.000 + 0.118 + 0.000	0.050	0.000
STORY3	B1381	W21X62	0.055 = 0.000 + 0.055 + 0.000	0.031	0.000
STORY2	B1381	W21X62	0.135 = 0.000 + 0.135 + 0.000	0.073	0.000
STORY1	B1381	W21X62	0.112 = 0.000 + 0.112 + 0.000	0.058	0.000
STORY3	B1382	W21X50	0.058 = 0.000 + 0.058 + 0.000	0.023	0.000
STORY2	B1382	W21X50	0.141 = 0.000 + 0.141 + 0.000	0.056	0.000
STORY1	B1382	W21X50	0.111 = 0.000 + 0.111 + 0.000	0.044	0.000
STORY3	B1384	W21X50	0.043 = 0.000 + 0.043 + 0.000	0.021	0.000
STORY2	B1384	W21X50	0.102 = 0.000 + 0.102 + 0.000	0.052	0.000
STORY1	B1384	W21X50	0.075 = 0.000 + 0.075 + 0.000	0.040	0.000
STORY4	B1391	W14X22	0.041 = 0.000 + 0.041 + 0.000	0.028	0.000
STORY3	B1391	W21X50	0.079 = 0.005 + 0.071 + 0.003	0.012	0.000
STORY2	B1391	W21X50	0.136 = 0.007 + 0.129 + 0.000	0.018	0.000
STORY1	B1391	W14X22	0.170 = 0.022 + 0.147 + 0.001	0.039	0.000
STORY3	B1392	W14X22	0.246 = 0.049 + 0.186 + 0.012	0.037	0.001
STORY2	B1392	W14X22	0.213 = 0.005 + 0.194 + 0.014	0.036	0.001
STORY1	B1392	W14X22	0.244 = 0.205 + 0.038 + 0.000	0.036	0.001
STORY4	B1393	W14X22	0.049 = 0.000 + 0.049 + 0.000	0.029	0.000
STORY3	B1393	W14X22	0.340 = 0.000 + 0.340 + 0.000	0.080	0.000
STORY2	B1393	W14X22	0.664 = 0.000 + 0.664 + 0.000	0.150	0.000
STORY1	B1393	W14X22	0.652 = 0.000 + 0.652 + 0.000	0.150	0.000

Steel Beam Design - Capacity Check Output

Story Level	Beam Bay	Section Name	Moment Interaction Check Ratio = AXL + B33 + B22	Shear22 Ratio	Shear33 Ratio
STORY3	B1394	W14X22	0.025 = 0.000 + 0.025 + 0.000	0.011	0.000
STORY2	B1394	W14X22	0.051 = 0.000 + 0.051 + 0.000	0.027	0.000
STORY1	B1394	W14X22	0.070 = 0.000 + 0.070 + 0.000	0.040	0.000
STORY3	B1397	W21X50	0.335 = 0.000 + 0.335 + 0.000	0.100	0.000
STORY2	B1397	W21X50	0.529 = 0.000 + 0.529 + 0.000	0.178	0.000
STORY1	B1397	W21X50	0.521 = 0.000 + 0.521 + 0.000	0.178	0.000
STORY3	B1399	W21X62	0.109 = 0.000 + 0.109 + 0.000	0.061	0.000
STORY2	B1399	W21X62	0.292 = 0.000 + 0.292 + 0.000	0.114	0.000
STORY1	B1399	W21X62	0.263 = 0.000 + 0.263 + 0.000	0.113	0.000
STORY3	B1400	W21X50	0.114 = 0.000 + 0.114 + 0.000	0.045	0.000
STORY2	B1400	W21X50	0.225 = 0.000 + 0.225 + 0.000	0.090	0.000
STORY1	B1400	W21X50	0.213 = 0.000 + 0.213 + 0.000	0.090	0.000
STORY3	B1401	W21X50	0.107 = 0.000 + 0.107 + 0.000	0.044	0.000
STORY2	B1401	W21X50	0.218 = 0.000 + 0.218 + 0.000	0.090	0.000
STORY1	B1401	W21X50	0.226 = 0.000 + 0.226 + 0.000	0.091	0.000
STORY3	B1402	W21X62	0.109 = 0.000 + 0.109 + 0.000	0.061	0.000
STORY2	B1402	W21X62	0.295 = 0.000 + 0.295 + 0.000	0.114	0.000
STORY1	B1402	W21X62	0.275 = 0.000 + 0.275 + 0.000	0.113	0.000
STORY3	B1403	W21X50	0.110 = 0.000 + 0.110 + 0.000	0.046	0.000
STORY2	B1403	W21X50	0.225 = 0.000 + 0.225 + 0.000	0.089	0.000
STORY1	B1403	W21X50	0.217 = 0.000 + 0.217 + 0.000	0.089	0.000
STORY3	B1414	W21X50	0.082 = 0.000 + 0.082 + 0.000	0.028	0.000
STORY2	B1414	W21X50	0.167 = 0.000 + 0.167 + 0.000	0.062	0.000
STORY1	B1414	W21X50	0.162 = 0.000 + 0.162 + 0.000	0.055	0.000
STORY3	B1415	W21X50	0.184 = 0.000 + 0.184 + 0.000	0.063	0.000
STORY2	B1415	W21X50	0.411 = 0.000 + 0.411 + 0.000	0.135	0.000
STORY1	B1415	W21X50	0.380 = 0.000 + 0.380 + 0.000	0.117	0.000
STORY3	B1416	W21X50	0.244 = 0.000 + 0.244 + 0.000	0.077	0.000
STORY2	B1416	W21X50	0.374 = 0.000 + 0.374 + 0.000	0.135	0.000
STORY1	B1416	W21X50	0.371 = 0.000 + 0.371 + 0.000	0.137	0.000
STORY3	B1417	W21X50	0.419 = 0.000 + 0.419 + 0.000	0.145	0.000
STORY2	B1417	W21X62	0.605 = 0.000 + 0.605 + 0.000	0.236	0.000
STORY1	B1417	W21X62	0.602 = 0.000 + 0.602 + 0.000	0.235	0.000
STORY3	B1418	W21X50	0.314 = 0.000 + 0.314 + 0.000	0.101	0.000
STORY2	B1418	W21X50	0.506 = 0.000 + 0.506 + 0.000	0.178	0.000
STORY1	B1418	W21X50	0.496 = 0.000 + 0.496 + 0.000	0.178	0.000
STORY3	B1419	W21X50	0.379 = 0.000 + 0.379 + 0.000	0.125	0.000
STORY2	B1419	W21X62	0.553 = 0.000 + 0.553 + 0.000	0.205	0.000
STORY1	B1419	W21X62	0.554 = 0.000 + 0.554 + 0.000	0.205	0.000
STORY3	B1420	W21X50	0.242 = 0.000 + 0.242 + 0.000	0.077	0.000
STORY2	B1420	W21X50	0.373 = 0.000 + 0.373 + 0.000	0.135	0.000
STORY1	B1420	W21X50	0.371 = 0.000 + 0.371 + 0.000	0.137	0.000
STORY3	B1421	W21X50	0.427 = 0.000 + 0.427 + 0.000	0.146	0.000
STORY2	B1421	W21X62	0.604 = 0.000 + 0.604 + 0.000	0.237	0.000
STORY1	B1421	W21X62	0.602 = 0.000 + 0.602 + 0.000	0.236	0.000
STORY3	B1422	W21X50	0.334 = 0.000 + 0.334 + 0.000	0.106	0.000
STORY2	B1422	W21X50	0.533 = 0.000 + 0.533 + 0.000	0.185	0.000
STORY1	B1422	W21X50	0.516 = 0.000 + 0.516 + 0.000	0.183	0.000

Steel Beam Design - Capacity Check Output

Story Level	Beam Bay	Section Name	Moment Interaction Check Ratio = AXL + B33 + B22	Shear22 Ratio	Shear33 Ratio
STORY3	B1424	W21X50	0.172 = 0.000 + 0.172 + 0.000	0.056	0.000
STORY2	B1424	W21X50	0.299 = 0.000 + 0.299 + 0.000	0.104	0.000
STORY1	B1424	W21X50	0.319 = 0.000 + 0.319 + 0.000	0.110	0.000
STORY3	B1425	W21X50	0.339 = 0.000 + 0.339 + 0.000	0.116	0.000
STORY2	B1425	W21X50	0.644 = 0.000 + 0.644 + 0.000	0.203	0.000
STORY1	B1425	W21X50	0.643 = 0.000 + 0.643 + 0.000	0.202	0.000
STORY3	B1426	W21X50	0.255 = 0.000 + 0.255 + 0.000	0.083	0.000
STORY2	B1426	W21X50	0.441 = 0.000 + 0.441 + 0.000	0.151	0.000
STORY1	B1426	W21X50	0.439 = 0.000 + 0.439 + 0.000	0.151	0.000
STORY3	B1428	W21X50	0.235 = 0.000 + 0.235 + 0.000	0.071	0.000
STORY2	B1428	W21X50	0.349 = 0.000 + 0.349 + 0.000	0.122	0.000
STORY1	B1428	W21X50	0.357 = 0.000 + 0.357 + 0.000	0.125	0.000
STORY3	B1429	W21X50	0.415 = 0.000 + 0.415 + 0.000	0.145	0.000
STORY2	B1429	W21X62	0.605 = 0.000 + 0.605 + 0.000	0.236	0.000
STORY1	B1429	W21X62	0.604 = 0.000 + 0.604 + 0.000	0.236	0.000
STORY3	B1430	W21X50	0.346 = 0.000 + 0.346 + 0.000	0.102	0.000
STORY2	B1430	W21X50	0.535 = 0.000 + 0.535 + 0.000	0.180	0.000
STORY1	B1430	W21X50	0.532 = 0.000 + 0.532 + 0.000	0.181	0.000
STORY4	B1433	W14X22	0.014 = 0.000 + 0.014 + 0.000	0.002	0.000
STORY3	B1433	W14X22	0.112 = 0.000 + 0.112 + 0.000	0.053	0.000
STORY2	B1433	W14X22	0.203 = 0.000 + 0.203 + 0.000	0.098	0.000
STORY1	B1433	W14X22	0.213 = 0.000 + 0.213 + 0.000	0.097	0.000
STORY4	B1434	W18X35	0.006 = 0.000 + 0.006 + 0.000	0.002	0.000
STORY3	B1436	W21X50	0.126 = 0.000 + 0.126 + 0.000	0.043	0.000
STORY2	B1436	W21X50	0.264 = 0.000 + 0.264 + 0.000	0.094	0.000
STORY1	B1436	W21X50	0.240 = 0.000 + 0.240 + 0.000	0.081	0.000
STORY3	B1437	W21X50	0.119 = 0.000 + 0.119 + 0.000	0.055	0.000
STORY2	B1437	W21X50	0.253 = 0.000 + 0.253 + 0.000	0.115	0.000
STORY1	B1437	W21X50	0.226 = 0.000 + 0.226 + 0.000	0.101	0.000
STORY3	B1438	W21X50	0.083 = 0.000 + 0.083 + 0.000	0.026	0.000
STORY2	B1438	W21X50	0.191 = 0.000 + 0.191 + 0.000	0.060	0.000
STORY1	B1438	W21X50	0.206 = 0.000 + 0.206 + 0.000	0.048	0.000
STORY3	B1439	W21X50	0.106 = 0.000 + 0.106 + 0.000	0.046	0.000
STORY2	B1439	W21X50	0.220 = 0.000 + 0.220 + 0.000	0.089	0.000
STORY1	B1439	W21X50	0.214 = 0.000 + 0.214 + 0.000	0.088	0.000
STORY3	B1440	W21X50	0.102 = 0.000 + 0.102 + 0.000	0.049	0.000
STORY2	B1440	W21X50	0.217 = 0.000 + 0.217 + 0.000	0.095	0.000
STORY1	B1440	W21X50	0.214 = 0.000 + 0.214 + 0.000	0.095	0.000
STORY3	B1441	W21X50	0.154 = 0.000 + 0.154 + 0.000	0.025	0.000
STORY2	B1441	W21X50	0.243 = 0.000 + 0.243 + 0.000	0.058	0.000
STORY1	B1441	W21X50	0.128 = 0.000 + 0.128 + 0.000	0.048	0.000
STORY3	B1442	W21X50	0.067 = 0.000 + 0.067 + 0.000	0.024	0.000
STORY2	B1442	W21X50	0.140 = 0.000 + 0.140 + 0.000	0.055	0.000
STORY1	B1442	W21X50	0.113 = 0.000 + 0.113 + 0.000	0.040	0.000
STORY3	B1443	W21X101	0.197 = 0.000 + 0.196 + 0.000	0.021	0.000
STORY2	B1443	W21X101	0.222 = 0.001 + 0.220 + 0.000	0.041	0.000
STORY1	B1443	W21X101	0.291 = 0.000 + 0.291 + 0.000	0.029	0.000
STORY3	B1444	W21X101	0.245 = 0.001 + 0.245 + 0.000	0.025	0.000

Steel Beam Design - Capacity Check Output

Story Level	Beam Bay	Section Name	Moment Interaction Check Ratio = AXL + B33 + B22	Shear22 Ratio	Shear33 Ratio
STORY2	B1444	W21X101	0.246 = 0.001 + 0.245 + 0.000	0.043	0.000
STORY1	B1444	W21X101	0.315 = 0.001 + 0.314 + 0.000	0.031	0.000
STORY4	B1445	W14X22	0.031 = 0.000 + 0.031 + 0.000	0.025	0.000
STORY4	B1446	W14X22	0.010 = 0.000 + 0.010 + 0.000	0.009	0.000
STORY3	B1520	W21X50	0.481 = 0.000 + 0.481 + 0.000	0.162	0.000
STORY2	B1520	W21X50	0.878 = 0.000 + 0.878 + 0.000	0.276	0.000
STORY1	B1520	W21X62	0.684 = 0.000 + 0.684 + 0.000	0.266	0.000
STORY1	B1542	W21X50	0.048 = 0.000 + 0.048 + 0.000	0.014	0.000
STORY3	B1543	W21X50	0.023 = 0.000 + 0.023 + 0.000	0.010	0.000
STORY2	B1543	W21X50	0.018 = 0.000 + 0.018 + 0.000	0.010	0.000
STORY1	B1543	W21X50	0.029 = 0.000 + 0.029 + 0.000	0.014	0.000
STORY3	B1544	W21X50	0.006 = 0.000 + 0.006 + 0.000	0.002	0.000
STORY2	B1544	W21X50	0.006 = 0.000 + 0.006 + 0.000	0.002	0.000
STORY1	B1544	W21X50	0.011 = 0.000 + 0.011 + 0.000	0.003	0.000
STORY3	B1545	W21X50	0.006 = 0.000 + 0.006 + 0.000	0.002	0.000
STORY2	B1545	W21X50	0.006 = 0.000 + 0.006 + 0.000	0.002	0.000
STORY1	B1545	W21X50	0.011 = 0.000 + 0.011 + 0.000	0.003	0.000
STORY1	B1546	W21X50	0.013 = 0.000 + 0.013 + 0.000	0.005	0.000
STORY3	B1553	W21X50	0.007 = 0.000 + 0.007 + 0.000	0.012	0.000
STORY2	B1553	W21X50	0.014 = 0.000 + 0.014 + 0.000	0.022	0.000
STORY1	B1553	W21X50	0.014 = 0.000 + 0.014 + 0.000	0.022	0.000
STORY3	B1555	W21X50	0.026 = 0.000 + 0.026 + 0.000	0.007	0.000
STORY2	B1555	W21X50	0.032 = 0.000 + 0.032 + 0.000	0.014	0.000
STORY1	B1555	W21X50	0.022 = 0.000 + 0.022 + 0.000	0.009	0.000
STORY3	B1558	W21X50	0.007 = 0.000 + 0.007 + 0.000	0.003	0.000
STORY2	B1558	W21X50	0.005 = 0.000 + 0.005 + 0.000	0.003	0.000
STORY3	B1559	W21X50	0.037 = 0.000 + 0.037 + 0.000	0.011	0.000
STORY2	B1559	W21X50	0.035 = 0.000 + 0.035 + 0.000	0.010	0.000
STORY3	B2566	W21X50	0.241 = 0.000 + 0.241 + 0.000	0.077	0.000
STORY2	B2566	W21X50	0.380 = 0.000 + 0.380 + 0.000	0.136	0.000
STORY1	B2566	W21X50	0.369 = 0.000 + 0.369 + 0.000	0.137	0.000
STORY3	B2567	W21X50	0.417 = 0.000 + 0.417 + 0.000	0.145	0.000
STORY2	B2567	W21X62	0.606 = 0.000 + 0.606 + 0.000	0.233	0.000
STORY1	B2567	W21X62	0.603 = 0.000 + 0.603 + 0.000	0.236	0.000
STORY1	B2571	W21X50	0.052 = 0.000 + 0.052 + 0.000	0.014	0.000
STORY3	B2572	W21X50	0.007 = 0.000 + 0.007 + 0.000	0.012	0.000
STORY2	B2572	W21X50	0.014 = 0.000 + 0.014 + 0.000	0.022	0.000
STORY1	B2572	W21X50	0.014 = 0.000 + 0.014 + 0.000	0.022	0.000
STORY3	B2573	W21X50	0.003 = 0.000 + 0.003 + 0.000	0.002	0.000
STORY2	B2573	W21X50	0.008 = 0.000 + 0.008 + 0.000	0.008	0.000
STORY1	B2573	W21X50	0.010 = 0.000 + 0.010 + 0.000	0.005	0.000
STORY3	B2574	W21X50	0.007 = 0.000 + 0.007 + 0.000	0.004	0.000
STORY2	B2574	W21X50	0.006 = 0.000 + 0.006 + 0.000	0.004	0.000
STORY3	B2575	W21X50	0.036 = 0.000 + 0.036 + 0.000	0.010	0.000
STORY2	B2575	W21X50	0.038 = 0.000 + 0.038 + 0.000	0.011	0.000
STORY1	B2576	W21X50	0.013 = 0.000 + 0.013 + 0.000	0.006	0.000
STORY3	B2578	W21X50	0.006 = 0.000 + 0.006 + 0.000	0.002	0.000
STORY2	B2578	W21X50	0.008 = 0.000 + 0.008 + 0.000	0.002	0.000

Steel Beam Design - Capacity Check Output

Story Level	Beam Bay	Section Name	Moment Interaction Check Ratio = AXL + B33 + B22	Shear22 Ratio	Shear33 Ratio
STORY1	B2578	W21X50	0.008 = 0.000 + 0.008 + 0.000	0.002	0.000
STORY3	B2579	W21X50	0.029 = 0.000 + 0.029 + 0.000	0.014	0.000
STORY2	B2579	W21X50	0.031 = 0.000 + 0.031 + 0.000	0.015	0.000
STORY1	B2579	W21X50	0.039 = 0.000 + 0.039 + 0.000	0.018	0.000
STORY3	B2580	W21X50	0.006 = 0.000 + 0.006 + 0.000	0.002	0.000
STORY2	B2580	W21X50	0.008 = 0.000 + 0.008 + 0.000	0.002	0.000
STORY1	B2580	W21X50	0.008 = 0.000 + 0.008 + 0.000	0.002	0.000
STORY3	B2721	W21X50	0.177 = 0.000 + 0.177 + 0.000	0.062	0.000
STORY2	B2721	W21X50	0.412 = 0.000 + 0.412 + 0.000	0.135	0.000
STORY1	B2721	W21X50	0.382 = 0.000 + 0.382 + 0.000	0.118	0.000
STORY3	B2756	W21X50	0.299 = 0.000 + 0.299 + 0.000	0.119	0.000
STORY2	B2756	W21X50	0.589 = 0.000 + 0.589 + 0.000	0.221	0.000
STORY1	B2756	W21X50	0.589 = 0.000 + 0.589 + 0.000	0.221	0.000
STORY3	B3189	W21X50	0.030 = 0.000 + 0.030 + 0.000	0.007	0.000
STORY2	B3189	W21X50	0.046 = 0.000 + 0.046 + 0.000	0.015	0.000
STORY1	B3189	W21X50	0.044 = 0.000 + 0.044 + 0.000	0.009	0.000
STORY3	B3190	W21X50	0.088 = 0.000 + 0.088 + 0.000	0.040	0.000
STORY2	B3190	W21X50	0.187 = 0.000 + 0.187 + 0.000	0.086	0.000
STORY1	B3190	W21X50	0.167 = 0.000 + 0.167 + 0.000	0.074	0.000
STORY3	B3200	W21X50	0.237 = 0.000 + 0.237 + 0.000	0.093	0.000
STORY2	B3200	W21X50	0.455 = 0.000 + 0.455 + 0.000	0.172	0.000
STORY1	B3200	W21X50	0.452 = 0.000 + 0.452 + 0.000	0.171	0.000
STORY3	B3201	W21X50	0.495 = 0.000 + 0.495 + 0.000	0.158	0.000
STORY2	B3201	W21X68	0.635 = 0.000 + 0.635 + 0.000	0.235	0.000
STORY1	B3201	W21X68	0.643 = 0.000 + 0.643 + 0.000	0.239	0.000
STORY3	B3202	W21X50	0.451 = 0.000 + 0.451 + 0.000	0.155	0.000
STORY2	B3202	W21X62	0.651 = 0.000 + 0.651 + 0.000	0.254	0.000
STORY1	B3202	W21X62	0.653 = 0.000 + 0.653 + 0.000	0.256	0.000
STORY3	B3203	W21X50	0.057 = 0.000 + 0.057 + 0.000	0.039	0.000
STORY2	B3203	W21X50	0.137 = 0.000 + 0.137 + 0.000	0.078	0.000
STORY1	B3203	W21X50	0.145 = 0.000 + 0.145 + 0.000	0.079	0.000
STORY3	B3204	W21X50	0.118 = 0.000 + 0.118 + 0.000	0.045	0.000
STORY2	B3204	W21X50	0.221 = 0.000 + 0.221 + 0.000	0.087	0.000
STORY1	B3204	W21X50	0.203 = 0.000 + 0.203 + 0.000	0.085	0.000
STORY3	B3205	W21X62	0.102 = 0.000 + 0.102 + 0.000	0.059	0.000
STORY2	B3205	W21X62	0.266 = 0.000 + 0.266 + 0.000	0.110	0.000
STORY1	B3205	W21X62	0.243 = 0.000 + 0.243 + 0.000	0.110	0.000
STORY3	B3206	W21X50	0.124 = 0.000 + 0.124 + 0.000	0.047	0.000
STORY2	B3206	W21X50	0.228 = 0.000 + 0.228 + 0.000	0.090	0.000
STORY1	B3206	W21X50	0.210 = 0.000 + 0.210 + 0.000	0.087	0.000
STORY3	B3207	W21X50	0.119 = 0.000 + 0.119 + 0.000	0.047	0.000
STORY2	B3207	W21X50	0.224 = 0.000 + 0.224 + 0.000	0.090	0.000
STORY1	B3207	W21X50	0.206 = 0.000 + 0.206 + 0.000	0.087	0.000
STORY3	B3208	W21X62	0.102 = 0.000 + 0.102 + 0.000	0.059	0.000
STORY2	B3208	W21X62	0.265 = 0.000 + 0.265 + 0.000	0.110	0.000
STORY1	B3208	W21X62	0.242 = 0.000 + 0.242 + 0.000	0.109	0.000
STORY3	B3857	W21X50	0.108 = 0.000 + 0.108 + 0.000	0.050	0.000
STORY2	B3857	W21X50	0.223 = 0.000 + 0.223 + 0.000	0.096	0.000

Steel Beam Design - Capacity Check Output

Story Level	Beam Bay	Section Name	Moment Interaction Check Ratio = AXL + B33 + B22	Shear22 Ratio	Shear33 Ratio
STORY1	B3857	W21X50	0.219 = 0.000 + 0.219 + 0.000	0.095	0.000
STORY3	B3858	W21X50	0.108 = 0.000 + 0.108 + 0.000	0.050	0.000
STORY2	B3858	W21X50	0.223 = 0.000 + 0.223 + 0.000	0.096	0.000
STORY1	B3858	W21X50	0.218 = 0.000 + 0.218 + 0.000	0.095	0.000
STORY3	B3859	W21X62	0.053 = 0.000 + 0.053 + 0.000	0.027	0.000
STORY2	B3859	W21X62	0.096 = 0.000 + 0.096 + 0.000	0.062	0.000
STORY1	B3859	W21X62	0.091 = 0.000 + 0.091 + 0.000	0.050	0.000
STORY3	B3860	W21X62	0.100 = 0.000 + 0.100 + 0.000	0.054	0.000
STORY2	B3860	W21X62	0.166 = 0.000 + 0.166 + 0.000	0.097	0.000
STORY1	B3860	W21X62	0.166 = 0.000 + 0.166 + 0.000	0.095	0.000
STORY3	B3861	W21X62	0.102 = 0.000 + 0.102 + 0.000	0.060	0.000
STORY2	B3861	W21X62	0.265 = 0.000 + 0.265 + 0.000	0.111	0.000
STORY1	B3861	W21X62	0.242 = 0.000 + 0.242 + 0.000	0.109	0.000
STORY3	B3863	W21X50	0.057 = 0.000 + 0.057 + 0.000	0.020	0.000
STORY2	B3863	W21X50	0.091 = 0.000 + 0.091 + 0.000	0.032	0.000
STORY1	B3863	W21X50	0.091 = 0.000 + 0.091 + 0.000	0.045	0.000
STORY3	B3864	W21X50	0.093 = 0.000 + 0.093 + 0.000	0.030	0.000
STORY2	B3864	W21X50	0.133 = 0.000 + 0.133 + 0.000	0.043	0.000
STORY1	B3864	W21X50	0.123 = 0.000 + 0.123 + 0.000	0.038	0.000
STORY3	B3865	W21X62	0.073 = 0.000 + 0.073 + 0.000	0.037	0.000
STORY2	B3865	W21X62	0.120 = 0.000 + 0.120 + 0.000	0.064	0.000
STORY1	B3865	W21X62	0.124 = 0.000 + 0.124 + 0.000	0.063	0.000
STORY3	B3866	W21X62	0.068 = 0.000 + 0.068 + 0.000	0.034	0.000
STORY2	B3866	W21X62	0.123 = 0.000 + 0.123 + 0.000	0.061	0.000
STORY1	B3866	W21X62	0.127 = 0.000 + 0.127 + 0.000	0.061	0.000
STORY3	B3869	W21X62	0.066 = 0.000 + 0.066 + 0.000	0.034	0.000
STORY2	B3869	W21X62	0.123 = 0.000 + 0.123 + 0.000	0.062	0.000
STORY1	B3869	W21X62	0.128 = 0.000 + 0.128 + 0.000	0.062	0.000
STORY3	B3870	W21X50	0.061 = 0.000 + 0.061 + 0.000	0.018	0.000
STORY2	B3870	W21X50	0.088 = 0.000 + 0.088 + 0.000	0.046	0.000
STORY1	B3870	W21X50	0.123 = 0.000 + 0.123 + 0.000	0.068	0.000
STORY3	B3871	W21X50	0.065 = 0.000 + 0.065 + 0.000	0.018	0.000
STORY2	B3871	W21X50	0.099 = 0.000 + 0.099 + 0.000	0.045	0.000
STORY1	B3871	W21X50	0.120 = 0.000 + 0.120 + 0.000	0.067	0.000
STORY3	B3872	W21X62	0.066 = 0.000 + 0.066 + 0.000	0.034	0.000
STORY2	B3872	W21X62	0.123 = 0.000 + 0.123 + 0.000	0.061	0.000
STORY1	B3872	W21X62	0.128 = 0.000 + 0.128 + 0.000	0.062	0.000
STORY3	B3873	W21X62	0.103 = 0.000 + 0.103 + 0.000	0.060	0.000
STORY2	B3873	W21X62	0.271 = 0.000 + 0.271 + 0.000	0.111	0.000
STORY1	B3873	W21X62	0.256 = 0.000 + 0.256 + 0.000	0.110	0.000
STORY3	B3874	W21X50	0.124 = 0.000 + 0.124 + 0.000	0.048	0.000
STORY2	B3874	W21X50	0.229 = 0.000 + 0.229 + 0.000	0.090	0.000
STORY1	B3874	W21X50	0.213 = 0.000 + 0.213 + 0.000	0.088	0.000
STORY3	B3875	W21X50	0.124 = 0.000 + 0.124 + 0.000	0.048	0.000
STORY2	B3875	W21X50	0.183 = 0.000 + 0.183 + 0.000	0.085	0.000
STORY1	B3875	W21X50	0.209 = 0.000 + 0.209 + 0.000	0.087	0.000
STORY3	B3876	W21X62	0.102 = 0.000 + 0.102 + 0.000	0.059	0.000
STORY2	B3876	W21X62	0.156 = 0.000 + 0.156 + 0.000	0.084	0.000

Steel Beam Design - Capacity Check Output

Story Level	Beam Bay	Section Name	Moment Interaction Check Ratio = AXL + B33 + B22	Shear22 Ratio	Shear33 Ratio
STORY1	B3876	W21X62	0.242 = 0.000 + 0.242 + 0.000	0.109	0.000
STORY3	B3879	W21X50	0.110 = 0.000 + 0.110 + 0.000	0.050	0.000
STORY2	B3879	W21X50	0.182 = 0.000 + 0.182 + 0.000	0.091	0.000
STORY1	B3879	W21X50	0.220 = 0.000 + 0.220 + 0.000	0.095	0.000
STORY3	B3880	W21X50	0.057 = 0.000 + 0.057 + 0.000	0.039	0.000
STORY2	B3880	W21X50	0.137 = 0.000 + 0.137 + 0.000	0.078	0.000
STORY1	B3880	W21X50	0.146 = 0.000 + 0.146 + 0.000	0.079	0.000
STORY3	B3881	W21X50	0.118 = 0.000 + 0.118 + 0.000	0.045	0.000
STORY2	B3881	W21X50	0.221 = 0.000 + 0.221 + 0.000	0.087	0.000
STORY1	B3881	W21X50	0.203 = 0.000 + 0.203 + 0.000	0.085	0.000
STORY3	B3882	W21X62	0.102 = 0.000 + 0.102 + 0.000	0.059	0.000
STORY2	B3882	W21X62	0.266 = 0.000 + 0.266 + 0.000	0.110	0.000
STORY1	B3882	W21X62	0.243 = 0.000 + 0.243 + 0.000	0.110	0.000
STORY3	B3883	W21X50	0.108 = 0.000 + 0.108 + 0.000	0.050	0.000
STORY2	B3883	W21X50	0.223 = 0.000 + 0.223 + 0.000	0.096	0.000
STORY1	B3883	W21X50	0.219 = 0.000 + 0.219 + 0.000	0.095	0.000
STORY3	B3884	W21X62	0.096 = 0.000 + 0.096 + 0.000	0.053	0.000
STORY2	B3884	W21X62	0.161 = 0.000 + 0.161 + 0.000	0.096	0.000
STORY1	B3884	W21X62	0.160 = 0.000 + 0.160 + 0.000	0.095	0.000
STORY3	B3885	W21X62	0.048 = 0.000 + 0.048 + 0.000	0.026	0.000
STORY2	B3885	W21X62	0.097 = 0.000 + 0.097 + 0.000	0.062	0.000
STORY1	B3885	W21X62	0.090 = 0.000 + 0.090 + 0.000	0.050	0.000
STORY3	B3942	W18X35	0.139 = 0.000 + 0.139 + 0.000	0.048	0.000
STORY2	B3942	W18X35	0.217 = 0.000 + 0.217 + 0.000	0.086	0.000
STORY1	B3942	W18X35	0.227 = 0.000 + 0.227 + 0.000	0.087	0.000
STORY3	B3943	W18X55	0.091 = 0.000 + 0.091 + 0.000	0.038	0.000
STORY2	B3943	W18X55	0.137 = 0.000 + 0.137 + 0.000	0.066	0.000
STORY1	B3943	W18X55	0.145 = 0.000 + 0.145 + 0.000	0.067	0.000
STORY3	B3944	W18X55	0.084 = 0.000 + 0.084 + 0.000	0.037	0.000
STORY2	B3944	W18X55	0.130 = 0.000 + 0.130 + 0.000	0.065	0.000
STORY1	B3944	W18X55	0.160 = 0.000 + 0.160 + 0.000	0.069	0.000
STORY3	B3945	W18X35	0.125 = 0.000 + 0.125 + 0.000	0.046	0.000
STORY2	B3945	W18X35	0.201 = 0.000 + 0.201 + 0.000	0.084	0.000
STORY1	B3945	W18X35	0.216 = 0.000 + 0.216 + 0.000	0.086	0.000
STORY3	B3948	W21X50	0.087 = 0.000 + 0.087 + 0.000	0.041	0.000
STORY2	B3948	W21X50	0.160 = 0.000 + 0.160 + 0.000	0.078	0.000
STORY1	B3948	W21X50	0.159 = 0.000 + 0.159 + 0.000	0.078	0.000

Steel Beam Design - Special Seismic Requirements

Steel Beam Design - Special Seismic Requirements

Story Level	Beam Bay	Section Name	Section Class	Connection Shear End-I	Connection Shear End-J
STORY3	B162	W21X50	Seismic	8.91	10.76
STORY2	B162	W21X50	Seismic	19.70	23.16
STORY1	B162	W21X50	Seismic	16.36	20.14
STORY3	B164	W21X50	Seismic	6.14	3.39
STORY2	B164	W21X50	Seismic	14.96	7.62

Steel Beam Design - Special Seismic Requirements

Story Level	Beam Bay	Section Name	Section Class	Connection Shear End-I	Connection Shear End-J
STORY1	B164	W21X50	Seismic	11.78	5.87
STORY3	B165	W21X62	Seismic	7.89	7.82
STORY2	B165	W21X62	Seismic	18.26	18.46
STORY1	B165	W21X62	Seismic	14.30	14.49
STORY3	B166	W21X50	Seismic	4.07	5.46
STORY2	B166	W21X50	Seismic	9.21	13.36
STORY1	B166	W21X50	Seismic	7.18	10.47
STORY3	B168	W21X50	Seismic	5.10	4.43
STORY2	B168	W21X50	Seismic	12.38	10.20
STORY1	B168	W21X50	Seismic	9.57	8.09
STORY4	B178	W14X22	Compact	2.57	2.63
STORY3	B178	W21X50	Seismic	7.378E-01	4.567E-01
STORY2	B178	W21X50	Seismic	7.064E-01	3.629E-01
STORY1	B178	W14X22	Compact	3.01	2.34
STORY3	B179	W14X22	Compact	3.43	1.71
STORY2	B179	W14X22	Compact	3.45	1.99
STORY1	B179	W14X22	Compact	3.46	2.08
STORY4	B180	W14X22	Compact	2.73	2.76
STORY3	B180	W14X22	Compact	7.51	7.52
STORY2	B180	W14X22	Compact	14.20	14.19
STORY1	B180	W14X22	Compact	14.19	14.20
STORY3	B181	W14X22	Compact	7.140E-01	1.35
STORY2	B181	W14X22	Compact	1.81	2.02
STORY1	B181	W14X22	Compact	2.30	1.51
STORY3	B182	W21X50	Seismic	14.50	6.621E-01
STORY2	B182	W21X50	Seismic	28.06	6.867E-01
STORY1	B182	W21X50	Seismic	29.23	7.841E-01
STORY3	B183	W21X50	Seismic	21.46	23.40
STORY2	B183	W21X50	Seismic	38.38	42.36
STORY1	B183	W21X50	Seismic	38.51	42.22
STORY3	B184	W21X50	Seismic	26.54	20.17
STORY2	B184	W21X50	Seismic	45.82	37.87
STORY1	B184	W21X50	Seismic	45.42	38.27
STORY3	B185	W21X50	Seismic	33.74	37.01
STORY2	B185	W21X62	Seismic	58.40	64.27
STORY1	B185	W21X62	Seismic	57.74	64.68
STORY3	B186	W21X62	Seismic	14.85	15.66
STORY2	B186	W21X62	Seismic	28.37	28.93
STORY1	B186	W21X62	Seismic	28.31	28.42
STORY3	B187	W21X50	Seismic	11.74	7.09
STORY2	B187	W21X50	Seismic	22.10	13.70
STORY1	B187	W21X50	Seismic	21.69	13.82
STORY3	B188	W21X50	Seismic	7.18	11.65
STORY2	B188	W21X50	Seismic	14.83	20.97
STORY1	B188	W21X50	Seismic	13.88	21.62
STORY3	B189	W21X62	Seismic	15.49	15.03
STORY2	B189	W21X62	Seismic	22.39	22.08
STORY1	B189	W21X62	Seismic	28.41	28.32

Steel Beam Design - Special Seismic Requirements

Story Level	Beam Bay	Section Name	Section Class	Connection Shear End-I	Connection Shear End-J
STORY3	B190	W21X50	Seismic	10.96	5.55
STORY2	B190	W21X50	Seismic	20.06	11.20
STORY1	B190	W21X50	Seismic	20.99	10.02
STORY3	B202	W21X50	Seismic	6.68	3.55
STORY2	B202	W21X50	Seismic	14.78	7.76
STORY1	B202	W21X50	Seismic	13.13	5.31
STORY3	B203	W21X50	Seismic	15.00	14.32
STORY2	B203	W21X50	Seismic	32.07	31.10
STORY1	B203	W21X50	Seismic	27.73	26.84
STORY3	B205	W21X50	Seismic	34.52	33.24
STORY2	B205	W21X62	Seismic	58.84	58.45
STORY1	B205	W21X62	Seismic	59.43	57.86
STORY3	B206	W21X50	Seismic	22.90	23.82
STORY2	B206	W21X50	Seismic	30.10	30.90
STORY1	B206	W21X50	Seismic	41.52	42.18
STORY3	B207	W21X50	Seismic	29.50	29.04
STORY2	B207	W21X62	Seismic	52.30	50.72
STORY1	B207	W21X50	Seismic	50.68	51.22
STORY3	B208	W21X50	Seismic	18.25	6.39
STORY2	B208	W21X50	Seismic	32.18	14.24
STORY1	B208	W21X50	Seismic	32.45	13.96
STORY3	B210	W21X50	Seismic	24.50	22.22
STORY2	B210	W21X50	Seismic	31.33	29.67
STORY1	B210	W21X50	Seismic	42.82	40.87
STORY3	B211	W21X50	Seismic	34.21	38.68
STORY2	B211	W21X68	Seismic	61.92	63.70
STORY1	B211	W21X68	Seismic	60.80	64.83
STORY3	B212	W21X50	Seismic	13.37	5.95
STORY2	B212	W21X50	Seismic	24.62	11.43
STORY1	B212	W21X50	Seismic	26.17	9.88
STORY3	B213	W21X50	Seismic	27.32	26.43
STORY2	B213	W21X50	Seismic	47.98	46.74
STORY1	B213	W21X50	Seismic	47.81	46.90
STORY3	B214	W21X50	Seismic	19.39	17.82
STORY2	B214	W21X50	Seismic	34.77	33.47
STORY1	B214	W21X50	Seismic	35.01	33.22
STORY3	B215	W21X50	Seismic	27.30	30.49
STORY2	B215	W21X62	Seismic	49.87	51.75
STORY1	B215	W21X62	Seismic	49.49	52.12
STORY3	B216	W21X50	Seismic	16.69	5.24
STORY2	B216	W21X50	Seismic	28.75	12.48
STORY1	B216	W21X50	Seismic	29.70	11.53
STORY3	B217	W21X50	Seismic	34.36	33.40
STORY2	B217	W21X62	Seismic	59.51	57.78
STORY1	B217	W21X62	Seismic	59.57	57.72
STORY3	B218	W21X50	Seismic	24.19	22.52
STORY2	B218	W21X50	Seismic	42.66	41.03
STORY1	B218	W21X50	Seismic	42.85	40.84

Steel Beam Design - Special Seismic Requirements

Story Level	Beam Bay	Section Name	Section Class	Connection Shear End-I	Connection Shear End-J
STORY3	B219	W21X50	Seismic	35.39	37.51
STORY2	B219	W21X68	Seismic	61.61	64.01
STORY1	B219	W21X68	Seismic	60.48	65.14
STORY3	B220	W21X50	Seismic	10.10	9.86
STORY2	B220	W21X50	Seismic	19.22	18.81
STORY1	B220	W21X50	Seismic	19.06	18.96
STORY4	B221	W14X22	Compact	9.591E-02	2.489E-01
STORY3	B221	W14X22	Compact	4.93	6.63
STORY2	B221	W14X22	Compact	9.77	12.38
STORY1	B221	W14X22	Compact	9.86	12.29
STORY4	B222	W18X35	Slender	3.673E-01	3.673E-01
STORY3	B279	W21X50	Seismic	10.08	9.60
STORY2	B279	W21X50	Seismic	21.89	20.97
STORY1	B279	W21X50	Seismic	18.77	17.73
STORY3	B280	W21X50	Seismic	10.66	13.11
STORY2	B280	W21X50	Seismic	23.84	27.50
STORY1	B280	W21X50	Seismic	20.25	24.14
STORY3	B281	W21X50	Seismic	6.50	9.559E-01
STORY2	B281	W21X50	Seismic	14.52	2.49
STORY1	B281	W21X50	Seismic	11.98	2.05
STORY3	B282	W21X50	Seismic	5.75	10.75
STORY2	B282	W21X50	Seismic	10.29	20.97
STORY1	B282	W21X50	Seismic	10.16	20.85
STORY3	B283	W21X50	Seismic	6.20	10.92
STORY2	B283	W21X50	Seismic	11.16	21.31
STORY1	B283	W21X50	Seismic	10.75	21.72
STORY3	B290	W21X50	Seismic	6.44	9.504E-01
STORY2	B290	W21X50	Seismic	14.26	2.79
STORY1	B290	W21X50	Seismic	12.52	2.59
STORY3	B291	W21X50	Seismic	3.73	5.66
STORY2	B291	W21X50	Seismic	10.09	13.06
STORY1	B291	W21X50	Seismic	6.51	9.69
STORY3	B293	W21X101	Compact	3.13	6.77
STORY2	B293	W21X101	Compact	4.02	13.35
STORY1	B293	W21X101	Compact	6.02	9.50
STORY3	B295	W21X101	Compact	4.39	7.94
STORY2	B295	W21X101	Compact	3.63	14.02
STORY1	B295	W21X101	Compact	6.24	9.72
STORY4	B300	W14X22	Compact	8.709E-01	8.985E-01
STORY4	B301	W14X22	Compact	8.963E-01	5.033E-01
STORY3	B1379	W21X50	Seismic	10.78	8.89
STORY2	B1379	W21X50	Seismic	23.15	19.71
STORY1	B1379	W21X50	Seismic	20.15	16.35
STORY3	B1380	W21X50	Seismic	6.15	3.37
STORY2	B1380	W21X50	Seismic	14.98	7.60
STORY1	B1380	W21X50	Seismic	11.78	5.87
STORY3	B1381	W21X62	Seismic	7.90	7.82
STORY2	B1381	W21X62	Seismic	18.27	18.45

Steel Beam Design - Special Seismic Requirements

Story Level	Beam Bay	Section Name	Section Class	Connection Shear End-I	Connection Shear End-J
STORY1	B1381	W21X62	Seismic	14.30	14.49
STORY3	B1382	W21X50	Seismic	4.09	5.44
STORY2	B1382	W21X50	Seismic	9.22	13.36
STORY1	B1382	W21X50	Seismic	7.19	10.47
STORY3	B1384	W21X50	Seismic	5.05	4.47
STORY2	B1384	W21X50	Seismic	12.40	10.18
STORY1	B1384	W21X50	Seismic	9.57	8.08
STORY4	B1391	W14X22	Compact	2.66	2.55
STORY3	B1391	W21X50	Seismic	1.99	2.86
STORY2	B1391	W21X50	Seismic	3.46	4.30
STORY1	B1391	W14X22	Compact	1.81	3.68
STORY3	B1392	W14X22	Compact	3.47	1.84
STORY2	B1392	W14X22	Compact	3.39	1.90
STORY1	B1392	W14X22	Compact	3.43	2.08
STORY4	B1393	W14X22	Compact	2.78	2.71
STORY3	B1393	W14X22	Compact	7.50	7.54
STORY2	B1393	W14X22	Compact	14.19	14.21
STORY1	B1393	W14X22	Compact	14.20	14.19
STORY3	B1394	W14X22	Compact	9.894E-01	1.01
STORY2	B1394	W14X22	Compact	2.55	1.24
STORY1	B1394	W14X22	Compact	3.80	4.648E-01
STORY3	B1397	W21X50	Seismic	23.66	23.05
STORY2	B1397	W21X50	Seismic	42.11	41.58
STORY1	B1397	W21X50	Seismic	42.12	41.57
STORY3	B1399	W21X62	Seismic	15.16	15.35
STORY2	B1399	W21X62	Seismic	28.60	28.70
STORY1	B1399	W21X62	Seismic	28.43	28.30
STORY3	B1400	W21X50	Seismic	10.71	8.11
STORY2	B1400	W21X50	Seismic	21.43	14.37
STORY1	B1400	W21X50	Seismic	21.27	14.23
STORY3	B1401	W21X50	Seismic	8.28	10.55
STORY2	B1401	W21X50	Seismic	14.56	21.24
STORY1	B1401	W21X50	Seismic	14.11	21.68
STORY3	B1402	W21X62	Seismic	15.21	15.30
STORY2	B1402	W21X62	Seismic	28.49	28.81
STORY1	B1402	W21X62	Seismic	28.46	28.55
STORY3	B1403	W21X50	Seismic	10.98	5.52
STORY2	B1403	W21X50	Seismic	21.20	10.06
STORY1	B1403	W21X50	Seismic	21.00	10.01
STORY3	B1414	W21X50	Seismic	3.59	6.66
STORY2	B1414	W21X50	Seismic	7.76	14.78
STORY1	B1414	W21X50	Seismic	5.30	13.14
STORY3	B1415	W21X50	Seismic	14.33	15.00
STORY2	B1415	W21X50	Seismic	31.10	32.08
STORY1	B1415	W21X50	Seismic	26.83	27.74
STORY3	B1416	W21X50	Seismic	6.30	18.34
STORY2	B1416	W21X50	Seismic	14.40	32.02
STORY1	B1416	W21X50	Seismic	13.88	32.54

Steel Beam Design - Special Seismic Requirements

Story Level	Beam Bay	Section Name	Section Class	Connection Shear End-I	Connection Shear End-J
STORY3	B1417	W21X50	Seismic	33.37	34.39
STORY2	B1417	W21X62	Seismic	57.82	59.47
STORY1	B1417	W21X62	Seismic	57.96	59.33
STORY3	B1418	W21X50	Seismic	23.84	22.87
STORY2	B1418	W21X50	Seismic	42.26	41.43
STORY1	B1418	W21X50	Seismic	42.17	41.52
STORY3	B1419	W21X50	Seismic	28.86	29.55
STORY2	B1419	W21X62	Seismic	51.22	51.54
STORY1	B1419	W21X62	Seismic	51.64	50.88
STORY3	B1420	W21X50	Seismic	6.34	18.29
STORY2	B1420	W21X50	Seismic	14.42	31.99
STORY1	B1420	W21X50	Seismic	13.90	32.51
STORY3	B1421	W21X50	Seismic	33.20	34.56
STORY2	B1421	W21X62	Seismic	57.69	59.60
STORY1	B1421	W21X62	Seismic	57.89	59.40
STORY3	B1422	W21X50	Seismic	25.23	21.48
STORY2	B1422	W21X50	Seismic	43.90	39.79
STORY1	B1422	W21X50	Seismic	43.42	40.27
STORY3	B1424	W21X50	Seismic	5.94	13.37
STORY2	B1424	W21X50	Seismic	11.37	24.69
STORY1	B1424	W21X50	Seismic	9.90	26.15
STORY3	B1425	W21X50	Seismic	26.28	27.47
STORY2	B1425	W21X50	Seismic	46.61	48.10
STORY1	B1425	W21X50	Seismic	46.74	47.97
STORY3	B1426	W21X50	Seismic	19.71	17.49
STORY2	B1426	W21X50	Seismic	35.78	32.46
STORY1	B1426	W21X50	Seismic	35.83	32.41
STORY3	B1428	W21X50	Seismic	5.18	16.75
STORY2	B1428	W21X50	Seismic	12.38	28.85
STORY1	B1428	W21X50	Seismic	11.51	29.73
STORY3	B1429	W21X50	Seismic	33.38	34.38
STORY2	B1429	W21X62	Seismic	57.75	59.54
STORY1	B1429	W21X62	Seismic	57.71	59.58
STORY3	B1430	W21X50	Seismic	22.51	24.21
STORY2	B1430	W21X50	Seismic	41.00	42.69
STORY1	B1430	W21X50	Seismic	40.85	42.85
STORY4	B1433	W14X22	Compact	9.735E-02	2.354E-01
STORY3	B1433	W14X22	Compact	3.74	5.03
STORY2	B1433	W14X22	Compact	7.40	9.28
STORY1	B1433	W14X22	Compact	7.52	9.18
STORY4	B1434	W18X35	Slender	3.673E-01	3.673E-01
STORY3	B1436	W21X50	Seismic	10.23	9.45
STORY2	B1436	W21X50	Seismic	22.37	20.49
STORY1	B1436	W21X50	Seismic	19.23	17.28
STORY3	B1437	W21X50	Seismic	13.00	10.78
STORY2	B1437	W21X50	Seismic	27.31	24.04
STORY1	B1437	W21X50	Seismic	23.96	20.43
STORY3	B1438	W21X50	Seismic	6.718E-01	6.20

Steel Beam Design - Special Seismic Requirements

Story Level	Beam Bay	Section Name	Section Class	Connection Shear End-I	Connection Shear End-J
STORY2	B1438	W21X50	Seismic	2.61	14.19
STORY1	B1438	W21X50	Seismic	1.57	11.50
STORY3	B1439	W21X50	Seismic	5.63	10.88
STORY2	B1439	W21X50	Seismic	10.20	21.06
STORY1	B1439	W21X50	Slender	10.11	20.90
STORY3	B1440	W21X50	Seismic	6.51	11.56
STORY2	B1440	W21X50	Seismic	11.80	22.52
STORY1	B1440	W21X50	Seismic	11.63	22.42
STORY3	B1441	W21X50	Seismic	3.334E-01	5.82
STORY2	B1441	W21X50	Seismic	3.00	13.70
STORY1	B1441	W21X50	Seismic	1.41	11.34
STORY3	B1442	W21X50	Seismic	5.63	3.77
STORY2	B1442	W21X50	Seismic	13.05	10.10
STORY1	B1442	W21X50	Seismic	9.57	6.63
STORY3	B1443	W21X101	Compact	6.75	3.14
STORY2	B1443	W21X101	Compact	13.20	4.06
STORY1	B1443	W21X101	Compact	9.30	5.82
STORY3	B1444	W21X101	Compact	7.93	4.39
STORY2	B1444	W21X101	Compact	13.93	3.70
STORY1	B1444	W21X101	Compact	9.87	6.39
STORY4	B1445	W14X22	Compact	8.668E-01	9.026E-01
STORY4	B1446	W14X22	Compact	8.851E-01	5.146E-01
STORY3	B1520	W21X50	Seismic	29.37	38.41
STORY2	B1520	W21X50	Seismic	51.54	65.35
STORY1	B1520	W21X62	Seismic	50.19	67.12
STORY1	B1542	W21X50	Seismic		
STORY3	B1543	W21X50	Seismic	2.36	
STORY2	B1543	W21X50	Seismic	2.11	
STORY1	B1543	W21X50	Seismic	3.28	
STORY3	B1544	W21X50	Seismic		
STORY2	B1544	W21X50	Seismic		
STORY1	B1544	W21X50	Seismic		
STORY3	B1545	W21X50	Seismic		
STORY2	B1545	W21X50	Seismic		
STORY1	B1545	W21X50	Seismic		
STORY1	B1546	W21X50	Seismic		
STORY3	B1553	W21X50	Seismic	2.76	2.74
STORY2	B1553	W21X50	Seismic	5.24	5.21
STORY1	B1553	W21X50	Seismic	5.24	5.21
STORY3	B1555	W21X50	Seismic		
STORY2	B1555	W21X50	Seismic		
STORY1	B1555	W21X50	Seismic		
STORY3	B1558	W21X50	Seismic		
STORY2	B1558	W21X50	Seismic		
STORY3	B1559	W21X50	Seismic		
STORY2	B1559	W21X50	Seismic		
STORY3	B2566	W21X50	Seismic	18.23	6.40
STORY2	B2566	W21X50	Seismic	32.21	14.21

Steel Beam Design - Special Seismic Requirements

Story Level	Beam Bay	Section Name	Section Class	Connection Shear End-I	Connection Shear End-J
STORY1	B2566	W21X50	Seismic	32.45	13.96
STORY3	B2567	W21X50	Seismic	34.38	33.38
STORY2	B2567	W21X62	Seismic	58.78	58.50
STORY1	B2567	W21X62	Seismic	59.36	57.93
STORY1	B2571	W21X50	Seismic		
STORY3	B2572	W21X50	Seismic	2.76	2.74
STORY2	B2572	W21X50	Seismic	5.24	5.21
STORY1	B2572	W21X50	Seismic	5.24	5.21
STORY3	B2573	W21X50	Seismic		
STORY2	B2573	W21X50	Seismic		
STORY1	B2573	W21X50	Seismic		
STORY3	B2574	W21X50	Seismic		
STORY2	B2574	W21X50	Seismic		
STORY3	B2575	W21X50	Seismic		
STORY2	B2575	W21X50	Seismic		
STORY1	B2576	W21X50	Seismic		
STORY3	B2578	W21X50	Seismic		
STORY2	B2578	W21X50	Seismic		
STORY1	B2578	W21X50	Slender		
STORY3	B2579	W21X50	Seismic	3.34	
STORY2	B2579	W21X50	Seismic	3.52	
STORY1	B2579	W21X50	Seismic	4.37	
STORY3	B2580	W21X50	Seismic		
STORY2	B2580	W21X50	Seismic		
STORY1	B2580	W21X50	Seismic		
STORY3	B2721	W21X50	Seismic	14.68	14.64
STORY2	B2721	W21X50	Seismic	31.17	32.00
STORY1	B2721	W21X50	Seismic	26.62	27.95
STORY3	B2756	W21X50	Seismic	28.13	2.41
STORY2	B2756	W21X50	Seismic	52.49	2.48
STORY1	B2756	W21X50	Seismic	52.41	8.28
STORY3	B3189	W21X50	Seismic	1.74	1.17
STORY2	B3189	W21X50	Seismic	3.55	1.10
STORY1	B3189	W21X50	Seismic	2.22	1.66
STORY3	B3190	W21X50	Seismic	9.55	8.01
STORY2	B3190	W21X50	Seismic	20.43	17.69
STORY1	B3190	W21X50	Seismic	17.61	14.91
STORY3	B3200	W21X50	Seismic	21.97	2.25
STORY2	B3200	W21X50	Seismic	40.68	3.38
STORY1	B3200	W21X50	Seismic	40.48	7.50
STORY3	B3201	W21X50	Seismic	37.42	35.33
STORY2	B3201	W21X68	Seismic	63.87	61.53
STORY1	B3201	W21X68	Seismic	65.06	60.34
STORY3	B3202	W21X50	Seismic	36.68	34.05
STORY2	B3202	W21X62	Seismic	63.97	58.63
STORY1	B3202	W21X62	Seismic	64.58	57.83
STORY3	B3203	W21X50	Seismic	9.19	8.88
STORY2	B3203	W21X50	Seismic	18.60	15.71

Steel Beam Design - Special Seismic Requirements

Story Level	Beam Bay	Section Name	Section Class	Connection Shear End-I	Connection Shear End-J
STORY1	B3203	W21X50	Seismic	18.73	15.57
STORY3	B3204	W21X50	Seismic	7.33	10.74
STORY2	B3204	W21X50	Seismic	13.62	20.70
STORY1	B3204	W21X50	Seismic	13.95	20.09
STORY3	B3205	W21X62	Seismic	14.86	14.57
STORY2	B3205	W21X62	Seismic	27.70	27.76
STORY1	B3205	W21X62	Seismic	27.28	27.63
STORY3	B3206	W21X50	Seismic	11.23	6.84
STORY2	B3206	W21X50	Seismic	21.40	12.91
STORY1	B3206	W21X50	Seismic	20.72	13.32
STORY3	B3207	W21X50	Seismic	6.97	11.10
STORY2	B3207	W21X50	Seismic	13.04	21.27
STORY1	B3207	W21X50	Seismic	13.43	20.61
STORY3	B3208	W21X62	Seismic	14.88	14.55
STORY2	B3208	W21X62	Seismic	27.75	27.71
STORY1	B3208	W21X62	Seismic	27.32	27.58
STORY3	B3857	W21X50	Seismic	11.75	6.32
STORY2	B3857	W21X50	Seismic	22.72	11.60
STORY1	B3857	W21X50	Seismic	22.58	11.47
STORY3	B3858	W21X50	Seismic	11.74	6.33
STORY2	B3858	W21X50	Seismic	22.70	11.61
STORY1	B3858	W21X50	Seismic	22.57	11.47
STORY3	B3859	W21X62	Seismic		6.11
STORY2	B3859	W21X62	Seismic		14.66
STORY1	B3859	W21X62	Seismic		11.00
STORY3	B3860	W21X62	Seismic	2.33	10.66
STORY2	B3860	W21X62	Seismic	1.20	21.33
STORY1	B3860	W21X62	Seismic	4.12	21.35
STORY3	B3861	W21X62	Seismic	14.40	15.03
STORY2	B3861	W21X62	Seismic	27.60	27.86
STORY1	B3861	W21X62	Seismic	27.57	27.33
STORY3	B3863	W21X50	Seismic	4.78	3.10
STORY2	B3863	W21X50	Seismic	6.76	1.77
STORY1	B3863	W21X50	Seismic	5.34	1.41
STORY3	B3864	W21X50	Seismic	3.71	7.20
STORY2	B3864	W21X50	Seismic	2.95	10.10
STORY1	B3864	W21X50	Seismic	5.577E-01	8.58
STORY3	B3865	W21X62	Seismic	9.45	7.20
STORY2	B3865	W21X62	Seismic	16.08	14.50
STORY1	B3865	W21X62	Seismic	15.89	14.69
STORY3	B3866	W21X62	Seismic	7.97	8.66
STORY2	B3866	W21X62	Seismic	15.37	15.20
STORY1	B3866	W21X62	Seismic	15.26	15.31
STORY3	B3869	W21X62	Seismic	7.97	8.66
STORY2	B3869	W21X62	Seismic	15.53	15.05
STORY1	B3869	W21X62	Seismic	15.64	14.93
STORY3	B3870	W21X50	Seismic	4.34	9.288E-01
STORY2	B3870	W21X50	Seismic	5.31	1.75

Steel Beam Design - Special Seismic Requirements

Story Level	Beam Bay	Section Name	Section Class	Connection Shear End-I	Connection Shear End-J
STORY1	B3870	W21X50	Seismic	2.46	6.74
STORY3	B3871	W21X50	Seismic	7.152E-01	4.30
STORY2	B3871	W21X50	Seismic	1.50	6.18
STORY1	B3871	W21X50	Seismic	6.55	2.64
STORY3	B3872	W21X62	Seismic	8.67	7.96
STORY2	B3872	W21X62	Seismic	15.36	15.21
STORY1	B3872	W21X62	Seismic	15.02	15.55
STORY3	B3873	W21X62	Seismic	14.29	15.13
STORY2	B3873	W21X62	Seismic	27.37	28.09
STORY1	B3873	W21X62	Seismic	27.53	27.65
STORY3	B3874	W21X50	Seismic	11.31	6.76
STORY2	B3874	W21X50	Seismic	21.25	13.06
STORY1	B3874	W21X50	Seismic	20.86	13.18
STORY3	B3875	W21X50	Seismic	6.77	11.30
STORY2	B3875	W21X50	Seismic	14.23	20.09
STORY1	B3875	W21X50	Seismic	13.30	20.73
STORY3	B3876	W21X62	Seismic	14.92	14.50
STORY2	B3876	W21X62	Seismic	21.23	21.20
STORY1	B3876	W21X62	Seismic	27.35	27.56
STORY3	B3879	W21X50	Seismic	11.77	6.29
STORY2	B3879	W21X50	Seismic	21.67	12.64
STORY1	B3879	W21X50	Seismic	22.60	11.44
STORY3	B3880	W21X50	Seismic	9.17	8.90
STORY2	B3880	W21X50	Seismic	18.58	15.73
STORY1	B3880	W21X50	Seismic	18.77	15.53
STORY3	B3881	W21X50	Seismic	7.33	10.74
STORY2	B3881	W21X50	Seismic	13.62	20.70
STORY1	B3881	W21X50	Seismic	13.94	20.10
STORY3	B3882	W21X62	Seismic	14.85	14.57
STORY2	B3882	W21X62	Seismic	27.69	27.77
STORY1	B3882	W21X62	Seismic	27.28	27.63
STORY3	B3883	W21X50	Seismic	11.74	6.33
STORY2	B3883	W21X50	Seismic	22.71	11.61
STORY1	B3883	W21X50	Seismic	22.57	11.47
STORY3	B3884	W21X62	Seismic	3.57	10.76
STORY2	B3884	W21X62	Seismic	4.32	21.44
STORY1	B3884	W21X62	Seismic	2.83	21.53
STORY3	B3885	W21X62	Seismic		6.32
STORY2	B3885	W21X62	Seismic		14.64
STORY1	B3885	W21X62	Seismic		11.01
STORY3	B3942	W18X35	Seismic	7.57	4.95
STORY2	B3942	W18X35	Seismic	13.64	10.12
STORY1	B3942	W18X35	Seismic	13.78	9.97
STORY3	B3943	W18X55	Seismic	7.99	4.93
STORY2	B3943	W18X55	Seismic	14.05	10.11
STORY1	B3943	W18X55	Seismic	14.26	9.90
STORY3	B3944	W18X55	Seismic	5.11	7.81
STORY2	B3944	W18X55	Seismic	10.30	13.86

Steel Beam Design - Special Seismic Requirements

Story Level	Beam Bay	Section Name	Section Class	Connection Shear End-I	Connection Shear End-J
STORY1	B3944	W18X55	Seismic	9.53	14.63
STORY3	B3945	W18X35	Seismic	5.15	7.36
STORY2	B3945	W18X35	Seismic	10.37	13.39
STORY1	B3945	W18X35	Seismic	10.14	13.62
STORY3	B3948	W21X50	Seismic	9.67	9.73
STORY2	B3948	W21X50	Seismic	18.39	18.53
STORY1	B3948	W21X50	Seismic	18.36	18.54

Steel Brace Design - Capacity Check Output

Steel Brace Design - Capacity Check Output

Story Level	Brace Bay	Section Name	Moment Interaction Check Ratio = AXL + B33 + B22	Shear22 Ratio	Shear33 Ratio
STORY3	D47	W16X50	0.007 = 0.004 + 0.003 + 0.000	0.002	0.000
STORY2	D47	W16X50	0.010 = 0.010 + 0.000 + 0.000	0.002	0.000
STORY1	D47	W16X50	0.080 = 0.080 + 0.000 + 0.000	0.002	0.000
STORY3	D49	W16X89	0.007 = 0.007 + 0.000 + 0.000	0.001	0.000
STORY2	D49	W16X89	0.022 = 0.022 + 0.000 + 0.000	0.001	0.000
STORY1	D49	W16X89	0.045 = 0.045 + 0.000 + 0.000	0.001	0.000
STORY3	D50	W16X89	0.004 = 0.004 + 0.000 + 0.000	0.001	0.000
STORY2	D50	W16X89	0.011 = 0.011 + 0.000 + 0.000	0.001	0.000
STORY1	D50	W16X89	0.030 = 0.030 + 0.000 + 0.000	0.001	0.000
STORY3	D53	W16X50	0.012 = 0.009 + 0.003 + 0.000	0.002	0.000
STORY2	D53	W16X50	0.011 = 0.008 + 0.003 + 0.000	0.002	0.000
STORY1	D53	W16X50	0.011 = 0.008 + 0.003 + 0.000	0.002	0.000
STORY3	D82	W16X50	0.004 = 0.002 + 0.003 + 0.000	0.002	0.000
STORY2	D82	W16X50	0.014 = 0.014 + 0.000 + 0.000	0.002	0.000
STORY1	D82	W16X50	0.079 = 0.079 + 0.000 + 0.000	0.002	0.000
STORY3	D83	W16X89	0.008 = 0.008 + 0.000 + 0.000	0.001	0.000
STORY2	D83	W16X89	0.020 = 0.020 + 0.000 + 0.000	0.001	0.000
STORY1	D83	W16X89	0.041 = 0.041 + 0.000 + 0.000	0.001	0.000
STORY3	D84	W16X89	0.008 = 0.008 + 0.000 + 0.000	0.001	0.000
STORY2	D84	W16X89	0.016 = 0.016 + 0.000 + 0.000	0.001	0.000
STORY1	D84	W16X89	0.032 = 0.032 + 0.000 + 0.000	0.001	0.000
STORY3	D85	W16X50	0.012 = 0.009 + 0.003 + 0.000	0.002	0.000
STORY2	D85	W16X50	0.012 = 0.009 + 0.003 + 0.000	0.002	0.000
STORY1	D85	W16X50	0.013 = 0.010 + 0.003 + 0.000	0.002	0.000
STORY4	D111	W16X50	0.033 = 0.033 + 0.000 + 0.000	0.002	0.000
STORY3	D111	W16X50	0.059 = 0.059 + 0.000 + 0.000	0.002	0.000
STORY2	D111	W16X50	0.082 = 0.082 + 0.000 + 0.000	0.003	0.000
STORY1	D111	W16X50	0.117 = 0.117 + 0.000 + 0.000	0.003	0.000
STORY4	D112	W16X50	0.058 = 0.058 + 0.000 + 0.000	0.002	0.000
STORY3	D112	W16X50	0.053 = 0.053 + 0.000 + 0.000	0.003	0.000
STORY2	D112	W16X50	0.095 = 0.095 + 0.000 + 0.000	0.004	0.000
STORY1	D112	W16X50	0.116 = 0.116 + 0.000 + 0.000	0.005	0.000
STORY4	D113	W16X50	0.042 = 0.042 + 0.000 + 0.000	0.002	0.000
STORY3	D113	W16X50	0.048 = 0.048 + 0.000 + 0.000	0.002	0.000
STORY2	D113	W16X50	0.079 = 0.079 + 0.000 + 0.000	0.003	0.000

Steel Brace Design - Capacity Check Output

Story Level	Brace Bay	Section Name	Moment Interaction Check Ratio = AXL + B33 + B22	Shear22 Ratio	Shear33 Ratio
STORY1	D113	W16X50	0.099 = 0.099 + 0.000 + 0.000	0.004	0.000
STORY4	D114	W16X50	0.035 = 0.035 + 0.000 + 0.000	0.002	0.000
STORY3	D114	W16X50	0.034 = 0.034 + 0.000 + 0.000	0.002	0.000
STORY2	D114	W16X50	0.042 = 0.042 + 0.000 + 0.000	0.003	0.000
STORY1	D114	W16X50	0.092 = 0.092 + 0.000 + 0.000	0.003	0.000
STORY3	D117	W16X50	0.025 = 0.015 + 0.010 + 0.000	0.003	0.000
STORY2	D117	W16X89	0.340 = 0.340 + 0.000 + 0.000	0.005	0.000
STORY1	D117	W16X89	0.835 = 0.835 + 0.000 + 0.000	0.005	0.000
STORY3	D118	W16X89	1.125 = 1.125 + 0.000 + 0.000	0.005	0.000
STORY2	D118	W16X89	1.264 = 1.264 + 0.000 + 0.000	0.005	0.000
STORY1	D118	W16X89	0.217 = 0.217 + 0.000 + 0.000	0.005	0.000
STORY3	D119	W16X89	0.963 = 0.963 + 0.000 + 0.000	0.005	0.000
STORY2	D119	W16X89	1.179 = 1.179 + 0.000 + 0.000	0.005	0.000
STORY1	D119	W16X89	0.213 = 0.213 + 0.000 + 0.000	0.005	0.000
STORY3	D120	W16X89	0.015 = 0.011 + 0.004 + 0.000	0.003	0.000
STORY2	D120	W16X89	0.431 = 0.431 + 0.000 + 0.000	0.005	0.000
STORY1	D120	W16X89	0.845 = 0.845 + 0.000 + 0.000	0.005	0.000
STORY3	D121	W16X89	0.017 = 0.012 + 0.005 + 0.000	0.003	0.000
STORY2	D121	W16X89	0.521 = 0.521 + 0.000 + 0.000	0.005	0.000
STORY1	D121	W16X89	0.983 = 0.983 + 0.000 + 0.000	0.005	0.000
STORY3	D122	W16X89	1.053 = 1.053 + 0.000 + 0.000	0.005	0.000
STORY2	D122	W16X89	Pu > Pe (B1 is undefined)		
STORY1	D122	W16X89	0.222 = 0.222 + 0.000 + 0.000	0.005	0.000
STORY3	D123	W16X89	1.073 = 1.073 + 0.000 + 0.000	0.005	0.000
STORY2	D123	W16X89	Pu > Pe (B1 is undefined)		
STORY1	D123	W16X89	0.229 = 0.229 + 0.000 + 0.000	0.005	0.000
STORY3	D124	W16X89	0.018 = 0.013 + 0.005 + 0.000	0.003	0.000
STORY2	D124	W16X89	0.522 = 0.522 + 0.000 + 0.000	0.005	0.000
STORY1	D124	W16X89	0.971 = 0.971 + 0.000 + 0.000	0.005	0.000
STORY3	D125	W16X89	0.022 = 0.022 + 0.000 + 0.000	0.001	0.000
STORY2	D125	W16X89	0.031 = 0.031 + 0.000 + 0.000	0.001	0.000
STORY1	D125	W16X89	0.050 = 0.050 + 0.000 + 0.000	0.001	0.000
STORY3	D126	W16X89	0.005 = 0.004 + 0.001 + 0.000	0.001	0.000
STORY2	D126	W16X89	0.009 = 0.009 + 0.000 + 0.000	0.001	0.000
STORY1	D126	W16X89	0.030 = 0.030 + 0.000 + 0.000	0.001	0.000
STORY3	D127	W16X89	0.004 = 0.003 + 0.001 + 0.000	0.001	0.000
STORY2	D127	W16X89	0.020 = 0.020 + 0.000 + 0.000	0.001	0.000
STORY1	D127	W16X89	0.051 = 0.051 + 0.000 + 0.000	0.001	0.000
STORY3	D128	W16X89	0.017 = 0.017 + 0.000 + 0.000	0.001	0.000
STORY2	D128	W16X89	0.021 = 0.021 + 0.000 + 0.000	0.001	0.000
STORY1	D128	W16X89	0.030 = 0.030 + 0.000 + 0.000	0.001	0.000
STORY3	D129	W16X89	0.014 = 0.013 + 0.001 + 0.000	0.001	0.000
STORY2	D129	W16X89	0.008 = 0.007 + 0.001 + 0.000	0.001	0.000
STORY1	D129	W16X89	0.046 = 0.046 + 0.000 + 0.000	0.001	0.000
STORY3	D130	W16X89	0.045 = 0.045 + 0.000 + 0.000	0.001	0.000
STORY2	D130	W16X89	0.034 = 0.034 + 0.000 + 0.000	0.001	0.000
STORY1	D130	W16X89	0.025 = 0.025 + 0.000 + 0.000	0.001	0.000
STORY3	D131	W16X89	0.006 = 0.006 + 0.000 + 0.000	0.001	0.000

Steel Brace Design - Capacity Check Output

Story Level	Brace Bay	Section Name	Moment Interaction Check Ratio = AXL + B33 + B22	Shear22 Ratio	Shear33 Ratio
STORY2	D131	W16X89	0.019 = 0.019 + 0.000 + 0.000	0.001	0.000
STORY1	D131	W16X89	0.042 = 0.042 + 0.000 + 0.000	0.001	0.000
STORY3	D132	W16X89	0.003 = 0.002 + 0.001 + 0.000	0.001	0.000
STORY2	D132	W16X89	0.009 = 0.009 + 0.000 + 0.000	0.001	0.000
STORY1	D132	W16X89	0.023 = 0.023 + 0.000 + 0.000	0.001	0.000

Steel Brace Design - Special Seismic Requirements

Steel Brace Design - Special Seismic Requirements

Story Level	Brace Bay	Section Name	Section Class	Connection Force End-I	Connection Force End-J
STORY3	D47	W16X50	Seismic	6.05	6.36
STORY2	D47	W16X50	Seismic	5.67	5.98
STORY1	D47	W16X50	Seismic	-24.27	-23.99
STORY3	D49	W16X89	Seismic	-13.73	-13.42
STORY2	D49	W16X89	Seismic	-23.53	-23.23
STORY1	D49	W16X89	Seismic	-40.27	-39.97
STORY3	D50	W16X89	Seismic	-5.77	-5.47
STORY2	D50	W16X89	Seismic	-17.74	-17.44
STORY1	D50	W16X89	Seismic	-29.87	-29.56
STORY3	D53	W16X50	Seismic	14.25	14.55
STORY2	D53	W16X50	Seismic	10.79	11.07
STORY1	D53	W16X50	Seismic	10.65	10.93
STORY3	D82	W16X50	Seismic	2.87	3.17
STORY2	D82	W16X50	Seismic	-5.34	-5.04
STORY1	D82	W16X50	Seismic	-24.15	-23.87
STORY3	D83	W16X89	Seismic	-14.09	-13.79
STORY2	D83	W16X89	Seismic	-22.05	-21.74
STORY1	D83	W16X89	Seismic	-36.88	-36.58
STORY3	D84	W16X89	Seismic	-9.25	-8.95
STORY2	D84	W16X89	Seismic	-21.81	-21.50
STORY1	D84	W16X89	Seismic	-31.29	-30.99
STORY3	D85	W16X50	Seismic	12.80	13.10
STORY2	D85	W16X50	Seismic	11.99	12.27
STORY1	D85	W16X50	Seismic	13.67	13.95
STORY4	D111	W16X50	Seismic	-9.15	-8.24
STORY3	D111	W16X50	Seismic	-35.31	-42.10
STORY2	D111	W16X50	Seismic	-36.44	-53.50
STORY1	D111	W16X50	Seismic	-91.37	-56.91
STORY4	D112	W16X50	Seismic	-17.77	-16.45
STORY3	D112	W16X50	Seismic	-44.13	13.81
STORY2	D112	W16X50	Seismic	-70.80	-8.79
STORY1	D112	W16X50	Seismic	-93.36	-8.25
STORY4	D113	W16X50	Seismic	-15.05	-13.82
STORY3	D113	W16X50	Seismic	-42.50	-17.89
STORY2	D113	W16X50	Seismic	-63.79	-14.03
STORY1	D113	W16X50	Seismic	-85.74	-10.76
STORY4	D114	W16X50	Seismic	-9.72	-8.93

Steel Brace Design - Special Seismic Requirements

Story Level	Brace Bay	Section Name	Section Class	Connection Force End-I	Connection Force End-J
STORY3	D114	W16X50	Seismic	-29.55	-27.36
STORY2	D114	W16X50	Seismic	-31.33	-35.50
STORY1	D114	W16X50	Seismic	-80.74	-37.36
STORY3	D117	W16X50	Seismic	22.87	27.22
STORY2	D117	W16X89	Seismic	-51.74	55.41
STORY1	D117	W16X89	Seismic	-87.91	-82.10
STORY3	D118	W16X89	Seismic	-91.26	-86.60
STORY2	D118	W16X89	Seismic	-108.05	-101.77
STORY1	D118	W16X89	Seismic	-73.48	-68.10
STORY3	D119	W16X89	Seismic	-80.85	-75.26
STORY2	D119	W16X89	Seismic	-102.97	-96.62
STORY1	D119	W16X89	Seismic	-73.23	-67.88
STORY3	D120	W16X89	Seismic	31.36	36.86
STORY2	D120	W16X89	Seismic	-56.72	-50.20
STORY1	D120	W16X89	Seismic	-87.77	-82.02
STORY3	D121	W16X89	Seismic	37.25	42.73
STORY2	D121	W16X89	Seismic	-71.33	-64.83
STORY1	D121	W16X89	Seismic	-110.71	-104.83
STORY3	D122	W16X89	Seismic	-92.55	-86.94
STORY2	D122	W16X89	Seismic	-120.66	-114.31
STORY1	D122	W16X89	Seismic	-84.63	-79.27
STORY3	D123	W16X89	Seismic	-93.71	-88.14
STORY2	D123	W16X89	Seismic	-120.34	-113.95
STORY1	D123	W16X89	Seismic	-86.31	-80.98
STORY3	D124	W16X89	Seismic	38.33	43.76
STORY2	D124	W16X89	Seismic	-71.35	-64.84
STORY1	D124	W16X89	Seismic	-110.04	-104.17
STORY3	D125	W16X89	Seismic	-17.36	-16.76
STORY2	D125	W16X89	Seismic	-24.52	-23.95
STORY1	D125	W16X89	Seismic	-39.45	-38.89
STORY3	D126	W16X89	Seismic	11.10	11.71
STORY2	D126	W16X89	Seismic	9.58	10.14
STORY1	D126	W16X89	Seismic	-23.53	-22.97
STORY3	D127	W16X89	Seismic	5.72	6.33
STORY2	D127	W16X89	Seismic	-15.56	-15.00
STORY1	D127	W16X89	Seismic	-39.91	-39.35
STORY3	D128	W16X89	Seismic	-13.60	-13.00
STORY2	D128	W16X89	Seismic	-16.30	-15.74
STORY1	D128	W16X89	Seismic	-23.46	-22.90
STORY3	D129	W16X89	Seismic	30.75	31.31
STORY2	D129	W16X89	Seismic	15.56	16.13
STORY1	D129	W16X89	Seismic	-35.73	-35.17
STORY3	D130	W16X89	Seismic	-34.98	-34.42
STORY2	D130	W16X89	Seismic	-27.00	-26.44
STORY1	D130	W16X89	Seismic	20.39	20.81
STORY3	D131	W16X89	Seismic	-6.06	-5.45
STORY2	D131	W16X89	Seismic	-15.02	-14.46
STORY1	D131	W16X89	Seismic	-33.09	-32.53

Steel Brace Design - Special Seismic Requirements

Story Level	Brace Bay	Section Name	Section Class	Connection Force End-I	Connection Force End-J
STORY3	D132	W16X89	Seismic	6.86	7.47
STORY2	D132	W16X89	Seismic	8.00	8.56
STORY1	D132	W16X89	Seismic	22.50	23.07

Appendix F

MQP : Mixed use Parking garage / steel structure

Job # : Lobby Truss Analysis & Design

loads :

SD : 10 psf ← assumed

Dead : 10 psf ← glazing

live : 0.0 psf ←

snow : 55 psf ← worcester area un-factored

Wind Pressure : Worcester Area,

100 Mph :

Wind Ward : 18 psf

Leeward : -11 psf

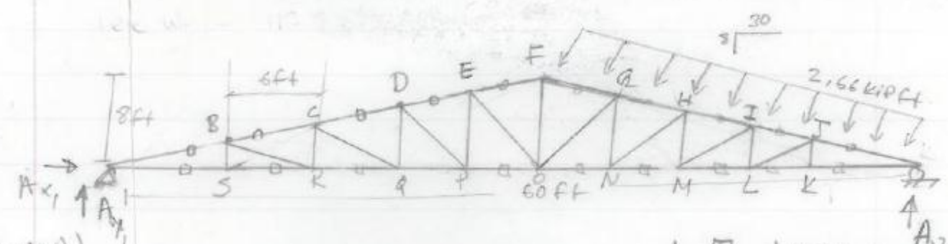
$U = 1.2 (D) + 1.0(W) + 0.5 (S) = 1.2 \times 20 + 1 \times 18 + 0.5 \times 55 = 70\ Psf$

$U = 1.2 D + 1.6(S) + 0.5 (W) = 1.2 \times 20 + 1.6 \times 55 + 0.5 \times 18 = 121\ Psf$

Maximum Ar of each truss : 22 ft × 60 ft span

linear ft load =

121 psf × 22 ft = 2.66 kip.ft



(wind ward)

2.66 kip ft × 31.05 ft = 82.6 kips

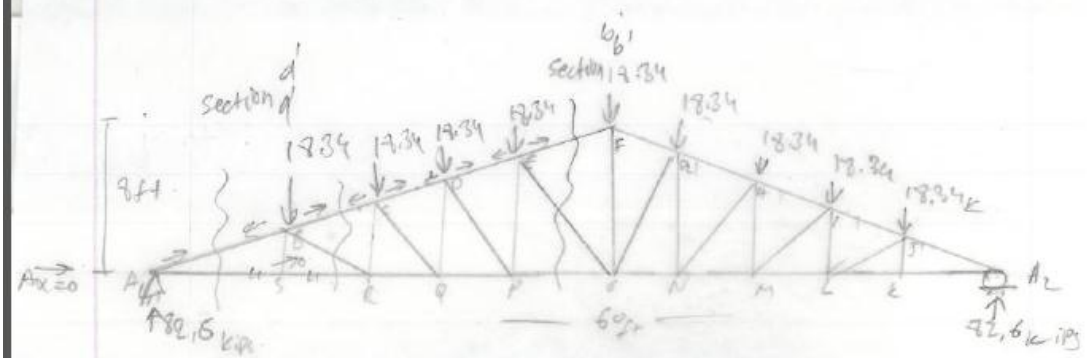
$A_{1y} = A_{2y} = 82.6$ kips , $A_{1x} = 0$

□ tension members

o compression members

Total w = 165 kips / # joints = 18.3 kips/joint

Redrawing the truss :



analyze the truss by method of sections:

Section a':

FA,B

82.6

FA,B sin a = [82.6 k]

FAB cos θ_1 = -FSA

θ_1 = 14.9°

FAB = [320 kips] Compression.

FSA = [-310 kips] tension

18.34

FBC

320 kip

FBR

$\Sigma F_y = 0$

$\Sigma F_x = 0$

$-FBC \times \cos(14.93) + 320 \times \cos(14.93) - FBR \cos(14.93) = 0$

$-18.34 - 320 \times \sin(14.93) - FBR \sin(14.93) + FBC \sin(15°) = 0$

$+FBC \cos(14.93°) + FBR \cos(14.93°) = +310$

$FBC \sin(14.93°) - FBR \sin(14.93°) = 100.94$

$2 FBC \sin(14.93)(\cos 14.93) = [177.4]$

[FBC = 356.3 kips] Compression

[FBR = -35.47] tension

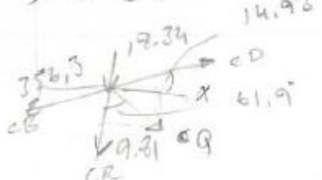
[FRC = +9.14] compression

FRQ = FRS + FRB cos 14.93°

310 + 34.27 = [344.3 kip] tension.

$\Sigma F_x = 0$

Joint C :-



$F_{CD} \times \cos(14.93) - 356.3 \times \cos(14.93) + F_{CQ} \sin 61.9° = 0.$

$-18.34 - 9.81 - F_{CD} \sin 14.93 - F_{CQ} \cos 61.9° + 356.3 \times \sin 14.93 =$

$F_{CD} \times \cos(14.93) + F_{CQ} \sin(61.9°) = 344.3k$

$-F_{CD} \sin(14.93) + F_{CQ} \cos(61.9°) = 63.65$

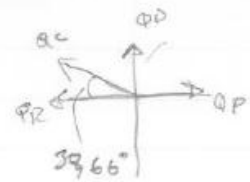
$F_{CD} \cos(14.93) \cos(61.9°) + F_{CD} \sin(14.93) \sin(61.9°) = 344.3 \times \cos(61.9) + 63.65 \times \sin(61.9°)$

$= 0.6824 F_{CD} = 218.3$

$F_{CD} = \boxed{320\ k}$ compression.

$F_{CQ} = \boxed{-39.9\ kips}$ tention

Joint Q :



$F_{RQ} = -344.3$ kips (T)

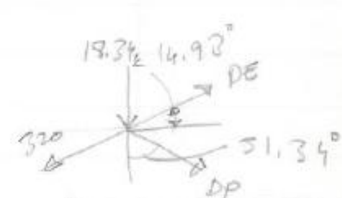
$F_{CQ} = -39.9$ kips (T)

$F_{QD} = \boxed{24.93}$ Compression.

$F_{QP} = QR + QC \times \cos(38.66°)$

$F_{QP} = \boxed{-375.5\ kips}$ tension

Joint D :-



$F_{CD} = 320$ kip Com

$F_{DQ} = -24.93$ Tension

$\boxed{F_{DP} = 24.51\ kips}$ compression.

$F_x = 0$:-

$F_{DP} \times \sin(51.34) + F_{DE} \cos(14.93°) - 320 \times \cos(14.93) = 0$

$= 0) -18.34 - F_{DP} \cos(51.34) - 320(\sin 14.93) + F_{DE} \sin(14.93) = 0.$

$F_{DP} \sin(51.34) + F_{DE} \cos(14.93) = 309.2$

$-F_{DP} \cos(51.34) + F_{DE} \sin(14.93) = 100.78$

$\Rightarrow \boxed{340\ kips = F_{DE}}$ compression.

(top cord)

Compression members

$F_{AB} = 320k$

$F_{BC} = 356k$

$F_{CD} = 390k$

$F_{DE} = 340k$

$F_{EF} = 365k$

Bottom cord:

tension members:-

$F_{AS} = -310$ kips

$F_{SR} = -310$ kips.

$F_{RQ} = -344$ kips.

$F_{QP} = -375.5$

$F_{PO} = -415$ kip

tension and com. webs:-

web max:

tension

-85 kips

compression:-

65 kips.

tension cord:

$P_{n,s} \frac{415}{0.9} = 461$ kips, W 12 x 45 tension.

yield @ 590 kips

Rapture @ 479 kips.

W 8 x 10 web

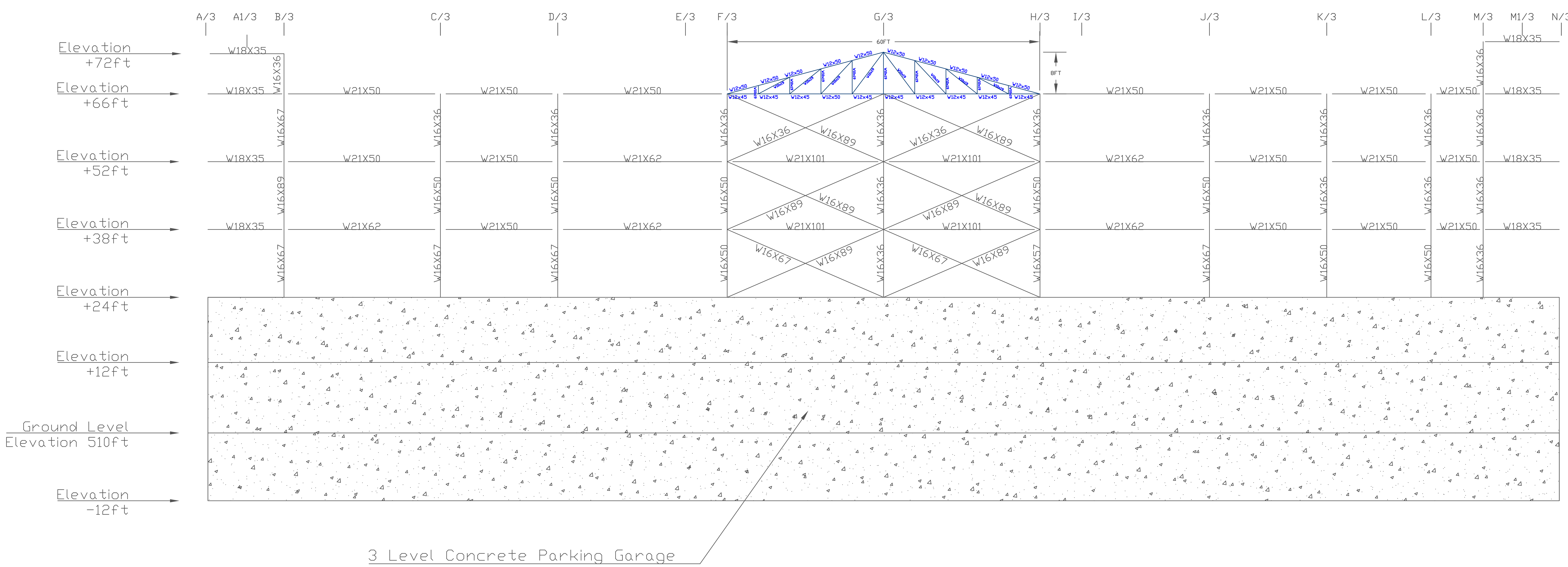
Yield @ 133 kips

Rapture @ 108 kips.

compression cord:-

W 12 x 45 provides enough capacity for [illegible]

365 kips required



General Notes

This drawing presents the steel columns, bracing and the floor elevation data referenced to ground level at the cross section 3 from east to west direction.

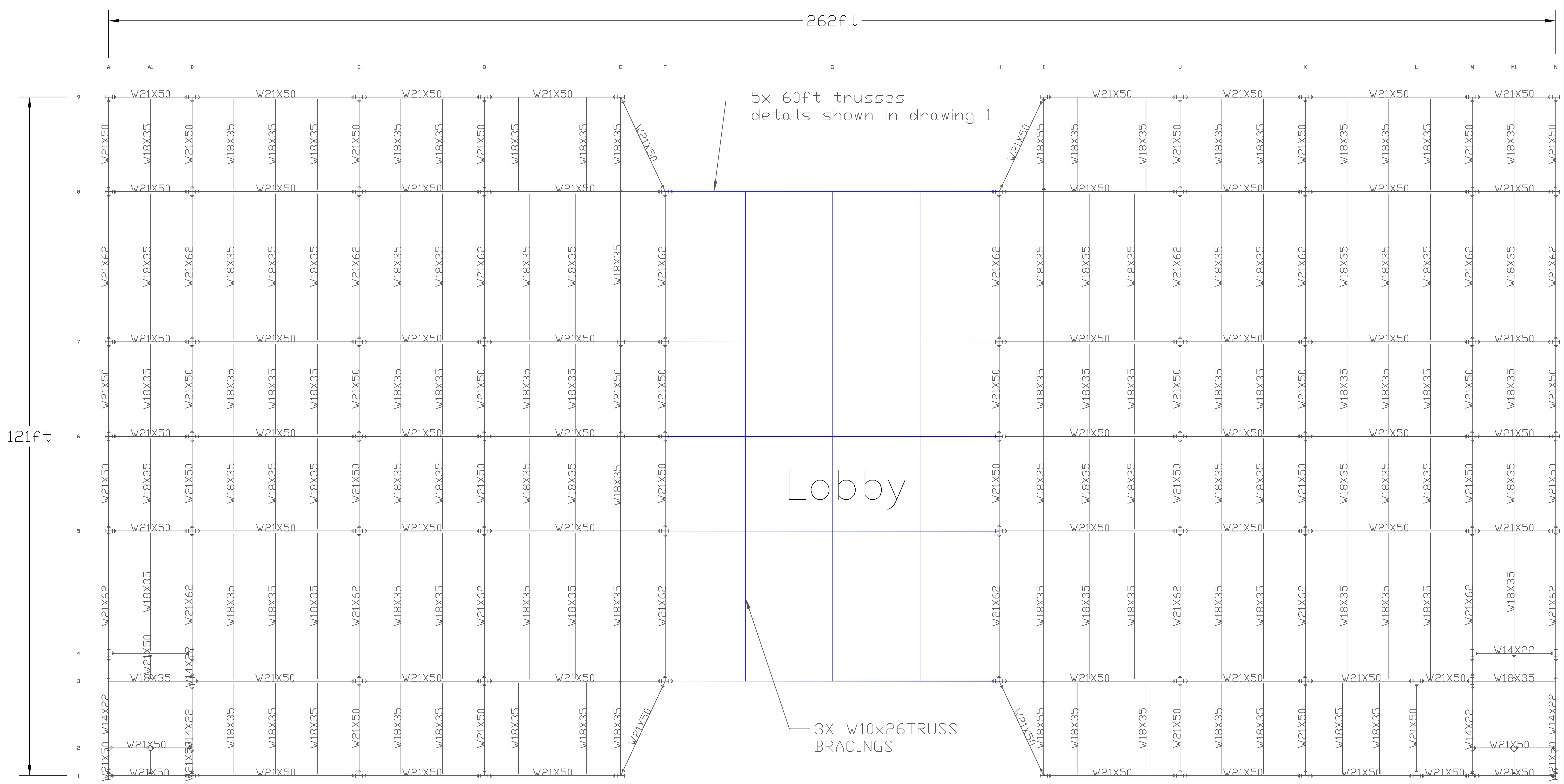
No.	Revision/Issue	Date

Firm Name and Address

Project Name and Address

Steel Structure Elevation View
Worcester Polytechnic Institute
School Of business Building
Boyonton St.

Project: East View
Date: April 26th 2012
Scale: As Noted
Sheet: 05



General Notes

This Drawing presents the Plan view for the level 3 of the steel building at elevation 66ft above ground. the lobby truss is shown at the center of the building. more details on the truss system is presented in drawing 1.

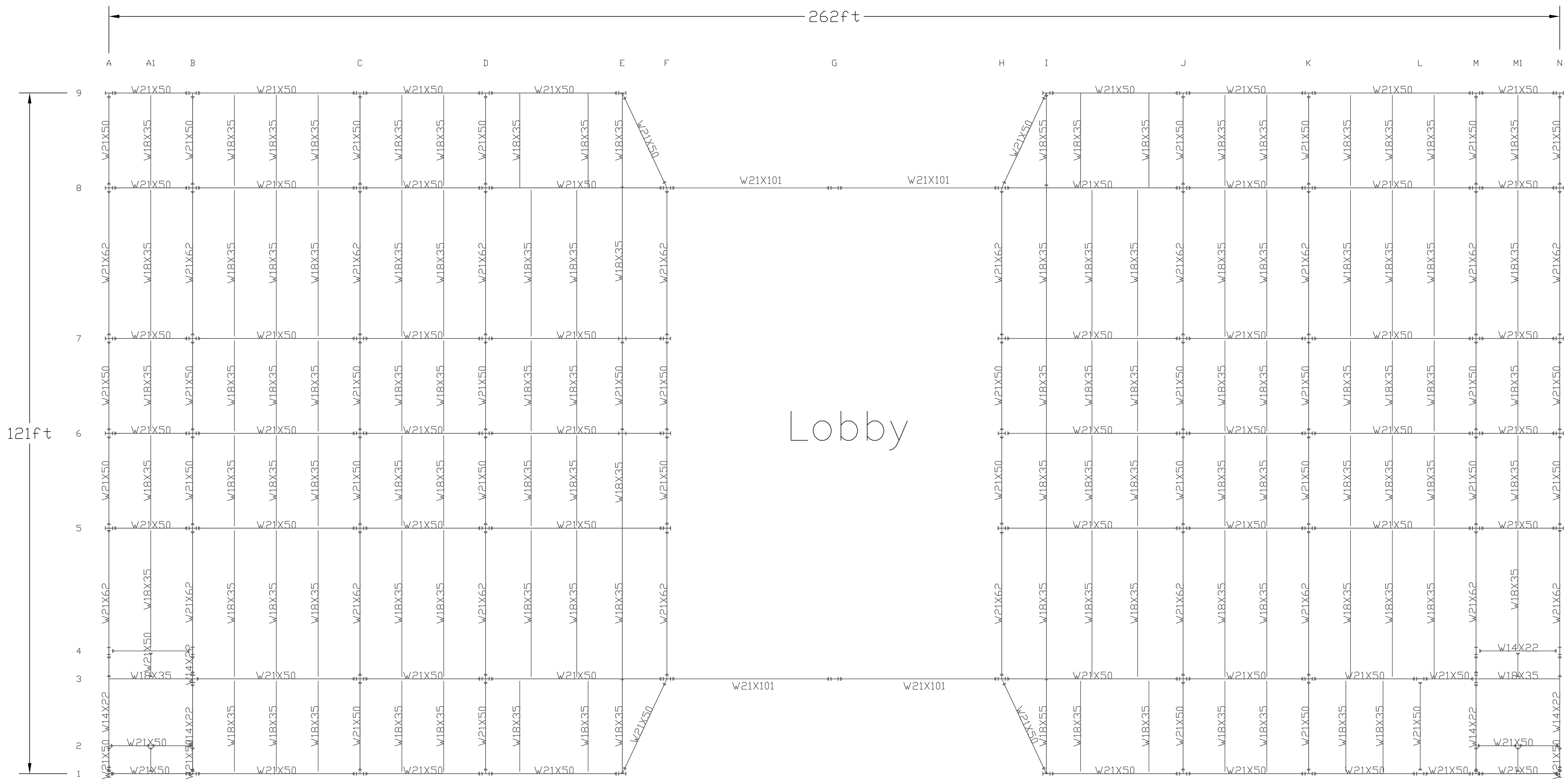
Load Cases on the frame:
Snow
Roof Live
Self Weight
Wind
Earth Quake

No.	Revision/Issue	Date

Firm Name and Address

Project Name and Address
3rd Floor Frame Plan Viewe
Worcester Polytechnic Institute
School Of business Building
Boyonton St.

Project: 3rd Floor Plan View
Date: April 26th 2012
Scale: As Noted
Sheet: 02



General Notes

This Drawing presents the Plan view for the level 2 of the steel building at elevation 52ft above ground. the lobby truss is shown at the center of the building. more details on the truss system is presented in drawing 1.

Load Cases on the frame:
Live, Dead
Self Weight
Wind, EQ

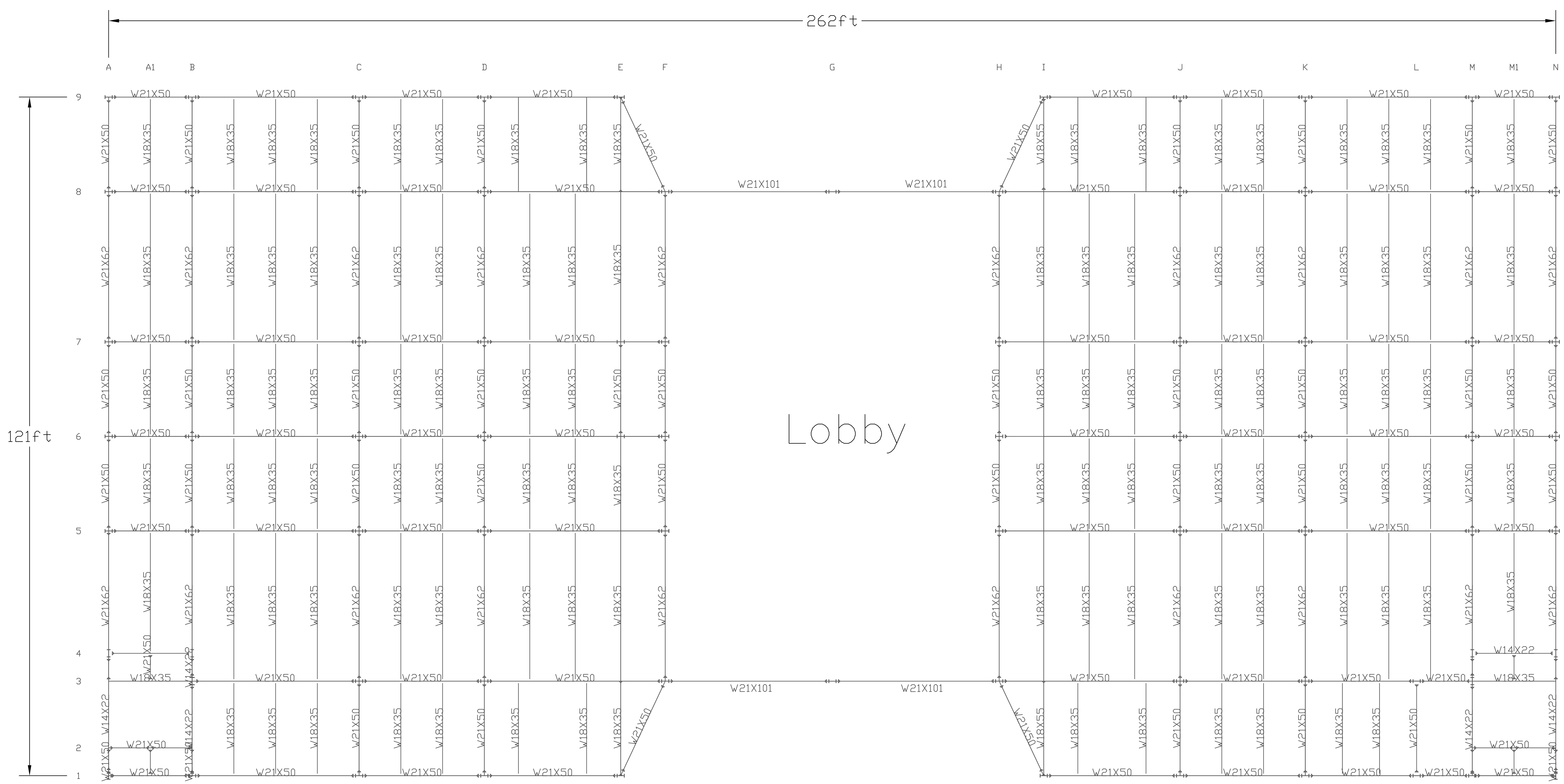
No.	Revision/Issue	Date

Firm Name and Address

Project Name and Address

2rd Floor Frame Plan Viewe
Worcester Polytechnic Institute
School Of business Building
Boyonton St.

Project: 2rd Floor Plan View
Date: April 26th 2012
Scale: As Noted
Sheet: 03



General Notes

This Drawing presents the Plan view for the level 1 of the steel building at elevation 38ft above ground. the lobby truss is shown at the center of the building. more details on the truss system is presented in drawing 1.

Load Cases on the frame:
Live, Dead
Self Weight
Wind, EQ

No.	Revision/Issue	Date

Firm Name and Address

Project Name and Address

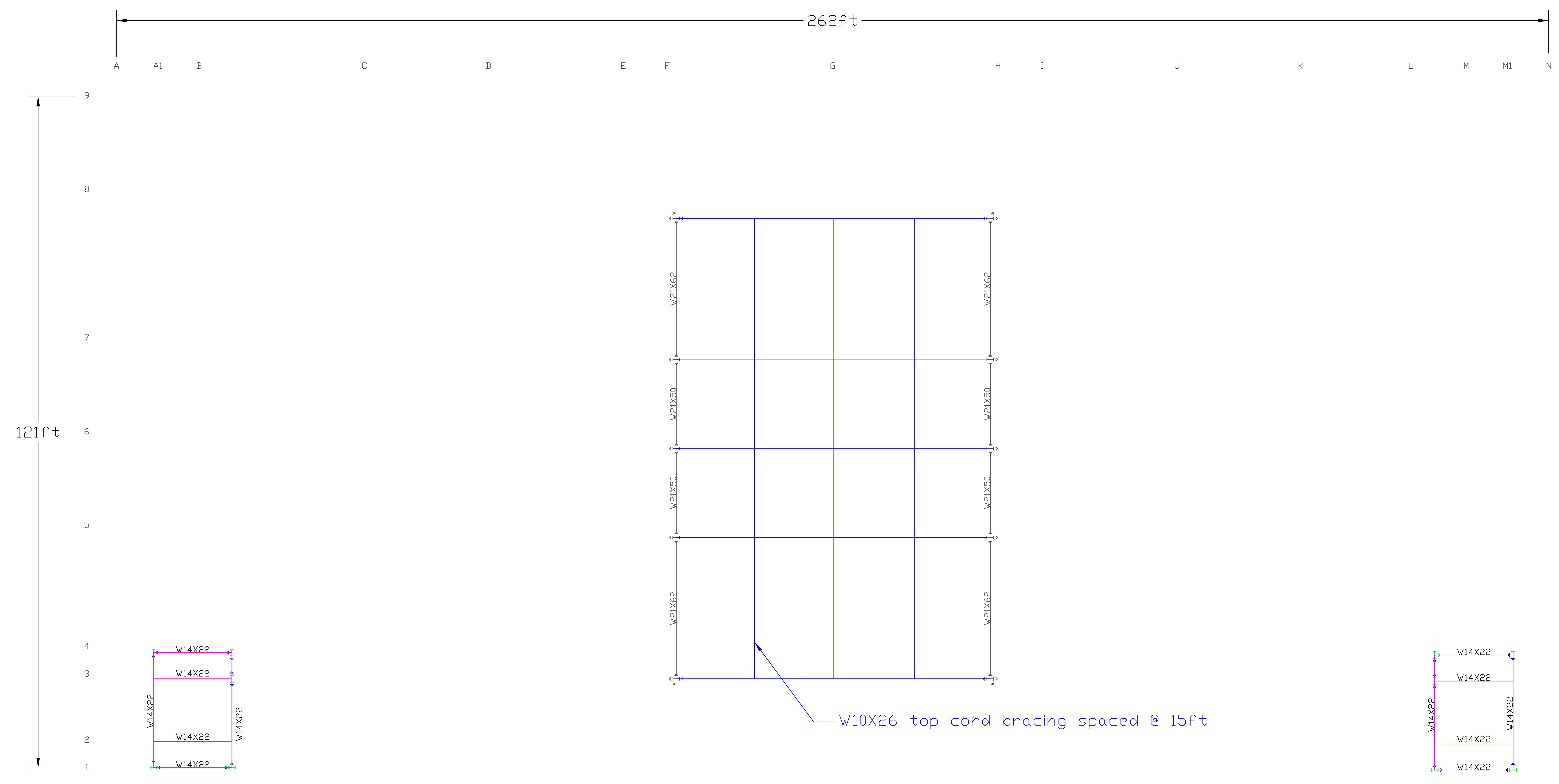
1st Floor Frame Plan Viewe
Worcester Polytechnic Institute
School Of business Building
Boyonton St.

Project: 1st Floor Plan View

Date: April 26th 2012

Scale: As Noted

Sheet: 04



General Notes

This Drawing presents the Plan view for the level Lobby and Stairs of the steel building at elevation 72ft above ground. the lobby truss is shown at the center of the building. more details on the truss system is presented in drawing 1.

Load Cases on the frame:
Snow
Roof Live
Self Weight
Wind
Earth Quake

No.	Revision/Issue	Date

Firm Name and Address

Project Name and Address

Plan View
Worcester Polytechnic Institute
School Of business Building
Boyonton St.

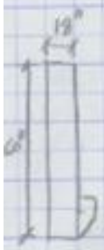
Project: 3rd Floor Plan View
Date: April 26th 2012
Scale: As Noted
Sheet: 05

Appendix G

East Side Girder Design

Girder design

$d = 58''$ $h = 60''$
$b = 18$
$L = 30\text{ ft}$
6 corbels
40 kips each



Beam self wt $= 145\text{ pcf}\left(\frac{60}{12}\right)\left(\frac{18''}{12}\right) = 1.088\text{ kip/ft}$

Corbel self wt $= \dfrac{145\text{ pcf}\left[(8in)(8in)(8in) + \left(\frac{1}{2}\right)(8in)(8in)(8in)\right]}{30\text{ ft}\ (1728\text{ in}^3)} = 2.15\text{ lbs/ft}$

6 corbels $= 12.9\text{ lb/ft} = .0129\text{ kip/ft}$

$P_{corbels} = \dfrac{6\ (40\text{ kips})}{30\text{ ft}} = 8\text{ kip/ft}$

$W_T = 1.2(1.088\text{ kip/ft} + .0129\text{ kip/ft}) + 8\text{ kip/ft} = 9.32\text{ kip/ft}$

$M_n = \frac{1}{8}wL^2 = \frac{1}{8}(9.32\text{ kip/ft})(30\text{ ft})^2 = 1048.5\text{ kip-ft}$

$M_n = bd^2R$

$R = \dfrac{M_n}{bd^2} = \dfrac{1048.5\text{ kip-ft}}{(18in)(58in)^2}\left(\dfrac{12in}{1ft}\right) * 1000 =$

$R = 207.79\text{ psi}$

for 4ksi concrete

$\rho = 0.0055$ — Design of Concrete Structures 13th edition Nilson, Darwin Table A.5a pg 738

$A_s = \rho bd = .0055(18in)(58in) = 5.742\text{ in}^2$

Use 6 #9 Bars

Stirrup Design

$V_u = \dfrac{9.32\text{ kip/ft}(30\text{ ft})}{2} = 139.8\text{ kip}$

$\phi V_n = \phi 2\sqrt{f'_c}\,b_w d + \dfrac{\phi A_v f_y d}{s}$ — Use $S = 12\text{ in}$

$A_v = \dfrac{[137.8\text{ kip} - \phi 2\sqrt{f'_c}\,b_w d]s}{\phi f_y d} = \dfrac{[139.8\text{ kip} - (.75)(2)\sqrt{4\text{ ksi}}(18)(58)]12}{.75(60)(58in)}$

$A_v = .187\text{ in}^2$ — **Use #4 @ 12in o.c.**

$\phi V_c = 99.04\text{ kip}$ — 10 ft on each end

Girder torsion design Page # 2

$V_{u1} = 139.8$ kips $\quad e = 1.083$ ft $= 13$ in $\quad W_{corbel} = 1.2(.0129\ \frac{kip}{ft}) + \frac{6(40\ kip)}{30\ ft}$

$= 8.0155$ k/ft

$T_{u1} = W_{corbel}(e)(30ft)\frac{1}{2} = 8.0155\ \frac{kip}{ft}(1.083ft)(30ft)\frac{1}{2}$

design torsional moment $\quad T_{u1} = 130.$ kip-ft

Critical design Section

$$V_{u2} = V_{u1}\left(\frac{\frac{L}{2} - d}{L/2}\right) = 139.8\ kip\left(\frac{\frac{30}{2} - \frac{58in}{12in}}{\frac{30}{2}}\right) = 94.75\ kips$$

$$T_{u2} = T_{u1}\left(\frac{\frac{L}{2} - d}{L/2}\right) = 88.11\ kip\text{-}ft$$

Cross Sectional Area check

$b_w d = 18in\,(58in) = 1044 in^2$

$X_0 = b_w - 2(cover) = 18 - 2(2in) = 14in$

$Y_0 = h - 2(cover) = 58 - 2(2in) = 56in$

$A_{oh} = X_0 Y_0 = 784 in^2$

$A_o = \phi A_{oh} = .85(784) = 666.4 in^2$

$P_h = 2(X_0 + Y_0) = 2(14 + 56) = 140 in$

$$\sqrt{\left(\frac{V_{u2}}{b_w d}\right)^2 + \left(\frac{T_{u2}(b_w)(P_h)}{1.7(A_{oh}^2)}\right)^2} \le \frac{\phi}{1000}\left(2\sqrt{f'_c} + 8\sqrt{f'_c}\right)$$

$$\sqrt{\left(\frac{94.75}{18(58)}\right)^2 + \left(\frac{88.11(18)(140)}{1.7(784in^2)}\right)^2} \le \frac{.75}{1000}\sqrt{2\sqrt{4000} + 8\sqrt{4000}}$$

$.22 \le .474$ Section is adequate

$$A_T = \frac{T_u S}{2\phi A_o f_{yv} \cot\theta} = \frac{(130\ kip\text{-}ft)S}{2(.75)(666.4in^2)(60)1} = .0265 \text{ one leg}$$

let $\theta = 45°$ $\quad$ two legs $= .0525$

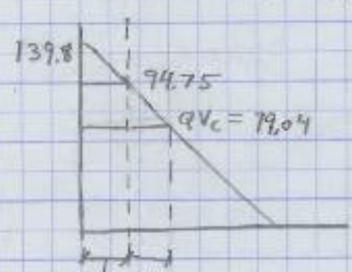
Girder torsion design Page #2

$\phi V_c = .75(2\sqrt{f'_c})\, b_w d = \dfrac{.75(2)\sqrt{4000}\,(18)(58)}{1000} = 99.04\,kip$

Web reinforcement for transverse shear

$A_v = \dfrac{(V_u - \phi V_c)s}{\phi f_{yv} d} = \dfrac{139.8 - 99.045}{.75(60)(58in)} = .01565$

distance $V_u(\text{Section}) = \left(\dfrac{d}{V_{u_1} - V_{u_2}} * V_{u2} - \phi V_c + 58in\right)\dfrac{1}{12in} = 4.37ft$



Total Area to be provided by the two legs

$0 \le x \le 4.37ft \quad 2A_T + A_v = 2A_T(\text{two legs})\left(1 - \dfrac{x}{15}\right)$

$4.37ft \le x \le 15ft \quad 2A_T + A_v = 2A_T\left(1 - \dfrac{x}{15}\right)$

$X = V_u\,\text{section} + X_{Location}$

For #4 stirrups $2A_T + A_v = .4in^2$

Solve for s and substitute $X_{Location}$

S	$X_{Location}$
11.89 in	d
15.74 in	6 ft
16.47 in	8 ft

Torsional Max Spacing $= \dfrac{P_h}{8} = 17.5in$

Shear Max spacing = 24in

Stirrup location

irder Design

Longitudinal steel for torsion

$$\frac{A_T}{S} = A_T\left(1-\frac{d}{(h/2)}\right) = .036\left(1-\frac{4.83ft}{15ft}\right) = .0176 > \frac{25 b_w}{f_{yv}} = .0075 \text{ ok}$$

(minimum)

$$A_l = \frac{A_T}{S} P_h \frac{f_{yv}}{f_{yl}} \cot^2\theta = .0176 (140in)\left(\frac{60ksi}{60ksi}\right) \cot^2(45°) = 2.46 \ in^2$$

$$A_{l,min} = \frac{5\sqrt{f'_c}\,A_{cp}}{f_{vy}(1000)} - \frac{A_T}{S} P_h \frac{f_{yv}}{f_{yl}} = \frac{5\sqrt{4000}\,(60" \times 18")}{60 ksi (1000)} - .0176\,(140in)\left(\frac{60}{60}\right)$$

$A_{l,min} = 3.22 in^2$ Controls **Use 8 #6 Bars**

by ACI spacing can't exceed 12in

Skin Reinforcement ACI 10.6.7 if $\frac{h}{2} > 36in \Rightarrow \frac{h}{2} = 30'$ No skin reinforcement required

Development length
$f'_c = 4000 PSI$

Flexture: #9 Bars $47 d_b = 47(1.128) = 53in$

Shear Stirrups: #4 Bars $38 d_b = 38(.5) = 19in$

torsion Stirrups: #4 Bars $38 d_b = 38(.5) = 19in$

torsion longitudinal: #6 Bars $38 d_b = 38(.750) = 28.5in$

Base Plate Design

Base Plate design

$f'_c = 6000\text{ psi}$ W16×67 $F_y = 50\text{ ksi}$ Concrete Column 20"×30"

$P_u = 500\text{ kips}$ Plate $F_y = 36\text{ ksi}$

A_2 = Area of concrete support $A_1 = BN$ (of Plate)

$$\sqrt{\frac{A_2}{A_1}} \approx 2.0$$

$$A_1 = \frac{P_u}{\phi_c\, .85 f'_c \sqrt{\frac{A_2}{A_1}}} = 75.4\text{ in}^2 < 16.3(10.2) = 166.26 \text{ (Governs)}$$

$$\Delta = \frac{.95d - .8b_f}{2} = \frac{.95(16.3) - .8(10.2)}{2} = 3.66\text{ in}$$

$$N = \sqrt{A_1} + \Delta = \sqrt{166.26\text{ in}^2} + 3.66 = 16.55\text{ in} \Rightarrow 28\text{ in}$$

$$B = \frac{A_1}{N} = \frac{166.26\text{ in}^2}{17\text{ in}} = 9.78\text{ in} < b_f \Rightarrow 20\text{ in}$$

Check concrete bearing strength

$$\phi_c P_p = \phi_c\, 0.85 f'_c A_1 \sqrt{\frac{A_2}{A_1}} = .65(.85)(6)(20(28))2 = 3712.8 > P_u \text{ OK}$$

Plate thickness

$$m = \frac{N - 0.95d}{2} = \frac{28 - .95(16.3)}{2} = 6.26\text{ in}$$

$$n = \frac{B - .8b_f}{2} = \frac{20 - .8(10.2)}{2} = 5.92\text{ in}$$

$$n' = \frac{\sqrt{d b_f}}{4} = \frac{\sqrt{10.2(16.3)}}{4} = 3.22$$

$\ell = 6.26\text{ in}$

$$t_{required} = \ell \sqrt{\frac{2P_u}{.9F_y BN}} = 4.92\sqrt{\frac{2(500k)}{.9(36)(20)(28)}} = 1.47\text{ in}$$

Plate is **PL 1½ × 28 × 20**

Appendix H

3.5 Pretopped Double-Tee Load Tables (cont.)

Strand Pattern Designation

Number of strand (18)
S = straight D = depressed

18 S - D 1

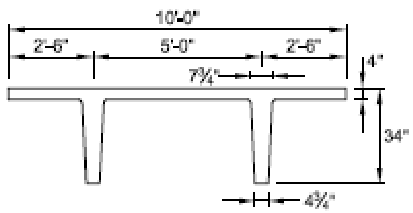
Number of depression points
Diameter of strand in 16ths

Because these units are pretopped and are typically used in parking structures, safe loads shown do not include any superimposed dead loads. Loads shown are live load. Long-time cambers do not include live load.

Key
187 – Safe superimposed service load, lb/ft²
0.8 – Estimated camber at erection, in.
1.2 – Estimated long-time camber, in.

Check with regional producers for availability.

10'-0" × 34"



f'_c = 5000 psi

f_{pu} = 270,000 psi

½-in.-diameter regular strand

Section Properties

	Normalweight	Lightweight
A =	855 in.²	855 in.²
I =	80,780 in.⁴	80,780 in.⁴
y_b =	25.07 in.	25.07 in.
y_t =	8.93 in.	8.93 in.
S_b =	3222 in.³	3222 in.³
S_t =	9046 in.³	9046 in.³
wt =	891 lb/ft	683 lb/ft
DL =	89 lb/ft²	68 lb/ft²
V/S =	2.32 in.	2.32 in.

10DT34

Table of safe superimposed service load, lb/ft², and cambers, in. — **Normalweight — No Topping**

Strand pattern	y_s(end) y_s(center) in.	Span, ft 44	46	48	50	52	54	56	58	60	62	64	66	68	70	72	74	76	78	80	82	84	86	88	90	92	94
128-S	7.00	187	166	147	130	115	102	90	79	70	61	53	46	40	34	28											
	7.00	0.8	0.9	0.9	0.9	0.9	09	0.9	0.9	0.8	0.7	0.7	0.6	0.4	0.3	0.1											
		1.2	1.2	1.2	1.3	1.3	1.3	1.2	1.2	1.1	1.0	0.9	0.7	0.5	0.3	0.1											
148-S	8.00		193	172	153	137	122	109	97	86	76	67	59	52	45	39	34	28									
	8.00		1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.0	0.9	0.9	0.8	0.6	0.5	0.3	0.1									
			1.4	1.4	1.5	1.5	1.5	1.5	1.5	1.5	1.4	1.3	1.2	1.0	0.8	0.6	0.3	0.0									
168-S	9.00			194	174	156	139	125	112	100	90	80	71	63	56	49	43	37	32	27							
	9.00			1.2	1.2	1.2	1.3	1.3	1.3	1.3	1.2	1.2	1.1	1.0	0.9	0.8	0.6	0.4	0.2	0.0							
				1.6	1.7	1.7	1.7	1.7	1.7	1.7	1.7	1.6	1.5	1.4	1.2	1.0	0.8	0.5	0.2	−0.2							
188-S	10.00				192	172	155	139	125	113	101	91	82	73	65	58	51	45	39	33	27						
	10.00				1.3	1.4	1.4	1.4	1.4	1.4	1.4	1.4	1.3	1.2	1.1	1.0	0.9	0.7	0.5	0.3	0.0						
					1.8	1.8	1.9	1.9	1.9	1.9	1.9	1.9	1.8	1.7	1.5	1.3	1.1	0.8	0.5	0.2	−0.2						
188-D1	14.39							184	167	152	138	125	114	103	94	85	77	69	63	56	50	44	38	32	27		
	4.00							1.8	1.9	1.9	1.9	1.9	1.9	1.8	1.8	1.7	1.6	1.5	1.3	1.2	1.0	0.7	0.4	0.1	−0.3		
								2.4	2.5	2.5	2.5	2.5	2.5	2.4	2.3	2.2	2.1	1.9	1.7	1.5	1.2	0.8	0.4	−0.1	−0.6		
208-D1	15.50								188	172	156	143	130	119	107	96	86	78	71	65	59	53	47	41	35	30	25
	4.25								2.0	2.1	2.1	2.2	2.2	2.2	2.1	2.1	2.0	1.9	1.8	1.6	1.4	1.2	1.0	0.7	0.4	0.0	−0.4
									2.8	2.8	2.9	2.9	2.9	2.9	2.8	2.7	2.5	2.4	2.2	2.0	1.8	1.5	1.2	0.7	0.2	−0.3	−0.9

10LDT34

Table of safe superimposed service load, lb/ft², and cambers, in. — **Lightweight — No Topping**

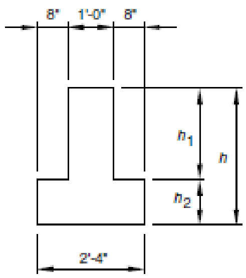
Strand pattern	y_s(end) y_s(center) in.	Span, ft 46	48	50	52	54	56	58	60	62	64	66	68	70	72	74	76	78	80	82	84	86	88	90	92	94	96	98	100	102
128-S	7.00	181	162	145	131	117	106	95	85	77	69	62	55	49	44	39	34	30	26											
	7.00	1.5	1.5	1.6	1.6	1.7	1.7	1.7	1.7	1.7	1.7	1.7	1.6	1.5	1.4	1.2	1.0	0.8	0.6											
		2.0	2.0	2.1	2.2	2.3	2.3	2.3	2.4	2.3	2.3	2.3	2.2	2.0	1.8	1.6	1.3	1.0	0.6											
148-S	8.00		188	169	152	137	124	112	102	92	83	75	68	61	55	49	44	39	35	31	27									
	8.00		1.8	1.8	1.9	2.0	2.0	2.1	2.1	2.1	2.1	2.1	2.0	2.0	1.9	1.8	1.6	1.4	1.2	1.0	0.7									
			2.4	2.5	2.5	2.6	2.7	2.8	2.8	2.8	2.8	2.8	2.7	2.7	2.6	2.4	2.2	1.9	1.5	1.2	0.7									
168-S	9.00			189	171	155	141	128	116	105	96	87	79	72	65	59	53	48	43	38	34	30	26							
	9.00			2.0	2.1	2.2	2.3	2.3	2.4	2.4	2.4	2.4	2.4	2.4	2.3	2.2	2.1	2.0	1.8	1.5	1.3	1.0	0.6							
				2.7	2.8	2.9	3.0	3.1	3.2	3.2	3.2	3.2	3.2	3.1	3.1	3.0	2.8	2.6	2.3	2.0	1.6	1.1	0.6							
188-S	10.00				188	170	155	141	128	117	107	97	89	81	73	67	61	55	50	45	40	36	32	28						
	10.00				2.3	2.4	2.5	2.5	2.6	2.7	2.7	2.7	2.7	2.7	2.6	2.6	2.5	2.3	2.2	2.0	1.7	1.5	1.1	0.8						
					3.1	3.2	3.3	3.4	3.4	3.5	3.5	3.6	3.6	3.5	3.5	3.4	3.2	3.1	2.9	2.6	2.2	1.8	1.3	0.8						
188-D1	14.39						200	183	167	154	141	129	119	109	101	92	83	76	69	62	56	52	48	44	40	37	33	30	26	
	4.00						3.0	3.1	3.3	3.4	3.5	3.6	3.6	3.6	3.6	3.6	3.6	3.5	3.4	3.2	3.1	2.9	2.6	2.4	2.1	1.7	1.3	0.9	0.3	
							4.1	4.2	4.4	4.5	4.6	4.7	4.7	4.7	4.7	4.6	4.5	4.4	4.2	3.9	3.6	3.4	3.1	2.8	2.4	2.0	1.5	0.8	0.0	
208-D1	15.50								187	172	158	146	134	123	113	103	94	86	78	71	65	58	53	48	44	41	37	34	31	29
	4.25								3.5	3.7	3.8	3.9	4.0	4.1	4.2	4.2	4.2	4.2	4.1	4.0	3.8	3.7	3.4	3.2	2.9	2.6	2.3	1.9	1.4	0.9
									4.8	4.9	5.1	5.2	5.3	5.4	5.4	5.4	5.4	5.3	5.1	4.9	4.7	4.4	4.0	3.7	3.3	2.9	2.5	2.0	1.4	0.8

Strength is based on strain compatibility; bottom tension is limited to $12\sqrt{f'_c}$; see pages 3–8 through 3–11 for explanation.
Shaded values require release strengths higher than 3500 psi.

Figure 64: 10' Pretopped Double Tee [34]

Appendix I

3.11 Inverted-Tee Beam Load Tables



f'_c = 5000 psi

f_{pu} = 270,000 psi

1/2-in.-diameter,
low-relaxation strand

Normalweight Concrete

Section Properties								
Designation	h in.	h_1/h_2 in.	A in.²	I in.⁴	y_b in.	S_b in.³	S_t in.³	Wt lb/ft
28IT20	20	12/8	368	11,688	7.91	1478	967	383
28IT24	24	12/12	480	20,275	9.60	2112	1408	500
28IT28	28	16/12	528	32,076	11.09	2892	1897	550
28IT32	32	20/12	576	47,872	12.67	3778	2477	600
28IT36	36	24/12	624	68,101	14.31	4759	3140	650
28IT40	40	24/16	736	93,503	15.83	5907	3869	767
28IT44	44	28/16	784	124,437	17.43	7139	4683	817
28IT48	48	32/16	832	161,424	19.08	8460	5582	867
28IT52	52	36/16	880	204,884	20.76	9869	6558	917
28IT56	56	40/16	928	255,229	22.48	11,354	7614	967
28IT60	60	44/16	976	312,866	24.23	12,912	8747	1017

1. Check local area for availability of other sizes.
2. Safe loads shown include 50% superimposed dead load and 50% live load.
   800 psi top tension has been allowed, therefore, additional top reinforcement is required.
3. Safe loads can be significantly increased by use of structural composite topping.

Key
8510 – Safe superimposed service load, lb/ft
0.2 – Estimated camber at erection, in.
0.1 – Estimated long-time camber, in.

Table of safe superimposed service load, lb/ft, and cambers, in.

Designation	Number strand	y_s (end) in.	Span, ft 16	18	20	22	24	26	28	30	32	34	36	38	40	42	44	46	48	50
28IT20	9	2.44	6510	5070	4040	3280	2710	2260	1900	1610	1380	1180	1020							
			0.2	0.3	0.4	0.4	0.5	0.5	0.6	0.7	0.7	0.7	0.8							
			0.1	0.1	0.1	0.1	0.1	0.1	0.0	0.0	0.0	0.0	–0.1							
28IT24	18	2.73	9610	7500	5990	4880	4030	3370	2850	2420	2080	1790	1550	1350	1170	1020				
			0.2	0.3	0.3	0.4	0.4	0.5	0.6	0.6	0.7	0.7	0.7	0.8	0.8	0.8				
			0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.0	0.0	–0.1	–0.2				
28IT28	13	3.08			8350	6820	5650	4750	4030	3450	2970	2580	2250	1970	1730	1530	1350	1190	1060	
					0.3	0.3	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.8	0.8	0.9	0.8	0.8	
					0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.0	0.0	–0.1	–0.2	–0.2	
28IT32	15	3.47				9040	7520	5330	5380	4620	4000	3490	3050	2690	2370	2110	1870	1670	1490	1330
						0.3	0.4	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.8	0.9	0.9	0.9	0.9
						0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.0	0.0	0.0	–0.1
28IT36	16	3.50					9830	8290	7070	6090	5280	4610	4060	3580	3180	2830	2530	2270	2040	1830
							0.3	0.4	0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.8	0.9	0.9	0.9
							0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.0	0.0	0.0	–0.1
28IT40	19	4.21							8630	7440	6460	5640	4960	4390	3890	3470	3100	2780	2500	2250
									0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.8	0.8	0.9	0.9
									0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1
28IT44	20	4.40								9180	7980	6990	6160	5460	4860	4340	3890	3500	3160	2850
										0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.7	0.8	0.8	0.8
										0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.0
28IT48	22	4.55									9710	8520	7520	6670	5950	5330	4790	4320	3900	3540
											0.4	0.5	0.5	0.6	0.6	0.7	0.7	0.8	0.8	0.9
											0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1
28IT52	24	5.17										9980	8820	7830	6990	6270	5640	4100	4610	4190
												0.5	0.5	0.6	0.6	0.6	0.7	0.7	0.8	0.8
												0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1
28IT56	26	5.23												9300	8310	7460	6730	6080	5520	5020
														0.5	0.6	0.6	0.7	0.7	0.8	0.8
														0.2	0.2	0.2	0.2	0.2	0.2	0.2
28IT60	28	5.57													9640	8660	7820	7080	6430	5850
															0.6	0.6	0.7	0.7	0.8	0.8
															0.2	0.2	0.2	0.2	0.2	0.2

Appendix J

Corbel Design

Load on Corbel	43.44	kips	
Moment load arm (e)	4	in	
Moment	14.48	kip-ft	
V_u	43.44	kip	
$V_n = A_{vf}F_y(1.4).75$	63	kip	
(b) Width from face of column or beam	8	in	
$V_n \leq .2f'_cA_c$	64	kip	OK
$V_n \leq 800A_c$	64	kip	OK
Nuc	8.688	kip	
Corbel depth requirment .5d > b, due to Slope at bottom of corbel	6	in	OK
Front face height	8	in	
A_c= A concrete plane	80	in^2	
f'_c	4000	psi	
d solved	9.05	in^2	
d choosen	10	in^2	
Total height of corbel	16	in	
Spacing total height of steel	6.66666667	in	
e/d	0.4	< 1.0 by ACI Code	
1.4 = coefficient of friction			
.75 reduction factor			
A_{vf}	0.68952381	in^2	
M_u	155.304	kip-in	
Assume (a)	2.1		
Af	0.38560894	in^2	
a check	2.09558824		
Revised Af	0.43128108	in^2	
Choosen A_f	1	in^2	
A_n	0.19306667	in^2	Additional area of steel
Choosen A_n	0.2	in^2	
Case (a) $A_{sc}>(2/3)A_{vf}+A_n$	0.65968254		
Case (b) $A_{sc}>A_f+A_n$	0.62434775	Governs	
Min A_{sc}	0.21333333		
Chosen A_{sc}	**0.93**	**in^2**	**3 #5 Bars**

$A_h > 0.5*A_{sc}-A_n)$	0.365	in^2	Governs
$A_h>.5*A_f$	0.5	in^2	
$A_h>.5*(2/3)A_{vf}$	0.33333333	in^2	
Chosen A_h	**0.66**	**in^2**	**3 #3 hoop Bars at 2in on center**
d_b	0.875	in	#7 Bar
l_{dh} Basic development Length	**16.601958**	**in**	
l_{dh} Past the face of the column or beam	**7.8019101**	**in**	
hook extension	**10.5**	**in**	
Steel Cover	2	in	
Width of corbel (From front)	8	in	< .5d = 8 in
Steel Plate size			
Dimension from front face of corbel. Choosen by bearing area of double tee	6	in	
Width (b dimension - 1in to prevent the plate from overloading the corner of the corbel	6	in	
Lenth of leg on face of corbel	2.41333333	in	
Choose	3	in	
m	1.15		
n	1.6		
n'	1.5		
l = largest m,n,n'	2.2		
Plate Thickness	0.60042509	in	
Use 5/8 in plate	0.625		

Appendix K

Bar number Diameter

Bar designation number	Nominal diameter in inches	Area
3	0.375	0.11
4	0.5	0.2
5	0.625	0.31
6	0.75	0.44
7	0.875	0.6
8	1	0.79
9	1.128	1
10	1.27	1.27
11	1.41	1.56
14	1.693	2.25
18	2.257	4

Base Level

Interior Column

Lenth	34.5	ft	34.5
b	30	in	24
h	30	in	24
Area	900	in^2	576
Trib Area	825	ft^2	825
Slab d	10	in	10
slab	91.67	psf	91.66666667
Slab load	75.625	kips	75.625
P Load	900	kip	625.625
P(Dead)	356.7	k	
P(Live)	268.3	k	
Pu	857.32	k	
M1 (Dead)	6.8	kip-ft (Single Curvature)	
M2 (Dead)	10.2	kip-ft (Single Curvature)	
M1 (Live)	325.7	kip-ft (Single Curvature)	200
M2 (Live)	425.7	kip-ft (Single Curvature)	200

Mu	693.36	kip-ft		
f'_c	4	ksi	4	
I	67500	In^4	27648	
r = (.3*B)	8.7	in	6.92820323	
l/r	47.8		59.75575286	
k	1.0		1	
kl/r	47.8		59.75575286	
Upper limit	24.8		22	

kl/r > 22 Column is slender

Because steel column is above and footing is below let

ψ_a	1.67	Hand Calculations		
ψ_b	1.67	Hand Calculations	compare with K = 1.0	
k	0.70	pg. 316 in DoCS		
kl/r	33.46		79.83368582	
M2, Min	107.17	kip-ft (not controlling)	112.5	
Cm	0.91		0.906037115	
β	0.50		1	
E	3604996.53	psi	3604996.533	
EI	64921237581.45	psi	48667453190	
P_c	7629.39	kip	1004.858107	
ϒ amplification factor	1.07		-4.665521723	
M1u	347.10	kip-ft	-1519.560425	
m2u	453.67	kip-ft	-933.1043446	
Pu	857.32	kip	625.625	
e	0.50		0.31968032	
K_n	0.37		0.417751736	
R_n	0.08		-0.311533235	
Cover	3.00	in	2	
ϒ	0.80		0.833333333	
A_{st}	**0.02**	**From Graph A.7**	**0.02**	
Required Reinforcement				
A	13.50	in^2	11.52	
# of Bars	**8.00**	**# 9**	8	15
	4.00	**#10**	5.08	
Max Spacing reqirements				

	d of ties	0.50	# 4	0.5
16d	18.05	in		8
48d	24.00	in		24
b or h	30.00	in		24
Maximum Allowable Spacing				
	18.05	in		8
Final Design Spacing				
	10	**in**		

Base Level

Exterior perimiter Column

Lenth(U)	11.5	ft		
b	30			
h	30			
double tees	68	psf		
double tees load	28.05	kips		
Area	900.00	in^2		
Trib Area	412.50	ft^2		
Slab d	10.00	in		
psf of slab	49.50	psf		
Slab load	20.42	kips		
P(Dead)	356.70	k		
P(Live)	268.30	k		
Pu	933.84	k		
M1 (Dead)	6.80	kip-ft (Single Curvature)	**Beam h**	
M2 (Dead)	10.20	kip-ft (Single Curvature)	**Beam b**	
M1 (Live)	325.70	kip-ft (Single Curvature)	**Beam Lenth**	
M2 (Live)	425.70	kip-ft (Single Curvature)	**Load to column**	
Mu	693.36	kip-ft		
f'_c	4.00	ksi		2
I	67500.00		2	
r	8.66		30	
l/r	15.93		26.4	
k	1.00			
kl/r	15.93			
kl/r≤ 22 Neglect Sldnerness				
Upper limit	24.82			

kl/r > 22 Column is slender
Because steel column is above and footing is below let

ψ_a	1.67			
ψ_b	1.67			
k	0.83			
kl/r	13.23			
M2, Min	116.73	kip-ft		
Cm	0.91			
β	0.50		EI/l	19784220971
E	3604996.53	psi	k	0.285344822
EI	64921237581.45	psi		
EI/l	5645325007.08	*Ratio of Stiffness*		
P_c	48839.61	kip		
ϒ amplification factor	0.93	1		
M1u	325.70	kip-ft		
m2u	425.70	kip-ft		
Pu	933.84	kip		
e	0.46			
K_n	0.40			
R_n	0.07			
Cover	3.00	in		
ϒ	0.80			
A_{st}	**0.014**	**From Graph A.11**		
Required Reinforcement				
A	12.6	in^2		
# of bars	**8**	**# 9**		
	4	**# 10**		
Max Spacing reqirements				
d of ties	0.5			
16d	18.048	in		
48d	24	in		
b or h	30	in		
Maximum Allowable Spacing				
	18.048	in		

Final Design Spacing

10 **in**

Appendix L

Table 18: Shear Wall Design

User:	Project: ASDIP Reference Manual	Page #	
Demo Version - Trial License	Descrip: Example	Date:	02/23/2012
	Engineer: Your Name		

ASDIP v 4.1.2 — **SHEAR WALL SEISMIC DESIGN** — www.asdipsoft.com

GEOMETRY

Wall Thickness Tw	**18.0**	in
Wall Total Length Lw	**26.1**	ft
Wall Total Height Hw	**35.0**	ft
Lateral Displacement du	**2.0**	in

FLEXURE AND AXIAL LOAD DESIGN

Generate the interaction diagram of the wall section using the INT-WALL program.

Neutral Axis Position c = kL ..	**64.5**	in
Nom. Bending Strength Mn	**136330**	k-ft

SHEAR DESIGN

2 Acv (f'c)½ Parameter	713.1	kip	
Req. Number of Courtains	1		
Use 2 Courtains of Reinforcement			
Max. Allowed Bars Spacing ...	18	in	
Min. Vertical Reinf. Area Asv	0.54	in²	
Min. Horiz. Reinf. Area Ash ..	0.54	in²	
Hw/Lw Aspect Ratio = 1.3 , Alfa c = 3.0			
Use # 6 @ 12 in. Vert. Bars, Asv = 0.88 in²			*OK*
Use # 5 @ 10 in. Horiz. Bars, Ash = 0.74 in²			*OK*
Under-Strength ø Factor	0.60		
Design Shear Strength øVn ..	1341	kip	OK

COMBINED FACTORED LOADS

Vertical Axial Force Pu	**625.0**	kip
Horizontal Shear Force Vu ...	**631.0**	kip
Overturning Moment Mu	**200**	k-ft

MATERIALS

Concrete Strength f'c	**4.0**	ksi
Main Steel Strength fy	**60.0**	ksi
Ties Steel Strength fyh	**60.0**	ksi

BOUNDARY ELEMENTS DESIGN

Use the Stress Ratio Method of Analysis

Max. Stress / 0.2 f'c Ratio	0.15		
Special Boundary Elements Not Required			
Boundary Member Width	**408.0**	in	
Boundary Member Length	**22.0**		NA
Minimum Member Length	NA	in	
Use 36 # 11 Vertical Bars , Rho = 0.0063			*OK*
Concrete Clear Cover	**2.0**	in	
Stirrups Bar Size #	**4**		
Use Stirrups Vertical Spacing	**5.0**		NA
Stirrups Max. Vert. Spacing ..	NA	in	

	hc (in)	Ash (in²)	Legs	sh (in)
Long. Dir.	403.5	NA	NA	NA
Short Dir.	17.5	NA	NA	NA

Date: 23-Feb-2012		Time: 01:49 AM	
Project: **ASDIP Reference Manual**			
Descrip: **Example**			
Engineer: **Your Name**			
COMBINED FACTORED LOADS:			
Vertical Axial Force Pu	(kip) =		**625.0**
Horizontal Shear Force Vu	(kip) =		**631.0**
Overturning Moment Mu	(k-ft) =		**200**
GEOMETRY:			
Wall Thickness Tw	(in) =		**18.0**
Wall Total Length Lw	(ft) =		**26.1**
Wall Total Height Hw	(ft) =		**35.0**
Lateral Displacement du	(in) =		**2.0**
MATERIALS:			
Concrete Strength f'c	(ksi) =		**4.0**
Main Steel Strength fy	(ksi) =		**60.0**
Ties Steel Strength fyh	(ksi) =		**60.0**
SHEAR DESIGN:			
2 Acv (f'c)½ Parameter	(kip) =		713.1
Number of Curtains	(Reqd : 1) =		**2**
Max. Allow. Bar Spacing	(in) =		18.0

	Bar Size #		Spacing (in)	Asmin/As Ratio
Vertical	**6**	@	**12.0**	0.61
Horizontal	**5**	@	**10.0**	0.73

GREAT ! , STEEL AREA IS OK !		
Hw/Lw Aspect Ratio = 1.34 ,	Alfa c = 3.0	
Under-Strength ø Factor	=	0.60
Design Shear Strength øVn	(kip) =	1341
Vu / øVn Shear Capacity Ratio	=	0.47
GREAT ! , SHEAR CAPACITY IS OK !		

Demo Version - Trial License			
SHEAR WALLS SEISMIC DESIGN			
FLEXURE AND AXIAL LOAD DESIGN:			
Generate the interaction diagram of the wall section using the INT-WALL program.			
To open, select Design | Interaction Diagram			
Neutral Axis Position c = kL	(in) =		**64.5**
Nom. Bending Strength Mn	(k-ft) =		**136330**
BOUNDARY MEMBER DESIGN (Note 1):			
Displ. or Stress Method?	(D/S) =		**S**
c / Lw/(600(du/Hw)) Displ. Ratio	=		0.86
Max. Stress / 0.2 f'c Ratio	=		0.15
SPECIAL BOUNDARY MEMB. NOT REQD !			
Boundary Member Width	(in) =		**408.0**
Boundary Member Length	(in) =		**22.0**
Minimum Member Length	(in) =		NA
GREAT !, MEMBER LENGTH IS OK !			
- LONGITUDINAL STEEL:			
Longit. Reinf. = **36**	Bars #		**11**
Longitudinal Steel Area	(in²) =		56.2
Reinforcement Ratio Rho	=		0.0063
Reinf. Ratio Rho = 400/fy	=		0.0067
NO BOUNDARY REQUIREMENTS			
- CONFINEMENT STEEL:			
Concrete Clear Cover	(in) =		**2.0**
Stirrups Bar Size	(#) =		**4**
Stirrups Max. Vert. Spacing	(in) =		NA
Stirrups Vertical Spacing sv	(in) =		**5.0**

	Ash (in²)	Legs	sh (in)
Long Direction	NA	NA	NA
Short Direction	NA	NA	NA

Notes:

1.- Even when special boundary members are NOT required, some code requirements must still be satisfied.

2.- Generate the interaction diagram of the wall section using the INT-WALL program. To open select Design | Interaction Diagram from the menu.

Appendix M

Footing Design					
P	800	kips			
P unfactofed	666.666667	kips			
Base	15	ft			
Footing depth	28	in			
Column base	18	in			
f'c	4000	psi			
fy	60	ksi			
q_a	4	ksf		**Check**	
depth of footing	5	ft			
unit weight of concrete and soil		pcf			
q_e		ksf			
q_e		ksf			
A_{req}	166.666667	ft^2			
Base	12.9099445	ft			
Base Choosen	15.00	ft		**Check**	
q_u	4	ksf		3.55555556	
d	26	in		**Check**	
b_o	176	in			
	44	in			
V_{ul}	**846.22222**	kip			
V_c	1157.64661	kip			
ΦV_c	**868.23495**	kip		**Check ΦV_c > V_{ul}**	
	4.25	ft			
V_{u2}	**255**	kips			
V_c	591.978378	kips			
ΦV_c	**443.98378**	kip		**Check ΦV_c > V_{u2}**	
			if so d is also adequtate for on-way shear		
	6.70833333	ft			
M_u	16200.625	kip-in			
a	2	in		**Check**	
A_s	12.000463	in^2			

$A_{s,\ min}$	14.7994594	in^2			
$A_{s,\ min}$	15.6	in^2	Governs		
# 8 bars	0.79	in^2			
Development length	78.5	in			
Requred Development length	41	in	Table A.10	**Check**	
# of # 8 bars	19.7468354	Use	**20**	**Check**	9 in o.c.
Dowels					
# 8 bars	0.79	in^2			
Diameter	1	in			
Requred Development length	19	in	Table A.11	**Check**	
lap splice	30	in			
Total length of dowel	56	in			
Required Depth of footing	29.5	in	**<**	**30**	**in**
For retaining wall Use		20	#8 bar	Both longitudinal and transverse, top and bottom.	

Appendix N

Foundation Steel Layout Design

Location (ft)	Moment	R	Row	A_s	# of # 9 bars
15	1048.5	207.7883472	0.0055	5.742	6
12	838.8	166.2306778	0.0045	4.698	5
10	699	138.5255648	0.0035	3.654	4
8	559.2	110.8204518	0.003	3.132	4
7	489.3	96.96789536	0.0025	2.61	3
6	419.4	83.11533888	0.0025	2.61	3
4	279.6	55.41022592	0.0015	1.566	2

Retwall as beam

Span	30	ft
Width	1	ft
Soil unit wieght	65	pcf
Ground slope angle	20	degrees
OCR	1	assumed
b	1	ft

16	2.156	ft/in
	2.15625	

K_o	0.3069378	0.5

H (ft)	YK_oH (lb/ft)	Moment (Kip-ft)	d (ft)	R	ρ	A_s	#8 Bars
34.5	1121.25	126.140625	2.25	192.25823	0.0049	1.5876	2.01
30	975	109.6875	2.25	167.18107	0.005	1.62	2.05
28	910	102.375	2.25	156.035665	0.005	1.62	2.05
26	845	95.0625	2.25	144.890261	0.004	1.296	1.64
24	780	87.75	2.25	133.744856	0.004	1.296	1.64
22	715	80.4375	2.25	122.599451	0.004	1.296	1.64
20	650	73.125	2.25	111.454047	0.003	0.972	1.23
18	585	65.8125	2.25	100.308642	0.003	0.972	1.23
16	520	58.5	1.25	288.888889	0.008	1.44	1.82
14	455	51.1875	1.25	252.777778	0.008	1.44	1.82
11	357.5	40.21875	1.25	198.611111	0.0055	0.99	1.25
9	292.5	32.90625	1.25	162.5	0.005	0.9	1.14
7	227.5	25.59375	1.25	126.388889	0.005	0.9	1.14
5	162.5	18.28125	1.25	90.2777778	0.005	0.9	1.14
3	97.5	10.96875	1.25	54.1666667	0.005	0.9	1.14
1	32.5	3.65625	1.25	18.0555556	0.005	0.9	1.14

Appendix O

Table 19: Final Cost Estimate

Estimate Name:	**Steel Building Estimate** WPI 21 Boynton Street , Worcester , Massachusetts , 01609	
Building Type:	**College, Classroom, 2-3 Story with Face Brick with Concrete Block Back-up / Steel Frame**	
Location:	**National Average**	
Story Count:	**3**	
Story Height (L.F.):	**14**	
Floor Area (S.F.):	**96678**	
Labor Type:	**Union**	
Basement Included:	**No**	
Data Release:	**Year 2012**	Costs are derived from a building model with basic components.
Cost Per Square Foot:	**$307.42**	Scope differences and market conditions can cause costs to vary significantly.
Building Cost:	**$29,766,218**	**Parameters are not within the ranges recommended by RSMeans.**

		% of Total	Cost Per S.F.	Cost
A Structure Base		**2.00%**	**$4.56**	**$441,200**
A1030	**Slab on Grade**		**$3.40**	**$329,000**
	Slab on grade, 8" thick, non-industrial, reinforced			
A2020	**Basement Walls**		**$1.16**	**$112,200**
	Foundation wall, CIP, 4' wall height, direct chute, .099 CY/LF, 4.8 PLF, 8" thick			
	Foundation wall, CIP, 4' wall height, direct chute, .148 CY/LF, 7.2 PLF, 12" thick			
B Shell		**20.51%**	**$46.79**	**$4,522,850**
B1010	**Floor Construction**		**$26.87**	**$2,597,250**
	Steel column, W16, 400 KIPS, 14' unsupported height, 80 PLF			
	Steel column, W14, 300 KIPS, 14' unsupported height, 50 PLF			
	Floor, composite metal deck, shear connectors, 5.5" slab, 35'x35' bay, 29.5" total depth, 75 PSF superimposed load, 121 PSF total load			
	Fireproofing, gypsum board, fire rated, 3 layer, 1.5" thick, 10" steel column, 3 hour rating, 27 PLF			
	Fireproofing, gypsum board, fire rated, 3 layer, 1.5" thick, 14" steel column, 3 hour rating, 35 PLF			
B1020	**Roof Construction**		**$4.14**	**$400,800**
	Floor, steel joists, beams, 1.5" 22 ga metal deck, on columns, 35'x35' bay, 25" deep, 30 PSF superimposed load, 52 PSF total load			
	Floor, steel joists, beams, 1.5" 22 ga metal deck, on columns, 35'x35' bay, 25" deep,			

	30 PSF superimposed load, 52 PSF total load, add for column			
B2010	**Exterior Walls** Brick wall, composite double wythe, standard face/CMU back-up, 8" thick, perlite core fill		**$8.33**	**$804,600**
B2020	**Exterior Windows** Aluminum flush tube frame, for 1/4"glass,1-3/4"x4", 5'x6' opening, no intermediate horizontals		**$4.60**	**$444,500**
	Glazing panel, plate glass, 1/4" thick, clear			
B2030	**Exterior Doors** Door, aluminum & glass, with transom, narrow stile, double door, hardware, 6'-0" x 10'-0" opening		**$0.67**	**$64,500**
B3010	**Roof Coverings**		**$2.18**	**$211,200**
	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			
	Flashing, aluminum, no backing sides, .019"			
	Gravel stop, aluminum, extruded, 4", mill finish, .050" thick			
C Interiors		**15.58%**	**$35.54**	**$3,436,000**
C1010	**Partitions**		**$4.50**	**$435,500**
	Concrete block (CMU) partition, light weight, hollow, 6" thick, no finish			
C1020	**Interior Doors** Door, single leaf, kd steel frame, hollow metal, commercial quality, flush, 3'-0" x 7'-0" x 1-3/8"		**$5.97**	**$577,000**
C1030	**Fittings**		**$5.98**	**$578,500**
	Chalkboards, liquid chalk type, wood frame & chalktrough			
	Cabinets, school, counter, wood, 32" high			
C2010	**Stair Construction**		**$3.28**	**$317,000**
	Stairs, steel, cement filled metal pan & picket rail, 20 risers, with landing			
C3010	**Wall Finishes**		**$3.98**	**$384,500**
	2 coats paint on masonry with block filler			
	Painting, masonry or concrete, latex, brushwork, primer & 2 coats			
	Painting, masonry or concrete, latex, brushwork, addition for block filler			
	Ceramic tile, thin set, 4-1/4" x 4-1/4"			
C3020	**Floor Finishes**		**$4.97**	**$480,500**
	Carpet, tufted, nylon, roll goods, 12' wide, 36 oz			
	Carpet, padding, add to above, maximum			
	Vinyl, composition tile, minimum			
	Vinyl, composition tile, maximum			
	Tile, ceramic natural clay			
C3030	**Ceiling Finishes** Acoustic ceilings, 3/4"mineral fiber, 12" x 12" tile, concealed 2" bar & channel grid, suspended support		**$6.86**	**$663,000**

D Services		46.61%	$105.94	$10,276,000
D1010	**Elevators and Lifts**		$5.09	$492,500
	2 - Hydraulic, passenger elevator, 2500 lb, 2 floors, 100 FPM			
	Hydraulic passenger elevator, 2500 lb., 2 floor, 125 FPM			
D2010	**Plumbing Fixtures**		$24.81	$2,399,250
	Water closet, vitreous china, bowl only with flush valve, wall hung			
	Urinal, vitreous china, wall hung			
	Lavatory w/trim, wall hung, vitreous china, 19" x 17"			
	Lab sink w/trim, polyethylene, single bowl, double drainboard, 54" x 24" OD			
	Service sink w/trim, vitreous china, wall hung 22" x 20"			
	Water cooler, electric, wall hung, wheelchair type, 7.5 GPH			
D2020	**Domestic Water Distribution**		$2.32	$224,500
	Gas fired water heater, commercial, 100< F rise, 600 MBH input, 576 GPH			
D2040	**Rain Water Drainage**		$0.62	$59,500
	Roof drain, CI, soil,single hub, 6" diam, 10' high			
	Roof drain, CI, soil,single hub, 6" diam, for each additional foot add			
D3050	**Terminal & Package Units**		$29.55	$2,856,750
	Rooftop, multizone, air conditioner, schools and colleges, 25,000 SF, 95.83 ton			
D4010	**Sprinklers**		$13.23	$1,312,000
	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			
	Spray Fireproofing			
D4020	**Standpipes**		$0.33	$32,000
	Dry standpipe risers, class III, steel, black, sch 40, 6" diam pipe, 1 floor			
	Dry standpipe risers, class III, steel, black, sch 40, 6" diam pipe, additional floors			
D5010	**Electrical Service/Distribution**		$2.55	$247,000
	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**		$19.65	$1,899,000
	Receptacles incl plate, box, conduit, wire, 10 per 1000 SF, 1.2 W per SF, with transformer			
	Wall switches, 1.0 per 1000 SF			
	Miscellaneous power, 1.2 watts			
	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			
	Fluorescent fixtures recess mounted in ceiling, 2.4 watt per SF, 60 FC, 15 fixtures @ 32 watt per 1000 SF			
D5030	**Communications and Security**		$7.06	$682,500

	Communication and alarm systems, includes outlets, boxes, conduit and wire, sound systems, 30 outlets			
	Communication and alarm systems, fire detection, addressable, 25 detectors, includes outlets, boxes, conduit and wire			
	Fire alarm command center, addressable with voice, excl. wire & conduit			
	Communication and alarm systems, includes outlets, boxes, conduit and wire, master clock systems, 20 rooms			
	Communication and alarm systems, includes outlets, boxes, conduit and wire, master TV antenna systems, 30 outlets			
	Internet wiring, 8 data/voice outlets per 1000 S.F.			
D5090	**Other Electrical Systems**		**$0.73**	**$71,000**
	Generator sets, w/battery, charger, muffler and transfer switch, gas/gasoline operated, 3 phase, 4 wire, 277/480 V, 100 kW			
E Equipment & Furnishings		**1.69%**	**$3.86**	**$373,000**
E1090	**Other Equipment**		**$3.86**	**$373,000**
	80 - T.V. systems, VHF reception & distribution, 100 outlets			
	10 - T.V. systems, VHF reception & distribution, 30 outlets			
	25 - T.V. systems, VHF reception & distribution, 12 outlets			
	6 - Sound system, speaker, ceiling or wall, excl rough-in wires, cables & conduits			
	3 - Sound system, amplifier, 250 W, excl rough-in wires, cables & conduits			
	25 - Detection Systems, smoke detector, duct type, excl. wires & conduit			
	50 - Detection Systems, heat detector, smoke detector, ceiling type, excl. wires & conduit			
	150 - Seating, lecture hall, pedestal type, maximum			
	100 - Seating, lecture hall, pedestal type, minimum			
	150 - School furniture, classroom, movable chair & desk type, maximum			
	100 - School furniture, classroom, movable chair & desk type, minimum			
	50 - Auditorium chair, fully upholstered, spring seat			
	80 - Auditorium chair, veneer back, padded seat			
	100 - Auditorium chair, all veneer construction			
	100 - Lockers, steel, baked enamel, single tier, 60" or 72", minimum			
	1 - Flagpoles, aluminum, tapered, ground set, 70' high, excludes base or foundation			
	20 - Emergency lighting units, nickel cadmium battery operated, twin sealed beam light, 25 W, 6 V each			
	2 - Hydraulic Passenger Elevators, for number of stops over 2, add			
	1 - Master time clock system, master controller, clocks & bells, 20 room, excl. wires & conduits			
	1 - Library furniture, carrels, hardwood, 36" x 24", max			
F Special Construction		**0.00%**	**$0.00**	**$0**
G Building Sitework		**13.61%**	**$31.03**	**$3,000,000**
SubTotal		**100%**	**$227.72**	**$22,049,050**

Contractor Fees (General Conditions,Overhead,Profit)	**25.00%**	**$56.93**	**$5,512,263**
Architectural Fees	**7.00%**	**$15.94**	**$1,543,434**
User Fees	**3.00%**	**$6.83**	**$661,472**
Total Building Cost		**$307.42**	**$29,766,218**

Square Foot Cost Estimate Report

Estimate Name:	**Parking Garage Estimate** WPI 21 Boynton Street , Worcester , Massachusetts , 01609	
Building Type:	**Garage, Parking with Precast Concrete / R/Conc. Frame**	
Location:	**National Average**	
Story Count:	**3**	
Story Height (L.F.):	**14**	
Floor Area (S.F.):	**96678**	
Labor Type:	**Union**	
Basement Included:	**No**	
Data Release:	**Year 2012**	Costs are derived from a building model with basic components.
Cost Per Square Foot:	**$77.01**	Scope differences and market conditions can cause costs to vary significantly.
Building Cost:	**$7,446,000**	**Parameters are not within the ranges recommended by RSMeans.**

		% of Total	Cost Per S.F.	Cost
A Substructure		10.90%	$12.41	$1,216,000
A1010	**Standard Foundations**		**$3.08**	**$298,000**
	Strip footing, concrete, reinforced, load 11.1 KLF, soil bearing capacity 6 KSF, 12" deep x 24" wide			
	Spread footings, 3000 PSI concrete, load 600K, soil bearing capacity 6 KSF, 10' - 6" square x 33" deep			
A1030	**Slab on Grade**		**$2.25**	**$218,000**
	Slab on grade, 6" thick, light industrial, non reinforced			
A2010	**Basement Excavation**		**$4.80**	**$480,000**
	Excavate and fill, 30,000 SF, 4' deep, sand, gravel, or common earth, on site storage			
A2020	**Basement Walls**		**$2.28**	**$220,000**
	Foundation wall, CIP, 4' wall height, direct chute, .099 CY/LF, 4.8 PLF, 8" thick			
B Shell		47.70%	$26.89	$2,600,500
B1010	**Floor Construction**		**$18.94**	**$1,831,500**

	Precast concrete column, 18" sq, tied, eccentric load, 600K, 14' story height, 375 PLF, 4 load levels			
	Precast concrete column, 20" sq, tied, eccentric load, 800K, 14' story height, 475 PLF, 4 load levels			
	Concrete I beam, precast, 12" x 24", 370 PLF, 15' span, 5.92 KLF superimposed load			
	Concrete I beam, precast, 18" x 36", 790 PLF, 25' span, 6.44 KLF superimposed load			
	Concrete I beam, precast, 24" x 44", 565 PLF, 30' span, 8.66 KLF superimposed load			
	Precast concrete double T beam, 2" topping, 24" deep x 8' wide, 50' span, 75 PSF superimposed load, 165 PSF total load			
B2010	**Exterior Walls**		**$7.95**	**$769,000**
	Exterior wall, precast concrete, flat, 8" thick, 4' x 8', white face, low rise			
C Interiors		**10.00%**	**$5.63**	**$545,000**
C1010	**Partitions**		**$1.40**	**$135,500**
	Concrete block (CMU) partition, light weight, hollow, 8" thick, no finish			
	8" concrete block partition			
C1020	**Interior Doors**		**$0.25**	**$24,000**
	Door, single leaf, kd steel frame, hollow metal, commercial quality, flush, 3'-0" x 7'-0" x 1-3/8"			
C2010	**Stair Construction**		**$0.27**	**$26,500**
	Stairs, CIP concrete, w/landing, 12 risers, w/o nosing			
C3010	**Wall Finishes**		**$0.20**	**$19,500**
	Painting, masonry or concrete, latex, brushwork, primer & 2 coats			
C3020	**Floor Finishes**		**$3.51**	**$339,500**
	Traffic deck, vinyl and neoprene membrane, sprayed, 2 to 4 mils thick			
D Services		**25.10%**	**$24.72**	**$2,390,500**
D1010	**Elevators and Lifts**		**$4.40**	**$425,000**
	1 - Traction geared elevators, passenger, 3500 lb, 6 floors, 200 FPM			
	Hydraulic passenger elevator, 3500 lb., five floors, 10' story height, 125 FPM			
D2010	**Plumbing Fixtures**		**$0.04**	**$4,000**
	Water closet, vitreous china, bowl only with flush valve, floor mount			
	Lavatory w/trim, wall hung, PE on CI, 19" x 17"			
D2020	**Domestic Water Distribution**		**$0.07**	**$7,000**
	Electric water heater, commercial, 100< F rise, 50 gallon tank, 9 KW 37 GPH			
D2040	**Rain Water Drainage**		**$1.91**	**$184,500**
	Roof drain, steel galv sch 40 threaded, 3" diam piping, 10' high			
	Roof drain, steel galv sch 40 threaded, 3" diam piping, for each additional foot add			
	Roof drain, steel galv sch 40 threaded, 4" diam piping, 10' high			
	Roof drain, steel galv sch 40 threaded, 4" diam piping, for each additional foot add			
D4010	**Sprinklers**		**$14.54**	**$1,406,000**
	Dry pipe sprinkler systems, steel, ordinary hazard, 1 floor, 10,000 SF			

	Dry pipe sprinkler systems, steel, ordinary hazard, each additional floor, 10,000 SF			
	Spray Fireproofing			
D4020	**Standpipes**		**$0.13**	**$13,000**
	Dry standpipe risers, class III, steel, black, sch 40, 4" diam pipe, 1 floor			
	Dry standpipe risers, class III, steel, black, sch 40, 4" diam pipe, additional floors			
D5010	**Electrical Service/Distribution**		**$0.38**	**$36,500**
	Service installation, includes breakers, metering, 20' conduit & wire, 3 phase, 4 wire, 120/208 V, 400 A			
	Feeder installation 600 V, including RGS conduit and XHHW wire, 400 A			
	Switchgear installation, incl switchboard, panels & circuit breaker, 400 A			
D5020	**Lighting and Branch Wiring**		**$3.07**	**$297,000**
	Receptacles incl plate, box, conduit, wire, 2.5 per 1000 SF, .3 watts per SF			
	Miscellaneous power, to .5 watts			
	Motor installation, three phase, 200 V, 15 HP motor size			
	Motor feeder systems, three phase, feed to 200 V 15 HP, 230 V 15 HP, 460 V 40 HP, 575 V 50 HP			
	Fluorescent fixtures recess mounted in ceiling, 0.8 watt per SF, 20 FC, 5 fixtures @32 watt per 1000 SF			
D5030	**Communications and Security**		**$0.12**	**$11,500**
	Communication and alarm systems, fire detection, addressable, 12 detectors, includes outlets, boxes, conduit and wire			
	Fire alarm command center, addressable without voice, excl. wire & conduit			
D5090	**Other Electrical Systems**		**$0.06**	**$6,000**
	Generator sets, w/battery, charger, muffler and transfer switch, gas/gasoline operated, 3 phase, 4 wire, 277/480 V, 7.5 kW			
E Equipment & Furnishings		**6.40%**	**$3.62**	**$350,000**
E1030	**Vehicular Equipment**		**$1.26**	**$122,000**
	Architectural equipment, parking equipment, automatic gates, 8 FT arm, 1 way			
	Architectural equipment, parking equipment, booth for attendant, deluxe			
	Architectural equipment, parking equipment, ticket printer/dispenser, rate computing			
E1090	**Other Equipment**		**$2.36**	**$228,000**
	280 - Precast concrete parking bumpers, wheel stops, precast concrete, 6" x 10" x 6' - 0", includes 2 dowels per each			
	280 - Pavement markings, parking stall, paint, white, 4" wide			
	1 - Parking control equipment, parking control software, min			
	1 - Parking equipment, collector station, pay on foot			
	1 - Parking equipment, ticket spitter with time/date stamp, mag stripe encoding			
	1 - Parking equipment, ticket spitter with time/date stamp, standard			
	1 - Parking equipment, fee computer			
	1 - Parking gates, barrier gate with programmable controller			
F Special Construction		**0.00%**	**$0.00**	**$0**
G Building Sitework		**21.97%**	**$20.69**	**$2,000,000**

SubTotal	**100%**	**$93.96**	**$9,102,000**
Contractor Fees (General Conditions,Overhead,Profit)	**25.00%**	**$23.49**	**$2,275,500**
Architectural Fees	**6.00%**	**$5.64**	**$546,120**
User Fees	**3.00%**	**$2.82**	**$273,060**
Total Building Cost		**$125.90**	**$12,196,680**

Total Building Cost	$41,962,898